Assessment of the seismic capacity of triumphal arches

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ABSTRACT: In this paper three triumphal arches of Naples churches are analysed both through FEM nonlinear analysis and limit analysis for deriving indications on the seismic capacity of these important structural elements. The three cases considered in this paper are in fact quite different in the geometry, in the behaviour and in the ultimate multiplier of horizontal load. Quite general quantitative indication can be inferred even in these very different cases.

1 INTRODUCTION

The triumphal arch is a recurrent constructive element in churches, particularly the basilica-type churches, and is defined as the element that separates (or connects) the zone of nave and aisles (hall) from the zone of crossing, transepts, and chancel. In several cases two elements that can be appointed as triumphal arch can be identified in the church, namely the arch between the hall and the crossing/transepts, and the arch between the crossing/transepts and the chancel.
It is a masonry wall, with either a single arched opening and quite large abutment piers, or a main arch flanked by two lower and narrower openings. Due to its arched shape (semicircular, pointed or elliptic) and the generally huge dimensions, this element evokes the triumphal arch of the roman empire age, thus the origin of its name. Generally it is made of either wall-facing wedge-shaped stones or is decorated by mosaics, marble elements, enriched mouldings stucco finishes.

This element has a particular importance in the seismic analysis of basilica type churches: damage surveys conducted on churches subjected to earthquakes (Doglioni et al., 1994), showed that very often the triumphal arch experiences severe damage resulting from in-plane horizontal actions, which leads to partial or total collapse. This effect is mainly connected with the structural scheme of the basilica plan churches, which features high stiffness concentration at the intersection between the nave and transept, thus attracting high seismic forces.

In this paper three triumphal arches of Naples churches are analysed for deriving preliminary indications on the seismic capacity of these important structural elements.

2 ANALYSIS METHODS

2.1 FEM analyses

Linear and nonlinear analyses of the selected case studies, the triumphal arches of some important churches of the historic center of Naples, have been carried out through FEM. For the sake of brevity in the following only the nonlinear analyses are discussed.

A smeared crack model has been utilized for accounting for the brittle behavior of masonry, as it is implemented in computer code ABAQUS (Hibbit et al.,1997). It is well known that the computational mechanics of brittle structures can be approached in two different ways: the discrete and the smeared crack models. While in the former approach cracking is accounted for by modifying the geometry and by keeping the interior of the body linear elastic, in the latter approach the geometry is maintained fixed and the cracking process is completely introduced via constitutive law, by affecting the material stiffness at every integration point. The ABAQUS "concrete" model is a fixed multi-crack model based on a simple yield surface with isotropic hardening and associated flow when the state of stress is predominantly compressive, and uses damaged elasticity to account for the cracking, the occurrence of which being defined by a so-called "crack detection surface". This failure surface is assumed to be a simple Coulomb line written in terms of the first and second stress invariant.

This concrete model has proven to be able to reasonably predict the masonry behavior in monotonic loading, as long as proper material definition is provided (Giordano et al., 2001). The concrete model basically requires the stress-strain curve in compression to be defined in tabular form as function of plastic strain, the shape of the failure surface via the *FAILURE RATIOS option and the post-cracking tensile behavior defined by the *TENSION STIFFENING option. This last feature actually makes no sense for masonry, but a small amount of tensile resistance should be anyway provided to avoid numerical instability problems.

In order to correctly calibrate the model parameters, a curve-fitting procedure has been adopted using the results of various experimental tests performed at the ISMES Laboratory, in Bergamo (Italy), on tuff masonry panels (Giordano et al., 2001). It is worth pointing out that all case studies herein examined are made of tuff masonry blocks, as typically occurs in the historic buildings of Naples.

2.2 Limit analysis

Due to the inherent complexity of the masonry behaviour, characterised by highly nonlinear response in compression state and by nearly no tensile strength, the use of tools for simplified analysis are particularly advisable. Such tools in fact allow to obtain, though in an approximate way, information on the strength of masonry structures and constitutes both a starting point for the assessment of the behaviour of masonry elements and a check instrument for the appraisal of the result obtained with more sophisticated nonlinear numerical approaches.

In this context, the limit analysis, firstly applied to masonry structure by Heyman (1969), represents a particularly effective tool for deriving an estimate of ultimate strength capacity.

The hypotheses on the masonry behaviour, as sketched by Heyman (1969), are: no tensile strength; infinite compression strength; absence of sliding at failure. Under these hypotheses, the masonry material becomes an assemblage of rigid parts, held up by mutual pressure, and the collapse of the structural elements is characterised by the development of non dissipative hinges which transform the structure in a mechanism. The term mechanism indicates a displacement distribution in the structure produced by inelastic deformations (the formation of hinges), which occur in a finite number of sections due to disconnections and cracking.

In this paper, the collapse multiplier of the triumphal arches subject to the self weight and the tributary roof loads, and to a distribution of horizontal loads (which simulate seismic-type loads) of gradually increasing resultant value, has been evaluated by applying the kinematic theorem. In particular, on the basis of geometrical considerations and also taking into account the results of the linear and nonlinear FEM analyses, the zones where locating the four hinges have been established, thus a class of collapse mechanism has been identified. Then, by varying the exact position of each hinge in the above defined zone, a class of collapse multipliers has been obtained. Finally, the actual collapse multiplier has been selected as the minimum among the computed ones.

