Fatigue behaviour of field-welded rib joints in orthotropic steel bridge decks

Fatigue Behaviour of Field-Welded Rib Joints in Orthotropic Steel Bridge Decks

Comportement à la fatigue des connexions de raidisseurs soudés dans des dalles orthotropes de ponts en acier

Ermüdungsverhalten in situ geschweisster Rippen orthotroper Stahlfahrbahnplatten

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SUMMARY

This paper describes the results of several research programs related to fatigue behaviour of field-welded rib joints in othotropic steel bridge decks. Constant as well as variable amplitude tests were carried out on several connections. Eurocode 3 fatigue design lines are considered. Fatigue cracks growing from welded details in an existing bridge and subsequent activities are reported. Finally, a fatigue design method which uses standard design curves and influence lines related to the static design load is introduced.

RÉSUMÉ

Cet article présente les résultats de plusieurs programmes de recherche relatifs au comportement à la fatigue des connexions de raidisseurs soudés dans des dalles orthotropes de ponts en acier. Des essais de fatigue sous amplitudes constante et variable ont été effectués sur différents types de connexions. Les courbes de fatigue de l'Eurocode 3 ont été considérées. Des fissures de fatigue apparues dans des détails soudés de ponts existants sont présentées, ainsi que les actions entreprises pour y remédier. Finalement, une méthode de calcul utilisant les courbes de Wöhler standardisées et les lignes d'influence relatives au modèle de charge statique est introduite.

ZUSAMMENFASSUNG

Dieser Artikel stellt die Resultate verschiedener Forschungsprogramme vor, die das Ermüdungsverhalten auf der Baustelle geschweisster Rippen orthotroper Fahrbahnplatten von Stahlbrücken betreffen. Verschiedene Arten solcher Verbindungen wurden sowohl unter konstanter als auch variabler Amplitude getestet. Dabei wurden die im Eurocode 3 enthaltenen Ermüdungsfestigkeitskurven berücksichtigt. Über Ermüdungsrisse, die in bestehenden Brücken von geschweissten Konstruktionsdetails ausgehen, wird ebenso berichtet, wie über die in solchen Fällen zur Auswahl stehenden Instandstellungsmassnahmen. Schliesslich wird eine Methode für die Ermüdungsbemessung vorgestellt, basierend auf normierten Wöhlerkurven und Einflusslinien in Verbindung mit den statischen Bemessungslasten.

1. INTRODUCTION

In most large orthotropic steel bridge decks, field splices are necessary because transportation of the complete bridge from the shop to the site is seldom possible. In general, both longitudinal and transverse field splices have to be made. In the longitudinal splices, only the deck plate has to be connected. This is mostly done by butt welding and as the weld is accessible both from above and from below, a good quality of the weld can be achieved. The same applies to the transverse butt splice weld in the deckplate. At the transverse splice the longitudinal ribs have to be connected as well.
With closed ribs, mostly used in modern bridges, the most appropriate way of splicing is by welding but as the welds can only be made from the outside in an unfavourable overhead position, the quality of those welds will be dubious. Depending on the location of the splice in the deck, the load on the splice can have a fluctuating part due to the traffic load, dominating the static loading, so conditions for fatigue damage are present.
As the splice in the ribs frequently occurs in a bridge deck, an investigation of the fatigue behaviour of field splices is worth while. Besides cracks were discovered in several field welded splices in a rib of a bridge which was 15 years in duty.

2. FATIGUE TESTS ON FULL SCALE TEST SPECIMENS

In this paper, the results of the fatigue tests are plotted in a S-N-diagram on a log-log scale. The regression mean-line minus twice the standard deviation is used to make comparisons with the fatigue design curves, defined by the Eurocode 3.

2.1. Constant amplitude tests

2.1.1. Dutch research

In 1974 [1], a research programme was set up to investigate the design of field splices in orthotropic steel bridge decks with respect to economy and fatigue strength. Laboratory tests were conducted at IBBC-TNO and at the Stevin Laboratory. The specimens for the bending tests were single trapezoidal rib specimens 2/325/6, steelgrade Fe 510. Three types of field splices were selected, as shown in figure 1.

- Type A : Butt splice with back-up strips (4 mm root gap)
- Type B : Lap splice with fillet welds
- Type C : Butt splice with a thick joint plate.

The conclusion (fig.1) can be drawn that the fatigue behavior of type C is the best (EC 125) and of type B is the worst (EC 56), with type A in between (EC 80). Nevertheless, with respect to type C, it must be noted that apparently this type is very sensitive to the quality of the workmanship, for the first series of specimens made with less care gave very bad results. That is one of the reasons why type A is often used.

As all tests were carried out at high stress range levels ($\geq$ 120 MPa), where the shape of the S-N curve amounts 3, no information is available at the lower stress regions and on the fatigue limit. This stress range region is very important, for measurements carried in ECSC research [2,3] showed for this type of connection a maximum stress range of $\pm$ 80 MPa.

In 1986 [4], an ECSC research program started at the Stevin Laboratory in which among other things, constant amplitude tests were carried out on the type A detail at stress ranges beyond 2.10^{6}-3.10^{7} cycles. Besides, the effect of the root-gap of the weld has been examined by varying the gap between the stiffener and the splice in 0, 2 and 4 mm. Furthermore tests were executed with an improved form of the weld (V-groove). The fatigue results are gathered in figure 2.

