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Non Linear Modelling of the Elliptical Dome of Vicoforte

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ABSTRACT: The elliptical dome of the Sanctuary of Vicoforte (Cuneo, Italy) is by far the largest elliptical dome the world over. The dome, erected in 1732, has suffered since the beginning of significant structural problems, related in part to differential settlements of the foundations and, to a large extent, to the daring structural configuration of the system itself. A severe and extended network of cracks affects the dome-drum region. This paper presents the results of a non-linear finite element analysis. The analysis is limited to the dome-drum system. A continuum anisotropic non-linear constitutive model is employed. Two load conditions are considered. The first one considers rigid foundations; the internal actions of the structure derives only from the mass forces. The second one considers, besides the mass forces, a set of settlements of the foundations, whose extent is defined on the basis of historical and geotechnical analyses. The results are discussed with regards to the entire set of studies performed on the construction in recent years.

1 INTRODUCTION

The elliptical dome of the Sanctuary of Vicoforte (Cuneo, Italy) is the fifth biggest in the world in terms of overall dimensions (Fig. 1). With its 37.15 m and 24.80 m long axes, it is also by far the largest elliptical dome the world over. Erected in 1732 on top of a slender drum constructed in preceding years, the dome has suffered since the beginning of significant structural problems, arising partly from differential settlements of the foundations due to an unfortunate selection of the site, and, to a larger extent, from the daring structural configuration of the system itself. A severe and extended network of cracks affects the dome-drum region.


Figure 1 : Internal and external view of the dome.

The structural behaviour of such a complex masonry structure is definitely influenced by the masonry pattern. In general, it is well known that masonry domes are frequently built by disposing bed joints along the parallels. If, on the one side, the behaviour along a meridian is similar to that of an arch, on the other hand, complex mechanisms are induced on parallels. In particular, the lower third of the dome profile is usually subjected to circumferential tensile stresses, acting on the plane of mortar head joints. The capability of the dome to carry such stresses depends mainly on the capability of mortar bed joints to carry shear stresses; in fact, the tensile strength of mortar head joints is usually very low (even lower than that of mortar bed joints). The overall shear strength of mortar bed joints depends on cohesion and, particularly, on friction. Cohesion being often very low, especially in ancient structures, a fundamental role is played by friction. The latter is function of (a) the compressive stresses acting on the plane of mortar bed joints, (b) the interlocking of the masonry pattern (i.e. the ratio between the height and the thickness of bricks), and (c) the friction coefficient of the interface. In addition to the typical behaviour of masonry domes, the complex masonry pattern of Vicoforte should be considered. The drum-dome system is characterized by series of masonry relieving arches; moreover, the big oval openings at the base of the dome are constituted by thick masonry rings. The masonry pattern changes at each of these structural elements, varying the stress flow. The dome is provided with cerclage annular iron ties as indicated in the sequel.

The complexity of masonry constructions calls for global and very detailed models able to represent the both the overall spatial configuration and the entire set of architectural elements having a structural relevance. These types of models are particularly important for domes, whose structural behaviour is strictly connected to their global shape, in particular in the rather infrequent cases of non axially symmetric domes. To this aim, finite element modelling has been recognised as a suitable tool, that is able to provide synthetic (albeit non exhaustive) information on the global behaviour of the structure, especially when associated with non linear constitutive laws.

The modelling strategy proposed here aims to describe the orthotropic non linear behaviour of masonry, taking in account the changes in masonry pattern within the structure. The analysis is referred exclusively to the drum-dome system and considers two different load cases: the gravity loads; the gravity loads plus a set of settlements at the base of the drum defined on the basis of historical analyses and detailed geometrical survey.

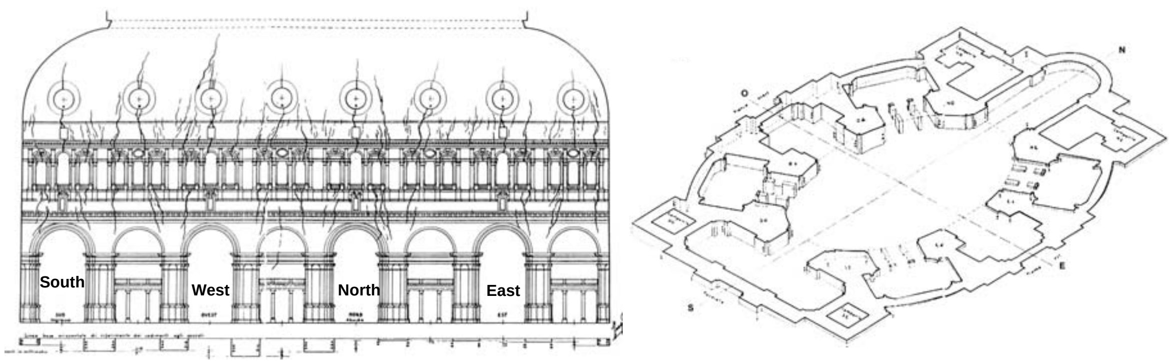

Figure 2 : (a) Crack pattern; (b) Axonometric view of total settlements.

