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Remaining Service Life of a Riveted Railway Bridge
Durée de vie restante d'un pont-rail riveté
Restnutzungsdauer einer genieteten Eisenbahnbrücke

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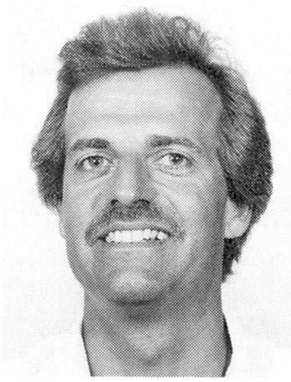

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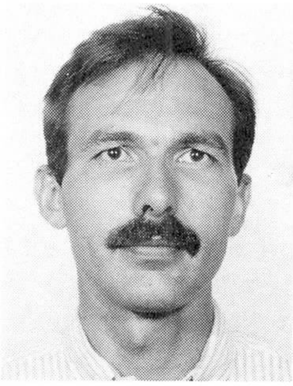

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SUMMARY
The remaining service life of a riveted wrought-iron bridge from the last century is assessed. The case study concentrates on the application of probabilistic concepts for evaluating fatigue safety. The theoretical studies are complemented by strain measurements on the most fatigue critical members and by a thorough visual bridge inspection. Based on the results of this study, allowable rail traffic load and optimum inspection intervals are established.

RÉSUMÉ
La durée de vie restante d'un pont-rail riveté en fer-puddlé du siècle passé est évaluée. L'étude se concentre sur l'application de méthodes probabilistes pour vérifier la sécurité à la fatigue. Ces études théoriques sont complétées par des mesures d'élongation sur des éléments de pont les plus critiques concernant la fatigue et par une inspection visuelle du pont. Sur la base des résultats de cette étude, la charge de trafic admissible et les intervalles d'inspection sont établis.

ZUSAMMENFASSUNG
Die Restnutzungsdauer einer genieteten, schweisseisernen Eisenbahnbrücke aus dem letzten Jahrhundert wird untersucht. Die Fallstudie konzentriert sich auf die Anwendung probabilistischer Methoden für die Ermittlung der Ermüdungssicherheit. Die theoretischen Studien werden ergänzt mit Dehnungsmessungen an ermüdungskritischen Bauteilen und einer visuellen Inspektion. Basierend auf den Ergebnissen dieser Studie werden zulässige Bahnlasten und optimale Inspektionsintervalle festgelegt.

1 INTRODUCTION

Riveted bridges were built until the 1950s. There are thousands of riveted bridges around the world which are still in service. Most old bridges have been restricted to reduced permissible loads or repaired; critical elements have been reinforced or even replaced. Some bridges are subjected to more severe loading than was envisaged during their design due to increased traffic and higher axle loads. Economically, it is not possible nor justified to replace all bridges when they reach their "design lives". Often there is a considerable reserve and a remaining service life may be justified provided that inspection guidelines are followed.

This paper deals with the investigation of the reliability and remaining service life of a 118 year-old bridge on a branch railway line. Standard engineering methods revealed various bridge members to be fatigue critical. Since fracture of a chord member could result in collapse of the bridge, a severe load restriction was imposed on the bridge. This load restriction hampered the freight service on the line resulting in substantial financial loss. As a consequence, a thorough investigation of the structural reliability was conducted applying, the method for evaluating the fatigue safety as developed in [1] and summarized in a companion paper [2].

2 DESCRIPTION OF THE BRIDGE

The investigated bridge crosses the Rhine river in Northern Switzerland near Lake Constance. It was built in 1875 and comprises riveted wrought iron members. The straight through truss bridge is a continuous girder bridge over four spans with a total length of 254 m (Fig. 1). The two main girders are of a truss-type construction with parallel chords and cross-wise diagonals. The single track is not ballasted; the timber sleepers are fixed directly to the stringers.

The bridge structure has been strengthened several times to allow for higher traffic loads. At the end of the last century, all major elements of the bridge were reinforced. In 1936, the stringers were strengthened for a second time, and in 1964 several truss nodes of the main girders were reinforced using high-strength bolts. Apart from some local corrosion, the present state of the bridge structure is satisfactory.

