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# Fatigue Life of Bridge Beams Subjected to Controlled Truck Traffic

*Résistance à la fatigue de poutres de ponts soumises à des essais de passage de camions*

*Dauerfestigkeit von Brückenträgern für Testlastenzüge*

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## Introduction

Although almost no fatigue failures of highway bridges have been reported, the increasing frequency of heavy trucks on modern highways suggests that the past experience may not be indicative of future trends. A fairly extensive body of laboratory data has been available on the fatigue life of structural members. However, the scarcity of failures in the field left unanswered the question of how to apply the results of the laboratory research to practical design problems, and gave rise to some doubt concerning the need for fatigue considerations in the design of highway bridges.

To explore the subject of the fatigue life of highway bridges, the American Association of State Highway Officials undertook tests of eighteen bridges under repeated overstress. The experiment was a part of the AASHO Road Test, a study of highway pavements and bridges under controlled truck traffic.

The principal objective of the bridge research program was to gather information on the effects of repeated overstress. In the past, many investigations of the fatigue life of beams were made on small laboratory specimens. Such tests covered wide ranges of pertinent variables. Could these laboratory tests be applied to bridges in actual service? Given characteristics of the materials and a reasonable estimate of the number, weight, and arrangement of the vehicle loads, could one make a reasonably good prediction of the usable life of the structure?

The combination of the precise knowledge of the materials, the design of the structures and the control of the loading made possible to study these and other items at the Road Test. The tests afforded the rare opportunity to compare the results of simpler laboratory experiments with the behavior of actual bridge structures. This paper deals only with five bridges with steel beams that showed fatigue distress.

## Description of Tests

### *Bridge Tests*

Each superstructure consisted of three steel beams and a reinforced concrete deck. The beams were fabricated from 18-in. deep wide-flange sections rolled of A 7 structural steel. In three bridges the beams were independent of the slab. A treatment of the top surface of the top flanges inhibited formation of natural bond between the slab and the steel beams. In two bridges the steel beams were connected to the slab with channel connections.

Beams of one noncomposite bridge and of the two composite bridges had partial-length cover plates on the bottom flange only, while the beams of the other two noncomposite bridges had partial-length cover plates on both the top and bottom flanges. All plates were welded to the flange with $^5/_{16}$-in. continuous fillet welds along the longitudinal edges. The ends of the plates were square and had no end welds.

The mean yield point of the flanges of the wide-flange beams varied from 32,500 to 37,900 psi and the mean ultimate strength from 59,500 to 64,900 psi from bridge to bridge. The chemical composition of the steel was 0.20—0.24% Carbon, 0.41—0.71% Manganese, 0.007—0.014% Phosphorus and 0.023 to 0.040% Sulfur. The estimated mean residual stresses in the flanges, caused by rolling the wide-flange sections, varied from a compression of 7,400 psi to a tension of 11,300 psi.

Individual mean values of the material properties for a particular beam, bridge, or slab and their standard deviations may be found in an earlier report[1]). The report contains also a description of the test methods and details of the construction procedures.

During a two-year period of test traffic, the bridges were tested with tractor semi-trailer trucks which traveled around the loops at approximately 30 mph. The time interval between individual passages of the vehicles over a bridge varied from one to two minutes. This resulted in over half a million passages of the test vehicles. Details of the test structures and experiments may be found elsewhere[2]).

The maximum stresses in the steel beams occurred just off the ends of the cover plates. The stresses at these critical locations varied during the vehicle trip. A typical variation of stress at the end of a cover plate during the trip of a vehicle over a bridge is illustrated in Fig. 1.

After a vehicle crossed a bridge, the bridge continued to vibrate causing alternate upward and downward deflections of decreasing amplitude. These

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[1]) "The AASHO Road Test, Report 2, Materials and Construction", *Highway Research Board,* Special Report 61 B, 1962.

[2]) "The AASHO Road Test, Report 4, Bridge Research", *Highway Research Board,* Special Report 61 D, 1962.