3 SELECTION OF THE CASE STUDIES

The three triumphal arches examined in this paper respectively belong to three basilica plan churches, the church of San Ippolisto Martire, the church of San Giovanni a Mare and the church of San Giovanni Maggiore. While the first church is located in Irpinia, the other two represent two of the several basilica plan churches of the historic centre of Naples, which are currently being analysed by the authors in the context of a wide research activity.

Beyond the historical, architectural and social values of the churches selected for the analysis, the choice was also related to some specific geometrical aspects of the triumphal arches which stimulate the structural study: in fact, the four churches analysed have quite different geometrical layout of the triumphal arches: in the first case, a slender, single opening element; in the second case a non symmetrical element, with a main semicircular arch, flanked by two lower and narrower pointed arches; in the third case, a large semicircular arch with very wide and stocky lateral abutments.

According to several studies conducted on the structural behaviour of the triumphal arch (Lagomarsino et al., 1999), the classification of this element can be based on the main geometrical dimensions and on their ratios. In particular, the geometrical parameters which can be considered of primary importance in affecting the structural behaviour of triumphal arches are: piers height: , H, piers width, B, arch rise (radius for circular arch), f or R, arch span, L, thickness at crown, s. The correlation between the relative values of geometrical parameters and the type of structural behaviour of the arch, which justifies the geometrical approach utilised in the structural classification, has a twofold motivation: generally speaking, in masonry structure the compressive effect of gravity, which is strictly connected to the geometrical dimensions, helps stability by decreasing the dangerous tension stress produced by lateral loads; further, with specific reference to thrusting structure like arches and vaults, the development of a mechanism is affected by the load eccentricity at basis, which, in turn, depends on the ratios between geometrical parameters (Doglioni et al., 1994).

Therefore, though the above classification of arches based on the geometrical ratio still holds, the analysis of actual case studies reveals quite complex geometries and poses crucial problems in the definition of the geometrical parameters. On this specific aspect, a study aimed to derive a parametric expression of the ultimate strength as a function of the geometrical characteristics is being developed by the authors.

In the following the results of FEM and limit analyses are reported.

4 THE TRIUMPHAL ARCH OF THE S. IPPOLISTO MARTIRE CHURCH

The S. Ippolisto Martire church in Atripalda (Avellino, Italy), was built between 1584 and 1612, on a previous paleo-christian basilica of the IV century a.c.. In the XVIII century several additions and restorations were made, including the façade, which consists of two orders on which the triangular tympanum with a central rose window is imposed. In the first order the portal is characterised by a semicircular arched doorway. The plan and the transversal section at the triumphal arch between the crossing/transepts and the chancel, are provided in figure 1.

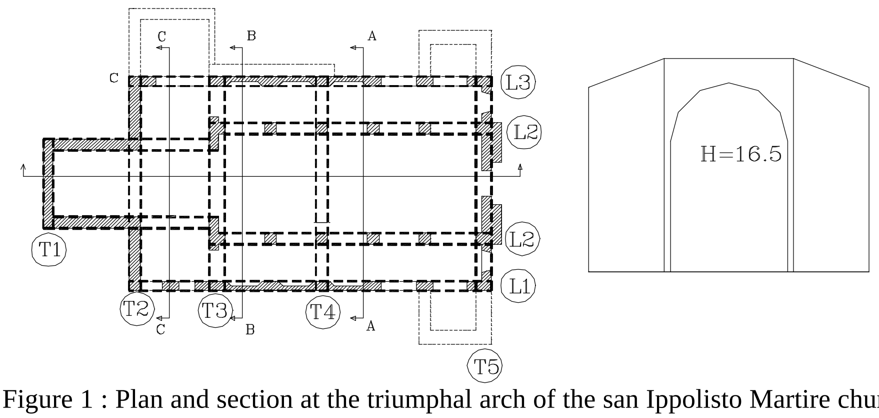


Figure 1 : Plan and section at the triumphal arch of the san Ippolisto Martire church

The nave of the church is 11.6 m wide, 28 m long and has a maximum height of 16.5 m, while the aisle are 5.0 m wide and 8.5 m high. The masonry walls have thickness varying between 1.0 and 1.2 m, the columns of the nave arcade have rectangular section 1.2 x 1.2 m. The chancel has a rectangular plan shape, 8.8 x 11.6 m, and height variable between 14.8 and 18.0 m. The structural elements are made of tuff masonry. The structural system of the roof of the nave is king-post timber roof, while the lateral aisles are covered by quadripartite vaulting systems with four diagonal ribs. Following the 1980 Irpinia earthquake, the crossing and transepts of the church, which were surmounted by heavy reinforced concrete roof system built after a previous earthquake of 1930, were destroyed.

In figure 2 the main geometrical dimensions of the triumphal arch are provided. It is worth pointing out that this arch is a quite slender structural element, due to the small arch thickness at crown (s=1.9 m) and to the slenderness of piers (H/B=2.1). The semicircular opening is quite large, equal to 46% of the total surface of the element.

