Zeitschrift: IABSE reports = Rapports AIPC = IVBH Berichte

Band: 73/1/73/2 (1995)

Artikel: Assessment of a 135 year old riveted railway bridge

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DOI: https://doi.org/10.5169/seals-55306

Assessment of a 135 Year Old Riveted Railway Bridge
Évaluation d'un pont-rail riveté, de 135 ans
Beurteilung einer 135 Jahre alten genieteten Eisenbahnbrücke

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SUMMARY

This paper deals with the assessment of the remaining service life of the oldest existing wrought-iron railway bridge in continental Europe. This bridge is part of a railway line currently under investigation for future use by the Zurich suburban railway. The evaluation, based on actual loads and fracture mechanics methods, has shown that this 135 year old bridge is still safe and serviceable for many years to come.

RÉSUMÉ

L'article examine la durée d'utilisation restante du plus vieux pont en fer puddlé d'Europe continentale. Le pont se situe sur une ligne potentielle du réseau express régional zurichois. L'évaluation est basée sur les charges actuelles et sur la mécanique de la rupture; elle montre que ce pont d'un âge de 135 ans pourra encore être maintenu en service pendant de nombreuses années.

ZUSAMMENFASSUNG

Der vorliegende Bericht behandelt die Untersuchung über die Restnutzungsdauer der ältesten bestehenden schweisseisernen Brücke auf dem europäischen Festland. Die Brücke liegt auf einer Strecke, die in Zukunft von der Zürcher S-Bahn befahren werden soll. Die auf den wirklichen Bahnlasten und auf der Bruchmechanik basierende Untersuchung hat gezeigt, dass diese 135 jährige Brücke auch in Zukunft noch genutzt werden kann.

1. INTRODUCTION

Riveted bridges were built over a period of more than 100 years up to the 1950s. There are thousands of riveted bridges around the world still in service. Some of them are considered "historical" and should be preserved as architectural heritage. Economically, it is not justified to replace a bridge when it reaches its "design life". Often the design life is an arbitrary value and there is considerable reserve. An important remaining service life may be justified provided that corresponding inspection guidelines are followed.

This paper deals with the investigation of the oldest existing wrought-iron railway bridge in continental Europe. This bridge is part of a railway line currently being under investigation for future use by the Zurich suburban railways. The main objective of the present investigation was to evaluate the consequences of this new use for the structural safety and remaining fatigue life of this historic structure.

The assessment of the bridge was conducted by proceeding in stages (Fig. 1). A first assessment of structural and fatigue safety was performed based on current code provisions to identify critical members in the structure. The effect of actual loads of past and future rail traffic on service stresses in the structure was studied to assess the fatigue safety in the second step on the basis of the fatigue limit. Fracture mechanics methods were applied in the final step to investigate critical crack length and fatigue crack propagation and their influence with respect to failure of structural elements.

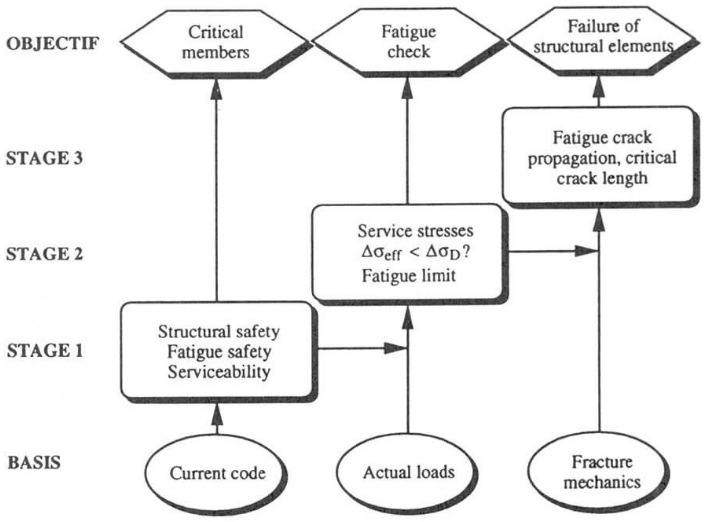

Fig. 1 : Assessment by proceeding in stages

2. DESCRIPTION OF THE BRIDGE

The investigated bridge (Fig. 2) crosses the river Rhine in northern Switzerland to carry a railway line between Koblenz (Switzerland) and Waldshut (Germany). It was built in 1859 and comprises riveted wrought iron members. The straight lattice-truss bridge is one of the last examples of a construction type that was typical for the railroad construction boom in Europe during the third quarter of the last Century.

