LOAD TEST EXAMINATIONS OF THE BUDAPEST SOUTHERN RAILWAY BRIDGE

A. SZITTNER, M. KÁLLÓ, L. KÖRÖNDI and F. SZÉPE

Department of Steel Structures
Technical University, H–1521 Budapest

Received: May 20, 1991.

Abstract

In this paper, the authors give an account of the results obtained by static and dynamic measurements on the bridge structures, and draw a comparison between those and the calculated results. In the second part of the account, the methods used for the examination of the fatigue and life expectation, respectively, relating to the structural elements of the bridge, as well as the results obtained are described.

Keywords: Railway, bridge, load test, fatigue.

Introduction

The history of the Southern Railway Bridge over the Danube was briefly summarized in the paper of G. NEMESKÉRI-KISS in the copy of October 1990 of the 'Közlekedésépítés- és Mélyépítéstudományi Szemle' (Scientific Review of Civil Engineering) [1]. The author of this paper gave an account of the design and construction of the 'fifth bridge' opened to traffic in 1948 and 1953, respectively, which is in service also today, as well as author referred briefly to *the future* of the two single track structures independent from each other. This latter point is promoted by the examinations with respect to the expected life conducted by the Department of Steel Structures at the Technical University of Budapest by the commission of the MÁV (Hungarian State Railways). The examination was justified by the fact that the first bridge in this place was in service only for 36 years when the bridge had to be replaced after it had been repaired in more hundred places. The second bridge was destroyed in the war after a service life of 31 years but the floor girders had to be repaired already prior to that time. The present structure of the right-hand track is 43 years old, however, it is known [2] that some years ago several marks of failures were observed on the stringers of this bridge due to which the bearing of the ties had to be restructured later on.

The continuous four-span structure opened to traffic in 1948 was designed to bear an ideal load 'A' as planned by dr. Imre Korányi on the basis of the Standard Specification for Railway Bridges (VH 38) issued in 1938. The other bridge opened to traffic in 1953 is of the same structure. According to the Standard Specification for Railway Bridges issued in 1976, which was undoubtedly in force at the time of examinations, the *load-bearing capacity* of the main girders of the major steel bridges dimensioned to load 'A", as specified in VH 38, meets the standard copiously, while that of the floor system meets the standard just in a satisfactory measure. The floor system of the bridge meeting the standard just satisfactorily will no more meet the standard specified in VH 76 with respect to *fatigue;* as far as the durability, i. e. the remaining fatigue life is concerned, no satisfactory results can be achieved even by static calculations more accurate than the specified one. In such cases — according to our up-to-date knowledge — the changes in stress measured in the structure should be compared with experimental laboratory results obtained with respect to fatigue; this experimental method is considered to be the relatively most reliable one in contrast to other speculative theoretical methods.

In the first part of this paper, the load test of the Southern Railway Bridge performed in 1987 is described, while in the other part, the evaluation of the stress measurements performed on the floor structure will be summarized. In another paper, account will be made of the laboratory fatigue tests on stringers removed from the Southern Railway Bridge, and on the stringers of a dismounted bridge over the river Sajó.

Load tests

The railway bridge specifications — like the VH 76, too — prescribe the regular inspection and the periodical load test of the bridges in service. The first load test of the right-hand track structure was conducted prior to opening the bridge to traffic on 18 September 1948, then at the time of supervision on 7 September 1965, and finally on 8 October 1987 in cooperation with the Department of Steel Structures. The basic task of the load test in conformity with stipulations was this time, too, the measurement of deflection on the main girders, which was completed by detailed stress measurements on the main girder and on the floor structure.

The first load test was carried out with 4 locomotives of serial number 424, the second one with those of serial number 411, while the last load test was realized with 4 locomotives of serial number M62. Due to the discrepancy between the loading vehicles, there is no possibility to draw a direct comparison between the measured deflection values. Instead, in all three cases the ratios of the measured and calculated deflections were compared with each other, and it was stated that no conclusion could be drawn from the results of deflection measurements with respect to any fault in the state of the main girders.

In the middle of the side span of the main girder, stress measurement was performed between the cross-girders No. 7 and No. 8 next to the contraction joint in the stringer on the bottom chord.

When the side span was loaded, in the southern main girder a normal force of 3134 kN, and in the northern main girder a normal force of 2914 kN were calculated on the basis of stress measurements, while at the same time, a bar force of 3330 kN could be calculated with theoretical locomotive weight. The deviation of such an extent between the calculated and measured bar forces can generally be experienced due e. g. to the contribution of the railway stringers with the main girders, nevertheless the deviation of 7% between the bar forces in the chords of the two main girders was remarkable. Such magnitude of asymmetry in the main girder cannot result from either the casual excentricity of the track centre line, or the variance of the weight of wheels alone. The deviation could be brought about by the fact that the three longitudinal girders, the cable channel and the reinforced- concrete slab of the pavement fixed to the northern main girder near the bottom chord take part in the contribution with the main girder.

Static Examination of Stringers

At the beginning of this account it was noted that the floor structure of the bridges dimensioned to load 'A' as specified in VH 38 just meets the load-capacity requirement specified in VH 76, but it is not satisfactory with respect to fatigue. In order to increase the reliability of the data obtainable by measurement for the fatigue examination of the floor system, the static experimental examination of the individual elements of it needed, as well as it is required to check the measured data by calculation.

Since the calculation of the stresses arisen in loaded bridges takes place generally by means of influence lines, the most useful method would be the measuring control of the ordinates of the influence lines. However, the structure cannot be loaded, in practice, by a unit load of adequate size, therefore instead of checking the unit-load influence lines, we are restricted only to the determination of the influenceline of the loading train. In the course of this, after having positioned measuring elements in the points chosen for the determination of the stresses in question, and moving a well-defined group of loads along the girder at given distances, the induced effect should be measured at each load position. These ordinates of the train influence line as measured at each load position can be compared to those obtained by calculation, and in this way, we can acquire information about the reliability of the computational model, too.

Influence lines of this kind were determined also on the stringers of the Southern Railway Danube Bridge. As a first step, the influence lines of stresses were calculated supposing a continuous beam with rigid or elastic supports. The two calculation models did not result in a significant deviation due to the given geometric and rigidity conditions of the floor beams (stringers and cross-girders), therefore the behaviour of the stringer can be approximated by assuming rigid supports, as well (*Fig. 1*).

Stresses in the cross- sections of the stringers were measured indirectly with the help of 6 – 6 pieces of resistance strain gage. In *Fig. 1*, as an example, the layout of the measuring points on the southern stringer positioned between the cross-girders No. 0 and No. 1 in the Buda-side span, as well as the train influence lines of stresses arising in the first midspan cross-section of the stringer can be seen, which were calculated with the help of the two static models, or computed on the basis of the measured greatest and average strain values, on the upper and bottom flange plates, by the effect of a single M62-type locomotive moving from left to right direction, as shown in the Figure.

