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CONSTRUCŢII. ARHITECTURĂ

CRITERIA FOR THE ASSESSMENT OF EXISTING HIGHWAY BRIDGES

BY

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In the western part of Romania, on the highway still are in operation, after two World Wars and other events, some old highway bridges erected on the beginning of the last century. Some of these structures are *witnesses of the past*, representing technical monuments with emblematic character. The duty of the administation is to maintain these structures in service. The paper presents an overview on the evaluation methodology of the remaining service life on the bases of the new principles. The procedure to establish safe inspection intervals is also pointed out.

1. Introduction

Rehabilitation and maintenance of existing steel bridges is one of the most important present problems. During service, bridges are subject to wear. In the last decades the initial volume of traffic has considerably increased. Therefore many bridges require a detailed investigation and control. The examination should consider the age of the bridge and all repairs, the extent and location of any defects etc.

Romania has a highway network of 153,057 km on which there are placed 3,192 bridges (Romanian Highway Administration Report 2003). From the total number only 83 ($\approx 3\%$) are steel bridges.

There is a reduced number of old steel highway bridges, generally older than 80...90 years, mainly in the western part of Romania, which are witnesses of the past. Even if these structures are in operation the technical condition of these bridges is not satisfactory. Some aspects can be noticed:

a) *Insufficient clearance*: (the width, B = 5.0...6.0 m, (Fig. 1) and therefore vertical members of the main truss girders damaged by the impact with vehicles (Fig. 2).

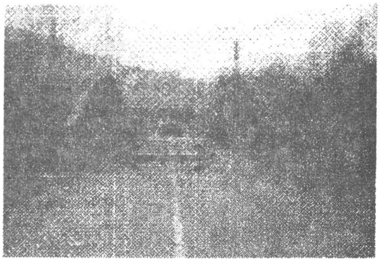


Fig. 1.– Frontal view of an old highway bridge – insufficient clearance.

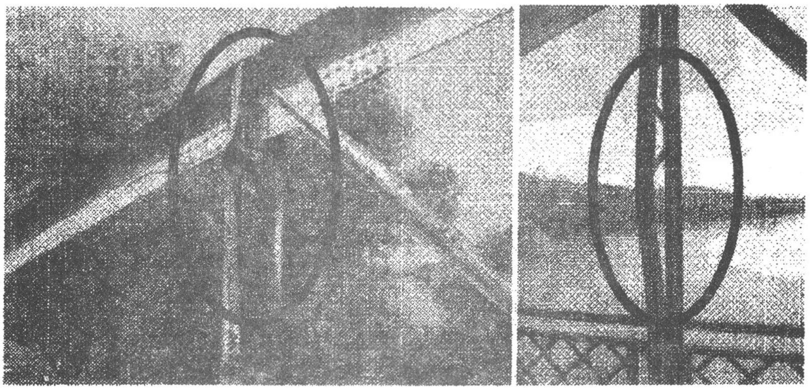


Fig. 2.– Degradations of the structural elements due to the impact with vehicles.

b) The *maintenance* of these structures is not continue (Fig. 3), the owners have changed during history (different administrations), the *corrosion* is important (Fig. 4).

c) Generally the complete lack of *documentation*.

d) Insufficient knowledge of *material qualities*.

e) All these structures are *riveted*.

f) Tendency of the administration to *replace* these structures.

In comparison with the existing railway bridges, the situation is different. First of all in case of the railways the number of steel bridges is dominating, the maintenance is better, documentation is generally available. Consequently the rehabilitation of old existing highway bridges is not current and some recommendations, according to the experience obtained by the rehabilitation of some representative structures, can be done. It should be mentioned that all these structures have an historical and artistical value (Fig. 5).

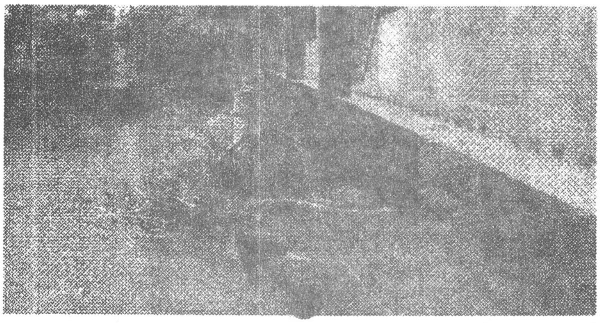


Fig. 3.– The problem of maintenance work.

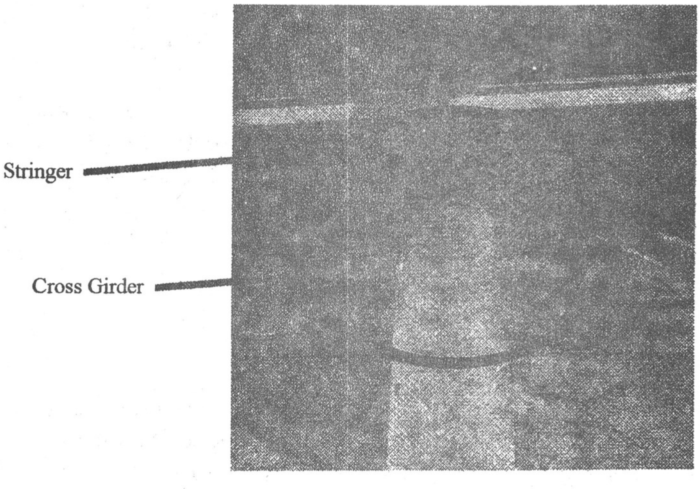


Fig. 4.– The problem of corrosion.

