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STUDY CASE

Refurbishment of Existing Steel Structures – an Actual Problem

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Abstract

Many steel structures erected at the end of the XIX and begin of the XX-centuries still are in function. Some of these structures, particularly bridges, have already achieved an age of ninety, hundred or even more years and 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. Information about the safety of the structure, the remaining life, the costs for maintenance etc. are important. Nobody will take the responsibility for failure of a structure as a result of budget restriction. The aim of the paper is to emphasize the importance of refurbishment of existing steel structures, part of sustainable development.

Keywords

Existing steel Structures and Bridges, refurbishment, sustainable development, in situ tests, strengthening, eccentricities, aqueduct, cracks, remaining fatigue life

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1 Introduction: Refurbishment of existing steel structures as an integral part of sustainable development

The life of a structure is not unlimited. The refurbishment of existing steel structures and steel bridges is an integral part of the sustainable development. In the developed societies, as they progress, the feeling grows that it is necessary to maintain the existing architectural heritage. Rehabilitation of heritage buildings is a way of sustainable development and also an act of culture [1].

Sustainable development has evaluated as a concept through several decades of active international scientific debate and has acquired distinct political connotations in the context of globalization. Sustainability is the study of the concepts of sustainable development and environmental science. There is an additional focus on the present generations responsibility to regenerate, maintain and improve planetary resources for use by future generations. According to the Brundtland Report: "*Sustainable development is development that meets the needs of the present without compromising the ability of future generations to meet their own needs*" [2].

Existing structures are subjected to processes of degradation in time, which leads to a situation in which they became not able to fulfil the initial purpose. Very often, there is also the need to change and improve the conditions offered by the existing buildings or to adapt them to new functions (the case of disaffected industrial buildings).

Refurbishment of existing buildings with architectural value (enabling the delivery of modern facilities), may be less expensive than a new structure; the degree of refurbishment can vary, from simple repairing to changing the existent structure.

It is more economically to refurbish than to rebuild; this aspect is sustained generally by tax reductions.

Existing constructions can be transformed in green constructions that means in structures environmentally responsible and resource efficient throughout the life cycle, from siting to design, construction, operation, maintenance, rehabilitation and deconstruction.

Other constructive aspects of the refurbishment are:

- Positive socio – economic impact for the region which would be able to obtain the maximum benefit from the rehabilitation of the structures (old structures have generally also touristic value);
- Safety – the rehabilitation program will be conceived to improve the safety of the existing structure, including safety at all construction stages;
- Exemplary work sites and an environmental management system have to be considered.

2 General considerations in refurbishment

The estimation of the carrying capacity of existing structures is a complex matter. One of the most important aspects is the experience of the expert. In a first step the expert have to inspect carefully the structure and to make an estimation, based on simplified analysis methods and a preliminary evaluation about the technical condition of the structure. During the visual inspection the corrosion state, must be also evaluated. Figure 1 presents the main steps in the evaluation of the existing structures.

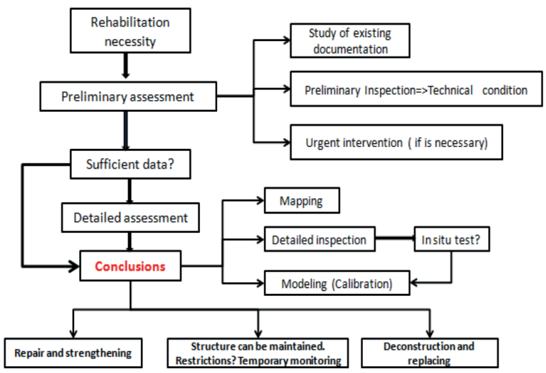


Fig. 1 Flowchart regarding the refurbishment of existing steel structures

The expert must see and inspect obligatory the structure; he can ask for some low tech NDT (Non Destructive Tests) tests, or even destructive ones in order to establish the material characteristics, like colour penetration test, magnetic particle inspection etc. An expert with experience can immediately see if in riveted bridges there are leaks (traces) of rust that means rivets are weak; in this area cracks are probably, but heavy to detect because they are generally covered. Usually riveting connections have a good behaviour in time.

Until the beginning of the XX century, the steel factories had own rules without a general standardization, resulting a large dispersion of steel characteristics; sometimes for the same structure it is possible to have different steel qualities.

In present in the technical literature, there are – in generally – sufficient data regarding the material qualities, in function of the year when the structure was put in function. In these direction the railway Administrations from Germany, Switzerland, Austria and Hungary, have performed 667 tests [3], on the material collected from existing structures. For wrought iron (puddle steel) and steel produced before 1900, the following values can be accepted:

Ultimate tensile strength fu = 320 … 380 N/mm²

Yielding stress $f_y = 220\ N/mm^2$ (survival probability of 95%)

Young modulus $E = 200\ 000\ N/mm^2$

For the partial safety factor the following values are prescribed: for wrought iron $\gamma_R = 1{,}2$ and $\gamma_R = 1{,}1$ for the old steels produced before 1900.

For steel grades after 1925 the following values are recommended:

Ultimate tensile strength $f_u = 370\ ...\ 460\ N/mm^2$

Yielding stress $f_y = 240\ N/mm^2$ (survival probability of 95%)

Young modulus E = 200 000 N/mm².

$\gamma_R = 1{,}1$

It is interesting to mention that, the former Romanian Standard for the Design of Railway Steel Bridges [4], recommend for existing structures produced after 1900, still in a satisfactory technical condition, the following values for the allowable stresses:

$\sigma_a^{\,I} = 150\ N/mm^2 \rightarrow$ mild steel

$\sigma_a^{\,I} = 140\ N/mm^2 \rightarrow$ wrought iron.

Destructive sampling shall be avoided because they are expensive and the load carrying capacity can be affected. Nevertheless if there are doubts about the material quality, tests are necessary! The most important aspect is to establish from the beginning if there is wrought iron or mild steel, especially for old bridges. The Romanian Railway Administration has replaced all the existing wrought iron bridges with new ones. Always an expert must chose the element from which sampling is possible (Fig. 2), without affecting the resistance and stability of the element (structure).

