Rehabilitation design of the Katholikon of Varnakova Holy Convent in Phocis

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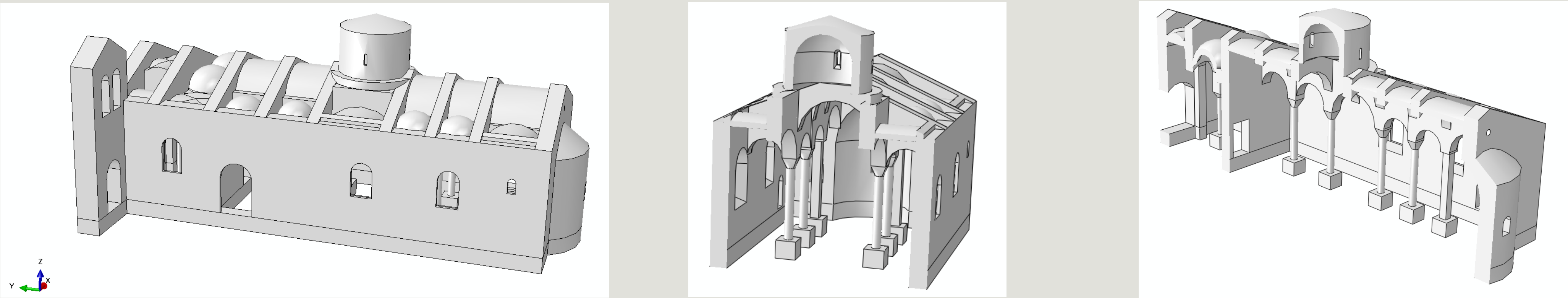
DESIGN PHASES (1/1)

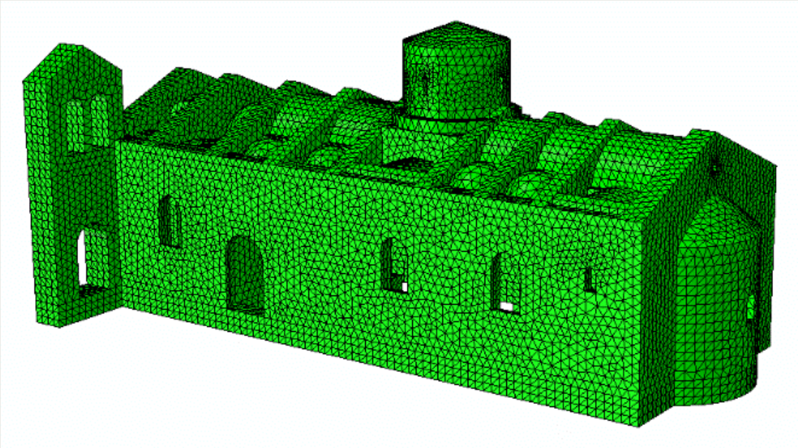
- Assessment of structural behavior under earthquake actions
- Rehabilitation design (2020) was proposed the roof was covered by tiles
- Opening of the roof structure and investigation of the area between dome structure and roof → Modifications to the structural model
- New rehabilitation design (2021) was proposed that was accepted by the Authorities

Assessment – Current State (1/1)

Comparison between analysis results with the pathology of the current state of the structure



Structure geometry – current state



Numerical model

Calibration

Comparison between the fundamental frequencies of the model and the measured frequencies as determined by microtremors measurements in situ

Analyses

Time history dynamic analyses for Limit States SD (Significant Damage) and NC (Near Collapse)

Probabilistic Seismic Hazard Analysis – Time History Selection (1/3)

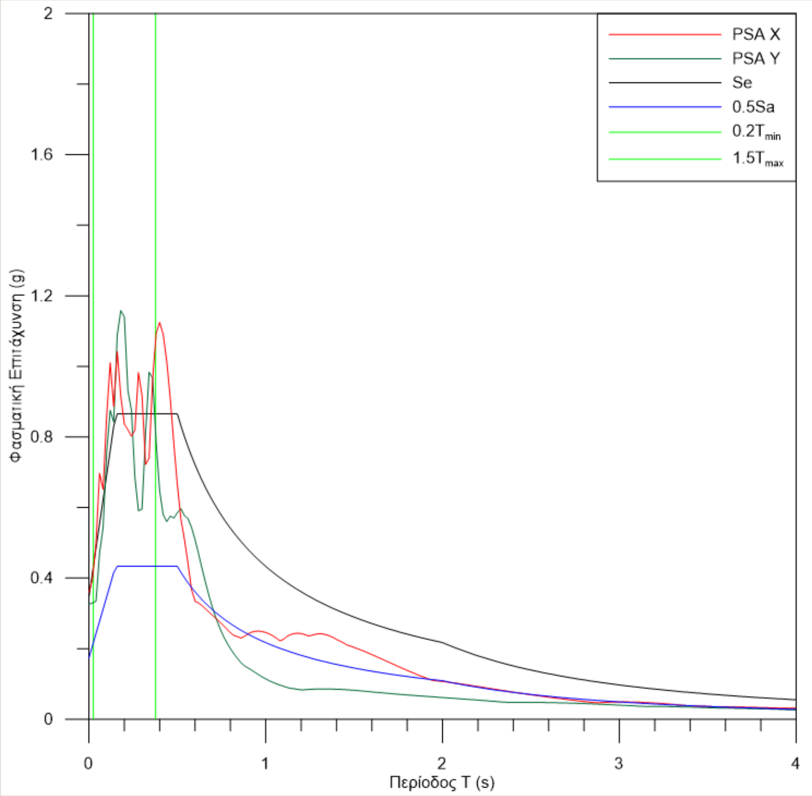
- Eurocode 8 (EC8)
  Seismic hazard zone Z2 → a_{gR} = 0.24 g for SD Limit State
  Importance factor for Consequence Class CC3-a, γ_I = 1.20
  Geotechnical study: Soil Category B, S = 1.20

Limit State	Consequence Class CC3-a – T_R (years)	PGA determination	Peak Ground Acceleration, PGA (g)
NC – Near Collapse	2500	$1.17 \cdot PGA_{SD}$	0.4044
SD – Significant Damage	800	$\gamma_I \cdot S \cdot a_{gR} = 1.20 \cdot 1.20 \cdot 0.24$	0.3456

- **7 triaxial ground motion acceleration time history records** from the PEER NGA-West2 and the ORFEUS ESM databases were selected **for each Limit State**
- Scaling of ground motions according to EC8

PROBABILISTIC SEISMIC HAZARD ANALYSIS – TIME HISTORY SELECTION – SD LIMIT STATE (2/3)

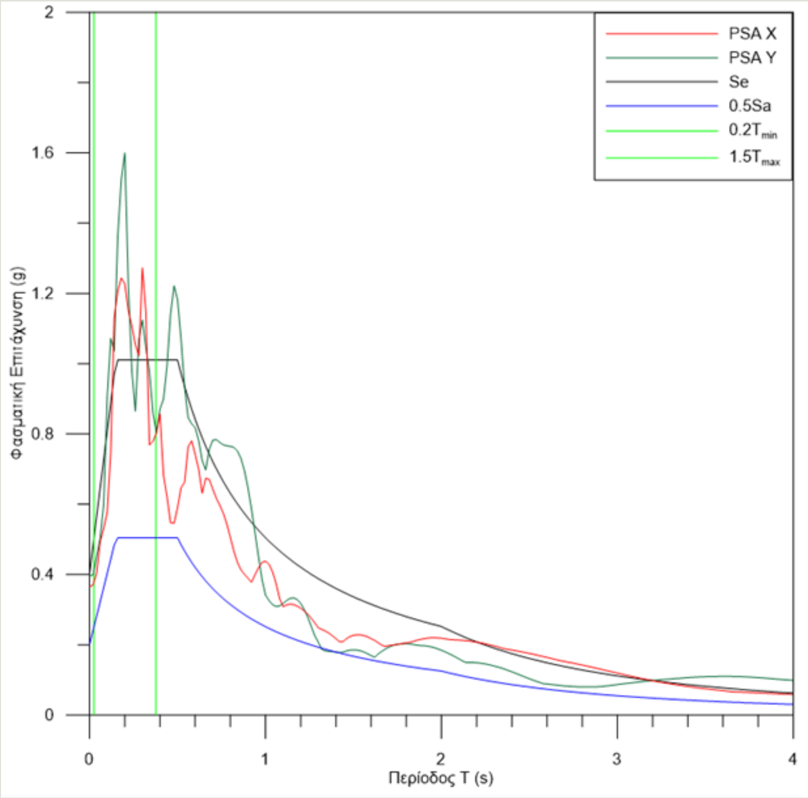
#	Database number	Seismic Event	Year	Station	M_w	Scaling Factor
1	IT.AQK	L_Aquila	2009	L'Aquila-V. Aterno-Aquil Parking	6.1	1.15
2	IT.TLM1	Friuli_1st_Shock	1976	Tolmezzo Centrale-Diga Ambiesta 1	6.4	1.12
3	IV.T1244	Central_Italy	2016	Spelonga	6.5	1.50
4	RSN 763	Loma Prieta	1989	Gilroy – Gavilan Coll.	6.93	–
5	RSN 1510	Chi-Chi, Taiwan	1999	TCU075	7.62	1.25
6	RSN 3473	Chi-Chi, Taiwan-06	1999	TCU078	6.3	–
7	RSN 4213	Niigata, Japan	2004	NIG023	6.63	1.25



