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

A. Cappini and G. Stagnitto
*Dipt. Structural Mechanics, University of Pavia, Italy*

A. Pederzani
*Anastasi SA Ingegneria, Locarno, Switzerland*

C. Rossi and P.P. Rossi
*R. teknos s.r.l., Bergamo*

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 surveys were used to analyse the structural characteristics of the masonries and flat-jack tests to measure the state of stress and the deformability characteristics. The vibration modal shapes of the bridge were than 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 (Fig,1a shows a photograph taken during the construction phase). After a century, significant damages were observed, mainly due to the water seepage through the waterproofing layer of the bridge (Fig.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.

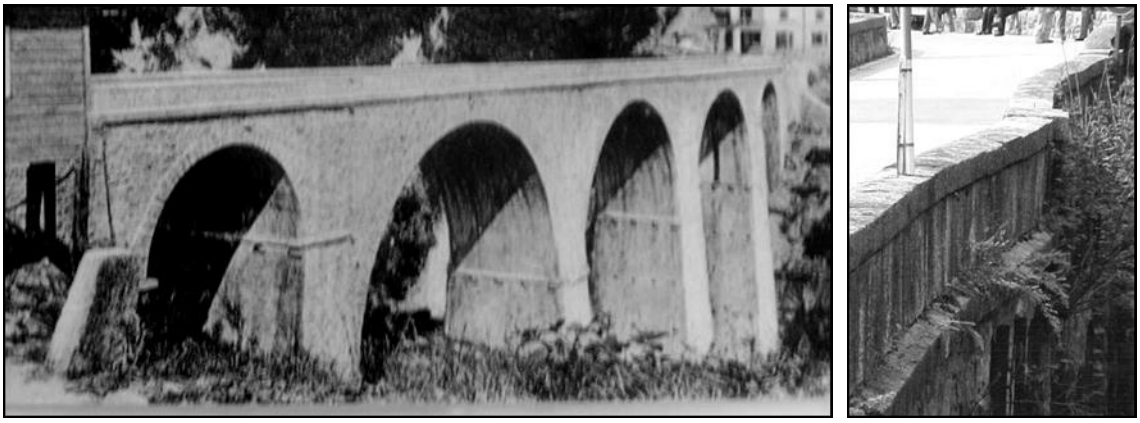
In order to analyze the static capability of the bridge according to the actual safety standards, a wide experimental investigation was carried out as well as a static and dynamic numerical analysis.

## 2 EXPERIMENTAL INVESTIGATIONS

After a preliminary historical research and a detailed examination of the original design drawings of the bridge (Fig.2), the damages and the crack pattern observed in the supporting structures (pillars and arches) suggested to carry out a detailed experimental investigations program. All the investigations were carried with the aid of specialized climbers without any scaffolding (Fig.3).

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

The composition and the structural characteristics of the masonry were investigated by means of coring and video-camera surveys. 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 (Fig.5). The foundation level of the pillars was determined by means of vertical corings and video-camera survey.



(a) (b)