2 HISTORY AND DESCRIPTION OF THE CONSTRUCTION

The Sanctuary of Vicoforte began to be constructed in 1596 based on a project by architect Ascanio Vittozzi. The soil on which the foundation structures are rested consists of a variable silt-clay layer, which, in its turn, is rested on a semilithoid marly substratum declining in the south-west direction. Three of the eight pillars supporting the drum and the dome have foundations resting on the marly substratum, and the other five rest on a layer of the silty-clayey complex up to 3.5m thick. In 1615, when Vittozzi died, the construction had reached the level of the impost of the big arches giving access to the chapels, but work had to be stopped due to the strong settlements. A network of drainage channels having been built, work was resumed and dragged on for several years, reaching about mid height of the drum structure. In the early eighteenth century, construction works were entrusted to architect Francesco Gallo (1672-1750), who demolished the drum and levelled its impost because of excessive deformations. The new drum was completed in 1711. For the construction of the dome, a supporting structure called *Ponte Reale* (*"Royal Bridge"*) was built. The dome was erected in 1731 in less than a year's time by proceeding by overlapping rings. In 1735, when the lantern top was completed, the Sanctuary was inaugurated. Settlements of the foundations continued to occur throughout the entire building process and during the following centuries, despite the repeated actions taken to dry the silty-clayey soil by expanding the network of drainage channels. Fig. 2b represents the total historical settlements ascertained at the base of the monument with maximum values of ca. 330 mm at one of the main western pillars vs. a few mm in the eastern sector. In 1985-87, in order to strengthen the dome, a system of tangential iron ties was embedded 3 m below the impost level.

2.1 Architecture, technology and building systems

From the structural viewpoint, the dome of the Sanctuary of Vicoforte can be likened to the "rotunda model", which is particularly effective to counter the radial thrusts arising from the dome's dead weight and to transfer to the tiburio the loads carried by the outer surface of the dome. By enabling the buttresses to rise above the dome impost level, the tiburio provides a more effective means to counter radial thrusts. The space between the tiburio and the vault is filled, up to a variable level, with light-weight materials (sand and pottery shreds), while at the buttresses locations, full masonry sections are present which ensure structural continuity. The top of the tiburio is connected to the vault by a reverse arch. The vault is equipped with three sets of iron annular ties positioned immediately above and below the oval windows and at the top of the tiburio, totalling 140 cm^2 of cross section. This set-up - consisting of tiburio, filling material, reverse arch and strengthening rings - is able to counter the horizontal deformations of the dome, from the impost level to about half the height of the dome, providing a slender and sufficiently light structural ensemble. The dome, whose thickness varies from 2.20 m at the impost to 1.27 m at the top, is reinforced by eight ribs on the extrados radially located in correspondence of the buttresses. The huge, heavy sandstone lantern (density: 2500 Kg/m^3), while generally enhancing the level of stresses in the dome, contributes to the integrity of the its upper region, in that it induces in the closing ring of the vault a state of tri-axial compression which prevents the meridian cracks from propagating to the top. The drum is very slender and transparent, owing both to its limited thickness (2.20 m) and to the presence of the large triple windows. It is possible to discern an overall structural skeleton consisting of the eight dome ribs, the buttresses and the pillars. The arch ring of the oval opening and the relieving arches above the triple windows of the drum help to transfer the loads from the intermediate zones onto the buttresses and pillars. The 4 buttresses adjacent to the major axis of the oval are hollow (one of them locates the helicoidal stair); as determined from the analyses performed by various models reported in the parallel paper (Chiorino et al. 2006) this architectural feature is justified by the fact that these buttresses are subject to smaller radial thrusts due to the dome's dead weight. The eight pillars supporting the weight of the dome-drum system are strengthened in the radial directions by the walls separating the chapels.

