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THE DIGITAL SURVEY AND STRUCTURAL BEHAVIOUR OF CHURCH OF ST. ASTVAZAZIN IN ARENI, ARMENIA

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Abstract

The assessment of the seismic behavior of historic constructions should start from a thorough historical study of the building, including a precise and accurate e geometric survey and the mechanical characterization of the materials. In these years the geometric survey has improved more and more, thanks to the development of new technologies such as the digital photogrammetry, which can provide the exact geometry of the product. The correct geometry, together with the information concerning the sequence of transformations suffered by the building, provides a reliable representation of the construction. Finally, the mechanical characterization of the materials allows to model the behavior of the structure both in its elastic and inelastic range. This paper presents a study made on the church of Areni (Church of Sant'Astvazazin in Areni, Armenia), which was built in 1321 by the architect Momik. After having carried out an accurate historical research, a geometrical survey has been made by combining the techniques of photogrammetry and laser scanner; the reconstruction of the 3D geometrical model allowed to obtain the Finite Element model to use for the structural analysis. Two types of analyses were carried out: a static (pushover) and a dynamic (time history) one. The obtained results have been analyzed and compared with the historical documentation about of the damages suffered by the church due to the last earthquake.

Keywords: Historic structure, digital photogrammetry, 3D FE model, Pushover analysis, Time history analysis.

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1 INTRODUCTION

The behavior of historic buildings is characterized by the construction techniques, related to the material properties and the architectural styles. To assess the structural behavior is very important achieving an accurate knowledge of the building, including the geometric, historical and mechanical properties of the material and the behavior of the various structural elements.

The current framework of knowledge of the behavior of historic buildings, and in particular that of churches, is the result of observing the damage sustained by earthquakes in the past. The correlation between the type of church or structural element and the damage has led to identify evaluation procedures [1-4] at different levels of approximation.

The international and national regulations [5,6] define the assessment process with particular attention to determine the structural performance under seismic actions.

The purpose of this work is to describe the process undertaken to arrive at the evaluation of the structural performance of the church of Areni.

The site offers the opportunity to study the architectural structure and, at the same time, to observe how the architectural composition is a synthesis of the expressive and static aspect of the project itself.

During the period of Mongol domination between 1243 and 1344, the Vaiodzor territory was implemented and there was a flourishing of the building activity from the beginning of the 1300s up to the middle of the century. The Areni church (Santasvtzazink / Our Lady Mother of God) [7,11] was built in 1321 by the architect Momik [12].

The Areni church (see Figure 1) is composed by a central plan with the presbytery portion flanked by narrow chapels and a nearly square-shaped hall, characterized by the presence of two poly-lobed pillars which confer a perceptive unity of the hall into the composition tripartition. The roof is a geometrically double-pitched pavilion whose center is occupied by a slender tambour pierced by openings and ending with a conical cover.

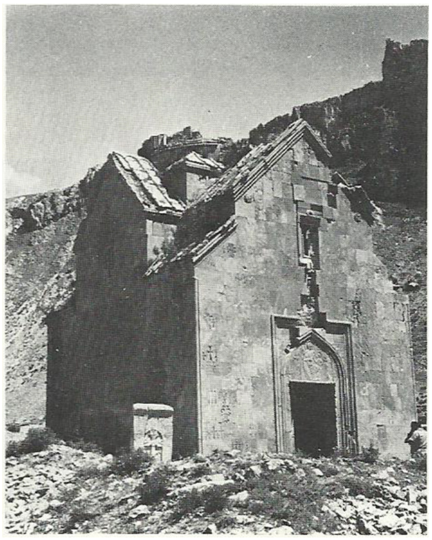
The rigorous external symmetry opens up to a cruciform geometry where the door is set against a full part with only one window. The external rigor is broken down into the inner box with ascending heights that gradually conquers the hall center and an arrangement between full and empty that seems to obey to a rigid ternary rhythm that reverberates throughout the structure.

The church was built with a local stone; all the walls consist of a rubble masonry technology with an inner core made of stone flakes put mostly horizontally; the two outer vestments, about 10 cm thick, are well connected with the inner part. This information has been deducted by observing portions of masonry belonging to similar buildings, partially demolished. Since no trustworthy information on the mechanical characteristics of the masonry is available, the mechanical parameters used for numerical analysis were obtained from generic tabulated values [13].

The problems related to the survey were dealt through the acquisition of geometric data (performed with photogrammetry), the transition from the geometric model to the physical mathematical model to use for the structural analyzes carried out by the approach to the finite element method (FEM).

The evaluation process starts from the survey carried out in situ by means of a traditional celerimetric survey flanked by photogrammetry, which allows to obtain a three-dimensional model of the church. Subsequently, through three-dimensional modelers, a simplified model of the church has been obtained which, through the use of passages within software to the finite elements, allows to obtain the model on which the structural analyzes are to be performed.