The characters of the calculated and measured train influence lines are satisfactorily similar to each other, however, it is remarkable that the tensile stresses on the bottom flange plate are higher than those measured on the side in compression. This fact is generally explained by the contribution with the main girders, however, this phenomenon may be brought about by other causes, too. In *Fig. 1*, axial tensile stress (σ_N) calculated from the measured strains is changing nearly regularly with the loading of the stringer.

This observation refers to the fact that the neutral line of the cross-section is nearer to the upper exterior fibre than it is traditionally accepted due supposedly to the contribution of the wind- brace of the stringer, and it may also occur that the section in compression of the riveted stringer is originally somewhat more rigid than the section in tension.

Measuring elements can be located only on the accessible, free surfaces of the structure, i. e. between the ties and between rivets on the riveted stringer of the railway bridge. With this cross-section of the stringer, the stress calculation can be performed on the basis of the gross sectional modulus without neglecting the rivet holes. The result will be: the *smallest calculable stress;* the *stress measurable* between rivets — in case of stresses calculable with due accuracy — will be identical with the former, or somewhat higher than that. Since, however, the experimental results of

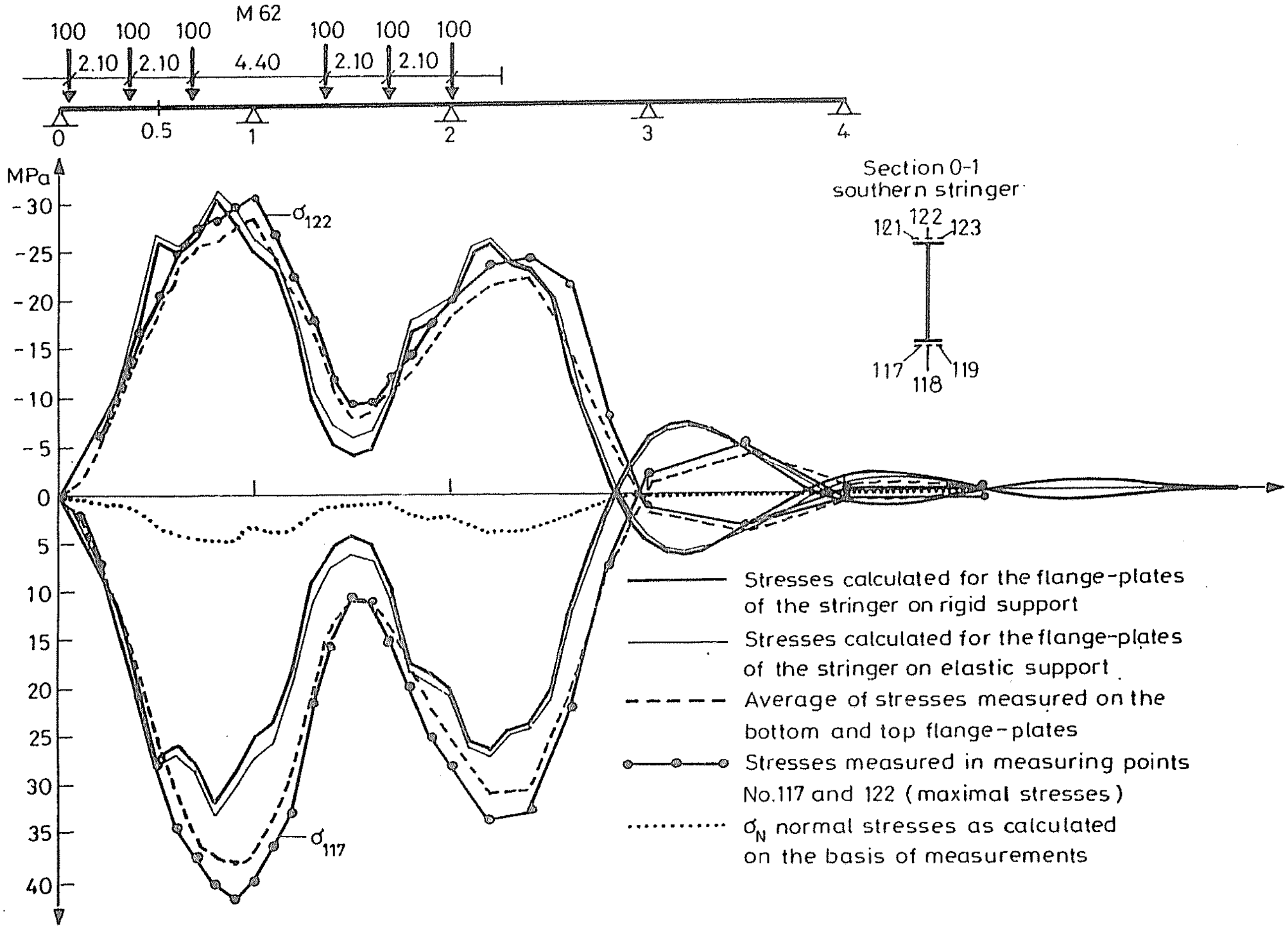


Fig. 1. Measured and calculated influence lines of stresses in cross-section 0.5 of the southern stringer

the fatigue tests involve logically stress values calculated on the basis of an effective cross-section, consequently, the *stresses measured* on the structure in the course of fatigue tests should also be converted into stresses expressed in terms of the effective cross-section, i. e. they should be increased: by multiplying them, in extreme cases, by the ratio of the gross- and the effective sectional modulus, respectively.

For conducting the fatigue test, a continuous registration and evaluation of the change of stresses caused by the actual traffic would be required in a number of measuring points. This can be implemented economically only with great difficulty, therefore — as an economical solution — the continuous measurement can be restricted to one or two measuring points only, per cross-sections which can be controlled also by calculations.

The location of the measuring elements on the stringer is limited by structural causes, because it cannot generally be guaranteed that the measurable cross-section be, at the same time, actually the most unfavourable cross-section with respect to fatigue. Instead we should be content with the examination of its neighbourhood; the influence of neglect can be estimated by calculation: it proves to be not more than some percentages.

Summarizing the remarks associated with the static examination of the stringers, it can be stated that the stresses measured on the stringers should be increased to a great extent so that they be compared reliably with the laboratory results, or for want of them, with the standard specifications related to the fatigue test.

Examination of Contraction-Joint in the Stringer

The Southern Railway Danube Bridge is a continuous trough-type truss with one break in the stringers of each span. The contraction joint in the stringer has a layout of the so-called plate joint. The broken butt end of the stringer is supported on the sliding saddle loading the cantilever clamped in the cross girder. The butt end of the stringer is secured against uprise by the plate joint connecting the cantilever and the cross brace of the stringer, enabling the rotation and horizontal displacement of the stringer end. This constructional form applied very often in our country is very sensitive to the inaccuracies in design, manufacture and mounting. It can occur also with new structures that the plate joint will not prevent the butt end of the stringer from rising upwards, and in this case, the plate joint will be subject to tension and the cross brace to bending; by the effect of continuously running traffic, the sliding saddle will be worn off heavily, and this wear can be even increased by the slight deformation of the stringer end. As a result, the stringer end in an unloaded state will float, while in a loaded state sink some millimetres, and thereby it will press the plate joint. The sinking of the permanent way impaired to such an extent can disturb the traffic, and the structural elements in the contraction joint can even suffer failure. (Otherwise, similar phenomena can occur with other anchorage mechanism, too, considered stress-released in an unloaded state.)