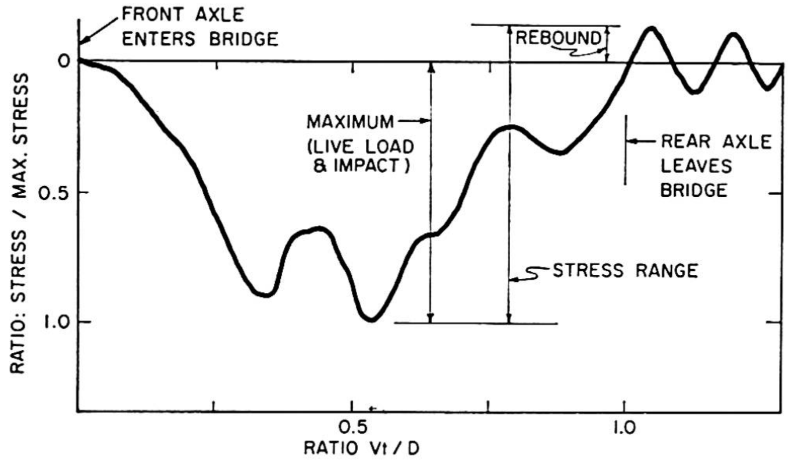


Fig. 1. Variation of stress near end of cover plate on passage of vehicle.

deflections caused stress fluctuations (Fig. 1). The maximum amplitude of the first negative half cycle of the stress fluctuation was designated the rebound stress. The rebound stress was of a sign opposite to that of the stress observed while the vehicle was on the bridge and thus increased the range of the fluctuating stress. The ratio of the sum of the maximum transient stress and the rebound stress to the maximum transient stress was in excess of 1.1 for non-composite bridges and less than 1.1 for composite bridges.

### *Laboratory Tests*

The fatigue characteristics of steel beams with partial-length cover plates with no welds across the end were evaluated by studies of the results of flexural fatigue tests of ten small welded beams having similar welded details as the bridge beams. Details of the experiments are given elsewhere[3]).

The beams were fabricated from A 373 plate steel. The yield point of the beam flanges was 38,700 psi and the ultimate strength was 63,400 psi. The steel had 0.18—0.23% Carbon, 0.53—0.94% Manganese, 0.007—0.023% Phosphorus and 0.019—0.030% Sulfur. These properties were similar to those of the beams in the Road Test bridges.

All beams were 12-in. deep and 11-ft. long I-beams. Six specimens were built up of two $^3/_4$-in. thick flanges welded to a $^3/_{16}$-in. web; $^1/_2$-in. thick cover plates were attached to both the tension and the compression flanges with $^1/_4$-in. fillet welds. Four specimens were built up of two $^3/_8$-in. thick flanges welded to a $^1/_4$-in. web and a $^1/_4$-in. thick cover plate attached to the tension flange with $^3/_{16}$-in. fillet welds. The welding was done manually with low hydrogen electrodes conforming to AWS Specification E-7016.

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[3]) Munse, W. H., and Stallmeyer, J. E.: "Fatigue in Welded Beams and Girders", *Highway Research Board,* Bulletin 315, 1962.

The specimens were tested in flexure on a span of 8 ft. 6 in. in an Illinois walking beam fatigue testing machine. The load was applied at a rate of 180 cycles per minute as two concentrated loads located 6 in. each side of midspan.

## Test Results

### *Bridge Structures*

The stresses caused by dead loads and by regular test traffic at the critical sections in the bridge beams are summarized in Table I. The minimum stress was obtained as the difference between the dead load stress and the rebound stress (Fig. 1). The dead load stresses were calculated with actual weights of the materials and were checked by strain measurements on several bridges. The stress range was obtained as the sum of the maximum live load and impact stress and the rebound stress (Fig. 1).

The stress range varied from one passage of the test vehicle to another as is shown by the standard deviations in Table 1. However, the variations in the stress range were smaller than one would expect to find under mixed traffic on bridges in the highway system.

Fatigue cracks were first discovered late during the period of the regular test traffic. The number of vehicle trips at the time the cracks were found is listed in the last column of Table 1. The fatigue cracks first appeared in the bottom surface of the rolled section at the toe of the welds connecting the cover plate and were usually $^1/_4$ to $^1/_2$ in. long. The cracks were determined by visual inspection of the critical areas with a magnifying glass.

Except for the crack at the approach end of the plate on the interior beam of Bridge 2 B, the cracks changed either very little or not at all during the remainder of the traffic. The one crack on Bridge 2B spread through one quarter of the bottom flange as shown in Fig. 2. By the end of regular test traffic Bridges 1A, 2B and 3B were subjected to approximately 556,000 vehicle trips. Bridge 1A had ten fatigue cracks, Bridge 2B had five cracks and Bridge 3B had two cracks at that time. Two fatigue cracks were found on Bridge 9A and one on Bridge 9B: these two bridges were subjected to approximately 478,000 vehicle trips.

