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General Principles Regarding the Rehabilitation of Existing Railway Bridges

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1. Introduction

Rehabilitation and maintenance of existing steel constructions, especially steel bridges is one of the most important actual problems [1],[2],[3],[8],[14]. The infrastructure in Romania and in other East – European countries has an average age of about seventy to ninety years. Many of these structures are still in operation after damages, several phases of repair and strengthening. Replacement with new structures raises financial, technical and political problems. The budget of the administration gets smaller. The present tendency to raise the speed on the main lines to a level of $v \leq 160 \text{ km / h}$ must be emphasized (Figure 1).

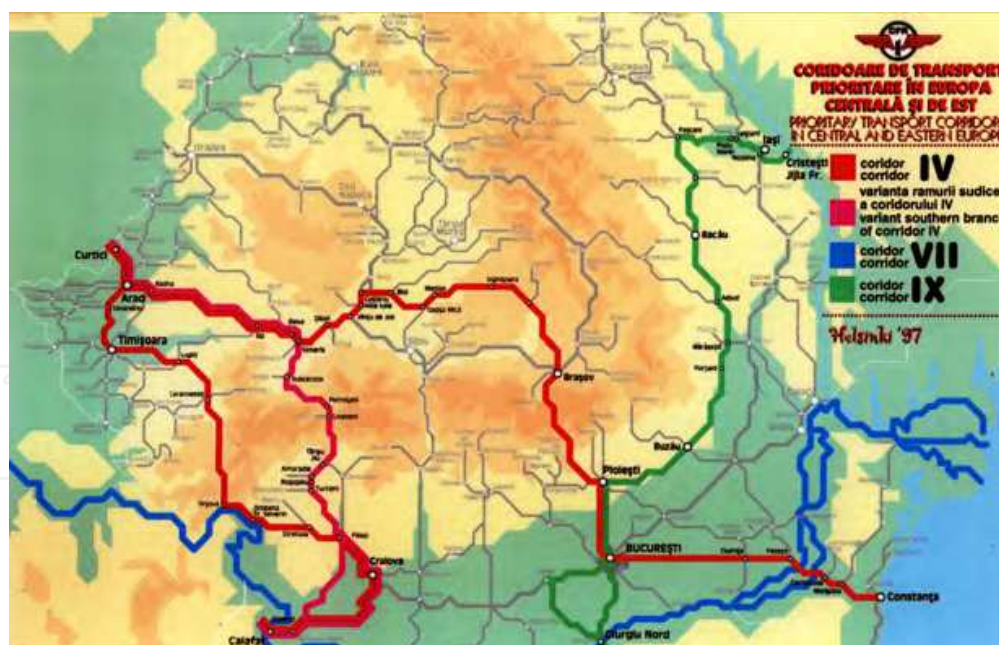


Fig. 1. The European corridors crossing Romania's territory: IV, VII and IX

During service, bridges are subject to wear. In the last decades the initial volume of traffic has 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 (Figure 2).

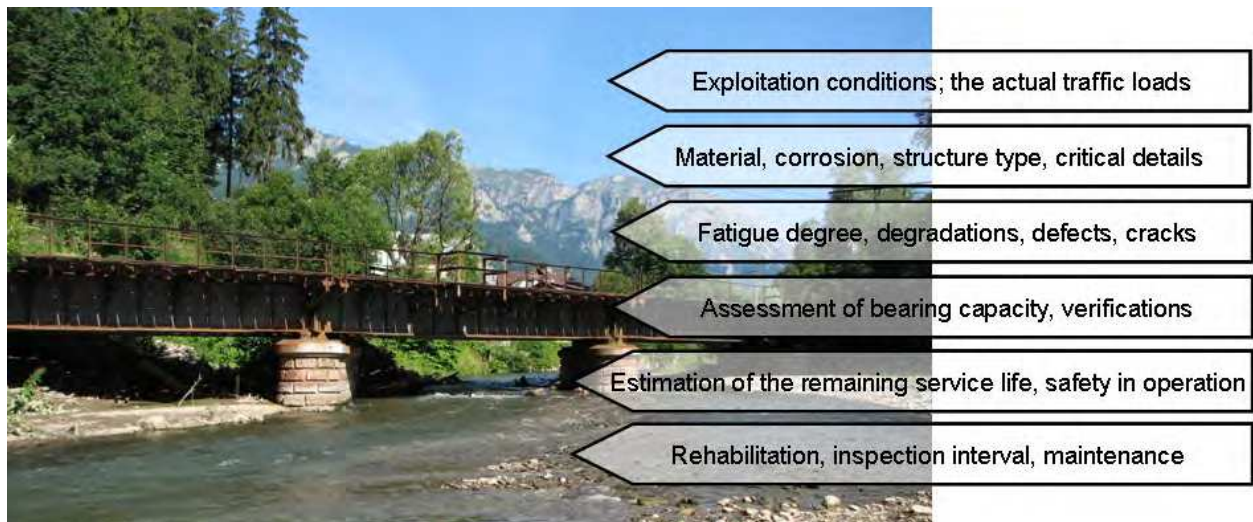


Fig. 2. Assessment and control of existing steel bridges

Carefully 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 check of existing structures should be based on the complete bridge documentation (drawings with accuracy details, dimensions and cross sections of all structural elements, information about structural steel, stress history. However, in many cases these documentations are incomplete or missing. But these informations can be recovered due to the carefully investigations and inspections of the structures, experimental determination of the material characteristics and stresses in structural elements, full scale in situ tests (static and dynamic), calibration of structure and spatial static analysis.

Today, the budget of the administration and the owners (i.e. the railways and highway companies) get smaller. In consequence it is necessary to invest the available money where there will be the greatest benefit. Therefore, those responsible for the decisions need information about the safety of the structure, the remaining life, the costs for maintenance etc. Nobody will take the responsibility for failure of a structure as a result of budget restrictions.

Bridge life is generally given by fatigue; difficult is the estimation of the loading history. For bridges where the stress history is known the fatigue life may be calculated using the Miners's rule and an appropriate S-N curve; also the assumption of the same spectrum for bridge life (or for certain periods) must be made. During the process of assessment the fatigue life of old riveted bridges, is important to establish the proportion of the whole fatigue life that has been already got through. For stringers and cross girders of existing railway bridges the number of 10^7 cycles is exceeded. It might be affirmed that for such bridges if no fatigue cracks can be detected, no fatigue damage has occurred! Subsequently, if the loading spectrum remains the same in the future as in the past, fatigue cracking might not take place! In almost all the cases the loadings increased; in this situation survival for 100 years (or more) without cracking would not justify the assumption that no damage has yet occurred! Minor cracking is difficult to detect during usual inspections.

2. Technical condition of existing bridges

Rehabilitation and maintenance of existing steel bridges is one of the most important actual problems.

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

The evaluation of the current technical condition of an existing steel bridge structure with the help of in situ evaluation depends, in a high percentage, on the engineer's qualification. In the case of in situ inspection it is recommended to insist on the appeared fatigue defects, of riveted or welded connections (the expert must 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 deformations due to traffic, bridge bearings. The expert can use non-destructive techniques to determine the integrity of base material or structural components. The non destructive testing of the inspection of in service damage in bridge elements are the following:

- *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;
- *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;
- *liquid penetration inspection* – is a simple method including the qualification of the personnel; it uses penetrate liquids with fluorescent pigment and UV – light in order to indicate the surface defections;
- *radiographic inspection* – the method is applied for hidden defects and it uses Gamma or Roentgen 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;
- *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 cannot be used for elements made of multiple plates (riveted sections).
- *Eddy Current testing* – this method can detect surface defects but can also be used for thickness inspection.

The usual simplified analysis methods do not always give the lowest resistance values for the structure, but usually the more refined assessment methods which give greater resistance are expensive. According to the experience of the expert a progressive analysis can be applied. In a first step simple classical methods can be applied [2]. If they fail, more sophisticated methods can be used, until either it is shown that the bridge is adequate, or it is concluded that strengthening is needed. An engineer with experience can jump over some time consuming steps which do not give any benefit (an interesting proposal is that the engineer will be paid on a percentage on the saving basis [4]). Actual loads are lower than

those used for design purposes. Fatigue tests on elements taken from demolished structures gives – generally - greater fatigue life than the values according the codes [5].

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.

By differences of more than 5 % of the cross section – due to corrosion, the actual values must be introduced.

However, from the overall examination of a large number of bridges many defects can be pointed out. The defects are widespread, having a heterogeneous character from the point of view of location, development and development tendency; their amplification was also due to the climate and polluting factors that caused the reduction of the cross section due to corrosion. Statistically, in 283 from among 1088 welded bridges, and in 356 from among 3201 steel riveted bridges cracks were detected and repaired. It is not allowed to weld cracks! Old bridges can have welds executed in the early years; a special attention must be paid to these parts. Generally the riveted connections have a good behavior in time due to the initial pre-stressing force which can reach 70 – 80 N/mm². In Table 1 some typical defects in stringers, cross girders, main girders wind bracings and orthotropic deck and their repair are presented [6],[7].

In figure 3 there is presented a crack of a joint plate from a wind bracing and in figure 4 also cracks in the lower flange joint of a double T girder, near to the bearing.



Fig. 3. Crack in the joint plate



Fig. 4. Crack in the lower flange near to the bearing

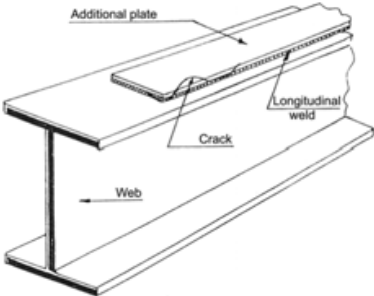
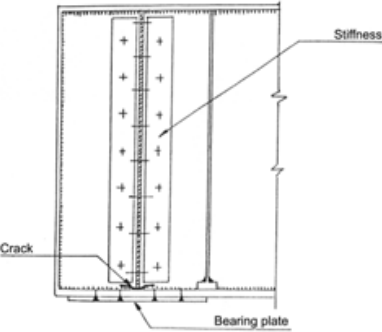
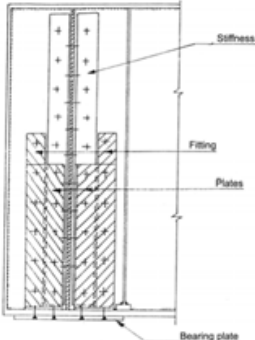
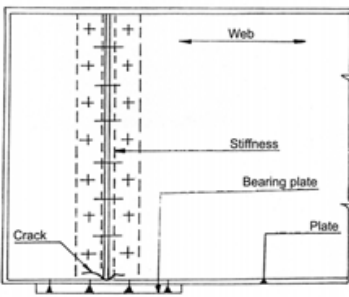
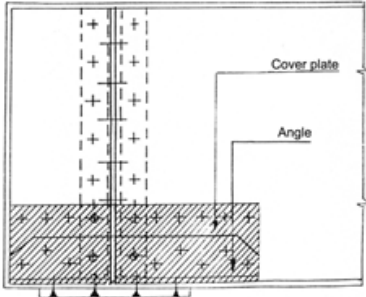
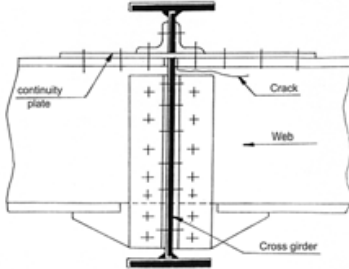
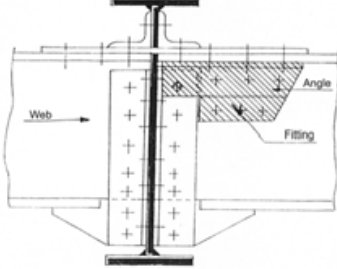
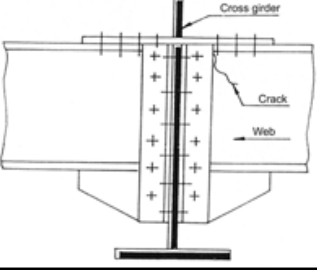
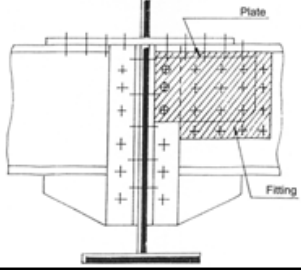
Element	CRACK	REPAIR
0	1	2
Main girder		<div>⇒ Grinding</div> <div>⇒ New additional plate</div> <div>⇒ Observation</div>
Main girder		
Main girder		
Stringer		
Stringer		

