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The Monumental Bridge of Monte Carmelo (Italy): Strategies for the Historical and Architectonical Preservation

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ABSTRACT: The monumental bridge of Monte Carmelo (Loano - Savona, Italy) was built between the end of the XVI century and 1610. It is an unusual example of "panoramic" bridge (besides representing the access to the homonymous convent, it was originally used as a *promenade*). Its recent building history is marked by two events: a retrofitting intervention in the 50's; a relevant increase in transient loads since the 80's, due to a higher traffic flow. In last years, such structural changes led to an evident deterioration of the structure, stressing the previously occurred damages and activating new ones. Recently, safety assessment and retrofitting intervention was required. On the one hand, even if the original static behaviour of bridge was better than the current one, the removal of the previous retrofitting interventions would produce further damages on the original structure, incompatible with preservation issues. On the other hand, the safety and serviceability of the bridge requires new intervention. The proposed design strategies are the synthesis of structural and architectonical issues.

1 INTRODUCTION

The monumental bridge of Monte Carmelo (Loano - Savona, Italy), built between the end of the XVI century and 1610 (Carattino 1972), is an interesting structure because of different reasons. From the architectonical point of view, it represents a rare case of "panoramic" bridge (besides representing the access to the homonymous convent, it was originally used as a *promenade*). From the structural point of view, it is a clear example of the risk which ancient structures can be often subjected to as a consequence of anthropic modifications.

Its constructive history is marked by two events: a retrofitting intervention in the 50's; a relevant increase in transient loads since the 80's, due to an higher traffic flow associated with the construction of a new road passing through the bridge. The intervention in the 50's has substantially modified the original structure: above the arch system, several reinforced-concrete beams were built along the transversal direction (in correspondence of each pier); on these beams, a thick reinforced-concrete slab was constructed, substituting the original extrados structure. The aim of this intervention, very common in Italy at that time, was the strengthening of the bridge extrados structure in order to ensure the motor-vehicles flow.

In the last years, these structural changes, introduced in a previously damaged condition, led to an increase of the static-functional deterioration of the bridge. Recently, safety evaluations and retrofitting intervention was required.

2 STRUCTURAL SURVEY, DAMAGE AND DETERIORATION

2.1 Geometrical and building features

The bridge has a total length of 158 m and uniform width of approximately 7 m. It is constituted of 12 regular stone masonry arches, with medium span of 5 m. It rises 10.80 m from its lowest foundation level. The slope was required because of the difference in level between the convent and the downhill. The piers of the bridge are very slender, giving to the structure an elegant appearance. The slope of the deck involves that each arch stands up at two different levels, the piers having variable heights (also as a consequence of the ground profile). Thus, it can be stated that the bridge is characterized by certain structural irregularities (Fig. 1).

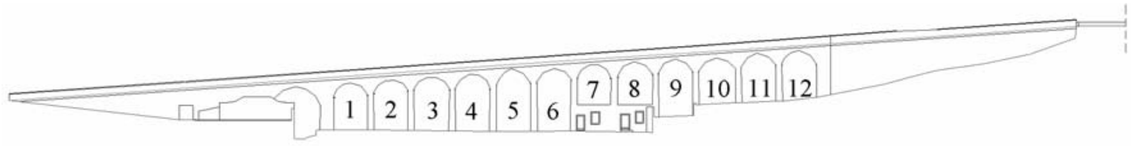


Figure 1 : Scheme of the longitudinal profile of the bridge.

The quality of the stone masonry is low. Actually, it is constituted mainly by poor rubble or river stones, connected by mortar and small stone chips. Masonry appears, therefore, quite uneven and chaotic (Fig. 2a).


(a)

(b)

Figure 2 : (a) Masonry typology and evidence of the reinforced concrete beams loading spandrel walls; (b) Inspection well on the deck, where it can be observed the reinforced concrete slab.

As previously reported, a retrofitting intervention realized between 1950 and 1960 modified the original structure of the bridge. The real nature of such an intervention was not well known until specific investigation was carried out (only some concrete elements were visible with naked eyes, such as the parapets and the head of some concrete beam). The intervention consisted in:

- construction of reinforced concrete beams positioned in correspondence of each pier, thus loading the spandrel walls; the height of beams is 0.6 m;
- realization of a reinforced concrete slab in place of the original deck (thickness 0.25 m). The slab is supported not only by the concrete beams, but also by the spandrel walls and, in some cases, the crown of arches is loaded; the reinforcements of the slab are bi-directional (Fig. 2b).