The number of stress cycles, $N$, at first detection of fatigue cracks varied from 478,000 to 606,000. The corresponding $\log N$ was 5.679 to 5.783. Hence, the total variation in the logarithmic life was only 1.8%. Because of the small variation on the logarithmic life, the regression line in Fig. 3, including all test data, can be considered as corresponding to the average $\log N$ or 530,000 stress cycles.

It can be seen from Fig. 3 that an increase in the minimum stress from 5,000 to 20,000 psi caused no change in the number of cycles to fatigue cracking

*Table 1. Fatigue Strength of Steel Bridges*

| Bridge | Beam | Critical Section | Min. Stress ksi | Stress Range | | Cycles to Cracking |
|---|---|---|---|---|---|---|
| | | | | Mean ksi | Std. Dev. ksi | |
| 1 A | Interior | Approach | 10.7 | 13.3 | 1.20 | 556,900 |
| | Center | Approach | 13.8 | 13.6 | 0.92 | 536,000 |
| | Exterior | Approach | 16.5 | 12.8 | 0.94 | 536,000 |
| | Interior | Exit | 10.5 | 15.5 | 1.17 | 557,300 |
| | Center | Exit | 13.6 | 15.7 | 0.98 | 536,000 |
| | Exterior | Exit | 16.1 | 15.1 | 1.04 | 536,000 |
| 2 B | Interior | Approach | 14.0 | 15.9 | 1.75 | 531,500 |
| | Center | Approach | 18.0 | 15.8 | 1.74 | 531,500 |
| | Exterior | Approach | 21.1 | 15.2 | 1.80 | 606,000 |
| | Center | Exit | 18.0 | 15.4 | 1.39 | 531,500 |
| 3 B | Center | Approach | 15.0 | 14.0 | 2.08 | 535,500 |
| | Exterior | Exit | 17.9 | 13.4 | 1.59 | 557,800 |
| 9 A | Exterior | Approach | 9.4 | 16.1 | 1.44 | 477,900 |
| | Exterior | Exit | 9.2 | 16.4 | 0.21 | 477,900 |
| 9 B | Center | Approach | 6.7 | 17.6 | 1.40 | 477,900 |

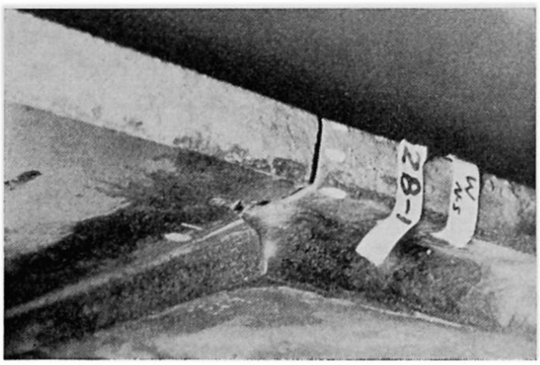

Fig. 2. Propagation of fatigue crack.

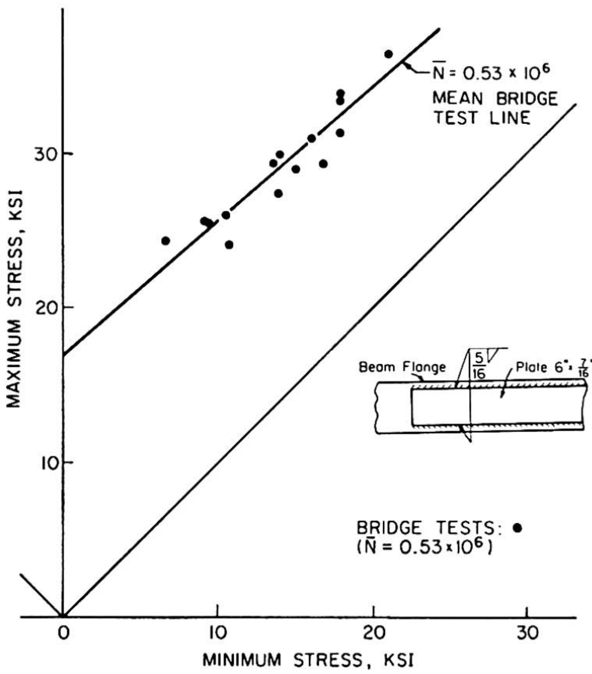


Fig. 3. Results of bridge tests.

when accompanied by a decrease in the stress range from 16,500 to 14,500 psi. Apparently a large change of the minimum stress accompanied by no change in the stress range may be expected to produce only a small change in the number of cycles to fatigue cracking. In other words, the minimum stress appeared to have only a small effect on the fatigue life.

