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Structural analysis for the reconstruction design of the old bridge of Mostar

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Abstract

The paper presents the structural analysis performed for the reconstruction design of the Old Bridge (Stari Most) of Mostar, in Bosnia Herzegovina. The bridge survived for more than 400 years, from 1566 till 1993, when it was destroyed by shelling, during the conflict between the Muslims of Bosnia and the Croats. The design was developed in 2000 and 2001 under the supervision of an International Commission of Experts. In the paper brief historical notices and information about topographic and geotechnical surveys and material tests are first given, then results of numerical analyses are presented. The main aim of the structural analysis was the assessment of the load bearing capacity of the bridge for all load cases prescribed by present European Recommendations. Numerical analyses allowed putting into evidence the excellent structural performance of the bridge. The first stone of the bridge was laid on June 27, 2002.

1 Introduction

The Old Bridge of Mostar, an arch stone bridge with a span of about 29 m over the Neretva River (Figure 1), will be rebuilt with the same shape and dimensions and using the same construction techniques. The bridge was designed by the Ottoman architect Mimar Hayruddin and its construction was completed in 1566. It was destroyed in November 1993 by heavy horizontal shootings from a tank positioned on the river bank downstream, during the conflict between the Muslims of Bosnia and the Croats. Only abutments and small portions of the arch vault survived, even if they were very damaged. The reconstruction of the bridge is part of a Pilot Project promoted by the Town of Mostar under the supervision of UNESCO and with funds managed by the World Bank with donations of many countries. In April 2000 the Florentine company General

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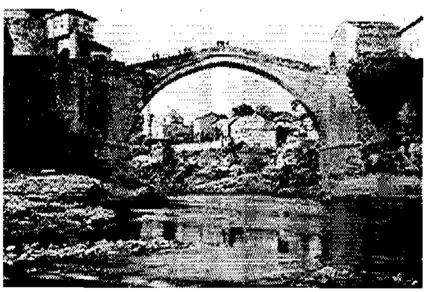
Engineering (GE) won the tender for the reconstruction design and the Department of Civil Engineering of Florence supported GE for the structural design. The design was developed in 2000 and 2001 [1]; step after step design documents - drawings and calculations - were checked and approved by an International Commission of Experts, named ICE.
The structural analysis of the bridge was performed in two phases. In the first phase the bearing structure was supposed to coincide with the arch vault alone and a linear stress-strain law was adopted for masonry. In the second step both linear and non linear finite element (FE) analyses of the whole bridge were performed, considering that all elements (arch, spandrels, slab) act structurally. Material parameters for all elements of the bridge were assessed experimentally. Load cases and combinations were defined according to Eurocode 1 for a footbridge of first category.



(a)

(b)

Figure 1: The Stari Most before (a) and after (b) the war (Figure 1a from "Stari Most", 19 October 1912, Albert Kahn "Archives de la Planète", Department Hauts-de-Seine, Paris)

2 Geometry, materials and old construction techniques

Original design drawings of the bridge are not available, so the most important data sources for the recognition of the bridge's geometry, stone by stone, are a topographic survey dated 1955 and a photogrammetry dated 1982. Moreover GE performed a photogrammetry and a topographic survey of bridge's ruins.
Main geometric dimensions of the old bridge were: span of 28.71 m and 28.62 m on the north and south side, respectively, and arch rise of 12.06 m. The eastern springing was about 0.12 m lower than the western; the cross-section of the arch had a uniform profile, virtually rectangular, and was about 4.00 m wide and 0.80 m thick. The arch consisted of 111 rows and each row was formed by two to five voussoirs. The average dimensions of voussoirs were 0,40 x 0,80 x 1,00 m. The intrados of the arch could be approximated by half a circle with a radius of about 14,89 m and 14,77 m on the north and south side, respectively. The bridge shape was almost symmetrical, apart from some small irregularities due to ordinary construction errors or settlements.
Arch voussoirs were made of a local oolithic limestone, called Tenelija Cursta, whose compressive strength is of about 20 MPa. Voussoirs had been assembled

