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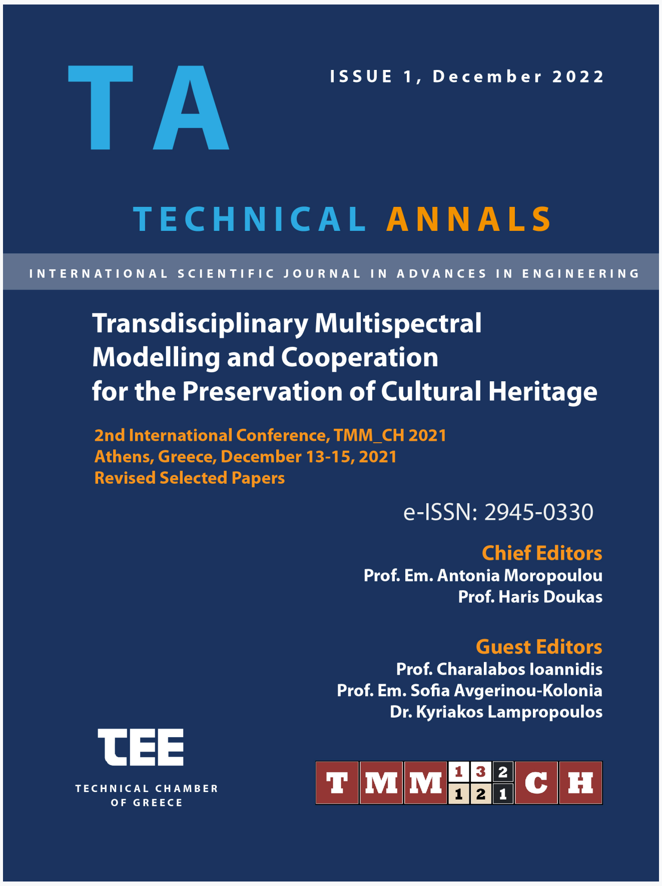

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A pointed falsework or a false decentering: Restoration and consolidation of Tsipiani bridge

Angelos Papageorgiou[1], Lampros Lolos[2]

[1] Department of Architecture, University of Ioannina, 45110 Ioannina, Greece
[2]Department of Civil Engineering, Democritus University of Thrace, Greece
architecture.uoi.gr

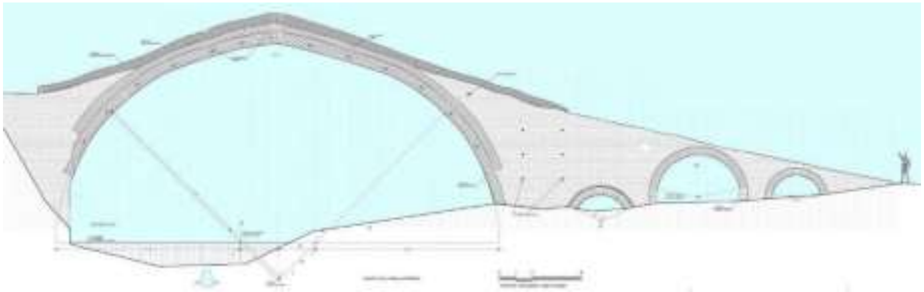
Abstract. Tsipiani bridge (1875) is located at the East Zagori region in Epirus Greece, near the village of Miliotades. It is a peculiar single-arched bridge, the arch of which appears pointed, with an opening of 26 m. Traditional stone bridges have no wedge-shaped voussoirs, and the final round shape is formed by variable mortar amount amongst the flat sandstones of the area. The current stake is the salvation of the bridge. In the institutional framework of protection, the exact wording of the question is «whether or not» the study is approved. Bridges do not stand by democratic procedures but by the decisive intervention of the engineer corresponding to that of the master builder who in this case seems to have carried out a false decentering. In this case our proposal was the method of "stitching" which is used to reinforce existing structures made of stone or brick to increase the resistance of the masonry against compressive, shear and tensile forces and to connect loose parts in the body of the masonry.

Keywords: Stone Bridge, voussoir, decentering, stitching.

1 A pointed bridge

Tsipiani bridge (1875) is located at the East Zagori region in Epirus Greece, near the village of Miliotades. It is a peculiar single-arched bridge, the arch of which appears pointed, with an opening of 26 m and a height of intrados 12,20 m as can be shown in Fig.1 (a and b). Traditional stone bridges have no wedge-shaped elements, known as voussoirs, and the final round shape is formed by variable mortar amount between the flat sandstones of the area. The average thickness of mortar is 1 cm while the sandstones are 6 cm thin. According to a 90's approach of the civil engineer Stathis Papavranousis the pointed shape of the arch is due to the compression of the mortar of the central area during an early decentering and not at the intention of the master builder. The geometry of the falsework is largely verified by the topographic measurement and the static con- tribution of the mortar has been largely discussed.

