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THEORETICAL AND EXPERIMENTAL EXAMINATION PERFORMED DURING THE LATEST REPAIRING OF THE CHAIN BRIDGE

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Abstract

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All the bridges in Budapest over the Danube were blown up in 1945. With the recreation of the Erzsébet bridge in 1964, the rebuilding of the bridges built before 1945 was accomplished with reuse in part of the original elements of the old bridges. The reconstruction was started with the half-provisional Kossuth bridge which was later dismounted. The process went on with the reconstruction of the Szabadság bridge (the former Ferenc József bridge) in 1946. The other bridges followed each other in succession: rebuilding of the Margit bridge, the built of the half-wide Árpád bridge, the rebuilt of the Petőfi bridge and in honour of the 100 year anniversary of its original construction the Széchenyi Chain Bridge was reopened in 1949. During the time after the post-war reconstruction of the Danube bridges, the intensity of traffic was multiplied due to the magnitude of the applied load and the number of vehicles. The influence of the developed loading was augmented by the salting of roadways, started in 1964. Due to this fact and to the catastrophe of the Reichsbrücke in Vienna in 1976, the controlling of the bridges over the Danube in Budapest was initiated, which otherwise was also necessitated by previous service-life of 20 – 25 years (at the original elements much more!).

In the course of the reconstruction of bridges, the Department of Steel Structures at TU Budapest was in a permanent consultative communication with UVATERV, which directed the reconstruction work.

The contribution of our Department to the reconstruction of Margit bridge, Szabadság bridge and Chain bridge was especially important.

Now, I should like to give a brief account about our work in the course of repair of the Chain Bridge.

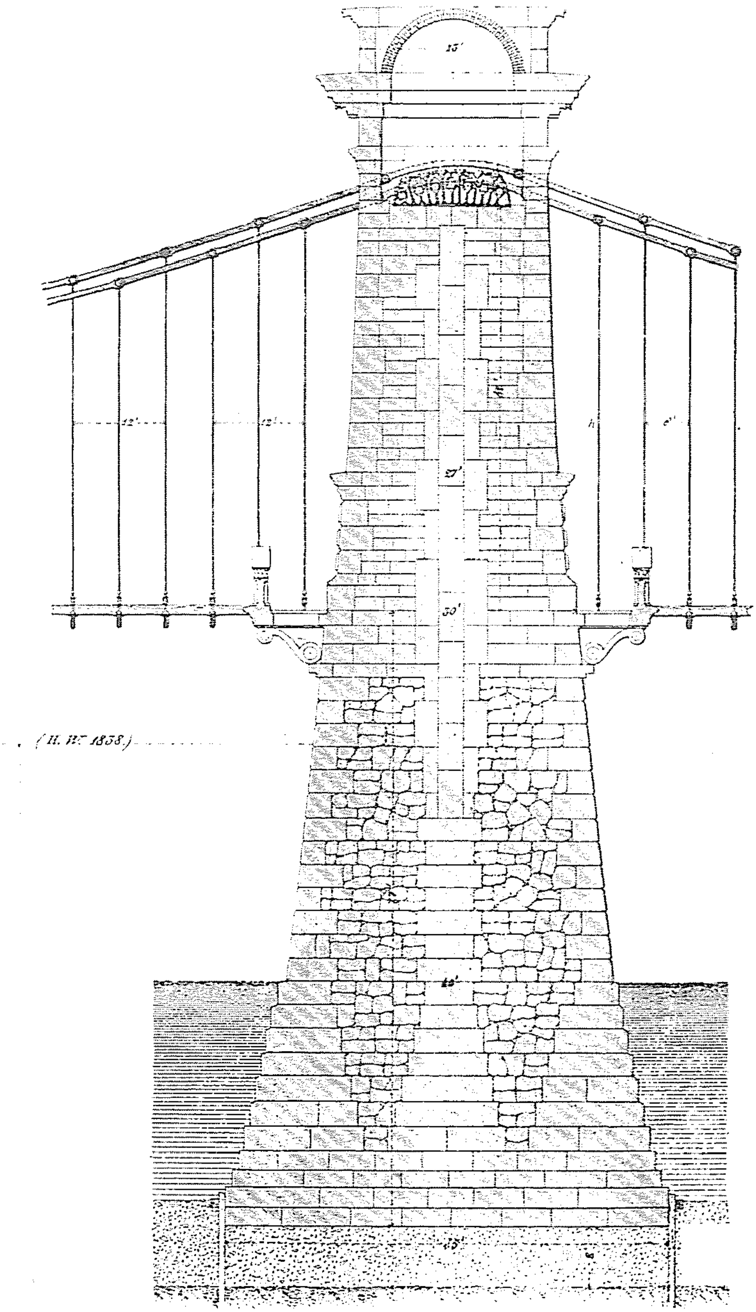


Fig. 1. Schematic drawing of the pier surroundings of the Chain Bridge constructed in 1849

