EXPERIMENTÁLNA ANALÝZA ÚNAVOVEJ PEVNOSTI A ŽIVOTNOSTI ŠPECIÁLNYCH MOSTNÝCH SÚSTAV

EXPERIMENTAL ANALYSIS OF FATIGUE STRENGTH AND LIFE TIME OF SPECIAL BRIDGE ASSEMBLIES

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Abstrakt

Príspevok uvádza prezentáciu a analýzu výsledkov experimentálnych vyšetrovaní únavovej pevnosti a životnosti špeciálnych mostných sústav. V laboratórnych podmienkach boli skúšané dva typy týchto zostáv, ktoré možno hodnotiť z hľadiska ich únosnosti a odolnosti proti únavovým účinkom. Výsledky výskumu sú konfrontované s normatívnymi výpočtovými postupmi a odporúčaniami.

Kľúčové slová: mostná sústava, únosnosť, únavová pevnosť, životnosť.

Abstract

The paper gives an insight into the original static and fatigue tests of special temporary steel bridge assemblies. Two types of these systems were tested and can be evaluated in view of their carrying capacity and resistance to fatigue effects. The results are discussed in relation to latest standards procedures and recommendations.

Keywords: bridge assembly, load carrying capacity fatigue strength, life time.

INTRODUCTION

Plate girders, used in many cases for bridges and industrial structures, are permanently subjected to repeated or even dynamic loading. Generally, it is accepted that the postcritical resistance capacity could not be fully utilised because of unfavourable dynamic and fatigue effects. However, when dealing with dynamic response of any structure, the effects of changes both in stiffness and the damping should be taken into account. The consequence is the mixture of favourable and unfavourable effects. In the case of slender cross-sections the flanges and webs of plate girders are buckling in dependence on geometrical and material properties and on the level and mode of loading. Thus, the stress-strain relations are non-linear and the response is influenced by the interaction of the webs, flanges and stiffeners in individual fields of plate girders, with concentration of strains and stresses in the most vulnerable points. [2,3,4,5,9].

The paper presents the results of experimental research that was devoted to the investigation of special bridge assemblies of actual size. These bridge assemblies could be used for temporary bridges during the construction period or for solutions of transport problems when the consequences of natural disasters or man faults do not allow using of regular bridges.

TESTS OF ASSEMBLED ELEMENTS OF TEMPORARY STEEL BRIDGES

Two assembly bridge systems were tested in the laboratory conditions. In the first system one from the tested elements was previously used as a part of the temporary bridge during the

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reconstruction of the tram bridge for the period of two years. Remaining tested elements did not fully answer to conditions of safety due to initial production defects. The second system was later developed with the intention to replace the first system. There was no room to test full bridge assemblies, therefore only individual assembly elements were tested in special configuration. The aim of tests was to simulate the design loading and stresses in investigated connecting bolts and areas near connections.

TEMPORARY BRIDGE SYSTEM 1

The dimensions and configuration of tested parts and supporting bridge parts are in Fig. 1. Bridge elements were produced from two different steels: flanges were from high strength steel 15 422.5 with design strength f_{yd} = 400 MPa, thin web was from steel 11 523.1 with design strength f_{yd} = 290 MPa.