Fundamental aspects of the following steps were the definition of seismic actions, in particular the definition of the seismic hazard of the area obtained through bibliographic information as no on-site surveys were carried out to determine the stratigraphy and consequently the type of subsoil.



a b

Figure 1: a) The respective photos, taken during the missions, show the front of the church of Areni; b) Areni following the collapse of the tambour and the part above the wall box, on the right. See Cuneo P. [7], p. 394.

The definition of the mechanical characteristics of the materials was carried out alongside the Italian regulations, as at present a campaign of investigations on materials aimed at defining the main characteristics is being planned in order to carry out a plan of restoration and consolidation of the church.

The evaluations were carried out considering two types of non-linear analysis using the Abaqus software. The first assessment of the structural behavior was carried out by means of a pushover analysis, considering a load proportional to the masses and applying the N2 method [14], after which dynamic nonlinear analyzes were carried out using the three components of time histories of accelerations related to events presenting PGA values compatible with the area under examination and the type of subsoil defined. The results obtained allow us to highlight the behavior compatible with the damage panel as shown in Figure 1b.

2 3D SURVEY

The survey operations in the on-site campaigns have taken into account specific external factors: the limited time of stay, the difficulty of access to the various sites and the work on the laptop, for which it has been preferred to privilege a direct survey of three-dimensional photogrammetric processing with a topographic survey campaign focused on returning the external profiles and essential internal geometries. According to the integrated protocol, elaborates of excellent metric reliability have been obtained and these generated a basis to control and manage the models for thematic analyzes (Total station “Leica”, Bosh laser level pll360, Nikon camera D530).

On the basis of the architectural elaborates based on the integrated survey method and the models extracted from the photogrammetric process, it was possible – through a three-dimensional modeling software – to reconstruct the architecture polygonal geometry of church (see Figure 2).

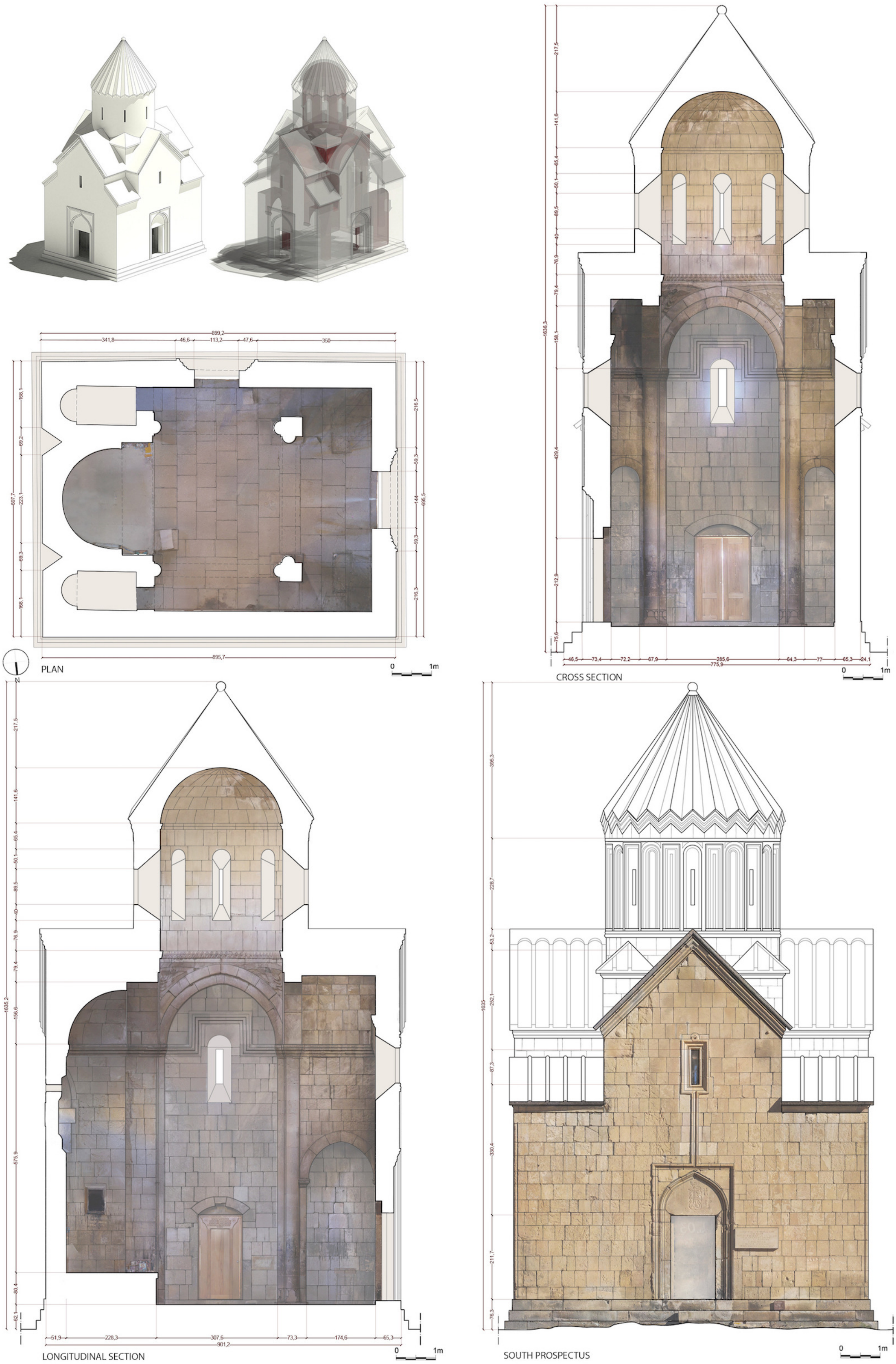


Figure 2: Architectural elaborations obtained through the integration of the three survey systems: direct survey, photogrammetric survey and topographic survey.

