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FATIGUE TESTS ON THE OLD STRINGERS OF THE RAILWAY BRIDGES

A. SZITTNER, L. KRISTÓF and L. KALTENBACH

Department of Steel Structures,
Technical University, H-1521 Budapest

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Abstract

Both elements in one of the sections of the stringers of the Southern Railway Bridge in Budapest were replaced. With the use of the removed stringers, as well as those of another railway bridge closed to traffic, laboratory experiments were conducted. In their course fatigue test of the basic steel material was performed by the step method using specimens cut out of different places of the longitudinal girder, which was followed by the fatigue test performed on one of the complete on-stringers of the Southern Railway Bridge. The results are reported in this paper.

Keywords: fatigue test, riveted railway bridge.

In connection with the supervision of the Southern Railway Bridge in Budapest, as it was reported in one of our papers, the checking of the bridge structure for fatigue took place, or it was estimated what reserves (safety) the main girders and the stringers have with respect to fatigue on the basis of the actual service load. As a result of these examinations, the statement was made that the safety of the elements of the main girders was satisfying, and the fatigue failure of the elements of the main girders was not expectable in the course of the expected life of 80–100 years of the bridge in the case of a proper bridge maintenance.

However, the safety is already much smaller in the case of the stringers, and after representative measurements of 100 trains, as well as the processing of the traffic data placed by Hungarian State Railways MÁV, the statement was made – which otherwise was in full harmony with the opinion of other European railways – that by the time the bridge, or the stringers, respectively, reached a life-time of 50 years, the safety against fatigue would be reduced to such an extent that the occurrence of fatigue could be expected at some places of the longitudinal girder subject most to load.

With those said above taken into consideration , we recommended that by the time the bridge reached a life-time of about 50 years, the exchange of the floor girders should be under way, which – with respect to the few-year difference in the life of the two adjacent bridges – could be performed successively in the most effective way after a third bridge had been built.

In the course of the supervision of the Southern Railway Bridge it was determined that the breaks in the stringer were badly worn out due to the smaller structural deficiencies. Therefore, the MÁV ordered that the section of the stringer worn out most between joints No. 7 and 8 of the first opening on the Buda side should be replaced.

Since this section of the stringer was lifted out without any considerable damage to the girder, the MÁV considered it to be verified to perform such experimental examinations on this section which could provide a basis for having the expected answer to the question: what damage to the stringer had occurred during its life-time of the previous 40 years with respect to fatigue.

The replacement of the old bridge on the railway line between Bánréve and Ózd preceded the supervision of the Southern Railway Bridge by some years, so that two stringers could be removed from it with a similar purpose prior to its smelting-down.

The examinations on the two pairs of stringers were performed parallel with each other, and in the following, we give an account of the examination results.

The Previous Life of the Examined Stringers

The material of the two stringers removed from sections No. 7 and 8 (the place of the break in the stringer) of the Southern Railway Bridge is mild steel, the year of its construction was 1948, consequently, the examination took place after a service life of 40 years. The examined stringer is an 8-span continuous, multi-span girder with an upper stay plate and haunch. The dimension of the web plate is 940–10 mm, that of the chord angle steels is 4 × 90–90–9, while that of the upper flange plate is 260–11 mm. Due to the original fastening of the bridge ties, the upper flange plate is asymmetric. Owing to the repeated bending caused by the bridge ties, cracking or fracture could be experienced in many places of the overhanging parts, therefore the bridge tie fastening was modified in 1964. The bottom flange plate is a symmetric plate of nominal dimension 210–11 mm, however, the actual thickness of both examined girders was 12 mm. The span of the examined stringer was 6305 mm, while the diameter of the fastening rivets was $d = 23$ mm.

The old trussed railway bridge over the river Sajó between Ózd and Bánréve ($l = 41.2$ m) was constructed in 1911–12 of mild steel. There are no data available about what elements were replaced after the damage to the bridge during World War II. The dimensions of the two stringers removed for examination agree with those given in the plan. It had an upper stay plate but no bottom stay plate was constructed. The dimension of the web plate: 600–10 mm, while the chord angle steels are unequal angle steels (L-bars) of dimensions 40 × 80–100–10 mm. The upper and bottom flange plates are of a symmetric layout, their dimensions are: 250–11 mm. The bridge was closed to traffic in 1987.