The significant influence lines of the contraction joint of the stringer at the joint No. 8 of the bridge are shown in *Fig. 2,* particularly in *Fig. 2a:* the stresses measured in the upper chord of the cantilever clamped in cross girder No. 8 and supporting the contraction joint of the stringer, *Fig. 2b:* the vertical motion of the saddles at the break of the stringer (measuring point No. 91: northern, point No. 92: southern stringer), and *Fig. 2c:* the influence lines of normal stresses measured on the plate joint.

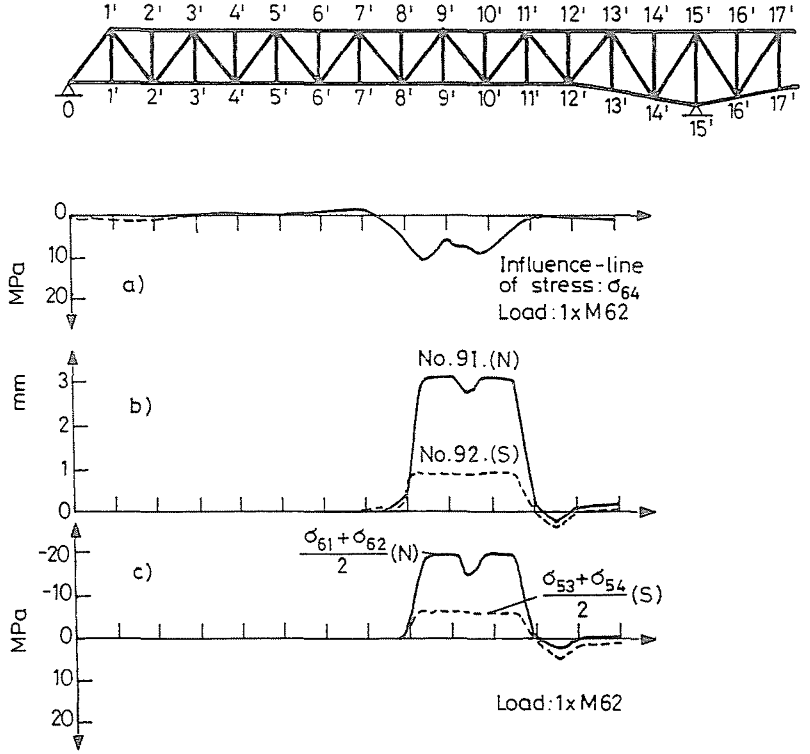


Fig. 2. Influence lines of stresses in the vicinity of the contraction joint in stringer

The couple of worn-off stringers were replaced by new ones, and the removed stringers were subjected to fatigue test. Results of the laboratory fatigue tests performed by the Department of Steel Structures are described in another paper.

Fatigue Tests

General Aspects

In the technical literature, the phenomenon of fatigue has been known for a long time, and the dimensioning for fatigue was already prescribed in the Railway Bridge Standard Specification VH 26 (1926). These prescriptions were modified by recent regulations parallel with the elaboration of the up-to-date results.

The meaning of the phenomena lies in the fact that the structure (element, connection) can suffer failure even prior to reaching the ultimate strength limit if the load is repeated very often. This phenomenon can be described by the Wöhler curve, which will show how the number of load cycles (N) causing fatigue failure is changing in the case of varying maximum stress ($\sigma_{\min}=$const.). The preferred point of the curve is value $\sigma_{\max}$ belonging to load cycle number $2 \cdot 10^6$; this value is called the fatigue limit belonging to a given $\sigma_{\min}$.

In the sequence of fatigue investigation on welded construction some observations deviating from the former experience were gained. The most important ones are:

— fatigue behaviour is mainly a function of $\sigma_{\max}-\sigma_{\min}=\Delta\sigma$, and it is nearly independent from the mean stress value,
— fatigue behaviour is almost independent from the strength of steel,
— 'fatigue curve' will be a straight line in the log-log system ($S-N$ diagram),
— fatigue straight line continues over load cycle number $2 \cdot 10^6$, too (without or with break), and it will possibly turn to a horizontal line only at a higher load cycle number,
— in the log-log system, the fatigue 'curve' can be defined by value $\Delta\sigma$ belonging to load cycle number $N = 2 \cdot 10^6$, and by the negative reciprocal of the slope of the line ($k=-1/\mathrm{m}$).
— the fatigue lines are parallel with each other, and the individual structural solutions are advised to assign to different $\Delta\sigma$ values belonging to load cycle number $N=2 \cdot 10^6$ and increasing according to Renard's series.

In the recommendation Eurocode 3, i. e. the fatigue lines at load cycle number $N = 2 \cdot 10^6$ were plotted with a slope $k=3$ starting from values $\Delta\sigma = 35$, 45, 56, 71, 90, 112, 140 and 180 MPa up to a validity limit of $N=5 \cdot 10^6$, where the straights break to a slope of $k=5$, and with another break at load cycle number $N = 5 \cdot 10^7$ they will go into a horizontal line ($k=\infty$) [3].

The $S-N$ diagrams published in the literature differ from each other in the slope of the straight lines, in the validity range of them, and in the development of the 'fatigue-strength' series.

In the standard specifications, the 'fatigue-strength' series is given according to the character of the connection. In the literature, the slope of the $S-N$ lines is assumed with value $k=3$ for the welded structures, and with $k=4$ for the riveted structures. According to the prescription of the Standard VH 76, the stress range $\Delta\sigma$ is to calculate from the service load multiplied by the dynamic factor. The service load is considered at the stringers as 60%, while in the main girder as 50% of the standard load. When $\Delta\sigma_{f,\mathrm{perm}}$ (index number of fatigue stage) belonging to load cycle number $2 \cdot 10^6$ is determined, the standard permits the increase or decrease, respectively, of the index number by $1-2$ values (by $10-20$ MPa) depending on the density of traffic.

