

Structural analysis and strengthening intervention of the multispan stone masonry bridge of Ribellasca, between Italy and Switzerland

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ABSTRACT: The multispan stone-masonry bridge was built in 1906 in the Ribellasca valley, along the border between Italy and Switzerland. The bridge is 100 m long and its five arches are supported by stone-masonry pillars which reach the maximum height of 25 m.

After a preliminary historical analysis of the construction and restoration phases, a wide experimental investigation, including in situ and laboratory tests, was carried out in order to analyse the static and dynamic behaviour of the bridge. Coring and video-camera survey were used to analyse the structural characteristics of the masonries and flat-jack tests to measure of the state of stress and the deformability characteristics. The vibration modal shapes of the bridge were then determined by in situ dynamic test. All the experimental investigations as well as the installation of the accelerometers for the dynamic test were carried out with the aid of specialized climbers without any scaffolding.

Owing to the difficulties of calculation of an historical bridge, it seemed advisable to compare the results of two different types of structural analysis: “rigid block analysis” proposed by Gilbert and “non-linear analysis with finite element model” which was carried out by using the mechanical parameters obtained by the experimental investigations.

The two types of calculation which were used for the structural analysis of the bridge lead to convergent results and clearly show that the structure does not respect the safety standards required by the regulations in force. For this reason a strengthening intervention has been proposed and the design of the intervention has been developed in order to induce negligible modifications of the actual structural behaviour of the bridge and to preserve at the same time the visual impact of the structure.

1 FOREWORD

The multispan masonry arch bridge of Ribellasca, between Italy and Switzerland was built in 1906 (in figure 1a a photograph taken during the construction phase is shown) and after a century, significant damages were observed, mainly due to the water seepage through the waterproofing layer of the bridge (figure 1b).

The bridge is 100 m long and 5 m large and its five arches (three of them with a span about 20 m) are supported by stone-masonry pillars which reach the maximum height of 25 m.

In order to analyze the static capability of the bridge according to the actual safety standards, an experimental investigation was carried out as well as a static and dynamic numerical analysis.

2 EXPERIMENTAL INVESTIGATIONS

After a preliminary historical research and a detailed examination of the original design drawings of the bridge (figure 2), the damages and the crack pattern observed in the supporting struc-

tures (pillars and arches) advised to carry out a detailed experimental investigations program. All the investigations were carried with the aid of specialized climbers without any scaffolding (figure 3)

At first, a detailed geometric survey was carried out in order to obtain a three-dimensional model of the bridge (figure 4)

The composition and the structural characteristics of the masonry were investigated by means of coring and video-camera survey. Some radial corings starting from the intrados of the arches have revealed the significant degradation of the original masonry and in particular of the mortar which covers the zone of the arches over the pillars (figure 5).

The foundation level was determined by means of vertical corings and video-camera survey.



Figure 1 : (a) Photograph taken during the construction of the bridge of Ribellasca in 1906.
(b) Particular of the damages observed today (very large transversal deformation of the lateral walls).