For the specimens with a root gap of 4 mm it appears that, the specimens tested at stress ranges > 120 MPa fall within the spread band of the previous tests [1]. At 105 MPa the fatigue strength is far more than one would expect. The fatigue cracks appeared over 9 million cycles instead of about 3 million cycles. The specimen tested at a stress range level of 90 MPa did not fail after almost 30 million cycles.
Comparing the results it appears furthermore, that a weld with a root gap of 0 or 2 mm can result in a fatigue strength far below the strength of the weld with a root gap of 4mm (a factor 12 - 18).
Changing the weld geometry by using a V-groove did not give the expected improvement.

2.1.2. Comparison with foreign research

In 1988 an Japanese IIW document [5] was published which contained, analogous to the Dutch researches [1,4], fatigue results on trapezoidal ribs. Also in this research, a great influence of the largeness of the root gap on the fatigue strength was found. Summing up these two test programmes, it can be concluded that Class 71 according the Eurocode 3 can be recommended for field splices type A, with a root gap ≥ 3 mm (fig.3a). If the root gap < 3 mm the classification decreases to a Eurocode Class 36 (fig.3b). Naturally it is better to avoid the last situation, but it is known that some of the existing bridges contain the type A detail with very small root gaps.

Simultaneous to the Dutch ECSC research [4] the Italian partners in this joint ECSC research investigated the fatigue behaviour of field splices in ribs of orthotropic decks [6]. They studied a triangular shape of the trough instead of a trapezoïdal one. Following details were tested (fig.4a):
- Butt splice using a V-groove with a root gap of 6 mm and a backing strip.
- Butt splice using a X-groove with a root gap of 4 mm, without backing strip.

From the results as given in figure 4a it can be concluded, that the V-groove specimens conducted better than the X-groove type. In both cases the behaviour is better than the Dutch [1,2] and Japanese [3] researches. However it must be noticed that all the welding of the Italian research have been checked by means of visual and magnetic controls. The butt welding, moreover, have been 100% X-rayed and repaired if necessary.

A triangular shape was also studied in the United Kingdom [7] in 1982. The following details were tested (fig.4b):
- Butt splice with backing strip and a root gap of 12 mm.
- Three series of butt splices with a sealing plate of 19 mm. The welding procedure as well as the influence of stress relieving was studied.

From the results as given in figure 4b it can be concluded that the fatigue strength of these testspecimens is comparable with those of the Italian research. Besides, the behaviour of the welded connections with a sealing plate of 19 mm is better than the one with a backing strip (Type E).

Gathering the Italian and UK-tests together, it can be noticed that perhaps for the triangular shapes containing a weld with backing strip and a root gap of 6 - 12 mm, a Eurocode Class 112 can be considered (fig.4c).

2.2. Variable amplitude tests

In above mentioned Dutch ECSC programme [4], also some variable tests were carried out. Special attention was performed to long life (low stress) fatigue results and testing the Miner summation by comparing variable and constant amplitude tests.

2.2.1. Load-spectra for the variable amplitude test.

Axleload-spectra and stress-spectra measured in the ECSC-research [2,3] together with the computer assessment programme of the University of Liege, were used. For the computer simulation , the traffic flow of the Rheden Bridge and a theoretical influence line were used to simulate a test-load spectra for the field splice in a longitudinal rib. Analysis of the simulated spectra showed, that the low stress ranges cause only 7% of the total damage of the spectra. These stress range classes amounts however 84 % of the total number of cycles (fig.5a). So leaving out these classes, resulted in a 'reduced spectrum' saving a lot of testing time without attacking the potential fatigue damage of the spectrum (fig.5b).
Besides the above mentioned simulated test-load spectrum, for one of the specimens a measured stress-spectrum was used. This spectrum (fig.6) was measured on the the Forth Bridge in the United Kingdom [2,3]. Considering the constant amplitude tests the stress range level of the actual measured and simulated were raised to a higher level for the variable amplitude tests.

2.2.2. Analysis of the spectra using Miner

To compare the variable amplitude test results with the constant amplitude ones, the applied stress-spectra were analyzed in two different ways.

a. An equivalent stress range $\Delta\sigma_e$ was calculated in a way that n-cycles of that stress range have the same fatigue damaging potential as n-cycles of the stress-spectrum, using a third power relationship;

$$\Delta\sigma_e = \left(\frac{1}{n_i} \Sigma n_i \Delta\sigma_i^3\right)^{1/3} \text{ MPa.}$$

b. The University of Liege proposed [3] the following equivalent stress $\Delta\sigma_m$ and belonging number of cycles n_m;

$$\Delta\sigma_m = \frac{\Sigma n_i \Delta\sigma_i^3}{\Sigma n_i \Delta\sigma_i^4} \text{ MPa} \quad \text{and} \quad n_m = \frac{\Sigma n_i \Delta\sigma_i^3}{\Delta\sigma_m^3}.$$

