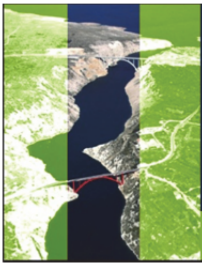

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ASSESSMENT OF MASONRY ARCH BRIDGE WITH CONCRETE DECK

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Abstract: *The assessment of a masonry arch bridge, built more than 110 years ago and widened twice with concrete decks during it service life, is presented in this paper. Conservative methods for the assessment of the load-carrying capacity of masonry arch bridges are shortly overviewed. Additionally the bridge deck cantilever is evaluated due to the bad condition of the second widening deck, taking into account the possibility of vehicle impact on the guardrail. Results are discussed and the deck cantilever replacement with the adequate new reinforcement is proposed.*

1 INTRODUCTION

Numerous masonry arch bridges are in daily use on highways and railways. Many of them are historic structures over 100 years old, still forming a vital part of the transport infrastructure. Deficiencies and degradation during years of service together with time-variability of loadings and particularly increase of traffic load result in different reliability levels of these bridges. Adequate assessment of masonry arch bridges is a key to their continued service to ensure that strengthening is used only when necessary.

2 ASSESSMENT METHODS OVERVIEW

The classic approach to determining the stability of arch bridges dates back to the work of Pippard starting from a two-hinge arch for which the minimum load applied at a fixed position, which causes the arch to turn into a mechanism, is determined and as further extended by Heyman whose theory assumes that the thrust line must become tangential to intrados or extrados in four locations at which point the structure becomes a mechanism [1]. More recent works [2] are based on the rigid block theory which is considered as the basic model for the understanding of the fundamental behaviour of brick arches but this theory uses too many simplifications and assumptions, which frequently leads to large deviations from actual conditions. The more realistic solutions need elasto-plastic behaviour of the material to be considered.
Nowadays, several methods are available for the assessment of the load-carrying capacity of masonry arch bridges. Conservative methods such as Pippard's elastic method, original and modified MEXE method, and Pauser's method often underestimate the load carrying capacity which may result in uneconomical or unnecessary mitigation measures being taken to maintain the bridge. Nevertheless these methods may provide general information on the bridge load carrying capacity which gives us a reference point for the next level of assessment.
On the other hand the use of sophisticated new methods is generally hindered by the difficulty in providing suitable input parameters or prolonged data processing. Finite element methods (RING, MArch, ASSARC,…) are the ones used most often to study structural stability of masonry arch bridges because of their high accuracy. But to arrive at a good match between real and predicted behaviour these FE models (specially the 2- and 3-dimensional models) require a large amount of input data such as many material properties. These methods will be used in the highest levels of assessment procedure as described in [3].
Methods used for assessment of the bridge described in this paper are shortly presented. The safe axle load (two wheel loads side by side) W_A

$$W = \frac{\dfrac{256 \cdot f_c \cdot h \cdot d}{L} - 128 \cdot \rho \cdot L \cdot h \cdot \left(\dfrac{1}{21} + \dfrac{h+d}{4 \cdot a} - \dfrac{a}{28 \cdot d} \right)}{\left(\dfrac{25}{a} + \dfrac{42}{d} \right)}; \; W_A = 2 \cdot W \tag{1}$$

according to the Pippard's elastic method [4] is based on limiting the compressive stress at the crown extrados under the combined dead and live load. The arch is assumed to be parabolic in shape with span/rise ratio L/a of 4, with dispersal of loading occurred only in transverse direction with a 45° load spread angle, effective width of $b = 2 \cdot h + 30$ cm, effective depth h + 15 cm (the thickness of the fill plus half depth of arch barrel in the crown), with the fill having no structural strength with the same density as the arch ring (ρ = 21.44 kN/m^3) and compressive stress limit of f_c = 1400 kN/m^2 and tensile stress limit of f_t = 700 kN/m^2. The Military Engineering Experimental Establishment found that equation (1) given for an idealised arch could be fitted quite well, for given values of allowable and limited stresses, by a nomograph or a formula for the provisional axle loading (maximum allowable axle load on an axle forming part of a double axled bogie):