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

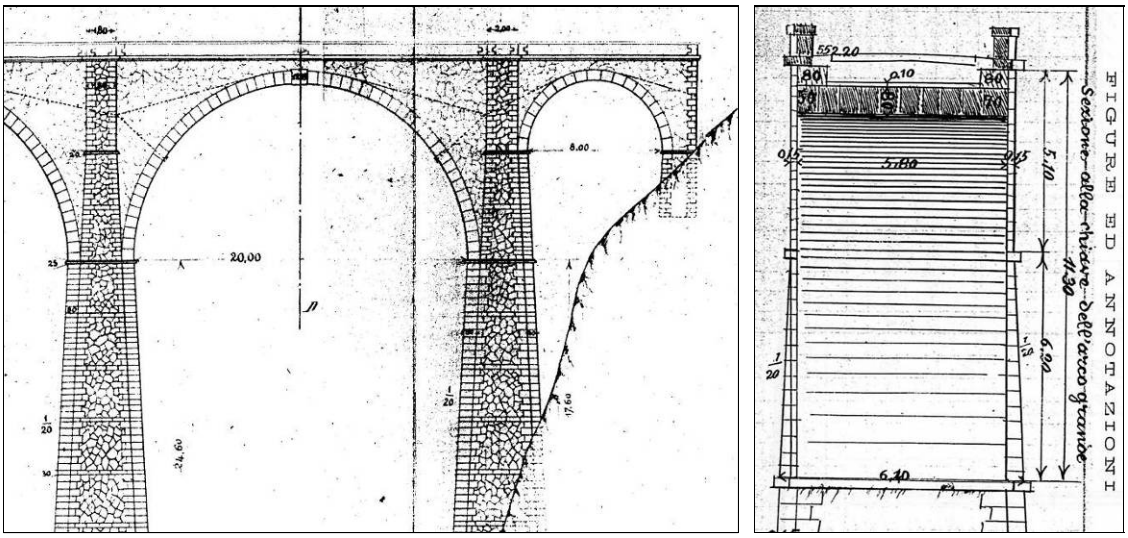



Figure 2 : Two particulars taken from the original design drawings.

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 Fig.6. By using two parallel flat-jacks, the deformability characteristics of the masonry were determined (Fig.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 1/1.5 $N/mm^2$.

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

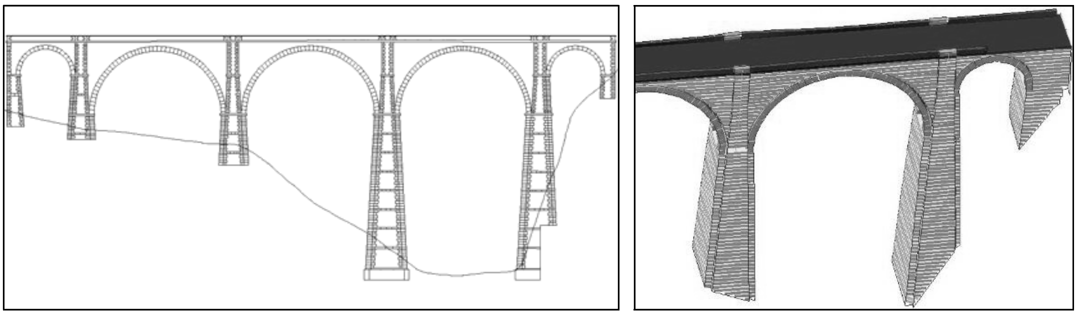


Figure 4 : Geometric survey and three-dimensional model of the bridge.

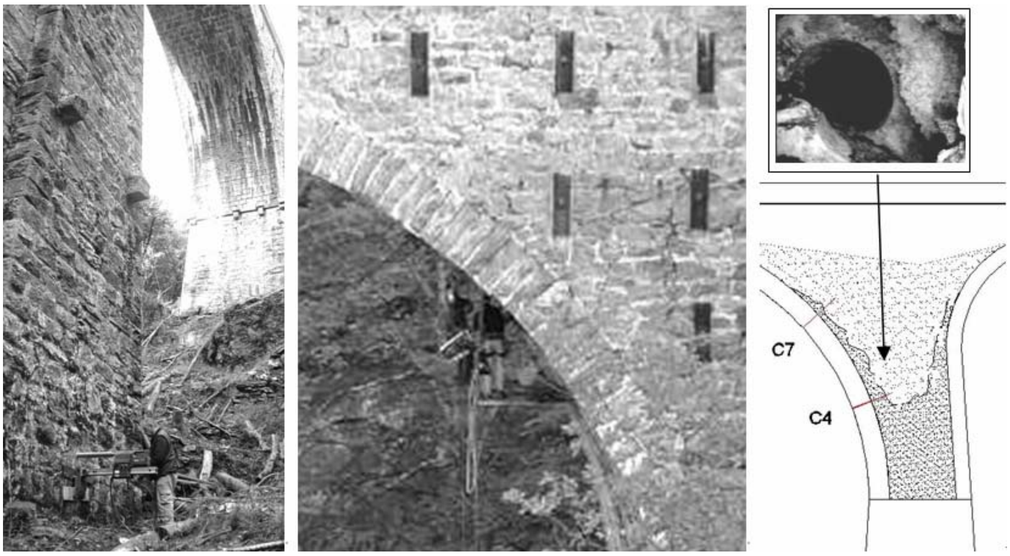
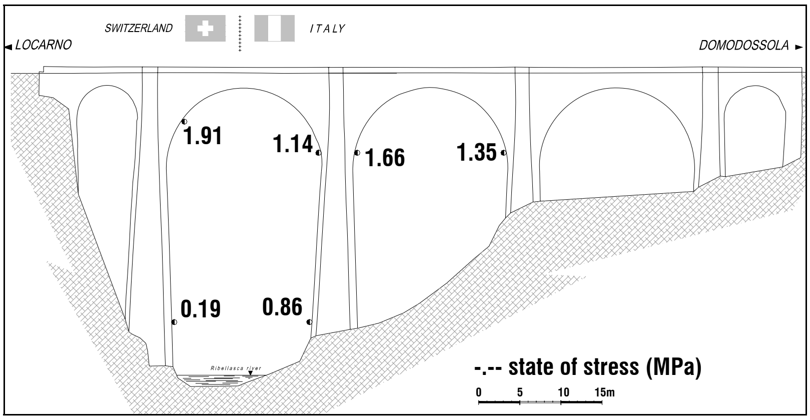