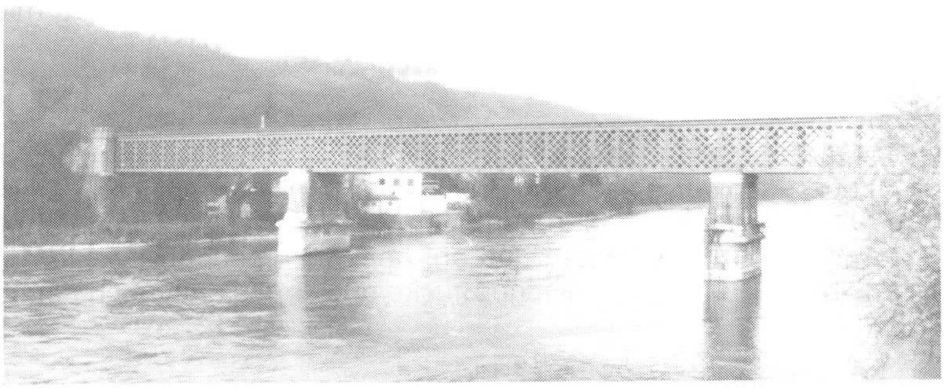

Fig. 2 : Bridge over the Rhine at Koblenz - Waldshut

The wrought iron structure was conceived as a continuous girder over three spans of 37.5m, 55m and 37.5m with a total length of 130m supported by abutments and piers in natural stone masonry. The single track is not ballasted; the timber sleepers are fixed directly to the stringers. The bridge was initially designed to carry two tracks; however, it only ever carried one track. During its service life the structure was well maintained and, apart from some local corrosion, the present state of the bridge structure is good. For the excellence of its maintenance and restoration, this bridge was given in 1994 a Brunel Award which is the most important award for railway architecture.

3. VERIFICATION ACCORDING TO THE CURRENT CODE

The first stage (see figure 1) concerned an overall assessment according to the current codes. Firstly, structural safety was assessed at different sections of the bridge in various elements. The action effect S_d was calculated using the UIC 71 model which is the rail traffic load model currently used in Europe. For wrought iron, a characteristic value of yield strength of 220 MPa is generally chosen, and for the rivets, a shear resistance of 225 MPa and an ultimate stress of 510 MPa is considered [1]. These values have to be divided by the resistance factor γ_R of 1.20 in order to obtain the design value for resistance R_d. As a result, all elements were found to satisfy the requirements of structural safety. The fatigue safety of all bridge members can be represented by the following ratio n_{fat} between fatigue resistance and fatigue action effect:

$$n_{fat} = \frac{\Delta\sigma_c/\gamma_{fat}}{\Delta\sigma_e}$$

The fatigue resistance $\Delta\sigma_c$ is represented by the detail category as defined at 2 million cycles. Detail category 71 with a fatigue limit of 52 MPa at $N_D = 7 \cdot 10^6$ is chosen for riveted wrought iron bridge elements [1]. For details causing local failure, the fatigue resistance factor $\gamma_{fat} = 1.15$ is considered, and for elements leading to total collapse of the structure, γ_{fat} is 1.25.
The fatigue action is represented by the equivalent stress range $\Delta\sigma_e$:

$$\Delta\sigma_e = \alpha_r \cdot \alpha_N \cdot \Delta\sigma(\phi\, Q_{UIC})$$

The live load stress range $\Delta\sigma(\phi\, Q_{UIC})$ is the difference between the extreme values of static live load stresses due to the UIC rail traffic model including the dynamic coefficient ϕ. The correction factor α_r accounts for the cumulative fatigue damage caused by the stress spectrum of actual traffic of main railway lines. The factor α_N accounts for the number of trains in the past [2].
The bridge members were compared, based on this deterministic method, and a ranked list identifying fatigue critical bridge details was established. Details with $n_{fat} < 1$ required further investigation, fatigue safety was assumed verified if $n_{fat} > 1$. The lowest value of 0.83 was obtained for the lower chord at section M1 of the main girders (Fig. 3). Consequently, the chord member at M1 will be more thoroughly investigated in the subsequent chapters.

4. FATIGUE ASSESSMENT FOR FUTURE URBAN RAIL TRAFFIC

The aim of the second stage of the assessment (see figure 1) is to compare the maximum stress range due to actual traffic loads, past and future, with the fatigue limit of riveted connections.

