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A structural analysis in seismic archaeology: the walls of Noto and the 1693 earthquake

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Abstract

A crucial problem for seismic archeology is how to recognize seismic effects and how to date them. On an experimental basis, we proposed that the problem be reversed, and that we begin at the other end: *i.e.* by analyzing already known seismic effects on ancient structures, testified by written sources, to be able to «calibrate» the types of possible observations and any subsequent elaborations. The choice of the walls of Noto was suggested by the fact that Noto was abandoned following the earthquake of 1693 (I_0 = XI MCS, *Me* 7.5) which had already been studied in depth as part of an ING research programme (1988-92). Moreover, just after recent research, this event proved to be reconstructed with a high quality standard. Photogrammetric measurements were made on several parts of the town walls to plot a numerical model aimed at ascertaining specific aspects of the earthquake damage. An estimate of the ground acceleration during the earthquake has been attempted via non-linear finite-element analyses of a building located by the main city gate. The analyses show that, in order to obtain the building vault collapse, a ground acceleration of 0.5 to 0.7 g had to be reached during the earthquake. This result, typical of a strong earthquake such as the one of 1693, proves that an approach based on finite element analysis and a sound engineering judgment may be systematically applied to historical earthquake sites to obtain some estimates of ground acceleration in historical earthquakes. On the whole, this work aimed at starting up the second development phase of the great event of 1693 of which the macroseismic effects are known. In the meantime, some possibilities of tackling structural analyses in seismic archaeology are being explored.

Key words *historical seismology – seismic archaeology – Noto (Sicily)*

1. Introduction

An attempt to apply a methodology of identifying seismic effects in strictly archeological contexts or on still-standing ancient architectural structures has been underway for over a decade in the Mediterranean area. The importance of these applications is self-evident: the identification of hitherto unidentified seismic effects could extend the chronological window concerning the seismic activity of a particular region or help to add further detail to the picture we already have of it. The difficulties, however, are considerable, and of a methodological nature (Karcz and Kafri, 1978a,b; Stiros, 1988; Ward-Perkins, 1989; Guidoboni, 1991). Hitherto, in our view, research on the matter has been limited to pointing out subjective elements of what is actually observed on the ground. This greatly limits the value of the contribution that archeologists and seismologists may make, if we exclude some exceptional cases in which the evidence of collapses of seismic type is confirmed by such characteristic features as the presence of skeletal remains *in situ*, as at Thebes (Stiros and Dakoronia, 1989) or at Kourion (Cyprus) (Soren and Leonard, 1989), or signs of post-seismic restoration as at Pompeii, sealed, as is well known, by the ash and detritus deposited by the volcanic explosion 17 years after the earthquake in 62 A.D. (Adam, 1989). In numerous other cases, however, uncertainty in identifying seismic collapses hampers the utilization of the available data. Nonetheless, even supposing that this problem can be overcome, would it be advantageous to introduce quantitative parameters in seismic archeology? This is a question we posed, because the introduction of quantitative elements in analysis requires a multidisciplinary approach which is not always easy to achieve, nor do we yet know how «strategic» it might be.

We wished to test the hypothesis in an optimal situation: for experimental purposes, we decided to reverse the problem and begin at the other end: *i.e.* by analyzing already known seismic effects on ancient structures, testified by written sources, in order to be able to «calibrate» the types of observation and any subsequent elaborations. We adopted this approach to ascertain whether the cost/benefit ratio in seismic archeology may become positive through the production of quantitative data utilizable in scientific analysis to improve knowledge on historical earthquakes. Our analysis is also aimed at making its indirect contribution to a more comprehensive aspect of the relationship between qualitative and quantitative data in historical macroseismics analyses. These two aspects often have appeared as not «intermingling» valuation modes – or feasible only by means of particular conventions (macroseismic scales).

In our opinion, qualitative data and their wealth of information are receiving higher consideration today, as well as the capability of linking these data to numeric evaluations. We should remember that a historical investigation (even of high territorial detail) on seismic effects is only the basis of data that can be used to locate the epicentre: this is the case of the great earthquake of 1693, whose seismogenetic structure is not yet known. Historical, structural, and geological analyses should contribute to this determination (each of them according to its specialistic point of view, but able to tend to the same purpose). This work is intended to be the second examination phase of knowledge of the 1693 event. Structural analyses intend to offer some elements to explain, in a more general sense, a new approach within seismic archaeology.

2. The walls of Noto and the earthquake of 1693

The choice of the walls of Noto was suggested to us by the particular features of the case: the site was abandoned following the earthquake on 9th and 11th January 1693, which has already been studied in depth as part of a research project conducted by the Istituto Nazionale di Geofisica (Boschi *et al.*, 1995a,b). The earthquake in question was one of the most destructive in Italian seismic history. Detailed historical research has revealed that the area affected by it was very extensive (fig. 1). This was by any standards a large-scale seismic event: the area of strongest seismic effects (greater or equal to VIII degree MCS) is approximately 14000 km^2. Due to the vicinity of the sea the observable seismic effects did not extend beyond the grade VII isoseism. The interpreted scenario of the effects involves nearly the whole of Sicily. On the basis of the areas which felt the various intensity levels, a macroseismic equivalent to the magnitude, whose value is 7.5 (see recent catalogue Boschi *et al.*, 1995b). The high detail guaranteed by research has also allowed a better outline of the epicentral area (XI MCS). The seismic effects in Noto are uncertainly assigned between X and XI degrees of the MCS scale.

