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RESCUE OF A STONE BRIDGE WITH RESPECT TO CURRENT CONDITION AND STANDARDS

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Abstract. A three span stone masonry bridge dating back to the middle of the 19$^{\text{th}}$ century, still in the roadway network, is assessed to prevent its demolition. Its industrial heritage value and ecological concerns were the principal reasons for the assessment. The carbon footprint of the stone arch replacement would be approximately 200 t CO_2 emission owing to 430 m^3 reinforced concrete in the new structure. Besides the cultural monument, considerable energy, CO_2 emissions and natural resources can be saved in accordance with the sustainable development goals. Standards, guides and commercial software do not provide an adequate support for the assessment of masonry arch bridges, therefore an innovative two phase application of a commercial linear sttructural analysis code and an original in-house code was developed for the purpose. Thousands of stone ach bridges still in service worlwide give the case study more general importance.

Keywords: Natural resources preservation, sandstone, bridge, arch, stress, displacement.

1. Introduction

The three-span sandstone arch bridge was built in 1857 and remains an important part of the second-class roadway network, see Figure 1. It was considered for a reconstruction with total demolition of the arches. The owner planned to remove the arches and top parts of the piers and replace them with a reinforced concrete structure that would be a reminder of the arches of the stone bridge.

In order to prevent an irrecoverable loss of the industrial heritage, an extensive load capacity assessment was conducted for the bridge with the replacement of the pavement. Standard assessments determine load rating based on the limit states analysis. The load models used for the purpose differ in various countries substantially, for instance either the ultimate (in Europe) or serviceability limit states (in the US) are preferred. Instead, the check of the available building standard's criteria is used herein. The conditions of the EN 1990 Part 2 Traffic Loads on Bridges have been checked to testify that the bridge with its valuable stone arches can remain a part of the roadway network without limitations. A number of clauses of the EN 1996-1-1 could be applied even though the standard is not designed for bridges.

"There is a multitude of assessment methods available for masonry arch bridges, yet there is no widely accepted framework for their application", this quotation from [1] is a sad fact. Tens of papers report on sophisticated methods based on advanced material and structural models. None of them are acceptable for the design practice for two principal reasons,

Figure 1. Front view of the sandstone bridge at Ponikla.

(a) the application requires specific high competencies of the designer and specialized software

(b) the input material and structure data are not available for the models.

The present approach is simple: the assessment method can use only the data available in a common design environment and with reasonable cost. This admits relatively simple material models. The finite element technology is preferred since it can model the components of the bridge and their interactions in a way that other methods and codes (modified MEXE, plasticity theorems like Ring [2], Castiligliano theorem like CTAP code [3] and others) cannot. At the same time it is widely available. The absence of design standards or other "widely accepted framework" gives the load rating of an important bridge the character

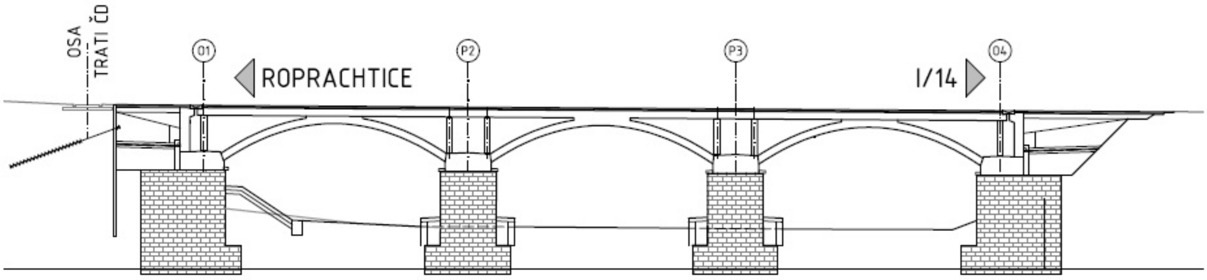

Figure 2. New design of the bridge.

of a research task. An assessment method was developed in the course of the solution of the bridge. The objective of the contribution is to outline the principal steps of the method and its application to similar bridges. The method is tailored to masonry bridges management practice and demostrates its application. The sample solution documents the potential of the method for sustainable infrastructure management and ecological impact.

2. The original restoration concept, ecological impact

Scheduled for reconstruction, the owner planned to remove the arches and top parts of the piers and replace them with a reinforced concrete structure that would remind the arches of the stone bridge, see Figure 2. However, considerable economical savings and benefits for the environment can be achieved when the arches are preserved. The construction of the new reinforced concrete structure would require the destruction and deposition of $230\,\mathrm{m}^3$ high quality stone masonry built with craftsmanship which would be lost forever. Removal and deposition of $400\,\mathrm{m}^3$ backfill would cost money and ecological load. The carbon footprint would be left owing to $430\,\mathrm{m}^3$ reinforced concrete with approximately 200 t of CO_2 emission.