Table 1. Typical defects in welded and riveted steel bridges

Element	CRACK	REPAIR
Stringer		
Stringer		
Cross girder		
Cross girder		
Orthotropic deck		
Orthotropic deck		
Stringer		

Table 1. Typical defects in welded and riveted steel bridges - continuation

Element	CRACK	REPAIR
Stringer		
Stringer		
Stringer		
Stringer		
Twin girders		
Twin girders		<p>⇒ Hole at the end of the crack</p> <p>⇒ Replacement</p>
Wind bracing		<p>⇒ Replacement of the gusset</p>

Table 1. Typical defects in welded and riveted steel bridges - continuation

Signs of cracks and defects are rust traces, which occur by friction between jointed plates of the elements. These are of relevance for hidden constructive elements.

Defects of the bearings are frequent as well. In figure 5 there are presented two examples.



Fig. 5. Defects on bearings

Due to deficient maintenance the riveted structures are strongly corroded, especially in the lower zones (figure 6). These corroded surfaces represent also a critical detail for fatigue. Fatigue under corrosion factor is an aspect treated only qualitatively and not quantitatively. The interaction between the two aspects is obvious. There can also found inefficient joints, like weakened rivets (figure 7).



Fig. 6. Strong corrosion at the lower flange – railway bridge on a main line



Fig. 7. Inefficient joints – railway bridge on a main line

3. Characteristics of materials

The following facts show that a material analysis for old riveted bridges is very useful:

- Old bridges are in many cases erected using material with very poor welding qualities and basing on railway administration data and specialized literature it is known that cast iron was used to build bridges;
- The specialized literature doesn't offer enough information about this structural steel;
- The structural material comes from several producers (for South Eastern Europe mostly from Reschitz – Romania and Győr – Hungary).

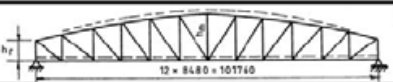
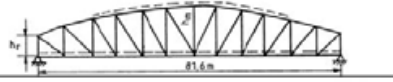
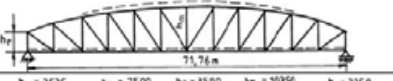

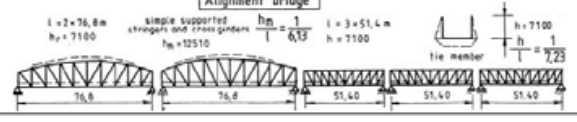
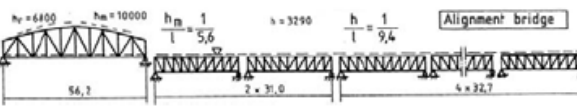
Span	Year of construction	Position	Type of structures
101,76 m	1912 (St.E.G. Reschitz)	Mehadia	 <div>Alignment bridge $h_r = 71.00$ $h_m = 15.000$ $\frac{h_m}{l} = \frac{1}{6.76}$ (interrupted stringers)</div>
81,6 m	1911 (St.E.G. Reschitz)	Valea Cernei	 <div>Alignment bridge $h_r = 50.00$ $h_m = 12.000$ $\frac{h_m}{l} = \frac{1}{6.8}$ (interrupted bridge)</div>
71,76 m	1913 (St.E.G. Reschitz)	Balta Saratã	 <div>Alignment bridge $h_r = 4.300$ $h_m = 9.000$ $\frac{h_m}{l} = \frac{1}{7.9}$</div>
50,65 + 66,20 + 36,70 + 2x30,40 m	1912 (St.E.G. Reschitz)	Şag replaced 1997	 <div>Skew bridge $\alpha = 12^\circ 30'$</div>
2x76,80 + 3x51,42 m	1912 (Győr)	Arad replaced 2000	 <div>Alignment bridge $l = 2 \times 76.80$ $h_r = 7100$ $h_m = 12510$ $\frac{h_m}{l} = \frac{1}{6.13}$ $l = 3 \times 51.42$ $h_r = 7100$ $h_m = 7100$ $\frac{h_m}{l} = \frac{1}{7.25}$ simple supported stringer and cross girders</div>
56,2 + 2x31,0 + 4x32,7 m	56,2 m 1927 (Reschitz) 2x31,0 + 4x32,7 m 1907	Brănişca	 <div>Alignment bridge $h_r = 6400$ $h_m = 10000$ $\frac{h_m}{l} = \frac{1}{5.6}$ $h_r = 3290$ $\frac{h_m}{l} = \frac{1}{9.4}$</div>

Table 2. The bridges on which the present material study basis

The presented study's results can be extended to Middle and South Eastern Europe when the history of communication ways and the state of old railway and highway steel bridges in this region is regarded. In this context the following event can be mentioned: on the 1st of January 1855 the "Kaiserliche und Königliche Privilegierte Österreichische Staatseisenbahngesellschaft" (St.E.G.) took over all steel producers in Banat. The investments in Reschitz (in present Resita) turn the steel mill into an important bridges' factory. The production of steel bridges reached 3960 tonnes in 1910. Bridges made by St.E.G. Reschitz are still in use in Romania, Austria and Hungary. Between 1911 – 1913, 1620 tonnes of bridge structures made of cast iron were replaced in the western part of Romania (Banat), namely on the railway segment Timișoara – Orșova. In this sense the material study took into account bridges from this region, built around 1911. Following material analysis were performed in order to determine the characteristics of the material: chemical analysis, metallographic analysis, tensile tests, Brinell tests, Charpy "V Notch" tests. The samples were taken from secondary elements, but also in some cases (Bridges in Arad and Şag which were replaced) from main elements: stringers, cross girders, main girders [7]). The results of the chemical analysis are presented in the following table.

No.	Bridge / Specimen number & position	Chemical composition						
		C %	S %	Mn %	P %	Si %	Ni %	N %
1.	Valea Cernei Bridge / S1-VCB / Secondary elements	0,12	0,034	0,39	0,021	0,023	-	-
2.	Valea Cernei Bridge / S2-VCB / Sec. Secondary elements	0,11	0,020	0,32	0,024	0,010	-	-
3.	Mehadia Bridge / S1-MB / Secondary elements	0,11	0,014	0,50	0,028	0,050	-	-
4.	Balta Sărată Bridge / S1-BSB / Secondary elements	0,07	0,072	0,38	0,016	0,010	-	-
5.	Şag Timiş Bridge / S1-STB / Secondary elements – Span I	0,16	0,058	0,46	0,035	0,035	-	-
6.	Şag Timiş Bridge / S2-STB / Secondary elements – Span II	0,14	0,066	0,63	0,030	0,061	-	-
7.	Şag Timiş Bridge / S3-STB / Secondary elements – Span III	0,13	0,060	0,50	0,051	0,112	-	-
8.	Şag Timiş Bridge / S4-STB / Secondary elements – Span IV	0,18	0,054	0,48	0,066	0,056	-	-
9.	Şag Timiş Bridge / S5-STB / Secondary elements – Span V	0,18	0,017	0,44	0,059	0,010	-	-
10.	Brănişca Bridge / S1-BB / Secondary elements	0,12	0,070	0,39	0,013	0,060	-	-
11.	Brănişca Bridge / S2-BB / Secondary elements	0,11	0,055	0,38	0,014	0,060	-	-
12.	Arad Bridge / S1-AB / Secondary elements – Span I	0,14	0,061	0,53	0,038	0,069	-	-
13.	Arad Bridge / S2-AB / Secondary elements – Span II	0,19	0,033	0,64	0,051	0,018	-	-
14.	Arad Bridge / S3-AB / Main elements – Stringers Span III	0,089	0,032	0,531	0,009	0,018	0,067	-
15.	Arad Bridge / S4-AB / Main elements – Cross girders Span III	0,058	0,059	0,485	0,017	0,018	0,037	-
16.	Arad Bridge / S5-AB / Main elements – Main girder Span III	0,056	0,032	0,493	0,001	-	0,031	-
17.	Arad Bridge / S6-AB / Secondary elements – Span IV	0,18	0,035	0,46	0,063	0,078	-	-
18.	Arad Bridge / S7-AB / Secondary elements – Span V	0,1	0,047	0,43	0,020	0,030	-	-
19.	St 37 (STAS 500/2-80)	0,25	0,065	0,85	0,065	0,07	0,30	0,015
20.	St 34 (STAS 500/2-80)	0,17	0,055	0,60	0,055	-	-	-
21.	INCERTRANS Bucharest Research for cast iron	0,04... 0,11	0,014...0, 043	0,15... 0,48	0,121...0, 32	0,07... 0,31	-	-
22.	German Research for cast iron*	0,16	0,056	0,100	0,470	0,100	0,007	-

* German research – Stahlbau 05.1985 (Prof.Dr.Ing. D. Kosteas, Ing. W. Stier, Ing. W. Grap)

Table 3. Chemical analysis results

The statistical interpretation of the tensile tests results shows a minimal value for the yield stress of 230 N/mm².

The impact tests on Charpy V Notch specimens lead to conclusion that the transition temperature is situated in many cases in the range from -10°C to 0°C (figure 8).

By analyzing the laboratory results it can be concluded that the steel is a mild one, that could be associated to the present steel types St 34 or St 37.1 (S 235 according to the Eurocodes). Also, on the two dismantled bridges – Arad and Şag – fracture mechanics tests were made [7] in order to establish the integral value J_c according to ASTM E813-89 (figure 9), the CTOD and to determine the fatigue crack growth rate and the material constants C and m according to ASTM E647-93. For these tests compact specimens CT (thickness 8 mm) as well as bending specimens for CTOD, have been used.

They have been obtained from the stringers, cross girders and main girder – lower chord. The minimal value of material toughness in term of J-Integral for these old riveted steel bridges is $J_{crit} = 10 \dots 20$ N/mm for a temperature of -20°C.

The method of fatigue assessment for structural elements with defects, was developed basing on the possibility of crack propagation modeling under fatigue loads and with the help of known laws.

The method is founded on the recommendations of the BS 7910:1999.