The walls of Noto represent a structure of extreme archeological and architectonic interest (fig. 2) and are as a whole fairly well pre-

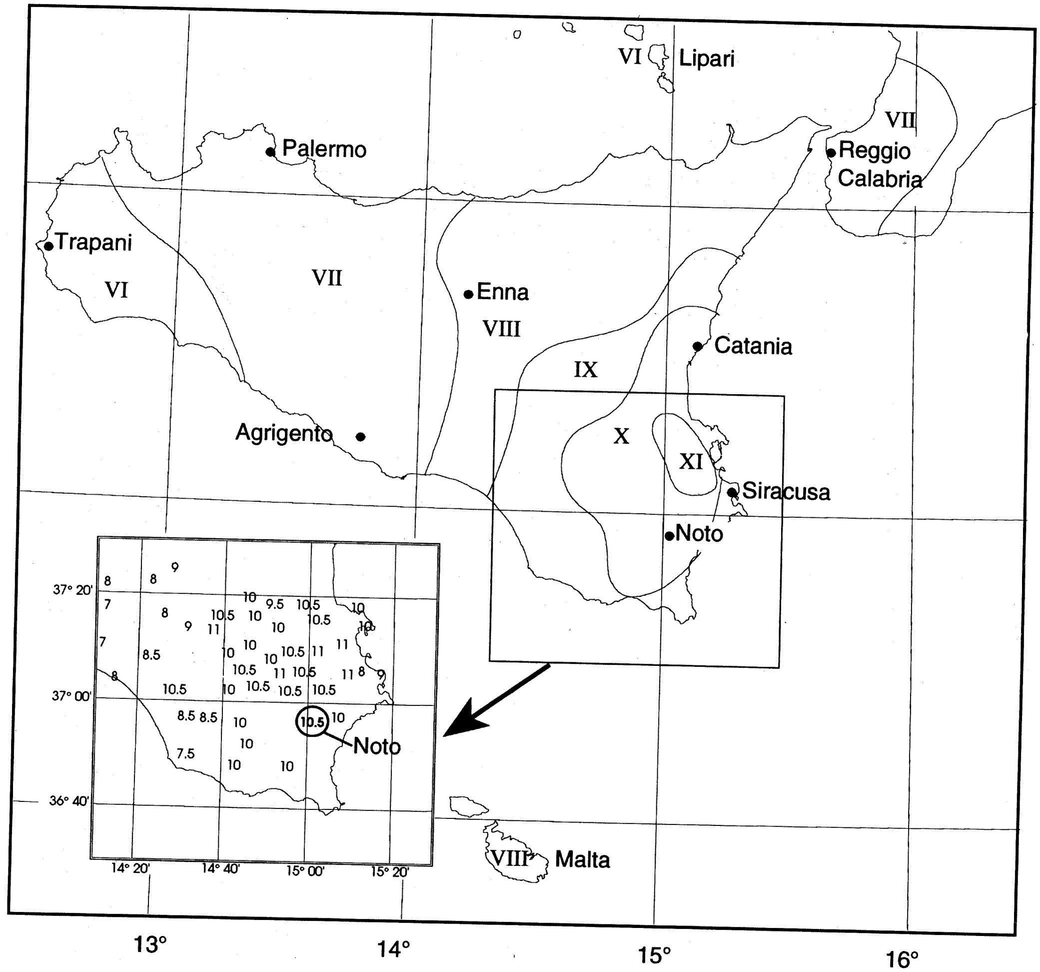

Fig. 1. Sicily: simplified picture of the effects of the earthquake of January 9th, 1693 in MCS scale: $I_0 = XI$ MCS, *Me* 7.5; estimated sites: No. 181. The site of Noto lies in an area of seriously destructive effects (X grade MCS). Source: Boschi *et al.* (1995b).

served. A partial restoration, which is very easy to identify, was carried out a decade ago on parts threatened by falling masonry. The walls had at least four phases in their construction, attested by the written sources: in the 11th century there was a period of reconstruction when the former Arab fortress became Norman; a second phase is attested in 1431; a third in 1542 (the date also of a strong earthquake in Eastern Sicily, see Boschi *et al.* 1995a); and the fourth and last in 1671, in the period of the Franco-Spanish war.

