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Session E2

Fatigue Issues in Bridges
Problèmes de fatigue dans les ponts
Ermüdungsfragen im Brückenbau

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Cumulative Damage Assessment in Structural Steel Details

Appréciation des dommages cumulatifs sur des détails de structures métalliques

Beurteilung kumulativer Schäden in Stahlbaukonstruktionsdetails

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SUMMARY

An approach is presented trying to unify both design and damage assessment methods for high and low cycle fatigue, based on the results of an extensive experimental research programme. Interpreting the stress range in a structural component as the one associated to the real strain range in an ideal perfectly elastic material, high and low cycle fatigue test data can be fitted by the same Whöler lines usually given in recommendations for the fatigue design of steel structures. Local buckling can be regarded as a notch effect, intrinsic to the various shapes, and related to their geometrical properties.

RÉSUMÉ

A partir d'un programme exhaustif de recherche expérimentale, l'auteur essaie de combiner entre elles des méthodes d'étude et des procédés d'appréciation des dommages provoqués par la fatigue résultant de cycles alternés de charges faibles et fortes. Étant donné que l'amplitude de la sollicitation d'un élément structural peut être saisie à partir de l'amplitude de l'allongement réel d'un matériau idéalement élastique, il est possible de représenter les données de fatigue de ces cycles de charges alternées par les mêmes lignes de Whöler, telles qu'elles sont habituellement recommandées pour le dimensionne-ment à la fatigue des constructions métalliques. Par ailleurs, un voilement local peut être considéré comme résultant de l'effet d'une concentration de contraintes, spécifique aux différentes sections et aux propriétés géométriques.

ZUSAMMENFASSUNG

Auf der Grundlage eines umfangreichen experimentellen Forschungsprogramms wird versucht, Entwurfsmethoden und Verfahren zur Schadensbeurteilung für Ermüdung unter hohen und niedrigen Lastwechselzahlen miteinander zu kombinieren. Indem die Spannungsamplitude in einem Bauteil als reale Dehnungsamplitude in einem idealelastischen Werkstoff aufgefasst wird, können die Ermüdungsdaten für hohe und niedrige Lastwechselzahlen durch dieselben Whölerlinien repräsentiert werden, wie sie üblicherweise für die Ermüdungsbemessung von Stahlkonstruktionen empfohlen sind. Oertliches Beulen kann als Kerbeneinfluss, spezifisch für die unterschiedlichen Formen und geometrischen Eigenschaften, aufgefasst werden.

1. INTRODUCTION

Eurocode-3 [1] defines fatigue as "damage in a structural part, through gradual crack propagation caused by repeated stress fluctuations". Depending on a number of factors, these load excursions may be introduced either under stress or strain controlled conditions. Depending on the number of cycles sustainable to failure, and on their amplitude, we can distinguish failure for high or low cycle fatigue.
Failure by high cycle fatigue is characterised by a large number of withstandable cycles with a nominal stress range $\Delta\sigma$ in the elastic range. This is a well known effect [2], although only a limited number of typologies of connections and of structural details can be considered at present thoroughly investigated, and general aspects of this problem and the basic methodologies for assessment and design, can be considered well established.
Low cycle fatigue is characterised by a small number of cycles to failure, with large plastic deformations. In general, low cycle fatigue problems in civil engineering structures arise either under seismic loading or in pressure vessels or under severe thermal cycling. Cycles with large amplitudes in the plastic range are usually connected with local buckling in structural members. At present, knowledge of low cycle fatigue behavior of civil engineering connections is not jet as well established as high cycle fatigue one; in particular there is no generally recognised design or damage assessment method, and a clear definition of a collapse criterion is lacking.
In this paper a procedure is described, trying to unify design and damage assessment methods for structural details under high and low cycle fatigue. After discussing the proposed approach, its experimental validation based on constant and variable amplitude cyclic test results will be presented. By transforming the nominal strain range $\Delta\varepsilon$ in an equivalent stress range ($\Delta\sigma^{*}=\Delta\varepsilon\ E$) computed by considering the material as indefinitely linear elastic, the experimental test data obtained under cycles with a constant amplitude in the plastic range can be interpolated by the same Stress range-Number of cycles to failure (S-N) lines usually given in recommendations for the (fatigue) design of steel structures (e.g. [1]). Furthermore, a linear damage accumulation model [16] (Miner's rule), together with the rainflow cycle counting method, can be adopted for the damage assessment under variable amplitude loading.

2. THE PROPOSED APPROACH

The proposed approach to unify design and damage assessment procedures for steel structures under low and/or high cycle fatigue, extensively discussed in [3], is based on the two following assumptions:

1. To know, for a given structural detail (cycled under strain controlled conditions), the relationships between the number of cycles to failure N_f and the cycle amplitude Δs, expressed in terms of generalised displacement components (i.e. of displacements Δv or of rotation $\Delta\phi$ or of deformation $\Delta\varepsilon$). These relationships have the same meaning in high and low cycle fatigue with the following difference:
   - in high cycle fatigue the component is subjected to cycles in the elastic range;
   - in low cycle fatigue the component is subjected to cycles in the plastic range .
2. Damage accumulation in a structural detail is a linear function of the number of cycles sustained by the component itself. This means that Miner's rule [16], can be applied also in low cycle fatigue.

Consequence of the second assumption is the definition of a unified failure criterion for both high and low cycle fatigue; of course appropriate S-N curves corresponding to the desired safety level should be identified. Consequence of the first assumption is the possibility to interpret low cycle fatigue with the same laws commonly accepted for high cycle fatigue. In fact, in high cycle fatigue (under stress controlled conditions) a structural component is subjected to load cycles having a constant amplitude $\Delta\ F<F_y$ (the yield strength, theoretically computed or experimentally evaluated) and the nominal stress range induced by the external load $\Delta\sigma=\Delta\sigma(\Delta F)$ (computed either theoretically or with conventional methods) is correlated to N_f, independently from the yield strength of the material.
In order to generalise this approach, for an indefinitely linear elastic material, it can be written:

$$\Delta\sigma = \frac{\Delta F}{F_y}\sigma(F_y) \qquad (1)$$

In low cycle fatigue (under strain controlled conditions), a structural component is subjected to displacement cycles having a constant amplitude $\Delta\ s<s_y$ (associated with the attainment of the yield stress in the material, that may be theoretically computed or experimentally evaluated). If the material can be regarded

as an elastic perfectly plastic one (as in the case of steel), and the hypothesis of concentrated plastic hinge can be considered realistic (as for steel members under seismic loading [4]), it can conventionally be assumed that strains are proportional to the generalised displacement component s, and it can be stated that:

$$\frac{\Delta\varepsilon}{\varepsilon_y} = \frac{\Delta s}{s_y} \tag{2}$$

As discussed in [3], equation (2) defines the nominal strain range in a particular way [3], taking into account the local reduction of stiffness at plastic hinge location by an equivalent uniform stiffness reduction along the total beam length, and can be rewritten as follows:

$$\Delta\sigma^* = E\Delta\varepsilon = E\frac{\Delta s}{s_y}\varepsilon_y = \frac{\Delta s}{s_y}\sigma(F_y) \tag{3}$$

Equation (3) is similar to equation (1), valid for high cycle fatigue.

3. EXPERIMENTAL VALIDATION

3.1 Re-processing the test data

For re-processing test data according to equation (3), the following parameters must be defined: the number of cycles to failure N_f, the values of the generalised displacement component (s_y) and of the stress level corresponding at first yield (σ (F_y)).
To define N_f, a collapse criterion must be adopted, either assumed conventionally a-priori [4,7-9], or identified test by test corresponding to failure. The value of the generalised displacement component corresponding to first yield can be theoretically computed or conventionally defined, for example reprocessing test data according to ECCS Recommended procedures [6]. The nominal stress level at first yield(σ (F_y)) can be determined either experimentally by tensile tests, or theoretically. In the second case, both F_y and, for flexural members, the dimension of the plastic hinge should be defined.
Once determined N_f, test data can be re-processed to plot in a log-log scale N_f vs. $\Delta\sigma^*$ given by eq. (3). The domain *log* ($\Delta\sigma^*$=$E\Delta$ ε) *vs. log N* is the usual domain for the S-N curves adopted by various International Codes for (high cycle) fatigue design of steel structures. In order to verify the assumption of equivalence of $E\Delta$ ε - N_f curves for high and low cycle fatigue, hereafter it is tried to interpret the experimental test data of low cycle fatigue tests performed on structural steel members and joints, by means of the *S-N* curves proposed by EC-3 [1] extended in the low cycle fatigue range by means of eq. (3).

3.2 Members

3.2.1 Constant amplitude tests

In order to experimentally validate the proposed approach, tests were performed at the Structural Engineering Department of Politecnico di Milano [4], on full scale cantilever members (1.6 m long) of the commercial shapes HE120A, HE140A, HE220A, HE220B and IPE300 using an equipment [5] capable of applying horizontal cyclic actions in a quasi-static way. Presently, the testing programme is continuing, in order to enlarge the data-base and tests are carried out also considering the presence of an axial load [7,8].
To date, 40 tests were performed imposing to the specimens displacement cycles with a constant amplitude (11 on HE220A shapes, 12 on HE220B and 11 on IPE300, 3 on HE120A, 3 on HE140A) . Furthermore, 3 tests on HE120A, and 8 tests on HE220A were performed on specimens, subjected to an axial load. In high cycle fatigue, strength categories implicitly account for different notch effects, i.e. for different local stress concentrations due to geometry of the detail and/or defects caused by fabrication procedures. It is supposed that the same consideration holds also in the case of low cycle fatigue: local buckling can in fact be regarded as a notch effect, because it induces local stress concentrations in the buckled area (at plastic hinge location). In fact, as discussed in [4,7,8] and in good agreement with previous results [10-12], the different geometries of the cross sections make the specimens more or less vulnerable by local buckling

effects. This means that each shape, as a function of its geometrical properties, can be considered as belonging to a definite fatigue strength category, because intrinsically affected by a more or less pronounced notch effect. It must be expected that the different shapes belong to different fatigue strength categories. Figure 1 shows the test data for beam specimens [4] failed by cracking in the base material at plastic hinge locations, fitted by the S-N lines of Eurocode-3. The parameter which seems to govern the fatigue behavior of beams is the d/t_w ratio. In fact, it can be noticed that test data for HE120A specimens (having lowest d/t_w) can be fitted by line for category 90, those for HE220B and HE140A (having similar d/t_w) can be fitted by EC-3 line for category 80, those for HE220A by category 71, while those for IPE300 (having largest d/t_w) by category 63.
The tested specimens evidenced two different failure modes: by cracking in the base material at plastic hinge locations, or by cracking at welding of the reinforcement plates to the specimens. It must be expected that different fatigue strength curves apply to different failure modes.In fact, as shown in fig. 2, HE220A and HE220B specimens can be fitted by category 63 line, while IPE300 specimens, despite the same welded detail was adopted for all shapes, are fitted by line 56 and show a lower fatigue strength[4]. Independently on the fatigue resistance category pertinent to each shape, it is important to notice that the slope of the line fitting (in a log-log plot) the low cycle fatigue test data, reprocessed according to eq. (3), is nearly -3, in good agreement with the results of research on high cycle fatigue.

Fig. 1 Fatigue strength of beams failed at plastic hinge

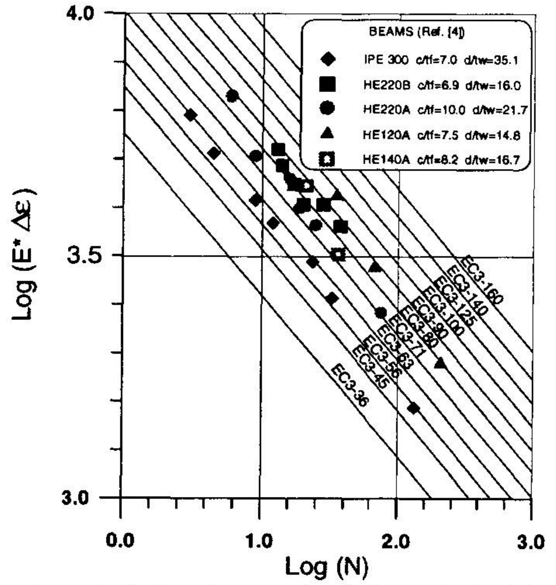


Fig. 2 Fatigue strength of beams failed at weldings [4]

