Method to back decisions on residual safety of bridges

Method to Back Decisions on Residual Safety of Bridges

Méthode pour la détermination de la sécurité restante des ponts existants

Methode zur Bestimmung der verbleibenden Sicherheit von bestehenden Brücken

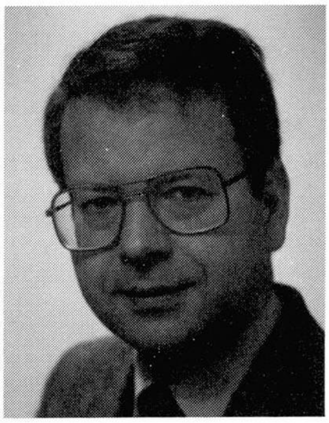
Winfried DAHL
Professor
Institute of Ferrous Metallurgy
Aachen, Fed. Rep. of Germany

Otfried SCHUMANN
Civil Engineer
Dep. of Bridges and Constr. Eng.
Duisburg, Fed. Rep. of Germany

Gerhard SEDLACEK
Professor
Inst. of Steel Construction
Aachen, Fed. Rep. of Germany

Winfried Dahl, born 1928, received his degree in Physics at the Georgia-Augusta-University in Göttingen, FRG. Since 1969, he has been the head of the Institute of Ferrous Metallurgy at the TU Aachen.

Dipl.-Ing. Otfried Schumann, born 1927, received his civil engineering degree at the Technical University of Berlin-Charlottenburg. Since 1977, he has been the head of the Dep. of Bridges and Construction Engineering of the town Duisburg.



Gerhard Sedlacek, born 1939, obtained his PhD at the TU Berlin. After work in the steel construction industry, he became the head of a department of bridge building. Since 1976 he has been Professor of the Institute of Steel Construction of the TU Aachen.

SUMMARY

This paper demonstrates a fracture-mechanics based method to determine the safety of old steel bridges for modern traffic loading and to prepare an inspection programme which enables extended service life of bridges to be justified. The method is illustrated by means of a practical example.

RÉSUMÉ

Cet article présente une méthode basée sur la mécanique de la rupture, et qui permet de déterminer la sécurité d'anciens ponts en acier soumis aux charges du trafic moderne et de préparer un programme d'inspection qui permette d'assurer une prolongation de la durée de service des ponts. La méthode est illustrée à l'aide d'un exemple pratique.

ZUSAMMENFASSUNG

Der Beitrag zeigt eine auf der Bruchmechanik basierende Methode, die erlaubt, die Sicherheit alter Stahlbrücken abzuschätzen, die modernen Verkehrslasten unterworfen sind, sowie Inspektionsmassnahmen zu planen, mit deren Hilfe ein sicherer Betrieb garantiert werden kann. Die Methode wird an einem praktischen Beispiel dargestellt.

1. Introduction

A great part of existing steel bridges have been built in the last century, fig. 1, and since then have undergone several phases of repair or strengthening after damages in the world wars or due to changes of service requirements.

For those bridges very often the question of the actual safety for modern traffic loads and the remaining service life is put forward.

This paper describes a procedure how the residual safety and service life of old steel bridges may be determined and how a basis may be established on which economic decisions for further strengthening or replacement by a new bridge may be taken.

The procedure is described by the example of an old roadway bridge, the Karl-Lehr-Bridge in Duisburg. It may however be applied to any other steel bridge.

2. Problem

The structural system and the cross-section of the Karl-Lehr-Bridge can be taken from fig.2. The bridge was built up in 1907 as a part of the Hohenzollern-Bridge over the river Rhein in Cologne and was blasted in the 2nd world war. After the war it was repaired and shipped on pontons to the harbour of Duisburg where it was installed for bridging a canal. In 1976 it got a new orthotropic deck to enhance the capacity to the loading class BKL 60 in DIN 1072.

In 1984 a tram on the bridge ran off the rails and hit a hanger, which exhibited cracks which were classified as "brittle". This behaviour alarmed the town authority that asked for an expertise that should answer the following questions:

1. Is the bridge sufficient safe for actual service conditions?
2. If so, what is the expected residual life and what are the requirements for inspection and maintenance to assure the expected residual life.

3. General conditions concerning "brittleness" and "ductility"

For answering the above mentioned questions a definition of different failure modes in view of "brittleness" and "ductiltity" is necessary.

The technical stress-strain curve obtained for tension coupon tests, see fig. 3, reveals a maximum, which can be explained by the stability criterion

$$\delta R_t = A_m \cdot \delta\sigma_w + \sigma_w \cdot \delta A_m = 0$$

hence
$$\frac{\delta A_m}{A_m} = -\frac{\delta\sigma_w}{\sigma_w}$$

where A_m is the actual cross sectional area
σ_w is the true stress, taken from the true stress-strain curve in fig. 4.

From fig. 5 it can be derived that the elongation before reduction of area in the specimen is the reduced the curve the higher the yield strength of the material is.

