POSSIBLE SOLUTIONS BEFORE AND IMMEDIATELY AFTER EARTHQUAKE.

CONCLUSIONS AND RECOMMENDATIONS FOR MAINTAINING AND INTERVENTIONS

SPEAKER: PROF. CLAUDIO MODENA

DEPARTMENT OF STRUCTURAL & TRANSPORTATIONS ENGINEERING
UNIVERSITY OF PADOVA, ITALY

OUTLINE OF THE PRESENTATION




SUMMARY

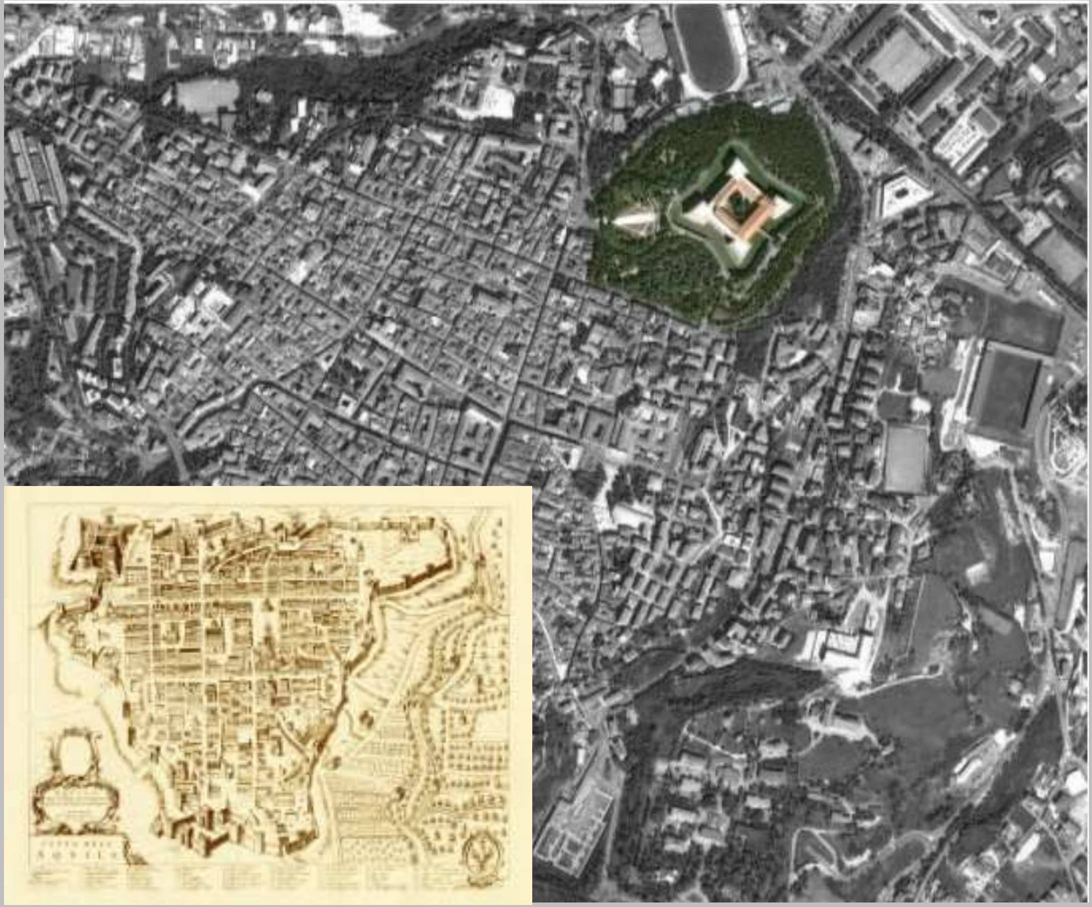
- THE SPANISH FORTRESS - L'AQUILA
- S.MARCO CHURCH - L'AQUILA

- TOMB OF DAVID AND CENACULUM - JERUSALEM
- S. MARIA ASSUNTA CATHEDRAL - REGGIO EMILIA

THE SPANISH FORTRESS

Possible Solutions before and immediately after Earthquake

Prof. Claudio Modena



The Spanish Fortress

The Spanish Fortress is located in the north-east part of the city of L'Aquila. It is one of the most impressive Renaissance castles in Central and Southern Italy. In 1528 Viceroy Filiberto d'Orange ordered to build a fortress in the highest North spot of the city, according to the project of a famous Spanish architect, Don Pirro Aloisio Escrivà. The construction started in 1534

MAIN OBSERVED DAMAGES:

- OUT OF PLANE OVERTURNING OF THE LONGITUDINAL WALLS
- COLLAPSE OF THE UPPER PART OF THE MAIN FAÇADE AND OF THE ROOF
- SHEAR DAMAGES AND COLLAPSES IN THE TRANSVERSE WALLS
- DAMAGE TO ARCHES AND VAULTS
- LOCAL COLLAPSE OF FLOORS AND VAULTS

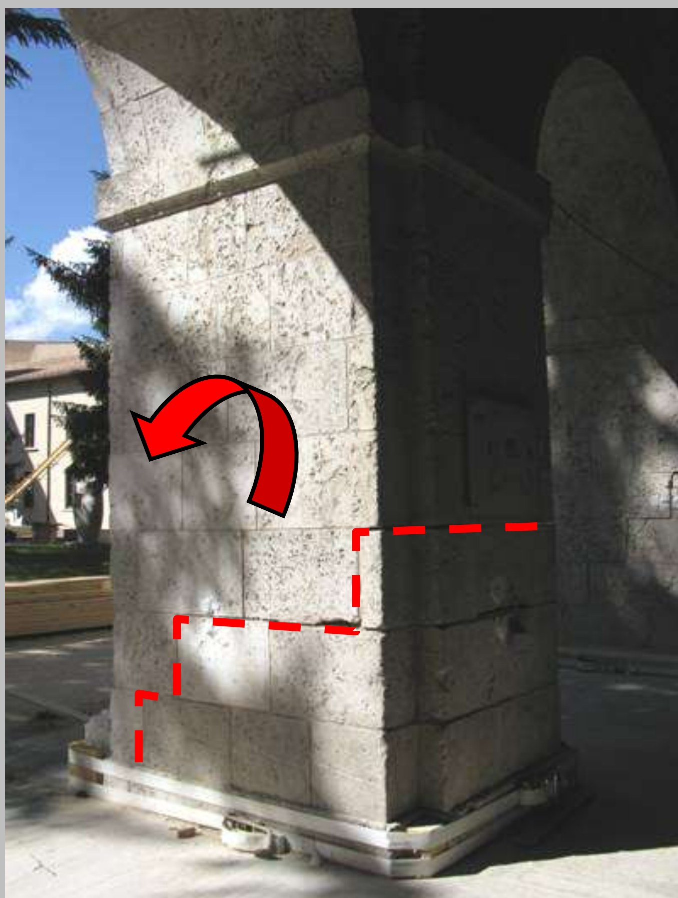
DAMAGE DESCRIPTION: S-E WING

Possible Solutions before and immediately after Earthquake

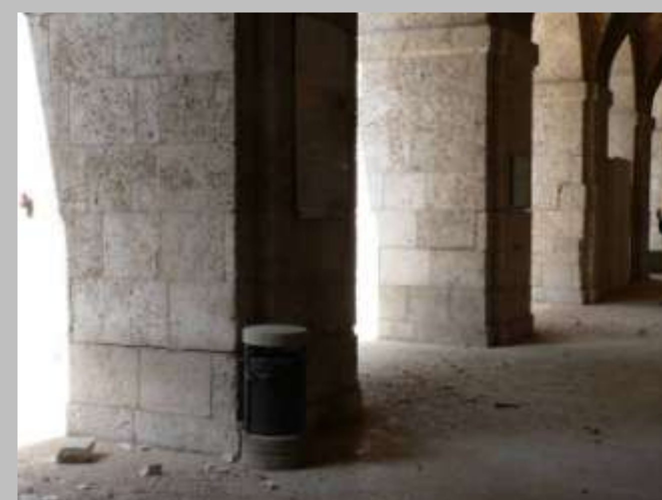
Prof. Claudio Modena

MAIN OBSERVED DAMAGES:

- OVERTURNING MECHANISM OF PILLARS
- DAMAGES ON PILLARS DUE TO CRUSHING



DAMAGE DESCRIPTION: S-E WING



MAIN OBSERVED DAMAGES:

- LOCAL COLLAPSES OF THE LONGITUDINAL WALLS
- DAMAGE TO ARCHES AND VAULTS

DAMAGE DESCRIPTION: S-E WING

MAIN OBSERVED DAMAGES:

- OVERTURNING MECHANISM OF THE LONGITUDINAL WALLS
- DAMAGES ON VAULTS
- SHEAR CRACKS ON TRANSVERSE WALLS
- DETACHMENT OF FLOORS AND TRANSVERSE WALLS FROM THE LONGITUDINAL FACADES

DAMAGE DESCRIPTION: S-W WING

IN SITU EXPERIMENTAL INVESTIGATIONS

UNIVERSITY OF PADOVA — DEPARTMENT OF STRUCTURAL AND TRANSPORTATION ENGINEEREING
PROF. ENG. C. MODENA, DR. ENG. F. CASARIN, ENG. F. LORENZONI, ENG. F. ZEN

POLITECNICO OF MILAN — DEPARTMENT OF STRUCTURAL ENGINEERING - MASONRY AND CULTURAL HERITAGE
PROF. ARCH. L. BINDA,, L. CANTINI, S. MUNDA, M. CUCCHI, M. ANTICO

ISTITUTO SUPERIORE PER LA CONSERVAZIONE ED IL RESTAURO — DR. ROBERTO CIABATTONI, DR. CARLO CACACE

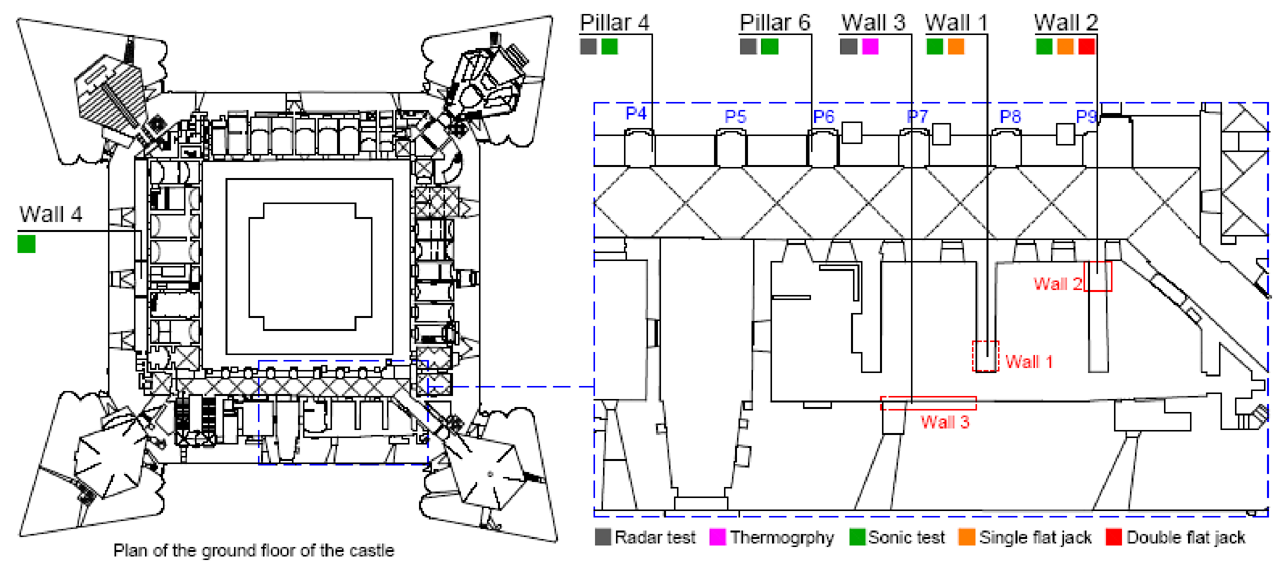
Experimental tests

- Dynamic identification tests
- Sonic tests
- Sonic tomography
- Active thermography
- Flat jack tests
- Radar tests

Structural health monitoring

- Control of the environmental parameters
- Static monitoring
- Dynamic monitoring

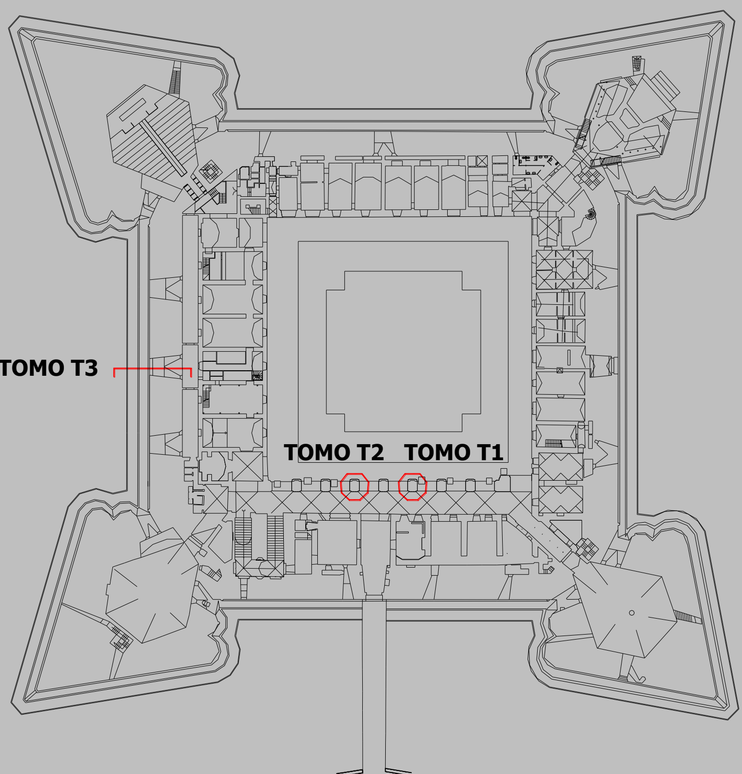
TESTS LAYOUT

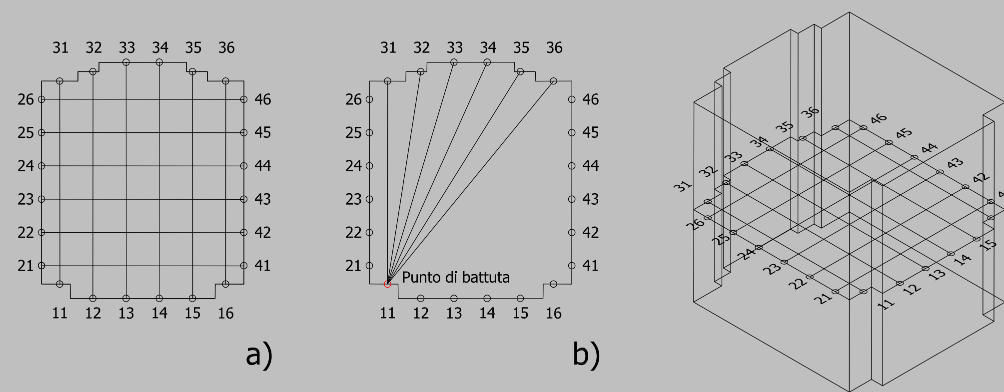


Plan of the ground floor of the castle

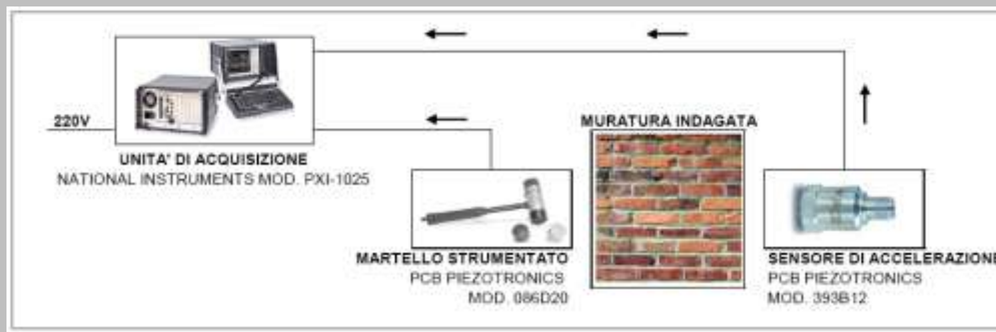
RESEARCH ACTIVITY

SONIC TOMOGRAPHY





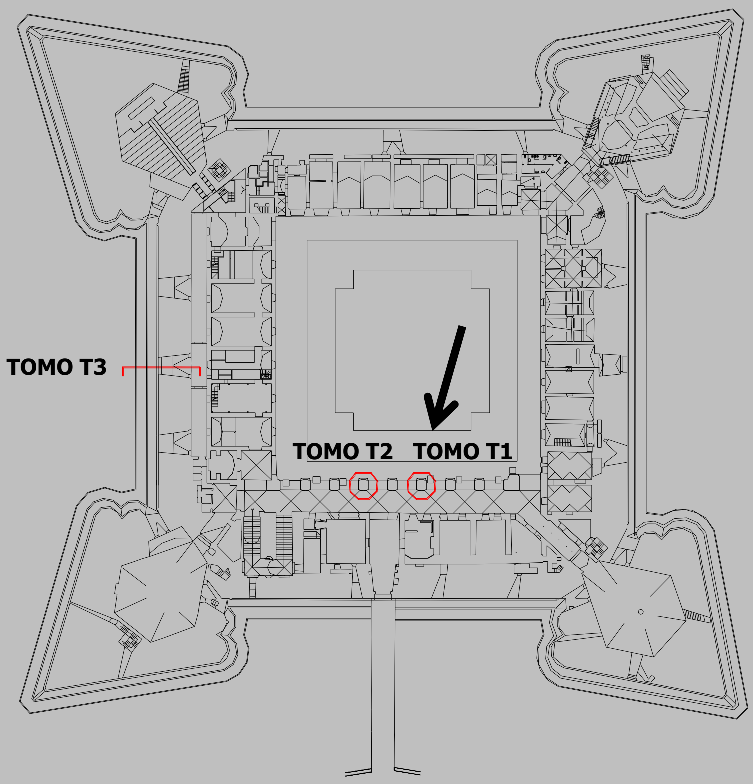
Griglia dei punti di acquisizione



RESEARCH ACTIVITY

SONIC TOMOGRAPHY

The sonic tests were performed on two pillars of the main wing of the fortress: one damaged and the other one not damaged by the earthquake.



RESEARCH ACTIVITY

SONIC TOMOGRAPHY

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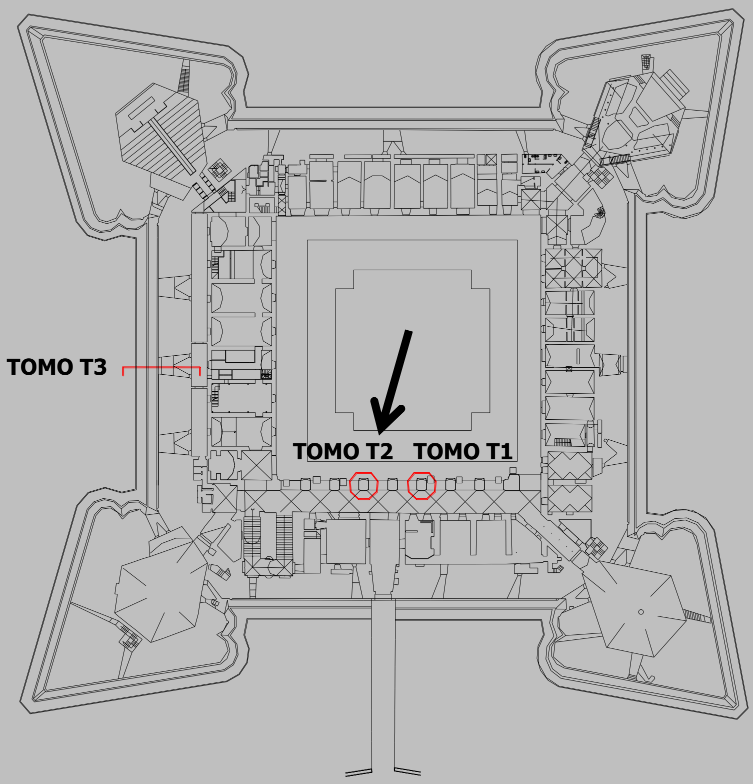
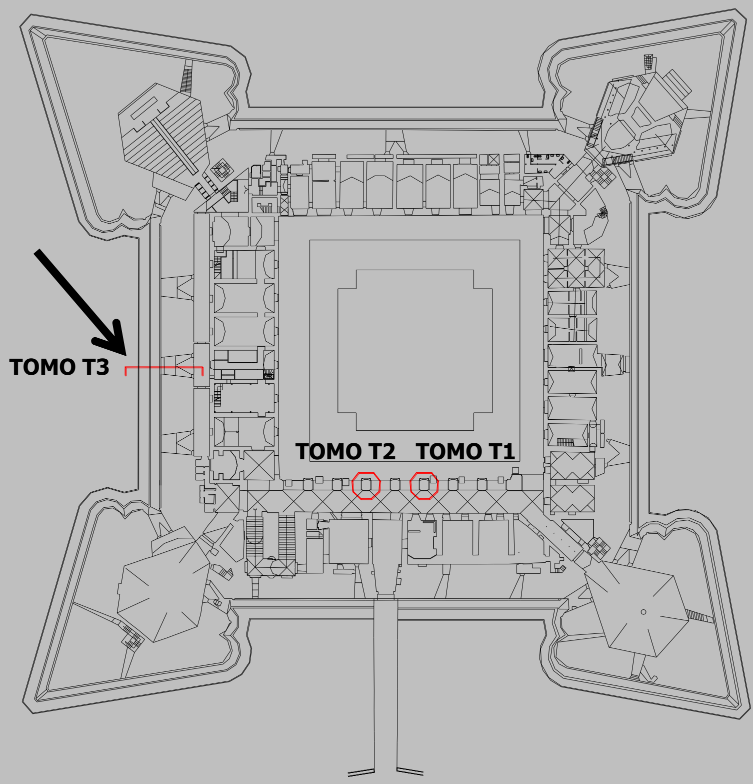


RESEARCH ACTIVITY

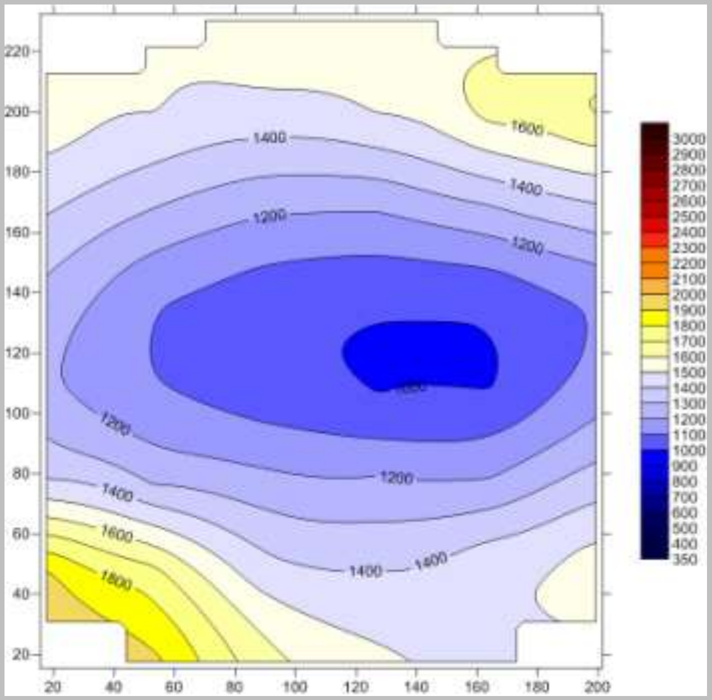
SONIC TOMOGRAPHY

The third tomography involved the vertical section of a masonry wall in the South-West wing of the. The aim was to collect qualitative data of masonry in this part of the castle that was heavily damaged by the earthquake.

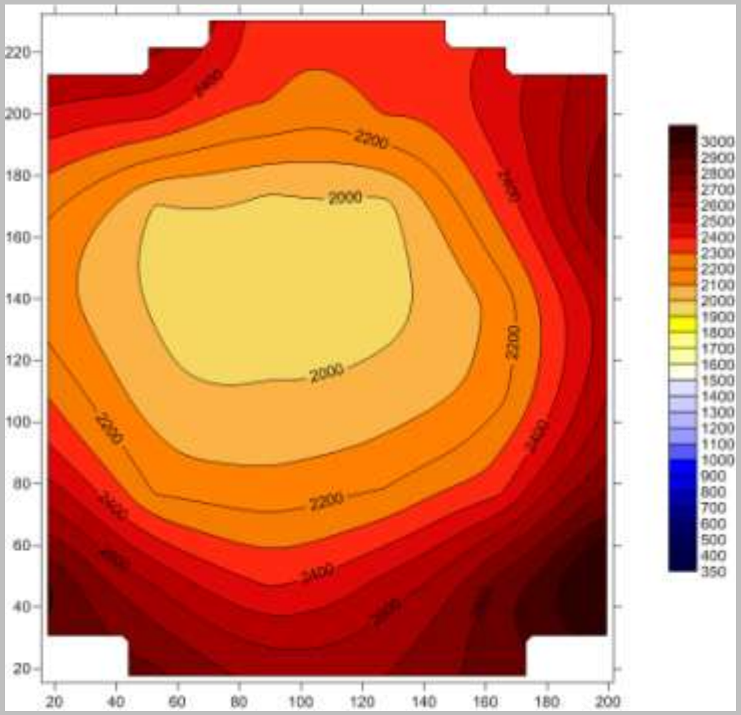


RESEARCH ACTIVITY

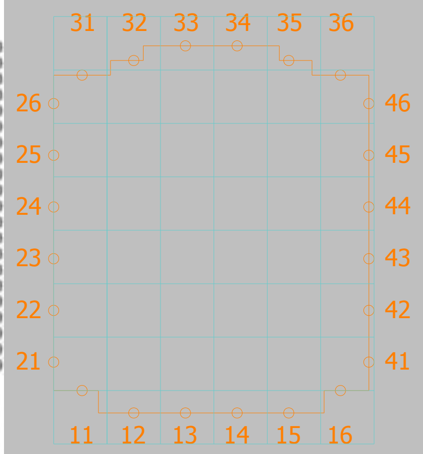
SONIC TOMOGRAPHY



TOMO 1: DAMAGED PILLAR

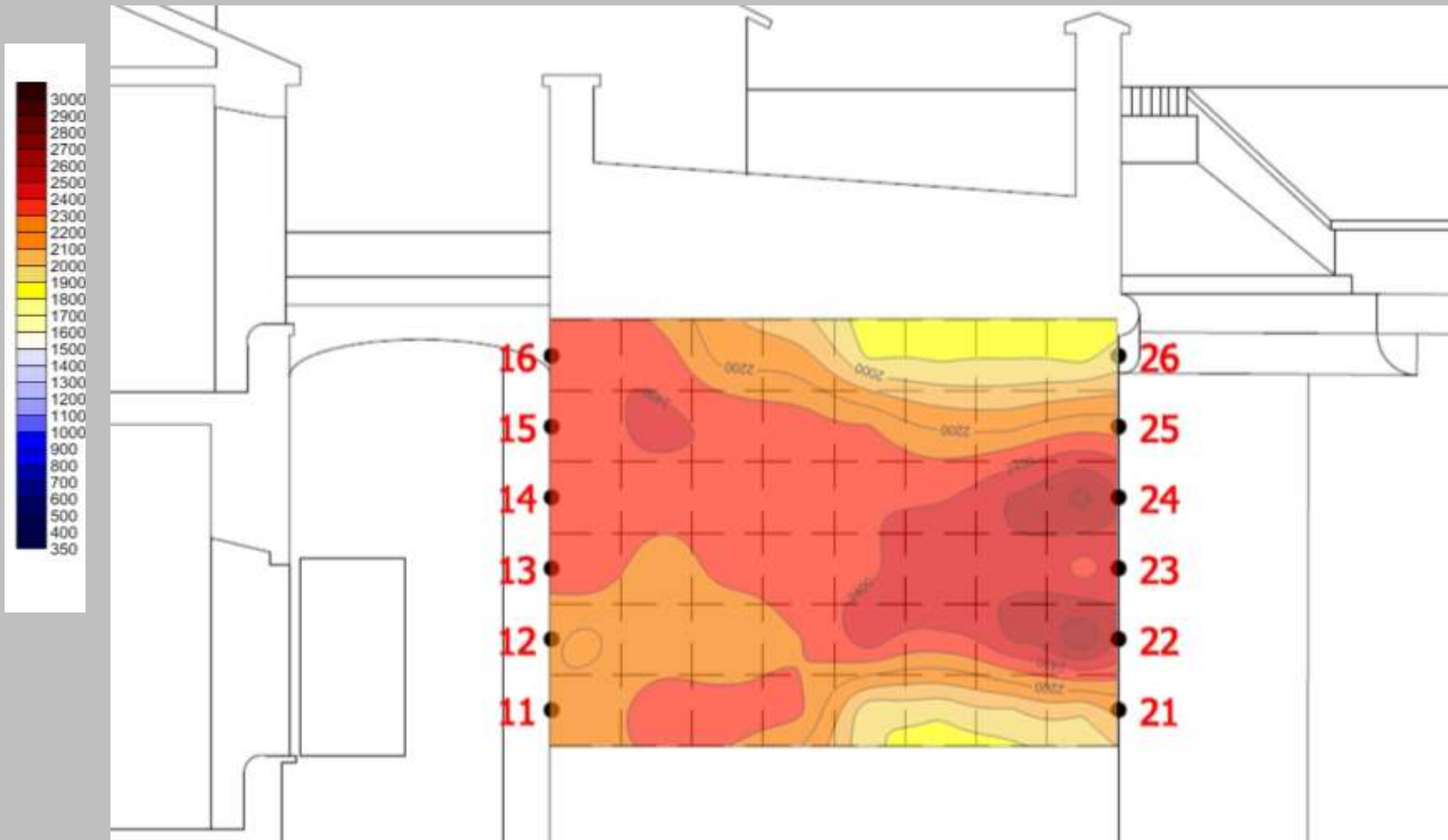


TOMO 2: UNDAMAGED PILLAR



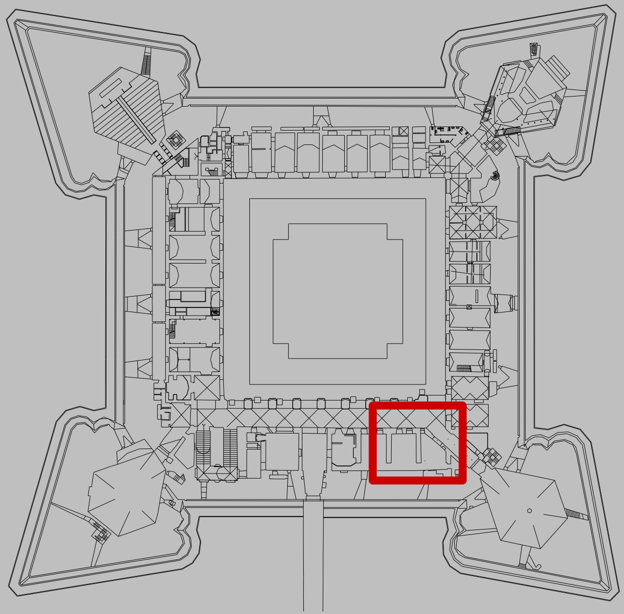
RESEARCH ACTIVITY

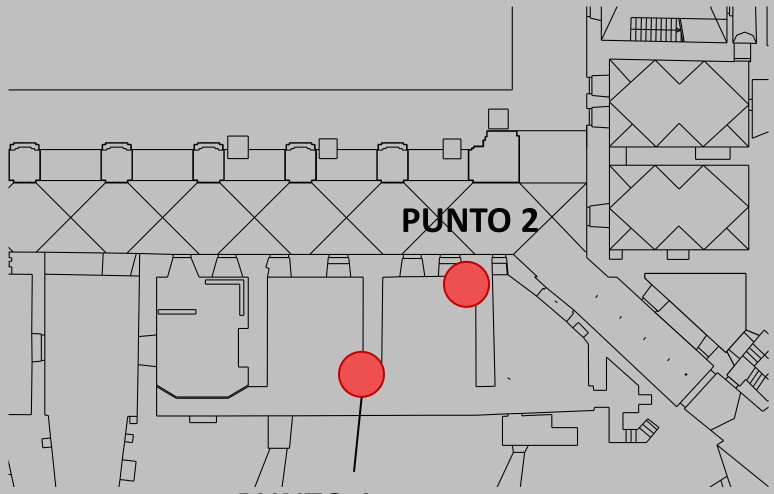
SONIC TOMOGRAPHY



RESEARCH ACTIVITY

SONIC TEST - FLAT JACK TEST





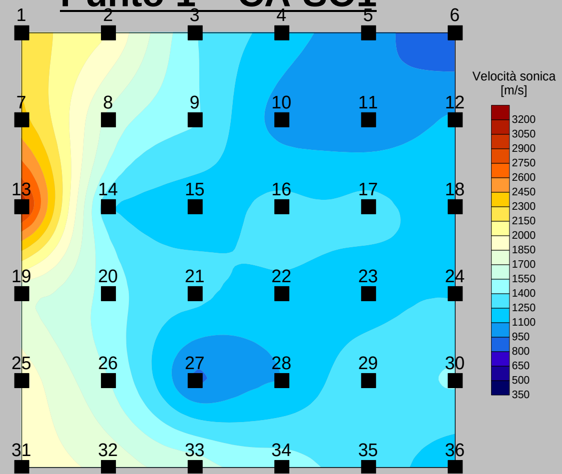
Sonic tests

Flat jack tests

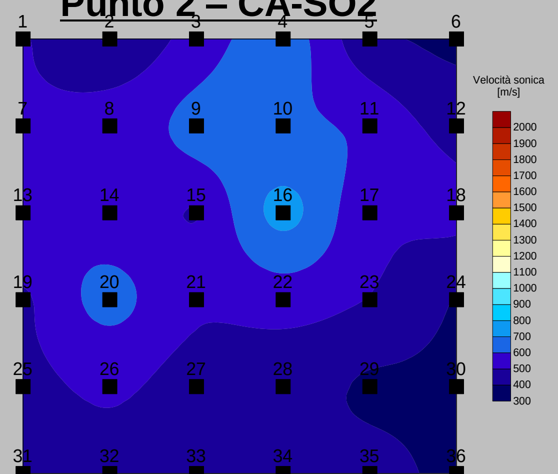
Prof. Claudio Modena

RESEARCH ACTIVITY

Punto 1 – CA-SO1



Punto 2 – CA-SO2



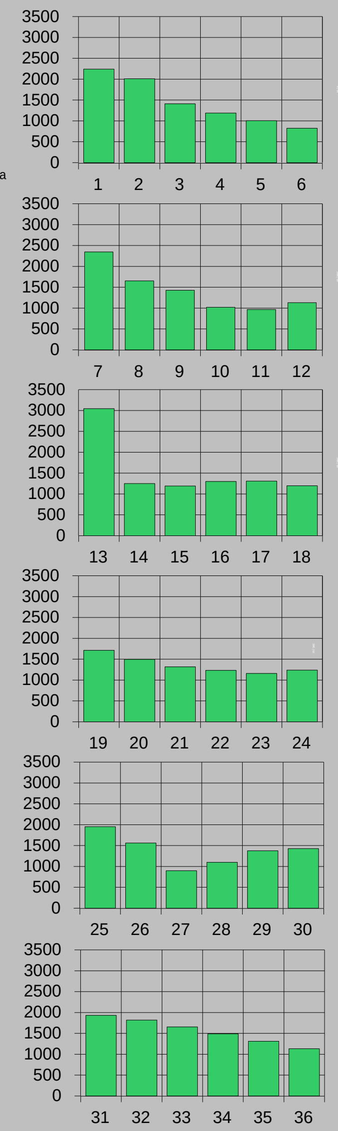
Punto 1 - Risultati in m/s

Velocità Max	punto 13	3046.41
Velocità min	punto 6	823.48
Velocità Media		1453.45
Deviazione Standard		454.11

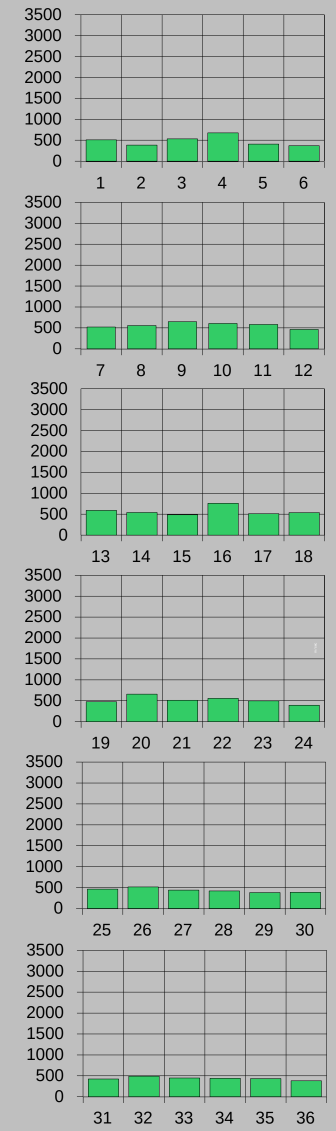
Punto 2 - Risultati in m/s

Velocità Max	Punto16	759.69
Velocità min	punto 6	371.30
Velocità Media		500.67
Deviazione Standard		93.30