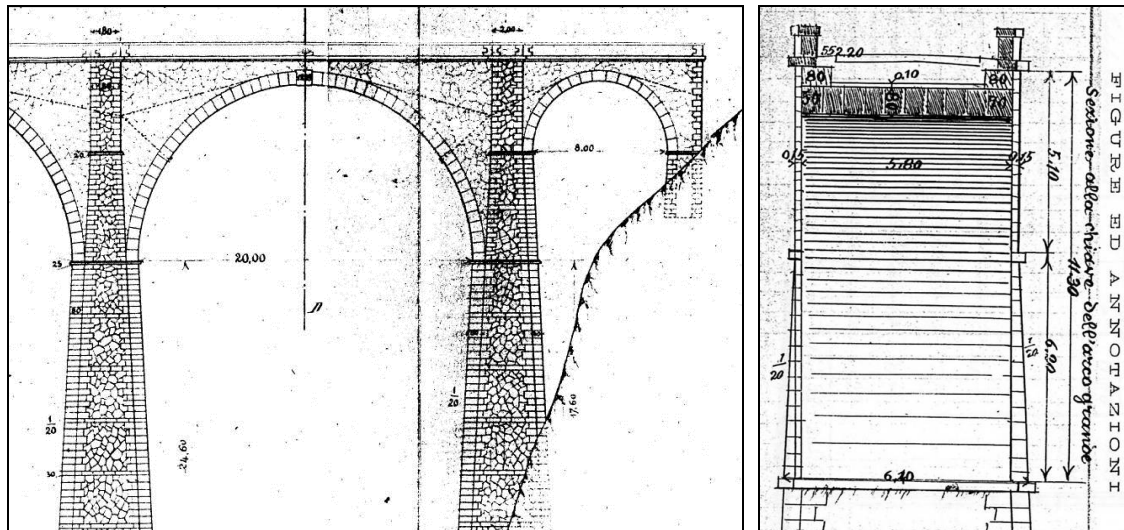


Figure 2 : Two particulars taken from the original design drawings.

The state of stress of the bridge was measured at different heights by using the flat-jack testing technique. The values of the state of stress measured at the base of the pillars and of the main arches are shown in figure 6. By using two parallel flat-jacks, the deformability characteristics of the masonry were determined (figure 6).

The mean value of the strength of the masonry, determined by flat-jack tests, is about 4.0 N/mm^2 . Starting from this mean value it is possible to determine a characteristic value of the strength of 3.0 N/mm^2 (about 75% of the mean value). Accepting a safety factor variable from 2 to 3, the design strength of the masonry is included in the range between $1 \div 1.5 \text{ N/mm}^2$.

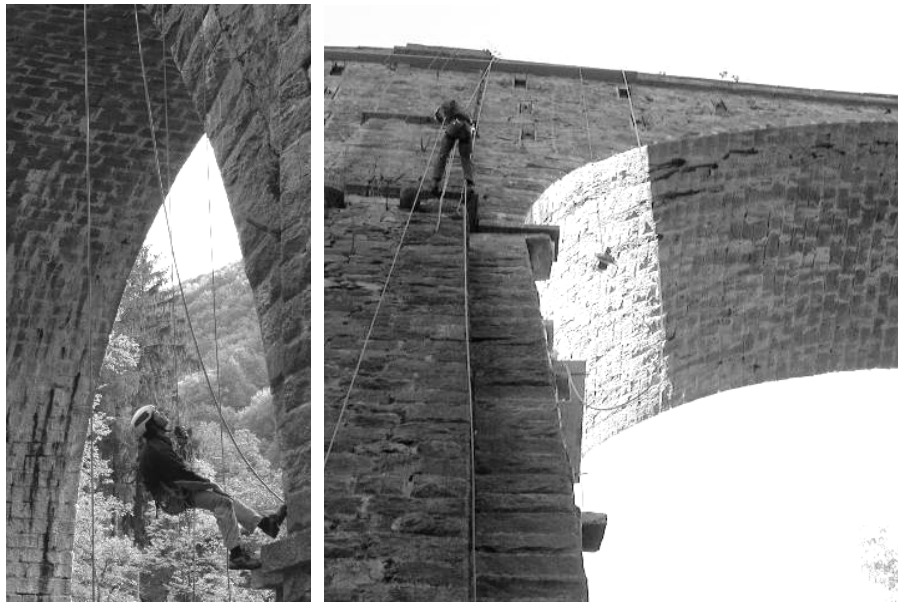


Figure 3 : Crack pattern survey and experimental investigations carried on with the aid of climbers.

