Static and dynamic investigations on the roman amphitheatre (Arena) in Verona

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Summary

Deterioration phenomena are heavily damaging the roman amphitheatre located in Verona, built at the beginning of the Christian era, and the fear exists that they can undermine the stability of its main structural components. More and more strict investigation and surveillance programs are then being adopted in order to allow for adequate repair and maintenance programs being set up while ensuring acceptable safety conditions for its very active use. Hundred thousands people in fact, besides visiting the monument attend every year summer musical performances.

1. Introduction

Major concerns regarding the stability and structural safety of the huge roman construction derive from the actual conditions of the complex systems of vaults and of the free standing, curved and perforated wall called "the wing" (approximately 30 m high, the remain of the peripheral circle of dry stone masonry) that characterise the monument. The vaults have been almost all repaired and partially reconstructed during the past centuries (starting from the sixteenth century - when the stone seats, disappeared during the previous centuries, started to be completely replaced - till now). The "wing" has been subjected in the fifties to a strengthening intervention based on the use of the prestressing technique. In the first case the rain penetration inevitably causes the weakening of mortars and consequently detachments and falls of stones. In the second case, the actual structural behaviour is not completely clear and adequate protection of the steel wires against oxidisation is not ensured.

The investigations and surveillance programs are based on both experimental and theoretical investigation procedures. NDT techniques (core drilling, single and double flat jack tests, carried out by ISMES) have been used to determine the mechanical material properties and the state of stress of masonry. Dynamic investigations combined with structural identification procedures in order to identify damages and calibrate numerical models of the investigated structures , have been performed by the University of Padua. The experimental part of the dynamic investigations are based mainly on forced vibrations (pulse hammer and vibrodyne). The theoretical analyses are mainly based on physical models.

One interesting aspect of the mentioned investigations and surveillance programs was their preliminary application as a tool for taking decisions regarding the use of the monument. After having characterised as previously described some typical vaults (the critical structure as far as the overall safety of the monument is concerned) a net of accelerometers have been installed on them and connected to an automatic recording system. This allowed for real time evaluations of the actual structural response of the vaults system to the more severe load conditions which could occur while thousands of people are attending the musical performances. The possibility of taking in due time adequate safety measures ensured by such monitoring system was considered a viable alternative to the closure of the monument for the entire (presumably very long) time required to complete the preliminary investigations and to design and execute all the required repair and maintenance interventions.

In the following, the methodologies that have been adopted to carry out the investigations and the more relevant results that have been so far obtained for assessing actual efficiency of the prestressed "wing" are briefly described in order to give a synthetic but complete idea of the work that is being carried out.

2. Main structural characteristics of the "wing"

From the structural point of view the "wing" is very simply a slender corbel with no lateral supports or confinements then that very uncertainly, and in any case unilaterally, offered by the remaining vault (Fig. 1). The lack of structural redundancy and of any kind of protection against the action of adverse environmental agents (e.g. rain, freeze-taw cycles, wind, soil differential settlements), besides of course what could it have suffered due to damages caused in the past and to the lack of maintenance, make it very high the risk inadequate structural safety conditions.