Seismic Event RSN 763-Loma Prieta

Probabilistic Seismic Hazard Analysis – Time History Selection – NC Limit State (3/3)

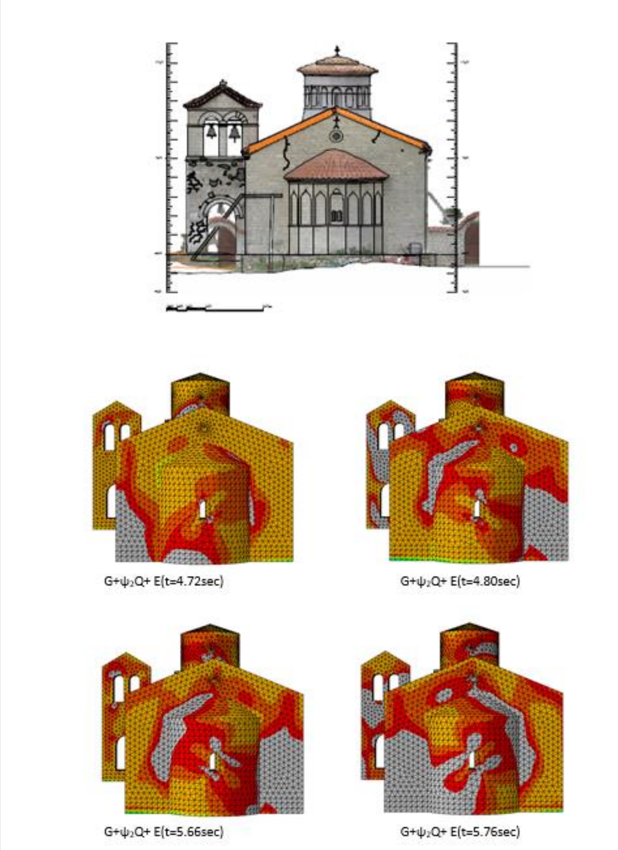
#	Database number	Seismic Event	Year	Station	M_w	Scaling Factor
1	3A.MZ102	Central_Italy	2016	Accumoli Madonna delle Coste-ENEA	6.5	–
2	IV.T1299	Central_Italy	2016	Amatrice, Casale Bucci	6.5	1.20
3	RSN 139	Tabas, Iran	1978	Dayhook	7.35	1.10
4	RSN 150	Coyote Lake	1979	Gilroy Array #6	5.74	1.15
5	RSN 802	Loma Prieta	1989	Saratoga – Aloha Ave	6.93	1.20
6	RSN 982	Northridge-01	1994	Jensen Filter Plant Admin. Building	6.69	1.10
7	RSN 1013	Northridge-01	1994	LA Dam	6.69	1.35



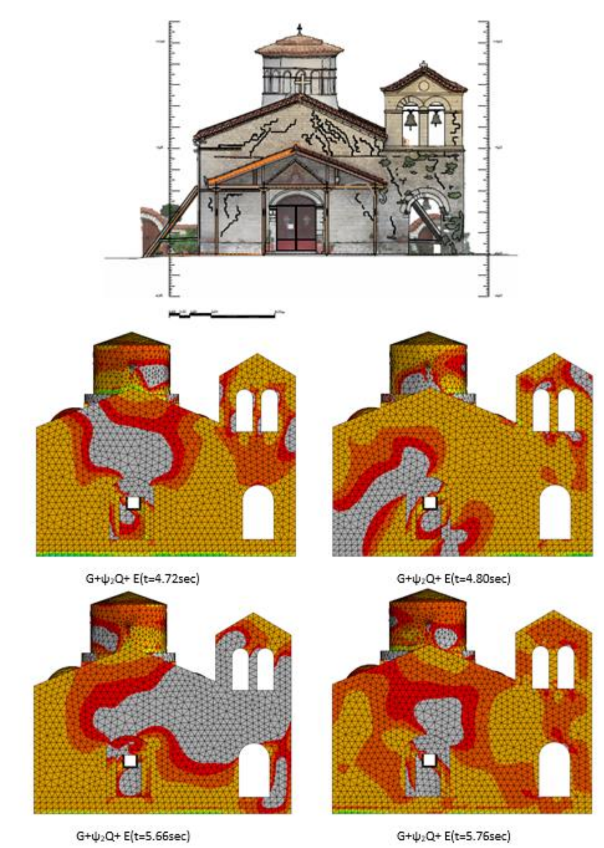
Seismic Event 3A.MZ102

Assessment – Current State – Results – SD Limit State (1/2)

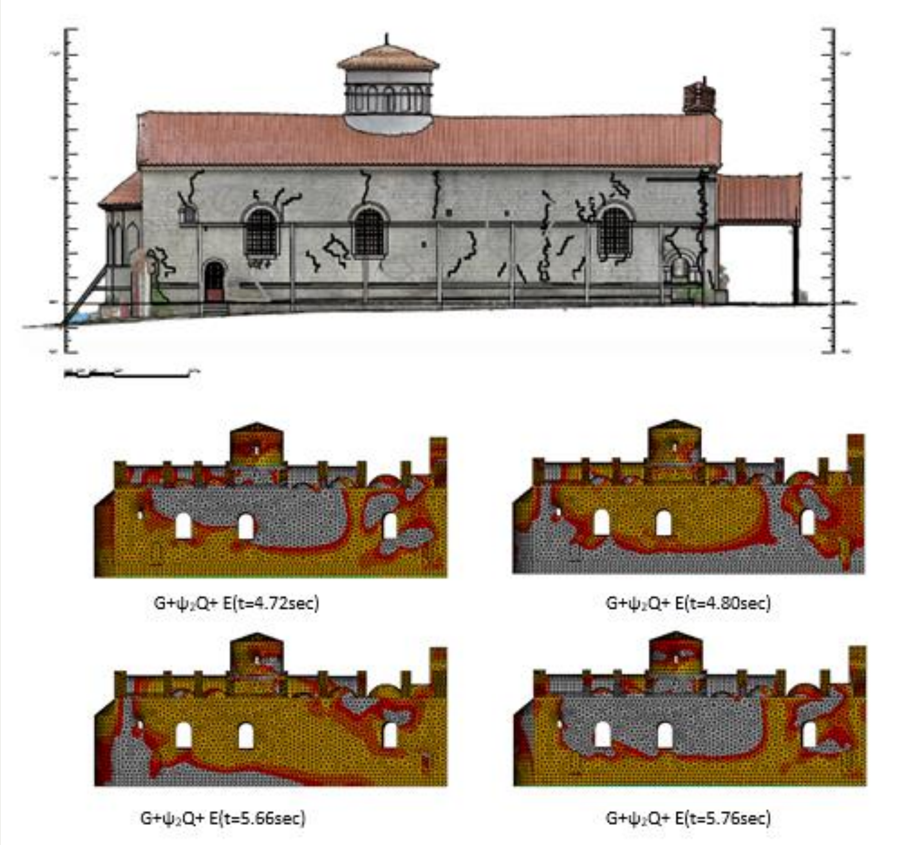
- Principal tensile stresses of masonry check from G+E load combination
- Masonry tensile strength = 0.1 MPa.



