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Full Scale Laboratory Fatigue Tests on Riveted Steel Bridges

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Summary

Thousands of old riveted structures, made of wrought iron as well as early mild steel are still in service. However, the fatigue behaviour of deteriorated riveted steel bridges is not well understood. In this paper we present the results of full scale tests on large members and parts of riveted bridges built between 1890 and 1906. The tests on three types of riveted bridge include strain measurement under traffic loads, full scale laboratory tests and small scale or Non Destructive Testing methods (NDT). Advanced material investigations have been carried out, including crack propagation tests, fracture toughness tests and tensile tests.

1. Introduction

It is well known that a great number of old bridges are still performing now as they did 100 years ago. There is often a need to keep them in service because of economic reasons or as historical monuments. Extending the life of a bridge requires knowledge of crack growth behaviour in order to assess the future safety of a given bridge against fatigue damage during its future life.

The paper presents only a first step, but the results obtained can be used to develop rational rating concepts through knowledge of critical structural details, crack growth behaviour of details with built-in defects and the influence of the low fracture toughness which is common to wrought iron elements and whole bridges. We introduce advanced assessment methods using NDT and small destructive techniques based on our experience during full scale laboratory tests and site testing on the same bridges under traffic loads before they were decommissioned.

For thousands of old steel bridges built between 1850 and 1900 riveting was the only connection method. Structural members were normally built up from many relatively thin plates (< 15 mm) or angle sections. Wrought iron has proven to be resistant to corrosion because of its layered structure. Consequently, such structures are more redundant and are not as susceptible to corrosion as more recent materials. Residual stresses as found in modern welded structures created by high temperature differences during the welding procedure are not found in riveted structures. Nonetheless, riveted bridges may have been war damaged or subjected to impact by lorries or cranes, producing defects in the structure which are not easily visible.

During the 1890's in Germany calculation methods using allowable stresses for static loading up to 105 N/mm^2 and for traffic loads up to 70 N/mm^2 were adopted in design (Hütte, 1892). In the course of time, for example during wartime, old bridges were subjected to higher loading than they were designed for, because of increased axle loads and heavier war traffic. It is often very difficult to determine the load history of these old structures.

Most test results reported in the literature are based on small test specimens in order to get information about material behaviour under fatigue loading or the behaviour of a connection. At BAM, we performed three series of full scale tests in order to determine the fatigue critical details in a girder or a steel bridge and to investigate fatigue crack initiation at specific points. These tests allowed a study of the special effects of fatigue loading in a structure when the cross girders are not removed from the structure to be tested independently. Full scale tests in Lausanne and Karlsruhe reported in the literature have been taken into consideration in the assessment of our results. The results of our tests have been used in order to adapt methods for crack identification by NDT as the basis for using a fracture mechanics approach in the bridge rating procedure.

2. Full scale tests

2.1 *Configuration of the test specimen*

All bridge elements tested have been taken from three bridges in Berlin. The first bridge, built in 1902 on the Berlin Underground railway U 1, was made of early mild steel and had been in good condition with respect to corrosion. Figure 1 shows the testing arrangement for the first specimen, a main truss girder, which was supported such that the cyclic load was applied to the end of a cantilevering section of the girder, as shown in Fig. 1A. Two girders of this type have been tested.

The second specimen was a web plate girder taken from a road bridge crossing the Teltow canal between Berlin and the former GDR. Built in 1906 during the construction of the canal, the bridge was 40 years out of service, but obviously in relatively good condition. The bridge was made of mild steel. It has been taken out of service because the bridge was too small after unification of Germany in 1990. The two point loading test arrangement of the 10 m long cross girders is shown in Fig. 1B. Three girders of this type were tested.

Lastly, a suburban train bridge made of wrought iron built in 1890 has been tested. The bridge was cut into three parts transversely so that the cross girders remained connected to the main girders. Each part of the bridge was supported on the main girders, and a two point test load was applied to the cross girders in order to study their behaviour under the real loading and end-connection conditions (Fig. 1C). Four cross girders of this type were tested.

The full scale tests were followed by tension tests, chemical analysis, crack growth tests and microstructure analysis. Details are available in the BAM test reports.

2.2 Equipment and test method

All test specimens were subjected to constant amplitude load cycles. The nominal stress ratio was 0.1 in all tests. The maximum stress difference had to be higher than 71 N/mm^2, corresponding to the ECCS 71 detail category (AASHTO D). Our main goal was to get results for high stress ranges in the region of the transition from m=3 to m=5 on the S- N (Wöhler) curves.