As it is well known, the original projects of the Chain Bridge were completed by the English engineer W. Thirney Clark, while the construction was supervised by Adam Clark. The original bridge was opened to traffic in November 1849 and this date is associated with the jubilee session, too. The bridge structure (*Figs. 1* and *2*) constructed to handle the traffic of the XIX. century, contained cast-iron cross girders, a road-way of wooden structure, a dense setup of hangers and no stiffening girders [1], could not meet the increased traffic any longer, therefore the old bridge structure was replaced with a new one in compliance with the contemporary circumstances in the years 1913 – 1915 on basis of the plans of A. Kherndl and J. Beke as well as I. Gállik. As far as I know, only one of the eye-bars remained from the original bridge structure, which is shown now on the ground floor of the central building of TU Budapest. Some other individual structural elements are housed in the Museum of Transport at Budapest and in the Castle of Krasznahorka.

The supporting structure of the bridge represents a suspension bridge with double-row chain on which the stiffening girders are suspended at a distance of 3.55 m in the side span, and a distance of 3.63 m in the middle span. The stiffening girders in all the three spans are simply supported beams. The suspension forces were adjusted so that the stiffening girder should be nearly moment-free with respect to the dead-load [2].

During the II. World War, this bridge too, was blown up. The anchorage chamber on the Pest side was destroyed but on the Buda side it remained relatively undamaged. The eye-bars still applicable during the reconstruction were rectified and installed again. The stiffening girder and the roadway were fabricated anew.

In the course of the repairing started in 1987 it was shown that the corrosion in the anchorage chamber was very advanced due to the defective isolation (*Fig. 3*), but a direct measure of the corrosion could not be answerable in the narrow gaps of 29 mm between the eye-bars.

Thickness Measurements of the Eye-bars

The first task in connection with the reconstruction of the Chain Bridge was to determine the minimum cross sectional area of the bunches of chains consisting of 12 or 13 eye-bars, damaged by the corrosion. The corrosion was the stronger in the anchorage member, therefore the examination was restricted only on the first eye-bars.

The measurements in the gaps of 29 mm between the eye-bars could be initiated, when the lateral surface of the eye-bars was sandblasted with a special sandblast-head developed especially for this purpose. For the

