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# Fatigue Strength of Riveted Connections

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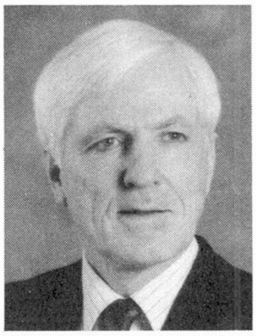

## Summary

The fatigue life behavior of riveted members is a matter of considerable interest, even though new riveted structures have not been built in the past several decades. The life of the large stock of riveted bridges that still exist must be extended, while at the same time maintaining a safe condition. Test results on full-size flexural members and axially loaded members are reported. These test results are compared with the design recommendations of several European and North American standards. Recommendations are presented for a fatigue life classification.

## 1. Introduction

Because new riveted structures have not generally been built in the past several decades, less attention has been paid to their fatigue strength behavior than to structures containing contemporary fastening elements such as bolts or welds. The behavior of riveted members is a matter of considerable importance to owners and regulatory authorities, however. Evaluation of the remaining fatigue life of a riveted structure has been impeded by the lack of a reasonable data base of test results of full-size members. However, recent work in both North America and Europe has addressed this need for better fatigue strength data for this category . Although more data are always welcome, the data base is now sufficiently large to be able to present the foundations for fatigue strength evaluation of riveted members with reasonable confidence.

## 2. Determination of Fatigue Life

### 2.1 Basis of Fatigue Strength Analysis

The fatigue life of any fabricated steel structure can be described using the following three factors:

- the range of stress at the detail under examination;
- the number of cycles of loading at the detail;
- the detail classification.

Although some comments will be made in this paper about the first two issues, attention will be focused on the detail classification, especially as it applies to riveted shear splices.

All fabricated steel structures contain metallurgical or fabrication-related discontinuities, and most also include stress concentrators such as weld toes. Consequently, fatigue failure is usually the result of slow crack growth from an existing discontinuity at a stress concentration. A description of the fatigue crack growth phenomenon can be made on the basis of a fracture mechanics model [1]. However, it is not practicable to use the fracture mechanics approach in design, and almost all design specifications simply arrange standard structural details into categories relative to their expected fatigue life. The expected fatigue life is established on the basis of test results, sometimes aided by fracture mechanics examination and sometimes requiring engineering judgement.

## 2.2 Fatigue Failure in a Riveted Connection

For a riveted connection, the experimental evidence is that in shear splices the great majority of fatigue failures relate to the connected material, not the rivet itself. Thus, the fatigue life can be expected to be a reflection of such features as the size of the hole relative to the part, the method of hole forming (drilled, punched, or sub-punched and reamed), the bearing condition of the rivet with respect to the hole, and the clamping force provided by the rivet group.

At the present time, the influences of clamping force, bearing condition, and the method of hole formation have not yet been examined in any systematic way. The influence of the hole size, *per se*, is not likely to be a strong one as long as the hole sizes and plate thicknesses normally used in structural practice pertain. Thus, the best data available are tests on riveted connections of proportions that are consistent with usual structural practice and are of full size, or at least large size. For the time being, the effects of clamping force, bearing condition, and hole formation must simply be part of the data pool. For this reason, and because the "defect" presented by a riveted connection is not severe, it is to be expected that the scatter of data will be relatively large.

## 2.3 Treatment of Riveted Details in Representative Standards

Not all specifications address the fatigue life evaluation of riveted connections. The short review that follows is not intended to be comprehensive, although it is believed to be representative.

**Eurocode 3** [2]— Riveted shear splices are not mentioned in this standard. If it is assumed that a riveted shear splice is the same as a splice that uses non-preloaded high-strength bolts, then Eurocode 3 prescribes the use of Detail Category 112. The slope of the fatigue life vs. number of cycles is taken as 3 until the constant amplitude fatigue limit is reached (which is at 5 million cycles). For Detail Category 112, this corresponds to a stress range of 83 MPa. A slope of 5 can then be used until the cutoff limit is reached (100 million cycles, 45 MPa). The Eurocode rule is plotted in Fig. 1, together with the test data that will be described later.