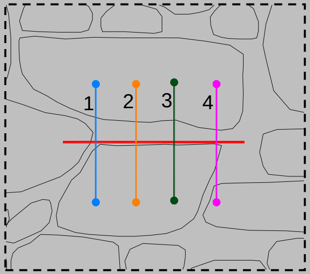
Punto 1 – CA-SO1



Punto 2 – CA-SO2



RESEARCH ACTIVITY



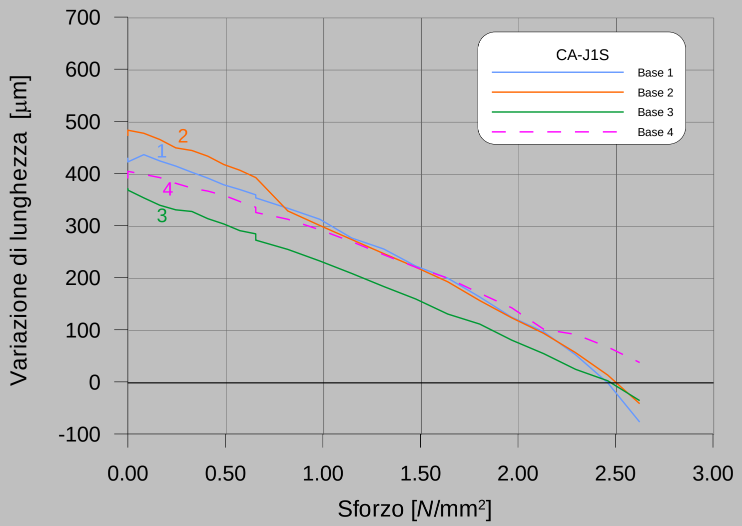
Point 1

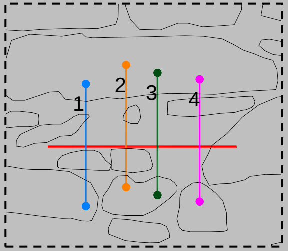
State of stress

↓

2.47 *N*/mm²

Single flat jack tests - CA-J1S-J2S



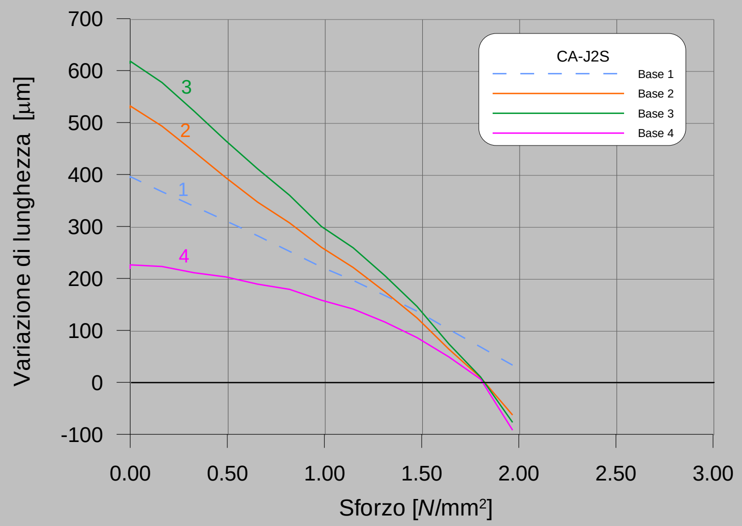


Point 2

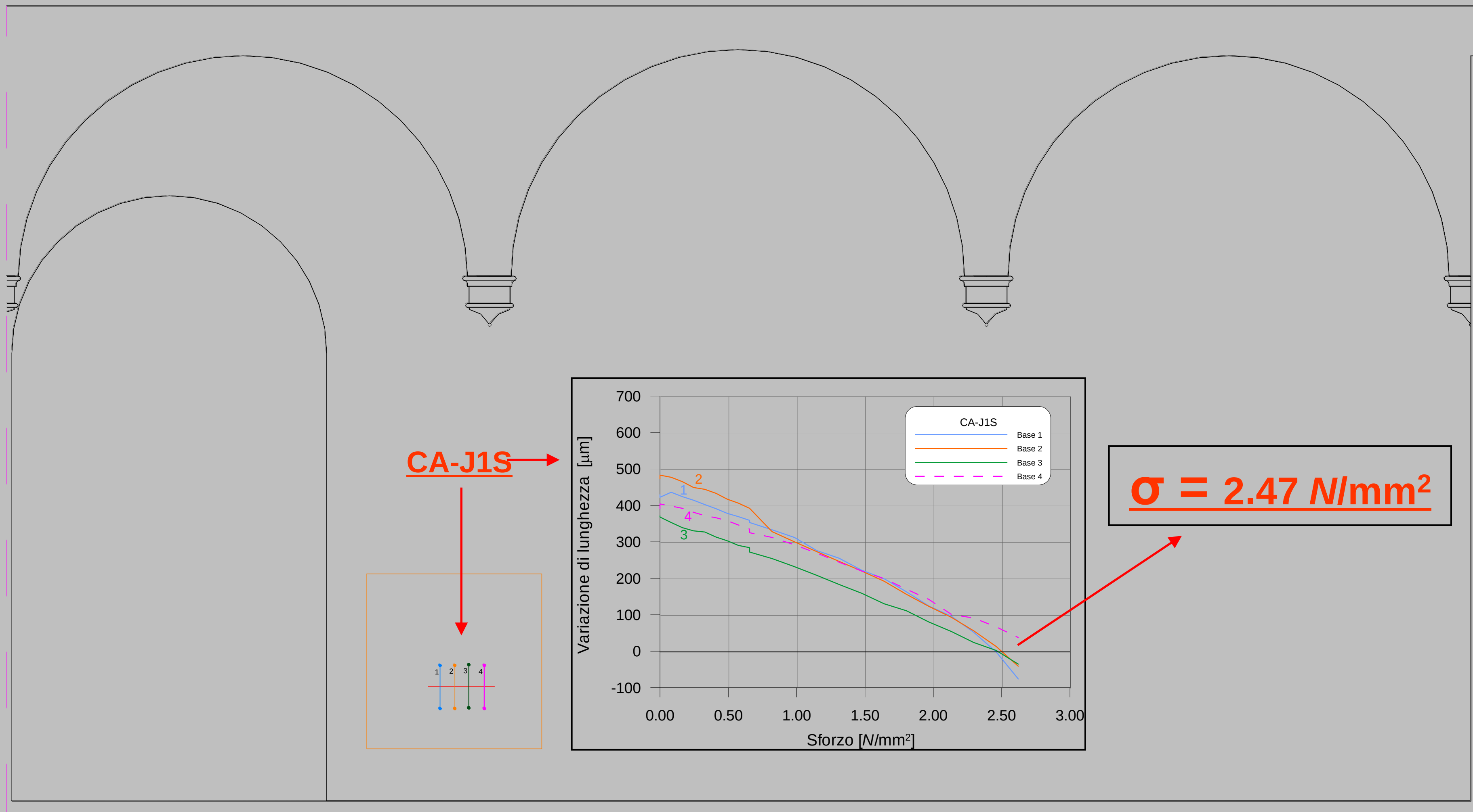
State of stress

↓

1.82 *N*/mm²

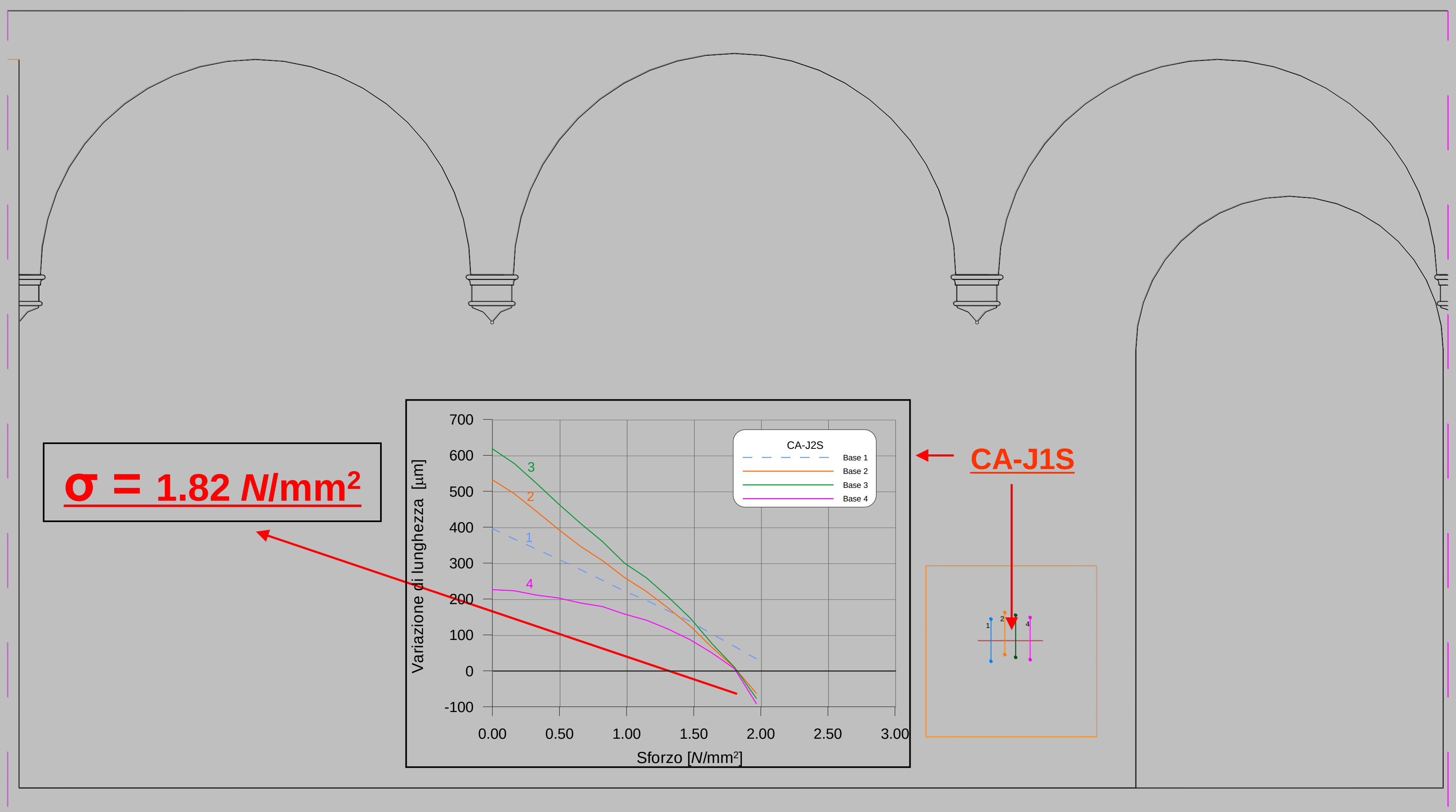


Prova con martinetto piatto singolo CA-J1S-Sotto il peduccio della volta



lato sala 3

Prova con martinetto piatto singolo CA-J2S di fianco all'apertura

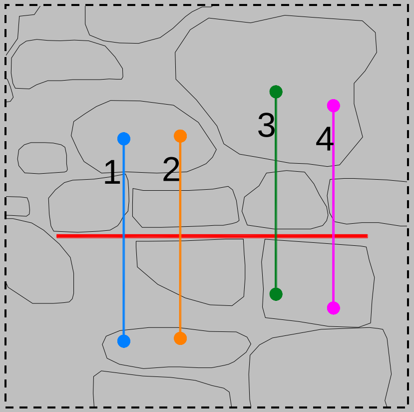


lato sala 2

RESEARCH ACTIVITY

Single flat jack test - point 3



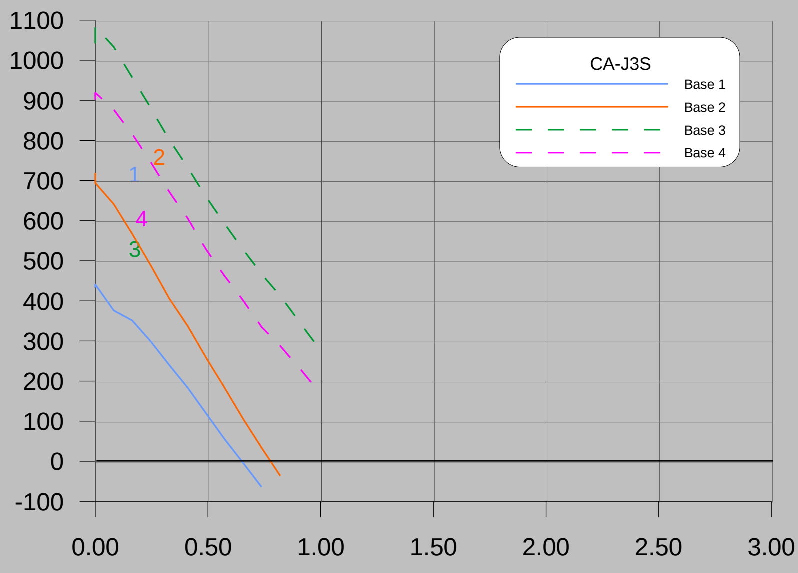


Point 3

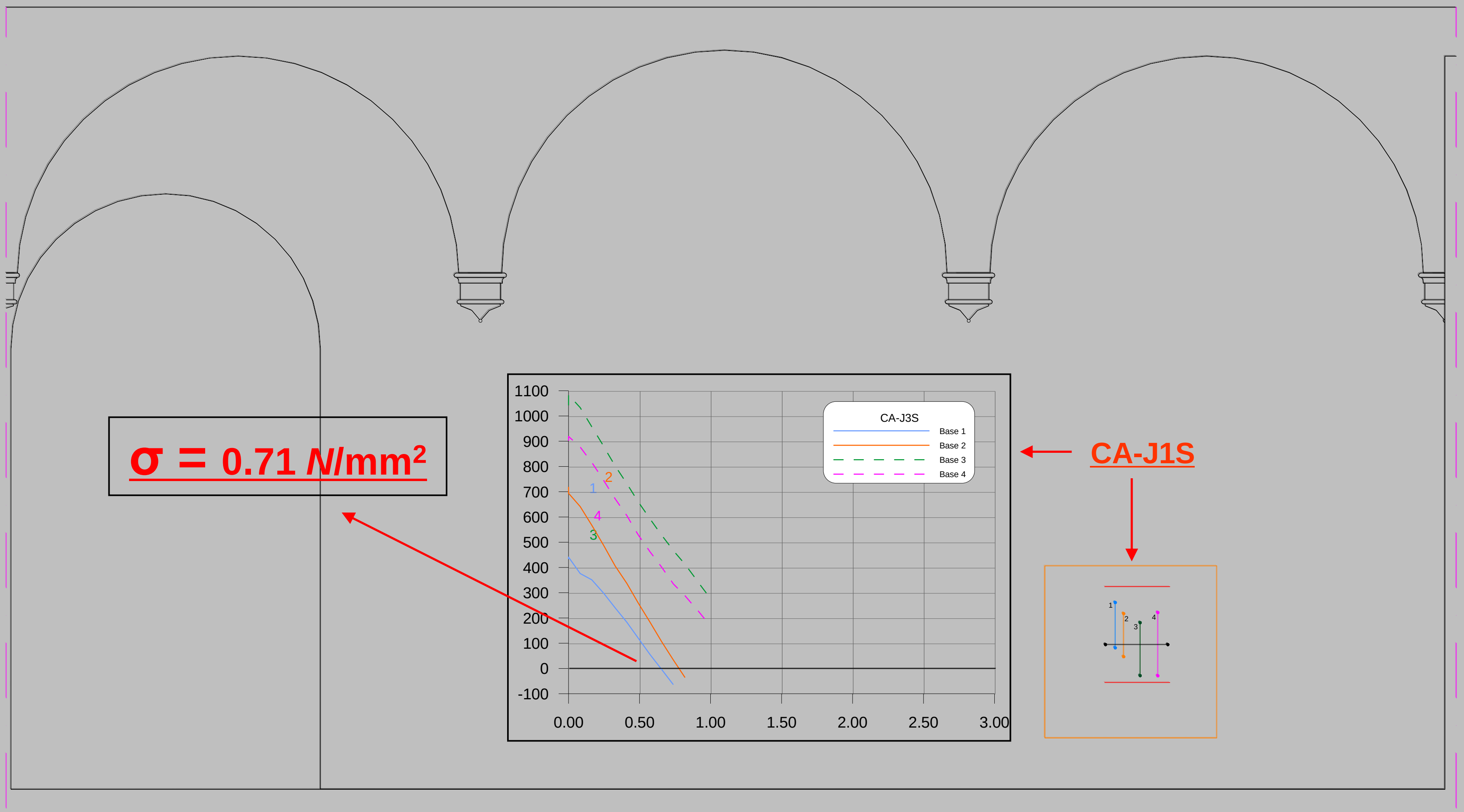
State of stress

↓

0.71 *N*/mm^2



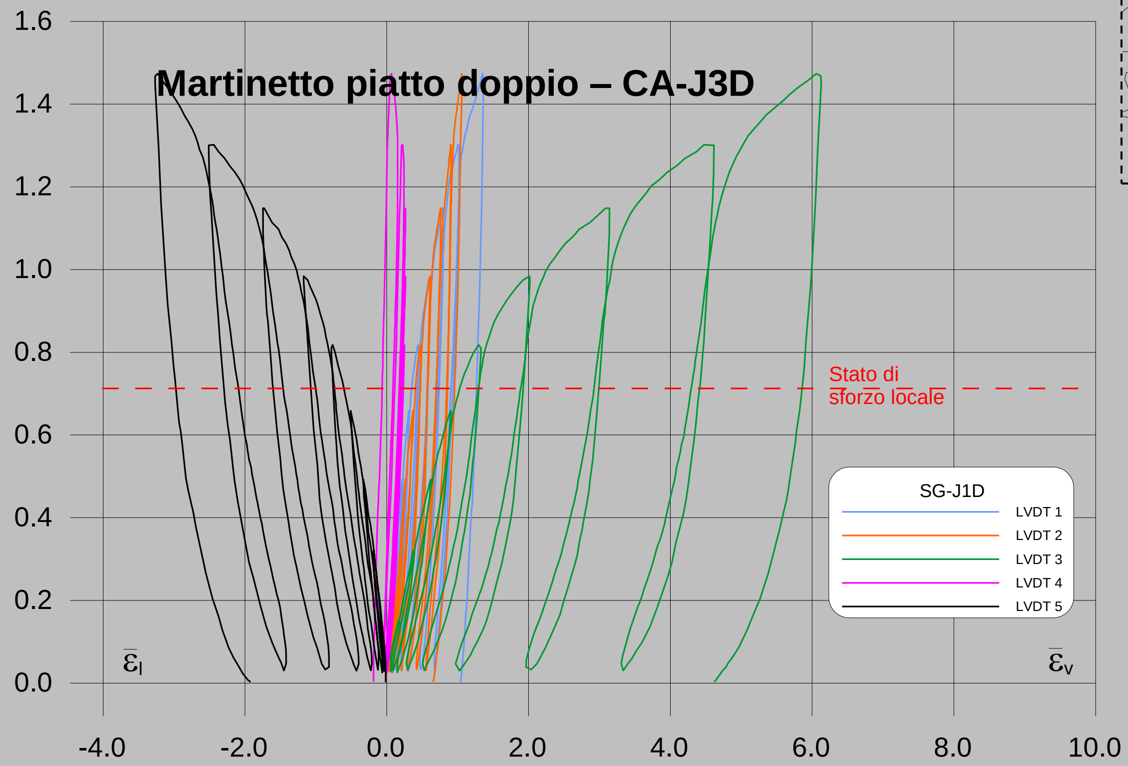
Prova con martinetto piatto singolo CA-J3S

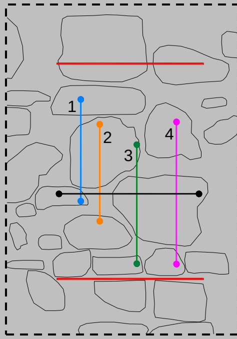


lato sala 4

RESEARCH ACTIVITY

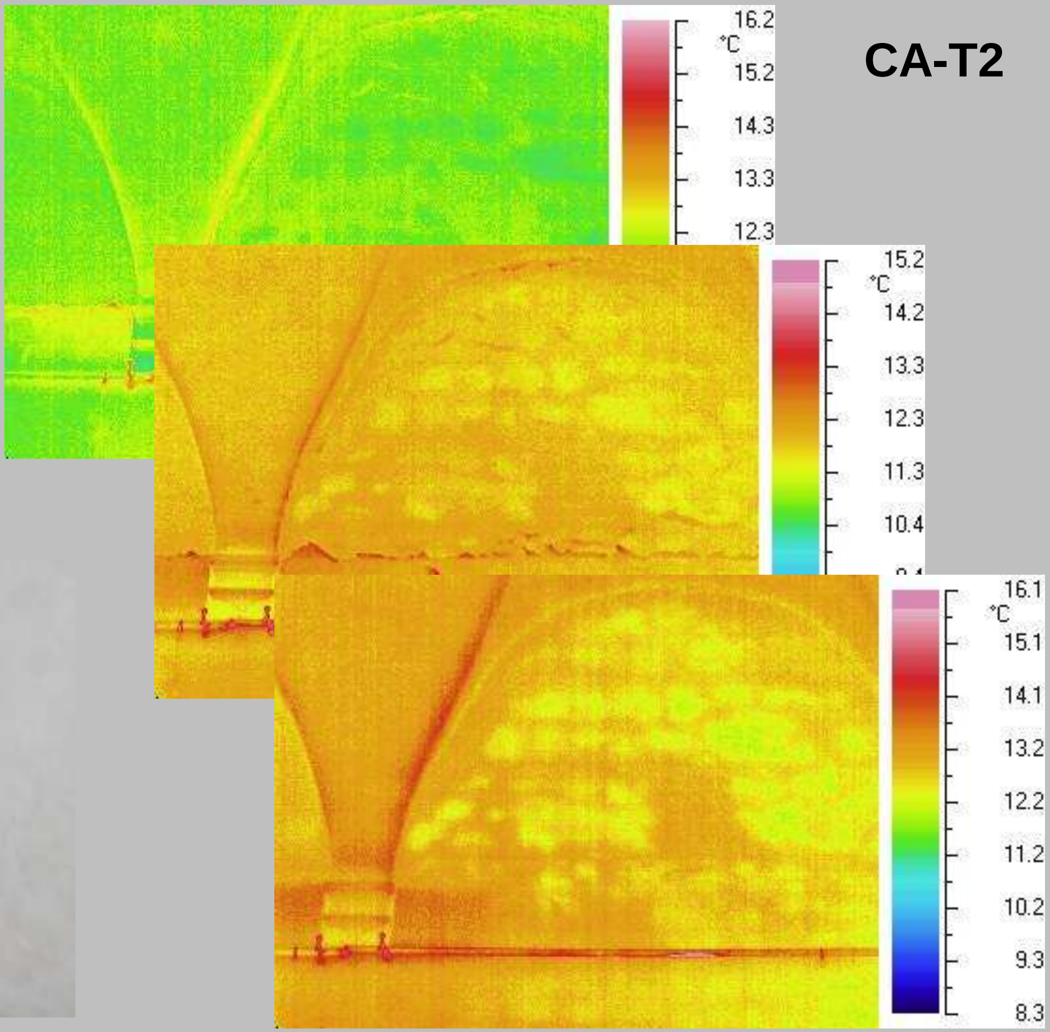
SONIC TEST - DOUBLE FLAT JACK TEST





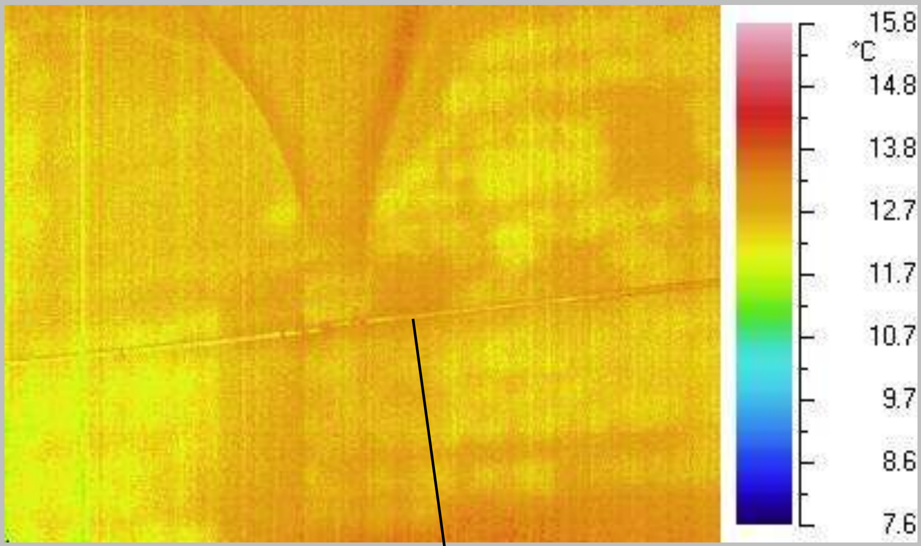
RESEARCH ACTIVITY

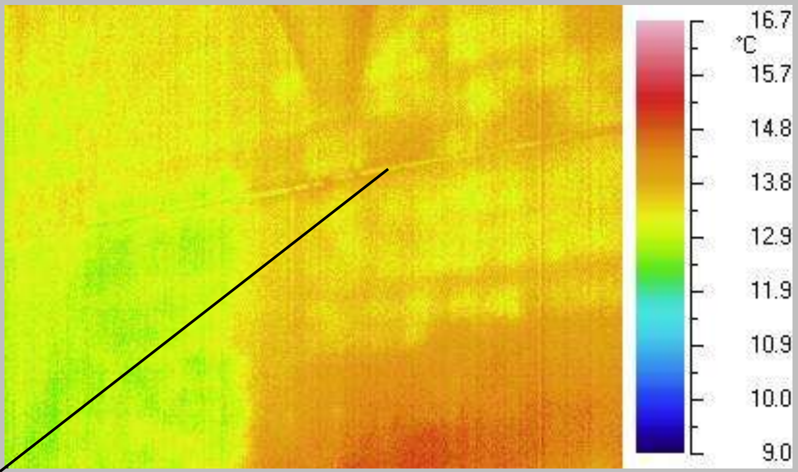
TERMOGRAPHY



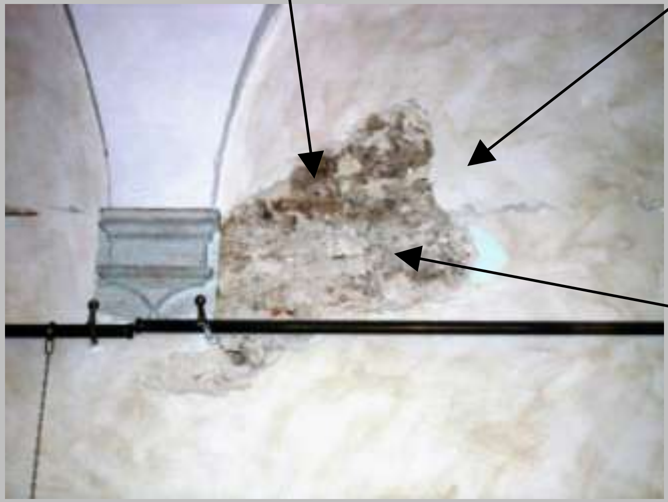
CA-T2

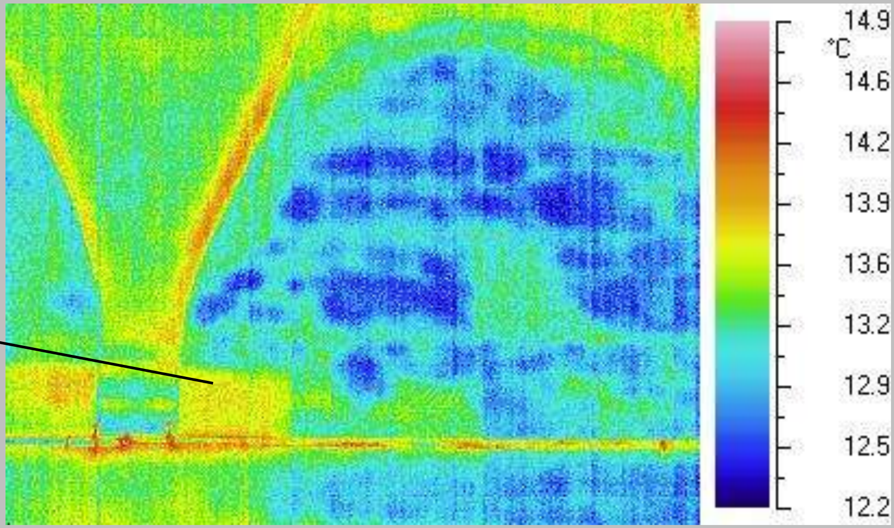
CA-T2





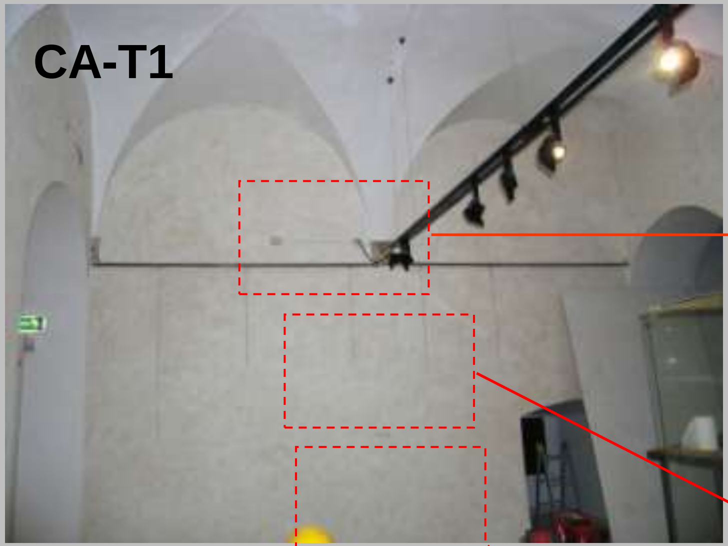
Identificazione di un’area composta da mattoni-e di blocchi grandi di pietra sotto il peduccio della volta

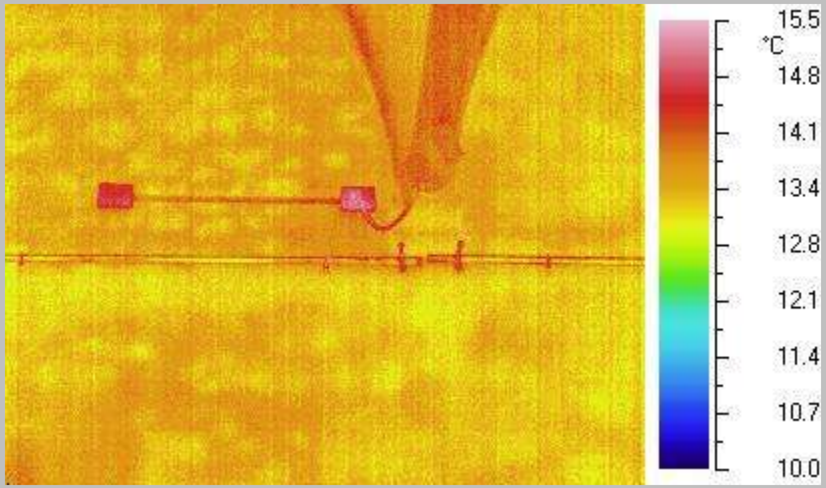




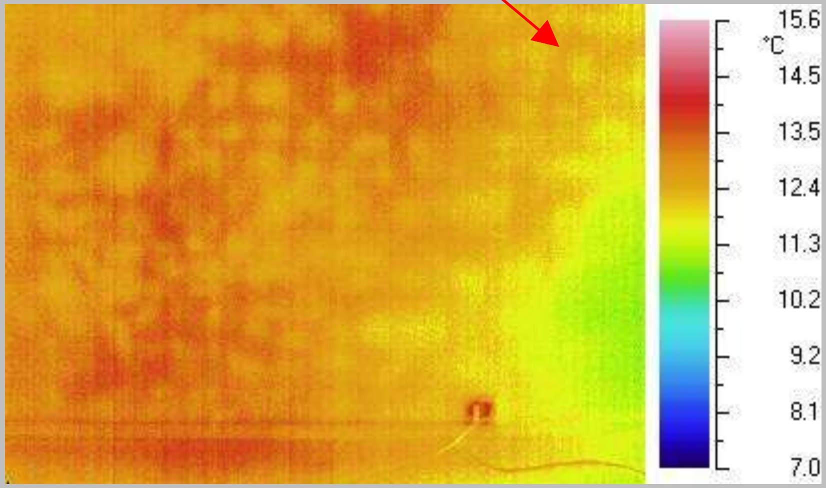
RESEARCH ACTIVITY

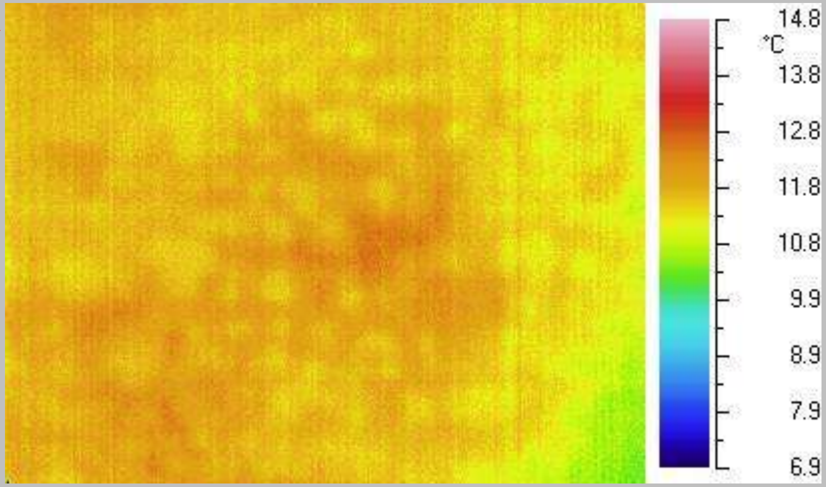
TERMOGRAPHY





Ricerca di un vuoto, non riuscita

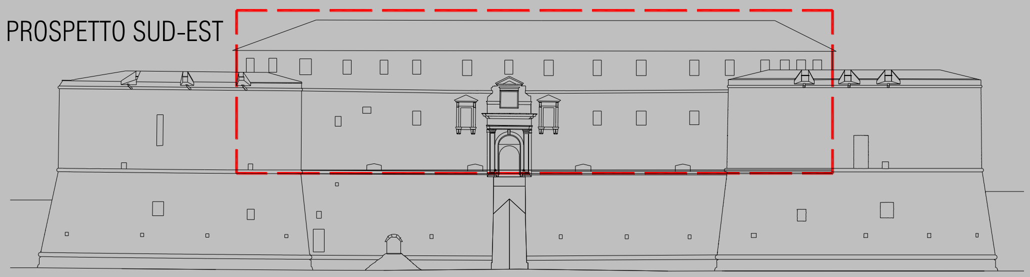


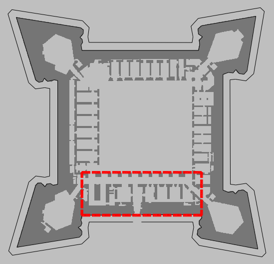


RESEARCH ACTIVITY

DYNAMIC IDENTIFICATION TESTS

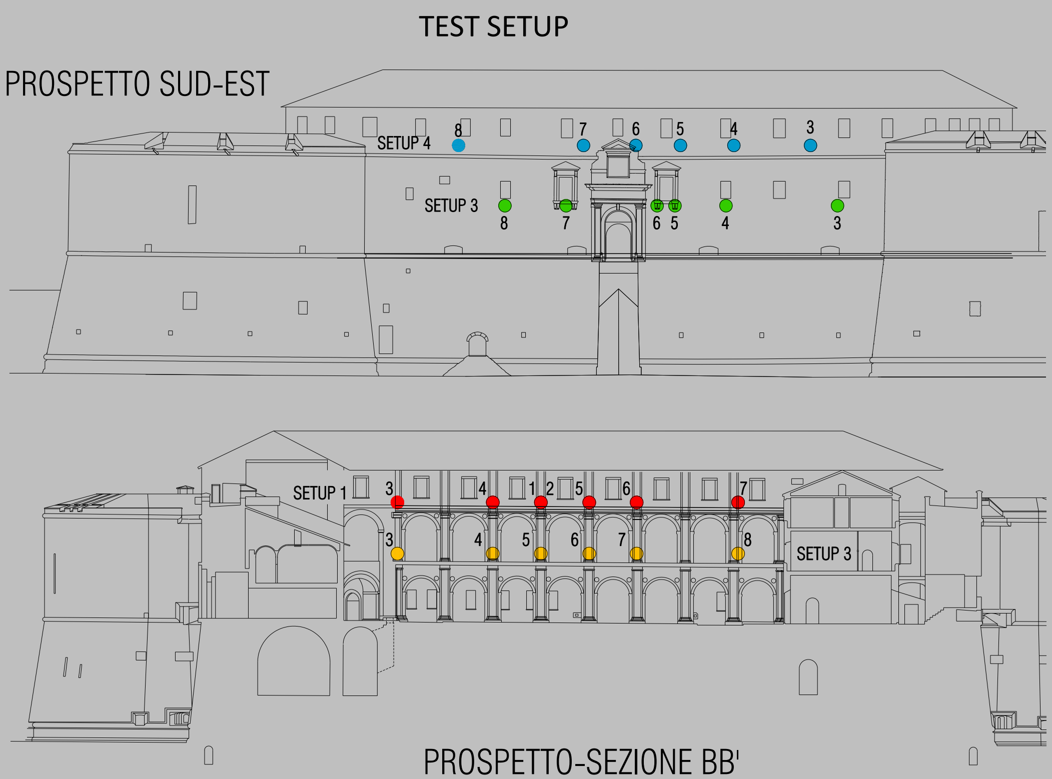
On September 8th and 9th, 2009 dynamic identification tests have been performed in the South-East wing of the Spanish fortress. The investigations consisted in the measurement of environmental vibration in several points of the structure. Tests were aimed at identifying the dynamic behavior of this part of the building (natural frequencies and mode shapes) at the present state - i.e. following the severe state of damage caused by the earthquake of April 2009 and subsequent emergency provisional interventions - by means of environmental vibration tests (excitation source: wind, traffic, etc.).





RESEARCH ACTIVITY

DYNAMIC IDENTIFICATION TESTS



The aim of the test was to evaluate the behavior of the perimeter walls toward inside (courtyard) and outside (ditch) the fortress, after the overturning mechanisms activated by the earthquake. From a structural point of view it was important to know if the two walls (also thanks to the provisional tie-rods system) have different dynamic behavior after the seismic event.

In order to maximize the amplitude of the signals, the accelerometers - in the considered setup - were positioned on the upper floors of the building (first and second floor).

Five configuration setups were performed with 27 points of acquisition. For each setup no more than eight sensors were used, including two fixed reference sensors, arranged in two directions (X and Y) parallel to the ground and perpendicular to each other (channels 1 and 2 at the second floor)

RESEARCH ACTIVITY

DYNAMIC IDENTIFICATION TESTS




Fissaggio dei sensori e registrazione dei segnali

The aim of the test was to evaluate the behavior of the perimeter walls toward inside (courtyard) and outside (ditch) the fortress, after the overturning mechanisms activated by the earthquake. From a structural point of view it was important to know if the two walls (also thanks to the provisional tie-rods system) have different dynamic behavior after the seismic event.

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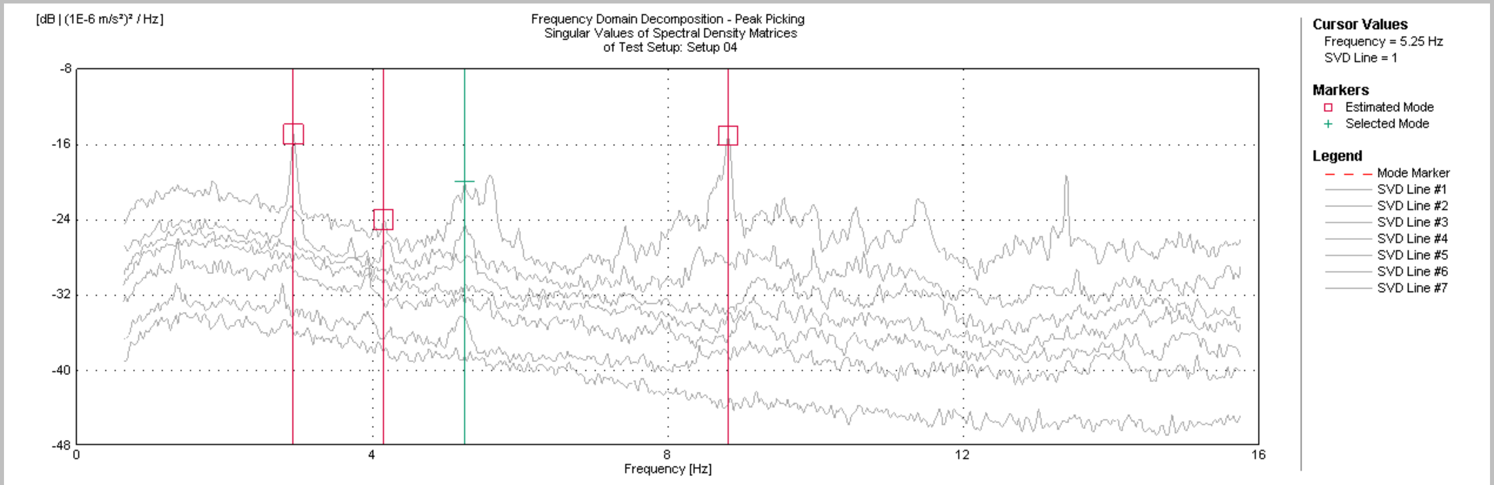
RESEARCH ACTIVITY

DYNAMIC IDENTIFICATION TESTS

TEST RESULTS

The identification of the modal parameters (natural frequencies and associated mode shapes) was obtained by means of identification techniques of the output signals (Operational modal analysis). The recorded vibrations were produced by the environmental stresses produced by the wind and the urban traffic.

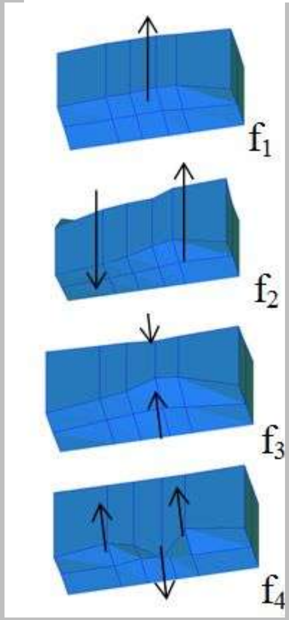
The analysis of the results clearly identify the first out-of-plane bending mode, corresponding to the frequency of 2.93 Hz, emerged in all acquisitions (*Fig.37* and *Tab.2*). The identification of the global vibration mode of the structure indicates that the building, in spite of the high level of damage and the disconnection of the perimeter masonry walls, has still a unitary dynamic response, probably thanks to the provisional emergency system of steel cables. Another vibration mode is detected by the analysis of individual data for each setup, and it is an out of phase bending mode recorded at the frequency of 5.2 - 5.4 Hz.



