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A FIRST LEVEL STRUCTURAL ANALYSYS TOOL FOR THE SPANISH RAILWAYS MASONRY ARCH BRIDGES

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Abstract. *This paper presents a first level analysis tool developed to aid the assessment of the Spanish railway bridges net –RENFE–. Explicit formulae are proposed, which yield ultimate point and uniform loads in terms of simple geometrical parameters and material strength. Formulae are derived from a parametric study carried over some hundred of prototypical bridges. Limitations to applicability are based on the ranges of such a study. Some conclusions can be outlined about safety level of empirically designed bridges of the XIX century. The tool has the advantage of its simplicity and is intended to be used in the context of management and maintenance of a large number of railway bridges, in which technical authorities often have the need of quick preliminary structural evaluation.*

Results of proposed formulae are compared with those of a well known rigid block analysis software showing acceptable agreement. Finally, recommendations are given for the use of this tool, as a means of discriminating well conditioned bridges from those which may require a more detailed inspection and evaluation, sometimes leading to strengthening or repair. Limitations and complementary verifications for the method are also reminded.

1 INTRODUCTION

In the re-classification process of the European railway lines[1] for accommodating higher axle loads, train speeds and an increased volume of containerised road traffic, it is necessary to re-assess old masonry arch bridges from a new technical point of view and with the help of the newly available structural analysis methods. In this context, it is necessary to have specific assessment tools in accordance with the peculiar structural behaviour of these bridges, the little data available and its reliability.

Current condition of masonry bridges varies from good to very bad, but statistics have identified a relatively large number of bridges in a medium and bad condition and a tendency for accelerated deterioration. In contrast to this general tendency, masonry arch bridges have proven durability. Their life-cycle costs seem to be significantly more economical than for the majority of other structure types. In addition, many of them belong to the architectural heritage of the railways, their substitution and refurbishment therefore should be considered very carefully. Fortunately it has become a generally accepted view that maintenance strategies should promote solutions that concentrate on preservation and restoration of these structures instead of their replacement.

The Spanish railways, RENFE, is involved in different studies trying to develop an assessment methodology for masonry bridges, including visual inspection and destructive testing, structural and material analysis, repair techniques, development of specific structural analysis tools for different level of accuracy, etc. In this tasks, collaborate the University and engineering consultants.

2 FIRST LEVEL ANALYSIS IN THE CONTEXT OF THE ASSESSMENT IN A HUGE NETWORK

The total number of RENFE masonry bridges (span ≥ 2.0 m) is 3,144. It approximately corresponds to 50% of the total. Annual maintenance cost represents 0.3 % of the replacement value of the stock[2]. Maintenance and inspection strategy implemented in RENFE is outlined in figure 1. Assessment is carried in two stages, at the end of which a decision has to be taken about the future use of the bridge and the actions needed to make it possible.

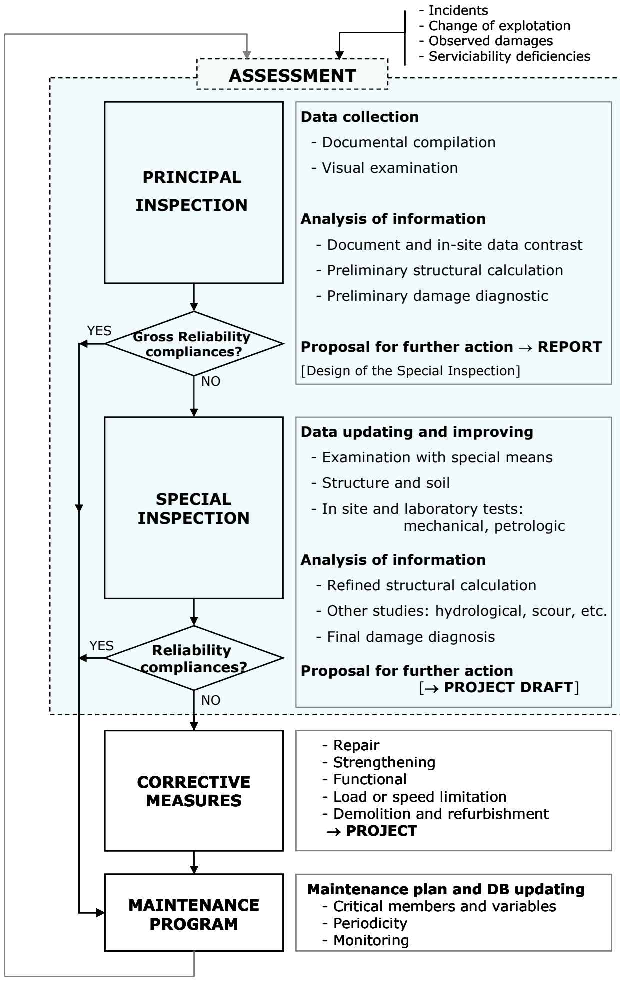

Figure 1: *Maintenance scheme for railway masonry*

Assessment may be carried out as a part of the maintenance program or as a consequence of special circumstances, such as incidents, observed damages or the need for a temporal or permanent change of exploitation conditions.

