SEISMIC RETROFIT
OF UNREINFORCED
MASONRY HERITAGE
CHURCHES IN THE
PHILIPPINES

INTERNATIONAL SYMPOSIUM

Photo by Raymond Rodolfo

National Commission for Culture and the Arts
National Museum, Philippines
Bakas Pilipinas
ICOMOS Philippines
University of Santo Tomas - Center for Conservation of
Cultural Property and Environment in the Tropics

JANUARY 13 – 14, 2016

Third Session: STRUCTURAL ANALYSIS

Analysis Methods for Unreinforced Heritage Masonry

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CONTENTS

• General issues on appropriate modeling strategies

• Cases studies:

    - Church of S. Maria del Pianto in Padova

    - Santa Maria Assunta Cathedral in Reggio Emilia

    - San Vigilio Cathedral in Trento

    - Scrovegni Chapel in Padova

    - Frari Bell Tower in Venice

   - Aggregate building in L'Aquila

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Models for seismic safety assessment

Structural modelling and seismic analysis methods

For existing masonry buildings it is possible to consider **different analysis methods**, according to the considered **appropriate model** which describe the structure and its seismic behaviour.

It is possible to consider:

- **Macro-elements models**
- **Equivalent frame models**
- **Finite elements models**
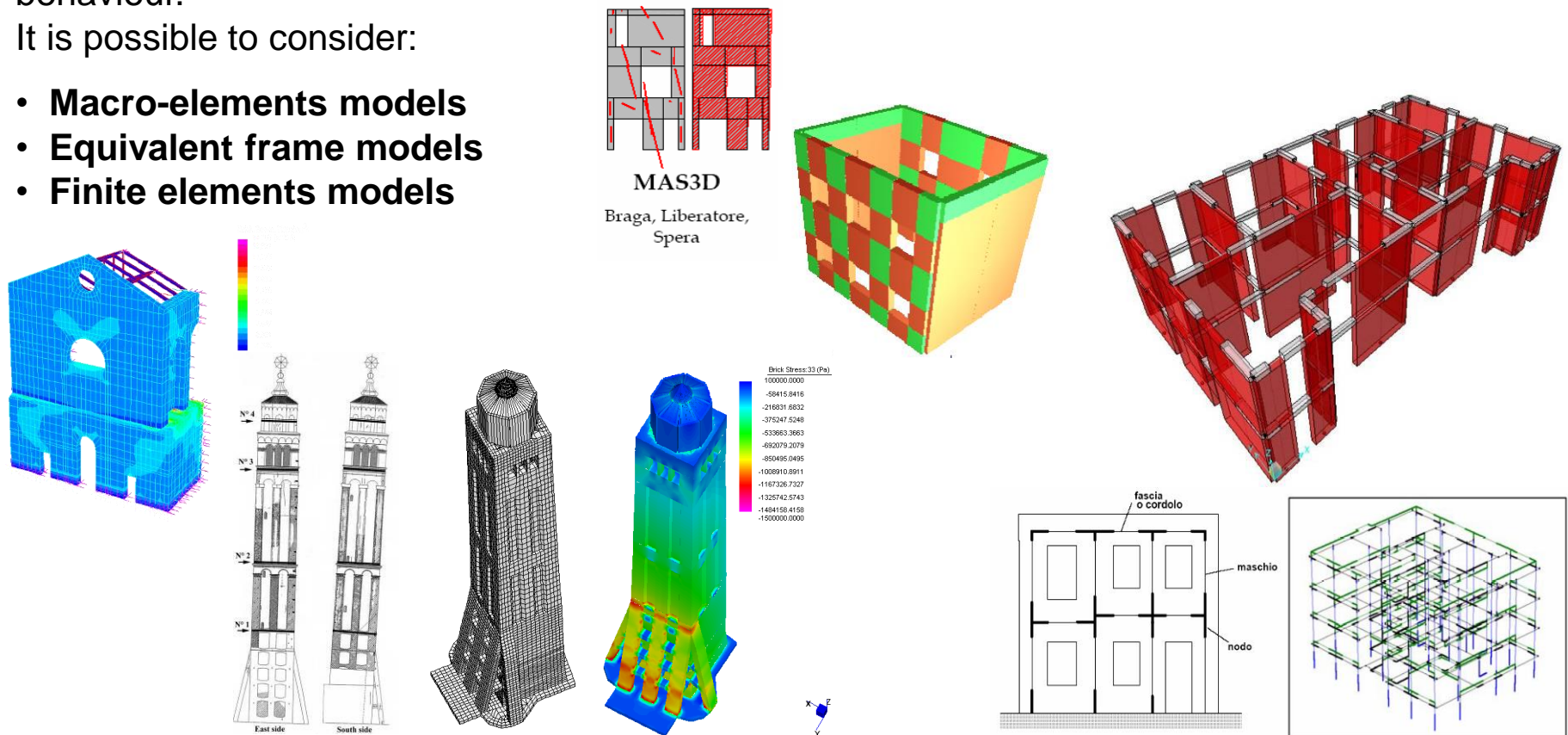
MAS3D
Braga, Liberatore, Spera

Models for seismic safety assessment

Structural modelling and seismic analysis methods

The **Finite Element Method** (FEM) is a powerful tool to study stresses and displacement in solids. A mathematical description of the material behaviour, which yields the relation between the stress and strain tensors in a material point of the structural element, is necessary for this purpose.

Constitutive models of interest for practice are normally developed according to a phenomenological approach in which the observed mechanisms are represented in such a manner that simulations are in reasonable agreement with experiments.

Several examples of non linear relatively **simple 2D or 3D models** can be found (e.g. structural elements as churches' triumphal arches, vaults, or structures as chimneys, bell towers). Relatively few studies considering full scale complex structures, for their seismic assessment, are on the contrary available.
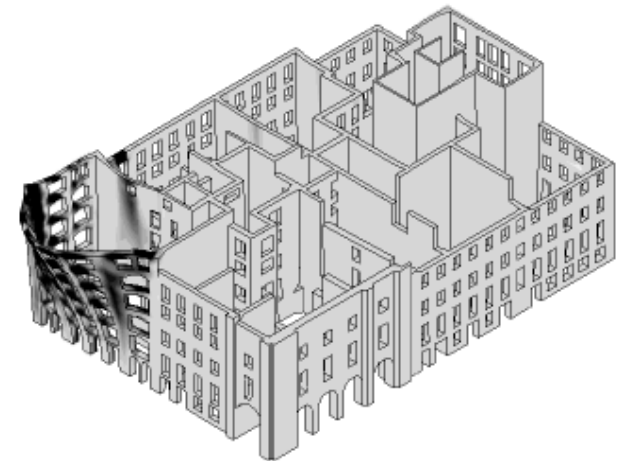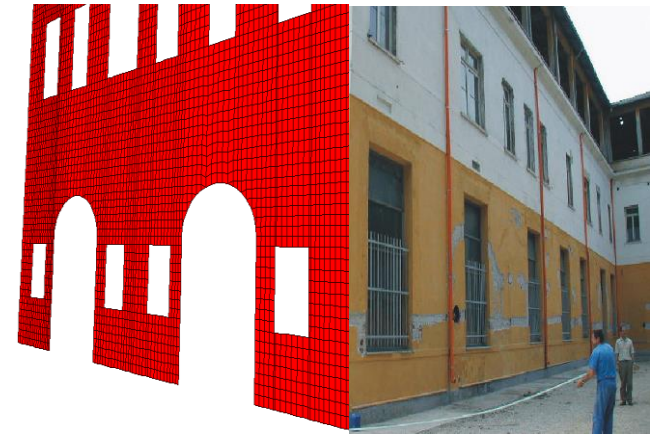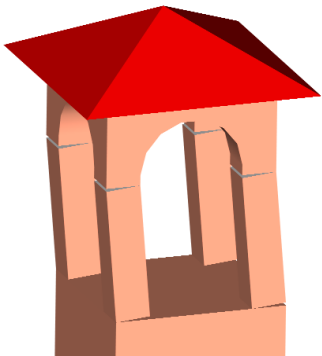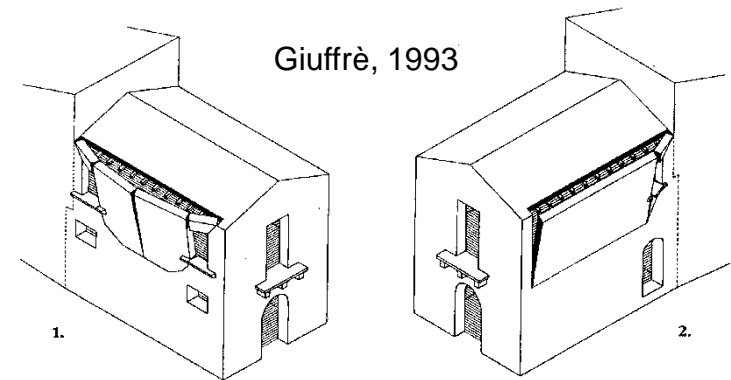
Models for seismic safety assessment