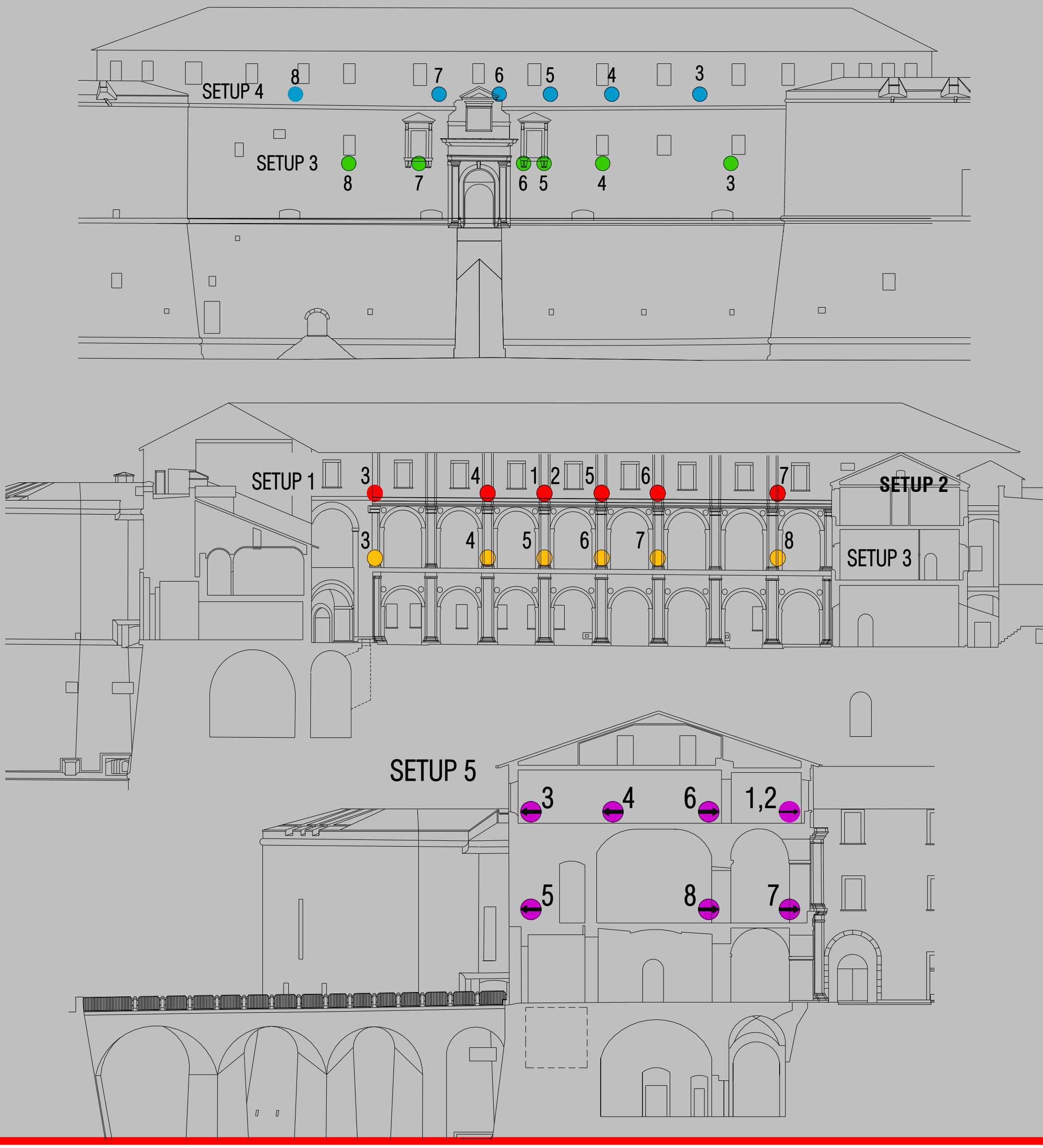
Setup 1+2+3+4 - f [Hz]
2.93
4.15
5.25
8.82

POSSIBLE SOLUTIONS BEFORE AND IMMEDIATELY AFTER EARTHQUAKE

PROF. CLAUDIO MODENA

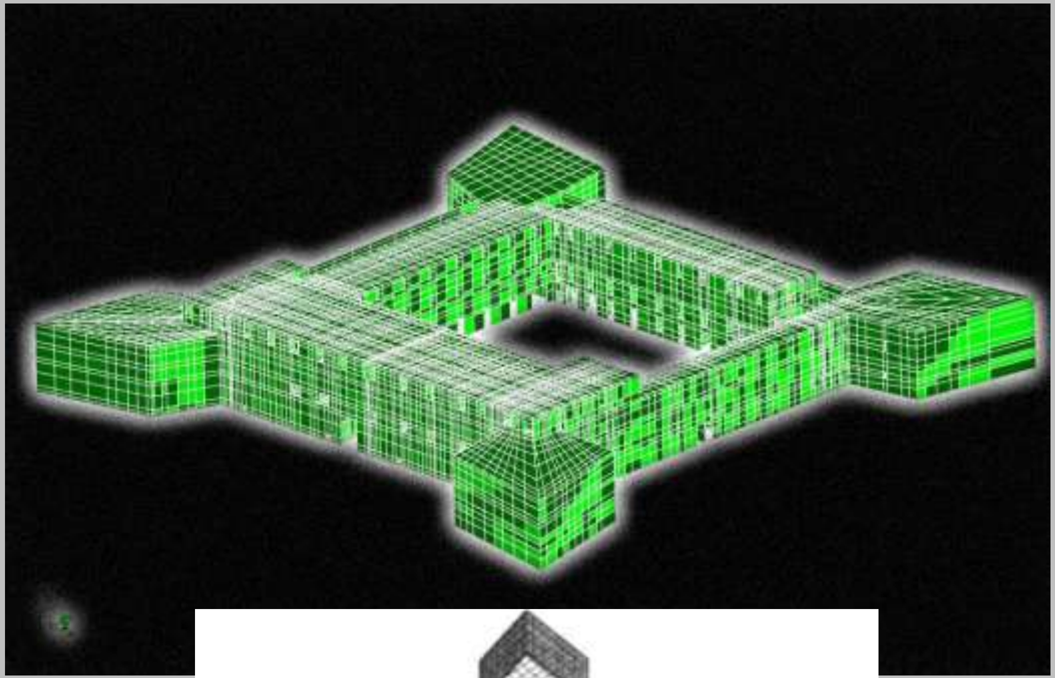


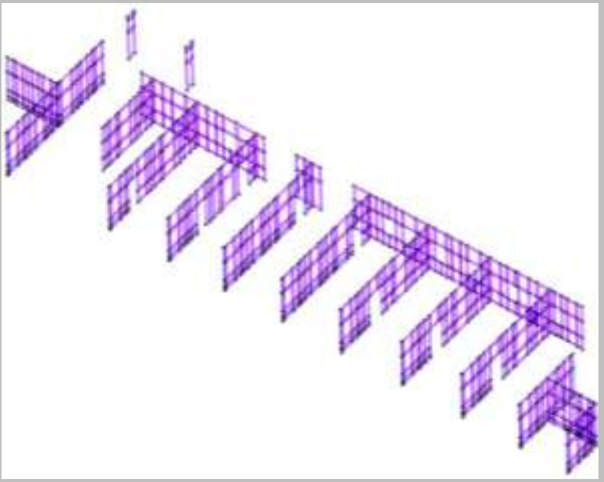
Setup 1+2+3+4 - f [Hz]
f1 = 2.93
f2 = 4.15
f3 = 5.25
f4 = 8.82

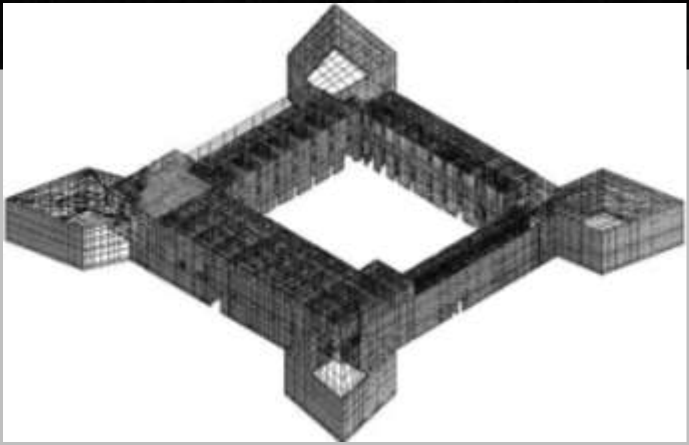


RESEARCH ACTIVITY

FINITE ELEMENT ANALYSIS



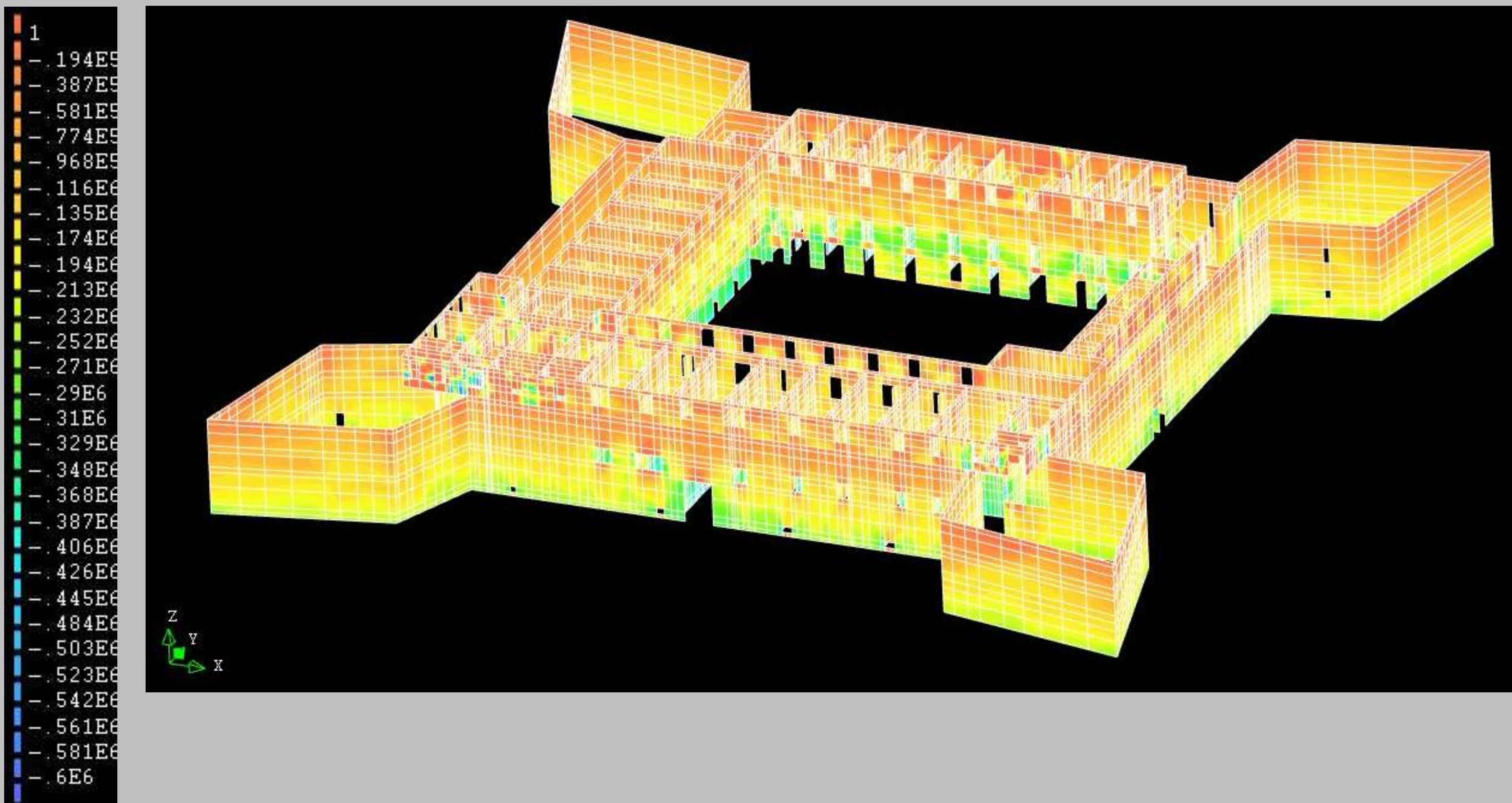




Materiale	Densità di massa [Kg/m^3]	Modulo elastico di Young [MPa]
Murature portanti	1800	3000
Cinta esterna	1800	3000
Pilastri del porticato	1800	3000
Volte in laterizio	1600	3500
Solaio in laterocemento	1500	11000
Solaio in laterizio e putrelle d'acciaio	1900	11000

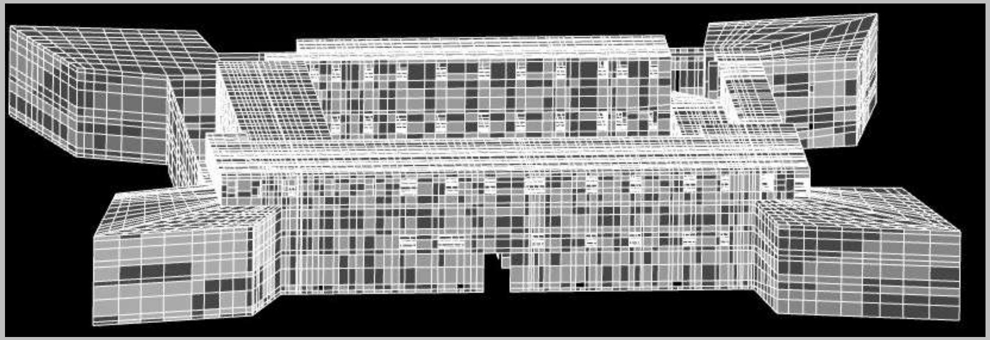
RESEARCH ACTIVITY

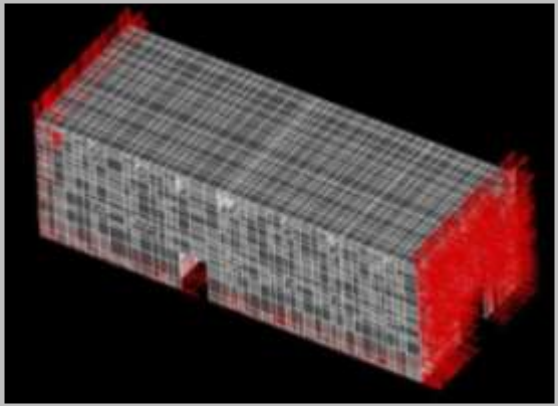
FINITE ELEMENT ANALYSIS



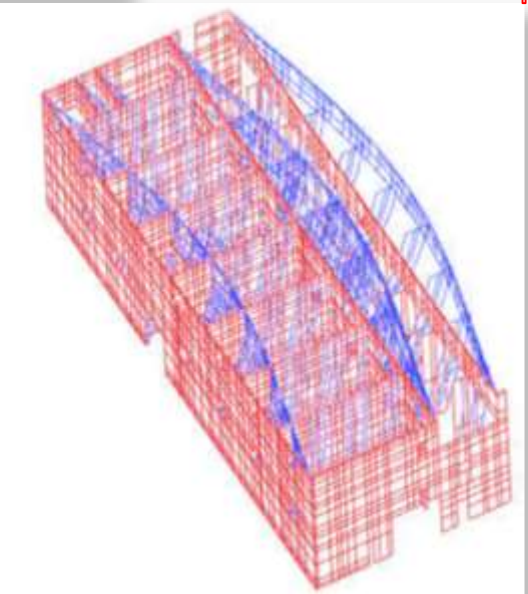
RESEARCH ACTIVITY

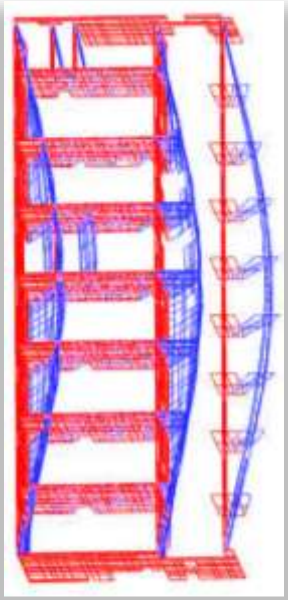
NATURAL FREQUENCY ANALYSIS



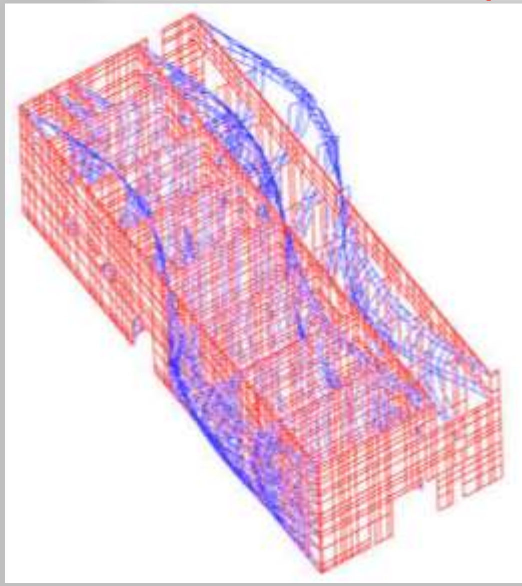


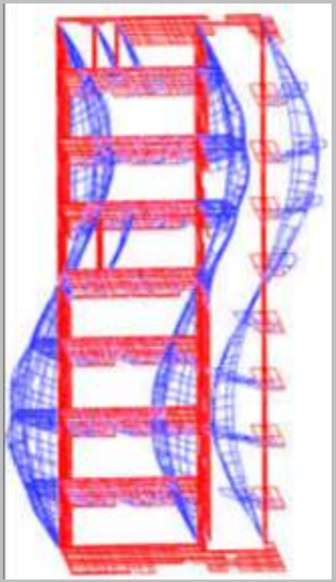
Mode shape at 2.76 Hz





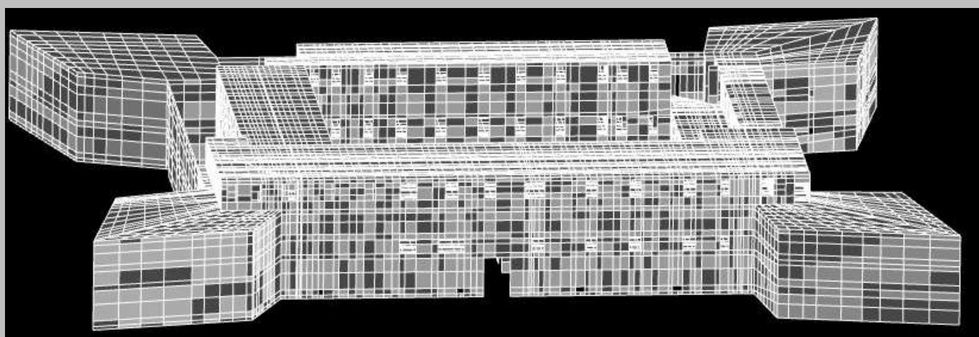
Mode shape at 4.21 Hz

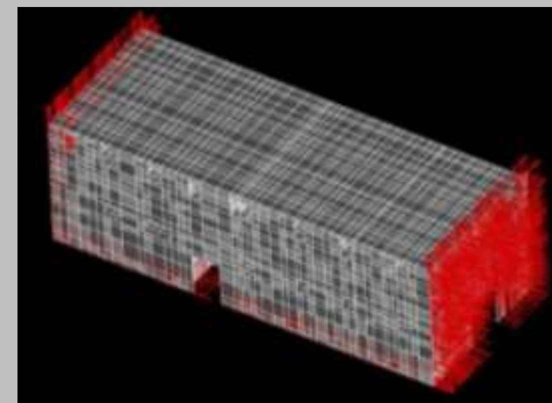




RESEARCH ACTIVITY

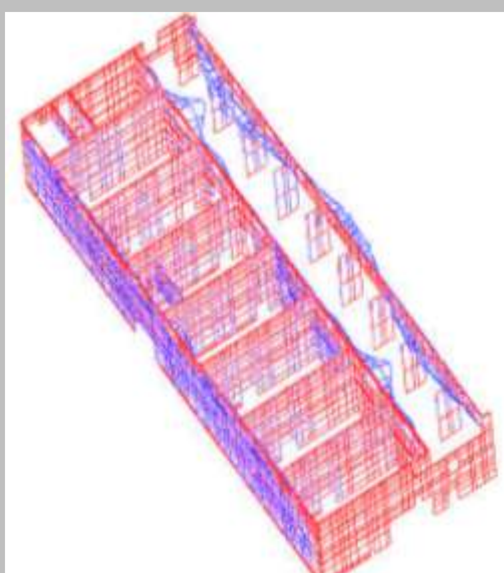
NATURAL FREQUENCY ANALYSIS

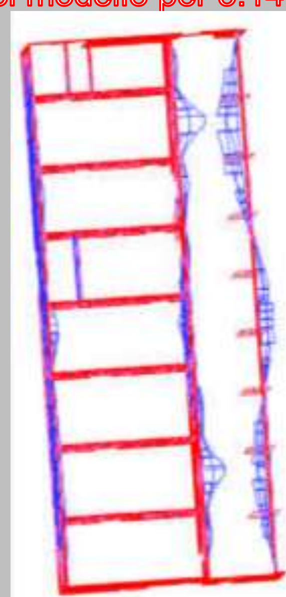


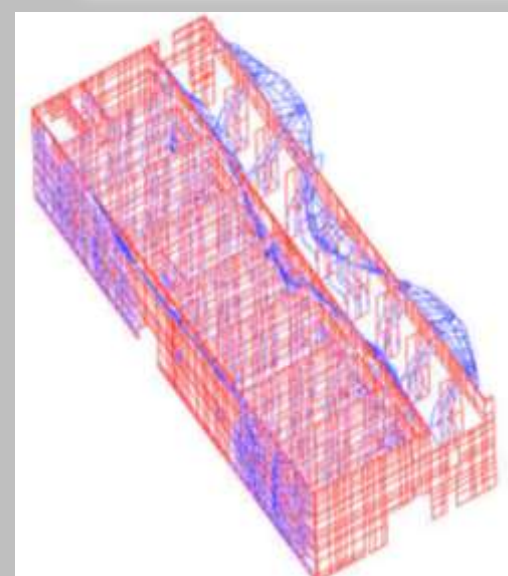


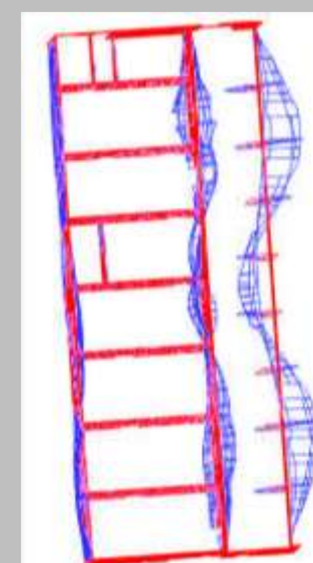
Forma modale ottenuta col modello per 5.14 Hz

Forma modale ottenuta col modello per 8.65 Hz



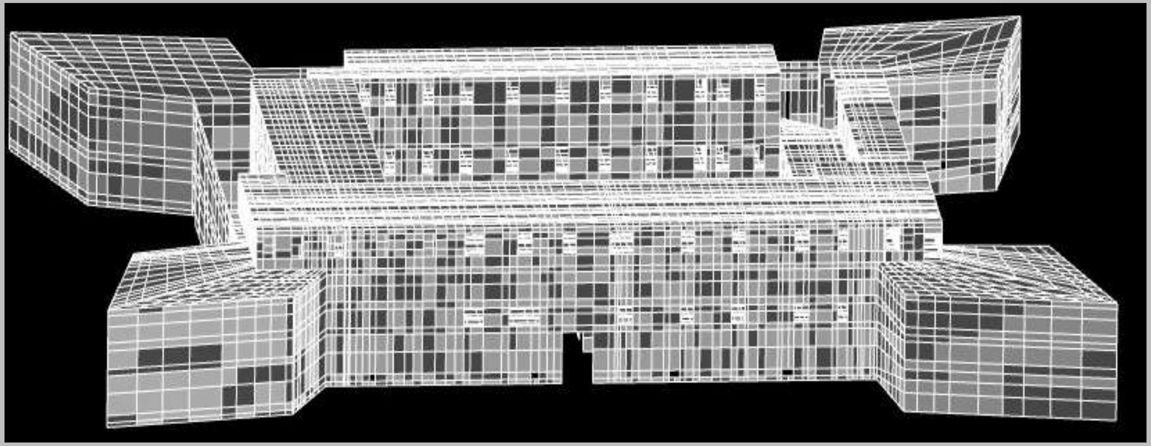


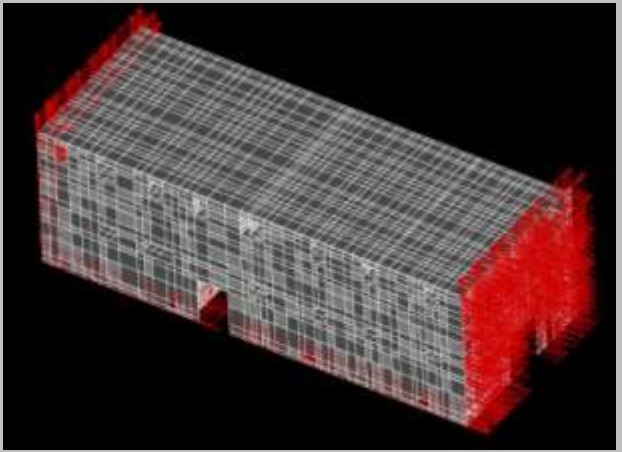




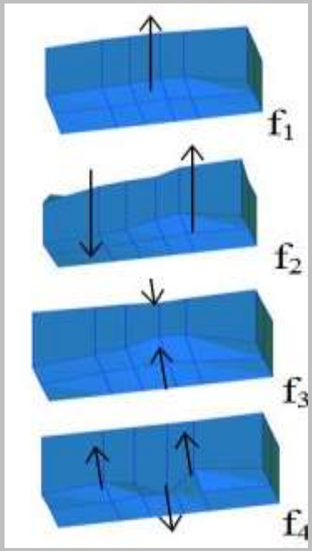
RESEARCH ACTIVITY

NATURAL FREQUENCY ANALYSIS





	Experimental results	FEA results	Difference [%]
f_1 [Hz]	2.88	2.76	4.3
f_2 [Hz]	4.11	4.21	2.5
f_3 [Hz]	5.21	5.14	1.4
f_4 [Hz]	8.47	8.65	2.1



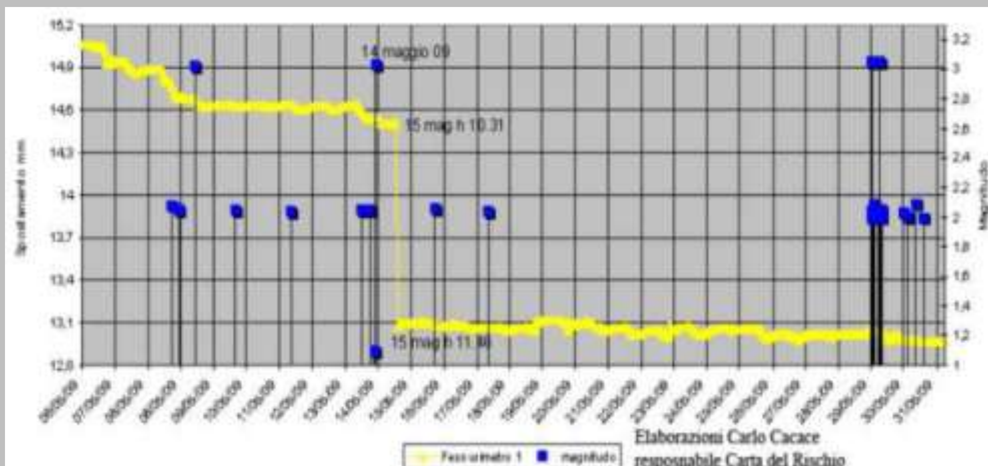
SPANISH FORTRESS, L'AQUILA: MONITORING SYSTEM

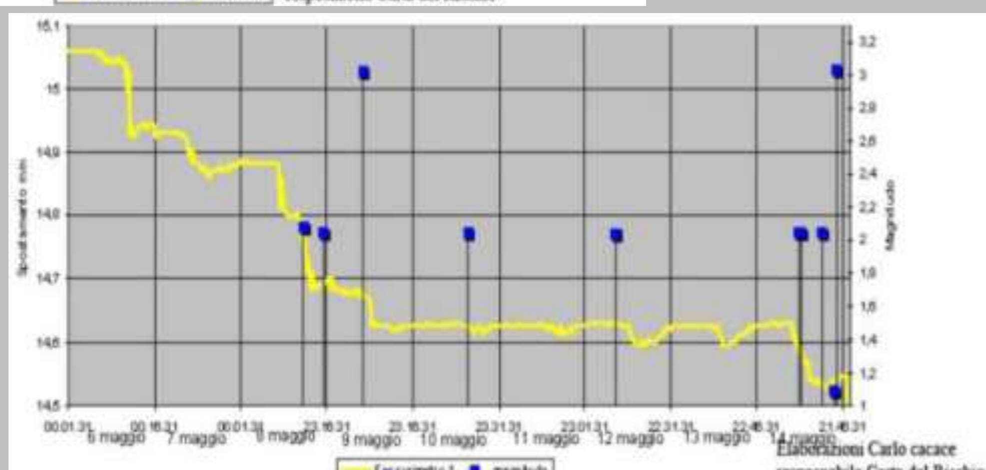
STATIC MONITORING SYSTEM

PROF. CLAUDIO MODENA

SPANISH FORTRESS, L'AQUILA: MONITORING SYSTEM

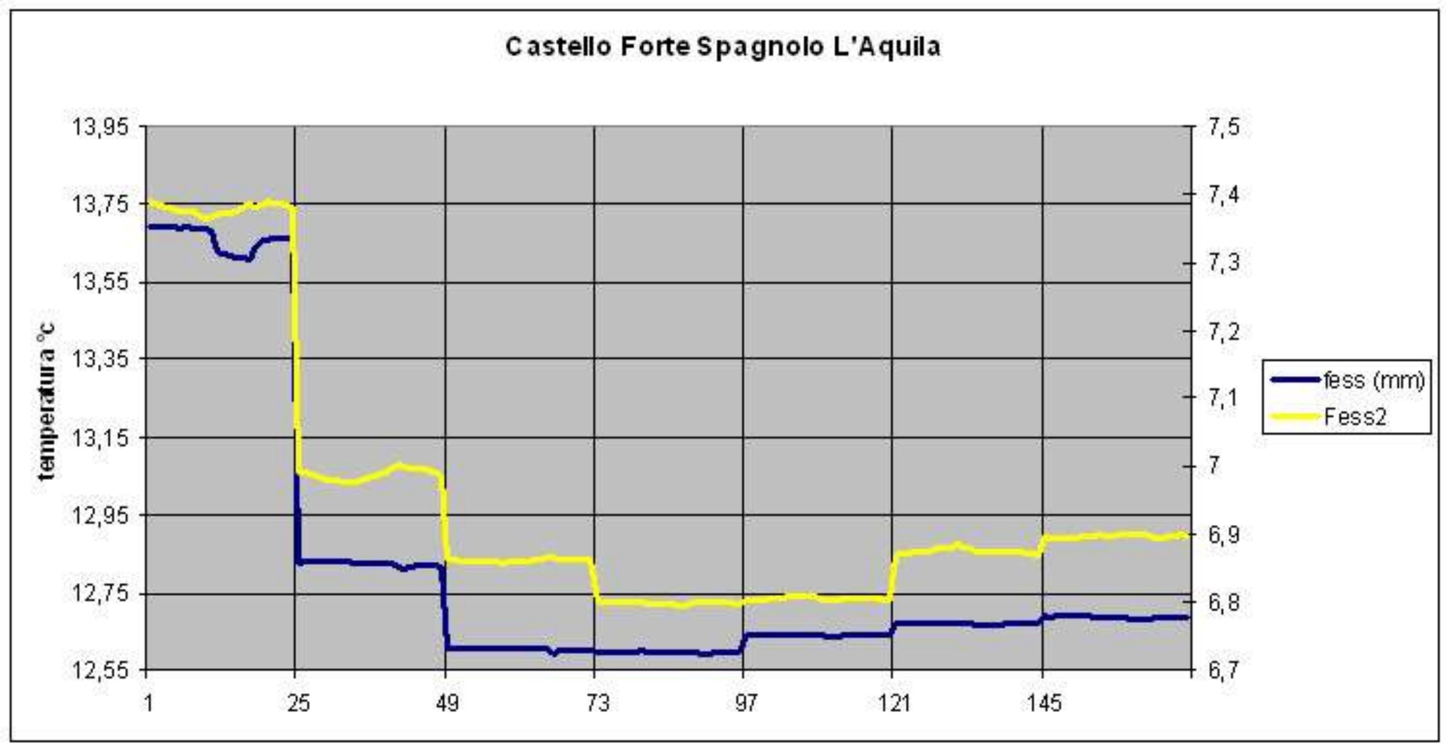
STATIC MONITORING SYSTEM





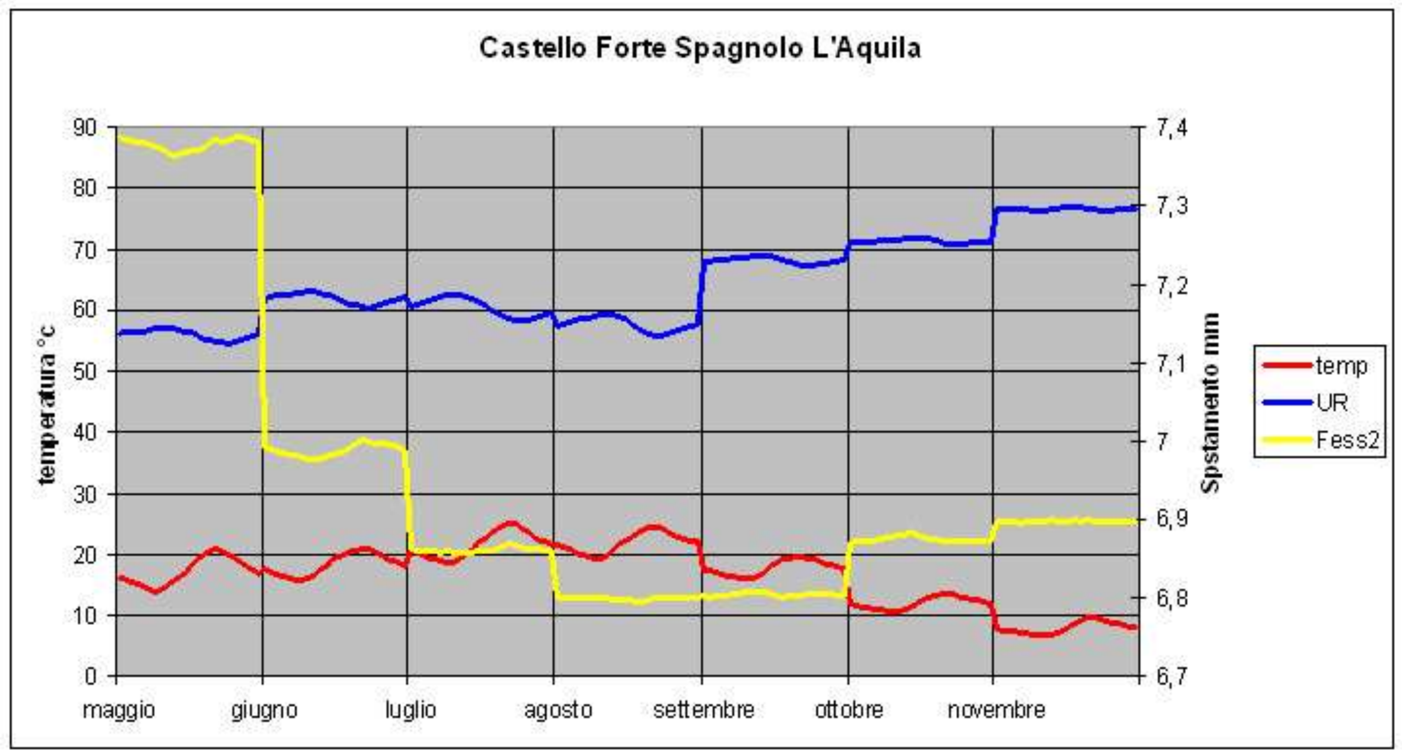
SPANISH FORTRESS, L'AQUILA: MONITORING SYSTEM

STATIC MONITORING SYSTEM



SPANISH FORTRESS, L'AQUILA: MONITORING SYSTEM

STATIC MONITORING SYSTEM

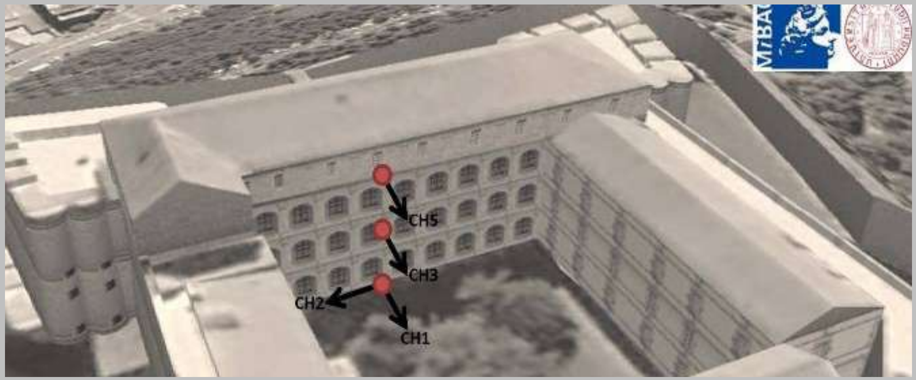


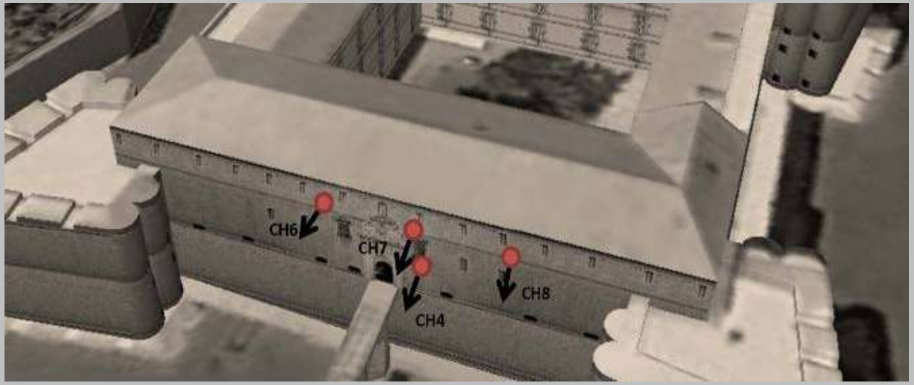
SPANISH FORTRESS, L'AQUILA: MONITORING SYSTEM

DYNAMIC MONITORING SYSTEM

In December 2009 a dynamic monitoring system was installed in the Fortress, composed by an acquisition unit connected to eight high sensitivity piezoelectric accelerometers. The central unit, located at the second floor of the fortress, in the South-East wing, is provided with a Wi-Fi router for remote data transmission.

A couple of reference sensors is fixed at the base of the structure for the record of the ground acceleration both in operational conditions and during seismic events. The positioning of the acceleration sensors on the elevation of the S-East wing was decided as a result of the structural dynamic identification.

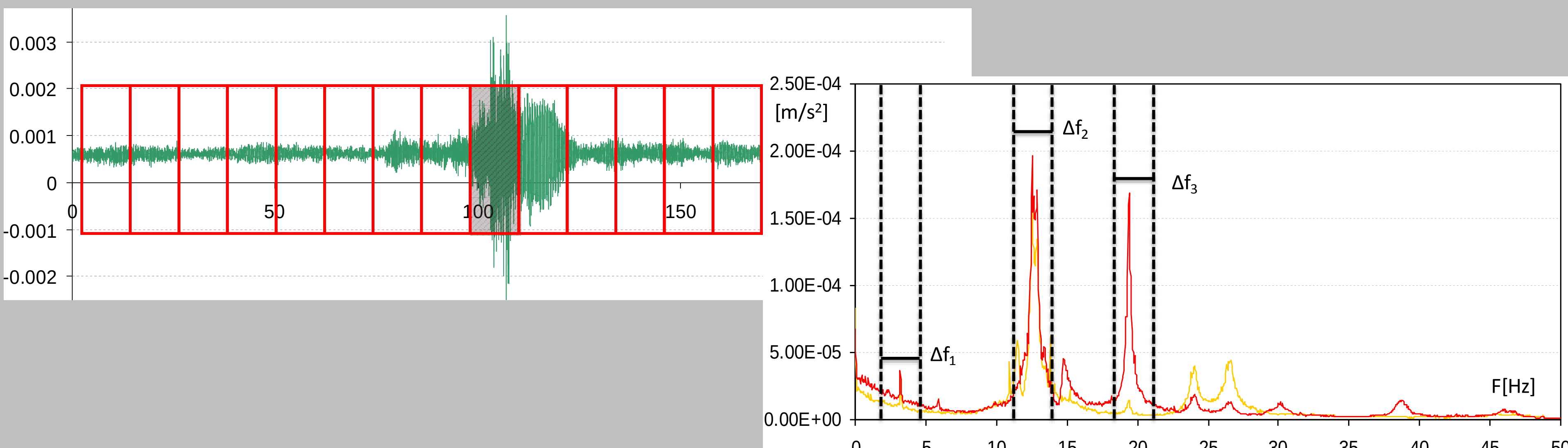




SPANISH FORTRESS, L'AQUILA: MONITORING SYSTEM

DYNAMIC MONITORING SYSTEM

Dynamic data are being collected both at fixed time intervals ("long" acquisition, corresponding to 131'072 points, or to 21'51'' of record at a sampling frequency of 100 SPS, each 1-24 hours) to allow successive dynamic identification of the structure with different environmental conditions, and on a trigger basis (shorter records, 3'35'' at a sampling frequency of 100 SPS), when the signal, on one of the acceleration channels, gets over the predefined threshold

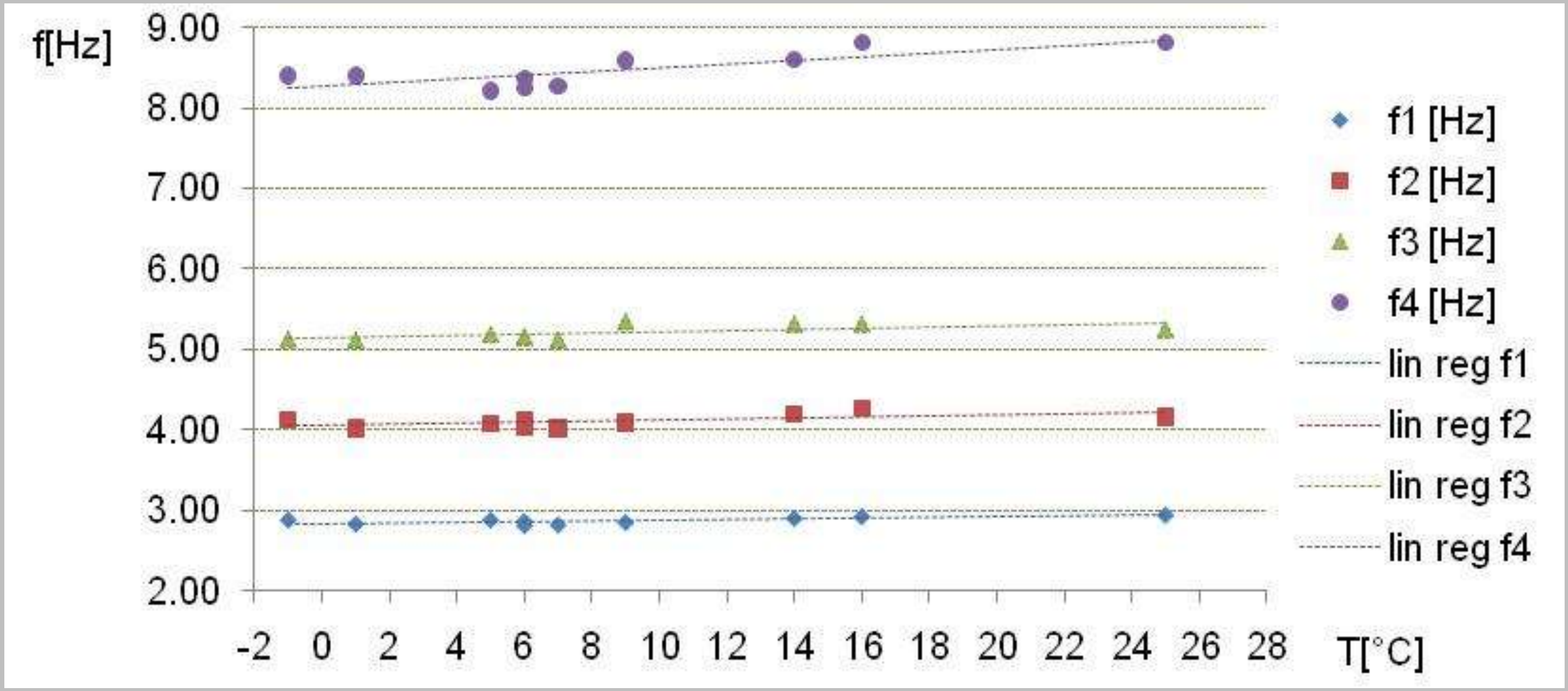


In order to avoid possible disturbance caused by noises with spectral components far from the frequencies of interest for the monitored structure (e.g. a noise with high frequency components, which can disturb the signal, making difficult the time domain based control), the control can be operated in the frequency domain.