The goal of the output product was to be able to give rise to a model of satisfactory geometric reliability that had at the same time good computational management. The dense cloud has been generated by the three-dimensional photographic modeling program (Agisoft Photo Scan Professional). Subsequently a polygonal model through a 3D modeling program has been chosen.

The model reliability (see Figure 2) has been controlled through three protocol lines that were gradually crossed: the topographic model with the direct survey, the photographic model with the topographic and the direct survey with the photographic. The controls have been facilitated by a laser level that has univocally determined which the horizontal section plan and the lifting plan were (see Figure 3).

The obtained models are made up of numerous contiguous nurbs flat and curved surfaces that describe the inner and outer shell progression of the structure. Only the full wall elements were taken into consideration. To be able to define with a good reliability which the internal full structures, not visible directly through the survey were, it was necessary to compare with historical photos and coeval buildings designed by the same architect.

Once the model was reconstructed and verified, it was possible to extract from the 3D modeling software the IGES file, that is the wireframe structure generated by the intersection of the various surfaces, this file was used to obtain the 3-D Finite Element Model (FE Model).

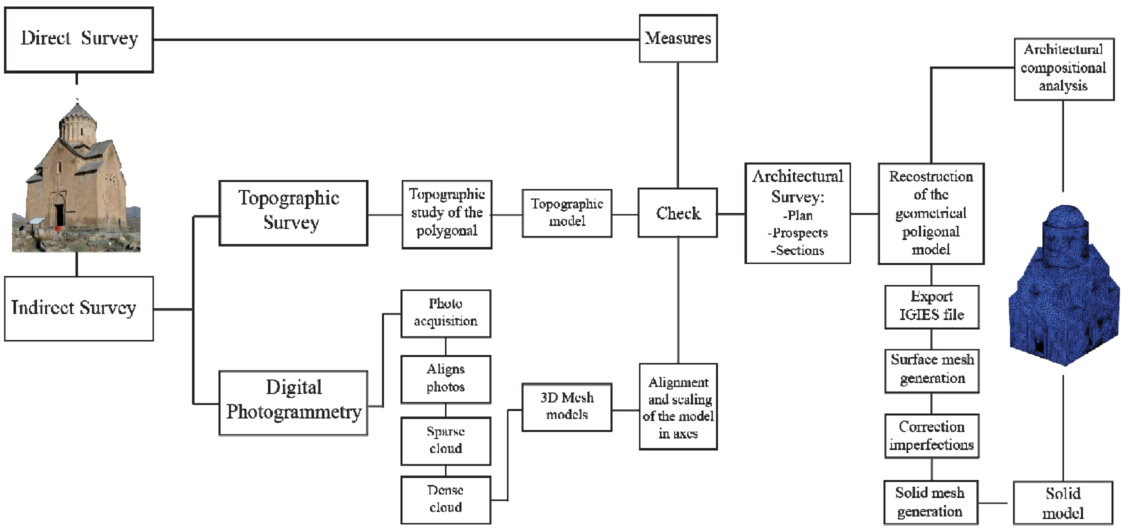


Figure 3: Scheme of the procedure adopted to obtain 3-D FE model.

3 FE MODEL

A non-linear modelling strategy has been adopted to study the non-linear mechanical behavior of the Church, under seismic actions. All structural elements have been discretized by isoparametric 4-nodes elements and the masonry material is assumed to be homogeneous and is represented through a Concrete Damage Plasticity (CDP) model.

The generation of the 3-D mesh has been obtained working with IGES file: first the surface mesh was generated according to parameters that allowed the creation of a surface with a controlled and limited number of polygons. In this way, the surface mesh was reliable and faithful to the first geometric model. Starting from this surface mesh (external and internal), it was possible to generate the 3-D mesh (see Figure 4).

The CDP, implemented in Abaqus software [15], was conceived by Lubliner [16] in to model the concrete but it owns all the necessary features needed to be adopted for macroscopic computations on masonry structural elements. The model, as the masonry behavior, is characterized by distinct tensile and compressive strengths and different post peak behaviors with damage parameters, as showed in Figure 5. The degradation mechanisms involve the opening and closing of previously formed cracks, under cyclic loading conditions. The CDP parameters adopted in the numerical simulations are reported in Table 1, Table 2 and Table 3.