Structural modelling and seismic analysis methods

The **effective response of an existing masonry building to horizontal actions** can be hardly defined, in the majority of cases, by just considering the global behaviour of the structure

Main causes:

- Lack of connection between walls
- Lack of connection between walls and floors
- Low in plane stiffness of floors
- Masonry composition
- Existing deterioration/damages

Giuffrè, 1993

Salò-Garda lake earthquake (24/11/2004)

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Models for seismic safety assessment

Structural modelling and seismic analysis methods

A wide research was performed to appreciate the reliability of the proposed models, also in **comparison** with "traditional" global assessment methods used for masonry buildings: in general, global analytical procedures applied to historical masonry building can be misleading in the interpretation of the actual behaviour of the analyzed buildings.

External walls

Out of plane   **In plane**

Discontinuity

Horizontal structures

Terraces and chimneys

Irregularities in plan and in elevation

Buildings interactions
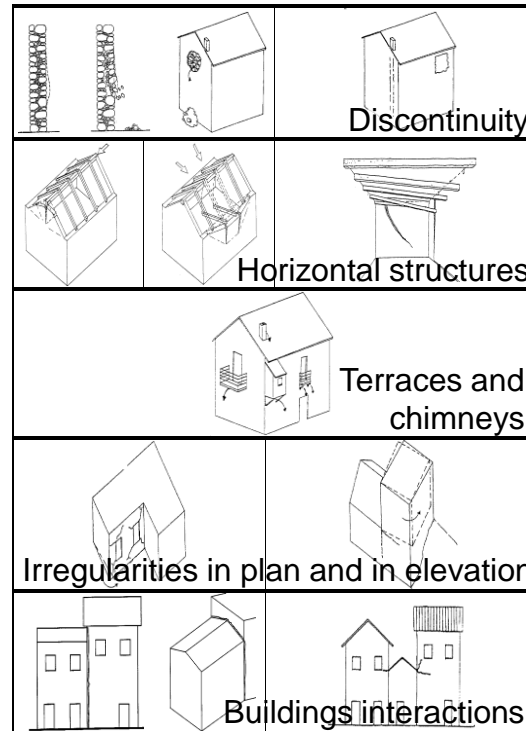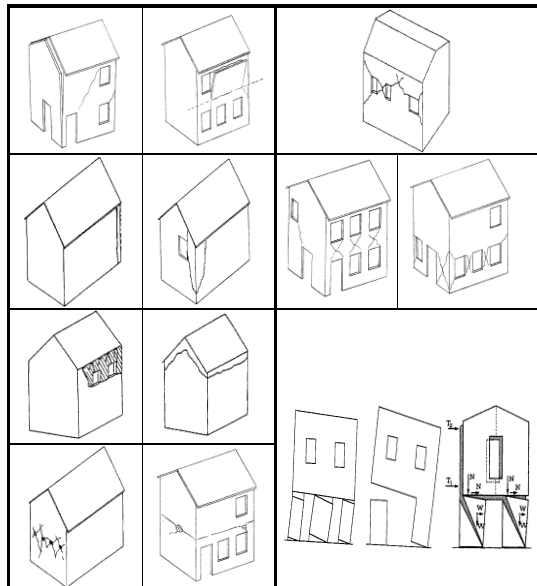
Analytical approach procedures considering the modelling of the elementary failure mechanisms with the limit analysis of **local rigid body kinematic mechanisms of structural macroelements** (portions of the buildings with homogeneous constructive characteristics and structural behaviour) found a better match with the observed damage

Internal walls

Building-soil interaction

Models for seismic safety assessment

Structural modelling and seismic analysis methods

The adoption of suitable interpretative models can not disregard the **structural typologies** of the considered buildings (**isolated or aggregate buildings, churches…**). Several **abaci** graphically depicting the more common failure modes, based on a vast damage mechanism classification work after the recent seismic events and referred to specific constructive typologies, were defined.

Further input data:

- the construction of the building following "correct" **empirical rules**

- the **historical response** of the building to past seismic events

Local models

Aggregate buildings

Observation of the damages of the churches

Façade Overturning
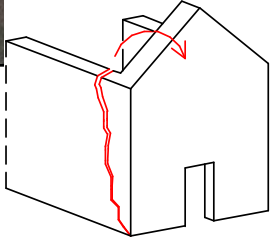
EXAMPLES
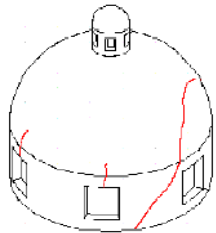
Hall transverse response

Dome
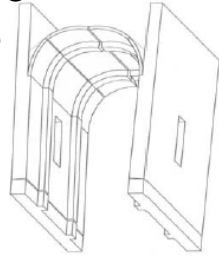
Bell tower
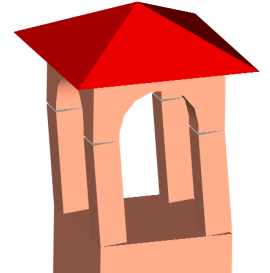
Façade in-plane mechanisms
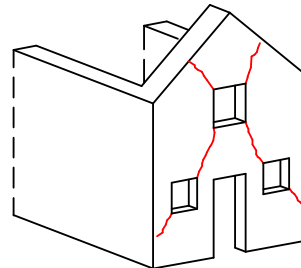
Apses overturning