(a)



(b)

Fig. 1. Tsipiani bridge (a) Schematic representation, (b) side photo of the structure

2 Inspection

The average thickness of the rings is for the upper ring 41cm and for the lower ring 66cm. Metal keys are transversely placed on the body of the lower ring. Their purpose is to improve the compressive and shear strength of cross-sections through biaxial stress After a visual inspection that was done, four are the estimated and probable problems of stability, endurance, and static insufficiency that this bridge presents:

i. Extensive loss of sandstone bonding mortar all over the outer surface of the bridge body, see (Fig.2)

Fig. 2. Extensive loss of sandstone bonding mortar

ii. The failure in the geometry of the arch in its upper left (downstream) part which occurred during its construction. The arch section tends towards its chord as can be seen in the (Fig.3).

Fig. 3. The arch section tends towards its chord

iii. The loss of contact between the two rings throughout the upper part of the arch due to the above-mentioned failure can be seen in (Fig. 4). The consequence of this is that the loads of the deck are transferred exclusively from the upper ring, subjecting it to intensive sizes disproportionate to its geometry.

Fig. 4. Loss of contact between the two rings

iv. The revelation of the foundation to the right (downstream) of the bridge body with the simultaneous revelation of the rock on which it is founded can also be seen in (Fig. 5).

Fig. 5. Revelation of the foundation

Due to (i) the initial mechanical operation of the "stone-mortar" system has been reduced to an unknown degree.

Because of (ii) and (iii), as shown by the static solutions, the eccentricity of the thrust line in the larger upper part of the arch exceeds in both rings ½ of their cross section. Specifically, for the upper ring we have a stability problem for the permanent actions (Same Weight) and for the design earthquake, while for the lower ring only for the design earthquake. The above results in the creation of internal joints [1], [2], [3], even- tually turning the larger upper part of the arch into a mechanism.

The revelation of the foundation (iv) does not currently raise the issue of the stability of the bridge, without this ruling out a future problem due to its possible under excavation.

3 Solution

The proposed solution was chosen to be compatible with the architectural and traditional constraints while maintaining the shape of the bridge, which is part of the history of the area. The aim of the solution is to reduce as much as possible the problems of stability, endurance, and static insufficiency that it presents. The following are suggested:

(i) The replacement of the remaining and the lost external bonding mortar of the stones. The joints are expected to have a penetration depth in the body of the bridge 3-5cm. Of course, the final depth of the joints depends on the condition of the individual parts of the bridge body, but deep joints are not appropriate because they can lead to loosening of the coherence [2].

(ii) The connection of the two rings of the arch with reinforcing bars, which will have blades at their ends. The new agglomerate that will emerge aims to achieve the static function that a (single) ring 107cm thick (average thickness) would have. At the same time, the gap created between them will be filled with mortar.

(iii) It is recommended that the transversely placed metal keys on the body of the lower ring be temporarily not moved, due to the unpredictable behavior of the stone blocks by the disturbance. After successful rehabilitation, if their presence is deemed unnecessary, it is recommended their gradual removal while monitoring changes in the intensive condition (eg local cracks).

(iv) Construction of a stone wall around the revealed foundation if an excavation problem arises in the future. This wall, if the need for its construction is judged, will be founded at the level where the rocky ground will be found.

The above solutions concern the static image of adequacy presented by the Tsipiani bridge macroscopically (visual control) but also computationally (paragraph 8).

4 Computational model – Data

The simulation of the stone bridge was done with surface finite elements 2.85m thick (average bridge thickness) while the simulation of the rings in the length that have lost contact with each other was done with rods for the two different phases, before and after the restoration of the arch.

For the prearch restoration phase, the upper ring was simulated with 34 0.41m (h) x2.85m (b) cross-section bars and the lower ring with 34 0.66m (h) x2.85m (b) cross-section bars. For the postarch restoration phase, the new single ring was simulated with 33 bars of cross section 1.07m (h) x2.85m (b).

The absence of laboratory data on the mechanical characteristics of our body forces us to refer to the international literature for their conservative determination. According to [4], [5], [6, [7], a measure of elasticity E = 3.00 GPa is chosen and a Poison ratio ν = 0.20. The coefficient of seismic behavior was obtained equal to q = 1.00. The same weight was set g = 22 KN / m3 while the compressive strength of the carrier for the solution was considered infinite [8], [9], and for the control of the cross sections equal to 7.72MPa (Table 1). Finally, the institution was considered to be rooted in its foundation.

5 Justification of computer model – Comments

There are three proposed methods for solving such vectors [1]. Linear elastic analysis with finite elements, "limit block analysis" and non-linear analysis with finite elements. Static solution is done by linear elastic finite element analysis [7], [1]. The static "limit block analysis" according to [1], [2], [3], which is based exclusively on Heyman's theory [8] cannot be applied here because the bridge is not subject to any significant mobile load, while the solution with non-linear finite element analysis [6] would be accurate with the strict condition of the laboratory determination of the mechanical characteristics of the bridge but also the knowledge of the history of the plastic movements, otherwise the solution with estimates it is very likely to lead to erroneous results [10].