From 1875 until 1968, a mixed freight and passenger traffic of about 25 trains per day was crossing the bridge. Since 1969, this branch line has been exclusively in service for a daily traffic of 15 freight trains. To date, a total of 850'000 trains have crossed the bridge which is a moderate traffic compared to main lines with a daily traffic of about 120 trains.

3 LOAD CARRYING CAPACITY

A structural analysis is performed first to determine the load carrying capacity of the bridge. The action effect S is calculated using actions and load factors according to the Code SIA 160 [3] with the load model corresponding to UIC 702 [4]. The resistance R for wrought iron is chosen based on [5, 6, 7] which include test results for wrought iron specimens from bridges built in Switzerland in the last century. The load carrying capacity is expressed by

$$\mu = \frac{R/\gamma_R}{S} \tag{1}$$

μ : load carrying capacity ratio
R : resistance
γ_R : resistance factor (=1.20)
S : action effect

For $\mu > 1$, the investigated element fulfills the structural safety required by SIA Code. For $\mu < 1$, the load carrying capacity in terms of admissible rail traffic load is established. The structural analysis reveals that the main girders are the governing elements regarding load carrying capacity of the bridge. The minimum value for μ, equal to 0.80, is obtained for the top chord members over the piers.

4 FATIGUE SAFETY

4.1 Proceeding in stages

The fatigue safety is verified using a method described in [1], proceeding in stages with increasing level of sophistication:
1. Simplified deterministic method (chap. 4.2)
2. Simplified probabilistic method (chap. 4.3)
3. Inspection intervals (chap. 4.4)
4. Detailed probabilistic method (chap. 4.5)

The aim of the first stage is to identify the fatigue critical members of the structure. The probability of failure p_f due to fatigue is calculated in stages 2 and 4. The probability of crack detection $\bar{p}(cd)$ during inspection is evaluated in stage 3, and subsequently linked to p_f to obtain the probability of rupture p_{rupt} :

$$p_{rupt} = p_f \cdot \left[1 - \bar{p}(cd)\right] \tag{2}$$

p_{rupt} : probability of rupture
p_f : probability of failure
$\bar{p}(cd)$: probability of crack detection in a construction detail

The probability of rupture can also be expressed by means of the reliability index β_{rupt} according to the standard normal distribution. Finally the reliability of a structural element is compared to the target value β_t :

$$\beta_{rupt} > \beta_t \tag{3}$$

β_{rupt} : reliability index with respect to rupture
β_t : target reliability index

The target value β_t for fatigue safety is adopted based on proved values assumed in the design of structures where a reliability index of 4.7 is often used; this corresponds to a probability of rupture of about 10^{-6} per year. Assuming a design life of 100 years, p_t drops to 10^{-4} or β_t = 3.7 over the entire service life of the structure. The probability of rupture due to fatigue should not be larger than structural safety. Consequently, a target value β_t = 3.7 is considered in this study for the total collapse of the structure resulting from fatigue failure of the detail being assessed.

4.2 Simplified deterministic method

The fatigue safety of all bridge members is checked by the following ratio:

$$\nu_{fat} = \frac{\Delta\sigma_C/\gamma_{fat}}{\Delta\sigma_e} \quad (4)$$

with

$$\Delta\sigma_e = \alpha_{SIA} \cdot \alpha_N \cdot \Delta\sigma(\Phi \cdot Q_{fat}) \quad (5)$$

ν_{fat} : fatigue safety ratio
$\Delta\sigma_C$: fatigue resistance to $2 \cdot 10^6$ cycles (= 67 N/mm^2) [5, 7, 8]
$\Delta\sigma_e$: fatigue load effect refered at $2 \cdot 10^6$ cycles
γ_{fat} : fatigue resistance factor
α_{SIA} : correction factor from the SIA Code 161
α_N : correction factor taking into account the number of trains in the past [1]
$\Delta\sigma(\Phi \cdot Q_{fat})$: stress difference due to the fatigue load defined by UIC (International Union of Railways) multiplied by the dynamic factor

The resistance factor γ_{fat} is adopted to obtain a reliability index β_t of 3.7 [1]. For redundant details causing local failure, γ_{fat} = 1.20 is chosen, and for elements leading to total collapse of the structure, γ_{fat} = 1.34 is considered.