East façade



West façade

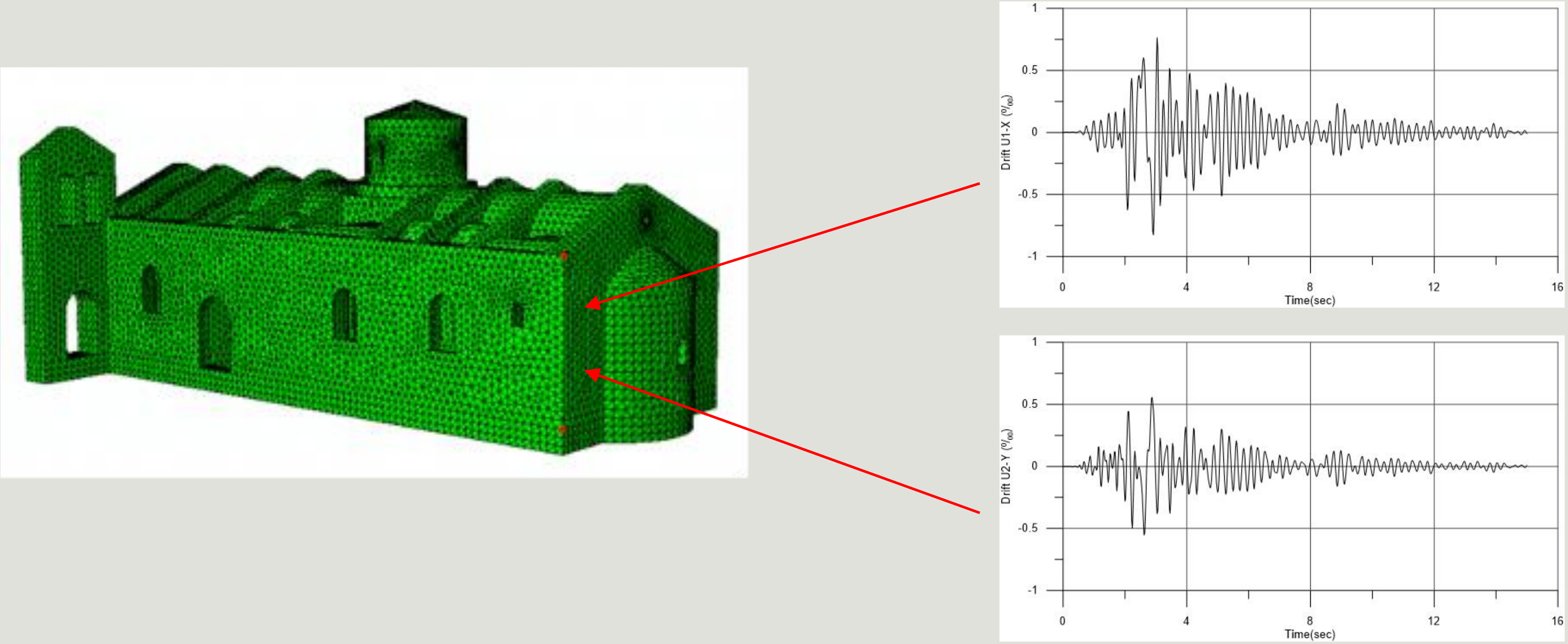


North façade

Tensile strength exceedance in the entire masonry shell, dome structure and bell tower

Assessment – Current State – Results – NC Limit State (1/2)

- Failure criterion: limitation of interstorey drifts at critical locations from G+E load combination
- Interstorey drift limit for NC limit state: 3%



No exceedance of interstorey drift limit was observed

Assessment – Current State – Conclusions (1/1)

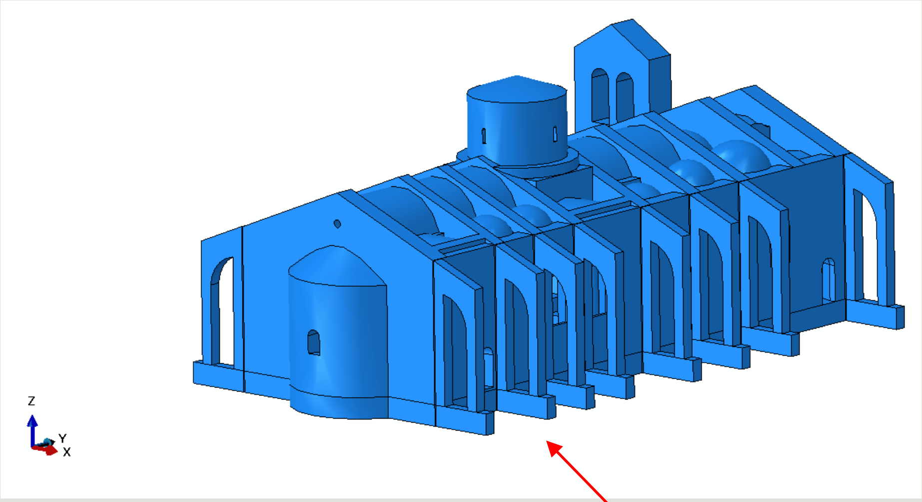
- The load bearing structure of the Katholikon is seismically vulnerable, consisting of masonry walls and arched colonnades.
- The observed pathology is verified by the numerical model.
- The design of new load bearing structures is proposed for the enhancement of the seismic behavior of the Katholikon.

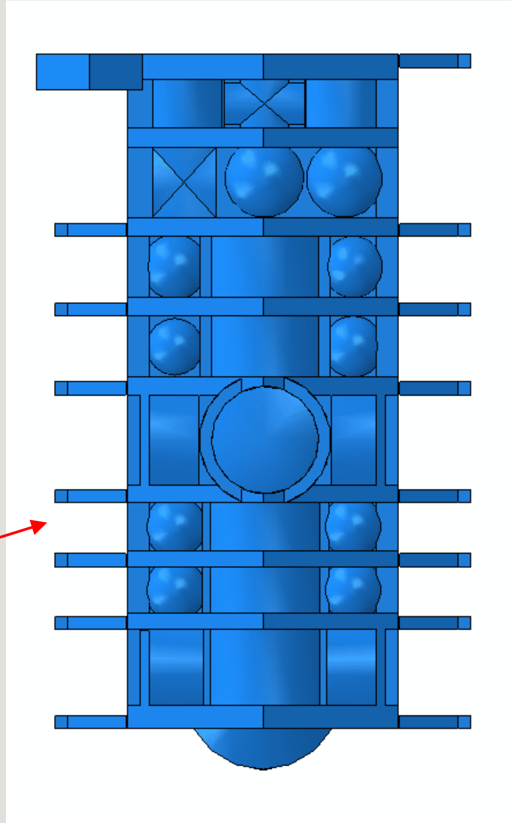
Rehabilitation Design 2020 (1/1)

The following interventions were proposed for the enhancement of the seismic behaviour of the Katholikon:

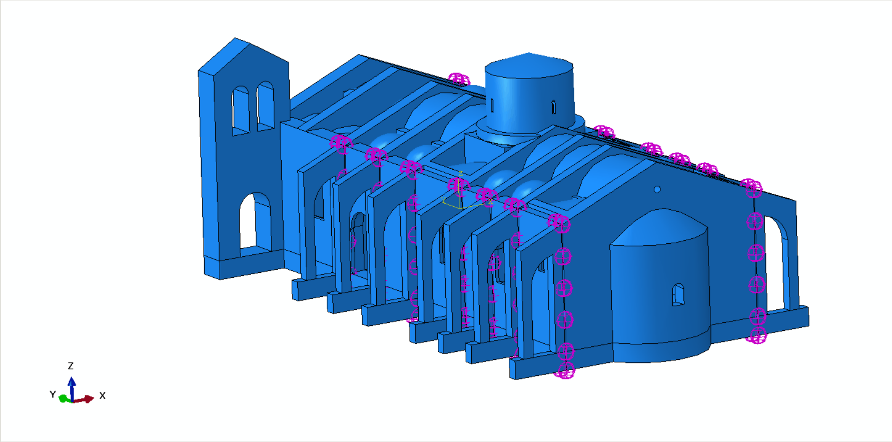
1. Global grouting of all masonry and domes
2. Stone stitching in the roof dome structure
3. Deep repointing on the outer and inner surface of the masonry
4. Installation of steel ties in the arches
5. Construction of new buttresses in the shape of arches along the perimeter of the north and south walls. The buttresses would be constructed of stone masonry with a core of reinforced pozzolanic concrete. The connection with the Katholikon would be realized using Ø14 dowels.
6. Construction of reinforced pozzolanic concrete surface foundation for the new struts

NUMERICAL MODEL – STRENGTHENED STRUCTURE (1/2)