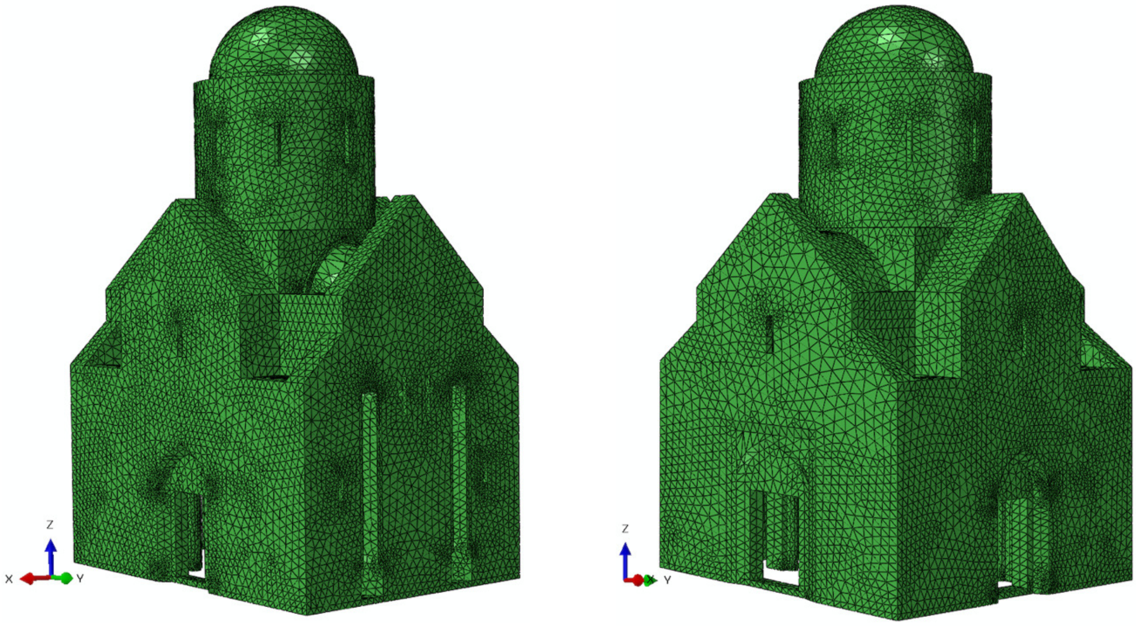


Figure 4: FE Model

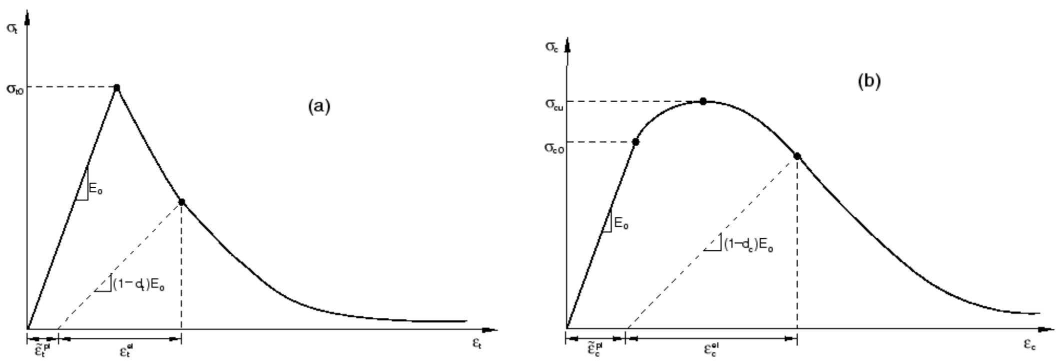


Figure 5: Non-linear behaviour in uniaxial tension and compression (CDP model).

σ_{cu} [MPa]	σ_{tu} [MPa]	E [MPa]	ν [--]
1.3	0.25	1230	0.1

Table 1: Mechanical properties adopted in the CDP model.

Ψ [°] Dilatance angle	K_c DP correction parameter	e eccentricity	f_{b0}/f_{c0} Biaxial strength ratio	μ Viscosity parameters
36	0.666	0.1	1.16	0

Table 2: Additional CDP parameters adopted in the numerical simulations.

Compression		Tension	
d_c [--]	Anelastic strain [--]	d_c [--]	Anelastic strain [--]
0	0	0	0
0.9	0.035	0.9	0.03

Table 3: Relation between damages in tension and compression and inelastic strains.

Different analyses have been performed: four non-linear static analyses (pushover ±x, ±y) and two non-linear dynamic analyses assuming two different ground motions, selected by the database European Strong-Motion database, which are spectrum compatible to the elastic spectrum provided by the European Code, EC8 [17], for the site, as shown in the following section.

4 SEISMIC INPUT

The information available in the technical literature, relating on the seismogenicity and the tectonics of the Armenian area [18,19], highlights that the area of ARENI has a seismic hazard of reference between 0.4g and 0.5g.