First stage (*Principal Inspection*) deals with the tasks of the anamnesis of the structure: compiling information about the construction of the bridge, interventions and other incidents, and the present state of the structure after a visual examination. As part of the analysis, preliminary damage diagnosis and structural calculation have to be made. If there are doubts – with the information available– about bridge safety, data is to be improved and updated by several means, which should be defined and valuated in the inspection report. If, even through the unavoidable uncertainties of the data and methods used, reliability (safety + serviceability + durability) of the bridge can be assured, then the 2^{nd} stage can be omitted.

Second stage improves and updates information as to feed a more complex analysis process to confirm reliability compliances. The result can be positive and no measures have to be taken. On the other hand, if the bridge undergoes damages with structural significance, or in the absence of severe damages, if the carrying capacity is below a safety minimum–as compared with traffic loads–, then remedial measures have to be studied. This study has the form of a project draft with technical, economical and service interference evaluation of the different possible alternatives. These can be load or speed limitation (temporarily only), repair (structural or durable damages), strengthening (insufficient carrying capacity) or complete substitution.

Once decision is taken, a detailed design of remedial measures is redacted and executed. In any case, the management database is updated with provisions for future inspection.

3 DEVELOPMENT OF A FIRST LEVEL STRUCTURAL ANALYSYS TOOL

Every technician familiar with existing structures maintenance and assessment knows that a specific case may be difficult and radically different from others. But it is also true that, when managing hundreds of structures, many cases are similar one each other and *simple*. Structural calculation may help in discriminating problematic cases from well conditioned ones. This is the scope of the 1st level analysis tool herein presented. It consists of closed-form expressions for the carrying capacity of a bridge (maximum point load and maximum distributed load) in terms of the basic geometrical parameters. The method is a multi-variable interpolation of a big number of calculations performed over a variety of structures. Details of calculations and hypothesis are explained below.

3.1 Range of application

A simplified tool must be restrained by rigid conditions to delimitate the field of application. It does not necessary means that the number of cases is strongly restricted, as practice shows. The limitations come from the parametric limits established in the parametric study[3] and are the following:

1. Plan geometry is straight and rectilinear (not skewed and not curved)
2. Vault bond is single-header (not multi-ring)
3. Consecutive spans are regular (equal to each other)

4. Abutments are undeformable
5. The bridge does not sustain severe structural damages
6. Spandrels are solid (not hollow)
7. Span is in the range $2.0 \leq L \leq 20.0$ [m]
8. Minimum rise to span ratio is: $f/L \geq 1/6$
9. Vault depth at crown to span ratio c/*L* is limited to the values given in table 1:

Table 1: *c/L limitation for different span ranges*

L [m]	2.0 – 5.0	5.0 – 7.5	7.5 – 10.0	10.0 – 15.0	15.0 – 20.0
c/L ≥	0.10	0.09	0.07	0.06	0.05

10. Masonry backfill is present with a height above extrados springers varying linearly with rise-to-span ratio: $h_{backfill} = 0.60 \cdot f$ for deep vaults (f/L=1/2), and $h_{backfill} = 0.30 \cdot f$ for shallow vaults (f/L=1/6)
11. Spandrel high at crown must be in the range: $0.25 \leq h_0 \leq 2.00$ [m]
12. Maximum pier height is 10 m.
13. Pier depth (at springing) to span ratio is limited to $b_p/L \geq 1/6$.

3.2 Calculation of the ultimate point load (axle) in masonry arch bridges by rigid block analysis y plastic analysis

Rigid block and plastic analysis were run over prototype-structures taking into account mechanism failure (hinges and shear) involving one and two vaults[3]. Limitations 12 and 13 were made *a posteriori* in order to discard multi-arch (two vaults) failures. The point load was applied in different positions from *L*/5 to *L*/2 (figure 2). The effective vault width is conservatively taken as 3.00 m. Load dispersion through the fill was taken under a limiting angle of 30º. Active and passive pressures were considered at the vicinity of springings. Dead loads were accounted for and factorized by 1.00.