Test Program

The tests were performed in two directions:

a) tests of the material,
b) tests of the structure.

The material testing was performed on the middle section of about 2 m of the stringers according to the cut-out program of the specimens (*Figs. 1* and *2*).

The places from which the specimens were cut out were selected so that the comparison between the results obtained from those may provide a possibility of comparing the effects caused by the different internal forces. Accordingly, the specimens of symbol A were cut out of the sides of the flange plates, as well as the bottom side of the web plate so that the edge of the rivet hole should coincide with that of the processed specimen for the tensile test, with the exception of the 'undisturbed' sample cut out of the middle part of the web plate.

The specimens of symbol B were cut out of the horizontal flange of the chord angle and the bottom edge of the web plate so that the original rivet hole should coincide with the axis of the specimen, and the processed width of the specimen laid out in the form of a tensile specimen should be three times of the rivet hole.

The specimens of symbol C were cut out of the middle part of the flange plates, as well as the nearly unloaded, 'undisturbed' middle section of the web plate. Into those specimens, new holes of dimensions identical with those of the original rivet holes with the specimens B were drilled.

The specimens of symbol D were of the Charpy type with dimensions 10 × 10 mm, which were cut out of the flange plates and the middle part, or the edge of the web plate, respectively, in rolling direction and perpendicular to it. The cut-out operation of the impact specimens also took place so that the specimen should be fitted to the edge of the rivet holes, with

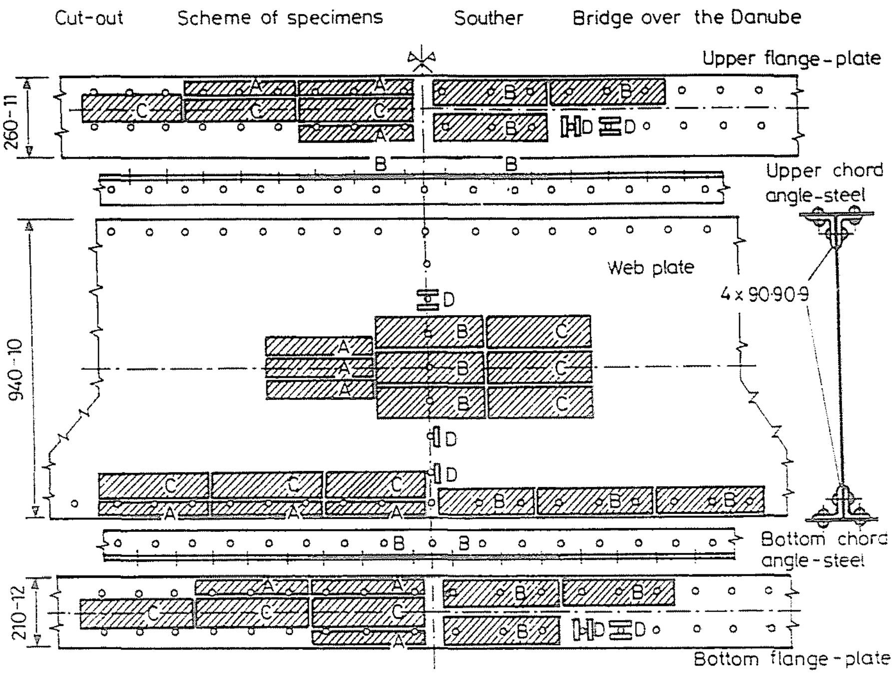

Fig. 1. Southern Railway Bridge. Cut-out plans of specimens

the exception of the 'undisturbed' samples cut out of the middle part of the specimen.

Static Test of the Steel Material

From among the specimens pre-treated in this way, 1 piece off each type was tested as a tensile specimen according to the Hungarian Standard MSZ 105. An exception to this was the tensile specimen with a hole which is not covered by the standard, and consequently which is not a correct one with respect to the tensile strength and yield strength while the ultimate strain cannot be evaluated.