Figure 5 : Coring and video-camera surveys at the base of a pillar and on the intrados of the arches to analyse the composition and the structural characteristics of the masonry and of the filling materials



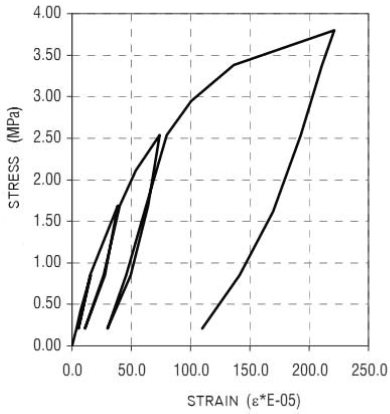




Figure 6 : Measure of the state of stress and of the deformability of the masonry by flat-jack technique: values of the state of stress measured and stress-strain curve of the masonry

In order to analyze the dynamic behaviour of the structure, a dynamic test was carried on. Three-dimensional accelerometers (Fig.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:

(1) ordinary vehicular traffic;

(2) impulses generated by a truck passing over a relief built on the road;

(3) 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 confining steel bars, avoided any alteration iduced by the vibrations to 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 Fig.7b.

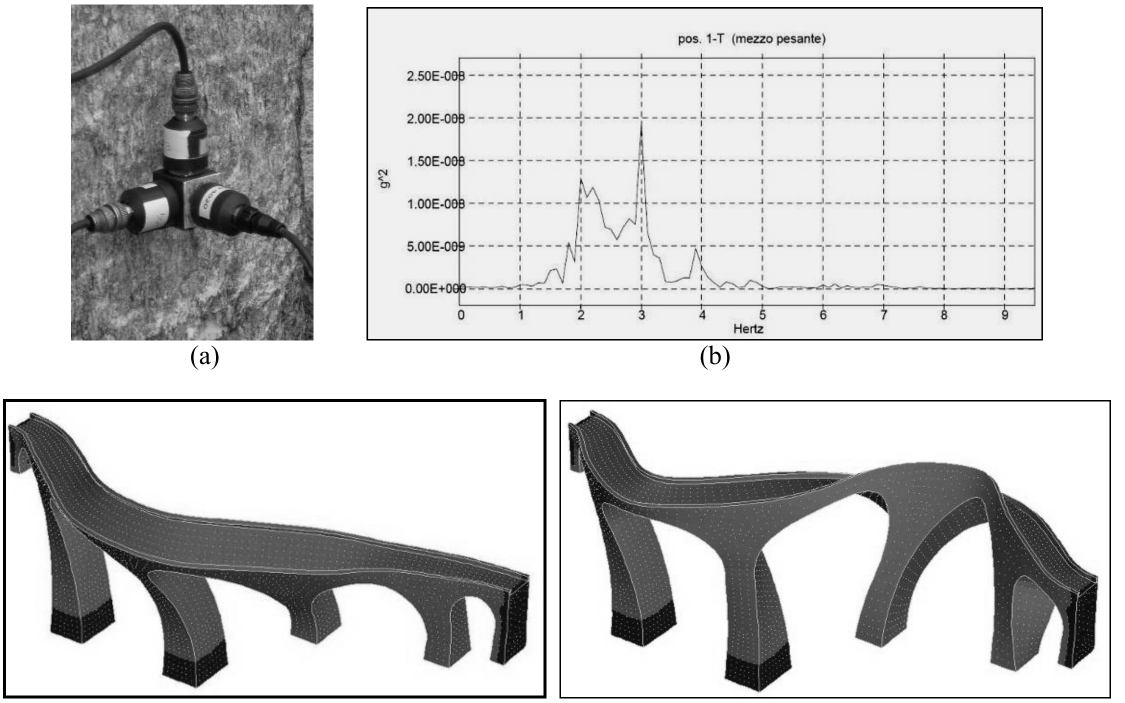
From the literature and from some studies of the Department of Structures of the University of Genova, the significant vibration modes of a multispan arch bridge are the following:

(1) 1st vibration mode, vertical (or global symmetric), characterized by in phase transversal vibrations of the keystones of the arches (Fig.7c).

(2) 2nd vibration mode, vertical (or global antisymmetric), characterized by controphase transversal vibrations of the keystones of the arches (Fig.7d).

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 N/mm$^2$, determined by the experimental stress-strain relations.




(a) (b)

(c) (d)

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.

## 3 NUMERICAL ANALYSIS

The failure analysis of multispan bridges can be executed with the cinematic method proposed by Prof. M. Gilbert (Gilbert M. et al. 1997, Gilbert Matthew et al. 1998, Melbourne C. et al. 1995): 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) (Brencich A. et al. 2001a, Brencich A. et al. 2001b, Brencich A. et al. 2004, Lagomarsino S. et al. 1999, A. Cauvin et al. 1993) for the single and for the multispan bridge cases.

The diagram in Fig.9 shows the failure load versus the position of the live load; different type of arches are considered (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 less abated arches (like in the case of Ribellasca Bridge).

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

In the calculations, the dead load of the road pavimentation was considered.

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

Fig.10 shows the collapse cinematism when the live load is applied on the keystones. The minimum collapse load is on the $4^{th}$ arch near the boundary.

For strength of 2 $N/mm^2$ the collapse load is equal to about 350 kN, close to the design value without any amplification.

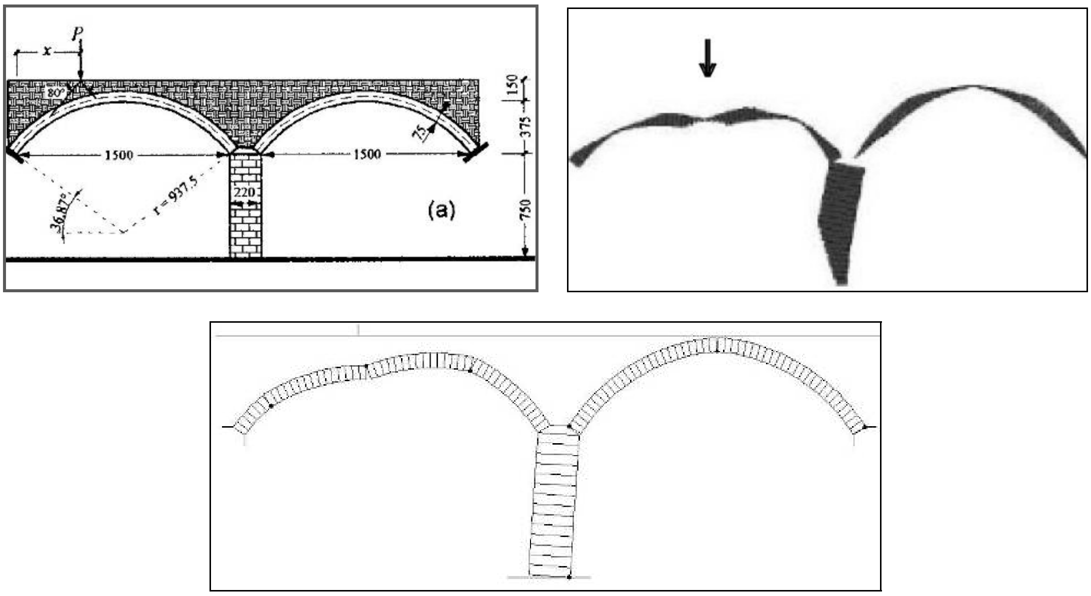



Figure 8 : (Left) Deformation behaviour at collapse determined by the Castigliano's method applied to the elastoplastic case (DISEG method); (Right) collapse behaviour determined by the cinematic method, for a two span arch proposed by Melbourne and Wagstaff (Brencich A. et al. 2004)