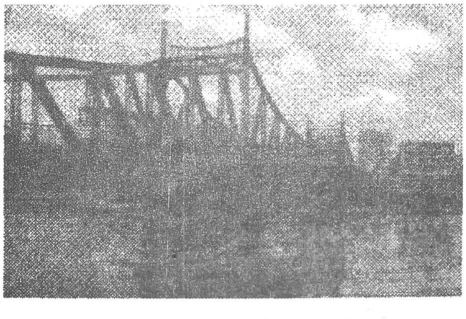


Fig. 5.– Traian Bridge in Arad – witness of the past.

A continuous maintenance, which generally must increase in time, is important in order to assure the safety in operation of the existing structures (Fig. 6).

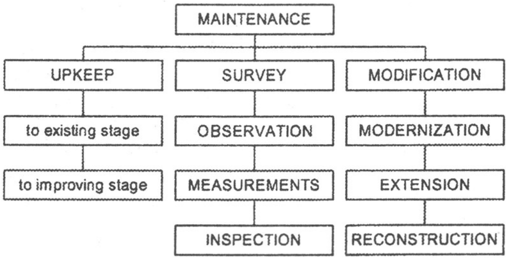


Fig. 6.– Maintenance of the existing bridges.

2. Present Verification Methodology and the Proposal for a Complementary Method

The rehabilitation of bridges represents a complex matter. The Romanian Highway Administration adopted a qualitative verification methodology based on the appreciation by the expert of the technical condition of the structures (AND 522-2002) [1]. Some quality indexes are defined; finally the technical condition of the structure is given by

$$I_{ST} = \sum_{i=1}^{5} C_i + \sum_{i=1}^{5} F_i. \tag{1}$$

a) The quality index, C, is a sum of five aspects: C_1 is referring to the main girder; C_2 is referring to the deck elements; C_3 is referring to the infrastructure and bearings; C_4 is referring to the bed river; C_5 is referring to the quality of the deck surface.

b) For the functional requirements also five aspects are relevant: F_1 expresses the traffic condition on the bridge; F_2 expresses the loading class of the highway; F_3 takes into consideration the year of construction and structure type; F_4 is referring to the quality of fabrication, erection and operation conditions; F_5 is referring to the maintenance of the structure.

For every index marks (from 1 to 10) are given. Finally the technical condition of the structure results from the total sum and can be ranged in one of the following categories:

a) Very good technical condition.
b) Good technical condition.
c) Satisfactory technical condition.
d) Unsatisfactory technical condition.
e) The present technical condition can not assure the safety of the structure.

Based on these conclusions the administration can take a decision regarding the time planning of maintenance works.

This method is suitable for concrete bridges and relatively new (20...30 years old) steel bridges. For older steel bridges it can have only an informative character. A more refined methodology must be adopted. In this direction the experience obtained by the verification of steel railway bridges can be used (Fig. 7).

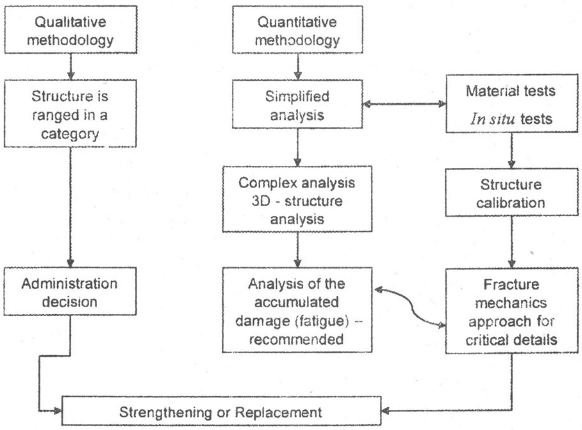


Fig. 7.– Proposal for present methodology improvement in the case of the verification of old existing highway bridges.

Following the examination of the existing documentation a simple analysis of the structure is recommended. This can lead to some immediate restrictions in circulation. *In situ* tests are possible and not as expensive as railway bridges tests. The results can be used for the calibration of the structure. Material tests from secondary elements – or if it is possible from main elements – are useful.

The analysis of technical conditions of these bridges can also contain non destructive tests. They are possible only after the removing of the deck and cleaning of the structure. In this phase a detailed and carefully inspection of the structure by the expert is compulsory [2].

Taking into account the year of construction, the following assumptions can be made [3]:

a) approx. 1900...1920, mild steel with a low carbon content;

b) after 1920 mild steel with the qualities of St 37.

Still existing wrought iron bridges were generally replaced immediately after the Second World War.

Charpy tests made with samples from existing bridges are relevant; due to the large dispersions of values; for useful conclusions a large number of Charpy tests are necessary. Generally, steel fabricated in this time (providing from the same steel plants, like Reşiţa or Györ) has the same qualities as the steel used in railway bridges [4]:

a) % C is situated in the range 0.09...0.16%;

b) the yielding stress is 230 N/mm^2;

c) the 27J transition temperature is around 0°...+5°C based on Charpy V Notch tests.