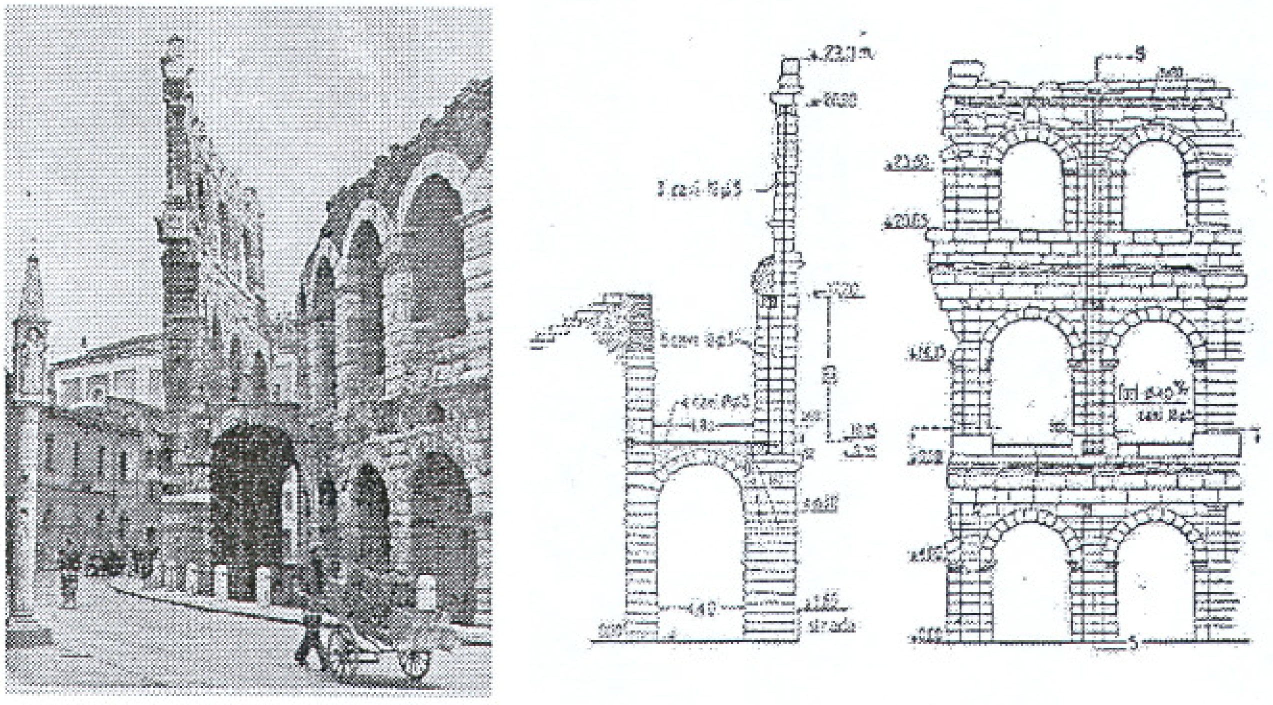

Figure 1 : view and typical section of the wing

It was around the thirties that the people responsible for its conservation became aware of the necessity of strengthening intervention as it was clear that, even due to a pronounced inclination, the wall could not resist the relevant horizontal action which could be expected in that area due to wind and earthquake motions (the region is in fact subjected to a moderate seismic risk).

The debate (long and very interesting) on how to intervene (at least three or four different proposals were discussed) was interrupted by the WW2, when temporary masonry buttresses were erected to protect the wall as much as possible against explosions (Fig. 2).
When the problem of removing the buttresses and at the same time strengthening the "wing" was faced after the war, at the beginning of the fifties, the use of the prestressing technique was

considered a very smart solution of the always complex problem of preserving the monument while ensuring acceptable structural safety conditions. The intervention, as described by the designer himself [1], consisted essentially in introducing harmonic steel wires in proper vertical holes (40 mm in diameter according to [1]) drilled in the stone (Figure 3, a - 18 wires, 5 mm in diameter, in each of the 6 holes of each pillar).