Classification of damage mechanisms for churches
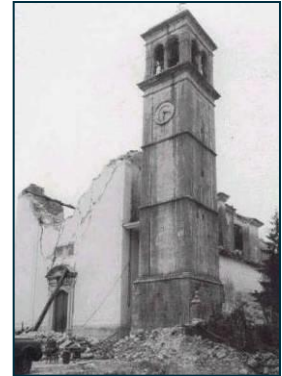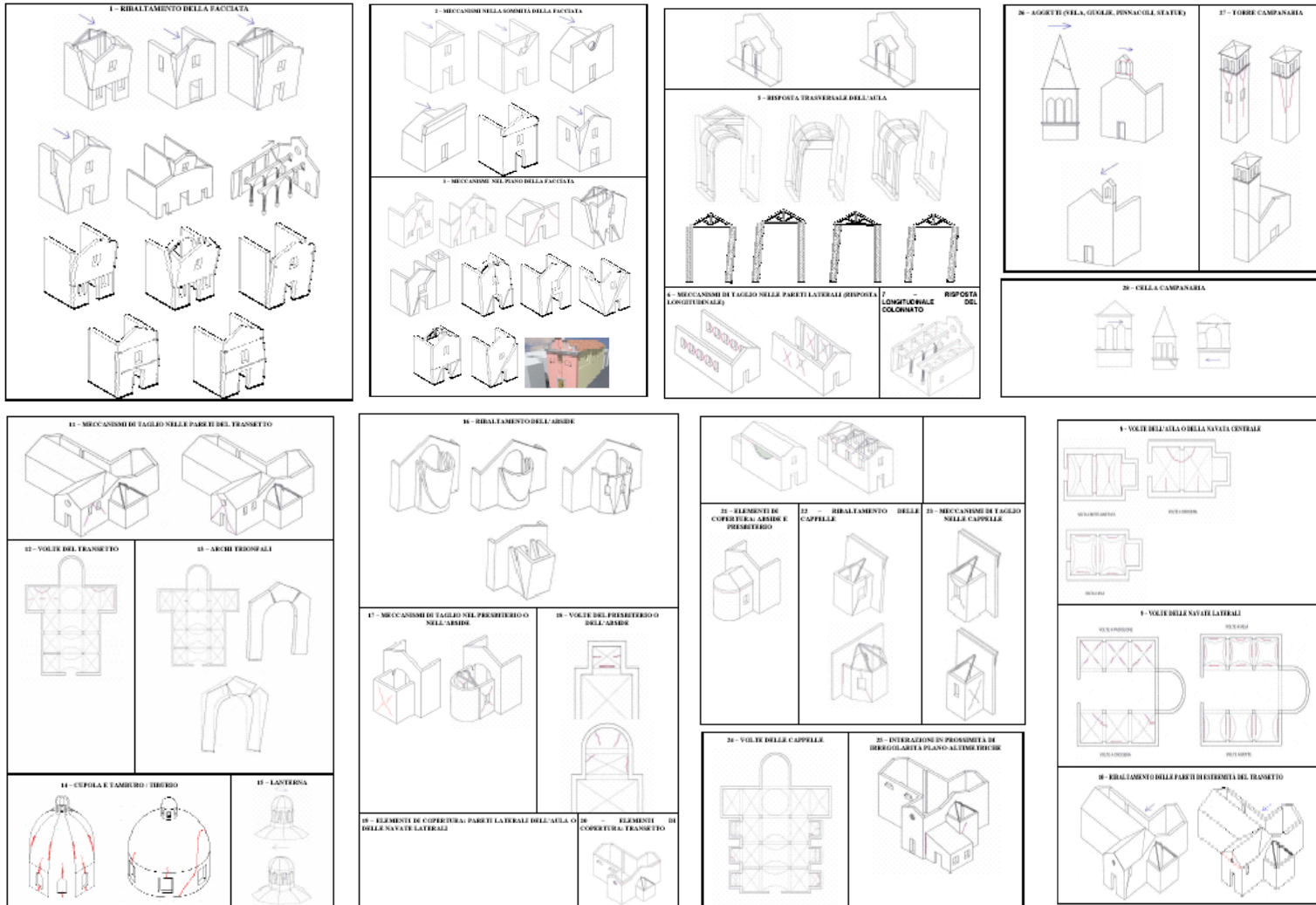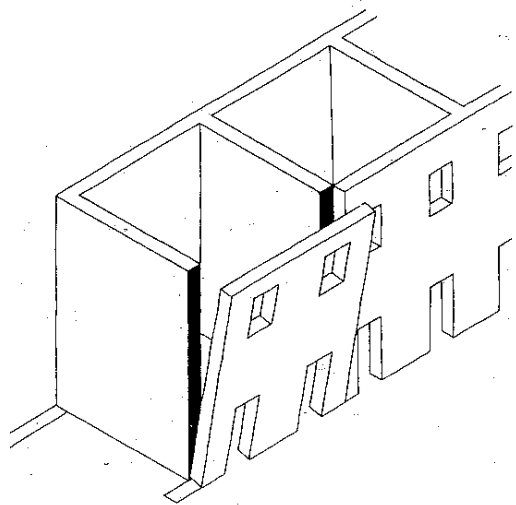
Local behavior

"FIRST MODE" MECHANISMS

Out-of-plane collapse of the walls due to absence of connections with perpendicular walls and floors
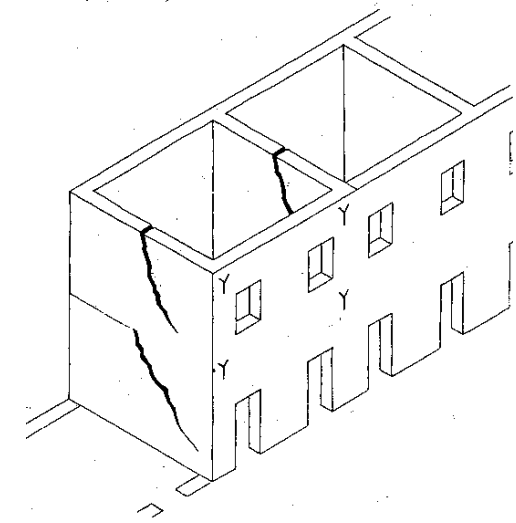
Global behavior

"SECOND MODE" MECHANISMS

In-plane cracking of the walls, in presence of effective constraints, bearable force levels much higher than in the previous case.

OUT-OF PLANE MECHANISMS

Vertical strips of masonry:

Simple overturing
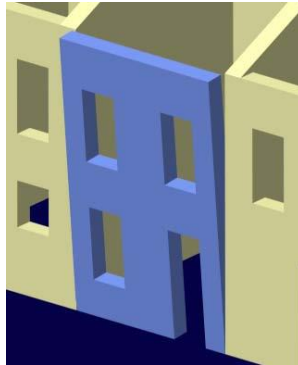
Composed overturning

Vertical bending
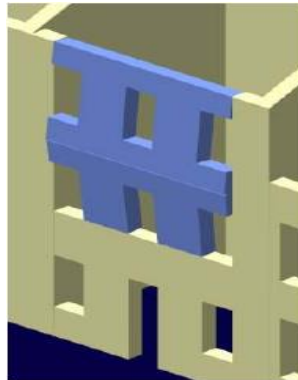
Horizontal bending
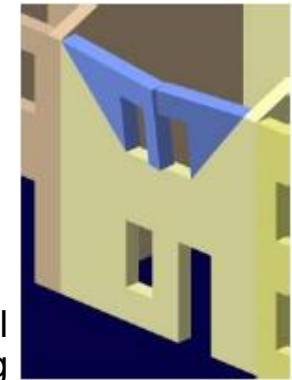
Horizontal strips of masonry:
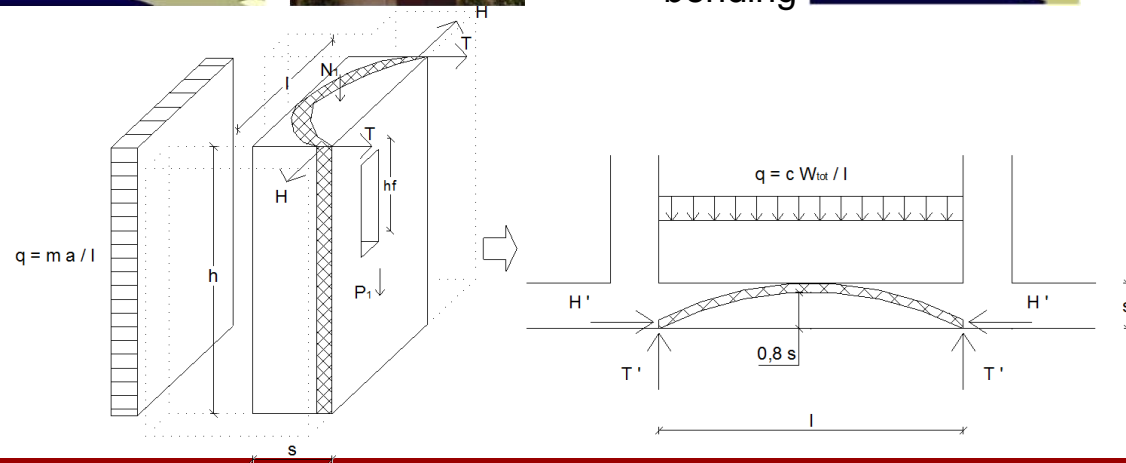
IN PLANE MECHANISMS

- Scarce quality of masonry and presence of openings and discontinuities
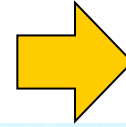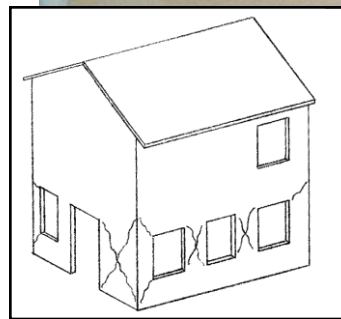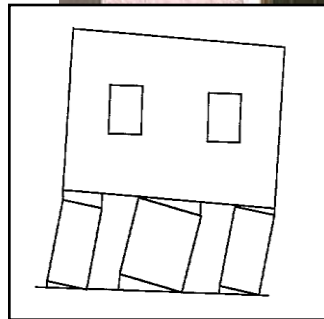
- Shear collapse of the walls
- Rotation and in plane overturning

Models for seismic safety assessment

Structural modelling and seismic analysis methods

It is in general inappropriate to propose an assessment tool instead of another, since the strategy adopted depends on several factors.