Buttresses

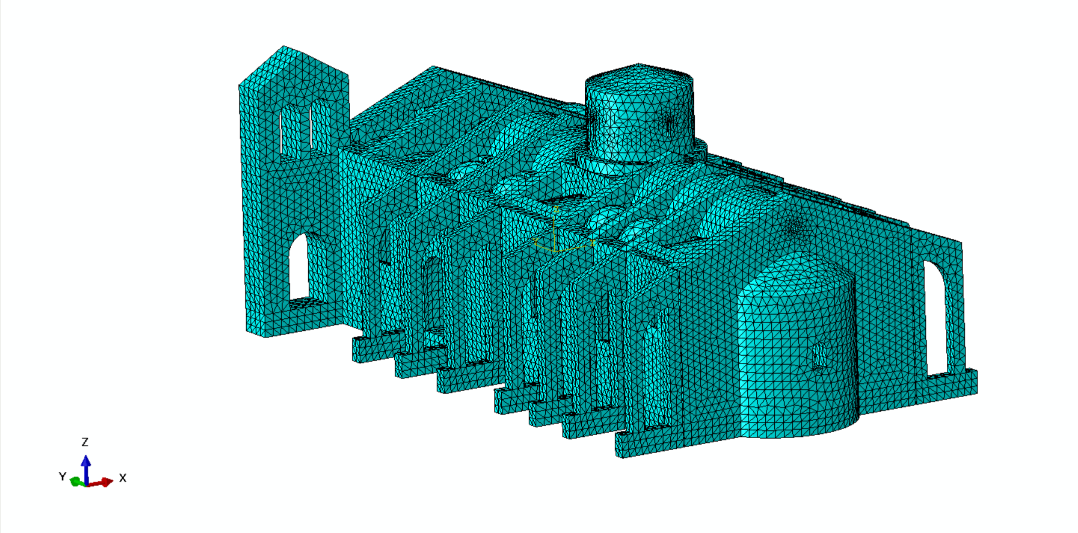


Buttress connection to the existing structure

Material mechanical properties considered in the model

Material	Modulus of Elasticity (GPa)	Poisson's ratio	Density (Self-weight) (Mg/m^3)
Grouted masonry	1.50	0.25	2.00
Struts-Stone-dressed RC	21.00	0.25	2.50

NUMERICAL MODEL – STRENGTHENED STRUCTURE - ANALYSES (2/2)



- 343803 nodes, 206776 3-D finite elements, and 1031409 degrees of freedom
- Modal analysis
- Vertical loads (G) – Elastic analysis
- Response spectrum dynamic analysis and time history dynamic analysis using triaxial acceleration records along X, Y, and Z directions – Elastic analysis
- 7 triaxial accelerograms for SD-Significant Damage Limit State
- 7 triaxial accelerograms for NC-Near Collapse Limit State

PROBABILISTIC SEISMIC HAZARD ANALYSIS – TIME HISTORY SELECTION – SD & NC LIMIT STATE (1/1)

- New time history selection based on the dynamic characteristics of the strengthened structure. Scaling according to EC8.
- For SD Limit State, the Katholikon can develop significant but repairable damage, corresponding to behaviour factor q=1.5.

SD Limit State Ground Motions

#	Database number	Seismic Event	Year	Station	M_w	Scaling Factor
1	3A.MZ29	Central_Italy 20161030_0000029	2016	Amatrice Caseificio storico - INGV	6.5	1.30
2	IT.AQV	L'Aquila	2009	L'Aquila - V. Aterno - Centro Valle	6.1	1.35
3	IT.MCV	Central_Italy 20161026_0000095	2016	Montecavallo	5.9	1.30
4	IT.PCB	Central_Italy 20170118_0000034	2017	Poggio Cancelli (Base Diga)	5.5	1.20
5	IV.T1214	Central_Italy 20161030_0000029	2016	Forca Canapine	6.5	–
6	RSN 240	Mammoth Lakes-04	1980	Convict Creek	5.7	1.30
7	RSN 619	Whittier Narrows-01	1987	Garvey Res. - Control Bldg	5.99	1.30

NC Limit State Ground Motions

#	Database number	Seismic Event	Year	Station	M_w	Scaling Factor
1	3A.MZ102	Central_Italy	2016	Accumoli Madonna delle Coste -ENEA	6.5	–
2	IV.T1299	Central_Italy	2016	Amatrice, Casale Bucci	6.5	1.20
3	RSN 139	Tabas, Iran	1978	Dayhook	7.35	1.10
4	RSN 150	Coyote Lake	1979	Gilroy Array #6	5.74	1.15
5	RSN 802	Loma Prieta	1989	Saratoga – Aloha Ave	6.93	1.20
6	RSN 982	Northridge-01	1994	Jensen Filter Plant Administrative Building	6.69	1.10
7	RSN 1013	Northridge-01	1994	LA Dam	6.69	1.35

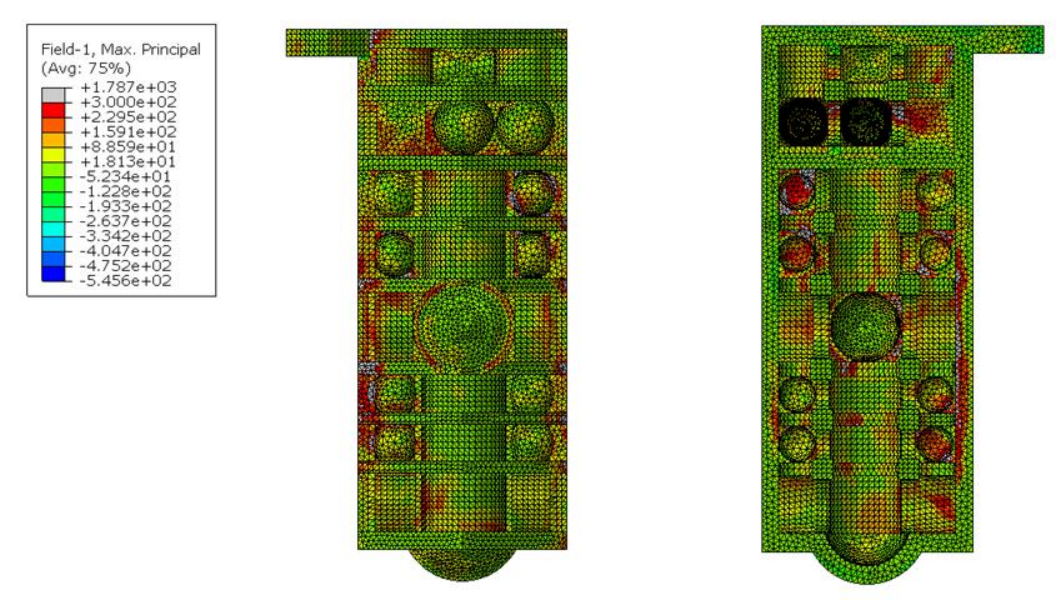
Results – SD Limit State – Masonry (1/3)

- Failure criterion: Masonry principal tensile stresses from $G+E_i/q$ load combination
- Grouted masonry tensile strength = 0.3 MPa
- For each triaxial ground motion, the envelope of principal compressive and tensile stresses is calculated (non-simultaneous values)
- Mean values of the principal tensile σ_I and compressive σ_{II} stresses envelopes of the 7 ground motions
- The principal stresses due to vertical loads are added to the mean values

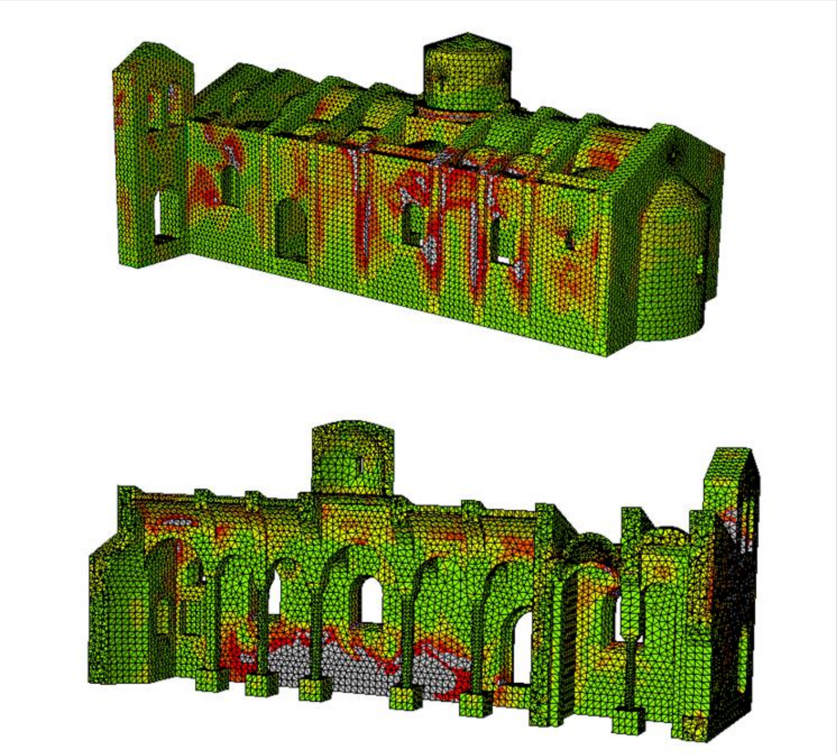
$$\overline{\sigma}_I = \frac{1}{7}\sum_{i=1}^{7} \sigma_{I,E_i/q}$$

$$\overline{\sigma}_{II} = \frac{1}{7}\sum_{i=1}^{7} \sigma_{II,E_i/q}$$

RESULTS – SD LIMIT STATE – MEAN VALUE OF PRINCIPAL TENSILE STRESSES ENVELOPE (2/3)