On the basis of the tensile tests of the Southern Railway Bridge, the steel material is satisfactory with respect to the tensile strength, yield point and ultimate strain. However, a very unfavourable result was yielded for the specific impact value, for which a minimal value of 27 Joule is stipulated in Standard MSZ 6280 in the case of a material of grade 37 at the temperature 0 °C. Particularly, all the test pieces of lateral direction without exception, while from among the specimens of longitudinal direction the ones positioned on the upper flange plate remain under the stipulated

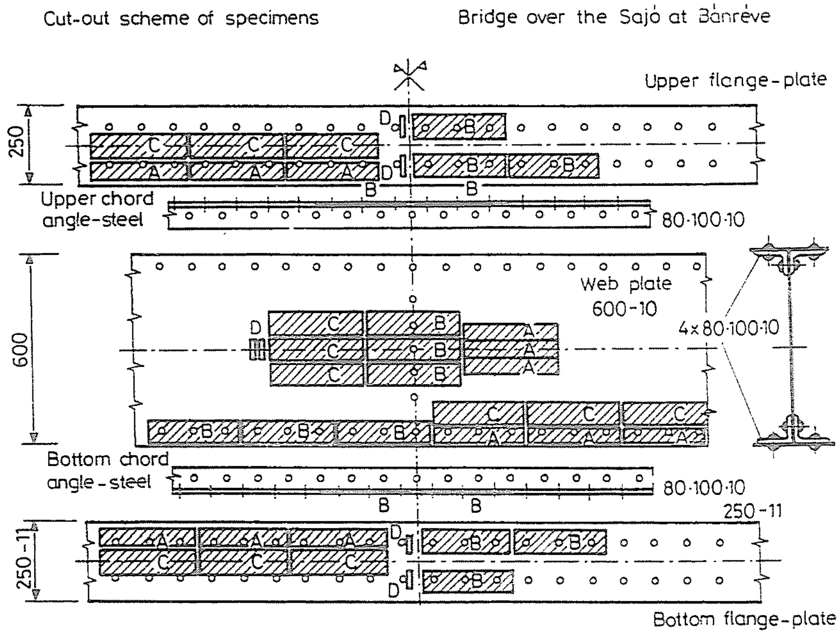

Fig. 2. Bridge over the River Sajó at Bánréve. Cut-out plans of specimens

value in many cases to a considerable extent. The very low impact value involved the brittle fine grained surface fracture without contraction.

The specimens cut out of the bridge over the River Sajó, with the exception of specimens: 2 pieces AGK 1, 2 pieces CGK 1, and 1 piece CGK 1 cut out of the web plate of bridge No. 2, all complied with the specification in standards MSZ 6280 and MSZ 500, respectively, from the viewpoint of tensile strength, yield point and ultimate elongation.

On the unfavourable results of the impact value, two comments can be mentioned:

– at the time of the construction of the two bridges tested, the killing of the mild steel was not usual, or it was not prescribed, probably, this fact could have been proved by chemical analysis,

– due to the increased load especially in the vicinity of rivet holes, the crystalline structure will be modified unfavourably (aging), and this, in turn, involves a reduction in the impact value. This seems to be proved by the fact that in the case of the specimens cut not out of the vicinity of the rivet hole, the impact value satisfied the specification of the standard.

Fatigue Test of the Steel Material

To determine the fatigue behaviour of the steel material, step method fatigue tests were performed on the specimens removed from the longitudinal girders of the bridges in question, as described earlier, because a full S–N diagram cannot be recorded with such a small amount of specimens. Besides, we thought that the comparison between the different fatigue results obtained under equal circumstances from the similar specimens removed from the different components could provide some orientation concerning how the damage already occurred at some places (previous life) affected measure of survival (reserve) remained yet in the structure.