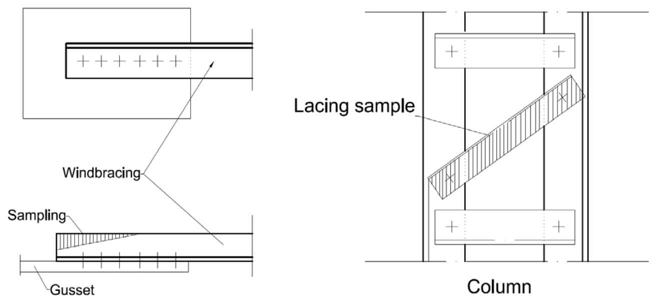


Fig. 2 Sampling from existing structures

In the XIX century for riveted structures (including bridges), wrought steel were used.

Chemical and metallographic analysis can identify wrought steel, also called puddle steel, showing their characteristic lamellar microstructure consisting of ferrite matrix and slag layers [5]. It must be mentioned that, old mild steel contain sometimes N – Nitrogen, which causes ageing effects by a temperature of 250°C, when by bad workmanship the snap-tool penetrates into the parent material. Aging has as result brittle cracks.

The first developments in welded connections for steel constructions (bridges) appeared in the '30 of the last century, replacing the traditional riveted connections. The first welded bridge in Romania was fabricated and erected in Resita in 1931. In the 1950's and 1960's, steel welded structures (bridges) fabricated in the plants under controlled conditions and erected on site by high strength bolts or welding, become the usual way in the construction of these structures [6].

Corrosion of existing steelwork may be one of the most important problems, especially for steel bridges (Fig.3).

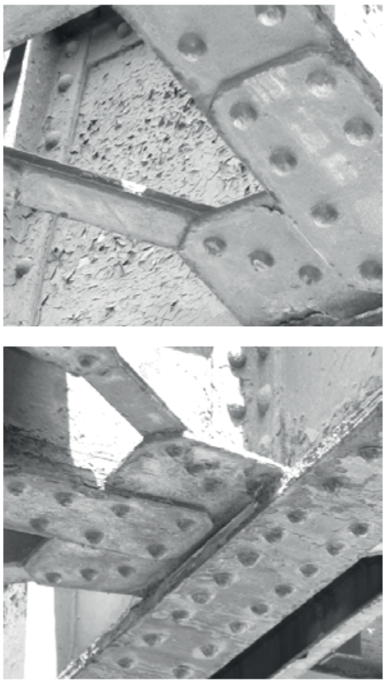


Fig. 3 Corrosion and inadequate painting

Once in 90 seconds a ton of steel is destroyed, 350 million Euro/year are allocated for steel structures maintenance. Every year about 40% of steel put in work is lost by corrosion.

Steel is an entirely recyclable material by re-melting it can be reused. In a first step, by the calculation of the carrying capacity of corroded steel structures, an overall decreasing of cross section with 10% can be accepted.

The surface must be prepared by wire brushing or scraping and for extended surfaces by abrasive (sand) blast cleaning.

Reduction of the dead load is generally possible; heavy floors (in building and bridges) can be replaced with lighter materials, like composite decks.

The load carrying capacity need to be assessed in all stages, including re-establishment and deconstruction; construction procedures are more critical in the different refurbishment phases than in new structure, especially by replacement of elements.

3 Main steps in refurbishment

Existing steel structures can be evaluated using the safety concept existing to the time of the structures erection [7], [8] – generally the safety concept of allowable stress. Nevertheless checking according to the Eurocodes is strongly recommended. For the majority of existing steel structures the documentation is missing (exception are the Railways, they have generally complete archives). In consequence the expert have to do some in situ measurements mapping the structure, which is not always easy, taking into account the accessibility on the site.

The next step is to perform simple stress verification based on usual calculus methods. These results corroborated with the technical condition of the structure, allows to take a decision; the structure can be used in continuation (even with some restrictions), the next evaluation step is necessary, or the structure must be disaffected immediately.

In the second stage a complete verification based on a spatial calculus model are usually performed. In function of the results some reinforcements can be done. It is important to mention that many existing structures are riveted; the reinforcement is not simple.

Generally the reinforcement of the structures <u>is not</u> recommended if [9]:

- the additional material is more than 40 % from the weight of the existing structure or 30 % of a new one;
- the rehabilitation cost is higher than the price of a new structure.

Exceptions are the historical structures, monuments of the engineering art; in this situation every case must be analysed separately.

It is important to emphasize that a refurbished structure is not a new one.

If the strengthening of the structure is realized without disabling completely the structure (usual case), the stresses in an element are presented in Fig. 4.

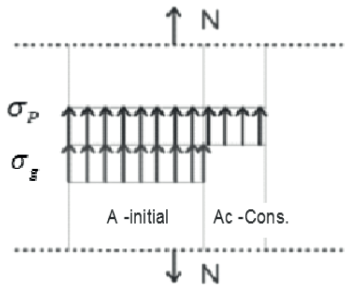


Fig. 4 Stresses in a strengthened tensioned element

For a tensioned member with the initial cross section A, strengthening material A_c, are added.

$$N = N_g + N_p \tag{1}$$

where, g –represents the dead load and p – the live load (both including the load coefficients). With

$$\sigma_g = \frac{N_g}{A} \quad \sigma_p = \frac{N_p}{A + A_c} \tag{2}$$

finally resulting:

$$\frac{N_g}{A} + \frac{N_p}{A + A_c} \leq R \tag{3}$$

Using the notation:

$$\sigma_{old} = \sigma_g + \sigma_p \frac{A_c}{A + A_c} \tag{4}$$

and

$$\sigma_{new} = \sigma_p \frac{A_c}{A + A_c} \tag{5}$$

where σ_{old} are the stresses taken by the existing (old) material and σ_{new} by the added (supplementary) material and with $n = A_c / A$ and $m = \sigma_g / \sigma_g$, finally we obtain the ratio

$$\alpha = \frac{\sigma_{new}}{\sigma_{old}} = \frac{n}{m(1-n)+1} \tag{6}$$

To explain the result, we can analyse two realistic situations:

- a railway bridge with n = 0.5 and m = 0.2; it results $\alpha = 0.385$
- a highway bridge with n = 0.5 and m = 0.5, it results $\alpha = 0.285$

In conclusion, even if we introduced 50% more new material in comparison with the old (existing) one, it takes only 38.5% in railway, respectively 28.5% in highway bridges compared to the old, existing material. Paradoxical, if n has a lower value (we add less new material), the strengthening is more effective. And obviously if the dead load stresses σ_g are lower than the live load ones, the consolidation is more effective! The same conclusions are valid for compression members.