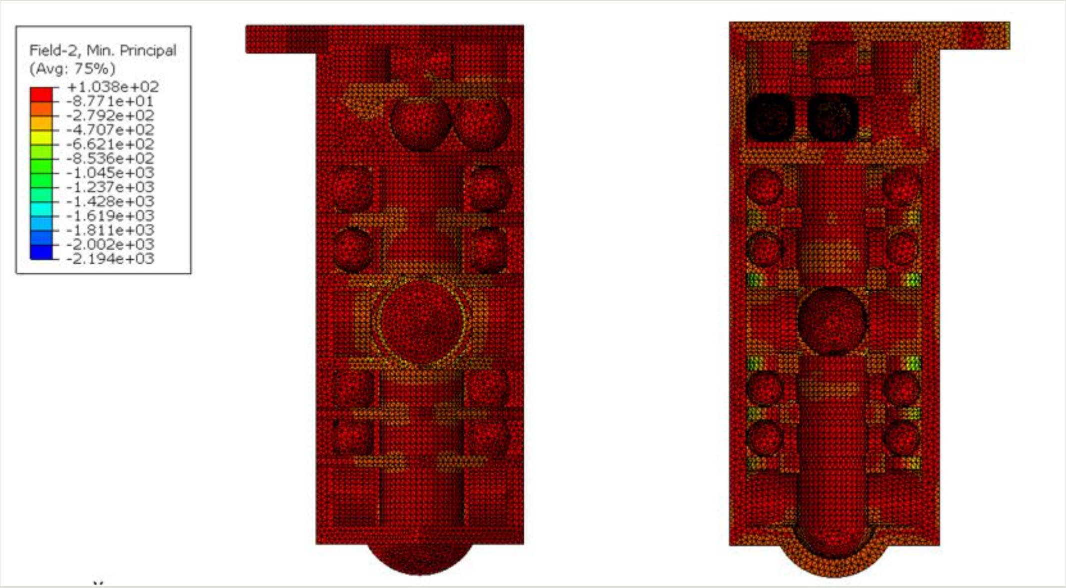
Bottom View – Top View



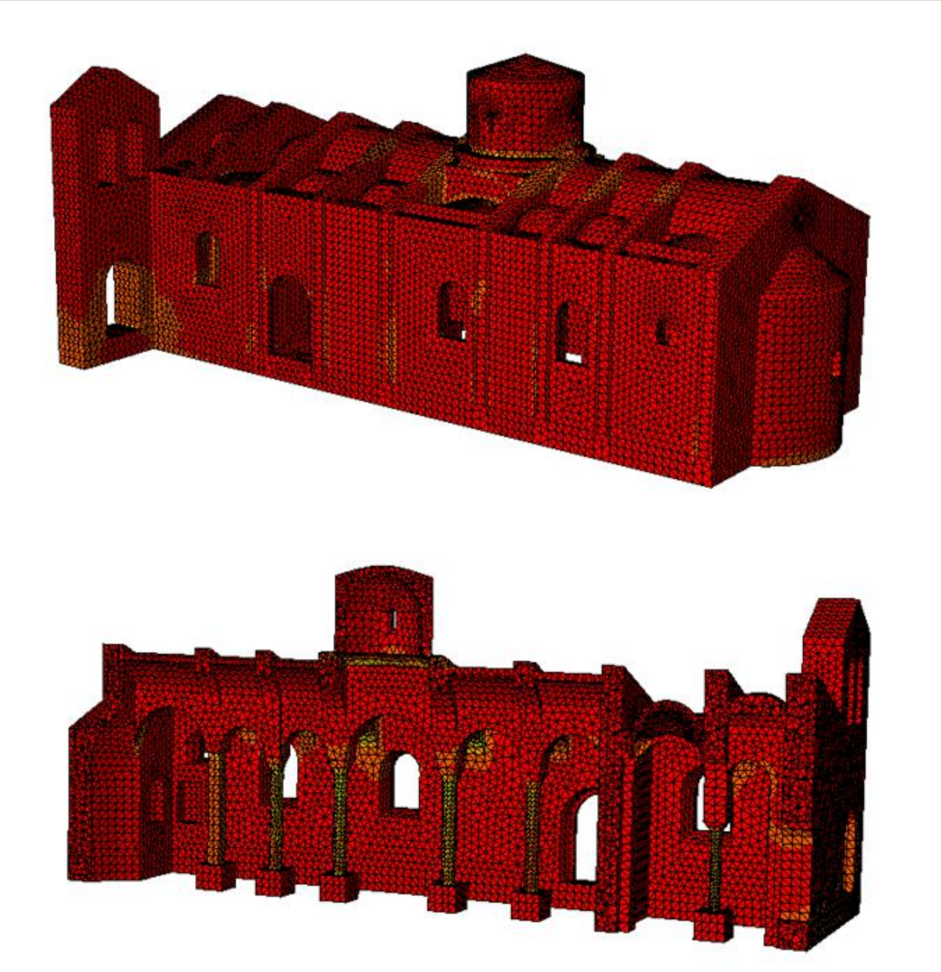
North Wall

No extensive damage of the strengthened masonry was observed

Results – SD Limit State – Mean Value of Principal Compressive Stresses Envelope (3/3)



Bottom View – Top View

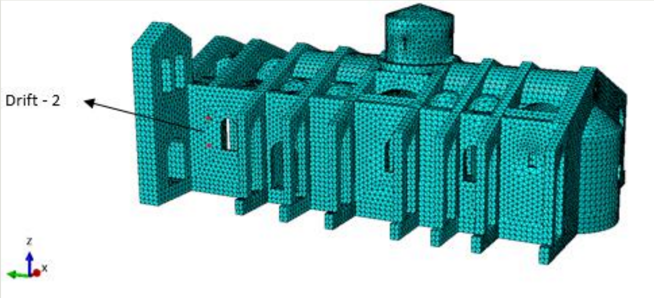


North Wall

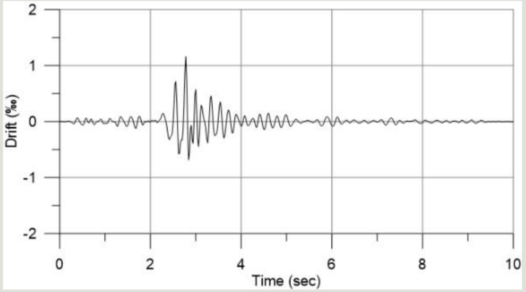
No compressive failures were observed

Results – NC Limit State – Interstorey Drifts (1/1)

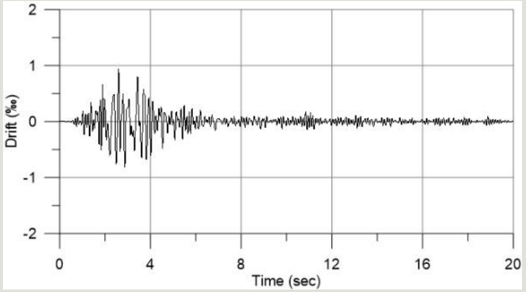
- For NC Limit State, the Katholikon sustains extensive, irreparable damages, which narrowly will not lead to its collapse.
- Masonry interstorey drift checks were carried out at 10 locations due to the G+E load combination.