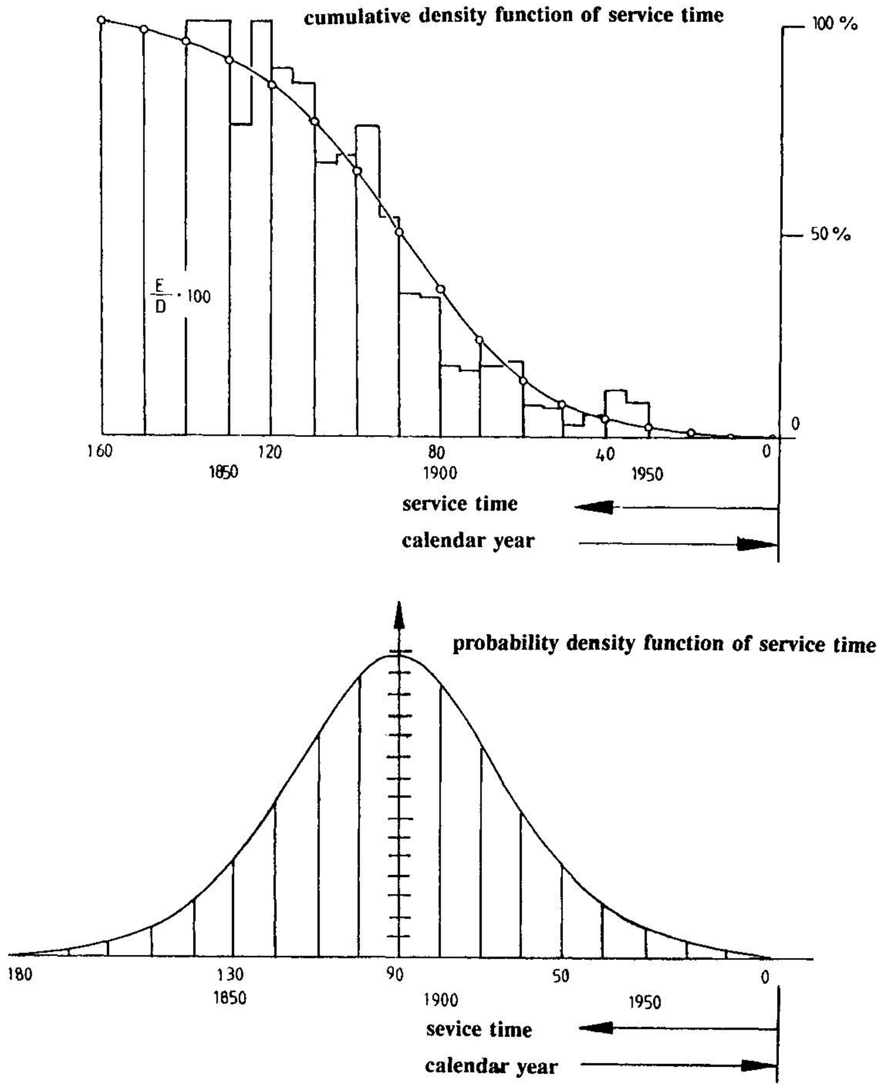
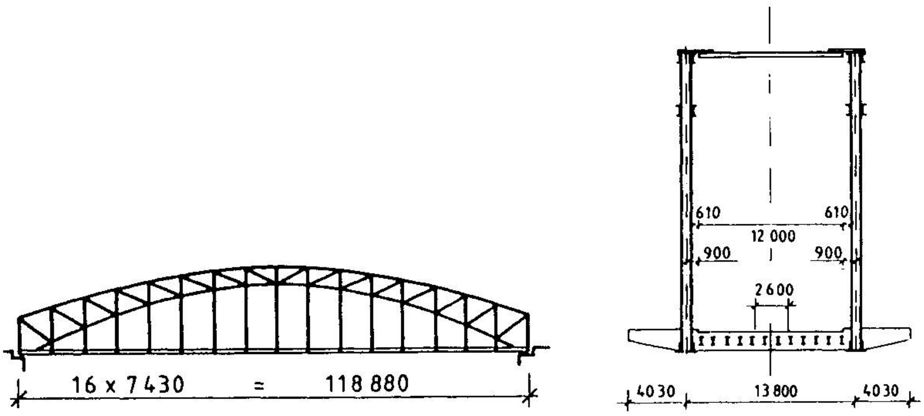


Figure 1: Distribution function of service time of existing railway bridges in steel