A more difficult problem is the fatigue assessment of the structure. Even if in the usual standard for highway bridges [5] this verification is not foreseen, the damage accumulation methodology applied for railway bridges can be adopted. In this direction, for the Wöhler curve the assumptions made by the Swiss Railways for existing bridges [2] (Fig. 8) can be adopted.

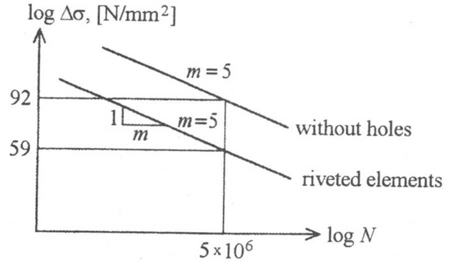


Fig. 8.– Wöhler curves adopted by the Swiss Regulation.

Difficult is evaluation of the stress history; also the assumption of the same spectrum for bridge life (or for certain periods) must be made. The applied stress range, the geometry of the detail and the number of stress cycles has a decisive effect on the remaining fatigue life of the structures. Information's about the circulation in different periods are necessary. With the stress history and the appropriate Wöhler curve, the accumulated damage can be evaluated using the relation

$$D = \sum_i \frac{n_i}{N_i} \leq 1. \tag{2}$$

For bridges with usual spans situated on the National Highway Network, taking into account the circulation in the past, fatigue problems were not decisive. It is worth to mention that the Swiss standard SIA 161 offers a very informative diagram in this sense.

Taking into account the importance of safety assessment of steel existing bridges the norm has lately evolved a lot; the first prescriptions based on simple criteria of damage accumulation determined by traffic loads on the bridge, later there appeared complementary criteria which take into account the appearance and propagation of cracks caused by material fatigue.

The classical fatigue concept is based on the assumption that a constructive element has no defects or cracks. However, discontinuities and cracks in the components of structures are unavoidable, basically because of the material fabrication and the erection of structures.

The presence of cracks in structural elements modifies essentially their fracture behaviour. Fracture, assimilated in this case as crack dimensions, growth process under external loadings, will be strongly influenced by the material toughness. In those cases in which defects are present in the structural elements, the small stress ranges also contribute to the propagation of crack from the initial crack to the critical one (failure probability). This phenomenon of crack propagation under different stress ranges levels can be studied with the help of the fracture mechanics method.

The authors proposed a complementary method based on the fracture mechanics basic concept

$$J_I \leq J_{Ic}. \tag{3}$$

In practice two situations can be distinguished:

a) $D < 0.8$ – the probability to detect cracks is very low. The inspection intervals (generally between 3...6 years) can be established on criteria independent of fatigue. Nevertheless, a special attention must be paid to critical details.

b) $D \geq 0.8$ – cracks are probable and possible. An *in situ* inspection and the analysis of critical details are strongly necessary. Also a fracture mechanics approach is recommended.

Generally, for the fracture mechanics complementary method the following steps must be taken (these are additional to the steps included by the classical method):

a) the material toughness (J_{Ic}) and crack growth tests which supplemented the conventional tests (chemical and metallographic analysis, tensile, Charpy V notch, bending and Brinell tests);

b) identification of critical details;

c) size and location of defects – NDT;

d) idealization of defects;

e) determination of the critical crack size based on material toughness and failure assessment diagram;

f) life prediction analysis.

The complementary methodology [6] is conceived as an advanced, complete analysis of structural elements containing fatigue defects, being founded on fracture mechanics principles and consisting in two steps, *i.e.* determination of defects' acceptability with the help of Failure Assessment Diagrams and determination of final acceptable values of defect dimensions. This is followed by a second step which in fact represents a fatigue evaluation of the analysed structural elements based on the present stress history recorded on the structure. It is also related on the initial and final defect dimensions and the FM parameters, namely the material characteristics, C and m, from the Paris relation (crack growth under real traffic stress) and further on the exact determination of the number of cycles, N, required in order that a fracture take place, respectively the determination of the remaining service life of the structural elements (years, months, days).

The assessment of flaw acceptability based on failure assessment diagrams (FAD) and material toughness is created taking into account the recommendations given by the BS 7910:1999 [7] and based on a two criteria method (Fig. 13).

At the old riveted steel bridges the usual incubated fatigue cracks are situated at the rivet hole or at the plate edge. A good initial fatigue crack length which can be detected at corrosion conditions is of 5.0 mm.

The crack cases assumed for these structures (riveted bridges) are illustrated in Fig. 9.

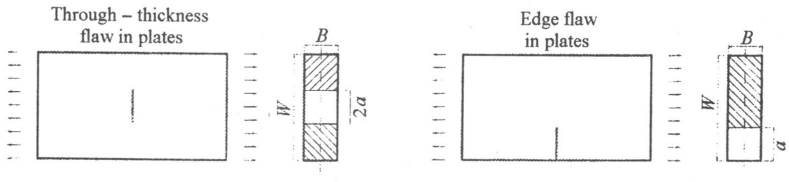


Fig. 9.– Theoretical crack models.

In the case of welded structures three cases of flaws can be considered:
a) through thickness flaws – idealization in a rectangular flaw (Fig. 10);
b) surface flaws – idealization in a semi-elliptic flaw (Fig. 11);
c) embedded flaws – idealization in a elliptic flaw (Fig. 12).