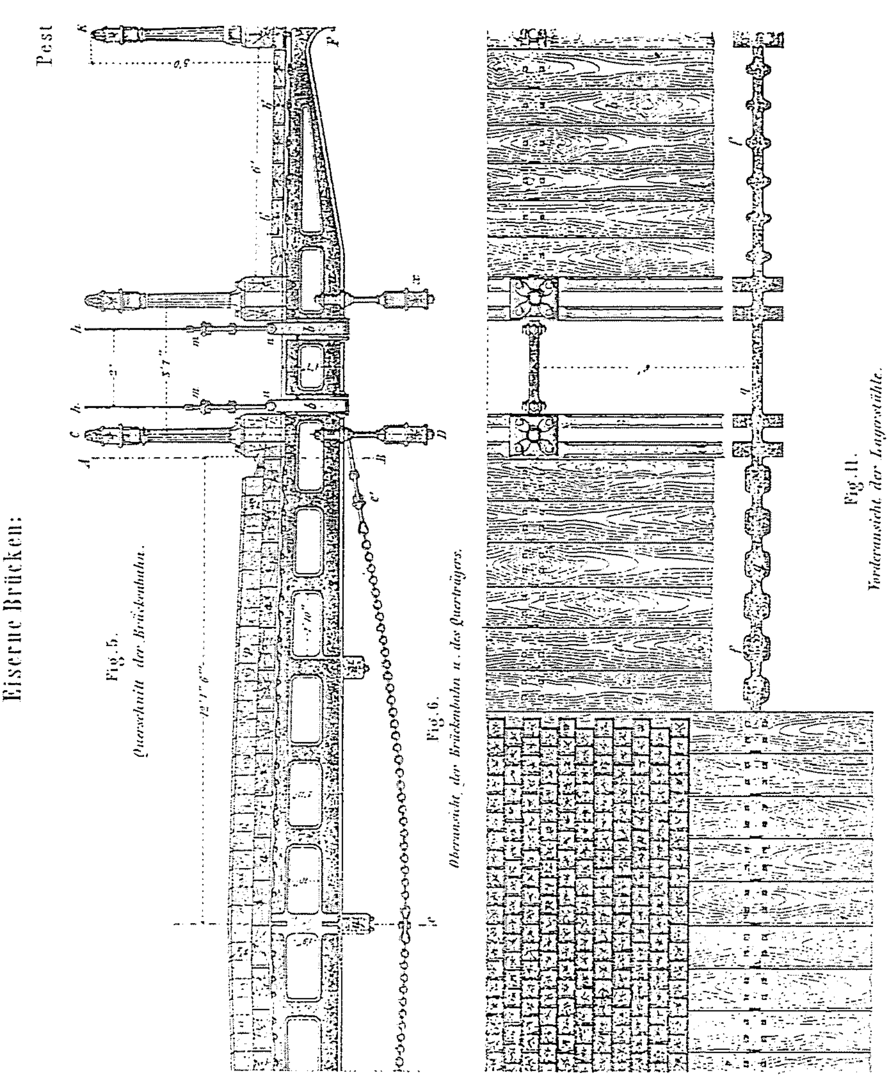


Fig. 2. Drawing of the floor system of the Chain Bridge constructed in 1849

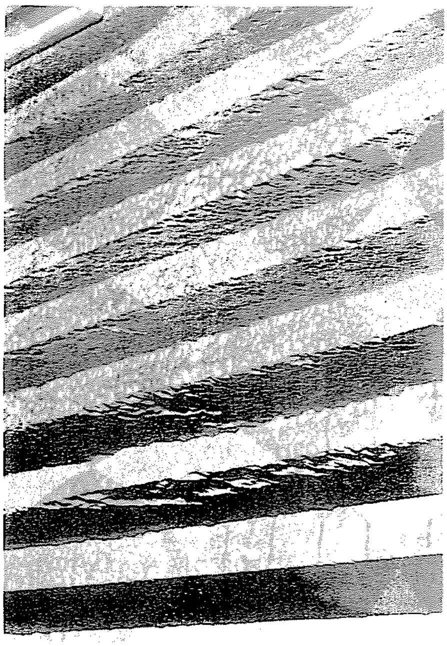


Fig. 3

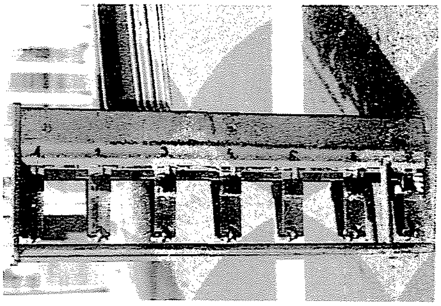


Fig. 4

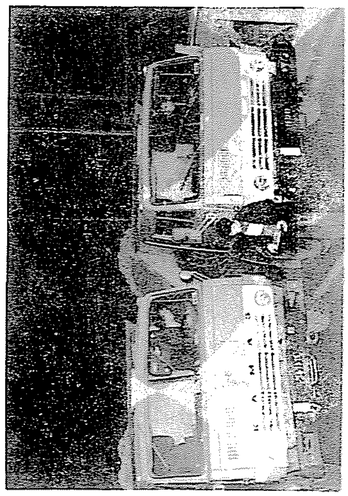


Fig. 5

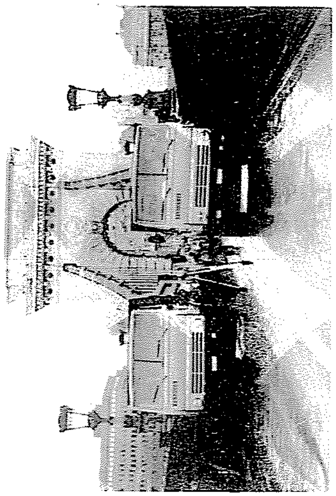


Fig. 6

measuring of the thickness, a special monitoring system was developed, by help of which the determination of the residual thickness remaining after the corrosion damage of the eye-bars in 7 places simultaneously along their height of 38 cm was possible (*Fig. 4*). The essential part of the instrument is a closed frame fixed to a rod at its lower end, and connected removable and easily re-adjustable at its upper end. The measuring springs are coupled to the frame-columns tilted towards each other so that the free distance between the feeler-rollers at the ends of the measuring springs should be about 15 mm. On the bottom of the measuring cantilever springs, the resistance strain-gages are bonded on both sides at the clamping. When the measuring instruments together with the measuring springs were adapted on the eye-bar to be measured, the measuring springs became deformed as cantilever beams according to the thickness of the eye-bar. The deformation is directly proportional to the bending moment in the clamping points of the cantilever, or to the strain of the exterior fibre due to the moment.