1907 - **year of construction (Hohenzollern-Bridge)**

after 1945 - **repairing (Karl - Lehr - Bridge)**

1976 - **Strengthening-Orthotropic deck**

Figure 2: Structural system and cross-section of the Karl-Lehr-Bridge

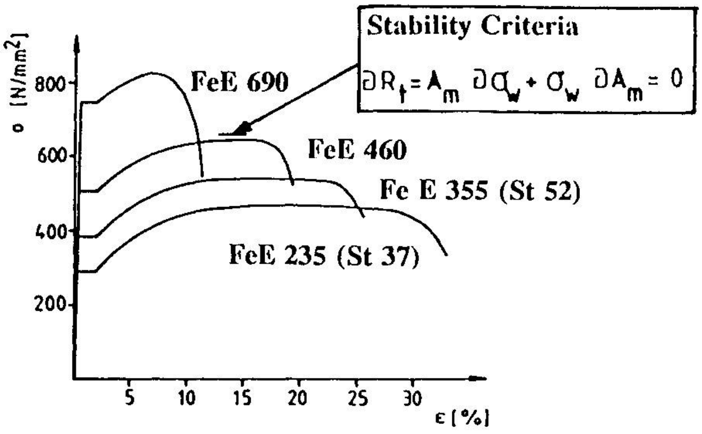


Figure 3: Technical stress-strain-curve

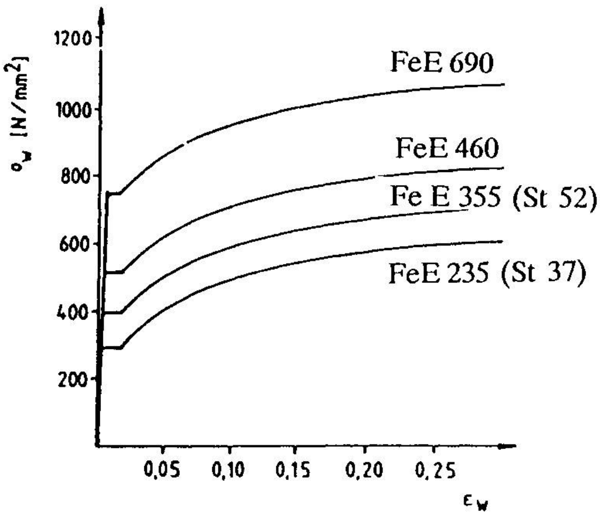


Figure 4: True stress-strain-curve

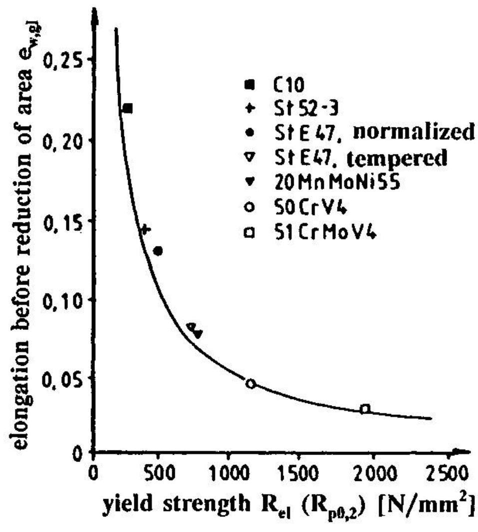


Figure 5: Elongation before reduction of the area $e_{w,gl}$ plottet versus yield strength R_{el}

A structural member may show different failure modes, which may be best distinguished for the example of a plate with a central crack in tension, fig. 6:

1. Unfavourable failure is exhibited when fracture occurs before net section yielding with only local yielding at the crack tips.

   In this case all actual stresses in the net section comprising the stresses from external loads including notch effects, residual stresses and stresses due to other restraints have to be taken into account.

   This failure mode commonly is called "brittle" failure.

2. If failure in structural applications occurs by fracture after net-section yielding, only the nominal stresses due to external loads in the net section are relevant, and notch effects, residual stresses and stresses due to other restraints may be neglected.

   This and all following modes are called "ductile" failures.

3. Another ductile failure mode, that may be required for plastic zones in plastic hinges or dissipative zones in seismic design is achieved by fracture in the net section after gross-section yielding.

   In this case the stresses from external loads only in the gross section are controlling the design of the member and the net section is capacity-designed.

The failure mode is mainly influenced by the material, the temperature, the loading rate and the shape of the structural member (state of stress).

For the safety assessment of old steel bridges of the kind of the Karl-Lehr-Bridge in general the first and the second mentionend failure modes are relevant, as the assessments have to be carried out for design situations with low temperatures and assumed cracks.

4. General procedure

The following steps are necessary to determined the actual safety and residual life of the structure.

1. Establishment of a failure scenario, where the consequences of failure of the different bridge elements for different design situations are investigated. Those bridge elements are identified as vital elements the failure of which would cause an immediate overall collapse.

   For the Karl-Lehr-Bridge the vital element proved to be the tension tie, see fig. 7, as all other members are either redundant or stressed so little that they do not produce risks.

2. As far as the vital elements may fail by fracture due to tension loads, they are assessed in the following way:

   a) Several loading cases are determined with combinations of self weigth, traffic loads including dynamic impact and temperature and with or without residual stresses and restraints depending on the expected failure mode.

      For these loading cases and a crack situation in the vital element that is assumed such that the crack sizes just reach the size of detectibility the applied fracture mechanics action effects in terms of J_{appl}, see chapter 5, is calculated.

   b) From miniaturized plate samples which are drilled from the vital elements at locations where they don't reduce the safety, the fracture mechanics material resistance in terms of J_{crit} for different temperatures is determined, fig. 8, which allow to carry out a safety check by

      $$J_{appl} \leq J_{crit}.$$

      From this safety check and by varying the crack length assumtions a critical crack length can be determined which indicates what amount of crack extension beyond the size of

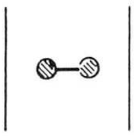
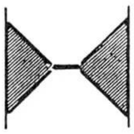
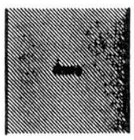
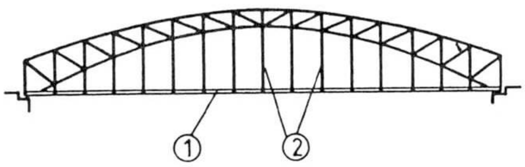
yielding pattern	failure mode	design values
	fracture before net-section yielding brittle	applied stress distribution in the net-section + residual stresses + restraints
	fracture after net-section yielding ductile	applied nominal stress distribution in the net-section
	fracture of the net-section after gross-section yielding ductile	applied nominal stress distribution in the net-section

Figure 6: Definition of failure modes and the applied design values of stresses dependant on the ductility level

Identification of those bridge elements where the consequence of failure would cause an immediate overall collapse of the bridge (→ vital elements)



1. Tention ties of the bridge
2. Hangers (redundant)
3. All other members are either redundant or stressed so little that they do not produce any risk

Figure 7: Vital elements of the Karl-Lehr-Bridge

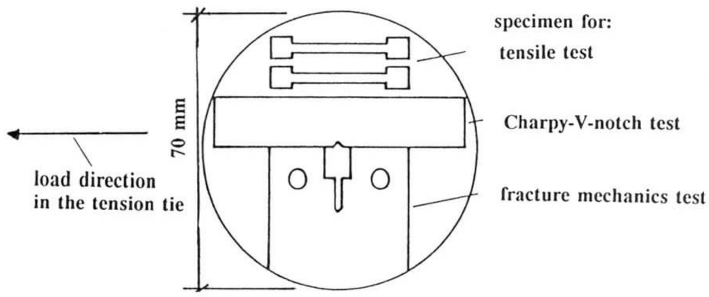


Figure 8: 'Miniaturized test element and available test specimens for different tests

detectibility is tolerable.