$$\mathrm{PAL} = 740 \cdot (d+h)^2 / L^{1,3} \tag{2}$$

involving only the arch span L and the total depth $h+d$ at the crown, and this idea was adapted as MEXE method. In this equation d is the thickness of the arch barrel adjacent to the keystone and h is the average depth of fill, at the quarter points of the transverse road profile, between the road surface and the arch barrel at the crown, including road surfacing.
This provisional assessment is then modified by factors (span/rise factor F_{sr}, profile factor F_p depending on the arch rise at the quarter points and the rise at the crown, material factor F_m depending on arch barrel and filling material and dimensions, joint factor F_j depending on width, condition and construction of the joints and condition factor F_{cM} depending on an objective assessment of the importance of the various cracks and deformations) which allow for the way in which the actual arch differs from the ideal [5].
In this way the modified axle load which represents the allowable loading (per axle) on the arch from a double axled bogie configuration with no 'lift-off' from any axle (all the wheels of the vehicle are assumed to be in full contact with the road surface at all times) is determined:

$$\mathrm{MAL} = F_{sr} \cdot F_p \cdot F_m \cdot F_j \cdot F_{cM} \cdot \mathrm{PAL} \tag{3}$$

For converting this result to other axle configurations and for situations where axle 'lift-off' (circumstances when the wheels of a multiple axled bogie can partially lose contact with the road surface and transfer some of their load to other axles in the bogie) may occur additional axial factors A_f are given.
In the work under reference [4] new equations for working out the safe axle load of an arch that incorporate the effects of axial strain energy are developed, with which lower carrying capacities have been predicted for relatively small span bridges:

$$\begin{aligned} W &= \frac{\dfrac{256 \cdot f_c \cdot h \cdot d}{L} - 128 \cdot \rho \cdot L \cdot h \cdot \dfrac{1}{1+\lambda}\left\{\left(\dfrac{1}{21}+\dfrac{h+d}{4 \cdot a}\right)-\dfrac{a}{28 \cdot d}\left[1-7 \cdot \lambda-\dfrac{42 \cdot (h+d)}{a} \cdot \lambda\right]\right\}}{\dfrac{1}{1+\lambda} \cdot \left[\dfrac{25}{a}+\dfrac{42}{d} \cdot \left(1+\dfrac{32}{7} \cdot \lambda\right)\right]}; \\ W_A &= 2 \cdot W; \\ \lambda &= \frac{5}{32} \cdot \left(\frac{d}{a}\right)^2 \int_0^1 (\cos\alpha)^{\frac{4}{3}}\, dx \end{aligned} \tag{4}$$

Whilst the MEXE method would give the same value for the safe axle load for a given (h+d), these new equations give very different safe axle loads depending upon the relative values of h and d.

3 MASONRY ARCH BRIDGE

Masonry arch bridge comprising three stone segmental arches was built more than 110 years ago [6, 3]. The original width of the bridge was 5.85 m. In the year 1950 the bridge was widened to 7.0 m with an additional 15 cm thick concrete deck, and in 1962 to a total of 9.0 m with an additional 16 cm thick concrete deck. The length of the first concrete deck cantilever is 50 cm, and of the second one is 107.5 cm.

Figure 1: Masonry arch bridge example

Arch span is 8.65 m with a rise of 1.97 m. Parabolic arch line was carefully measured on the site to enable the calculation of the line of thrust. The bridge is constructed with closed vertical spandrel walls, two arch pier-walls have variable thickness from the top to the bottom and two arch abutments have vertical front walls and parallel wing walls.

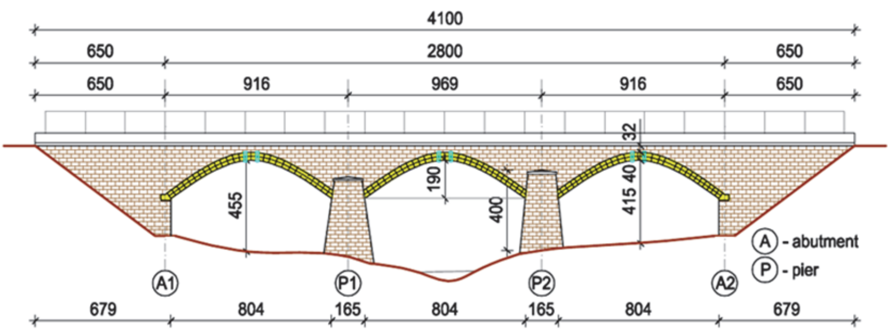


Figure 2: Longitudinal disposition of the bridge

3.1 Bridge condition

The foundations of the bridge are on sound solid rock and no scour or settlement is visible. The abutment, pier-walls and arch show no signs of insufficient capacity for the loads and traffic for which the bridge is in service today. There are no signs that the arch is too thin, distorted or that ring separation occurred and also there are no signs of arch stone erosion or movement. The arches show only slight signs of old water seepage. Judging from the black marks on the surface a lot of water is seeping through the structure. But it seems that no major structural damage occurred yet.