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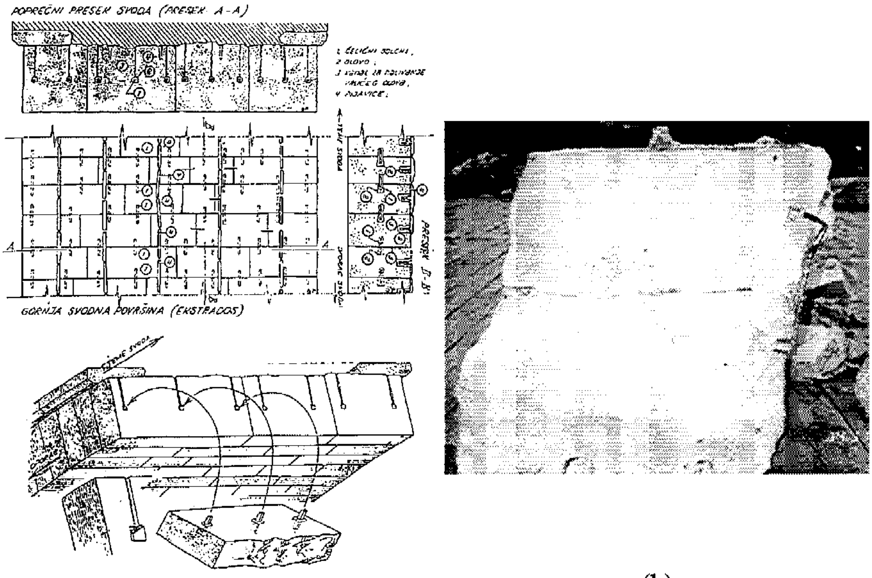
with a minimum of mortar and with iron dowels and cramps (Figure 2); mortar joints were very thin with thickness included between two and eight millimetres. Each voussoir was linked to those of the preceding row through one or more iron dowels. Each dowel was inserted in a purposely carved slot and sealed with melted lead. Iron cramps were put longitudinally across joints of adjacent rows, at the arch extrados. Other cramps were put at the lateral sides of stone blocks to link transversely voussoirs inside each row.



(a) (b)

Figure 2: (a) Scheme of voussoir connections from [2]; (b) photo of two stone blocks with dowels and cramps.

Cornices, spandrels and parapets were made of Tenelija Mekša, whose compressive strength is lower than Tenelija Cursta, while the slab and the pavement were made of Krecnjak limestone. River banks are of Breca, a very porous and coarse conglomerate.

3 Actual state of conservation

The arch vault of the bridge was destroyed by shelling in November 1993, but some portions close to springings are still on site (Figure 1b). In 1996 arch stones, which still remained near the bridge location in the Neretva river, were recovered by divers and placed on a platform nearby. Only a few percent of stones from the vault were recovered, most of them as individual stones. In the abutment walls there are vertical and diagonal cracks, through joints and/or across stone blocks.

Abutments consist of different stone types and mortars: walls facing the river are made of Tenelija, while Breca was widely used in the wing walls. Thick crusts of gypsum cover the lower part of abutment walls.
In the past foundations underwent settlements, because caves were excavated below them by the slow solution of limestone by water. Attempts have been undertaken to stabilise foundations using grouted micropiles and injections, but those measures were probably not adequate. The heads of injection pipes are still visible on the right bank.
Mechanical properties of abutments were assessed through a wide geotechnical campaign performed by the Company Conex [3]. Main results of this campaign were: at abutments the rock is very heterogeneous and permeable, with a high influence of erosion; the rock under the foundations needs to be consolidated because of the presence of caves; the fill of abutments, of poor quality, was eroded and it is required to consolidate it. The cantilever portions of the arch were deeply damaged by shootings and attacked by percolating water. The high permeability of the rock and the fill inside abutments favoured the erosion by water, so that the mechanical properties of many stones were reduced significantly. On the contrary the lower portions appear in a good state of conservation.

4 Main items of the bridge reconstruction

Ancient portions still on site were severely damaged by shelling in 1993 and have to be dismantled. Those blocks which appear intact can be relocated in their original positions only after assessing their mechanical properties (e.g. through ultrasound tests). Abutments, which support the bridge thrust, will be repaired before any dismantling operation: tie rods or anchor bars will be used to strengthen walls which are leaning outwards, and mortar injections will be executed to repair cracks. The reuse of old stones, recovered by divers, is a difficult task especially due to their small quantity. The number of voussoirs in the old bridge was 456, but only 162 arch stones were recovered and are on the platform nearby. There are only 24 pieces from cornices, 44 pieces from spandrels and 19 pieces from parapets. Moreover those stones which appear intact can have internal cracks produced by shelling and falling into the riverbed. Other limits to the reuse of old recovered stones are their scarce quality and the difficult recognition of their original positions.