3. Photogrammetric measurements

Photogrammetric measurements were carried out on various parts of the walls to plot a numerical model aimed at ascertaining specific aspects of the earthquake damage. The method

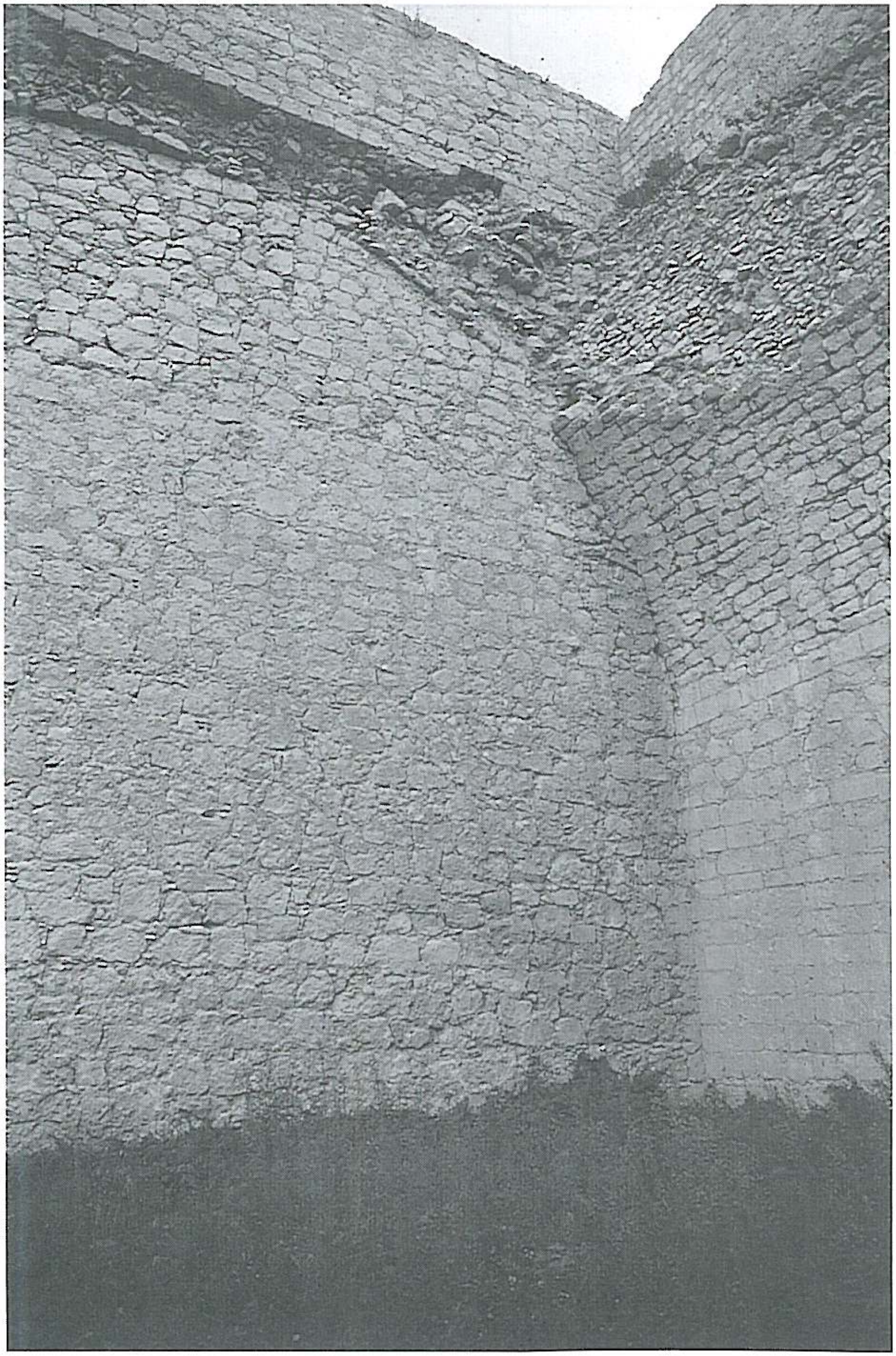

Fig. 2. Old town of Noto (Sicily) abandoned after the 1693 earthquake. The analyzed building is the small barrack in the south-eastern corner of the walls.

was chosen because it seemed, on the basis both of experience and knowledge, to be the one that provided the best feasibilities of revealing structural deformations.

The photogrammetric survey of the subject was carried out by means of a Rolleiflex 6000 metric camera with the negative format of 60×60 mm and a 40 mm nominal focus Zeiss Planar lens. The photos were taken at a distance of approximately 10 m, adopting a photo base of 4 mm. Prior to taking the photos, 4 self-adhesive targets were attached to the subject; their positioning was determined by intersections, and served for the absolute orientation of the model in the plotting phase. To determine the coordinates of the points of support, we proceeded to draw intersections forwards from a measured base, placed in front of the subject and having a width of approximately 4.37 m. The instrumentation used consisted of a millimetre steel tape and a Wild T2 theodolite.