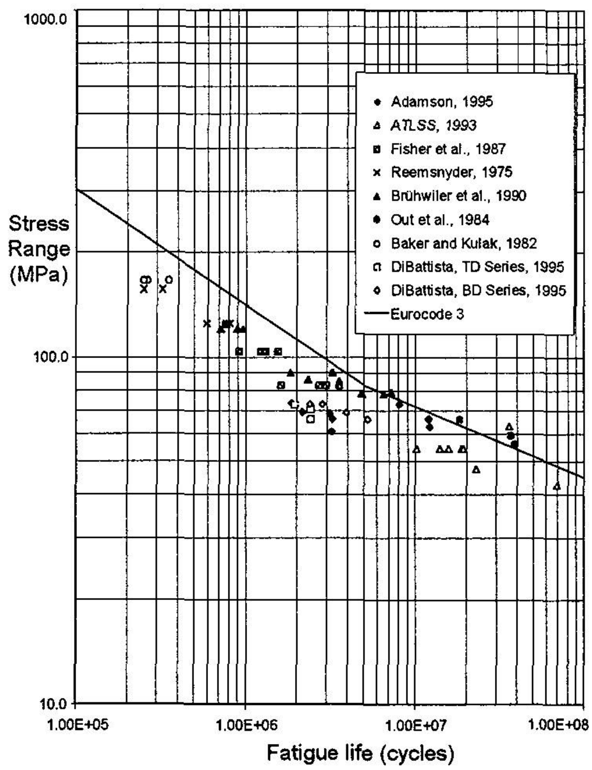


Fig. 1 Test Results + Eurocode 3

**British Standard** (BS5400, Part 10) [3] — With the advent of the EUROCODES, development of national standards in Europe has ceased. However, riveted shear splices are specifically mentioned in BS5400, and it is likely that the information is still being used. The riveted shear splice detail is described in BS5400 as their Class D. A slope of 3 is used until the constant amplitude stress range of 53 MPa is reached at 10 million cycles. If variable amplitude cycles are present in this low stress range region (likely the case with a bridge), then a slope of 5 is used for all stress ranges less than 53 MPa. Figure 2 shows the permissible fatigue life rule and the test data.

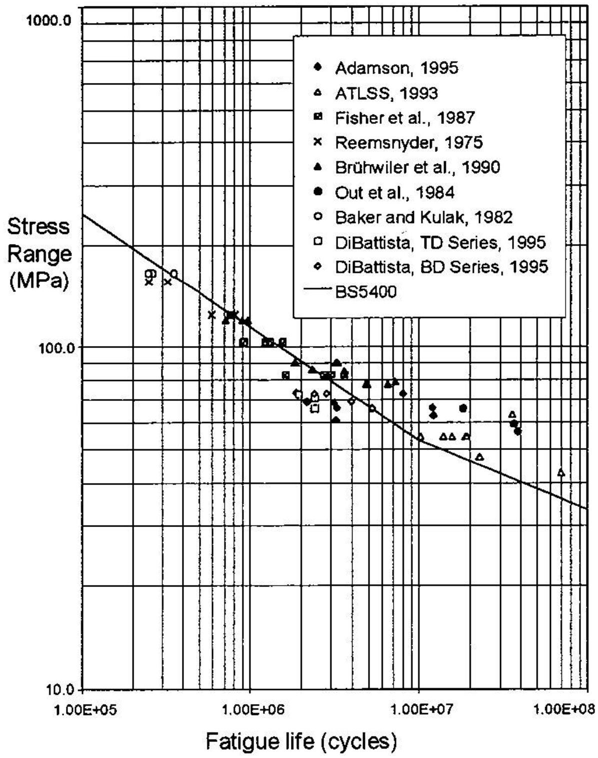

Fig. 2 Test Results + BS5400

**American Association of State Highway and Transportation Officials** (AASHTO) [4] — Riveted shear splices are designated as AASHTO Category D. The stress range vs. number of cycles relationship has a slope of 3 and the constant amplitude stress range is 48 MPa. If the effective stress range is below the variable amplitude fatigue limit (17.7 MPa), then it is assumed that no fatigue damage occurs for these stress ranges. Figure 3 shows the AASHTO permissible fatigue life rule and the test data.

**American Railway Engineering Association** (AREA) [5] — This standard is widely used by railroads in North America. Of the standards reviewed, it is the one that most closely reflects the recent test data, which are reviewed later in this paper.

The rules in the AREA Manual distinguish between cases in which it can be identified that the rivets are tight and have developed a normal level of clamping force and those situations in which the slip resistance is deemed to be low. No advice is given as to how the tightness is to be determined, however. If the decision is made that the slip resistance is low, then AREA Category D is to be used. At 2 million cycles, the permissible stress range for this category is 71 MPa. The slope of the fatigue life line is to be taken as 3 until the stress level of 41 MPa is reached, which occurs at 10 million cycles. Figure 4 shows the Category D line and the test data.