In these formulae : $\Delta\sigma_i$ = individual stress range

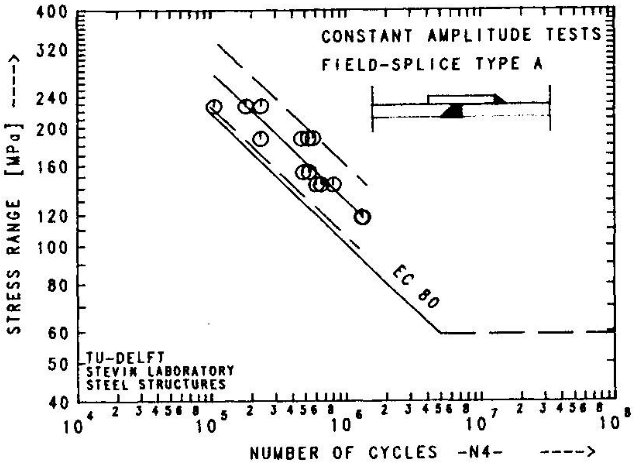
n_i = individual number of cycles belonging to $\Delta\sigma_i$

$\Delta\sigma_m$ = equivalent stress range

n_m = number of cycles belonging to $\Delta\sigma_m$.

In this paper both methods are be used (fig. 5 and 6).

The fatigue results of the variable amplitude tests are presented in figure 7. It appears, that there is a very good agreement between the constant amplitude tests and the variable amplitude tests using the Miner calculation. Furthermore no cracks were found in the specimen with a maximum stress range of the spectrum of 132 MPa after testing for 48 million cycles. Comparing the results of the tests applying the "measured-stress-spectrum" and the "simulated stress-spectrum", the difference seems to be very small.



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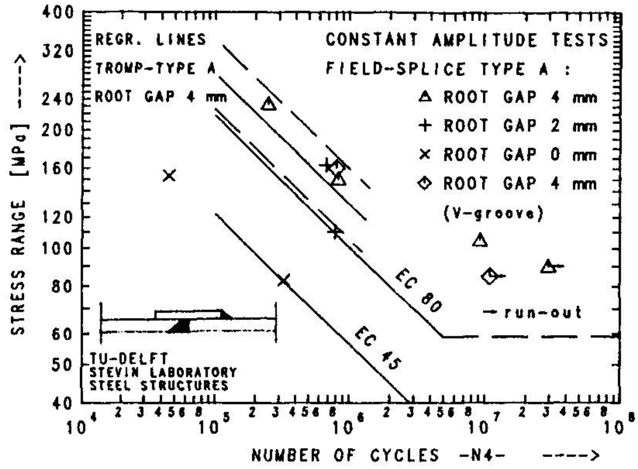
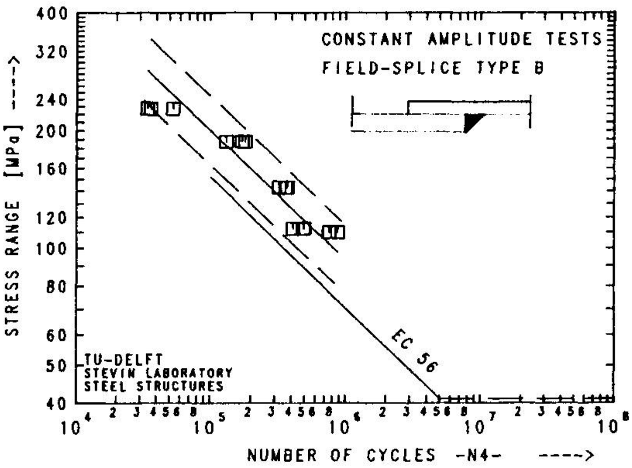


Test results Kolstein [Ref.4.]
Figure 2.



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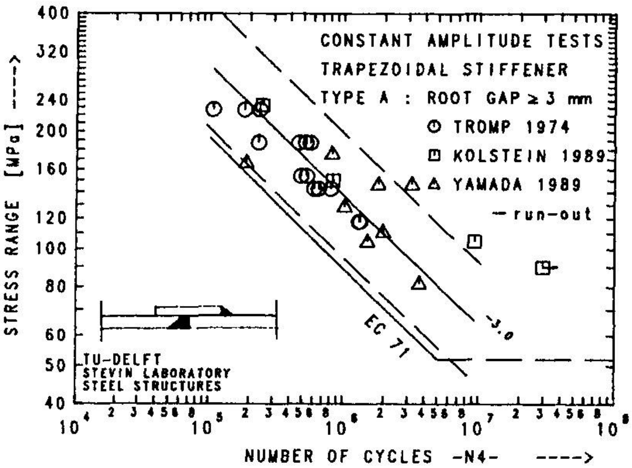

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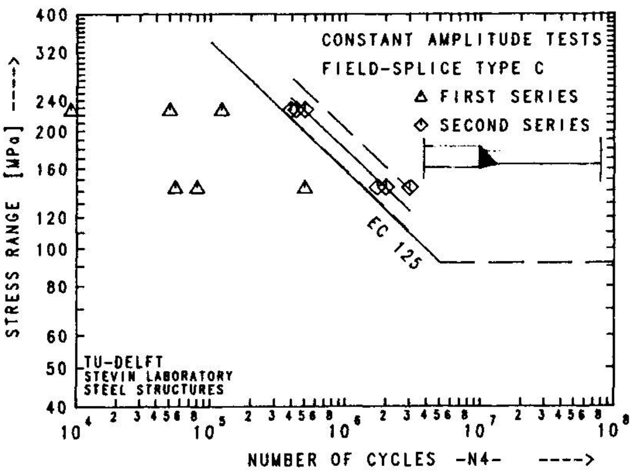