### *Laboratory Beams*

The results of the laboratory fatigue tests of beams with partial-length cover plates are given in Table 2. Six beams were tested at the minimum stress of 400 psi and four were tested at the minimum stress of 15,400 psi. The maximum stress was so chosen that one-half of the specimens, at each level of minimum stress, failed above 1,000,000 cycles and the other half below 400,000 cycles. All fatigue cracks started at the toe of the longitudinal weld connecting the cover plate and propagated transversely and vertically through the beam flange.

*Table 2. Laboratory Fatigue Tests of Beams*

| Test Specimen | Minimum Stress, ksi | Maximum Stress, ksi | No. of Cycles To Failure | No. of Cycles To Last Inspection Prior to Failure |
|---|---|---|---|---|
| CPDF-1 | 0.4 | 13.2 | 1,431,500 | 1,000,100*) |
| CPDG-1 | 0.4 | 13.3 | 2,819,300 | 2,308,500**) |
| CPCD-1 | 0.4 | 14.3 | 1,308,600 | 1,126,400 |
| CPDF-2 | 0.4 | 23.9 | 291,800 | 220,700 |
| CPAD-1 | 0.4 | 24.9 | 256,700 | 160,800 |
| CPCD-2 | 0.4 | 26.0 | 260,200 | 241,600**) |
| 1 | 15.6 | 28.0 | 1,378,400 | 1,312,800 |
| 4 | 15.4 | 27.7 | 1,524,500 | 1,150,900 |
| 2 | 15.4 | 36.0 | 350,200 | 307,400 |
| 3 | 15.4 | 36.7 | 285,200 | 222,100 |

*) Specimen had a crack 1 ½ in. long.
**) Specimen had a crack ¼ in. long.

The number of cycles at which the crack became visible in the laboratory specimens was not determined. However, at the last inspection prior to failure small cracks, $^1/_4$ to $1\,^1/_2$ in. long, were found in three beams while no cracks were found in the remaining seven beams. Apparently, had the inspections been more frequent, cracks in the seven specimens would have been found somewhere between the "last inspection" and the "failure".

The number of cycles to last inspection is listed in Table 2 in addition to the number of cycles to failure. As all last inspections were made relatively close to failure, the number of cycles to last inspection will be considered as the number of cycles to fatigue cracking.

## Analysis of Test Results

### *Laboratory Tests*

The experimental data from Table 2 are plotted in Fig. 4 in which the stress range is given as a function of the logarithm of the number of cycles to cracking. A separate plot is included for each minimum stress level.

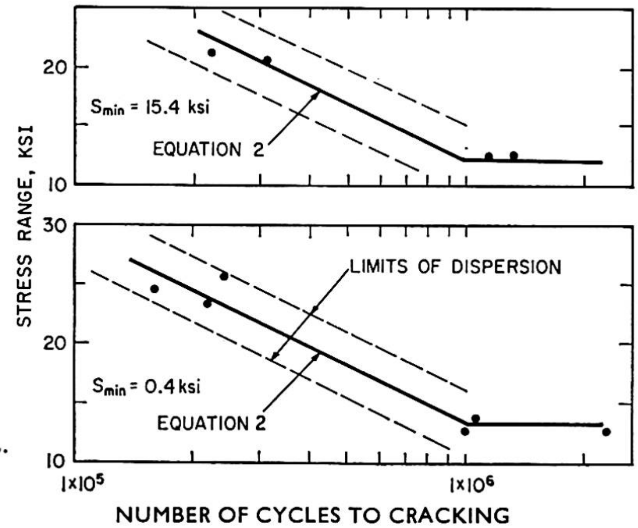


Fig. 4. S-N diagrams of laboratory data.

The relationship between the stress range and the logarithm of the number of cycles to failure was assumed to be represented by two straight lines: a sloping line up to 1,000,000 cycles (fatigue strength) and a horizontal line beyond 1,000,000 cycles (endurance limit).

The sloping line was expressed by the following mathematical model:

$$\log N = A + B\,S_r + C\,S_{min} \pm E\,, \tag{1}$$

in which
$S_r$ = $S_{max} - S_{min}$; range of stress, ksi
$S_{min}$ = minimum stress, ksi
$S_{max}$ = maximum stress, ksi
$N$ = number of cycles to failure
$A, B, C$ = empirical constants
$E$ = estimate of experimental error

Coefficients $A$, $B$, and $C$ of Eq. 1 were evaluated by regression analysis of the laboratory data.