The results of the static solution will be interpreted based on the assumptions of the theory of Castigliano [9] and Heyman [8]. The assumption of infinite compressive strength of the carrier is necessary for the solution. Solving an estimate of a value for compressive strength turns the problem into a nonlinear one [1] and if the estimate is not correct, as mentioned above, it leads to erroneous results. The number of ring simulation bars, according to [11], is sufficient to simulate their behavior.

As can be seen from the static analysis of the model, the eccentricity of the thrust line at the critical cross-section of the arch is now less than ½ of the new cross-section and thus at least part of the surface of the critical cross-section of the arch is operated under compression [2]. This, in combination with the control that the compressive stresses do not exceed the value of 7.72 MPa, certify the adequate operation of the arch, as it emerged after the proposed interventions, but also of the body as a whole. A prerequisite, of course, is the successful application of the armatures connecting the rings together.

The construction was considered to be fixed at the level of the foundation. More precise solution using a model on elastic ground requires the determination of soil characteristics after geotechnical research.

6 Upper and lower ring connection armature

As mentioned above, the solution of connecting the rings with reinforcement rods was chosen, which will have anchor plates at their ends. The difficulty arises in determining their density (number of bars / m2) because it is a purely empirical method [2], [12].

In the upper part of the arch are placed bars generally 4Φ12 / m2, B500C [2], [3], which at each end have blades measuring 100x100x20mm. In the rest of the arch where it is not possible to place blades on both sides, 4Φ12 / m2 bars, S500s with a length of 2.30m are placed, on which a dimension blade is placed at their free end.

According to [12], if the operation of the arch after the installation of the rods is not sufficient (eg unacceptable deformations, cracks, etc.) additional rods must be added.

The holes that will be drilled for the placement of the reinforcements will have a diameter of 20mm. The mortar pressed into the hole will be pure cement mortar with a watercement ratio of 1.0: 1.5. [2]

7 Compressive strength of load-bearing masonry - Selection of mortar for grouting

Table 1 below is from [5] and has been derived from the application of his semi-empirical equations [13] to determine the strength of load-bearing masonry if the compressive strength of stones and mortar is known. Here the calculations have been made for compressive strength (fb) of the stones from 15 to 22 MPa and quality mortars M1.2, M2.5 and M5 according to Eurocode 6.

Table 1. Compressive strength of load-bearing masonry for use of M1.2, M2.5, M5 cements and stones with fb = 15 to 22 MPa

Stones	Mortar		
f_{bk}	M1.2	M2.5	M5
15	5.45	6.08	7.30
16	5.78	6.40	7.62
17	6.10	6.73	7.95
18	6.42	7.06	8.27
19	6.75	7.38	8.60
20	7.07	7.71	8.92
21	7.40	8.03	9.25
22	7.72	8.35	9.57

The compressive strength of the stones, the compressive strength of the existing grout, the depth of grouting and the mechanical characteristics of the stone-mortar system, before and after grouting, are virtually unknown.

A schematic representation of Papavranousis 1996 [11] static model approach can be seen in (Fig. 6).

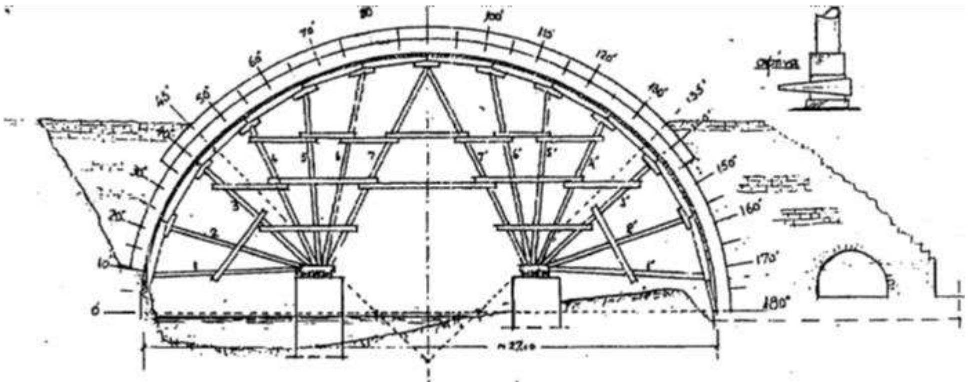


Fig. 6. A schematic representation of Papavranousis [13] approach

According to [11] the binder was created from river sand and lime in a large proportion. This allows us to estimate its relatively low strength. The following composition is proposed for the new mortar: 1 part Portland cement, 3 parts lime and 9 parts sand, ie M2.5 quality mortar ([2]).

The compressive strength of moderately strong unbreakable sandstone in uniaxial compression is less than 50 MPa [14]. Compressive strength is obtained for the stones of the 22 MPa system due to uncertainties about their condition.

From the above and from the results of Table 1 it is estimated that the compressive strength of the load-bearing masonry has as an estimated minimum the value of 7.72 MPa.

8 Calculations - Check at critical sections of the arch after restoration

The Position of checking points of critical sections of the arch after restoration can be seen in (Fig. 7).