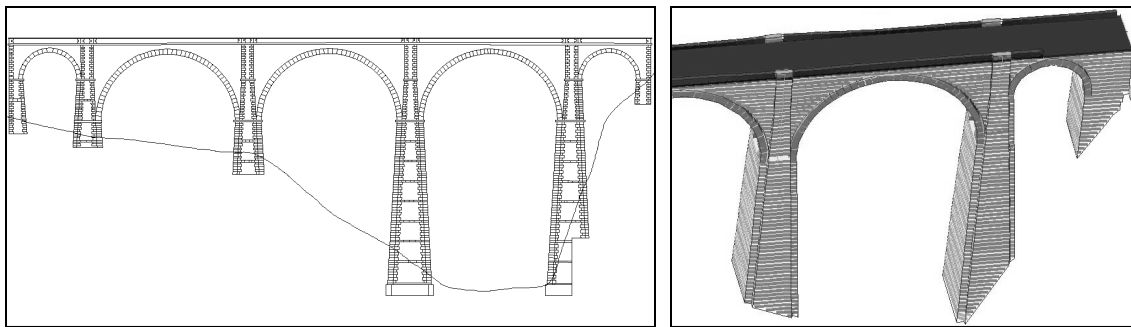


Figure 4 : Geometric survey and three-dimensional model of the bridge.

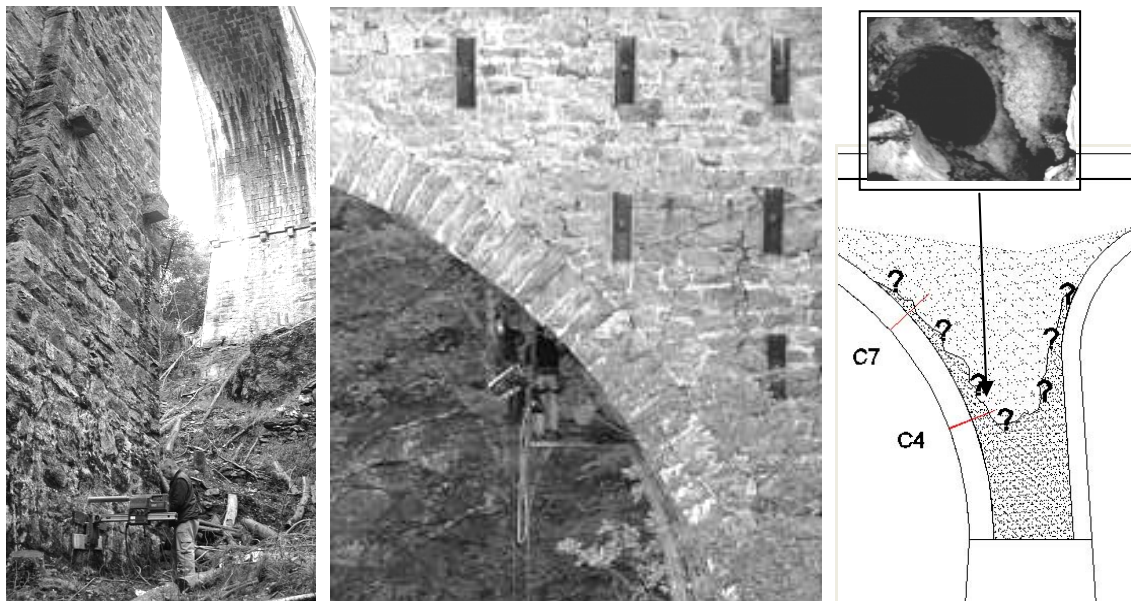


Figure 5 : Coring and video-camera surveys at the base of a pillar and on the intrados of the arches to analyse the composition and the structural characteristics of the masonry and the filling materials