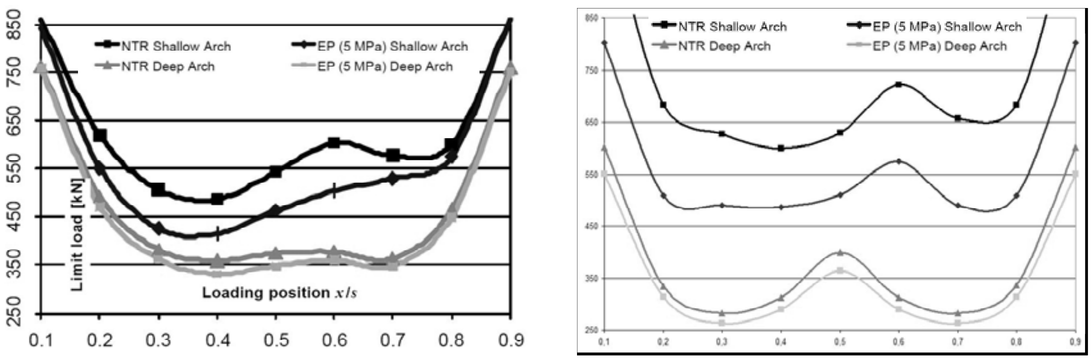



Figure 9 : Comparison of the limit load at failure versus the position of the live load, determined with DISEG's method (Brencich A. et al. 2004) and the cinematic method

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 (Fig.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 Fig.12 the results of the finite elements analysis and the cinematic analysis are compared. The position of the plastic hinges and the values of the collapse load are similar.

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, which are 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 (Fig.13) made with thin concrete plats 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.

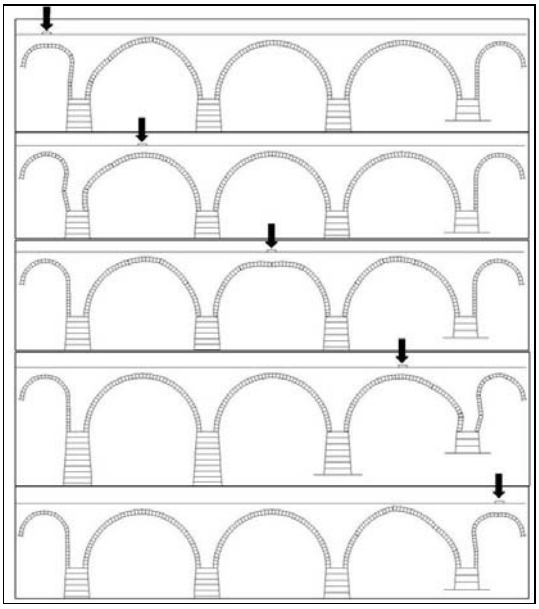


Figure 10 : Collapse cinematism when the live load is applied on the keystones. The minimum collapse load is on the 4th arch near the boundary.

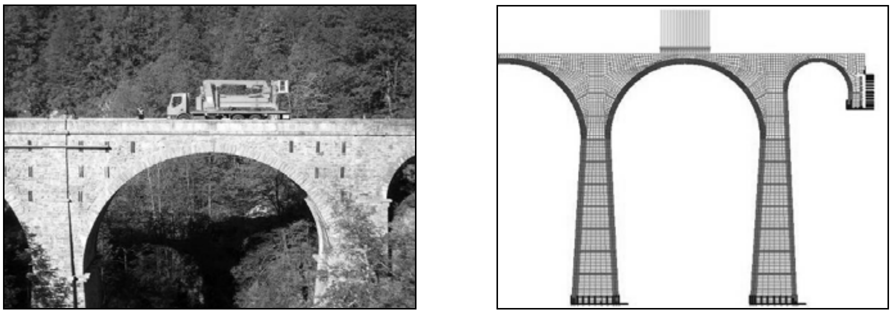


Figure 11 : Validation of the finite elements model by comparing the calculated and experimental (measured with the flat-jack technique) variation of vertical stress due to a live load applied on the top of the main arch.