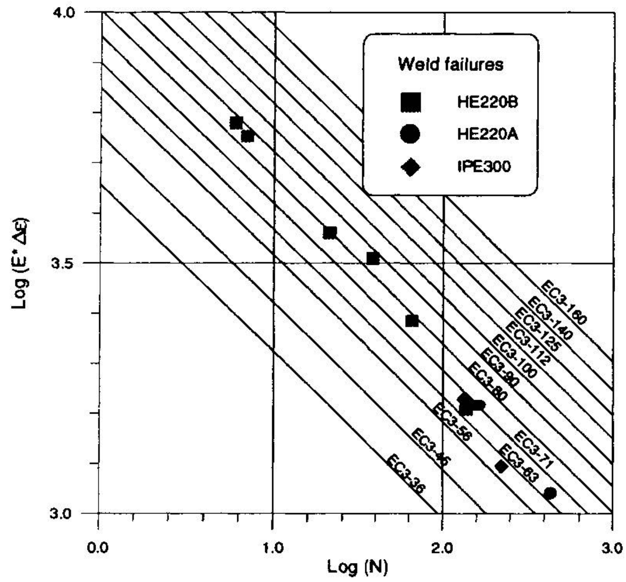


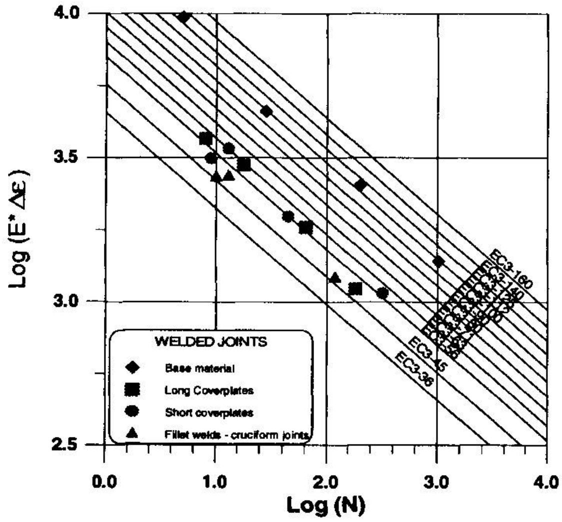
Table 1 - Damage indexes corresponding to specimen collapse in variable amplitude tests

TEST	EC-3 90	EC-3 80	EC-3 71	EC-3 63	Failure
HEA1	0.542	0.772	1.104	1.580	P.H.
HEA9	0.854	1.202	1.740	2.489	P.H.
HEA10	0.599	0.853	1.220	1.746	P.H.
HEA11	0.773	1.040	1.490	2.135	P.H.
HEB1	1.420	2.030	2.900	4.152	P.H.
HEB12	0.717	1.020	1.460	2.090	W
HEB13	0.523	0.744	1.060	1.524	W
IPE9	0.461	0.657	0.939	1.340	P.H.
IPE10	0.385	0.547	0.783	1.120	P.H.
IPE11	0.460	0.660	0.940	1.346	P.H.
IPE15	0.478	0.680	0.973	1.394	P.H.

3.2.2 Variable amplitude tests

At present, a total of 11 variable amplitude tests have been performed at Politecnico di Milano [4] (4 on HE220A shapes, 4 on IPE300 and 3 on HE220B). The experimental results were reprocessed by means of the rainflow cycle counting method and Miner's damage index [16] associated with collapse of each specimen was computed, based on the transformation given by equation (3) and on the EC-3 fatigue strength lines previously identified for the various profiles. The obtained results are summarised in table 1 where the failure mode (P.H.= at plastic hinge, W= at welding) and the damage index corresponding to the EC-3 lines are given. It can be noticed that Miner's rule [16] gives damage index values with scatters similar to those commonly accepted in random high cycle fatigue, and correctly allows prediction of failure in association with EC-3 line 71 for HE220A specimens and with line 63 for IPE300 specimens. For HEB specimens, depending on the failure mode, a correct prediction of failure is achieved respectively in association with EC-3 fatigue strength lines 80 and 63. These fatigue strength categories lead to damage assessments on the safe side, increasing the fatigue strength of HE220B specimens by one category results in damage index values nearer to unity.

Fig. 3 Fatigue strength of welded joints

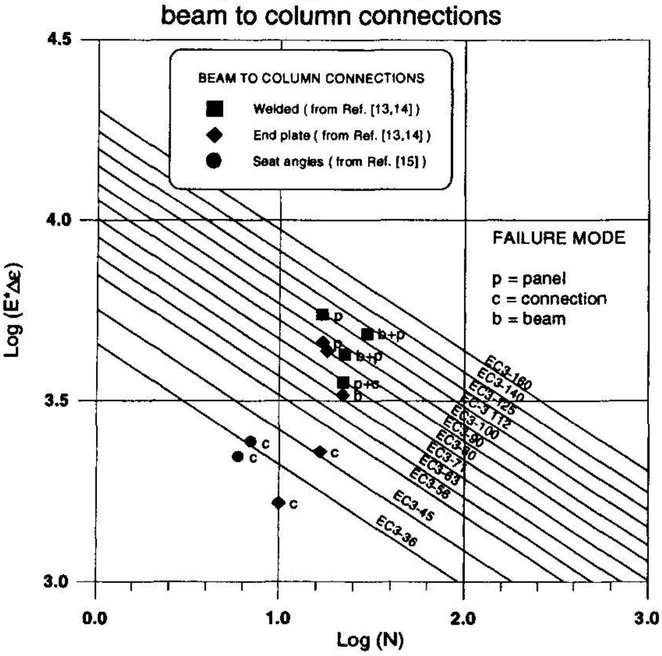


3.3 Connections

3.3.1 Welded joints

An experimental study was carried out at Politecnico di Milano on welded details commonly adopted in structural steelwork. Three types of connections were considerded: fillet welded coverplates (long and short ones) and fillet welds in cruciform joints. Small specimens (400 mm long) were manufactured and tested imposing axial strain cycles with a constant amplitude. Various tests were performed under different strain ranges. Fig. 3 shows some of these test results (together with results for unwelded specimens obtained during the same study), reprocessed according to eq. (3). The base material plots between EC-3 lines 100-125, 100mm ("long") coverplates between EC-3 lines 56-63, 50mm ("short") coverplates between EC-3 lines 45-63, fillet welds in cruciform joints between EC-3 lines 45-56. These results are in good agreement with the classification of the various details by EC-3, where base material is assigned a fatigue strength category ranging between 125 and 160, coverplates to categories ranging between 45 and 63, fillet welds in cruciform joints to category 36 (40-63 by the Italian C.N.R. 10011/86 Code). Of course, the small number of test data did not allow for any regression analysis. This fact, in addition to the absence of residual stresses (due to the small dimension of the specimens), account for eventual small differences between code classification and the results of the present study.

Fig.4 Fatigue strength of beam to column connections



3.3.2 Beam to column connection

It has been tried to apply the same procedure for defining the fatigue strength of beam to column connections[13-15], consisting of welded connections, end plate connections and double seat angle connections. As all these tests were performed under variable amplitude loading according to [6], in order to define the fatigue strength category of each connection re-analysis of test data was carried out adopting Miner's rule, and defining the pertinent EC-3 line a-posteriori, as the one giving, at collapse, a damage index approximately equal to 1.0.

It can be noticed that test data are grouped, depending on the failure mode. The end plate joints from Ref. [13,14] that failed in the connection show a low fatigue strength, similar to that of seat angle connections from Ref. [15]. The other connections (both welded and end plated) show a higher fatigue strength, although the failure mode seems to influence their fatigue behavior. Presently it is tried to enlarge the data base also by collecting and re-analysing test data presented in the literature. Even from these previous results it can however be concluded that the design of connections is critical for a good performance of structures under low cycle fatigue and that, in order to avoid brittle fracture, it should be tried to induce the formation of a plastic hinge in the members, away from the connections. This is of course in good agreement with other results presented in the literature (e.g. [18]).

4. CONCLUSIONS

If an equivalent stress range $\Delta\sigma^{*}=E\Delta\ \varepsilon$ is considered, associated with the actual strain range $\Delta\ \varepsilon$ in an ideal indefinitely elastic behavior of the material, the S-N lines given by Codes for high cycle fatigue can be adopted for interpreting the low cycle fatigue behavior of beams, welded joints and beam to column connections.

Miner's rule can be adopted, together with the previously defined S-N curves and with a cycle counting method (e.g. Rainflow) to define a unified collapse criterion, valid for both high and low cycle fatigue.

The application of these results and of the proposed method for damage assessment to steel structures under seismic loading may lead to an overcoming of seismic design methods based on the behavior factor as shown in [17].

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Survey on Fatigue Damage in Steel Railway Bridges in Thailand

Dommages par fatigue de ponts ferroviaires métalliques en Thaïlande

Ermüdungsschäden an Stahleisenbahnbrücken in Thailand

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SUMMARY

The results of the survey of fatigue damage of steel bridges of the State Railway of Thailand are presented. An investigation of the characteristic and the cause of cracks occurring in the welds of stringer necks of through girder bridges was conducted.

RÉSUMÉ

Les résultats de l'étude des dommages par fatigue des ponts métalliques du Chemin de Fer National de Thaïlande (SRT)sont présentés. Une recherche a été conduite sur les caractéristiques et les causes des fissures qui se produisent dans les soudures des raccords des sommiers des ponts à poutres transversales.

ZUSAMMENFASSUNG

Beschrieben wird das Ergebnis einer Untersuchung von Ermüdungsschäden an Stahlbrücken der thailändischen Staatsbahn (SRT). Untersucht wurden die Eigenschaften und die Ursachen von Rissen in den Schweissnähten der Längsträgerhälse von durchgehenden Balkenbrücken.

1.INTRODUCTION

The State Railway of Thailand (SRT) has concentrated its effort on the use and replacement of existing facilities to ensure stable transport and increase the transport capacity and speeds.

New kinds of damage are showing up even in newly-built bridges. These new kinds of damage include fatigue damage often found characteristically in welded structure bridges in Europe and the U.S. as well as in Japan.[1) They develop rapidly and are hard to treat. Therefore, if they are left alone as they are or treated only with conventional means, they may become a critical damage that may hinder stable transport.

These, however, can be prevented from developing into critical damage if appropriate approaches are taken.

The bridge which we examined is a through girder bridge located 73.057 km on the North line. It was built in 1969 (design load: DL-15) and rivets are used for splicing and connection.

The arrangement of stringers and the track and positions where fatigue cracks have occurred are shown in Fig.1. (the same figure used in bibliography 1)

2.OUTLINE OF DAMAGE

The model assumed so as to clarify the cause and to know the characteristics of damage is shown in Figs.2 and 3.

Fig.2 shows a crack developing about 700 mm horizontally along the toe on the weld between upper flange and web of stringer (we called "the neck's weld) just under the sleeper. It had, however, not reached the beam's end of the stringer.

Fig.3 shows a crack occurring in the beam end of a stringer advancing horizontally on the web along the bead toe. At this position, a stop hole is provided at the end of the crack, and found effective to some extent.

3.INVESTIGATING THE CAUSE OF DAMAGE

Investigation of the cause is an important acquirement in knowing the occurrence trend of damage of the same kind and in making measures more reasonable.

Studies were made on the basis of the data as given below.

3.1 Visual Inspection of Similar Bridges

The result of visual inspection is rearranged as follows to investigate causes.

(1) It was assumed that the crack start from just under the sleeper or from the bead toe on the web side of the neck's weld of the stringer end (some coming from the corner of the notch)

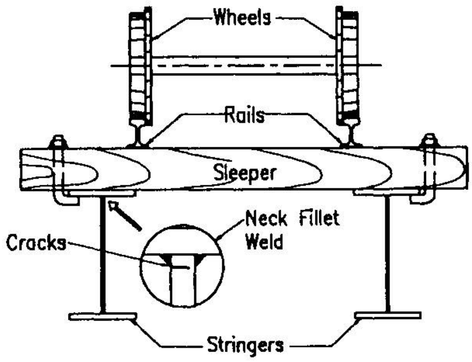

Fig.1 Wheels, track and stringer

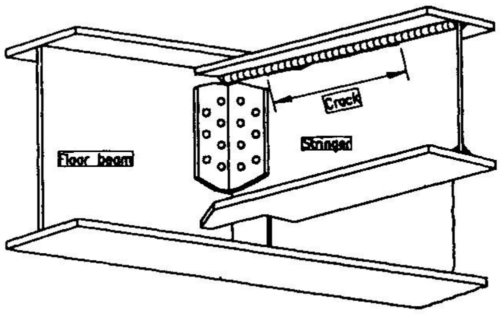

Fig.2 Crack occurring just under the sleeper

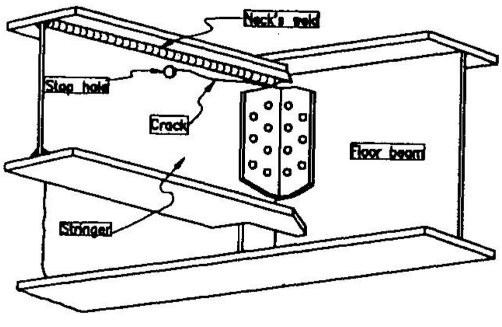

Fig.3 Crack occurring in the beam end

(2) The crack has grown nearly horizontally along the bead toe on the web side of the neck's weld or on the web near it.
(3) The crack along the neck's weld continued to progress without being stopped, parts of it extending beyond the patch plate or stop hole.

3.2 Measuring Actual Bridges

Stress behaviors were examined by sticking a triaxial gauge to the positions as shown in Fig.4 with due consideration to the foregoing.

The trains used for measurement include: The condition of contact between the sleeper and the flange is shown in parentheses.
(1) ALSTHOM: (under present conditions: in close contact inside; 0.2 to 0.5 mm apart outside) (See Fig.5)
(2) ALSTHOM: (liner inserted intentionally: 2 mm apart inside; in close contact outside) (See Fig.6)
(3) new-HITACHI (liner inserted intentionally: 2 mm apart inside; in close contact outside)
The axle load and spacing of ALSTHOM and new-HITACHI is shown Fig.7.

