1970


John W. Fisher

Follow this and additional works at: http://preserve.lehigh.edu/engr-civil-environmental-fritz-lab-reports

Recommended Citation
http://preserve.lehigh.edu/engr-civil-environmental-fritz-lab-reports/408

This Technical Report is brought to you for free and open access by the Civil and Environmental Engineering at Lehigh Preserve. It has been accepted for inclusion in Fritz Laboratory Reports by an authorized administrator of Lehigh Preserve. For more information, please contact preserve@lehigh.edu.
Low-Cycle Fatigue

FATIGUE STRENGTH OF WELDED
A514 STEEL BEAMS

by

John W. Fisher

This paper was prepared as part of a study of Low-Cycle Fatigue, sponsored by the Office of Naval Research, Department of Defense, under Contract N 00014-68-A-514; NR 064-509. Reproduction in whole or part is permitted for any purpose of the United States Government.

Fritz Engineering Laboratory
Lehigh University
Bethlehem, Pennsylvania

April, 1970

Fritz Engineering Laboratory Report No. 358.16
# TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>SUMMARY</td>
<td>1</td>
</tr>
<tr>
<td>INTRODUCTION</td>
<td>2</td>
</tr>
<tr>
<td>FATIGUE STRENGTH OF COVER-PLATED BEAMS</td>
<td>6</td>
</tr>
<tr>
<td>FATIGUE STRENGTH OF WELDED BEAMS WITHOUT ATTACHMENTS</td>
<td>8</td>
</tr>
<tr>
<td>STRESS ANALYSIS OF CRACK PROPAGATION</td>
<td>10</td>
</tr>
<tr>
<td>SUMMARY AND CONCLUSIONS</td>
<td>13</td>
</tr>
<tr>
<td>ACKNOWLEDGEMENTS</td>
<td>14</td>
</tr>
<tr>
<td>REFERENCES</td>
<td></td>
</tr>
<tr>
<td>TABLES AND FIGURES</td>
<td></td>
</tr>
</tbody>
</table>
SUMMARY

This paper summarizes 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.

Only two of the beam series are discussed herein. Emphasis is placed on ASTM A514 steel although comparisons are also provided for other grades of steel. The beam types 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.
In previous studies failure to properly control and measure the variables influencing the fatigue strength was usually the major reason for the apparently conflicting and contradictory claims on the significance of stress variables and material characteristics.

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. A theoretical stress analysis based on the fracture mechanics of crack growth substantiated the empirical exponential model that provided the best fit to the test data. In addition, the theoretical analysis was used to rationally explain the observed behavior of the experimental results.

**INTRODUCTION**

In the present design (1970) of steel bridges for fatigue, provisions have been specified in many instances on the basis of limited test data. Equally important to the fatigue life of highway bridges is the significance of such factors as the loading history to which the structures are subjected, the type of materials used, the design details, and the quality of fabrication.

Recognition of these facts lead to the bridge studies of 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.*

**Objectives and Design Variables**

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. 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). 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.

Although three grades of steel were examined - ASTM A36, A441 and A514 - the primary emphasis in this paper is on A514 steel. Comparisons are made with the other steels where appropriate.

Four different types of beams were examined including cover-plated 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 and a lower-bound to fatigue behavior. The

*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.*
second detail discussed is the plain welded beam without attachments, which provided a minimum notch producing detail and an upper bound for the behavior of welded beams.

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. All A514 steel beams had 11.4 cm (4½ in) wide cover plates which were 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 320 cm (10 ft.-6 in.) long and were tested on a 304.8 cm (10 ft.) span. The ends of the cover plate details were positioned in the shear spans 30.5 cm (12 in) from each load point. The plain-welded beams were loaded so that a 106.7 cm (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 ratio (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.

Experiment Design

Each beam series was arranged into factorial experiments. The basic factorial for each steel within each beam type was defined by the stress variables as illustrated in Table 1 for the cover-plated and plain-welded A514 steel beams. The same factorial existed for each type of steel and comparable factorials existed for the other types of beams and details.

All factorials with cover-plated beams had at least three specimens assigned to each cell. This permitted the variance of each cell to be estimated. Each cell of the plain-welded beam factorial had two specimens or replicates assigned, since it was considered that more than one fatigue crack was probable between the load points.

None of the experimental factorials were complete. Partial factorials for each series were developed because of known boundary conditions. The maximum values of stress had to be limited to the yield stress of the material. The lower values of stress range were not examined at all values of minimum stress because of excessive lives. At least 10 million cycles were applied before testing was discontinued and a fatigue limit was assumed to be reached.
The fact that all beams were fabricated symmetrically about their neutral axis provided information on the behavior of both the tension and compression flanges.

The order of testing of the specimens for each beam type was randomized so that the effect of uncontrolled variables such as temperature, humidity, laboratory and testing personnel would be random. The randomization of the uncontrolled variables allows the effect of these variables to be included as a random error in the analysis and prevents any systematic bias due to these effects on the controlled variables.