SPANISH FORTRESS, L'AQUILA: MONITORING SYSTEM

DYNAMIC MONITORING SYSTEM

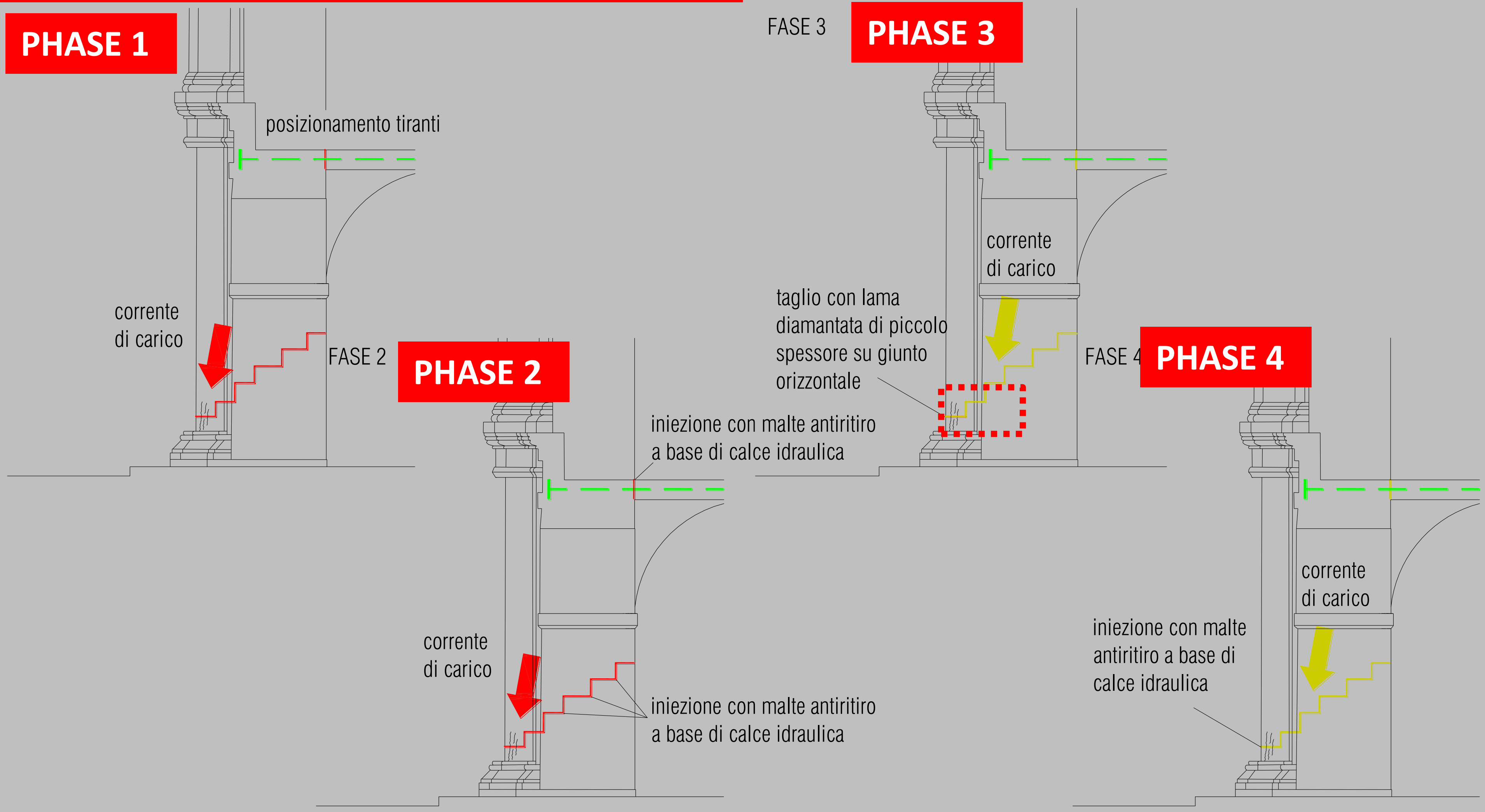
From the observation of the initial results, it seems that there exists a correlation between natural frequencies and environmental parameters, that is to say that frequencies tend to increase with the temperature



POSSIBLE SOLUTIONS BEFORE AND IMMEDIATELY AFTER EARTHQUAKE

PROF. CLAUDIO MODENA

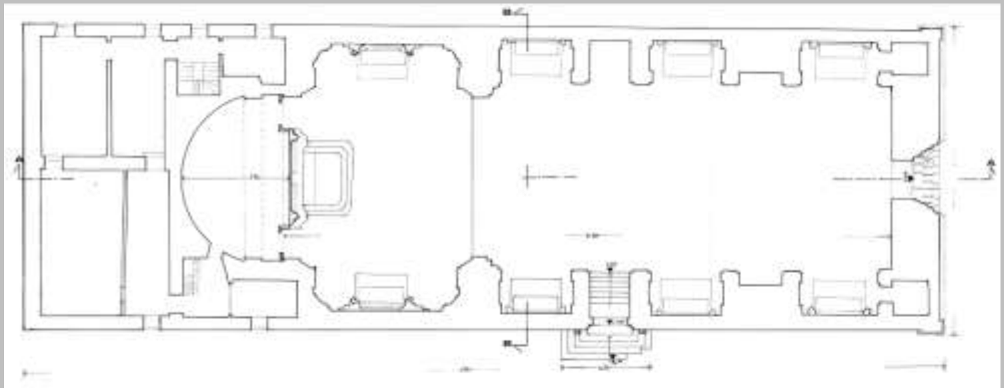
INTERVENTION PROPOSAL

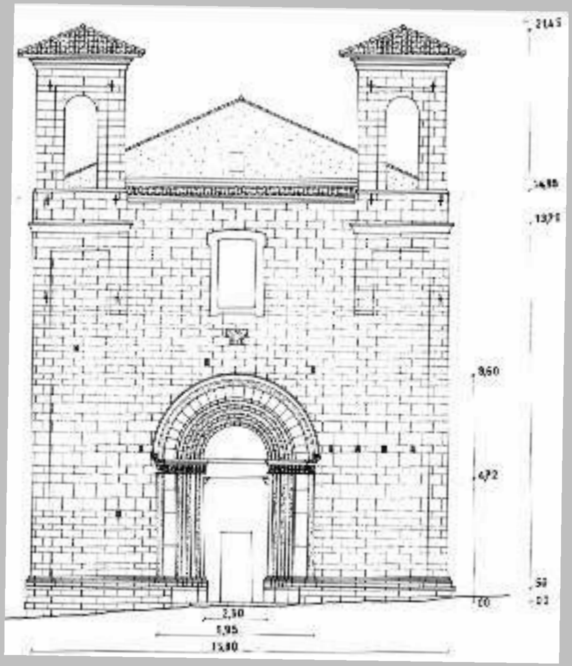


ST. MARCO CHURCH

THE S.MARCO CHURCH

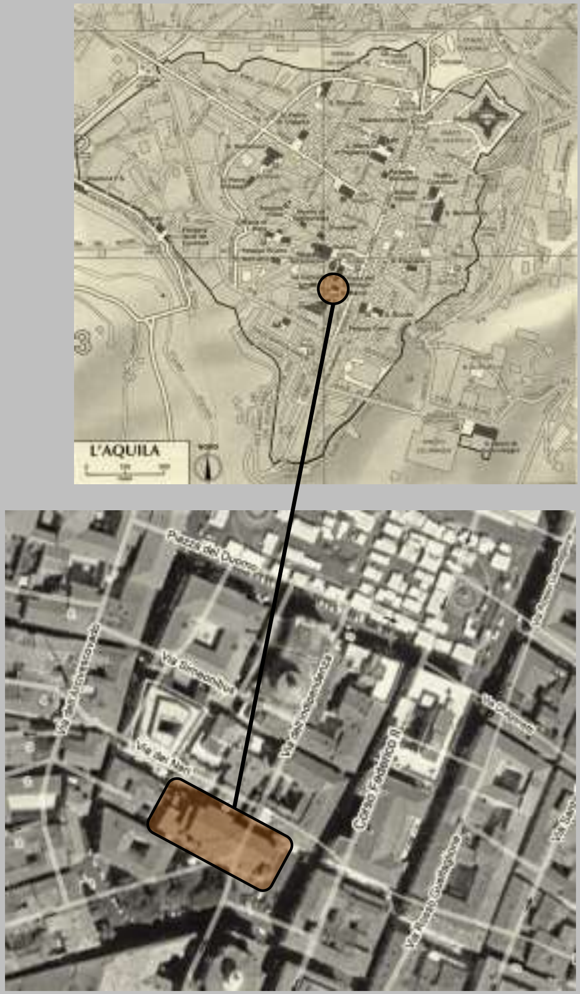
The St. Mark's church is one of the first churches built in L'Aquila in the second half of the 13th century, and it is located on the hearth of the city of L'Aquila, between "Via dei Neri" and "Piazza della Prefettura". Medieval traces are preserved mainly in the external walls and in the lateral entrance, dated from the 14th century. The main façade was built at the beginning of the XV century





HISTORICAL INVESTIGATION

The st. Mark's church is one of the first churches built in l'aquila in the second half of the 13th century, and it is located in the hearth of the city of l'aquila, between "via dei neri" and "piazza della prefettura". Medieval traces are preserved mainly in the external walls and in the lateral entrance, dated back to the 14th century. The main façade was built at the beginning of the xv century



This church suffered different transformations throughout time:

1315 - suspicion: part of the church is reconstructed after the earthquake;

XVI - construction of the lateral chapels attached to the nave lateral façades;

1750 - construction of the two bell towers and of the top part of the frontal façade (sommità).

HISTORICAL INVESTIGATION

INTERVENTIONS

1970

2005

2007

A - STRUCTURAL: SUBSTITUTION OF THE OLD WOODEN ROOF;

B - STRUCTURAL: R.C. ELEMENTS OVER THE PRESBYTERY;

A AND B

HISTORICAL INVESTIGATION

1970

2005

2007

INTERVENTIONS

A - STRUCTURAL: SUBSTITUTION OF THE OLD WOODEN ROOF;

B - STRUCTURAL: R.C. ELEMENTS OVER THE PRESBYTERY;

A - NON-STRUCTURAL: REMOVAL OF THE OLD EAVES AND CONSTRUCTION OF NEW ONES;

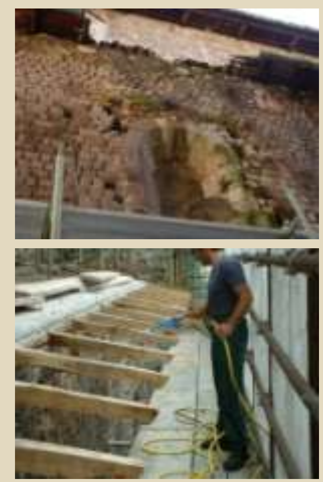
B - STRUCTURAL: CONSTRUCTION OF THE NEW WOODEN ROOF ON THE BELL TOWERS;

C - NON-STRUCTURAL: ISOLATION OF THE CHURCH ROOF;

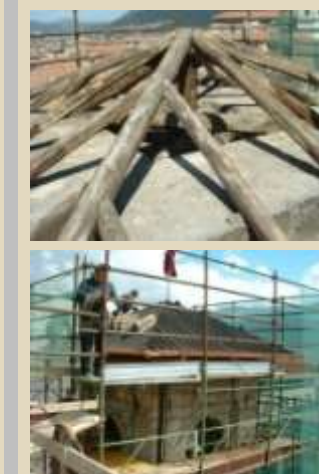
D - NON-STRUCTURAL AND STRUCTURAL: MAINTENANCE OF THE LATERAL AND FRONTAL FAÇADES AND CONSOLIDATION OF THE FRONTAL FAÇADE;

E - STRUCTURAL: SUBSTITUTION OF THE BELL TOWERS PRE-EXISTENT IRON TIES;

A



B



C

HISTORICAL INVESTIGATION

1970

2005

2007

INTERVENTIONS

A - STRUCTURAL: SUBSTITUTION OF THE OLD WOODEN ROOF;

B - STRUCTURAL: R.C. ELEMENTS OVER THE PRESBYTERY;

A - NON-STRUCTURAL: REMOVAL OF THE OLD EAVES AND CONSTRUCTION OF NEW ONES;

B - STRUCTURAL: CONSTRUCTION OF THE NEW WOODEN ROOF ON THE BELL TOWERS;

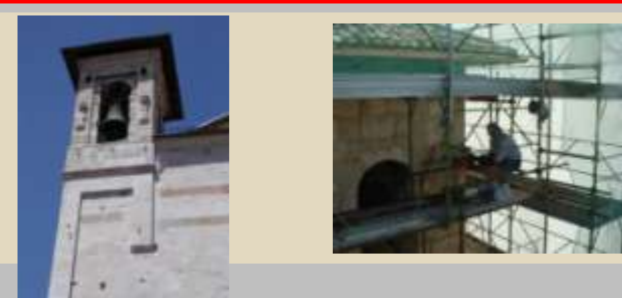
C - NON-STRUCTURAL: ISOLATION OF THE CHURCH ROOF;

D - NON-STRUCTURAL AND STRUCTURAL: MAINTENANCE OF THE LATERAL AND FRONTAL FAÇADES AND CONSOLIDATION OF THE FRONTAL FAÇADE;

E - STRUCTURAL: SUBSTITUTION OF IRON TIES OF THE BELL TOWERS;

D

E



HISTORICAL INVESTIGATION

INTERVENTIONS

1970

A - STRUCTURAL: SUBSTITUTION OF THE OLD WOODEN ROOF;

B - STRUCTURAL: R.C. ELEMENTS OVER THE PRESBYTERY;

2005

A - NON-STRUCTURAL: REMOVAL OF THE OLD EAVES AND CONSTRUCTION OF NEW ONES;

B - STRUCTURAL: CONSTRUCTION OF THE NEW WOODEN ROOF ON THE BELL TOWERS;

C - NON-STRUCTURAL: ISOLATION OF THE CHURCH ROOF;

D - NON-STRUCTURAL AND STRUCTURAL: MAINTENANCE OF THE LATERAL AND FRONTAL FAÇADES AND CONSOLIDATION OF THE FRONTAL FAÇADE;

E - STRUCTURAL: SUBSTITUTION OF THE BELL TOWERS PRE-EXISTENT IRON TIES;

2007

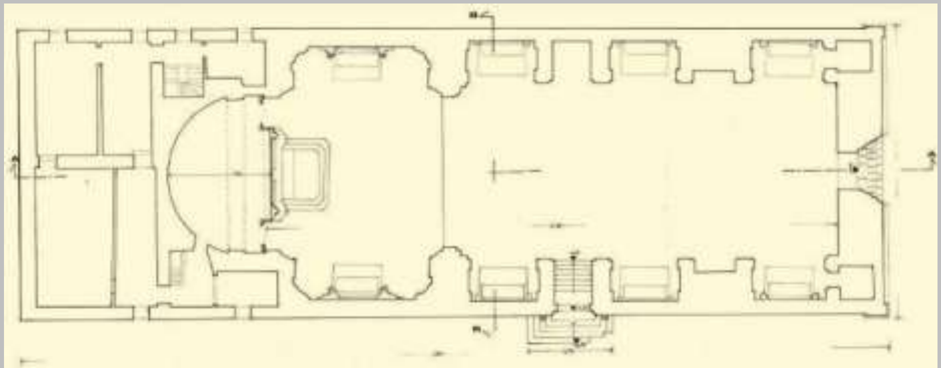
A - STRUCTURAL: APPLICATION OF THE CARBON FIBERS TO THE ARCHES INNER FACE

A

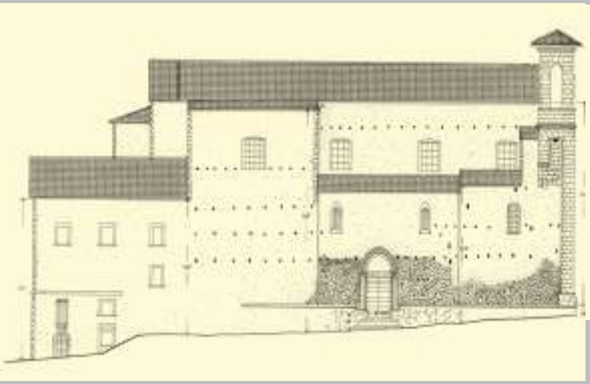
DESCRIPTION OF THE BUILDING: SURVEY

A geometrical survey of the church was performed based on the available elements such as topographic surveys, which were later validated during the technical inspections to the building, through control measurements.

Plan

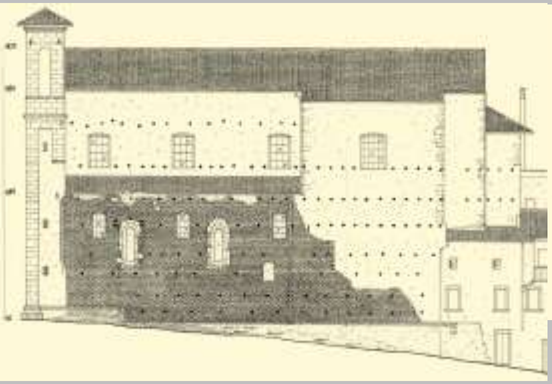


EVANGELI FAÇADE

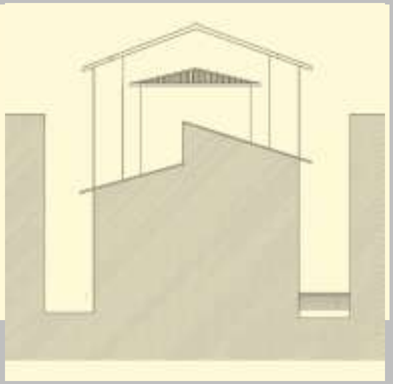


FRONTAL FAÇADE

EPISTOLA FAÇADE



POSTERIOR FAÇADE



DESCRIPTION OF THE BUILDING: CONSTRUCTIVE ELEMENTS

- ROOF (REINFORCED CONCRETE AND CLAY ELEMENTS);
- DOME (BRICK MASONRY);
- BEARING WALLS (MULTIPLE LEAF MASONRY) - NON HOMOGENEITY OF THE MASONRY DUE TO DIFFERENT CONSTRUCTION PHASES;
- VAULTS OVER THE AISLES (MIXED BRICK AND STONE MASONRY);
- BARREL VAULT (BRICK MASONRY ARCHES AND SPANS BETWEEN ARCHES IN REED);
- APSES (MIXED BRICK AND STONE MASONRY);
- ARCHES (AT THE EXTRADOS OF THE ARCHES WOODEN TIE BEAMS ARE INSERTED IN THE MASONRY AS CONFINEMENT).



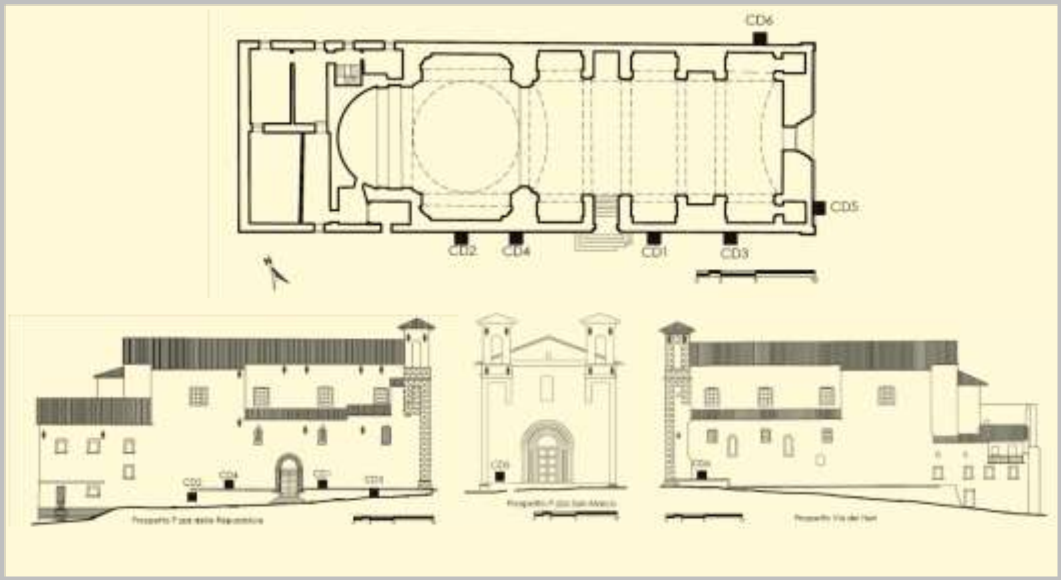





DESCRIPTION OF THE BUILDING: MATERIAL QUALITY

The survey of the different types of masonry present in the church was also done during the technical inspections. TO SYSTEMATIZE THE INFORMATION COLLECTED, A SPECIFIC FORM WAS DEVELOPED AND FILLED - "SCHEDA DI 1º LIVELLO PER IL RILIEVO DELLA TIPOLOGIA E DELLA QUALITÀ DELLA MURATURA" FOR EACH TYPE OF MASONRY PRESENT IN THE STRUCTURE.

LOCATION



TYPE OF MATERIALS

Sample	Description		Characteristics				
	Ord. 3274/2005) (Circ. NTC08, 2009)	Reluis	f_m	τ_0	E	G	W (kN/m³)
SP1/SP2/SP3/SP4	Uncut stone masonry with facing walls of limited thickness and infill core	Uncut stone masonry, of variable dimensions, with prevailingly horizontal layers	200 300	3.5 5.1	1020 1440	340 480	20.0
SP5/SP6	Dressed rectangular (ashlar) stone masonry	Dressed rectangular (ashlar) non-soft stone masonry	600 800	9.0 12.0	2400 3200	780 940	22.0

Possible Solutions Before and Immediately After Earthquake

Prof. Claudio Modena

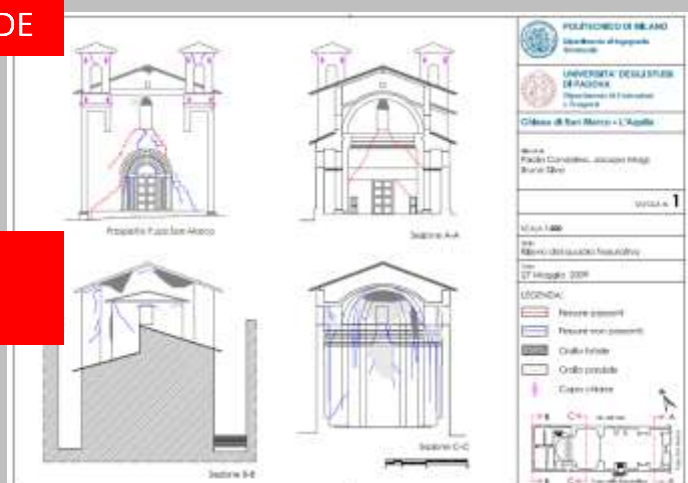
SURVEY AND DESCRIPTION OF THE CARCK PATTERN

The several technical inspections to the church during the months following the earthquake allowed the creation of detailed internal and external crack pattern maps. The interpretation of the crack pattern can be of great help in understanding the state of damage of the structure, its possible causes and the type of survey to be performed, provided that the development history of the building is already known. In this case the crack pattern maps allowed an easy and intuitive assessment of the damage during the seismic event.

EPISTOLA FAÇADE

FRONTAL FAÇADE

POSTERIOR FAÇADE



EVANGELI FAÇADE

DAMAGE SURVEY

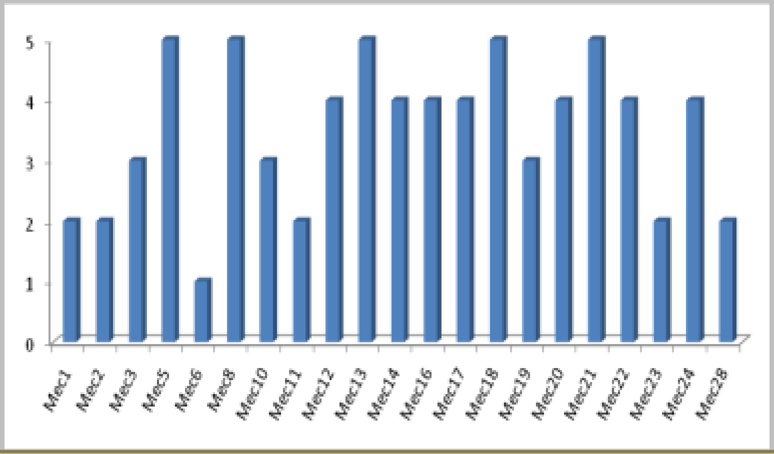
The church was severely damaged by the 6th of April 2009 earthquake.

After the earthquake the church reported severe damage in the apsidal and transept area, where a critical crack pattern was noticed in the external walls, which manifested a visible outward overturning, involving the four pillars sustaining the dome. Also the transversal response of the church proved to be inadequate, since the most part of the vaults collapsed, such as a big portion of the external wall, at the clerestory level. Severe damage was finally reported in the vaults of the apse, of the presbytery, in the triumphal arch.

ACTIVATED DAMAGE MECHANISMS

1ST LEVEL DAMAGE SURVEY FORM FOR CULTURAL HERITAGE - CHURCHES

DAMAGE LEVEL OF THE DIFFERENT ACTIVATED MECHANISMS



DAMAGE INDEX

$$I_d = \frac{d}{5.n}$$

$$n = 22$$

$$d = 73$$

$$I_d = 0.66$$

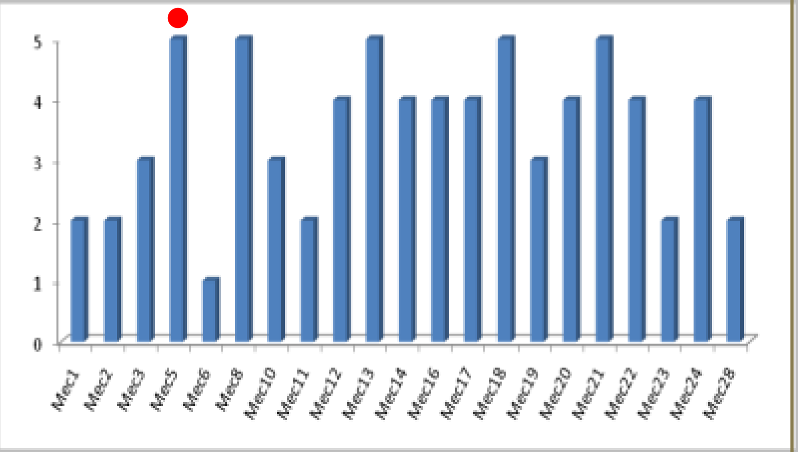
DETAILED ANALYSIS OF THE MECHANISMS, OUTLINING A SET OF POSSIBLE CAUSES THROUGH THE CORRELATION OF THIS MECHANISMS WITH ALL THE PREVIOUSLY GATHERED INFORMATION (CRACK MAP, INTERVENTIONS, TRANSFORMATIONS, ETC...)

ACTIVATED DAMAGE MECHANISMS

MECHANISM

M5. TRANSVERSAL RESPONSE OF THE AULA

DAMAGE LEVEL OF THE DIFFERENT ACTIVATED MECHANISMS

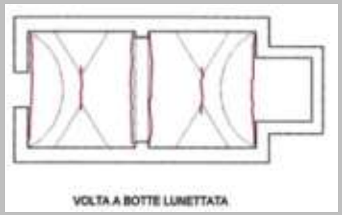


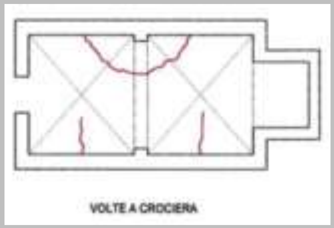
PHOTOS

PHOTOS

ACTIVATED DAMAGE MECHANISMS

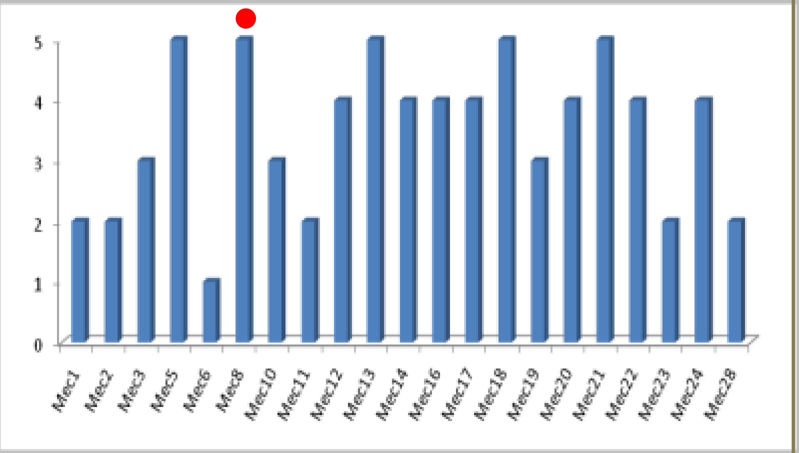
MECHANISM





M8. CENTRAL NAVE VAULTS

DAMAGE LEVEL OF THE DIFFERENT ACTIVATED MECHANISMS



PHOTOS

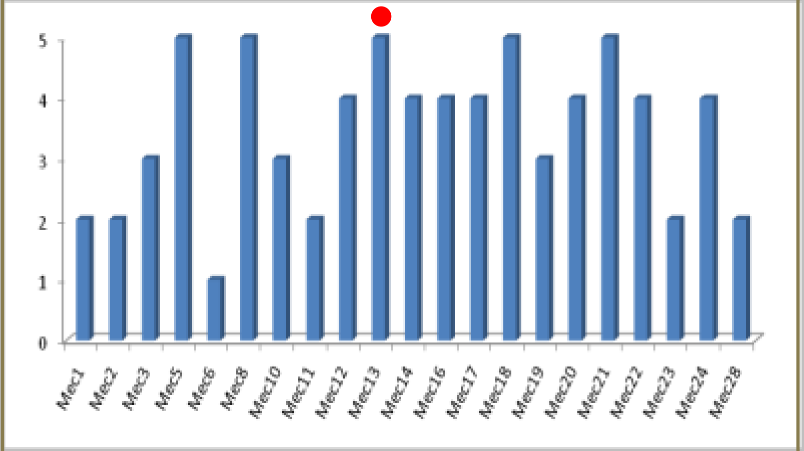
PHOTOS

ACTIVATED DAMAGE MECHANISMS

MECHANISM

M13. "TRIUMPHAL ARCH"

DAMAGE LEVEL OF THE DIFFERENT ACTIVATED MECHANISMS

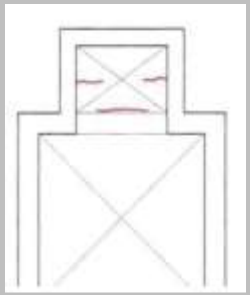


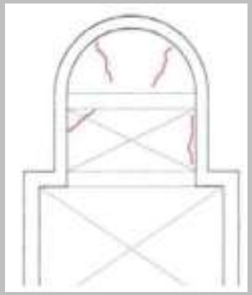
PHOTOS

PHOTOS

ACTIVATED DAMAGE MECHANISMS

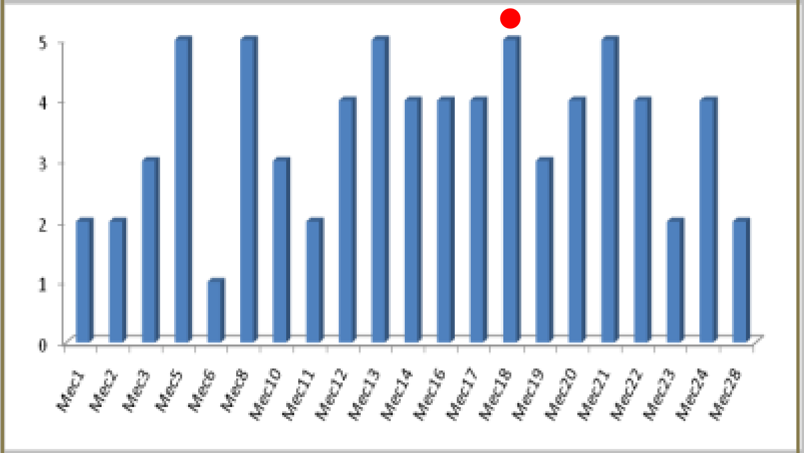
Mechanism





M18. Presbytery or apses vaults

Damage level of the different activated mechanisms

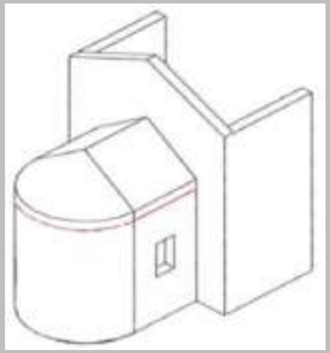


Photos

Photos

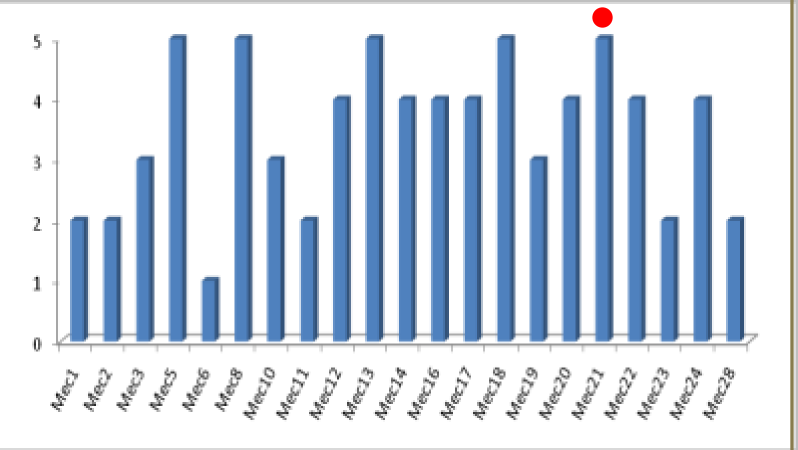
ACTIVATED DAMAGE MECHANISMS

MECHANISM



M21. COVERING ELEMENTS: APSES

DAMAGE LEVEL OF THE DIFFERENT ACTIVATED MECHANISMS



PHOTOS

THE MONITORING SYSTEM

Static: 5 Displacement transducers

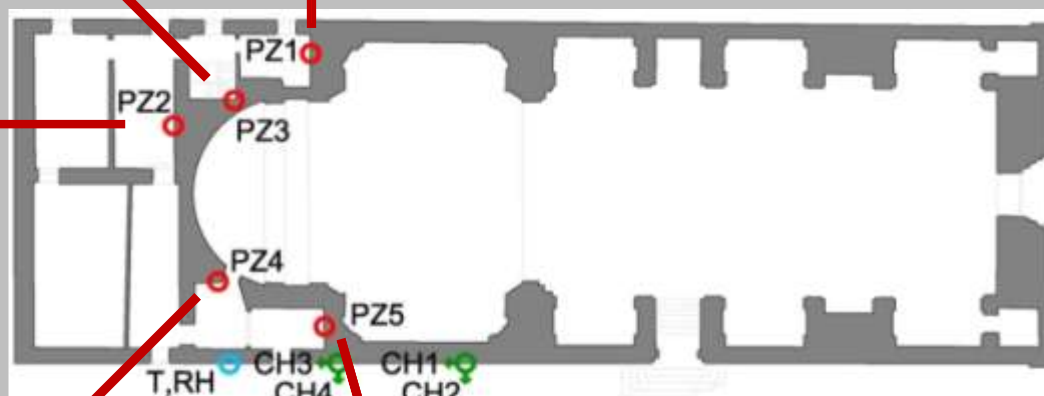


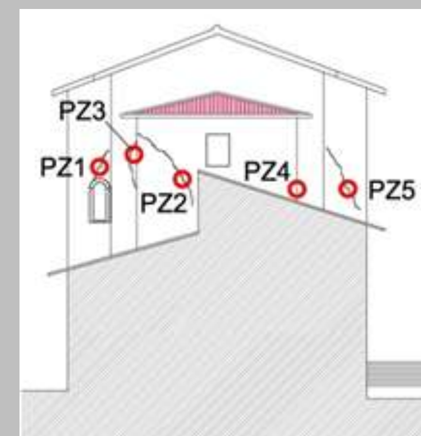
Dynamic: 2+2 accelerometers

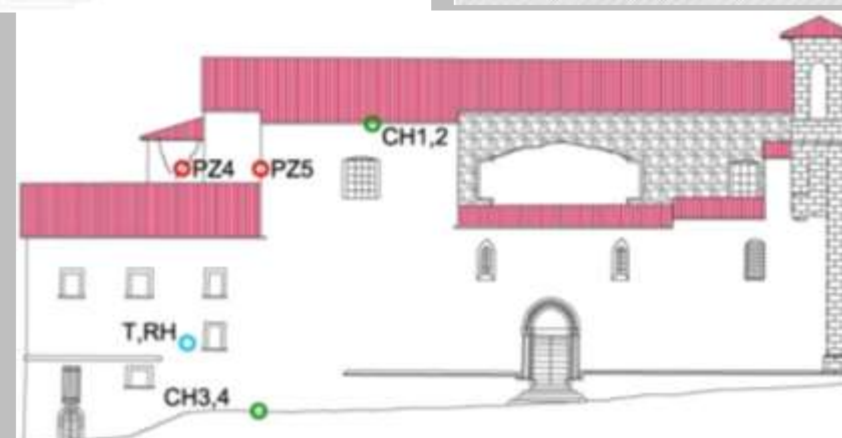

Prof. Claudio Modena

THE MONITORING SYSTEM

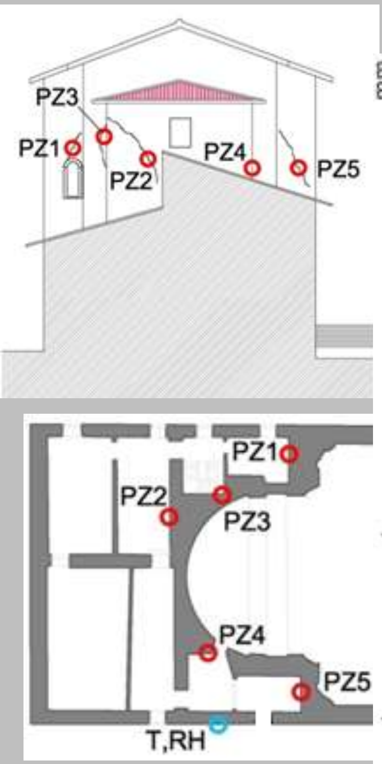
- **STATIC** 5 displacement transducers
- **DINAMIC** 2+2 accelerometers
- **CLIMA** 1 temperature/relative humidity sesnsor

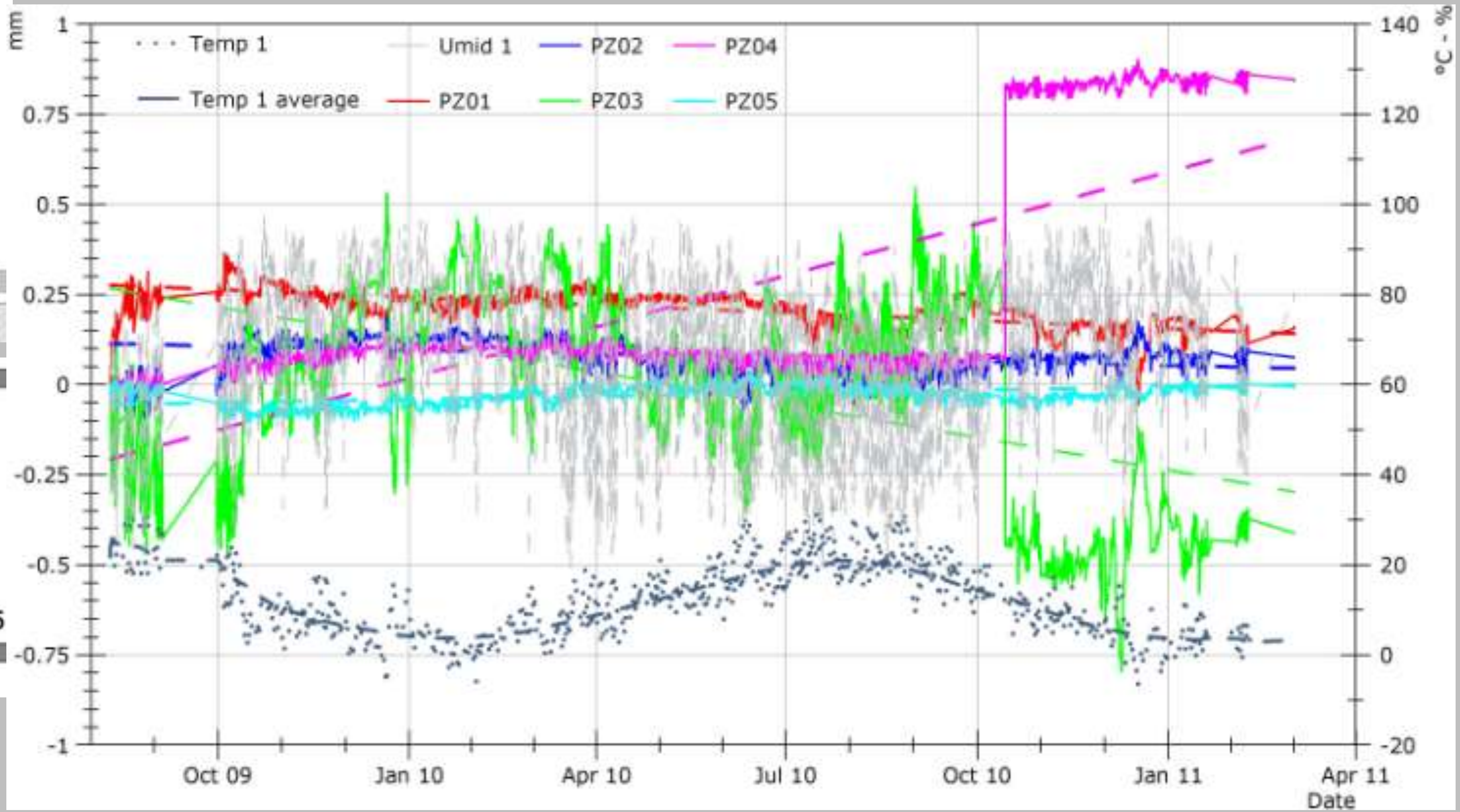






THE MONITORING SYSTEM

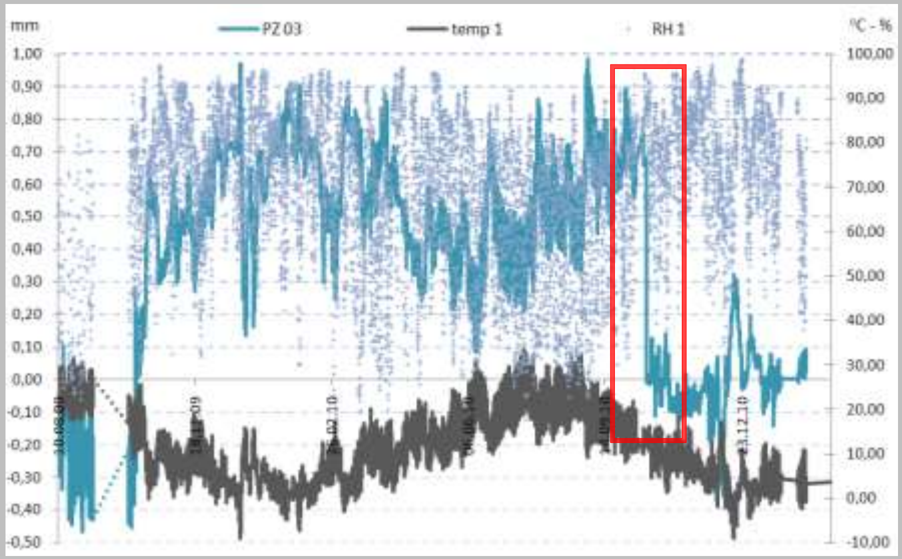


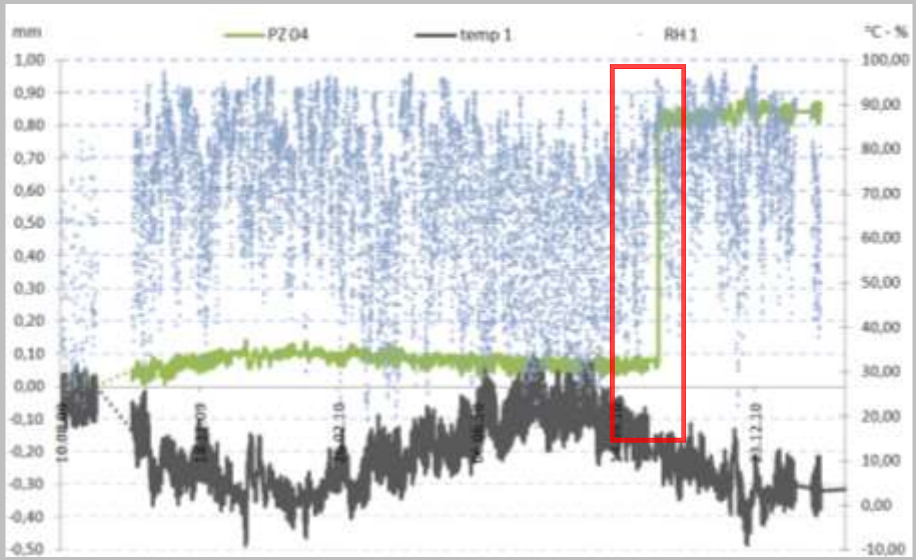


- PZ 2-4-5: constant trend → Stability of the crack pattern
- PZ 1-3: non constant trend → Settling and climate factors