The prescriptions of VH 76 can be applied also to the checking of existing structures, though the consideration of the history and the expected life by such approximations can involve pretty great neglect, and in many cases, it can yield results not justifying the serviceability of the structure with respect to fatigue. But beyond the question whether the structure (connection, material) is at the moment adequate or not, there is another important question to be answered, i. e. to what extent the fatigue load capacity of the structure has been exploited in the course of its service life so far, in other words, how far the structure can still be used in service according to the prescriptions mentioned (how long is the residual fatigue life), or what the safety of the structure is with respect to the present time or a time given in the future. These questions can be possibly answered with the help of the theory of cumulative damage, which is known as the Palmgren–Miner hypothesis. The point of this hypothesis is that load cycle number (n) obtained on different stress ranges and the fatigue load cycle number (N), relating to the same stress ranges, obtained from the fatigue curve allow the determination of the stage of damage belonging to each stress range (n_i/N_i). Summarizing these damage values calculable on different stress ranges, the cumulative damage $\sum(n_i/N_i)$ is obtained. In the case of a correct fatigue curve, the fatigue failure will occur when the condition:

$$\sum \frac{n_i}{N_i} = 1$$

is fulfilled (*Fig. 3*).

By adopting the Palmgren–Miner hypothesis and in knowledge of the stress range spectrum, the stage of damage and the safety against fatigue

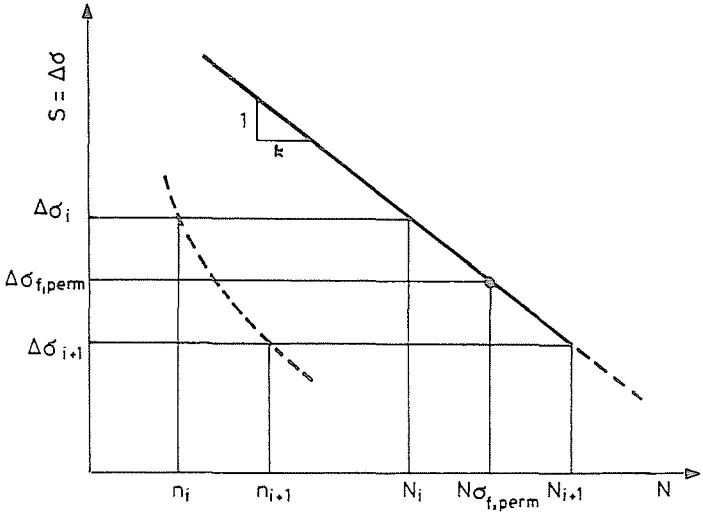


Fig. 3. Interpretation of the basic quantities of cumulative damage

can be determined. The equation of the fatigue curve defined in the log-log system by a straight line will be:

$$N = \frac{C}{\Delta\sigma^k} = \frac{N_{\Delta\sigma_{f,\,\mathrm{perm}}} \cdot \Delta\sigma^k_{f,\,\mathrm{perm}}}{\Delta\sigma^k},$$

where N is the load cycle number belonging to the different stress ranges $\Delta\sigma$,

C is a constant value for each stage of fatigue (for each 'curve'), and calculable with the knowledge of the point with coordinates $(N_{\Delta\sigma_{f,\,\mathrm{perm}}};\ \Delta\sigma_{f,\,\mathrm{perm}})$ given in prescriptions.

The *stage of damage* with the above values will be:

$$\sum \frac{n_i}{N_i} = \frac{\sum \left(n_i \cdot \Delta\sigma_i^k\right)}{N_{\Delta\sigma_{f,\,\mathrm{perm}}} \cdot \Delta\sigma^k_{f,\,\mathrm{perm}}},$$

while the *safety against fatigue* defined as the reciprocal of damage:

$$\overline{n} = \frac{1}{\sum \frac{n_i}{N_i}} = \frac{N_{\Delta\sigma_{f,\,\mathrm{perm}}} \cdot \Delta\sigma^k_{f,\,\mathrm{perm}}}{\sum \left(n_i \cdot \Delta\sigma_i^k\right)}.$$

For example, in the case of riveted girders (VH 76):

$$N_{\Delta\sigma_{f,\mathrm{perm}}} = 2 \cdot 10^6,$$

$$\Delta\sigma_{f,\mathrm{perm}} = 100\,\mathrm{MPa} \qquad \text{and} \qquad k = 4\,,$$

taken as a basis, the safety will be as follows:

$$\overline{n} = \frac{2 \cdot 10^6 \cdot 100^4}{\sum\left(n_i \cdot \Delta\sigma_i^4\right)} = \frac{2 \cdot 10^{14}}{\sum\left(n_i \cdot \Delta\sigma_i^4\right)}\,.$$

If the safety calculated in this way is smaller than 1, the structure cannot be operated without danger.

Naturally, the result of examination depends, among others, on the fact, too, how value k and the value of fatigue stress given (permissible) for load cycle number $2 \cdot 10^6$ is assumed. A further question is whether mean stress or approximate peak stress are reckoned with when determining values $\Delta\sigma_i$ found in the summation, and to what extent the applied dynamic factor is harmonized with the real dynamic effect.

With the above problems taken into consideration, the determination of the fatigue state is much more ensured by experimental examination which can be fulfilled on the basis of actual load and the effective behaviour of the structure.

Examination on the Southern Railway Bridge

For determining the fatigue load of the Southern Railway Bridge, a representative series of measurements was performed with the help of 100 trains passing through the bridge. In the course of measurements, the strain values were measured in the characteristic points in the midspan of the stringers in the section between the joints 0 and 1 Near the Buda side, on the stay plate of the stringer's cantilever, as well as on the rail flange between the ties at the end of the bridge. Measurement results were recorded continuously by an instrumentation tape-recorder and an 8-channel strip-chart recorder (*Fig. 4*).

Determination of the Formation of Loading Trains

The data of the representative estimate involving 100 sets of trains were processed in two ways:

a) the train formation, the weight of axles and the axle distances were defined,
b) by continuous observation of the alternation in the stresses, the density spectra of stresses, i. e. the cycle number of the stress amplitudes arisen under the influence of the actual load were assumed.