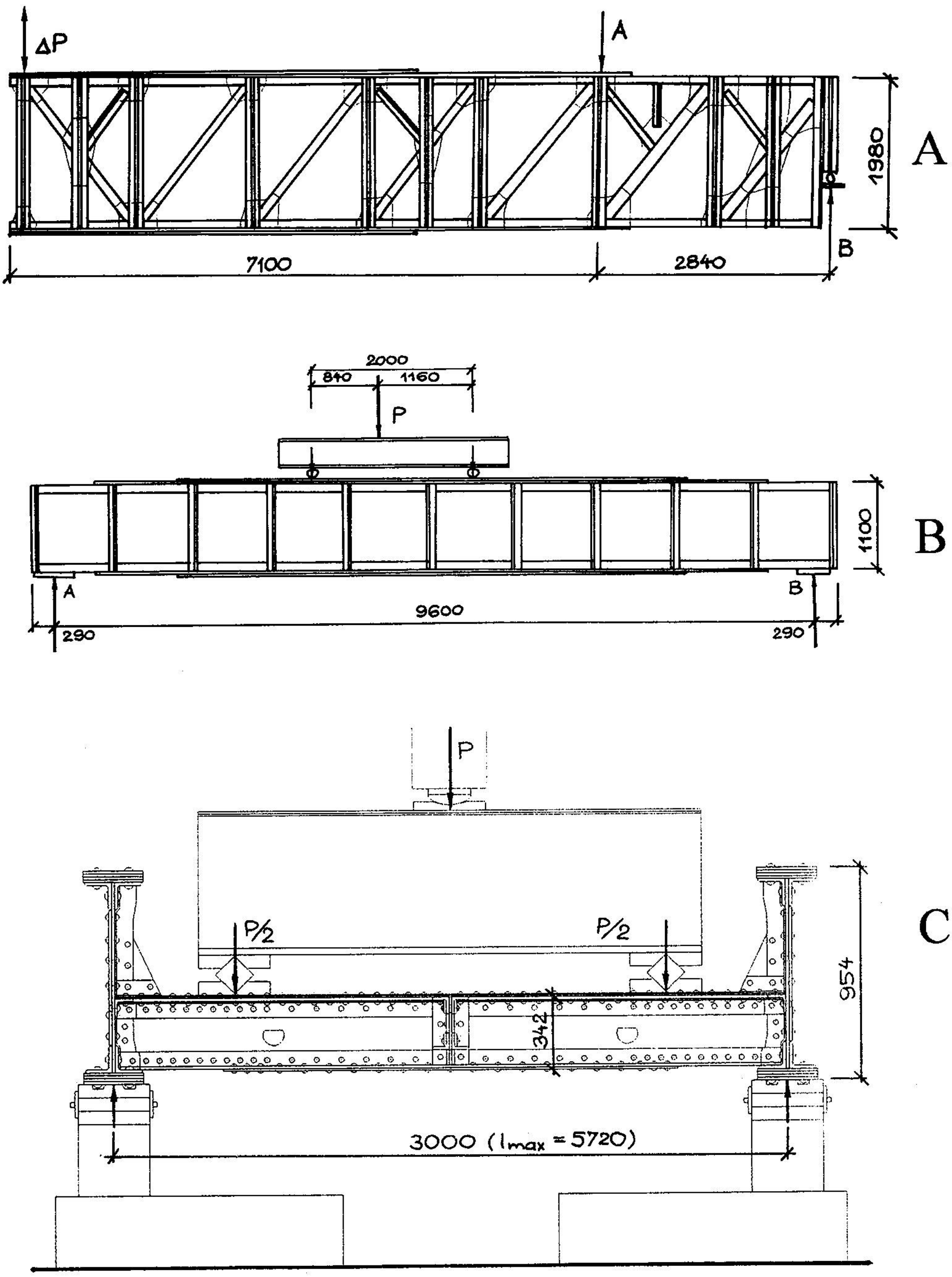

***Fig. 1** Load arrangements : **(A)** main truss girder, **(B)** riveted plate cross girder, and **(C)** bridge parts made of wrought iron*

We did not take into account the load history of the bridges during the tests because the measured strain range under traffic loading was low. During the first and the third test series we had the opportunity to measure strains under suburban train traffic before the bridges were dismantled. Measured maximum stresses under traffic loading were lower than 29 N/mm^2 for both bridges. According to fracture mechanics, these stresses cannot initiate or propagate cracks, although later they are taken into consideration for discussing the test results. The third bridge was in service for less than 40 years.