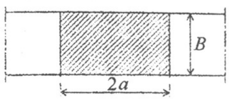


Fig. 10.– Through thickness flaw.

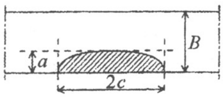


Fig. 11.– Surface flaw.

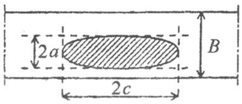


Fig. 12.– Embedded flaw.

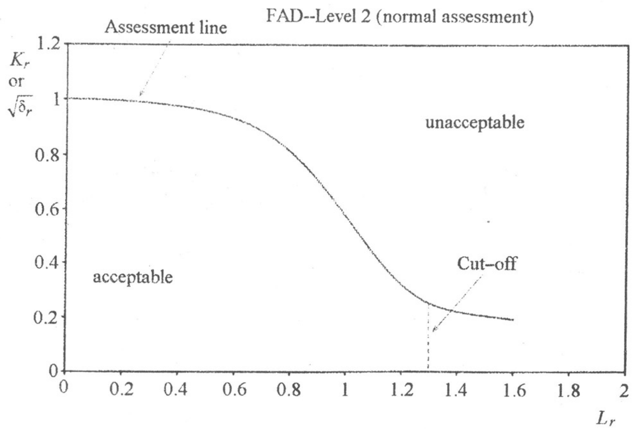


Fig. 13.– Failure Assessment Diagram, level 2.

Knowing these data the second step can be started. It is practically a fatigue assessment for structural elements with defects, which are based on the possibility of modelling, on the propagation rate of crack dimensions under fatigue loads and with the help of known laws. The method is founded on the recommendations of the BS 7910:1999 applied for old riveted steel bridges and it uses the Paris relation. It can also be applied for other bridge structures, which contain defects namely

$$(4) \qquad \frac{da}{dN} = C\Delta K^m, \quad (5) \quad \Delta K = K_{\max} - K_{\min} = Y(\sigma_{\max} - \sigma_{\min})\sqrt{\pi a}\,.$$

The simulation procedure of the crack growth from the initial crack length, a_0, to the final crack length, a_f (critical crack), for the determination of the total number of cycles until the failure occurs and implicitly the remaining service life.

On the basis of results obtained with the help of this two-step method, conclusions can be drawn regarding the maintenance programme to be applied to the structure, the reinforcement measures, the measures of traffic limitation, respectively speed, or the need of closing the circulation or replacement of the structure. At the same time it is to be underlined the fact that the knowledge of the crack propagation type and speed of a defect, which is represented by the remaining fatigue life (years, months, days), as well as of the danger it represents leads to the possibility to extend the of service life of bridge structures under safe conditions.

The main steps for the assessment of the remaining fatigue life are presented in Fig. 14 [6]

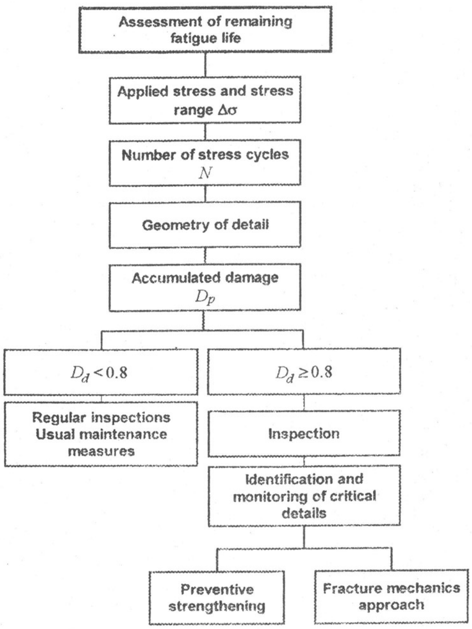


Fig. 14.– Assessment of the remaining fatigue life.

3. Non-Destructive Inspection Methods for Bridges

Careful inspection of the structure is the most important aspect in evaluating the safety of the bridge. On the accuracy of the *in situ* inspection depends the level of evaluation. The estimation of the current technical condition of an existing steel bridge structure with the help of *in situ* evaluation depends, in a high proportion, on the engineer's qualification. In the case of *in situ* inspection it is recommended to insist on the appeared fatigue defects, deformations, welded connections (the expert must to insist on the connections between the stringers with cross girders and cross girders with main girders), critical details (which are included in standards or catalogues), corrosion level, the structure behaviour due to traffic, bridge bearings. The expert can use non-destructive techniques to determine the integrity of basic material or structural components. The main non-destructive methods for bridge inspection are the following:

a) *visual inspection* – the most common method which includes microscopes, mirrors, portable video cameras, robotic crawlers; this method is very useful in case of surface cracks;

b) *magnetic particle inspection* – this method is also very simple and does not need high qualification personnel, but can be applied just in case of ferromagnetic materials (not for austenitic steels); the method consists in the magnetization of the high stress elements or critical details and indicates directly the surface discontinuity through forming a distorted magnetic field, which can be detected under proper lighting conditions;

c) *liquid penetration inspection* – is a simple method including the qualification of the personnel; it uses penetration of liquids with fluorescent pigment and UV-light in order to indicate the surface defections;

d) *radiographic inspection* – the method is applied for hidden defects and it uses Gamma or Röntgen radiation; the inspected element is placed between the radiation and the film; the interpretation of the radiographic images should be done by experts, otherwise defects could be ignored;

e) *ultrasonic inspection* – this testing is used for flaws and cracks in the material thickness, on the surface or hidden defects; highly qualified personnel is needed; high frequency sound waves are introduced into a material and they are reflected back from surfaces or flaws; this process is recorded by an oscilloscope; this method can not be used for elements made of multiple plates (riveted sections);

f) *Eddy Current testing* – this method can detect surface defects but can also be used for thickness inspection.