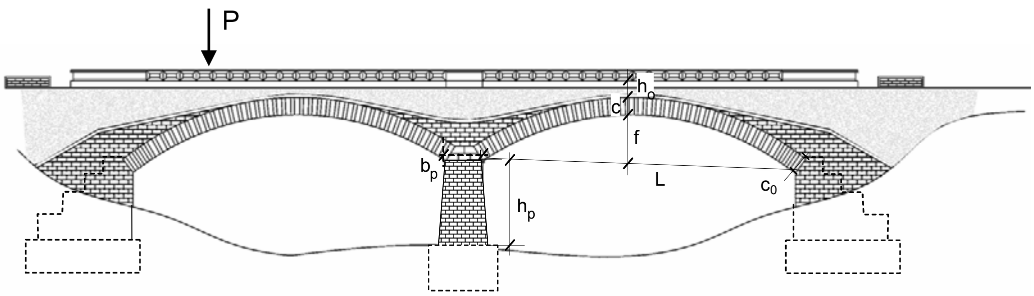

Figure 2: *Calculation sketch for ultimate point load*

The study was conducted over 790 prototype-structures selected from the possible variations of the parameters shown in table 2. No account was made for the influence of the masonry crushing strength and failures associated to the material. This topic is treated under uniform load in 3.3.

Table 2: *Parameters investigated for the determination of the point load*

L[m]	c/L	f/L	h_o[m]	h_p[m]	b_p/L	μ	γ [kN/m^3]	γ_{fill} [kN/m^3]
5.00	0.10	1/2	0,25	2.0	1/4	0.60	20.0	18.0
7.50	0.09	1/4	0.50	5.0	1/6	0.80		
10.00	0.07	1/6	2.00	10.0	1/8			
12.50	0.06							
15.00	0.05							
17.50								
20.00								

3.3 Calculation of the maximum uniform live load in masonry arch bridges

Non linear uniaxial analysis using SOFISTIK® according to the methodology proposed in Martin-Caro[3] were run over prototype-structures taking into account crushing failure of masonry involving one vault. The uniform load was extended to the hole length (q1, fig. 2) of the bridge or just to one half (q2, fig. 3). Active and passive pressures were considered at the vicinity of springings. Dead loads were accounted for and factorized by 1.35 and 1.00.

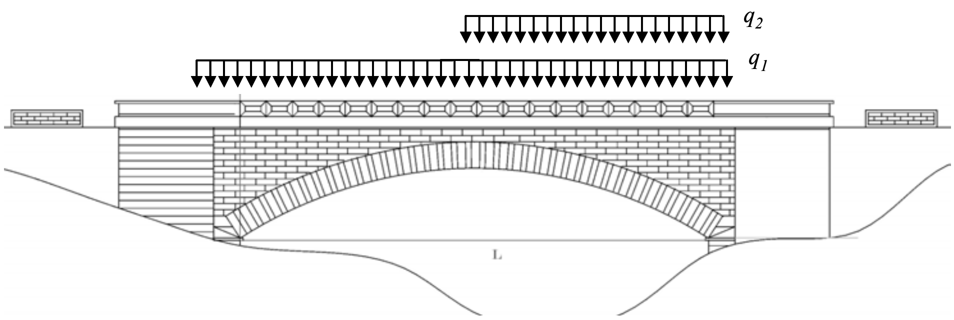

Figure 3: *Calculation sketch for ultimate uniform load*

A total of 96 calculations were run varying the geometric parameters and the crushing strength f_c as shown in table 3. Other issues varied were position of live load and factoring of dead loads.

Table 3: *Parameters investigated for the determination of the live*

L[m]	f/L	ho[m]	f_c [N/mm²]	γ [kN/m^2]
5,0	1 / 2	0,40	4,0	20,0
10,0	1 / 6		6,0	
20,0			8,0	
			10,0	

Conservatively, section depth at crown was considered to be the least one for each span value as given in table 1, i.e.: c/L= 0.10 for L=5.00 m, c/L= 0.07 for L=10.00 m and c/L= 0.05 for L=20.00 m.