The arch barrels show signs that some kind of repointing works has been done, probably simultaneously with the works on the last widening in 1962. The appreciation is that some loose or friable mortar was removed and filed with new material. The old and ″newer″ mortar show little signs of cracking and seem in relatively good order. Only slight longitudinal cracking can be seen with almost none transversal cracking [6]. The type of masonry is regular laid natural stone.

Based on visual inspection, most of spandrel walls are in good order with one local exception on the upstream side near the abutment. There the spandrel (wing) wall shows local signs of wall movement - bulging and part of the stone has a changed colour. There is a possibility that this bulging occurred during the road widening and the adjustment of the structure to the new load. There are no signs that this movement occurred recently or that it is progressing.

The first widening with a 15 cm thick concrete deck shows usual signs of deterioration of concrete after approximately 60 years of use. The second widening with a 16 cm thick concrete deck is in a very bad condition due to unsufficient concrete cover provided in the design and poor execution of works.

Figure 3: The poor condition of the bridge widening decks

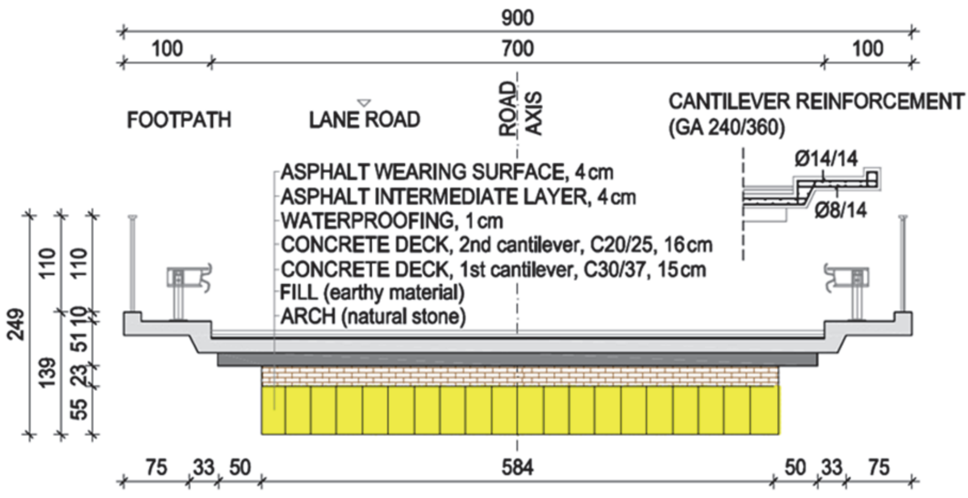
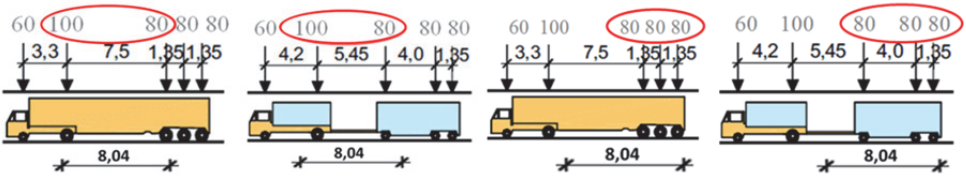


Figure 4: Cross-section of the existing bridge

3.2 Arch assessment

Assessment of the bridge arch [7] was performed using Pippard's method (resulting with 18.7 tonnes), modified MEXE method for one (resulting with 31.2 tonnes), two (resulting with 15.6×2=31.2 tonnes) and three axle bogie (resulting with 18.7×3=56.1 tonnes) and according to new formulas developed for the short span bridges (resulting with 17.7 tonnes) as described in section 1. Based on these calculations the gross vehicle weight and the maximal axle load are rounded off to the nearest 0.5 tonne. The gross vehicle weight is 31 tonnes and the maximal axle load is 15 tonnes.

Comparing these results with representative vehicle models [8, 9] on Croatian roads, all vehicles satisfy maximal allowable axle load, but vehicles models 5 (tug trucks) and 6 (trucks with trailers) do not satisfy the gross vehicle weight limit. But taking into account the axle distance (see Figure 5) and the span of the bridge (8.04 m) these vehicles satisfy the gross vehicle weight limit, as well.