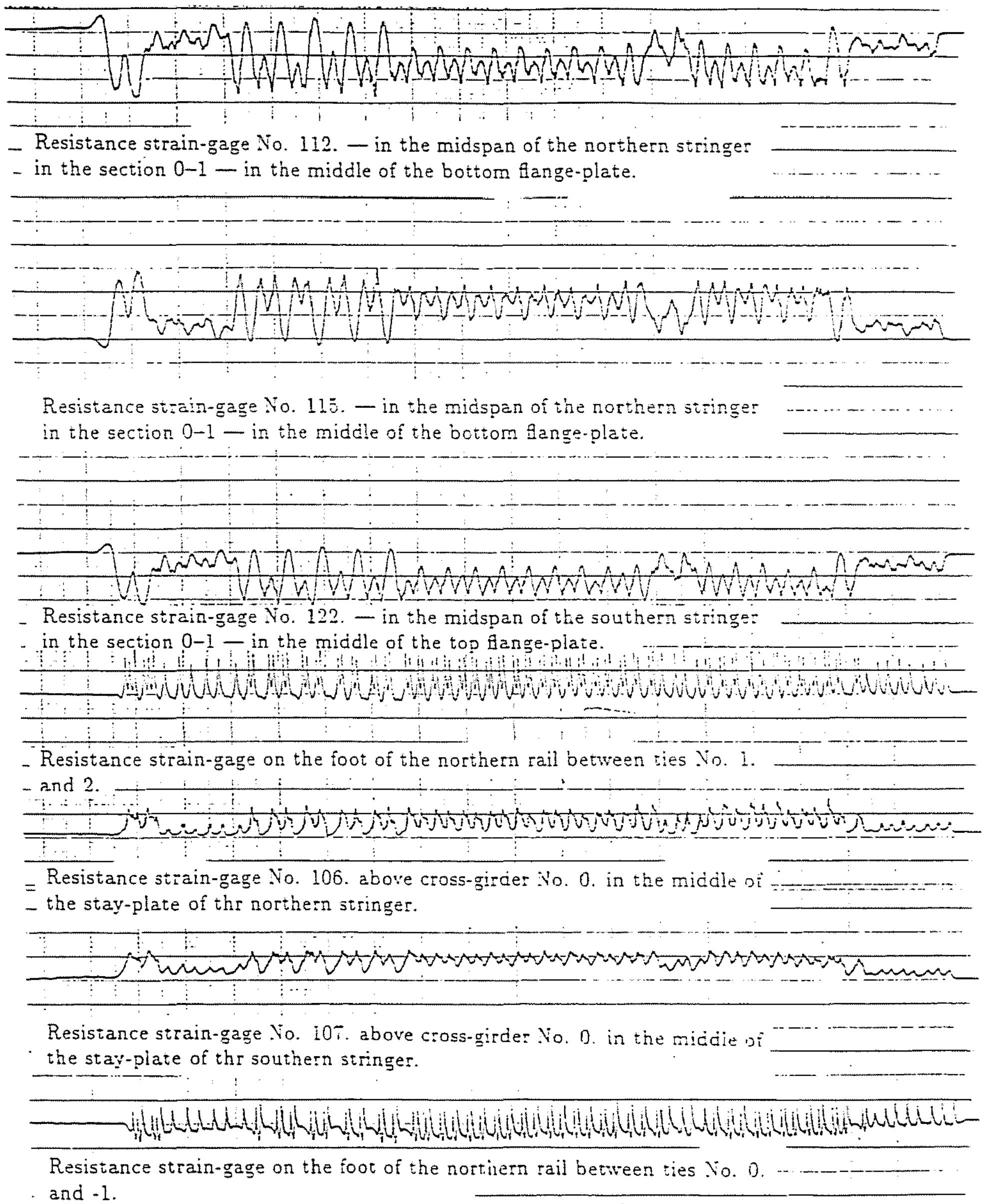

Resistance strain-gage No. 112. — in the midspan of the northern stringer
in the section 0–1 — in the middle of the bottom flange-plate.
Resistance strain-gage No. 115. — in the midspan of the northern stringer
in the section 0–1 — in the middle of the bottom flange-plate.
Resistance strain-gage No. 122. — in the midspan of the southern stringer
in the section 0–1 — in the middle of the top flange-plate.
Resistance strain-gage on the foot of the northern rail between ties No. 1. and 2.
Resistance strain-gage No. 106. above cross-girder No. 0. in the middle of the stay-plate of thr northern stringer.
Resistance strain-gage No. 107. above cross-girder No. 0. in the middle of the stay-plate of thr southern stringer.
Resistance strain-gage on the foot of the northern rail between ties No. 0. and -1.

Fig. 4.

For determining the train formation, the succession of analogue signals of gages bonded to the rail-foot, recorded by the measuring tape-recorder was fed into a computers A/D converter, which running at maximal sampling rate, determined the subsequent peak values — each proportional with an axle load — and the time intervals between the peaks — proportional with the axle distances. The first four (or six) extreme values are associated with the locomotive axles, therefore the sum of those should agree with the total weight of the locomotive. The nominal weight of the locomotive and the axle arrangement were entered into the computer as initial data. The computer calculated the axle distances and the axle weight distribution of the locomotive passing through the bridge. In the following, the computer used the data of locomotive weight and length for determining the longitudinal arrangement and the axle weights of the other carriages of the train set, as well as the specific load of the train.

The results obtained are correct in case:

- the nominal and the actual weight of the locomotive are identical,
- the axle weight is distributed between the two wheels on the same axle in the ratio of 50%,
- the maximum value detected on a measuring point is influenced only by the weight of the wheel passing over that point,
- the force resulting through the wheel is not influenced by the irregularities of the wheel and track,
- the train travels over the measuring point at a uniform speed.

We are quite sure about the fact that these conditions will not be fulfilled totally, however, for the lack of a more exact solution, the evaluation of the train formation was performed according to those mentioned above with the knowledge of the uncertainties enumerated.

Fatigue Tests on the Main Girder

From among the bars of the main girder it is the chord members and the truss members in the vicinity of the support whose behaviour is characteristic of the structure, and since the influence line of these bars generally consists of one or more long sections of the same sign, therefore the most unfavourable load of these bars can be determined - – with a good approximation — from the specific load on the basics of the principle 'one train — one maximum'. After processing the traffic load of trains as stochastic specific load, the results can be converted into the terms of bar stresses and examined in the following accordingly. The effect of single engines running can be neglected at these bars.

Fatigue Tests on the Structural Elements Characterized by Short Sections of Influence Line

With the bars involving short influence line sections, or with cross- girders and stringers, respectively, the stochastic distribution of stresses arising from the most unfavourable load cannot be determined in a simple way like that mentioned above. The test can be conducted in two ways, nevertheless in both cases stress oscillations $\Delta\sigma$ should be determined:

a) It should be examined what kind of trains were in service during the time up to now of the bridge, and those trains are substituted by certain types of trains. With the help of a computer, it can be run off what stress ranges and with what periodicities occurred when a train was passing through the bridge. If these stress alternations are analyzed with respect to the previous load history of the structure then the present stage of damage can be determined, on the basis of the principle of cumulative damage. In case the value of damage caused by both the trains previously in service and those to be planned to use in the future is available, then the residual life of the structure, too, can be estimated.

b) As a second possibility, in the characteristic points of the structure where the greatest stresses are supposedly arisen, resistance strain gages are bonded on. Then the analogue signals of stresses arisen by the effect are recorded (as a representative sample of e. g. 100 passing trains), and they are evaluated stochastically with respect to the stress oscillations. Neglecting the expected change in the composition of traffic, the results obtained in the case of 100 train sets can be extrapolated with respect to the previous history or the further service, respectively, and in this way, conclusions can be drawn as for the remaining fatigue life of the structure.

When processing the measurement data of a sample consisting of 100 train sets, the following procedure was applied:

In the course of determining stress ranges $\Delta\sigma$, the local extreme values were defined by a computer program analyzing the extremes of the stresses by the method known in the technical literature by the terms 'rain-flow' or 'reservoir' method [4].