4. Case Study – the Bridge in Săvârşin

The bridge in Săvârşin over the Mureş River (Fig. 15) on the local highway DJ 707 A (km 1+271 m) is a remarkable structure with four spans (Fig. 16) erected in 1897. The steel superstructure has a typical composition for the time period in which it was built, that is: steel deck, main parabolic truss girder with descending diagonals and posts, with trough deck slab laid on profiles of the Zorres type.

Fig. 15.– The bridge in Săvârşin (1987).

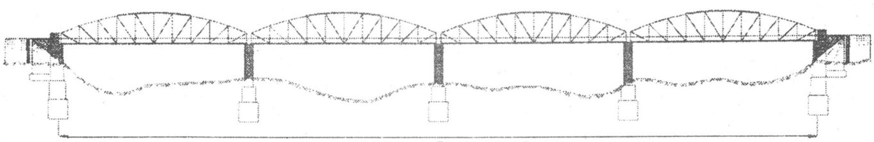


Fig. 16.– General view of the structure.

In Table 1 the cross section of the main girders elements is presented. The technical condition of the bridge is unsatisfactory, the elements are corroded and some verticals and diagonals are damaged by the impact with the vehicles. The existent floor beams, stringers and cross girders are simple supported elements. The deck consists on Zorres elements filled with ballast, supporting an asphalt surface. In present the structure has a special importance being the only crossing of the river in a large area. It can be also mentioned that in Săvârşin is the present summer residence of the former King of Romania, Mihai I.

Taking into account the importance of the structure, its historical value, the decision of strengthening of the structure was taken:

a) for the stringers the flanges were consolidated by supplementary plates (Fig. 17);

b) the cross girders were transformed in switch girders (Fig. 18);

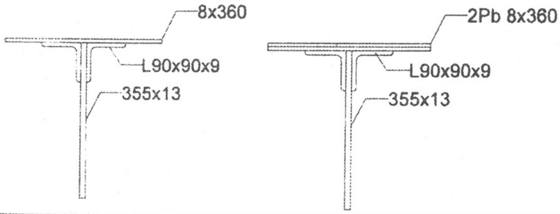
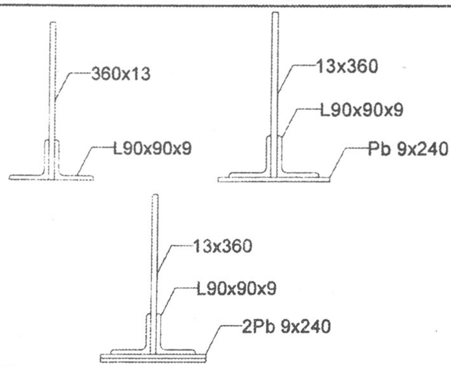
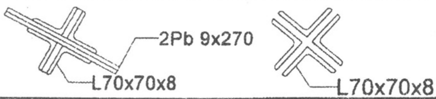
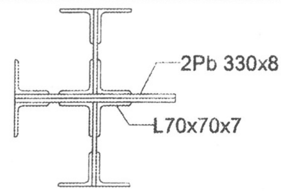
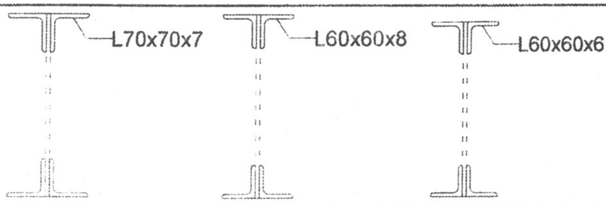
c) for the lower chord of the main girder a supplementary tie member was chosen (Fig. 18);

d) for the upper chord the direct strengthening with two angles, improving also the local stability, was chosen (Fig. 19);

e) diagonals and vertical members have to be first of all straighten, and strengthen by additional plates (Fig. 17);

f) the old floor system was replaced by a composite deck.

Table 1
Cross Section of the Main Girder Elements

Element	Cross - section
Upper chord (final panels)	
Lower chord	
Final diagonals	
Diagonals	
Final posts	
Current posts	

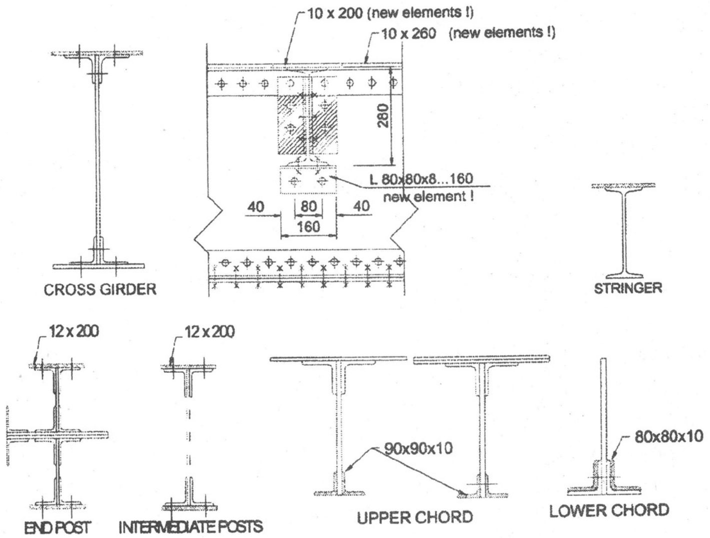


Fig. 17.– Direct reinforcement with additional elements.