Reinforcement can be done directly by adding of material (complex by riveted structures), or indirectly by changing the statically scheme (if it's possible), which is more efficient. For usual steel constructions which change their destination (e.g. industrial buildings becoming exhibitions, theatre halls or commercial buildings) the last solution combined with an adequate architectural conception can have as result spectacular and emblematic buildings.

By welded structures the first solution is easy to apply; however for old steels welding is not recommended.

For civil engineering steel structures the refurbishment can extend - by adequate maintenance - the service life for an unlimited period.

For bridges due to fatigue, the service life can be extended for limited period of 20–40 years, by evaluating the remaining fatigue life.

In situ tests of the structure are very relevant, especially for important structures and complicated statically schemes, but there are expensive and time consuming. In situ tests measuring stresses and deformations [10], are used often for existing steel bridges (Fig. 5).

(a) (b)

Fig. 5 In situ tests on bridges:(a)-highway bridge; (b)-railway bridge

Important data about the technical condition of the structure can be obtained and the calculus model can be calibrated (validated). The existence of a Romanian Standard – in this direction – can be mentioned [11]. In situ tests can remove the doubts about the safety of the structure. In Fig. 6, a Sports – Hall with 2000 places is presented; the owner asked for a test; in every joint of the roof lower chord, four bags with sand were hanged, realizing 1,2 of the design load. Stresses with strain gauges and deformations were measured. The general behaviour of the structure was satisfactory.

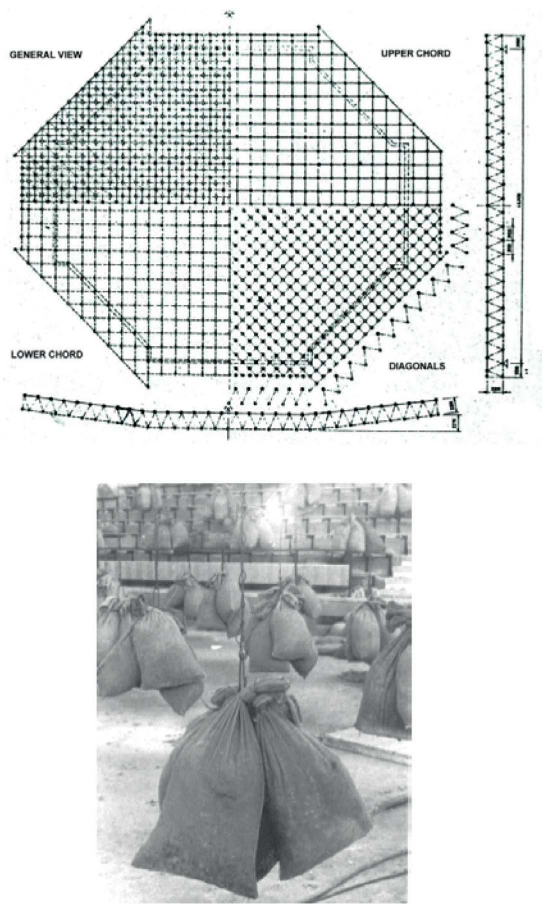


Fig. 6 In situ test of Sports Hall double layer with L = 60 m and B = 50 m.

In the case of existing bridges the fatigue assessment of the structure is difficult. The damage accumulation methodology can be adopted. By choosing the adequate Wöhler curve (for example in the case of riveted structure see [3], [5]) and reconsidering the traffic on structure, a stress history can be recovered. With both elements, the accumulated damage can be evaluated:

$$D = \sum \frac{n_i}{N_i} \leq 1 \tag{7}$$

It can be mentioned that the Swiss Standard SIA 161 [12], propose in this direction a very informative diagram for railway and highway bridges, which allows to reconsidering the accumulated damage in the past. Recommendable is to save the structure, only if the cumulative damage - taking into account the traffic in the past is maximum $D = 0,6 – 0,7$; in this situation the structure can be strengthened and the remaining fatigue life can be extended. This is generally the case of highway bridges, where heavy vehicles passed rarely in the past. Considering the present and the probable future traffic, the remaining fatigue life must be estimated. If $D > 0,7$ local strengthening can assure for a limited time, the safety of the bridge.

The general affirmation:" *...the bridge is old, consequently the structure is fatigued"*, is not correct.

A next step is the fracture mechanics FM approach [13] [14].

For old riveted bridges cracks usually underneath the head of a rivet can be often detected; by dismantling of old bridges, cracks with openings of a_0 = 2-3 mm, are relatively frequent. In welded structured cracks are easier to detect. With Fracture Mechanics considering the ductility characteristics of the material (constants C and m), the critical crack length a_{crit} can be determined, respectively the maximum number of load cycles, resulting the residual service life. The cumulated damage due to the traffic in the past (difficult to evaluate) is fulfilled by the assumption of the initial crack size [15],[16]. This method can determine also the interval between two inspections [17].