The choice of the more suitable methodology/ies should be then made taking into account their advantages and limitations and, more important, the **definition of the aims and limits of the analysis to be performed**.

Standing however the several uncertainties denounced, it seems sensible to use **more than a single technique** to the evaluation of the seismic response of a historical masonry structure, possibly arising from different studies and underlying theories, to finally perform a final comparison of data emerged, thus increasing the degree of confidence in the obtained results.
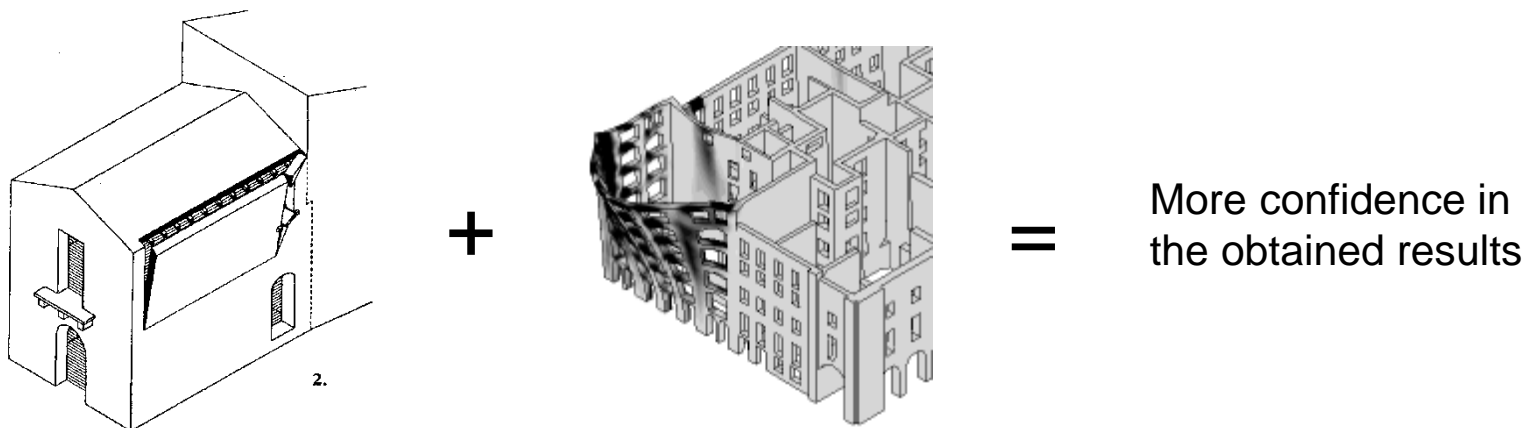
More confidence in the obtained results

Models for seismic safety assessment

Structural modelling and seismic analysis methods

In the case of cultural heritage buildings the assessment of the structure capacity and seismic safety must be considered at local and global level, using suitable analysis methods.
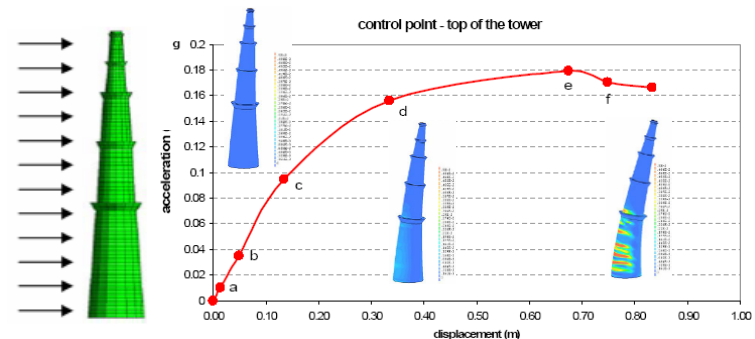
It is possible to consider:

- **linear static analysis**
- **modal dynamics analysis**
- **non linear static analysis**
- **non linear dynamics analysis**

Example of not linear dynamic analysis (using 3muri software)
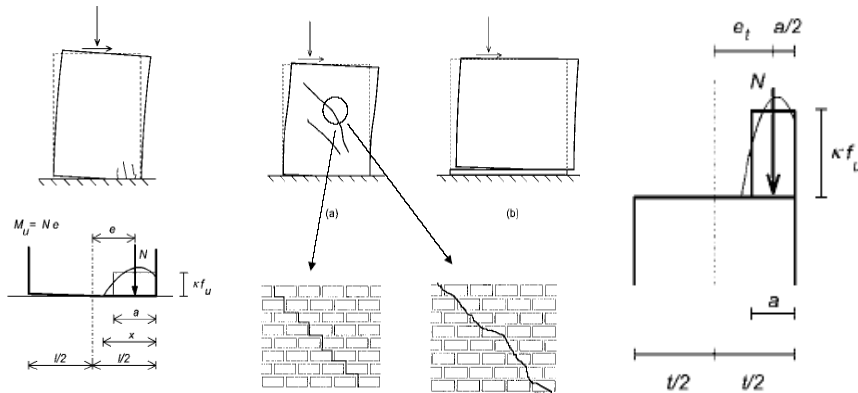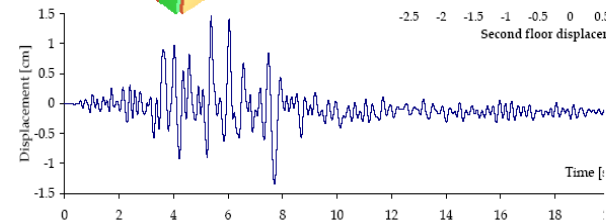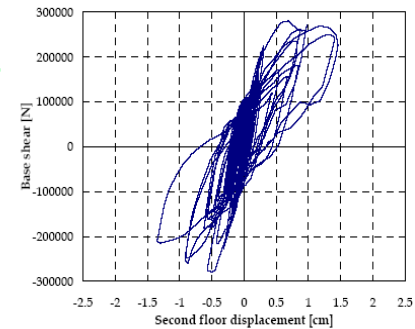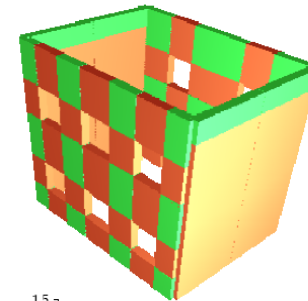
In-plane and out-of-plane mechanisms (bending, shear, sliding…) that have to be verified when considering a linear static analysis

control point - top of the tower

Non linear static analysis

SIMPLIFIED METHODOLOGIES FOR SEISMIC VULNERABILITY ASSESSMENT ON LARGE SCALE

Historical Centres:

- minor building as a cultural heritage

- masonry buildings

- traditional rules of art and intervention techniques

- aggregations of building
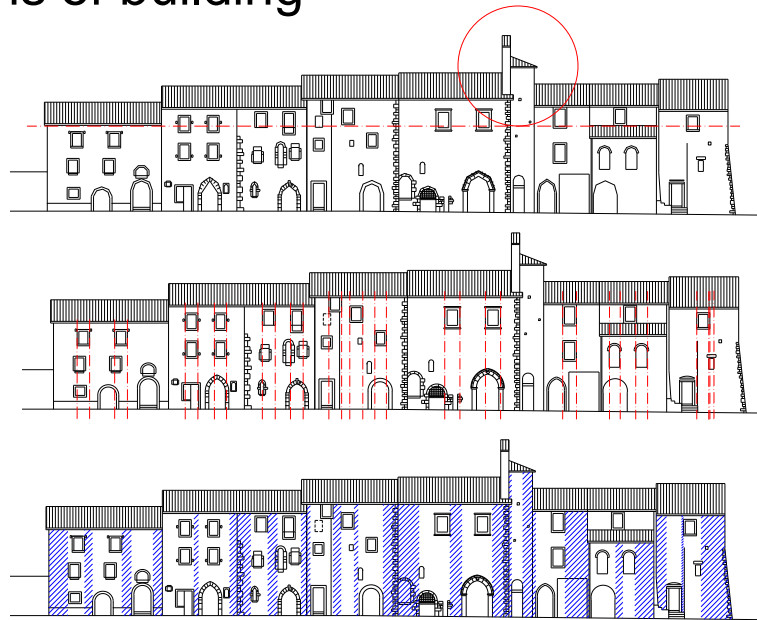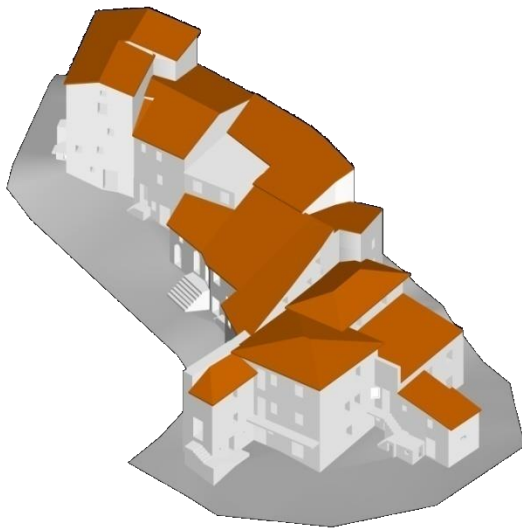
Simplified methodologies for seismic vulnerability assessment

Seismic vulnerability analysis of historic buildings at a **territorial level**

⬇

Evaluation carried out on numerous samples

⬇

Simplified methodologies and models, that are reliable, based on empiric parameters

- **Levels of analysis**

- **Methods of analysis**

Campo di Brenzone

Simplified methodologies for seismic vulnerability assessment

Automatic procedures for the systematic assessment of the existing masonry buildings vulnerability developed by the University of Padova:

Vulnus: global seismic vulnerability analysis (vulnerability assessment and damage probability) of isolated or clustered masonry buildings through different in plane and out of plane mechanisms combinations and qualitative informations.

c-Sisma: local analysis of vulnerability through the application of single kinematic models applied to the more significant macroelements. It also performs the safety analysis according to the Italian regulation

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THE CHURCH OF 'MADONNA DEL PIANTO' (Padova)

XVIII Century church by Frigimelica with central plan and some irregularities due to following resets done in the first half of the XX Century