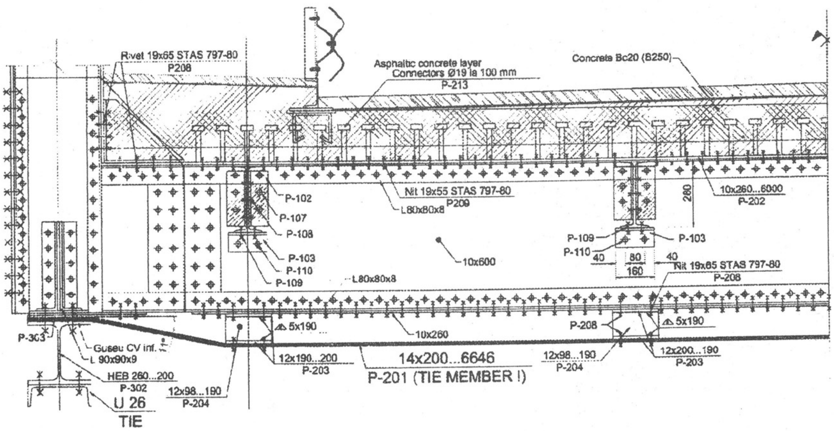


Fig. 18.– Strengthening of the structure (cross section).

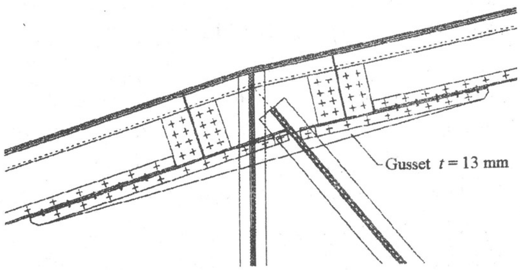


Fig. 19.– Upper chord strengthening.

All these operations are difficult and suppose a high technical level of all *in situ* works.

The decision for these rehabilitation works is based on the conclusions regarding the residual safety of the bridge.

A fatigue verification was firstly made for the assessment of the remaining fatigue life. The verification was performed according to the Swiss SIA 161-1990 [8] using the relation

$$\Delta\sigma_e \leq \frac{\Delta\sigma_c}{\gamma_{\text{fat}}}, \tag{6}$$

where: $\Delta\sigma_e = \alpha\Delta\sigma(Q_{\text{fat}})$; $\gamma_{\text{fat}} = 1.1$; α – traffic coeficient; $\Delta\sigma = \sigma_{\max} - \sigma_{\min}$ – stress range $= 80$ N/mm^2 (admitted stress range in the case of riveted elements).

For example in the case of the main girder – lower chord (middle span) the verification is the following:

$$\Delta\sigma = 735\,\text{daN/cm}^2, \quad \alpha = 0.52 - \text{secondary highway (Fig. 20)}$$

$$\Delta\sigma_e = 0.52 \times 735 = 382\,\text{daN/cm}^2 < \frac{800}{1.1} = 727\ \text{daN/cm}^2.$$

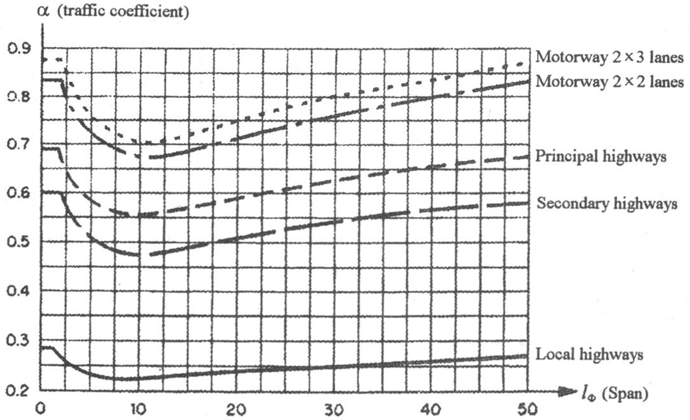


Fig. 20.– Diagram for establishing the factor α.

By taking into consideration all investigated cases it has been concluded that the fatigue in the case of this structure is satisfactory, meaning that it permitted strengthening.

For the assessment of the remaining service life it was practically impossible to perform an analysis based on the classical method of the damage accumulation hypothesis PLM. This can be explained by the fact that it is very difficult to recognize the stress history of the structure. Approximations made in the establishing of the past traffic lead to irrelevant results. Fatigue life calculations based on the classical method sometimes lead to the conclusion that there is no remaining service life, although there are no cracks observed in the structural elements. That was the reason for which the complementary method based on fracture mechanics principles was chosen.