Based on this deterministic method, the bridge members are compared, and a ranked list identifying fatigue critical bridge details is established. Details with $\nu_{fat} < 1$ require further investigation and inspection. Fatigue safety is verified if $\nu_{fat} > 1$.

The lowest ν_{fat}-values are calculated for truss elements of the main girders. This is an uncommon result because frequently the stringers are the most fatigue critical elements of old bridges with non-ballasted track. This result is explained by the various strengthening measures; stringers and cross girders were reinforced more than the main girders.

Due to the continuity of the bridge over four spans, the influence line for the chord members (Fig. 4) shows both high compressive and tensile stresses at the zero-moment locations. The fatigue action is thus greater than the action effect that was considered for structural safety during design and strengthening. Consequently, the simplified deterministic method identifies the chord members at zero-moment locations between the nodes 7 and 8, 12 and 13 as well as 19 and 20 as the most fatigue critical members of the main girders (Fig. 1).

The minimum value for v_{fat} (= 0.69) considering 850'000 trains is calculated for chord member 18/19. This is considerably smaller than 1, and a more thorough investigation based on probabilistic concepts is therefore performed.

4.3 Simplified probabilistic method

The reliability index for fatigue failure of a bridge member is evaluated as follows:

$$\beta_f = \frac{m_R - m_S(N_{fut})}{\sqrt{s_R^2 + s_S^2}} \tag{6}$$

m_R : mean of the fatigue strength (= $\log\Delta\sigma_C + 2\cdot s_R$)
$m_S(N_{fut})$: mean of the fatigue load effect as a function of the number of future trains N_{fut}
s_R : standard deviation of the fatigue strength (= 0.11)
s_S : standard deviation of the fatigue load effect (= 0.04)

The background and the assumptions as well as the results and the correction factors needed to calculate m_S are presented and discussed in [1, 2].

The reliability index β_f as a function of the number of future trains N_{fut} is calculated. The probability of crack detection due to inspections is not considered, thus $\bar{p}(cd) = 0$; and from equation (2) follows: $\beta_{rupt} = \beta_f$. The result is shown in Figure 2 with N_{fut} represented on a log-scale.

The reliability index is now compared to the target value $\beta_t = 3.7$. The calculated β-value of chord members 18/19 is smaller than β_t; a more detailed analysis is necessary. There are two ways to influence the calculated reliability index:
(1) to optimise the **inspection intervals**, considering the probability of crack detection, and
(2) to refine the assumptions and calculate p_f using a **detailed probabilistic method**.

4.4 Inspection intervals

By a fracture mechanics analysis, the critical crack length of the most likely crack in chord member 18/19 is calculated using the R6-method [9]. Assuming fracture toughness values K_{IC} of 1500 N/mm$^{3/2}$ and 2000 N/mm$^{3/2}$, critical crack sizes a_c of 70 and 100 mm respectively are calculated. Subsequently, the probability of detecting a crack shorter than the critical crack is assessed. For this, the relation between the probability of crack detection and visible crack length must be known for steel bridges. Due to the lack of relevant information, an estimate based on literature regarding offshore structures is adopted [1].

Figure 3 shows the probability of crack detection $\bar{p}(cd)$ as a function of the number of trains between two inspections $N_{p,insp}$. Inspection intervals can be fixed; for example for

$\bar{p}(cd) = 0.95$, the chord member must be inspected after every 17'000 trains. Putting $\bar{p}(cd)$ equal to 0.95 in equation (2) a value of 3.3 is obtained for β_{rupt} for the present state, which is still smaller than the target value (Fig. 2).

4.5 Detailed probabilistic method

For the detailed probabilistic analysis, refined rail traffic models for the past and future are needed. The rail traffic model in the past is based on statistical data provided by the Swiss Federal Railways regarding the number of trains and towed load. The rail traffic model for the future considers the expected freight traffic on this line.