4.1 Load models for past and future traffic

The past traffic on the bridge was significantly different from normal main line traffic. There was a moderate total of only 750'000 trains crossing the bridge in the period from 1859 until today. Most of the trains were light passenger trains, but some 30'000 heavy freight trains (maximal axle load: 180 kN, maximal distributed load: 60 kN/m) also crossed the bridge. The loads of this heavy freight train were thus considered in the calculation of the maximum stress range due to the past traffic. For the future a daily traffic of 40 passenger trains is planned. To represent this traffic, the loads of today's suburban passenger trains are taken into account. Various rail traffic models were developed accordingly.

4.2 Calculation of stress-time histories

The passage of these traffic models was then simulated to determine stress histories. The stress-time history due to the future suburban passenger trains is given as an example for section M1 in Figure 4 showing a maximum stress range of 31 MPa. The heavy freight train representing the past traffic gave rise to a maximum stress range of 59 MPa.

The stress history in Figure 3 illustrates why section M1 is the fatigue critical location in the main girder: due to the continuous action of the main girders over three spans, the influence line for bending moments at M1 shows both tensile and compressive stresses. The resulting fatigue action in terms of stress range is thus greater than the design stress for structural safety. This fatigue relevant stress range was not considered when the main girder was designed in the last century; section M1 was designed to account for static structural safety only.

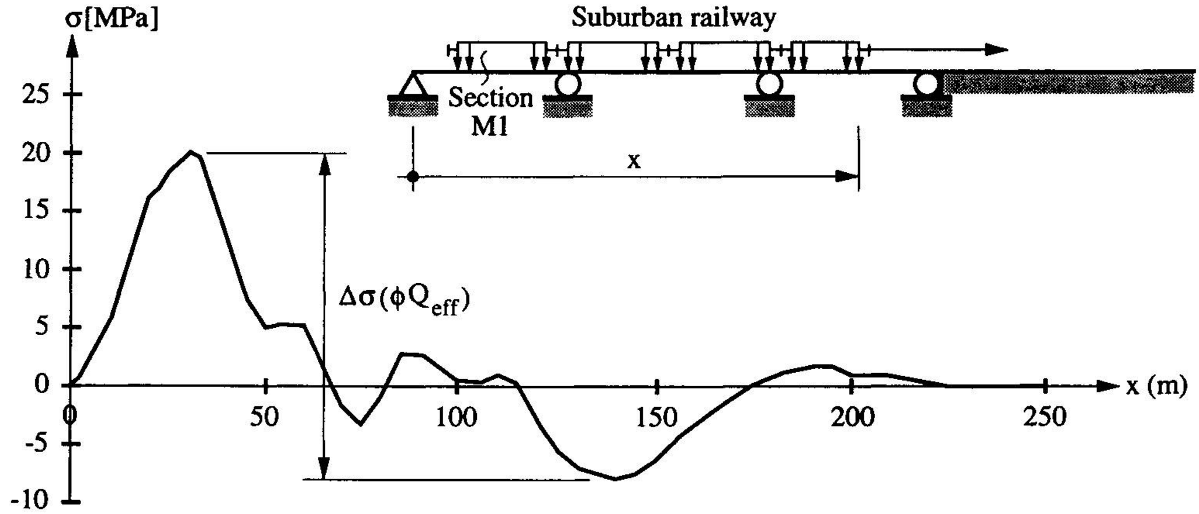

Fig. 3 : Simulated stress-time history due to the passage of suburban passenger trains

4.3 Fatigue damage due to past traffic

The simulated maximum stress range due to the actual traffic was then compared to the constant amplitude fatigue limit of riveted connections of 52 MPa. The fact that the maximum stress range from heavy freight traffic is 59 MPa means that a part of the past stress spectrum was above the fatigue limit, and thus, there is a theoretical fatigue damage todate. The limit below which no crack propagation occurs is therefore no longer constant, but decreases with increasing damage. This decreasing limit is called the damage limit $\Delta\sigma_{th}$ representing a reduction of the constant amplitude fatigue limit [2],[3] (Fig. 4):