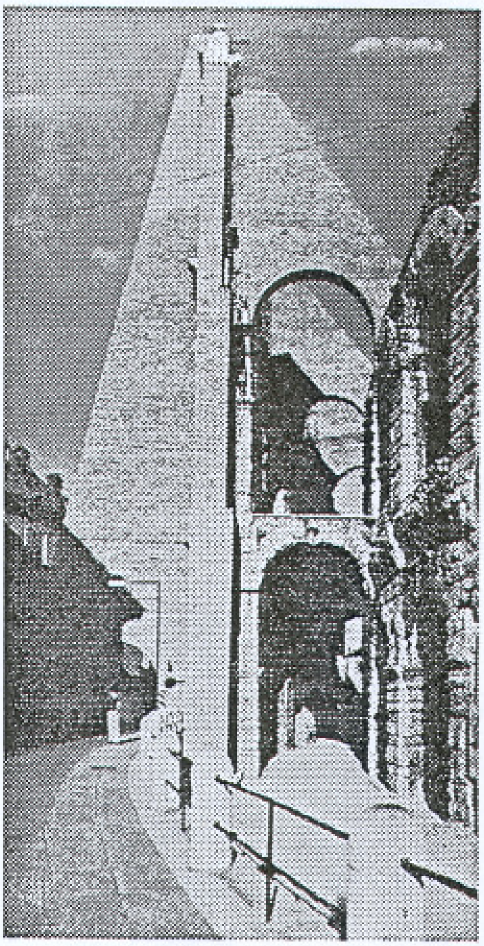

Fig. 2: view of the WW2 buttresses

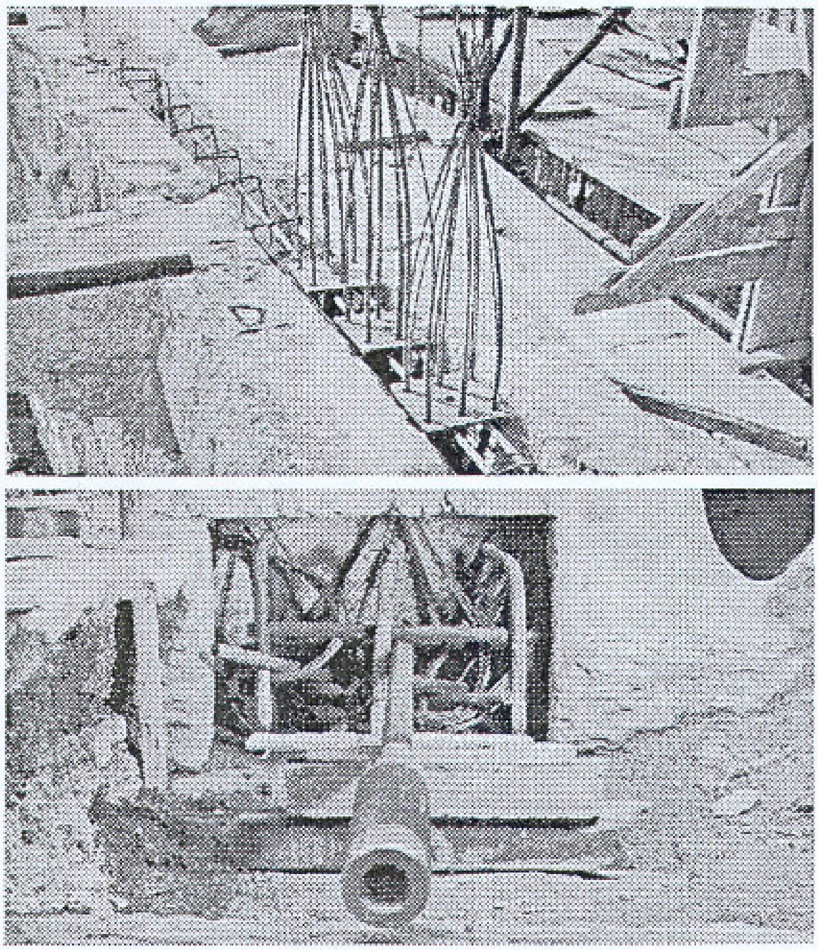

Fig. 3 a, b: details of the 1952 intervention

The wires were anchored in a concrete block at the bottom, in a niche (Figure 3 b) prepared in the wall at the level of 9.0 m - i. e. at the extrados of the vault that links the Wing to the Arena - and, at the top, half in a r.c. beam, built in a channel cut in the stone at the level of 26.9 m (Figure 3 a)- and half by means of special plates located at level 17.2 m (where a set-back is formed). The wall was then strongly concreted, also thanks to the extensive injections of a water-cement admixture that were assumed to be capable to fill every cavity, and particularly the artificial holes, of the masonry [3].