The multi-stage fatigue took place according to the principles of the cumulative fatigue damage (Miner-hypothesis). In the course of the test, the following assumptions were made:

a) the S–N diagrams are parallel straight lines (S–N lines) in the log–log systems, and
b) the straight lines are continuous ones, neither a directional change, nor breakage can be experienced in them,
c) the straight lines can be given by the fix point, by the value of the stress ranges belonging to cycle number 2.10^6, the stress difference $\Delta\sigma$ belonging to cycle number 2.10^6 called in the previous fatigue limit, with the slope of the straight line: $m = 1/k$,
d) the evaluation took place with different values of k, particularly:
   with $k = 3$ as recommended by Fisher for the welded beams,
   with $k = 3.75$ corresponding to the prescription of the UIC,
   with $k = 4$ related to the riveted structure in VH 76, and
   with $k = 5$ according to the recommendation of Maarschalkerwart.

In the course of the evaluation of the fatigue tests, the 'equivalent fatigue limit', or the straight line belonging to it were determined by a computer program in MPa units, for which the assumption

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

will be fulfilled with the Palmgreen–Miner hypothesis assumed as valid. The experiments were carried out with the use of a servo-hydraulic test machine of the Mohr–Federhaff Losenhausen HUS 40 type.

The parameters of the step method were the following:

initial level with specimens of type A	210 MPa,
with specimens of type B	100 MPa, then
with others	135 MPa
rise of the level of steeps for all specimens	15 MPa,
the cycle number belonging to the individual levels	1.10^5.

From among the results obtained in the course of computer-aided evaluation, only the values of the equivalent fatigue limit obtained by values $c = 1$ (multiplication factor c refers to the proportion of the peak stress along the cross-sectional area of the specimen to the average stress) and $k = 4$ (specified for the riveted beams in VH 76) will be imparted here.

Table 1 contains the results of the fatigue tests performed on the steel material of the Southern Railway Bridge, and

Table 2 contains the results of the fatigue tests performed on the steel material of the bridge over the River Sajó.

In the Tables, the following were given in each column: the results of the equivalent fatigue limit obtained for specimens of

type A (specimen without holes),
type B (specimen with the original rivet holes),
type C (specimen with bored holes),

as well as the ratios of the results between columns A and C, and columns C and B were given in the last two columns.

From the results reported, the following conclusions can be inferred:

Longitudinal girder of the Southern Railway Bridge (Table 1)
The results obtained with specimens A reflect – though only to a small extent – the differences due to their original positioning. The lower value obtained for the upper chord is well explained by the considerable seizure in the former bearing surface of the tie in the place of the cut-out specimen. Summarizing the results, we can say that the comparative fatigue limit deviates by ±5 % from the average value of 200 MPa.

With the specimens B of a cross-sectional area reduced by the original rivet holes, the unexpected result was obtained that the lowest 'fatigue limit' was observed with the specimens cut out of the middle of the web plate. The cause of this phenomenon is given clearly in photo No. 3 taken of specimen GK 3. Since the original hole in the specimen has an extraordinary rough surface, it was probably processed without 'reaming', and this fact verifies the lower value. Since a hole of such a layout can occur anywhere, the low fatigue limit is determined by the technology used rather than by the position of the hole.

The fatigue limit realized on the underside of the web plate and on the bottom flange plate are nearly identical this time, too, and ratio A/B is 1.65 in both cases. This ratio is somewhat smaller with the upper flange plate (1.61), which is verified by the seizure in the upper chord caused by the sleeper-tie.

The fatigue limit of specimens C equipped with 'laboratory' holes and with dimensions identical with those of specimens B is much more favourable in comparison to the fatigue limit of specimens B, which is reflected also in ratio C/B. This fact shows the importance of the task that the final dimensions of the rivet holes should be developed by reaming after occasional boring or punching.

Summarized all those said above: the fatigue tests performed on the steel material of stringer 7–8 of the Southern Railway Bridge resulted in an equivalent fatigue limit of 200 MPa ± 10 MPa for the blank specimens. This value is 40 % with fatigue tests performed on the existing rivet holes, while in the case of the rivet holes of identical dimension as bored in the laboratory showed a reduction of 25–30 %. According to the data found in literature, the fatigue limit of the riveted joint is even somewhat smaller than that, but no possibility of testing was provided for us.