Assuming small detectable fatigue defects (cracks) emanating from the rivet holes and using the two steps fracture mechanics analysis (described in § **2**) one can determine the remaining fatigue life and the inspection intervals for this old riveted bridge. The application of this procedure begins with the information about the structural steel. In this sense the authors lean on the conventional experimental tests performed on samples taken from the structure and also on researches on the material from this period (mild steel) made by the authors themselves [4],[9] as well as taken from the special literature [10], [11].

The laboratory tests have lead to the following results:

a) the material is a mild steel similar to the present St37.2n (STAS 500/2 - 80);

b) yield stress is $\sigma_y = 236$ N/mm^2;

c) tensile stress is $\sigma_{\text{ult}} = 370$ N/mm^2.

For the material toughness in terms of J_{cr} a minimal value of 20 N.mm, at a temperature of −20°C, was chosen.

Based on these values and with the help of the failure assessment diagram the critical crack value for the case main girder – lower chord was determined, $a_{\text{cr}} = 36.1$ mm for $\sigma_{\text{appl}} = 118.3$ N/mm^2.

In order to determine the residual service life for the main girder and also the safe inspection interval one needs information about crack propagation.

The experimental tests on CT specimens have shown that for the oldest mild steels the values of the material constants from the Paris relation are in the following intervals: $m = 2.05...5.65$, $C = 2.2 \times 10^{-11}...10^{-18}$.

Relatively large value of m correspond to very small values of C, for example for $m > 4$ it results $C \cong 10^{-15}...10^{-18}$.

For the following life prediction procedure has been chosen: $m = 3$ and $C = 3 \times 10^{-12}$. The choice has been made according to the experience based on laboratory investigations on material produced in Reşiţa and Györ.

The considered stress history of the Săvârşin Bridge in the case of the main girder is represented in Fig. 21.

Table 2

$\Delta\sigma$ daN/cm^2	Main girder			
	A30	Trucks	Bus	Total
100	0	3	2	5
150	1	6	3	10
200	2	8	5	15
250	3	3	1	7
300	4	3	0	7
350	1	1	0	2
400	3	0	0	3

$\Delta\sigma_e$, [daN/cm^2]=252,14 Total/day 49

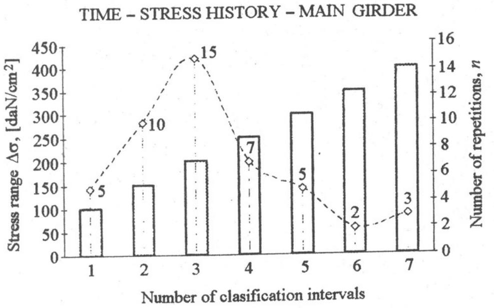


Fig. 21.– Stress history.

In order to determine the remaining service life it is important to know how long it will take the crack to grow from the minimum detectable size to the critical value. Two cases were studied:

a) through thickness flaw with initial size $a_0 = 5$ mm, which is undetectable because it is situated under rivet head (this case has been studied because no cracks with higher value were detected), and

b) through thickness flaw with initial size $a_0 = 15$ mm, which is detectable during a visual inspection appearing sufficiently beyond the rivet head.

In the first case, which is the real one, the remaining fatigue life is estimated at a value of 64.1 years. But by a regular inspection a crack greater than the rivet head is detected, then the manager must know that propagation of the crack till the critical value will occur in 6.9 years. In this situation the safe inspection intervals can be calculated with the relation

$$\Delta T_{\text{insp}} = \frac{N_{\text{RFL}}}{N_{\Delta a}},$$

where: N_{RFL} is the remaining fatigue life calculated for the structural element, $N_{\Delta a}$ – the number of cycles computed for a crack extension during the inspections of 5 mm. In this case the safe inspection intervals are situated between 6 months and 1 year.

The life prediction computing was performed with the help of a soft developed by one of the authors [12] (Fig. 22).

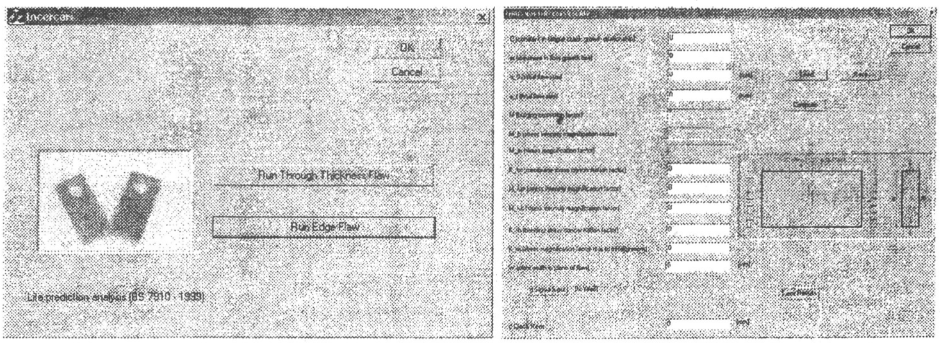


Fig. 22.– Soft for life prediction analysis.

Some phases from the rehabilitation works at the Săvârşin Bridge are shortly presented in what follows.

1. The works began and follows by removing the pavement and then by straightening of the deformed posts. (Figs. 23,...,25).