Unfortunately some of the most critical issues of the intervention are ignored or contradictorily described in the cited reports [1, 2, 3], so that it is very difficult now to draw any conclusion on the actual and present efficiency of the intervention itself.

This is the case of the very critical stability conditions occurring a the base of the wing, where the overturning moment induced by horizontal forces has the maximum value and the stabilising effect of the tensioned wires suddenly is missed. They are ignored in [1], where moreover only horizontal forces acting towards the outside of the amphitheatre are considered, and imprecisely examined in [2]. A photographic documentation exists showing that the rigid block, formed by the two upper prestressed orders, is really bonded to the rest of the wing and to the main body of the Arena as schematically indicated in Figure 1. Steel (not prestressed) rods are placed horizontally, on the extrados of the vault (at the 9.00 m level), and vertically, from the concrete block into the first-order pillars. The eccentric position of the vertical ties evidently reflects the assumption made on the direction of the horizontal forces. Why such assumption was made, where exactly the ties are, their number and dimensions, are unknown.

Even the position of the prestressed wires is a matter of evident contradictions. According to [1] the total prestressing force should be eccentric, in order to counteract horizontal forces acting towards the outside of the Arena, but it is not clear what the eccentricity is. According to [2] the masonry should be concentrically prestressed, and this seems to be confirmed, even if not uncontroversially, by some photographic documentations.

The efficiency of the protection of steel against oxidisation that should be ensured by the water-cement injection is also finally very doubtful. Particularly the possibility that such admixture could really penetrate (without using all the additives and injection devices employed in modern prestressed structures) the 40 mm in diameter, approximately 20 m long holes, where 18 wires, 5 mm in diameter, are inserted is questionable.

The assessment of what actually the safety conditions are of the "wing" under horizontal forces acting towards both the inside and the outside of the amphitheatre, evidently requires that a number of very important questions are answered, particularly regarding:

a) the actual distribution of stresses induced by the prestressing forces, compared to the design values as they can be drawn from [1];
b) the actual position, dimension and anchorage efficiency of the steel rods;
c) the possible oxidisation attacks to both steel wires and steel rods;
d) the overall structural behaviour of the wing , and particularly the interaction with the main body of the amphitheatre.

3. Preliminary non destructive investigations

NDT methods are being used to answer the first three of the above questions. Flat-jack tests, combined with cores drilling allowing a precise knowledge of the masonry composition and construction technique, offer a very practical tool for getting reliable informations on issue a). Direct visual inspection are possible, and in fact being executed, to solve point b). For the time being, however, no practical test method has been proposed allowing direct measurements regarding the possible presence and extension of oxidisation processes on the steel wires and rods inserted into masonry. In this respect, the only viable alternative to direct controls appeared to be the use of stress measurements and local inspections (by drilling cores) on the state and composition of the internal parts of masonry as indirect indications of possible occurrence of chemical alteration and consequent reduction of the sections of steel wires and rods.

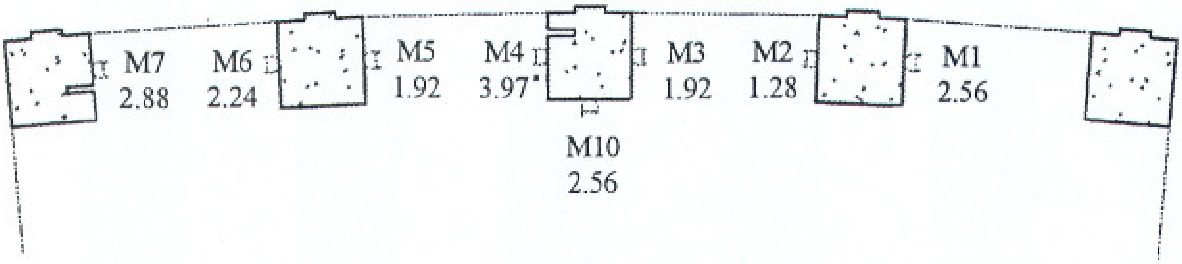

Fig. 4: position of flat-jack tests and corresponding values of the measured stresses in MPa
The principal results of the flat-jack tests are summarised in Figure 4, where the measurements executed at a level approximately 1.5 m above the extrados of the vault (Figure 1) are given.