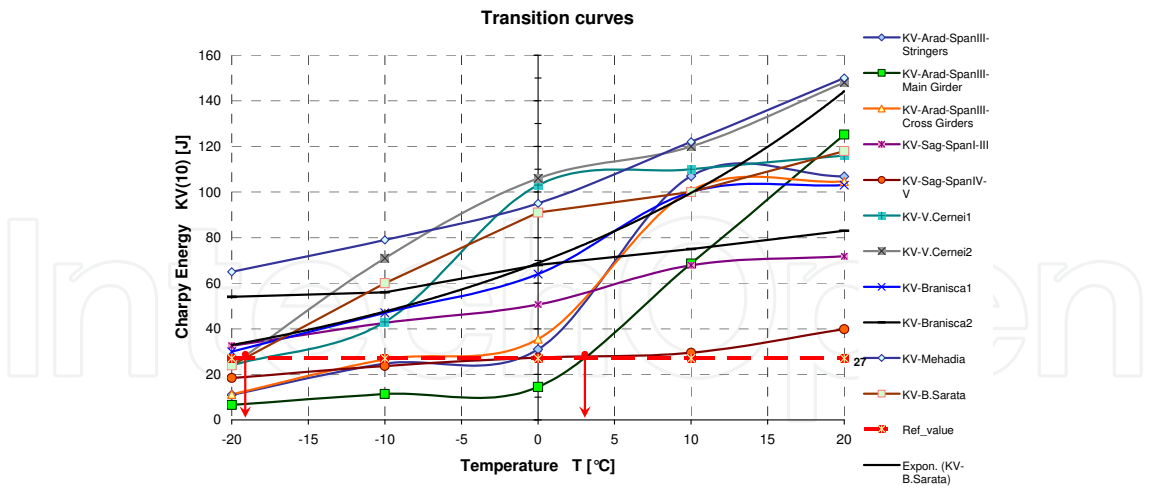


Fig. 8. Transition curves for the analyzed bridge structures



Fig. 9. Experimental tests for determining of J values

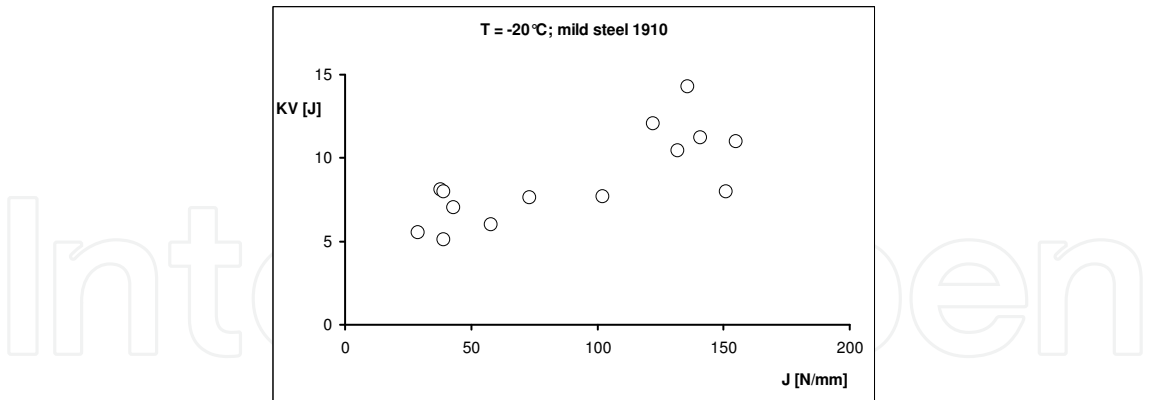


Fig. 10. Charpy V Notch Energy vs. material toughness J_c (Arad Bridge 1912)

In the present state of knowledge it is generally accepted that the fatigue failure of materials is a process containing three distinct steps: (1) initiation of defect (crack), (2) crack propagation in material, (3) separation through complete failure of the material in two or more pieces. Practically, the safety service life of an element under fatigue conditions can be expressed as follows (figure 11):

$$N_f = N_i + N_p \tag{1}$$

N_i = number of cycles necessary for the initiation of the defect (crack)
 N_p = number of cycles necessary for the propagation of the defect until the occurrence of failure.

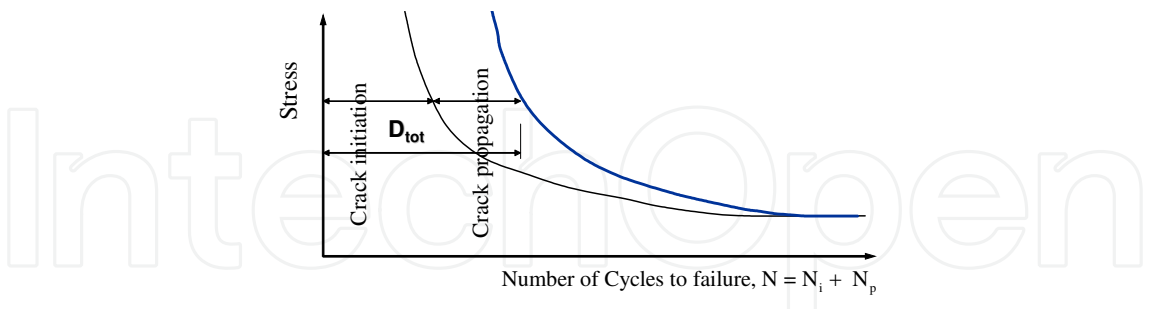


Fig. 11. Fatigue life of structural elements

The evaluation of crack propagation conditions can be accomplished with the help of characteristically values, which are founded on fracture mechanics concepts: material toughness express by the stress intensity factor K or J integral value and the crack growth rate da/dN (crack growth for each load cycle). A relation of the following type can express the crack growth rate (Figure 12):

$$\frac{da}{dN} = f(\Delta K, R, H) \tag{2}$$

da/dN - crack extension for one load cycle; ΔK - stress intensity range, established basing on the stress range $\Delta\sigma$; R - stress ratio; H - indicates the stress history dependence.

The crack growth rate da/dN , defined as a crack extension - da obtained through a load cycle dN (it can also be defined as da/dt , in which case the crack extension is related to a time interval), represents a value characteristic of the initiation phases respectively the stable crack propagation. It has been experimentally observed that the connection between the crack growth rate and stress intensity factor variation represents a suitable solution for the description of the behavior of a metallic material containing a crack, as in the case of steel. In a logarithmic graphical representation of the crack growth rate da/dN versus the stress intensity range ΔK a curve as the one in the following figure (Fig. 12), is obtained.

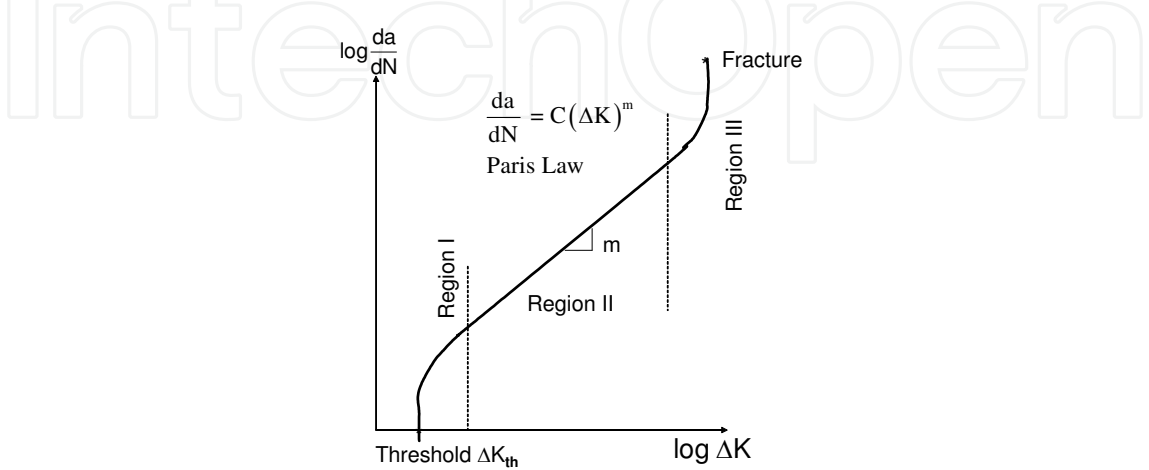


Fig. 12. Logarithmic representation of the fatigue crack growth in steel

The calculation of the structural elements remaining service life can be done basing on the Paris law, more precisely by integration of this law:

$$N = \int_0^N dN = \int_{a_0}^{a_{crit}} \frac{da}{C \cdot \Delta K^m} \quad (3)$$

N - number of stress cycles necessary in order that the crack extends from its initial dimension a_0 to the critical value a_{crit} , where failure occurs;

a - crack length;

C, m - material constants from the crack propagation law;

ΔK - stress intensity factor range.

This integral can be numerically calculated by taking into account a critical detail knowing the crack values (initial and critical), basing on the following relation:

$$N = \int_{a_0}^{a_{crit}} \frac{da}{C \cdot \Delta \sigma^m \cdot Y^m \cdot (\pi a)^{-m/2}} \quad (4)$$

The number of cycles N_i obtained with the help of relation (4) represents the remaining service life of the detail, by regarding the initial length a_0 up to the critical length a_{crit} , by admitting stable crack propagation (Figure 13).

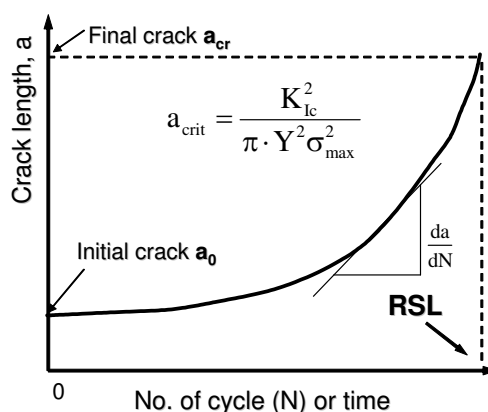


Fig. 13. Principles for determining the remaining service life

The critical crack value can be calculated basing on the K criterion respectively on the J or $CTOD$ criterions or with the help of the failure assessment diagram.

The C and m material constants from the Paris law are experimentally determined by fracture mechanical tests. In this sense in most cases compact specimens $C(T)$, three point bended specimens $SEN(B)$ and middle central panels $M(T)$ are used. Such a standard which describes the test methods for the determination of the crack growth rate, is the American Standard ASTM E 647 (Standard Test Method for Measurement of Fatigue Crack Rates).

Basing on the determined values da/dN and ΔK the program presented below, also automatically determines the C and m material constants by the Paris relation:

$$\ln\left(\frac{da}{dN}\right) = \ln C + m \cdot \ln \Delta K \quad (5)$$

The experimental tests on 26 CT specimens from two old bridges have shown that for the oldest mild steels the values of the material constants from the Paris relation are in the following intervals: $m = 2,05 \dots 5,65$ and $C = 2,2 \times 10^{-11} \dots 10^{-18}$.

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



Fig. 14. Experimental tests in order to establish the material constants C and m

4. Present verification concept

During service bridges are subjected to repeated loadings causing fatigue. Therefore many bridges require an inspection. The examination should consider the age of the bridge and all repairs, the extent and location of any defects etc. [8]. A continuous maintenance, which generally must increase in time, is important in order to assure the safety in operation of the existing structures. The present methodology includes the following stages [9]:

- Step 1.** estimation of the loading capacity of the structure based on a detailed inspection; analysis of drawings, inspection reports, repairs, reinforcements, analysis of the general behavior of the bridge (displacements, vibrations, corrosion and cracks). In this phase the stresses in the structure can be calculated with the usual simplified hypothesis;
- Step 2.** the accurate determination of the stresses in the structure and of the remaining safety of the elements. This phase includes: tests on materials, computer aided analysis of the space structure, remaining safety calculated on the base of the real time - stress history;
- Step 3.** in situ static and dynamic tests.