5 Structural analysis

The bridge will be rebuilt with the same shape and dimensions of the old bridge, using the same materials and techniques. The only differences will be: a new type of mortar with elasticity and waterproofing characteristics will be used, dowels and cramps will be made of stainless steel, with the exception of those visible. It could seem superfluous to perform structural analyses of such a structure, which had survived to floods and earthquakes for about four and half

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centuries, undergoing only small damages. Nevertheless the structural analysis is required to assess the stress and strain patterns of the new "Stari Most", to discover eventual hidden defects or to confirm its excellent structural performance. Only non linear analyses can allow evaluating the safety factor of the bridge under design loads prescribed by present Recommendations.

Preliminary static analyses were performed considering only the arch vault as structural element, while other elements, like spandrels and stiffening rib, were supposed to contribute to loading but not to act structurally. The arch was modelled with "beam" elements using the well-known computer-code SAP2000 [4] and linear elastic analyses were performed under dead weights. The line of thrust was found to lie inside the middle-third of the arch cross section, unless two limited portions: one at the crown and one on the eastern side. The last was in correspondence of a significant curvature irregularity, which was due to settlements after removal of the centering in the original construction or to foundation settlements. Nevertheless tensile stresses obtained in these two arch portions were not realistic, because they were higher than the tensile strength of the masonry and the former bridge would have exhibited significant cracking patterns under permanent loads. A refined FE model was required to get more accurate stress and strain patterns taking into consideration that spandrels, central rib and slab act structurally together with the arch vault.

Then more accurate numerical analyses were performed through a three-dimensional FE model with eight-node solid elements (Figure 3), using the code ANSYS [5]. Each element is capable of plastic deformation, cracking in three orthogonal directions, and crushing. This model included the whole bridge (arch vault, spandrels, stiffening rib, slab, parapets) and was extended at both ends to include a 6.00 m deep portion of abutments.

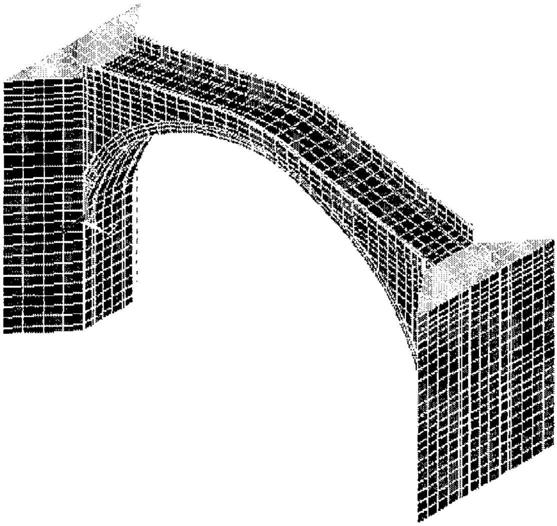

Figure 3: 3D FE model of the whole bridge.

From a first series of linear analyses with the 3D model, tensile regions were more limited than those obtained from preliminary analyses of the arch vault alone. The three-dimensional model provided a better description of stress and strain patterns than the beam model: the line of thrust was found to lie almost completely inside the middle-third of the arch cross section.

The non linear behaviour of masonry under permanent, live and thermal loads was simulated through a series of linear analyses changing the nodal connectivity of the FE model. After each analysis, in those regions where stresses were higher than the masonry tensile strength, elements were disconnected in order to simulate the formation of discrete cracks. The numerical process stopped when all tractions were lower than the tensile strength. Therefore, for each load combination, a different nodal connectivity and different restraint conditions were defined.

Under flood and earthquake actions non linear analyses were performed using the smeared crack approach. A combination of the Drucker-Prager yield surface and of the William-Warnke failure surface [6] was used. The William-Warnke failure criterion accounts for both cracking and crushing failure modes trough a smeared model.

From tests on specimens driven out from boreholes, both strength and modulus of elasticity of masonry exhibited a large variation for all bridge portions survived to shelling. Therefore for each structural element three different values (minimum, mean and maximum) of the modulus of elasticity were considered in numerical analyses. Values of masonry strength and modulus of elasticity are listed in Table 1.

Table 1. Strength and modulus of elasticity of masonry from experimental tests.

Structural element	$f_{c\ mean}$ (MPa)	E_{min} (MPa)	E_{mean} (MPa)	E_{max} (MPa)
Arch	8	6000	8000	10000
Stiffening rib	5	3000	5000	5000
Lateral spandrels	6	5000	6000	8000
Slab	6	5000	6000	8000
Upper part of abutments	4	-	4000	-
Lower part of abutments	4	-	15000	-

5.1 Load cases and combinations

Load cases and combinations were defined according to Part 3 of Eurocode 1 [7], which refers to the definition of pedestrian and other actions specifically for footbridges. According to Eurocode 1 wind and thermal actions were not taken into account as simultaneous. Moreover, as the bridge is not protected from bad weather, traffic loads were considered incompatible with significant wind and/or snow. Eurocode 8 [8] was referred to for earthquake actions.