The most important offense against sustainable building would probably be the life span of the new reinforced concrete structure. The stone arches are 170 years old and still in service in spite of the neglected maintenance in the last decades. With adequate protection against water leaks they can serve for 150 more years. The service life of the proposed reinforced concrete structure willl certainly be substantially shorter, as in particular the struts above the piers with notch hinges are susceptible to early deterioration. The ecological aspects of the reconstruction speak for the preservation of the stone arches.

3. Load capacity of stone arch bridges

3.1. Material models

The dominant material failures and limit states in the stone arch bridges are

(a) the tension cracking in the bed joints,

(b) the compression failure in the bed joints.

The former occurs already at the service load levels and may result in gradual development of virtual hinges and stability loss of the arch with the growing live load. It must be represented in the material model in order to assess the redistribution of the internal forces. The trouble with this limit state is that there are no generally accepted quantitative criteria worldwide. Recently the Czech Guide TP199 of the Ministry of Transport of the Czech Republic introduced an admissible relative depth of the cracks in the bed joints. The other trouble is that the predestined orientation of the cracks makes the no-tension models in general purpose codes almost unusable. This failure mode can effectively be included only in beam elements of the 2D models or shell elements of the 3D models where the kinematic assumptions and dominant cross-section normal stresses facilitate the specific no-tension model. The thick shell and beam finite elements with the no-tension property are derived in [4].

The latter limit state is a kind of a brittle failure, partially contained in case of the arch bridge with fill. The failure criterion for this limit state is the compression strength of the masonry. This property is the only one that can be determined in a diagnostic survey with acceptable cost and reliability. EN 1996-1-1 provides support for determining the masonry compression strength from the compression tests of the core bored specimens of the stones and by nondestructive boring tests of the mortar (PZZ 01 device). The tests were performed for the Ponikla bridge masonry arches and the compression design strength was determined $f_d = 2\,\mathrm{MPa}$. The value is rather conservative since the sandstone ashlars of the vault are in good condition.

The backfill and pavement are linear elastic. In order to eliminate extensive backfill failures, the Drucker/Prager criterion is checked in the fill elements and indicated by asterisks in the graphic output.

The elastic properties used in the FEM models are summarized in Table 1. They are deliberately conservative for the backfill. A review of the backfill properties was conducted in the published literature [5] and these are the values at the low end. During the method development, a parametric study showed that softer fill always implies worse stresses in the vault drawing

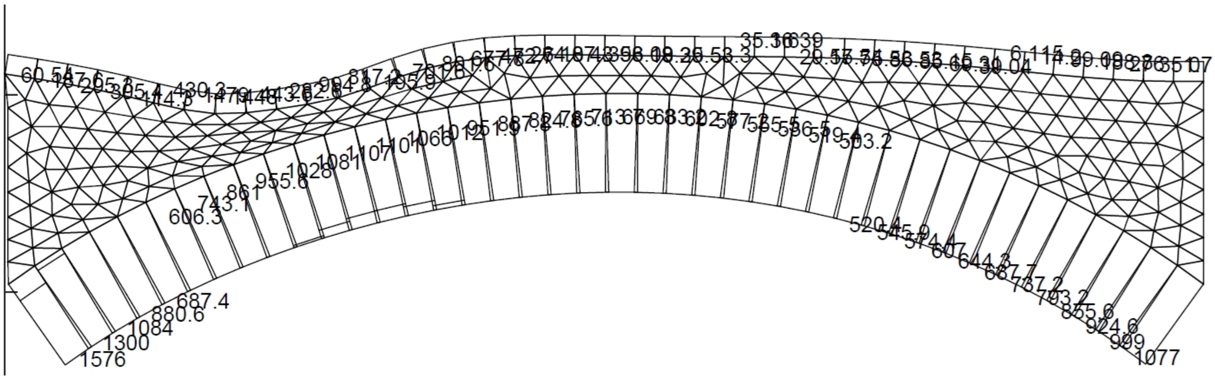

Figure 3. The minimum stresses [kPa] and crack depths in the Ponikla central span arch. The displacements are 1000 times scaled up.