The calculated reliability index as a function of the number of future trains is shown in Figure 2. β is equal to 3.4 for N_{fut} = 150'000. Fixing the inspection intervals so as to give a 95 % probability of crack detection, a reliability index β_{rupt} of 4.15 is obtained using equation (2), which is greater than the target value β_t of 3.7. Fatigue safety could also be verified for even more trains; yet the bridge owner is primarily interested in the reliability of the bridge for the next 25 years.

Figure 2 shows a striking increase in β from the simplified to the detailed probabilistic method. This is explained by the significantly smaller towed load and length of the trains on this branch line when compared to the traffic load models used in the simplified method. For main lines with usually higher loads and longer trains, there is a better agreement between service and traffic model loads, and consequently, the difference in β between the simplified and the detailed probabilistic method is smaller.

5 FIELD TESTING

The main objective of the field testing is to verify the structural models. Top and bottom chord members between the nodes 18 and 19 (Fig. 1) have been equipped with strain gauges, and the strain history due to an engine of six axle loads equal to 177 kN has been recorded. There is a small difference between the measured and calculated influence lines for the bottom chord member 18/19 (Fig. 4); such good agreement is frequently observed for truss girders.

6 BRIDGE INSPECTION

The field investigation included a thorough visual bridge inspection. Cracks have been detected at details with sharp corners located at the reinforced parts of the stringers. These cracks may not affect the structural integrity because crack propagation is likely to stop. Such "undangerous" cracks are usually not revealed by analysis, which emphasizes the importance of bridge inspection and study of construction details.

"Dangerous" fatigue cracks affecting the structural integrity (such as those investigated in the foregoing sections) could not be detected. This is in agreement with the theoretical investigation predicting that - at present - any fatigue crack is likely to be shorter than the crack that could be detected by visual inspection.

7 CONCLUSIONS

1. Comprehensive assessment of the remaining service life of railway bridges includes theoretical studies of the structural reliability, bridge inspection and field testing.
2. A method for the evaluation of fatigue safety which proceeds by stages of deterministic and probabilistic calculations of increasing sophistication has been demonstrated. Probabilistic methods enable the consideration of the scatter of the parameters that influence both the fatigue strength and damage accumulation.
3. Based on fracture mechanics and probabilistic concepts, inspection intervals are assessed and considered in the evaluation of structural reliability.
4. By means of strain measurements on fatigue critical members, the load carrying behaviour of the structure is evaluated and a more accurate (and usually lower) value for the stress range is determined, which is the most dominant parameter influencing the fatigue safety.

As a consequence of this study, the severe load restriction imposed on the bridge was removed for the next 170'000 trains. In addition to the main inspection every 5 years, a supplementary inspection of the fatigue critical chord members of the main girders is now conducted at yearly intervals. This thorough evaluation is cost effective because it now allows for practically unrestricted rail traffic for the next 25 years.