Besides the above described structural intervention, a reinforced concrete parapet has been introduced, replacing the original masonry one. Fig. 3 synthetically shows the current structural configuration of the bridge.

During the inspection of the bridge, the poor quality of the infill, characterized by a strong decohesion (sand, gravel, loose soil) has been observed. It has not been possible to analyse the nature of the backfill. The estimated average arch thickness is about 0.3 m.

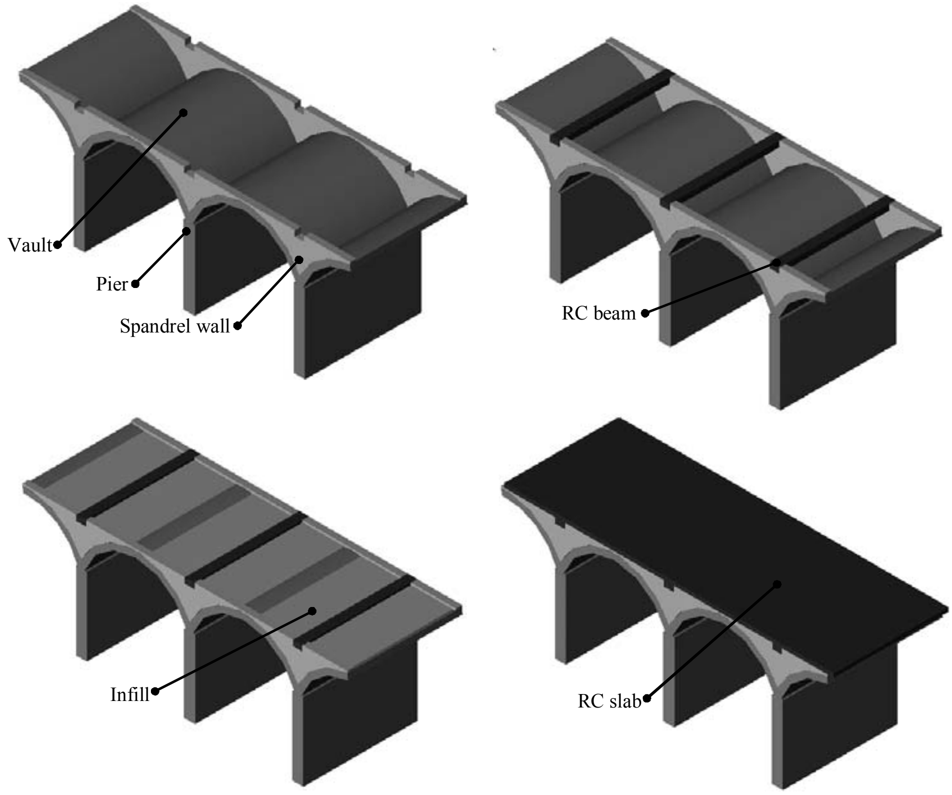


Figure 3 : Scheme of the current structural elements of the bridge.

2.2 Analysis of deformations and damage state

2.2.1 Deformation of arches and piers

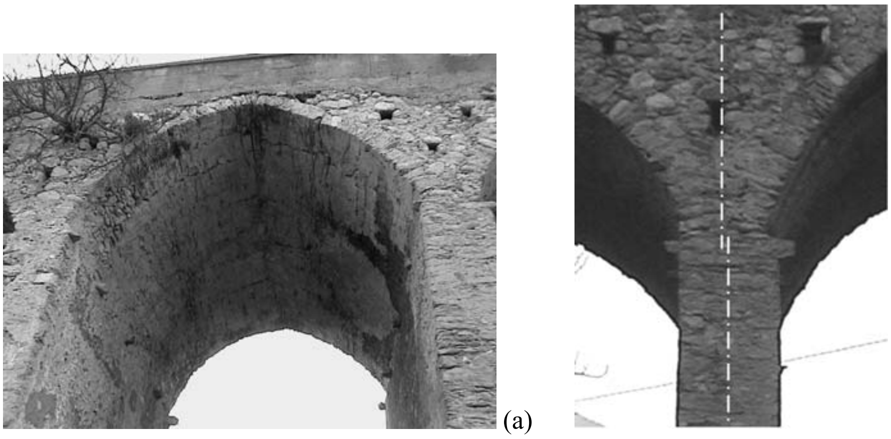
The geometrical survey of the bridge has been carried out in order to point out deformations meaningful from the structural point of view. In particular, the three following phenomena have been observed in some spans:

- leaning of piers;
- out-of-alignment of the springings of arches and the top of piers (Fig. 4a);
- remarkable deformations of the arch rings (Fig. 4b).

Such phenomena are probably connected each other. Actually, the out-of-alignment and the piers' leaning can be observed in correspondence of those spans in which arches are deformed.

The cause of the above described phenomena are of different types. The slope of the deck may be considered a "predisposing" cause: as a consequence of the asymmetry of arches, the thrusts acting on springings are not perfectly equilibrated. "Activating" causes may have been the seismic events suffered from the bridge throughout its history. In particular, the following earthquakes should be considered: the earthquake in 1831 and its principal after-shock, whose macroseismic intensities were VI and VIII respectively (Mercalli 1897); the earthquake in 1887 of magnitude M_s = 6.4, as to the parametric catalogue NT4.1 (Camassi and Stucchi 1997). The sussultatory components of the seismic shock could have instantly reduced the compressive stress acting on the hingings of the arches, allowing the arch base to move with respect to the piers.

A remarkable damage was observed in the 5th span (Fig. 4), where the hinging at the crown of the vault may be associated with the effect of the earthquake: the slender piers may have oscillated out-of-phase, bringing nearer and farther the arch springings. The 5th span is exactly in the middle of a set of spans (0-9) with similar dynamical characteristics; the above described phenomenon could have been emphasized from such a specific position.



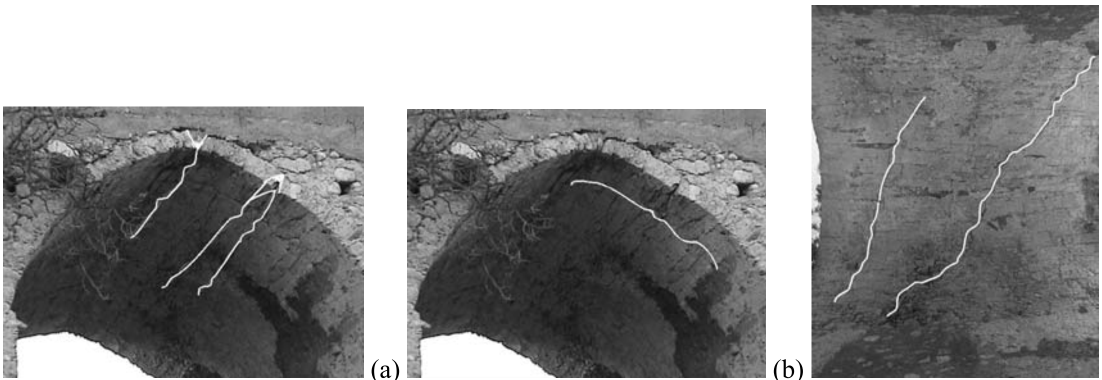
(a) (b)

Figure 4 : (a) Deformation of the arch rings; (b) Out-of-alignment of piers and arches.

2.2.2 Crack pattern in vaults

Three different types of cracks have been observed:

- cracks associated with the formation of hinges in arches (Fig. 5a). Such cracks, in most of the cases, are more clearly visible on the external rings than on the intrados of vaults (such observation is meaningful as far as safety issues are concerned);
- cracks parallel to the longitudinal axis of the bridge, 0.5 m far from the external rings of the vaults (Fig. 5b). These cracks are common in masonry arch bridges. They are usually caused from: the effect of the spandrel walls loading vaults; the building realization of the vaults themselves, involving a different care in the construction of the intrados and of the external arch rings;
- diagonal cracks, at the intrados of the vaults (Fig. 5c); such cracks are uncommon for masonry arch bridge and may be associated mainly to seismic actions. The more remarkable crack is visible under span 9th, while other considerable cracks can be observed under spans 10th and 11th.