	$\boldsymbol{E}$ [GPa]	$\boldsymbol{G}$ [GPa]	$\boldsymbol{\nu}$	$\boldsymbol{\rho}$ [t/m^3]
Arch	10	5	0.25	2.0
Backfill	0.1	0.043	0.15	1.8
Pavement	31	12.917	0.2	2.5
Spandrel	0.1	0.04	0.25	2.2
Piers/Abutments	10	4	0.25	1.8

Table 1. Elastic properties considered in this case study.

an important conclusion. The fill material constants from the table are on the safe side and can be used in bridge assessment when nothing is known about the fill quality. The value 0.4 GPa is recommended when at least a compacted cohesionless material can be assumed. Selection of the Young moduli of the arch parts is facilitated by a convenient property of the FEM model, see the next section [6].

The short summary of the material models and properties exposes the uncertainties involved which inevitably imply low reliability of the analytical models of the masonry arch bridges. On the other hand, most of them were built in the 19$^{\text{th}}$ century according to empiric rules with ample reserve of load capacity. Decades of their service testify to their robustness and reliability. These aspects compensate the low reliability of the analytical models, asserts Boothby in [7].

3.2. Finite element bridge models

It is difficult to account in a single complex model for the interaction of the spans and piers, the transverse asymmetry of the load and structure and non-linear arch behavior. The assessment method therefore uses different FEM models for the tasks.

The checks required by the standards are performed in a non-linear 2D (plane strain condition) model of a single span featuring the no-tension material specified above. The simplest Timoshenko beam elements [4] are used for the stone arch so that the normal stresses in the cross-section planes are used in the above failure conditions. The backfill is meshed by standard CST triangles and the pavement by linear Timoshenko beam elements. The meshing is shown in Figure 3. Clamped arch springings are assumed. The results are slightly non-conservative owing to the plane strain 2D idealization which neglects all transverse effects. Correction factors are used to make up for the error, see the next paragraph. The linear elastic no-tension material model secures robust convergence of the equilibrium iterations so that unskilled users can use the model and code. The model also implies that stresses and crack depths do not change when the Young moduli of all materials are multiplied by a common factor. A non-commercial code was developed for the purpose and will be available as an Octave/Matlab script when a minimum user interface is completed.

The other two models are a 3D linear model of one span with the same boundary conditions as in the non-linear model and a 3D model of the whole bridge including piers and abutments. They are primarily used to assess the differences between the 2D and 3D models and the interaction of the arches, piers and abutments.The differences between the 2D and 3D models are expressed in terms of correction factors for the 2D solutions. The factors are then used to correct the results of the non-linear model. This is not quite consistent but the bed joints cracking is localized to relatively small volumes of the vault and the imperfection is thus accepted.

A front view of the bridge and its 3D FEM model are shown in Figures 1 and 4. The spans are all approximately 11.4 m and the full width of the bridge is 7.6 m. The single span 3D model is an extraction of the full bridge separated by adjacent pier's symmetry planes. On the planes, the symmetry boundary conditions are applied (zero normal and free in-plane

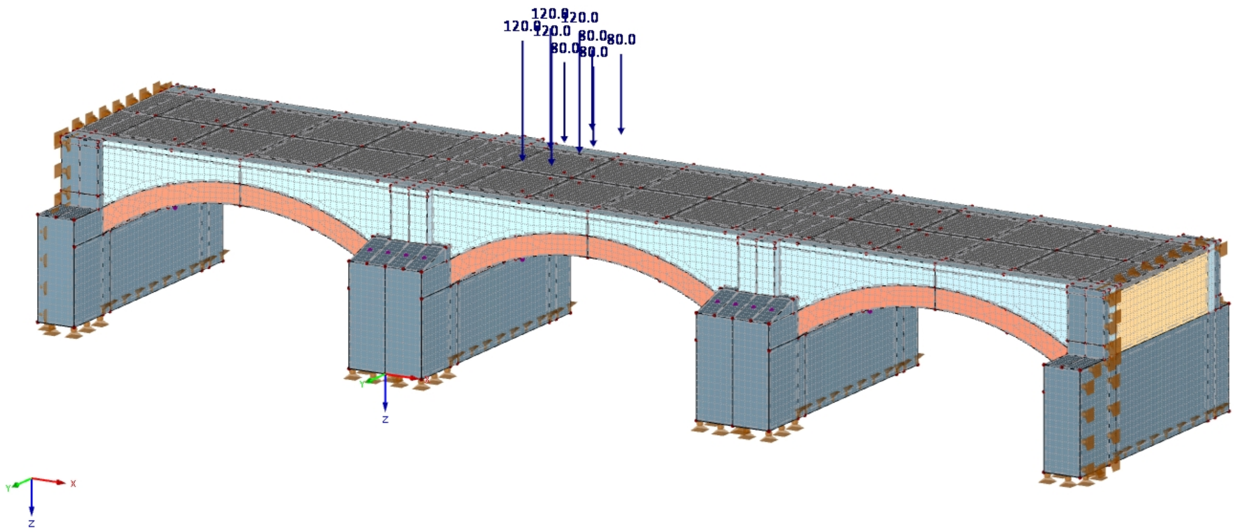

Figure 4. Full 3D bridge model with two tandem axle forces.

displacements). The elements are based on isoparametric bricks/quadrilaterals enriched by rotational degrees of freedom at all nodes. The commercial package RFEM from Dlubal Software [8] was used in the sample solution. Any other package can be used with an adequate modeler. The model's build itself is rather easy since details of the sructure can be omitted. Extraction and processing of the relevant stresses and other data may be more demanding and depends largely on the available software.