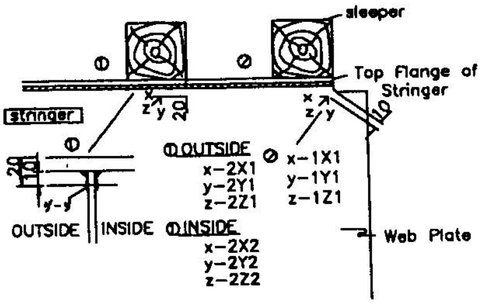

Fig.4 Position of strain gauges

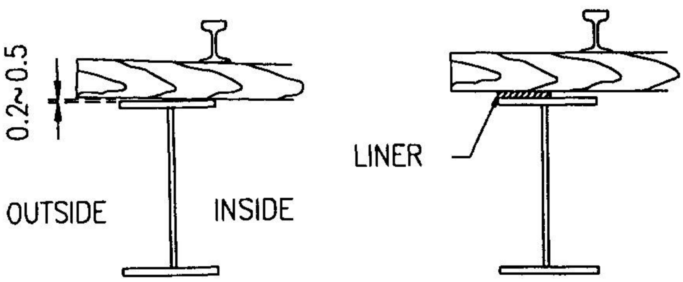

Fig.5 Present condition Fig.6 Liner inserted intentionally

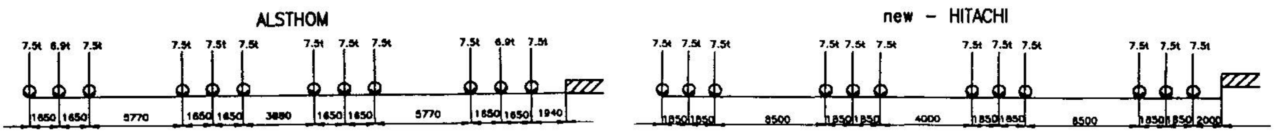

Fig.7 Axle load and axle spacing

The following are items given special attention and the results of measurement
(1) Stress in the neck of the stringer just under the sleeper. Measurements were made for ALSTHOM (the sleeper is touching the top flange). Fig. 8 shows three-component stress for the front and the back faces.

(2) Stress in the coped corner notch in the stringer end
The train for which measurements were made was new-HITACHI whose axle load is the greatest this line. (The liner plate was inserted just under the sleeper only in the halfway.)
In Fig.9, the upper three waveforms show triaxial stress in the girder end notch, and the lower two waveforms show vertical stress in the front and the back of the web in the halfway of the stringer.
(3) Effects of contact conditions between the sleeper and the flange
Stress acting on the neck just under the sleeper is affected greatly by the contact condition between the sleeper and the upper flange. Under present conditions, the sleeper and the flange are in contact on the inside of the stringer, but there is a gap of 0.2 to 0.5 mm in the outside. Lifting the sleeper manually makes the gap expand to 1 to 2 mm. Therefore, external force from the sleeper is absorbed by the inside of

the flange when loaded with a train.

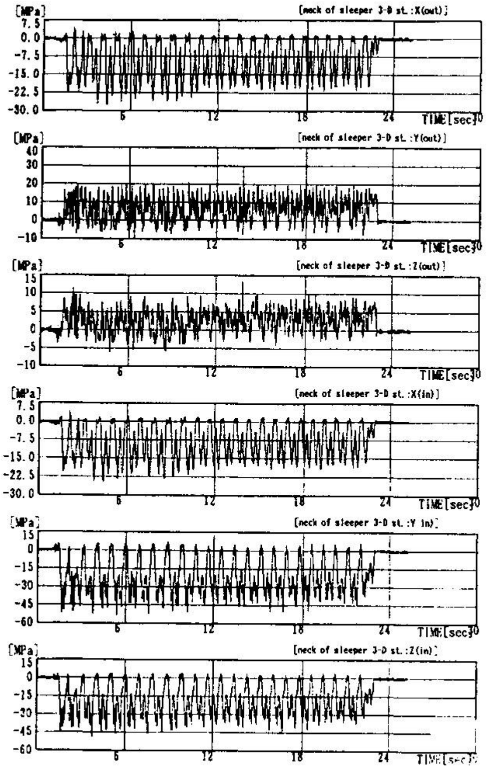

Fig.8 Stress in front and back of the neck just under the sleeper

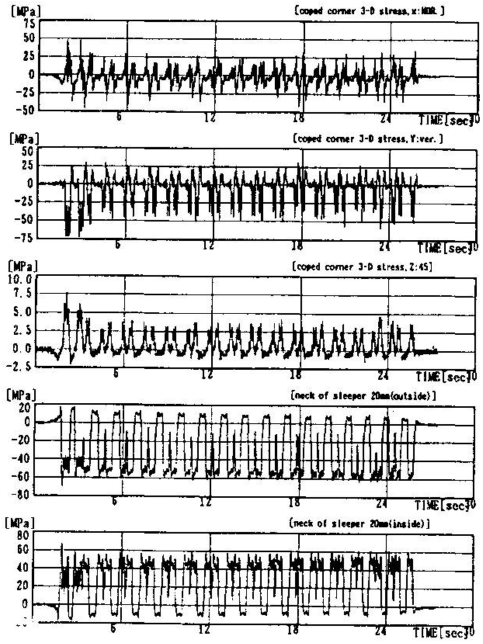

Fig.9 Triaxial stress in the girder end notch and vertical stress of the web

3.3 Evaluating Fatigue Damage in the Neck's Weld

The method and standards for evaluating fatigue were based on the fatigue design guidelines of Japanese Society of Steel Construction[2]. The result of fatigue damage evaluations is shown in Fig.10 and Fig.11.
Based on the results above,the following can be said:

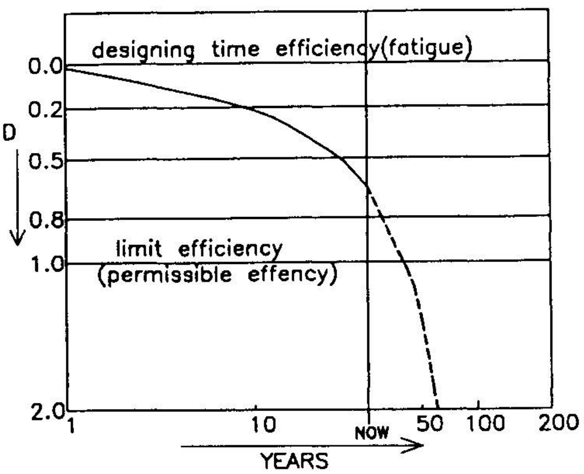

Fig.10 Fatigue crack occurring life of the stringer's crack(under the present condition)

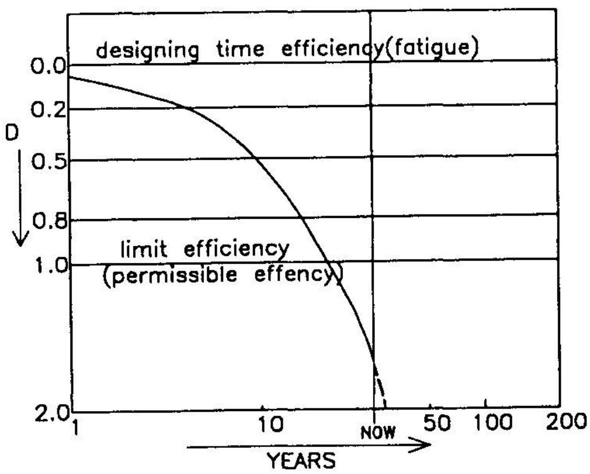

Fig.11 Fatigue crack occurring life of the stringer's neck(liner inserted)

(1) Vertical stress of about 65 MPa is acting on the neck's weld. It was greater than the stress in the axial direction of the bridge, and was in the reverse phase in the inside and the outside of the web.
(2) Vertical stress due to out-of-plane bending of the stringer neck was as large as 75 MPa.
(3) Vertical stress in the notch of the stringer's end was 90 to 100 MPa and main stress was similarly great.
(4) Fatigue crack occurrence life was calculated to be 9 years for vertical stress acting on the neck's weld. Fatigue crack occurrence life when the sleeper was in loose contact (when the sleeper was completely deviated) was calculated to be 8 years.

4. Estimating Fracture Life of Rails

For analysis, a stress waveform was obtained by simulating a stress when a typical train is run over rails where the sleeper just above a crack in the weld of a stringer's neck ceases to support the rails, and on the assumption that those rails are continuous beams with a span of 1,000 mm. The typical train was a train drawn by new-HITACHI.
Considering the live load history of this line, fatigue accumulated in the rails were obtained to calculate crack occurrence life.
Load and fatigue strength were treated in the same way as those for railroads in Japan.[3,4]

4.1 Waveforms of Stress Acting on Rails

In the analysis of the waveform of stress acting on rails, based on the fact that the crack in the neck's weld was 700 mm long, the rail was assumed as a 1,000 mm span continuous beam with one sleeper lost. In that condition, axial force acting on the rail was obtained by simulation. On this basis, stress "on the railhead in the weld joint" and "bolt holes in the joint bar" was obtained. Stress acting on the bolt holes was obtained by the FEM analysis because it could not be obtained by a simple beam stress calculation. Fig.12 shows the stress waveform and stress spectrum in top-flange of the rail obtained by simulation.

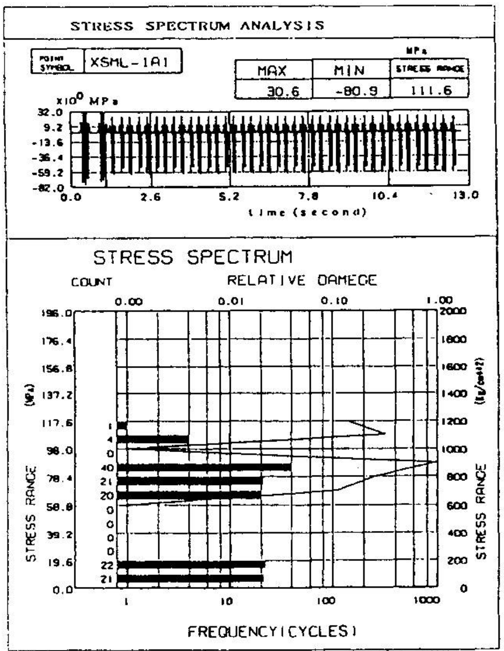

Fig.12 Stress waveform and stress spectrum obtained by simulation

4.2 Dynamic Effects of External Force

There is no clear measurements on "a" above. On the average, however, "about 30% higher than static external force"was considered a reasonable value and was used.
Concerning "b" above, recent research results [3,4] show that load used for evaluation of fatigue of thermit-welded rails with undulations was about 1.7 times those without undulations.

4.3 Stress Acting on Rail Joints

Stress acting on joints was obtained with due consideration to the conditions described above.
(1) Weld joints
Rails here are joined by thermit welding, and undulated surfaces of rails cause impact.
(2) Bolt holes
Stress distribution at a hole's periphery was obtained by FEM analysis.
The model used for the analysis is a two dimensional plane element, and the rigidity of

top and bottom flanges was considered only for board thickness. As a result, large stress was acting in the tangential direction at the hole's periphery at 45 degrees from the hole's center, that stress was used here.

4.4 Fatigue Strength of Joints Used for Evaluation

The fatigue strength curve of rails used for evaluation of fatigue was the one as shown in Fig.13 [3,4] which is usually used by former Japanese National Railways.
The S-N curve used to evaluate general parts was for corrosion fatigue (pH3.5).

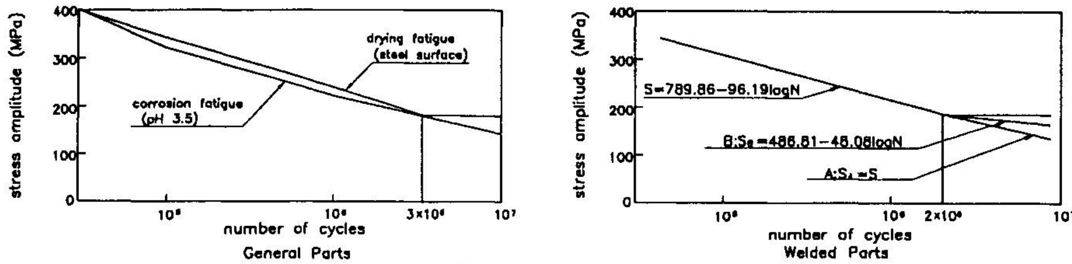

Fig.13 Rail's fatigue strength curve

4.5 Result of Estimation of Remaining Life

The time when fatigue will cracks occur on rails under said conditions was estimated as follows:

(1) The ordinary part of a rail, that is, the part without either the weld nor bolt holes does not pose any fatigue problem at once, even if, for example, sleepers are spaced one meter apart.

(2) Weld joints
In case there is no crack in the neck's weld: more than 200 years.
In case there are cracks in the neck's weld: 11 years.

(3) Bolt holes
Whether there is a crack in the neck's weld or not: 4 years.

5.Conclusions

(1) Summary of evaluation results
Stress acting in the neck of the stringer stress (out-of-plane bending to the web) due to eccentric loading from the sleeper, and is influenced by the way the sleeper and the upper flange contact with each other.

(2) Emergency breakdown repair
Immediate treatment of existing cracks is needed to prevent rail fractures and serious accidents associated with a buckling of a stringer.