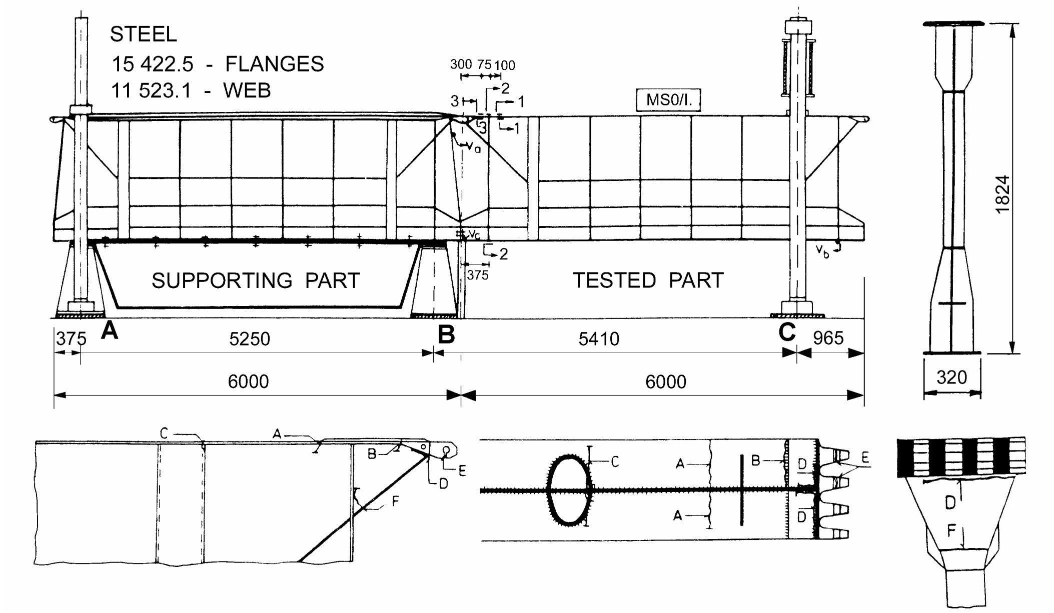

Fig.1 Static and geometric scheme of tested system 1. Position of fatigue cracks after test

In the first stage the static successive and repeating loading was applied like concentrated load in section C by one or two jacks until $F = F_u$, where F_u is a theoretical ultimate static load, its effect corresponds to the effect of design load of full scale bridge structure. In total it was 25 loading steps as follows: 0-100-200-300-400-500-600-650-700-750-500-250-0-750-0-750-0-750-0-250-500-750-500-250-0 kN. Deflections and strains were measured in chosen points of flanges, webs and stiffeners. Measured deflections included: the B support sag v_B, total vertical deflection v_C in section C, horizontal displacement u_B at the eye bolt connection in upper part of section B. During static tests the good behaviour of tested system was proved. Stress-strain diagram was linear, but stiffening sheet appeared to be rather weak. The effect of residual stresses in flanges was remarkable.

Fatigue tests run with constant harmonic loading, f = 5Hz, F_{min}= 0.33F_u, F_{max}= 0.67 F_u. During these fatigue tests the dependence of strains on the level of loading was observed together with their changes as consequence of the initiation and development of fatigue cracks. 6 types of cracks appeared and successively increased (Fig. 1 - cracks A, B, C, D, E, F):

A - originated along fillet welding of strap plate to tension flange,
B - uprose along butt welding of the lock to the flange of component,
C - originated in tension flange at the connection of closed transverse stiffener,
D - created in edge skew stiffener along the connection of the lock with stiffener,
E - originated in the place of teeth for shank connection,
F - uprose in edge skew stiffener in the place of its width change.

It should be mentioned that at the beginning of fatigue tests the levels of decisive strains were in limits 0.6 - 1.3 milistrains and at the initiation of cracks it was ap. 1.3 - 2.0 milistrains. The decisive cracks and total numbers of cycles N from tests of system 1 are in Table 1. Five assembly elements were tested in this case, each of them twice in cantilever composition with exchange of the ends. It means position (a) or (b) in Table 1. The element, previously used in the tram bridge had already visible fatigue cracks in the place of connections. The other elements were taken from stocks.

Results of fatigue tests - temporary bridge system 1 **Table 1**

Indication		MS0	MS1	MS2	MS3	MS4
(a)	cracks	C	A,E	D	F	B
(a)	N (mil.)	1.15	1.16	1.17	0.585	1.01
(b)	cracks	A	D	F	F	F
(b)	N (mil.)	0.68	1.82	0.75	0.26	0.565

Either it was expected that some cracks could appear in compression parts of a web, due to stiffened fields it was not so. More sensitive places were near the lock as the actual types of existing cracks suggested. These details appeared to be more sensitive to dynamic actions, then, for new systems another detailing was recommended, see also [6].

TEMPORARY BRIDGE SYSTEM 2

Temporary bridge system 2 has shown some improvements in its behaviour under dynamic loading. It was based on the knowledge from the investigation of temporary bridge system 1. The general view of the system 2 is in Fig. 2.

As can be seen from Fig. 2, it was possible to apply four available assembly parts (I, II, III, IV) in four testing setups: MS1 (I-left, II-right), MS2 (II-left, I-right), MS3 (III-left, IV-right), MS4 (IV-left, III-right). From static analysis the ultimate carrying capacity was determined as $F_u = 922.554$ kN. This ultimate loading was controlled by postcritical load carrying capacity of stiffened fields in this composition referred to combined bending and shear effects. Geometrical and material characteristics used in calculations were those supplied by the producer. Comparison of theoretical and experimental deflections during static preloading is in Table 2.