Individual tests were terminated either at an increase of the maximum deflection of more than 0.2 mm between measurements or once a crack propagated through the entire section of a element. In some of the tests, the test was continued until the crack grew into the web plate.

The girders from the second series were cut into smaller specimens (lengths of about 1m) and the fatigue tests were continued with the same load range as in the main tests. These results are included in Fig. 8. During the third test series (wrought iron), we continued the test on a cracked element at the stress level which was measured under traffic load.

3. Results

3.1 Material investigations

The materials were analysed by several tests. **Tensile test**: Except for the truss girder (gusset plate) all standard test specimens were provided only in the main rolling direction. All results are summarised in Table 1. The yield stresses of mild steel and wrought iron do not differ very much. The Young's modulus of wrought iron is on average about 10-15% lower than that of mild steel; in the tests we got a value of about 195 000 N/mm^2, which is similar to results from literature and old scientific papers written in the for last century. This Young's modulus has been taken for calculating stresses in our wrought iron tests. Ultimate elongation is lower, and in particular, macroscopic necking does not occur. This behaviour is a consequence of the microstructure of wrought iron. Wrought iron behaves like a composite material with layers of ductile ferrite matrix with brittle slag layers. This structure is a result of the forging and rolling process.

The **microstructure** at the crack- tip- opening region for mild steel and wrought iron was investigated. Fatigue striations for the length of a crack during one load cycle were found on the fracture face in both materials. In mild steel, for example, a crack was stopped and divided into two tip- cracks in the CTO- region after leaving the rivet head because of the material's ductile behaviour. In wrought iron a split level (terrace) crack was found in a web plate after finishing the test; the crack was not visible from outside. This crack showed typical nodal lines stepped in the plates area.

Crack growth tests of wrought iron were carried out. In these tests, the influence of lower temperatures was not taken into consideration. All test results for cracks perpendicular to the rolling direction were below the Barsom curve for contemporary steels. Rolfe and Barsom (1987) proposed this empirical approximation for steels with a yield strength greater than 250 N/mm^2. Test results are shown in Figure 2 in relation to this curve. Tests for cracks oriented parallel to the rolling direction produced results above this curve, meaning that such cracks propagate faster. The cyclic stress intensity ΔK depends on the stress range; a higher stress range results in a lower cyclic stress intensity. The yield cyclic stress intensity ΔK_{th} of the investigated wrought iron was ~ 10 MPa·m$^{1/2}$ for a nominal stress ratio of 0.1.