2.2 The crack pattern

The Sanctuary of Vicoforte is affected by a widespread system of cracks (Fig. 2a encompassing various zones of the structure. From the historical records of the Sanctuary we find that cracking phenomena began to occur in the early stages of its construction (always due to settlements of the foundations). Particularly significant is the cracking pattern in the dome-drum system. Two zones, in particular, cause concern: the zone between the drum windows and the oval openings in the dome and the zone at the base of the buttresses. The former displays a finely spaced network of cracks running in the subvertical direction, mostly concentrated in the masonry portion just above the impost. The cracks become thicker and bigger in the proximity of the openings. In the dome impost zone we find 84 cracks adding up to a total width of 416 mm. The two main cracks, situated on the north and the west sides, start from the triple windows of the drum and propagate beyond the oval openings of the vault. The crack to the west is the biggest with a maximum width of 48 mm and it almost reaches the closing ring of the dome. The sums of the openings of the meridional cracks in the north and west sectors reach 82 and 78 mm, respectively. The other zone of interest, at the base of the buttresses (especially the hollow ones), displays diagonal cracks resulting from shear stresses induced by the radial thrusting action of the dome (Chiorino et al. 2006).

3 MODELLING THE STRUCTURE

The constitutive law here employed, as briefly described in paragraph 3.2, is a plane stress model involving different in-plane mechanisms. Since 4-node non-linear shell elements were used, the description of the structure required a complex work of geometrical synthesis (Fig. 3).

3.1 Geometrical features

The geometric model was produced based on the results of a recent topographic survey performed over the entire complex with the exception of the dome that was subjected to an earlier photogrammetric survey of the intrados surface, while the thickness of the vault was determined with endoscopic measurements.

3.1.1 Geometrical description of the structure: from 3D reality to a shell-based model

Translating 3D geometry into shell-based surfaces necessarily entailed some simplifications. The vault was modelled by defining a mean surface, based on the data obtained from the geometric survey. Each buttress was modelled with three shells, of appropriate thickness to simulate the moments of inertia and the area of the actual structure with sufficient approximation. The original strengthening rings were modelled by means of beam elements, connected to the shell nodes so as to ensure that the lengths of the ties would be approximately the same as in the original elements. The effects of the filling material between the tiburio and the vault were not taken into account, as it proved impossible to determine its characteristics in all the areas where it was present. The lantern, not included in the model, was analysed separately to determine the loads transferred to the dome.

3.1.2 The masonry pattern

The inelastic orthotropic model requires that the reconstruction of the masonry be carried out by paying special attention to vaulted or arch/architrave portions. From the historical records and an examination of the extrados, we find that the dome was constructed with the main mortar joint beds arranged along the parallels and slanted according to the curvature of the dome. This pattern stops around the oval windows in the proximity of which the main beds are arranged radially. It has also been ascertained that under the outer surface of the drum there are relieving arches with their imposts on the buttresses, positioned at an intermediate height between the triple windows and the dome impost. The architraves around the different drum windows and on the buttresses were modelled in keeping with common building practice. The rest of the masonry structure of the drum and buttresses, in the absence of specific findings, was modelled with the mortar beds arranged horizontally.

3.1.3 Actions and boundary conditions

Actions and boundary conditions assumed in the analyses contained in paragraphs 4.1 and 4.2 are, respectively, the dead weight and the deformations at the drum base due to the settlements of the foundations and the deformations of the underlying structure. The deformations at the drum base were introduced as imposed vertical displacements; their values were obtained from a reasonable estimate of the amount of differential settlements occurred after the completion of the dome with reference to a geometric levelling of the base cornice (Garro, 1962). This choice is justified by the fact that, from the historical records (Vacchetta, 1984), it appears that before building the drum and the dome the base impost was reconstructed and levelled. Horizontally, restraints were placed at the base to prevent translation movements in either direction at the four buttresses adjacent to the major axis. This was inferred from a FEM analysis performed on a 3D

model encompassing the entire structure of the Sanctuary: in those zones, the model revealed negligible horizontal displacements (Chiorino et al. 2006).
Two different analyses were performed: in the first one, only gravity loads were considered; in the second, the additional effects of the foundation differential settlements were studied.