The intersection angles were measured by performing three reiterations on all four points of orientation. During the process of relative orientation, the mean quadratic differential of the residual parallaxes in y, measured on 11 points of the model, never exceeded the value of 0.0009 mm. This degree of precision, which found no specific application in this phase of the research, supplied a mathematical model on which further calculations are planned. The absolute orientation was realized by feeding the calculated and compensated coordinates of the orientation points into the computer linked with the plotting instrument by means of the Starnet programme. The plotting was carried out using a Digicart 40 analytical plotting instrument, in service at the Institute of Topography of the University of Bologna.

4. Quantitative assessment of ground acceleration

A quantitative assessment of the ground acceleration during the earthquake was attempted by means of a finite element analysis of a building located by the city gate (fig. 3). This building has a trapezoidal shape of $24.2 \times 18.7 \times 20.3 \times 24.0$ m sides (fig. 4) and a height of 15.3 m, with massive outer walls and a vaulted roof. This structure was selected for its structural simplicity and because its collapse mode, easily identifiable with the vault collapse, could reasonably be attributed to the earthquake event.

It is important to note that the choice of the structure to analyze is a crucial part of the investigation and requires a sound structural en-

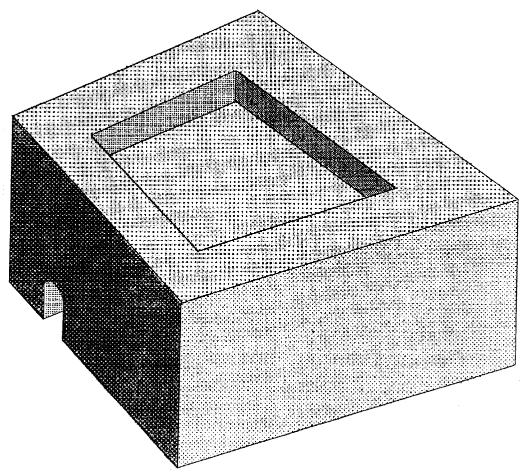

Fig. 3. Assonometric view of the analyzed building.

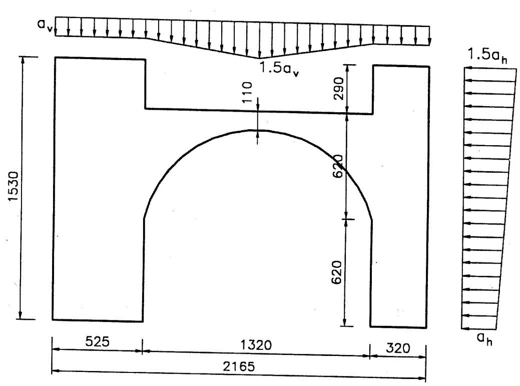

Fig. 4. Middle section of the analyzed building.

gineering judgment to select a significant structure. In fact, meaningful quantitative assessments on the earthquake intensity may be obtained only if the structure to be analyzed has the following characteristics: a) the structural layout is known with reasonable accuracy; b) the structural collapse may be attributed to the earthquake event, and not to later degradation (due to vegetation growth, spoliation etc.); c) the structure is sufficiently strong to withstand a moderate earthquake.

Although a three dimensional dynamic finite element analysis of the whole building might appear desirable to obtain some quantitative estimate of the earthquake ground acceleration, this approach was discarded because it was far too complex and beset by uncertainty, both related to the model geometry, the material characteristics, and the dynamic excitation to impose on the model. Hence, a simpler nonlinear equivalent static finite element analysis was performed on a plane model of the middle building section (fig. 4).

The analyses were conducted under the following assumptions:

1) Plane strain behaviour of the structure. This hypothesis is justified by the massive thickness of the end walls of the building, which prevent out-of-plane deformations taking place.

2) In order to perform a static equivalent analysis, a linear variation of the horizontal acceleration acting on the structure was assumed, with an acceleration amplification at the top of the building equal to 1.5 the ground acceleration (fig. 4). The assumption of such a limited acceleration amplification is justified considering that, due to the extremely high stiffness of the building, a higher amplification is not likely to be present during ground shaking.

3) A linear vertical acceleration amplification is assumed for the vault, while no amplification is assumed for the outer walls.

4) The material of which the structure is made (Limestone blocks, filled walls and mortar) is assumed to be homogeneous with an elastic Young's modulus E = 8000 MPa, a Poisson's coefficient $\nu = 0.2$, and a weight density γ = 20 kN/m^3. These values, which are typical of masonry structures, were assumed arbitrarily and should therefore be validated by appropriate experimental tests. However, in the context of the present study, their variation may be neglected, as the objective of our investigation is limited to determining a limit ground acceleration through an equivalent static nonlinear analysis.

5) The non-linear material behaviour may be represented by a perfectly plastic Drucker-Prager Yield criterion (fig. 5). This criterion is obtained by combining Mises yield criterion and Coulomb friction, hence allowing materials with non-symmetric tensile and compressive yield strength, like masonry, to be represented. The possibility of adopting a no-tension material behaviour was at first considered. This choice was eventually discarded because, however small, the tensile strength of masonry structure is larger than zero, and a zero value may lead to extremely conservative and unrealistic results.

Furthermore, numerical integration of no-tension problems may lead to numerical instability and a lack of convergence in the finite element solution.