(a) (b) (c)

Figure 5 : (a) Transversal cracks; (b) Parallel cracks; (c) Oblique cracks (span 9th).

2.2.3 Spandrel walls

The spandrel walls of the bridge are characterized by a widespread degradation. Masonry is upset and out-of-plane deformations are well evident. The cause of such a damage state is the introduction of the above mentioned reinforced concrete beams. These beams, actually, load directly the spandrel walls (not designed to carry specific vertical loads) producing a stress concentration. Moreover, the material degradation is increased from the presence of vegetation.

2.2.4 Other damage states

In the upper side of the bridge, on both the wing walls, acting as retaining walls for the embankment, a remarkable crack pattern can be observed (deep sub-vertical fractures). The phenomenon appears to be associated to movements of the walls themselves with respect to the upper span. Such a hypothesis is confirmed by the presence of a subsidence in the deck and evident cracks in parapets, both located in correspondence of the beginning of the wing walls, probably coincident with the end of the concrete slab.

2.3 Material degradation

2.3.1 Degradation of masonry

The masonry of the bridge is degraded, with widespread cracks and upsets of stone elements. “Predisposing” cause of such damages is the poor quality of materials and the lack in maintenance (Fig. 6a). However, the current use of the bridge could be another possible “activating” cause. Originally, the bridge has been realized for light and pedestrian traffic. The recent building development of the area produced an increase in traffic. Thus, highest static and dynamic loads burden on the bridge, producing also remarkable vibrations. Probably, these latter are stressed by the presence of the reinforced concrete slab introduced in the 50’s, that produced an increase in stiffness of the deck, reducing the damping capacity of the structure. Vibrations could produce, by the time, degradation phenomena in masonry structures.

2.3.2 Degradation of reinforced concrete structure

The reinforced concrete structure presents some degradation phenomena, particularly located in the external ends of the slab and in the tie-beams connecting the parapets. The “spalling” of the reinforcements is well evident (Fig. 6b). The phenomenon may be associated to an inadequate thickness of the surrounding concrete.


(a)

(b)

Figure 6 : (a) Degradation of a pier’s masonry; (b) Spalling in the reinforced concrete slab.

3 MODELLING AND STRUCTURAL ANALYSIS

In order to investigate the stress and strain states of the structure, the bridge has been analysed by mean of a three dimensional finite element model. Various analyses, considering different load conditions, have been performed. In particular, the following conditions have been considered:

- dead and permanent loads, before and after the introduction of the reinforced concrete structure;
- dead and permanents loads (in the current configuration of the bridge) plus transient loads representing the passing through vehicles in different positions.

The model was realized by mean of the numerical code *Ansys release 5.7.*

3.1 The FEM model: geometry and employed elements

The model of the bridge is based on the geometrical survey; thus, the current geometry is considered, including deformations accumulated from the structure throughout its history (leaning of piers, deformations in arch rings, etc…).

It is well known that, although from a visual examination and dimensional survey it is possible to ascertain the geometry and the material used for external construction, for a masonry bridge it is not possible to ascertain current material thickness or the characteristics of the structure and material within the bridge. For these reason, the internal dimensional and mechanical characteristics of the structure have been hypothesized on the basis of the external evidences.

The structure has been modelled by mean of shell elements (piers, barrel vaults, spandrel walls, concrete slab), beams (reinforced concrete beams, tie-rods) and solids elements (infill and backfill). The structural role of the backfill was neglected, since the inspection showed its small size and its scarce mechanical quality. Thus, the infill and the backfill act only as weights loading the structure. The no more existent masonry parapets (replaced with the current concrete ones) have been simulated by mean of concentrated forces applied on the spandrel wall.

Adequate boundary conditions have been applied to the model (translations and rotations are not allowed on the base of piers and on the abutments).