(3) Causes of damage
This time cracks were found in through plate girder bridges, and their causes include:
a.Joints of sleepers were loosened, causing eccentric load in the neck.
b.Legs of beads are slightly short.
c.Shapes of beads are slightly defective.
d.The structural detail of the connection between a stringer and a floor-beam is not very good.
e.Track irregularity causes lateral oscillations while it is run over by a train.

(4) Main measures
Possible measures include improving the sitting condition of sleepers, preventing the twisting of the neck, and reducing bending stress.
Adjustable pads(resin mortal) are sometimes placed under sleepers to make them fit well with flanges and to provide sleeper supports.

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Fatigue Safety of Existing Steel Bridges

Sécurité à la fatigue de ponts métalliques

Ermüdungssicherheit von Stahlbrücken

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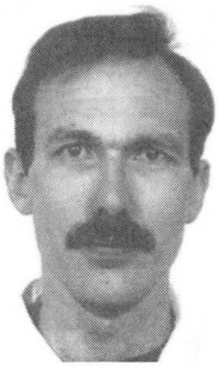

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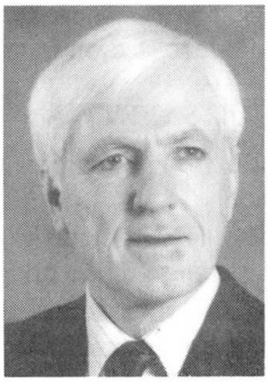

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SUMMARY

Today, mainly two types of steel bridges are fatigue critical, railway bridges from the beginning of the 20th century and highway bridges built in the 1950s and 1960s. In this paper a practical method is presented to calculate the remaining fatigue life of existing struc-tures that is closer to the real behaviour than previous methods. The method helps to make better decisions regarding maintenance, replacement and rehabilitation. The use of the proposed method is illustrated by an example.

RÉSUMÉ

Aujourd'hui, deux types de ponts métalliques sont critiques vis-à-vis de la fatigue: les ponts-rails construits au début du 20e siècle et les ponts-routes datant des années 50 et 60. Cet article présente une approche pratique et simple permettant aux ingénieurs de calculer la durée de vie restante de structures existantes, ceci d'une manière plus proche de la réalité qu'il n'était possible auparavant. De plus cette méthode aide à prendre des décisions concernant la maintenance, les remplacements et les réparations. Un exemple permet d'illustrer l'utilisation de la méthode proposée.

ZUSAMMENFASSUNG

Heute sind vielfach zwei Arten von Stahlbrücken hinsichtlich Ermüdung gefährdet: die zu Beginn des 20. Jahrhunderts erbauten Bahnbrücken und die in den 50er und 60er Jahren errichteten Strassenbrücken. Im vorliegenden Beitrag wird ein praxisnahes Verfahren vorgestellt, welches dem Ingenieur erlaubt, die Restlebensdauer bestehender Konstruktionen zuverlässiger abzuschätzen als dies bis anhin möglich war. Zusätzlich können mit diesem Verfahren auch bessere Entscheidungsgrundlagen für Unterhalt, Ersatz und Wiederinstandsetzung zur Verfügung gestellt werden. Der Nutzen des vorgeschlagenen Verfahrens wird anhand eines Modellbeispiels veranschaulicht.

INTRODUCTION

Today, there are mainly two types of bridges the remaining fatigue life is often questioned - railway bridges from the beginning of this century and highway bridges built in the 1950's and 1960's. The following discussion will be focused on the first type of bridges. Often, the structural capacity of these bridges is still satisfactory due to a very conservative design at this time, but fatigue becomes often very critical. Therefore it is important to estimate reasonably the remaining fatigue life of these structures. On the same time engineers must evaluate alternatives by making assumptions about the traffic in the future and they must be enable to make proposals for reinforcements and rehabilitations or traffic restrictions.

The objectives of this paper are to give some indications how fatigue safety can be assessed and the remaining fatigue life can be estimated. Therefore the procedure for the assessment will be discussed and a damage accumulation model based on fracture mechanics will be introduced.

ASSESSMENT OF FATIGUE SAFETY

For design and assessment of existing structures the following three main factors must be considered in order to verify fatigue safety:

- **Applied stress ranges:** The applied stress ranges are a function of the service loads, including impact, and the structural response of the bridge to those loads.
- **Geometry of the detail:** The stress concentration caused by the geometry of the detail, the manufacturing procedure, and the crack shape due to a given stress direction all influence the fatigue life. These will be considered by assigning a detail to a predefined category.
- **Number of stress cycles:** The number of stress cycles applied in the past directly influences the remaining fatigue life of a structure.

An assessment of fatigue safety may be required as a result of observations (e.g., increasing in displacements of vibrations characteristics or the occurrence of corrosion or cracks) or because of changes in service conditions (e.g., increase of axle loads or number of vehicles) or for a legal reason when a assumed service life is reached. In the assessment of fatigue safety the following three stages can be distinguished:

- **Stage 1 - Identification of fatigue critical details:** It is recommended that a thorough study of the available documents be made and a detailed inspection of the bridge be carried out. A list of priorities of fatigue critical details can be established on the basis of a calculation that uses the current design rules.
- **Stage 2 - Calculation of remaining fatigue life:** In addition to inspection and maintenance, a reliable estimation of the remaining fatigue life based on an appropriate calculation is needed.
- **Stage 3 - Monitoring of fatigue critical details:** For structures subjected to fatigue, regular inspections can be essential.

CALCULATION OF REMAINING FATIGUE LIFE

For existing structures a sophisticated procedure must be recommended where the damage increase of each stress range can be taken into account. This is important because the traffic conditions have normally changed since construction. The axle loads have gotten heavier and the number of vehicles, i.e., the traffic frequency, has increased significantly. Therefore the traffic has become more fatigue aggressive.

For the verification of the fatigue safety normally fatigue strength curves are used. Depending on the geometry and the manufacturing process the corespondent fatigue category for the investigated detail can be identified. The damage increase d_i per stress $\Delta\sigma_i$ is defined as the inverse of the number of stress ranges N_i which could be applied for a constant amplitude $\Delta\sigma_i$. Failure would be assumed when the accumulated damage $D = \Sigma d_i$ is 1.0.

This calculation gives good results as long as all stress ranges are above the constant amplitude fatigue limit $\Delta\sigma_D$. When some stress ranges are below $\Delta\sigma_D$, as illustrated in Fig. 1, there are several alternative approaches that can be used:

a) **Constant slope:** The approach used in North America is to calculate the equivalent stress range, using the root mean cube, and compare this to the allowed stress range for a given number of stress cycles.

b) **European approach:** Below the constant amplitude fatigue limit $\Delta\sigma_D$ a different slope m' = 2m-1 is taken into account, considering that the stress ranges below $\Delta\sigma_D$ do not contribute to the damage increase in the same way as the stress ranges above $\Delta\sigma_D$ do [1].

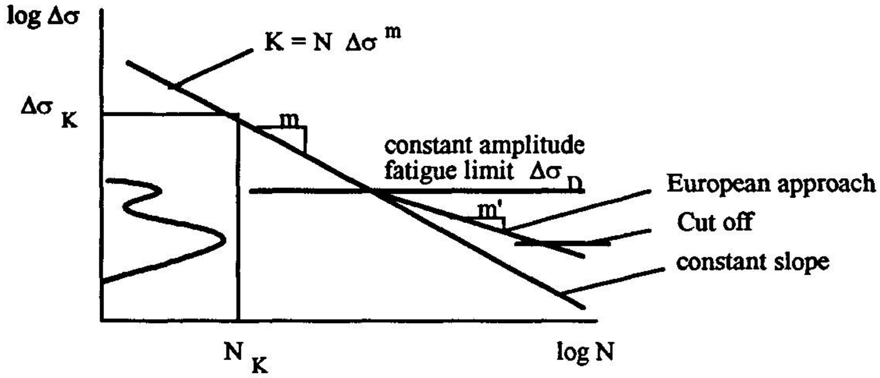

Figure 1 : Stress range spectrum with different fatigue strength curves

Both methods will give conservative results. However, for a reliable and economic assessment of the fatigue safety of existing structures it is important to have a better understanding of the way the stress ranges below the constant amplitude fatigue limit contribute to the damage increase. Therefore a damage accumulation, called FM-model, based on fracture mechanics has been developed.

In fracture mechanics it can be observe that with an increasing crack the crack rate per stress cycle increases. On the same time smaller and smaller stress ranges contribute to the crack propagation. The stress range below no further damage occurs is called the damage limit $\Delta\sigma_{th}$. It decreases with the increase of the damage D, for more information see [2].

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

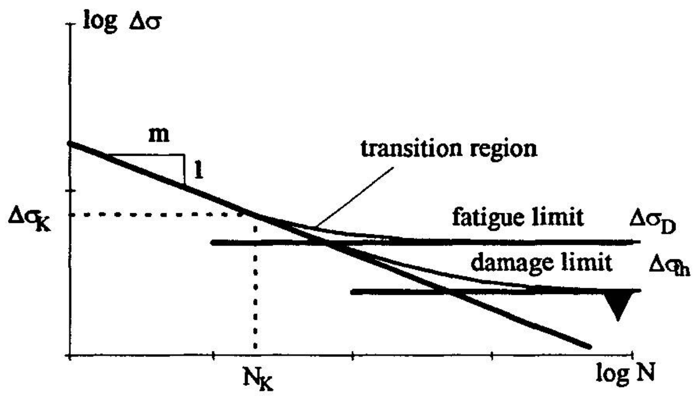

Figure 2 : Decreasing of the damage limit including the rounding of the transition region

The decreasing of the damage limit $\Delta\sigma_{th}$ is illustrated in Fig. 2. Between the constant slope region above the constant amplitude fatigue limit $\Delta\sigma_D$ a transition region can be observed. A similar observation can be made in fracture mechanics and therefore the damage increase per stress cycle can be quantified.

The prediction of the decreasing of the fatigue limit can be compared with fracture mechanics and it will be seen that with the FM-Model the fatigue life is underestimated by 5 to 8 % referred to the fracture mechanics calculation. The range depends on the assumed detail and the applied stress range spectrum [2].

APPLICATION

In order to illustrate the application possibilities the basic information from a railway bridge built in 1900 will be used. In an other situation the results can be completely different.

The main assumptions of the railway bridge are:

- year of construction: 1900
- year of reference: 1995
- actual traffic volume: 60 trains per day
- traffic mixture: 50 % freight trains; 50 % passenger trains
- fatigue strength: category 67, i.e. fatigue strength at 2 million cycles: 67 MPa constant amplitude fatigue limit at 7 million cycles [3]
- static system: simple span beam with the influence line of the moment in the middle of the span
- section modulus: 25×10^6 mm^3

In the following figures the damage evolution of different damage accumulation models is shown. The FM-Model will be compared with the traditional approaches, the Constant Slope and the European model (Fig. 1). In Fig. 3 a constant traffic model is assumed. The traditional damage accumulation models show a constant increase of the damage, in the way it will be expected for design, the FM-Approach has a acceleration in the damage increase, because with increasing damage more and smaller stress ranges contribute to the damage increase. The differences between the three models is obvious because the majority of the stress ranges of the applied spectrum is below the constant amplitude limit. Based on the damage accumulation model used, the fatigue life will be estimated in a range of almost 100 years. If all stress ranges are above the constant amplitude fatigue limit and above the influence of the transition region the three curves are identical.

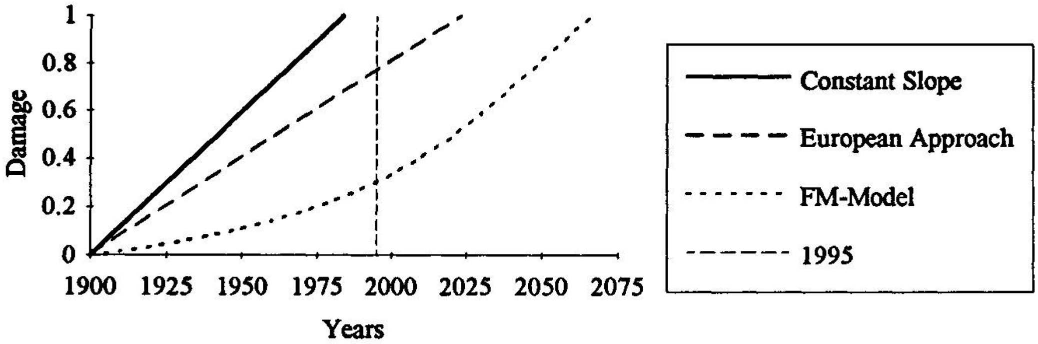

Figure 3 : Damage evolution for a constant traffic model

For the traffic in the past it is normally appropriate to consider a traffic evolution, where the increase of axle loads and the increase of the number of passages will be taken into account. For railway bridges the UIC (Union Internationale des Chemins de fer) proposes a traffic evolution like shown in Fig. 4. For each time period an other number of trains per day and other trains must be considered. The corespondent damage evolution is shown in Fig. 5. Due to the traffic evolution even the traditional methods change now the slope. Nevertheless, for each defined traffic model the damage increase is constant. However, the FM-model shows a continuous increase of the damage.

In Fig. 6 the influence of the percentage of freight trains is shown. The total number of trains is unchanged. The value at the left, 0 %, corresponds to exclusively passenger trains and 100 % to exclusively freight trains. The remaining fatigue life is given in years, where 0 corresponds to the reference year 1995 and 100 corresponds to 2095. Freight trains have heavier axle loads and are unfavorable with respect to the remaining fatigue life, this parameter can influence the remaining fatigue life significantly. Accurate information about the traffic is very important in order to calculate a reliable estimations of the remaining fatigue life.