A fairly fine 2D mesh, consisting of 998 shell elements (S4R), has been used to model the element, which has been subjected to the vertical load deriving from the self weight and form the roof loads, and to horizontal loads of increasing intensity, constantly distributed along the height of the element. In figure 3 (a) the deformed configuration at the last load increment of the analysis, with the vertical stress contour superimposed, is shown. In figure 3 (b) a vector representation of the principal stresses is provided, which clearly evidences the stress flow in the element. Both the plots allow to derive the stress state in the element, characterised by tensile components at the left side of both pier bases, at the intrados on the left haunch and at the extrados on the right haunch of the arch. In figure 4 (a) the same vector representation as figure 3 (b) is utilised for the principal strains, while in figure 4 (b) the plastic strain components are provided. From these figures it is evident that the maximum principal component (tensile) is always quite low, due to the brittle behaviour of the masonry material.

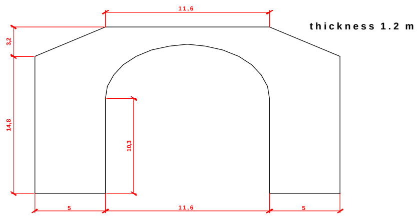
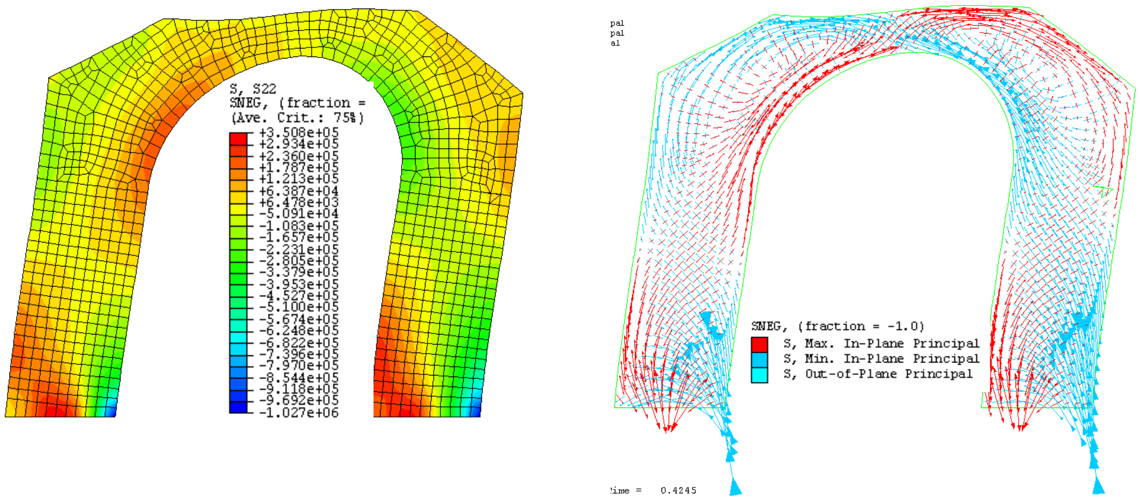


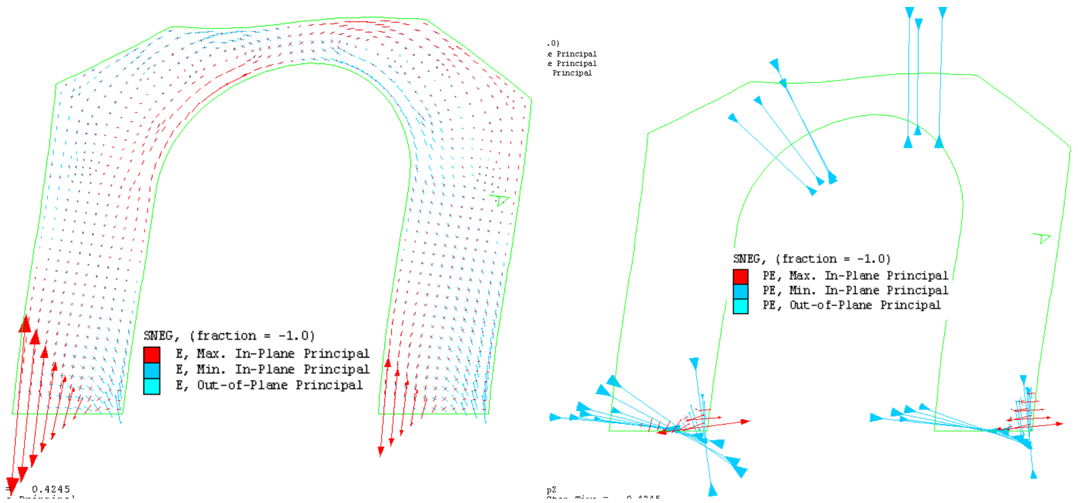
Figure 2 : Geometry and dimensions of the triumphal arch of the san Ippolisto Martire church



(a) (b)

Figure 3 : FEM analysis : (a) vertical stresses; (b) principal stresses.