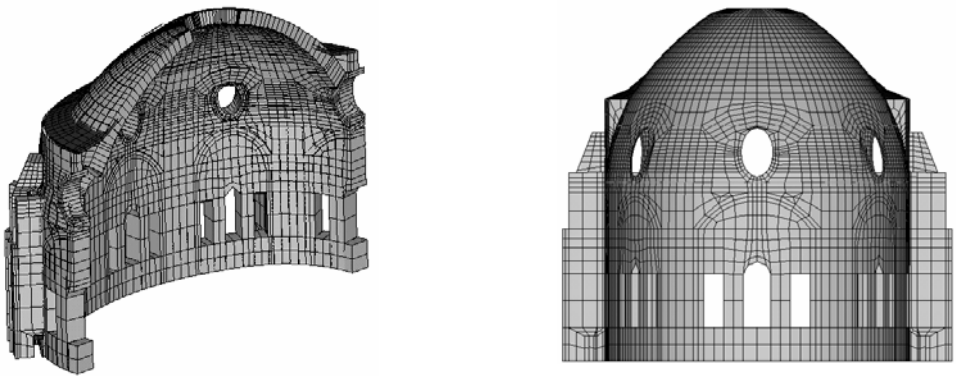

Figure 3 : FEM model of the dome-drum system.

3.2 Constitutive law

The model employed is based on a micromechanical approach (Calderini and Lagomarsino 2005, Calderini and Lagomarsino 2006). The plane stress hypothesis is assumed. Masonry is considered as a composite material, made up of periodic sets of blocks connected by mortar joints. Mortar bed joints are assumed to be mechanically resistant (tensile strength and cohesion); moreover, friction at the mortar-block interface is considered. Mortar head joints are considered as geometrical discontinuities; their mechanical behaviour is neglected. The masonry pattern is described by means of a single summary parameter, the angle of interlocking, that is the ratio between block width and block height.

Constitutive equations consider the non linear stress-strain relation in term of mean stresses and mean strains. The latter are produced by an elastic strain contribution, associated with a homogenized elastic continuum, and by an inelastic strain contribution depending on damage. Different in-plane damage mechanisms are considered, referred both to mortar joints and blocks. Considering the full set of possible damage mechanisms of joints, an emisymmetric condition on the inelastic strains of the mortar bed joints is imposed. Damage evolution is described by mean of an energetic approach (Rough-Curve approach). Moreover, the hysteretic behaviour of the masonry under cyclic loads is described by considering a Coulomb-type friction law on the mortar bed joints. In particular, two damage variables are considered for emisymmetric sets of mortar bed joints. This choice enables the model to describe the inelastic strains in the mortar head joints in terms of difference between the tangential inelastic strains of the two sets of mortar bed joints. A specific condition is imposed in order to avoid the interpenetration of head joints.

The constitutive equations are implemented in a general-purpose finite element code (ANSYS). The procedure for the integration of the constitutive equations in the finite load step has been developed based on a description involving five internal variables which evolve during the incremental analysis. The Newton-Raphson method is adopted for the solution. In particular, the model has been implemented in 4-node non-linear shell elements with transverse shear strain capability. Such elements have a number of integration points through the thickness. This feature makes it possible to describe the behaviour of the walls by mean of series of layers subjected to in-plane mechanisms; thus, a description of their out-of-plane behaviour can be obtained. The choice of 4-node elements, instead of the better performing 8-node elements, is related to the need of limiting the degree of freedoms of the model.

The model requires, besides the homogenized elastic moduli (E_x = 2.0 10^3 MPa, E_y = 1.6 10^3 MPa, G_{xy} = 8.0 10^2 MPa, ν = 0.2), the following inelastic mechanical parameters: friction coefficient ($\mu = 0.6$); tensile strength (σ_{mr} = 0.05 MPa) and cohesion (τ_{mr} = 0.1 MPa) of mortar joints; compressive strength of the masonry (σ_{Mr} = 5 MPa); tensile (σ_{br} = 0.5 MPa) and shear (τ_{br} = 2 MPa) strength of blocks; ratio between the elastic and inelastic shear strain at failure in mortar joints ($G_{xy}/\tilde{G}_{xy} = 2$); ratio between elastic and inelastic strain at failure of masonry in compression ($E_y/\tilde{E}_y = 1.5$); softening coefficient of mortar joints (β_m = 0.2) and blocks (β_b = 0.2); interlocking ratio (φ = 4). So far, limited experimental tests have been carried out on the materials of the structure. Except for the compressive strength of the masonry and the Young's modulus E_y (Barosso 1979), all the parameters have been qualitatively defined by considering typical values of historical masonry.