The 2×2 resistance strain-gages on the two measuring springs at the same height were connected in a full bridge-circuit to the measuring amplifier. Through this solution, it was achieved that the influence of measurement asymmetry was filtered out of the results.

The seven measuring-circuits for thickness measurements were connected to a computer-controlled measuring system, which monitored, collected, processed and recorded the measuring results in succession.

Before the processing of the measuring results, and even at the greater time-distance between the measuring monitoring, the measuring- circuits were calibrated. The calibration took place in the following system: the measuring instrument was placed on reference pieces of 20, 25 and 30 mm in succession, then the measurement results were collected by the computer and each of the detecting elements was individually calibrated so that one regression-line for each measuring-circuit was calculated. This step was repeated three times or more. If the deviation between the relevant results was under a certain limit, the measurement results and the coefficients were pointed out, and the measuring of the thickness could be initiated with these parameters.

The thickness measurement was placed on the individual eye-bars in succession, and the thickness was measured at a distance of 5 – 25 cm depending on the condition of the eye-bars. After each data entry, the seven results (thickness) were printed out in mm- dimensions, then the remained cross-section area of the examined eye- bar was determined and at last this remained area was expressed in the percentage of the nominal cross-sectional area, too. By this method, it was possible to determine the minimum cross-sectional area in every eye-bar and the minimum active area in every bunch of bars, respectively.

On the basis of about 40000 measurement data, it was stated that the weakest cross-sectional area of the anchorage elements among the 2×2 bunches of eye-bars on the Buda and Pest sides could be found:

in the northern bottom bunch of chains on the Buda side, and in the southern upper bunch of chains on the Pest side, where the cross-sectional area attacked by corrosion was 91, or 95% of the nominal cross-sectional area.

In the weakest eye-bar, the damaged cross-sectional area was 80% of its nominal area, which fact indicates the importance of the bridge supervision and maintenance, because the bridge cannot bear any more a corrosion damage like this one. Making known the results of thickness measurements, we suggested that the effective cross-sectional area of the bunch of chains should be reckoned with by the 0.90-fold value of the nominal cross-sectional area in the course of bridge controlling calculation. The instrument was devised in the workshop of our Department with the direction of L. Kaltenbach, while the computer-based system was developed by Dr. M. Kálló.

Load Test

In course of the reconstruction of the Chain Bridge, there were two load tests conducted. In the first load test, the suspension forces, and the load-distribution effect in the stiffening girder were determined. With the second load-test, the deformation of the bridge, the load-bearing capacity of the supporting chains and individual eye-bars, and the stress-condition of the stiffening girder, respectively, were examined.

The first load test was performed at night. The bridge was loaded by lorries weighing 20 tons each and placed at a distance of 7.5 m from each other in the following arrangement:

1×2 lorries beside each other in one row,

4×2 lorries beside and behind each other in 2 queues,

8×1 lorries behind each other in one queue (*Fig. 5*).

The second load was prescribed for the Chain Bridge by the Bridge Department of the Ministry of Transport (service load). This load consisted of a distributed load involving 18 kN/m (buses), and 0.5 kN/m (passenger cars) taken alternatively for each 24 m long section in the full width of the bridge (2 lanes), which load corresponds to a distributed load of 13.0 kN/m considering the dynamic factor with respect to the main girder and the stiffening girder. This load was applied with lorries weighing 200 kN each put behind each other in two queues and close to the curb so that the measurement of deformation (levelling) could be performed in the longitudinal axis of the bridge (*Fig. 6*). This load was substantially smaller than that applied with the first load test.

In the course of the first load test, by the effect of the load applied in both the middle- and the side-span, respectively, there were experienced smaller forces and a less unequal load distribution in the hangers than it was expected without any previous calculations. Therefore the measurements were repeated but practically no deviation was detected.

