Evaluating the linear behaviour of the historical Mehmed Pasha Sokolovic Bridge across the river Drina in Bosnia and Herzegovina

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Abstract A significant part of the stone arch bridges in the Balkans are composed of the vestiges of the period of Ottoman rule in the region (1354-1922). Based on the review of the literature created by the studies conducted by various researchers up to the present, a total of 308 bridges were identified including 121 in Bosnia and Herzegovina. Regarding the stone arch bridges which form an important part of the cultural heritage, the numerical calculation analyses which have been very common in recent years have both ensured a good understanding of the structural behavior of such structures and, accordingly, the identification of their construction systems, and provided the experts executing the restoration works of these bridges with important ideas in making sound decisions with regards to the intervention. This study will provide information on the comparative evaluation of structural analysis studies conducted using two different software (Lusas, Sap 2000) for the 230m-long Mehmed Pasha Sokolovic Bridge across the river Drina located in the city center of Visegrad in Bosnia and Herzegovina which is included in the UNESCO's World Cultural Heritage List and supervised by the General Directorate of Highways for Restoration Project and Implementation and correspondance between the findings obtained from each study and the damages observed on the bridge, and the contribution of these two studies to the restoration process of the bridge.

1 Introduction

The historical Mehmed Pasha Sokolovic (Drina) Bridge is located over the river Drina in the city center of Visegrad, in the east of Bosnia and Herzegovina which is located in the northwest of the Balkan Peninsula. The bridge was built by Mimar Sinan at the behest of the grand vizier Mehmed Pasha Sokolovic from 1571 to 1577, during the rule of Sultan Murad III and, once constructed, the bridge constituted the junction point of the road connecting Sarajevo to Istanbul and Croatia to Greece. This study will present the comparison of structural analysis studies conducted within the Restoration Implementation Project of the bridge in question and their contribution to the restoration process.

2 Repairs to the Bridge and its Current Status

The large-scale repairs on the bridge were conducted during the 19th and 20th centuries. For instance, in 1911 and 1912, the Austrians conducted repair works on the foundations of all abutments from A4 to A9, widened the foundations in a cascaded manner and used cement-based mortar during the repairs.

In the World War I, the abutments A3-A4 and the archs K3, K4 and K5 were demolished, the disappeared parts of the bridge were reconstructed in 1939, and in the World War II, the abutments A3, A4, A5, A6 and the archs K4, K5, K6 were completely and the archs K3, K7 were partially demolished, the richly decorated stela and the sofa were destroyed, the reconstruction of the demolished parts was started in 1949 and, instead of mortared rubble, concrete was used for the interior parts of the abutments. A new repair was performed on the foundations of abutments by M. Gojkovic in 1980-81; steel sheet piles were formed around the foundations of abutments A5, A6 and A8 and a concrete jacketing was applied in between (See Figure 2).

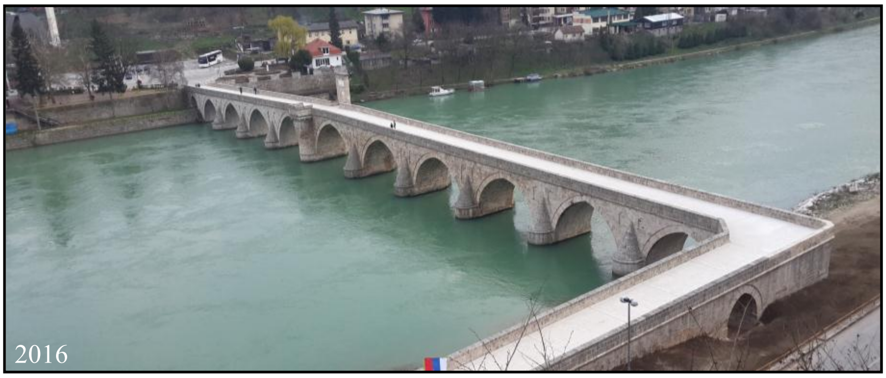

Figure 1. View of Downstream after Restoration

In 2003, the bridge was closed to vehicle traffic and, then, was included in the UNESCO's World Heritage List in 2007. Pursuant to the protocol signed between the Turkish International Cooperation and Development Agency (TIKA) on behalf of Turkey and the Commission to Preserve National Monuments on behalf of Bosnia and Herzegovina, the “Restoration Project and Implementation” of the bridge was undertaken by Turkey whereas the consulting and supervision services were undertaken by the General Directorate of Highways on behalf of TIKA, and the Restoration Implementation Projects of the bridge were achieved in 2010 and its restoration was completed as of the current year (Figure 1). [1]

3 Architecture of the Bridge and Environmental Conditions

The bridge which crosses over the Drina in the city center of Visegrad makes a right angle as it reaches the opposite shore and, then, attains the street level in curved manner. This curved part which connects the bridge to the road was designed along with the bridge as an ascending ramp to the bridge. Following the road connection, a three-arched small bridge was also constructed. The L-shaped bridge has 12 arches and is of a length of 228,6m in total, including 179,42m over the river, and of an average width of 7.25 m.