4 NUMERICAL ANALYSES

The use of an anisotropic damage model requires adequate tools to express the results, in particular in the case of a 3D structure. In fact, the use of different local coordinate systems for the elements (used for the description of the different masonry pattern within the structure) leads to a difficult interpretation of the local inelastic strains. In the following paragraphs, the results will be expressed in terms of: maximum principal inelastic strains, in order to consider only positive strains (crack openings) independently of the local reference system; strains in the local reference systems, along the two directions parallel and orthogonal to the mortar bed joints.

For the interpretation of the results, the hypothesis of smeared crack should be taken in account. In fact, in the model, the inelastic strains are diffused on a certain area and there is no direct evidence of cracks. The crack pattern can be derived only from the observation of the strain pattern.

4.1 Analysis of the dome-drum system under gravity loads

The analysis of the structure under gravity loads pointed out some intrinsic weaknesses of the structure. Fig. 4 shows the maximum principal inelastic strains of the structure. It can be noticed that:

- the damage pattern is quite symmetric;
- the highest, and broadly diffused, principal inelastic strains are at the oval openings and the relieving arches;
- inelastic strains are present above and below the openings at the base of the drum.

The meaning of such a damage state can be understood by comparing the principal strains with the local strains. As an example, let us consider the west side of the dome. Fig. 5 shows the local inelastic strains, in the three plane components of the shells. Only strains in those elements in which failure has been reached (in at least one integration point) are represented.

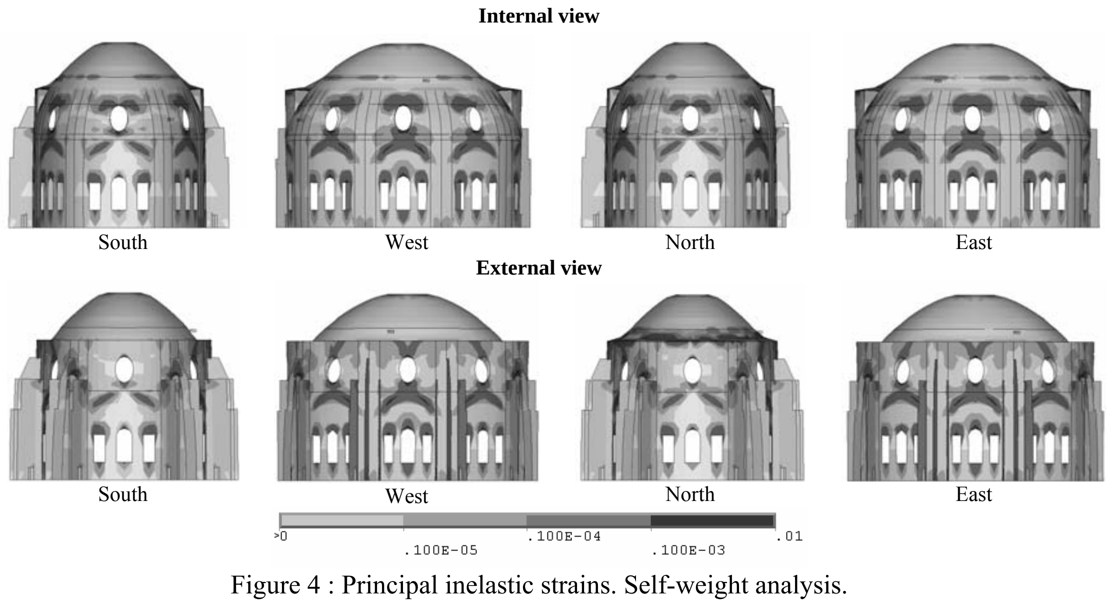

Figure 4 : Principal inelastic strains. Self-weight analysis.

The strains parallel to the bed joints are localized:

– At the base of the dome, below the arch ring of the oval openings. The effect of the friction is here well evident. Below the oval openings, in fact, the compressive stresses on the bed joints plane are very low; thus, the friction contribution is limited. The circumferential tensile stresses at the base of the dome, acting orthogonally to the head joints, can be balanced only by the cohesion of bed joints.
– Above the arch ring of the oval openings. In this case, the inelastic strains are associated with the absence of compression on the bed joints at the arch crowns, mainly due to the presence of the upper relieving arches and, locally, to the activation of a hinge (recognizable in the strains orthogonal to the mortar bed joints); the vertical tensile stresses arising from the weight of the upper wall produce the strains.
– At the springings of the relieving arches. The inelastic strains depend on the interaction of two factors: the absence of compression on the bed joints, due to their orientation (approximately 45° with respect to the vertical); the shear stresses acting on the arches as a consequence of the change in the pattern orientation within the wall (this is confirmed by the presence of inelastic shear strains in the same area).
– Below the openings at the base of the drum. In this case too, parallel strains are associated mainly with the absence of compression on the mortar joints below the openings and with the presence of circumferential tensile stresses, acting orthogonally to the head joints.
– Above the central openings (of each span) at the base of the drum. The presence of the upper relieving arches cause the forces to migrate along the buttresses; thus, below these arches the compressive stresses are very low. Also in this case the absence of compression, together with the presence of vertical tensile stresses produced by the weight of the upper wall, is the cause of damage. This is confirmed by the absence of parallel inelastic strains in the lateral openings of each span; being close to the buttresses, they carry higher compressive stresses.