After passaging the trains, the collected extreme values were grouped by a sub-program of processing further, and then the cycle number of occurrence in all the stress ranges (histogram) was plotted. The width of the groups was 5 MPa in our processing.

In the results of stochastic processing, the historical sequence cannot be traced back, however, this disadvantage can be eliminated by recording adequate cycles (100 trains, yearly or daily traffic, etc.). In the course of processing, the stochastic distribution of stress oscillations can be determined with respect to passenger, freight and other trains, too, as well as the single engine running, both separately and jointly.

Examinations Performed on Stringers

In *Fig. 5,* the stochastic processing of stress ranges greater than 5 MPa is shown on the basis of the diagram of stresses measured by strain gage No. 115 in the midspan of section No. 0 – 1 of the stringer in the centre line of the bottom flange (between two rows of rivets). From the results obtained in the course of measurements during traffic, it can be surely stated that the stress range measured in this point reached the value of 50 MPa only in exceptional cases. The measured stresses were multiplied by $1.2 \cdot 1.25 = 1.5$ due to the non-uniform load distribution in lateral direction and the stress concentration caused by the rivet hole. Since the stresses were measured during traffic, the measurement results contain the 'train factor' (dynamic factor), as well. In spite of this fact, the whole range of stress spectrum remains under the knee-point (fatigue limit) of the $S-N$-curve, and consequently, no damage would be caused according to the former conceptions.

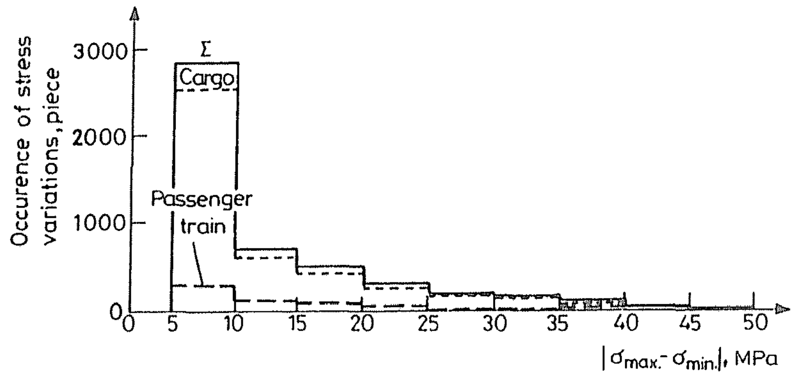
In order to extrapolate the measurement results of the '100 trains sample', the MÁV put at our disposal the traffic data of the trains having passed through the section between the stations Budapest- Ferencváros and Budapest-Kelenföld:

in 1986 (all the year round), and

in October 1987 (1 month),

which data were processed at the Computation Technique Institute of the MÁV. The data values of October 1987 multiplied by 12 showed a good agreement with the traffic data of 1986.

The single engine running does not enter into the traffic data of the MÁV, those were extrapolated on the basis of the measurement consisting of 100 samples of trains because the single engine running cannot be neglected any more with respect to the girder elements having short sections of influence line. The traffic data of the MÁV do not similarly contain the axle number of locomotives. This lack was corrected in a way that the



		Frequency number of stress variations											piece	%
Passenger train	piece	291	104	95	58	15	15	22	1	-	-	-	601	11.6
	%	48.4	17.3	15.8	9.7	2.5	2.5	3.6	0.2	-	-	-		
Cargo	piece	2567	623	432	255	201	168	85	15	3	1	-	4350	84.1
	%	59.0	14.3	9.9	5.9	4.6	3.9	2.0	0.3	0.1	0	-		
Passenger train+cargo	piece	2858	727	527	313	216	183	107	16	3	1	-	4951	95.7
	%	57.7	14.7	10.6	6.3	4.4	3.7	2.2	0.3	0.1	0	-		
Single engine	piece	119	12	7	21	28	14	10	4	1	-	1	217	4.2
	%	54.8	5.5	3.2	9.7	12.9	6.5	4.6	1.8	0.5	-	0.5		
Other trains	piece	4	2	1	-	-	-	-	-	-	-	-	7	0.1
	%	57.1	28.6	14.3	-	-	-	-	-	-	-	-		
Total sum	piece	2981	741	535	334	244	197	117	20	4	1	1	5775	100.0
	%	57.6	14.3	10.3	6.5	3.8	3.8	2.3	0.4	0.1	0	0		

Fig. 5. Stochastic processing of stress oscillations measured in cross-section 0.5 of the northern stringer in the middle of the bottom flange plate

Table 1
The loads of the Southern Railway Bridge in 1986

Service		Number of trains				
		passenger	cargo	other	single engine	totally
steam		—	—	34		34
electric		6637	13256	655		20548
motor		—	3	83		86
diesel		613	8465	780		9848
single engine						10595
totally	pieces	7250	21714	1532	10595	41112
	%	17.63	52.82	3.78	25.77	100

Table 1
(continued)

Service		Number of axles						
		passenger	cargo loaded	cargo empty	other	locomotive	single engine	totally
steam		79	545	264	26	170		1084
electric		274758	938881	445466	871	102740		1762716
motor		222	89	36	—	430		777
diesel		27184	578442	316858	850	49240		972574
single engine								60628
totally	pieces	302243	1517957	762624	1747	152580	60628	2797779
	%	10.80	54.26	27.26	0.06	5.45	2.17	100
			81.52					

average axle number of all locomotives was taken for 5 agreeing well with the data of measurement.

The data of the year 1986 processed in this way are shown in *Table 1.* The data of October 1987 were processed similarly. As a result of this, the yearly number of trains and axles:

— from the data taken in 1986: 41112 trains, 2797779 axles,
— from the data taken in Oct. 1987: 40756 trains, 2708080 axles.

The processing of the measurement consisting of 100 samples of trains, and the percentage ratio of certain types of trains are shown in *Table 2.*

Table 2
Evaluation of the traffic through the Budapest Southern Railway Bridge on the basis of the measurement of 100 samples of train sets in October 1987

		Number of trains					Number of axles				
		passenger	cargo	other	single engine	totally	passenger	cargo	other	single engine	totally
totally	pieces	16	54	2	25	97	700	5534	42	139	6415
	%	16.50	55.67	2.06	25.78	100	10.91	86.27	0.65	2.17	100

Detailed Examination of the Individual Elements of the Southern Railway Bridge

Examination of the main girder

The VH 76 prescribes the completion of the fatigue test on the main girder to be performed by applying the 0.5-fold value of the ideal load 'U' (UIC) as a service load, which should be increased by a train factor 1.42 — according to span $l = 98.52$ m. As a matter of fact, the stresses arisen in the bridge (in the main girder) under the influence of traffic are varying within a wide range, and do not generally reach the value calculated on the basis of ideal loads U or A multiplied by the train factor (1.42) and multiplied by the service load factor (0.5).

After processing the measurement data of the 100 train sample performed in October 1987 and the traffic data of the MÁV, it became possible to carry through the fatigue control with the actual or approximately actual traffic loads (passenger and cargotrains).