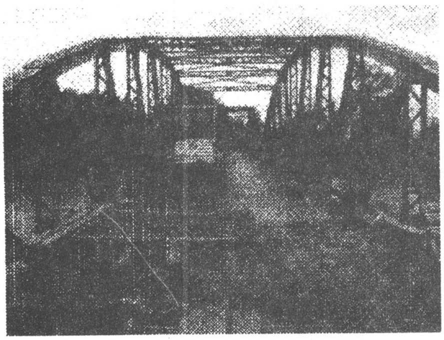


Fig. 23.– Removing the pavement.

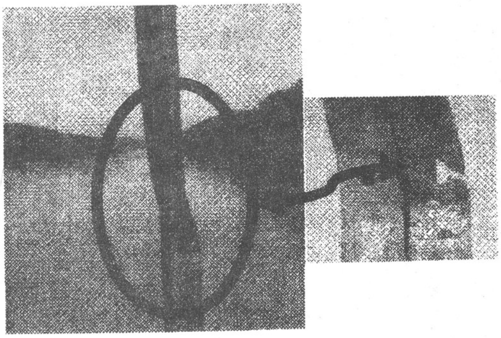


Fig. 24.– Deformation defect.

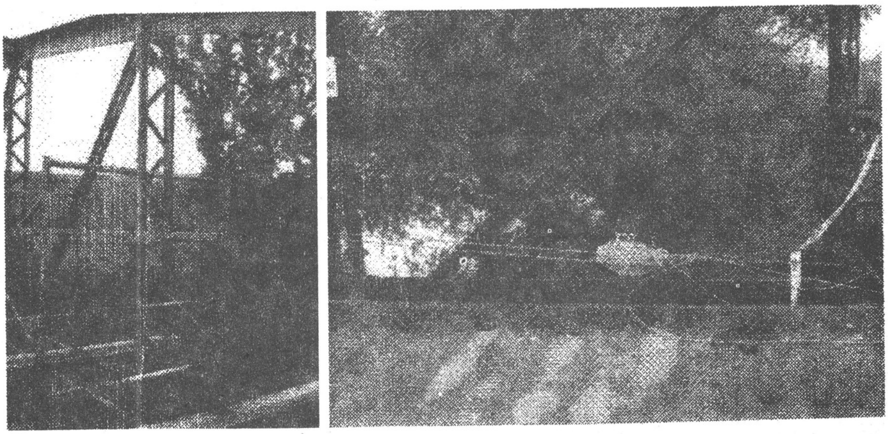


Fig. 25.– Warm straightening of the deformed posts.

2. Sand blasting of the structure and after that cutting the new consolidation elements and their positioning (Figs. 26 and 27).

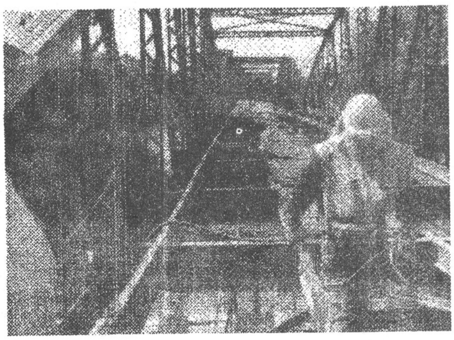


Fig. 26.– Sand blasting of the structure.

Fig. 27.– New consolidation elements.

3. Riveting of the new steel elements (Fig. 28).

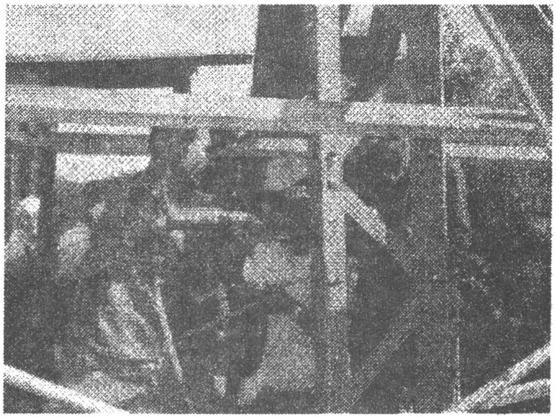


Fig. 28.– Riveting of the new steel elements.

5. Conclusions

The paper realizes an overview of the assessment methodology for old steel highway bridges leading to following conclusions:

1. Every case must be separately considered.
2. Nevertheless the rehabilitation of such representative structures is one of the main tasks of the bridge engineers.
3. This paper also presents a complementary method based on fracture mechanics for the assessment of safety in operation of old highway bridges which avoids the uncertainties of predictions made with the help of classical concept.
4. The procedure to establish safe inspection intervals is also pointed out.

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CRITERII PENTRU EVALUAREA PODURILOR METALICE EXISTENTE PE ŞOSEA

(Rezumat)

Pe reţeaua de drumuri din partea de vest a ţării se găsesc în exploatare, după două războaie mondiale şi alte evenimente, poduri metalice construite la începutul secolului trecut. O parte din aceste structuri reprezintă *mărturii ale trecutului* fiind monumente de artă tehnică şi având un caracter emblematic. Datoria administratorului acestor stucturi este de a le păstra în exploatare. Se prezintă o imagine de ansamblu asupra metodologiei de determinare a siguranţei în exploatare a podurilor vechi bazată pe pincipii moderne. De asemenea este prezentată şi procedura de stabilire a unor intervale de inspecţie a acestor structuri.