It should be especially noted that the fatigue limit of 200 MPa can be reduced to 100 MPa due to the careless operation, and in this way, the fatigue index 10 without any safety factor was reached as given for the riveted joints by the VH 76 ($\sigma_{f,\mathrm{perm}}$ =100 MPa). If even a minimal safety: 1.2 is accepted – with the considerable traffic load of the Southern Railway Bridge – then the reduction of index number 10 by 2 values, and consequently, the check of the longitudinal girder with the use of value $\sigma_{f,\mathrm{perm}}$ =80 MPa do not seem an unreal requirement. *This fact, too, confirms our earlier standpoint that the replacement because of the fatigue of the floor beams with the Southern Railway Bridge will be due by the turn of the second millennium. This date, however, can be considered a reasonable one regular supervision and continuous maintenance of the bridge are ensured due to the unfavourable experience gained by testing the existing rivet holes. In this way, it can be assumed that the damage disclosed and repaired in due time within the period till the end of the century can be eliminated at the expense of a shorter blocking of the bridge.*

Longitudinal girders of the bridge over the River Sajó between Bánréve and Ózd

The results contained in *Table 2* are self-explanatory, and are in consonance with the results related to the steel material of the stringers of the Southern Railway Bridge. Attention should be drawn to an outstanding value: with the bored specimen BOA 2 cut out of the bottom flange plate of longitudinal girder No. 1, a fatigue limit of 102.2 MPa was obtained. This low value can be explained well by the mechanical erosion shown in *Fig. 4*, and verifies the fatigue limit of 100 MPa found in the VH 76, or the further reduction of this value if an adequate safety is prescribed.

In this way, the fatigue tests performed on the specimens removed from the two different bridges verify in concert the prescriptions of the VH 76 because the fatigue failure even in the case of riveted bridge struc-

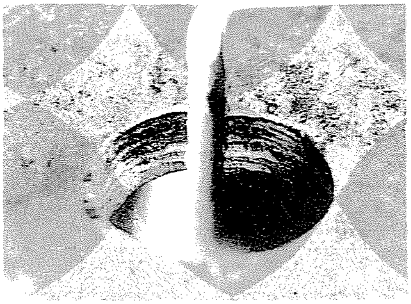

Fig. 3. Rough surface of rivet holes developed with high thread pitch

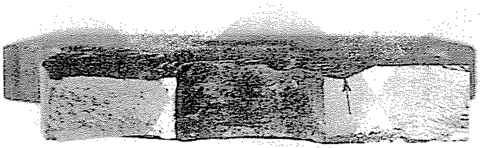

Fig. 4. Fatigue-failure started from a surface deficiency

tures can develop mainly as a consequence of some local defect (careless processing).

Fatigue Tests of Structure

Parallel to the fatigue tests performed on the stringer as a disintegrated material removed from section 7–8 of the Southern Railway Bridge, the other longitudinal girder was subject to fatigue test as a beam.

The disintegrated part of the stringer was fixed at one end in a supporting bracket which was fastened to the testing bench with the help of HSFG-bolts, while at the other end, a roller support was applied. The span of the stringer was adjusted to the actual span of 6305 mm.

The loading of the beam, as it can be seen in *Fig. 5*, took place with the use of two servo-hydraulic actuators of type MTS with a maximum

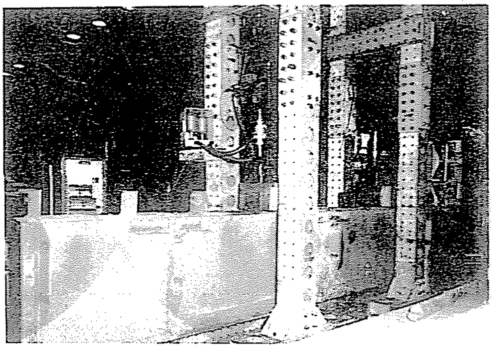

Fig. 5. Structural fatigue loading of the stringer of the Southern Railway Bridge in laboratory

capacity of 240 kN each fastened through pieces of sleepers to the sleeper pads spaced out at a distance of 2 m from each other, symmetrically to the middle.