18 tonnes < 31 tonnes

24 tonnes < 31 tonnes

Figure 5: Influence of axle distances of the axle loads of the representative vehicles (tug trucks) and 6 (trucks with trailers) acting on the bridge span

3.3 Cantilever assessment

Due to the bad condition of the second widening deck, the assessment of the deck cantilever was performed. Impact of a realistic vehicle with the mass of 10 tonnes on the guardrail type H1 and of a vehicle with the mass of 13 tonnes on the guardrail type H2 in an accidental load situation was investigated. Based on the inversed procedure [10] for defining an average vehicle impact force for the required containment level testing and guardrail deformation level the maximum limiting vehicle speeds were established [7].
For a vehicle with the mass of 10 tonnes the maximum limiting speed is 80 km/h and for a vehicle with the mass of 13 tonnes the maximum limiting speed is 20 km/h. The effect of the vehicle of the higher mass would jeopardize the safety of the bridge cantilever. Namely heavier vehicles would induce design bending moment M_{Sd} in excess of the resistance bending M_{Rd} calculated on the basis of as built cantilever reinforcement.

3.4 Results overview and proposal for counter measures

Conservative arch assessment methods result in a gross vehicle weight limit of 31 tonnes, but due to a poor concrete condition and inadequate as built reinforcement deck cantilever would not be able to resist the vehicle impact on the guardrail. Based on these assessment results repair of concrete deck is proposed.
Additionally, the existing deck cantilever was analyzed for the persistent load situation with the new European load model of traffic load (one wheel load of 120 kN and a uniformly distributed pedestrian load of 5 kN/m^2) and it was concluded that as built reinforcement is not sufficient. Cantilever replacement with adequate new reinforcement is proposed based on the analyses according to normative Eurocode traffic load models and realistic vehicle impacts on the guardrail type H1 and H3.

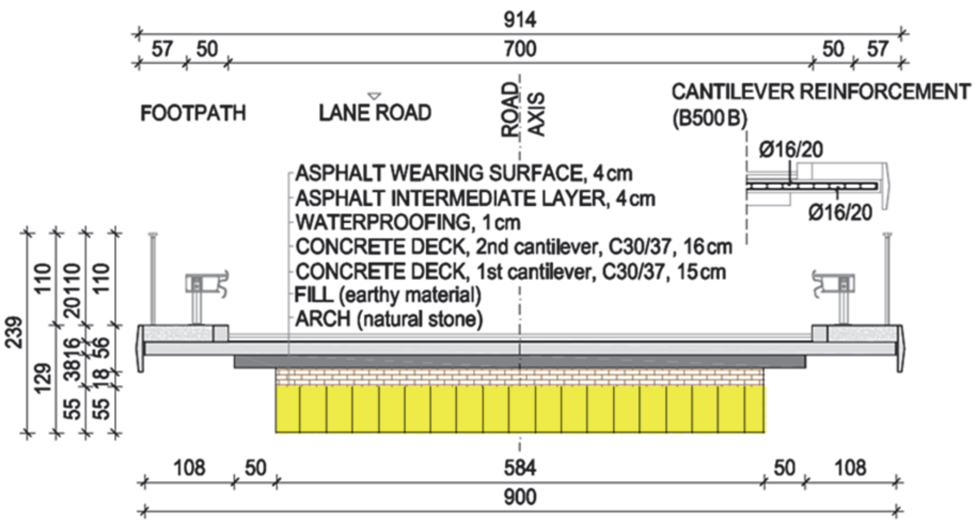


Figure 6: Cross-section of a proposed new bridge deck

4 CONCLUSION

Adequate assessment of the load-carrying capacity of masonry arch bridges is a key to their continued service to ensure that strengthening is used only when necessary. Today several assessment methods are available. Although conservative methods often underestimate the load carrying capacity of the arch, they may provide general information on the bridge which gives us a reference point for the next level of assessment.

In this paper the assessment of the masonry arch bridge, built more than 110 years ago and widened twice with concrete decks during it service life, is shortly presented. Arch assessment is performed using Pippard's method, modified MEXE method for one, two and three axle bogie and according to new formulas developed for the short span bridges and it results with the gross vehicle weight of 31 tonnes and the maximal axle load of 15 tonnes.

Due to the poor condition of the second widening deck, additionally the assessment of the deck cantilever was performed taking into account the possibility of vehicle impact on the guardrail. As for the vehicle with the mass of 13 tonnes the maximum limiting speed of only 20 km/h was established, it is clear that the effect of the vehicle of the higher mass would jeopardize the safety of the bridge deck cantilever.

Based on these assessment results cantilever replacement with an adequate new reinforcement is proposed as a repair measure. The new solution was verified with the analyses according to normative Eurocode traffic load models and realistic vehicle impacts on the guardrail type H1 and H3.

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