The analysis was based on the number of cycles to the last inspection before failure. For specimens with $N$ larger than 1,000,000, the value of $\log N = 6.0$ was used in the analysis. The following equation was obtained for the number of cycles to fatigue cracking:

$$\log N = 6.827 - 0.0620\,S_r - 0.0056\,S_{min} \pm 0.180\,. \tag{2}$$

The error term at the end of the equation is equal to twice the standard error of estimate.

Equation 2 applies only up to 1,000,000 cycles. The stress range corresponding to cracking at 1,000,000 cycles represents the endurance limit. Both Eq. 2 and the endurance limit are shown in Fig. 4.

An examination of the coefficients of Eq. 2 reveals the relative significance of the stress range and the minimum stress. For example, a decrease in the stress range of 10 ksi increases the logarithmic life by 0.62; but the same decrease in the minimum stress increases the logarithmic life only by 0.056 or less than one-tenth as much as the change in the stress range. The relatively small effect of the minimum stress on fatigue life is illustrated in Fig. 5 containing the two mean regression lines for the laboratory tests and the limits of dispersion.

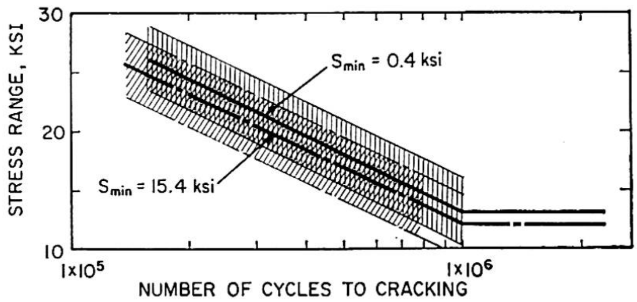


Fig. 5. Effect of minimum stress on fatigue life.

*Comparison of Laboratory and Bridge Tests*

Fatigue cracking of the bridge beams furnished data for quantitative comparisons with the results of the simpler laboratory tests.

In the laboratory tests the stress fluctuations followed a simple sine wave; in the bridge tests the shape of the time-stress curve was irregular including major stress waves caused by the weight of the moving vehicles, minor stress waves caused by vibration of the bridge, and rest periods corresponding to the intervals between the trucks and to periodic breaks in the test traffic. Finally, in the laboratory tests the duration of one stress cycle was of the order of one second or less while in the bridge tests the intervals of the major stress cycles were of the order of 40 seconds or more. Thus the stress histories of the bridge tests differed considerably from those of the laboratory tests.

In comparing quantitatively the results of the bridge tests with laboratory data, the only characteristics of the stress histories considered were the minimum stress and the stress range. The effects of the speed of loading, rest periods, and vibrations were disregarded because of lack of methods which would permit their inclusion in the analysis.

The results of the individual comparisons are shown in Fig. 6 in which the ratio of the observed to the computed number of cycles to fatigue cracking is plotted for every beam section with a fatigue crack detected during the traffic period. The individual comparisons are plotted corresponding to the sequential listing of the observed number of cycles to cracking in Table 1. The computed values were obtained by three methods: the bars in Fig. 6, each representing one beam cross-section, show the results for all three methods.

The method designated as "Miner's Hypothesis" accounted for the variations in the stress range with the aid of Miner's Hypothesis of cumulative damage[4]). In this analysis, use was made of the minimum stress and both the mean and the standard deviation of the stress range (Table 1). Details of this method may be found elsewhere[2]).

The method designated in Fig. 6 as "Mean Stress Range" neglected the variations in the stress range. Values of $N$ were calculated from Eq. 2, neglecting the error term, with the minimum stress and the mean stress range.

Finally, the method designated as "Rebound Neglected" used the same approach as the method "Mean Stress Range" except that the minimum stress was taken equal to the dead load stress and the stress range was taken as the live load and impact stress. In other words, the rebound stress (Fig. 1) was neglected in the computations.

Fig. 6 includes also the limits of dispersion of the test data computed from the error term in Eq. 2. It will be noted that all but two ratios based on Miner's Hypothesis and on Mean Stress Range fall within these limits. Furthermore, the values calculated with the aid of Miner's Hypothesis were in the best agreement with the laboratory data. Finally, the data in Fig. 6 demonstrate the need to consider the rebound stress in the analysis.