On the basis of the foregoing observations, it can be observed that the inelastic strains parallel to mortar bed joints are strongly connected with friction. Cohesion being very low, their occurrence depends mainly on the ratio between the circumferential tensile stresses and the rate of compression on the bed joints.

The strains orthogonal to the bed joints are localized:

– Above and below the arch rings of the oval openings. Owing to the presence of the rings, the mortar bed joints above and below the openings are nearly orthogonal to the circumferential tensile stresses at the base of the dome; thus, those joints are subjected both to the tensile stresses associated with the in-plane behaviour of the arch ring, and to the circumferential tensile stresses.
– At the crowns of the two sets of relieving arches. In this case too, owing to the presence of the relieving arches within the walls the mortar bed joints at crowns are nearly orthogonal to the circumferential tensile stresses at the base of the dome. As in the previous case, the joints are subjected both to the typical tensile stresses arising from the behaviour of the arch in its plane and to the circumferential tensile stresses of the dome. The difference between the internal and the external side, more evident above the oval rings, is justified by the fact that the highest relieving arches are present only in the dome, and not in the tiburio. The difference between the central oval opening and the lateral ones, visible also in the real crack pattern, derives from the elliptical shape of the dome.
– Above the openings at the base of the drum. In this case too, mortar bed joints are subjected both to the circumferential tensile stresses of the dome and to the in-plane tensile stresses of the arch (or of the lintels).

The inelastic shear strains substantially confirm the previous observations, in particular the ones related to the relieving arches. Because of the change in orientation of the mortar bed joints the vertical compressive stresses transmitted by the upper parts of the structure become local shear stresses on the arches.

In general, it can be stated that the inelastic strain pattern obtained through the non-linear analysis is compatible with the crack pattern of the real structure. In particular, a good correspondence can be found between the cracks above and below the openings. Moreover, the model points out the increase in damage at the minor axis of the dome. The more critical point concerns the relieving arches; in fact, in the real structure there is no conclusive evidence of damage at their backs.