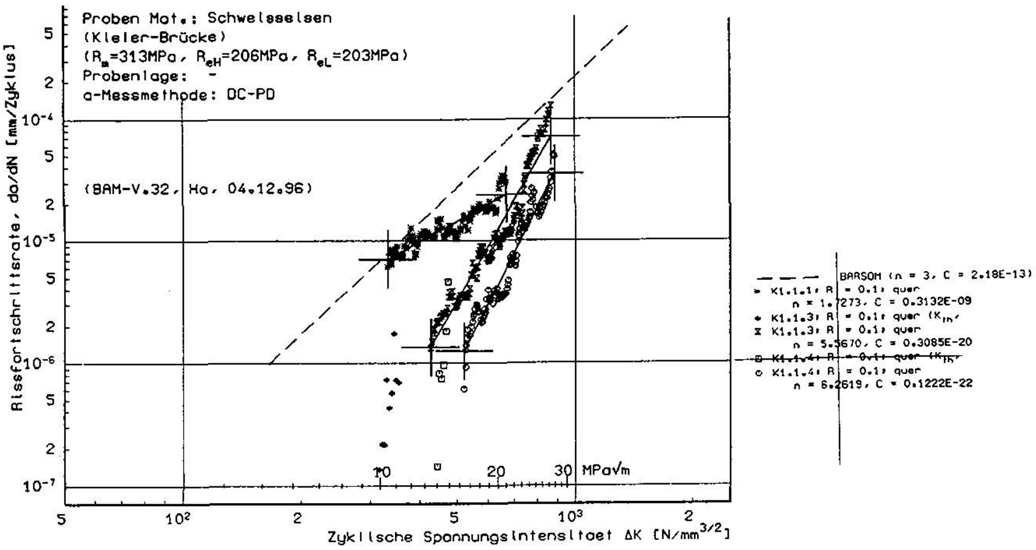

***Fig. 2** Results of crack growth tests on wrought iron specimens*

Material	Higher Yield Stress N/mm²	Lower Yield Stress, σ_Y N/mm²	Ultimate Tensile strength, σ_f N/mm²	Ultimate Elongation %	σ_Y / σ_f
1. Test, WB mild steel, 1902,	underground - 16 specimens	suburban line U1			
Mean in rolling direction	255	234	446	29,5	0,57
standard deviation s_x	25,4	23,3	10,8	2,4	
Mean 90° to rolling direction	244	236	442	27,5	0,55
standard deviation	19,5	17,3	7,9	1,6	
2. Test, Kn mild steel, 1906	road bridge				
Mean, angle profile (n=6)	321	296	371	17	0,86
standard deviation	11,1	3,0	6,4	3,1	
Mean, cover plate (n=12)	234	214	322	23,3	0,73
standard deviation	16,7	9,3	6,1	6,0	
3. Test, wrought iron, 1890	suburban line 5 specimens				
Mean, angle iron	243	233	339	15,5	0,72
standard deviation	2,82	14,58	24,0	2,6	
Mean, cover plates	211	207	315,6	18	0.67
standard deviation	7,3	5,6	7,4	2,7	

***Table 1** Tensile test results*

3.2 Results of fatigue tests

The results of all fatigue tests are given in Table 2. Only one truss girder (Suburban train viaduct, U1,mild steel, WB1, main girder) was repaired and the test was continued with the same maximum strain difference. The fatigue failure occurred in an untypical net cross-section on a hole drilled for a conveyer mechanism during dismantling. In the second test series (Road bridge, mild steel, Kn1-3, cross girder) a fatigue crack occurred in the constant moment region. For the wrought iron bridge test series (Suburban train bridge, S1, Kie 1-4), the cross girders were tested under their original support conditions. (Bridge parts up to 3,3m x 5,70m were supported at four points on their main girders).
All results fall in a S-N diagram above the ECCS category 71 with constant slope m = 5.

Te st specimen	***Net cross-section* range, Ds**	***Numbers of* cycles Mio cycles**	**Crack location (first cracked element)**
	N/mm^2		
(A) truss girder,	**mild steel, 1902**		
WB 1	80	2,600	at a new drilled hole (untypically, repaired)
WB 1,continued	80	3,600	in a gusset plate
WB 2	140	0,250	in a gusset plate
(B) plate girder	**mild steel, 1906**		
Kn 1	101	2,618	second cover plate, at rivet hole (in built failure)
Kn 2	123	0,361	second cover plate, at rivet hole (outside impact damage)
Kn 3	115	0,562	second cover plate, at rivet hole
(C) plate girder bridge	**wrought iron, 1890**		
Kie 1	130	0,586	at rivet hole under cutoff in web- plate
Kie 2	108	2,829	at rivet hole, angle on the end of a cower plate
Kie 3	97	2,691	at rivet hole, angle on the end of a cover plate
Kie 4	104	4,008 4,276	+ 0,2mm- at rivet hole under a cutoff in web- plate, (rivets on plates ends of the cross girders were replaced by prestressed bolts)

Table 2 *Results of full scale fatigue tests*

3.3 Characteristics of crack initiation and geometry of growing cracks

In all tests we noticed that crack initiation was not detectable. All cracks were identified by increasing strains or deflections, or after they reached the edge of an element. Most of the cracks were initiated at rivet holes, often caused by changes in the cross-section geometry. Stress concentrations may also occur at hidden defects within a built-up section. In the tests we also discovered cracks at such defects.

In the **truss girder** test series all fatigue cracks occurred in the gusset plates, indicated by increasing strains during the test. The cracks initiated in the region of the gusset plate near to the last rivets on the diagonal where the gusset is sandwiched between two diagonal elements. Details of the gusset plate with the crack after removing rivets and the diagonal elements are shown in Figure 3. After elongation of the rivet hole cracking began at the edge of the rivet hole. Additional factors contributing to crack growth, for example from rivet installation, were not found in our tests. As known from literature, at this time (1902) in Germany rivet holes had to be drilled; punching was not allowed in structures with traffic loads. Exceptions to this rule might have occurred.