CURRENT CONDITION
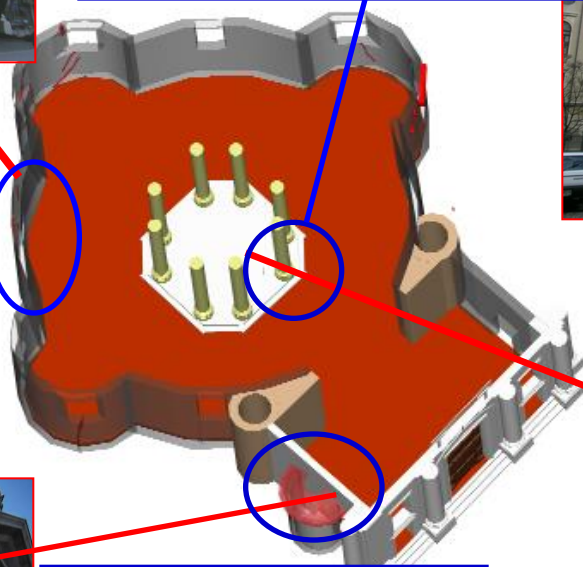
CRITICAL SURVEY

Example: the church of S. Maria del Pianto

Southern Apse

Wall C

Western Apse

Identification of the **macroelements**

Wall D

Wall B

Eastern Apse

Lateral Wall A
Western Bell Tower
Sacristy

Wall E

Lateral Wall F
Eastern Bell Tower
Baptismal Font

Façade
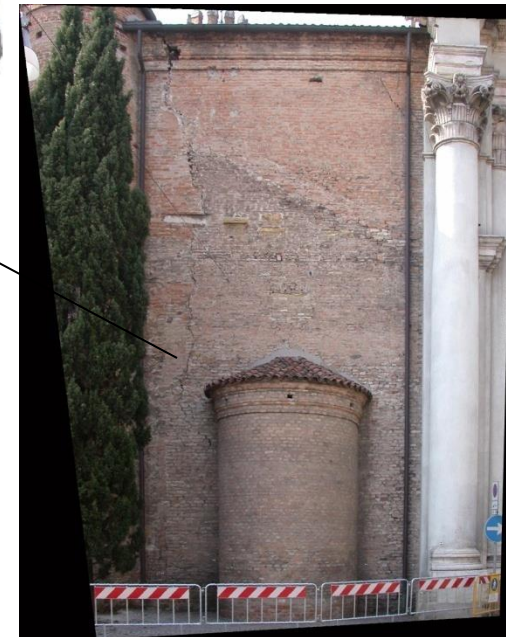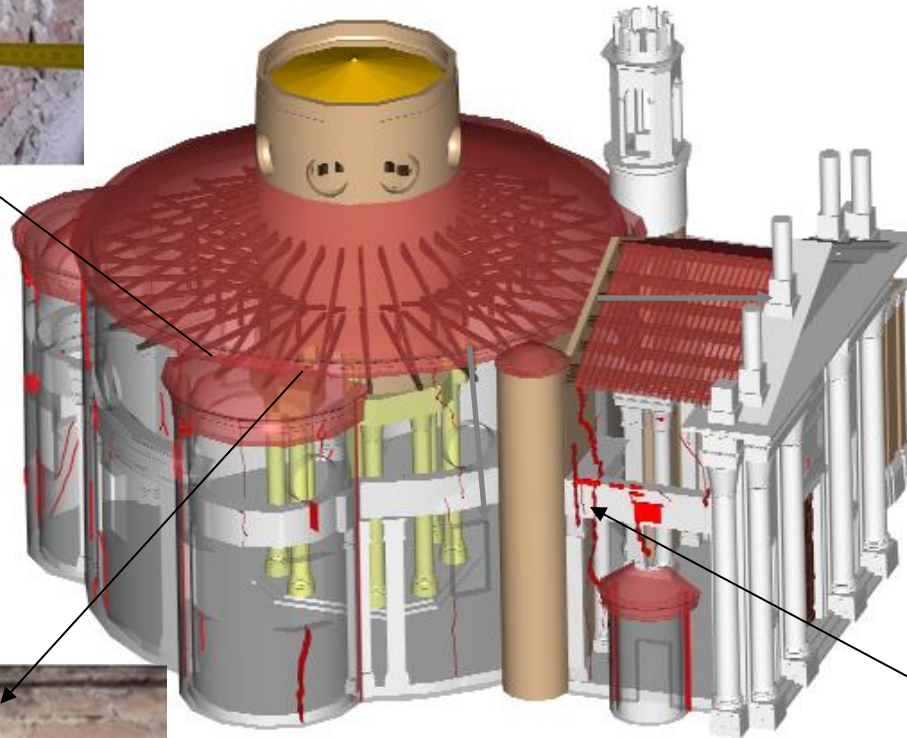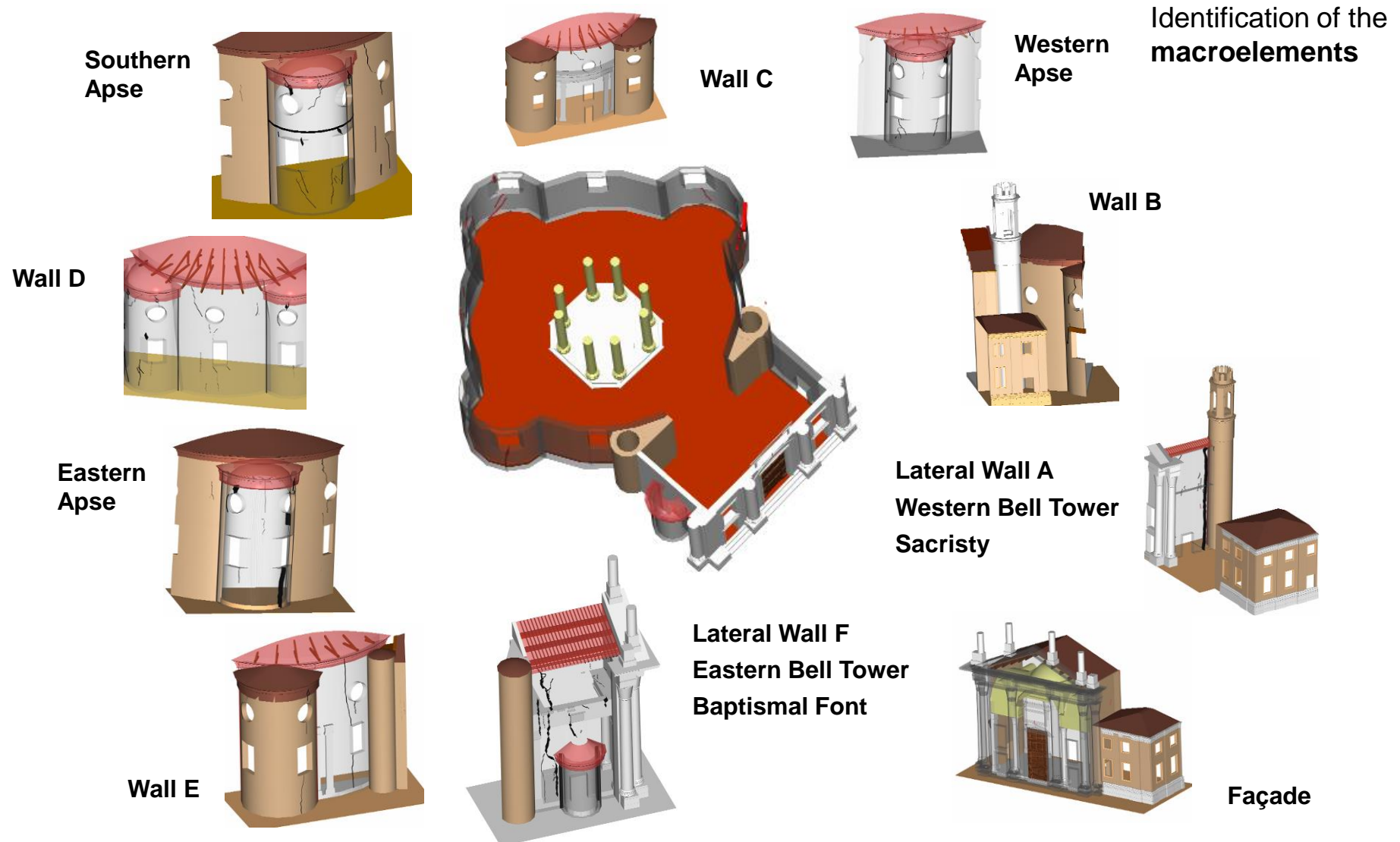
Example: the church of S. Maria del Pianto
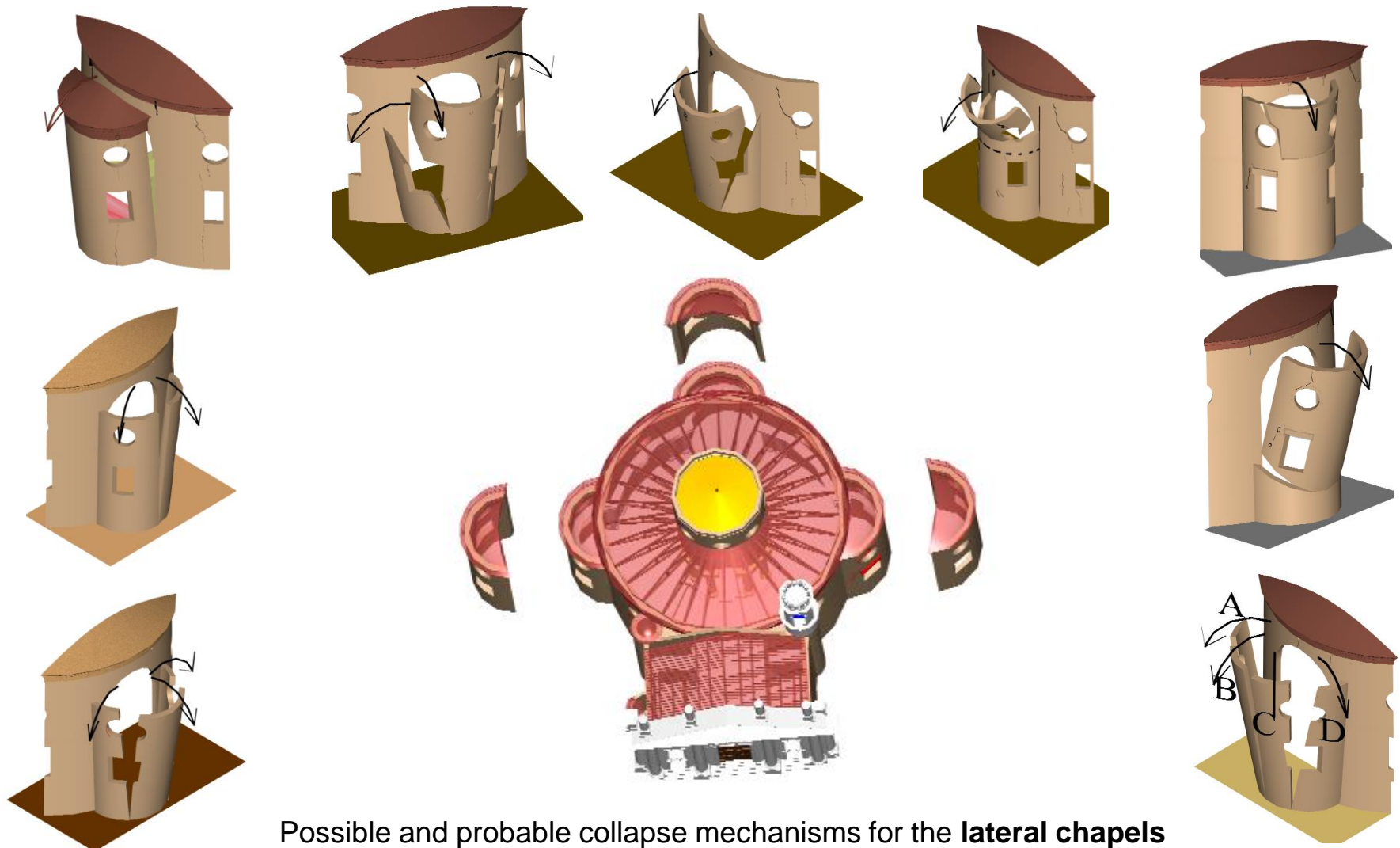
Possible and probable collapse mechanisms for the **lateral chapels**