The loading of the stringer started from 2 × 100 kN, and after reaching a cycle number $n = 10^5$, the load was increased by 10–10 kN, and so the loading was continued first till 2 × 200 kN, then after a retreat to 2 × 150 kN, the cycle number was completed to 2×10^5 on the individual load levels proceeding similarly by stages of 2 × 10 kN subsequently, it was increased up to 2 × 240 kN with a cycle number $\Delta n = 2.10^5$ and an increase in load of 2 × 10 kN with each stage. This load was already greater than that recommended by the factory as a dynamic operating load.

Prior to starting the fatigue tests, in the middle of the stringer on the bottom and upper flange plates, on the edge of the flange plates and above the web plate, a measuring basis of 100 mm was developed on each component mentioned to carry out strain measurement by a movable strain meter of Pfender type. Strain measurements were performed after the start and finish of each loading steps.

The strains measured at points 1–6 (1–3: upper flange 4–6: bottom flange were determined – with their average value calculated per flanges by averaging them according to the area of the stress diagram) – with the help of formulae:

$$\sigma_{\text{upper average}} = \frac{1.2\sigma_1 + 2\sigma_2 + 0.8\sigma_3}{4} ,$$

$$\sigma_{\text{lower average}} = \frac{\sigma_4 + 2\sigma_5 + \sigma_6}{4} .$$

The measured stresses as well as those calculated on the basis of load are shown in *Table 3* in the sequence of load steps.

During the experiment conducted in a proper way, neither fatigue failure nor any sign of it could be observed.

The loading process could not be increased any further because the capacity of the actuator (work-cylinder) ran out, neither the increase of the cycle number was considered to be a reasonable means with a load of 240 kN. Otherwise, the load of 2 × 240 kN applied to the longitudinal girders at a distance of 2.0 m corresponds to an axle load of 480 kN, which is very overloaded even with the dynamic factor taken into consideration.

Consequently, in the course of the fatigue test evaluation, it was possible only to determine a lower fatigue limit for which the actual fatigue load may be greater than, or at most equal to that.

In the course of evaluating the experiments, as a first step, the stress ranges were assumed according to the load steps, and they were assigned to values $\Delta\sigma$. Afterwards, with the use of partial results of the fatigue test performed on the basis of the actual traffic through the Southern Railway Bridge, and by reducing the cycle numbers belonging to the individual stress cycles down to 40 years, the previous life of the stringers, then the two kinds of the load effect were summarized in the form of value pairs $\Delta\sigma - \Delta n$. The evaluation was performed with values $k = 3$, $k = 3.75$, $k = 4$ and $k = 5$, then values $n_i\sigma_i^k$ were calculated for the experimental data, as well as for the data of experiment + previous life, and then they were compared to each other. From the results, it can be seen that the weight of the laboratory fatigue will be increased directly with the increase of k, the weight ratio will be 3:1 in the case $k = 4$, while in the case $k = 3$ it will be 3:2.

Further processing was performed with the use of different parameters, or the combination of those, respectively:
values $k = 3$, 3.75, 4 and 5 represent the effect of the angle of slope of the S–N diagram,

values $\sigma_{f,\mathrm{perm}} = 80$, 90, 100, 110 and 120 MPa represent the effect of the assumed index numbers of fatigue, while

values $c = 1.0$, 1.2, 1.25 and 1.5 represent the ratios of the local maximum stress and the average stress.

The run and the results of processing performed accordingly and described in the foregoing can be seen in *Table 4*. In the last column of the Table, the minimum safety related to the individual values is indicated.

From the results, the following conclusions can be deducted: the effect of $\sigma_{f,\mathrm{perm}}$, and the multiplication factor c taken on the safety is obvious, the safety will increase with the vise of $\sigma_{f,\mathrm{perm}}$, while it will be reduced with the increase of c, safety will grow with the increase of values of k.