3. From observations of the actual traffic situations on the bridge and extrapolation to future developments a fatigue load is defined which allows to determine the residual service time of the bridge on the basis of the crack propagation rate from the detectable crack to the critical crack size with a fracture mechanics model.

   The procedure developed in [1] is illustrated in fig. 9, in which the vital member resistance is plotted versus service life, due to fatigue induced crack propagation. It has been applied to different railway bridges, e.g. [2], [3].

The main advantages of this procedure are the following:

1. It can be demonstrated that cracks with detectable sizes can be accepted without catastrophic consequences and no collapse (without prewarning) may take place.

   If this check is not possitive, the member has to be strengthened with tough material or to be replaced before the next cold season (loss of toughness at low temperatures).

2. It can be demonstrated that the crack extension from the detectable crack size to the critical crack size takes sufficient time, to allow for economic intervals of inspection.

   If this check is not possitive a strengthening with tough material or a replacement should be considered.

3. In case of both checks being positive the inspections at safe intervals at the critical location of the vital elements will allow the following conclusions:

   - As long as no cracks are observed, the structure is fit for use for at least the service period up to the next inspection.
   - This statement can be repeated up to the inspection when first cracks are found.
   - In case cracks are found there is sufficient time left to react by replacing the members or the total bridge.

5. Justification of the procedure

The procedure is based on the J-Integral [4],[5], fig. 10

$$J = \int_{\Gamma} \left(W \cdot \delta y - T \cdot \frac{\delta u}{\delta x} \cdot \delta s\right)$$

where
W = Energy density
T = Vector of stresses
u = Vector of displacements
Γ = Integration path around the crack tip
δs = element of the integration path

The crack driving force in terms of the J-Integral J_{appl} can be calculated by FE-analysis with a grid of collapsed iso-parametric elements, fig. 11.

The material toughness in terms of the J-Integral J_{crit} can be evaluated from the fracture mechanics test specimens, see fig. 8 and can be interpreted as the energy which leads to crack tip opening just before crack extension, fig. 12. This represents a conservative assumption for the ultimate limit state, because it neglects the reserves that may be produced by stable carck extension after initiation.

The critical material resistance values J_{crit} obtained from the small scale test specimens taken from the tension tie and tested at a temperature T = -30°C are plottet in fig. 13, the lowest value found was 42

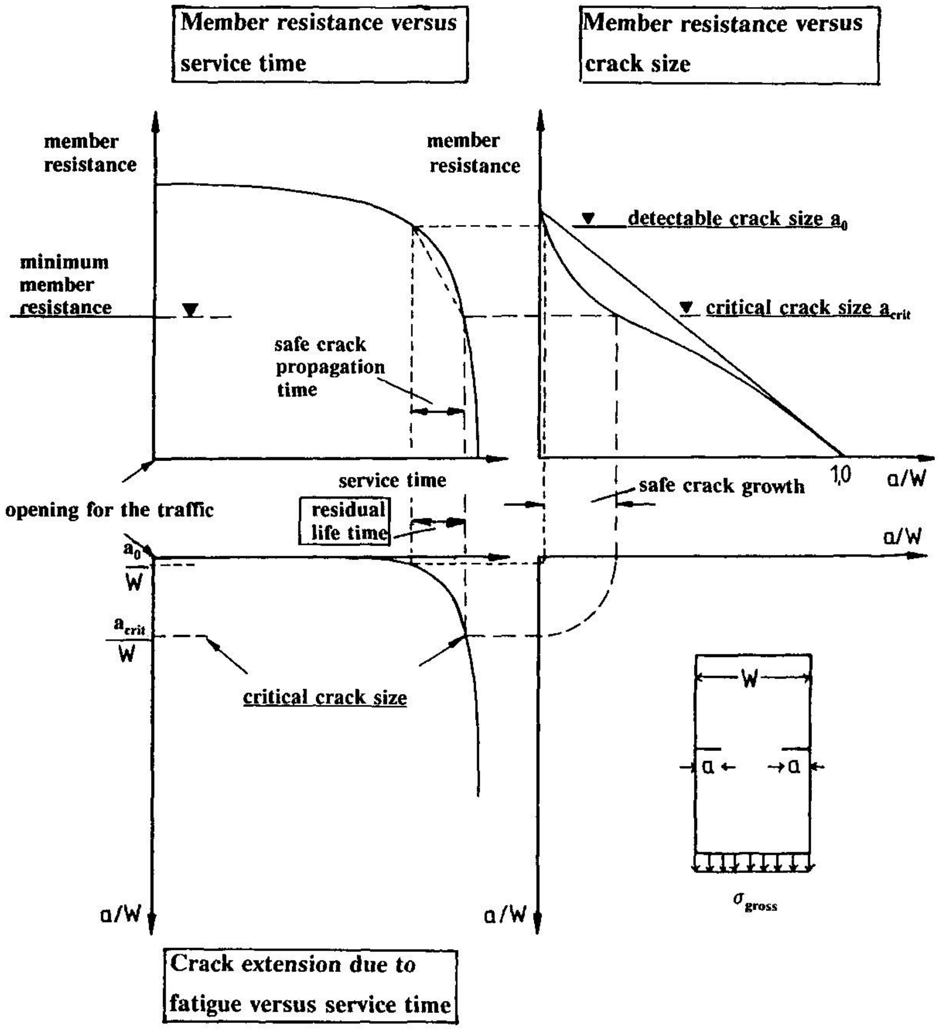


Figure 9: Relation between the member resistance and the crack sizes as a function of the service time

$$J = \int_{\Gamma} \left(W \cdot \delta y - T \cdot \frac{\delta u}{\delta x} \cdot \delta s \right)$$