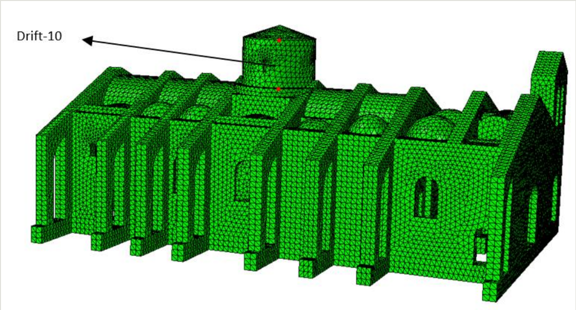
Location 2



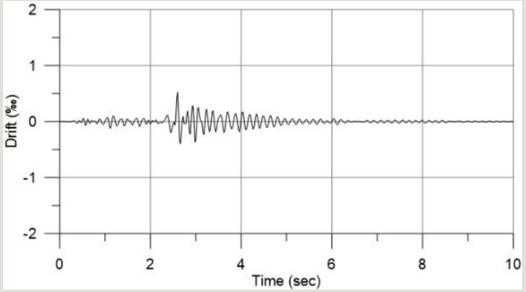
Ground Motion 1



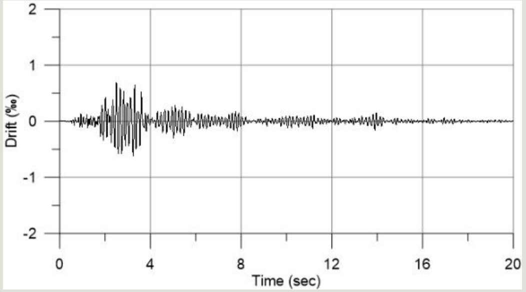
Ground Motion 2



Location 10



Ground Motion 1



Ground Motion 2

No exceedance of the interstorey drift limit was observed

2020 Study Conclusions (1/1)

The analysis results show that the construction of the proposed struts results in the following:

1. Limitation of both in-plane and out-of-plane displacements of the north and south walls.
2. The developed principal tensile and compressive stresses of masonry after global grouting lead to limited cracking of masonry.
3. Stresses and displacements of strut masonry are within allowable limits.
4. The ties which will connect the north and south walls will, in case of cracking, hinder their out-of-plane deviation.

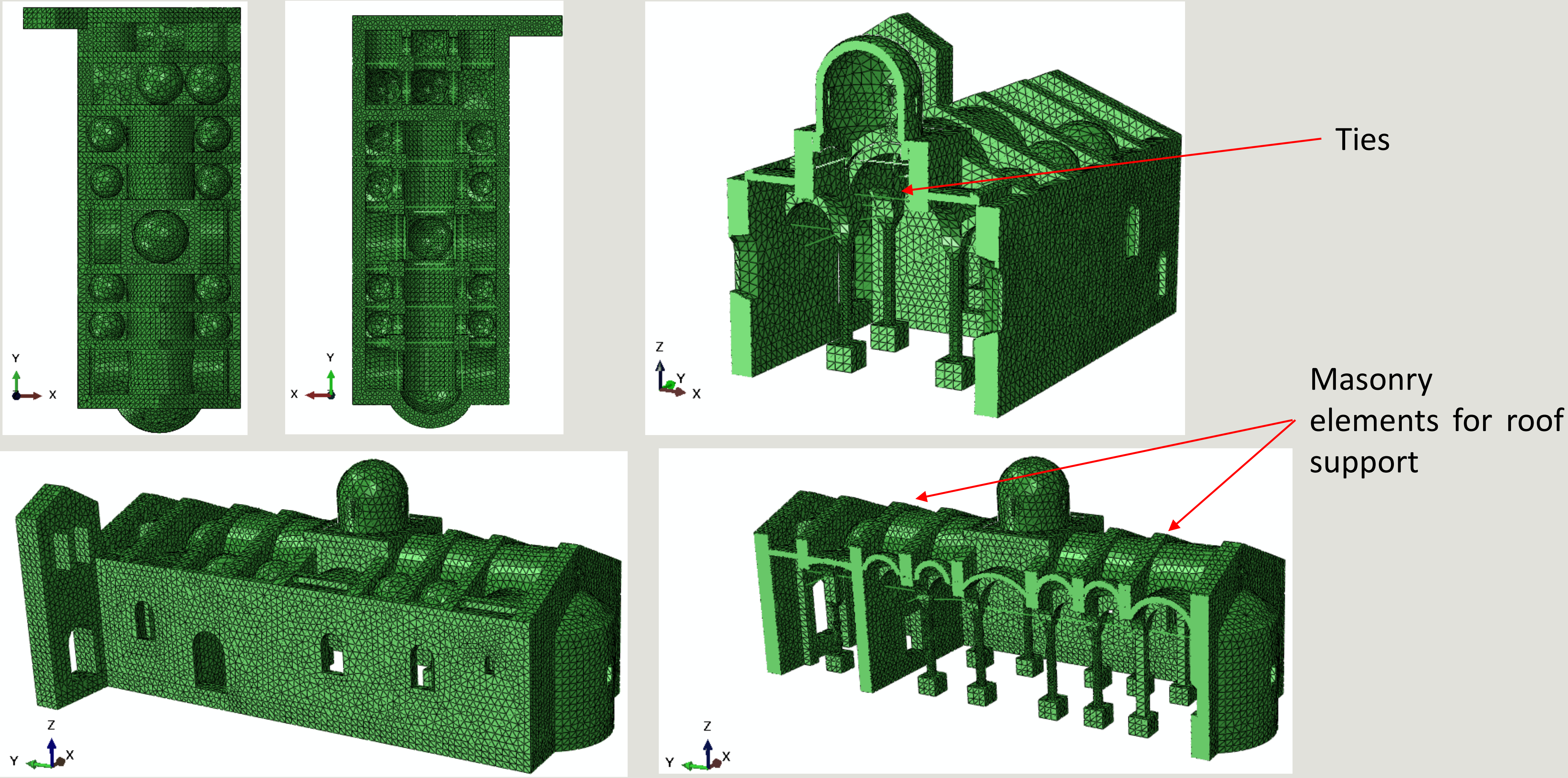
At the time of this study (2020), the opening of the roof and the investigation of the area between dome structure and roof was not possible. The findings after that investigation led to modifications of the rehabilitation design.

Rehabilitation Design 2021 – Final Measures (1/1)

2021: After the opening of the roof, the studied model was modified. The following interventions are proposed:

1. Global grouting of all Katholikon masonry and domes.
2. Installation of steel struts-ties in the arches.
3. Instead of placing stones over the timber structures, the roof will be reconstructed using plywood of 22mm thickness, onto which byzantine tiles will be screwed.
4. The plywood will be placed over GL24h spruce glulam beams of 8x16 cross-section.
5. The timber elements will be supported by reconstructed pediments over the arches; MasterEmaco S 285 TIX will be used as mortar.
6. Struts-ties will be installed at the base of the arches along both directions in the interior of the temple. The struts shall consist of rectangular stainless steel (AISI 304) hollow section 100x100x10, while the struts of circular stainless steel (AISI 304) bar of 30mm diameter.
7. The interior and exterior of the domes will be lined with stainless steel meshes in MasterEmaco S 285 TIX of 50mm thickness, as well as the interior of the sanctuary apse.
8. Installation of in-plane steel tie in the sanctuary apse.
9. Stainless steel hoops of 50mm thickness/500mm along the height of the columns, taking into account the crack patterns of the columns.
10. Concrete tie beam at column foundations.

NUMERICAL MODEL – STRENGTHENED STRUCTURE (1/3)