3.4 Development of explicit formulae for ultimate loads

Point Loads P_{ult}. In principle, a multivariable regression analysis is needed to obtain explicit formulae. Instead of that, a simplified approach is taken, directly uncoupling some variables. First, formulae are derived for different values of c/L within results obtained for deep vaults (f/L=0.5). Then correction factors are added to take into account influence of rise-to-span ratio and height of fill at crown h_o [m]. As it will be seen in 3.5, sufficient precision is achieved by this procedure. Expressions (1), (2) and (3) and provide the ultimate point load P_{ult} [kN] on the basis of and effective width of 3.00 m (conservatively deduced taking into account the rail, crossbeam dimensions and fill height).

$$P_{ult} = (AL^2 + BL + C)K_1 K_2 \qquad (L \text{ in [m]}, P_{ult} \text{ in [kN]}) \tag{1}$$

A, B and C are coefficients depending on depth-to-span ratio, c/L, given in table 4.
K_1 is a coefficient depending on rise-to-span ratio f/L, expression (2).
K_2 is a coefficient depending on height of fill at crown h_o, expression (3).

Table 4: *Parameters A, B and C for expression (1)*

c/L	*A*	*B*	*C*
0.10	24.432	15.796	792
0.09	29.891	-183.15	1092
0.07	20.000	-190.00	1020
0.06	14.927	-173.30	1095
0.05	3.3036	7.4106	345

$$K_1 = \begin{cases} 1 - 0.05(\frac{L}{f} - 4)^2 & \text{if } \frac{f}{L} \leq 0.25 \text{ and } \frac{h_p}{L} \geq \frac{1}{2} \\ 1 + 0.03(\frac{L}{f} - 2)^2 & \text{else} \end{cases} \tag{2}$$

$$K_2 = 1 + \left(\frac{h_0}{2.50}\right)^2 \qquad h_0 \text{ in [m]} \tag{3}$$

Figure 4 shows the values of P_{ult} vs L for single deep and shallow vaults according to 1 (γ=1.0; h_0=0.50) together with the curves of prototype bridges designed in agreement with empirical rules after Croizette-Desnoyers and Sejourné for railway bridges. If P_{ult} is to be compared with the maximum *real* point load per axle (typically in the order of 300 kN), these *historically engineered* bridges exhibit a safety global coefficient not less than 4 for deep vaults and in the order of 10 for shallow vaults. The minimum safety coefficient is obtained for spans in the range 10 – 15 m. It is interesting to note that, while empirical rules of the XIX century propose bigger depth values for shallow arches (span being equal), ultimate point load is bigger for these type of arches (span and depth supposed equal) than for deep ones, except for cases with slender piers, as show K_1 values.

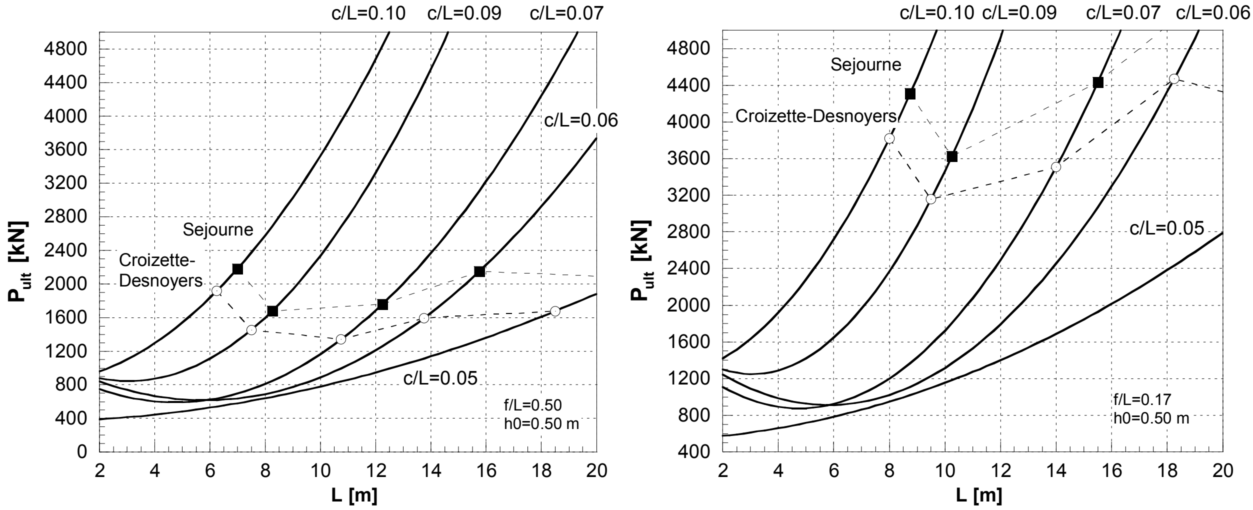

Figure 4: *Ultimate point load vs span for single vaults and results for empirically designed bridges (Croizette-Desnoyers and Sejourné). Left, deep arches (f/L=1/2); right, shallow arches (f/L=1/6)*

Uniform Loads q_{ult}. Results show a nearly linear dependency of q_{ult} on crushing strength f_c. Half span loading (q_2) is determinant and higher strengths. Typically, most unfavorable dead load factor is 1.35 for q_1 and 1.00 for q_2. Table 5 summarizes results.