There are no information regarding the stratigraphy of the ground where the building is located, and there are no seismic stations nearby; therefore, it is assumed that the spectrum of the recordings to use in the analyses is that of soil type B [17]. In order to obtain a set of 7 accelerograms for each component whose average PGA value is as close as possible to the hazard values of the area (Table 4), seven recorded events on soils of category B were selected from the European database [20].

Code	Name station	Event Name	Event Data	Distance [km]	PGA E [g]	PGA N [g]	PGA Z [g]
TK_1201	AI_049_BNG	TURKEY	01/05/2003	11.8	0.292	0.519	0.432
HL_AIGA	AIGIO	GREECE	15/06/1995	23.6	0.498	0.520	0.190
IT_AMT	AMATRICE	CENTRAL_ITALY	30/10/2016	26.4	0.532	0.401	0.324
IV_NRCA	NORCIA	CENTRAL_ITALY	30/10/2016	7.2	0.303	0.495	0.494
IT_AQG	L'AQUILA-V.ATERNO-COLLE GRILLI	L_AQUILA	06/04/2009	5.0	0.446	0.489	0.239
3A_MZ08	AMATRICE / RAN	CENTRAL_ITALY	30/10/2016	26.4	0.537	0.436	0.328
IT_NRC	NORCIA	CENTRAL_ITALY	30/10/2016	4.6	0.486	0.372	0.375

Table 4: The ground motions main information.

In this paper, for sake brevity, only two accelerograms sets have been used for dynamic analyses: the accelerograms and elastic spectra selected (horizontal and vertical components) are shown in the Figure 6 and Figure 7 respectively.

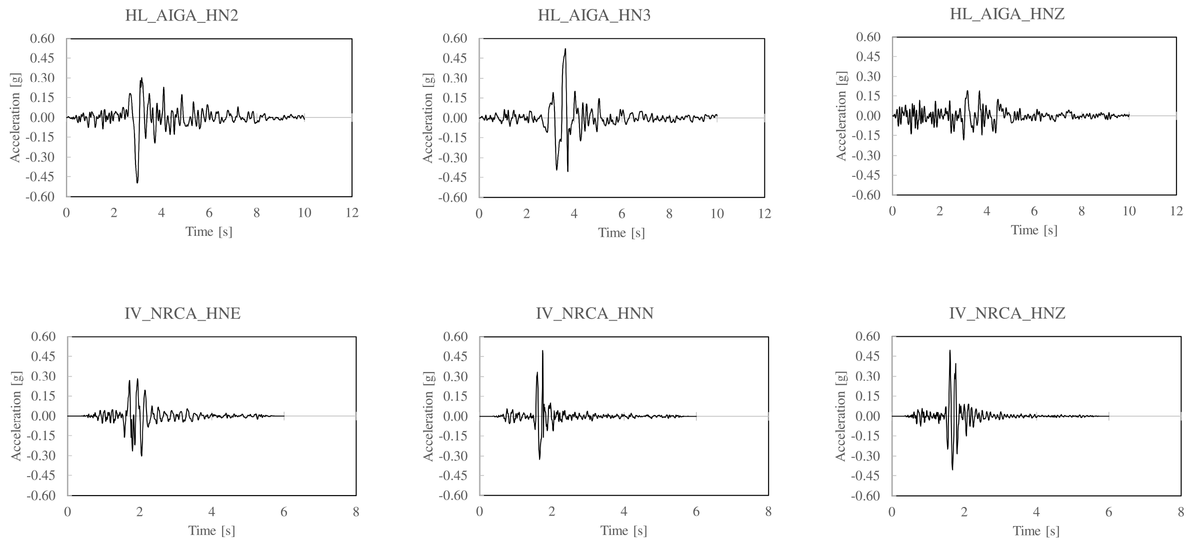


Figure 6: Recording accelerogram of the ground motions selected for the dynamic analysis.

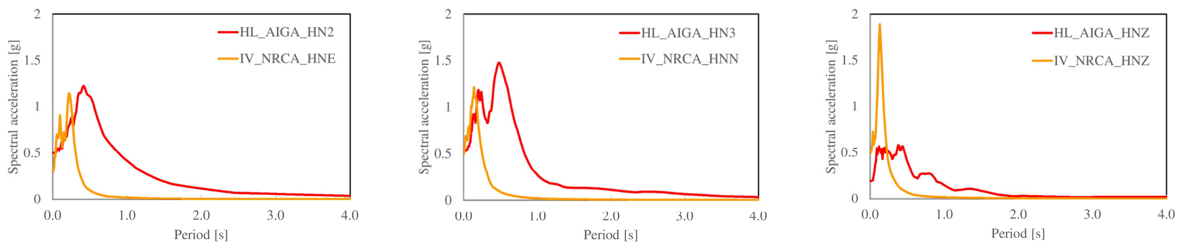


Figure 7: Elastic spectra of the ground motions selected for the dynamic analysis.