It would be more common to evaluate the fatigue life on the assumption that the level of clamping force in the rivets can be taken as "normal." In this option, the AREA rules provide two categories. When the holes might have been punched, the fatigue life is that of Category D (71 MPa permissible stress range at 2 million cycles) for any stress range above 62 MPa. For stress ranges just below 62 MPa, the fatigue life line is then shifted laterally until it hits the Category C line, after which it moves downward on that line until the variable amplitude fatigue limit of 41 MPa is reached. This is shown in Fig. 5. If the holes have either been drilled or were sub-punched and then reamed, a further improvement is available. This occurs in the region 53 MPa down to 41 MPa. See Fig. 6.

The AREA rules simply capture the boundary of the test data, as can be seen in these figures. The lengths to which this standard has gone to provide rules that are as close to the test data as possible reflects the importance of this region to owners: a large portion of the stress ranges experienced in North American rail operations are in the region of 40 to 70 MPa. For proper

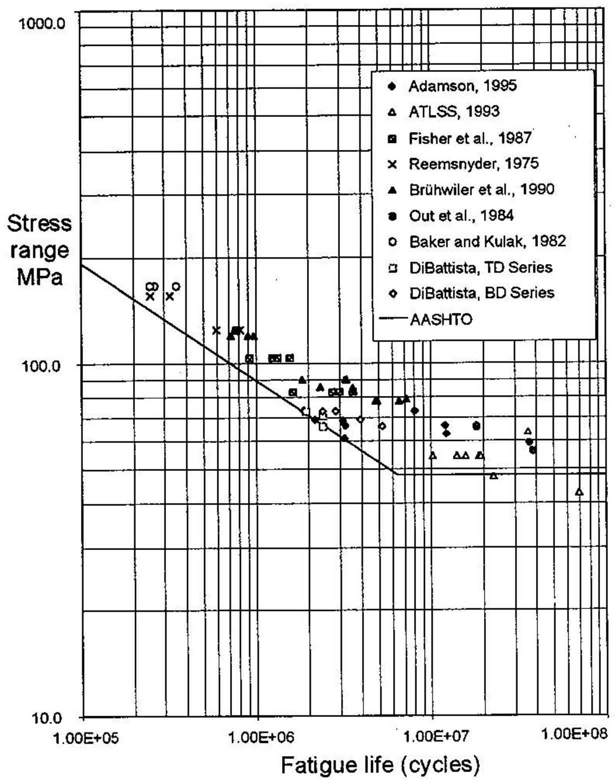


Fig. 3 Test Results + AASHTO

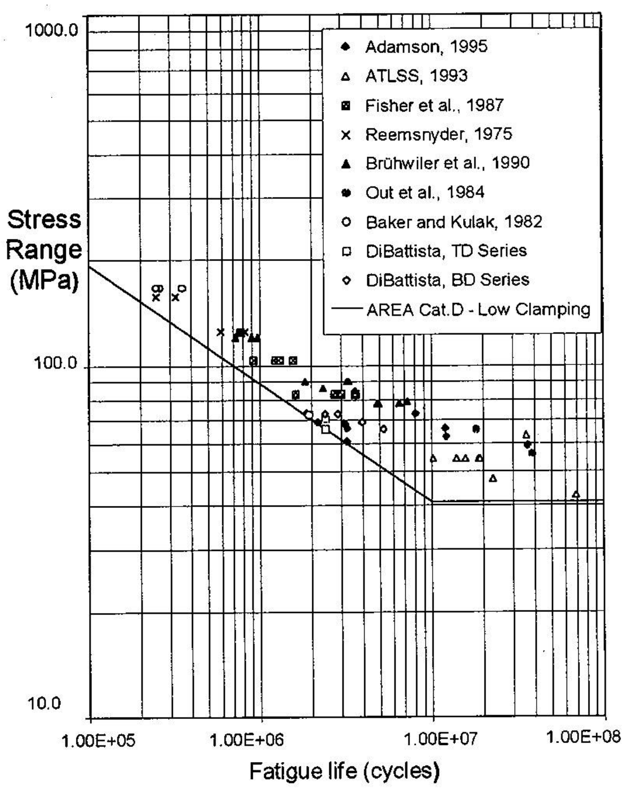