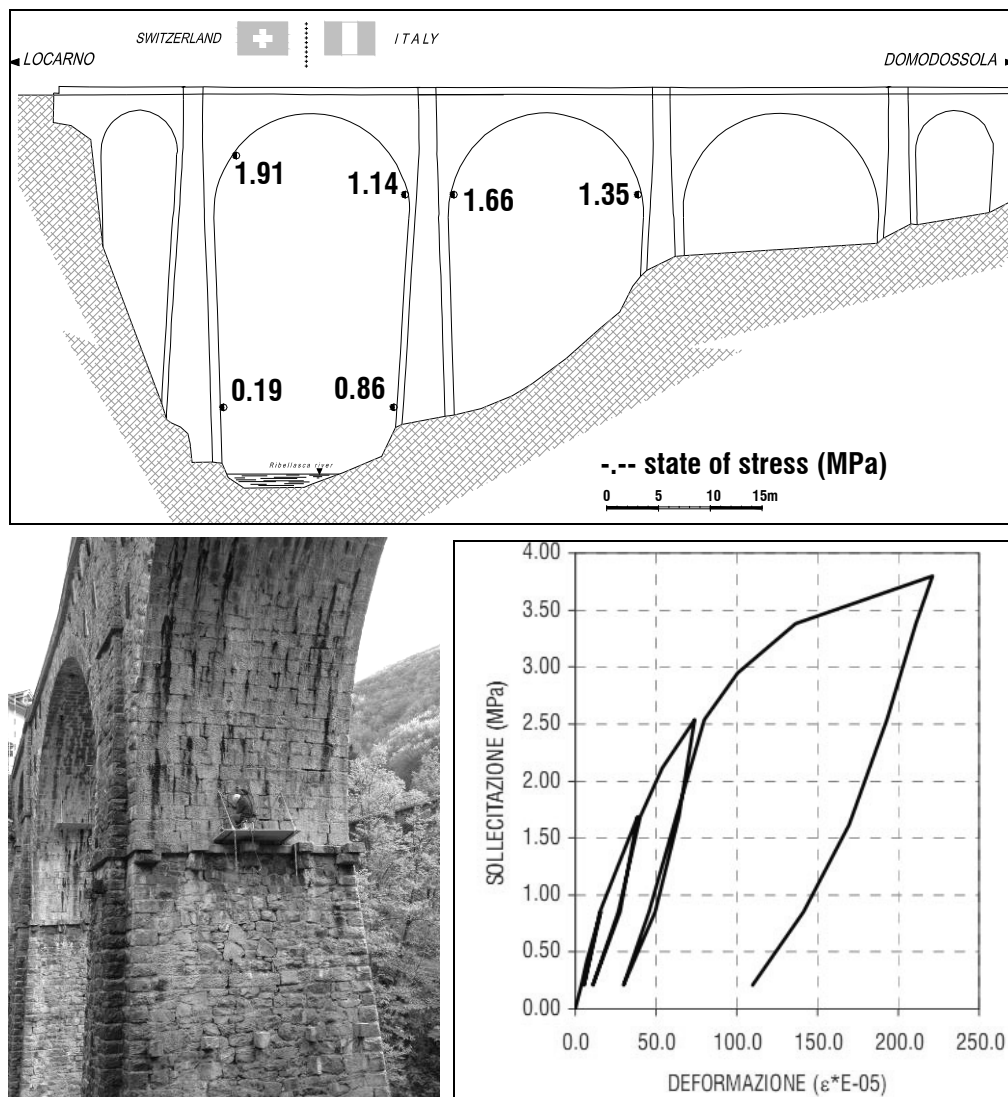


Figure 6 : Measure of the state of stress and of the deformability of the masonry by flat-jack technique: values of the state of stress measured and stress-strain curve of the masonry.

In order to analyze the dynamic behaviour of the structure, a dynamic test was carried on.

Three-dimensional accelerometers (figure 7a) were installed by the climbers in several points of the masonry structures of the bridge and the test was carried on in three different loading conditions:

- ordinary vehicular traffic;
- impulses generated by a truck passing over a relief build on the road;
- impulses generated by the impact of a bucket of an excavator.

A strengthening temporary intervention in the upper part of the bridge, by the installation of steel bars, avoided any alteration of the vibration due to the presence of a damaged and loosened structure.

The spectral analysis shows that in all the testing conditions, the natural frequencies of the structure are 2.0 and 3.0 Hz (see example of spectrum in figure 7b).