All these plots have been utilised for individuating the class of collapse mechanisms, namely the zones where the hinges are likely to occur in the masonry. It is well known that the collapse under horizontal load of a simple arched element, like the one herein examined, implies the development of an emi-symmetric mechanism, with formation of four hinges. However the mechanism and more generally the collapse mode depends on geometrical and mechanical factors. For the generic arch, three mechanism types can be activated: a local mechanism, which is characterised by the local failure of the arch only (four hinges in the arch); a global mechanism, which is characterised by the presence of two hinges at the base of the piers and two hinges in the arch; a semi-global mechanism, characterised by the presence of one hinge at the base of one pier and by three hinges in the arch. The local mechanism occurs when the pillars are very stocky (B/H>1), while the global and semi-global ones are typical of more slender structure.



(a) (b)

Figure 4 : FEM analysis : (a) principal strains; (b) plastic strains.

On the basis of the results of linear (not reported here) and nonlinear analyses, a global mechanism type has been a priori selected for the element, and four critical zones in the arch have been identified (left side of both pier bases, intrados on the left haunch, extrados on the right haunch). Varying the position of the hinges in these critical zones, the class of the cinematic multipliers ha been defined, and the minimum value has provided the collapse multiplier. The final collapse mechanism is represented in figure 5. It is fairly close to the deformed configuration obtained from the FEM analysis, and it is quite interesting to note that the inclination of the plastic strain vectors is directly related to the hinge aperture.

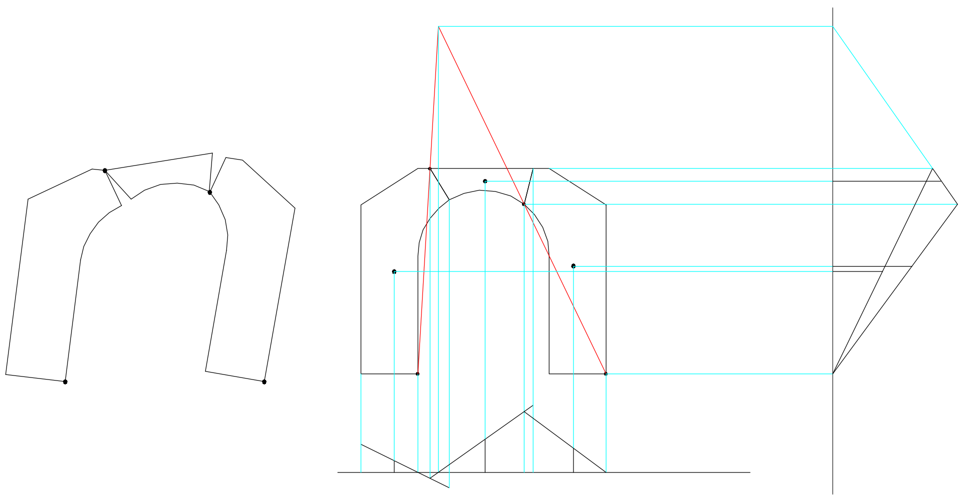


Figure 5 : Limit analysis: collapse mechanism.

In figure 6 the comparison between the push-over FEM analysis and the limit analysis is provided. In particular the curve force-displacement, in terms of horizontal force resultant, F, normalised to the vertical load W, vs the horizontal displacement of the arch's top right joint, is depicted in the graph together with the horizontal line corresponding to the collapse multiplier. The comparison between the collapse multiplier (F/W=0.28) and the maximum load capacity obtained via FEM (F/W=0.24) confirms that the former provides un upper bound of the element capacity.

It is worth noting that in the limit analysis unlimited compressive strength and no tensile capacity is assumed for the masonry material, while in the model used in the FEM analysis finite values of the compression and tensile strength have been adopted. In order to investigate if higher strength values could possibly lead the model to overcome the limit analysis multiplier, a sensitivity analysis, not reported here, has been carried out, by varying the compression and the tensile strengths, as well as their ratio. The results show that by increasing the compression strength of the material, the ultimate strength capacity of the element does not significantly vary, while a larger deformation capacity can be observed.

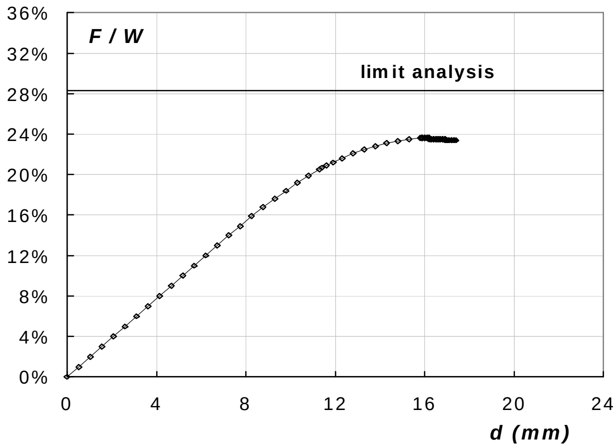


Figure 6 : FEM analysis vs. limit analysis results.

5 THE TRIUMPHAL ARCH OF THE SAN GIOVANNI A MARE CHURCH

The church of S. Giovanni a Mare is a classical Angioin style monument. It is one of the most ancient church of Naples: the first document reporting about the church is dated 1186. Even though several additions and alterations have strongly modified the original plan of the church during the centuries (Mele et al., 2001), the major elements featuring the basilica type churches are still recognizable, namely: the main nave, the two lateral aisles, the transept and the chancel. In figure 7 the church plan and a transversal section on the triumphal arch, are provided.