Example: the church of S. Maria del Pianto
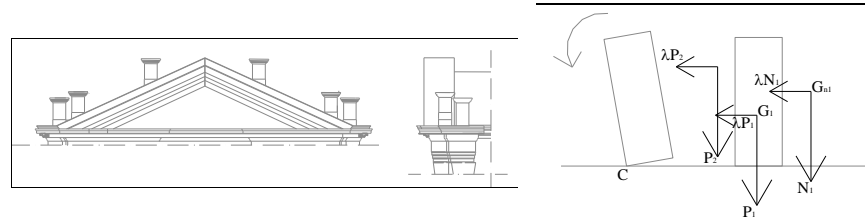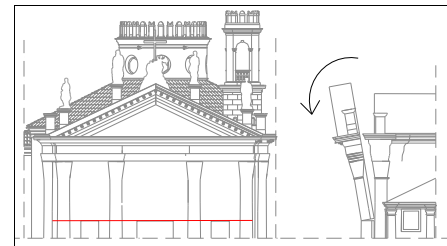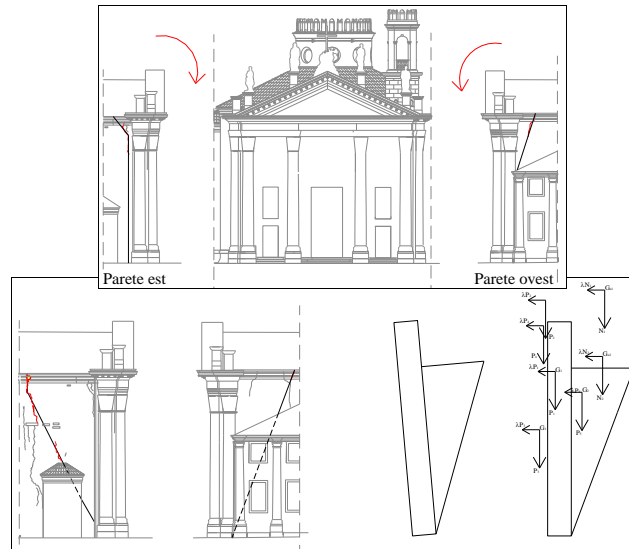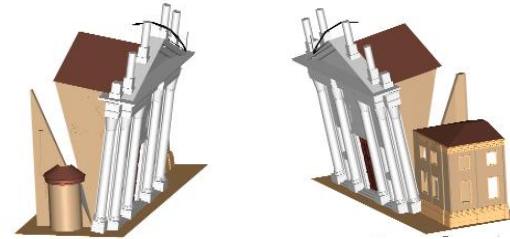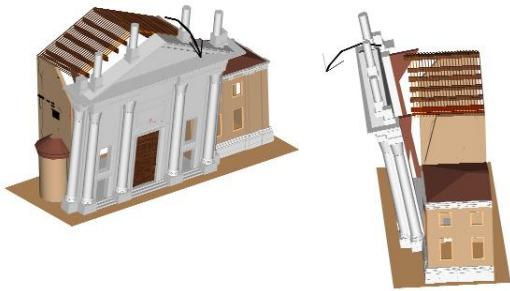
M_{st} = 52230 daN m

M_{inst} = 3984900 daN m

c = 0,0131

M_{st} = 236220 daN m

M_{inst} = 4441250 daN m

c = 0,05319

M_{st} = 2125 daN m

M_{inst} = 148300 daN m

c = 0,0143

M_{st} = 41137 daN m

M_{inst} = 1473034 daN m

c = 0,0279

Possible and probable collapse mechanisms for the **façade**

Example: the church of S. Maria del Pianto

From the analyses carried out, it was pointed out that the most vulnerable element is the **façade**, in case of **overturning with partial involvement of the lateral walls**.

This is also a possible mechanism, due to the presence of corresponding **crack pattern** close by the façade.
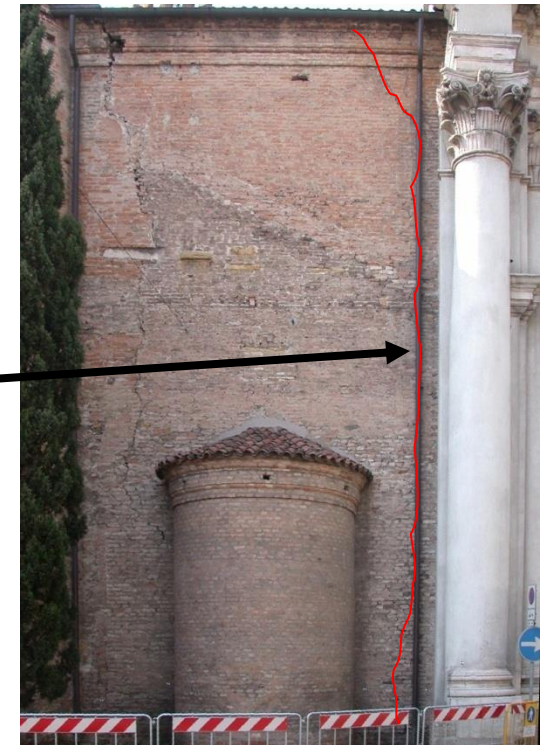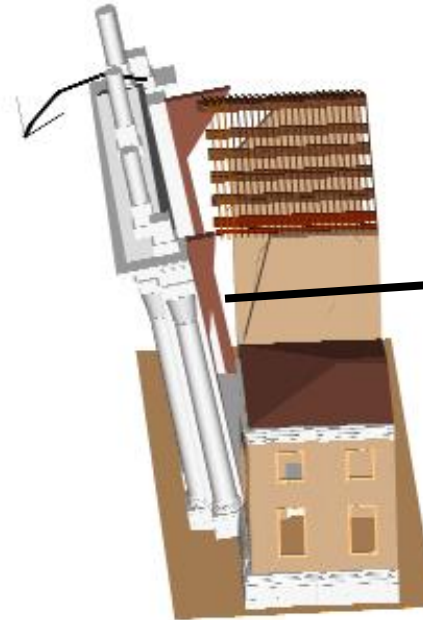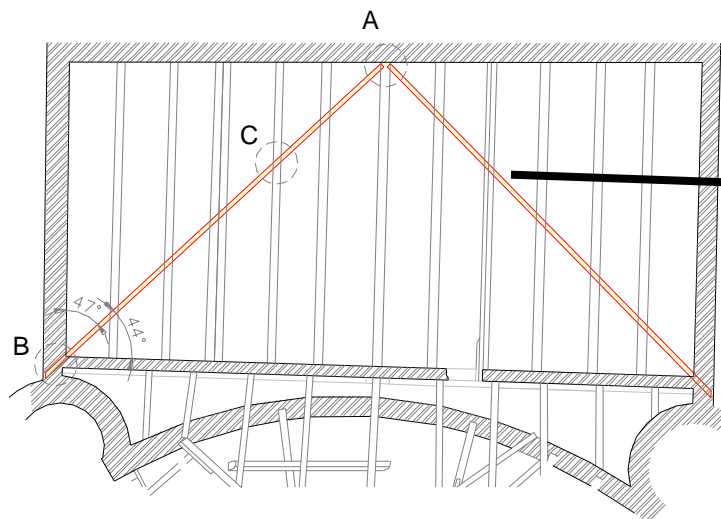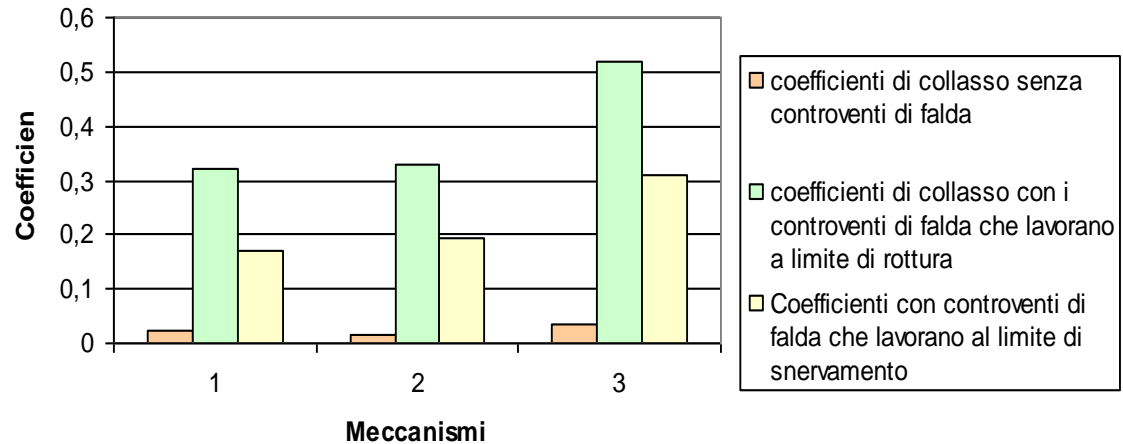
Example of seismic improvement: the church of S. Maria del Pianto
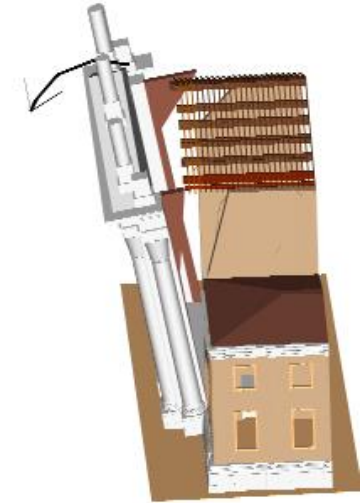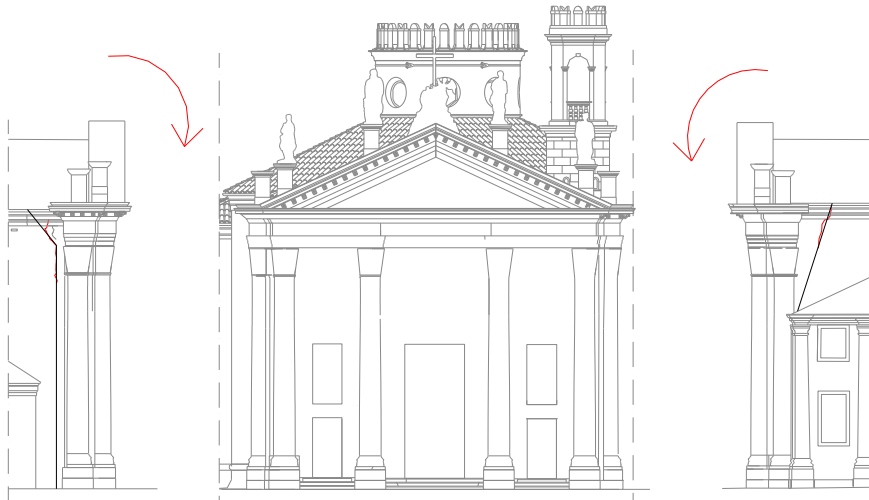
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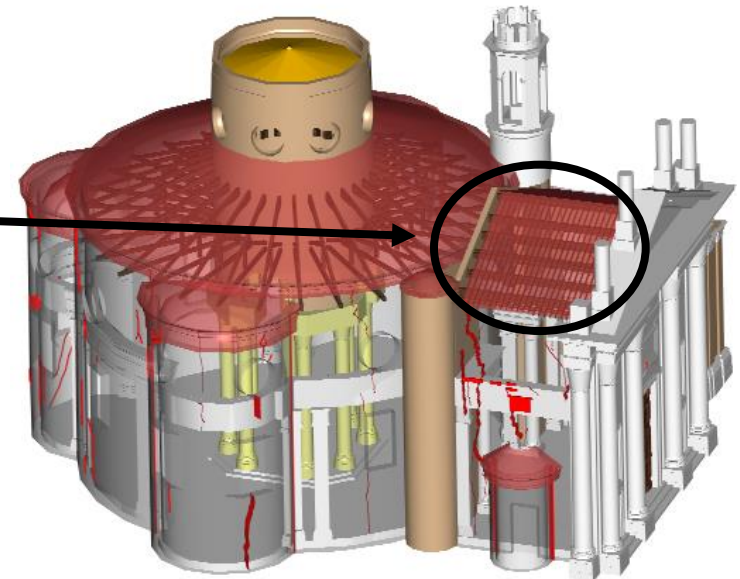
M_{st} = 52230 daN m