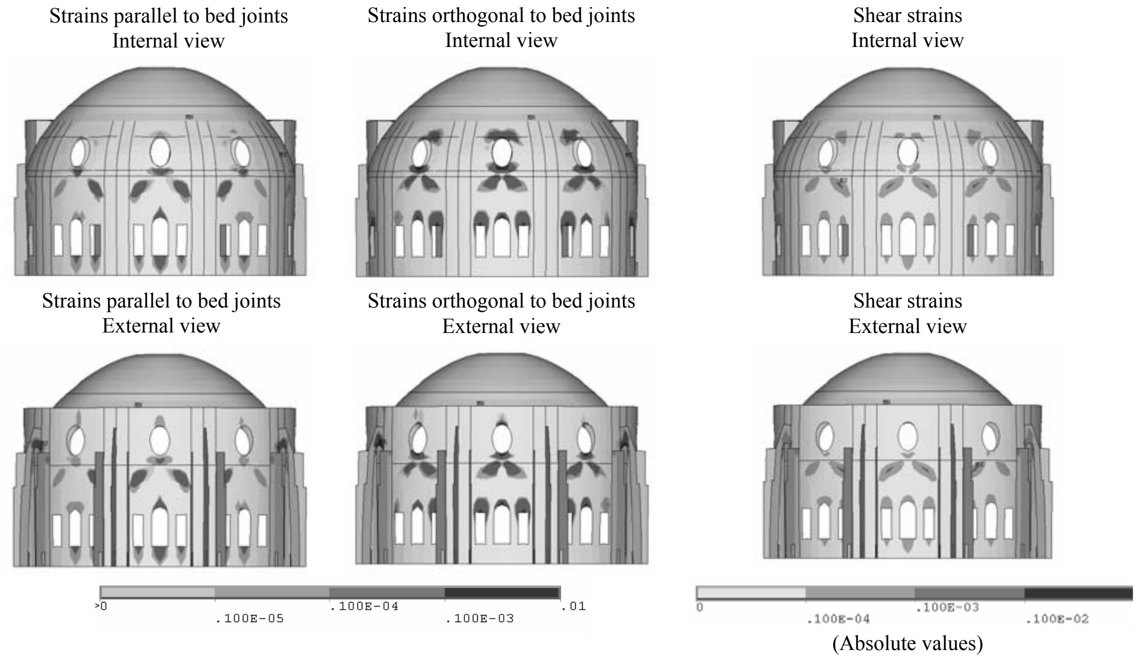

Figure 5: Inelastic strains in the local coordinate system of the elements (West side). Self-weight analysis.

4.2 Analysis of the drum-dome system considering the historical settlements

The settlements have been applied in terms of differential vertical displacements at the base of the drum. As previously stated, the displacements were defined from a reasonable estimate of the amount of differential settlements occurred after the completion of the dome with reference to a geometric levelling of the base cornice of the drum.

Fig. 6 shows the principal inelastic strains obtained. An increase in the damage state is well evident in the west side of the construction, where the highest differential displacements were applied. On the other sides the increase in damage is very limited.

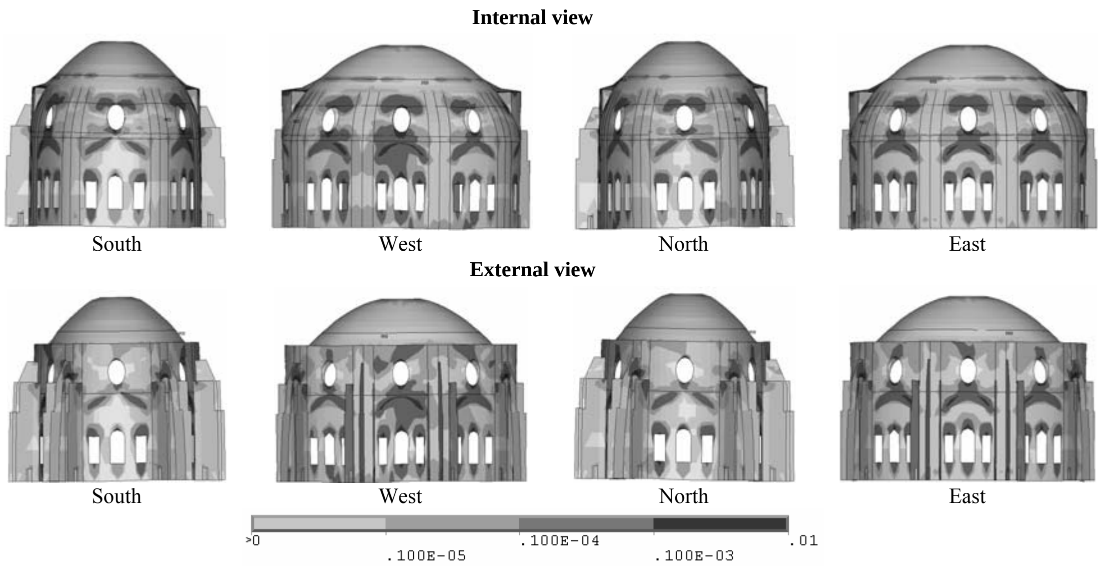

Figure 6: Principal inelastic strains. Settlement analysis.

The observation of the inelastic strains in the local reference system of the elements leads to the following considerations (Fig. 7, referred to the west side of the construction):

- a vertical crack is evident in the middle of the central span (see the strains parallel to the bed joints); it passes through all the openings (see in particular the external view) and slopes to the south in the highest part of the tiburio. The occurrence of parallel inelastic strains is due to the interaction between the circumferential tensile stresses of the dome and the shear stresses produced by the differential settlements;
- the inelastic strains orthogonal to the bed joints increase at the openings and the relieving arches;
- no evident increases occurred in inelastic shear strains.