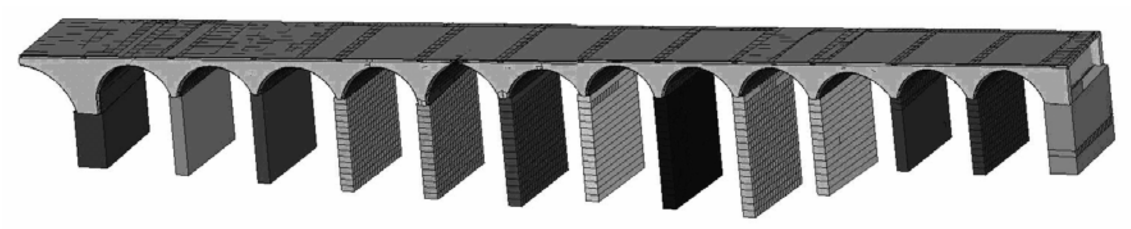


Figure 7 : Global representation of the FEM model.

3.1.1 *Materials*

Since the objective of the modelling was the investigation of the relationship between the transient loads and dead loads, the mechanical properties have been defined on the basis of qualitative considerations.

A linear elastic constitutive model has been adopted. This choice can be accepted because only service loads were considered. Table 1 shows the mechanical properties defined for the different structural elements.

Table 1: Mechanical properties of the FEM model.

Structural element	Material	Density ρ [kg/m^3]	Poisson coeficient ν	Young modulus E [N/m^2]
Spandrel walls Piers	Masonry	1700	0.2	$0.15 \cdot 10^{10}$
Arch rings	Masonry	1800	0.2	$0.15 \cdot 10^{10}$
Infill	Unbound material	1400	0.0	$0.05 \cdot 10^{10}$
Reinforced concrete slab and beams	Reinforced concrete	2500	0.2	$2.5 \cdot 10^{10}$

3.2 *Scheme of the numerical analyses*

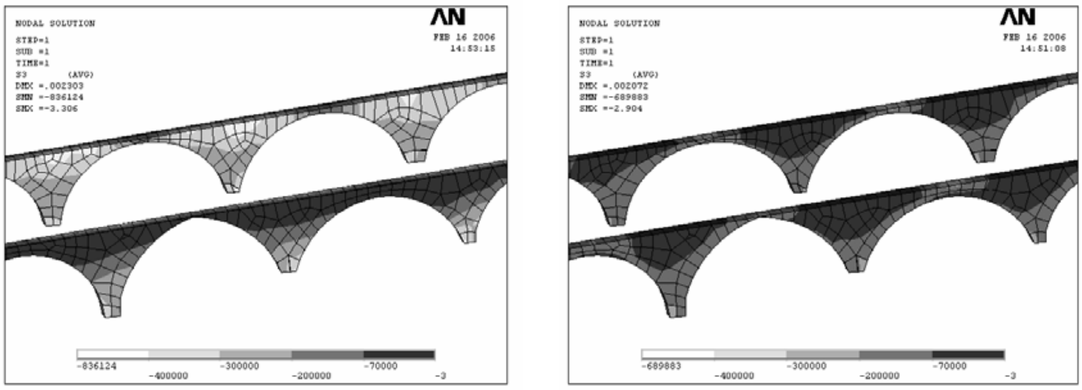
Different linear static analyses were performed. The following loads conditions have been considered:

1. permanent loads in the current configuration of the structure;
2. permanent loads in the original configuration (without the reinforced concrete structures introduced in the 50's);
3. permanent loads in the current configuration plus transient loads representing two double-axis vehicles, considering different positions and different values. Alternative loading condition.

3.2.1 *Comparison between the original and the current configuration (load conditions 1 - 2)*

In Fig. 8, the minima principal stresses (compression) in the spandrel walls are represented, before and after the introduction of the concrete structures. Besides an increase in stresses, in the current configuration it can be noticed the bi-axial bending of the spandrel walls associated with the presence of the reinforced concrete beams.

As far as vaults are regarded, even neglecting the typical tensile stresses at the intrados associated with the type of analysis performed (the building phases were not considered), it is important to point out the presence of tensile stresses. It means that, even considering only permanent loads, as a consequence of the deformed shape of arch rings and of the presence of concentrated loads on spandrel walls, the natural compressive stress state of arches have been altered. The maximum value of the vertical displacements at crown in the current configuration is 0.0032 m, while in the original one was 0.0019 m.