The first important question raised from test results is that the mean values of the measured stresses are approximately double then the design values given in [1]. It came out in fact, and this was a really remarkable contribution of the inspections made by drilling horizontal cores through the dry joints between the stone elements, that the actual resisting section of the masonry is reduced by approximately one half due to the construction technique has been used. As it was the rule for similar type of dry masonry (anathirosis), the huge stones have horizontal concave faces as schematically indicated in Figure 5 a, what makes evidently much easier to obtain good contact conditions without using mortar joints.

Important consequences derive from such situation as far as the axial and flexural stiffnesses, and then the overall structural behaviour of the wall are considered. This effect has been numerically quantified by using the mathematical model shown in Figure 5 b.

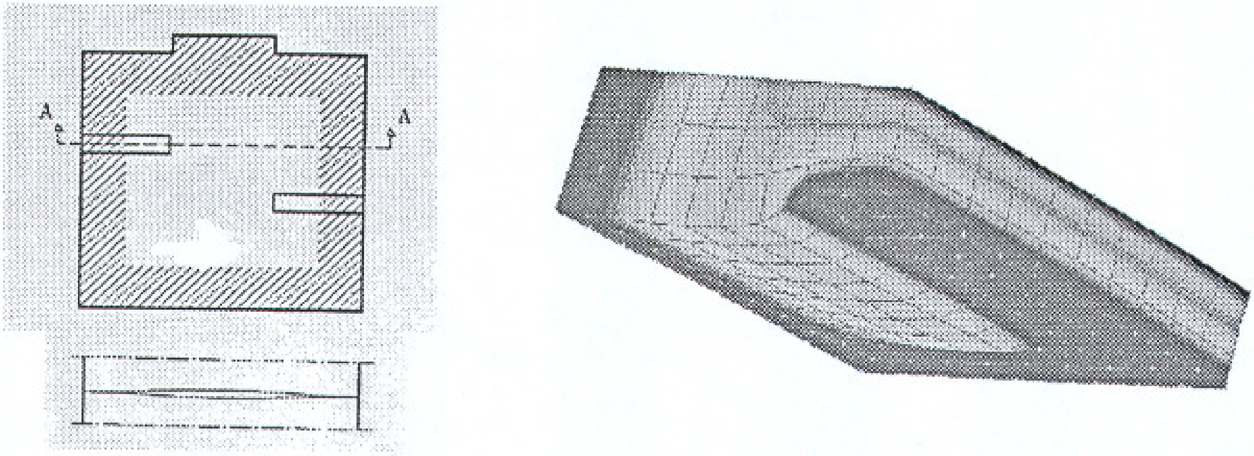

Fig. 5: cavity at the joints and mathematical model of its effect on the masonry deformability

Other important observations can be drawn considering the very uneven distribution of the measured compressive stresses, with significant variation of the mean values between the pillars and of the local values within each pillar. The uncertain and variable geometry of the actual resisting section, together with the uncertain structural behaviour of the perforated wall (especially under forces acting in its own plane, when a frame-like response can be activated) are certainly among the principal causes of this fact.

However no absolute guarantee can be given that the prestressing forces are of the same intensity and in the same position as they could be supposed to be according the original design and execution of the intervention. The supposition that the actual distribution of stresses could be and indirect sign regarding the state of conservation of the steel wires receives further support from the results of the visual inspections inside the cores drilled through the horizontal dry joints of the masonry. The space between the stone faces due to the concavity (Figure 5) is in fact empty, what seems really not reasonable if the holes containing the wires should actually be filled with injection material that is supposed to protect the steel. It must be taken into account in fact that the cavity is intersected by the vertical holes, and its minimum dimension is of the order of 15-20 mm, i.e. much greater then the dimension of the spaces left free in the vertical holes by the 18 wires.