The foundations of the bridge were constructed as staged, by placing 2 or 3-lined wooden grills, which are 20 x 20 cm in size and made of pine, between the rows of stone. The average width of the abutments are 7,6m and the length of its short side ranges from 4.27m to 3.73m. They have an average height of 4.10 m. The flood splitters are in the form of isosceles triangular prism in the upstream and in the form of nonagon in the downstream.

The archs have distinct spans and heights whereas the arch K6 has the widest span, 14,74m, and the arch K11 has the most narrow span, 5,20m. The width of the eaves stones of the archs are varied between 0,30 and 0,50m and their height is 0,85m. Starting from the stirrup level, the archs were built in the form of a double-centered pointed arch (Figure 1, 2).

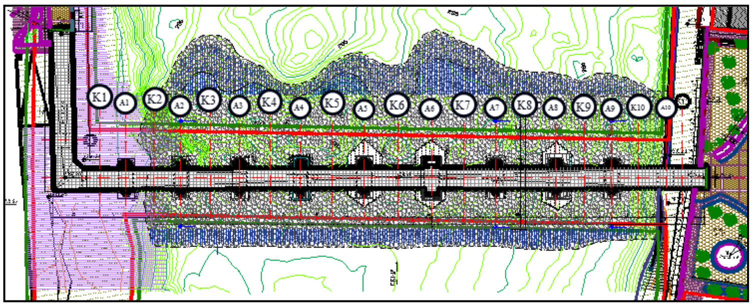

Figure 2. Restoration Layout Plan

The flow regime of the river Drina over which the bridge was built has undergone significant change owing to the large number of dams constructed on the river. Among these dams, the construction of Bajina Basta HPP on the downstream side of the bridge and the Visegrad HPP built about 2km from the upstream side of the bridge was completed respectively in 1966 and 1989. It is observed that, especially in rainy periods, the water spilled from these dams increase the scouring of the foundation. [1]

4 Soil and Building Materials

In order to identify the soil properties, support forms and building materials, a total of 12 exploratory drillings were opened, including 10 in the abutments, the depth of which ranged from 15m to 27m. The earth was excavated down to the bedrock under the foundations of the bridge abutments. Also, the drilling cores were subjected to a test in the laboratory at the university in Mostar. Although the foundation level is generally composed of gravelled soil, some differences were observed under the foundations in terms of the type of the bedrock in depth. Whereas the upstream is the bedrock limestone for the first three abutments on the left, it has a heterogeneous structure on the right side and lies in the sandstone and limestone formations. As a result of the geological surveys, it is stated that rock mass supporting the foundations is in a good condition and is able to perform its task. The mechanical values on the properties of the bedrock are given in Table 1.

Bedrock	σort. (Mpa)	σemn. (Mpa)	E (Mpa)	ε	γ (kg/m³)
Experimental	13-73	4,33-24,33	24444		2513-2673
Şekercioğlu [2]		4			
Türkçü [3]		3,5			
Ünay [4]	18-35		10000-55000		
Teymen [5]			20000		2590-2600
Goodman [6]				0,29-30	
Hunt [7]				0,25-0,33	
Hoek&Bray [8]					2300-2800
Acceptance		4	20000	0,45	2600

Table 1. Mech. Values of the Bedrock on which the Foundations were settled

Krečnjak	σort. (Mpa)	E (Mpa)	ε	γ (kg/m³)
Experimental	32,55-80,99			
Poor Foundation		5000	0,45	600
Acceptance		20000	0,45	2600

Table 2. Mechanical properties of the *Krečnjak* Stone Used in the Foundations

The values acceptable for the processed *krečnjak*, a limestone used in the foundations, are given in Table 2. The material values named as "poor foundation" due to the damage in some of the abutments were taken into account. The abutments A1 to A3 are generally settled on the bedrock whereas the abutments A4 to A9 are generally settled on the gravelled soil.

As a facing stone on other surfaces, a stone called locally "*sedra*" was used, which is a type of travertine. *Sedra* was also used in subsequent repairs made on the demolished sections of the bridge. The samples taken from *Sedra* stones were subjected to a test, and their compressive strengths as well as the average unit weights were identified. These values are shown in Table 3.