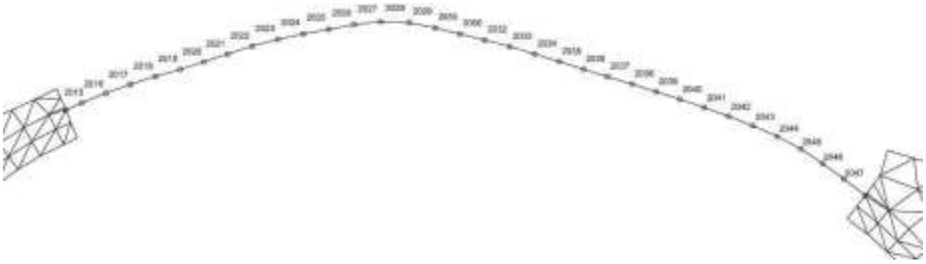


Fig. 7. Position of checking points of critical sections of the arch after restoration

Table Rod 2020 (Position of maximum positive torque against permanent actions for the left upper part of the arch).

Permanent Actions

Intensive design sizes: Msd = 111.32 KNm, Nsd = -1067.04 KN

Eccentricity: $e=\frac{Msd}{Nsd}=\frac{111.32}{-1067.04}=0.104<\frac{b}{6}=\frac{1.07}{6}=0.178$

The cross section is in complete compression.
Cross-sectional area = 1.07 * 2.85 = 3.05 m2
Compressive cross section voltages = 1067.04 / 3.05 = 350 KN / m2 = 0.35MPa <7.72 MPa
The cross section is sufficient

Seismic Actions
Maximum torque within the level (M2max)
Intensive design sizes: M2sd = 269.30, M3sd = -101.90 KNm, Nsd = -878.46 KN

Eccentricities: $e_{M2}=\frac{M2sd}{Nsd}=\frac{269.30}{878.46}=0.307<\frac{b}{2}=\frac{1.07}{2}=0.535$

$e_{M3}=\frac{M3sd}{Nsd}=\frac{101.90}{878.46}=0.116<\frac{b}{6}=\frac{2.85}{6}=0.475$

The cross section is in partial compression.
Active cross-sectional area = (1.07-2 * 0.307) * (2.85-2 * 0.116) = 1.193 m2 Compressive stresses of active cross section = 878.46 / 1.193 = 736 KN / m2 = 0.74MPa <7.72 MPa
The cross section is sufficient

Maximum off-level torque (M3max)
Intensive design sizes: M2sd = 167.34, M3sd = -416.86 KNm, Nsd = -1047.26 KN

Eccentricities: $e_{M2}=\frac{M2sd}{Nsd}=\frac{167.34}{1047.26}=0.160<\frac{b}{6}=\frac{1.07}{6}=0.178$

$e_{M3}=\frac{M3sd}{Nsd}=\frac{416.86}{1047.26}=0.398<\frac{b}{6}=\frac{2.85}{6}=0.475$

The cross section is in complete compression.
Cross-sectional area = 1.07 * 2.85 = 3.05 m2
Compressive cross section stresses = 1047.26 / 3.05 = 343 KN / m2 = 0.34MPa <7.72 MPa
The cross section is sufficient

Bar 2028 (Arch key - Position of maximum negative torque against permanent actions)
Permanent Actions
Intensive design sizes: Msd = -157.81 KNm, Nsd = -1017.83 KN

Eccentricity: $e=\frac{Msd}{Nsd}=\frac{-157.81}{-1017.83}=0.155<\frac{b}{6}=\frac{1.07}{6}=0.178$

The cross section is in complete compression.
Cross-sectional area = 1.07 * 2.85 = 3.05 m2
Compressive cross section stresses = 1017.83 / 3.05 = 334 KN / m2 = 0.33MPa <7.72 MPa
The cross section is sufficient

Seismic Actions
Maximum torque within the level (M2max)
Intensive design sizes: M2sd = -284.83, M3sd = 114.90 KNm, Nsd = -1244.85 KN

Eccentricities:

$$e_{M2}=\frac{M2sd}{Nsd}=\frac{284.83}{1244.85}=0.229<\frac{b}{2}=\frac{1.07}{2}=0.535$$
$$e_{M3}=\frac{M3sd}{Nsd}=\frac{114.90}{1244.85}=0.092<\frac{b}{6}=\frac{2.85}{6}=0.475$$

The cross section is in partial compression.
Active cross-sectional area = (1.07-2 * 0.229) * (2.85-2 * 0.092) = 1.632 m2 Compressive stresses of active cross section = 1244.85 / 1.632 = 762 KN / m2 = 0.76MPa <7.72 MPa
The cross section is sufficient

Maximum off-level torque (M3max)
Intensive design sizes: M2sd = -229.96, M3sd = -383.01 KNm, Nsd = -1119.31 KN

Eccentricities:

$$e_{M2}=\frac{M2sd}{Nsd}=\frac{229.96}{1119.31}=0.205<\frac{b}{2}=\frac{1.07}{2}=0.535$$
$$e_{M3}=\frac{M3sd}{Nsd}=\frac{383.01}{1119.31}=0.342<\frac{b}{6}=\frac{2.85}{6}=0.475$$