This methodology adopted by the Romanian standard is illustrated in Figure 15.

The calculation of remaining fatigue life is normally carried out by a damage accumulation calculation. The cumulative damage caused by stress cycles will be calculated; failure criteria will be reached.

$$D = \sum \frac{n_i}{N_i} \leq 1 \quad (6)$$

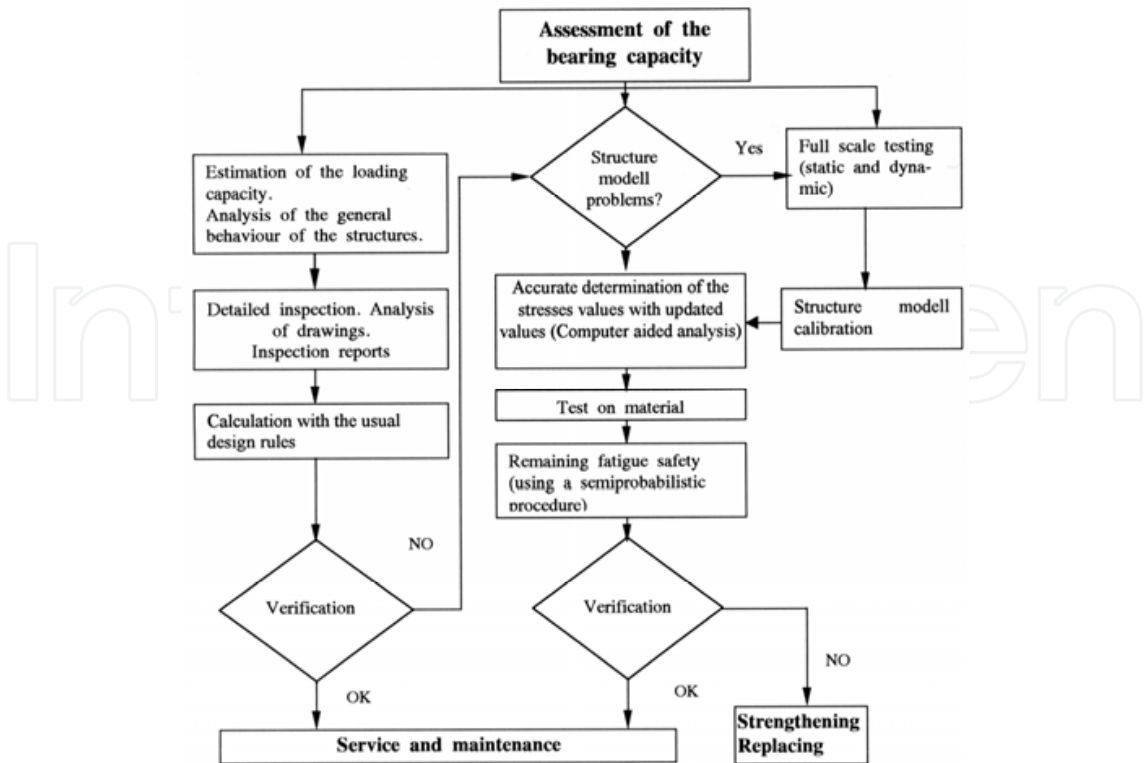


Fig. 15. Methodology of the Romanian standard SR 1911-98 [3]

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. It is very clear that the kind of fatigue cracks, which are initiated by structural non-homogeneity (possible non-metallic inclusions or other impurities), surface defects (including corrosion) and the stress factor, are present in the old riveted structures.

The presence of cracks in structural elements modifies essentially their fracture behavior. Fracture, assimilated in this case as crack dimensions growth process under external loadings, will be strongly influenced by the deformation capacity of material. The FM approach has acceleration in damage increase; with increasing damage a smaller stress range contribute to the damage increase. The authors proposed [9], a complementary method based on the fracture mechanics basic concept

$$J_I \leq J_{Ic} \tag{7}$$

in order to calculate the remaining fatigue life.

In practice two situations can be distinguished:

- $D < 0,8$ 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.
- $D \geq 0,8$ 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, the establishing of the maintenance program, the determination of inspection intervals, the inspection priorities of structural elements and finally the calculation with high accuracy of the remaining service life of old riveted bridges takes into account the following main data:

- type of structure and exploitation conditions (traffic events);
- information about structural steel (mechanical properties – yield strength, tensile strength, hardness, transition curve ductile – brittle and transition temperatures, chemical composition, metallographic analysis);
- determination of critical members and details;
- crack detection and inspection techniques for evaluation of the initial crack size – a_0 and crack configuration;
- recording of the stress spectrum for the critical members under the actual traffic loads;
- evaluation of the critical crack size – a_{crit} based on failure assessment diagrams;
- fracture mechanics parameter – K_{crit} , δ_{crit} , J_{crit} (fracture toughness);
- simulation of the fatigue crack growth;
- temperature, environment conditions.

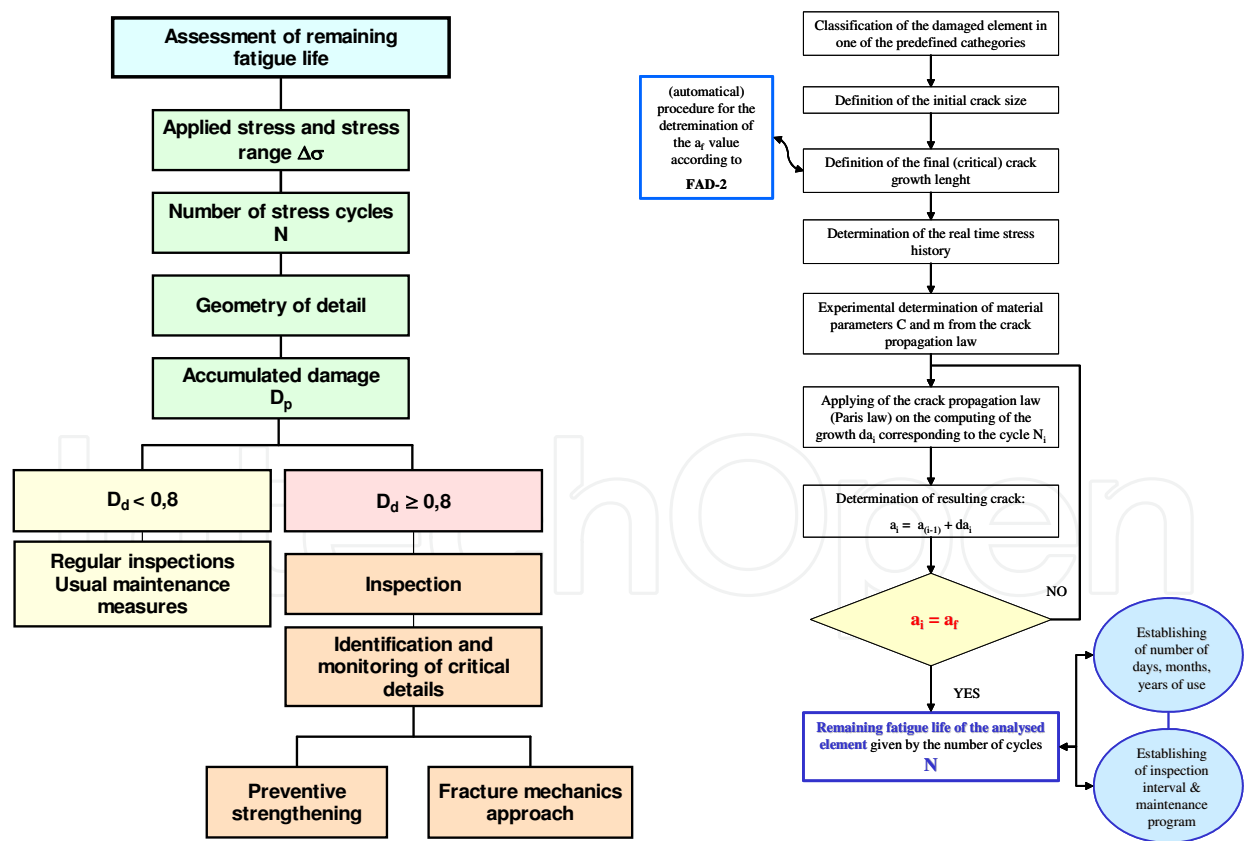


Fig. 16. Assessment of the remaining fatigue life and the crack growth procedure



Fig. 17. Soft for life prediction analysis

The methodology is conceived as an advanced, complete analysis of structural elements containing fatigue defects, being founded on fracture mechanics principles and containing two steps: namely one of determination of defects' acceptability with the help of Failure Assessment Diagrams (level 2) [10] and of determination of final acceptable values of defect dimensions; this is followed by a second step which in fact represents a fatigue evaluation of the analyzed structural elements basing on the present stress history recorded on the structure, 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 needed in order that a fracture take place, respectively the determination of the remaining service life of the structural elements (years, months, days).

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. In this situation the safe inspection intervals can be calculated with the following relation:

$$\Delta T_{\text{insp}} = N_{\text{RFL}} / N_{\Delta a} \quad (8)$$

where

N_{RFL} = the remaining fatigue life calculated for the structural element

$N_{\Delta a}$ = the number of cycles computed for a crack extension rate of 5 mm during two successive inspections.

The life prediction computing was performed with the help of a soft developed by one of the authors [7].

5. Case studies

Three European corridors cross Romania's territory: IV, VII and IX. Of a special interest for many European countries is the Pan-European Corridor IV „Berlin - Nürnberg - Prague - Budapest - Constanta /Istanbul/ Thessaloniki”. On Romania's territory the railway component of Pan - European Corridor IV has the following route: Curtici - Arad - Sighisoara - Brasov - Predeal - Campina - Bucharest - Constanta.

Due to the fact that the Campina - Predeal railway route crosses the sub-Carpathians area and the southern part of the Southern Carpathians it was necessary to adapt it to the difficult geographical conditions; actually this section is the most complicated part from the whole Romanian route. The railway line was built step by step beginning with 1879. The doubling of the 43 km long railway line Campina - Predeal was accomplished between 1939 - 1942. The line electrification was completed during 1961 - 1965.

Generally, the line is oriented from the south to the north. It follows the valley of the Prahova River crossing this river in 17 points.

This section includes 41 bridges. From this total number 22 are plate girder bridges, 6 are truss structures, 12 concrete bridges and 1 are conceived in the filler beams deck solution.