Controlling the measurement results, a method of approximate calculation was elaborated by Assistant Professor Dr. F. Papp briefing the essential concept of this method in a separate paper [4]. By using the method of computer simulation, the bridge was substituted by a planar framework of bars stiffened by beams, where — of course — a theory of second order was applied for calculations. With the second load test, the calculated network was modified, first of all, by reckoning with the brake structure in the middle of the bridge.

The results of the computer simulation were plotted in a formatised layout. As an example, the calculation results obtained for load position No. 13 are demonstrated here. In the *Fig. 7* the original network, the deformed (distorted) network, the load applied, the numerical value of the suspension forces and their distribution are represented graphically, as well as the reactions of the stiffening girder, and the results of the equilibrium can be seen. Showing the difference between calculated and measured results, the calculated and measured values of suspension forces arisen at load position No. 13 (8 lorries in a queue behind each other in the side-span along the southern lane), then at load position No. 10 (8 lorries behind each other in a queue in the middle span) were compared with each other (*Figs. 8* and *9*).

As the comparison shows, when the side-span is loaded, the suspension forces are arisen only within the side-span, while the suspension forces in the middle span are negligible.

When the middle span is loaded (load position No. 10), not only the hangers of the loaded middle span but also those of the side-span take part simultaneously in load bearing, though smaller forces arise in the side-span than in the loaded middle one.

The theoretical examinations were extended also for those of the suspension forces due to the dead-load. To analyze this problem, after the application of rated dead-load of 189 kN on the hanger No. 42 by computer simulation, a reduction of 7.8 mm in the length of hangers was entered with the help of a force of 30 kN. According to calculations, this operation induces considerable suspension forces only in the manipulated hanger (100%) and in the two adjacent hangers suspended on the same chain (ca 50 – 50%). These two latter forces, of course, are of reversed sign