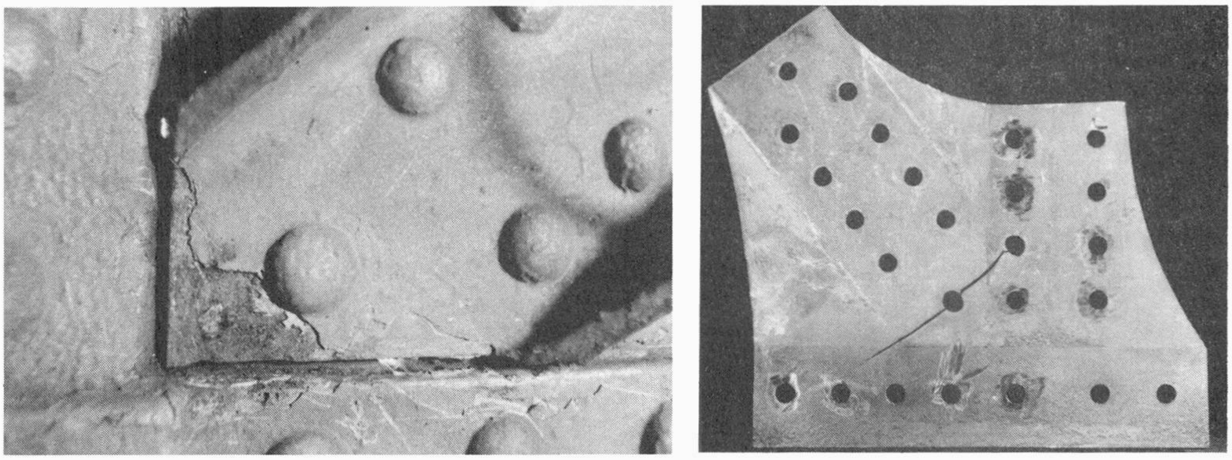

Fig. 3 *Details of a cracked cover plate* ***(A)*** *concealed within the structure,* ***(B)*** *after disassembly*

In the **plate girder** test series (mild steel, 1906, Kn), all cracks initiated in the cover plate, sandwiched between angles and additional cover plates. Cracks were found after deflection and strains increased or when the crack was long enough to reach the edge of the sheet plate. When cracks reached the edge, they were only visible when the girder was loaded. All tests were continued to find out if crack growth became unstable. The result was plastification at the crack tip and crack dividing into two tips, but no unstable growth. In one of these tests the crack tip in the web plate had been cooled down to -30°C on both sides of the web plate. The crack tip detail is shown in Figure 4.

Fig. 4 *Detail of crack tip in a web plate*

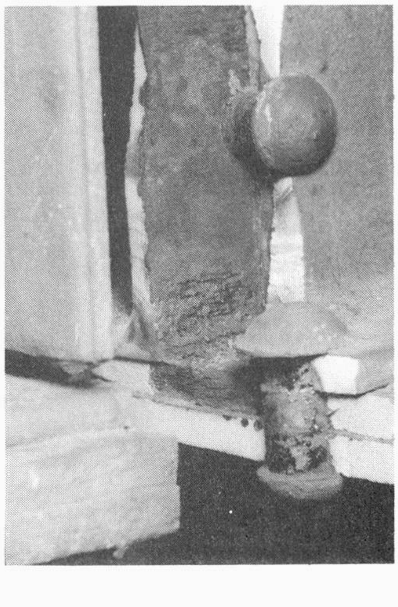

Fig. 5 *Crack in the tension flange*

The bridge crossing the Teltow canal "Knesebeck-bridge ", had presumably been damaged during World War II. Almost every bridge crossing the canal had been blasted during the last days of the war. We found old corroded failures in the cross section of cover plates and untypical cracking behaviour in the region of constant moments. In the tension flange of the second plate girder the crack was growing from an old corroded failure, as shown in Figure 5. The third web plate girder seemed to have been exchanged. In material tests we found higher tensile strength, appropriate to Fe 360, in girder Kn 3. By means of strain measurement or deflection measurement it was possible to detect crack growth.