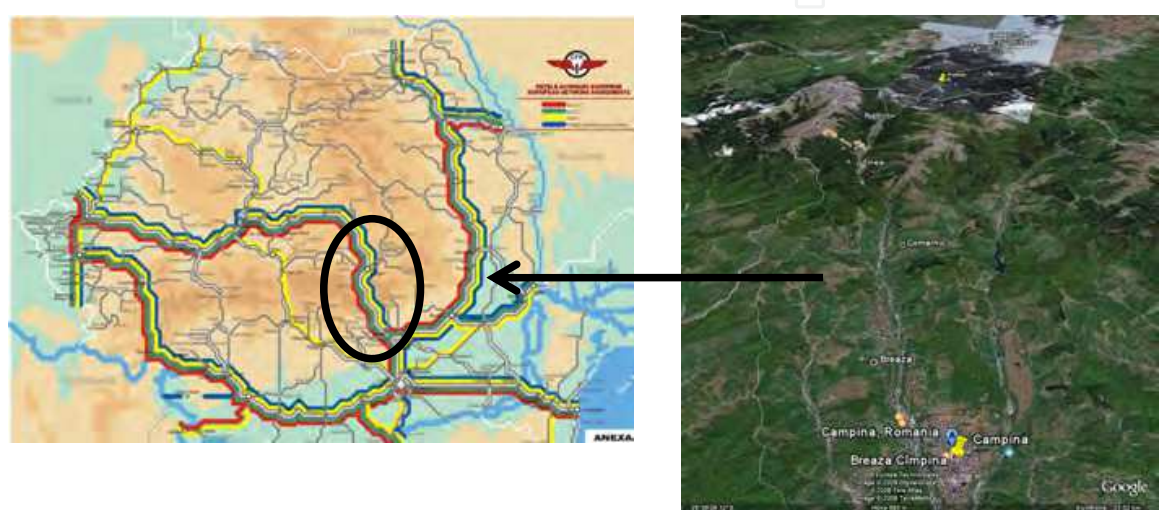


Fig. 18. Emplacement plan

All these structures were verified. In the following some aspects regarding the verification of the steel bridges are presented. The methodology which was adopted is showed in the figure 19.

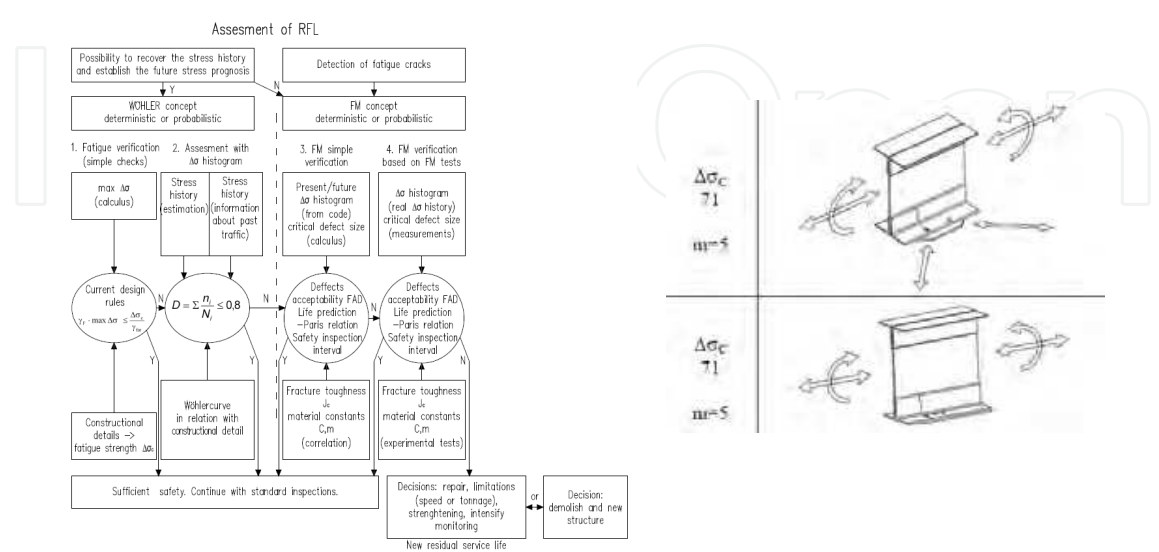


Fig. 19. The methodology and the constructive details for the evaluation of the RFL

After the stress analysis (Figure 20) and the fatigue verifications based on the Wöhler concept which were made in relation with the prescriptions of the Romanian standard SR 1911-1998, Swiss code 161 & SBB Richtlinie 2002 and the German code DS 805-2002, a life prediction analysis based on fracture mechanics principles was performed in order to evaluate the remaining fatigue life for these structures for the new traffic UIC conditions.

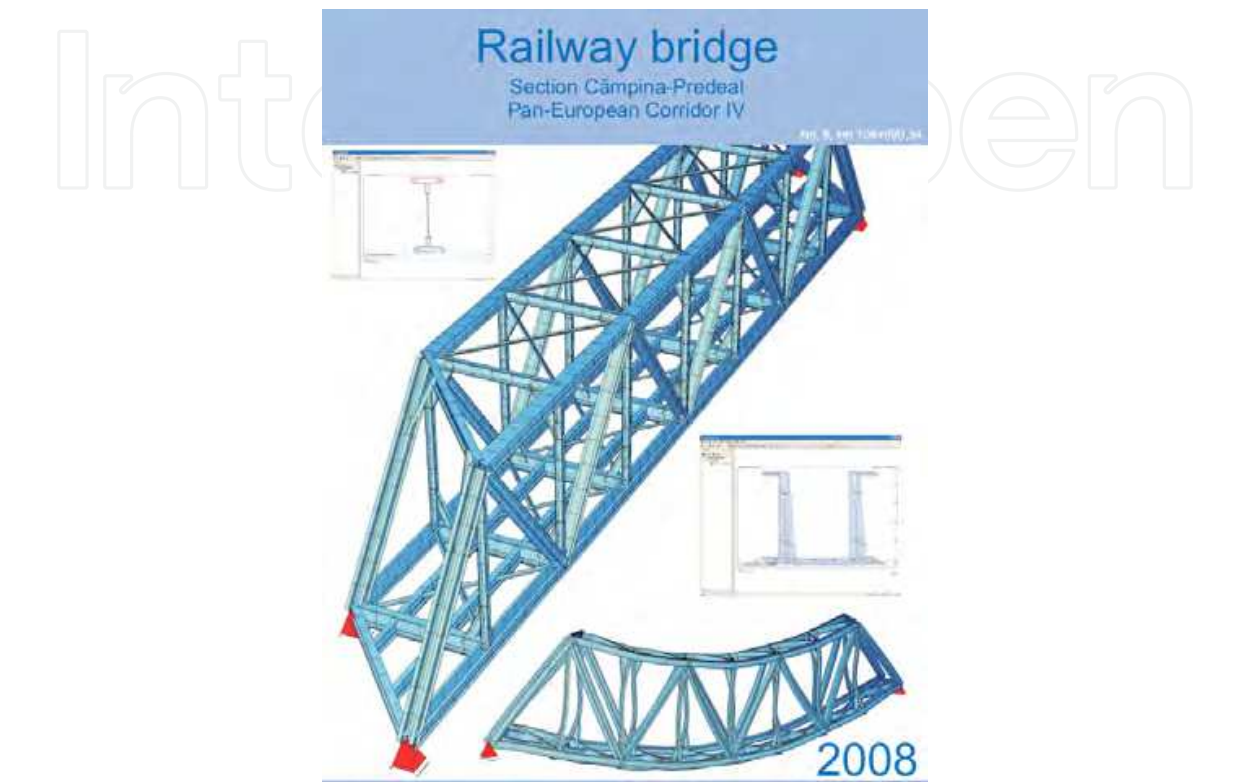


Fig. 20. The static model of the structure no. 6, km 108+690,34

For example, a plate girder bridge (no. 28, at km 125+323.25) is presented, which is a riveted structures crossing the Prahova River. The structure has four spans, one of 13.90 m and three of 14.10 m each and is skew (skew to the right - 48°37') and its superstructure is made out of independent plate girder decks.

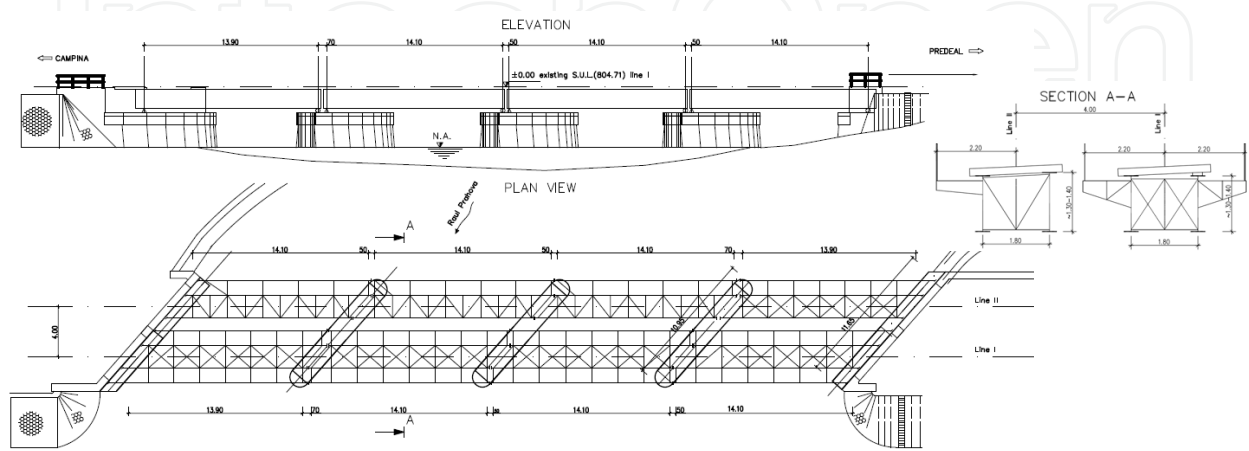


Fig. 21. The general disposition of the bridge no. 28

Calculation elements:

- Construction year – 1940.
- Due to lack of any documentation, the dimensions of the main girders elements were measured in the emplacement (Figure 22).

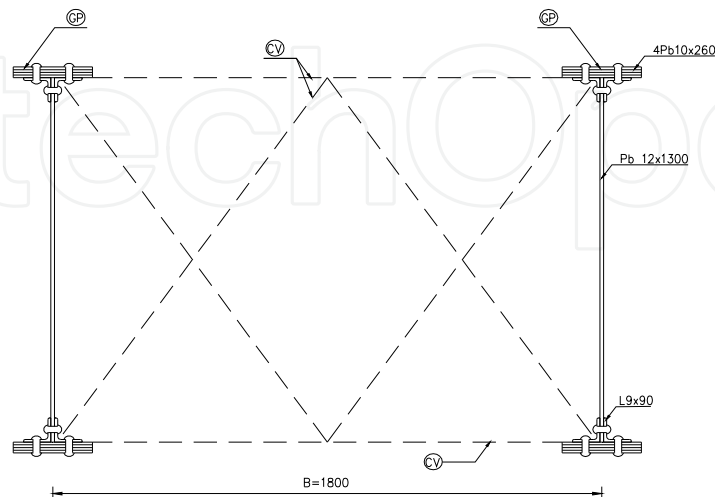


Fig. 22. Dimensions of the steel elements – Main girders

- The basic material to be considered is mild steel similar with St 37 (S235).
- The static scheme - simply supported girders.

The traffic on one main railway line was considered:

- the reference traffic $20-24 \times 10^6$ t/ line/ year
- the Calculation Centre of SNCF CFR S.A. for the year 2004: 13,7 t/ line / year => 12-16 t / line/ year.