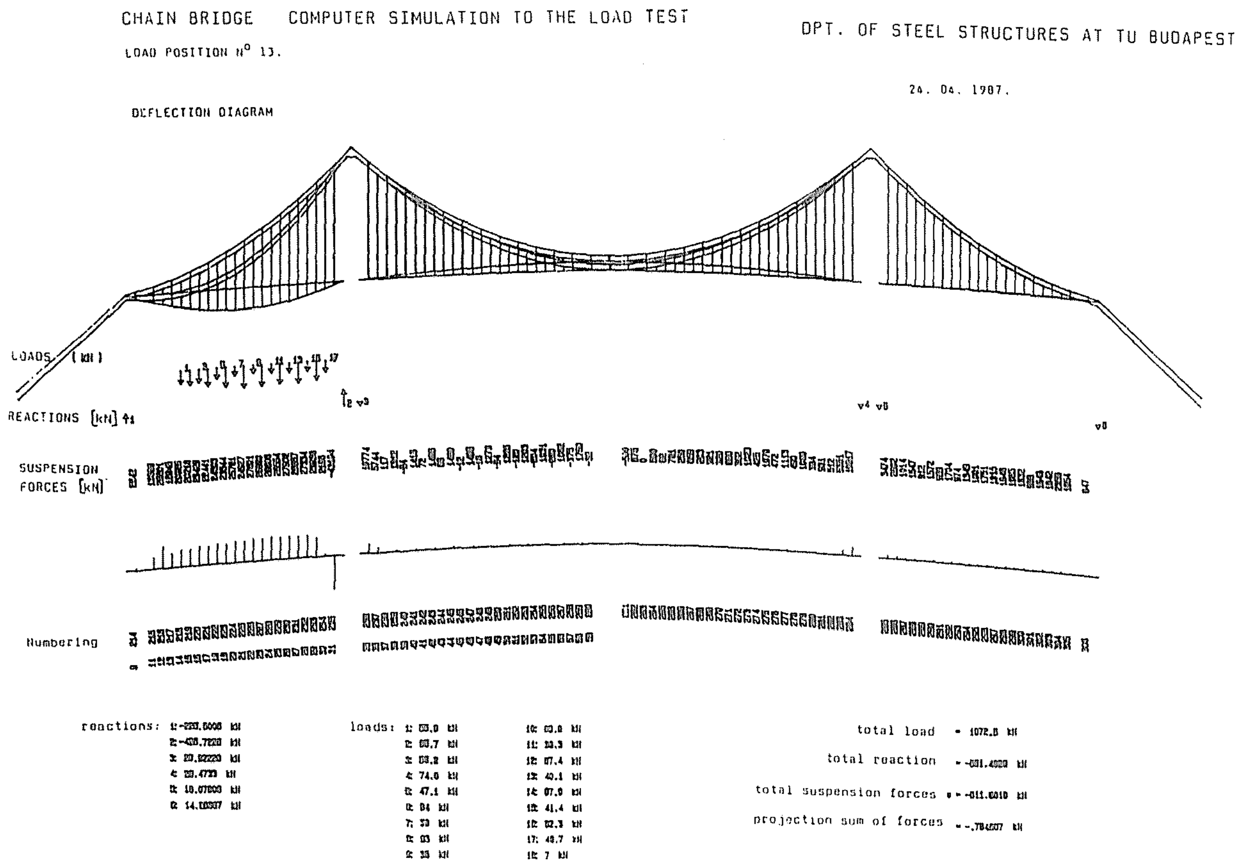


Fig. 7. Results obtained by the method of computer simulation at load position No. 13

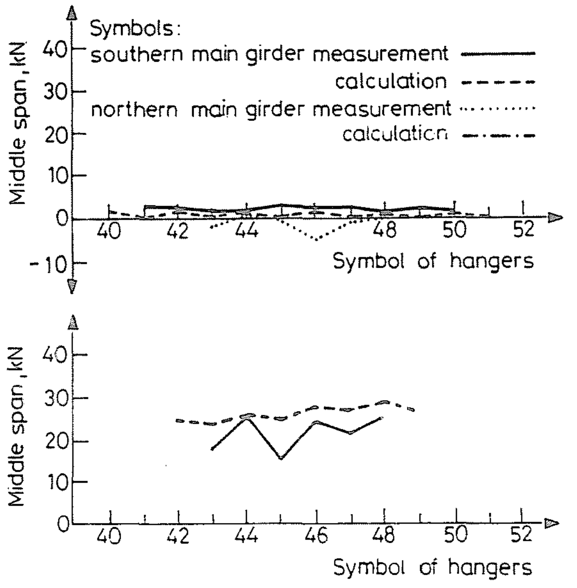


Fig. 8. Measured and calculated suspension forces in the hangers at load position No. 13 (side-span loaded)

as compared to the induced force mentioned above. As a consequence of the results obtained, the suspension forces can be controlled or regulated easily by hydraulic operating jacks. This relatively not too high sensitivity made possible the simple change or adjustment of the suspending spindle of the short hangers in the middle and at the end of the bridge. In the course of the second load test, the vertical displacements of the bridge were measured at the middle of the side-span and in the sixths of the middle span. The displacements were measured by levelling in the longitudinal axis of the bridge and by photogrammetric method on the northern main girder. As measuring points for photogrammetry, the chandeliers of the decorative lighting were switched on.

In *Table 1* presented here, the deflection values calculated and measured, respectively, at the different load positions by photogrammetric method and with the help of levelling were compared with each other. Deformation values obtained by levelling and photogrammetry, respectively, deviate from each other to a small extent, which can be explained by the not exact marking of the photogrammetric points.

The difference between the values of the calculated and measured vertical displacements will amount to 25 – 30% within the loaded spans, while it will amount to 40 – 60% within the unloaded ones. This can

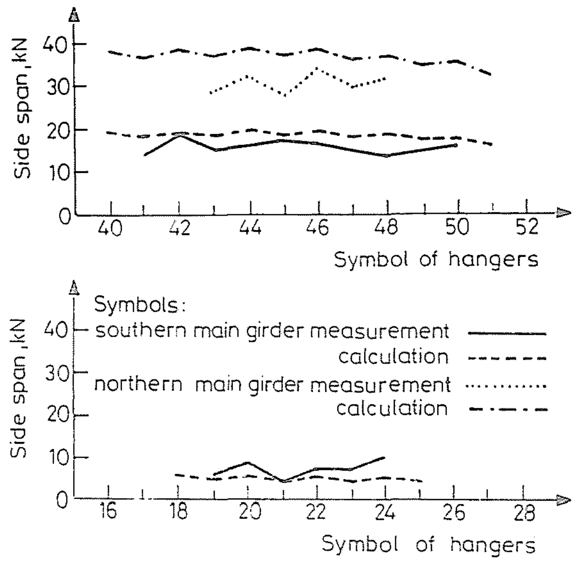


Fig. 9. Measured and calculated suspension forces in the hangers at load position No. 10 (middle span loaded)

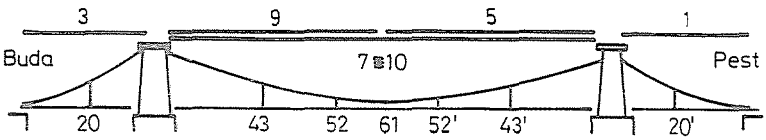
probably by attributed to the fact that the contribution effect between the floor system and the stiffening girders cannot be estimated with a required accuracy.