From the literature and from studies of the Department of Structures of the University of Genova, the significant vibration modes of a multispan arch bridge are the followings:

- 1st vibration mode, vertical (or global symmetric), characterized by in phase transversal vibrations of the keystones of the arches (figure 7c).
- 2nd vibration mode, vertical (or global antisymmetric), characterized by controphase transversal vibrations of the keystones of the arches (figure 7d).

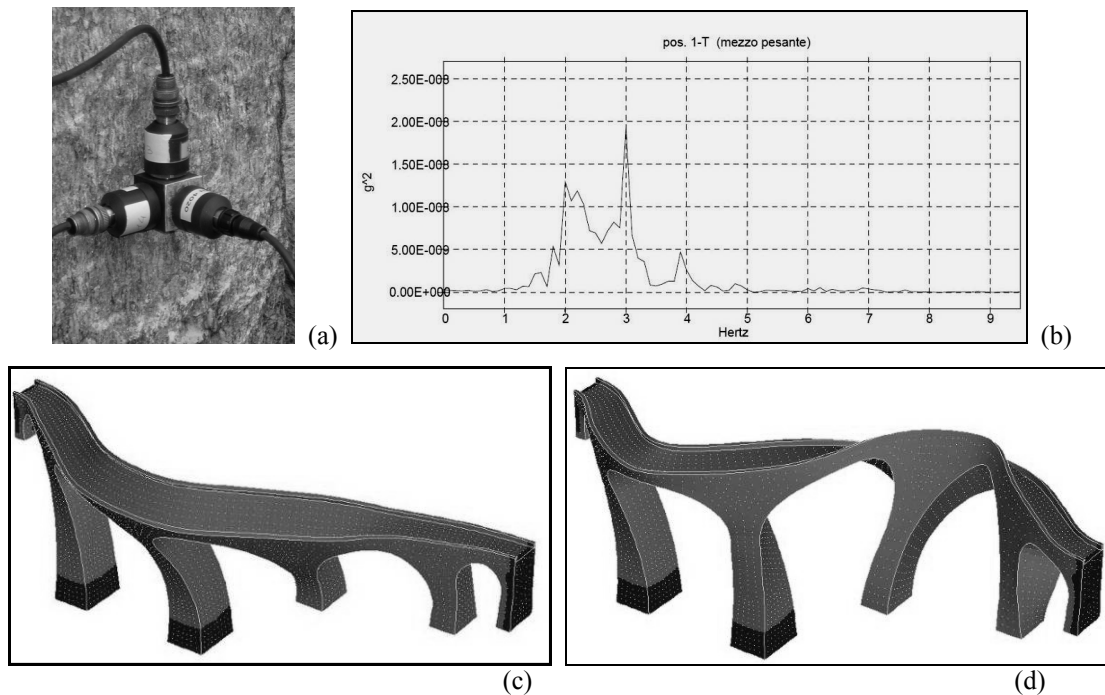


Figure 7: Dynamic test: (a) three-dimensional accelerometer installed on the masonry of the bridge; (b) natural frequencies of the structure; (c) 1st vibration mode: vertical or global symmetric; (d) 2nd vibration mode: longitudinal or global anti-symmetric.

The two theoretical vibration modes correspond to the experimental modal forms and are close to the first two modes obtained by a dynamic analysis executed with a three-dimensional numerical model of the bridge. In the analysis, the masonry was assumed as an elastic and isotropic material.

The numerical model was validated by the deformability tests carried out with the flat-jack technique, using the value of the deformability modulus $E = 4000 \text{ N/mm}^2$, determined by the experimental stress-strain relations.

3 NUMERICAL ANALYSIS

According to the results of experimental investigations on multispan bridges, it is observed that the spans near the one collapsing, are characterized by a vertical displacement upward. The interaction between adjoining arches is due to:

- failure that involves the pier;
- thrust of the material between the two arches.