Fig. 4 Test Results + AREA, Low Clamping

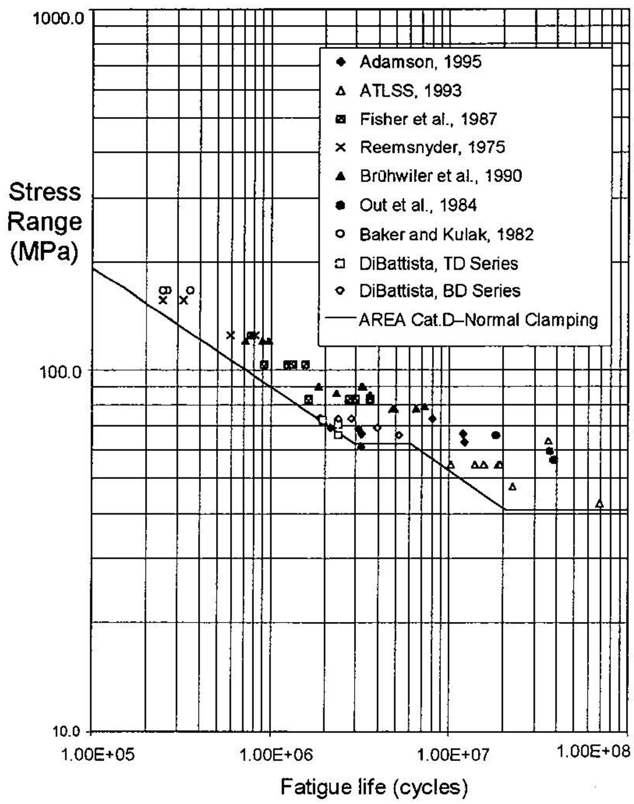


Fig. 5 Test Results + AREA, Normal Clamping

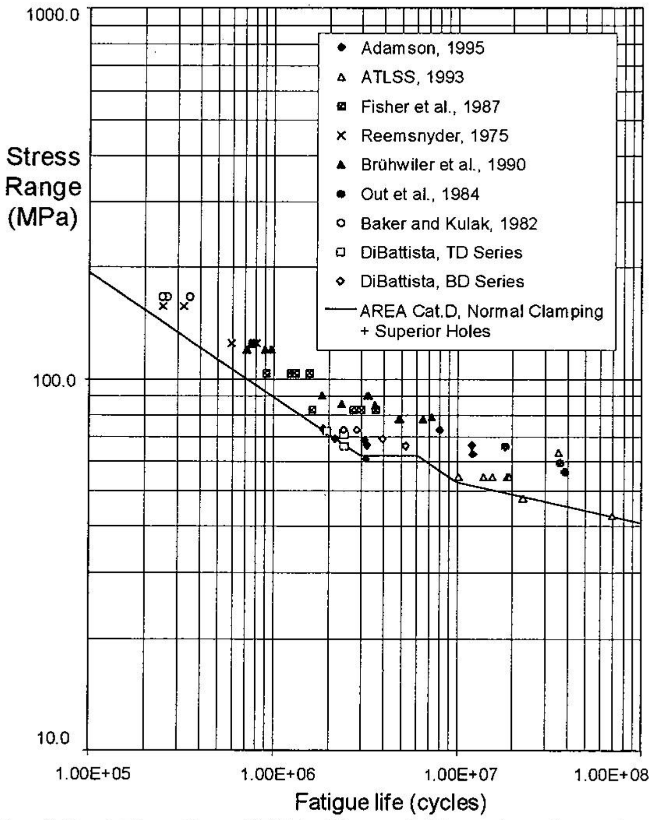


Fig. 6 Test Results + AREA, Normal Clamping, Superior Holes

evaluation of remaining fatigue life of existing bridges, it is of great importance to maximize the detail category in this region, while at the same time maintaining a suitable margin of safety.

In reviewing any of the design guidelines given in the foregoing summary, it is important to remember that the other components of a fatigue life evaluation—calculation of stress cycles, loading history, stress counting methods, and so on—are not necessarily the same in the standards cited.

# 3. Fatigue Test Data for Riveted Shear Splices

## 3.1 Fatigue Life Data

The fatigue life data shown in Figs. 1 to 6 (the data are identical in each figure) come from a variety of sources. In most cases, the members tested were taken from a structure that had been in service. Most specimens were tested in constant amplitude fatigue. All specimens were full-size. Details of the actual test conditions and member configuration must be obtained from the original source material. A brief description of the tests follows.