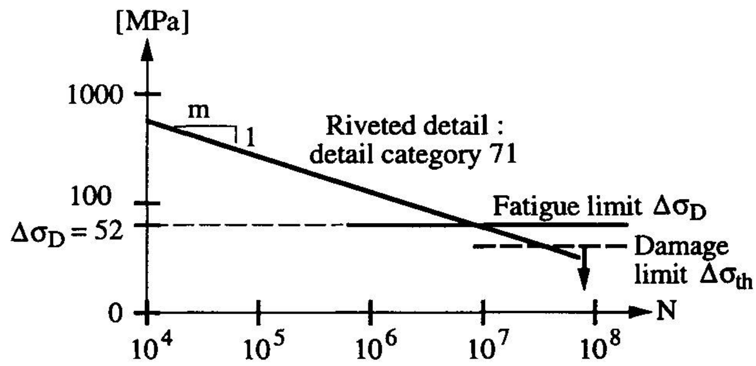

Fig. 4 : Definition of fatigue strength curve

$$\Delta\sigma_{th} = \Delta\sigma_D \cdot (1 - D)$$

In the present case, this reduction was found to be minor ($\Delta\sigma_{th}$=51 MPa compared to $\Delta\sigma_D$=52 MPa) and the fatigue damage effect due to the past traffic is thus almost negligible.

4.4 Fatigue safety for future traffic

With the predicted stress ranges $\Delta\sigma(Q_{fut})$ less than 31 MPa for the suburban trains, these will be significantly smaller than the fatigue damage limit $\Delta\sigma_{th}$. Consequently, the fatigue safety for future traffic is verified, since:

$$\Delta\sigma(Q_{fut}) < \frac{\Delta\sigma_{th}}{\gamma_{fat}}$$

The planned future suburban rail traffic should not lead to any further fatigue damage and, from the point of view of fatigue, a theoretically infinite number of future passenger trains could cross the bridge.

5. FRACTURE MECHANICS ASSESSMENT

5.1 Motivation

Normally, stage 3 of figure 1 would not be necessary for the assessment, since both stages 1 and 2 have shown adequate safety. However, in a riveted structure, there might exist crack-like defects due to the riveting process or flaws in the wrought iron material stemming from its fabrication.
Additionally, undetected cracks might be present although regular inspections were conducted. The question remains: What is the behavior of these defects under future traffic loading? As an illustration, a lamella of the chord member at section M1 is considered (Fig. 5).

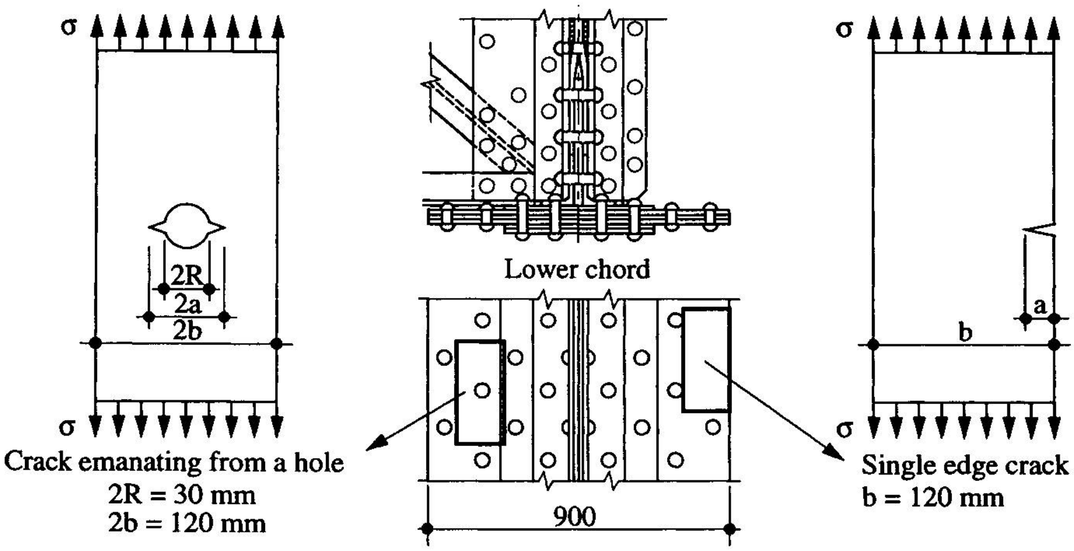

Fig. 5 : Modeling of cracks in the chord

5.2 Critical crack size

A first question is whether the critical crack can be detected before the member fails. The critical crack size a_{cr} is estimated using two criteria:

- With the *strength of materials* approach, failure occurs if, under maximum stress σ_{max} in the member, the net section stress in the cracked section reaches the yield strength.
- With the *linear elastic fracture mechanics* approach accounting with $Y(a_{cr})$ for the stress concentration at the crack tip, failure occurs if the stress intensity K becomes equal to the fracture toughness K_{IC} (Griffith' criterion):

$$K = Y(a_{cr}) \cdot \sigma_{max} \cdot \sqrt{\pi \cdot a_{cr}} = K_{IC}$$

The analysis revealed that a crack in section M1 having a length of 25mm larger than the rivet head on each side must appear before failure of the chord member occurs. Such cracks should be detectable during visual inspection, particularly since the fracture critical locations are known.

5.3 Fatigue crack propagation

A next question is whether undetected cracks will propagate due to the future passenger trains. The modified Paris law was used to study this:

$$da/dN = C \cdot (\Delta K^m - \Delta K_{th}^m)$$

da/dN : rate of increase in crack size per stress cycle
C : crack propagation constant

m : slope of the crack propagation curve
ΔK : stress-intensity factor range
ΔK_{th} : threshold stress intensity factor range

Assuming that the initial flaw at the time of construction was 1mm at the edge of the rivet hole and applying the maximum stress range of 59 MPa due to the past heavy freight traffic, the analysis shows that (using conservative values of the material constants C, m and ΔK_{th}) a crack would have propagated by only 1mm todate. Subsequently, a crack size of 2mm was assumed to be actually present in the structure. Further crack propagation under the future traffic was calculated with this crack size as a starting value. Figure 6 shows that after passage of 1 million passenger trains, representing about 33 years of future service life, this crack would only grow by an additional 5mm. Such a crack would still be hidden under the rivet head. It could therefore not be detected. However, it is very far from the critical crack size calculated above.

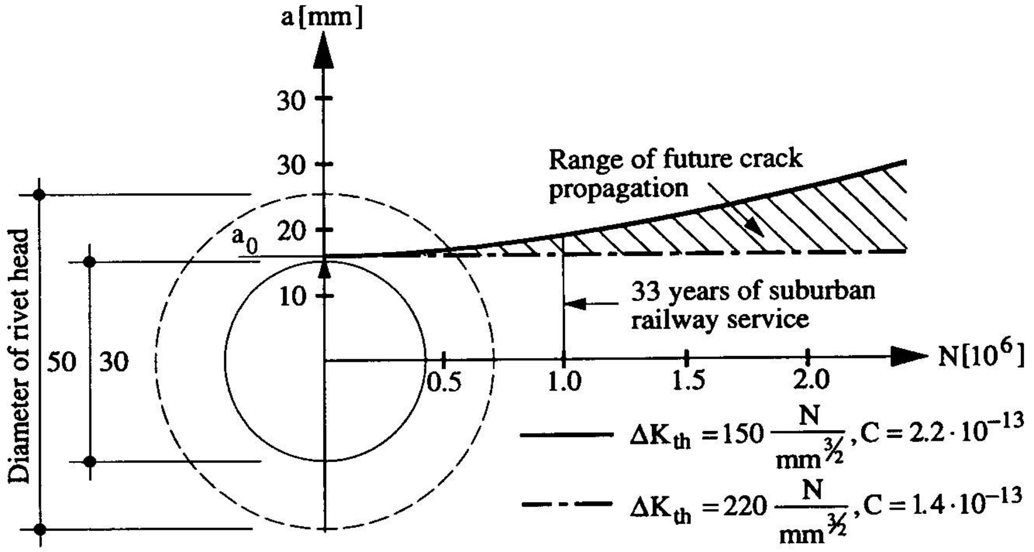

Fig. 6 : Fatigue crack propagation due to future suburban railway traffic (a_0= 17 mm)

6. CONCLUSIONS

Based on a procedure following three stages, the following conclusions can be drawn:

1. The overall evaluation has shown that this 135 year old bridge is still safe and serviceable, mainly due to the fact that it was originally designed for two tracks but in the event only carried one. Careful inspection and maintenance has ensured and will ensure that the bridge can be kept in service for many years to come.
2. Unlike during design, information about actual loads and section properties can be used during an assessment in order to more closely represent the actual behavior of a structure. More sophisticated methods like, in the present case, fracture mechanics can be efficient tools.
3. A thorough evaluation is generally cost effective because it may enable planned suburban passenger traffic to be carried without the need for costly interventions.

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