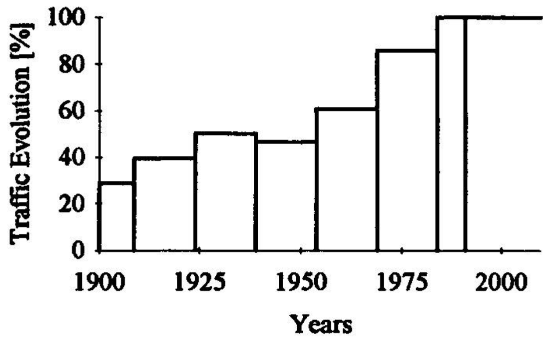

Figure 4 : Traffic evolution corresponding to UIC

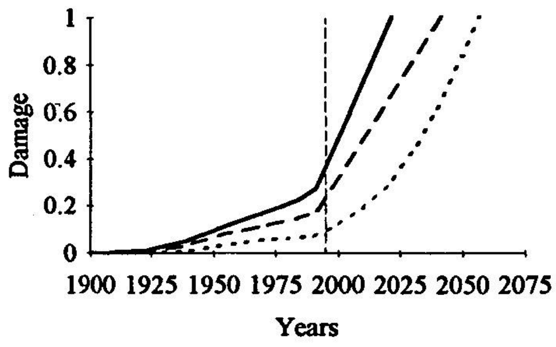

Figure 5 : Damage increase for a given traffic evolution (legend see Fig. 3)

Fig. 7 shows the influence of the traffic volume, expressed by the number of trains per day. Even when the number of trains must be estimated and an uncertainty of ± 10 % must be assumed, the calculated remaining fatigue life will not change significantly.

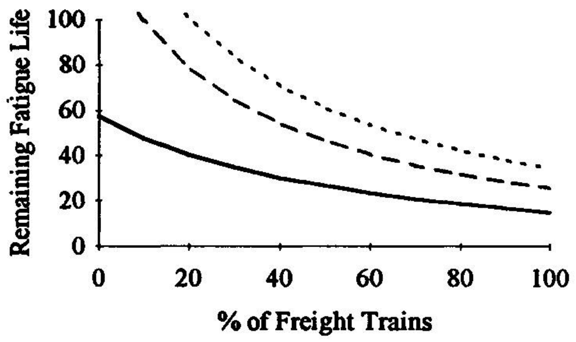

Figure 6 : Influence of the percentage of freight trains (legend see Fig. 3)

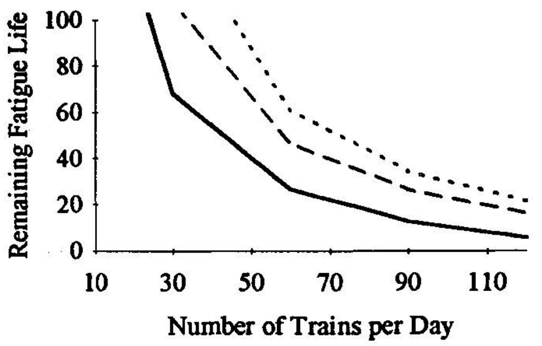

Figure 7 : Influence of the traffic volume (legend see Fig. 3)

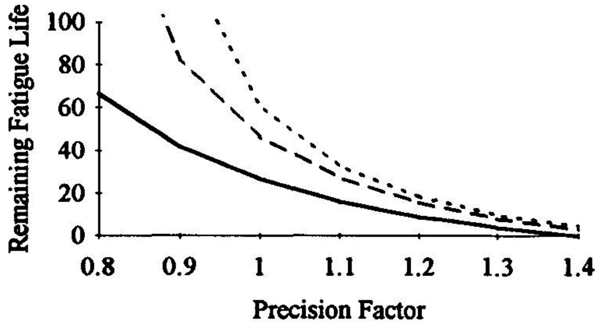

Figure 8 : Influence of the precision factor (legend see Fig. 3)

One of the most important parameters needed for the calculation is the influence line. There can be significant differences between calculated and measured values. In Fig. 8 the influence of the assumed influence line is shown by applying a precision factor.

The calculated remaining fatigue life is very sensitive to this parameter and it is highly desirable to carry out field measurements in order to create as accurate a picture of the influence line as possible.

In Fig. 9 the influence of a second track is shown. In all cases the total number of trains will be assumed to be the same. The first assumption correspond to a bridge where the total traffic in both directions is on the same track. This is compared to a second assumption that corresponds to a bridge with two tracks where on each track 50 % of the original traffic will be assumed. The influence of the stress ranges on the investigated detail of the second track is due to the lateral distribution 80 % of the influence of the first track; train crossings are excluded. In a third assumption different train crossings are assumed.

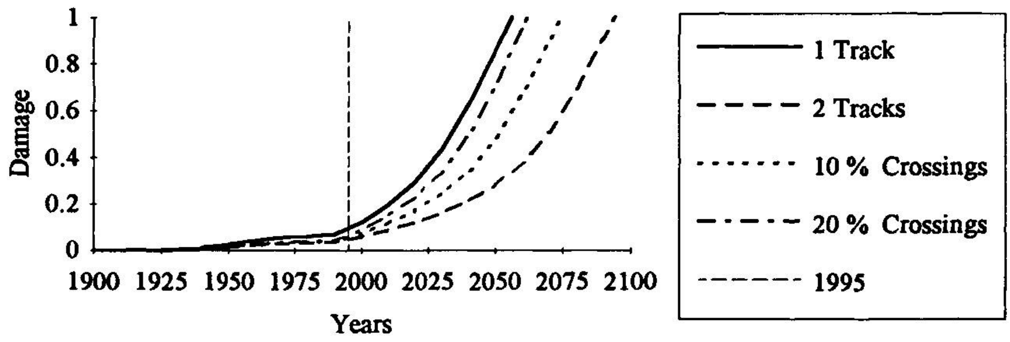

Figure 9 : Influence of parallel tracks and train crossings

CONCLUSIONS

The main foundings of the presented study can be summarized as follows:

1. For a reliable and effective assessment of the remaining fatigue life, three steps must be distinguished: Identification of Fatigue Critical Details - Calculation of the Remaining Fatigue Life - Monitoring of Fatigue Critical Details
2. For the calculations of the remaining fatigue life, a reliable and precise approach, based on fracture mechanics, called FM-Model, is presented. The prediction of remaining fatigue life with the FM-Model is much closer to the real behavior than with traditionally used approaches as the root mean cube method or the European S-N curves.
3. The applied stress ranges have a predominant effect on the remaining fatigue life. The stresses will normally overestimated with traditional static analysis. In order to elaborate a more realistic static model it is useful to carry out field measurements.

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Fatigue Life of Riveted Railway Bridges

Fatigue des ponts-rails rivetés

Dauerhaftigkeit genieteter Eisenbahnbrücken

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SUMMARY

Many old riveted railway bridges in Sweden from the turn of the century and the few decades thereafter are today still being kept in service, so their useful lifespan might be close to an end. However, as shown by several field investigations and extensive full-scale fatigue tests, for a well-maintained bridge the expected remaining fatigue life should be substantial.

RÉSUMÉ

Un grand nombre de vieux ponts-rails rivetés, construits en Suède à la fin du 19e siècle et au cours des décennies suivantes, sont toujours en service. Ces ouvrages devraient ainsi arriver au terme de leur durée de vie. Toutefois d'innombrables essais à la fatigue exécutés en plusieurs étapes, sur le site et en grandeur nature, montrent qu'il est possible de prolonger considérablement la durée de vie résiduelle des ponts correctement entretenus.

ZUSAMMENFASSUNG

Viele alte, genietete Eisenbahnbrücken aus der Jahrhundertwende und den folgenden Jahrzehnten sind in Schweden heute noch im Betrieb. Ihre Lebensdauer sollte bald abgelaufen sein. Wie jedoch in mehreren Felduntersuchungen und ausgiebigen Ermüdungsversuchen in Originalgrösse gezeigt wurde, kann die Restlebensdauer einer gut unterhaltenen Brücke beträchtlich sein.

1. BACKGROUND

Riveting was the dominating joining technique for the construction of railway bridges in Sweden from the last decades of the 19th century until the late 1930's. Many of the riveted railway bridges built during that period are today still being kept in service. Out of the 1100 railway bridges in steel that are being managed by the Swedish National Rail Administration around 600 are riveted structures. The average age of these riveted railway bridges is about 70 years and there are reasons to presume that their useful lifespan is close to its end.
However, some old riveted railway bridges are today replaced too soon due to general lack of knowledge about the expected lifespan. These particular bridges have often been judged to be unfit with reference to fatigue, although no analysis of the fatigue damage accumulation has been made. Further in those cases the assumed presence of fatigue cracks has not been confirmed. Additionally, no investigation has been made regarding the possibility to strengthen the structure in order to increase its load-carrying capacity and at the same time improve its resistance against fatigue.

Research concerning fatigue and lifespan expectancy of old existing riveted railway bridges in steel enables the railway authorities to plan and more accurately decide upon when and which of these bridges should be replaced or strengthened first. Over the last decades riveted railway bridges have sometimes been left without proper maintenance and only limited research has been made concerning these kind of structures. It may be stated that recent research has rather been focused on the development of new materials and structures than on further development and maintenance of existing structures. And, in these times of environmental awareness, more and more attention is also payed to the waste of our resources. Prolonging the useful lifespan of our old stock of riveted railway bridges by the help of this kind of research gives considerable economical savings not only to the railway authorities, but also to the society at large.

2. FIELD STUDIES

2.1 General

Several field investigations of riveted steel bridges have since 1989 been conducted at the Department of Structural Engineering, Chalmers University of Technology, In all, the field investigations have resulted in studies of 15 different riveted railway bridges built between 1903 and 1928.
The major concerns behind the decision to investigate the superstructure of these old bridges have been the following points:

- Static load-carrying capacity
- Magnitude of live load stresses
- Corrosion damages
- Hit damages or other defects
- Vertical and lateral deformations during train passages
- Dynamic amplification of the static response
- Fracture toughness of the steel
- Possibility that the steel is susceptible to ageing
- Expected lifespan according to a fatigue damage accumulation analysis
- Any visible fatigue cracks

2.2 Results

Among other findings, the major results from the field studies of the riveted railway bridges investigated by our department are:

- No major corrosion damages were found, despite neglected maintenance in some cases.
- No loose or faulty rivets were detected.
- No visible cracks were found.
- The maximum stress in the bridge superstructure was seldom exceeding 42 MPa, when the heaviest freight trains were passing.
- There is a good correspondence between theoretical strain values and the actual strain response.
- The dynamic amplification factor was found to be smaller that what can be assumed.
- Extensive ultrasonic crack controls have been performed, and not in a single case were fatigue cracks detected at or near the rivet holes.
- The fracture toughness of the steel has in general been inadequate with respect to present code requirements.
- The static load-carrying capacity has in many cases shown to be sufficient to carry modern "design traffic".
- A fairly accurate estimate of the remaining fatigue life has been possible through a thorough study of the loading history. A remaining fatigue design life of 25 years or more has been the result for the bridges which have been analyzed.

2.3 Discussion

The fracture toughness of the steel was generally found to be too low compared with the generally accepted ductility requirements (the only "negative" finding presented above!). If so, why has there not been any brittle fractures? The different possible answers to this question are:

- Low stresses in general (i.e. low probability of a propagating fatigue crack).
- Despite the stress level, the major part of the fatigue life (say 90 - 95 %) is before the initiation of a fatigue crack, and therefore the probability of a brittle fracture during a normal service-life period is negligible.
- The strain rate is low (the fracture toughness increases with decreasing strain rate).

- The plate thicknesses are normally small (thin plates are more ductile than thick plates – and the fracture toughness requirements of today are based on thick plates). A thicker plate can be expected to fail in a more brittle manner due to:
  - a large probability for triaxial stress conditions,
  - large grain sizes,
  - an inhomogeneous and anisotropic material,
  - the size effect,
  - a high residual stress level.
- There is normally no other visible defect in the primary tension members than the rivet holes, and this stress raiser is by no means large enough to be the single cause for the initiation of a brittle fracture.
- The clamping force is introducing a triaxial compressive stress state at the rivet hole which prevents any fracture tendencies.
- A riveted built-up bridge member shows an inherent structural redundancy through "separate part composite action". A propagating crack will, without exception, be stopped when passing from a member part to another.

3. FULL-SCALE FATIGUE TESTS

3.1 Test results

When the old riveted railway bridge over the Vindelälven at Vännäsby (built in 1896) was replaced by the railway authorities in 1993 it was decided that one should take the opportunity to carry out an extensive full-scale fatigue testing programme comprising the stringers and floor-beams from the bridge. In the first series 10 stringers were tested in four-point bending. The stringers were 5,5 meters long (after being flame cut from the bridge) with a depth of 830 mm:

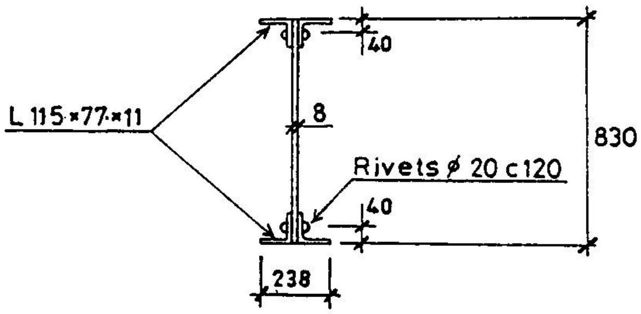

Fig.1 Dimensions of the built-up I-shaped stringers.