The other purpose of the full 3D model is to asses the interaction of the spans, piers and abutments. The dominant effect of the interaction is the widening of the arch subject to the local live load, see Section 3.3 for the live load model. The vertical displacements and rotations of the arch springings owing to the interaction are neglected in the method. The arch widening is obtained as the difference of the averaged horizontal displacements of the right and left springings. It is then used in the non-linear 2D model as an imposed displacement (kinematic load).

3.3. Load model

The load model LM1 of the EN 1991-1-1 is used. It is known to be critical for bridges with these space parameters. The same position of the two axle tandems is considered in all calculations, as shown in Figure 1. The position above the quarter span is generally acknowledged to be the most adverse for the arch. The uniform continuous live loads of LM1 are not shown in Figure 4 but applied on the loaded half of the span in the calculations. It is worth mentioning that LM4 class 1800 kN of the EN standard gives approximately the same total load on the half span but is almost uniformly distributed. The dead load is included via mass densities of the bridge components materials. The load factors of the ultimate limit state are used according to the EN and National Application Document. Loading in the 2D model is taken as the average over the bridge width of the 3D model loads.

3.4. Results for the design load of the Ponikla central span

The solution of the Ponikla bridge for the design loads of the EN 1991-2 is shown in Figure 3 with axle tandems of the LM1 allocated at a quarter span of the arch. The dead load of the arch, fill and pavement is accounted for. The third load component is the imposed horizontal displacement 0.13 mm of the right arch springing, fill boundary and pavement end. It equals the arch widening obtained in the full 3D linear model, see the last paragraph of Section 3.2.

The absolute values of the minimum stresses in the arch elements are inserted at the arch face where they occur (the maximum stresses are always nonnegative).

The same quantities are inserted at the lower and upper faces of the pavement although they are irrelevant with respect to the arch assessment. The 0.2 m thick reinforced concrete pavement is considered linear elastic in the model. The bending "waves" characteristic for beams on an elastic layer can be observed in the figure, note the alternating faces of the minimum stresses in the pavement elements.

The crack depths in the arch elements are indicated by the inserted line segments. The crack mouth widths in the bed joints can be computed from the crack depths, the voussoir lengths and the minimum strains in the elements when the Timoshenko kinematic assumption is employed. The arch model is a 1D continuous beam, where the arch elements lengths depend on the chosen mesh density. The voussoirs are approximately 0.4 m long whereas the elements just about 0.33 m.

The load intensities and material properties used in the non-linear plane strain 2D model are averages

Cross-section locations	0	0.25	0.5
Extrados stress correction factor	1.18	1.08	1.1
Intrados stress correction factor	1.05	-	0.94

Table 2. The 3D/2D correction factors for the compressive stresses at three cross-section locations.

across the vault width 7.6 m. However, the load model LM1 for two lanes is not symmetric with respect to the longitudinal symmetry plane of the bridge. The asymmetry effect is evaluated through comparison of the 3D model solutions for the true LM1 load distribution with the solution for the average uniform distribution in the transverse direction. The ratios of the two stress' sets are used as 3D/2D correction factors and are summarized in Table 2 for the four cross-section locations along the arch span. Only compressive stresses are considered.

The stress values from Figure 3 are multiplied by the correction factors from Table 2 in the last step of the method. The assessment of the bridge using the method determined its safety in the ultimate limit state as defined in EN 1991-1.

4. Conclusion

A method for the assessment and load rating of the stone masonry arch bridges is presented. It is designed for application in bridge management. A single material property, the vault masonry compression strength, is necessary. Users need no special skills beyond adequate experience with the application of the FEM codes. The safety assessment of the Ponikla bridge using available standards together with activities of other institutions and individuals in the cultural heritage protection helped to avert the demolition of the bridge's arches. The simple material model and the dedicated finite element can be used to check the complex structure against available standards conditions. An extension of the concept to 3D shells is currently under way and then the parallel linear 3D model will not be necessary for the application of the method. Preservation of the Ponikla bridge represents a contribution to the responsible consumption and production goal of the sustainable development policy.

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