Sedra **Stone**	$\sigma_{ort.}$ (Mpa)	E (Mpa)	ε	γ (kg/m^3)
Experimental	9,71-30,53			1900-2242
Teymen [5]		14200-17400		
Acceptance		15000	0,30	2000

Table 3. Mechanical Properties of the *Sedra* Stone

The backfill material was composed of 5 to 20 cm-sized *Krečnjak* gallets and a lime mortar partially consisting of *sedra* gallets up to the level 296,50 and of 2 to 10cm-sized Krečnjak gallets, coarse sand and a mixture of limestone gallets which are bigger than 10cm in size and slag. In such backfills, the mechanical values were selected using a combined approach (See Table 4).

Backfill	E (Mpa)	ε	γ (kg/m^3)
Bardet/gravel	150-300	0,30	
Sedra	15000		2000
Kedzi/gravel			1400-2100
Acceptance	7500	0,30	1800

Table 4. Mechanical Properties of the Backfill Material

Wood	Perpendicular to the fibers $\sigma_{emn.}$(Mpa)	Perpendicular to the fibers E (Mpa)	Parallel to the fibers E (Mpa)	ε	γ (kg/m^3)
TS 647 (coniferous)		300	10000		
TS 647 (oak)	3	600	12500		
TSISO9194					640-800 sert 410-570 yum.
Jasienko [9]		400	8000	0,4	
Acceptance	3		5000	0,4	600

Table 5. Mechanical Properties of the Wood Used in Foundations

The properties of the wood used in the grids under the encasements of the bridge are summarized in Table 5. Specimens were also collected from the mortars used in the bridge and were subjected to tests. On the basis of the tests performed, the average compressive strength of mortar specimens were found to be 6.43 MPa and their density was found to be 1358 kg/m^3. [10]

5 Damage Status

According to the assessment surveys conducted before the restoration of the bridge, the deteriorations that affect most the load bearing capacity on the material and the carrier system are those formed in the foundations.

These deteriorations have burst into sight in the form of outcrop of the foundations a result of the fall of the original thalweg, the displacement due to the abrasion by water and the partial losses (See Figure 4). Also, floral deterioration and joint discharges were observed (See Figure 5).

The water seeping from the bridge deck can both trigger the vegetation and lead to the disintegration of the mortar between the stones as a result of volume expansion due to the freezing water and cause to the decomposition of the stones. The deteriorations caused by the seeping water are shown in Figures 6 and 7. [1]

Figure 4. Displacement in Foundation

Figure 5. Joint Discharge and Vegetation

Figure 6. Effect of the Seeping Water

Figure 7. Water Seeping and Freezing into the Deck

6 Structural Analysis

6.1. First Method

The analyses were first made for the damaged status of the bridge and for the post-restoration period using the LUSAS software and, a total of 11 analyses were performed for five load combinations. Accordingly, the hydrostatic force calculated considering the case where the water level rises above the bridge was applied onto the upstream surface as the compressive force, and was considered as 32,5 kN/m² from the foundation to the arch's stirrup level and as 26,25 kN/m² from the arch's stirrup level to the highest level. The displacement values based on the current status are shown in Figure 8 whereas the tensile stress values are given in Figure 9. The maximum displacement values were obtained in the section where the richly decorated stela is located and calculated as 2,87mm. The highest tensile stress values were however obtained in the damaged foundation A6 on which the richly decorated stela is located and it was found to be 1,05Mpa. For the post-restoration status, the displacement value of the same section was found to be 2,82mm and the tensile stress was found to be 0,98Mpa in A3, A4 and A8.

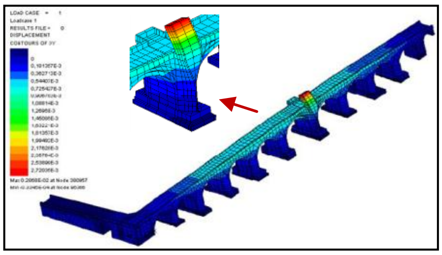

Figure 8. Pre-Rest. Displacement

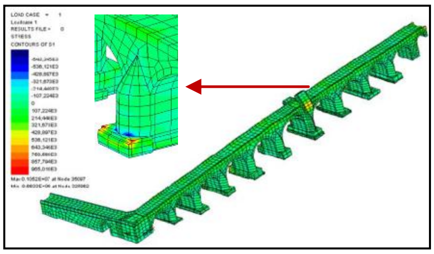

Figure 9. Pre-Rest. Tensile Stress

6.1.1 Earthquake Analysis

In terms of the seismicity of the Bosnia and Herzegovina, the map prepared by the European Seismological Commission and shown in Figure 13 was taken into account. It is seen that especially the near-coastal parts of the country are subject to a high risk of earthquake, however, this risk gradually decreases upcountry. The ground acceleration expected in case of an earthquake in the region indicated with an asterisk where the Mehmed Pasha Sokolovic Bridge is located is approximately 0.2g. This corresponds to a 3rd degree earthquake zone in Turkey in terms of maximum ground acceleration that will occur during a possible earthquake.