- Reemsnyder 1975 [6]: Full-scale fatigue tests on truss chords that had riveted gusset plate connections. The critical detail was that of the connection between the tension chord and a gusset plate.
- Baker and Kulak 1982 [7]: Full-scale fatigue tests on built-up hanger members. The arrangement was such that the rivets were not subject to any shear force, and thus the bearing stresses in the connected material were negligible.
- Out *et al.* 1984 [8]: Flexural tests on built-up railway stringers (beams) in which flange angles were riveted to a web plate. The critical detail was the continuous riveted connection between the web and the flange angles, and it was located in a region of constant moment.
- Fisher *et al.* 1987 [9]: Flexural tests on built-up railway stringers (beams) in which flange angles were riveted to a web plate. The configuration was similar to that of Out *et al.* except that a coverplate riveted to the outstanding leg of the flange angles was also present.
- Brühwiler *et al.* 1990 [10]: Tested riveted built-up plate girders, riveted lattice girders, and rolled mild steel girders that had a cover plate riveted to the flanges.
- ATLSS 1993 [11]: Flexural tests on built-up railway stringers (beams) in which the flange angles were riveted to the web plate. The critical detail was the continuous riveted flange-to-web connection.
- Adamson and Kulak 1995 [12]: Flexural tests on built-up railway stringers (beams). The configuration was similar to that described in the ATLSS 1993 program.
- DiBattista and Kulak 1995 [13]: Tension tests on diagonals taken from a railway truss bridge. The designations TD and BD refer to the connection at the top or the bottom of the member and reflect the way the net area is calculated. See the original source material. The cross-section of the member consisted of four angles connected to a web plate. The critical detail was the connection of the outstanding legs of these angles to gusset plates.

## 3.2 Evaluation of Fatigue Life Rules

Figures 1 through 6 contain the test data deemed to be most relevant for evaluation of the fatigue life of riveted shear splices and the individual fatigue life rules. The following observations can be made.

**Eurocode 3** —Use of a standard that does not specifically mention riveted shear splices can be criticized as inappropriate. However, there are a number of standards in force that do not mention riveted connections, and it may be that a decision has to be made by the designer as to a suitable category. In this case, Detail Category 112 appears to be a reasonable choice based on the information in the standard. Since the failure mode anticipated is cracking in the net section of the connected material, the strength of the fastener (i.e., rivet versus bolt) is not relevant. Non-preloaded high-strength bolts, which are specifically permitted by this standard, are in principle no different than a rivet with respect to fatigue of the connected material. Thus, it appears that selection of this detail category for use with a riveted shear splice is reasonable within Eurocode 3.

From the information in Fig. 1, it is obvious that use of Eurocode Detail Category 112 is not suitable for use as a predictor of the fatigue strength of riveted shear splices. The prediction line greatly overestimates the fatigue life.

**British Standard** (BS5400, Part 10) — Unlike the Eurocode 3 standard, this standard does distinguish between riveted shear splices and bolted shear splices. However, Fig. 2 shows that the fatigue life rule provided by this standard for riveted shear splices also is not satisfactory.

**AASHTO** — This standard also provides fatigue life rules for each of bolted and riveted shear splices. As seen in Fig. 3, the predictor line is a reasonable boundary to the test data, except for two data points at long lives (approximately 20 and 70 million cycles). It was noted earlier that the variable amplitude fatigue limit is given as 17.7 MPa. If that limit were to be used, the rule would capture these data, but it would be too conservative. Alternatively, it has been suggested (unpublished correspondence) that the AASHTO Category D line should be extended downward to a fatigue limit line at 41 MPa. This would be a reasonable choice, and it is the solution presented in the AREA rules, described following, for the case where slip resistance is deemed to be low. (Figure 4, which relates to the AREA rules, shows how such a fatigue life line would look relative to the test data.)

**AREA** — It has been described earlier that the AREA rules offer three choices for fatigue life of riveted shear splices. If the clamping force provided by the rivets is considered to be low, then their Category D is used, along with a fatigue limit line at 41 MPa (Fig. 4). All of the test points lie on or above this line. It is therefore a satisfactory solution, except that it might be that it is too conservative in the region of the knee of these two lines. More tests in this region would be helpful. Another solution, which will be discussed later, is to provide a transition fatigue life line between the two parts of the bi-linear fatigue life prediction shown in Fig. 4.