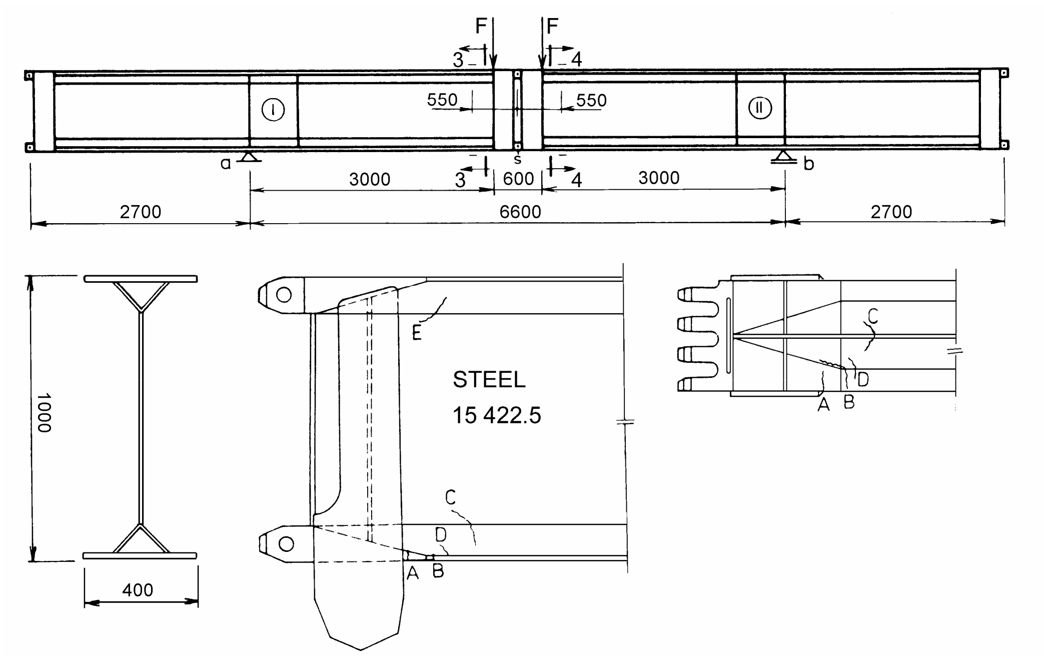

Fig.2 Static and geometric scheme of tested system 2. Position of fatigue cracks after test

Results of fatigue tests – deflections *v* - temporary bridge system 2 **Table 2**

<table>
<tr><td rowspan="5">Setup</td><td colspan="4">1st loading</td><td colspan="4">2nd loading</td><td rowspan="4">$v_{s,e}$</td><td rowspan="4">$v_{s,t}$</td><td rowspan="5">$\frac{v_{s,e}}{v_{s,t}}$</td></tr>
<tr><td colspan="8">F (kN)</td></tr>
<tr><td colspan="2">900</td><td colspan="2">0</td><td colspan="2">900</td><td colspan="2">0</td></tr>
<tr><td colspan="8">v_s</td></tr>
<tr><td colspan="8">(mm)</td><td colspan="2">(mm)</td></tr>
<tr><td>MS1</td><td>17.171</td><td rowspan="4">16.946</td><td>1.325</td><td rowspan="4">0.724</td><td>17.164</td><td rowspan="4">17.019</td><td>1.371</td><td rowspan="4">0.748</td><td rowspan="5">16.241</td><td rowspan="5">15.980</td><td rowspan="5">1.017</td></tr>
<tr><td>MS2</td><td>16.269</td><td>0.611</td><td>16.353</td><td>0.591</td></tr>
<tr><td>MS3</td><td>16.639</td><td>0.674</td><td>16.860</td><td>0.701</td></tr>
<tr><td>MS4</td><td>17.706</td><td>0.284</td><td>17.698</td><td>0.330</td></tr>
<tr><td>MS1 up to MS4</td><td colspan="4">$v_{s,e}$ = 16.222 mm</td><td colspan="4">$v_{s,e}$ = 16.271 mm</td></tr>
</table>

Fatigue tests of system 2 run with constant harmonic loading, f = 5Hz, F_{min}= 250 kN, F_{max}= 500 kN. This corresponds to stress range $\Delta\sigma$ about 100 MPa.