Possible Solutions before and immediately after Earthquake

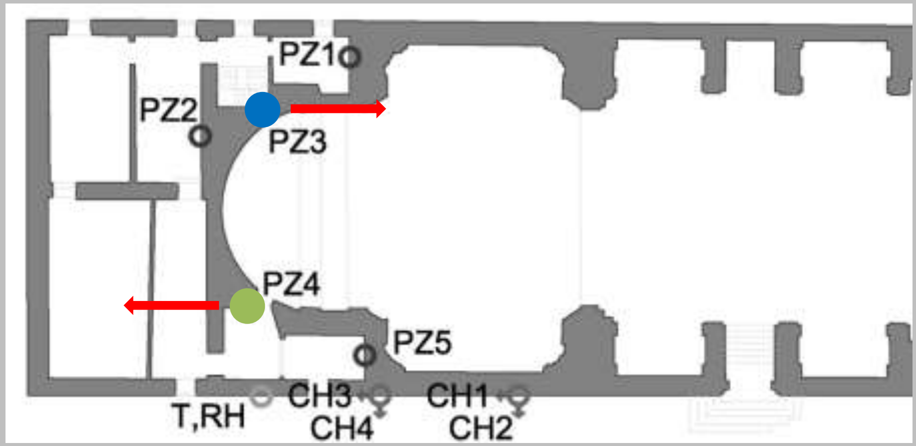
Prof. Claudio Modena



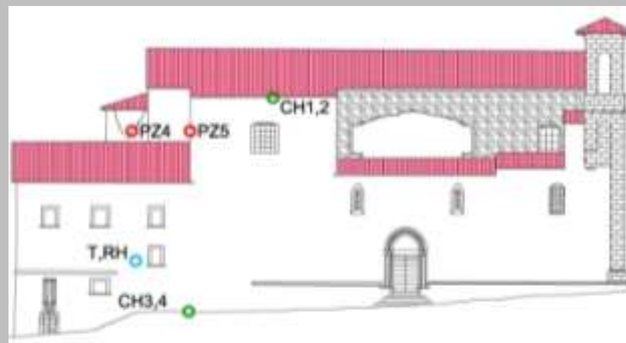


PZ 3 and PZ 4 (14.10.2010)

- same modulus ~ 0.80 mm
- opposite verse
- symmetric position in the apse
- no correlation with aftershocks

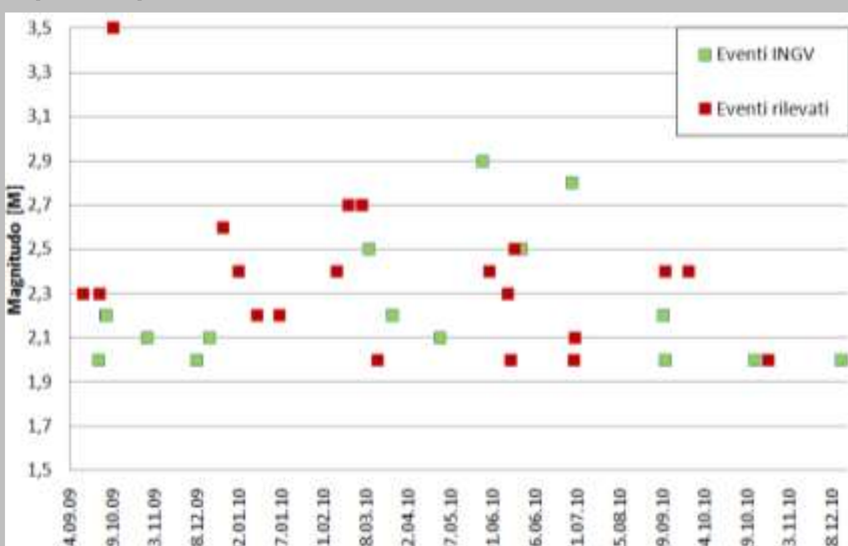


RIGID ROTATION OF THE MACROELEMENT

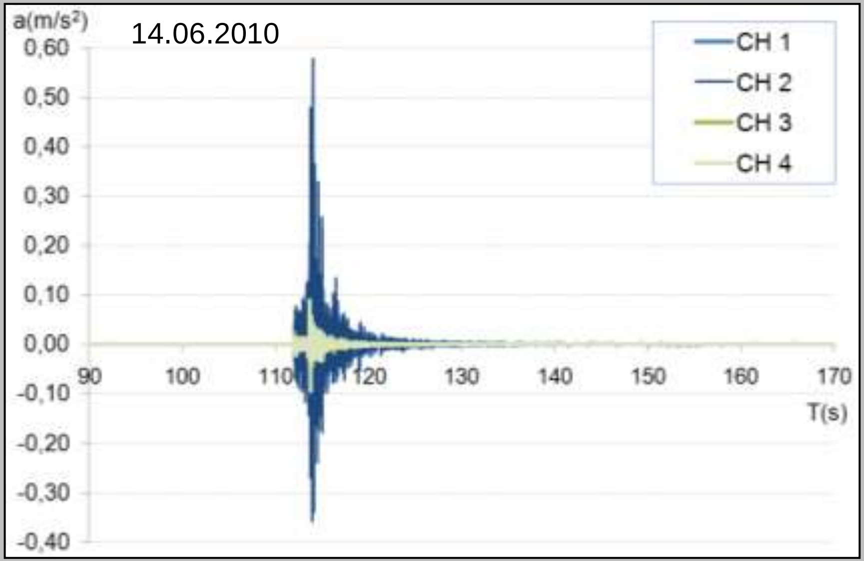


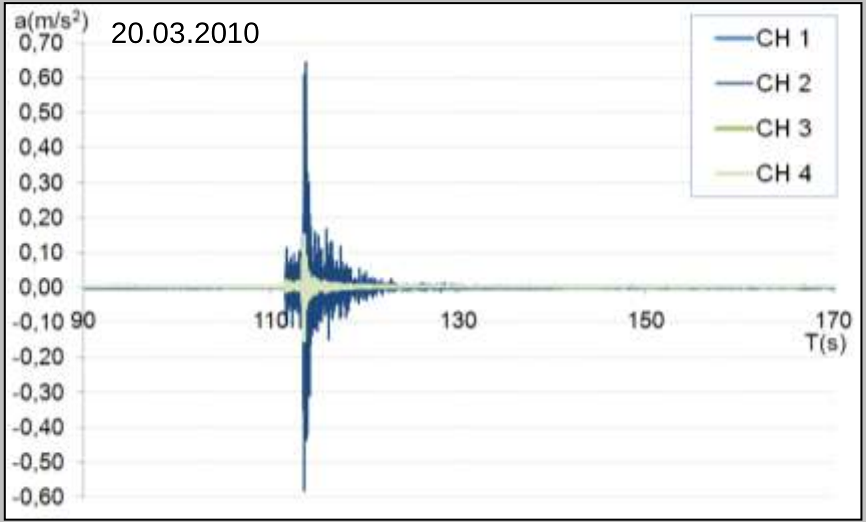
L'Aquila District

- 45 events registered by INGV (M>2)
- 24 events registered by the system (53%)

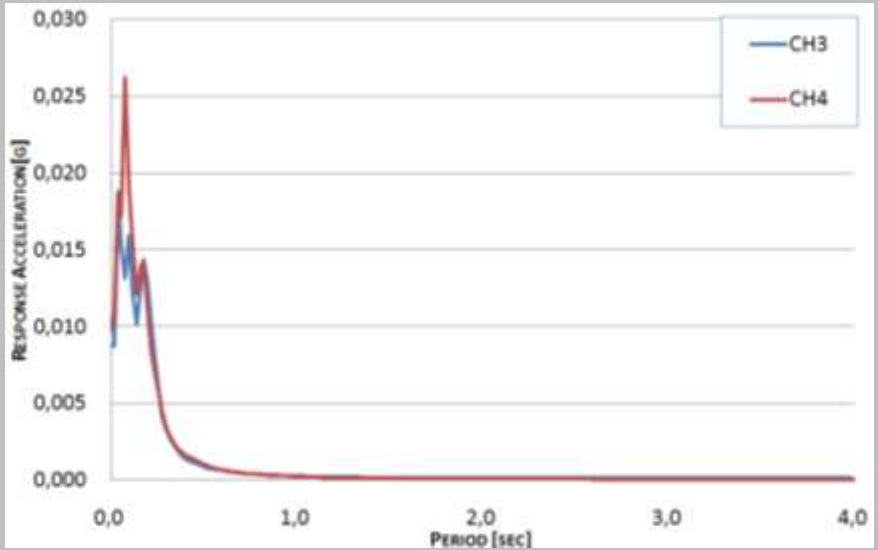


Data	Distanza [km]	Profondità [km]	Magnitudo [M]	Registrazioni CH3	Registrazioni CH4
2010/03/20	6.7	8.0	2.3	0.085547	0.094782
	5.5	9.0	2.2	0.03122	0.042285
2010/03/25	4.6	9.7	2.0	0.026153	0.018568
2010/05/30	6.3	11.2	2.4	0.039126	0.029275
2010/06/10	2.2	10.6	2.3	0.079613	0.063702
2010/06/12	12.4	12.8	2.0	0.008141	0.00742
	12.6	11.1	2.1	0.007932	0.012373
2010/06/14	6.3	10.4	2.5	0.080649	0.151613
2010/07/19	13.6	10.4	2.0	0.018472	0.021337
	13.8	10.1	2.1	0.007065	0.006093
2010/07/20	13.5	10	2.1	0.008766	0.012988
2010/09/11	13.3	9.9	2.4	0.015685	0.013646
2010/09/25	9.2	10.8	2.4	0.011675	0.013836
2010/11/03	7.1	8.7	2.0	0.013035	0.014509
2010/11/11	11.3	13.8	2.0	0.025378	0.029088





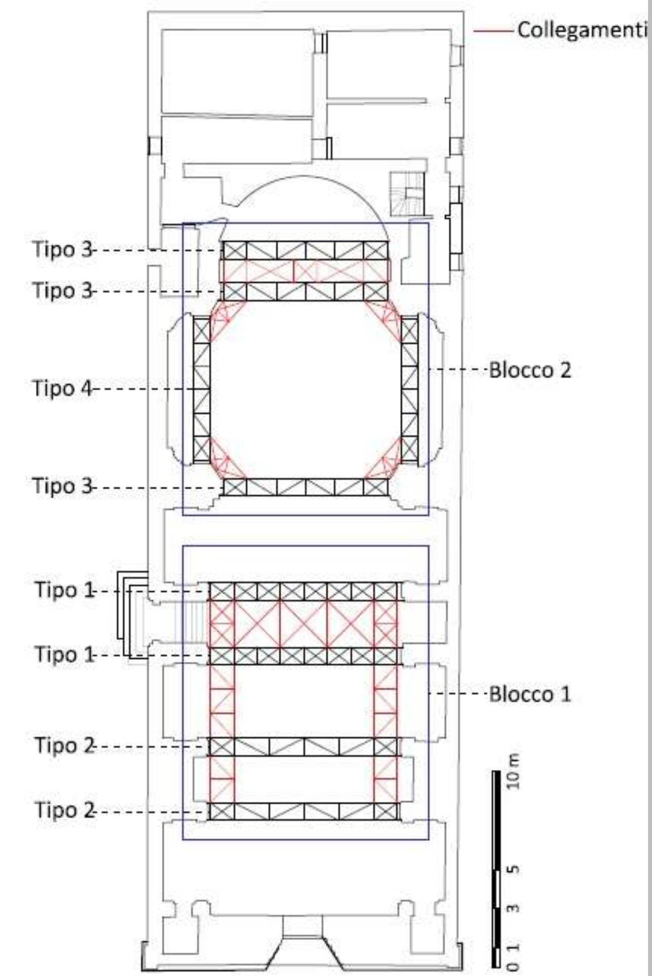
Data	Dist [km]	Prof [km]	m	CH 3	CH4
20.03. 10	6.7	8.0	2.3	0.085547	0.094782
14.06. 10	6.3	10.4	2.5	0.080649	0.151613

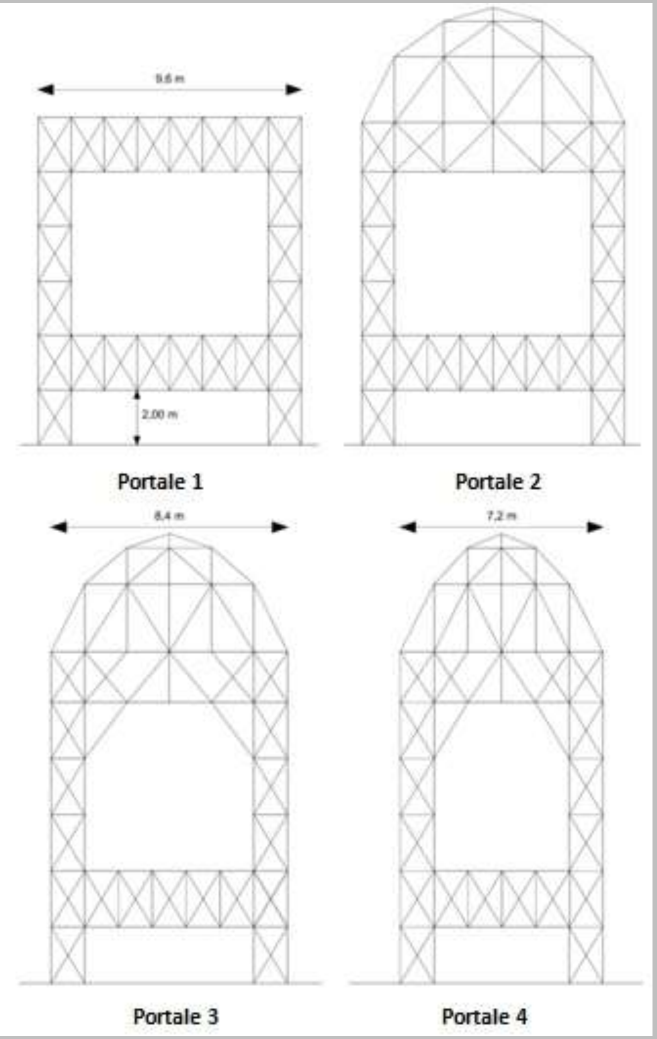


- Obtained input spectra

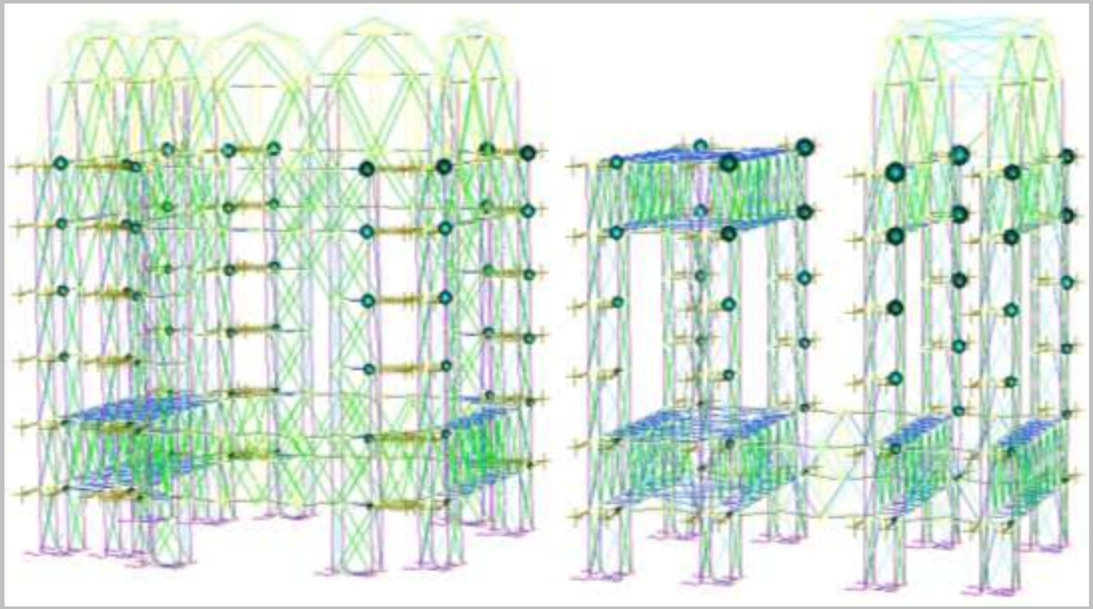
DESIGN OF THE INTERVENTION: INTERNAL SCAFFOLING

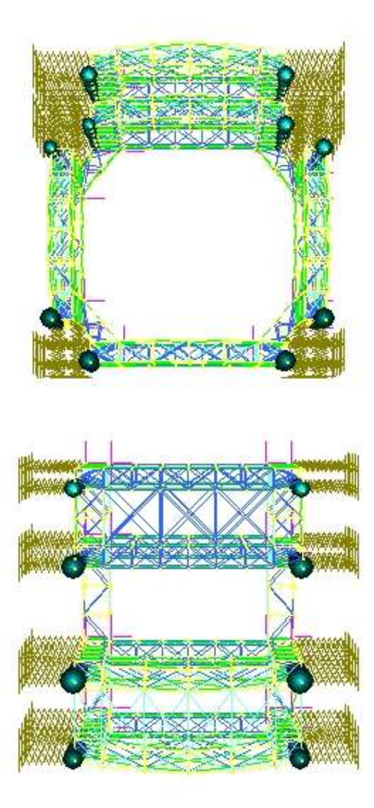
2 BLOCKS → 4 PORTALS



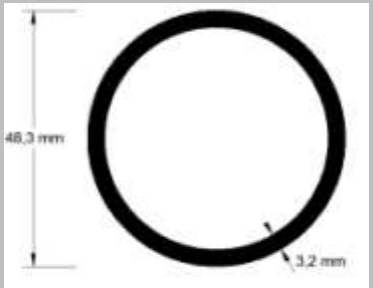


DESIGN OF THE INTERVENTION: INTERNAL SCAFFOLING





- FE modeling
- 4133 elements,

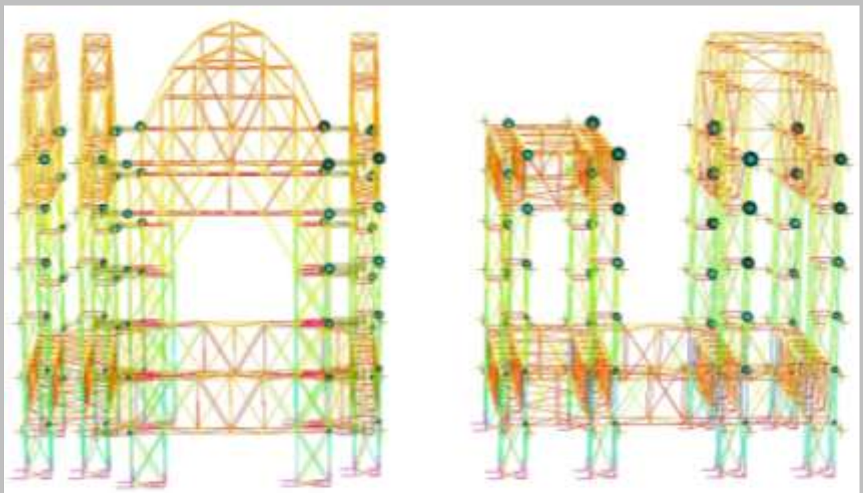


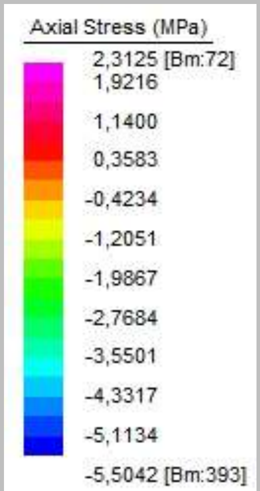
- Steel hallow tubes:

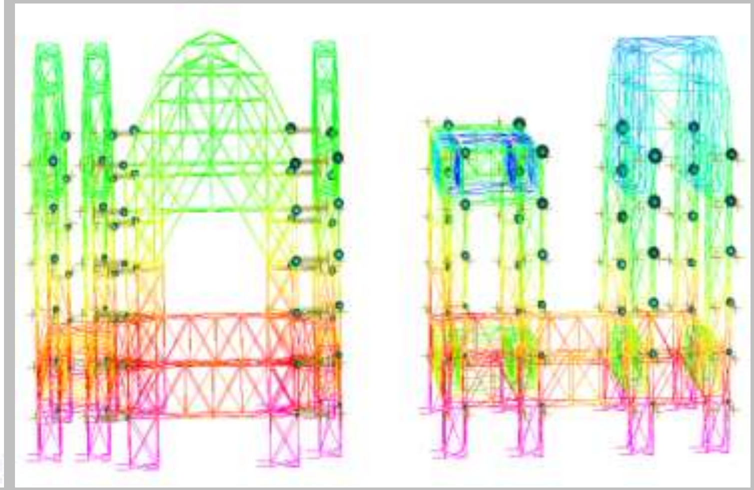
  D = 48,3 mm ; s = 3,2 mm

DESIGN OF THE INTERVENTION: INTERNAL SCAFFOLING

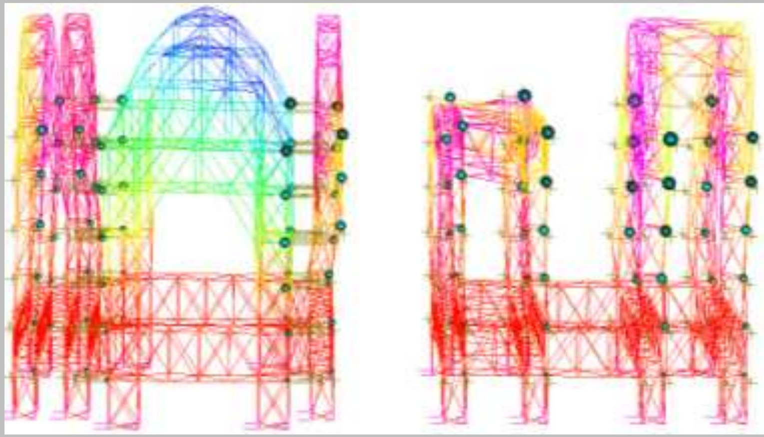
Static analysis

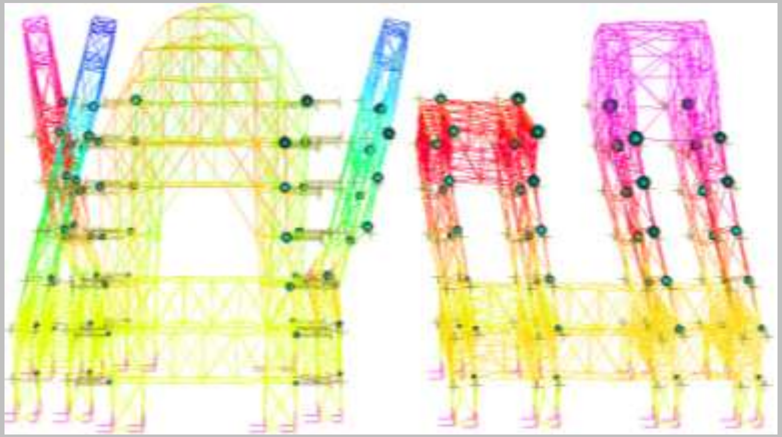






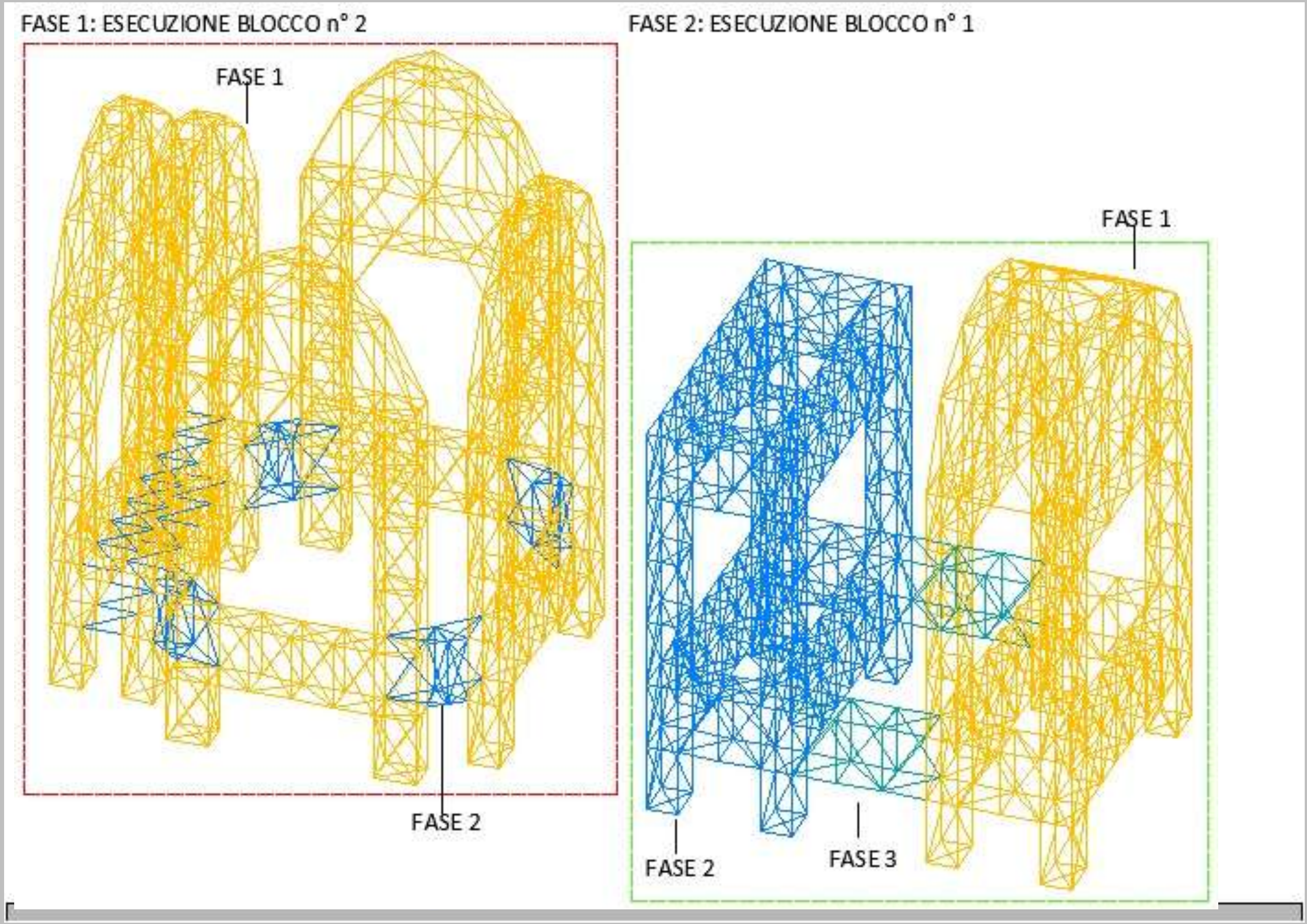
Linear Dynamic Analysis with response spectra





DESIGN OF THE INTERVENTION: INTERNAL SCAFFOLING

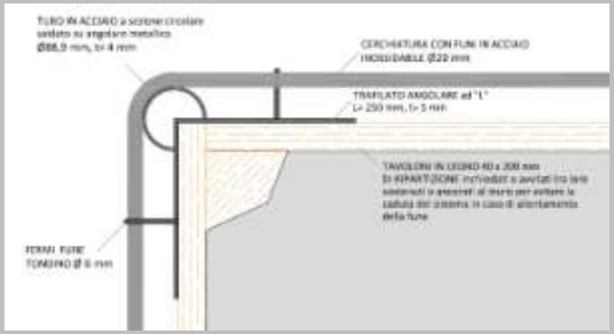
EXECUTION PHASES

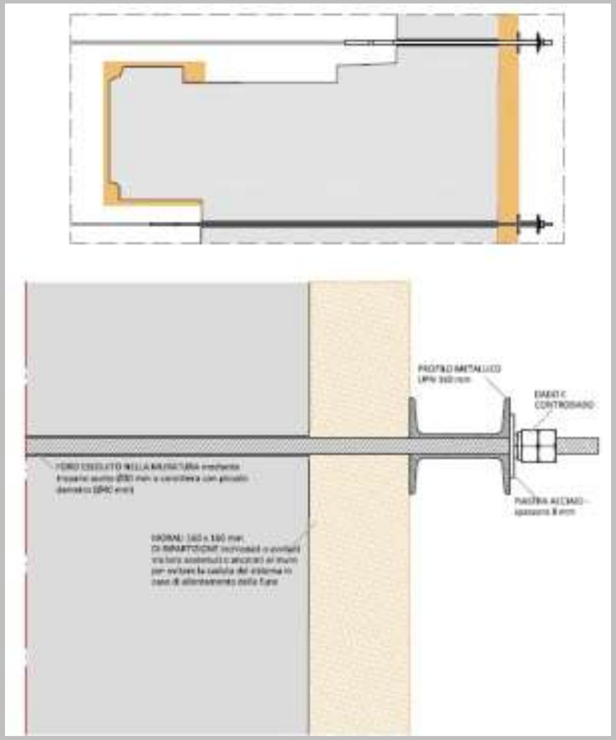


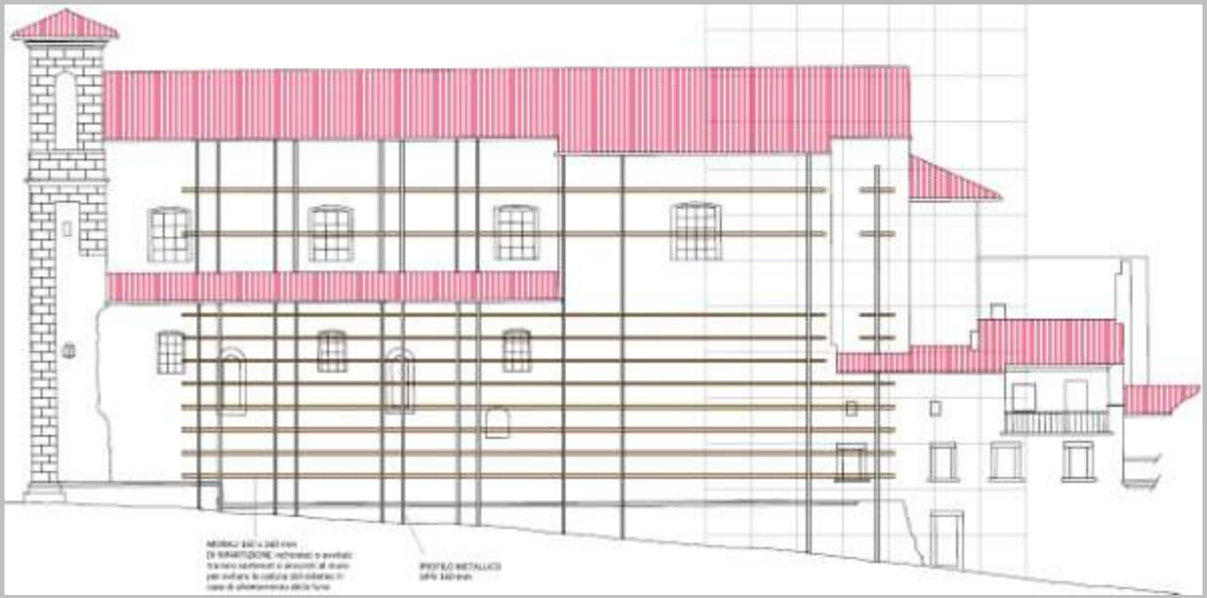
PROF. CLAUDIO MODENA

DESIGN OF THE INTERVENTION: INTERNAL SCAFFOLING

- System of ties at different hights to confine the structure
- Steel ties ϕ 16 anchored with 2 steel profiles UPN 160 mm
- 1 tie every 2 meters: 6 ties on the total height







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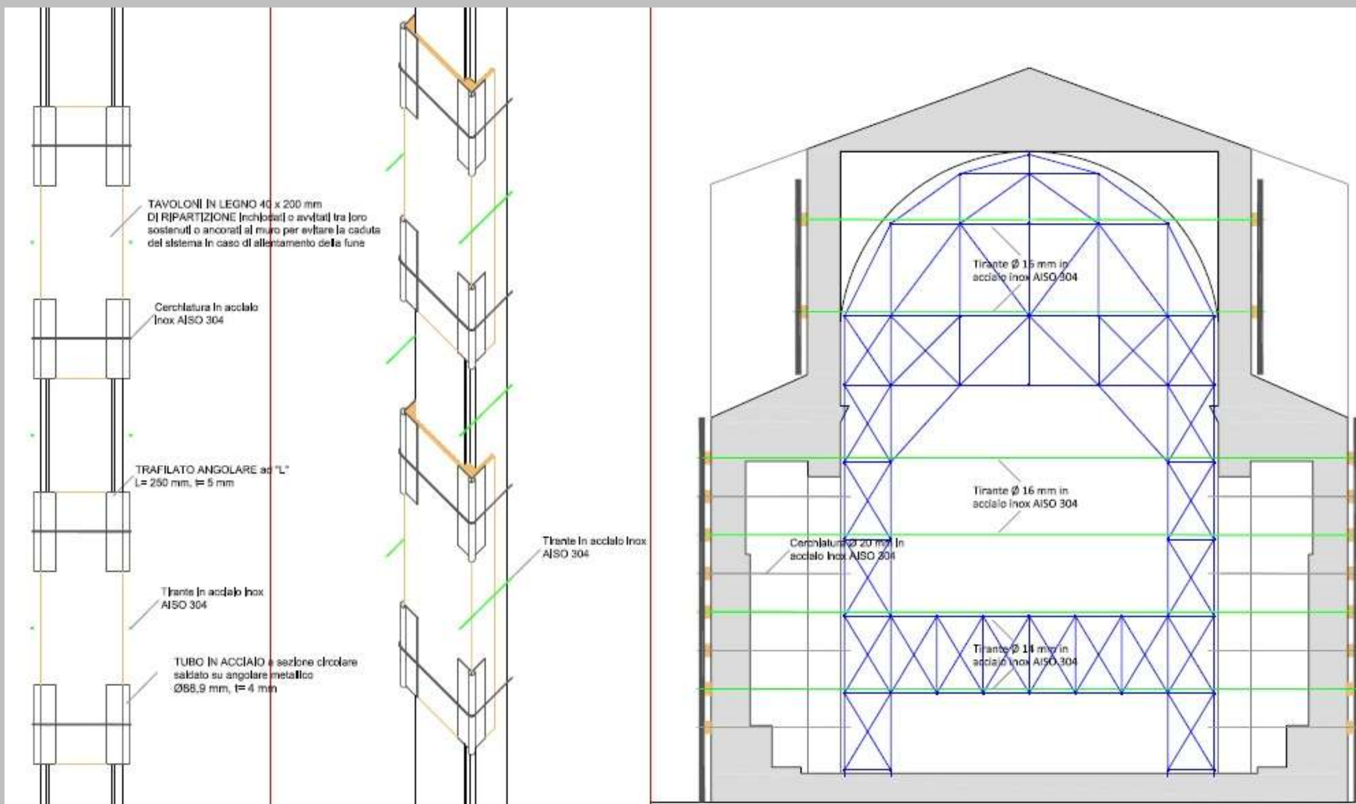
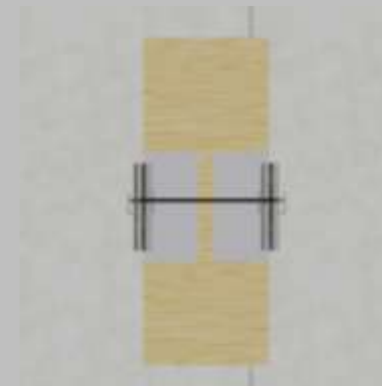
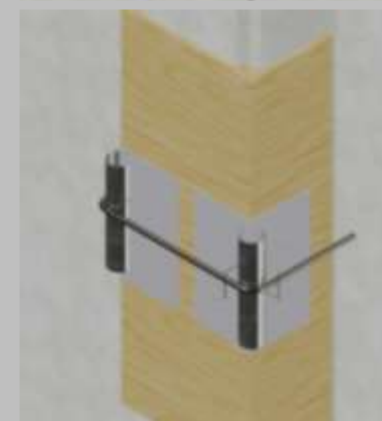
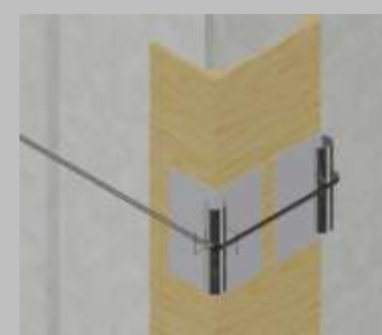
DESIGN OF THE INTERVENTION: INTERNAL SCAFFOLING









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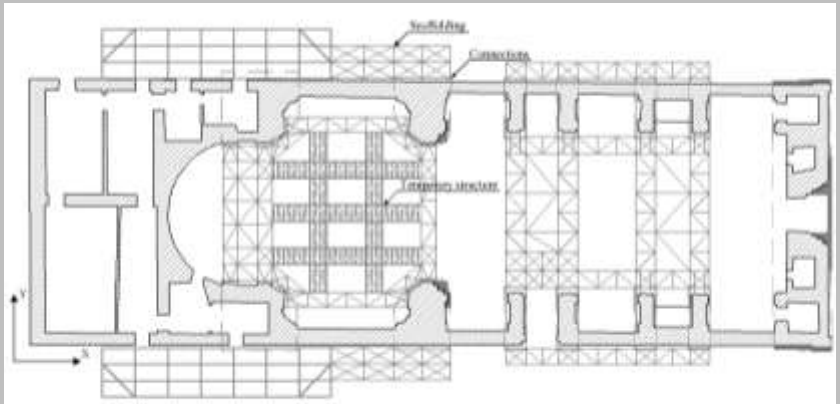
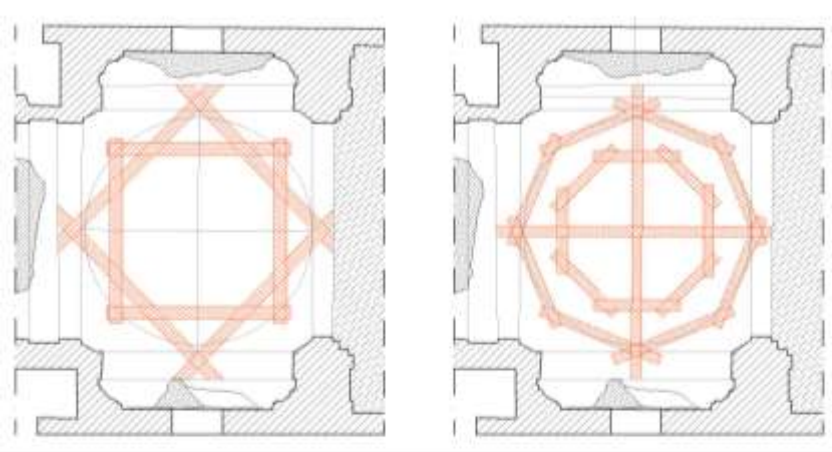
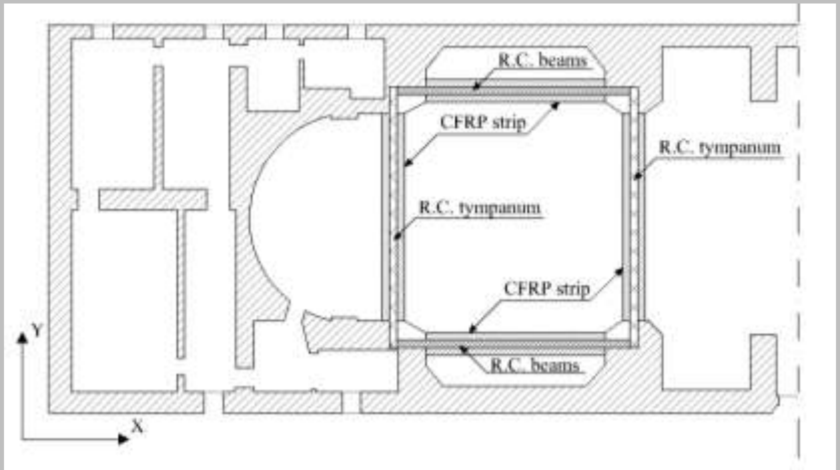
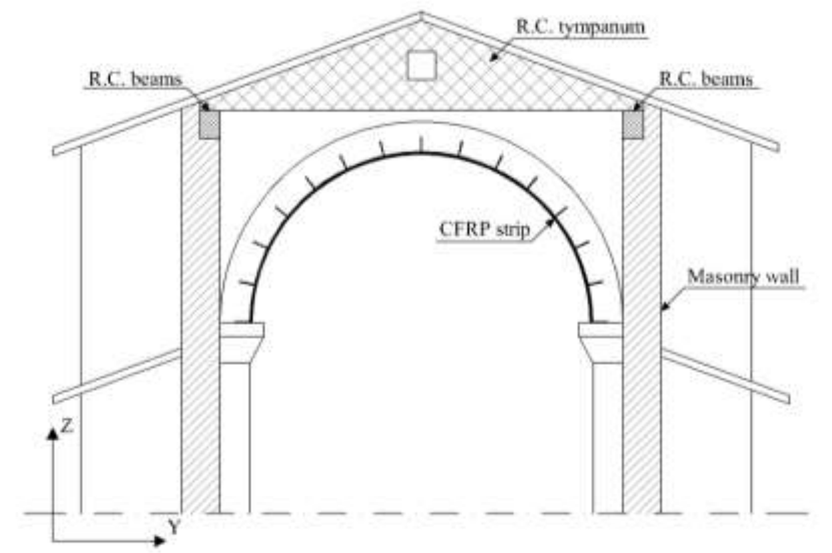
DESIGN OF THE INTERVENTION









THE TOMB OF DAVID AND CENACULUM - JERUSALEM, ISRAEL

The main object is the seismic analysis and the structural assessment of a part of the monumental historical complex on Mount Zion, located at the south-west corner of the old city of Jerusalem, outside the walls. In particular the study is concentrated on the structural unit that contains the Tomb of David on the ground floor and the Room of the Last Supper (Cenaculum) on the upper floor.