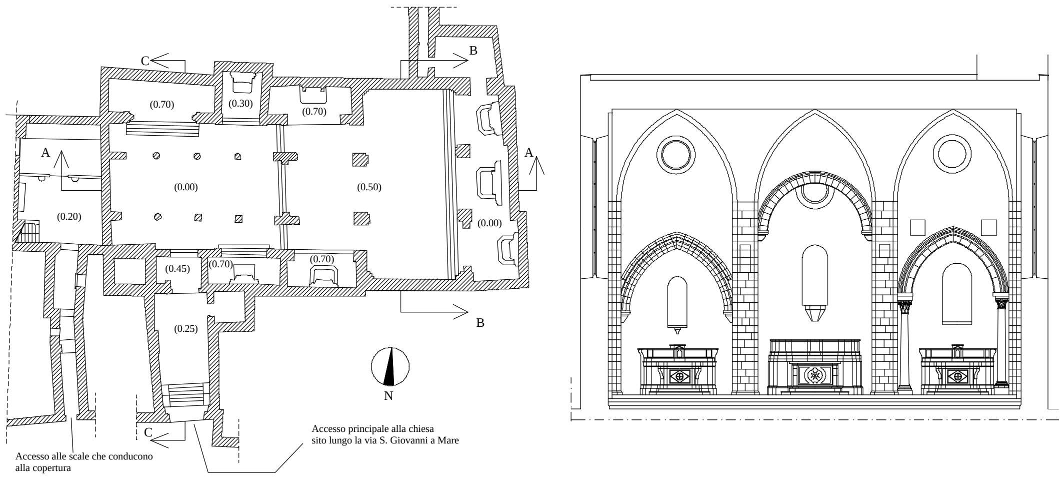


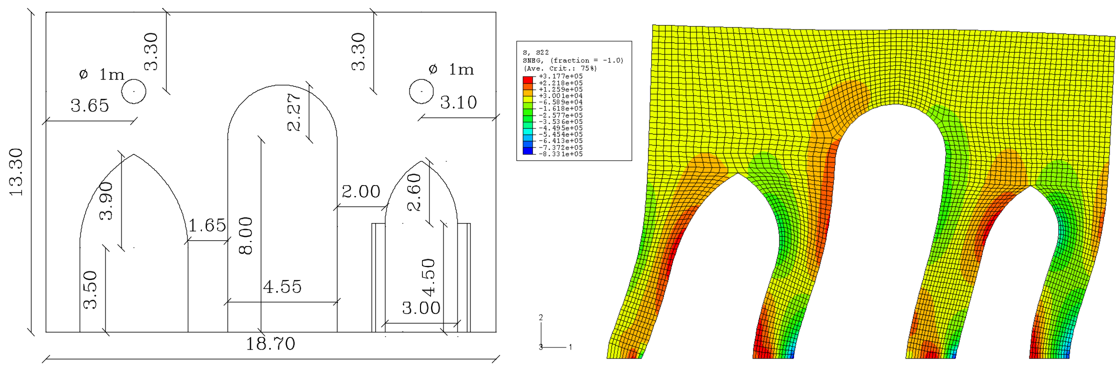
Figure 7 : Plan and section at the triumphal arch of the san Giovanni a Mare church.

The church is 37,35 m long and 18,7 m wide, has a maximum height of 13,3 m (roof of the transept) and a minimum height of 7 m (vaults of the lateral chapels). The nave width varies between 4,75 and 4.0 m, while the aisles are 2.35 m wide. An atrium is located ahead of the nave, at the west end of the church; the doorway is located on the lateral south elevation. The west part of the three naves (floor level 0.0 m) consists of four spans (three columns per two arcades), while the east part (floor level + 0.50 m), originally occupied by the first transept, consists of two lines of two pillars, which support pointed arches. The structural system of the roof of the church is made of several vaults: barrel, groin cross and quadripartite ribbed vaulting systems. The walls have thickness varying between 0.80 and 1.00 m, and are made of tuff masonry.

In figure 8 (a) the geometrical layout of the triumphal arch is provided. The element has a main semicircular arched opening, flanked by two lower and narrower pointed arches. The global height and width of the element are 13.30 m and 18.71 m, respectively. The piers supporting the arch sequence are quite slender, while the thickness of the arches at crown is considerable.

The 2D mesh utilised to model the element, under the vertical load deriving from the self weight (2702 kN) and form the roof loads (1784 kN), and to horizontal loads of increasing intensity, linearly distributed along the height of the element, consists of 2457 shell elements.

In figure 8 (b) the deformed configuration at the last load increment of the analysis, with the vertical stress contour superimposed, is shown, while in figure 9 the vector representation of the minimum (compression) and maximum (tensile) principal stresses is respectively provided. These plots allow to derive the stress state in the element, characterised by tensile components at the pier bases, at the left haunch of the three arches, and at the upper edge of the element above the main arch. In figure 10 (a) and (b) the vector representation of the principal strains and of the plastic strain components are respectively provided.