Finally, it must be considered that stress concentrations are possible even under moderate horizontal actions of the order of 4-5 MPa, and locally even greater due to the inevitably uneven contact conditions. Combined with the deterioration phenomena threatening the stone, this can explain local crashing effects (especially at the corners) which are damaging the stones.

4. Dynamic analyses

The overall structural behaviour of the wing, and particularly its interaction with the main body of the amphitheatre, is being experimentally analysed and theoretically modelled by using forced vibration tests and dynamic identification procedures.

Modal testing [4] has been already carried out by using ambient vibrations and forced, both impulsive and stepped-sine excitations [5]. A schematic view of the testing procedure and of the main results of the signal treatment is presented in Figure 6. Given the positions of the acceleration transducers, shocks are applied separately in any of such positions while the harmonic exciter has been placed near the accelerometer n. 3.

The impulse tests allowed for the identification of modal frequencies in a rather wide range of values (from 0 to 25 Hz in Figure 6). Stepped-sine tests allowed for very accurate estimation of few, lower frequencies (in the range between 1 and 3 Hz, as shown in Figure 6) and, most important for the following application of dynamic identification procedures [5,6], of the corresponding modal shapes.

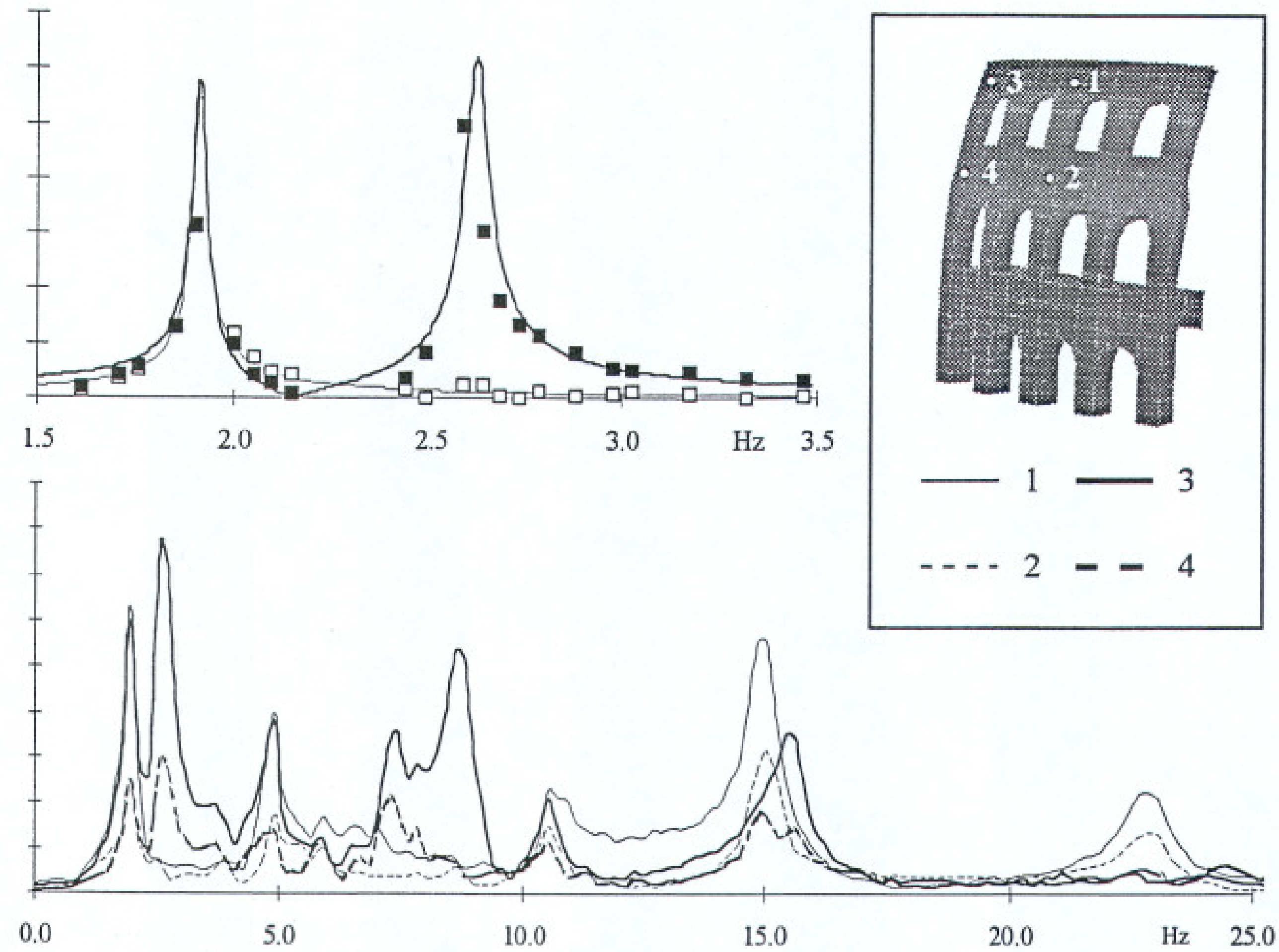

Fig. 6: FRFs obtained from an impulse test (evidencing the modal frequencies in the range 0-25 Hz) and from a stepped-sine test compared to the theoretical one generated by the identified model in the range 1-3 Hz.

Representative results of modal extractions, performed through simple fitting algorithms, are reported in Table 1.