Since there is no uniform prescription or experience for the assumption of these factors, with the assumption of those, the results (calculated safety) can be manipulated. The solution to the problem will be aggravated by an additional fact that – as it could be seen also from the results of the fatigue tests performed on steel material specimens – the fatigue behaviour is influenced decidedly by the local anomalies (erosion, notches during processing). This fault can occur in a place which has a determinant role with respect to fatigue (flange plate, edges of the web plate). This possibility of damage could be reckoned with by the increase in the prescribed safety, however, no proper experimental results for this are available so far. In some places, safety is rated 1.4–1.5 times greater than usual, however, these values can be applied only to loads or structures, respectively, built in a well defined 'empiric' way. A value greater than that should be probably assumed for fatigue safety.

The concrete evaluation will be aggravated by the fact, too, that the results reported here are based only on a single test, consequently, they can not be evaluated statistically. It should be mentioned in addition that our results – with the exception of failures – are only minimum values in comparison to which the actual fatigue behaviour is generally more favourable.

To summarize the results, it can be stated that the lower fatigue values (80, 90 MPa) obtained with the girder tested are not real ones, since the safety calculated by these values is opposed to the experimental results. With a fatigue limit of 100 MPa, as well as with values $k = 4$ and $c_1 = 1.25$ or $c_1 = 1.5$, the safety calculated on the basis of experimental results seems to be reasonable, therefore the application of such parameters to the fatigue test of riveted railway bridges is considered to be verified.

Table 1

type of spec / place	$\sigma_{fat}[N/mm^2]$						A/C	C/B
	A		B		C			
		average		average		average		
GA	198.3 199.8	199.05	112.2 129.1	120.65	139.7 142.7	141.2	1.41	1.17
GK	213.1 205.5	209.3	105.7 100.0	102.85	148.1 147.2	147.65	1.42	1.43
ÖA	209.0 191.3	200.15	118.0 124.7	121.35	154.2 148.1	151.10	1.32	1.24
ÖF	191.1 –	191.1	129.1 108.1	118.6	136.1 126.5	131.3	1.46	1.11

Table 2

type of spec place			$\sigma_{\mathrm{fat}}[\mathrm{N/mm^2}]$						A/C	C/B
			A	average	B	average	C	average		
GIRDER No. 1	web plate	GA	182.2 176.1	179.15	130.6 133.9	132.25	148.8 142.8	145.8	1.23	1.10
		GK	211.7 199.0	205.35	141.2 155.8	148.50	141.7 152.1	146.9	1.40	0.99
	flange plate	ÖA	175.9 198.4	187.15	102.2 134.1	118.15	141.2 135.2	132.2	1.35	1.17
		ÖF	– 192.2	192.20	152.0 129.0	140.50	142.3 137.0	139.65	1.38	0.99
GIRDER No. 2	web plate	GA	– 175.9	175.90	136.6 134.0	132.3	123.2 134.0	128.6	1.37	0.97
		GK	180.0 179.7	179.85	131.1 133.9	132.0	129.1 122.9	126.0	1.43	0.95
	flange plate	ÖA	220.6 198.3	209.45	121.2 142.8	132.0	145.4 155.9	150.65	1.39	1.14
		ÖF	199.6 188.5	194.05	123.4 142.5	132.95	165.5 138.6	152.05	1.28	1.14