(a) (b)

Figure 8 : Triumphal arch San Giovanni a Mare church: (a) geometry and dimensions; (b) deformed configuration and vertical stresses.

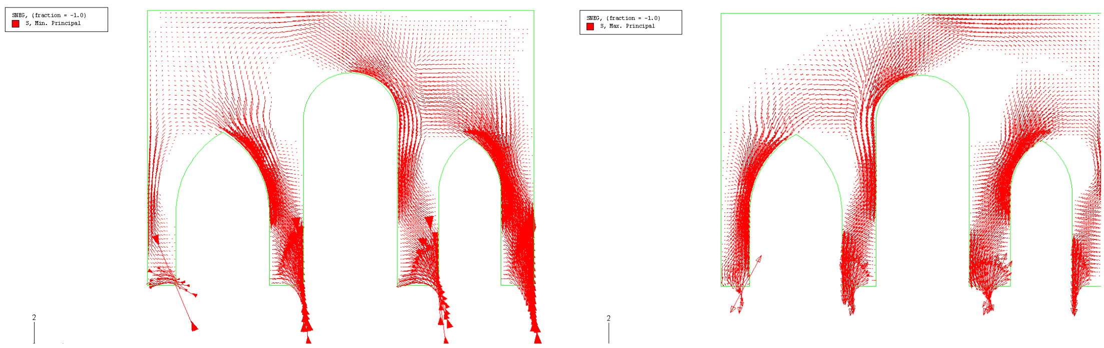
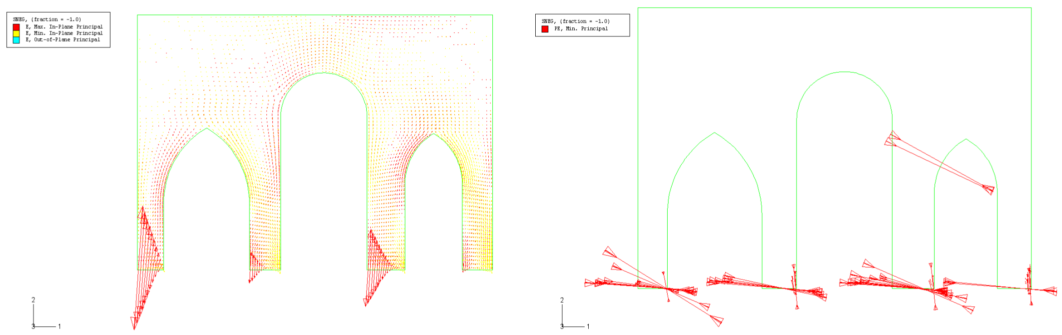
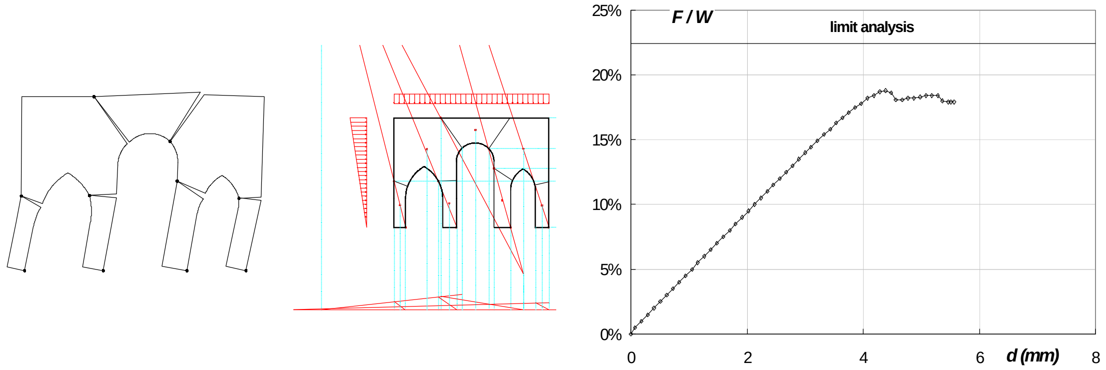


Figure 9 : FEM analysis: principal stresses.

As in the previous case, all these plots have been utilised for individuating the class of collapse mechanisms, namely the critical zones where the hinges are likely to occur. Varying the position of the hinges in these critical zones, the collapse multiplier has been computed as the minimum one. The final collapse mechanism, represented in figure 1 (a), closely match the deformed configuration obtained from the FEM analysis. Finally in figure 11 (b) the comparison between the FEM analysis and the limit analysis is provided, in terms of normalised horizontal force resultant, F/W vs. horizontal displacement. Also in this case the collapse multiplier value (F/W=22.4%) is slightly larger than maximum load capacity obtained via FEM (F/W=18.8%).



(a) (b)
Figure 10 : FEM analysis: (a) principal strains; (b) plastic strains.



(a) (b)
Figure 11 : (a) collapse mechanism; (b) comparison FEM analysis vs. limit analysis.