The cross section is in partial compression.
Active cross-sectional area = (1.07-2 * 0.205) * (2.85-2 * 0.342) = 1.43 m2
Compressive stresses of active cross section = 1119.31 / 1.430 = 782 KN / m2 = 0.78MPa <7.72 MPa
The cross section is sufficient

Bar 2037 (Position of maximum positive torque against permanent actions for the upper right part of the arch)
Permanent Actions
Intensive design sizes: Msd = 266.09 KNm, Nsd = -1067.37 KN

Eccentricity:

$$e=\frac{Msd}{Nsd}=\frac{266.09}{-1067.37}=0.249<\frac{b}{2}=\frac{1.07}{2}=0.535$$

The cross section is in partial compression.
Active cross-sectional area = (1.07-2 * 0.249) * 2.85 = 1,630 m2
Compressive stresses of active cross section = 1067.37 / 1.630 = 654 KN / m2 = 0.65MPa <7.72 MPa
The cross section is sufficient

Seismic Actions
Maximum torque within the level (M2max)
Intensive design sizes: M2sd = 481.95, M3sd = -86.79 KNm, Nsd = -944.19 KN
Eccentricities: eM2 = = = 0.510 <= = 0.535
eM3 = = = 0.092 <= = 0.475
The cross section is in partial compression.
Active cross-sectional area = (1.07-2 * 0.510) * (2.85-2 * 0.092) = 0.133 m2
Compressive stresses of active cross section = 944 / 0.133 = 7082 KN / m2 = 7.10MPa <7.72 MPa
The cross section is sufficient

Maximum off-level torque (M3max)
Intensive design sizes: M2sd = 307.67, M3sd = -289.32 KNm, Nsd = -961.70 KN

Eccentricities:

$$e_{M2}=\frac{M2sd}{Nsd}=\frac{307.67}{961.70}=0.320<\frac{b}{2}=\frac{1.07}{2}=0.535$$
$$e_{M3}=\frac{M3sd}{Nsd}=\frac{289.32}{961.70}=0.301<\frac{b}{6}=\frac{2.85}{6}=0.475$$

The cross section is in partial compression.
Active cross-sectional area = (1.07-2 * 0.320) * (2.85-2 * 0.301) = 0.966 m2
Compressive stresses of active cross section = 961.70 / 0.966 = 995 KN / m2 = 0.99 MPa <7.72 MPa
The cross section is sufficient

As revealed by the above calculations, the mean of the approximately straight section which resulted from the failure of the arch geometry (upper right section), is the most critical cross section.

9 Reinforcement procedure - Installation of reinforcement (root reinforcement)

9.1 General [2]

The method of root reinforcement (stitching) is used to reinforce existing structures made of stone or brick to increase the resistance of the masonry against compressive, shear and tensile forces and to connect loose parts in the body of the masonry. The technique was developed by the Italian Lizzi in 1952 to reinforce historic Italian structures that had been severely damaged during World War II.

It is a method of stabilizing the masonry by inserting steel reinforcing bars or anchors in a defined manner into the body of the masonry. The calculation of the strength of an element of masonry or brickwork to which the method of stitching has been applied is not practically possible as it depends on the existence of gaps, the variation of the strength of the mortar and the wall, the way of construction, etc., so the application stitching is more of an art than a science and the success of the method is based more on experience than on calculations.

Knowledge of the causes of wear, the general condition of the masonry and the permissible change in loads are some of the factors that determine the course of work for the application of the method while the absence of regulations makes it necessary to have experienced personnel about the method. The diameter of the holes that are drilled for the installation of the root equipment is of the order of 20 - 40mm, and their length varies depending on the thickness of the element and the nature of the construction problems, but must be sufficient to ensure the overlap of the reinforcement. , whose diameter ranges between 12-20 mm.

The number of holes or bars per unit area depends on the condition of the construction and the reason for the reinforcement. Approximately it is recommended to place 3 or 4 bars per m2, about three times the thickness of the masonry. Reinforced steel reinforcement ensures better cohesion and anchorage but in monuments and structures in wet environments it is recommended to use stainless steel.

The mortar pressed into the hole is usually pure cementitious with a watercement ratio of 1.0: 1.5. Mixing with sand is allowed only if there are large gaps in the body of the masonry. Epoxy or other polymeric resins can also be used if it is deemed necessary to use them for a large increase in the strength of the wall. However, their use is not recommended if the percentage of gaps in the masonry exceeds 3% -5% of its volume, so it is necessary to use grout with properties more compatible with the masonry, ie cement mortar. The method of root equipment is successfully applied in constructions with masonry thickness of 0.5-2.0m and finds application in the stabilization of arches that have undergone deformations.

In vulnerable structures such as the arched part of the bridge, the holes are drilled using electric rotary drills with a diamond head and water inlet to cool the head and remove drilling materials. The use of drills of this type does not cause major damage. The direction of drilling the holes is from bottom to top, i.e. from the lower sole of the arch to the deck.

Medium-sized structures can be drilled using electric rotary-impact drills. Vibrations from the use of these drills are not capable of causing damage to most structures, but in cases of very weak masonry, special care must be taken to avoid minor damage.