For existing structures problems not considered initially in design can influence the behaviour, like:

- Stresses and deformations in joints (secondary stresses due to the rigidity of joints);
- Restraint due to non-functional or corroded bearings (Fig 7);

Fig. 7 Non-functional bridge bearing

- Eccentricities;
- Deformations and stresses, caused by unforeseen interaction between longitudinal and transversal members due to high stiffness, or temperature. As an example, for a railway bridge with a large span (L > 100m), situated on a main railway line (Figure 8), in order to avoid interaction the main girder lower chord and stringers (important rigidity difference), the stringers are interrupted at L/3 where L is the span of the bridge [13].

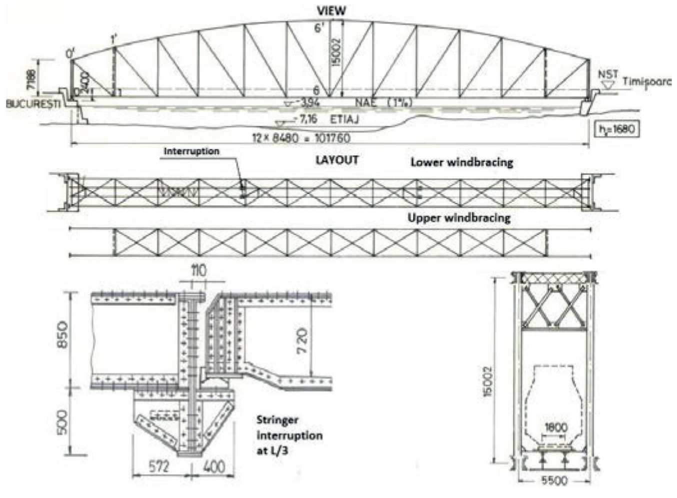


Fig. 8 Stringer interruption at L/3

- Unforeseen large traffic increase on the highway bridges and overload of structures by trailers with higher axle load than permitted (often recorded during bridge measurement).

A general repair can bring the structure to the initial performance (Fig. 9), or even better (in seismic areas).

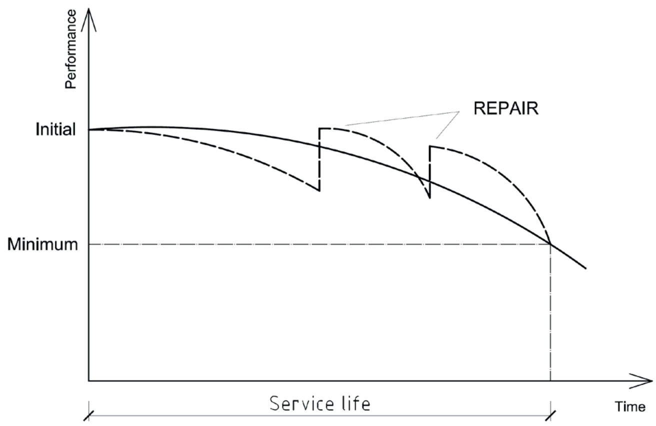


Fig. 9 Time performance of a structure

4 Case studies

In this paragraph three typical cases: an aqueduct, a roof truss girder and a railway bridge are presented.

The Siderurgic Group of enterprises of Reșița (founded in 1775) situated in the south of Banat is supplied with cooling water from a distance of 20 km with open channels. The channels are passing a valley by steel aqueducts. They have a trapezoidal form and are supported by two truss girders (H = 3100mm) with parallel chords and cross diagonals connected by floor beams (Fig.10). Aqueducts are supported by steel pyramidal columns having a slope of 2/1000 which insures free water flowing. By entering in the plant, water is directed by pipes into small hydroelectric station. The most important aqueducts is this from *Secu* with a total lengths L = 252m, with equal spans of 36,00 m and the piers with height of $H_{max}= 35$m. The construction was put into operation in 1911. During its lifetime, the maintenance was made every two years and general inspection every ten years. With the development of the production, the necessity to increase the quantity of cooling water appeared.

A study was performed and the conclusion was that it is possible to increase the debit from 4 m^3/sec. to 4,5 m^3/sec, which corresponded to an increasing of the live load from 26,8 kN/m to 30,0 kN/m. The solution consisted in raising the lateral walls with 30 cm (Fig. 11). With this occasion a general revision of the whole structure was made for truss girders, columns and foundations.

In first step tests on material were achieved in order to determine the mechanical properties (tensile strength and Charpy tests). The results showed that the material is mild steel similar to S 235.

The 3D model structure analysis was performed. The compressed diagonals situated in the bearings area, where strengthened. Also the diagonals from the middle of the spans, which initially were made of a single angle, were doubled. The deformations of the structure were measured in situ, in two situations: without and with water.

Additionally, the longitudinal slope and the deflection of the structure which remained in acceptable limits (L/400) were checked. In conclusion, the structure can be maintained in utilization without special measures. It is technical monument still in use (Fig. 12).