**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 illustrated in Fig. 2. During the first stage of growth, the crack grew through the flange in an elliptical shape. 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 at ends where 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 3 summarizes the test data for A514 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. 4. 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 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 Refs. 5 and 9.

**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. 5. 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, but grew out to the fillet weld surface and were analogous to stage 1 of crack growth at the end of a cover plate. After penetrating the outside fibers of the flange, the crack grew on two fronts toward the flange tips which was comparable to stage 2 in the cover-plated beams.

Most of the A514 steel beams showed a clear tendency to form multiple cracks in the tension flange. Many cracks also occurred in the compression flange with more cracks formed with a decreasing minimum stress.

The significance of the design factors, minimum stress and stress range are illustrated in Fig. 6. 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.

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. 7. 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 a
21.3 cm (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 welded beams with the reinforcement removed from groove-welded flange splices.

STRESS ANALYSIS OF CRACK PROPAGATION

For the prediction of macrocrack propagation the relationship proposed by Paris (13) was used. The relationship

\[ C \Delta K^m = \frac{da}{dN} \]  

(1)

expresses the change in crack length, \(a\), for a sinusoidal stress cycle \(N\) as a function of the change in \(K\) during that cycle and a constant of proportionality. The \(K\) value for a crack can be expressed in terms of the "corrected crack \(c\)" as

\[ K = \sigma \sqrt{\pi c} \]  

(2)

Since \(K\) is determined both by crack 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 stresses at the crack tip. Signes et al. (14) have shown the fatigue cracks at the toe of fillet welds start from small cracks at the weld toe. These cracks exist before cyclic loading is applied to the joint. The
applicability of Equation 1 has been illustrated for many types of materials and geometric configurations (13,15,16).

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 corrected crack size at initiation (\( c_i \)) and failure (\( c_f \)). Neglecting \( dc/da \) this yields

\[
A'S_r^n\Delta N = c_i^{-\alpha} - c_f^{-\alpha}
\]  

(3)

where \( \Delta\sigma = \lambda S_r \) is assumed constant and equal to the product of the stress concentration factor, \( \lambda \), and the applied stress range, \( S_r \); \( A' = A\alpha n/2 \lambda n \), \( \Delta N = N_f - N_i \), and \( \alpha = \frac{n}{2} - 1 \). The value of the corrected failure crack, \( c_f \), is large and can be neglected since \( \alpha > 0 \). The relationship between life \( \Delta N \) and the applied stress range is exponential in form which agrees with the results of the regression analysis.

The exponent of stress range, \( n \), was observed to vary between 2.80 and 2.87 when cover-plates were attached to A36, A441 and A514 steel beams. Hence, a variation in yield stress from 25.2 kg/mm\(^2\) (36 ksi) to 70 kg/mm\(^2\) (100 ksi) caused a negligible change in exponent. The variation in mean 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. No plasticity correction was used for computation of the corrected crack size \( c \).
Equation 3 could then be expressed as

\[ \Delta N = \left( \frac{1}{A'} \right) \left( c_{i}^{-1/2} \right) \sigma_{r}^{-3} \]

(4)

The crack growth in the cover-plated beams was analyzed by Frank and Fisher (5,9). They observed the cracks to be semi-elliptical in shape with a ratio of \( a \) to \( b \) (see Fig. 2) 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 3 was used to evaluate the parameter \( A' \) and the corrected initial crack size \( c_{i}^{-1/2} \). This yielded \( c_{i}^{-1/2} = 130/\sqrt{\pi} \) and \( A' = 1.02 \times 10^{-7}/\text{ksi}^{3} \sqrt{\pi} \).

Substituting these values into Eq. 4 gives the number of cycles for the crack to propagate through the flange as

\[ \Delta N = 1.28 \times 10^{9} \sigma_{r}^{-3} \]

(5)

Equation 5 is compared with the mean regression line for beams with end welded cover plates in Fig. 8. The predicted crack growth is in good agreement with the experimental results. Also shown in Fig. 8 is the mean regression curve for plain-welded beams together with the exponential 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.

The quantity of \( (1/A')(c_{i}^{-a}) \) is a measure of the notch effect of each type of specimen and detail. This quantity may be expressed as
\[ \frac{1}{A'}(c_1^{-a}) = \frac{1}{[A\lambda^{n/2} \alpha_0] c_1^a} \]

It is therefore inversely dependent upon the initial corrected crack size, \( c_1 \), the constant of crack growth, \( A \), and the stress concentration factor, \( \lambda \). The cover-plated beams represented the most severe condition of these parameters.

**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, a study and correlation with earlier work, and on theoretical studies based on the application of continuum mechanics to macrocrack propagation. Details of the study are given in Refs. 5, 9 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 25.2 kg/mm\(^2\) (36 ksi) to 70 kg/mm\(^2\) (100 ksi) did not exhibit significantly different fatigue strength for a given welded detail.
4. The logarithmic transformation of cycle life resulted in a normal distribution of the test data at nearly all levels of stress range for each welded beam detail.