ACKNOWLEDGEMENTS
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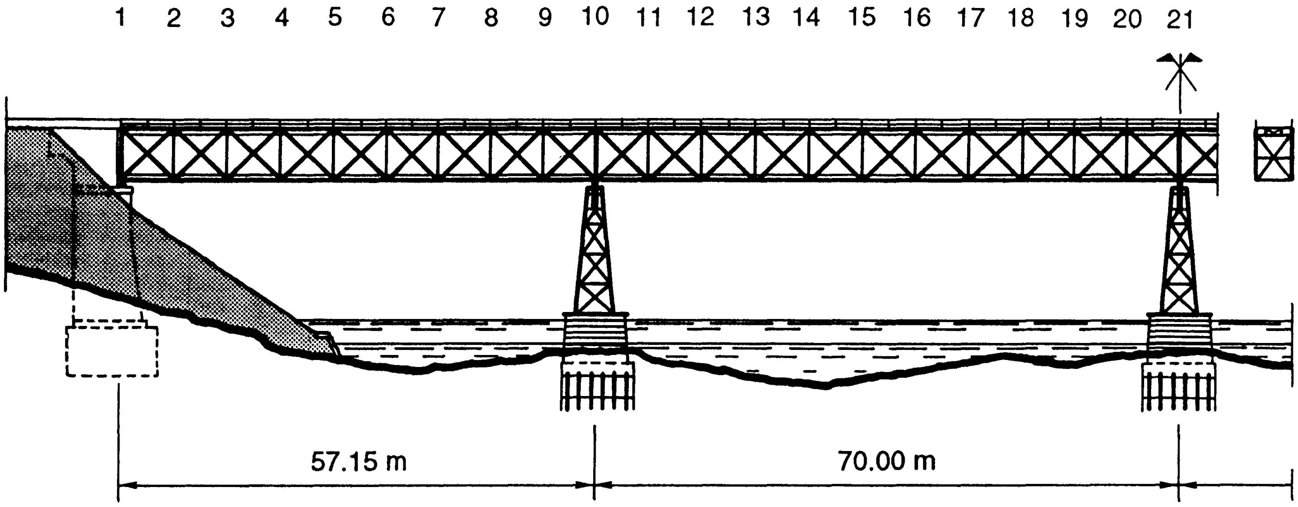

Figure 1 View of the investigated truss bridge (one half)

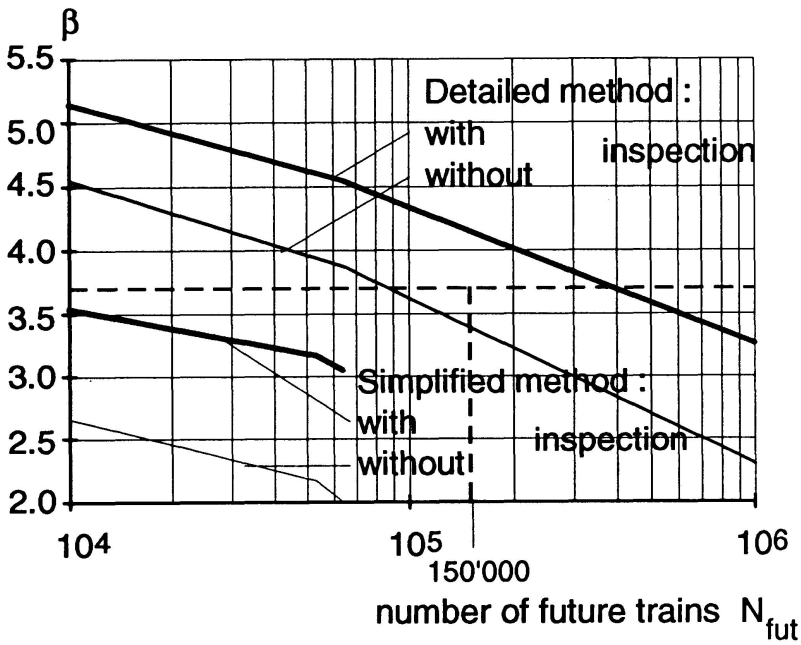

Figure 2 Reliability index for chord member 18/19 as a function of the number of future trains for simplified and detailed probabilistic approach