The failure analysis of multispan bridges can be executed with the cinematic method proposed by Prof. M. Gilbert ([1], [2], [3]): for each possible failure cinematism the corresponding load, responsible of the failure, is calculated; using the method of the virtual works, the cinematism characterized by the minimum load is determined.

For the Ribellasca bridge, calculations were carried out by using the Gilbert's method and the elastoplastic non linear analysis (Castigliano's method applied to the elastoplastic case) developed by the Structural Department of the University of Genova (DISEG) ([4], [5], [6], [7], [8]) for the single and for the multispan bridge cases.

The diagram in figure 9 shows the failure load versus the position of the live load; different type of arches are considered: abaited arches (height of 3.75 m and 5.00 m) and different strength conditions (infinite strength and strength equal to 5 MPa).

The values are coincident with the two methods in the case of arches less abated (like in the case of Ribellasca bridge).

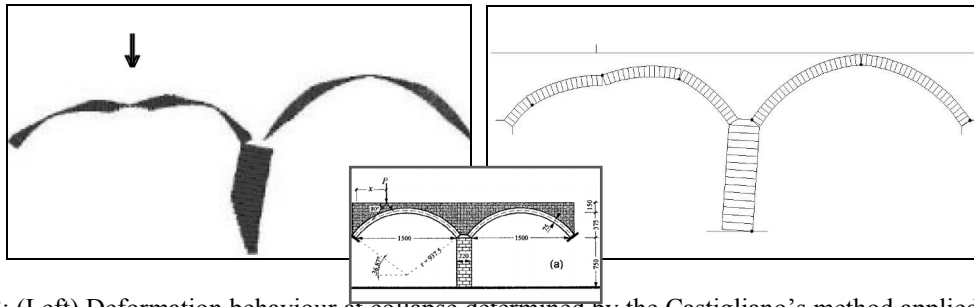


Figure 8: (Left) Deformation behaviour at collapse determined by the Castigliano's method applied to the elastoplastic case (DISEG method); (Right) collapse behaviour determined by the cinematic method, for a two span arch proposed by Melbourne and Wagstaff ([6]).

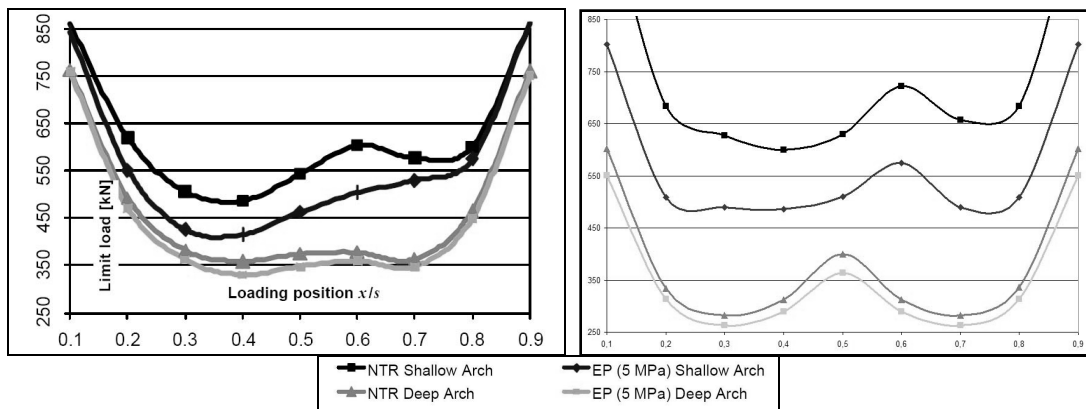


Figure 9: Comparison of the limit load at failure versus the position of the live load, determined with DISEG's method ([6]) and the cinematic method.