5. A theoretical stress analysis based on fracture mechanics for macrocrack propagation substantiated the experimental model that provided the best fit to the test data.

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, Pa. and the Department of Civil Engineering, Drexel University, Philadelphia, Pa. Lehigh University was the contractor for this study which commenced in 1966.

The preparation of this paper was supported by the Office of Naval Research, Department of Defense under contract N00014-68-A-0514; NRO64-509.

The author is indebted to his colleagues Karl H. Frank and Manfred A. Hirt, Research Assistants and Ben T. Yen, Associate Professor at Fritz Engineering Laboratory for their assistance and work throughout this study. Thanks are also due Fred Schmitt, Research Assistant and 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, IASBE, 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 Project 12-7, Fritz Laboratory Report No. 334.2 Lehigh University, Bethlehem, Pa., September 1969

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

9. FRANK, K. H. and FISHER, J. W.
The Fatigue Strength of Welded Coverplated Beams, Fritz Lab Report 334.1, Lehigh University, Bethlehem, Pa., March 1969

10. GURNEY, T. R.

11. REEMSNYDER, H. S.
12. HIRT, M. A. and FISHER, J. W.
The Fatigue Strength of Rolled and Welded Steel I-Beams,
Fritz Laboratory Report No. 334.3, Lehigh University,
Bethlehem, Pa., March 1970

13. PARIS, P. C.
The Fracture Mechanics Approach to Fatigue, Proc. 10th Sagamore
Conf., Syracuse Univ. 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

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

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
### TABLE 1

#### a. EXPERIMENT DESIGN FOR COVER-PLATED BEAMS

<table>
<thead>
<tr>
<th>$S_{min}$ kg/mm² (ksi)</th>
<th>Stress Range $S_r$</th>
<th>kg/mm² (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>5.6 (8)</td>
<td>CRC131</td>
</tr>
<tr>
<td></td>
<td>8.4 (12)</td>
<td>CRC132</td>
</tr>
<tr>
<td></td>
<td>11.2 (16)</td>
<td>CRC133</td>
</tr>
<tr>
<td>-4.2 (-6)</td>
<td></td>
<td>CWC131</td>
</tr>
<tr>
<td>1.4 (2)</td>
<td></td>
<td>CWC221</td>
</tr>
<tr>
<td></td>
<td></td>
<td>CRC231</td>
</tr>
<tr>
<td></td>
<td></td>
<td>CRC232</td>
</tr>
<tr>
<td>7.0 (10)</td>
<td>CRC311*</td>
<td>CRC311</td>
</tr>
<tr>
<td></td>
<td>CRC321</td>
<td>CRC321</td>
</tr>
<tr>
<td></td>
<td>CWC312</td>
<td>CWC332</td>
</tr>
<tr>
<td></td>
<td>CWC313</td>
<td>CWC333</td>
</tr>
<tr>
<td></td>
<td>CRC324</td>
<td>CRC334</td>
</tr>
</tbody>
</table>

* CR = Cover Plate on Rolled Beam
  CW = Cover Plate on Welded Beam

#### b. EXPERIMENT DESIGN FOR PLAIN WELDED BEAMS

<table>
<thead>
<tr>
<th>$S_{min}$ kg/mm² (ksi)</th>
<th>Stress Range $S_r$</th>
<th>kg/mm² (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>12.7 (18)</td>
<td></td>
<td>PWC131</td>
</tr>
<tr>
<td>16.9 (24)</td>
<td></td>
<td>PWC132</td>
</tr>
<tr>
<td>21.1 (30)</td>
<td></td>
<td>PWC131</td>
</tr>
<tr>
<td>25.3 (36)</td>
<td></td>
<td>PWC141</td>
</tr>
<tr>
<td>29.5 (42)</td>
<td></td>
<td>PWC151</td>
</tr>
<tr>
<td>9.8 (14)</td>
<td>PWC311</td>
<td>PWC311</td>
</tr>
<tr>
<td></td>
<td>PWC321</td>
<td>PWC321</td>
</tr>
<tr>
<td></td>
<td>PWC331</td>
<td>PWC331</td>
</tr>
</tbody>
</table>
Fig. 1 Details of beams with cover plates attached to each flange

- 229 mm (9 in.)
- 14 mm (5/8 in.) for CR, CB & CW Series
- 19 mm (5/4 in.) for CT Series
- 3200 mm (126 in.)
Fig. 2 Crack growth at the transversely welded end of cover-plated beams
Fig. 3. Effect of stress range and minimum stress on the cycle life for the welded end of A514 steel cover-plated beams.
Fig. 4 Effect of grade of steel on the fatigue strength of cover-plated beams.
Stage 1

Stage 2

Fig. 5: Crack growth in welded beams
Fig. 6 Effect of stress range and minimum stress on the fatigue strength of welded beams.
Fig. 7 Effect of grade of steel on the fatigue strength of welded beams
Fig. 8 Mean fatigue strength and 95% confidence limits for welded and cover-plated beams.