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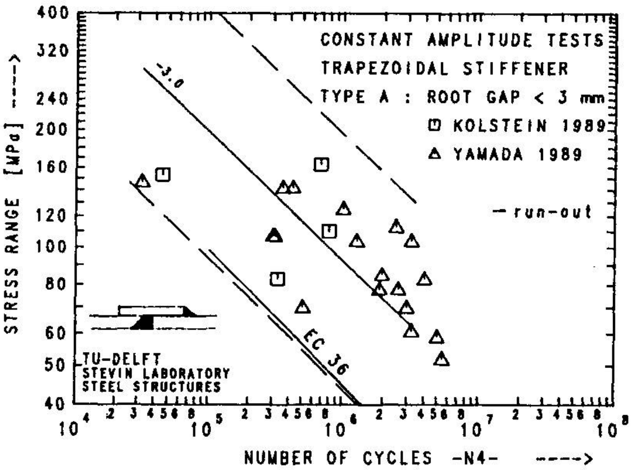
Test results by Tromp [Ref.1].
Test results by Kolstein [Ref.4].
Figure 1.



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Comparison test results; Tromp, Kolstein and Yamada [1,4,5].
Figure 3.

3. FATIGUE CRACKS IN AN EXISTING BRIDGE

During the course of routine maintenance work on the Muiden Bridge in 1985, in one of the 68 longitudinal ribs, cracks were found in several field-welded splices. For some of these connections about 70% of the total cross-section was cracked. The rib splice detail is similar to the type-A detail with a root gap of 2 mm. Due to the fact this bridge was used for only 15 years and the specific detail had been used in a lot of other bridges a research was urgently needed.

Calculations and measurements were executed to find reasons for this cracks and the remaining fatigue life of the bridge [8]. The fatigue calculations are made by using at first a derived load spectrum and in the second place a measured stress-spectrum for this detail.

3.1 Traffic loading during 15 years

Since the opening of the Muiden Bridge in 1970, traffic measurements had been carried out. These measurements concerned the counting of the vehicles. Three categories of vehicles are considered:
- B1: Light-weight vehicles (a passenger car, a delivery van, a motor-cycle),
- B2: Rigid commercial vehicles (a bus, a rigid truck without trailer),
- B3: Articulated commercial vehicles (a rigid truck and trailer, a trailer truck).

In the period 1970 - 1985 a total of $1,07.10^7$ vehicles was counted.

Using measurements from ECSC research [2,3], the following distribution of categories B2 and B3 over the fast and slow lane could be derived:

TYPE	TOTAL: $1,07.10^7$	FAST LANE	SLOW LANE
B2	4,2% → $4,5.10^6$	18% → $0,8.10^6$	82% → $3,7.10^6$
B3	2,9% → $3,1.10^6$	23% → $0,4.10^6$	87% → $2,7.10^6$

The number of axle loads ≥ 10 kN (N10) in the Slow Lane amounts 3,5 times the number of trucks B2 and B3, which results in N10 = $2,2.10^7$.

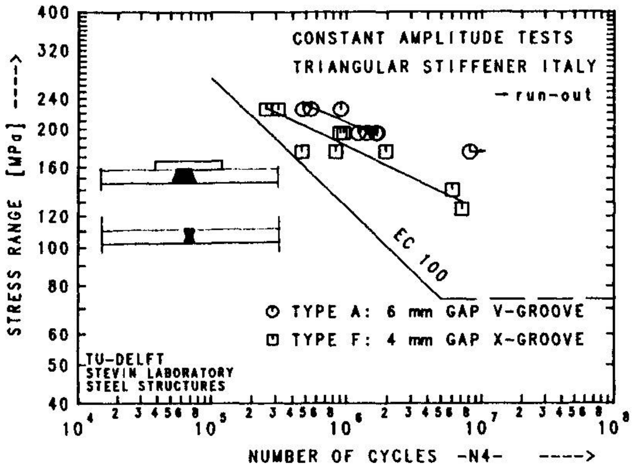
3.2. stress-spectrum of the field splice in a longitudinal rib

Relating the measured axle load spectra and stress-spectra of the Forth Bridge (fig.6) it appeared, that the number of stress range cycles ≥ 10 MPa (NR) is equal to 0,42.N10. That means for the Muiden Bridge a number NR = $0.928*10^7$. Assuming as a first approach that 10 kN wheel load results in a stress range of 15 MPa for the considered field splice, a stress-spectrum based on an load spectrum for the period 1970 - 1985 was determined. For the first fatigue calculation the measured axle load spectrum from the Rheden Bridge was used (SPECTRUM-I). A second fatigue calculation was made using directly the measured stress-spectrum of the Forth Bridge. In this case the number NR is the same as with the first calculation (SPECTRUM-II).
In figure 8 both stress-spectra and the Eurocode S-N curve EC 80 are plotted.

3.3. Calculated fatigue lives

Using Miner the following life-times could be calculated:
- SPECTRUM-I : Load spectrum Rheden Bridge and 10 kN → 15 MPa : 13,3 years,
- SPECTRUM-II: stress-spectrum Forth Bridge : 55,5 years.