The second choice offered by the AREA specification is depicted in Fig. 5. This case is described as applicable to conditions of "normal" clamping, and it is also intended to be used when it can only be assumed that the holes were punched. The fatigue life line shown in the figure has been selected to generally skirt the test data. It can be criticized on the basis that the prediction might be too generous in the region between, say, 4 million and 20 cycles. (Small changes in the placement of the predicted curve in this region can have a large impact on the fatigue life predicted.) As observed earlier, more test data in this region (at stress ranges less than about 60 MPa) are desirable. The possibility of using a transition curve can also be considered here.

The other choice available in the AREA specification is shown in Fig. 6. It is intended for use when it is known that the holes were either drilled or sub-punched and reamed. This prediction follows the lower boundary data points even more closely than the one shown in Fig. 5.

## 4. Issues Arising from the Examination of Rules and Data

### 4.1 Failure Criterion

A review of various experimental data reveals that not all researchers used the same criterion to define when fatigue failure of the member had taken place. Some reported the number of cycles to failure at a certain deflection criterion, some used the time at which the loading system was no longer stable, and some used cracking of an element as the basis for reporting fatigue life. In the last case, cracking could be the first detection of a crack or some later stage of cracking.

If the number of cycles is large, there might not be much difference between the various criteria. However, for shorter lives the distinction could be important, and of course it is generally desirable that all similar research results be reported on the same basis. The author suggests that cracking be used as the basis of reporting fatigue life. Based on the tests done by the author and his colleagues [12, 13], it is suggested that the fatigue life be reported as the number of cycles at the time the first crack has severed the element in which it appeared and a new crack has just started in a second element. For a tension member, the work of DiBattista and Kulak [13] showed that there will be relatively few cycles between this point and complete destruction of the member.

In riveted flexural members there will be more time between the point at which the failure criterion is reached and complete destruction because the cross-section is not uniformly stressed, as it is in the case of a tension member. As the crack grows, say from a bottom flange angle, the stress range will tend to decrease because the crack is moving upward in the member, but it will tend to increase because the amount of cross-section available to carry the forces is reduced. Overall, the failure criterion suggested seems to work well in these cases [12].

The proposed failure criterion was used in plotting the data in Figs. 1 to 6 whenever possible. This included at least the results from ATLSS [11], Adamson and Kulak [12], and DiBattista and Kulak [13].

### 4.2 Tension Members vs. Bending Members

The discussion about failure criteria has pointed out that the presence of a fatigue crack in a tension member is likely to be more serious than one in a flexural member. At the present time there are not enough data in each category to be able to separate the two groups, however. For the time being, all the data must be used together, but the consequences of fatigue cracking in each of these two different types of members should be kept in mind.

### 4.3 Effect of Staggered Hole Patterns

The limited amount of experience available shows that fatigue cracking will take place more or less orthogonally to the direction of the stress, even in the presence of a staggered hole pattern. This suggests that a cross-sectional area normal to the direction of the tensile stress in the member be used for the calculation of the stress range. However, it is reasonable to expect that the effect of staggered holes will be to increase the stress range that is driving the crack. One alternative then is to account for the effect of staggered holes as is done for static loading. Another option is simply to deduct both the holes in the plane of the crack and the full effect of the "staggered" holes, assuming that they are reasonably close to those in the plane of the crack. This was the option selected in calculating the stress range for the TD Series of DiBattista. (This means that the TD Series of DiBattista plot higher than they would using either of the other two alternatives. In Fig. 3, for example, the three TD points would plot below the fatigue life line of AASHTO if either of the other criterion were used.) The problem of just how to calculate the stress range in the presence of staggered hole patterns requires further investigation.

### 4.4 Criterion for Selection of a Fatigue Life Limit

It has been customary in selecting a fatigue life limit to use the mean less two standard deviations of the data (on the number of cycles). If the number of data is reasonably large, this means that the confidence limit is approximately 95%. The scatter in the riveted splice data is much greater than usually seen with welded details, however. So far, it has been usual to select a fatigue life line that is more or less at the lower limit of the data, such as has been done in the AREA standard [5]. There seems to be no reason to change this approach at the present time.