Table 3

Load		Upper flange		Bottom flange	
F	Δn	σ[MPa]			
[kN]		measured	calculated	measured	calculated
2 × 100	10^5	-33.0	-32.2	34.0	32.6
2 × 110	10^5	-40.6	-35.4	36.1	35.9
2 × 120	10^5	-40.0	-38.6	37.6	39.2
2 × 130	$4.13 \cdot 10^5$	-43.7	-41.9	45.8	42.4
2 × 140	10^5	-48.8	-45.1	48.4	45.7
2 × 150	$2 \cdot 10^5$	-52.9	-48.3	54.1	48.9
2 × 160	10^5	-56.2	-51.5	54.1	52.2
2 × 170	10^5	-57.9	-54.7	55.6	55.5
2 × 180	10^5	-62.2	-58.0	59.7	58.7
2 × 190	10^5	-67.8	-61.2	63.3	62.0
2 × 200	10^5	-72.5	-64.4	68.5	65.3
2 × 150	10^5	-48.8	-48.3	50.5	48.9
2 × 160	10^5	-56.4	-51.5	54.1	52.2
2 × 170	10^5	-58.3	-54.7	57.2	55.5
2 × 180	10^5	-62.2	-58.0	59.7	58.7
2 × 190	10^5	-67.8	-61.2	63.3	62.0
2 × 200	10^5	-72.5	-64.4	68.5	65.3
2 × 210	$2 \cdot 10^5$	-74.6	-67.6	71.7	68.5
2 × 220	$2 \cdot 10^5$	-74.2	-70.8	76.7	71.8
2 × 230	$2 \cdot 10^5$	-75.3	-74.1	80.3	75.0
2 × 240	$2 \cdot 10^5$	-83.2	-77.3	85.5	78.3

Table 4

		k=3			k=3.75			k=4			k=5		
$\sigma_{t,\,\text{perm}}$	C_j	$\sum c_i^k \sigma_i^k \cdot n_i \times 10^{12}$	$N_{\sigma_{t,\,\text{perm}}} \cdot \sigma_{t,\,\text{perm}}^k$	$\bar{n}$ safety	$\sum c_i^k \sigma_i^k \cdot n_i \times 10^{12}$	$N_{\sigma_{t,\,\text{perm}}} \cdot \sigma_{t,\,\text{perm}}^k$	$\bar{n}$ safety	$\sum c_i^k \sigma_i^k \cdot n_i \times 10^{12}$	$N_{\sigma_{t,\,\text{perm}}} \cdot \sigma_{t,\,\text{perm}}^k$	$\bar{n}$ safety	$\sum c_i^k \sigma_i^k \cdot n_i \times 10^{12}$	$N_{\sigma_{t,\,\text{perm}}} \cdot \sigma_{t,\,\text{perm}}^k$	$\bar{n}$ safety
80	1.0	1.2648	1.02	0.810	25.038	27.39	1.094	67.216	81.9	1.219	4353.12	65.5	1.505
90			1.46	1.153		42.60	1.702		131.2	1.952		118.1	2.713
100			2.00	1.581		63.24	2.526		200.0	2.975		200.0	4.594
110			2.66	2.105		90.42	3.611		292.8	4.356		322.1	7.399
120			3.45	2.732		125.30	5.004		414.7	6.170		497.6	11.431
80	1.2	2.186	1.02	0.468	49.600	27.39	0.552	136.851	81.9	0.599	10832.95	65.5	0.605
90			1.46	0.667		42.60	0.859		131.2	0.959		118.1	1.090
100			2.00	0.915		63.24	1.275		200.0	1.461		200.0	1.846
110			2.66	1.218		90.42	1.823		292.8	2.140		322.1	2.973
120			3.45	1.581		125.30	2.526		414.7	3.030		497.6	4.594
80	1.25	2.470	1.02	0.415	57.838	27.39	0.474	155.269	81.9	0.528	13286.1	65.5	0.403
90			1.46	0.590		42.60	0.737		132.2	0.854		118.1	0.889
100			2.00	0.810		63.24	1.094		200.0	1.288		200.0	1.505
110			2.66	1.078		90.42	1.563		292.8	1.886		322.1	2.424
120			3.45	1.399		125.30	2.166		414.7	2.671		497.6	3.746
80	1.5	4.269	1.02	0.240	114.524	27.39	0.239	340.289	81.9	0.241	33059.5	65.5	0.198
90			1.46	0.242		42.60	0.372		131.2	0.386		118.1	0.357
100			2.00	0.462		63.24	0.552		200.0	0.588		200.0	0.605
110			2.66	0.624		90.42	0.790		209.8	0.861		322.1	0.974
120			3.45	0.810		125.30	1.094		414.7	1.219		497.6	1.505

Address:

Dr. Antal SZITTNER
Dr. László KRISTÓF
Dr. László KALTENBACH
Department of Steel Structures
Technical University
H–1521 Budapest, Hungary