The finite element model adopted for the analyses is shown in fig. 6. It consists of 270 eight node isoparametric reduced integration elements and 919 nodes for a total of 1838 de-

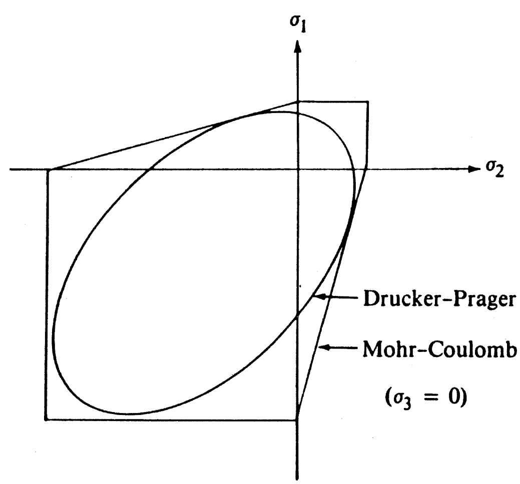

Fig. 5. Drucker-Prager and Mohr-Coulomb yield criteria in σ_1-σ_2 plane (Lubliner, 1990).

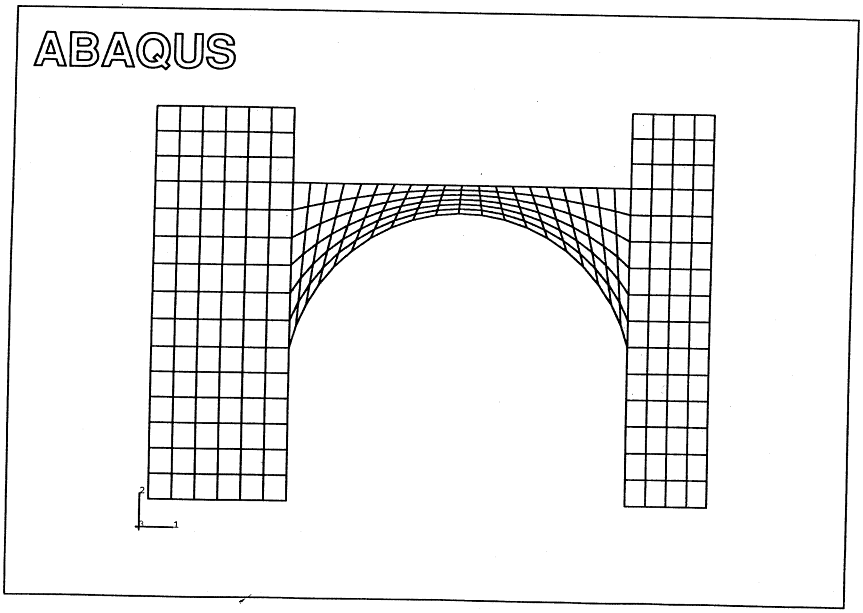

Fig. 6. Finite element model of the middle building section.

grees of freedom. The non linear analyses were performed with the non-linear finite element program ABAQUS (HKS, 1993).

4.1. *Finite element analyses results*

The finite element model shown in figs. 4 and 6 was analyzed considering different material yield limits and loading combinations as summarized in table I, which also shows the ground acceleration for which the structural collapse was reached in the non-linear finite element analyses.

Two different values of the material monoaxial tensile strength σ_t and of the friction angle β were chosen for the analysis (table I). The value $\sigma_t = 0.5\text{MP}a$ may be regarded as a realistic estimate of the average value of the masonry tensile strength, while the value $\sigma_t = 0.2\text{MP}a$ may be seen as a lower bound to the tensile strength. The friction angle value $\beta = 65$ was chosen so that the compressive strength σ_c of the material was sufficiently

Table I. Finite element analyses summary.

No.	σ_t (MPa)	β	σ_c (MPa)	Loading	a
1	0.50	65	3.00	$+\gamma a_h/g$	0.697 g
2	0.50	65	3.00	$-\gamma a_h/g$	0.636 g
3	0.50	50	1.16	$+\gamma a_h/g$	0.579 g
4	0.50	50	1.16	$-\gamma a_h/g$	0.503 g
5	0.20	65	1.20	$+\gamma a_h/g$	0.401 g
6	0.20	65	1.20	$-\gamma a_h/g$	0.329 g
7	0.20	50	0.46	$+\gamma a_h/g$	0.335 g
8	0.20	50	0.46	$-\gamma a_h/g$	0.282 g
9	0.50	65	3.00	$+\gamma a_v/g$	> 1.0 g

σ_t = Monoaxial tensile strength;
β = Drucker-Prager friction angle;
$\sigma_c = \dfrac{1 + 1/3\tan\beta}{1 - 1/3\tan\beta}\,\sigma_t$ = monoaxial compressive strength;
γ = 20kN/m^3 = material weight density, positive loading as for fig. 4;
a = ground acceleration in g.

high to ensure that the collapse mechanism of the structure would not be affected by the compression yield of the material. Finally, the value $\beta = 50$, leading to a lower σ_c value, may be considered a lower bound to the actual friction angle.