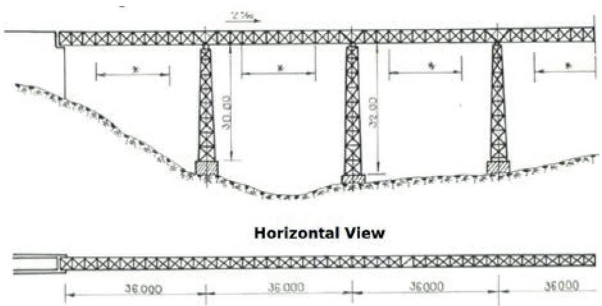


Fig. 10 Aqueduct Reșița, general view

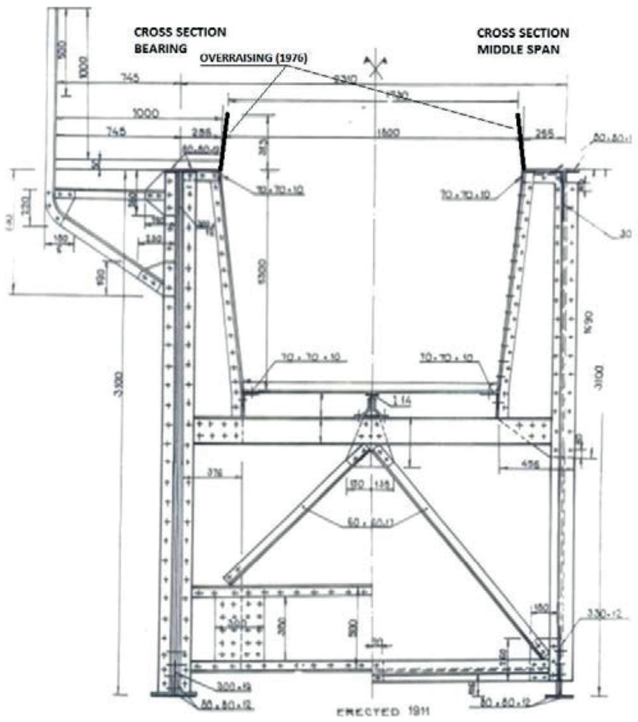


Fig. 11 Aqueduct Reșița, cross section

Finally a sustainable rehabilitation of the structure was realized with a reduced environmental impact.

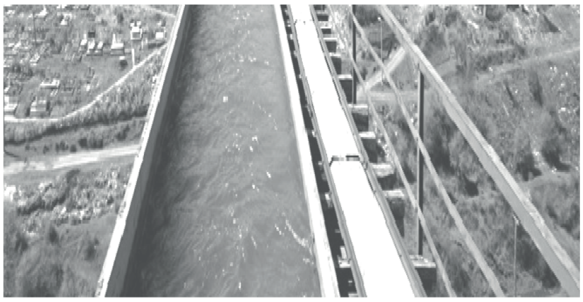
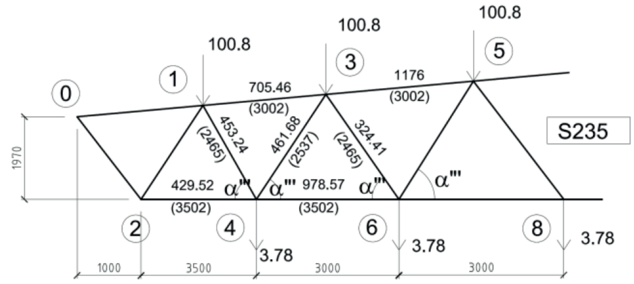


Fig. 12 Aqueduct Reșița, after refurbishment

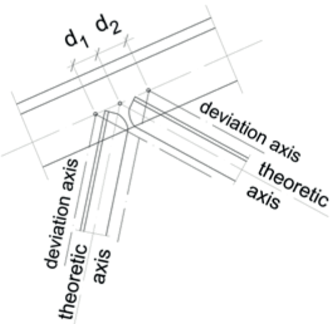
Roof truss girders for a single industrial building. In the case of eighteen roof truss girders, during the final control in the plant, eccentricities were detected, due to errors in fabrication. In the European Standards EN 1993-1-8-2011 is specified that "*where is eccentricity at intersections, the joints and member should be designed for resulting bending moments and forces*". It is obviously that the superposition of the stresses produced by the bending moments resulting from eccentricities with the stresses due to the rigidity of the joints is very unfavourable and cannot be accepted.

In figure 13 there are presented the truss beam geometry, elements cross section, forces and stresses. The deviations from the initial project d_1 and d_2 where measured for all the truss girders in each joint. In figure 14, the measured eccentricities for two joints 3 and 4, are given for all the eighteen trusses.



Element	Cross section	
		Area [cm²]
3-1	⊤	73.5
3-5	⊤	73.5
3-4	⊤⊤	38.4
3-6	⊤⊤	18.8
4-2	⊥	51.4
4-1	⊤⊤	24.6
4-6	⊥	51.4

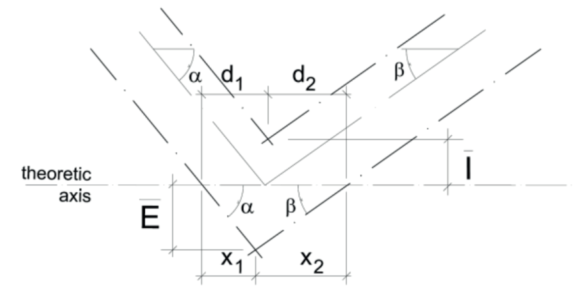
Fig. 13 Roof truss beam – elements cross sections, loads and stresses



Number	TRUSS		Eccentric d1 and d2				Eccentricity transformation			
			Joint							
			Joint 3		Joint 4		Joint 3		Joint 4	
			3-4	3-6	4-1	4-3	$\bar{I}$	$\bar{E}$	$\bar{I}$	$\bar{E}$
1	Truss Girder 1	+								3.33
		-			5					
2	Truss Girder 2	+				5				7
		-			5					
3	Truss Girder 3	+	10	15				3.33		
		-								
4	Truss Girder 4	+	20	10	5	20	6.66			10
		-								
5	Truss Girder 5	+			20	7		13	9	
		-	20							
6	Truss Girder 6	+	12			8	25			5.33
		-		25						
7	Truss Girder 7	+				5				7
		-			5					
8	Truss Girder 8	+	5				10			13
		-		10	20					
9	Truss Girder 9	+								
		-								
10	Truss Girder 10	+			10	8			1.33	
		-								
11	Truss Girder 11	+							17	
		-			10	35				
12	Truss Girder 12	+								
		-								
13	Truss Girder 13	+			5	40	7			23
		-		10						
14	Truss Girder 14	+	10		35		7		30	
		-				10				
15	Truss Girder 15	+	12	12		5				10
		-			10					
16	Truss Girder 16	+					5.33			
		-		8						
17	Truss Girder 17	+	25				27			
		-		15	23	23				
18	Truss Girder 18	+			15	15				
		-								

Fig. 14 Roof truss beam – in situ measured eccetricities

In order to systematize the calculus, the eccentricities d_1 and d_2 (figure 14) are transformed in the values $\bar{I}$ and $\bar{E}$ (figure 15).



ctgα=x₁/E ; ctgβ=x₂/E => ctgα+ctgβ=(x₁+x₂)/E

$x_1+x_2=d_1+d_2$ => $\bar{E}=(d_1+d_2)/(\text{ctg}\alpha+\text{ctg}\beta)$; identically for $\bar{I}$

Fig.15 Eccentricities d_1, d_2 and transformation in $\bar{I}$ and $\bar{E}$

With the transformed eccentricities $\bar{I}$ and $\bar{E}$, the bending moments $M_{\bar{I}}$ and $M_{\bar{E}}$ can be determined (figure 15).