The traffic expressed in pairs of trains per day in the peak month that took place on the line Câmpina – Predeal in the year 2004, was:

- 28 pairs of passenger trains / day;
- 17 pairs of freight trains / day;

Total – 48 assimilated pairs of trains / day

For the main girders of the steel decks, all the fatigue checks were performed in the section from the middle of the span.

According to the stress analysis, the maximal stress range for UIC 71 convoy is:

$$\max \Delta \sigma_{UIC} = 97.13 \text{ N} / \text{mm}^2$$

The resulted damage accumulated value (Miner rule) is $D = 0.98$.

Also, the complementary method of fracture mechanics was applied. For the material characteristics followings values were considered: the material is mild steel similar to the former steel St 34 - 37. n (Romanian standard - STAS 500/2 - 80); yield stress is $\sigma_y = 230$ N/mm²; tensile stress $\sigma_{ult} = 360$ N/mm².

For the material toughness in terms of J_{crit} a minimal value of 20 N/mm at a temperature of -20°C was chosen. For the life prediction procedure in the case of the material constants following values have been chosen: $m = 3$ and $C = 3 \times 10^{-12}$ (see also [11]).

A stress history was established using the following mixed traffic from EC 1. This is actually the future traffic which will be characteristic for the new rehabilitated railway corridor. All these trains were moved on the structure in order to establish the multi-block stress history.

Train type	No. of trains / day	Train weight [t]	Traffic volume [Mil. t/year]
1	12	663	2,90
2	12	530	2,30
3	5	940	1,72
4	5	510	0,93
5	7	2160	5,52
6	12	1431	6,27
7	8	1035	3,02
8	6	1035	2,27
	67		24,95

Table 4. Mixed traffic from EC 1

All these trains were moved on the structure in order to establish the multi-block stress history.