The longer fatigue life of the second calculation is perhaps too optimistic because the measured axle loads in the United Kingdom are lower than in the



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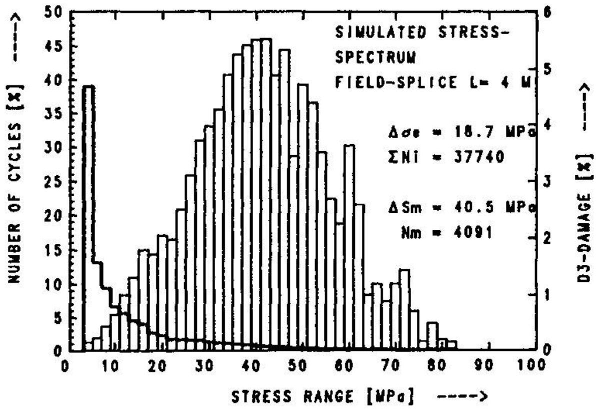

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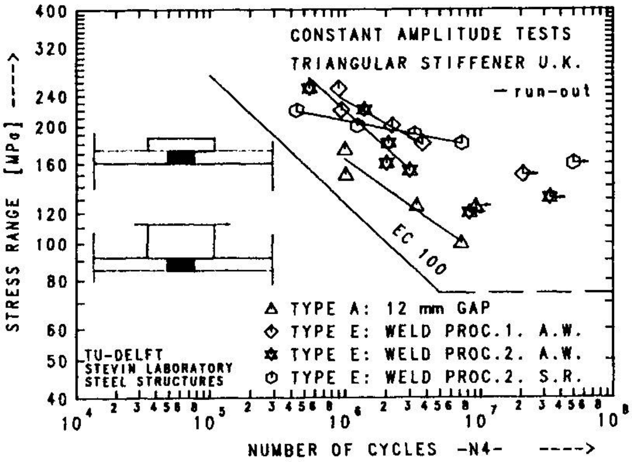

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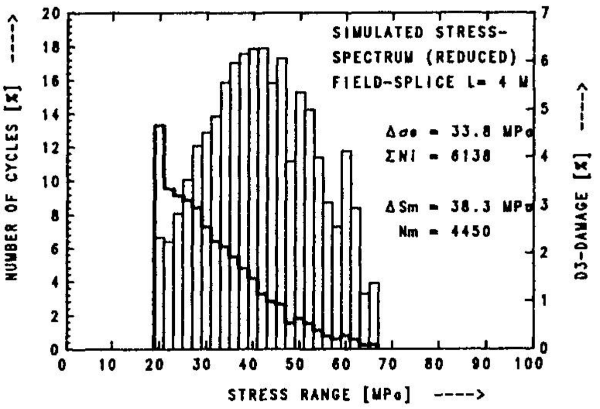

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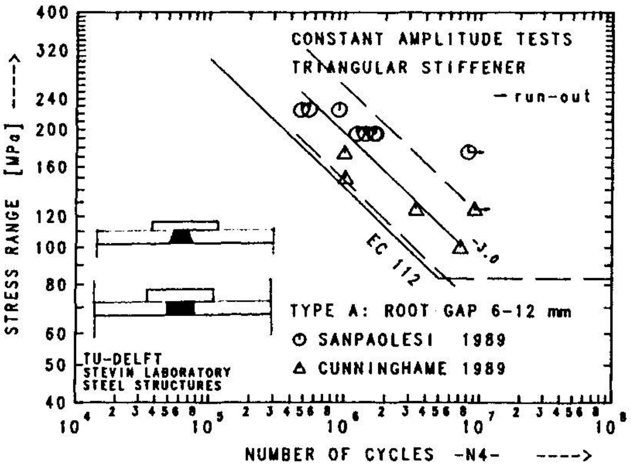
Simulated stress spectra [Ref.2,3].
Figure 5.



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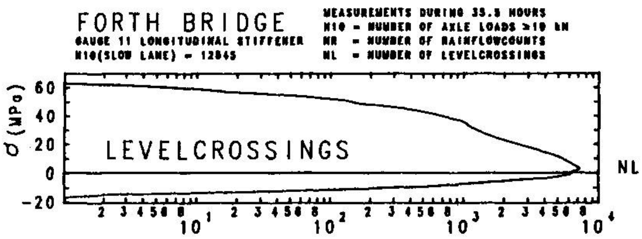
Test results Sanpaolesi [Ref.6]
and Cunninghame [Ref.7].
Figure 4.



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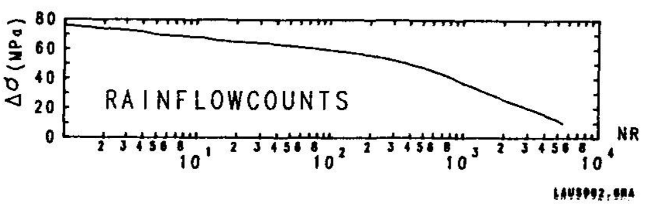

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Netherlands (fig.9). At this stage of the research it was clear that the cracks and fatigue calculation agree, which was a very unlucky situation.
To check the assumed relation load-stress, strain measurements on the Muiden Bridge were carried out.

3.4. Strain measurements

Strain gauges were fixed on several longitudinal ribs to check if the damaged rib was the most heavily loaded one. Three types of measurements were carried out:

- Measurements without interrupting the the real traffic,
- Static measurements with a calibrated truck,
- Dynamic measurements with a calibrated truck.