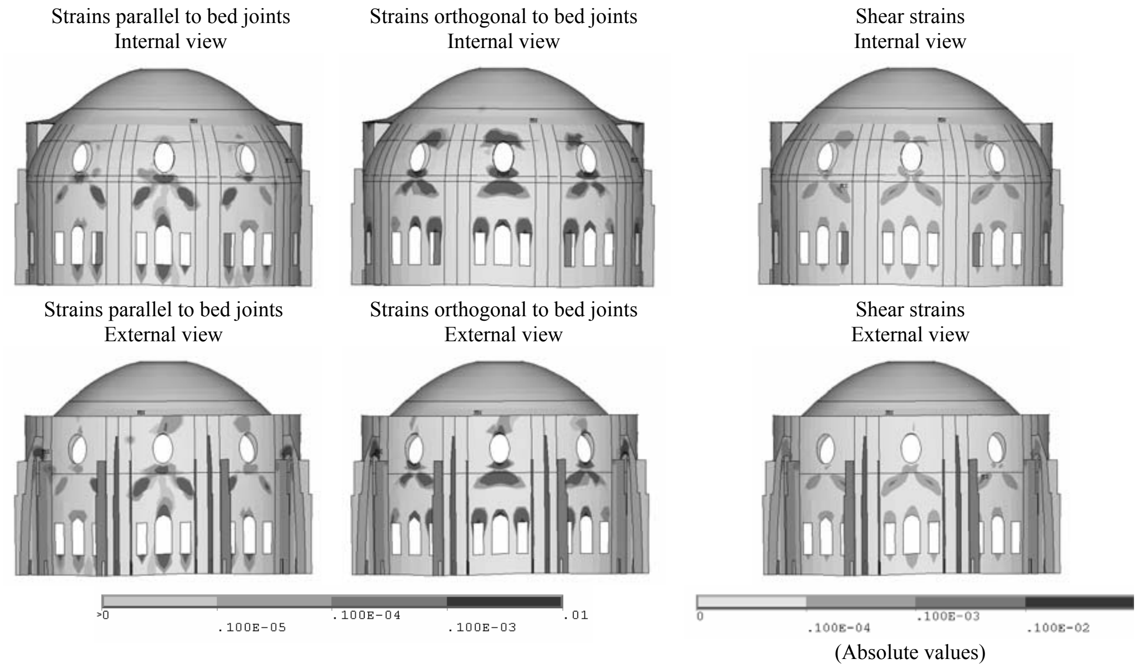

Figure 7: Inelastic strains in the local coordinate system of the elements (west side). Settlement analysis.

5 FINAL REMARKS

Albeit not exhaustive and despite some interpretation limits, the non-linear modelling of the dome-drum complex of the Sanctuary of Vicoforte made it possible to identify some significant aspects of the behaviour of the structure. The contribution offered by this model to the interpretation of structural behaviour lies in the possibility of adopting non linear anisotropic constitutive laws, mated to a commitment to modelling the internal conditions of the masonry structure by varying the local reference systems of the elements. The most significant aspect that has emerged is the role of the relieving arches present in the surfaces of the vault and the drum. These relieving arches, built in order to convey the loads towards the buttresses, and hence to relieve the surfaces of the drum perforated by the voids of the triple windows, turn out to be, to some extent, elements that tend to weaken the structure. In fact, on one hand they reduce the compressive loads acting on the main mortar joints, thereby attenuating the favourable effects of friction; on the other hand, the arrangement of the crown joints, orthogonal to the circumferential tensile stresses, creates a zone highly susceptible to damage precisely along the – already weak – axis of the openings above and below. Similar remarks can be made regarding the rings and the architraves defining the openings themselves; however, in this case, the architect could make no other choice, since the creation of an opening in a masonry structure always calls for a relieving system (whether an arch or an architrave).

The correlation with the crack distribution observed in the real structure is in general remarkably good. In fact, while the main meridional cracks follow the axis of the oval windows, they are flanked by a dense system of thinner cracks in the zones below the oval windows and in correspondence of the relieving arches. Abetter knowledge of the inner makeup of the masonry of the dome and the drum could improve this correlation. In fact, the composition of the masonry considered in the model, is presently based on what can be seen from the exterior of the masonry, but in the interior of the 2 metre thick structures the makeup of the masonry may very well vary.

It is important to underscore that the structure is subject to damaging even due to the effects of its dead weight alone, disregarding the effects of differential settlements. The presence of differential settlements seems to make the situation much worse, causing asymmetric damage which is particularly severe in the central span on the west side, where it is possible to recognise a continuous central zone where the material has reached failure. In general, from the qualitative standpoint, this last analysis simulates fairly well the current cracking configuration of the construction.

The analyses completed so far supply a partial contribution to an understanding of the structure. A more exhaustive interpretation of the structural behaviour of the construction may arise solely from the interaction of different modelling strategies (3D linear analysis, limit analysis, non-linear analysis using different constitutive laws), each of which can supply significant elements (Chiorino et al. 2006).

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