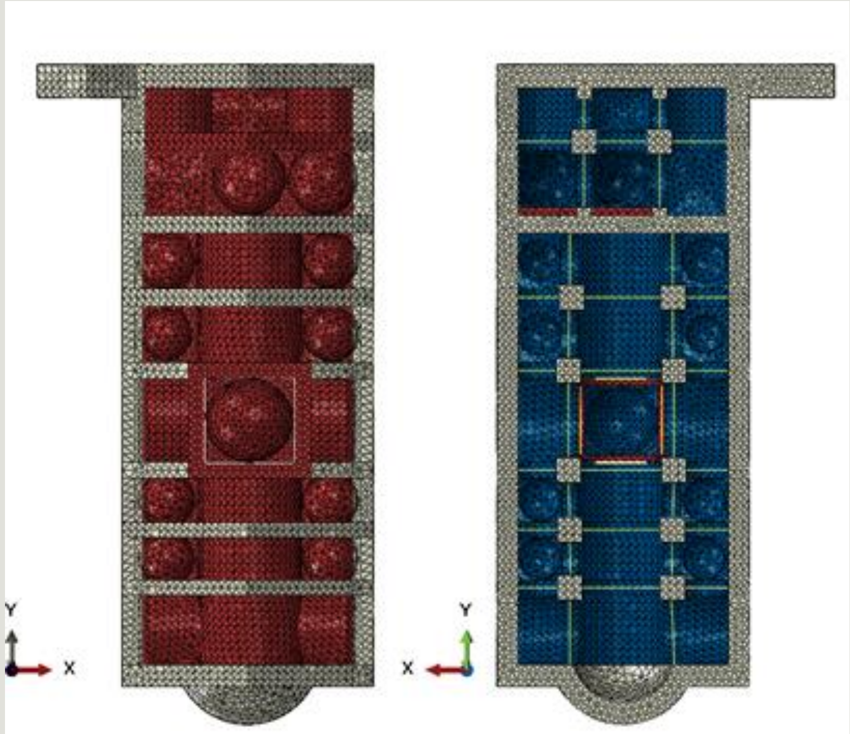


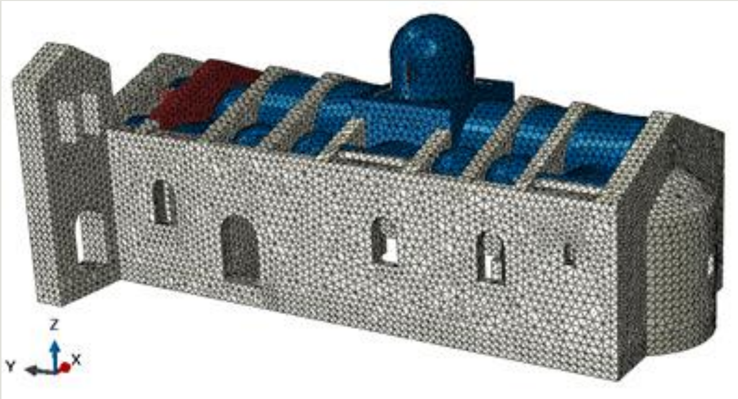
NUMERICAL MODEL – STRENGTHENED STRUCTURE – MATERIALS (2/3)

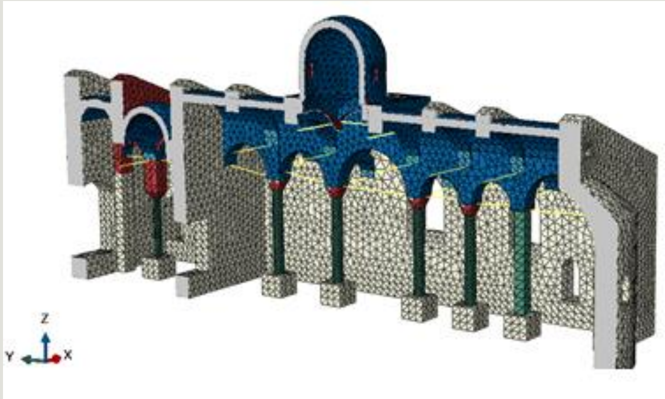
Material mechanical properties

Material	Modulus of Elasticity (GPa)	Poisson's ratio	Density (Self-weight) (Mg/m^3)	
Marble	30.0	0.30	2.00	
Porolith	1.0	0.25	1.50	
Jacket-strengthening	15.0	0.25	2.40	
Steel (ties)	210.0	0.30	7.80	
Strengthened masonry*	3.00	0.30	2.00	

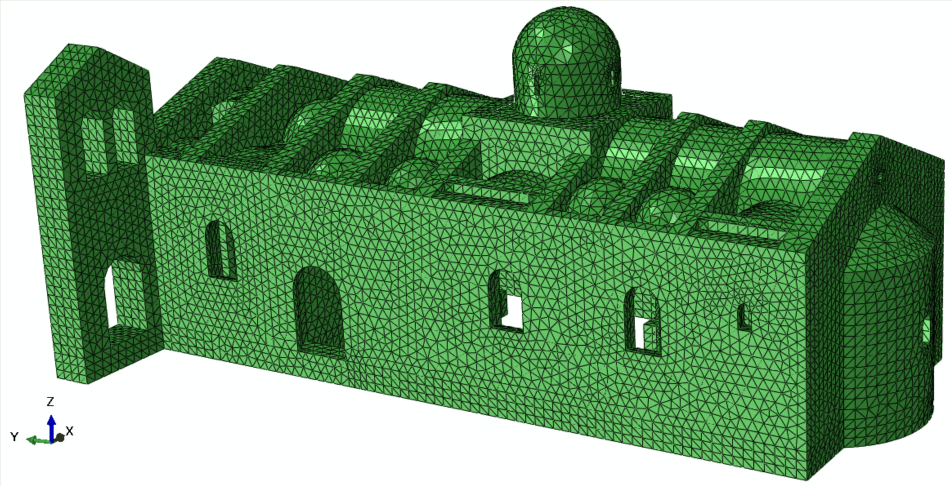
*non-linear behaviour







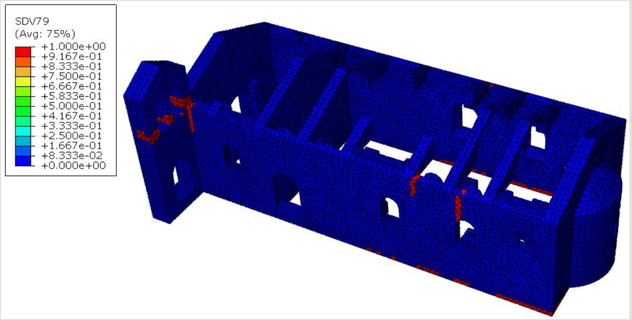
Numerical Model – Strengthened Structure – Analyses (3/3)



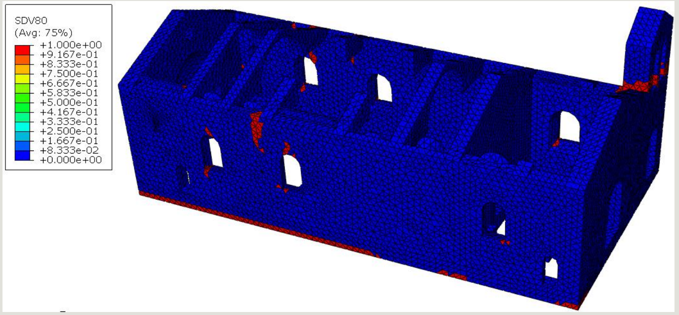
- 44834 nodes, 200753 3-D finite elements, and 134502 degrees of freedom
- Modal analysis – Elastic analysis – Checks
- Vertical loads (G) – Non-linear analysis
- Explicit integration Dynamic analysis using triaxial acceleration records along X, Y, and Z directions – Non-linear analysis
- 7 triaxial accelerograms for SD-Significant Damage Limit State
- 7 triaxial accelerograms for NC-Near Collapse Limit State

Results – SD Limit State – Ground Motion E_1 – Masonry (1/3)

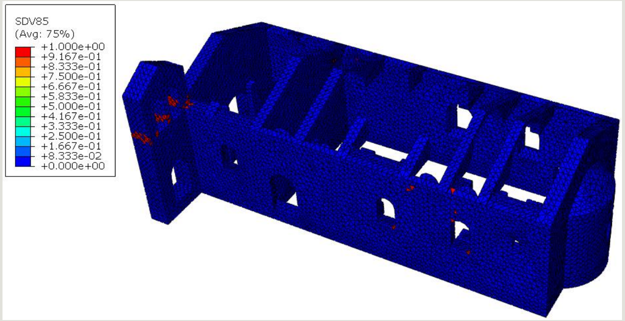
SD Limit State checks on equivalent strain and final cumulative damage indices for the Katholikon masonry (inelastic behaviour) and on stresses for the dome structure, the strengthening jacket and the ties και σε επίπεδο τάσεων για τη θολοδομία, το μανδύα ενίσχυσης της θολοδομίας και τους ελκυστήρες (elastic behaviour).



Tensile failures along X direction



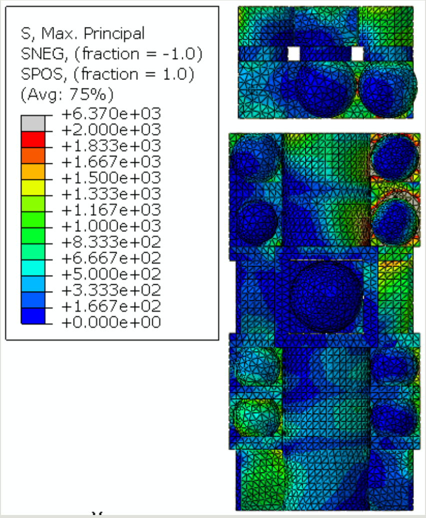
Tensile failures along Y direction



Shear failures in XY plane

- No extensive masonry damage was observed.
- Limited damages in tension and shear.
- No damage in compression.