In the third test series on parts of a **wrought iron plate girder bridge** (1890), typical for the Berlin Suburban railway, cross girders under their original support conditions were tested. Due to its special layered microstructure, unlike modern steels, the behaviour of wrought iron under constant amplitude fatigue load needs more tests. During the tests, in addition to strain and deflection measurements, we used Non Destructive Test methods such as magnetic particle tests, fibre optic sensors glued to the tension flange, an Acoustic Emission System and X-ray methods to detect cracks as early as possible. Initial readings from strain gauges taken every 15 minutes (1800 load cycles) indicated increasing strains. The optical fibre sensor cracked when the crack reached the edge of the angle. At that time the crack was not visible under good lighting conditions without loading (Figure 6). In the test Kie 1 the angle cracked in an unexpected cross-section under a drilled hole for a rainwater pipe. On other similar bridges, holes of this type had been framed with angles. This was not the case with this particular hole. The crack was also detected by a change in direction of the main stresses on the cracked girder in the joint between main and cross girder, measured by means of strain gauge rosettes. A terrace crack in the web plate was found after finishing the test and disassembly of the girder (Figure 7). Crack tips in the angle were found only by magnetic particle tests or by X-ray. The crack was indicated by increasing strains, fretting rust and a cracked old paint layer near the crack. After the point at which the measured strain had increased by 1%, it took 47 000 further load cycles until the web-to-flange angle was craked. Relating to traffic load (max. 30 N/mm^2 in net cross section) it means on this bridge 4,3 years of service for one full train every 10 minutes until the tension flange is damaged, if it is not assumed that low load cycles do not cause crack growth. Magnetic particle testing also discovered misaligned rivet holes which had been back-filled in near to the cracked cross-section. However, no cracks were found at these back-filled holes.

In the last cross girder test, the last rivets of the cover plates were replaced by prestressed bolts. In this case, from the point at which strains had increased by 1%, it took 2 million high-stress-range load cycles until failure of the web-to-flange angle.

Fig. 6 *Cracked angle in tension flange*

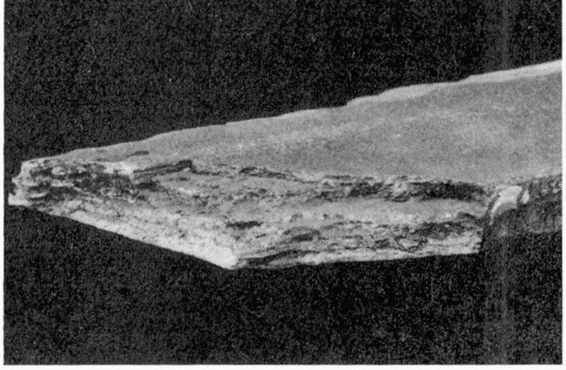

Fig. 7 *Terrace crack in web plate*

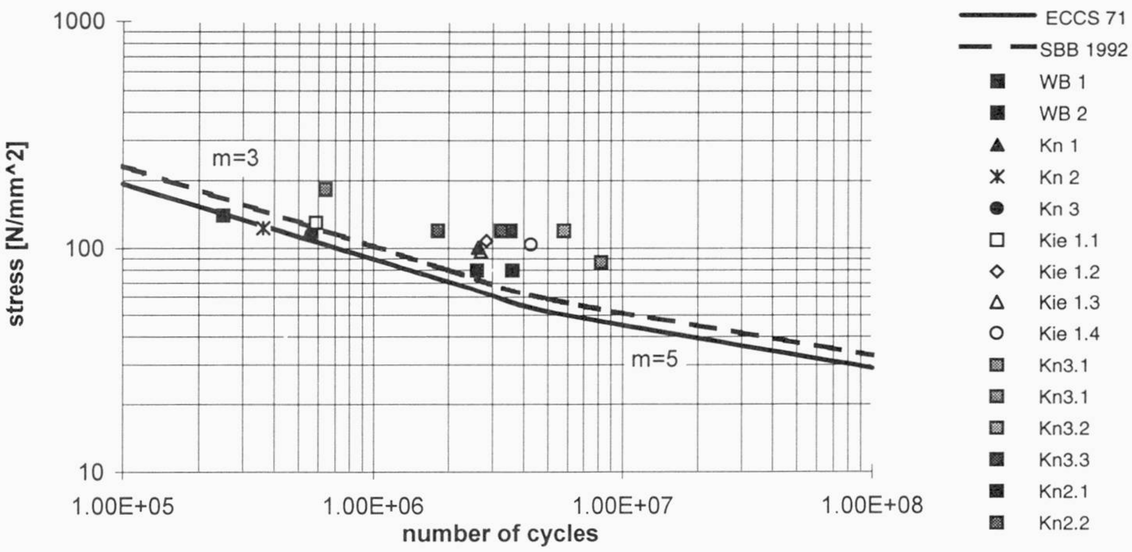

Fig. 8 Results of full scale tests in S-N-curve

In two tests the crack occurred in the plate under an additional plate at the first row of rivets. Crack growth was indicated by decreasing strains in the tension flange. All tests are summarized in the S-N-diagram in Figure 8.