With respect to the main girder, the following assumptions were made:

a) According to the experiences gained from the load tests, the measured and calculated bar stresses show a very good agreement with each other, therefore the analysis was conducted by the calculated stresses.
b) The stresses arising in the chords due to train load can be calculated with a distributed load whose intensity can be determined from the total weight of the locomotive and the hauled train set on the basis of the train length. The stresses calculated from the obtained bar forces were multiplied by 1.2 due to the non-uniform distribution in the cross-section, the dynamic factor was reckoned with — according to the measuring experiences — by a value of 1.05.
c) The sections of influence line of the same sign resulted from the continuous character of the bridge were not totalled because the partial loading of the bridge will generally not occur in service.
d) Dead load was neglected since it has not any effect when the difference $\sigma_{\max} - \sigma_{\min}$ is formed.
e) The stresses were calculated for both the full and the effective cross-sectional areas of the bar examined.
f) For the evaluation of fatigue, the spectrum of the stresses calculable from the actual loads (measurement of the 100 train sample) was reckoned with stress cycle numbers belonging to the individual $\Delta\sigma_i$ stress range values. In the value of load cycle number, only the number of passenger and cargotrains as extrapolated for 50 years was taken into consideration (the influence of locomotives and other trains is negligible).
g) With the fatigue test. we started from values $N = 2 \cdot 10^6$ and $\Delta\sigma_{f,\mathrm{perm}} = 100\,\mathrm{MPa}$ according to VH 76, however, stress values $\Delta\sigma_i < 100\,\mathrm{MPa}$, too, were taken into consideration with respect to the cumulative damage (fatigue curve is in force also below $\Delta\sigma_{f,\,\mathrm{perm}}$).

The detailed calculation of damage (determination of safety) for the bottom chord bar No. 6 – 8 of the main girder was summarized in *Table 3* with the use of the following initial data:
cross-sectional area:

$$A_{\mathrm{tot}} = 1049\,\mathrm{cm}^2, \qquad A_{\mathrm{eff}} = 870.4\,\mathrm{cm}^2, \qquad \frac{A_{\mathrm{tot}}}{A_{\mathrm{eff}}} = 1.2052\,,$$

'influence line area':

$$T_{\mathrm{IV}} = 106.00\,\mathrm{m}, \qquad T_{\mathrm{III}} = -32.19\,\mathrm{m}\,,$$

Table 3

q specific load [kN/m]	$n_i^{(50)}$ 10^9	$n_i \Delta\sigma^4_{i,\,\mathrm{tot}}$ $[10^6\,\mathrm{MPa}^4]$	$n_i \Delta\sigma^4_{i,\,\mathrm{eff}}$ $[10^6\,\mathrm{MPa}^4]$	$n_i(1.2\Delta\sigma_{i,\,\mathrm{tot}})^4$ $[10^6\,\mathrm{MPa}^4]$	$n_i(1.2\Delta\sigma_{i,\,\mathrm{eff}})^4$ $[10^6\,\mathrm{MPa}^4]$
9	20.5	492			
12	418.9	31793			
15	332.8	61672	$c_1 = 1.2052$	$c_1 = 1.2$	$c_1 = 1.4462$
18	245.8	94458			
21	266.4	189644	$c_1^4 = 2.1097$	$c_1^4 = 2.0736$	$c_1^4 = 4.3747$
24	102.4	124364			
27	61.5	119633			
$\sum$	1448.3	622056	1312369	1289895	2721320
Damage		0.0031	0.0066	0.0064	0.0136
'Safety'		321	152	155	73.5

area corresponding to load oscillation:

$$|T_{\mathrm{IV}}| + |T_{\mathrm{III}}| = 138.19\,\mathrm{m}\,,$$

parameters of the fatigue 'curve':

$$N_{\Delta\sigma_{f,\,\mathrm{perm}}} = 2 \cdot 10^6\,,$$
$$\Delta\sigma_{f,\,\mathrm{perm}} = 100\,\mathrm{MPa}\,,$$
$$k = 4\,,$$

multiplication factor of stress: 1.2,
multiplication factor of stress and cross-sectional area:

$$1.2 \cdot 1.2052 = 1.4462\,.$$

From the *Table 3* it can be seen that the damage calculated from a load of the 100 train sample on the basis of stress oscillations extrapolated for 50 years is very small with this chord $(6-8)$, consequently, the safety defined as the reciprocal of the damage value is satisfactorily great. In case, with respect to the dense traffic of the bridge, the estimated value of the fatigue stage (fatigue limit) is decreased by 1 or 2 magnitudes, the degree of safety can be still considered very good. With the Southern Railway Bridge, load of 110 trains/day can be registered involving single engine running, which can be considered as normal according to the practice of the DB (Deutsche Bundesbahn) and the prescriptions of the UIC. In addition, the reduction of the fatigue limit will not be justified by the condition of the structure, either. Similar results can be obtained with the truss members, too.

Summing up those said above, the main girder of the Southern Railway Bridge — with correct bridge inspection, track and bridge structure maintenance assumed — can reach the maximum service life (80 – 100 years) in case the present-day traffic is keeping on. If the traffic (especially the axle weights or the specific load) would be increased in the following years, then the expected life of the bridge should be reduced in a proportional degree.

Examination performed on stringers

With the supporting elements placed on each other coming nearer to the loading vehicle on the structure examined, the stress oscillations generally increase, and the problem of fatigue will be more and more important. From structural point of view, the cantilevers of the stringer would be in the most unfavourable condition (the rail and the rail fastenings are not examined here), but — as our measurements show — there is hardly any moment arising in the cantilever of the stringer due to the first cross-tie located very near the cross-girder, and consequently, only small stresses arise in the stay plate, too.

The examination of the stringers was carried out in the places of the resistance gages positioned in the midspan between the cross-girders No. 0 and 1, and used before during the static examination. The investigation was conducted by the method introduced at the main girder, with the following consideration taken into account:

a) the stress oscillations yielded from measurements consisting of 100 samples of train sets were processed stochastically,
b) according to the prescriptions in VH 76, permissible fatigue stress $\Delta\sigma_{f,\,\mathrm{perm}} = 100\,\mathrm{MPa}$ was assumed for the riveted structure, however — considered the contraction joints in the stringers and the dense traffic — the fatigue safety of the stringers was examined additionally by applying value $\Delta\sigma_{f,\,\mathrm{perm}} = 80\,\mathrm{MPa}$ given in the Table No. 17. of the Hungarian Standard MSz $-07-3702-87$, as well as value $\Delta\sigma_{f,\,\mathrm{perm}} = 90\,\mathrm{MPa}$ generally accepted in the literature. The negative reciprocal of the slope of the $S-N$ line was taken into account by value $k = 4$ in this case, too.
c) degree of damage $\sum(n_i/N_i)$ was determined on the basis of straight $S-N$ assumed without horizontal section and that of the spectrum of stress ranges,

d) the cycle number of stress oscillations evaluated on the basis of load consisting of 100 samples of train sets was extrapolated for 50 years with the traffic data relating to year 1986,
e) while processing the stress data, the mean value of the individual stress oscillation ranges was considered. Mean value 2.5 MPa related to the bottom level between 0 and 5 MPa was assumed with cycle number $200 \cdot 10^6$, which, after all, exerts no practical influence on the final result,
f) since the measurements were performed during traffic, the measured stresses contain the dynamic surplus, too,
g) as the estimated maximum stress, the following values were taken into consideration:

— stresses (stress oscillations) — measured between two rows of rivets in the middle of the flange plates — were increased by 20% due to the unequal distribution of stress in lateral direction, and
— they were increased by an additional 25% in consequence of the stress concentrations next to the rivet holes;
— altogether the measured values increased by 50% were taken into account.