5 ANALYSES AND RESULTS

5.1 Pushover analyses

The assessment of the seismic capacity of the church was obtained by the pushover analysis, applying a load distribution proportional to the mass, both in direction X and in direction Y (positive and negative). The pushover curve (force-displacement diagram), obtained from the analysis, regarding to multi-degree of freedom system (MDOF), was reduced to a perfectly elastic-plastic bilinear diagram.

In particular, the bilinear force-displacement diagram has been obtained assuming that the area under the diagram itself is equivalent to the area under the original curve and that the following assumptions are fulfilled:

• the maximum displacement of the bilinear curve is assumed at 85% of the maximum value of the cutting force at the base;

• the slope of the first linear branch was obtained using the secant line, with respect to the pushover curve, at the point corresponding to a base shear force equal to 60% of the maximum value.

The structural performances are evaluated with the N2 method [14], converting the MDOF system into an equivalent one, with only one degree of freedom (SDOF) through the modal participation factor (in this case equal to 1). The capacity curves respect to two main directions (X and Y), with positive and negative sign, are reported in Figure 8. The results highlight resistance and stiffness in Y direction greater than those in X direction, whilst the displacements are similar. At the same time, the behaviour of the church is independent of the sign (positive and negative) of the applied load distribution.

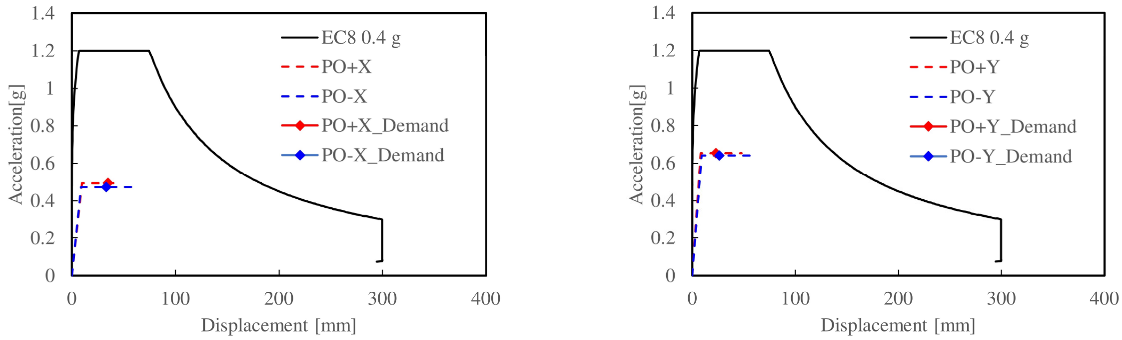


Figure 8: Bilinear Capacity curve in ADRS format.

In Table 5 are reported the main results obtained from the pushover analyses: the period of SDOF (T); the ratio between Fse (equivalent elastic force) and Fy (yield force), called q*; the yielding displacements (Dy); the ultimate displacement (Du) and the displacement of demand (Dd) obtained by applying the N2 method [14]; finally, the ratio between Du and Dd, i.e the safety factor. Figure 9 shows the tensile damage for each pushover analysis corresponding to the displacement D_d. All the contour plot shows that the areas mainly damaged are at the base and near the edges of the openings; conversely, there is no damage at the drum and the dome.