Table 5: Ultimate uniform load values [kN/m] on the basis of an effective ault width of 3.00 m

		L=5.00 m		*L*=10.00 m		*L*=20.00 m	
fc [N/mm²]	q_{ult} [kN/m]	*f/L*=1/2	*f/L*=1/6	*f/L*=1/2	*f/L*=1/6	*f/L*=1/2	*f/L*=1/6
4.0	q1	**1197**	**1097**	**797**	**721**	**721**	621
	q2	1463	1107	963	831	738	**480**
6.0	q1	1797	2094	**931**	1442	1101	952
	q2	**1625**	**1183**	1107	**1107**	**887**	**738**
8.0	q1	2795	2891	1397	2105	1383	1301
	q2	**1773**	**1849**	**1187**	**1773**	**1107**	**945**
10.0	q1	3392	3593	2218	2595	1996	1730
	q2	**1922**	**1922**	**1266**	**2074**	**1177**	**1044**

There is a little dependence on the rise-to-span ratio but it is less noticeable than the influence of f_c and is significant only for intermediate spans. In view of this, simplified expressions are proposed (table 6).

Table 6: *Expressions for q_{ult} [kN/m] in terms of f_c [N/mm²] on the basis of an effective vault width of 3.0 m*

L=5.0 m		*L*=10.0 m		*L*= 20.0 m
fc ≤ 6 N/mm²	*fc* > 6 N/mm²	*fc* ≤ 7 N/mm²	*fc* > 7 N/mm²	
$q_{ult} = 1{,}230$	$q_{ult} = 185f + 75$	$q_{ult} = 210f + -333$	$q_{ult} = 50f + 788$	$q_{ult} = 95f + 140$

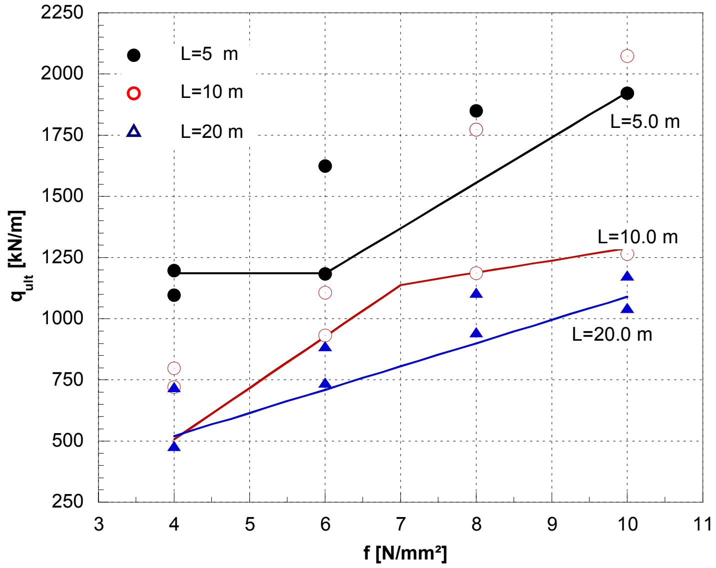

Figure 5: *Ultimate uniform load vs span for single vaults, calculation results and proposed interpolation*

Figure 5 shows the numerical results of calculations and the proposed expressions for interpolation (table 6). If ultimate load is to be compared with the maximum uniform loads (about 200 kN/m) a global coefficient of 2.5 or superior is achieved for compressive strengths over 4.0 N/mm², provided *c*/*L* ratios are coincident with the lower values considered in analysis (table 1).

3.5 Contrast of the results

For contrast purposes P_{ult} values obtained with proposed expressions have been compared with results using a rigid block analysis tool different from that used for the calculations. RING 1.1 was selected[4]. Table 7 summarizes 18 cases sampled and the results with both approaches. The mean ratio RING results to proposed formulae is 1.05 with a coefficient of variation of 12 %. Results are more disperse for short and medium spans. Figure 6 shows ratios.