Possible Solutions before and immediately after Earthquake

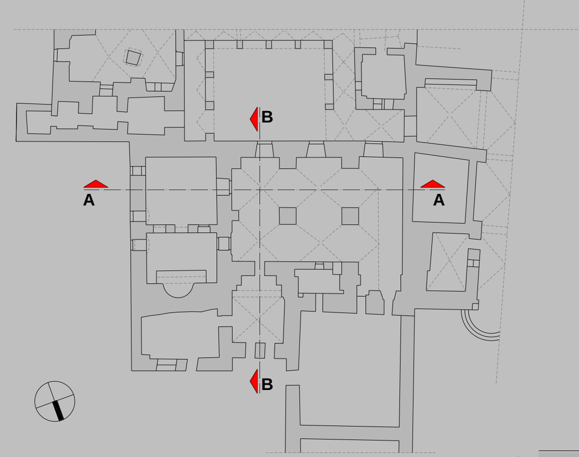
Prof. Claudio Modena

GEOMETRIC SURVEY

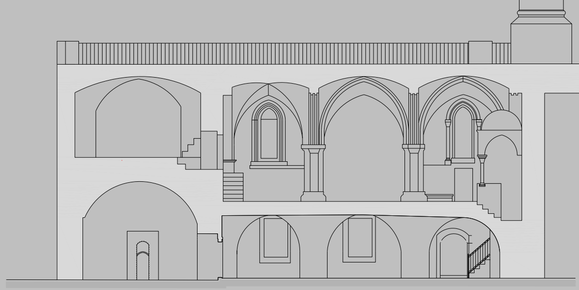
On the ground floor there is the "Tomb of David" with a system of groin vaults in the main entrance. The tomb is located in the eastern part of the floor and it is inserted in a room under a huge barrel vault

The "Room of the Last Supper" or "Cenaculum" is located on the first floor. It has a beautiful system of rib vaults supported by the perimeter walls of the room and two pillars in middle of it.

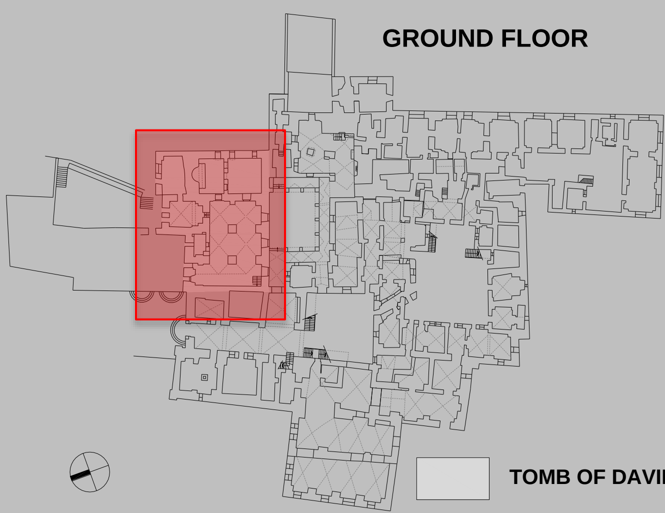
GROUND FLOOR: TOMB OF DAVID



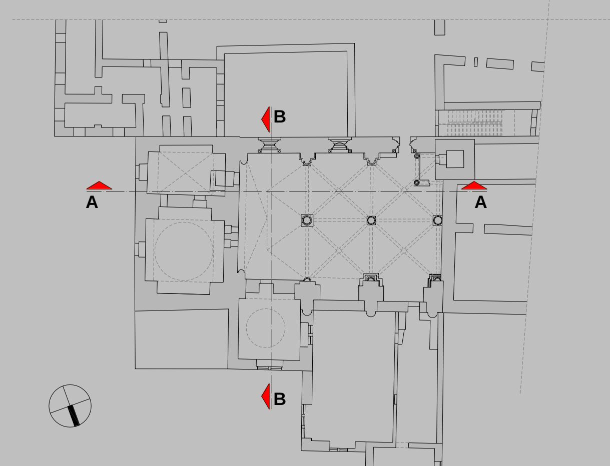
SECTION A-A



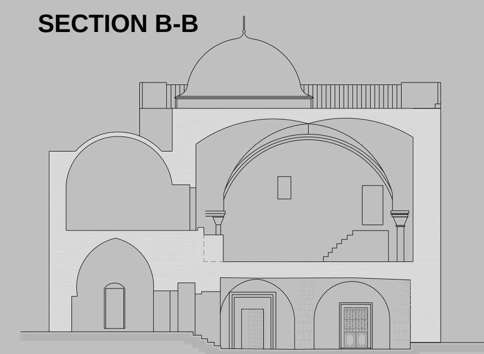
GROUND FLOOR



FIRST FLOOR: CENACULUM



SECTION B-B

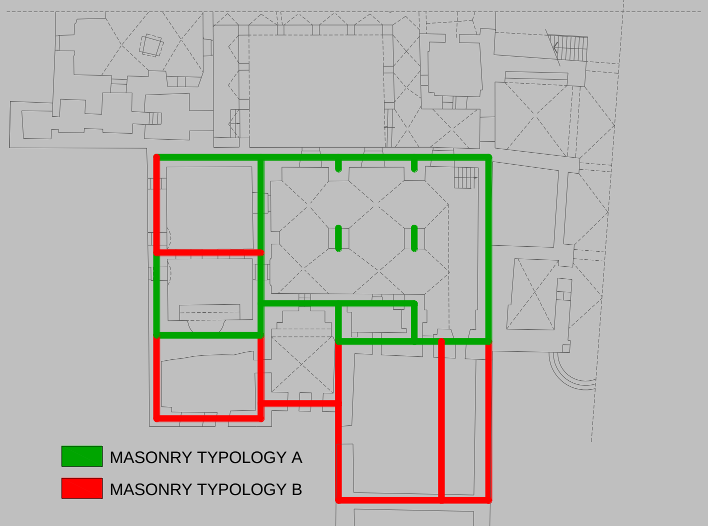


MATERIALS

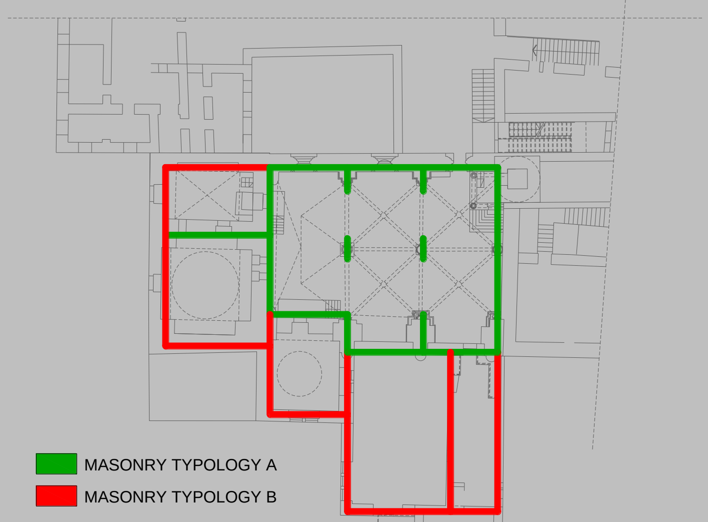
The mechanical properties of stone masonry are derived from the table C8A.2.1 of the *Circolare 2 febbraio 2009, n. 617 C.S.LL.PP. “Istruzioni per l’applicazione delle «Nuove norme tecniche per le costruzioni»*, which gives a range of values of the principal mechanical parameters of different kind of masonry. After a detailed critical survey of the masonry walls and in situ inspections of the building it was possible to identify two different masonry typologies

MECHANICAL PROPERTIES OF MASONRY [TAB. C8A.2.1]									
MASONRY TYPOLOGY A	fm [N·cm^{-2}]		τ_0 [N·cm^{-2}]		E [N·mm^{-2}]		G [N·mm^{-2}]		w [kN·m^{-3}]
Squared blocks stone masonry with good texture	min	max	min	max	min	max	min	max	
	260	380	5,6	7,4	1500	1980	500	660	21
MASONRY TYPOLOGY B	fm [N·cm^{-2}]		τ_0 [N·cm^{-2}]		E [N·mm^{-2}]		G [N·mm^{-2}]		w [kN·m^{-3}]
Irregular stone masonry with inner core	min	max	min	max	min	max	min	max	
	200	300	3,5	5,1	1020	1440	340	480	20

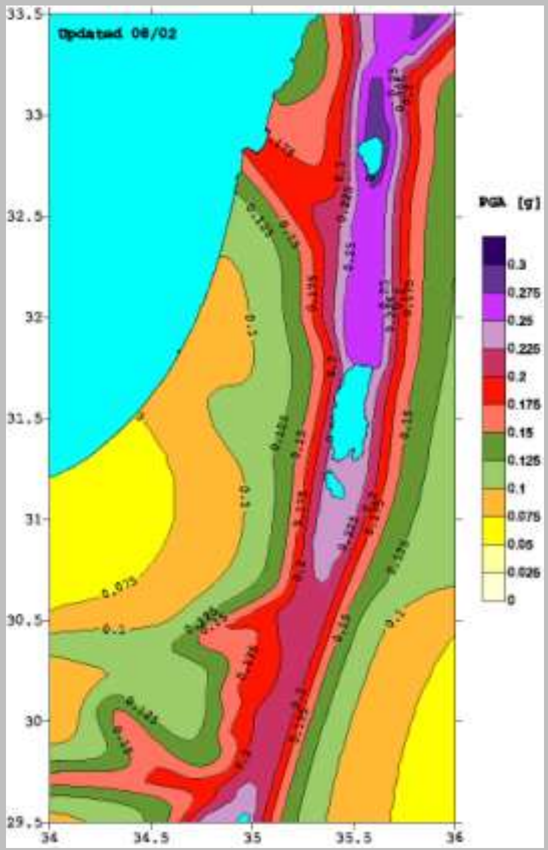
GROUND FLOOR



FIRST FLOOR

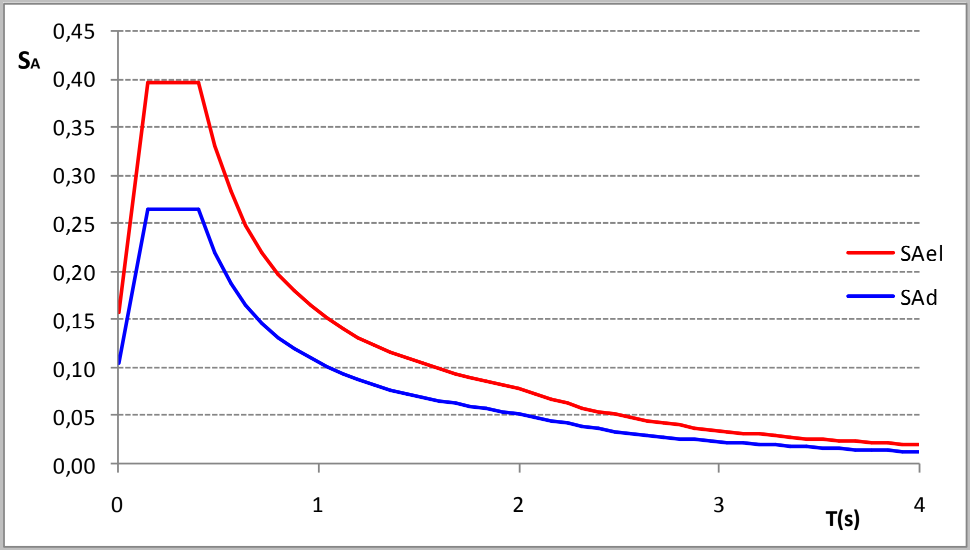


DEFINITION OF THE SEISMIC ACTION



	Peak ground acceleration a_{gR}	geographic coordinates	
		longitude	latitude
Jerusalem	0.132	35°12'E	31°47'N

PARAMETERS		VALUES
Ground Type	A	/
Reference peak ground acceleration on type A ground	a_{gR}	0.132 g
Soil Factor	S	1.00
Periods defining the elastic response spectrum	T_B	0.15 s
	T_C	0.4 s
	T_D	2.0 s
Importance Factor	γ_I	1.2
Behaviour Factor	q	1.5



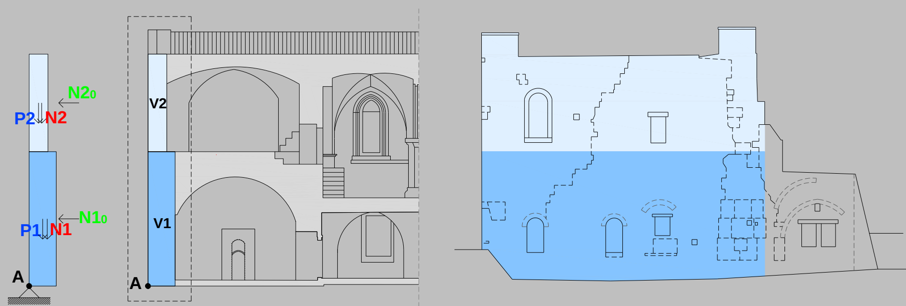
SIMPLIFIED ANALYSIS: LIMIT ANALYSIS METHOD

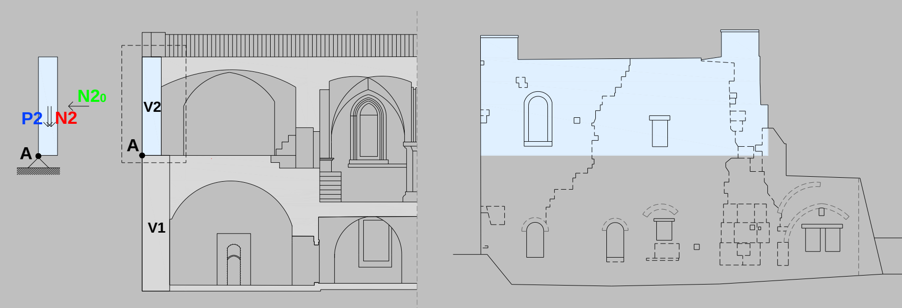
This method, proposed by the Italian code, is based on the failure mechanisms observed in masonry buildings after severe seismic events, and it is based on the evaluation of the limit analysis of masonry portions - considered as rigid blocks - subjected to their self weight (stabilising effect) and horizontal forces (earthquake action).

DEFINITION OF THE MACROELEMENTS

VERIFICATIONS

MACROELEMENT A → NOT VERIFIED!





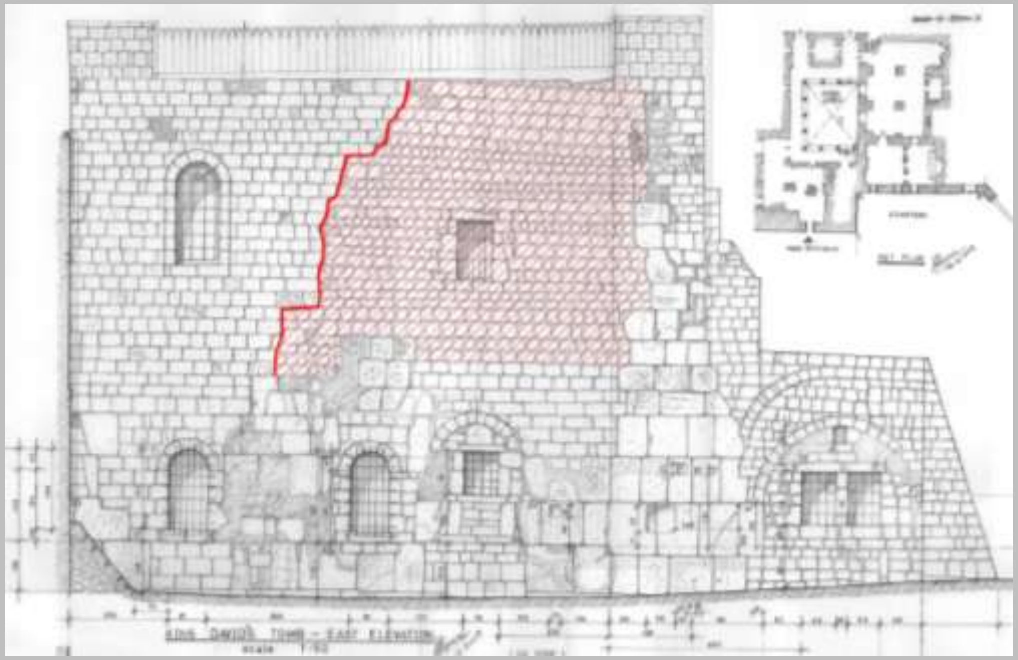
GLOBAL									
	LINEAR ANALYSIS								
	t [m]	M_S [kNm]	M_R [kNm]	α_0	M^* [t]	e^*	a_0^* [m/s²]	a_0^* [m/s²]	VERIFICATION: $a_0^* \geq a_0^*$
	0,261	1922,03	26362,14	0,073	405,75	0,79	0,668	0,777	NO
	NON-LINEAR ANALYSIS								
	θ [rad]	d_{k0} [m]	d_0^* [m]	d_u^* [m]	d_s^* [m]	VERIFICATION: $d_u^* \geq d_s^*$ [m]			
	0,073	0,382	0,482	0,193	0,079	YES*			

PARTIAL									
	LINEAR ANALYSIS								
	t [m]	M_S [kNm]	M_R [kNm]	α_0	M^* [t]	e^*	a_0^* [m/s²]	a_0^* [m/s²]	VERIFICATION: $a_0^* \geq a_0^*$
	0,093	171,35	4204,73	0,041	181,62	1,00	0,30	1,35	NO
	NON-LINEAR ANALYSIS								
	θ [rad]	d_{k0} [m]	d_0^* [m]	d_u^* [m]	d_s^* [m]	VERIFICATION: $d_u^* \geq d_s^*$ [m]			
	0,08	0,189	0,189	0,075	0,086	NO			

ALL THE OTHER MACROELEMENTS → VERIFIED!

DESIGN OF THE INTERVENTIONS

The limit analysis showed that the most vulnerable structural element in relation to the seismic risk is the eastern façade on the cemetery. The thrusts of the barrel vault on the ground floor and of the vault and cupola on the first floor are particularly high and induce to a precarious stability condition of the whole structural system. This is also testifies by the fact that most likely the façade has been reconstructed several times during centuries, since it is possible to recognize different kind of stone and different textures and arrangements of stones in the façade's elevation

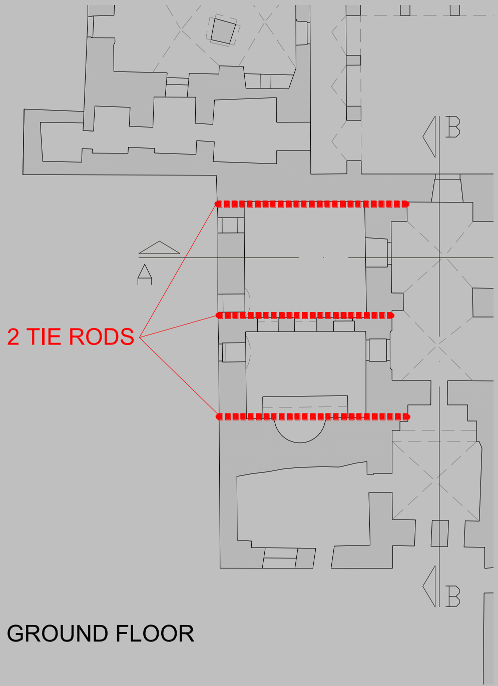


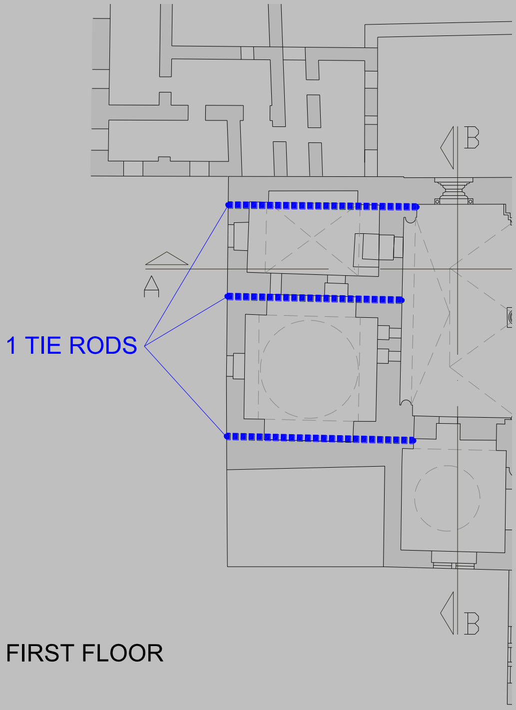
Possible Solutions before and immediately after Earthquake

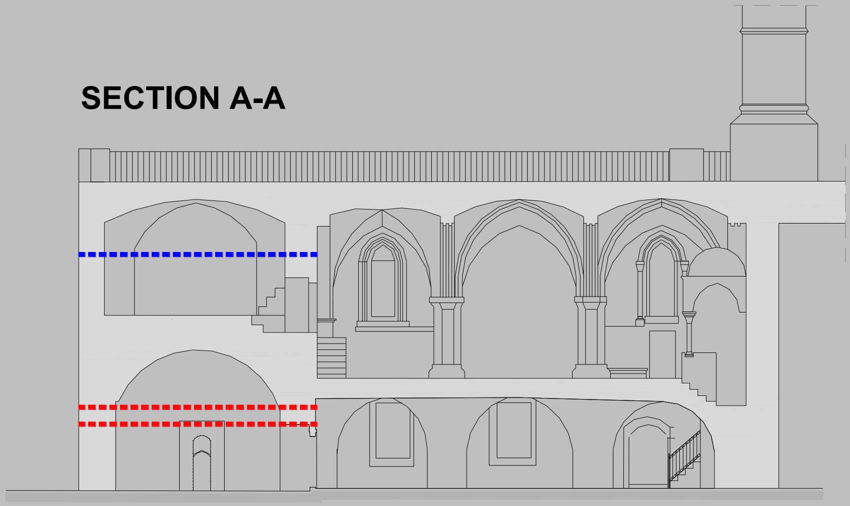
Prof. Claudio Modena

1. GROUT INJECTIONS

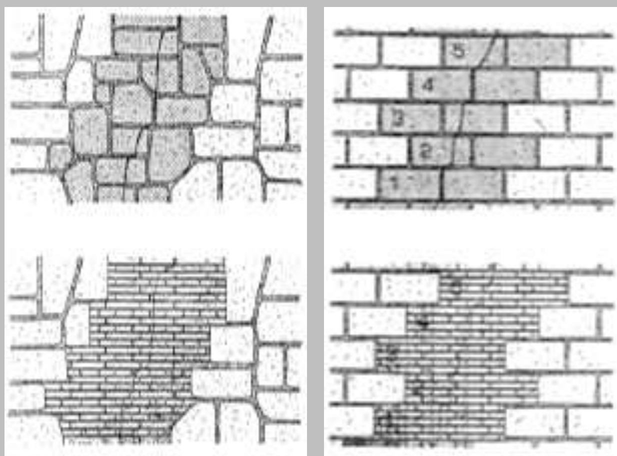
2. INSERTION OF TIES

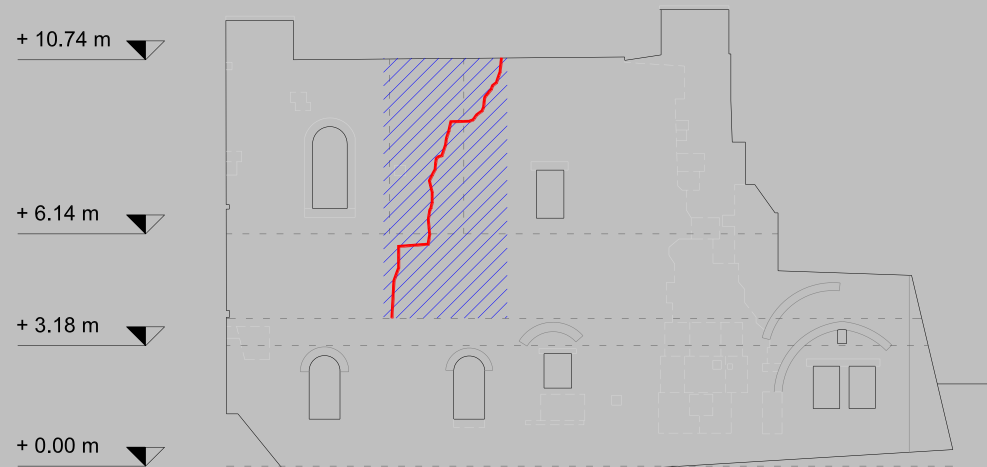


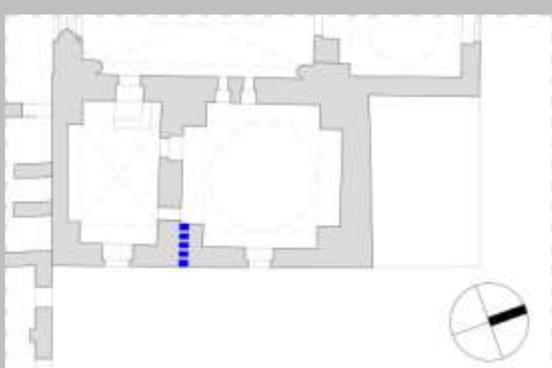


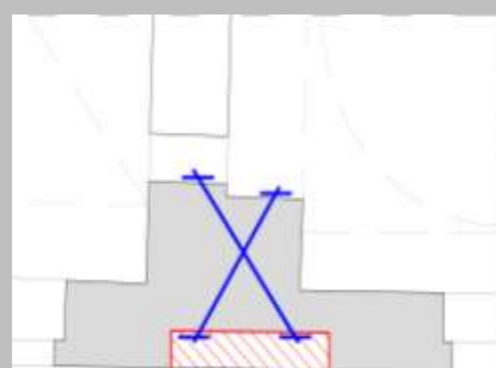


3. LOCAL REBUILDING ("SCUCI-CUCI") AND INSERTION OF TIE RODS IN THE WALL THICKNESS



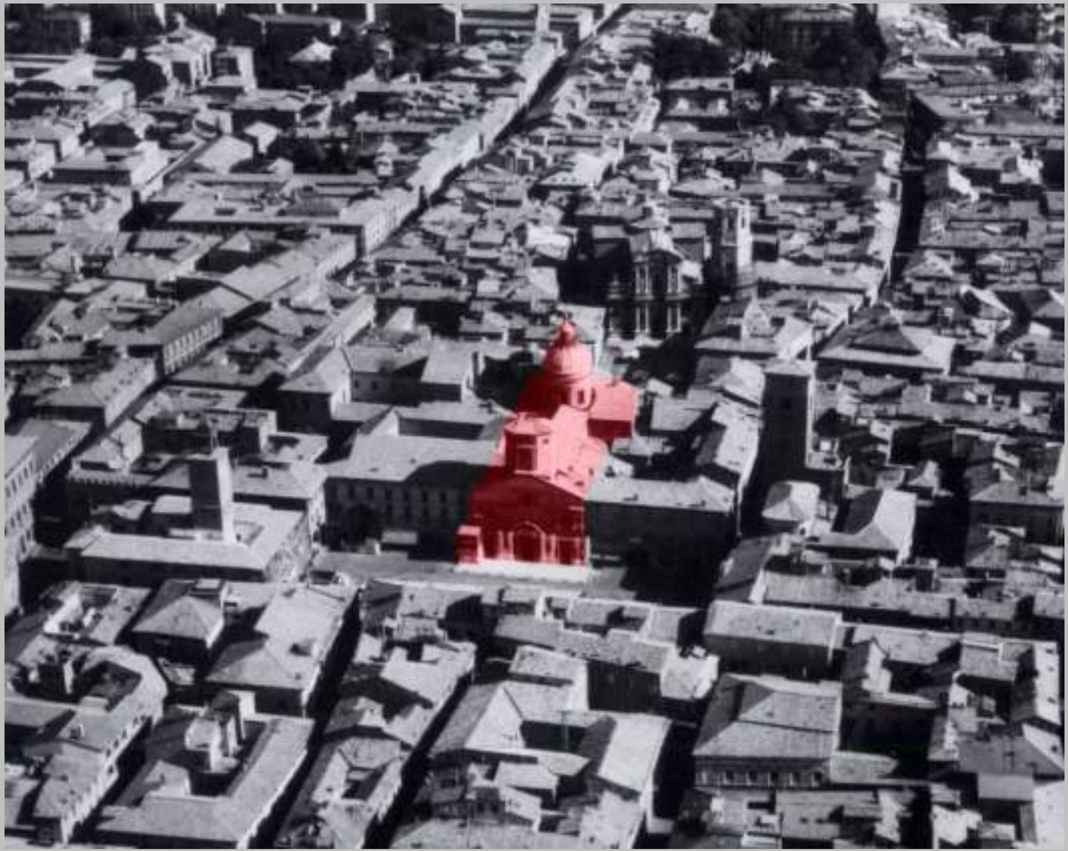






In addition to the local rebuilding in the area where the "scuci-cuci technique" is applied it is suggested to connect the external leaf of the masonry wall with the internal one in order to avoid the mechanism of layers' delamination (overturning of the external leaf) by inserting steel tie rods in the masonry thickness

INVESTIGATION AND MONITORING FOR THE DESIGN OF STRENGTHENING INTERVENTION ON SANTA MARIA ASSUNTA CATHEDRAL - REGGIO EMILIA



Main nave

Roman mosaic under the crypt

Main façade

Cupola from below

POSSIBLE SOLUTIONS BEFORE AND IMMEDIATELY AFTER EARTHQUAKE

PROF. CLAUDIO MODENA

1

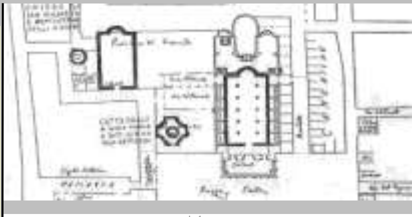
Adopted methology:

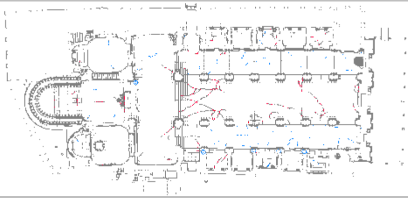
In a first instance an extensive knowledge of the fabric was achieved, considering

i) its construction history and structural evolution,
ii) damage manifested by the building after the historic seismic events and
iii) the present day crack pattern

→ **A complete damage survey of the building was carried out, as a redefinition of the geometrical survey provided, lacking in necessary data to feed the successive modelling phase**

iv) a wide on-site investigation campaign was carried out, including local analyses aiming at qualitatively defining the composition of the investigated structural elements (sonic pulse velocity tests) and at quantitatively characterize the dynamic response of relevant portions of the complex (AVT dynamic tests).



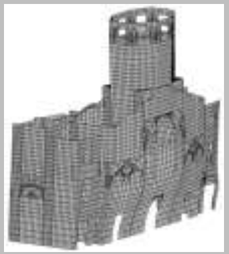


2

v) Finally the seismic assessment of relevant parts of the structure, involving different modelling strategies, was carried out.

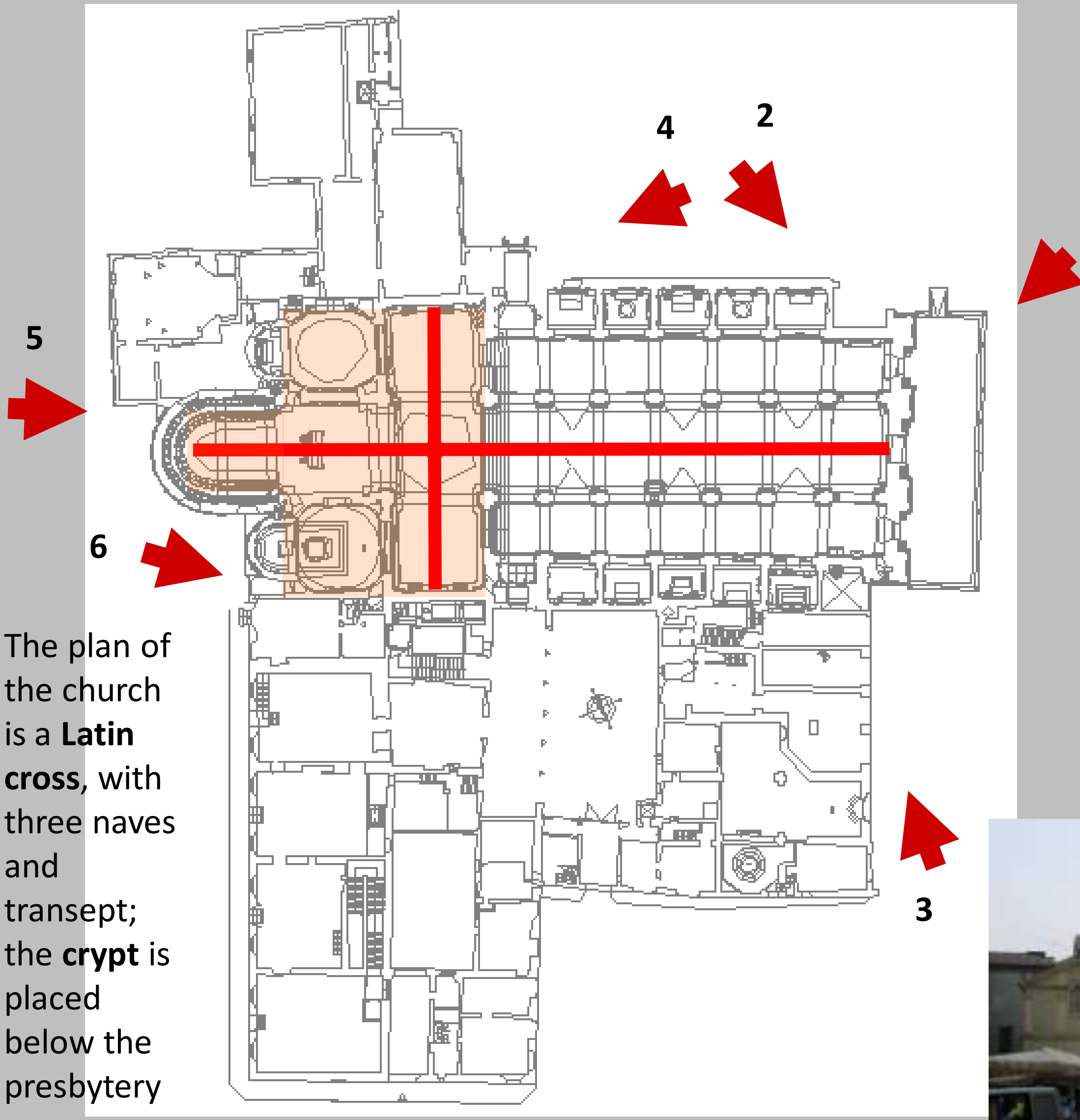
↓

- ***Global* linear elastic numerical models, calibrated on the basis of the results of the experimental phase, were used to feed *local* non linear numerical models**
- ***Push-over* analyses were carried out on the non linear models in order to obtain seismic capacity curves, to be confirmed by the actual seismic damage pattern**
- **Limit analysis was comparatively used to define the seismic capacity of selected *macroelements*, defined in the numerical models corresponding areas.**



POSSIBLE SOLUTIONS BEFORE AND IMMEDIATELY AFTER EARTHQUAKE

PROF. CLAUDIO MODENA



The plan of the church is a **Latin cross**, with three naves and transept; the **crypt** is placed below the presbytery

1

2

3

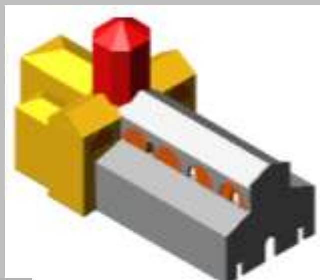
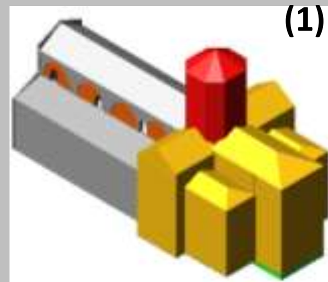
4

5

6

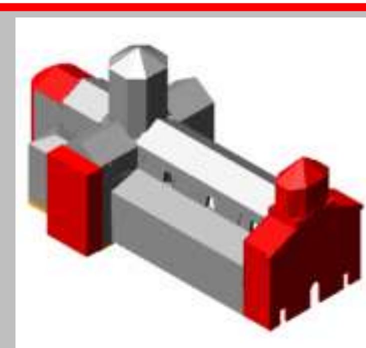
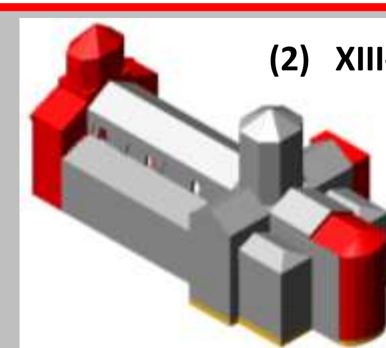
Historical notes

(1) IX-XIII c.

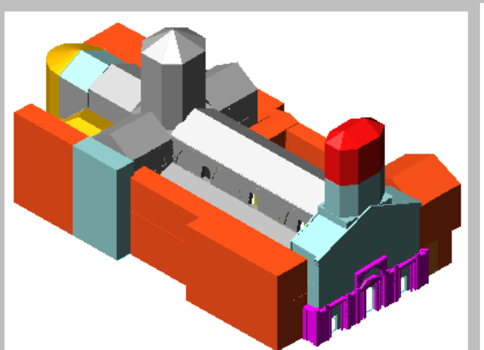
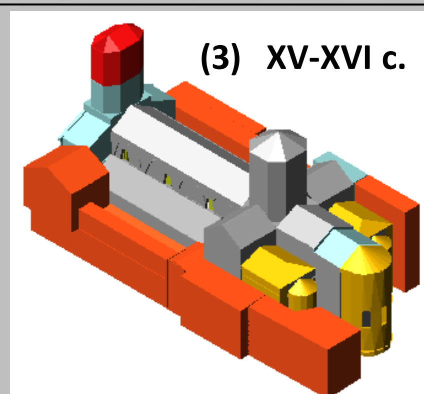
- Original structure (referred construction date 857 A.D.)
- X-XI centuries: erection of the transept and apses
- Early XIII century: construction of the dome lantern
- XIII century: construction of the crypt below the main chapel
- XIII century: closing of the clerestory windows

(2) XIII-XV c.

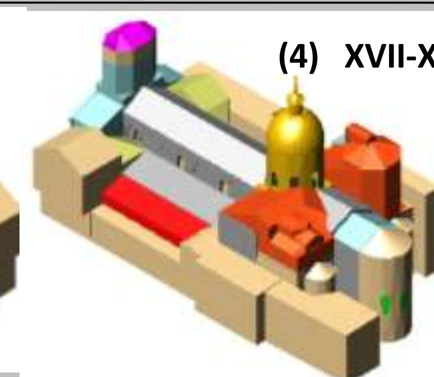
- Original structure (Romanesque church)
- after 1269: construction of the new façade with dome lantern
- after 1269: reconversion of the main apse (semicircular)
- after 1269: connection between transept and Bishop's Palace
- Late XIII century: widening of the crypt, below the transept

(3) XV-XVI c.

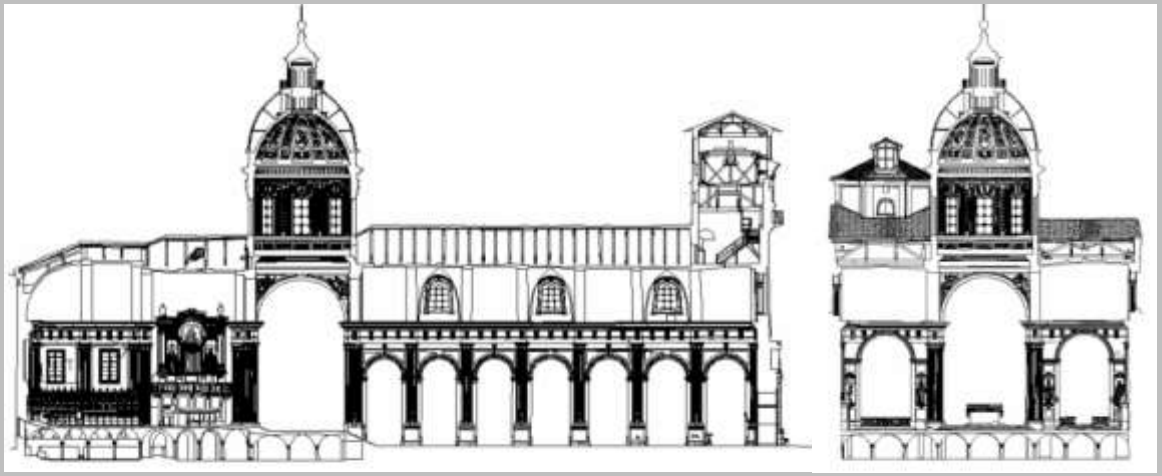



- Romanesque church
- After the late XIII century interventions
- 1451: raising of the façade's dome lantern
- 1505-1506: widening of the main chapel and construction of the two side chapels
- Half XV – end XVI centuries: erection of the aisles' chapels. The Bishop's and Canonicals' Palaces surround the Cathedral
- Half XVI century: marble veneers on the façade

(4) XVII-XIX c.

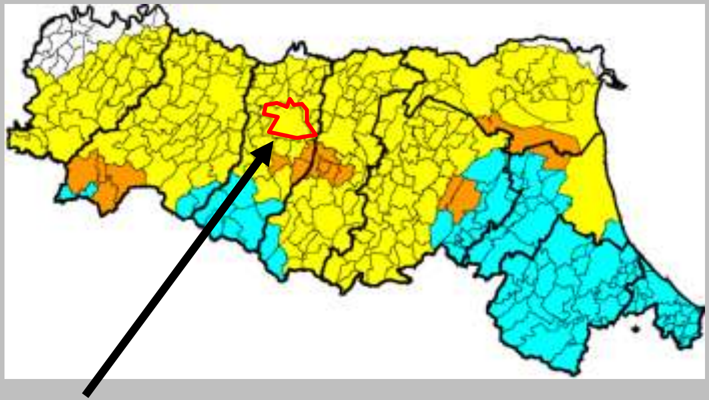



- Romanesque church
- After the late XIII century interventions
- After the XV-XVI centuries interventions
- Early XVII century: raising of the right aisle's chapels
- 1624-1624: substitution of the old dome lantern with a dome
- 1767, 1774: Modification of the transept's chapels
- 1832: lowering and reconstruction of a wall of the façade dome lantern
- Late XIX century: closing of the main apse's windows



The length of the church is 77,40 m, the width is 33,80 m, the span of the main nave is 10,15 m, the span of the two lateral naves is 6,50 m. The maximum height is reached at the top of the dome, with 44,60 m; the height of the front lantern is 33,80 m and the height of the roof above the central nave is 22,25 m.