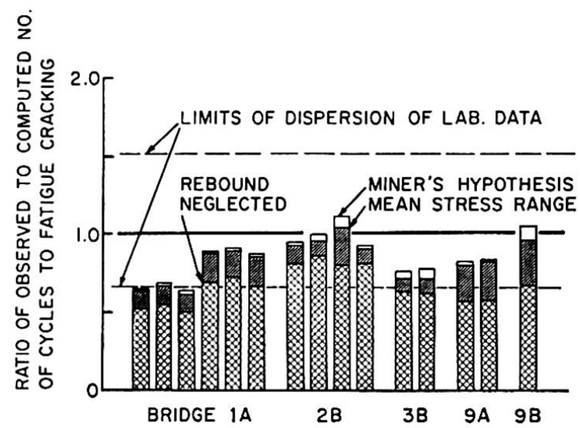


Fig. 6. Comparison of individual bridge test results with laboratory data.

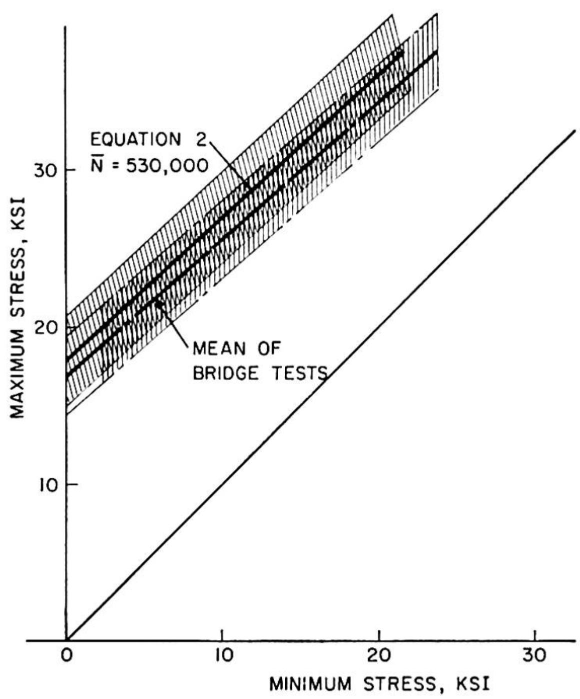


Fig. 7. Overall comparison of bridge and laboratory tests.

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[4]) MINER, M. A.: "Cumulative Damage in Fatigue", *Journal of Applied Mechanics*, September, 1945.

The results of the overall comparisons are shown in Fig. 7. The regression line for the bridge tests is compared with a line computed from Eq. 2 for 530,000 cycles. The means as well as the limits of dispersion, equal to twice the standard error of estimate, are included for both the bridge tests and the laboratory data. The scatter bands for the two series of tests show a substantial overlap.

The comparisons in Fig. 6 and 7 show that the results of the bridge tests were in excellent agreement with the results of the simple laboratory fatigue tests. However, the bridge beams appeared to be slightly weaker in fatigue than the laboratory specimens. Fortunately, the differences were too small to be of practical engineering significance.

The comparisons show further that it should be satisfactory to base the engineering analysis of the fatigue strength of structures solely on the magnitude of the minimum stress and the stress range. Of the two, the effect of the stress range appears to be considerably more important and therefore should be estimated with greater accuracy: it should include not only the live load and the impact stresses but also the rebound stress. Furthermore, for structures with large variations of the stress range it may be necessary to consider the effects of the so-called cumulative damage.

### *Comparison of Test Results with Design Specifications*

It has been shown in the preceding discussion that the fatigue strength of beams with partial-length cover plates is satisfactorily represented by Eq. 2. Thus the equation may be used to examine the design stresses permitted for such members by current design specifications. As all test beams were made of structural grade steel, the comparisons are limited to such steels.

Fig. 8 shows allowable stresses for 2,000,000 cycles used in Great Britain (steel BS 15)[5], U.S.A. (A 36)[6], U.S.S.R. (ST 3)[7] and West Germany (ST 37)[8]. The figure includes also the mean fatigue strength and the limits of dispersion of test data for 2,000,000 cycles.

The allowable stresses are represented by a sloping line (or a series of sloping lines) and a horizontal line. Except for the allowable stresses specified by the American Welding Society, there is little variation between the allowable stresses. The sloping lines fall slightly below and follow reasonably well the slope of the mean fatigue strength. However, they are within the scatter band of the test data.

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[5]) British Standard 153: 1958, "Steel Girder Bridges" (with Amendments 1, 2, 3 and 4), 1962.

[6]) American Welding Society, "Specification for Welded Highway and Railway Bridges", 1963.

[7]) TUPIN-SV-55, "Specification for the Design and Construction of Welded Railway Bridges", 1955.

[8]) DV 848, "Regulations for Welded Railway Bridges", 1955.