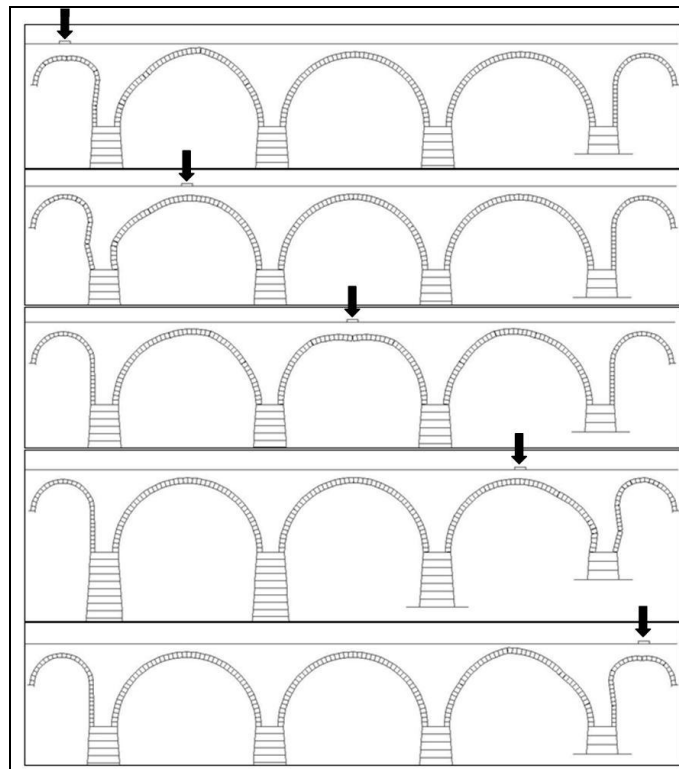


Figure 10: Collapse cinematism when the live load is applied on the keystones. The minimum collapse load is on the 4th arch near the boundary.

To verify the static inadequacy of the bridge of Ribellasca it was enough to consider the concentrated load of 400 kN of the Swiss code SIA 261 (model 1).

In the calculations all the dead loads of the road pavementation were considered.

Different value of masonry strength and different positions of the live loads were considered in the cinematic calculations.

Figure 10 shows the collapse cinematism when the live load is applied on the keystones. The minimum collapse load is on the 4th arch near the boundary.

For a strength of 2 N/mm² the collapse load is equal to about 350 kN, near to the design value without any amplification.

A finite elements model was then executed. The mechanical parameters of the masonry determined by the flat-jack tests were applied to the model. The validation of the model was carried out by comparing the states of stress calculated on the model and those obtained by flat-jack tests, and by comparing the numerical and experimental variation of the state of stress induced by a live load applied on the top of the main arch (Figure 11).

A non linear analysis was also carried out with the same finite elements model, using the stress-strain curve and the strength determined by the flat-jack tests. In Figure 12 the results of the finite elements analysis and the cinematic analysis are compared. The position of the hinges and the values of the collapse load are similar.

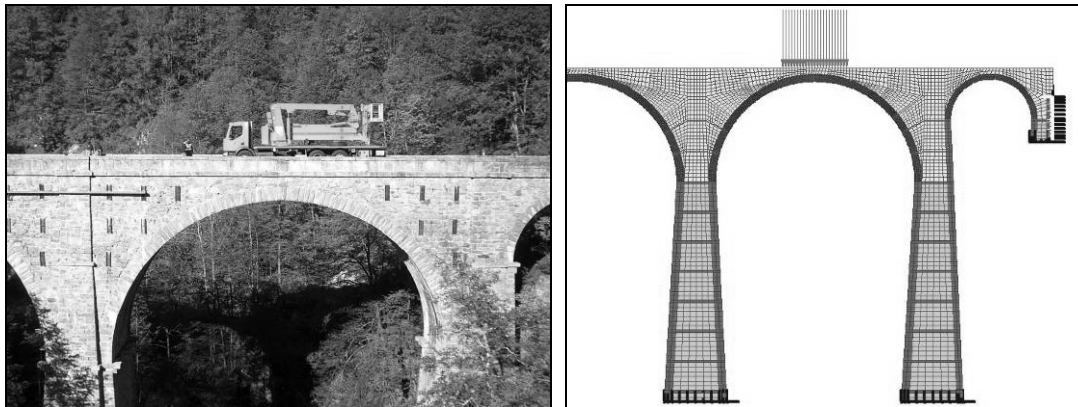


Figure 11: Validation of the finite elements model by comparing the calculated and experimental (measured with the flat-jack technique) variation of vertical stress due to a live load applied on the top of the main arch.