The measurement in the southern main girder was performed on two bunches of chains in the middle of the bridge, while on the upper chord of the stiffening girder in the middle of the Pest-side span and the middle span. The measured results were checked, this time too, by the method of computer simulation, but the network was modified. In addition to the calculation results associated with the first load test, the variation of bending moment arisen in the stiffening girder, too, was elaborated.

The calculation results were checked for the cases of 3 load positions:
side-span loaded,
middle span loaded in half length,
middle span loaded in full length
with respect to the suspension forces and the bending moments arisen in the stiffening girder.

From the results obtained it could be seen that the bending moments do not pass from the side span into the middle one, however, when the middle span is loaded, a considerable amount of moment will be arisen in

Table 1
Deformation measured by levelling (N), photogrammetric method (F) and calculation (Sz)



Load position	20 N	20 F	20 Sz	43 N	43 F	43 Sz	52 N	52 F	52 Sz	61 N	61 F	61 Sz
1 — 0	0.7	0		-8.3	-6		-15.8	-13		-19.3	-17	
			-5			-22			-35			-38
1 — 2	-0.6	0		-11.2	-8		-20.1	-14		-25.8	-21	
2 — 0	1.3	0		2.9	2		4.3	1		6.5	4	
3 — 2	68.3	62		-8.5	-5		-14.8	-11		-17.6	-16	
			108			-23			-35			-37
3 — 4	66.6	60		-8.6	-5		-15.7	-11		-18.9	-17	
4 — 2	1.7	2		0.1	0		0.9	0		1.3	1	
5 — 4	-24.8	-28		-32.1	-35		-7.2	-10		65.8	69	
			-48			-27			9			96
5 — 6	-24.5	-30		-33.0	-33		-9.2	-9		62.5	73	
6 — 4	-0.3	2		0.9	2		2.0	1		3.3	4	
7 — 6	-52.2	-44		70.9	74		118.1	123		132.2	150	
			-99			99			170			198
7 — 8	-49.9	-40		67.0	71		109.6	115		123.3	141	
8 — 6	-2.3	4		3.9	3		8.5	8		8.9	9	
9 — 8	-28.1	-26		102.8	108		126.0	130		65.5	74	
			-54			129			165			109
9 — 11	-29.1	-28		100.8	104		123.8	125		64.5	70	
10 — 8	-51.5	-58		71.2	73		117.5	120		132.2	148	
			-99			99			170			198
10 — 11	-52.5	-54		69.2	70		115.3	116		130.2	143	
11 — 8	1.0	4		2.0	3		2.2	4		2.0	5	
11 — 0	1.4	0		9.8	10		17.9	20		22.0	24	

the side-span, too, and furthermore, the stress in the stiffening girder will grow higher in the side-span than in the middle one.

In *Table 2,* the calculated and measured forces, the calculated and measured moments in the stiffening girder, as well as the measured suspension forces are shown. As far as the chain forces are concerned, the agreement is very good with both the bunches of chains and the couple of

Table 1
(continued)

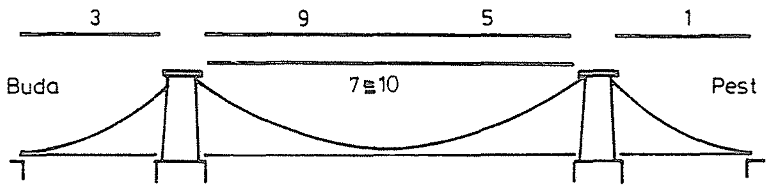
Load position	52' N	52' F	52' Sz	43' N	43' F	43' Sz	20' N	20' F	20' Sz
1 — 0	-17.8	-14		-11.6	-8		14.8	70	
			-35			-23			108
1 — 2	-24.5	-19		-15.9	-11		72.9	67	
2 — 0	6.7	5		4.3	3		1.9	3	
3 — 2	-16.8	-14		-10.3	-7		-1.3	0	
			-33			-21			-6
3 — 4	-18.1	-16		-11.0	-7		-2.0	0	
4 — 2	1.3	2		0.7	0		0.7	0	
5 — 4	120.7	124		97.5	100		-31.8	-18	
			157			127			-49
5 — 6	117.5	129		95.2	104		-30.9	-16	
6 — 4	3.2	5		2.3	4		-0.9	2	
7 — 6	110.7	124		62.9	73		-61.2	-58	
			170			100			-98
7 — 8	103.5	118		59.5	69		-59.8	-55	
8 — 6	7.2	6		3.4	4		-1.4	3	
9 — 8	-10.0	-6		-36.4	-32		-29.0	-20	
			19			-24			-53
9 — 11	-10.7	-6		-35.2	-32		-26.9	-15	
10 — 8	111.6	121		63.9	74		-61.0	-42	
			170			100			-98
10 — 11	110.9	117		65.1	71		-58.9	-37	
11 — 8	0.7	4		-1.2	3		-2.1	5	
11 — 0	19.1	18		9.5	9		-1.8	0	