6 THE TRIUMPHAL ARCH OF THE SAN GIOVANNI MAGGIORE CHURCH

The San Giovanni Maggiore church was erected in the 555-560 in Naples at a site previously occupied by a pagan sanctuary. It is one of the more ancient sacral sites reported in the *Liber pontificalis* of Naples and in other medieval documents, and from its origin has represented a reference church in the city. In figure 12 the plan and a transversal section of the church are provided. Also this church has been modified during centuries, was subject to several earthquake attacks (1456; 1732, 1805, 1980) which mostly damaged the transept zone.

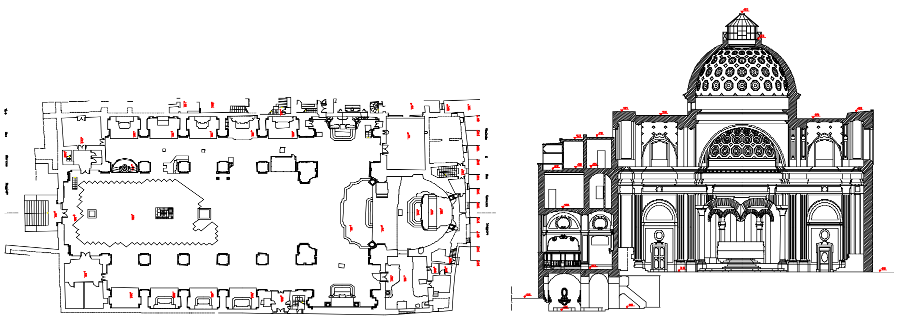
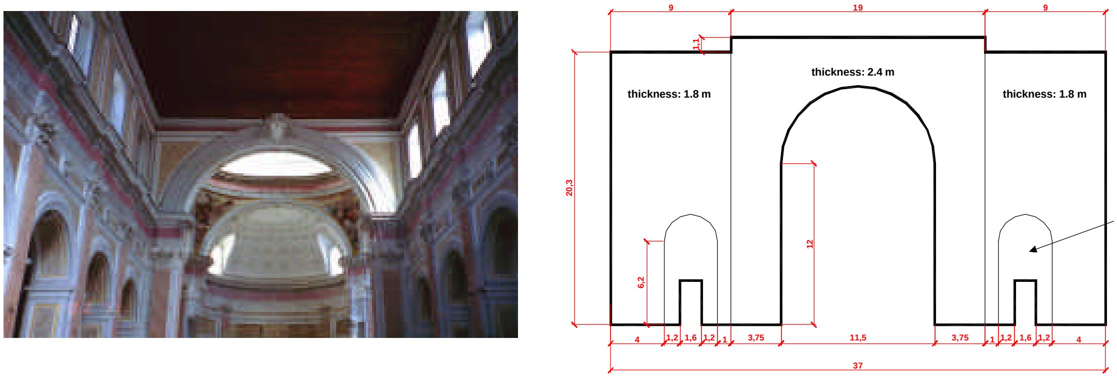


Figure 12 : Plan and transversal section of the san Giovanni Maggiore church.

The church has a typical basilica plan with the nave, two aisles and nine lateral chapels, transept with two big chapels, the tribune, chancel with semicircular apse. The global length of the church is 61 m, while the width is 37 m. The hall, consisting of the nave and aisles; is 37 m long, and has a maximum height of 25,90 m in the nave and 14.30 m in the aisles, at the apex of the respective roof systems. The height of the chancel is 19.80 m; the dome is imposed on a drum 2 m high; the height of the dome spring is 22 m, while the crown height is 32,70 m.

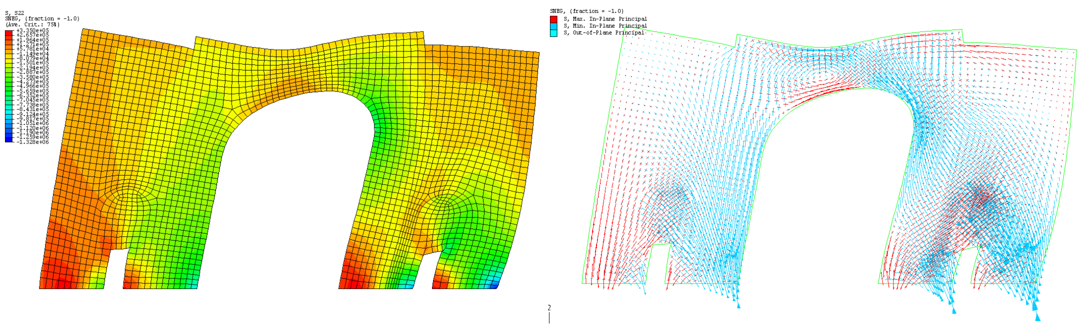
A view and the geometrical layout of the triumphal arch, are respectively provided in figures 13 (a) and (b). As immediately can be observed from figure 13 (b), this case is characterised by two peculiar geometrical aspects: the presence of wide, stocky abutment panels (H/B=0.91) and a relatively small thickness of the arch at crown; and the presence of zones in the element with different transversal thickness (central part 2.4 m, lateral panels 1.8 m, infilled minor arches 0.9 m). The semicircular opening is not very large, equal to 25% of the total surface of the element.