The strains and deflections were stabilised after static preloading and they remained at these values until the initiation of cracks. The initiation of cracks was firstly registered through the changes of strains near the critical sections. This allowed identify the crack initiation before it started to be visible. On the other hand, the changes in deflections had appeared after development of cracks in the tension flanges and web parts, pretty well before the total failure.

Fatigue tests – number of cycles and type of cracks - temporary bridge system 2 Table 3a

	Type of crack	Number of cycles N (millions)							
Testing setup		MS1				MS2			
Component		I		II		II		I	
Position		front	back	front	back	front	back	front	back
crack initiation crack visible failure	A 1				- 3.380 -				
crack initiation crack visible failure	C 1		- 2.400 -	2.200 2.500 3.380	2.200 2.380 3.380		- 3.050 -	- 3.390 -	- 1.990 -
crack initiation crack visible failure	C 2				- 2.380 -		- 3.050 -		
crack initiation crack visible failure	C 3				- 2.400 -				
crack initiation crack visible failure	C 4				- 2.400 -				
crack initiation crack visible failure	C 5				- 2.800 -				
crack initiation crack visible failure	D 1						2.300 2.745 3.690		

Fatigue tests – number of cycles and type of cracks - temporary bridge system 2 Table 3b

	Type of crack	Number of cycles N (millions)							
Testing setup		MS3				MS4			
Component		III		IV		IV		III	
Position		front	back	front	back	front	back	front	back
crack initiation crack visible failure	C 1	1.050 2.223	1.050 1.550 2.223			- 0.950 -		0.400 0.750 1.850	0.400 0.625 1.850
crack initiation crack visible failure	C 2						- 1.330 -		
crack initiation crack visible failure	D 1				- 1.250 -				
crack initiation crack visible failure	E 1	1.050 1.250 -	1.050 1.250 -						

In the temporary bridge system 2 the cracks of type A, C, D and E were registered. Following the data of Table 3a, 3b it is obvious that crack of type A at the fillet welding of strap plate to tension flange had appeared only once, while in case of previous bridge system 1 this crack was present in all cases of tests. The crack of type B along butt welding of the lock to the flange of component did not appear during the tests of bridge system 2.

The crack of type C had originated in welded connection of a web to the roof like angle part of tension flange, see Fig. 2. These cracks had appeared repeatedly along the fillet weld, even in places that were not too stressed. Later it had penetrated into the web and tension flange. It had decided about the failure of setups MS1-II, MS3-III and MS4-III.

The crack of type D had started in welded connection of a web to the roof like angle part of tension flange, but it propagated in majority into tension flange and only slightly into the web. It was deciding for the failure of setup MS2-II, and had appeared also in the case of MS3-IV, but there had not the decisive effect.

The crack of type E started in welding connection of the web to the roof like part of compression flange. It had appeared only in the case of MS3-III. Either it was not decisive, at the end of test it reached the length of 160 mm.

The comparison of Table 1 to Tabs 3a, 3b suggests that the fatigue resistance has increased in the case of bridge system 2. In general, the bridge system 2 has shown acceptable behaviour with option to be used for practical purposes in actual transport conditions.

LIMITATION OF WEB SLENDERNESS, FATIGUE ASPECTS

In both cases of tests exists the possibility of using latest standards approaches for the comparison with obtained results. The Slovak standard STN 73 1401: 1998 [10] gives for constant normal stress range the fatigue strength curves as given in Fig. 3.

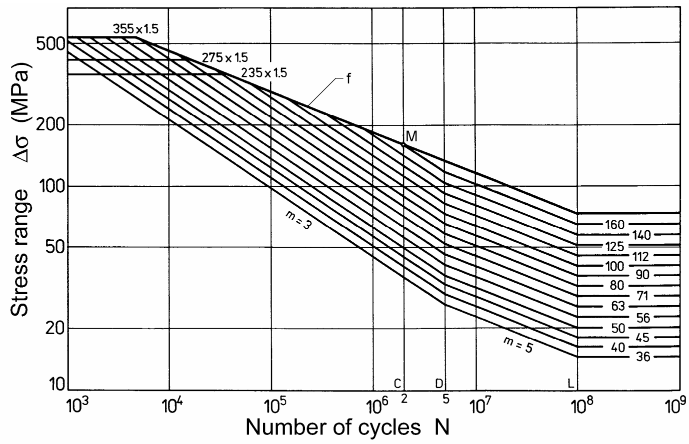

Fig.3 Fatigue strength curves for different detail categories at constant normal stress range

Taking into account that during the fatigue tests respective stress ranges in exposed critical sections were about 100 MPa the assessment of detail category as 90 to 100 gives the standard fatigue life time about 1.5 to 2 millions of cycles. In comparison with test results in Tables 3a, 3b it has reached acceptable agreement.