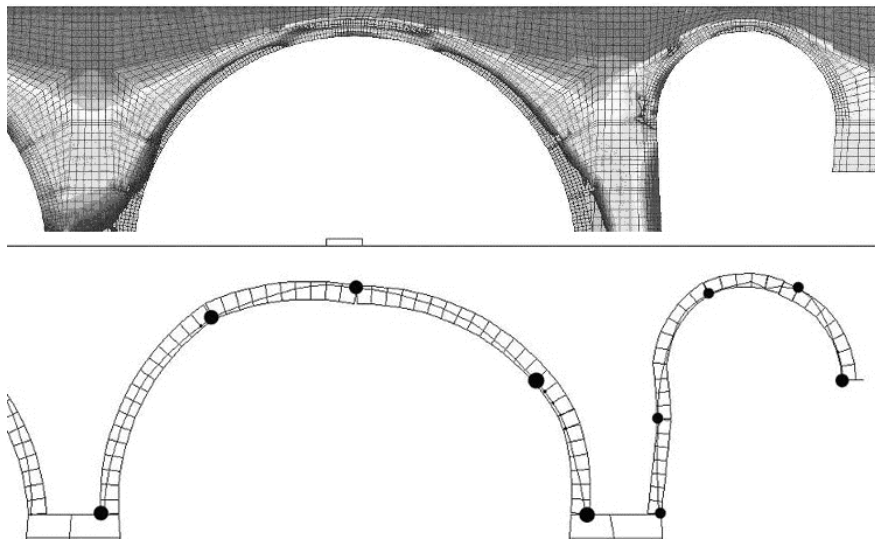


Figure 12: Comparison between the results of the finite elements analysis and the cinematic analysis

In the study of historical buildings it is advisable to use both finite elements method analysis and cinematic analysis. The advantage of the cinematic analysis is that it is a method based on pure equilibrium, and so it is independent from the building procedures and load histories, very difficult to determine in case of ancient buildings.

The experimental investigations and the numerical analysis allowed to design the strengthening works which at present have not been finished. The main intervention is the substitution of the filling material which covers the arches with a light boxed structure (figure 14) made with thin concrete plates which will be able to avoid the transversal deformation of the lateral walls of the bridge. In the meantime, the extrados surfaces of the arches will be consolidated by means of a concrete layer with steel mesh connected to the masonry of the arches by means of steel anchors fixed with epoxy. The structural conditions of the pillars are considered satisfactory; only local strengthening interventions with steel bars glued with epoxy into small diameter boreholes are foreseen.

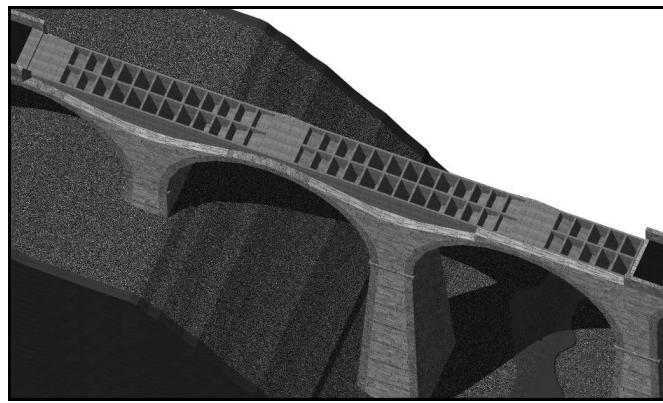


Figure 13: Scheme of the strengthening intervention

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