(a) (b)

Figure 8: Minima principal stresses in spandrel walls (internal and external view) – span 5^{th} and 6^{th} [N/m^2] (a) current configuration; (b) original configuration.

3.2.2 Analyses with permanent and transient loads (load condition 3 – current configuration)

Different analyses have been performed in order to investigate the increase in the compressive stresses in some structural elements (in particular, the spandrel walls) by varying the values of the transient loads (statically applied) and by considering different positions.

The loads considered are referred to two-axles vehicles, assuming the following categories of the Italian Road Code: category M1 (vehicles for carrying people, up to 9 persons); category M2 (vehicles for carrying people, over 9 persons - maximum weight 5 t); category N3 (vehicles for carrying wares). Two different loads have been assumed:

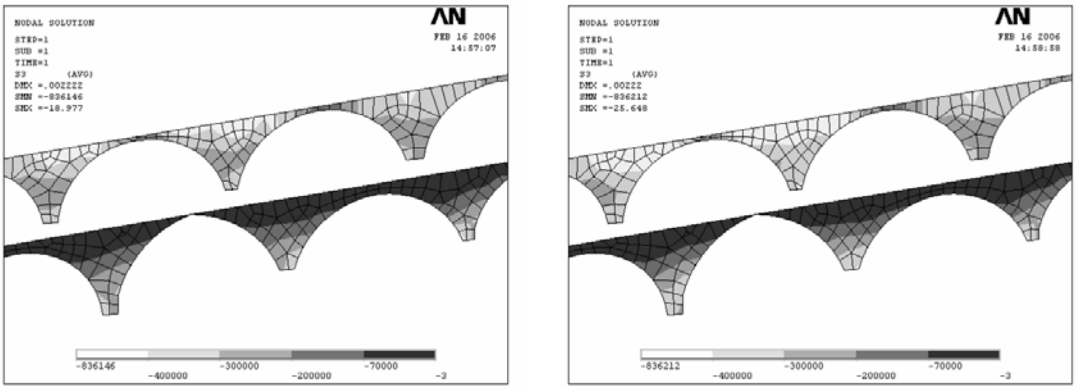
- 10000 N for axle (as a mean between the loads associated to the category M1 and M2);
- 40000 N for axle (as a reference value for the category N3);

Moreover, the two following positions are considered:

- position 1: the two axles are positioned across the crown of the fifth span (the more critical one, considering its deformed shape);
- position 2: the axles are positioned across the crown of the 8^{th} span (where the self-weight analysis revealed the presence of the highest tensile stresses).

The analyses showed that position 1 is the worst one for the structure. It can be noticed (Fig. 9) that the spandrel walls carry remarkable compressive stresses on the internal side; moreover, the bi-axial bending is stressed.

Finally, the numerical analyses pointed out that, in the current state and considering only the self-weight, the structures carry quite high stresses. Even if such stresses are reasonably far from the safety limits, however by comparing the current state with the original one it can be observed that the compressive stresses are higher in each structural element (vaults, piers, spandrel walls); moreover, the introduction of the concrete structure produced a stress concentration in the spandrel walls, as a consequence of the concentrated loads transmitted from the reinforced concrete beams. On the basis of these considerations, it is can be stated that the retrofitting intervention realized in the 50’s was inadequate and even harmful.



(a) (b)

Figure 9: Minima principal stresses in spandrel walls (internal and external view) in span 5 and 6 [N/m^2] - current configuration: (a) load case A -10000 N; (b) load case B - 40000 N.

4 STRUCTURAL DIAGNOSIS

The monumental bridge, in spite of its physical impressive dimension (considering both the number of spans and transversal dimension), was realized and built in order to carry relatively low loads (pedestrians or coaches). This is confirmed by the limited thickness of the vaults, the scarce quality of the materials employed and of the building techniques.