The results of the first series in the fatigue testing programme are presented in table 1:

Table 1 Results from the full-scale fatigue testing of the stringers.

Stress range σ_r [1]	Stress Ratio R	Stringer No	Number of cycles Crack detection	Additional cycles	Failure
40 MPa	0.24	6B	-	-	(20 000 000) [2]
40 "	0.24	7A	-	-	(20 000 000) [2]
60 "	0.18	12A	-	-	(10 000 000) [2]
60 "	0.18	12B	-	-	(10 000 000) [2]
80 "	0.14	2A	5 893 350	102 520	5 995 870
80 "	0.14	3B	2 375 500	0	2 375 500
80 "	0.14	5B	6 370 440	114 900	6 485 340
100 "	0.28	4B	1 488 920	148 950	1 637 870
100 "	0.28	5A	2 038 660	146 240	2 184 900
100 "	0.28	10B	2 027 250	0	2 027 250

1) Based on net section area and the stresses are derived at the outermost edge of the beam (i.e. $\sigma_r = M_r/W_{net}$) instead of the actual critical rivet locations.

2) The test was discontinued after a prescribed high number of load cycles.

3.2 Summary

The main findings from the first full-scale fatigue test series of 10 riveted built-up stringers are:

- The fatigue lives of the nine stringers tested were *in accordance* with or *above* the fatigue design curve given in the structural steel codes for riveted details (C = 71 / category D), despite the suffering of almost 100 years of loading!

- The fatigue cracks always started and subsequently propagated in the *lower (tension) flange.*

- The fatigue crack initiations were, in five cases out of six, at a *flange rivet connection* outside the mid-span region. Only in one case was the fatigue crack emanating from a *"neck" rivet hole* in the mid-span region.

- A substantial amount of loading cycles were required to propagate the fatigue crack from one L-profile to the other (in average about 100 000 cycles!). The riveted built-up composite I-beams (i.e. the stringers) thus showed an *inherent structural redundancy.*

- The fatigue cracks were difficult to observe in the beginning of the propagation period. They were *almost invisible to the naked eye* when the stringers were temporarily unloaded.

- The fractured stringers were *capable of carrying a considerable static load* (greater than the maximum load of the fatigue test) after completed cyclic loading.
- A series of tension tests of the steel material showed that the mechanical properties were comparable to a *mild steel* with a yield point higher than many other riveted bridge steels tested.
- A *markedly ageing effect* was found for the steel after cold deformation and subsequent heat treatment.
- A transition temperature of approximately +12 °C was the result of the Charpy-V impact notch testing. This temperature is much above the lowest service temperature.
- A chemical analysis confirmed that we have an ordinary *carbon steel* with a low content of nitrogen.
- The clamping ratio (i.e. rivet clamping stress in relation to the yield stress of the rivet) was found to be 0.42 on an average.
- The fatigue damage accumulated in the stringers was approximately found to be *negligible*.

4. CONCLUSIONS

Both the full-scale fatigue tests and the different field investigations clearly show that there is a substantial remaining fatigue life of typical riveted railway bridges still in use today:

- The full-scale fatigue tests showed that the common fatigue design curve for riveted details is *underestimating* the fatigue lives. However, this curve can be used as a safe estimate of the *remaining* fatigue life of old riveted railway bridges still in service today.
- The different field investigations gave the conclusion that the maximum stress ranges *seldom, if ever, exceed the fatigue limit* for riveted details. The fatigue damage accumulated in the bridges can therefore be assumed to be negligible.

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Fatigue Failure Mechanism of Steel Elements with Thin Webs

Mécanismes de rupture de fatigue des éléments en acier à paroi mince

Ermüdungsbrucharten von Stahlbauelementen mit dünnen Stegen

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SUMMARY

The paper contains the results of the extensive experimental investigation on the behaviour and failure mechanism of thin-walled steel elements subjected to repeated load. The presentation of recommendations and criteria for the limitation of slenderness of the web of hybrid plate girders is included as well.

RÉSUMÉ

L'article présente les résultats d'un vaste programme expérimental. Ce programme de recherche concerne le comportement et le mécanisme de rupture des éléments en acier à paroi mince sous charges répétées. Des recommandations sont faites et un critère pour la limite de l'élancement de l'âme de poutres mixtes est donnée.

ZUSAMMENFASSUNG

Der Beitrag behandelt Erkenntnisse aus umfangreichen experimentellen Untersuchungen hinsichtlich dem Verhalten und den Brucharten von dünnstegigen Stahlelementen unter wiederholter Belastung. Es werden Empfehlungen und Grenzwerte für die Schlankheit der Stege von hybriden Blechträgern vorgestellt.

1. INTRODUCTION

From the economical point of view it is necessary to design thin-walled steel elements. Therefore the design of steel elements with thin webs is recommended and demanded according to the elasto-plastic postcritical conceptions and limit state design method. Hybrid plate elements with different steels in flanges and webs of the cross-sections can be especially advantageous for economical design. For the safe and economical design of thin-walled hybrid steel elements it is very important to know their real load-carrying capacity and behaviour during repeated loading process. [1-3]

The thin webs of steel elements are laterally buckling under the static loading and they are laterally vibrating under the dynamic fatigue loading. As a result of this membrane and bending stresses occure which, together with residual stresses, can cause a formation and successive increase of fatigue cracks. Besides of that the thin webs of hybrid steel elements are loaded at the elasto-plastic stage. From practical point of view it is very important question of the limitation of the secondary stresses of webs in order to prevent arising of fatigue cracks during the life-time of elements. Therefore the extensive experimental programmes have been carried out for the investigation of load - stress - strain - deflection relationships and fatigue failure mechanisms of hybrid steel elements - girders and columns - with thin webs [4-6]. Similar investigations on girders were accomplished in the United States [7-8] and Japan [9].

According to this aim and allowable range the paper deals especially with realised investigation of hybrid plate girders.

2. EXPERIMENTAL INVESTIGATION

The complete experimental programme of the investigation consisted of the static and dynamic fatigue tests of 48 plate girders with symmetrical "I" cross-section of real practical dimensions, produced under common production-technological conditions. The test girders were different in geometrical dimensions, stiffened field ratios α, web slenderness β and material combinations of steels for flanges and webs (material and cross-section groups). One half of the test girders was used for static tests and the other half of similar girders for dynamic fatigue tests. Table 1 contains characteristics of fatigue test girders.

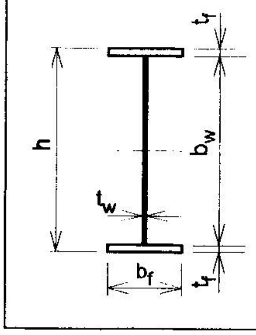
Cross-section	Dimensions $2\square b_f \times t_f +$ p. $t_w \times b_w$ (mm)	Span L (mm)	Stiffened field ratios $\alpha = a_w/b_w$	Slenderness $\beta = b_w/t_w$	Material combinations steel of flanges / steel of web: 11 523.1	13 221.1	15 422.5	16 224.1
					11 375.1			
	$2\square 160 \times 10(12)$ $+$p. 5×600	4900	1.0 1.25 1.5 2.0 3.5	120	A 125 A 126	B 123 B 124	C 125 C 126	D 123 D 124
	$2\square 180 \times 12$ $+$p. 5×750	5950		150	A 155 A 156	B 153 B 154	C 155 C 156	D 153 D 154
	$2\square 200 \times 12$ $+$p. 5×900	7000		180	A 185 A 186	A 183 A 184	C185 C186	D 183 D 184

Table 1. Characteristics of tested girders.

The cross-section dimensions in the individual groups were selected to give (at uniform t_w thickness and various b_w breadth values) slenderness ratios β_w of 120, 150 and 180. The flange dimensions were designed to ensure local stability. Element lengths were linked to b_w web breath ($L \approx 7b_w + 800$ mm). The intermediate panels had different side ratios ($1 \le \alpha \le 3.5$).

The flanges and webs were machine flame cut. Besides of that the tension flanges of girders were planed. The fillet welds between flanges and webs were made by automatic machine under flux, while stiffeners were hand welded.

Prior of testing the true geometrical and material characteristics of the flanges and webs were ascertained. All steels used for the experiments were in conformance with the requirements of the applicable standards, or exceeded them. The geometrical and material characteristics, as well as their corresponding deviations from design dimensions and standard material specifications are random variables. Hence, in evaluations of experimental results, either the directly determined or those generally probabilistically - statistically defined values of these variables could be used.

From the point of view of this investigation the information on lateral buckling of the webs in production and their influence on the overall behaviour of girders are particularly significant. Therefore, individualy tested girders were investigated for the lateral buckling of the webs in production. Scheme of girders and working arrangement of the tests are shown on Fig. 1.

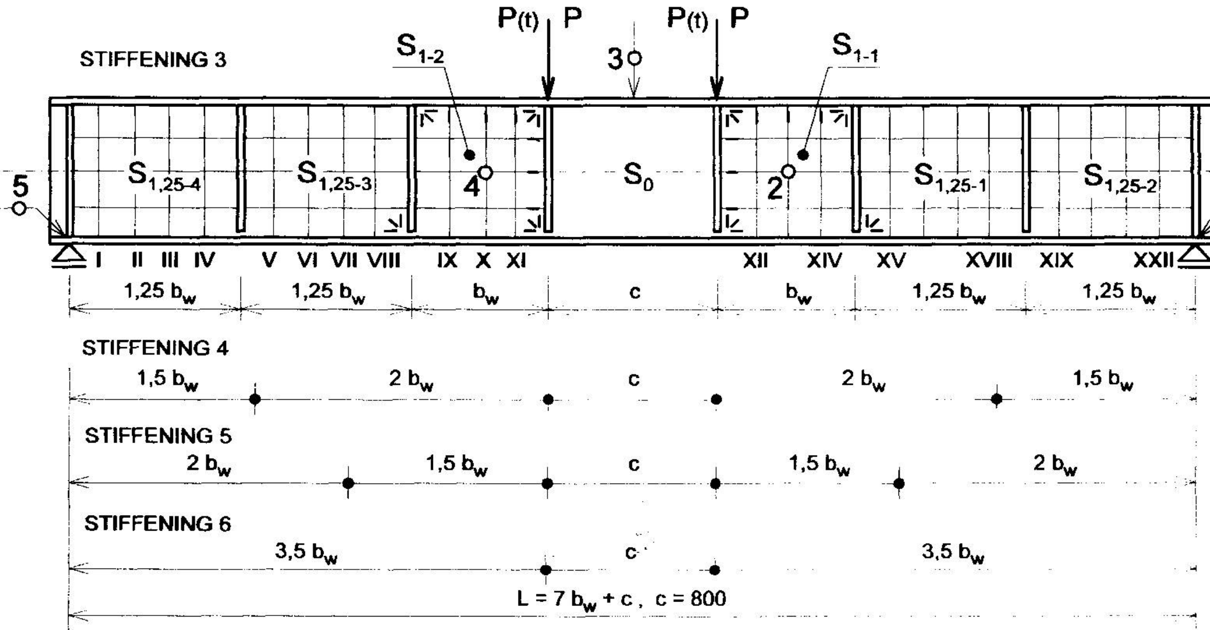


Fig. 1. Scheme of girders and working arrangement of the tests

The tested girders were sustained by special vertical and horizontal supports to prevent lateral buckling. The loading was realised by two hydraulic jacks. The tests of girders were divided into two parts. First of all, the girder was loaded by successive loading till to P_{max} and after that about five cycles between $P = 0$ and $P = P_{max}$ were repeated. Then the fatigue test could start. During fatigue test the girders were loaded by synchronise pulsate loads

$P(t) = P_0 + P_1 \sin(2\pi f t)$, where

$P_0 = 0.5(P_{max} + P_{min})$, $P_1 = 0.5(P_{max} - P_{min})$,

$P_{min} = (0.2 \div 0.4)P_u$, $P_{max} = (0.7 \div 0.8)P_u$, frequency $f = 5.0$ Hz,

P_u is the ultimate static loading including the elasto-plastic postcritical behaviour of the web

The behaviour of girders during the static and dynamic tests was ascertained by measurement, registration and evaluation of strains ε in the most stressed parts (78 tensometers). The supports sets down v_s - deflection pickups 1 and 5, deflection of girder v_c in the middle cross-section - deflection pickup 3 and buckling of the web w_w in the middle points of two stiffening panels - deflection pickups 2 and 4 were also measured. Besides of that we made the detailed measurement of static and dynamic buckling of the web w in different cross-sections and point - cross-sections I, II,..., XXII by movable pickup with graphic evaluation, or by inductive pickup, tensometer bridge and oscillograph, respectively.

The fatigue tests were realised in stages. After each 50 000 cycles the tests were interrupted. Incidental arising of the cracks was ascertained by the detailed observation of the girder with the help of magnifying glass and by direct graphic and tabulate evaluation of strains ε and deflections w_w and v_c at loads $P = 0$, P_{min}, P_{max}. The differences between last and previous values of strains ε in some points of girder indicate arising and placement of fatigue cracks. After arising of the cracks, we followed their development and influence on the global behaviour of girders. The tests continued until the total failure occurred.