The results in table I show that the maximum ground acceleration leading to the structural collapse varies from 0.282 g to 0.697 g. It is observed that the analyses with $\sigma_t = 0.2MPa$ lead to acceleration values much lower than the analyses with $\sigma_t = 0.5MPa$. This is due to the early appearance of tensile yield and premature collapse of the structure, due to an excessively low value of the tensile yield limit. Accordingly, results with $\sigma_t = 0.50MPa$ are considered to be more realistic and the maximum acceleration leading to structural collapse varies from approximately 0.5 g to 0.7 g, depending on the values of the friction angle and of the acceleration direction.

Figure 7a,b shows the minimum (SP1) and maximum (SP3) principal stress components on the displaced structure under the effect of gravity loading. It is noted that principal stress components under the structure's own weight, varying between −0.8MPa and 0.1MPa, are everywhere much smaller than the material yield limits, and that the behaviour under gravity loading is therefore linear elastic. Furthermore,

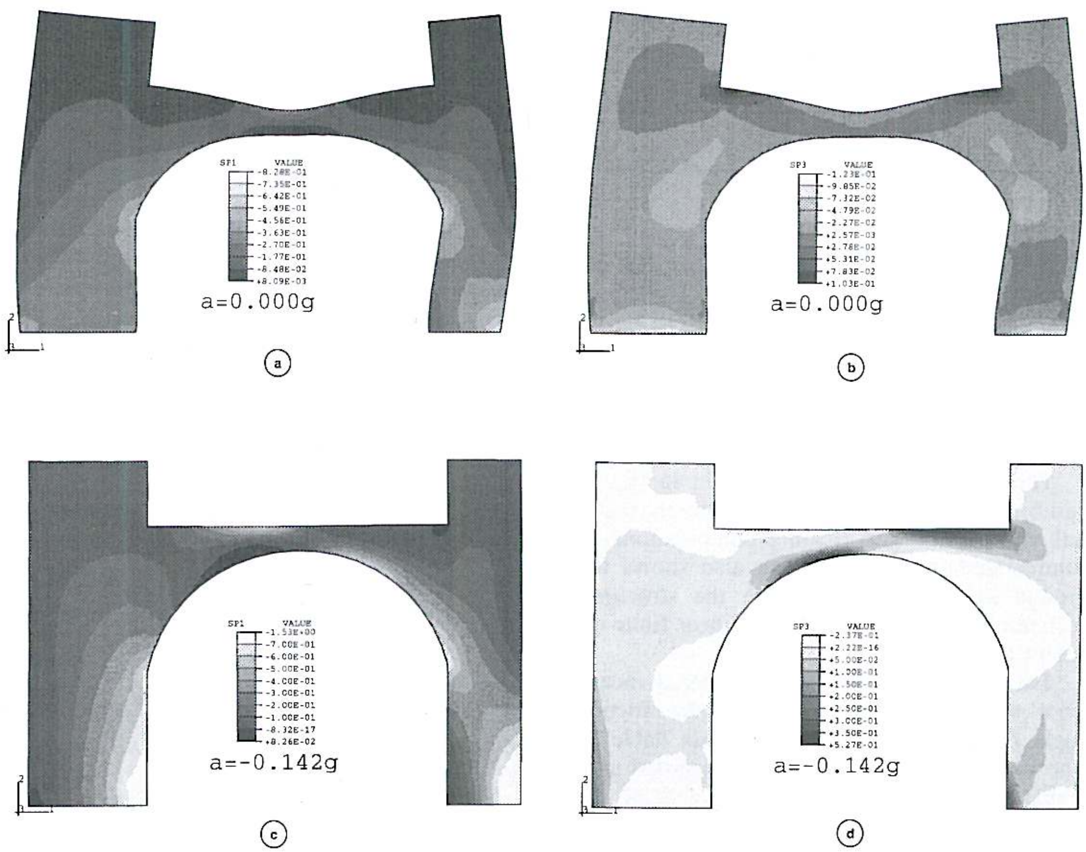

Fig. 7a-d. Stress contours: a) minimum principal stress (SP1) under gravity loading; b) maximum principal stress (SP3) under gravity loading; c) minimum principal stress (SP1) at first yield under negative acceleration; d) maximum principal stress (SP3) at first yield under negative acceleration.

the results of the non-linear analysis for increasing vertical acceleration (analysis 9 in table I) show that vertical ground accelerations have no influence on the structural collapse.

Figure 7c,d shows the minimum (SP1) and maximum (SP3) principal stress components on the displaced structure for the negative acceleration (*i.e.* ground motion from left to right) at which first yield appears ($a = -0.142$ g) for analysis 2 in table I. In particular, fig. 7d shows that at first yield there is a high tensile stress concentration at the upper right and lower left corners of the roof vault, leading to the development of plastic deformations in these regions.