The FEM model shown in Figures 6 and 7 has been then identified by using different approaches and numerical procedures [6], and particularly by both using a relative high number of modal frequencies, as they were obtained from shock tests, and only two modal frequencies but together with the corresponding modal shapes, obtained from the stepped-sine tests. The best fitting between experimental and theoretical responses was obtained, as expected, in the second case. A regular non-linear programming algorithm (i. e.: the conjugate gradient) was used in order to minimise the target function. The comparisons demonstrating the excellent results of the identification procedure are shown graphically in Figure 6 and numerically in Table 1.

	f_1	f_2	f_3	f_4	f_5	f_6	f_7	f_8
Real	1.92	2.61	4.83	5.87	6.10	7.10	8.62	10.65
Theoretical	1.91	2.63	5.01	5.85	6.42	6.91	9.15	10.68

Table 1: comparison between the first two experimental and theoretical modal shapes

What is interesting to note is that besides satisfying the fitting criteria of the experimental parameters used in the identification procedure, this led also to a good theoretical estimation of the higher modal frequencies, as it is shown in the comparison reported in Table2.

	f_1 = 1.92 Hz				f_2 = 2.61 Hz			
	1	2	3	4	1	2	3	4
Real	1	1	0.36	0.36	0.06	1	-	0.32
Theoretical	1	1	0.37	0.37	0	1	0	0.34

Table 2: comparison between experimental and theoretical modal frequencies

It is then expected that also the theoretically calculated higher modal shapes (a representative selection of which are shown in Figure 7) are good estimates of the real ones. In this case, more detailed modal testing could in the future allow for modal extraction of the higher modes, and consequently more refined estimates of the relevant parameters on which the structural response of the wing depends. This could very much facilitate the monitoring of possible modifications of the state of conservation of the structure (comprised the crucial prestressing system) simply by means of periodical repetitions the dynamic analyses.