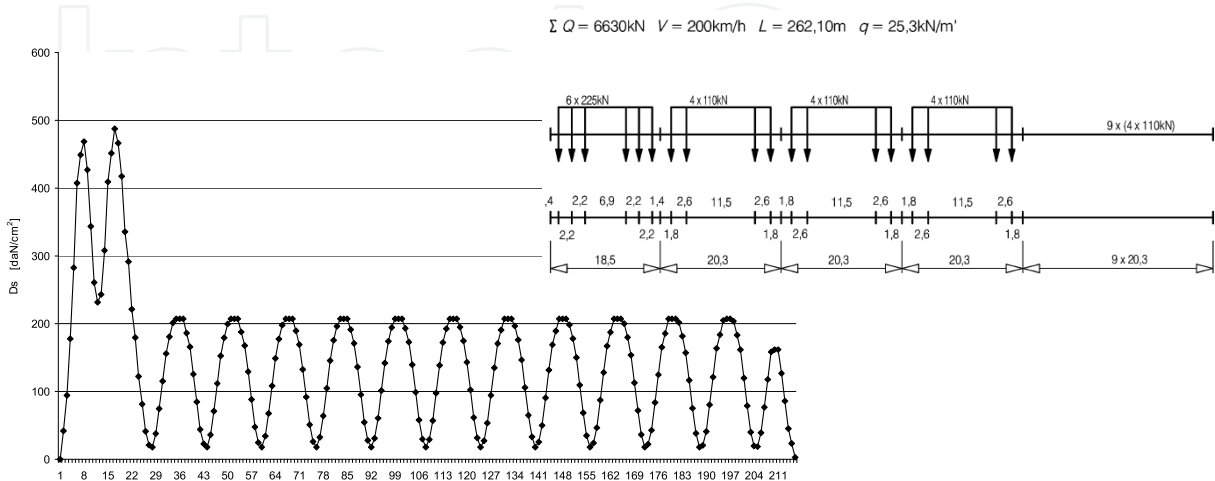


Fig. 23. The stress range diagram $\Delta\sigma$ for train type 1

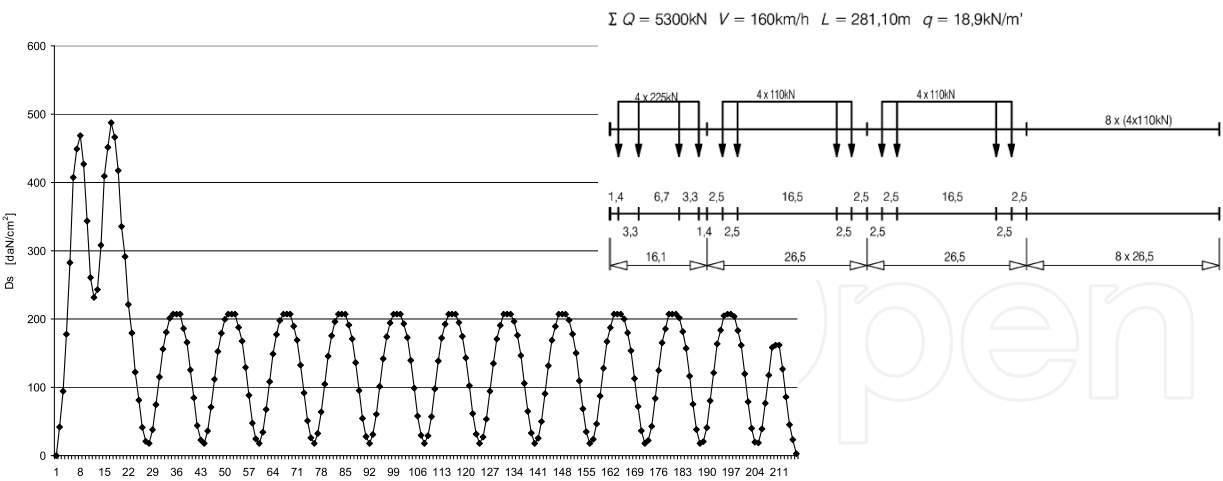


Fig. 24. The stress range diagram $\Delta\sigma$ for train type 2

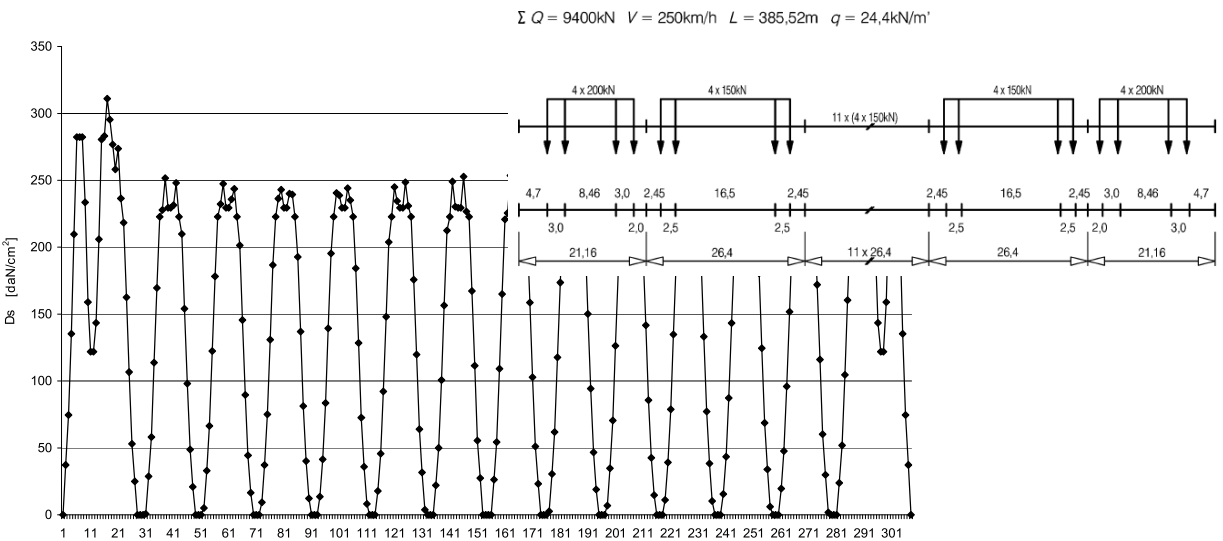


Fig. 25. The stress range diagram $\Delta\sigma$ for train type 3

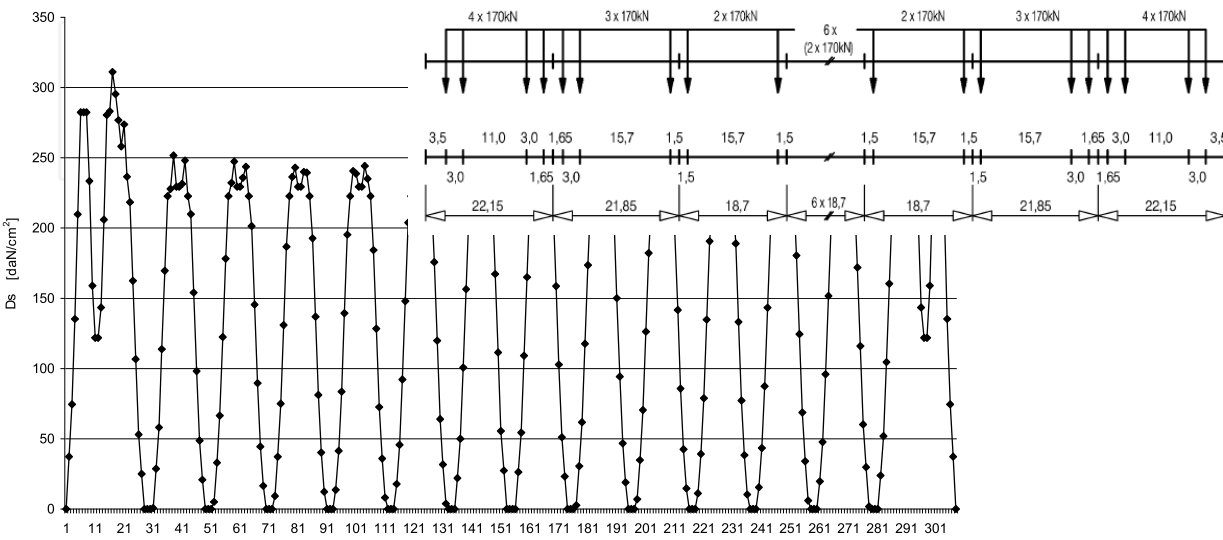


Fig. 26. The stress range diagram $\Delta\sigma$ for train type 4

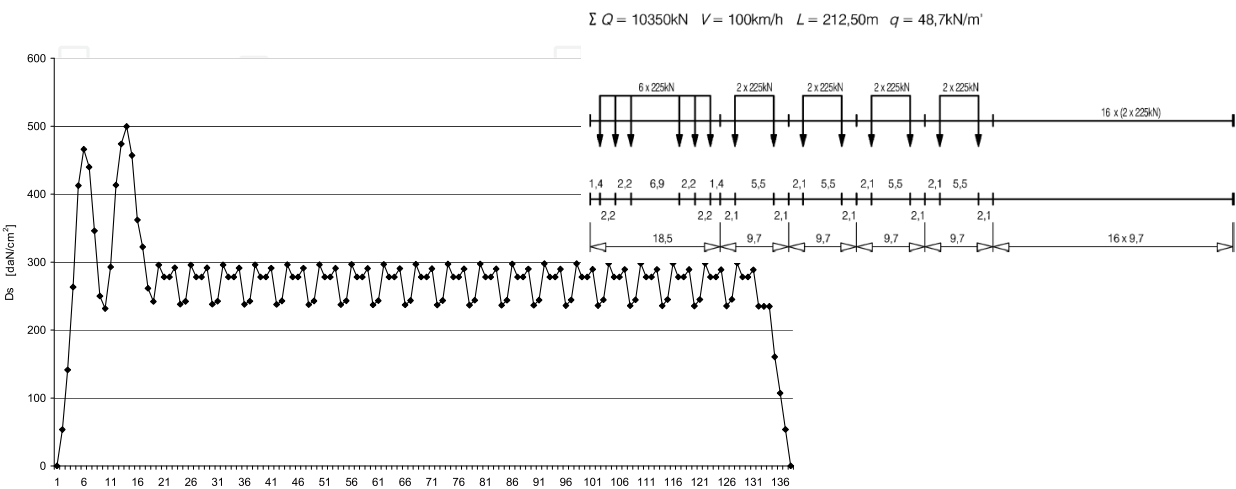


Fig. 30. The stress range diagram $\Delta\sigma$ for train type 8

$\Delta \sigma$ [daN/cm ²]	MAIN GIRDER (L/2) mix								
	T1	T2	T3	T4	T5	T6	T7	T8	Total
50	0	0	0	0	105	36	0	108	249
100	0	0	0	0	7	12	0	0	19
150	12	12	10	0	0	0	0	0	34
175	0	108	0	0	0	0	0	0	108
200	132	0	0	10	105	36	8	0	291
250	12	0	65	60	0	0	0	12	149
275	0	0	10	10	0	0	0	0	20
300	12	12	10	0	0	60	0	6	100
350	0	12	0	0	0	60	8	0	80
400	0	0	0	0	0	36	0	0	36
450	0	0	0	0	0	0	88	6	94
475	12	0	0	0	0	0	0	0	12
500	0	0	0	0	14	72	8	0	94
$\Delta \sigma_e$ [daN/cm ²] =						381.36	Total/day		1037

Table 5. Classification of the stress range intervals

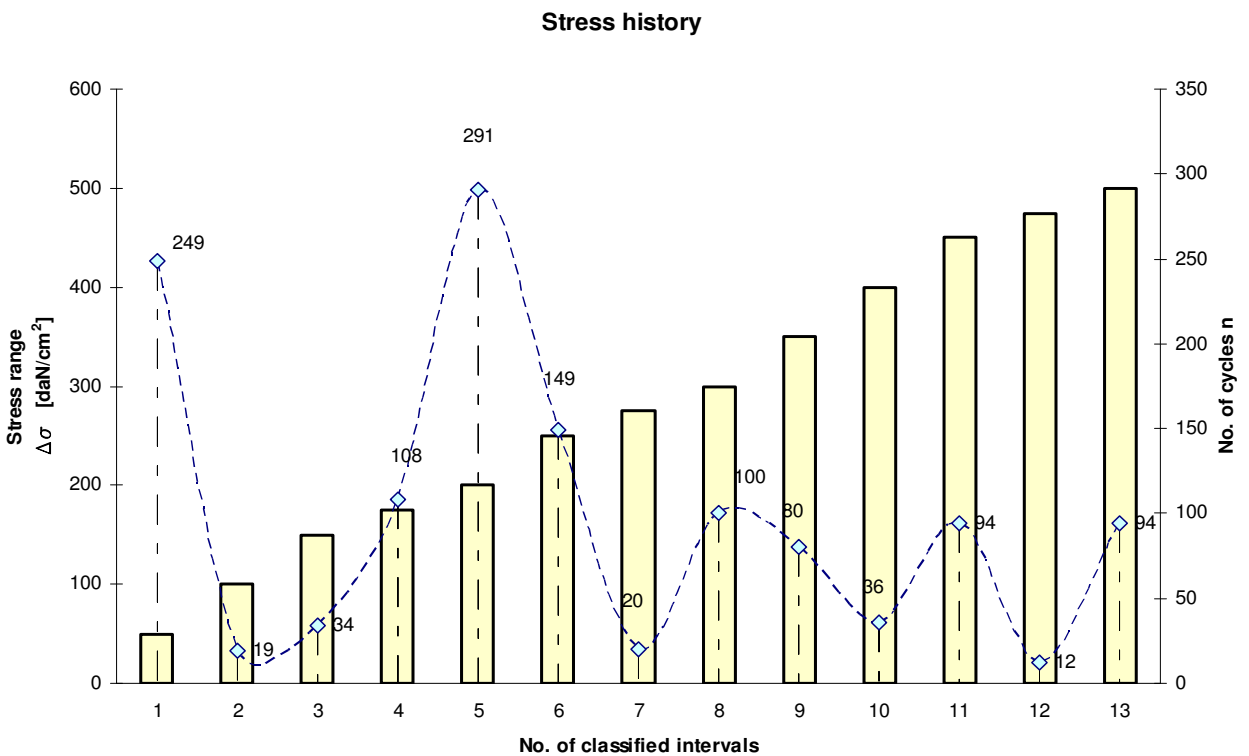


Fig. 31. Stress history

The crack case assumed for these structures (riveted bridges) is illustrated in figures 32; it is a trough thickness defect.

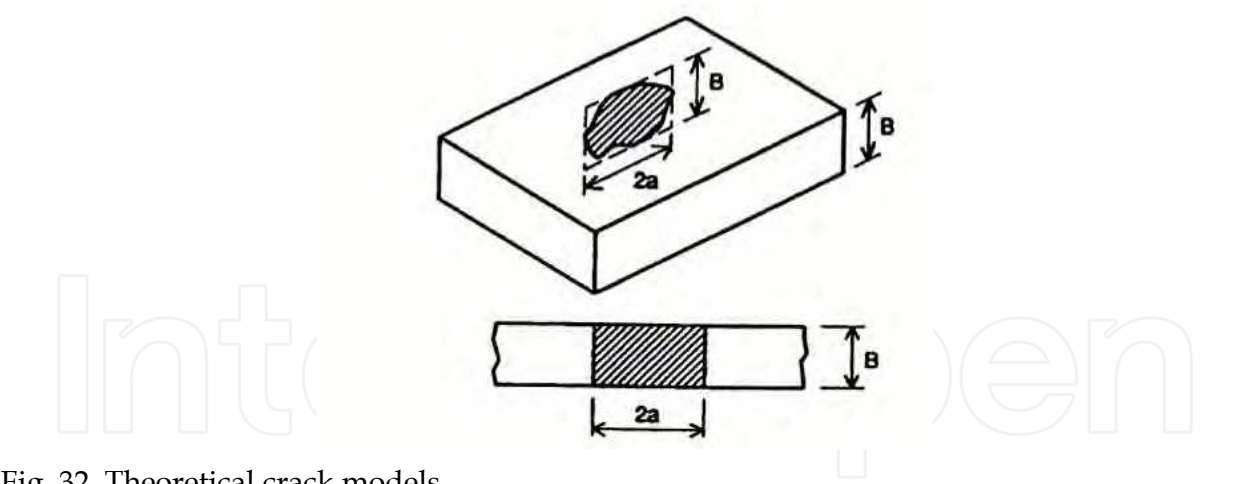


Fig. 32. Theoretical crack models

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:

- through thickness flaw with initial size $a_0 = 2.0$ mm which is undetectable because it is situated under rivet head in the web steel plate, and
- through thickness flaw with initial size $a_0 = 2.0$ mm which is also undetectable during a visual inspection appearing in the corner (lower flange).

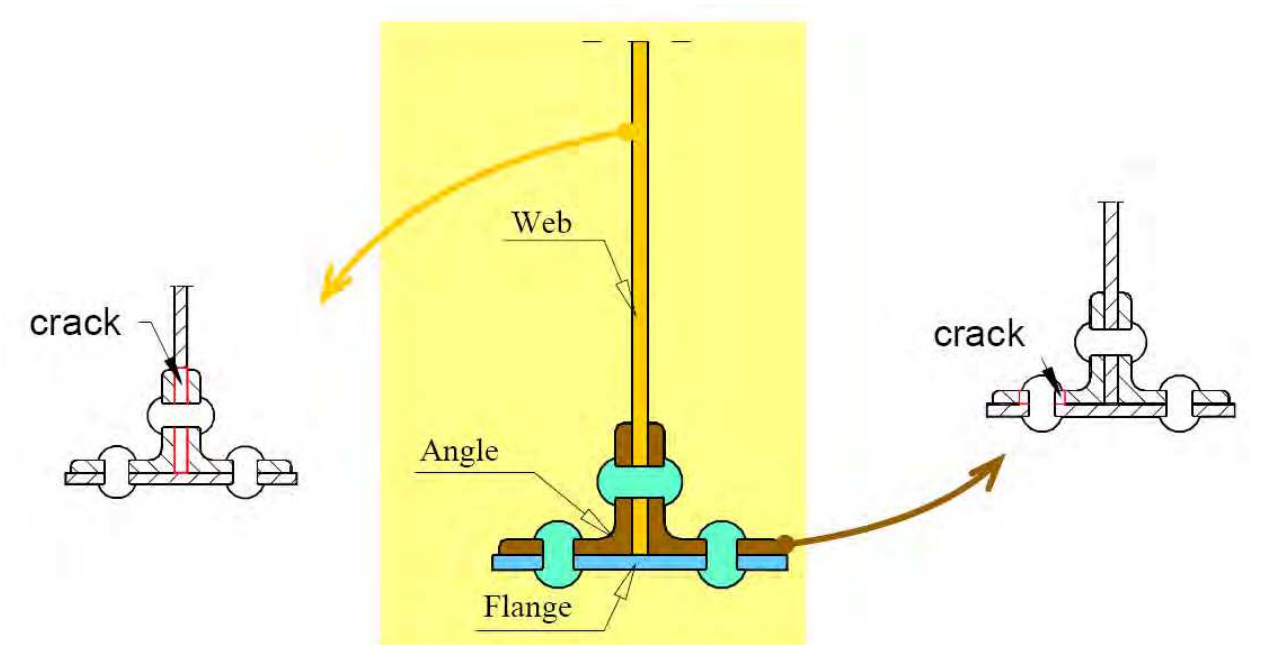


Fig. 33. The two studied defect cases

In the first case the remaining fatigue life is estimated at a value of 8.31 years (corresponding to a number of cycles of 1 566 000). In the second case the administrator must be acquainted with the fact, that propagation of the crack to the critical value will occur in 8.89 years ($N=1\,676\,000$ cycles).

In this case the safe inspection intervals, calculated with formula 8, are situated between 1.2 and 2.0 years.

Finally due to the high corrosion level (Figure 34) and also based on the fatigue assessment, the superstructure was proposed for the replacement.



Fig. 34. Corrosion attack details

Also all the other plate girder bridges were verified. They are presented in the next table.