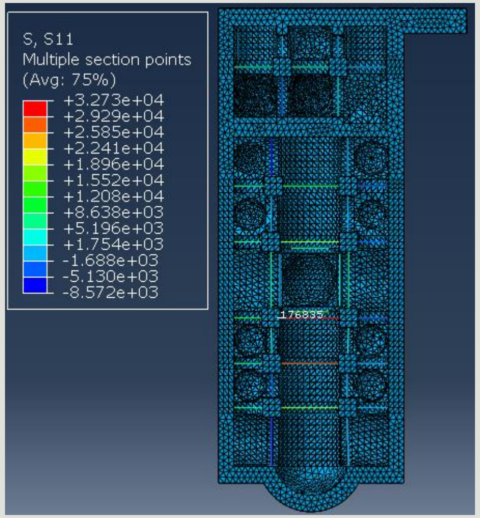
Results – SD Limit State – Ground Motion E_1 – Jacket, Porolith, Ties (2/3)



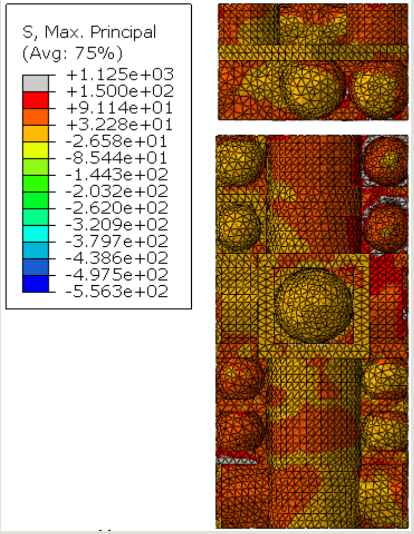
Jacket (elastic behaviour)

- Upper limit:

  dome jacket tensile strength = 2.0 MPa
- No tensile strength exceedance observed over a large area and the entire jacket thickness.



σ_{11} stress distribution in ties



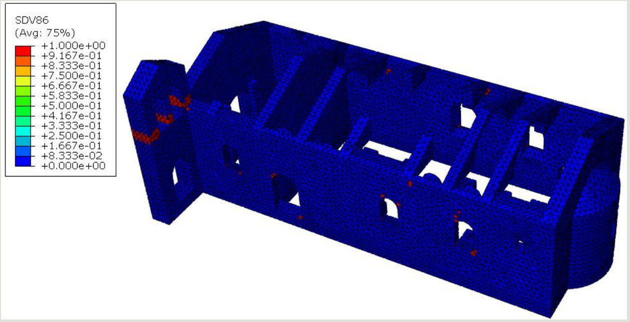
Max tensile stresses distribution

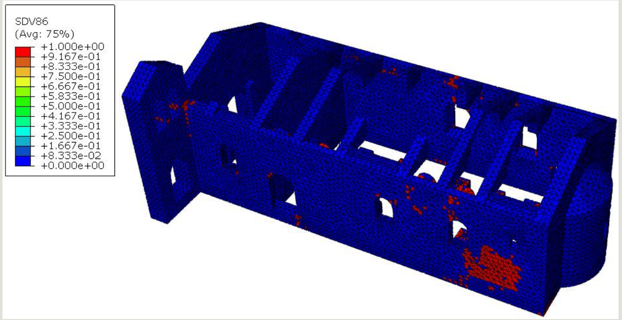
Porolith (elastic behaviour)

- Tensile strength = 150 kPa
- No tensile strength exceedance observed over a large area and the entire porolith thickness.

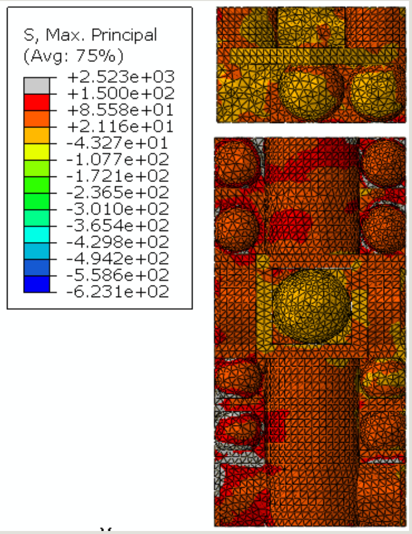
Results – SD Limit State – Ground Motion E_1 – With and Without Jacket (3/3)

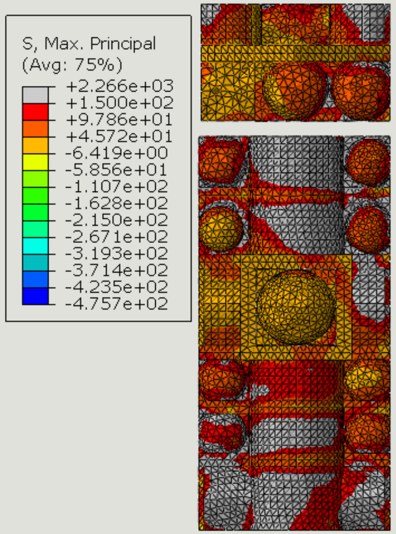
The **absence of a jacket** leads to: 1) more extensive tensile and shear damage in the Katholikon masonry and 2) exceedance of the porolith tensile strength over a large area and in the entire dome structure thickness.





Shear failure in YZ plane: with jacket without jacket

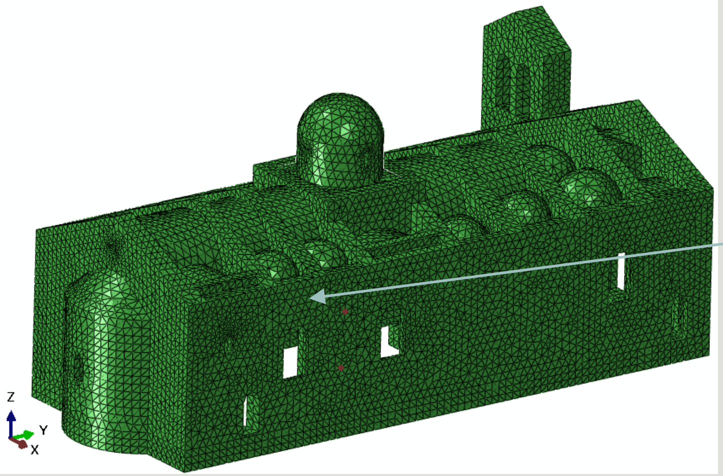




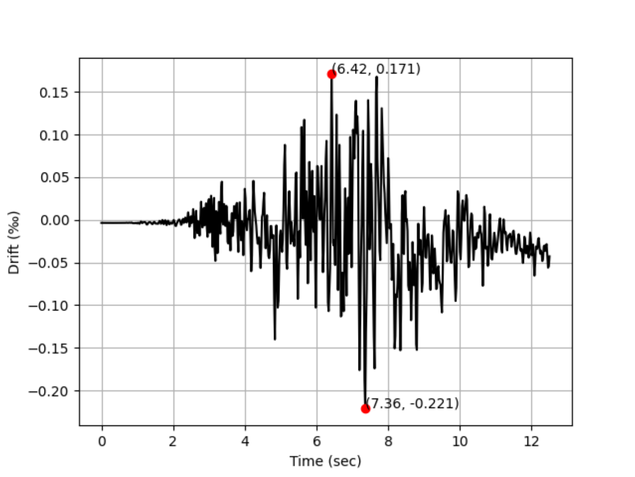
Max principal tensile stresses: with jacket without jacket

RESULTS – NC LIMIT STATE – GROUND MOTION E_1 (1/1)

- Extensive and irreparable damages, collapse avoidance
- Failure criterion: masonry interstorey drifts due to G+E load combination
- Interstorey drift limit 1‰
- No exceedance of interstorey drift limit



Location 17-Y direction



Conclusions (1/1)

- Global grouting of the Katholikon masonry and installation of ties, for PGA 0.24 g and behaviour factor q=1.5, results in the following:
    1. Limitation of both the in-plane and out-of-plane displacements of the north and south walls. For NC Limit State, the interstorey drifts of these walls are within allowable limits.
    2. The principal tensile and compressive stresses in the grouted masonry lead to its limited cracking.
    3. The ties hinder the out-of-plane deviation between the north and south walls in case of cracking.

- The suggested interventions are adequate for the enhancement of the seismic behaviour of the Katholikon of the Varnakova Holy Convent for earthquakes of 0.24 g PGA with the occurrence of repairable damages.

- In the case of the maximum considered earthquake, significant damages will appear in the masonry and the dome structure. The analysis shows that the interstorey drifts are within allowable limits; therefore, masonry collapse will not occur.

Thank you for your attention