For positive acceleration values (*i.e.* ground motion from right to left, analysis 1 in table I) similar acceleration values where found at first yield. Similar values were found for all the analyses listed in table I, leading to the conclusion that, regardless of the assumed material characteristics, the structural behaviour is linear elastic up to acceleration values of approximately 0.15 g. Hence, the structure could originally withstand moderate earthquakes without structural damage.

Figures 8a-d and 9a-d show the minimum principal stress (SP1, figs. 8a and 9a), maximum principal stress (SP3, figs. 8b and 9b), and maximum principal plastic strain (PEP3,

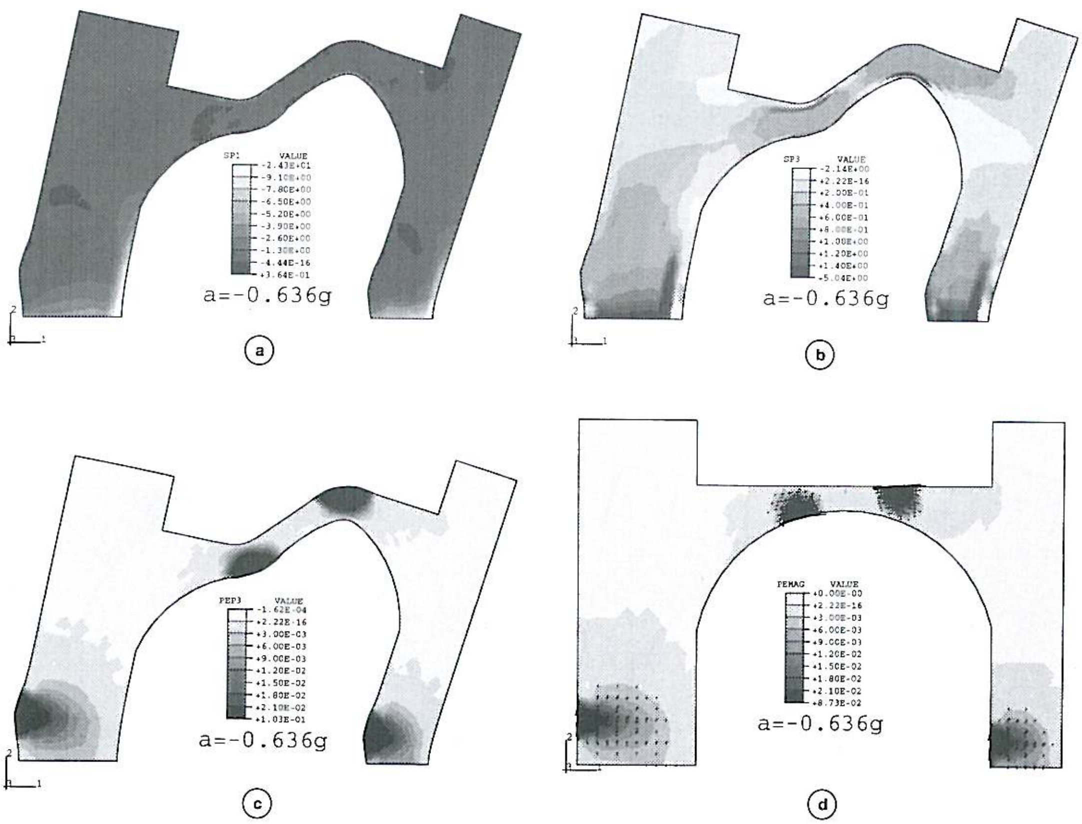

Fig. 8a-d. Stress and strain contours under maximum negative acceleration: a) minimum principal stress (SP1); b) maximum principal stress (SP3); c) maximum principal plastic strain (PEP3); d) plastic strain magnitude (PEMAG) and maximum plastic strain directions.

figs. 8c and 9c) contour maps superimposed on the displaced shape and the plastic strain magnitude (PEMAG) contour map superimposed on the maximum principal plastic strain direction (figs. 8d and 9d) for the negative and positive ground acceleration values leading to the structural collapse (a = 0.636 g and a = 0.697 g, respectively).

These figures show that the structural collapse coincides with the development of a panel mechanism, characterized by plastic hinges on both sides of the vault key-stone and at the wall bases, having maximum principal plastic strain components parallel to the outer surface of the arch and of the walls. Plastic hinges on the vault are characterized by tensile plastic deformations developing through the whole thickness of the arch, hence leading to the roof collapse, as may also be observed on the ruins of the building in Noto.

Figures 8a-d and 9a-d also show that for accelerations in opposite directions plastic damage develops on opposite sides of the same sections. Hence, for alternating acceleration directions, as during earthquake motion, damage would develop on both sides of the roof, leading to its collapse for acceleration values below those determined for acceleration in one direction.