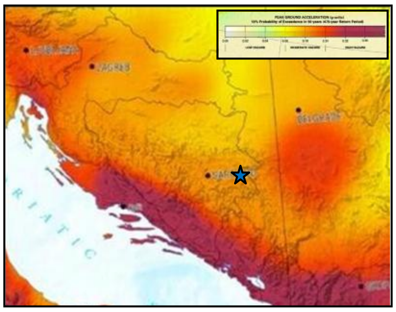

Figure 13. Seismicity of the Bosnia and Herzegovina

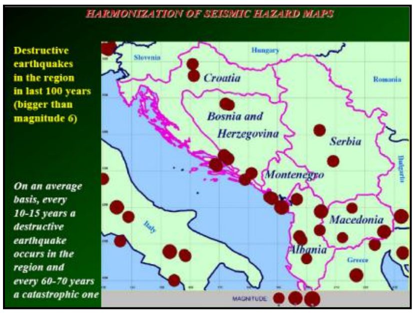

Figure 14. Earthquakes of magnitude above 6 in and around B&H

The earthquakes having a magnitude above 6 in and around Bosnia and Herzegovina which occurred during the last century are shown in Figure 14. The large-scale earthquakes generally occurred in near-coastal areas and Banja Luka, and no record of an earthquake with a magnitude above 6 was found in the region where the bridge is located. Accordingly, the earthquake analyses were conducted with accelerations of 0.2g and 0.3g using Eurocode-8 code.

The first 20 modes which will constitute a basis for the earthquake calculation and the first ten of their period and frequency values are presented in Table 6. In the first 20 modes, 80% of the active mass were exceeded. As expected in this type of masonry structures, the period values remained at very low levels.

Mode	1	2	3	4	5	6	7	8	9	10
Frequency	9,20	9,98	10,85	11,87	12,42	13,47	13,79	14,10	15,73	16,73
Period	0,11	0,10	0,09	0,084	0,081	0,074	0,073	0,071	0,064	0,060

Table 6. Frequency and period values of the first 10 modes

As a result of seismic analysis based on the design spectrum in the case of exposure to a 0.2g-accelerated motion, the highest tensile stresses occurred in the area where the richly decorated stela is located, shown with the arrow, and were obtained as 1.94 Mpa (Figure 15). This value rises to 2.91 MPa in 0.3 g-accelerated analysis. The deformations resulting from the pressure available in the bridge were concentrated in abutments A5 and A6 and they were found to be as low as 0,000024 (Figure 16). [10]

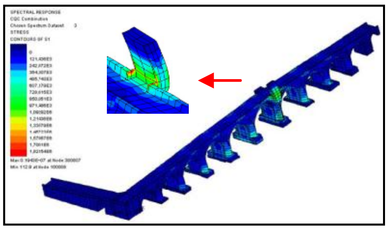

Figure 15. Pre-Rest. Tensile Stress

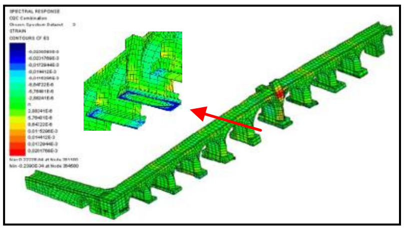

Figure 16. Pre-Rest. Deformations Resulting from the Pressure

6.2 Second Method

In this method, the analyses were individually performed for the damaged status of the bridge and after-restoration period using SAP 2000 software, and a total of 9 analyses were carried out for five different load combinations. The geometric information on the bridge previously modelled using the structural analysis software LUSAS was imported into SAP2000 software in MS-Excel format. Namely, the bridge model which was dissolved in SAP2000 software is exactly identical to the LUSAS model.