The use of compressed air drills is only permitted in solid structures, especially if they are made of very hard stone masonry and long holes must be drilled.

The next step follows after making several holes in one surface. The reinforcement enters the holes and the process of inserting the grout is prepared. First water enters the hole to remove loose materials and then the injection of grout, which starts from the lowest points and proceeds upwards.

The equipment for the cement mortar consists of a mixer, a storage tank of the mixture and a pump which can be motorized or manual. In cases of fine-grained grouts, it is better to use a hand pump that allows better control of the impregnation process. The filling of the holes with grout is done under low pressure, usually 1-2 atm, but the pressure gauge that measures the operating pressure must be placed close to the nozzle to measure the actual pressure.

At the beginning of the impregnation the pressure is up to 0.30MPa and is kept constant until the grout is absorbed. It is then raised to 0.40MPa and held steady for 5-10 minutes until the mixture solidifies and the excess water is drained.

High pressure can create problems in low strength stone blocks, so the above sizes will be taken into account at work.

Anchor systems can be used in a housing containing a strong expandable mortar so that when the housing is broken the hole with the material is filled.

9.2 Holes & reinforcement grid

Holes with a diameter of 20mm are drilled for the installation of rod-shaped reinforce- ment Φ12 / B500C. The holes are in a grid by X (along the arch) per 1m (distance measured at the lower foot of the arch) and by Y (across the width of the arch) per 0.42 m (6 pieces by width per meter of lower foot length).

Thus 6 rows of rods were created. The directions of the rows alternately are + 45o & - 45o with respect to the lower foot of the arch. The same address is maintained for the entire series.

This grid was chosen so that, if necessary, there would be free space available to thicken the reinforcement. Details are presented in the Reinforcement Plan.

9.3 Supports [15]

The restoration process requires the support of the bridge arch. In general, any damaged component must be secured immediately by supporting to discharge it. With the support, that is, an alternative way of charging and load relief of the damaged element is achieved.

The arch of said bridge even if we ignore its loading history (which is essentially unknown), due to 2 (ii) & 2 (iii), has at least two estimated plastic joints in the upper ring (upper arch tread) as can be seen in (Fig. 8), generative cause of which is the failure in geometry and the self-weight loading.

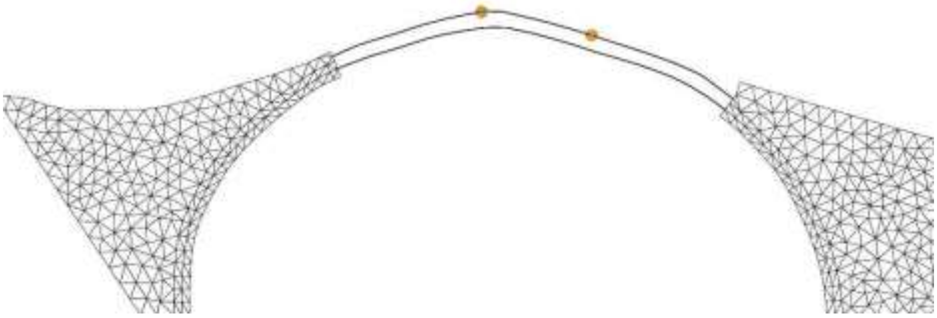


Fig. 8. The two estimated plastic joints in the upper ring

The two above joints have turned the upper foot into a mechanism and the fact that it does not collapse is due solely to the frictional forces acting between the stone blocks. Meanwhile, the distribution of the self weight principal tensile stresses σ_1 can be seen in the (Fig. 9).

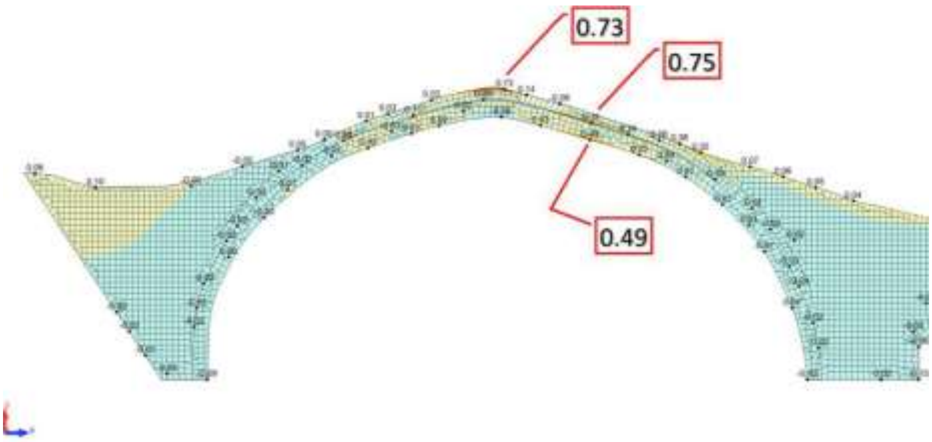


Fig. 9. Self-weight principal tensile stresses σ1 (MPa)

The role of support during the repair-reinforcement

. The support in principle ensures that the behavior of the lower arch will be the same as it would be if there was no failure in its geometry (ignoring other faults due to unknown charging history). During the restoration, both by drilling the holes and by pressurizing the grout to fill the hole, the possibility of disturbing the balance of the upper ring which would result in slipping cannot be ruled out. and consequently, the loading of the lower ring with the loads of the stone blocks of the upper ring, thus subjecting the lower ring to additional loading with unknown consequences (e.g. possible collapse of the lower ring as well).