Seven load cases were considered:

a. Dead and permanent loads;

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b. *Live load uniformly distributed over the bridge*: a uniformly distributed load of 5 kN / m^2 was applied on the whole bridge;
c. *Live load uniformly distributed over half the bridge*: a uniformly distributed load of 5 kN / m^2 was put on half the bridge, from one springing to midspan;
d. *Uniform thermal load of +15 °C*;
e. *Uniform thermal load of -15 °C*;
f. *Flood* (2500 m^3/s): forces due to flood were analysed assuming a discharge between 1500 m^3/sec and 2500 m^3/sec, with step of 250 m^3/sec. The discharge of 2500 m^3/sec is the maximum allowed between the arch abutments and produces a total force of 10560 kN on the bridge;
g. *Earthquake actions*: a simplified modal response spectrum analysis was performed.

Numerical analyses were performed combining in three different ways the modulus of elasticity of structural elements:

A. mean value of the modulus of elasticity for all structural elements,
B. maximum value of the elasticity modulus for the arch vault and minimum value for all the other structural elements,
C. maximum values of the modulus of elasticity for all structural elements.

All load combinations are listed in Table 2. The number in the first column refers to the load combination, while the capital letter in the last column (A, B or C) designates the combination of the modulus of elasticity. In the second column small letters refer to load cases listed above.

Table 2. Load combinations.

Load combination		Combination of Young modulus
1	1.0 a + 1.0 b	A, B
2	1.0 a + 1.0 c	A, B
3	1.35 a + 1.35 b	A, B
4	1.35 a + 1.35 c	A, B
5	1.35 a + 0.54 b + 1.5 d	A, C
6	1.35 a + 0.54 b + 1.5 e	A, C

Load combinations with flood or earthquake loads are described in Sect. 5.3.

5.2 Main results of FE analyses

For sake of brevity graphic outputs of stresses are given only for analysis no. 5C, which gave maximum compressions in the arch masonry (Figure 4).
If the modulus of elasticity of the arch is changed from its mean value to its maximum value (percentage variation of +25 %) and those of other elements

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from their mean value to the minimum value (- 40 %), stresses in the arch vault increase of about 10 %, while those in spandrels reduce of about 25 %.

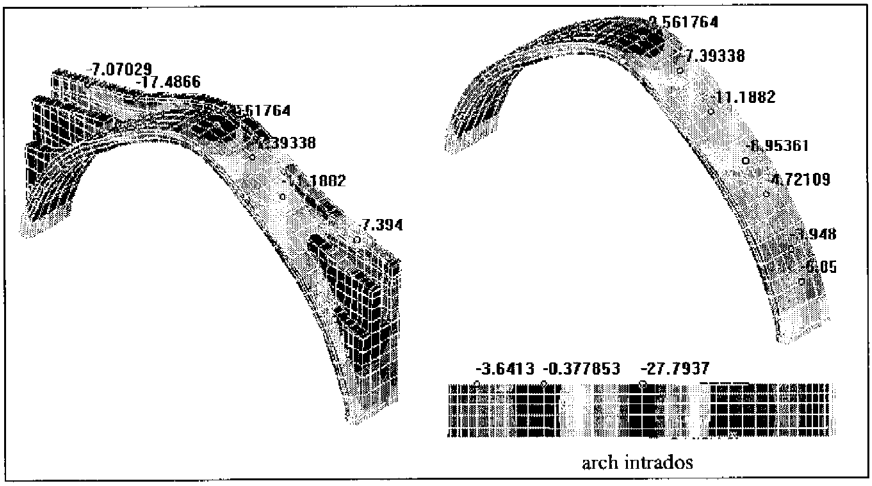

Figure 4: Maximum compressive stresses (in kg/cm^2) from analysis no. 5C.

The maximum compressive stress at the crown of the arch is –2.38 MPa in the analysis no. 5A and to –2.78 MPa in the analysis no. 5C (with a percentage variation of + 17 %), while compressive stresses in spandrels increase from –1.54 MPa (analysis no. 5A) to –1.88 MPa (analysis no. 5C) (with a percentage variation of + 22 %).