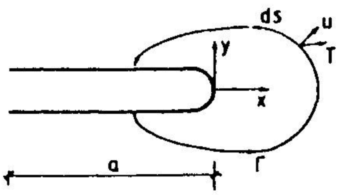


Figure 10: Definition of the integration path for the J-Integral calculation

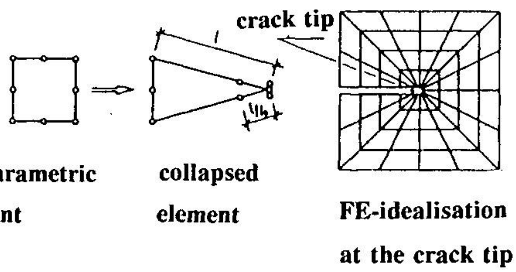


Figure 11: FE-elements and FE-grid for the calculation of J_{appl}

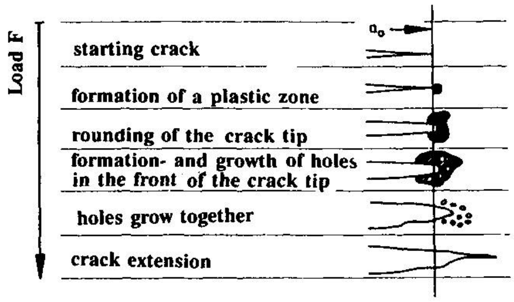


Figure 12: Procedure of crack propagation

temperature [°C]	J_C (N/mm)
- 30°	77
- 30°	100
- 30°	119
- 30°	125
- 30°	116
- 30°	85
- 30°	42
	$\bar{J}_c$ = 664:7 = 95 N/mm $\bar{\sigma}_J$ = 29.4 N/mm

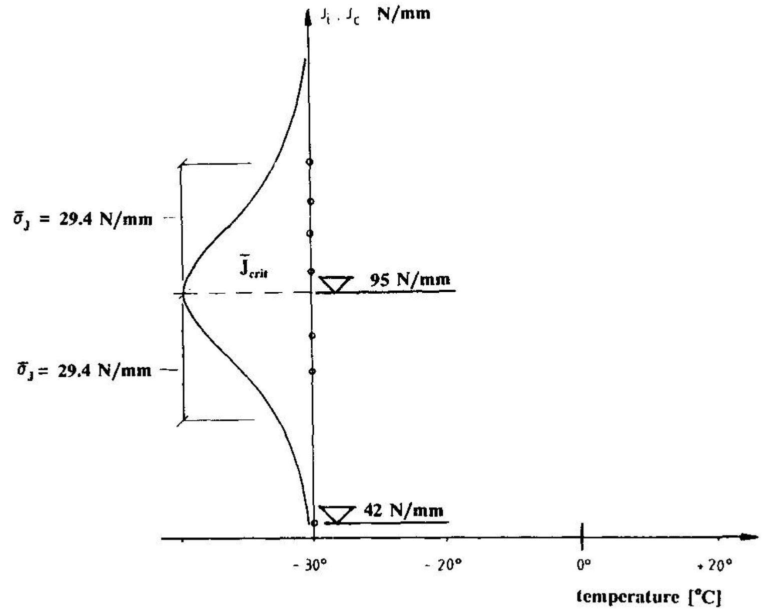


Figure 13: Critical material resistance values J_{crit} obtained from small scale test specimens

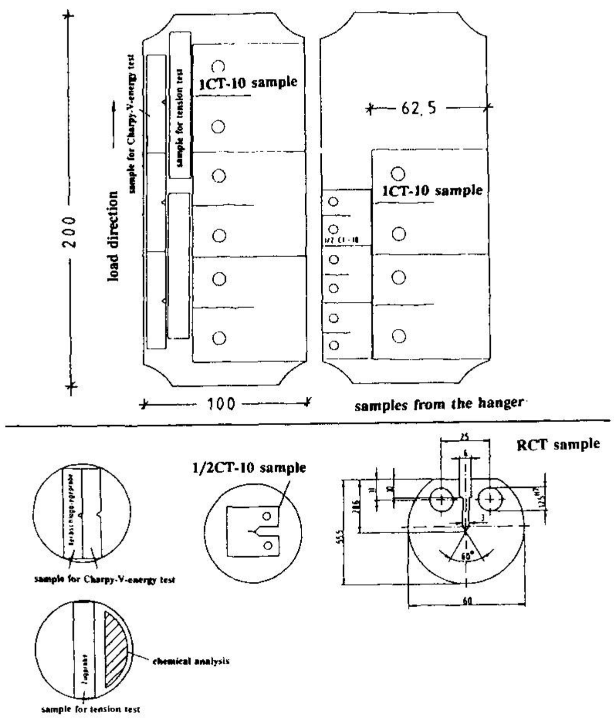


Figure 14:

Test specimen taken from the hangers

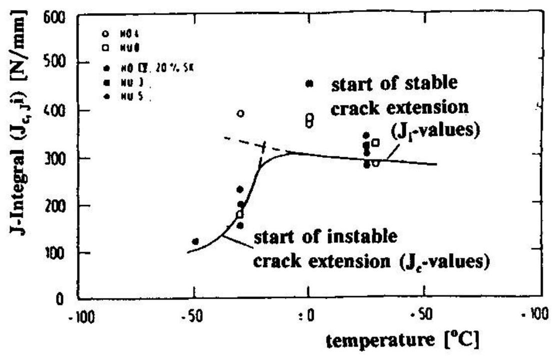


Figure 15: J-curve versus temperature

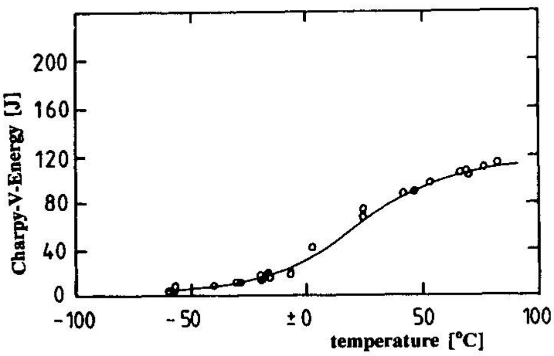


Figure 16: Charpy-V-energy-temperature curve

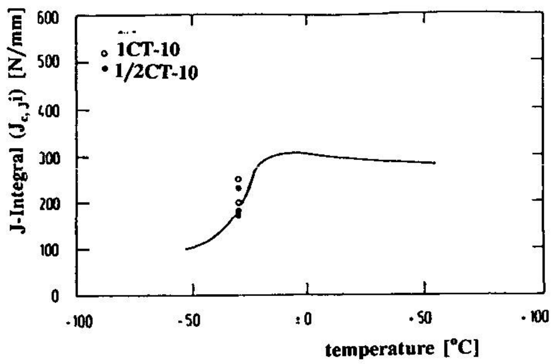


Figure 17: Comparison between 1CT-10 and 1/2CT-10 samples

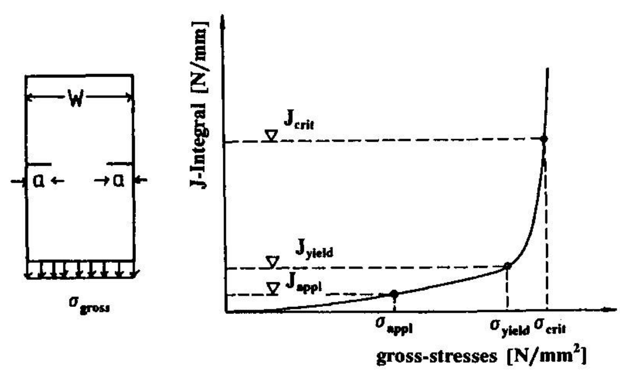


Figure 18: J_{appl}-curve

N/mm.

For the hangers of the bridge, which were not vital elements comparative tests could be made to determine possible scaling effects due to the miniatured specimen and a correlation to the results of Charpy-V-energy tests if there are any. For this reason test specimens of different sizes were taken from the hangers, see fig. 14. The 1CT-10 samples allowed to determine the J-Integral transition curve, fig. 15, which gives no correlation to the Charpy-V-energy-temperature curve fig. 16, which for safety assessments can only be used qualitatively.

A comparison between the results from 1CT-10 samples and 1/2CT-10 samples is given in fig. 17. It demonstrated that the 1/2CT-10 samples give conservative data.

For a given structural member and a given crack location and crack size the curve of J_{appl} versus the applied gross-stresses may be calculated if the true stress-strain curve of the material is known, fig. 18. On this J_{appl}-curve J_{yield}, where net section yielding is reached, and J_{crit}, where crack extension starts, are particular values. The case of brittle failure occurs if $J_{crit} < J_{yield}$, else net section yielding will occur before fracture is expected.

The reliability of the prediction of the ultimate resistance of structural members by the J-Integral method has been proved by a series of justification tests with big plates [6],[7].

6. Application and results of the procedure

Fig. 19 demonstrates the cross section of the tension tie which is riveted, and three alternative models (I, II, III) for the fracture mechanics assessment of this tie.

In assuming that fatigue cracks would initiate in the parent metal under the heads of the rivets [8], a starting crack length is considered which represents the rivet hole plus two cracks at both sides of the rivet grown prependicular to the direction of the applied stresses, that have reached the edge of the rivet head (a = 21.5 mm), fig. 20.

For the given true stress-strain curve the J_{appl}-curves for the models I, II, and III were calculated as plotted in fig. 21 and model II was chosen as relevant.

In fig.22 several J_{appl}-curves are given starting from the basic curve for a = 21.5 mm up to a = 75 mm.

The relevant load case was represented by a temperature of -30°C which was combined with the applied stress due to self weigth and full traffic load as specified in DIN 1072. This gives a total gross stress σ_{appl} = 114 N/mm^2 for the ductile failure mode and σ_{appl} = 234 N/mm^2 for the brittle failure mode, where residual stresses have to be added.

The safety check has been made for the following cases, see fig. 22:

1. For the minimum material value of J_{crit} = 42 N/mm the brittle failure mode is relevant and the tolerable crack length is a = 34.5 mm i.e. 13 mm crack growth beyond the edge of the rivet head.

2. In the second case, two of the four plates, that represent the tension tie, would fail, and the plates that survive would exhibit a material value J_{crit} = 70 N/mm, the behaviour of the latter could be classified as ductile failure mode and the applied stress they get would be 222 N/mm^2. For this case the tolerable crack length would be greater than a = 34.5 mm. So the first case with a = 34.5 mm is relevant.