Similar collapse mechanisms were obtained

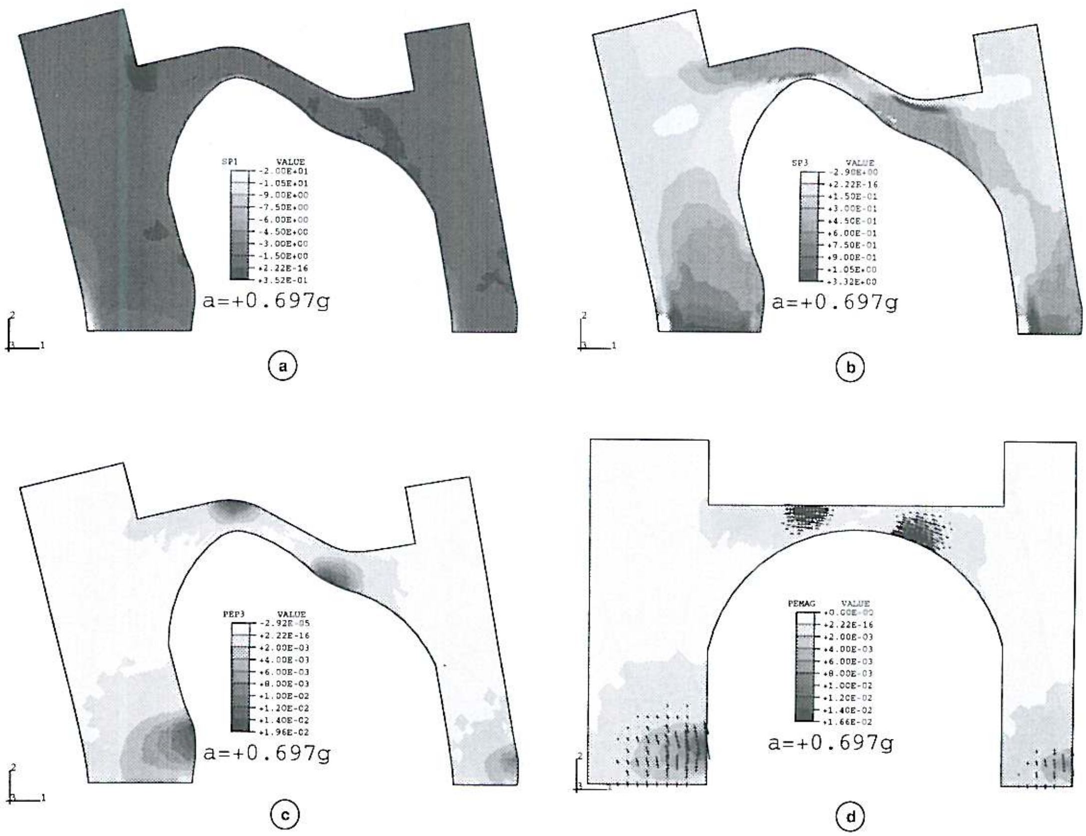

Fig. 9a-d. Stress and strain contours under maximum positive acceleration: a) minimum principal stress (SP1); b) maximum principal stress (SP3); c) maximum principal plastic strain (PEP3); d) plastic strain magnitude (PEMAG) and maximum plastic strain directions.

for the analyses with $\sigma_t = 0.5\text{MP}a$ and $\beta = 50$. In this case, however, the development of compression yielding was observed when approaching the collapse load. The development of compression yield is believed to be unrealistic, as the compressive strength of the structure should vary considerably from that resulting from the application of a Drucker-Prager yield condition with $\beta = 50$ ($\sigma_c = 1.16\text{MP}a$). Therefore, results obtained with $\beta = 65$ are considered more significant.

5. Conclusions

The results of the analysis have confirmed the validity and the interest of a multidisciplinary approach to the utilization of historical and archaeological records. It is necessary that basic data come from purpose-oriented historical research. This should allow both available documentation to be gathered, and historical contextual elements and aspects of the material culture, as buildings and their seismic response to be stressed critically; these aspects are important for the knowledge of a great seismic event. In this phase, from the non-linear analyses presented the following conclusions may be drawn:

– The proposed approach permits the lower and upper bounds of the ground acceleration necessary to cause the collapse of a building to be determined. Hence, if systematically applied to an array of meaningful structural elements coming from the same site, it may give objective and valuable insights into the ground accelerations of historical seismic events.

– For the building under examination, the structural collapse coincides with the development of a panel mechanism leading to the roof collapse for ground acceleration values between 0.5 and 0.7 g. The collapse mechanism found with the numerical analyses is in close agreement with observations made on the site of Noto.

The analysis presented must be considered only an approach to a quantitative assessment of historical seismic events. Further studies should include: numerical analyses on different structural elements; an experimental validation of the material characteristics adopted for the analysis; an in depth analysis of the ruins in Noto to identify whether damage at the wall bases, as shown by the numerical analyses, actually took place; static equivalent analyses under alternating acceleration direction with different maximum acceleration intensities; dynamic analyses adopting as input recently recorded seismic events in nearby areas.

The extension of these analyses to a meaningful sample of buildings, even of other localities struck by the 1693 event, can provide useful information to pinpoint some characteristics of the earthquake, such as its localization (still uncertain between earth and sea) and its propagation.

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