The role of support during the work is twofold.

. On the one hand to ensure the integrity of the lower ring from possible slipping of the upper ring on it during the work and on the other hand to protect the staff who will work under the arch to open or fill the holes. In short, during the repair-reinforcement works the role of the pillar is in principle passive (in business as usual conditions) with the possibility of turning it into an asset if specific failures occur.

The role of support after the work is completed

As mentioned, monitoring of the arch behavior (e.g. cracks due to sinking deformations) is required after the work is completed to determine if further reinforcement is required. In this case the role of the pillar is in principle passive (prevents the increase of deformations outside tolerable limits) with the possibility of turning it into an asset in the extreme scenario of failure of the restoration. In short, after the work is completed, the support is the only way to control and improve the proposed reinforcement.

10 Premeasurement of reinforcement bars and anchor blades

A side schematic representation of the reinforcement bars and anchor blades can be seen in (Fig. 10).

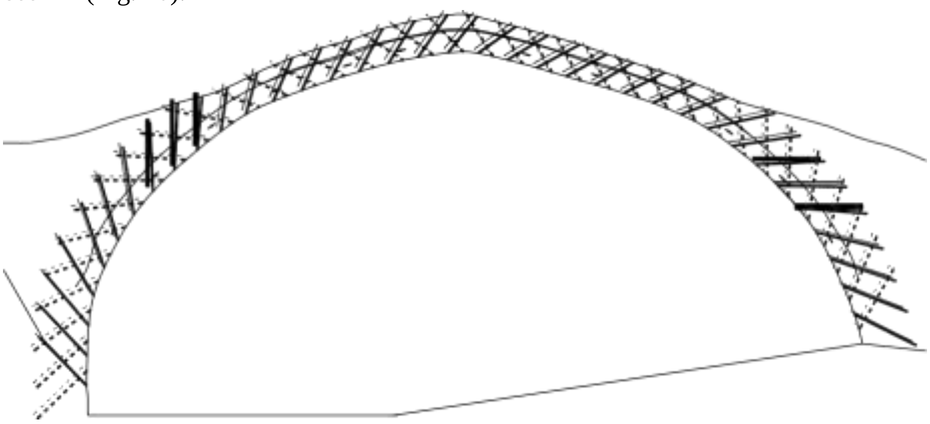


Fig. 10. A side schematic representation of the reinforcement

Metal rods $\Phi 12$ / B500C of different lengths and metal anchor blades
100x100x20mm
/ Fe 360 are installed.
Measurement of total length of metal bars
216 rods with an estimated total length of 420m
Total number of blades 100x100x20
327 pieces

11 Regulations

1. Regulation for Loading of Construction Works (BΔ 10-12-1945 Government Gazette 171 A / 1946)
2. Hellenic Earthquake Regulation 2000 (Government Gazette 2184 B / 20-12-99)
3. Amendments of EAK 2000: Government Gazette 781 B / 18-6-2003, 1154 B / 12-8- 2003, 447 B / 5-3-2004
4. Eurocode 6

12 Self-weight load - Comparative results of principal tensile stresses σ1

The results of principal tensile stresses σ1 due to Self-weight load before and after the application of reinforcement are shown in (Figures 11 and 12) respectively.

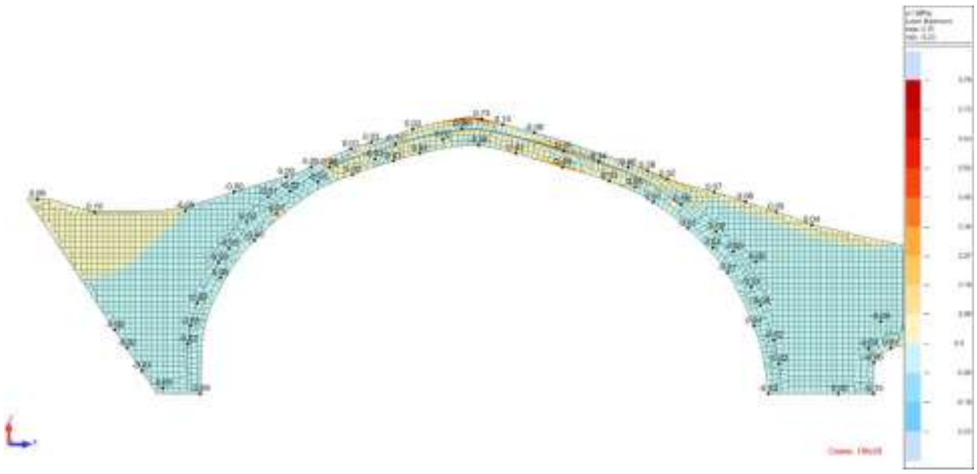


Fig. 11. Results before reinforcement: Self weight load. Principal tensile stresses σ1 (MPa)