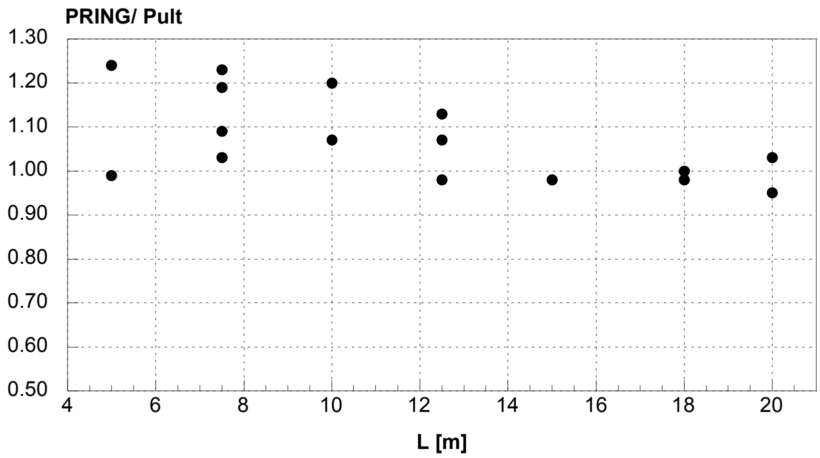

Figure 6: *Ratio of ring results to formulae ones for* P_{ult}

Table 7: *Sample cases calculated with proposed expressions and RING*

Case	L [m]	c [m]	f [m]	h0 [m]	P_{ult} [kN]	P_{RING} [kN]	P_{RING}/P_{ult}
1	5.00	0.5	2.5	0.5	1541	1525	0.99
2	5.00	0.500	1.25	0.50	1726	2135	1.24
3	7.50	0.750	3.75	0.50	2376	2438	1.03
4	7.50	0.750	1.88	0.50	2662	3175	1.19
5	7.50	0.675	3.75	0.50	1456	1580	1.09
6	7.50	0.675	1.88	0.50	1630	2010	1.23
7	10.00	0.700	5.00	0.50	1165	1393	1.20
8	10.00	0.700	2.50	0.50	1305	1400	1.07
9	12.50	0.750	6.25	0.50	1312	1405	1.07
10	12.50	0.750	2.50	0.50	1666	1790	1.07
11	12.50	0.875	6.25	0.50	1841	1798	0.98
12	12.50	0.875	2.50	0.50	2338	2640	1.13
13	15.00	0.900	7.50	0.50	1928	1890	0.98
14	15.00	0.900	3.00	0.50	2449	2390	0.98
15	18.00	0.900	9.00	0.50	1750	1750	1.00
16	18.00	0.900	3.60	0.50	2046	2000	0.98
17	20.00	1.000	10.00	0.50	1887	1790	0.95
18	20.00	1.000	4.00	0.50	2397	2475	1.03

4 CONCLUSIONS

In the context of management and maintenance of a large number of railway bridges, technical authorities often have the need of quick preliminary structural evaluation. A first-level evaluation tool developed for and adopted by RENFE[5] has been presented.

The method has the advantage of its simplicity (within a reasonable level of accuracy): explicit formulae are proposed to obtain the ultimate point and uniform load. In the formulae, just the few most important geometric parameters are involved. The ultimate load values, factorized by a safety coefficient (in the order of 2.0 to 3.0), may be compared with the maximum load per axle (or total weight of the design locomotive) and the maximum uniform load required by standards or abnormal transports. If no compliance is reached for such a *gross* reliability verification, the 2nd stage of evaluation should be carried, possibly leading to strengthening measures. If, on the contrary, carrying capacity of the *ideal* structure is satisfactory but there is concern about the condition of the bridge (damages) the special inspection may be required, possibly leading to repair measures.

Restrictive limitations are imposed for the applicability of the formulae; however the same fact that formulae are not applicable indicates that the case may be complex or non-standard. The results from the expressions may be compared with prototypical bridges designed according to empirical rules of the XIX century, allowing coming to conclusions about the safety level of an important part of the net structures.

Finally, it is obliged to remind the necessity of verifying the assumptions made and the stress-state under service loads. Topics that should be revised with the aid of other tools are the actual presence of rigid backfill (critical for deep arches), the thrusts on the abutments (here supposed undeformable), forces transmitted to foundation and soil, and stresses on masonry.

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