Table 4

$\Delta\sigma_i$ [MPa]	$n_i^{(50)}$ [10^6]	$n_i\Delta\sigma_i^4$ [10^{12} MPa^4]	$n_i(1.2\Delta\sigma_i)^4$ [10^{12} MPa^4]	$n_i(1.5\Delta\sigma_i)^4$ [10^{12} MPa^4]
57.5	0.022	0.240		
52.5	0.022	0.167		
47.5	0.087	0.443		
42.5	0.435	1.419	$c_1 = 1.2$	$c_1 = 1.5$
37.5	2.585	5.112		
32.5	4.295	4.792		
27.5	5.312	3.038		
22.5	7.365	1.888		
17.5	11.786	1.105	$c_1^4 = 2.0736$	$c_1^4 = 5.0625$
12.5	16.246	0.397		
7.5	65.088	0.206		
2.5	200	0.008		
$\sum$	313.243	18.815	39.015	92.251

The application of the calculation method developed on the basis of the above principles is shown in *Tables 4* and *5* with the use of stress oscillations measured by strain gage No. 115 (in the middle of the bottom flange plate) and with the use of different permissible fatigue stresses assumed according to VH 76. At measuring point No. 112 (on the upper flange plate) the situation is somewhat more favourable. The results obtained with the help of multiplying the measured stresses by 1.5 — approaching the reality — show that in case the traffic data of year 1986 and the performed measurement consisting of 100 samples of train sets are extrapolated for 50 years, then

with $\Delta\sigma_{f,\,\mathrm{perm}} = 100\,\mathrm{MPa}$ the calculated damage: $\frac{1}{\overline{n}} = 0.476$,

the safety: $\overline{n} = 2.10$,

with $\Delta\sigma_{f,\,\mathrm{perm}} = 90\,\mathrm{MPa}$ the calculated damage: $\frac{1}{\overline{n}} = 0.726$,

the safety: $\overline{n} = 1.38$,

with $\Delta\sigma_{f,\,\mathrm{perm}} = 80\,\mathrm{MPa}$ (according to prescription of Standard MSz–07–3702–87) the calculated damage: $\frac{1}{\overline{n}} = 1.163$,

the safety: $\overline{n} = 0.86$,

Table 5

serial number	$\Delta\sigma_{f,\,\mathrm{perm}}$ [MPa]	$N_{\Delta\sigma_{f,\,\mathrm{perm}}} \cdot \Delta\sigma^4_{f,\,\mathrm{perm}}$ [MPa^4]	C_1	$\sum n_i(c_1\Delta\sigma_i)^4$ [10^{12} MPa^4]	Damage $\frac{1}{\overline{n}}$ [—]	'Safety' $\overline{n}$ [—]
1	80	$8.192 \cdot 10^{13}$			0.230	4.35
2	90	$1.3122 \cdot 10^{14}$	1	18.815	0.143	6.97
3	100	$2 \cdot 10^{14}$			0.094	10.63
4	80	$8.192 \cdot 10^{13}$			0.476	2.10
5	90	$1.3122 \cdot 10^{14}$	1.2	39.015	0.297	3.36
6	100	$2 \cdot 10^{14}$			0.195	5.13
7	80	$8.192 \cdot 10^{13}$			1.163	0.86
8	90	$1.3122 \cdot 10^{14}$	1.5	95.251	0.726	1.38
9	100	$2 \cdot 10^{14}$			0.476	2.10

With serial No. 1 – 3, the spectrum of the measured stresses, with serial No. 4 – 6 and 7 – 9, the stress oscillations increased by 20 or 50%, respectively, were reckoned with.

With the fact taken into consideration that the examinations were carried out within the span of the stringer near the support of the main girder, which is in a more favourable position than the section of the stringer with a contraction joint in it, the conclusion is fully justified that *by the time of the 50th anniversary of the bridge construction, i. e. by 2000, the replacement of the stringers should be prepared, and it is advisable to carry it out successively on the two bridges.*

Considering that fatigue fractures are generally caused by some local failure reducing the fatigue strength and they are not calculable in advance, therefore the replacement of the stringers planned for a life of 50 years seems to be a realistic requirement only in case the continuous and documented inspection of the stringers is ensured and the floor system of the bridge is brought into a faultless state (replacement of the damaged ties and rail fastenings).

Proposal for the Determination of the Service Load and the Residual Fatigue Life of Existing Railway Bridges

As it was mentioned before, in the course of dimensioning the railway bridges the dimensioning for fatigue either took place or was omitted according to the prescriptions of the Railway Bridge Standard Specifications being in force at the time of design.

It can often be experienced that the existing bridge structures or structural elements, respectively, do not comply with the prescriptions of VH 76, or of the Standard MSz –07–3702–87. In these cases, it is advisable to recalculate the bridge structure with its actual antecedent service life taken into consideration, and to estimate its residual expected life. A more exact determination of the actual service load (recording of stress spectra) can take place — in our opinion — only statistically on the basis of the actual traffic, and with the help of measurements performed on the bridge or the railway line. The traffic registry of the MÁV is presently not suitable for the reproduction of service load, however, the data contained in it can be used for the determination of the number of trains in service along the examined lines, while the data are completed with single engine running.

With respect to the railway traffic, the damage can be divided into two parts: the previous service life and the residual life. In case of defined expected further life, the value of safety should be estimated. With a predetermined value of safety, the residual life can be calculated or prognosticated.

In case of reserves associated with the expected life of the structure are exhausted (e. g. the safety will decrease below 1.0 or other stipulated value), the maximum axle weight exceeds 20 tons, or deficiencies can be detected in the structure (corrosion, crack, loosening of rivets), the bridge inspection should be rendered more frequent, and in case of emergency, restriction in traffic or full blocking should be prescribed.

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Address:

Dr. Antal, SZITTNER
Dr. Miklós, KÁLLÓ
Dr. László KÖRÖNDI
Ferenc SZÉPE
Department of Steel Structures
Technical University Budapest
H–1521 Budapest, Hungary