+ sinking, - uprise

bunches. According to calculations, a chain stress of 106 MPa is arisen due to the dead-load, while a chain stress of 16 MPa is arisen due to the load applied by lorries. The deviation of the stress from this one as measured in the eye-bars is of not considerable value. In the case of smaller loads, the deviation between the chain forces showed a greater percentage probably due to the friction of joints, however, these measured stresses are not considerable as expressed in terms of absolute value.

In the case of the stiffening girder, the deviation between the beam moments measured and calculated is larger than that measured in the chains. The calculated moment is greater by 30 – 40% than the measured one. This can be explained partly by the neglection of the contribution effect of the floor slab, and partly by the fact that measurements could be performed only in the upper exterior fibre of the beam. The stresses mea-

Table 2
Stresses measured (M) and calculatied (Sz)



Load position	3 M	3 Sz	5 M	5 Sz	7 M	7 Sz
Bottom chain at point No. 51	-440	198	894	1065	2036	2009
Upper chain at point No. 52	-388	119	873	901	2101	1987
Bottom + upper	-828	237	1766	1966	4137	3996
21	-26.5	1.8	3.5	12.8	15.1	28.4
22	-11.9	1.8	13.1	16.7	18.4	29.8
23	-10.9	1.8	12.7	13.1	23.6	29.1
45	-21.0	1.6	35.8	29.7	41.1	45.8
46	-19.9	1.6	28.9	33.6	43.6	46.7
47	-14.3	1.6	37.7	29.8	44.8	45.6
Chord force	-38		1027		1813	
bending moment	108	-460	-2937	-3738	-5185	-7399
Chord force	-337		-872		-602	
bending moment	913	-241	2363	3344	1631	3283

sured in the hangers were in accordance with the results obtained during the first load test.

On the basis of the evaluation of those said above, i. e.

Table 2
(continued)

Load position	9		10		12 = 1	
	M	Sz	M	Sz	M	Sz
Bottom chain at point No. 51	928	983	2134	2009	65	124
Upper chain at point No. 52	1055	1137	1970	1987	122	122
Bottom + upper	1983	2120	4105	3996	187	246
21	24.3	16.9	29.6	28.4	16.8	15.3
22	19.3	14.3	27.9	29.8	23.7	15.4
23	62.2	17.3	33.7	29.1	29.0	15.3
45	14.7	16.9	43.4	45.8	-3.3	1.7
46	16.7	13.7	51.5	46.7	-3.3	1.7
47	17.4	16.5	49.0	45.6	-4.8	1.6
Chord force	654		1750		-2242	
bending moment	-1869	-3969	-4994	-7399	6411	798
Chord force	185		-577		20	
bending moment	-500	1812	1563	3283	-55	-319

the reduction of about 10% in the cross-sectional area determined from the measurement results,

from the stresses arisen in the chain, the hangers and the stiffening girder,

we agreed with the proposal of the UVATERV in our report, according to which: further traffic can be allowed for loads prescribed by the Ministry of Transport (buses+passenger cars) in case

the unrusting and re-painting of the bridge steel structure,

the repair of the anchorage chamber's insulation,

the replacement of the deck-slab, and
the repair of the masonry
have been performed.

However, we should like to draw the attention to the importance of the supervision and maintenance of the bridge, and especially to check the chains with special care, because in case this task would be neglected, a newer attack of corrosion damage on the chains could result in the reduction of their load-bearing capacity to such an extent which would endanger the serviceability of the bridge.

Note

This paper contains the material of the lecture delivered in course of the scientific session held by the Hungarian Academy of Sciences on the occasion of the 140 year anniversary of opening the originally constructed Chain Bridge to traffic.

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