Especial feature of STN 73 1401:1998 is that fatigue curves are limited with upper boundary with inclined line m = 5 originated from crossing with fatigue strength curve for structural detail with reference value of stress range $\Delta\sigma_C$ = 160 MPa at $N_C = 2 \times 10^6$ cycles, point M on Fig. 3. It considers the recommendations of IIW Doc. XIII-1539-96/XV-845-96 and test results including those of authors. They have shown that the inclination of fatigue curves is lower $m > 3$ in case of structural details with higher fatigue strength and lower notch effect [4] and [7]. Accounting for quasi-static effects the normal stress range should be $\Delta\sigma \leq 1{,}5\, f_y$ [10].

Latest Eurocode 3 Part EN 1993-1-9: 2005 contains similar procedures and requirements as STN 73 1401: 1998. Importance is given to the proper determination of fatigue stress range. Fatigue curves include both inclinations $m = 3$ and 5 [11]. However, validity of fatigue curves is not limited with upper boundary by other conditions expect that one - $\Delta\sigma \leq 1{,}5\, f_y$. General safety limits are left for national decision.

The fatigue strength and the life-time of plate girders with thin web depend besides of usual loading, material and construction influences also on the web vibration.

It was proved that fatigue crack can appear also in primary compressed regions. In case of thin walled structures the significant reason of that are local stability effects. Therefore, the slenderness β should be appropriately limited. On the basis of extensive research and results of other authors the limit slenderness of webs subjected to bending and compression is recommended be calculated as follows [3]

$$\beta_{uf} = \frac{280\sqrt{k_{MN}}}{\sqrt[7]{N}}\sqrt{\frac{235}{f_{yf}}} \tag{1}$$

f_{yf} is yield strength of flanges, k_{MN} is stability coefficient of the web, N is required number of cycles. Considering for bending k_{NM}=k_M =39.52 (the web clamped into flanges), then

$$\beta_{uf} = \frac{1760}{\sqrt[7]{N}}\sqrt{\frac{235}{f_{yf}}} \tag{2}$$

$$N_{uf} = \left(\frac{1760}{\beta}\sqrt{\frac{235}{f_{yf}}}\right)^7 \tag{3}$$

However, in the case of tested bridge assemblies, only bridge assembly 1 represented the structures where the slenderness effects in the web should be taken into account [6].

DISCUSSION OF RESULTS

The conception of considering the service variable amplitude loading is to some degree included in technical standards. One approach considers the definition of the effective constant amplitude stress that allows use equivalent constant amplitude relationships for the variable amplitude loading conditions. Equivalent stress amplitude is defined by the fatigue effect after 2×10^6 cycles, such effect being the same as the effect of variable amplitude stress. This procedure is accepted as a standard procedure, provided that there are sufficient and reliable data to define the equivalent stress amplitude. The other advantage of the standard STN 73 1401: 1998 is the introduction of separate fatigue curves for constant and variable amplitudes of stresses [7,10]. In case of railway bridges monitoring dynamic measurements [1,8] could be conveniently combined with the available data of Railway Companies that give the information about time schedules, masses, and lengths of trains for longer time periods up to several months or years.

CONCLUSIONS

The exact judgement and verification of fatigue strength and the life time for bridge structures and especially that of temporary assemblies is problematic also nowadays. In cases of original pretentious steel bridges the experimental verification is inevitable. The appropriate simulation of actual conditions and characteristics is, however, very important for reliable results. In general cases the fatigue life-time can be determined using the value of minimum life-time in the case of maximum loading together with the application of aggression effect of the whole loading spectrum. The fatigue life time is then appropriately increased. Modified proposed conception takes into account the influence of general loading process. This is more convenient than the application of usual linear cumulation hypotheses. The positive feature is that new standard´s approaches include the use of measured data that will help to define the expected total or remaining life time. Limit slenderness of webs should be verified to fulfil the demands of fatigue aspects. Recommended empirical dependences for limit slenderness apparently proved to be sufficiently safe. On the other hand, the probability of local failure near the different connections and/or locks suggests the individual judgement of the structure design and its detailing.

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