The analyzed stresses by counting levelcrossings are given in figure 10. It can be seen that the measured stresses on the Muiden Bridge are little lower than those measured on the Forth Bridge and the damaged rib (gauge 2) had to sustain the highest stresses.

The results of the measurements with the calibrated truck are summarized as follows:

static	front wheel rear wheel	10 kN → 11,8 MPa 10 kN → 10,0 MPa
dynamic	front wheel rear wheel	10 kN → 7,9 MPa 10 kN → 7,5 MPa

It is clear that the assumption of 10 kN wheel load resulting in a stress of 15 MPa is not realistic for this detail.

3.5 Second series of fatigue calculations

Using the load spectrum of the Rheden Bridge a second series of fatigue calculations was made. The results are summarized below:

- SPECTRUM III : 10 kN → 12 MPa : 28,6 years
- SPECTRUM IV : 10 kN → 10 MPa : 66,0 years
- SPECTRUM V : 10 kN → 8 MPa : 145,6 years

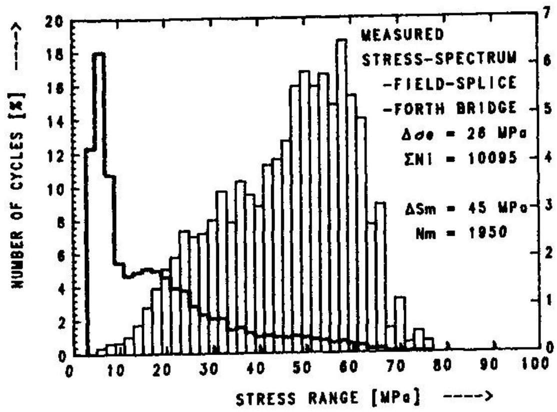
It can be concluded, that the calculated fatigue life for the field splice in the longitudinal rib of the Muiden Bridge is much longer than 15 years. Knowing that the calculations were made in the most unfavorable situation, all wheel loads in the same track, the number of years must be higher.

3.6 Quality of the welds

Examination of cracks showed, that the quality of the welds was very bad. It was concluded that the very bad execution of the welds was the main reason for the observed cracks in this detail. However it must be remembered that the root gap of the detail in this bridge was 2 mm and with the knowledge of the later executed laboratory tests [4,5], it is clear that the EC 80 is too high and more cracks are not excluded in the future.

4. FATIGUE DESIGN USING STANDARD FATIGUE DESIGN CURVES [9,10]

When the static design stresses $\Delta\sigma st$ are known, there is a possibility to have a relationship to the actual stress ranges due to real traffic. The actual stress ranges can be expressed in a proportion of the stress range $\Delta\sigma st$. This opens the way to investigate the fatigue problem for existing bridges but also



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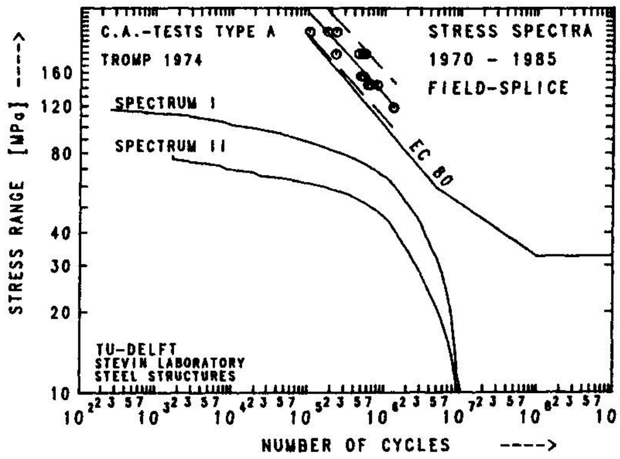
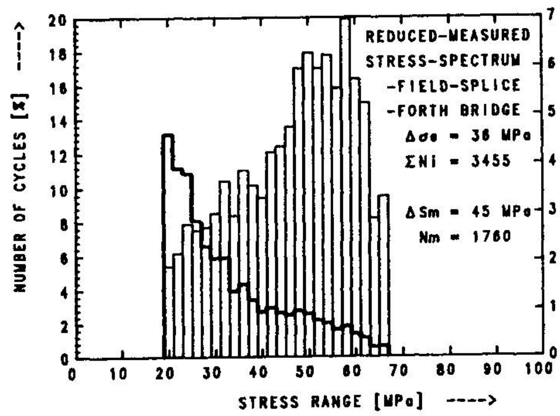


Stress spectra Muiden [Ref.8].
Figure 8.