These results highlight the sensitivity of the FE model to material parameters assumed in numerical analyses. Anyway the range of variation considered for the modulus of elasticity of all elements is so large that it can be considered to cover uncertainties on the actual value of the Young modulus of all elements. Therefore, obtained results represent limit values for actual stresses in the bridge.

Table 3. Maximum compressions (in MPa) at the arch crown.

Load Comb.	Comb. of the modulus of elasticity		
	A	B	C
1	-0.67	-0.74	-
2	-0.66	-0.73	-
3	-0.90	-1.00	-
4	-0.94	-1.05	-
5	-2.38	-	-2.78
6	-1.06	-	-1.12

The thrust at abutments in the analysis no. 1B has a horizontal component of about 4500 kN and a vertical component of about 3000 kN. On both the western and eastern side the FE model includes, as described above, a 6.00 m deep

portion of abutments, so that it allowed also to evaluate the diffusion of the thrust into abutments.
Maximum compressive stresses induced by the thrust in the horizontal direction are equal to about 0.12 MPa under permanent and live loads.
The maximum compression at the arch crown is 2.78 MPa (Table 3) and, for a safety factor of three, the minimum required strength of masonry arch is about 8.4 MPa. New voussoirs will be made with Tenelija blocks, which have a strength of 20 MPa, while the mortar type and the thickness of mortar joints will be defined only in the execution phase. Using formulas from literature [9] and tables from Eurocode 6 [10] and supposing a thickness of joints of 5 to 10 mm, the required strength of the masonry can be obtained if the mean strength of the mortar is higher than 5 MPa. Once the mortar type and the thickness of joints will be defined, experimental tests on full scale specimens of the masonry arch are desirable to check the actual strength of the masonry.
The structural behaviour of the bridge under the flood force was investigated through a non linear analysis, where dead loads were applied in the first step and the flood force was applied in successive five steps, starting with the force of 4940 kN corresponding to a discharge $Q=1500$ m^3/s and ending with the force of 10560 kN exerted by the maximum discharge $Q=2500$ m^3/s, which corresponds to the maximum water level at abutments. The obtained load-displacement diagram is linear up to $Q=1750$ m^3/s, then the diagram presents a hardening branch corresponding to a reduction of the bridge stiffness. For $Q \leq 2000$ m^3/s, compressive stresses induced by dead weights at the arch crown are higher than tensile stresses produced by the flood; tensile stresses arise only for $Q \geq 2250$ m^3/s. Numerical analyses showed that the maximum flood force does not produce the collapse of the bridge.
Effects of earthquake actions on the bridge were evaluated through a simplified modal response spectrum analysis, which can be used strictly speaking only when the response is not significantly affected by higher eigenmodes. For a given direction (longitudinal, transversal or vertical) the main mode of vibration was determined, that is the mode with the maximum participation factor. Then seismic loads in that direction, proportional to the total mass, were calculated using modal displacements of the main mode of vibration. Moreover seismic actions in the three directions were applied contemporarily to the FE model according to the combination factors prescribed by EC8.
The elastic response spectrum was defined on the basis of the soil characteristics: the bridge is located in a rocky region, which was considered to belong to class A of EC8. The behaviour factor q was taken equal to one, as non linear analyses were performed. These analyses showed that the bridge can resist the 90 % of the design seismic load in the horizontal direction, that is an acceleration of 0.31 g, and more than the design seismic load in the longitudinal and vertical direction: a maximum acceleration of 0.49 g in the longitudinal direction and of 0.29 g in the vertical direction.
The total shear produced by earthquake in the transversal direction is much lower than the flood, because the flood force is concentrated prevalently near springings, where the surface of spandrels exposed to water is very large.

6 Conclusions

The structural analysis for the reconstruction design of the old bridge of Mostar highlighted the excellent structural performance of the bridge, under all load cases and combinations, although it was designed and built more than 400 years ago. The stress and strain patterns under the load cases prescribed by Eurocode 1 were assessed, allowing to give indications on the required strength of materials to be used for the new masonry. The diffusion of the thrust at abutments was also studied to allow the correct design of the abutment consolidation. FE analyses showed that the bridge can resist to the maximum flood without collapsing, but at springings it can undergo some damages, which are detrimental to its durability. For this reason in the future a careful control of dams and reservoirs along the Neretva river upstream of the bridge is desirable to avoid these damages. In safety metal dowels and cramps were not considered in the analysis, because experimental data on metal connections were not available to reliably evaluate their contribution to the shear strength and stiffness of voussoir joints.

Acknowledgements

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