3. BEHAVIOUR AND FAILURE OF TESTED GIRDERS

The thin webs of girders are laterally buckling in dependence on the initial imperfections, geometrical parameters and stiffness of bordered flanges and stiffeners. They are buckling in shapes which correspond to the mechanism of their behaviour according to the process of loading. These shapes are formed already at the beginning of loading process. In accordance with the initial shape a new service shape can be formed by the successive fluent transition or by the sudden jump over of the web. The service shapes of buckling are decisive for analysis of load-carrying capacity and behaviour of the thin webs of steel girders.

Under the dynamic fatigue loading the thin webs of girders are laterally vibrating. At the beginning of loading process the service shape of web vibration is formed in a similar way to the case of quasistatic loading. During the usual service loading regime in girders the resonant effects do not arise. Therefore, the lateral web deflections which are received at the repeating loading are practically identical with the dynamic lateral web deflections. Under the fixed cyclic loading the levels of deformations, deflections and shape of lateral web vibration did not change at any tested girder till the beginning of fatigue failure. Referred knowledge is presented in Fig. 2.

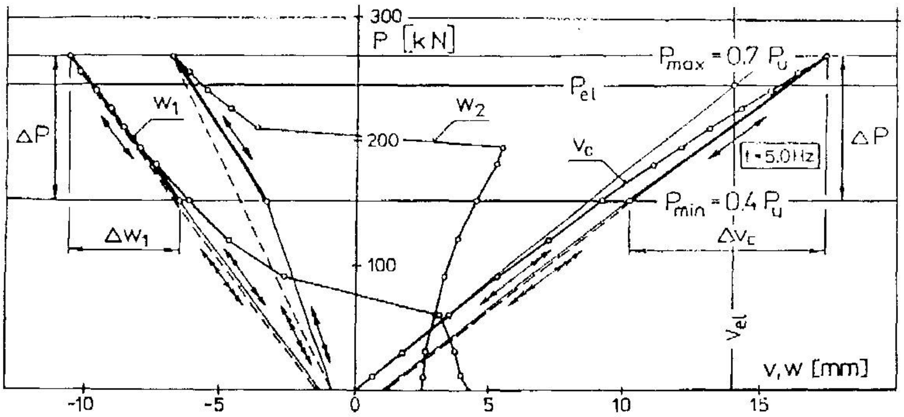


Fig. 2. Deflections w_1, w_2, v_c in dependence of load P. Jumps over of the web in the most loaded fields.

The ultimate load of the girder with account of the web buckling and the elasto-plastic postcritical behaviour of the web is $P_u = 382.2$ kN. During the first loading the sudden jump over appeared in two most loaded web fields - in the web field $S_{2\text{-}1}$ after $P = 60$ kN and in the web field $S_{2\text{-}2}$ after $P = 195$ kN. However, during the following repeating loading between $P = 0$ and $P = P_{max}$ (quasistatic loading) or between $P = P_{min}$ and $P = P_{max}$ (dynamic fatigue loading) any next jump over of the web did not appeared. Under repeating loading relatively quick stabilisation of deformations and deflections was set in and the web buckled or vibrated in the service shape. The girder failed by arising and quick development of fatigue crack in the tension flange after 1,000,000 cycles owing to production defect.

The result of realised experimental investigation proved that the lateral web vibration of steel girders can also cause the formation and successively increasing of the fatigue cracks. Fatigue cracks of the chosen test are presented in Fig. 3.

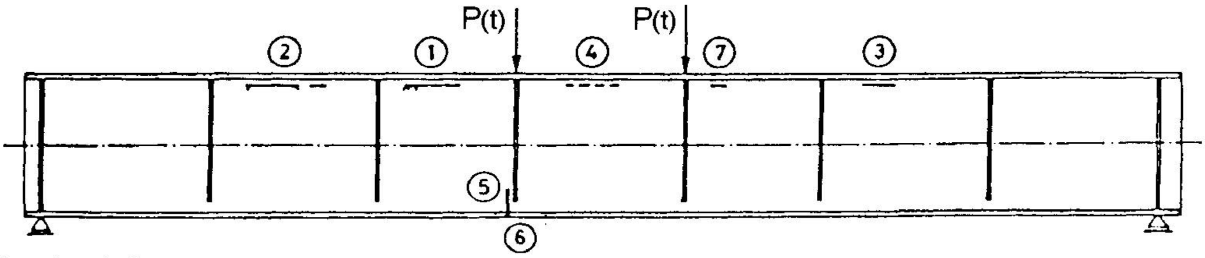


Fig. 3. Fatigue cracks in the thin web along welded connections with compression flange and transverse stiffener, crack in tension flange.

This girder is especially interesting with fatigue cracks along compressed flange in all internal web fields. The first cracks 1 and 2 had appeared after 300,000 and 400,000 cycles respectively. Later cracks were successively increasing, but only the initiation of the crack 5 along stiffener was decisive for the failure of the girder. The crack had spread very quickly to the tension flange in the form of crack 6 and the total failure of girder was completed after 1,342,000 cycles.

Seven fatigue cracks of 3 types have appeared in the girder. The crack of the type 1 is in the web under the compression flange, it is spreading along the fillet welding, it can pass also into the web depending from the membrane stresses. The crack of the type 2 is in the tension web part, it is passing along the stiffener to tension flange. The crack of the type 3 is in the tension flange, it appears after developing of the crack of type 2.

The intensity of the crack's propagation of different types of the cracks is the important factor of the complex life-time evaluation of steel girders. Due to our results there is difference in the crack's propagation velocity. The slowest cracks were those of the type 1, the fastest were cracks of the type 3.

Besides of mentioned fatigue cracks, which are characteristic for thin-walled girders, we could met also 2 another types of cracks as the results of production defects - e.g. due to defects in fillet welding, or due to the notch in the edge of tension flange.

4. LIMITATION OF WEB SLENDERNESS

According to the global results of realised investigation with taking account results of previous researches, the maximum slenderness of the web β_{uf} was designed for the steel girders subjected to fatigue loading in dependence on required life-time in cycles N and on calculated stress of flanges R_{df} (MPa):

$$\beta_{uf} = \frac{1760}{\sqrt[7]{N}} \sqrt{\frac{210}{R_{df}}} .$$

The values of the limit slenderness β_{uf} are in Table 2. It means that the slenderness of the web β should be less than β_{uf}. This limit criterion comes into consideration especially at the plate girders from high strength steels of homogeneous or combined cross-sections.

Life-time N (cycles)	Steel of flanges				
	11 375.1	11 523.1	13 221.1	15 422.5	16 224.6
	R_{df} (MPa)				
	210	290	352	424	548
	Slenderness β_{uf}				
100 000	339.80	289.16	262.46	239.14	210.35
200 000	307.77	261.90	237.72	216.60	190.52
500 000	270.01	229.77	208.55	190.02	167.15
1 000 000	244.55	208.10	188.89	172.10	151.39
2 000 000	221.50	188.49	171.08	155.88	137.12
5 000 000	194.32	165.36	150.09	136.75	120.29
10 000 000	176.00	149.77	135.94	123.86	108.95

Table 2. The limit slenderness of the webs β_{uf} for plate girders.

5. CONCLUSIONS

The realised tests have proved that the service shapes of buckling and vibration are decisive for the analysis of behaviour of thin-walled plate girders. The lateral web vibration of plate girders can cause formation and successive increasing of failure cracks which are situated near the welded connections of the web with flanges and stiffeners. Therefore, the fatigue strength of plate girders with thin web depends besides the usual loading, material and construction influences also on the lateral web vibration. Deformation and increasing of fatigue cracks in thin webs depends especially on the bending stresses along flanges and stiffeners and on their changes during loading process. The bending stresses of the web can be reduced above all by their slenderness β. Therefore, the web slenderness should be adequately limited according to presented proposals.

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Fatigue Testing of Modular Expansion Joints for Bridges

Essais à la fatigue de joints de dilatation modulaires des ponts

Ermüdungsversuche an modularen Dehnungsfugen von Brücken

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SUMMARY

Strain-gage testing on a modular expansion joint system revealed aspects of the load distribution which are difficult to predict from analysis. The system, with a partial-penetration connection detail, was subjected to over two-million cycles of loading without cracking. Subassemblies were also tested, and it was shown that the full-penetration connection detail can be designed as a Category C (Eurocode 90) detail. The stress range used to check the fatigue strength of these details in design must be the maximum range of principal stress, considering the combined stress in the detail including torsion and biaxial bending.

RÉSUMÉ

Des essais effectués sur un système de joints modulaires, pourvus de jauges de déformation, ont révélé des aspects difficiles à mettre en relief par le calcul. Constitué par un dispositif de liaison avec pénétration partielle, le système a été soumis à plus de deux millions de cycles d'efforts alternés, sans entraîner de fissures. D'autres types de joints à assemblage partiel avec pénétration totale ont été également testés, montrant que ce type de joint peut correspondre à celui de la catégorie C (Eurocode 90). Le domaine de contraintes des essais à la fatigue doivent englober la plage maximale de la contrainte principale, en tenant compte de la superposition de la flexion biaxiale et de la torsion.

ZUSAMMENFASSUNG

Versuche mit Dehnungsaufnehmern an einem modularen Fugensystem brachte Erkenntnisse, die durch Berechnung schwer zu gewinnen sind. Das System, bestehend aus einer Verbindungsvorrichtung mit teilweiser Durchdringung, wurde mehr als zwei Millionen Lastwechseln unterworfen, ohne Risse zu verursachen. Teilkonfigurationen wurden ebenfalls getestet, und es zeigte sich, dass die Verbindungsvariante mit voller Durch-dringung als Fuge nach Kategorie C des Eurocodes 90 gestaltet werden kann. Das Spannungsregime für die Ermüdungsprüfung muss die Maximalamplitude der Hauptspannung mit Berücksichtigung der Ueberlagerung von Torsion mit Doppelbiegung in der Fuge, umfassen.

1. INTRODUCTION

Drainage and debris through open expansion joints has been a major cause of corrosion damage in bridges. Modular bridge expansion joints (MBEJ), shown schematically in Figure 1, are sealed and prevent drainage through the deck joint. The transverse beams between the seals are called centerbeams, and each centerbeam is supported on an independent series of support bars which span in the longitudinal direction between support boxes which are cast into the concrete haunches. Inside the support boxes, the support bars slide between precompressed springs and bearings. The transverse beams at the edges which are cast into the concrete haunches are called edgebeams. Since expansion joints are subject to almost exclusively live load, the fatigue limit state will typically govern the design. However, there are no fatigue design specifications for MBEJ in the USA and they are procured by a lowest-bidder process. The competitive nature of the marketplace has led to decreased margin of safety against fatigue, and premature failures have occurred in a few cases. Several different types of fatigue tests were conducted to develop design guidance.

2. TESTS ON THREE-SEAL SYSTEM

Static strain-gage testing and fatigue test were conducted on the three-seal modular expansion joint as shown in Figure 2. A three-span length of this system (including seals, springs, bearings, and support boxes) was subjected to loads applied through a truck axle with wheels and tires. The fatigue test on the three-seal system was conducted with a 186 kN axle load, which corresponds to the HS20 truck (see Article 3.7.6 of the AASHTO "Standard Specifications for Highway Bridges" [1]) with a 30 percent impact factor applied. This axle load (hereafter referred to as the design load) is more than twice as large as would normally be allowed for the tires. The joint was inclined relative to the loading axis so that the loading contained a component parallel to the plane of the joint ("horizontal" load) equal to 20 percent of the load normal to the plane of the joint ("vertical" load). The horizontal load of 20 percent of the vertical load is consistent with field measurements and the recommendations of Tschemmernegg [2] as well as a specification from the State of Washington [3].

2.1 Static Strain-Gage Tests and Comparison to Analysis

Static tests were conducted at various seal gaps. The width of the contact area of the tire against the joint (in the direction along the centerbeams) was relatively constant at 560 mm because the sidewalls of the tire are relatively rigid. The length of the contact area increased with load up to about 330 mm at the design axle load. Typically, the measured stress ranges in the system can vary about 50 percent between locations that ought to have similar results. Our experience with strain-gage testing of complex systems indicates that this much variation is expected. Bending moment diagrams in both the centerbeams and support bars were "fit" to the measured strains, which helped to reduce the apparent variation in results. The support bar reactions were computed from the bending moment diagrams. A comparison of the bending moment in the primary centerbeam (i.e. that centerbeam directly under the tire) at different gap widths shows that the load in this centerbeam is not sensitive to the gap width. This finding is reasonable because the load on the centerbeam is limited to the product of the tire air pressure and the contact area with the centerbeam. For the 38 mm gap with air in the tires, the share of the load in the primary centerbeam decreased from about 55 percent at low loads to about 40 percent at the design load. The 40 percent share is in reasonable agreement with previously published design recommendations [1,2].