Every member in the joint m, will take a bending moment, proportional to its rigidity:

$$M_i = \frac{I_i / l_i}{\sum \frac{I_i}{l_i}} \quad M_i = k_i M \tag{8}$$

where $M = M_{\bar{I}}$ or $M_{\bar{E}}$

and $M_{\bar{I}} = \Delta S \cdot \bar{I}$ (9)

or $M_{\bar{E}} = \Delta S \cdot \bar{E}$ (10)

$\Delta S = S_{ml} - S_{mk}$ (11)

and k_i is the rigidity of the member i in the joint m.

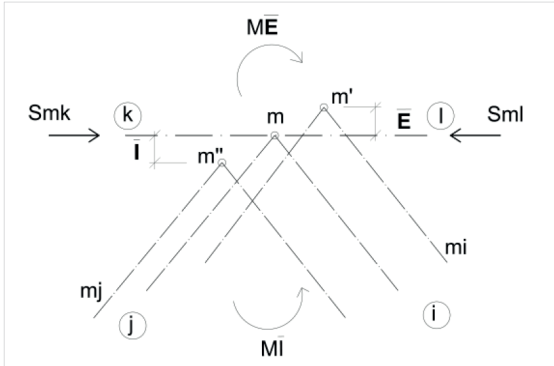


Fig. 16 Bending moments produced by the eccentricities $\bar{I}$ and $\bar{E}$

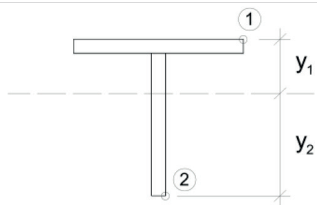
The supplementary stresses due to eccentricities, in the element I, joint m, are

$$\sigma_i = \frac{M_i}{W_i} \tag{12}$$

The carrying capacity reserve, for each member is given by

$$\Delta\sigma = f_y - \sigma_{eff} \tag{13}$$

where σ_{eff} is the effective stress in the analysed element and f_y is the yielding stress of the steel.



$$W_1 = \frac{I_z}{y_1}; \quad W_2 = \frac{I_z}{y_2}$$

Fig. 17 Elastic section modulus

Finally the stresses in the element i are:

$$\frac{k_i \cdot \Delta S \cdot \bar{I}}{W_i} \le \Delta\sigma \tag{14}$$

where Wi is the elastic modulus of the section (figure 17).

From the equation (14) the allowable eccentricity $\bar{I}_a$ can be obtained:

$$\bar{I}_a = \frac{\Delta\sigma \cdot I_z}{k_i \cdot \Delta S \cdot y_i} \tag{15}$$

Similar, the value of $\bar{E}_a$ can be calculated.

Comparing these admissible values for $\bar{I}_a$ and $\bar{E}_a$ with the values of eccentricities measured in situ, it can be establish the elements for which the stresses exceed the limit and for which are needed strengthening. For a better understanding a tabular form of the calculus is presented in figure 18.

It is underlined that the given algorithm can present an easy way to automatize the calculation.

Number	Element	Cross Section	Length [cm]	Ix [cm²]	Ix / l	k	Force [KN]	Stress σ [KN/cm²]	Allowable ecc. [cm] $\bar{I}a$	Allowable ecc. [cm] $\bar{E}a$
1	3-1	T 300 12 250 15	300.2	5105	17.01	0.476	-705.46	11.77	11.9	30.8
2	3-4	2L 100x100x10	253.7	354	1.40	0.039	-461.68	17.71	26.8	10.5
3	3-6	2L 70x70x7	246.5	84.8	0.344	0.0096	+324.41	17.26	41.6	16.3
4	3-5	T 300 12 250 15	300.2	5105	17.01	0.476	-1177.16	18.63	5.86	15.22
				Σ	35.75	Σ k=1.00				

Fig. 18 Calculation of the allowable eccentricities in joint 3

Example - JOINT 3

$$DS = S_{35} - S_{31} = 1176 - 705.46 = 470.54 kN$$

For element 3-5, $\Delta\sigma = 23,5-18.63=4,87\ KN/cm^2$

According with (15),

$$\bar{I}_a = \frac{\Delta\sigma \cdot I_z}{k \cdot \Delta S \cdot y_1} = 5,86\ cm\ ;\ \bar{E}_a = \frac{\Delta\sigma \cdot I_z}{k \cdot \Delta S \cdot y_2} = 15,22\ cm$$

Where $I_z / y_1 = W_1$ and $I_z / y_2 = W_2$ (figure 18)

$$DS = S_{46} + S_{42} = 978.57 - 429.52 = 549.15 KN$$

The two values $\bar{I}_a$ and $\bar{E}_a$ permit to establish very easy the elements where the stresses exceed f_y and which need strengthening.

Some strengthening proposals are presented in figure 19. Considering the bending moment distribution to other joints, the length of the strengthening elements can be equal to *l/3*. Regarding the welds, a_{min} is satisfactory.

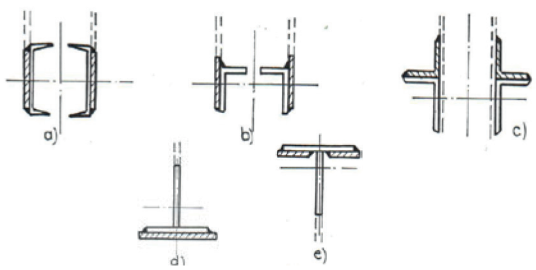


Fig. 19 Strengthening proposals

In conclusion, using the calculated allowable eccentricities, the decision which element must be strengthened, can be taken easily and directly.