The crack propagation calculation is based on measurements of the traffic situation to get the fatigue load; from these measurements the relevant damage-equivalent stress ranges were mainly caused by multiple traffic jams which yielded to $\Delta\sigma e$ = 52.1 N/mm^2 or ΔKe = 460 N/mm$^{3/2}$ with a number of 10 cycles per day. This includes a dynamic impact factor φ = 1.20 that was taken from the measurements as well. The "Paris"-coefficients were taken as given in fig. 23.

The calculation of the crack extension yielded to the curves in fig. 24, from which the limit curve in fig. 25 could be derived.

Though the total procedure includes a set of conservative assumptions, the final proposal was not

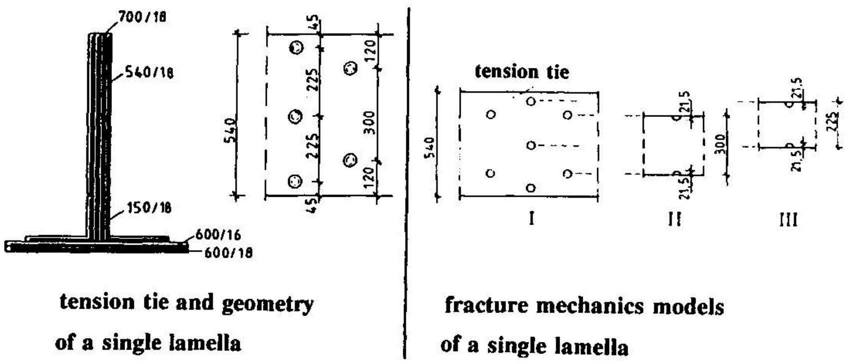


Figure 19: Cross section of the tension tie and three alternative models for the fracture mechanics assessment

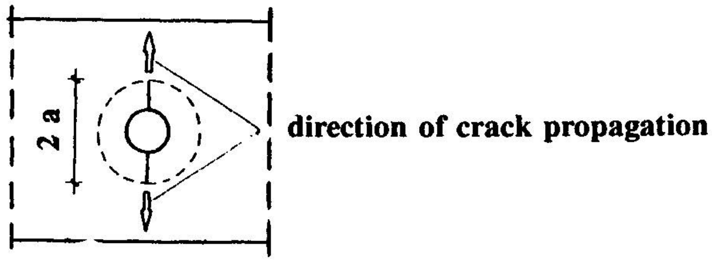


Figure 20: Definition of the starting crack size

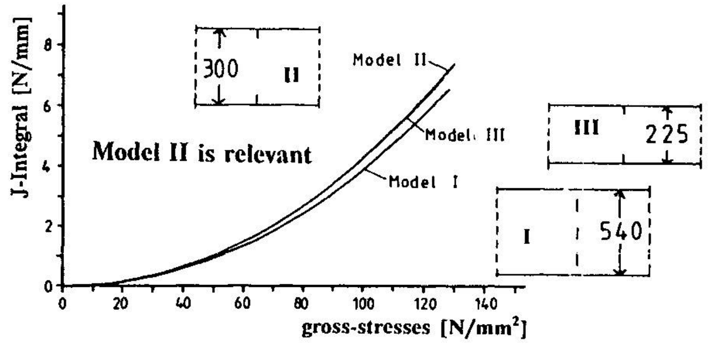


Figure 21: J_{appl}-curves for the fracture mechanical models I, II and III

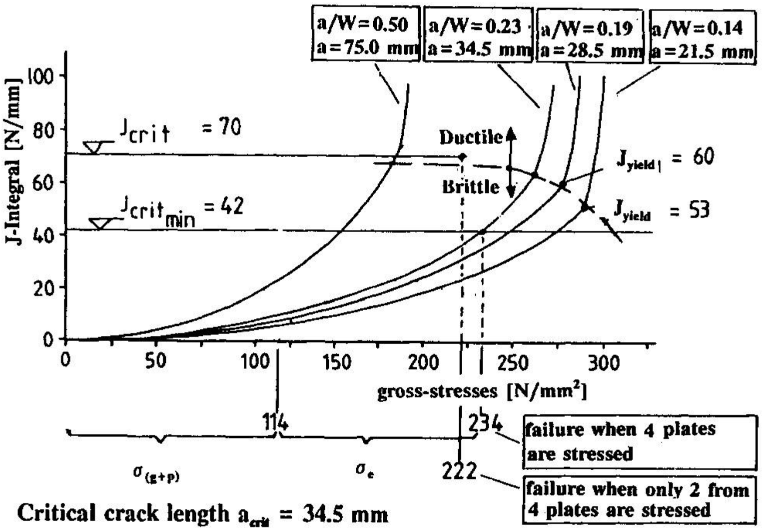


Figure 22: Determination of the critical crack size for the actual load situation