(a) (b)

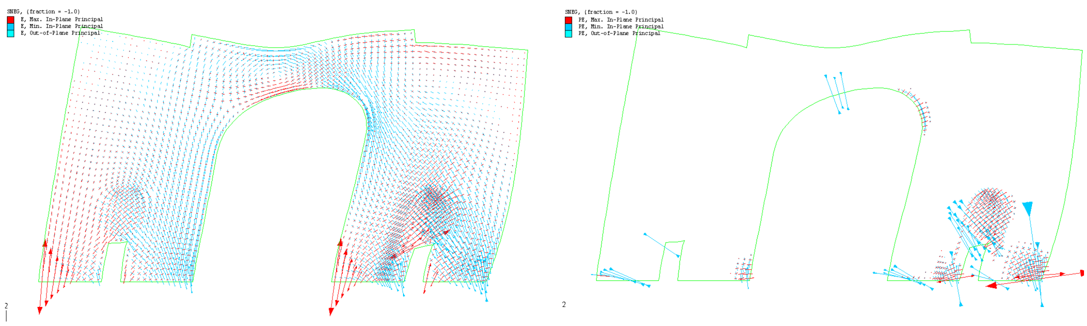
Figure 13 : Triumphal arch of San Giovanni Maggiore church: (a) view, (b) geometry and dimensions.

A 2D mesh, consisting of 2520 shell elements, has been used to model the element, which has been subjected to the vertical load deriving from the self weight (19080 kN) and from the roof loads (2939 kN), and to horizontal loads of increasing intensity, constantly distributed along the height of the element.



(a) (b)

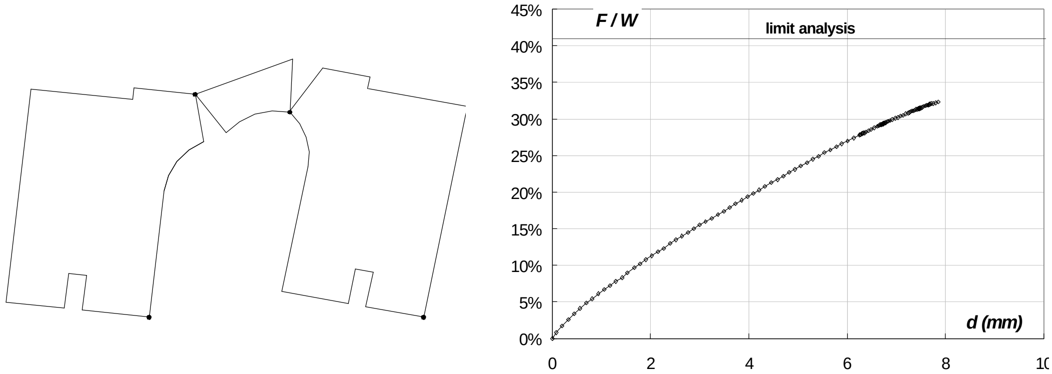
Figure 14 : FEM analysis: (a) vertical stresses, (b) principal stresses.



(a) (b)

Figure 15 : FEM analysis: (a) principal strains, (b) plastic strains.

In figure 14 (a) the deformed configuration at the last load increment of the analysis, with the vertical stress contour superimposed, is shown. From this figure it is evident that the behaviour of the element is strongly affected by the particular geometry and governed by the behaviour of the two lateral abutment walls. In the figures 14 (b), and 15 (a) and (b) the vector representations of principal stresses, principal strains, and the plastic strain components are respectively provided. All these plots allow to derive the deformation and collapse mode of the element, which occur, after the failure of the thin upper arched part, as a consequence of shear failure of the two stocky panels. In the plots is also evidenced the concentration of stress and strain in the zone of the in-filled minor arches, which, having a thickness smaller than the other parts, represent a weak point of the panels. As in the previous case, these plots have been utilised for individuating the class of collapse mechanisms and deriving the collapse multiplier as the minimum among the ones associated to the mechanism type. In figure 16 (a) the collapse mechanism is reported, while in figure 16 (b) the FEM analysis results are compared to the collapse multiplier. The lateral capacity of the element, determined both via FEM and limit analysis, is quite high (32% and 41%, respectively) especially if compared to the values derived for the previous cases. It is worth pointing out that these values are roughly close to the ones that could be obtained from the analysis of the single abutment panel.



(a) (b)

Figure 16 (a) collapse mechanism, (b) comparison FEM analysis vs. limit analysis.

7 CONCLUSIVE REMARKS

This paper demonstrates the importance of analysing real cases since the geometry of triumphal arches are quite complex and it is not immediate to assume a simplified model for structural analysis considerations and for consequent parametric analyses. The three cases considered in this paper are in fact quite different in the geometry, in the behaviour and in the ultimate multiplier of horizontal load. The latter, nondimensionalized to the vertical load, is approximately equal to 0.20 in two cases and to 0.30 in the third case, the larger value is due to the presence of wide abutments, which act as shear walls. A quite general and quantitative indication can be therefore inferred even in these very different cases: the nondimensional collapse multiplier is bound between 0.20 and 0.40. This indication might be very useful in the seismic assessment of churches and also in the design of retrofit solutions. A wider analysis extended to a large sample of real churches could result very useful for the assessment of seismic performance of churches.

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