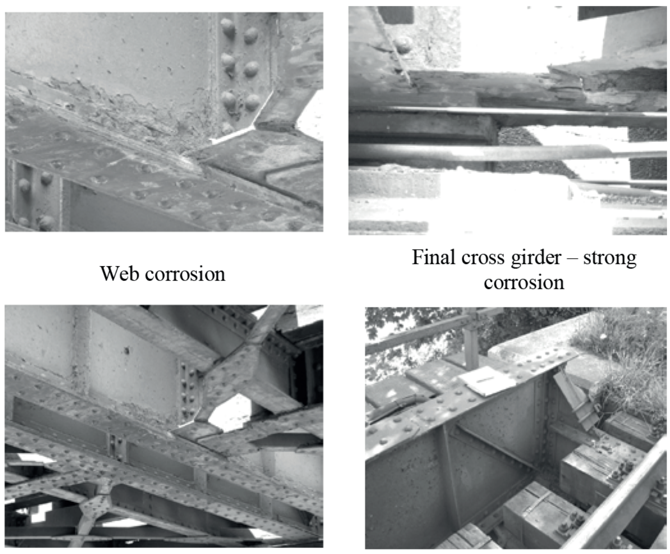
Existing railway bridge. The third example refers to a railway bridge situated on the main railway line from Bucharest to Brașov (figure 20). The underpass is situated in the centre of the Sinaia town, a well-known touristic zone. The structure is relatively new and was built in 1940, during the doubling of the line [18].

Fig. 20 Railway underpass; span L = 8,60 m.

The free height under the bridge is only 4,00 m. The superstructure has a classical composition: two main plate girders (situated to a distance of 3400 mm), stringers and cross girders and a general wind bracing. The wooden track ties are supported by the stringers placed by a distance of 1800 mm).

It is interesting to remark that the structure was designed for the heavy "N–convoy", according to the German rules from 1923.

The technical condition of the bridge is bad, the maintenance was neglected (figure 21).



Web corrosion

Final cross girder – strong corrosion

General corrosion

Deformed element

Fig. 21 Present technical condition of the structure

Due to the lack of documentation, the elements were measured on the site. A usual mild steel S 235 was considered. In figure 22 is presented the cross section of the bridge and the main riveted plate girder.

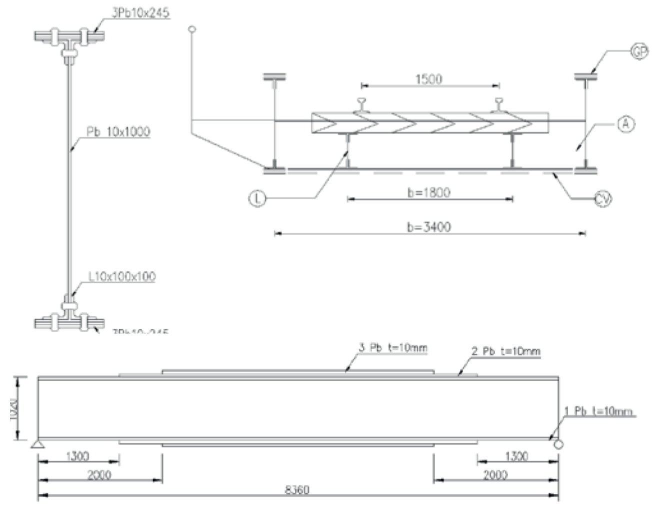


Fig. 22 Cross section of the bridge with riveted main girder

The assessment of the static strengths carried out with usual simple methods [20] and designed for UIC 71 convoy, had as result:

$$\sigma = \frac{M_{\max}}{W} = \frac{166929,6x10^2}{9504} \cong 17,56\frac{kN}{cm^2} < R \qquad (16)$$

More relevant is the fatigue assessment of the structure; it was carried out in accordance with:

a) The Romanian Standard for Steel Bridges SR 1911-98 [19].

The real traffic according to the Romanian Railway Company for the main line Bucharest – Brasov was 13,7 thousand tonnes/line/year (by reference traffic of 24 thousand tonnes/line/year). According with this standard the fatigue resistance is:

$$R_\sigma = \frac{\sigma_g + \frac{1}{\Phi}\cdot\psi\cdot\sigma_{\min T8.5}}{\sigma_g + \frac{1}{\Phi}\cdot\psi\cdot\sigma_{\max T8.5}} \qquad (17)$$

where σ_g → stresses due to the dead load and

$\sigma_{T8,5}$ → are stresses due to the railway convoy

ϕ - takes into account the traffic:

Ψ→ dynamic coefficient.

$$\Phi = \Phi_1 \cdot \Phi_2 \cdot \Phi_3 \rightarrow \Phi = 1,09x1,0x1,0 = 1,09$$

The fatigue resistance [20] is

$$R_\sigma^{L/2} = \frac{98,5}{98,5 + \frac{1}{1,09}x1,48x782} = 0,048$$

$$\Delta\sigma_{Ra} = 957,5\frac{daN}{cm^2}$$

$$\Delta\sigma^{L/2} = \frac{1}{1,09}x1,48x782 = 1062\frac{daN}{cm^2} >> 957,5\frac{daN}{cm^2}$$

$$\Rightarrow not \rightarrow fulfilled$$

For the evaluation of the residual service life, they were no data about the traffic in past on the structure.

b) In a second step the Swiss rules SIA 161 [3] were used. The equivalent stress range

$$\Delta\sigma_e \le \frac{\Delta\sigma_c}{\gamma_{fat}} \qquad (18)$$

where $\Delta\sigma_e = \alpha\Delta\sigma\ (Q_{fat})$ and $\Delta\sigma_C$ → is the fatigue resistance (Wöhler).

The value of $\Delta\sigma_e$ results in function of the load coefficient α given in the Swiss rules, taking into account the traffic in past for different categories of traffic (heavy, usual and light).