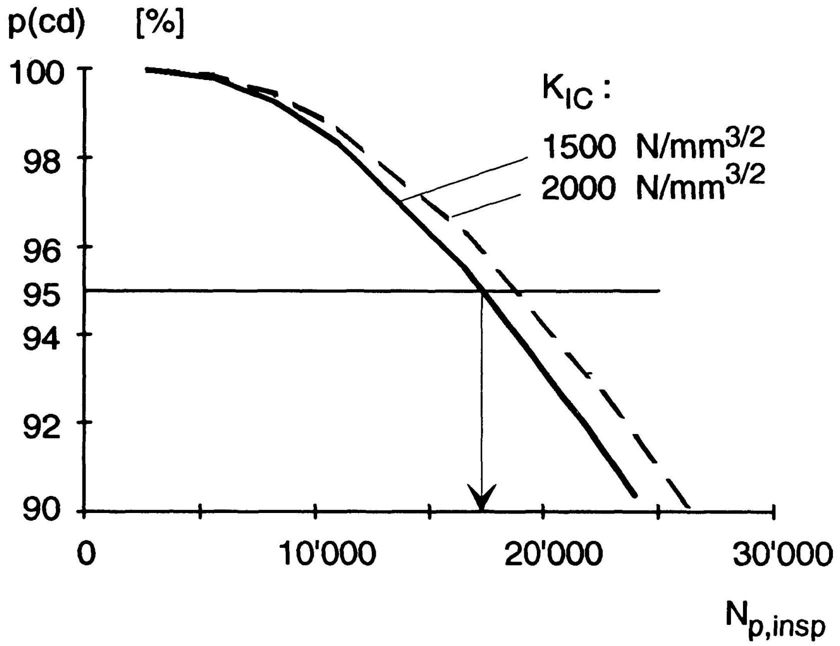

Figure 3 Probability of crack detection in chord member 18/19 as a function of inspection intervals

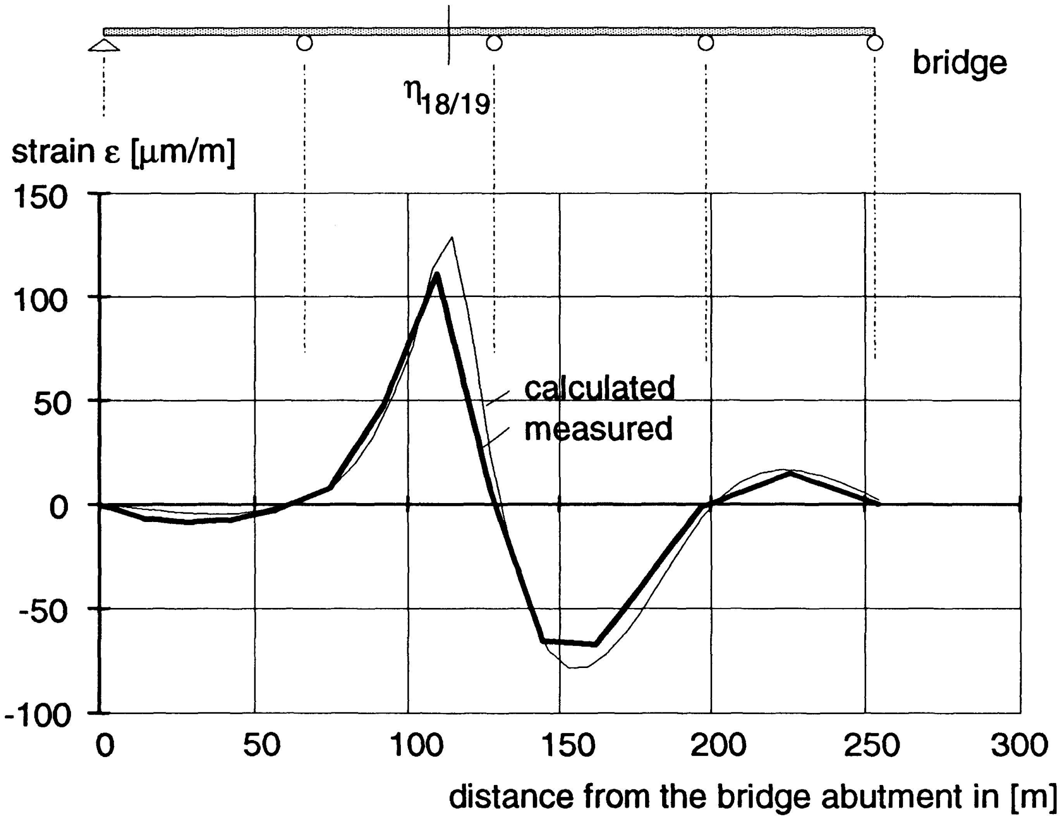

Figure 4 Comparison between calculated and measured influence lines for bottom chord member 18/19 due to the passage of the engine. (The stress values are evaluated from the strain readingsusing an elastic modulus of 200'000 MPa.)