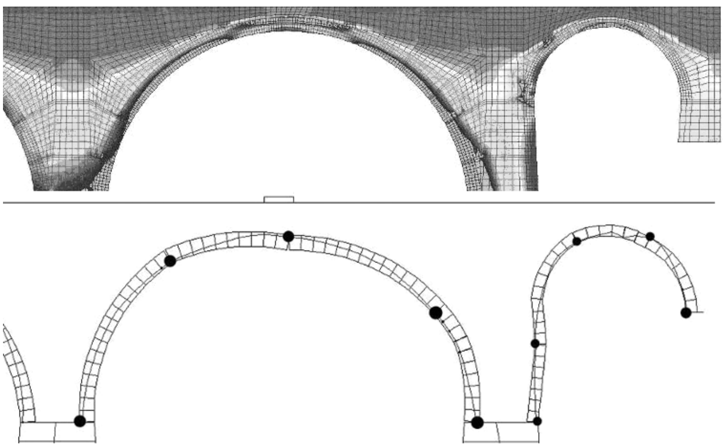


Figure 12 : Comparison between the results of the finite elements analysis and the cinematicanalysis

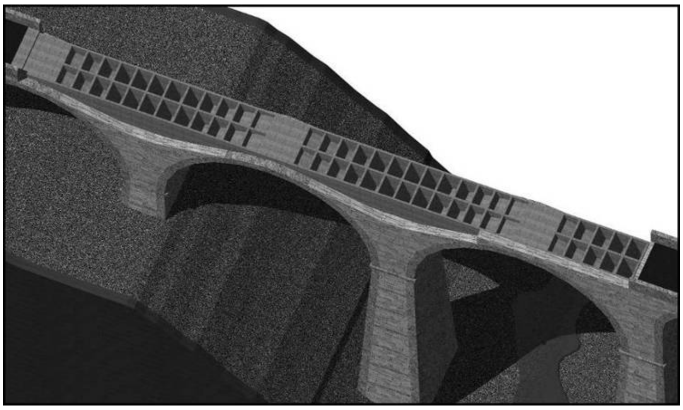


Figure 13 : Scheme of the strengthening intervention

## REFERENCES


Gilbert M., Melbourne C., Rigid-block analysis of masonry structures, The Structural Engineer, Vol. 72, No. 21, 1 November 1994.

Gilbert Matthew, On the analysis of multi-ring brickwork arch bridges,Proceedings of the Second International Arch Bridges Conference, Venezia, 1998

Melbourne C. and Gilbert M., The behaviour of multi-ring brickwork arch bridges, The Structural Engineer, Vol.73, No. 3, 7 February 1995.

Brencich A., Gambarotta L., De Francesco U., Elastic no tensile resistant-plastic analysis of masonry arch bridges as an extension of Castigliano's method,Proc. 9th Canadian Masonry Conference, Fredericton, New Brunswick, Bishoff, Dawe, Schriver, Valsangkar (eds.), 2001.

Brencich A., Gambarotta L., De Francesco U., Non linear elasto-plastic collapse analysis of multi-span masonry arch bridges,Proc. ARCH '01 – 3rd International Conference on Arch Bridges, Paris,C. Abdunur (ed.), 2001, pp. 513-522, Presses de l'Ecole Nationale des Ponts et Chaussées, Paris.

Brencich A., De Francesco U.,Assessment of Multispan Masonry Arch Bridges. I: Simplified Approach,Journal of Bridges Engineering, ASCE, November/December 2004.

Lagomarsino S., Resemini S. e Rossi B., Analisi teorico-sperimentale per la sicurezza dei ponti della linea ferroviaria Genova-Ovada, in Atti del Corso CIAS: Progetto, costruzione e controllo dei ponti: il ruolo della sperimentazione, Milano, 15 ottobre 1999, Trento, 1999, pp. 54-79.

A. Cauvin, G. Stagnitto, Strength assessment of ancient masonry vaults: integration of classical and modern methods of structural analysis, IABSE, Roma 1993

P.P. Rossi, Non destructive evaluation of the mechanical characteristics of masonry structures". Conference on non-destructive evaluation of Civil Structures and Materials, Boulder Colorado, USA, October 1990

F. Casarin, I. Cavaggioni, A. Lionello, C. Modena, C. Rosso, P.P. Rossi