The current improper use of the bridge, together with degradation phenomena (caused mainly by lack in maintenance), produced damages to the structure. The retrofitting intervention realized in the 50’s, even if designed with the aim of strengthen the bridge and make it able to carry heavier loads, has not produced the foreseen effects. Actually, such an intervention altered remarkably the structural configuration of the bridge, inducing the following effects:

- the compressive stress state in vaults decreased; this is not necessarily a positive effect from the structural point of view, because in arch structures the compression associated with permanent loads make the structure less sensitive to the variations of transient loads, to the presence of concentrated transient loads and to the displacements at the abutments;
- the compressive stress state in the spandrel walls considerably increased; actually, they carry nearly all the loads of the deck;
- the dynamical effect associated with passing loads is stressed from the presence of the reinforced concrete slab and beams. Actually, in the original structure, these actions burden the paving and the underlying infill; this latter played a energy-dissipation role. The reinforced concrete slab stiffened the deck and, therefore, the overall damping of the structure is today lower than the original one.

5 RETROFITTING INTERVENTIONS

Although damages and deterioration occurred in time, the bridge is not actually subjected to evolutive damage phenomena. However, the persistence of severe traffic loads may produce a progressive worsening of the most critical local conditions.

Even if the reinforced concrete structure introduced in the 50’s revealed its inaptitude and its noxiousness, conservation and economical issues suggest not to remove it. In fact, on the one side, the removal would produce further damages on the original structure; moreover, in any case, the 50’s intervention is not completely reversible (parts of the original structure were loss during the works, such the original parapets and portions of the spandrel walls). On the other side, the demolition costs would be enormous, not economically sustainable. Therefore, on the basis of conservation and safety issues, the following interventions are proposed:

- restrictions on the through traffic, both in terms of loads (maximum 35000 N) and speed; it is possible to restrict the traffic because a new road, not passing through the bridge, will be constructed;

– widespread consolidation of masonry, with the aim of infill the cracks and restore the mechanical behaviour of the material; local interventions are supposed in order to prevent the evolution of specific damage phenomena (e.g. the insertion of a transversal tie-rods, able to limit the out-of-plane movements of the spandrel walls or wing walls).

It is worth noting that the restriction on loads is the main action; actually, the permanence of the current use of the bridge (with no restrictions to heavy vehicles) would necessitate a stronger retrofitting intervention, unsuitable considering the conservation issues of the construction.

6 CONCLUSIONS

The structural diagnosis highlights that various aspects added up to the existing damage pattern. The intervention in the 50's has to be considered as a predisposing factor for the current static condition, in addition to the low masonry quality, to the lack of maintenance and the past damage pattern, possibly caused by seismic events (the area was struck by the earthquake in various occasions). In fact, the insertion of the reinforced-concrete slab induced a static-load increase in the bridge, in particular in the spandrel walls, which the beams weigh on. The current improper use of the structure may be considered as a motivating factor. Even if the bridge of Monte Carmelo was originally built for pedestrian or light weight non-motorized vehicles, from a few years it has to sustain heavy-vehicles flow (such as concrete mixers, lorries, etc.), causing the significant increase of static and dynamic loads. In particular, it is well-known that the vibration effects, induced by dynamic actions, may lead, through the years, to severe deterioration in masonry structures. These vibrations may be heightened by the presence of the r.c. slab of the 50's that causes the decrease of the energy-dissipation capacity for the bridge, stiffening the extrados structure.

The structural analyses, carried out through 3D numerical simulations (Finite Element Method), highlight that, in the current condition and taking into account only the dead load, the stress values seem to be quite high, even if the safety level do not appear to be inadequate. Nevertheless, the comparison between the stress state in the current configuration and in the original one (before the r.c.-slab building) shows that, apart from a general increase in all the structural elements, significant stress localization arises in the spandrel walls, near the beam supports. If the traffic loads (heavy vehicles) are taken into account, the stress state gets nearly dangerous values.

Therefore, the retrofitting design of the bridge has to be based on precise methodological concepts. Even if the r.c. slab caused negative static effects, its demolition seems to be unfeasible both from the economical point of view and because of the work complexity (in fact, unfortunately, it seems to be a non-reversible intervention, of which the scrapping may imply the partial ruin of large masonry portions). So, on the one hand, the preservation and restoration can not lead to the original condition, for which the static behaviour was better. On the other hand, the safety and usability of the whole bridge depend on new interventions.

The design strategies are the synthesis result of these structural and architectonical issues: if we realize that limiting the loading actions (through the regulation of the traffic flow) is the most important intervention, the strictly-structural retrofitting works can be reduced to the restoration of the masonry quality (where necessary), preserving at least the current configuration of the bridge.

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