### 4.5 Improvement of the Fatigue Life Limit Selection Process

The bi-linear fatigue life representation used in most standards is a reflection of the simplification usually applied to the crack growth model, which is assumed to be bilinear. Kunz [14] has proposed that a transition be recognized between the crack growth region described by the Paris law and the threshold stress intensity value. Application of this produces a similar transition between the sloping and horizontal portions of traditional fatigue life curves. This provides a better representation of test data in the region of the "knee" of the traditional fatigue life curves. This idea could be easily incorporated in a design standard and it should be explored further.

Kunz [14] has also proposed that the limit below which no crack propagation occurs is not a constant, but decreases with increasing damage (crack growth). This fracture mechanics approach to forecasting fatigue life offers the possibility of improved predictions, particularly when variable amplitude stress cycles are present. This, too, should be explored further.

## 5. Summary and Conclusions

The selection of a fatigue life category for riveted shear splices can now be made on the basis of a reasonable amount of data derived from full-size tests. The review of a representative number of existing design rules has shown that improvement is needed in some cases. The author makes the following recommendations:

- A suitable fatigue life category for riveted shear splices is that shown in Fig. 4.
- An improvement to this selection in the important region of the knee of the bi-linear representation can be made using the work of Kunz [14].
- The fatigue life of riveted shear splices should be reported as the number of cycles at the time the first crack has severed the element in which it appeared and a new crack has just started in a second element.

The author also makes the following suggestions for research that will improve the understanding of fatigue life predictions for riveted shear splices:

- The influence of bearing stress upon fatigue life.
- The influence of rivet clamping force.
- The effect of staggered hole patterns on fatigue crack growth.
- The influence of method of hole forming upon fatigue life.

## References:

1. Broek, D. (1989). *The Practical Use of Fracture Mechanics*. Kluwer Academic Publishers, London.

2. European Committee for Standardisation (1992). *Eurocode 3: Design of Steel Structures*, ENV 1993-1-1, Brussels.

3. British Standard BS 5400, Steel, Concrete and Composite Bridges (1980). *Part 10, Code of Practice for Fatigue*. British Standards Institution, London.

4. American Association of State Highway and Transportation Officials (AASHTO) (1992). *AASHTO LRFD Bridge Design Specifications*, SI Units, First Edition, Washington, D.C., 1994.

5. American Railway Engineering Association (AREA) (1996). *Steel Structures*. Chapter 15, Manual for Railway Engineering.

6. Reemsnyder, H.S. (1975). *Fatigue Life Extension of Riveted Connections*. Journal of the Structural Division, ASCE, Vol. 101, No ST12, pp. 2591-2608.

7. Baker, K. A. and Kulak, G. L. (1982). *Fatigue Strength of Two Steel Details*. Structural Engineering Report No. 105, Department of Civil Engineering, University of Alberta, Edmonton, Canada.

8. Out, J.M.M., Fisher, J.W., and Yen, B.T. (1984). *Fatigue Strength of Weathered and Deteriorated Riveted Members*. Transportation Research Record 950, Transportation Research Board, National Research Council, Washington, D.C., pp. 10-20.

9. Fisher, J. W., Yen, B.T., Wang, D., and Mann, J.E. (1987). *Fatigue and Fracture Evaluation for Rating Riveted Bridges*. National Cooperative Highway Research Program Report 302, Transportation Research Board, National Research Council, Washington, D.C.

10. Brühwiler, E., Smith, I.F.C., and Hirt, M.A. (1990). *Fatigue and Fracture of Riveted Bridge Members*. Journal of Structural Engineering, ASCE, Vol. 116, No. 1, pp. 198-214.

11. ATLSS (Center for Advanced Technology for Large Structural Systems) (1993). *Assessment of Remaining Capacity and Life of Riveted Bridge Members*. Draft Project Report to Canadian National Railways, Lehigh University, Bethlehem, Pennsylvania.

12. Adamson, D.E. and Kulak, G. L. (1995). *Fatigue Tests of Riveted Bridge Girders*. Structural Engineering Report No. 210, Department of Civil Engineering, University of Alberta, Edmonton, Canada.

13. DiBattista, J.D. and Kulak, G.L. (1995). *Fatigue of Riveted Tension Members*. Structural Engineering Report No. 211, Department of Civil Engineering, University of Alberta, Edmonton, Canada.

14. Kunz, P. (1992). *Probabilistic Method for the Evaluation of Fatigue Safety of Existing Steel Bridges*. Doctoral Thesis No. 1023, ICOM – Steel Structures, Swiss Federal Institute of Technology, Lausanne, Switzerland.

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