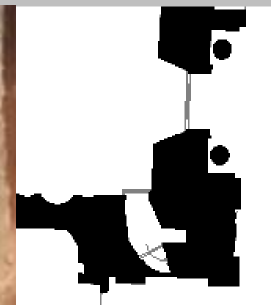
The **seismicity of the area** is well documented: medium-moderate seismic events are typical of the region.
In the last centuries, the earthquakes that struck the Reggio Emilia area **never exceeded the VIII degree on the MCS scale**. In particular, the principal seismic events recorded happened in **1465** (VI-VII), in **1547** (VIII) in **1996** (VII MCS - 6.1 Richter) and in **2000** (VI-VII MCS - 5.4-6.1 Richter).
Following the earthquake of 1832 the reconstruction of the wall of the façade lantern was necessary. The last significant seismic sequence appeared in 1996 and 2000, causing detachment of plaster and opening of fissures.



The structures of the Cathedral are generally composed by **clay brickwork masonry**. The diversified constructive phases comported the use of different materials, and some structural elements or parts (e.g. Romanesque pillars, lower façade veneers) are made of **stone**.



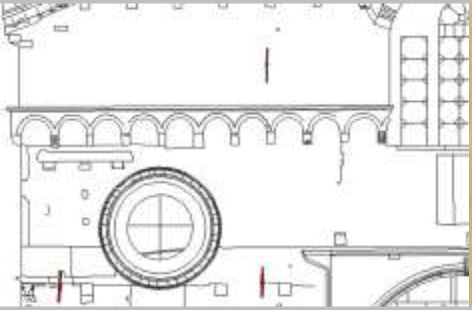




The **façade** presents towards the church square a rough and **heterogeneous external aspect**

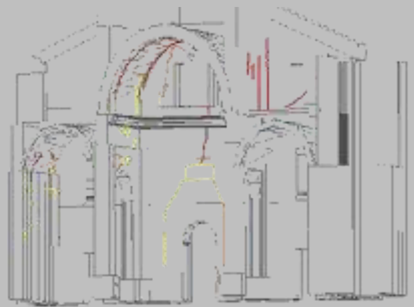
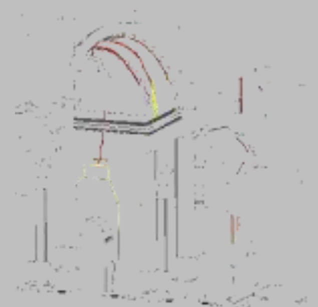


The masonry walls of the **clerestory** present a strongly **composite aspect**, being subject to past interventions comporting remarkable size windows closing

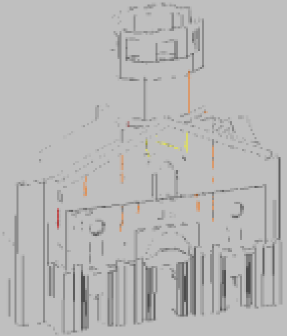



The Cathedral is generally provided with **connective systems** (metallic tie-beams), present in the central nave and in the aisles, in the arches connecting the dome sustaining pillars, in the transept, in the apses, in the dome lantern and in façade

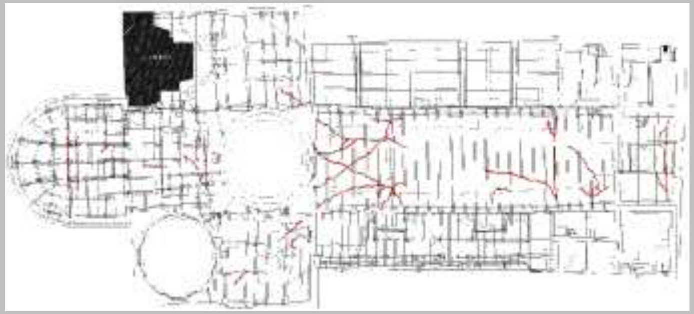
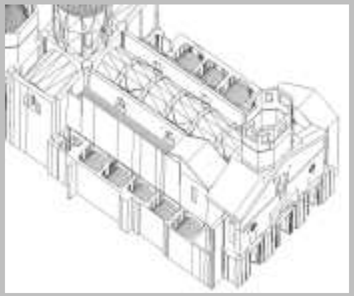
The **present damage pattern** shown by the Cathedral is highly indicative of the structural response of the building, and denounces the areas manifesting higher seismic vulnerability




Façade area


Nave vaults

Possible Solutions before and immediately after Earthquake

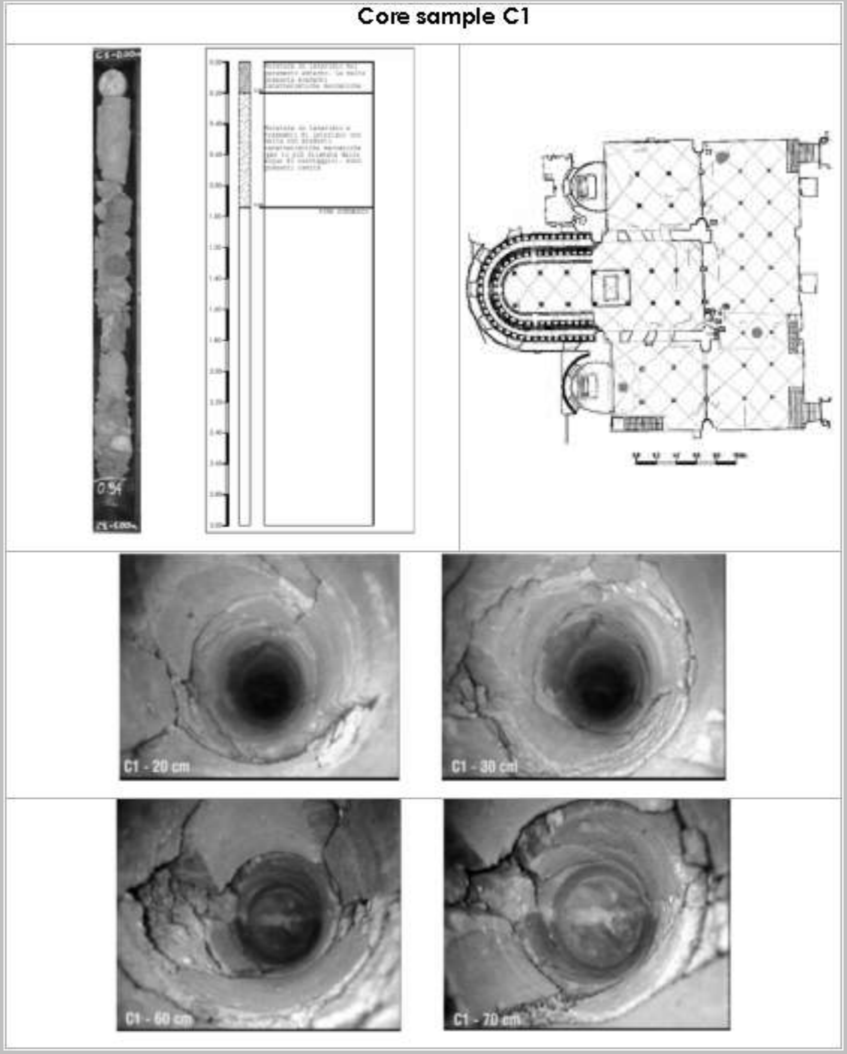
Prof. Claudio Modena

Experimental investigations and monitoring

A **diagnostic survey** was performed on the masonry structures of the Cathedral to increase the knowledge of morphology and consistency of:

- the masonry of the crypt's two main pillars
- a lateral pillar: 3 **horizontal core samples** (Cn)
- the masonry of the crypt's underlying room: 2 horizontal and 1 **vertical core sample** on the foundation structure (CFn)

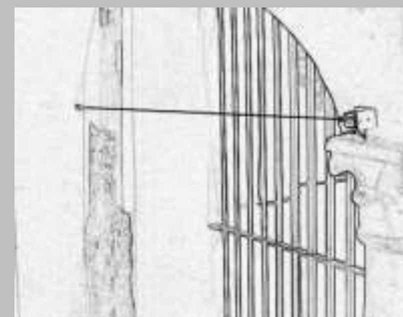
The holes, once extracted the sample, has been inspected with a video **endoscope**.
The **core samples** encountered brick masonry presenting in general poor mechanical characteristics.
In particular, inside the masonry of the pillars, some pieces of disarranged bricks and several cavities due to the scarce cohesion of the mortar, which has been often washed away by the water used for the perforation, were found.



Experimental investigations and monitoring

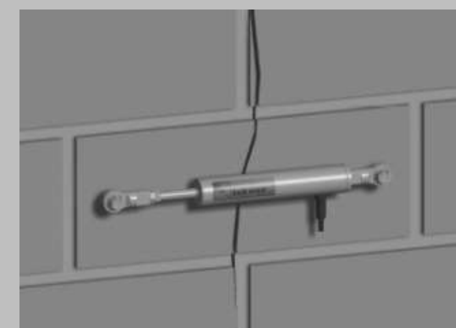
In addition, a **static monitoring system** aimed to detect the behaviour of the principal load-bearing structures, was positioned:

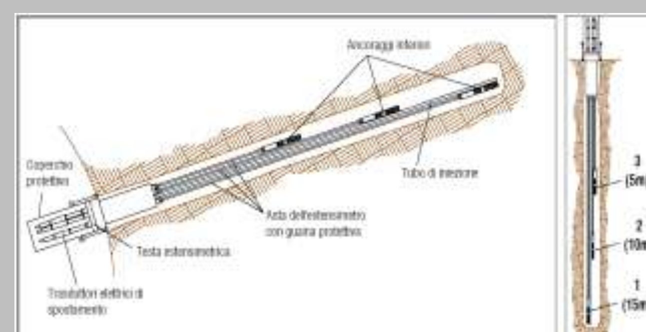
- 10 long base cable extensometers, to detect the **relative displacements** between the vertical structures (pillars, columns and perimeter walls);
- 13 electric extensometers, to measure eventual variations in the openings on the main **fissures**;
- 2 multi base extensometers, equipped with 3 measurement bases each, to evaluate the **settlements of the foundation soils** below the crypt's two main pillars;
- 2 measurement panels, equipped with switch and thermometer to control the **air temperature**.



long base cable extensometers

Electric extensometers



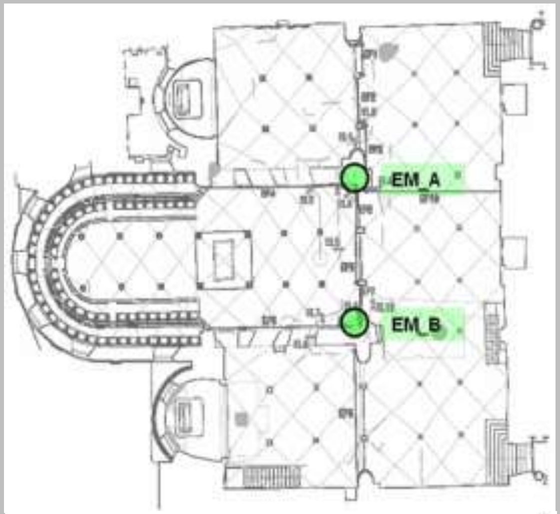


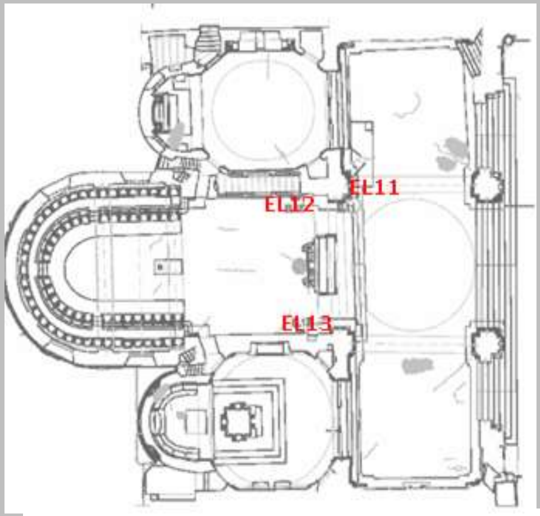
multi base extensometers

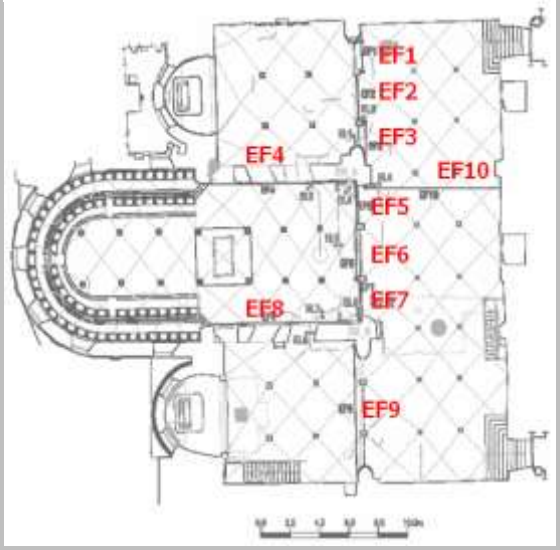
Location of sensors

The sensors are mainly placed at the **crypt level**, aiming at controlling the substructures of the pillars sustaining the dome.
Three potentiometric transducers are also placed in the **presbytery**.

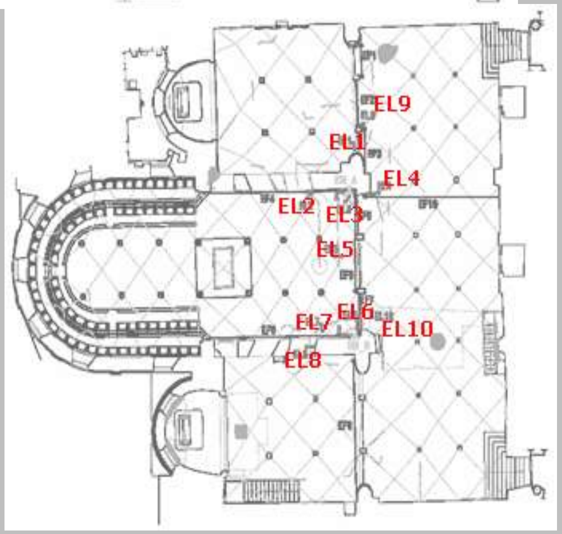
After a year and a half a **full year seasonal effects** can be appreciated.







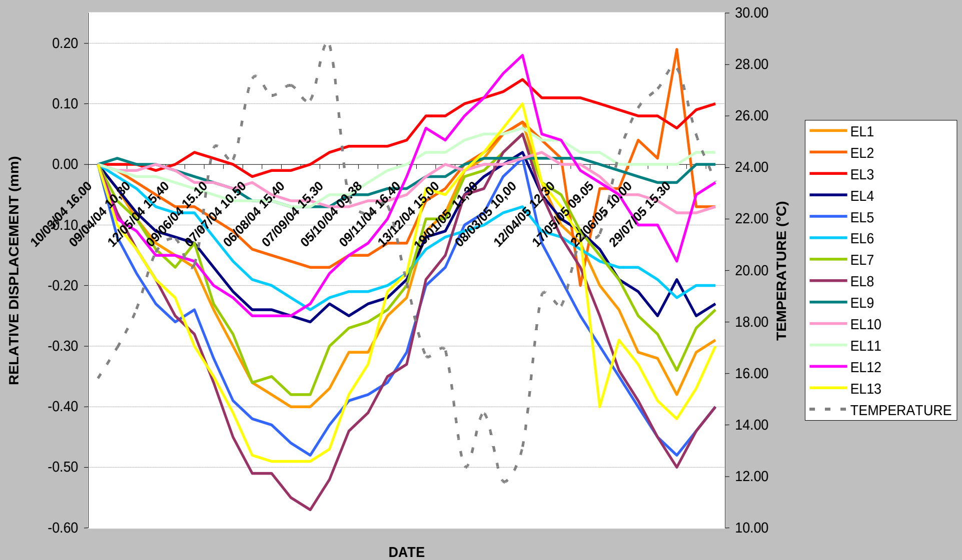
positioning of the **borehole rod extensometers** and of the of the **cable extensometers** at the crypt level



positioning of the **potentiometric displacement transducers** at the crypt and presbytery level

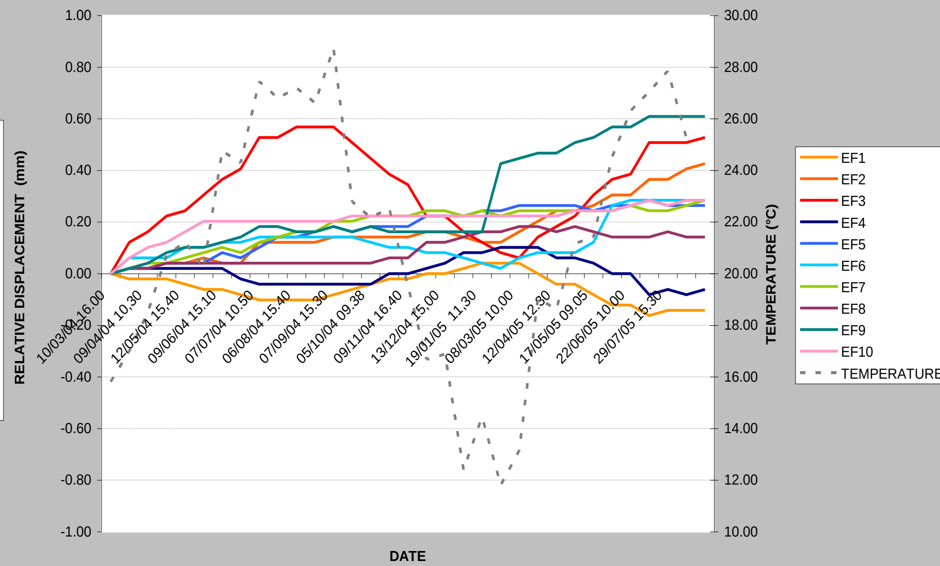
Crack mouth opening

linear potentiometric transducers EL1... EL13



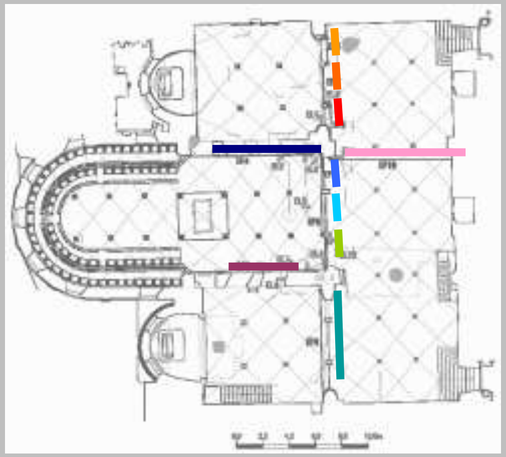
Relative distance

cable extension position transducers EF1...EF10

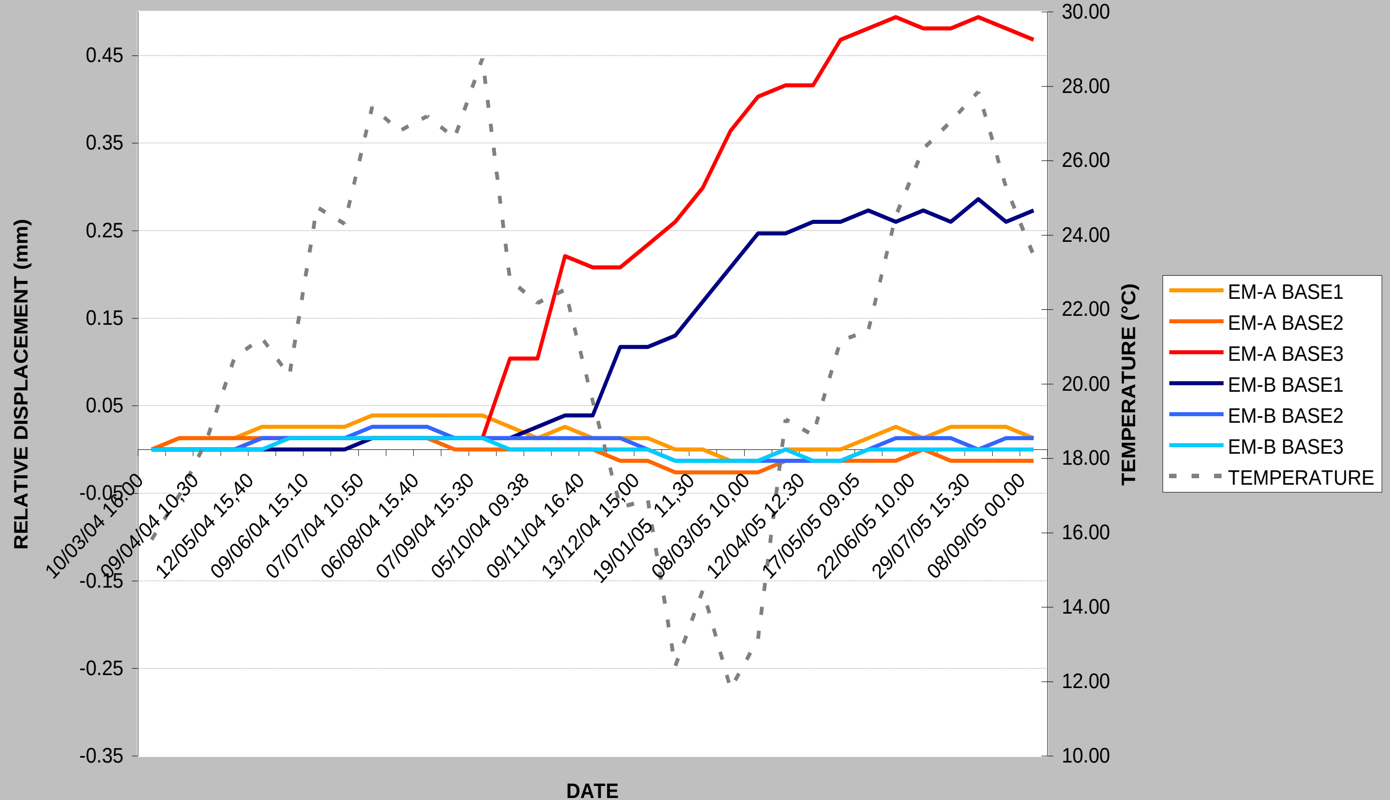


The **seasonal effect** is generally visible and the relative displacement are generally contained within **narrow variations** (the tenth part of a millimeter/half mm).

Since maintenance works are being carried out where the sensors are positioned, some of the curves were modified in order to remove inconsistent data caused by accidental sensors disturbance (EF3, EF6, EF8).



Borehole rod extensometers EM_A / EM_B



It is visible a general slight tendency to “open” (positive values are related to increase in terms of distance between the measuring fixed points): however, the recorded data present **reduced values**. Four over six of the borehole rod extensometers measuring heads did not detect significant variations: some remarkable residual displacement are noticed, on the contrary, on base 3 of EM_A (5 m deep) and on base 1 of EM_B (15 m deep).

Experimental investigations and monitoring

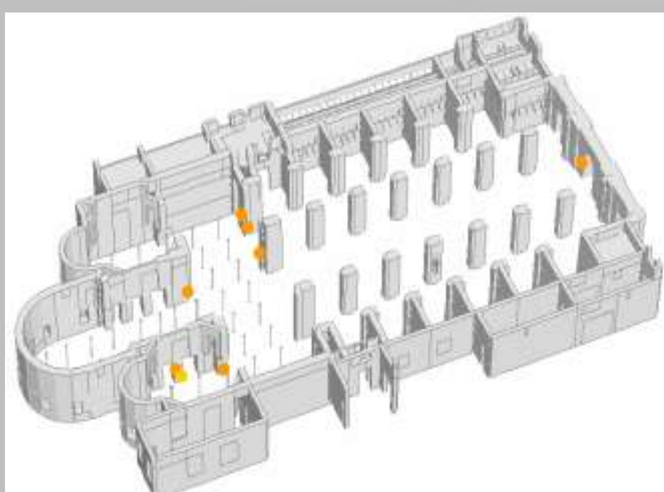
Several **investigation procedures** have been implemented; the attempt is to use non destructive techniques (**NDT**) as much as possible. With the exception of few **MDT** investigation techniques that can provide information about **quantitative** characteristics of materials and structural elements, most of the procedures can give only **qualitative** results

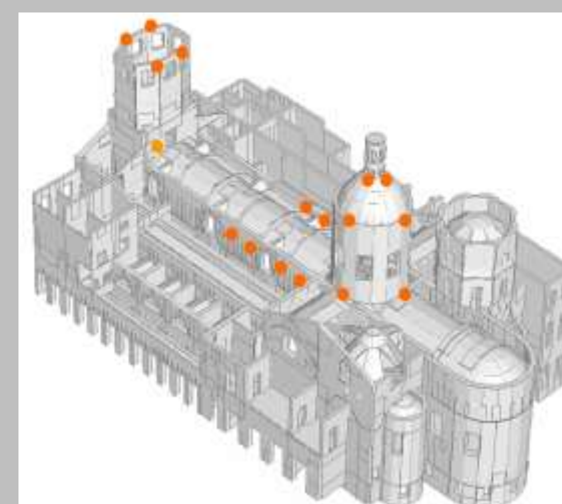
LOCAL, QUANTITATIVE	LOCAL, QUALITATIVE		GLOBAL, QUANTITATIVE
Quantitative determination of the mechanical characteristics of structural elements ▶	Execution of ND tests in portions previously evaluated by means MD test ▶	Extension of ND tests to wider areas of the structure ▶	Execution of ND dynamic tests to characterize the seismic response of the structure

Dynamic tests, control points

Direct sonic tests

Double Flat jack tests

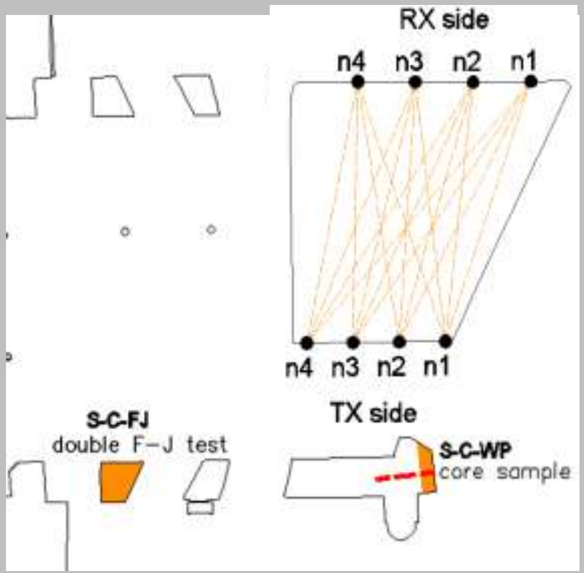


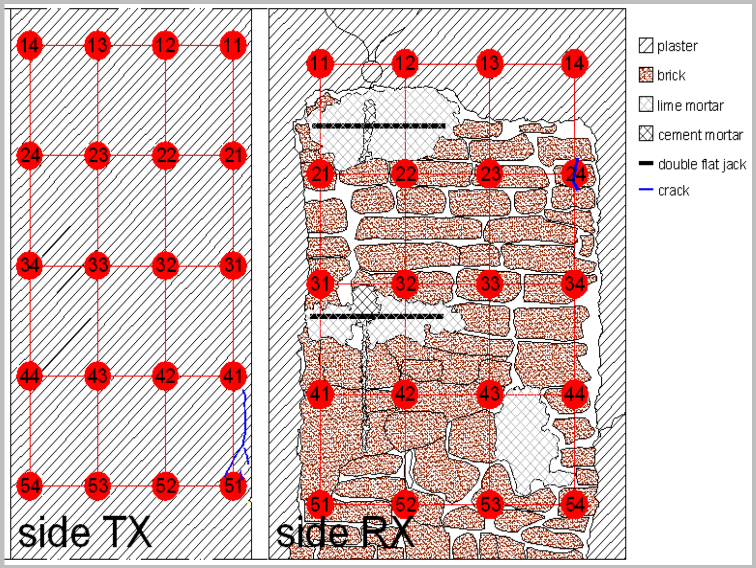


POSSIBLE SOLUTIONS BEFORE AND IMMEDIATELY AFTER EARTHQUAKE

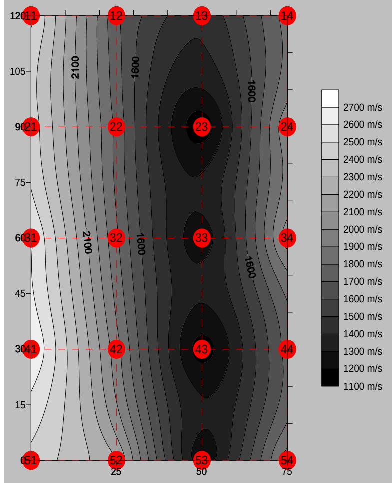
PROF. CLAUDIO MODENA

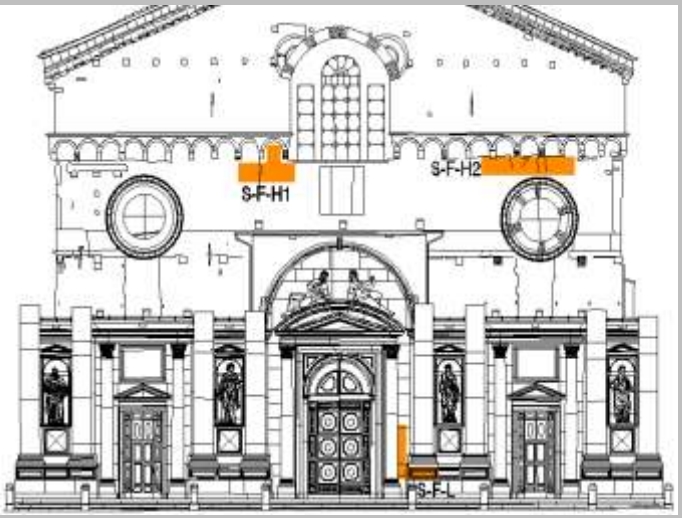
Sonic tests

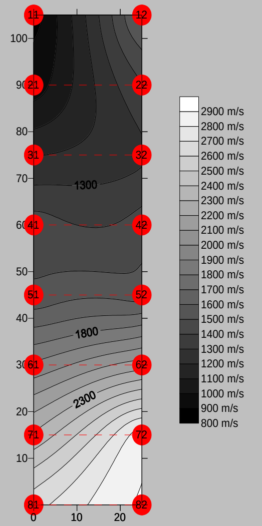




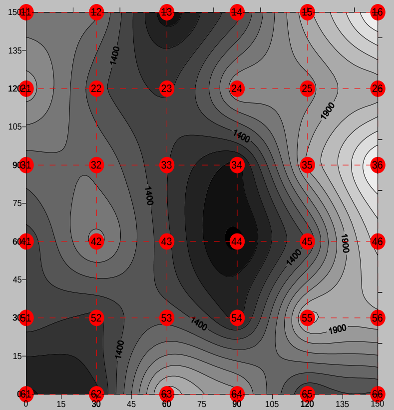
Crypt, test S-C-FJ

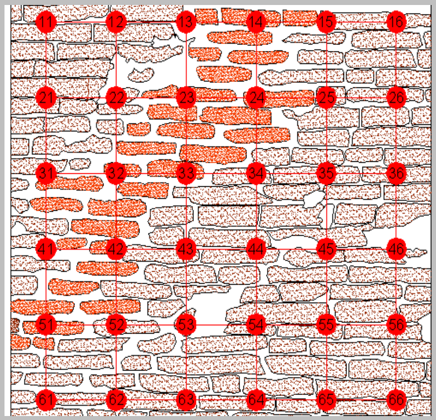






Façade area

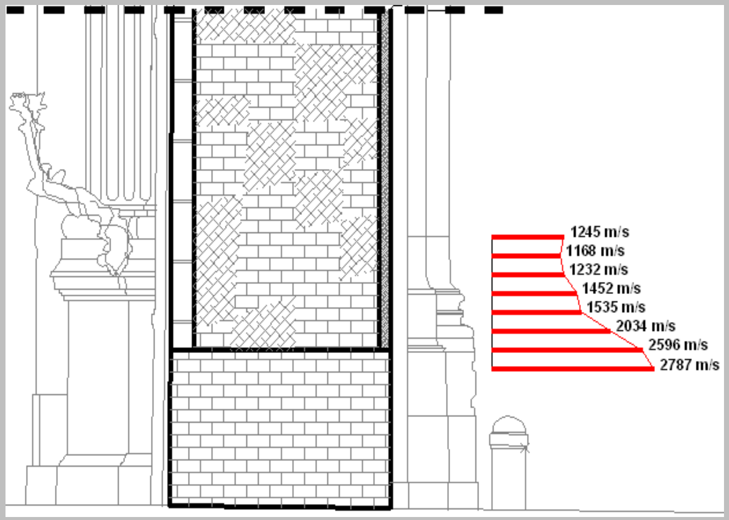




Sonic tests

Data emerged from the application of the method to wide portions of the complex denoted globally **fair velocity results**, indicating possible fair mechanical characteristics. In terms of minimum values encountered, seldom the velocity was lower than 800 m/s. No odd velocity values were found, denoting **absence of macroscopic inconsistencies** within the tested masonry portions investigated.
Several masonry walls tested denoted a **general velocity uniformity**, indicating a possible uniformity also in terms of mechanical characteristics, resulting in a positive **homogeneous** stress pattern.

Sonic investigations results proposed the impression that **the successive implementation of the kinematic mechanisms approach analyses**, considering the selected masonry portions (macroelements) as rigid bodies, **is possible**, since the tested masonry portions did not denounce severe structural deficiencies, possible cause of sudden collapse (e.g. masonry crumbling) before the beginning of the considered mechanisms.

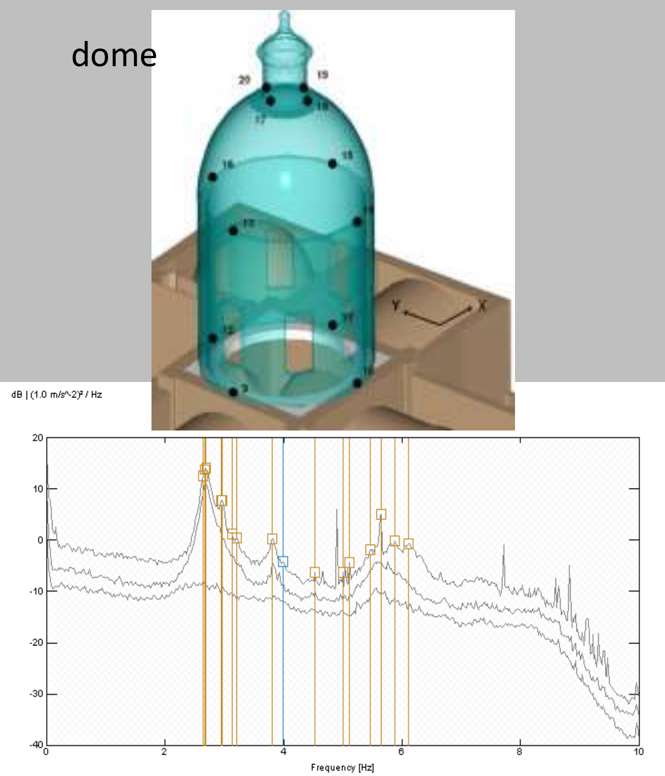


Sharp variation from the 1st two lower lines (average on 4 points equal to 2692 m/s) respect the upper ones (average on 12 points equal to 1444 m/s)

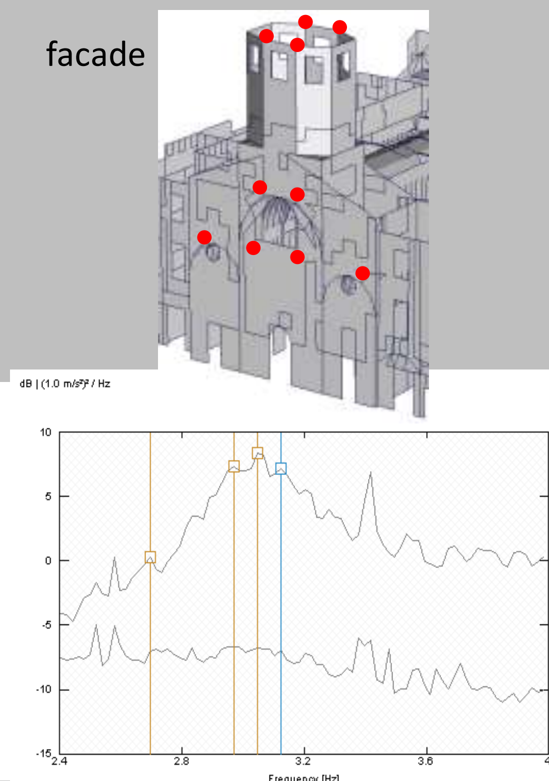
Dynamic tests

This testing methodology is currently the only one that allows to experimentally measure parameters related to the **global structural behaviour** of an historical construction. The obtained results are related to several **structural/physical parameters** (geometry, mass distribution, stiffness, connections effectiveness, presence of damage and boundary conditions). Dynamic investigations were carried out considering **ambient vibrations** (AVT).

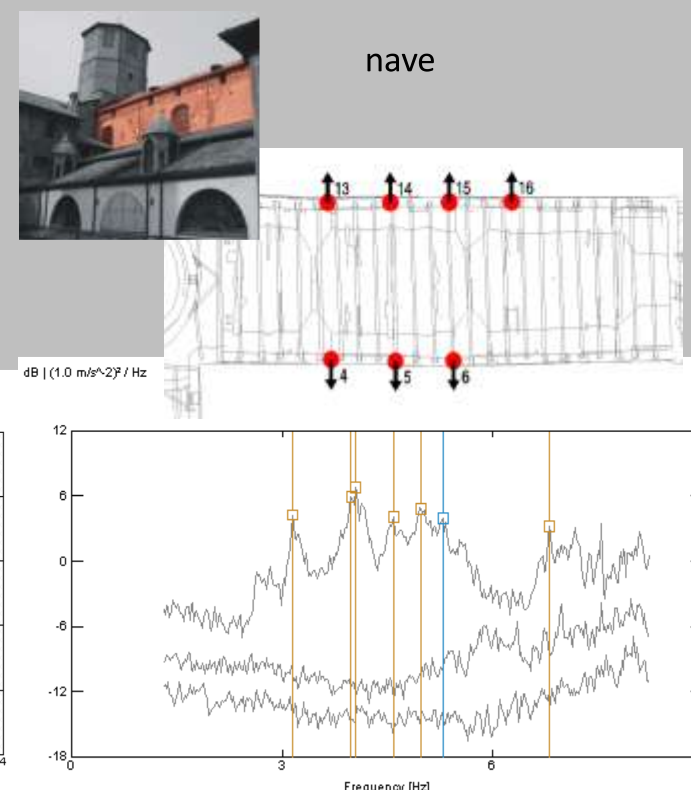
dome



facade



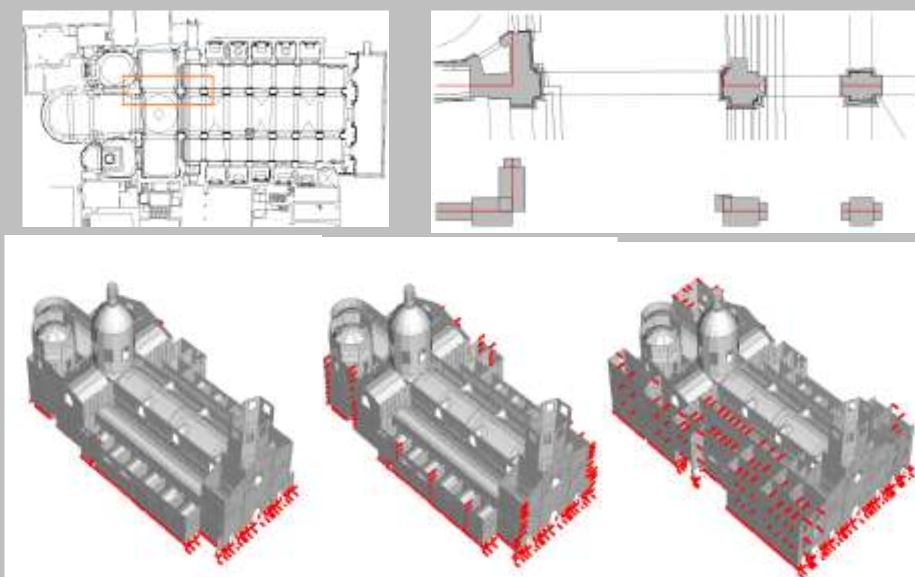
nave



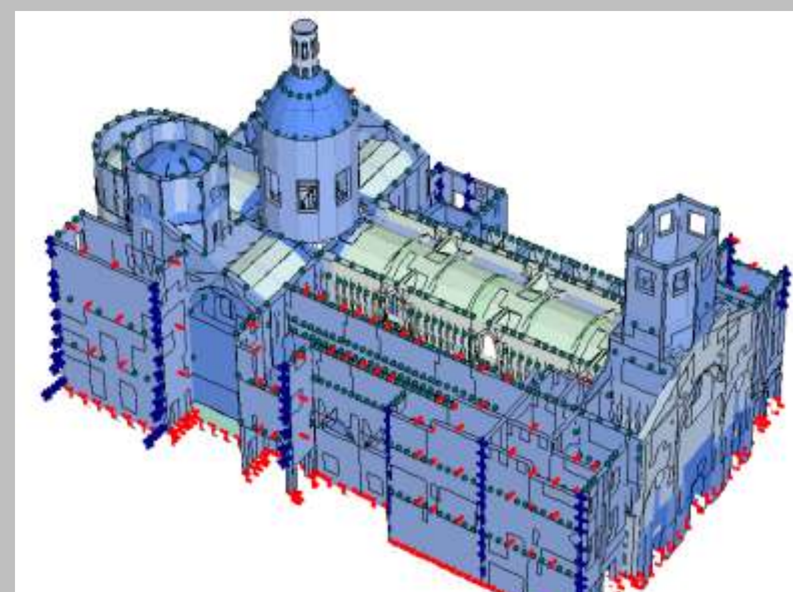
Structural modelling: FEM

The numerical simulation of Reggio Emilia Cathedral considered in the first instance several **linear elastic Finite Element models**. All of them represented the global Cathedral's structure, since the overall dynamic response can not be evaluated by considering partial models.

From the first model, considering the structures of the Cathedral as disconnected from the other adjacent buildings, several refinement were introduced, subsequently applying **lateral constraints** and then **adding portions of the surrounding structures**: the final model can be considered acceptable for dynamic identification purposes.