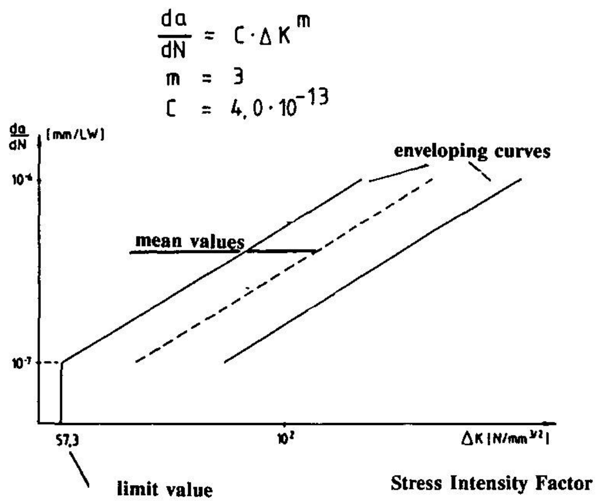


Figure 23: Determination of the crack propagation

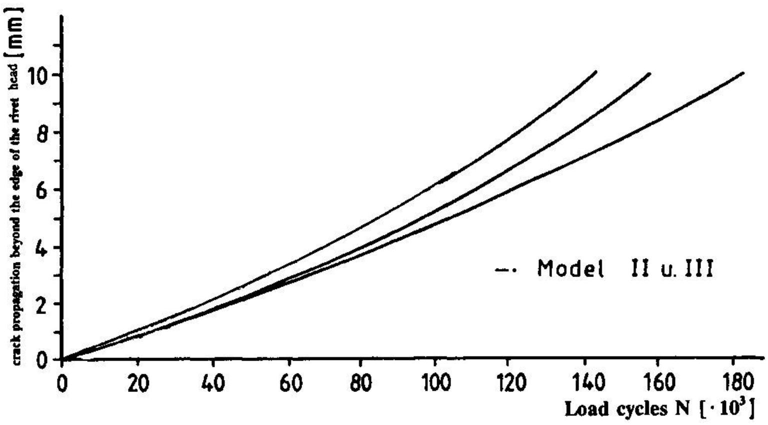


Figure 24: Crack propagation curve for the different fracture mechanical models

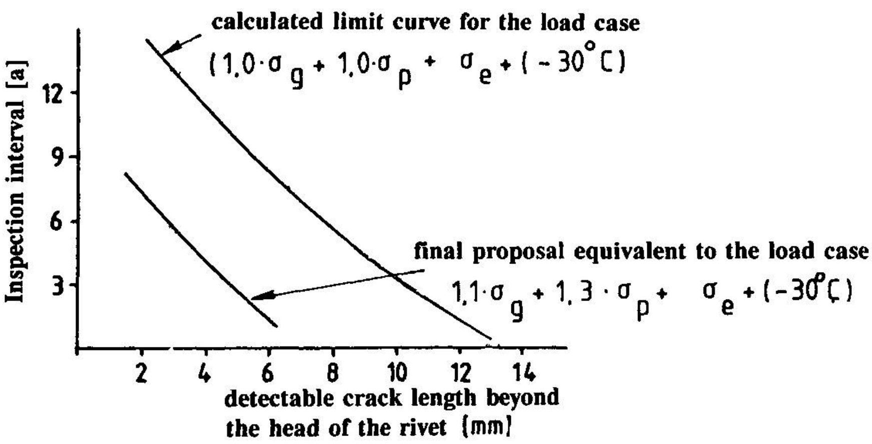


Figure 25: Crack propagation behaviour and inspection interval

based on the calculated limit curve but on a more conservative curve given in fig. 25. Details concerning the procedure can be found in [1].

The final result from all checks including the aforementioned fracture mechanics assessment was:

1. The bridge complies with the present design codes and is fit for the use with sufficient safety when no cracks at locations indicated in the crack control plan are found.

2. The inspection should be carried out every 1.5 years according to the crack control plan.

3. The bridge remains fit for the use with suffient safety as long as no crack is detected during inspection and the loading conditions do not change significantly.

7. Conclusions

A fracture mechanics based procedure for the determination of the safety and the residual life of old steel bridges is presented, that avoids the uncertainties of predictions that are linked to S-N-curves.

The method has already been applied to railway and roadway bridges, to guyed masts and rotors and is considered as a useful tool together with other engineering judgments.

At present the method is being simplified and made more operational in order to be standardized in a code that could be used for the safety assessment of old steel bridges.

8. Ackknowledgement

The afore mentioned work was mainly carried out by Dipl.-Ing. Johannson, member of the staff of Dipl.-Ing. Schumann, Dr.-Ing. W. Hesse and Dipl.-Ing. R. Hubo, Institute for Ferrous Metallurgy, Technical University Aachen and Dr.-Ing. J. Bild, Dipl.-Ing. H. Eisel and Dipl.-Ing. W. Hensen Institute for Steel Structures, Technical University Aachen.

The fruitful cooperation between the town authority in Duisburg and the institutes of the RWTH Aachen is highly appreciated.

9. References

1. BILD, J., Beitrag zur Anwendung der Bruchmechanik bei der Lösung von Sicherheitsproblemen im Stahlbau. Diss. RWTH Aachen, 1988.

2. DAHL, W., SEDLACEK, G., Untersuchungen zur Ermittlung der Sicherheit und Restnutzungsdauer der Karl-Lehr-Brücke in Duisburg. Expertice for the town Duisburg, 1986.

3. DAHL, W., SEDLACEK, G., Untersuchungen zur Ermittlung der Sicherheit und Restnutzungsdauer der Ackerfährbrücke in Duisburg. Expertice for the town Duisburg, 1986.

4. RICE, J.R. and TRACEY, D.M., J. Mech. Phys. Solids 17, pa. 201-217/1969.

5. CHEREPANOV, G.P., Crack propagation in continuous media. PMM Vol. 31, No. 3/1967, pa. 476-488.

6. DAHL, W., DORMAGEN, D., EHRHARDT, H., HESSE, W., TWICKLER, R., Anwendung bruchmechanischer Konzepte auf das Versagensverhalten von Großplatten. Nucl. Eng. and Design, Vol. 87/1985, pa. 83-88.

7. EHRHARDT, H., Untersuchungen zum Einfluß unterschiedlicher Fehlergeometrien auf das Versagensverhalten von Stahl auf der Grundlage von Großzugversuchen. Diss. RWTH Aachen, 1988.

8. BRÜHWEILER, E., ESSAIS DE FATIGUE SUR DES POUTRES A TRIPPLIS DOUBLE EN PER PUDDLE. Publication ICOM 159/1986.