Finally with $\gamma_{fat} = 1,0$ and $\psi = \frac{1,44}{\sqrt{L_\Phi} - 0,2} + 0,82 = 1,33$

and $M_{\max\max}^{UIC71} \cong 816kNm$ for one girder, $\Delta\sigma\ (Q_{fat}) = \sigma_{max} - \sigma_{min}$

$$\Delta\sigma(Q_{fat}) = \frac{1,33 \times 816 \times 10^2}{9504} = 11,45kN/cm^2;$$

$$\Delta\sigma_c = 7,0kN/cm^2$$

In conclusion:

$$\alpha\ \Delta\sigma = 0,98x114,5 = 112,3N/mm^2 >> 71N/mm^2$$

→Not satisfied.

It is important to mention that the Swiss rules have in this case only an informative character.

Fatigue verification according DS 805-2002

For the estimation of the residual fatigue life, the following steps are necessary:

Calculation of the relevant fatigue coefficient

$$\beta_{D,UIC} = \frac{adm\Delta\sigma_{Be,\kappa}}{\Phi \cdot \max\Delta\sigma_{UIC}} \qquad (19)$$

where: Φ - dynamic coefficient

$$\varphi = \frac{1,44}{\sqrt{l_\Phi} - 0,2} + 0,82 = 1,3343$$

$\max\Delta\sigma_{UIC}$ – stress range for UIC 71

$$\max\Delta\sigma_{UIC} = \frac{81600}{9504} = 8,59kN/cm^2$$

adm $\Delta\sigma_{Be,\kappa}$ - according to DS 804, Table 4 and 5, with

$$\kappa = \frac{\sigma_g}{\sigma_g + \Phi_{\max}\sigma_{UIC}} = \frac{0,985}{0,985 + 1,3343x8,59} = 0,0793$$

→ Constructive detail W II (DS 805)

$+0,0793 \Rightarrow Ds_{Be,k} = 72,57\ N/mm^2$

$$\rightarrow \quad \beta_{D,UIC} = \beta_{D,UIC} = \frac{72,57}{1,3343x85,9} = 0,635$$

Total cumulative damage beginning with year 1876

$$D_{Past,1876} = \alpha\left(\frac{1}{\beta_{D,UIC}}\right)^5 = 0,15\left(\frac{1}{0,635}\right)^5 = 1,45$$

Total cumulated damage in the past

$$D_{past} = \rho_1 \cdot \rho_2 \cdot \rho_3 \cdot \rho_4 \cdot D_{past\ 1876} \quad (20)$$

where:

ρ_1 – adjustment factor $D_{Past,1876}$ in function of the construction year of the structure $\rho_1 = 0{,}85$ (year 1942)

ρ_2 – adjustment factor taking into account the real traffic $\rho_2 = 0{,}64$

ρ_3 – adjustment factor considering the number of lines on the structure $(\rho_3 = 1{,}0)$

ρ_4 – adjustment factor taking into account the allowable speed $\rho_4 = 0{,}927$ (for $v < 70km/h$)

$D_{Past} = 0{,}85 \times 0{,}64 \times 1{,}0 \times 0{,}927 \times 1{,}45 = 0{,}731$

Estimation of the remaining fatigue life

$$R = \frac{1 - D_{Past}}{0{,}01 + D_{Future}} - A \leq 50\,years \quad (21)$$

The final result is ***– 5,5** years.*

In the same way the remaining fatigue life was determinate for the cross girder and for the stringer. For both elements the fatigue life is consumed.

The structure was transported in the steel plant and carefully analysed (Fig. 23); cracks and defects were detected confirming the above fatigue calculations (Fig. 24 and 25). The bridge is not too old (approximately 70 years), but due to the lack of maintenance and some defects, the safety of the bridge is not assured.

Fig. 23 The structure analyzed in the plant

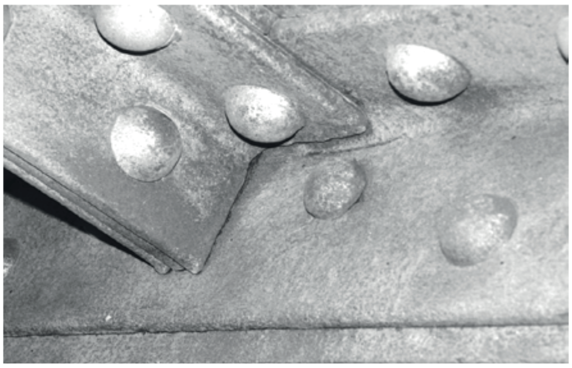


Fig. 24 Crack detected during the visual control

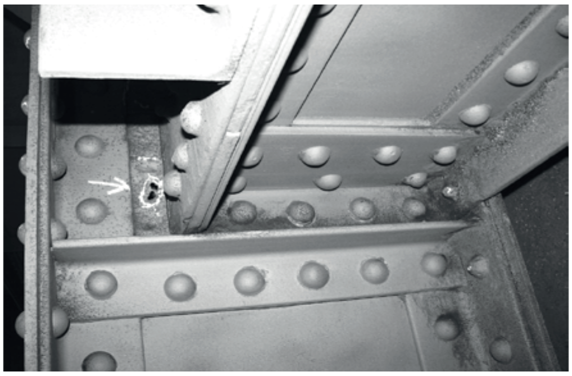


Fig. 25 Cracks and holes detected after cleaning during the visual control

It is important to mention that, even if the statically conditions are fulfilled (Eq. 16), the fatigue is decisive in this case. Finally the bridge was replaced with a new one.

5 Conclusions

On European level there is a tendency to extend by different measures the service life of the existent steel structures and to postpone investments by lack of funding. A special attention must be paid to old historical constructions. The affirmation *"take no measure for the moment"* cannot be accepted. A carefully cost-benefit-analysis have to be performed together with a refurbishment program of the structures. Rehabilitation of heritage steel structures is a way of sustainable development and also an act of culture in the environment.

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