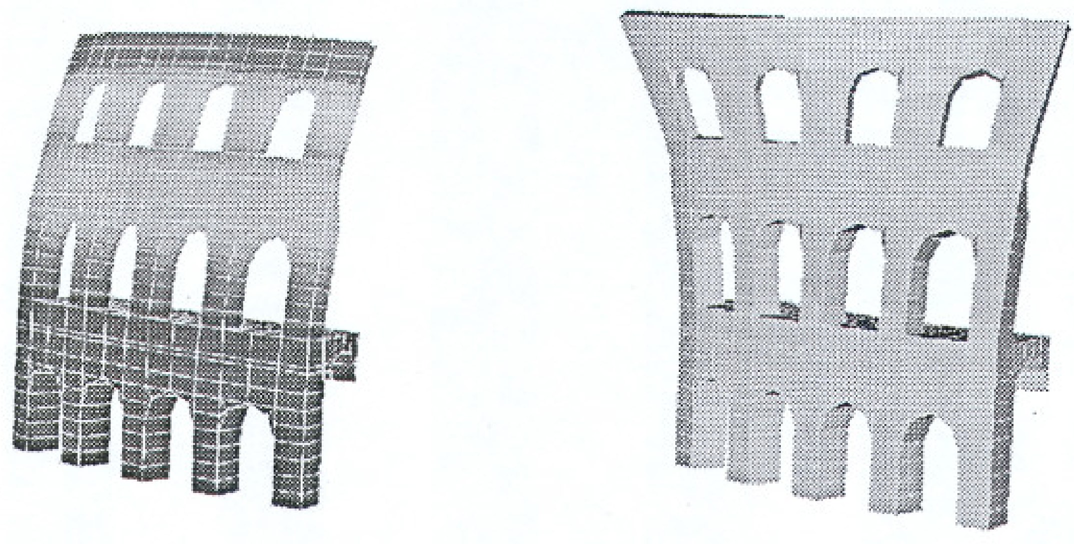

Fig. 7: example of theoretical modal shapes

In this respect the following important considerations will be taken into account in the future work in order to make as much as possible efficient the combined use of different modal testing and dynamic identification procedures as a tool for structural damage detection. This is in fact a very critical point of the surveillance program considering the limitations has been pointed out in chapter 3 of NDT methods from this point of view.

The first consideration regards the choice of the parameters that have been used so far for the dynamic identification and the values such parameters were given by the adopted procedure. As the model is as much simple as possible, i.e. linear elastic with rigid supports at the base and rigid connections between the vault and the main body of the Arena, it was assumed that the real deformability of the soil-foundation system and of the vault can be taken into account through adequate equivalent moduli of elasticity assigned to the masonry respectively of the first order of the wing and of the vault itself. Together with the modulus of elasticity of the masonry of the upper two orders of the wing, they are the three unknown parameters that have been used for the identification procedure.

The values of these parameters given by the analysis are 5700 MPa, 8000 MPa and 11900 MPa respectively for the first order of the wing, the vault and the upper part of the wing, which are evidently very low compared to the value of the modulus of elasticity of the stone (of the order of 60000 MPa). More calibrations will be certainly done in the near future, however it must be mentioned that approximate evaluations of the modulus of elasticity of the upper part of the wing, obtained by the single flat-jack test measurements, corrected by taking into account the effect of the cavity between the contact faces of the stone elements as indicated in chapter 3 (Figure 5), are not in contradiction with the results of the identification process.

The second consideration regards the type of signals can be used in the identification procedure. Very interesting could be in this respect the possibility of using ambient vibrations signals. The application of forced vibrations is in fact rather difficult and costly in the conditions the structure is, as an appropriate scaffolding must be every time installed and removed. Ambient vibrations, on the contrary, are being automatically and continuously recorded by the net of acceleration transducers that has been recently installed exactly in the same positions indicated in Figure 5, where the instruments were during the previously described tests.

Same preliminary attempts have been already done to verify this possible alternative and much easier procedure. Transducers responses are treated as random signals, and their frequency contents are described by the Auto-Spectral Density (PSD). Under the assumption that the ambient excitations (due mostly to wind and roadway traffic) have a flat PSD (white noise), FRF amplitude is proportional to the square root of PSD. Electrical resonance of the acquisition system, which are not negligible for low intensity signals, makes unfortunately unreliable the frequency responses up to 5 Hz. It was however demonstrated that at least qualitative comparison between theoretical and experimental responses are elsewhere possible.

6. References

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