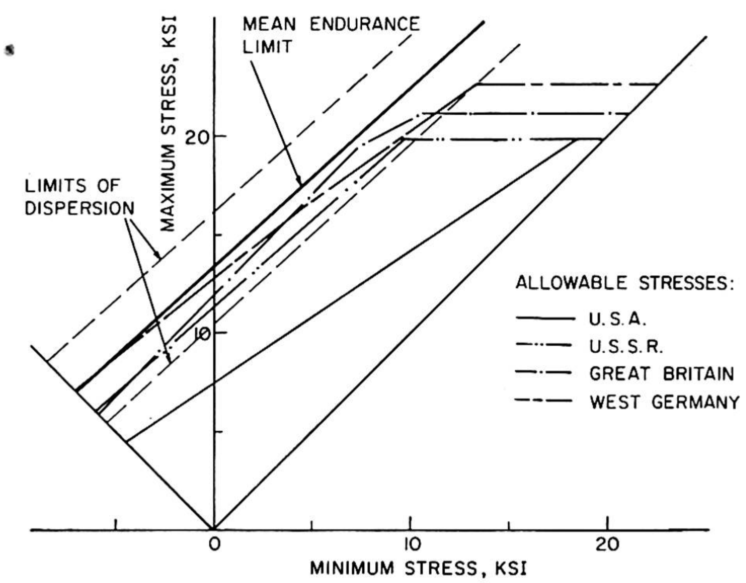


Fig. 8. Fatigue strength and allowable stresses for 2,000,000 cycles.

The horizontal lines represent the limitation imposed by the basic design stress, a condition independent of fatigue considerations. For the four mild steels considered, the basic design stress varied from 20 to 22.7 ksi.

The allowable stresses specified by the American Welding Society are considerably more conservative. While the factor of safety may be considered satisfactory for the case of full stress reversal, the divergence of the sloping line from the mean fatigue strength results in a prohibitively large factor of safety.

### *Generalized Design Approach*

Current design rules specify the allowable stresses for fatigue loading in terms of maximum and minimum stress. Separate sets of values are usually given for different weld details, different desired lives of the structure and different grades of steel. Studies carried out in conjunction with the tests reported here indicate that this seemingly endless procession of design equations may be replaced by a single formula based on classic concepts of fatigue strength and a different set of two empirical coefficients for each different category of weld details.

Several investigators have reported that the fatigue lives of beams of different steels, having the same type splice configuration, were approximately the same[9]). Recently GURNEY[10]) concluded on the basis of an extensive

[9]) GURNEY, T. R.: "Fatigue Strength of Fillet Welded Joints in Steel", *British Welding Journal*, March 1960; STALLMEYER, J. E., NORDMARK, G. E., MUNSE, W. H., NEWMARK, N. M.: "Fatigue Strength of Welds in Low Alloy Structural Steels", *Welding Journal*, Vol. 35, January 1956.

[10]) GURNEY, T. R.: "Fatigue Tests of Butt and Fillet Welded Joints in Mild and High Tensile Structural Steels", *British Welding Journal*, November 1962.

investigation that the fatigue strength of *similar welded details* in mild steel and high tensile steel can be represented by the same *S-N* curve. This point is illustrated in Fig. 9 presenting the results of fatigue tests of tension joints with transverse butt welds in mild (BS 15 and A 7, $Fy = 35{,}000$ psi)[9], [10]) low-alloy (BS 968 and A 242, $Fy = 54{,}000$ psi)[9], [10]) and constructional alloy (N-A XTRA 100, $Fy = 93{,}000$ psi)[11]) steels. The principal difference among the different grades of steel is the point at which the curve starts to deviate appreciably from the straight line. As this point of departure is generally above the yield point of the material, it is only of academic interest.

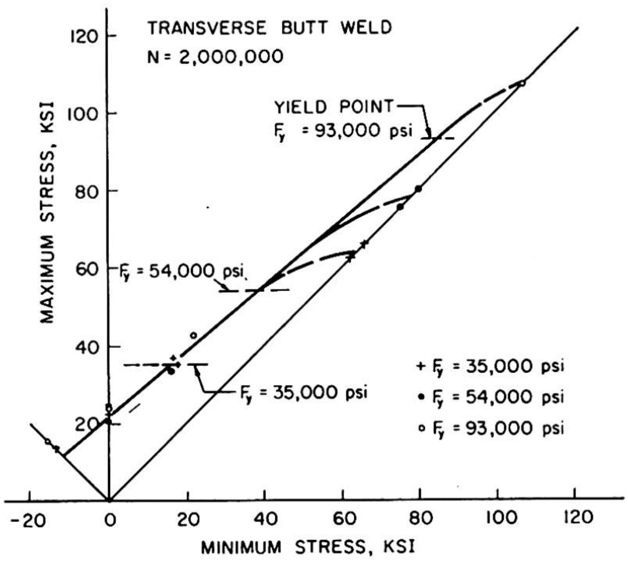


Fig. 9. Effect of steel strength on fatigue strength of welded joints.