A linear three-dimensional frame analysis was made of one centerbeam with the ends of the support bars pinned in both directions. This analysis is a stick model, i.e. the members all lie in the same plane and have infinitesimal thickness. Forty percent of the load was assumed to be acting on the centerbeam and the load included a 20 percent horizontal component. The three-dimensional analysis gives strong-axis bending results which are very similar to the results from a continuous beam model with rigid supports. The computed strong-axis bending is in good agreement with measurements at load levels up to 60 percent of the design load (i.e. in the range of realistic axle loads). However, at higher load levels, the rate of increase of the measured strong-axis bending moments with load decreases. Therefore, other load paths must develop as the load is increased which are not predicted by the analysis. Results for the weak-axis (horizontal) bending case show that the predicted horizontal moments in the centerbeam are about equal to 10 percent of the vertical moments. Part of the horizontal bending moments are transmitted to the support bars as the joints rotate. The ten percent ratio is generally consistent with the measurements, where the ratio varies from about 5 to about 10 percent.

The tires appeared significantly distressed at the design load and it was judged that they would probably soon fail if cycled at this load. Therefore, it was decided to perform the fatigue test with concrete in the tires. The footprint of the concrete-filled tires was formed to have a 280 mm footprint (between the AASHTO and actual measurement for the air-filled tires corresponding to the design load). For the midrange seal gap which was used in the fatigue tests (38 mm), there was reasonable agreement between the bending moments at the design load from tests with concrete-filled tires and the tests with air-filled tires.

2.2 Fatigue Test

The objective of this test was to demonstrate that the particular size modular expansion joint system that was tested could withstand more than two million cycles of the design loading. This objective differs from the typical objective of fatigue testing which is to get the number of cycles to failure to identify the appropriate S-N curve for certain details which failed. Valid tests to characterize the fatigue strength must be full-scale and, if there is biaxial loading, the proportion of the loads is important. In typical fatigue testing of components of the system, structural analysis is required to transfer the test results to the actual joint geometry. As discussed above, the static testing showed significant variation in measured strains which vary in a nonlinear manner with respect to load, and vary from the analytical results by as much as 50 percent. There are probably many different factors that cause this variation which are not considered in the analysis. The closer the configuration and design loading conditions are to the conditions in the test, the better. Therefore, due to the unreliability of stress analysis, the full-scale system proof test is advantageous because structural analysis is not required to interpret or use the results.

A partial-penetration connection detail was used for the centerbeam-to-support-bar joints in this system. The fatigue tests were conducted between 1 and 2 Hz (much slower than actual truck impact) with a very small minimum load (about 9 kN). After 2.05 million cycles were applied, the test specimen was disassembled and visually inspected for cracks. No cracks were found, indicating the test passed. A single fatigue test was also conducted on an elastomeric bearing. This test indicated that it is likely that the fatigue strength of the bearings is also sufficient to withstand more than 2 million cycles of HS20 loading with 30 percent impact.

3. SUBASSEMBLY TESTS

Fatigue tests were also conducted on three specimens, each of which consisted of three spans of a single centerbeam on four support bars as shown in Figure 3 A full-penetration welded connection detail was used in this subassembly test. The objectives of these subassembly tests were to characterize the fatigue strength of the full-penetration weld details in terms of the appropriate detail category or S-N curve. Two 280 mm long line loads centered 1830 mm apart were applied to the centerbeam to simulate axle loading. The specimens were inclined so that the load produced a horizontal component which was equal to 20 percent of the vertical component. The applied total load ranges were 390 kN, 260 kN, and 180 kN.

The centerbeam is in a state of biaxial bending. Therefore, the proper stress range for checking the centerbeam at the point of midspan maximum moment is the sum of the extreme fiber stress for vertical and horizontal bending. There is torsion in the centerbeam as well, but at this location the torsion causes only shear stresses and can therefore be ignored. The data from these base metal failures all fall above the Category A S-N curve, the equivalent of the Eurocode 160 S-N curve [4], as expected.

In the connection of the centerbeam to the support bar, the maximum stress range occurs where the applied stresses remain in compression. Cracks can form because there is generally high tensile residual stress so that the sum of residual plus applied stresses is at least partly in tension as the stress fluctuates. Figure 4 shows a typical crack which occur at the full-penetration connection detail. Note that the cracks are inclined at about 45 degrees. The cracks grow in the plane normal to the principal stress axis. It is clear that cracks are driven by a combination of the longitudinal bending stress in the centerbeam or support bar as well as the vertical stress range in the throat of the connection resulting from the reaction on the support bar and the bending stress due to the overturning moment in the centerbeam. The two components of the principal stress range are calculated and the total principal stress range may be approximated as the square root of the sum of the squares of the two components.

The S-N curve for the two types of cracks which occur at this connection is shown in Figure 5. Data from both types of cracks appear to belong to the same population. Most of the data are above the Category C (Eurocode 90) S-N curve, which is the expected fatigue strength for a full-penetration groove weld attachment. Before these cracks could grow to failure, the cracks developed in the centerbeams which prevented further loading. Therefore it is expected that the total number of cycles to failure would be above the Category C curve if the cracks had been able to continue to grow.

4. CONCLUSIONS

The system test demonstrated that this particular MBEJ, with a partial penetration connection detail, can withstand more than two-million cycles of the design loading. The subassembly testing characterized the fatigue strength of the details in terms of the appropriate detail category and design S-N curve. The stress range used to check the fatigue strength of these details in design must be the maximum range of principal stress, considering the combined stress in the detail including torsion and biaxial bending. The base metal, away from any weld, can be designed as an AASHTO Category A (Eurocode 160) detail. The full-penetration weld connecting the centerbeam to the support bar can be designed as a Category C (Eurocode 90) detail.

5. ACKNOWLEDGEMENT

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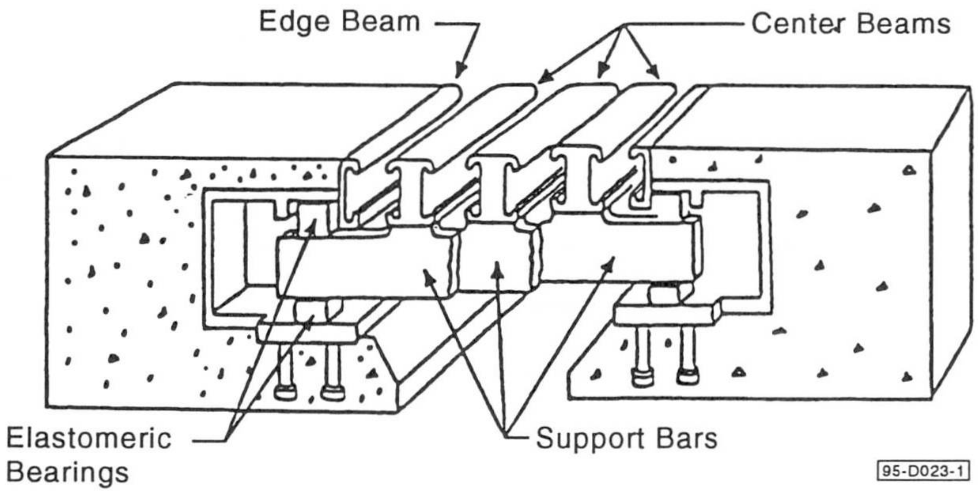

Figure 1: Schematic of modular expansion joint system

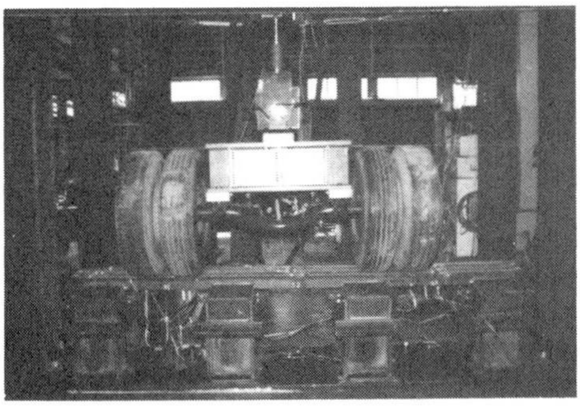

Figure 2: Full-scale system fatigue test

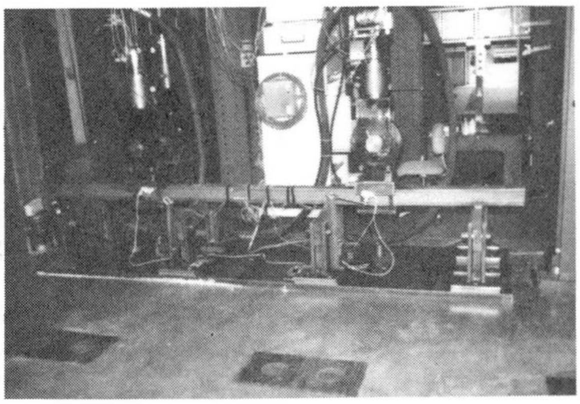

Figure 3: Single centerbeam fatigue test

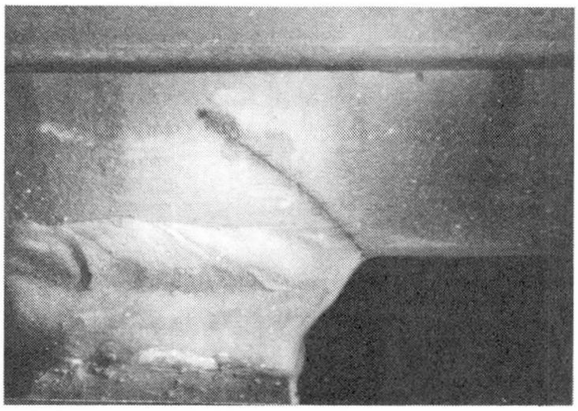

Figure 4: Typical crack in the centerbeam at the full-penetration weld connection.

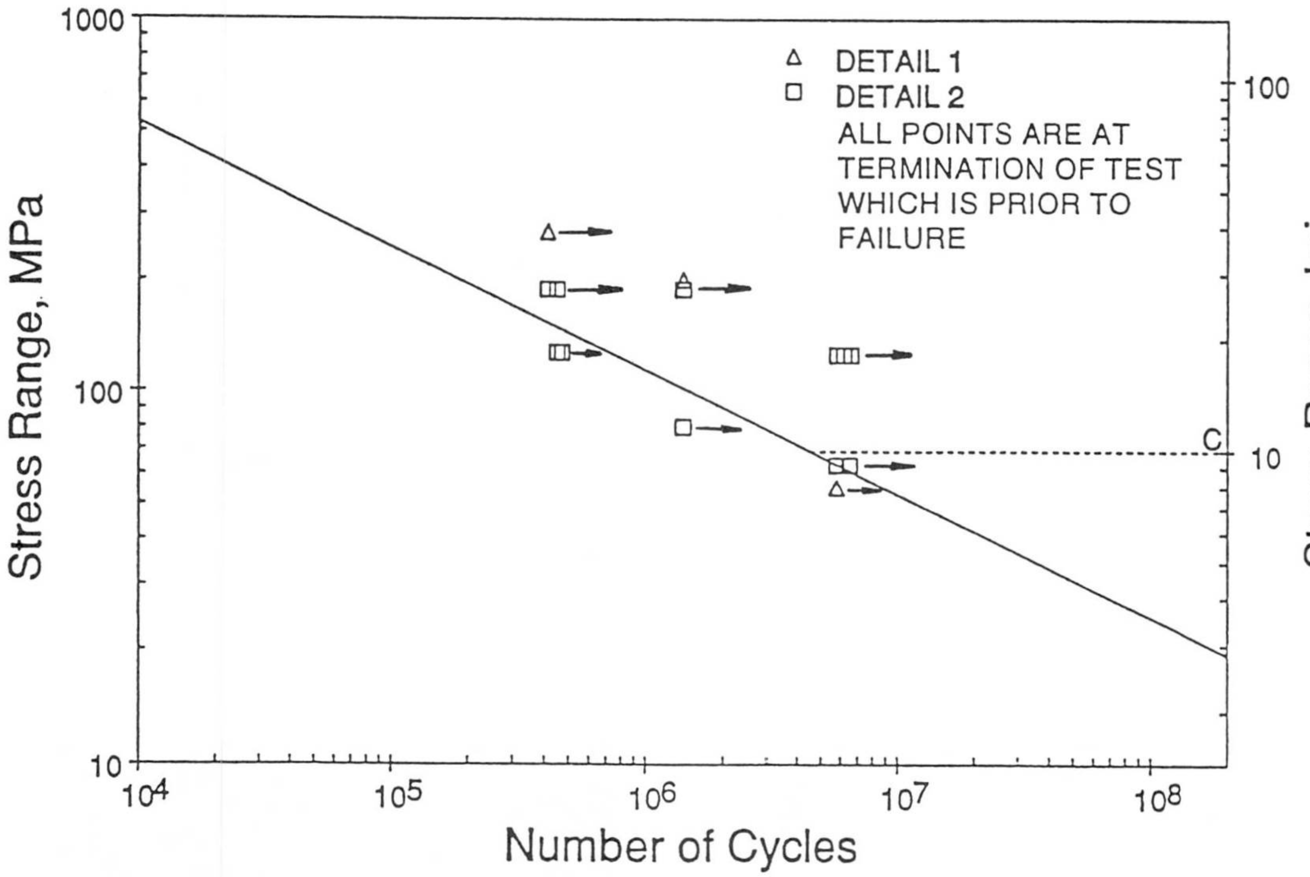

Figure 5: S-N data for cracks at the full-penetration connection detail plotted with theoretical life (Category C). Notes: 1) Detail 1 is cracking in the centerbeam at the connection (as shown in Figure 3) while detail 2 is cracking in the support bar at the connection. 2) The specimens had not actually "failed" and the cracks were still growing at the number of cycles indicated.