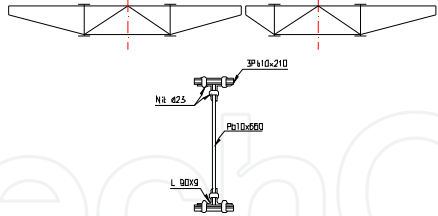

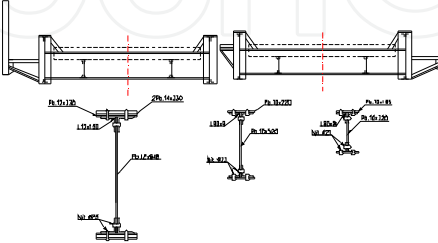

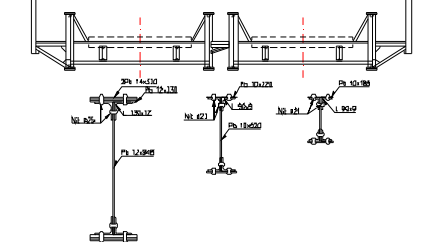

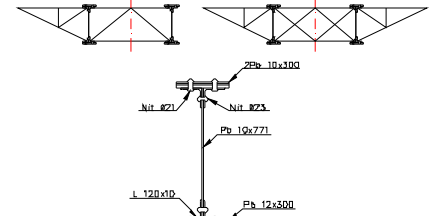

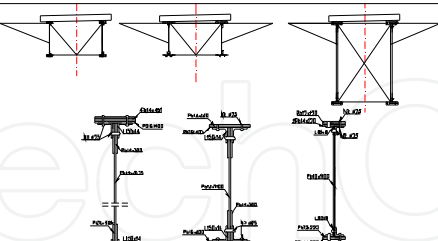

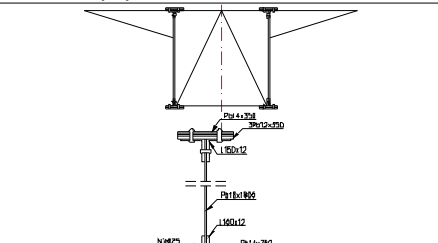

Bridge nr.	Constr. year	Span	Cross section type / main elements	Photo
4	1940	6,60 m		
5	1942	2 x 11,8 m		
8	1942	2 x 11,8 m		
9	1942	8,60 m		
13	1942	9,80 m + 26,90 m		
15	1942	19,00 m + 18,80 m		

Table 6. Analyzed plate girder bridges

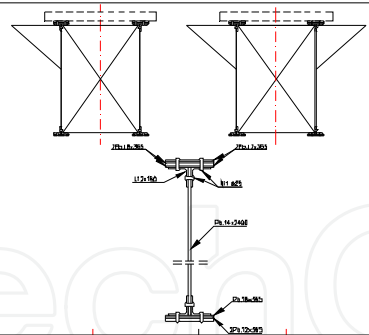

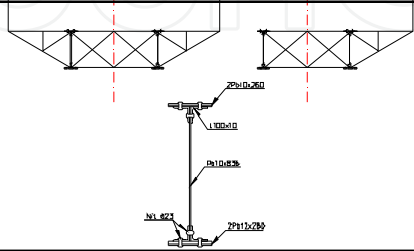

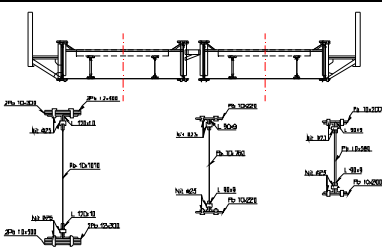

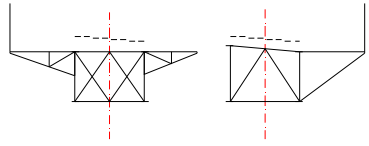

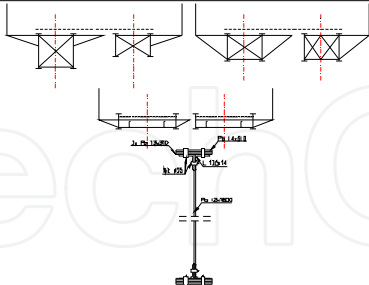

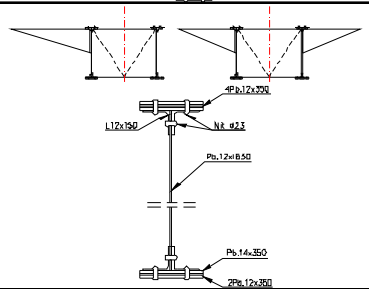

16	1942	26,90 m		
17	1880/ 1942	8,40 m		
20	1942	37,20 m		
22	1942	14,00 m		
24	1944	53,60 m		
25	1940- 1944	58,25 m		

Table 6. Analyzed plate girder bridges - continuation

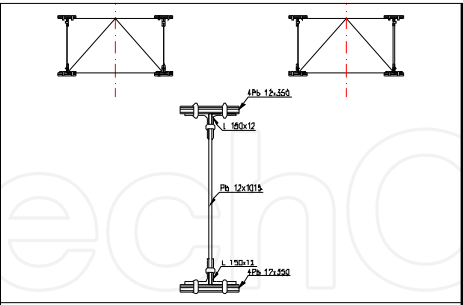

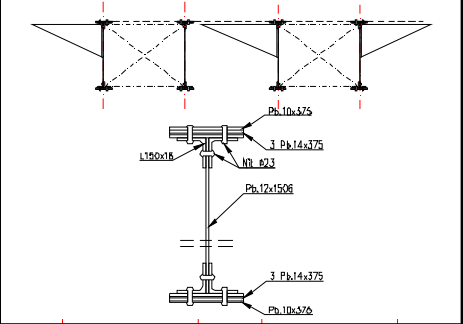

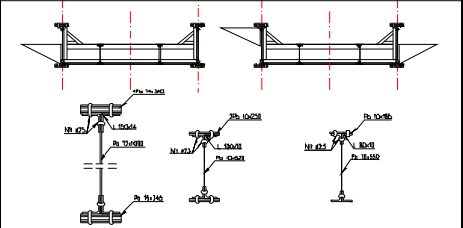

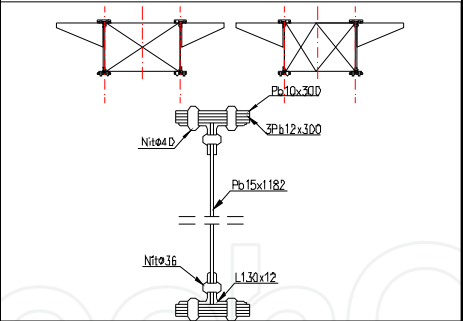

34	1966	14,20 m		
37	1960	23,00 m		
40	1942-1944	31,20 m		
41	1880/1942	16,00 m		

Table 6. Analyzed plate girder bridges - continuation

For the replacement different solutions can be proposed. A modern economical and robust structure is the Schmitt Stumpf Frühauf solution (VFT-WIB®), [12] – which has a high degree of prefabrication and goes along with a significant improvement of working conditions, as weather-independent working; also improved environment conditions for the workers while erecting formwork, placing re-bars and casting concrete is guaranteed. Furthermore a smaller amount of man-hours outdoors at the construction site is needed.

The constructive depth was optimized by a computer specialized program (Figure 35).

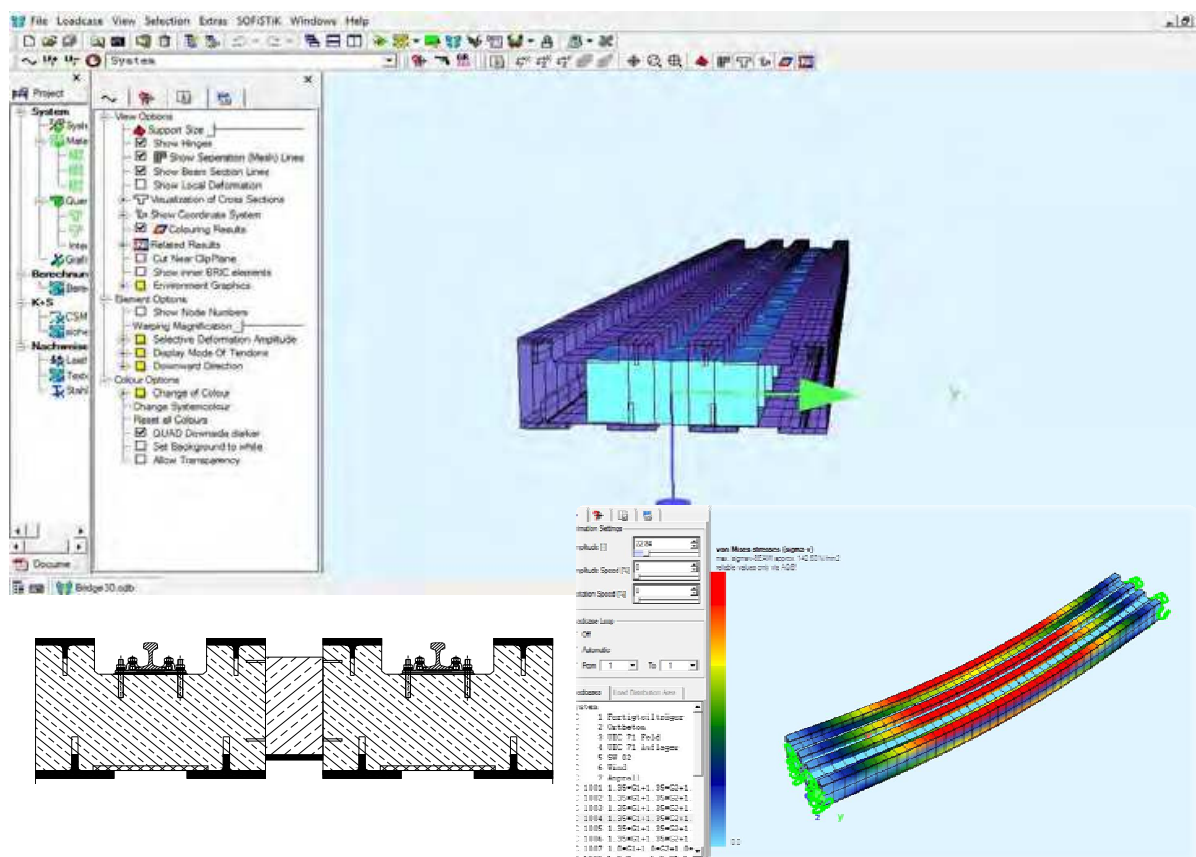


Fig. 35. Cross section of the new structure and calculation model

6. Conclusion

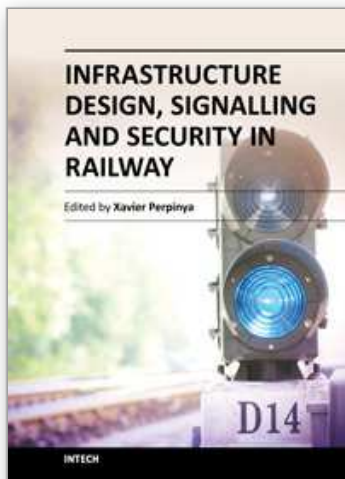
The progress recorded in the last decades allows on one part an accurate evaluation of the remaining safety of the structures and on the others part the proposal of new efficient, economical solutions with the high level sustainability.

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Railway transportation has become one of the main technological advances of our society. Since the first railway used to carry coal from a mine in Shropshire (England, 1600), a lot of efforts have been made to improve this transportation concept. One of its milestones was the invention and development of the steam locomotive, but commercial rail travels became practical two hundred years later. From these first attempts, railway infrastructures, signalling and security have evolved and become more complex than those performed in its earlier stages. This book will provide readers a comprehensive technical guide, covering these topics and presenting a brief overview of selected railway systems in the world. The objective of the book is to serve as a valuable reference for students, educators, scientists, faculty members, researchers, and engineers.

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