Therefore, the same sloping line representing a specific fatigue life is applicable to all steels irrespective of their yield point or ultimate strength. This sloping line can be represented by a general design formula. The horizontal cut-offs will be different for various steels since they are usually determined by applying a uniform factor of safety against the static strength of the material. This consideration is independent of the fatigue strength.

The general design formula for fatigue strength may be expressed in several different terms[12]) but for the purposes of this discussion the following form

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[11]) SCOTT, G. R., STALLMEYER, J. E., and MUNSE, W. H.: "Fatigue Strength of Transverse Butt-Welded Joints in N-A XTRA 100 Steel", University of Illinois, 1963.

[12]) For example, one of the better known variations of this formula is:

$$S_{max} = \frac{\alpha}{1 - \beta k},$$

where $k = S_{min}/S_{max}$ and the symbols "$\alpha$" and "$\beta$" are empirical coefficients.

was chosen:

$$S_r = C_1 - (1 - C_2)\, S_{min}, \tag{3}$$

$S_r$ = permissible stress range
$S_{max}$ = maximum stress
$S_{min}$ = minimum stress
$C_1$ = maximum permissible stress for a 0 to tension loading and $N$ cycles
$C_2$ = slope of the permissible stress line

Empirical coefficients $C_1$ and $C_2$ must be evaluated for each category of weld details. This may be done by a statistical analysis of the test data using the relationship given by Eq. 1. The coefficients $C_1$ and $C_2$ may then be evaluated as:

$$C_1 = \frac{\log N - A'}{B}, \tag{4}$$

$$C_2 = \frac{B - C}{B}, \tag{5}$$

where $A'$ is the empirical constant $A$ (Eq. 1) corrected for the error term (or its multiple). *The correction for the error term decreases the probability of fatigue failure and thus provides the necessary factor of safety.*

An item of particular interest is the magnitude of coefficient $C_2$: its value is often close to 1.0. For example, for beams with partial-length cover plates (Eq. 2) it is equal to 0.91. The substitution of $C_2 = 1.0$ in Eq. 3 provides an approximate rule for guarding against fatigue failure: the stress range must not exceed a certain allowable value which depends only on the category of weld detail and the desired life of the structure.

## Summary

Tests of five slab and beam steel bridges with partial-length cover plates are discussed. All bridges were subjected to 480,000 or 560,000 passages of test trucks. The number of stress cycles at fatigue cracking of steel bridge beams are compared with laboratory fatigue data.

Four current specification requirements for design against fatigue are compared with the results of these tests, and a generalized approach to the problem of fatigue design is outlined.

## Résumé

Les auteurs analysent les résultats d'essais exécutés sur cinq ponts, constitués d'une dalle reposant sur des poutres métalliques renforcées par des semelles sur une partie de leur longueur. Tous les ponts ont été sollicités par le passage de 480.000 ou 560.000 camions. On compare les essais sur ponts et ceux au laboratoire en considérant le nombre de cycles précédant la formation de fissures dues à la fatigue.

On compare également les résultats de ces essais aux sollicitations admissibles à la fatigue fixées par quatre règlements; on expose en outre, dans ses grandes lignes, une méthode généralisée de calcul de la résistance à la fatigue.

## Zusammenfassung

In diesem Beitrag werden an fünf Brücken mit stählernen Hauptträgern, gebildet aus durch aufgeschweißte Gurtplatten teilweise verstärkten I-Walzprofilen und einer Stahlbeton-Fahrbahnplatte, durchgeführte Versuche besprochen. Alle Brücken werden entweder 480 000 oder 560 000 Überfahrten von Testlastenzügen unterworfen. Die Lastwechselzahlen für das Auftreten von Ermüdungsrissen an den Stahlträgern werden mit den Werkstattergebnissen verglichen.

Vier zur Zeit geltende Dauerfestigkeitsvorschriften werden den Ergebnissen dieser Versuche gegenübergestellt und anschließend wird eine allgemeine Näherungslösung für den Dauerfestigkeitsnachweis gegeben.