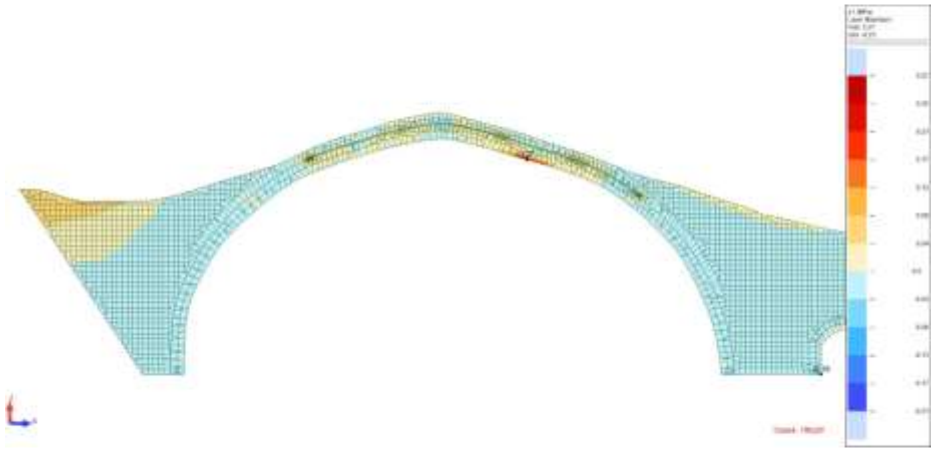


Fig. 12. Results after reinforcement: Self weight load. Principal tensile stresses σ1 (MPa)

13 Seismic design load - Comparative results of principal tensile stresses σ1

The results of principal tensile stresses σ1 due to Seismic design load before and after the application of reinforcement are shown in (Figures 13 and 14) respectively.

Fig. 13. Results before reinforcement: Seismic design load. Principal tensile stresses σ1 (MPa)

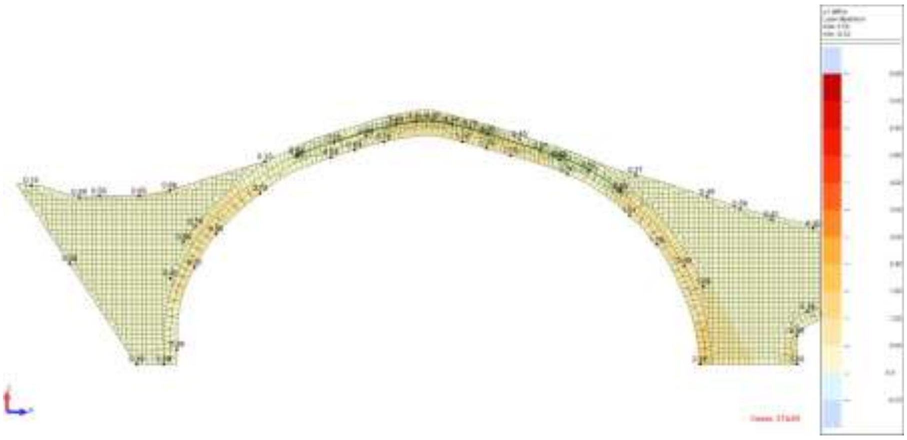


Fig. 14. Results after reinforcement: Seismic design load. Principal tensile stresses σ1 stresses σ1 (MPa)

14 The administrative adventure

The whole architectural and static study was first submitted to the Municipality of East Zagori in 2006. It was forwarded to the supervisor, then head of the Ephorate of Modern Epirus Monuments of the Ministry of Culture, who, according to the Ministry of Public Works [14], “omitted all his obligations”. The result was the automatic receipt of the study.

Nevertheless, the study was re-examined by the Council of Modern Monuments and Technical Works of Epirus. In its opinion [15] it is stated that "the arch consists of wedge-shaped slates… The condition of the bridge is quite good and does not present static problems to date…" Concludes the approval of the study with the following, inter alia, remark: "To take care of the cleaning of the stone structures from the vegetation… and not to do any work for the joining of the rings, because this is a given situation from the construction of the bridge, has acquired its static balance… »

Respectively, the General Directorate for the Restoration, Museums and Technical Works of the Ministry of Culture [16], points out: "To analyze the current situation… to carry out laboratory research on the spot… to take into account the remarks of the Service of Modern Monuments and Technical Works of Epirus, with which we fully agree. In the summer of 2014, mortar studies were carried out which did not add key elements to the static approach.

The solution to the problem of the arc that became a string lies in the logic and sensitivity of the Central Council of Modern Monuments that will examine the issue, hopefully before the collapse of Tsipiani bridge.

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10. , Castigliano approach: (a) Tangential movements between lithosomes are not allowed (b) the compressive strength of the interface between lithosomes is infinite (for cases where either the bonding cement is absent or weakened). Sofianos, A.: «Antochi ariktou petromatos», EMP – Sxoli Michanikon Metalion-Metalour- gon
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