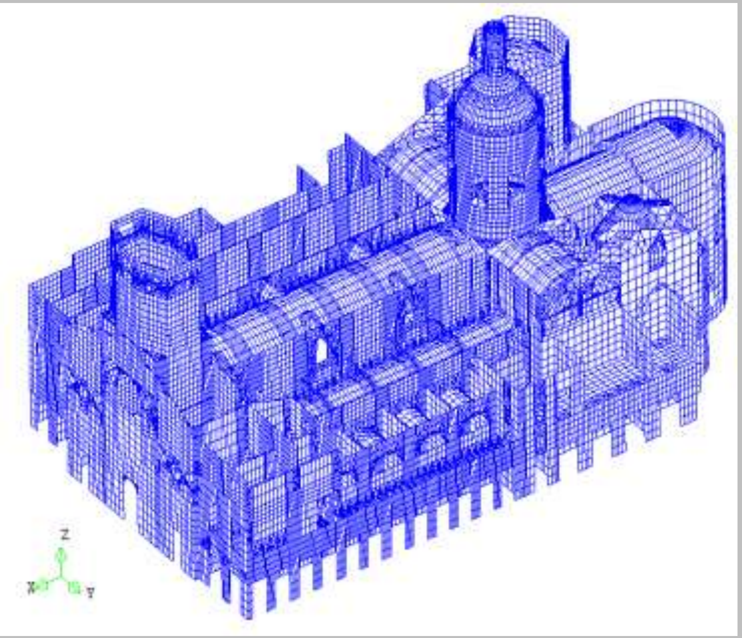
a) b) c)



Final FE elastic model

Possible Solutions before and immediately after Earthquake

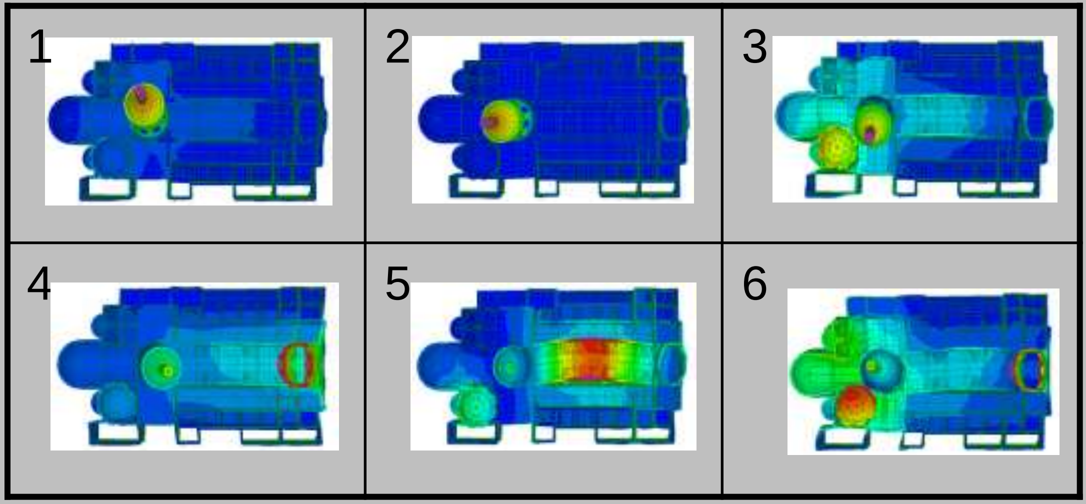
Prof. Claudio Modena



The **full model** comprises 60,158 elements with 161,218 nodes: besides the 2D rectangular curved shell quadratic elements, it was necessary to use triangular elements because of the complex model's geometry.

A **non linear behaviour** was adopted for all elements, with the exception of the vaults' infill, that was assumed linear elastic throughout the analyses.

The numerical model has been **validated** on the basis of the experimental results of dynamic tests: the complexity of the structure did not allow a simple comparison and only for the **first frequencies and corresponding mode shapes** it was possible to obtain fair good results.



Mode nr.	Frequency (Hz)	mass participation (%) Longit.	Transv.	Mode description
1	2,65	0,21	16,24	Transverse bending, dome
2	2,66	7,76	0,28	Longitudinal bending, dome
3	2,90	1,36	20,05	Transverse bending, dome
4	3,02	27,82	1,20	Longitudinal bending, façade
5	3,33	0,02	9,05	Transverse bending, nave
6	3,89	16,17	1,07	Apses area - Longitudinal bending, façade

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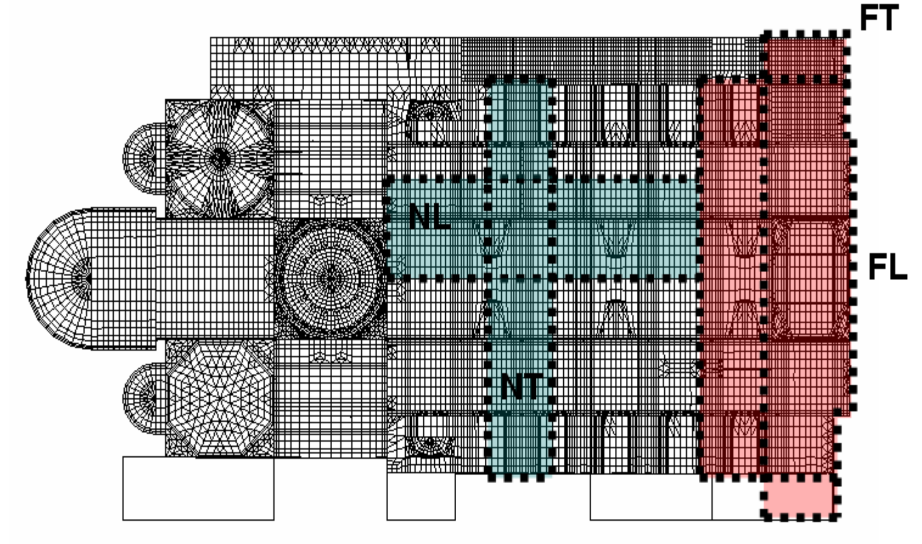
Significant portions of the full model were extracted and separately analyzed. The subdivision of the global model in sub-models was decided following the **macroelement** approach, in order to directly **compare the numerical results** to the outcomes of the limit analyses.

Attention was paid to the **structural unitariness** of the selected parts, in a way to have the possibility to separate them still not loosing significance the analysis: the Cathedral's parts considered in the sub-models were selected on the basis of geometrical, historical-constructive and damage pattern preventive evaluations.

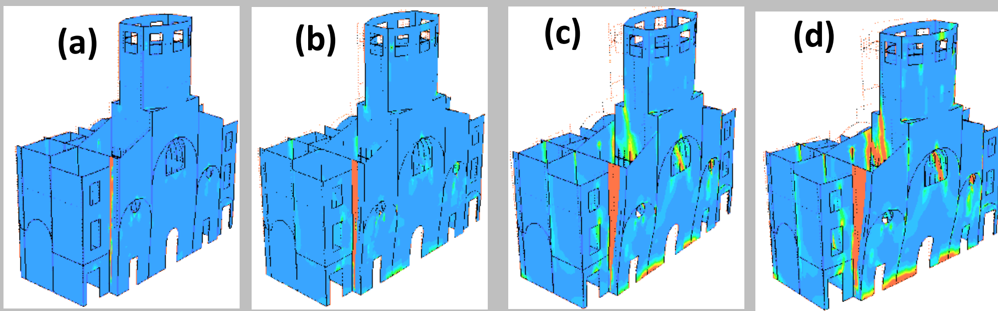
Two models for the **façade** longitudinal and transversal response (FL and FT) and two for the **nave** (NL and NT, longitudinal and transversal response) were considered.

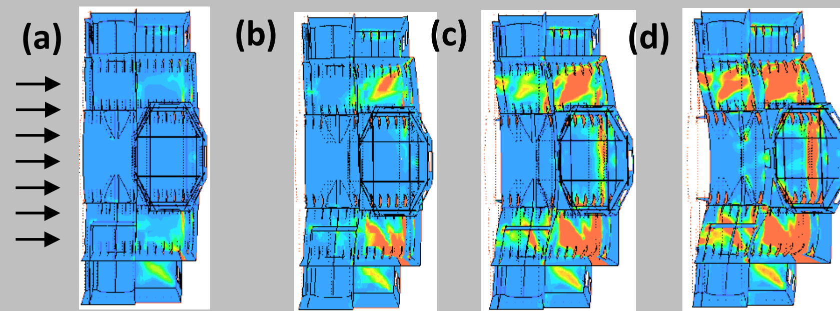


MODEL	FL	FT	NL	NT
MASS DENSITY - STRUCTURAL (kg/m³)	1800	1800	1800	1800
MASS DENSITY – VAULTS' FILLING (kg/m³)	1500	1500	1500	1500
YOUNG MODULUS (MPa)	2100	2100	2100	2100
POISSON'S COEFF.	0.2	0.2	0.2	0.2
TENSILE STRENGTH (MPa)	0.10	0.10	0.10	0.10
CRACK BANDWIDTH (m)	-	-	- 0,5	- 0,5
Gf[I] (J/m²)			100	100
COMPR. STRENGTH (MPa)	INF.	INF.	INF.	INF.
NUMBER OF NODES	38959	28282	12667	9767
NUMBER OF ELEMENTS	14683	10500	4806	3704

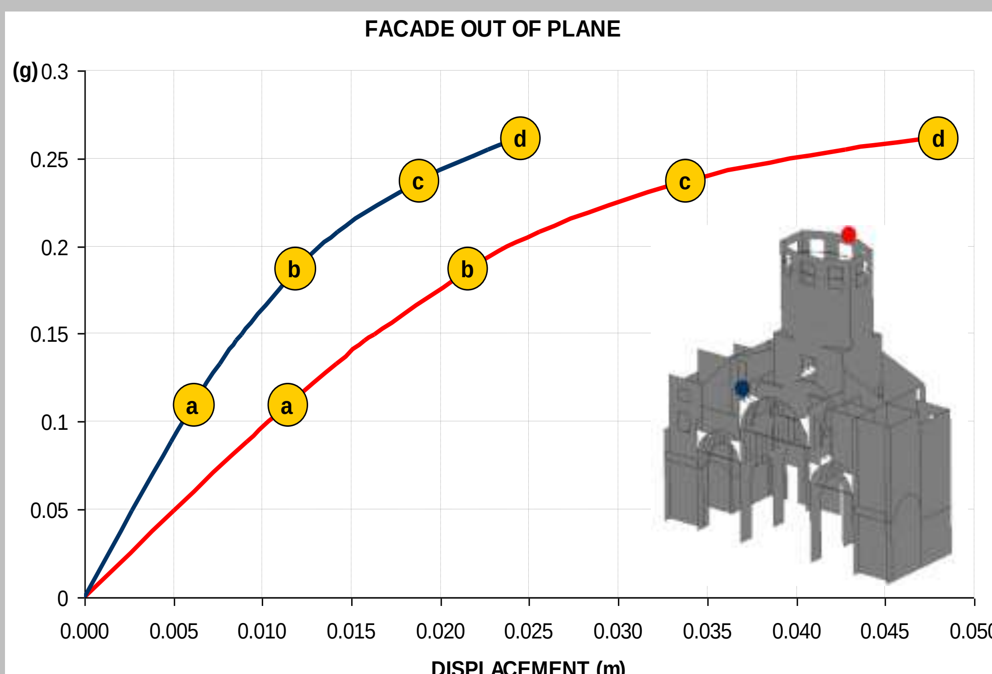
Model FL

The **horizontal mass proportional growing load** was applied towards the Cathedral's square.





Progression of the damage mechanism; deformation is magnified (x100)



The analysis stopped at a value of horizontal acceleration equal to 0.26 g. Defining the parameter k (**structural stiffness**) as:

$$k = \frac{\Delta F}{\Delta \delta} = \frac{m \cdot \Delta \alpha}{\Delta \delta}$$

the final k value, evaluated on the last two load steps, was approximately equal to 13% of the initial k, evaluated on the control point at the top of the dome lantern.

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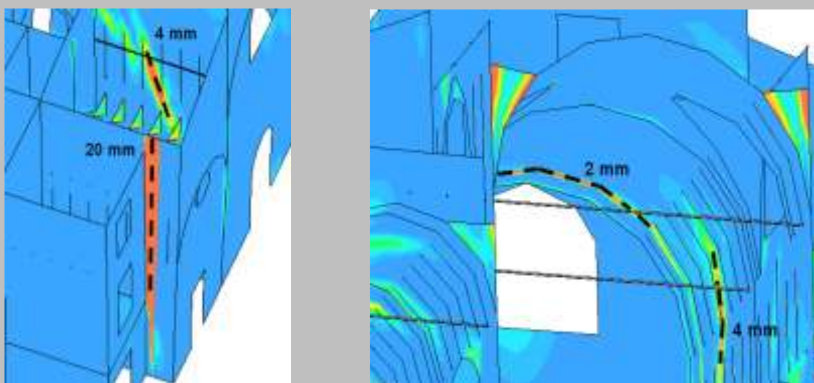
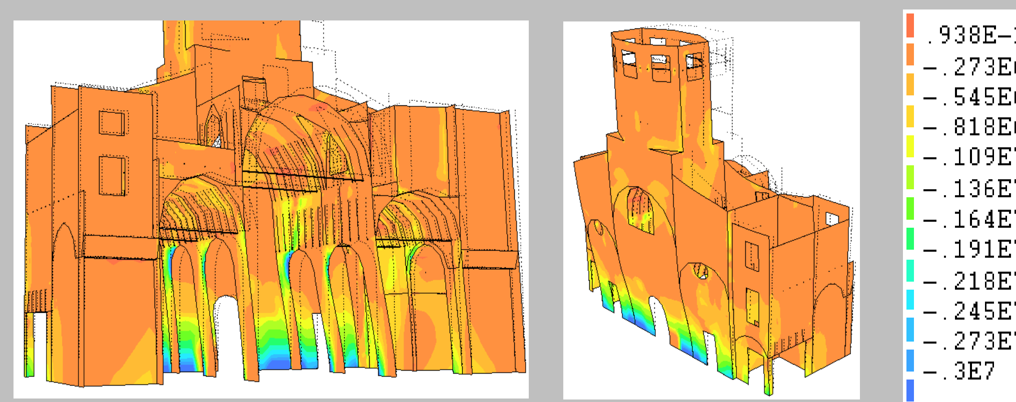
Model FL

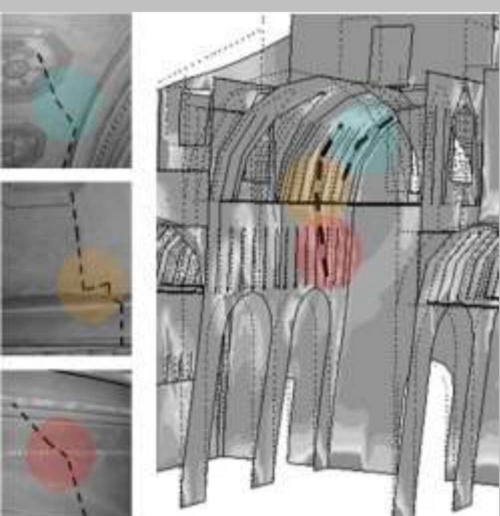
The **damage pattern predicted** by the analysis finds a remarkable correspondence with the **effective damage** denoted by the structures, in several positions:

Cracks opening - last step of the analysis (0.26 g)



Principal minimum stresses - last step of the analysis (0.26 g)





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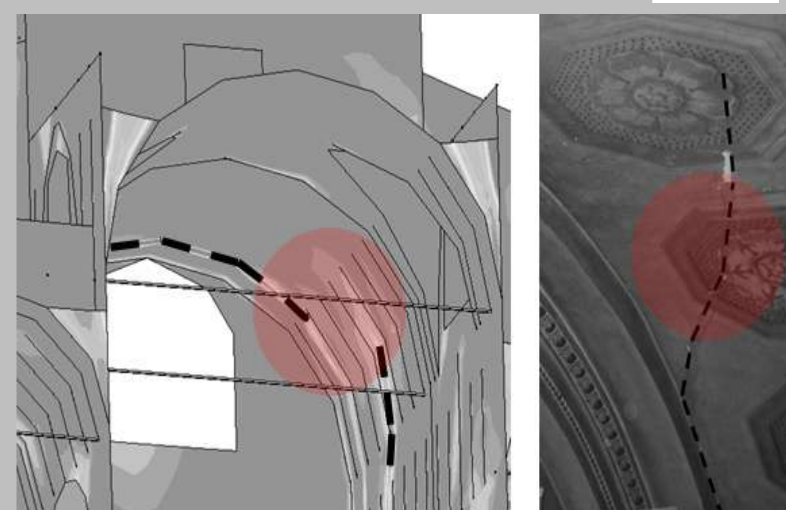
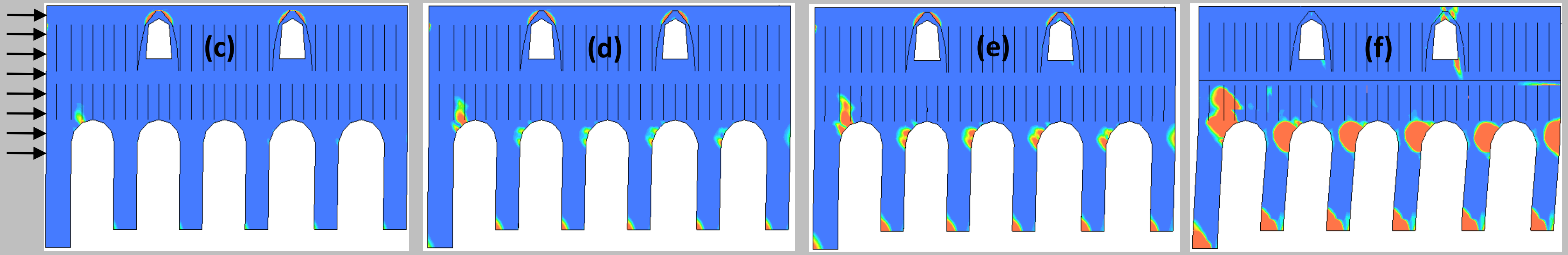
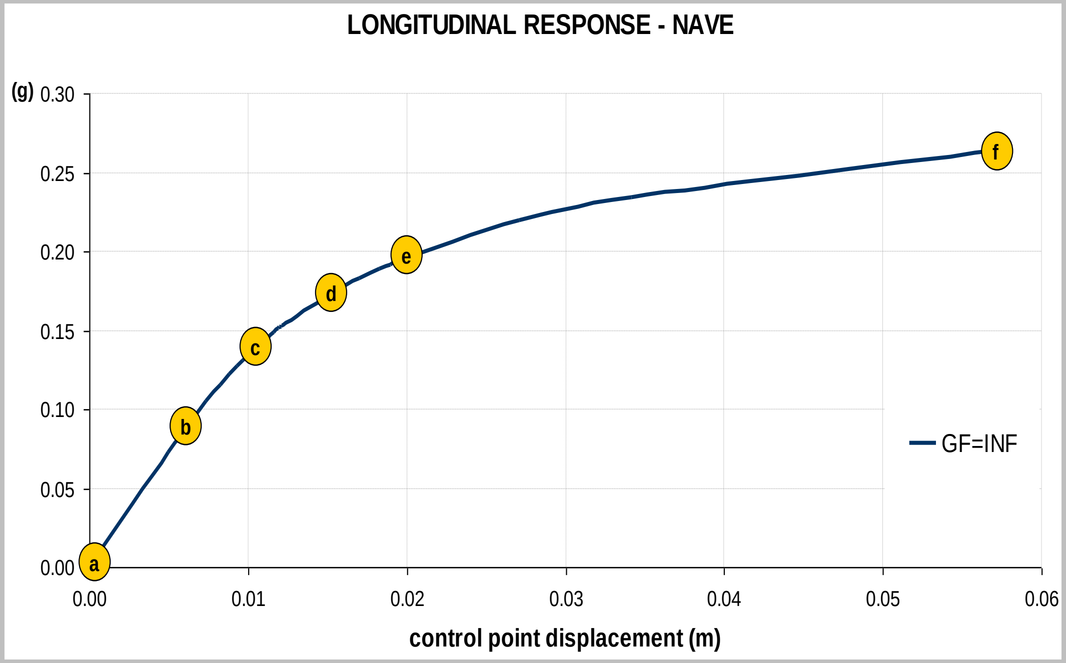


Model NL





The failure mode, following the application of growing horizontal loads, is determined by the overturning of the pillars

Damage identification – model prediction experimental verification

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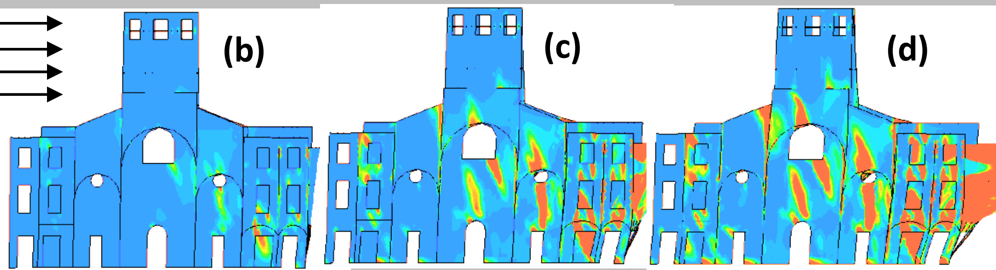
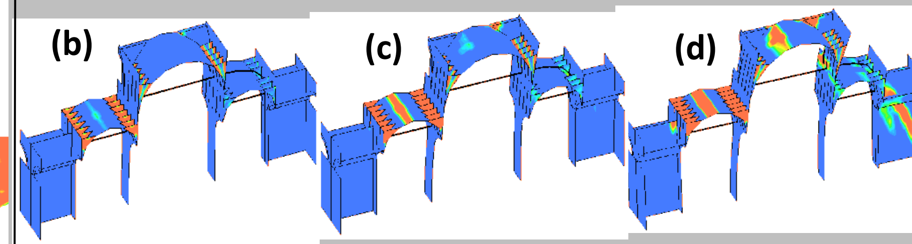
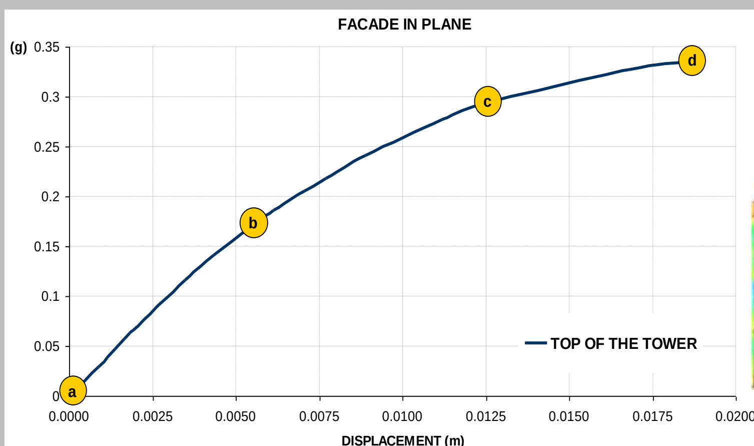
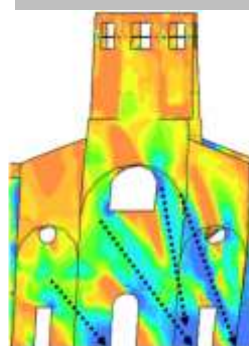
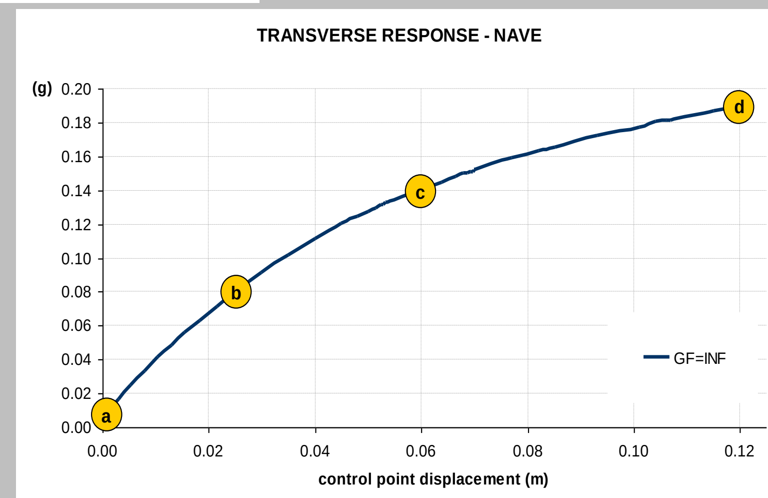
Model FT　　　　Model NT





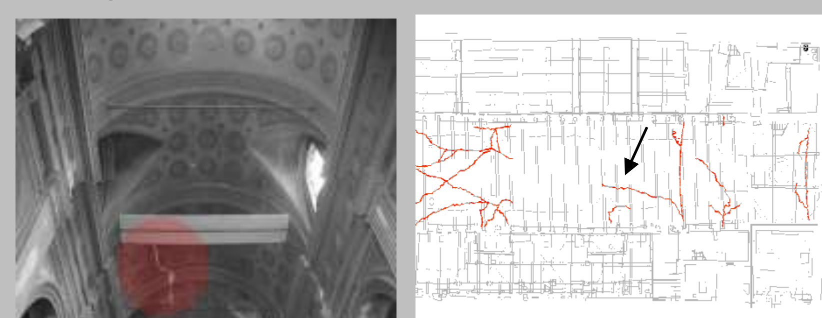






Damage identification

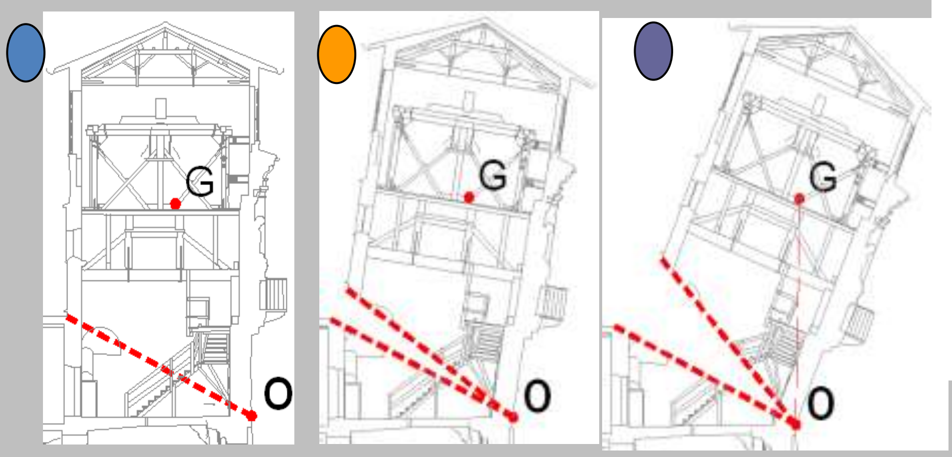
Damage identification

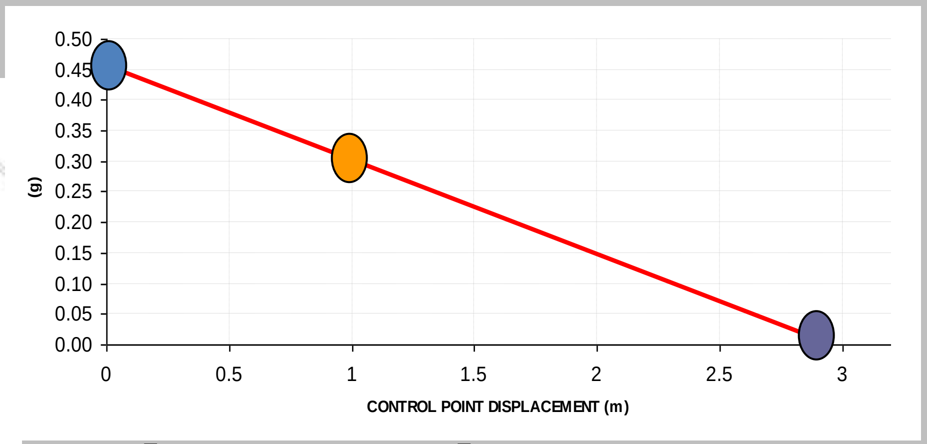


Structural modelling: Limit analysis

The **possible failures** implemented were selected on the basis of the damage pattern, of the typical seismic vulnerabilities, of the historical analysis and of the comparative evaluation of the numerical simulation results.

Several failure scenarios were evaluated, and corresponding safety factors (α_0 = c) and capacity curves were defined. The different α_0 values of the considered mechanisms give then an indication about the range of horizontal acceleration values that possibly will bring the onset of the envisaged failure scenarios.





$$\alpha_0\left[\sum_{i=1}^{n}P_i\delta_{x,i}+\sum_{j=n+1}^{n+m}P_j\delta_{x,j}\right]-\sum_{i=1}^{n}P_i\delta_{y,i}-\sum_{h=1}^{o}F_h\delta_h=L_{fi}$$

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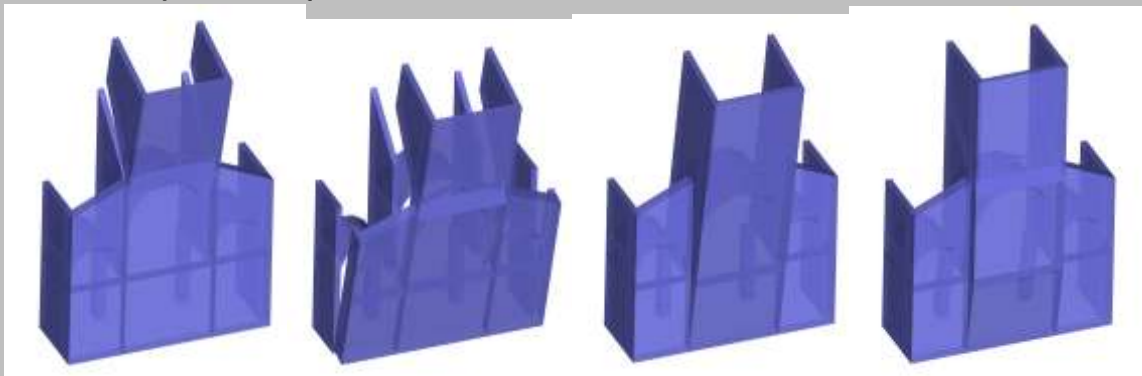
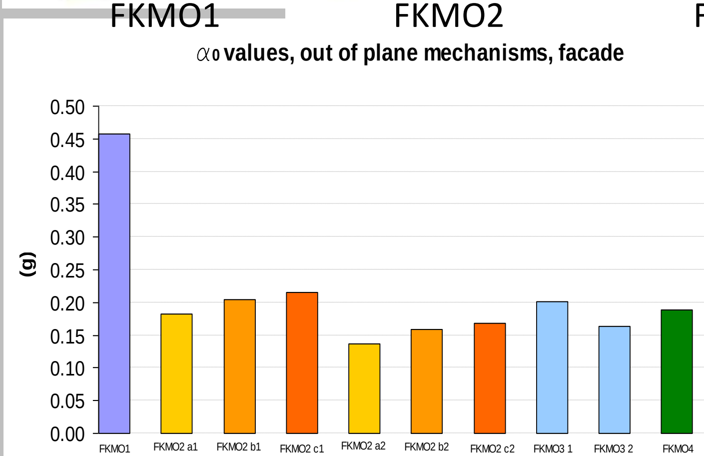
Out of plane façade failure

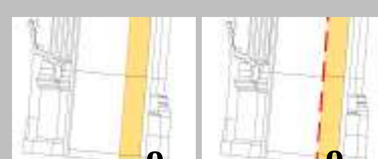


FKMO1 FKMO2 FKMO3 FKMO4



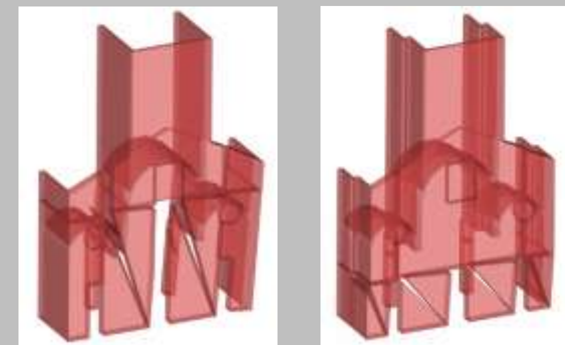
- FKMO1
- FKMO2a, case 1 and 2
- FKMO2b, case 1 and 2
- FKMO2c, case 1 and 2
- FKMO3, case 1 and 2
- FKMO4

FKMO2a FKMO2b FKMO2c

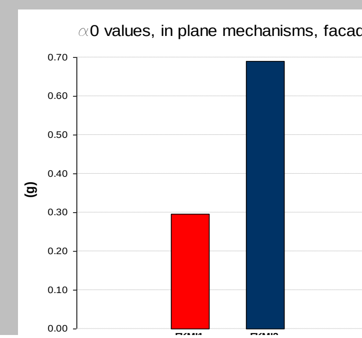


case 1 case 2

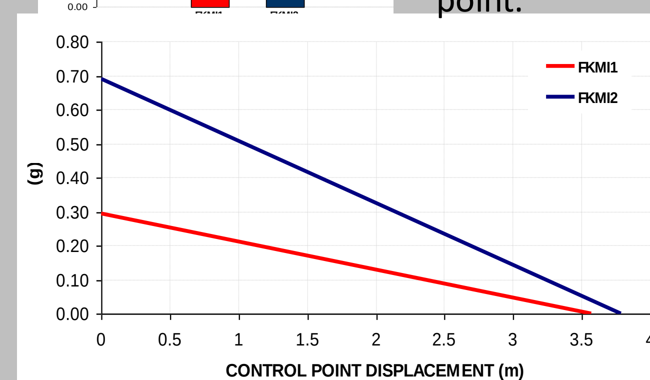
In plane façade failure



FKMI1 FKMI2

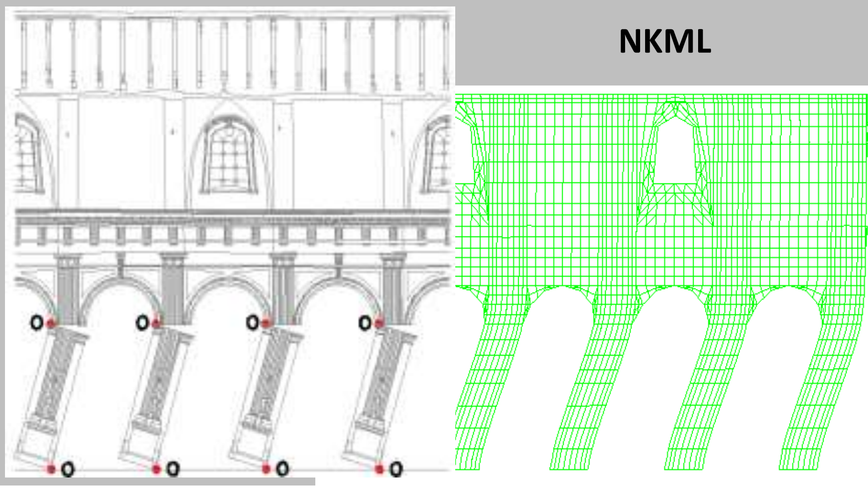


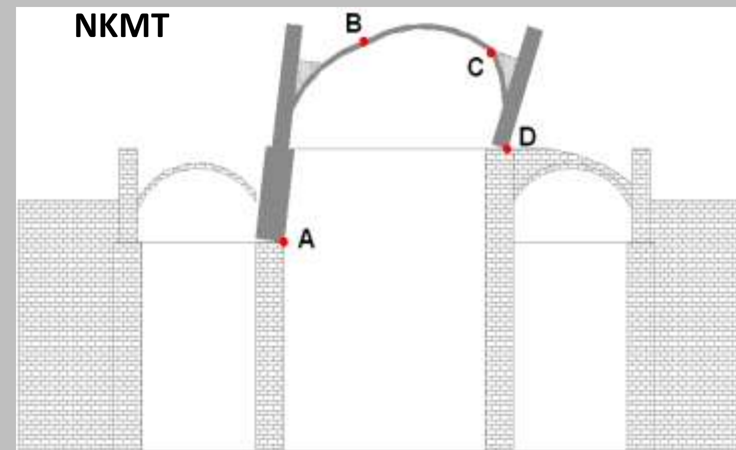
The mechanisms' centre of masses was considered as reference point.



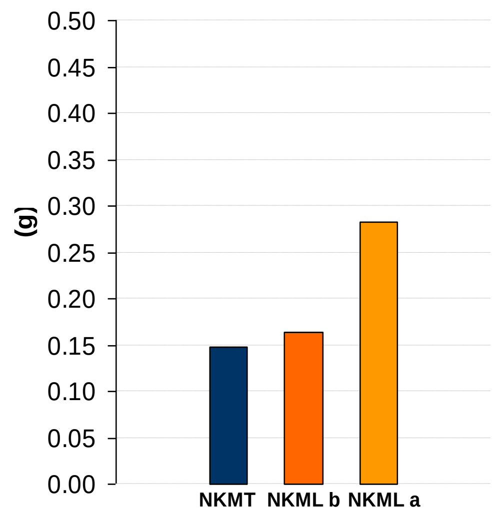
Nave longitudinal and transversal failure mechanisms

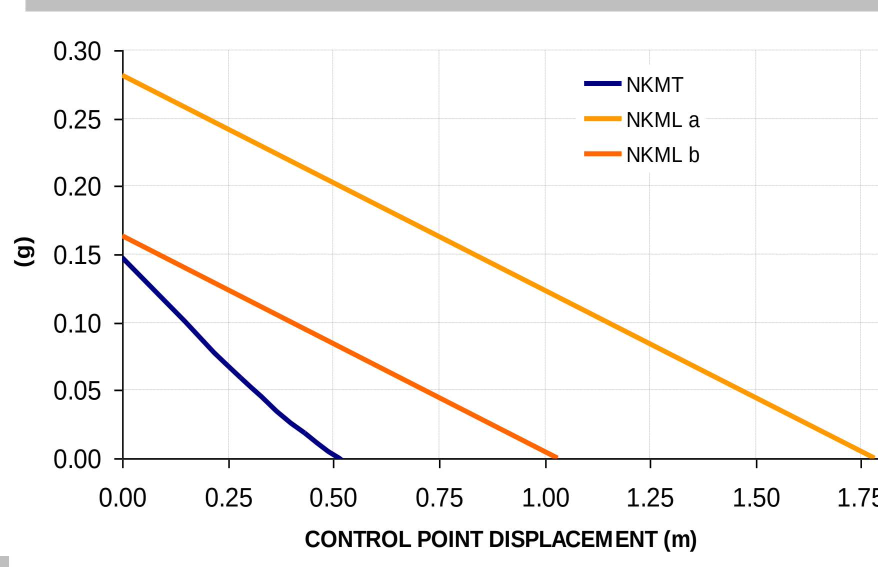
NKML





α0 values, nave mechanisms





Two kinematic mechanisms were considered, evaluating the nave longitudinal seismic capacity and the response of the barrelled vault of the main nave to transversal seismic actions

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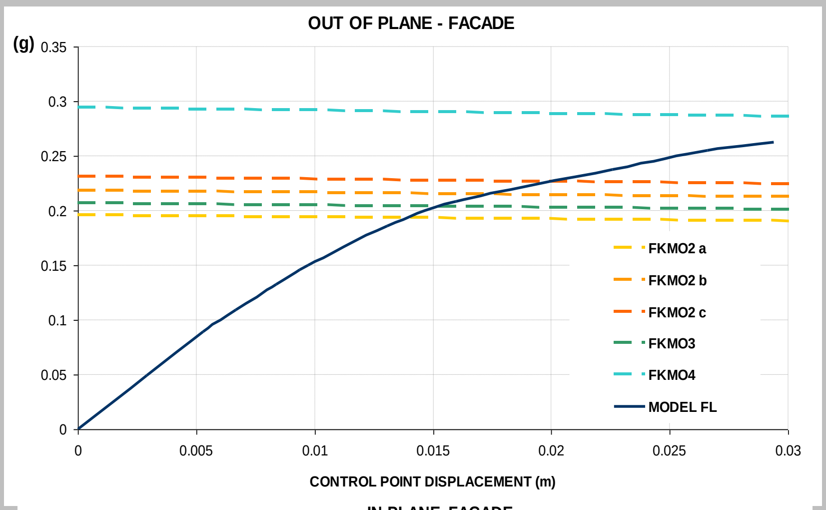
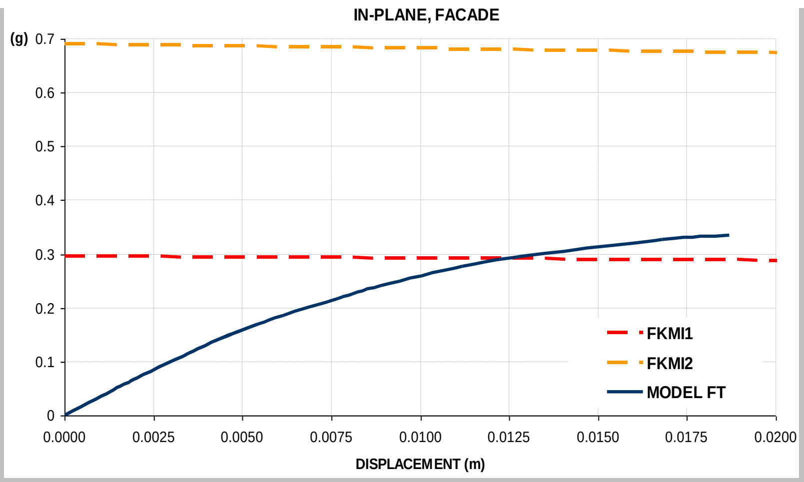
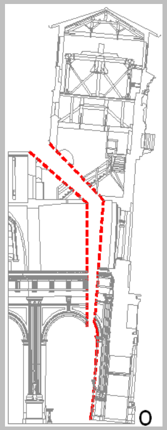
Both the limit analysis **kinematic approach** and the **FE modelling** strategy provided fairly **satisfactory results in terms of comparative evaluation**.

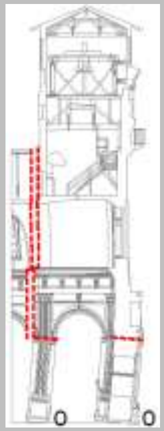
From the comparison between models, it is evident that, according to the numerical outcomes, the closer failure mode implemented with the limit analysis corresponds to model **FKMI1**



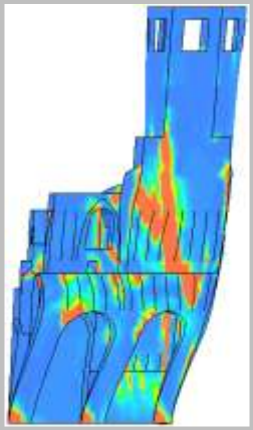




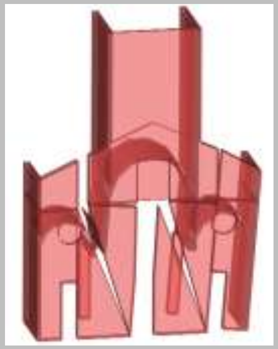
FKMO2



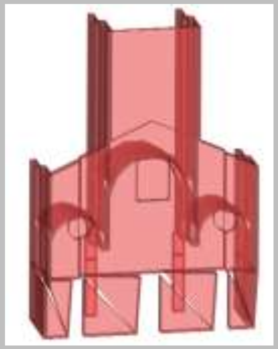
FKMO4



FL MODEL



FKMI1



FKMI2

FT MODEL

THANK YOU!

SPEAKER: PROF. CLAUDIO MODENA

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