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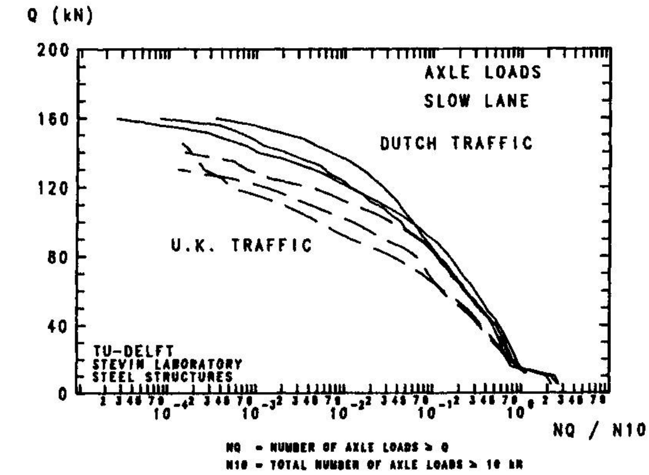
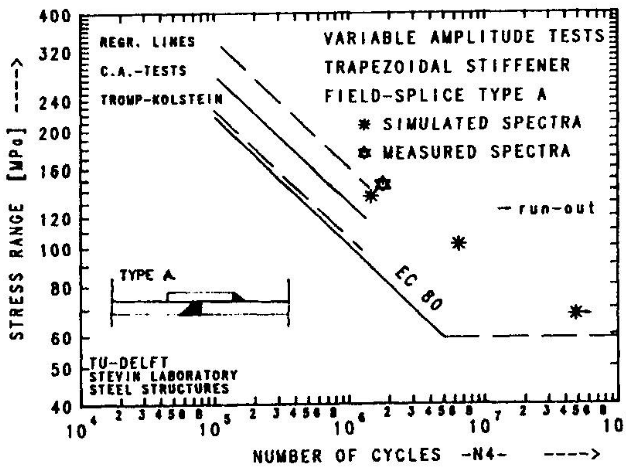
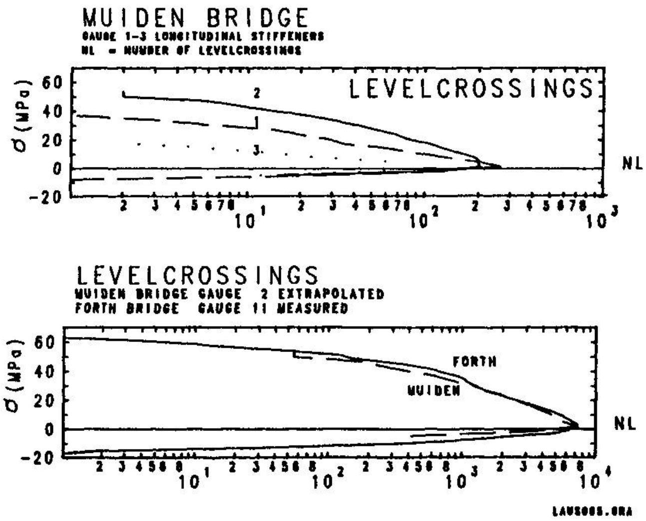
Measured stress spectra [Ref.2,3]
Figure 6.



Axle load spectra [Ref.2,3].
Figure 9.



Test results Kolstein [Ref.4].
Figure 7.



Measured stresses Muiden [Ref.8].
Figure 10.

for the design of new bridges [9]. Knowing the composition of a traffic, stress-time histories can be calculated by running that traffic over static influence lines.
Instead of the impact factor a so called fatigue-coefficient is used to represent the dynamic influence. It has been shown from measurements that the application of the usual impact factor led to results which are too pessimistic while the impact factor covers maximum values of the dynamic effect. In reality there is a scatter of stresses which have to be considered. A fatigue-coefficient being an average of the impact factor seems to be more realistic.
Using the rainflow counting method the stress-time histories are transformed into a number of stress cycles. After that the evaluated stress ranges of the different vehicles are expressed in a ratio to the the stress range $\Delta\sigma st$.
Doing this for several spans spectra of stress ranges can be composed for each span. To evaluate the fatigue life the cumulative damage hypothesis of Palgrem-Miner is applied, using the fatigue strength curves of the Eurocode.
Knowing the spectra with stress ranges expressed in a ratio to $\Delta\sigma st$, the fatigue life can be calculated for bridges of several spans and a chosen value $\Delta\sigma st$.
Figure provides the course of the procedure using an arbitrary S-N curve.
While all fatigue strength curves are similar, having the same slope and the kneepoint at the same number of cycles $N = 5.10^6$, a simplification in the calculations can be achieved by transforming the stress range $\Delta\sigma st$ into a dimensionless value, dividing $\Delta\sigma st$ by the value $\Delta\sigma\text{-}2.10^6$ of the considered S-N curve.
Now factors, k_1, applicable for all S-N curves can be evaluated to define the limiting stress range $\Delta\sigma st$ for an assumed life and the standard traffic.

$$\Delta\sigma_{st(lim)} = \frac{\Delta\sigma_{2.10^6}}{k_1 . \gamma}$$

Figure 11 shows the derivation and gives a set of factors k_1 for an assumed fatigue life of 50 years, respectively 100 years, for the mentioned standard traffic.