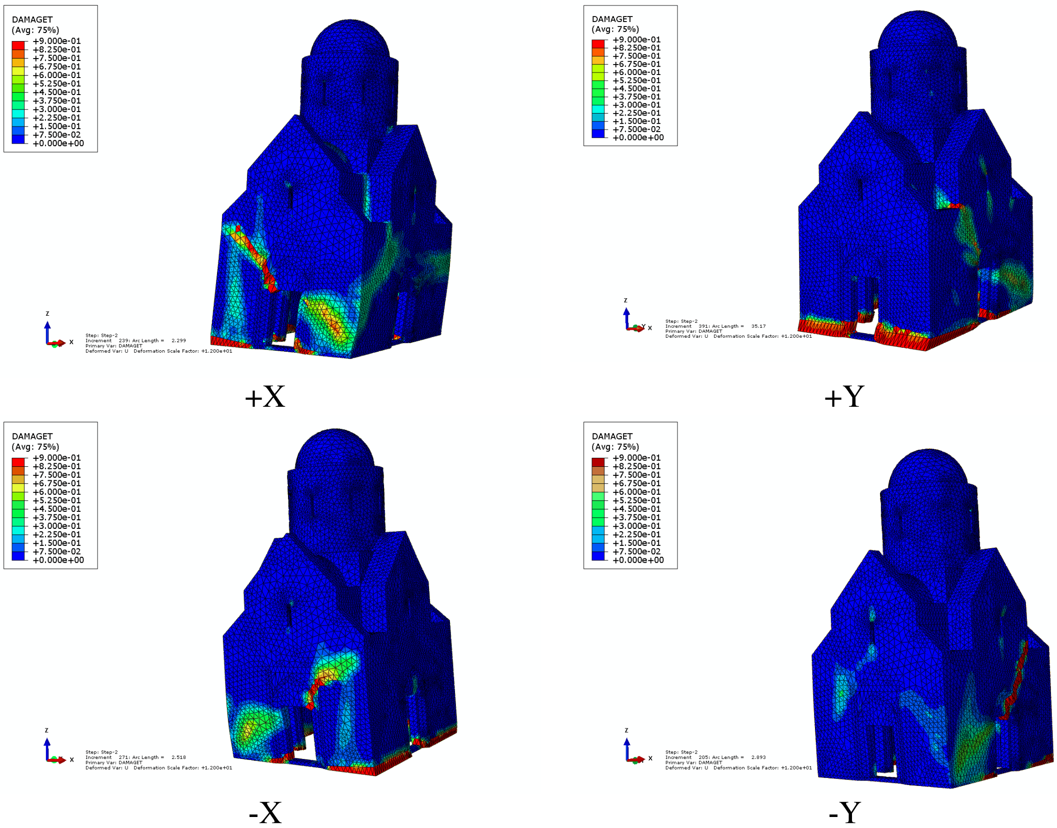


Figure 9: Tensile Damage corresponding to the D_d displacement.

Analisys	T [s]	q*	Dy [mm]	Dd [mm]	Du [mm]	Du/Dd
PO+X	0.280	2.43	9.6	35	47	1.35
PO-X	0.274	2.54	8.8	33	63	1.90
PO+Y	0.223	1.84	8.1	23	48	2.09
PO-Y	0.241	1.87	9.2	26	56	2.14

Table 5: Main Result for the pushover analysis.

5.2 Time History

Two dynamic analyses have been performed using the two sets of accelerograms shown in the Figure 6 and described in section 4; all three components (X, Y and Z) of the two sets were applied to the model. The coefficients α and β, needed to define the Rayleigh damping matrix [21, 22], have been obtained using the first two fundamental periods of the church and a damping factor equal to 2%.

Figure 10 shows the dynamic response of the church, in terms of the displacements (direction X and Y) of the node at top of the dome, subjected to the HL_AIGA set. Note that the maximum displacements in the x and y directions are 14 mm and 10 mm respectively.

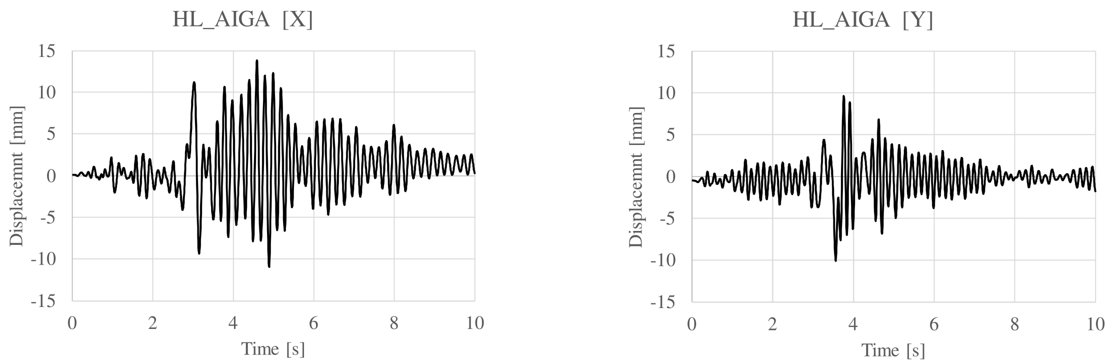


Figure 10: Diagram of Top displacement for the HL_AIGA event for the X and Y direction.

Figure 11a-b shows the displacements, always of the node placed at the top of the dome, projected on the X-Y plane, concerning respectively the two sets HL_AIGA and IV_NRCA. Note that the curve relative to the set IV_NRCA (see Figure 11a) shows displacements prevalently in the two main directions (x and y) whilst the curve related to the set HL_AIGA (see Figure 11b) is distributed over the whole plane.