According to the results of the analysis for hydraulic load, he displacement occurred in the area shown in Figure 17 in the pre-restoration status, and the maximum value was found to be 2,72mm. The tensile stress, however, is shown in Figure 18 and found to be 1,043Mp. As a result of seismic analysis based on the design spectrum in the case of exposure to a 0.2g-accelerated motion, the highest tensile stresses occurred in the area where the richly decorated stela is located, and were obtained as 0.136 Mpa (Figure 19).

This value rises to 0.204 MPa in 0.3 g-accelerated analysis. The compressive stress however was found to be a maximum of 0,227Mpa and obtained as 0,341Mpa for an earthquake of 0,3 g acceleration (Figure 20). [10]

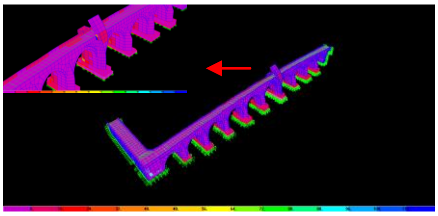

Figure 17. Displacement in Current Condition

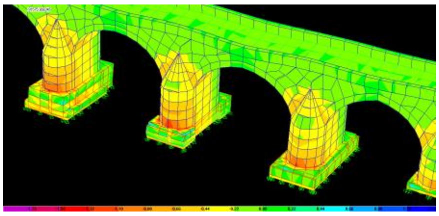

Figure 18. Tensile Stress in Current Condition

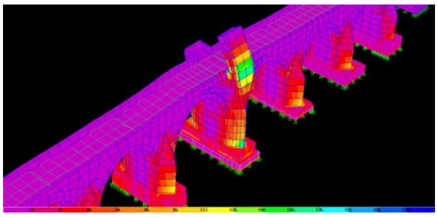

Figure 19. Pre-Restoration Tensile Stress

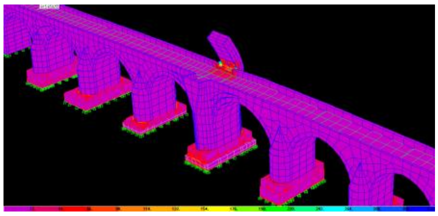

Figure 20. Pre-Restoration Compressive Stress

7 Results

For all of the analyses, the compressive stresses were compared with the allowable stresses, and no problem was observed in terms of exceeding the compressive strength. Except for the earthquake, the tensile stresses remained around 1MPa at LUSAS, however it has increased significantly in the event of earthquake and thus, the tensile strength values which are 1.94MPa for an acceleration of 0.2g and 2.91MPa for 0.3g has not reached the extreme levels. At SAP 2000, however, the values except for the hydraulic load remained below 0,56Mpa and reached 1,04Mpa in the hydraulic load analysis. In the literature, the tensile strength for this kind of stones is given as 2-6MPa. [4] Accordingly, a damage may be expected in a position in which the maximum value is attained during the earthquake conditions taken into consideration according to LUSAS. According to LUSAS, the shear stress values generally remained around 0.4MPa and attained 0.72MPa in the case of an earthquake of only 0.3g. In SAP 2000, however, the values were found to be 0,48 Mpa and below. Nevertheless, these values are also smaller than the permissible value. The shear strength for these types of rocks are given as 6 to 20 MPa. [4] Examining the deformation values obtained, the limit values were not exceeded in the current situation. In Eurocode-629, the maximum deformation of bending and stacking under pressure is taken as 0.0035. For wood, the limit value on the deformation of wood is envisaged to be 0.002 to 0.004 by California Building Code. It can be said that, in case of continuation of this situation without taking measures in the abutments, the problems will increase more and more in terms of the deformations and the safety of the bridge will become more critical. The problems in terms deformations are mostly expected to occur in the damaged abutments.

In SAP2000 software, the weights of materials are mentioned in the properties of the materials used. In other words, the user enters the values required for the materials to be used in the model. The weight data is automatically identified in LUSAS software. This is why the results obtained in linear analyses are incompatible with each other. It is also seen in the literature review that SAP2000 software is rather used for framed systems and is not much preferred for the analysis of solid models. It is also seen that especially the results of seismic analysis produced very low values in SAP 2000, however, there is a significant correlation between the values given by both software in other analysis. In addition, in case of a material nonlinearity for solid models, the SAP2000 software is not eligible for use.

Based on the results; the fact that the ground level in the river bed goes down to the foundation bed level poses a threat to the safety of the foundations and leads to scouring under the foundations. Since the foundation was designed as a rectangle, the cornerstones on the downstream side are strained in case of exposure to hydraulic effects. First all of the damages in the foundations are repaired and then the foundations are ensured to remain under the natural ground level through a stone support to all the riverbed as a raft foundation. [10] [11]

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