It can be concluded for a field splice in a longitudinal rib the static design stress range $\Delta\sigma_{st(lim)}$ with a cross-beam span of 4 meter and a weld class EC 80 must be $\leq$ 100 MPa. Considering the transverse distribution of the wheels and belonging influence line (fig.12) a fatigue reduction factor k_2 can be calculated by the following formula ;

$$k_2 = (\alpha_1.\beta_1^m + \ldots\ldots + \alpha_i.\beta_i^m)^{1/m}, \text{ in which:}$$

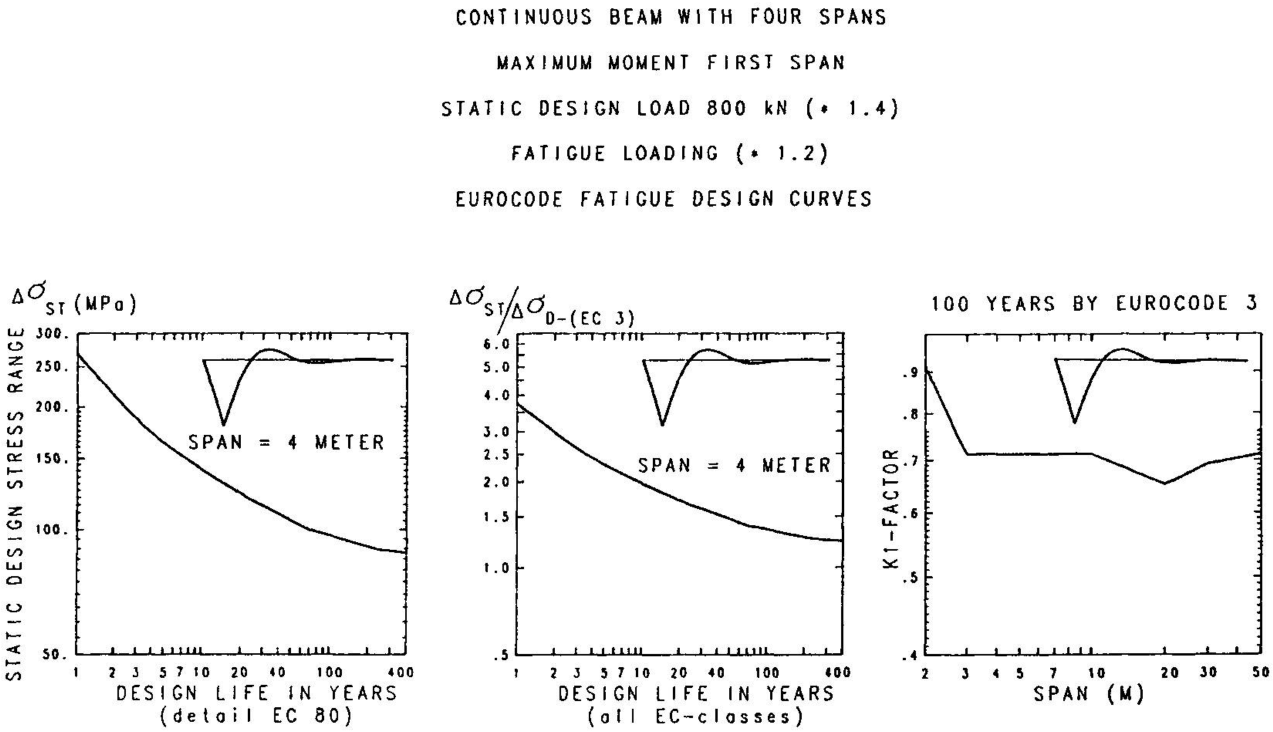
α = transverse distribution factor of the wheel load
β = influence line factor for a specific point
m = slope of the S-N curve.

Now: $$\Delta\sigma_{st(lim)} = \frac{\Delta\sigma_{2.10^6}}{k_1.k_2.\gamma} .$$

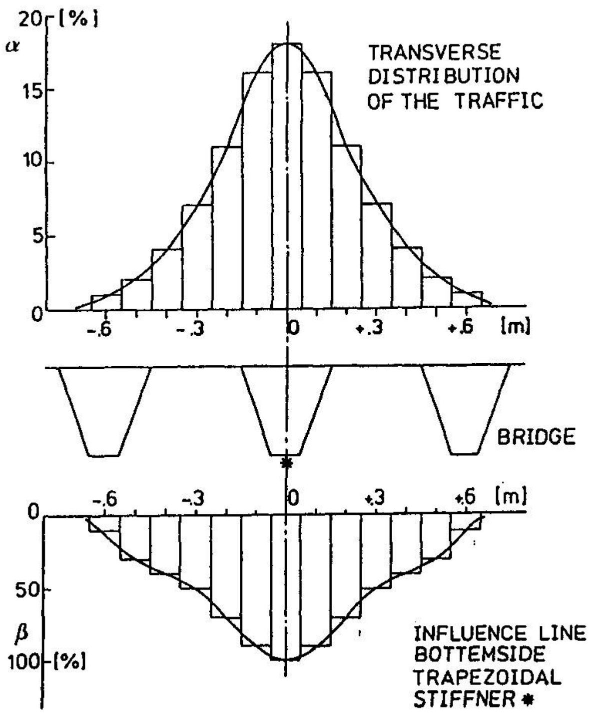
For above mentioned situation $\Delta\sigma_{st(lim)}$ must be $\leq$ 140 MPa.

5. NEED FOR FURTHER WORK

It can be concluded, that the remaining fatigue life of the field- welded splices in longitudinal ribs in existing bridges will be very actual the following years. Therefore it is necessary to develop economic repairing techniques for these welded connections. Based on the available results on these detail, a better design can be realized for new bridges. However it is required to be very accurate concerning the quality of the welds.



Fatigue design curves [Ref.9,10].
Figure 11.



Transverse load distribution and influence line [Ref.10].
Figure 12.

6. ACKNOWLEDGMENTS

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7. LITERATURE

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