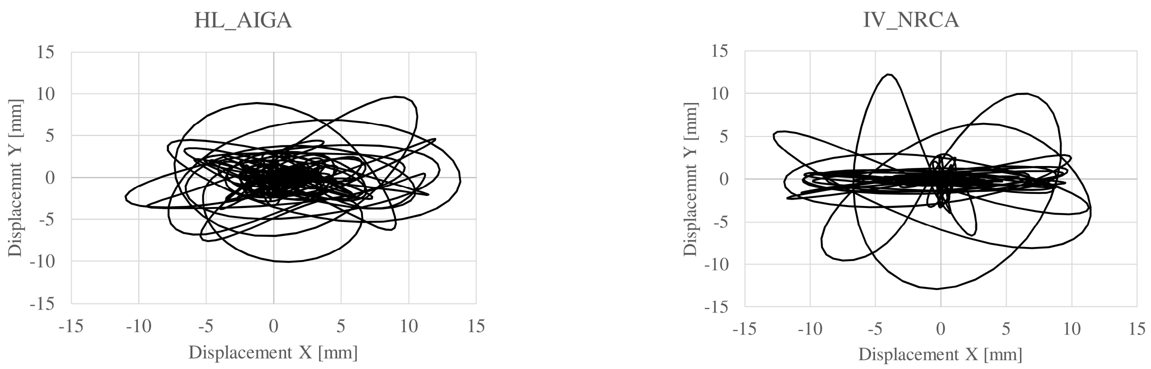
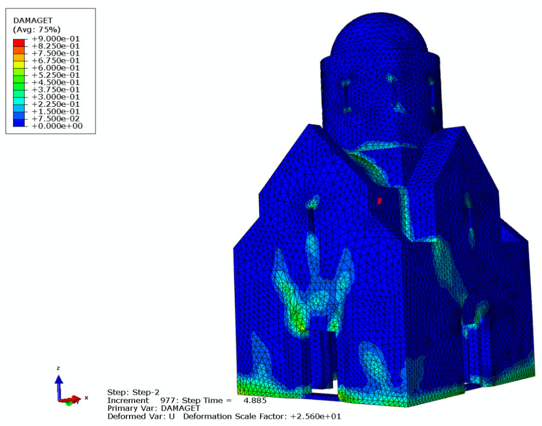
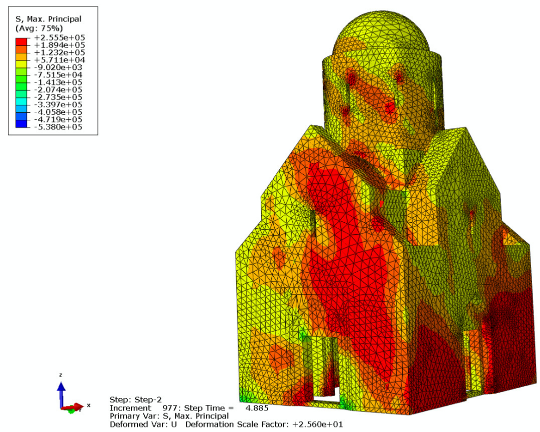


Figure 11: Diagram of Top displacement of the X and Y plane for the HL_AIGA and IV_NRCA events.

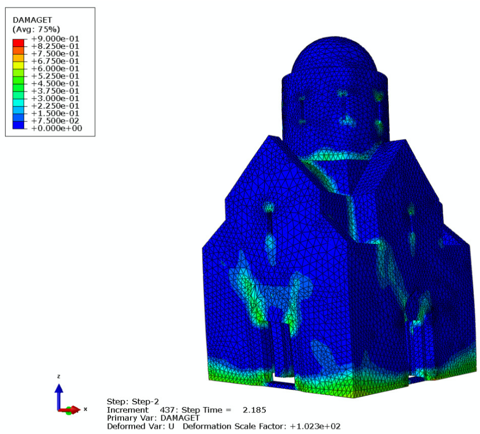
In order to show the distribution of the tensile damage and maximum stress, Figure 12 reports this distribution at maximum displacement value; it can be noted that the areas most stressed and therefore damaged are those near the base, the drum of the dome and the openings.



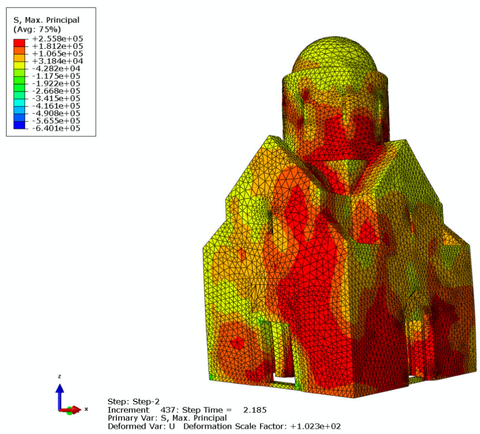
HL_AIGA (Tensile Damage)



HL_AIGA _(Maximum Stress)



IV_NRCA_(Tensile Damage)



IV_NRCA_(Maximum Stress)

Figure 12: Tensile Damage and Maximum Stress corresponding to the maximum displacement for the two events.

6 CONCLUSIONS

A case study was presented to show the knowledge path needed to assess the structural seismic performance of historic monumental buildings. This knowledge path was necessarily started from a careful historical research concerning to the constructive evolution and from an accurate survey. The survey of the church of Areni was carried out using both the traditional technique and the photogrammetry. The survey information was process in order to obtain a 3-D FE model. At the same time, important information was collected to define the soil and the seismic characteristics of the site, necessary to define the seismic actions. Finally, non-linear dynamic and static analyses were performed.

The results of the two analysis types show that the damage patterns are similar, except for the area at the base of the dome drum. The displacements achieved in the dynamic analysis are similar to those corresponding to the yield level obtained through the pushover analysis; however, these values are lower than the demand calculated with the N2 method. These first results allow to express an early evaluation of the seismic behaviour of the church of Areni under seismic action; however, in order to minimize the uncertainty of the results, further investigations will be the focus of future studies, such as *in situ* determination of the mechanical characteristics of materials and an accurate definition of the subsoil.

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