4. Non Destructive Testing methods

Fig. 9 Crack in a tension flange test Kie 3

Non Destructive Testing methods such as **X-Ray** make it possible to see a crack on a film. However, more tests are needed to develop this method for applying it to bridges in service. First tests on the Berlin Suburban railway are finished. Figure 9 shows a crack in the tension flange of cross girder Kie 1.3 (scanned X-ray- film). Of course, these tests are possible only for special cross-sections under special conditions (40 m diameter safety zone) because of the X-Ray emissions.

Testing should be carried by NDT specialists under the direction of the investigating engineer. In most cases, the costs of testing are outweighed by the savings in structural repair or replacement costs.

For detection and location of damage in large parts of wrought iron bridges we used glued **fibre optic sensors**. An only primarily coated optical fibre was fastened to the top and the bottom side of the bottom flange of the cross girder. Because fatigue cracks were expected near rivet heads the fibres were sensitised (coating) in this area before gluing. A threshold limit for crack formation in the range between 10 and 200 μm can be obtained. This method is also a cheap way of detecting existing cracks with a width of between 10 and 30 μm which open under traffic loads and which cannot be detected visually. The measurements made with a fibre optic sensor during test Kie 1 detected a crack (after 628 000 load cycles) as shown in Figure 10.

Fatigue cracks were also detected using the **magnetic particle method**. This method was used during laboratory tests after strains changed in a certain region, and it has been used on bridges in service to find cracks and their direction in the region of war damage or highly stressed rivet heads.

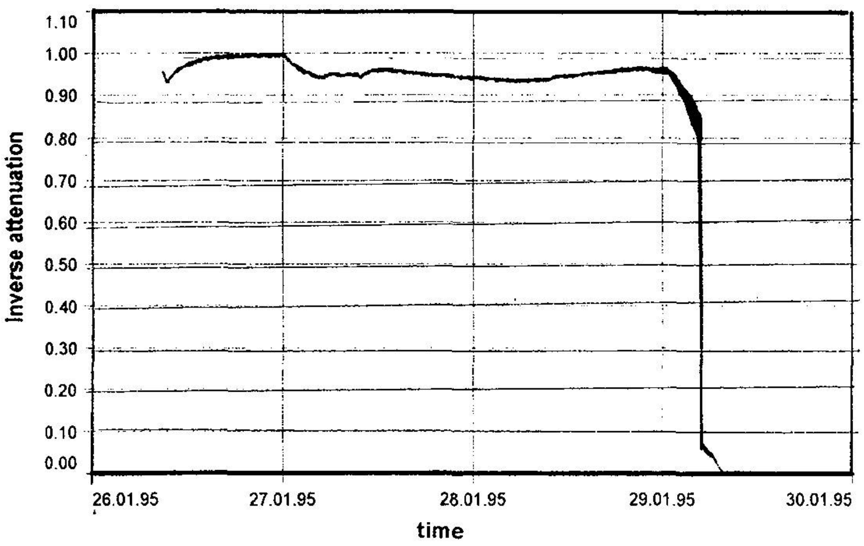

Fig. 10 Signal from a sensitised optical fibre before and after crack formation (Kie 1)

5. Concluding remarks

1. Results of testing riveted connections agree with the design code ECCS detail category 71 with slope m=5. Tests indicate that this detail category can be used for evaluating old bridges.

2. Some fatigue cracks initiated at old corroded impact damage or structural defects. Nonetheless, constant-amplitude fatigue limit detail category 71 N/mm^2 covers these test results.

3. Fatigue cracks which do not reach the edge of an element are not detectable unless NDT methods are used. The magnetic particle and X-ray methods enable the detection of existing cracks.

4. Cracks growing in tension elements can be indicated by a change in the direction of main stress, measured using strain gauge rosettes, for example during permanent tests on bridges.

5. The fatigue behaviour of wrought iron during standard tests is not worse than that of mild steel. By considering appropriate values for Young's modulus and yield stress as well as the rolling direction, wrought iron bridges can be assessed in the same manner as mild steel bridges.

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