M_{inst} = 3984900 daN m

c = 0,0131

Example of seismic improvement: the church of S. Maria del Pianto

Collapse coefficients in the previous state (orange) and simulating **bracings** (green), installed between 2003 and 2004 on the roof above the façade. This has significantly **improved the seismic response of the façade**, as shown by the increase of the collapse coefficient for the overturning.
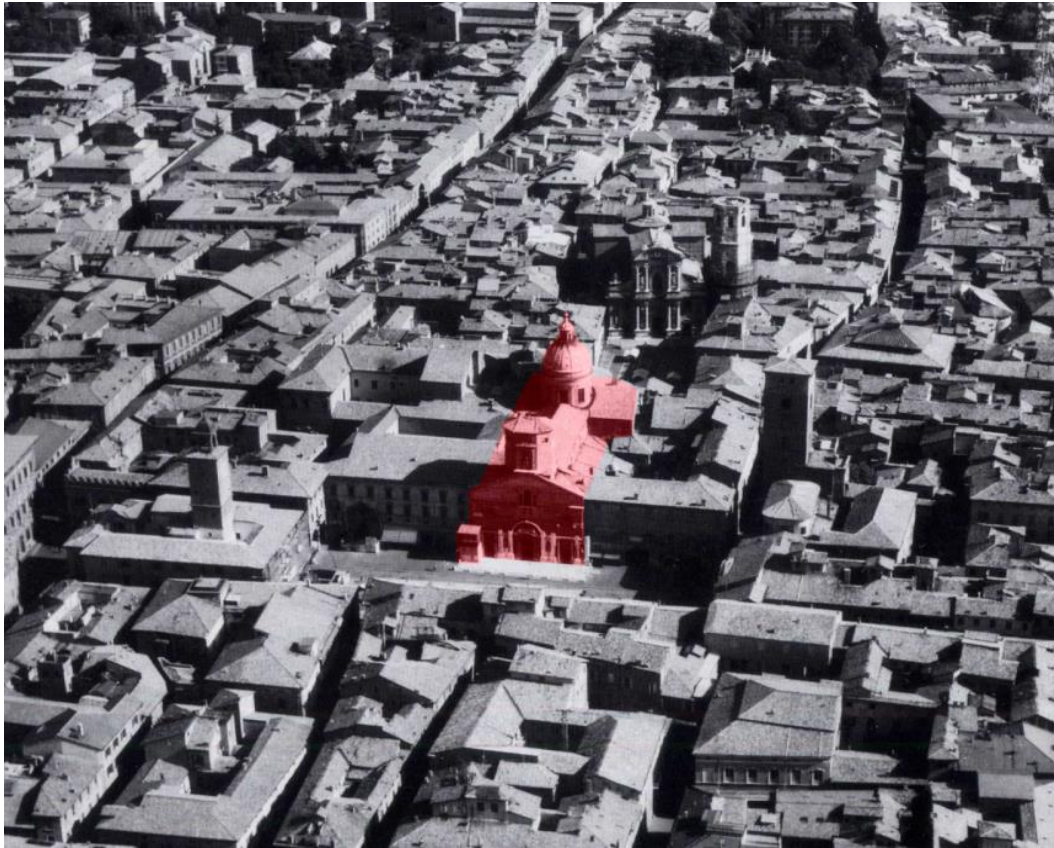

Legend:
- coefficienti di collasso senza controventi di falda
- coefficienti di collasso con i controventi di falda che lavorano a limite di rottura
- Coefficienti con controventi di falda che lavorano al limite di snervamento

X-axis: **Meccanismi** — Y-axis: **Coefficien**

Roof bracing

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Santa Maria Assunta Cathedral - Reggio Emilia
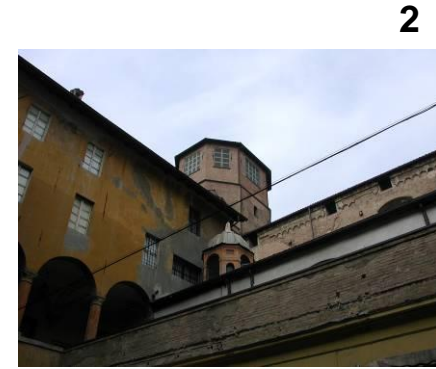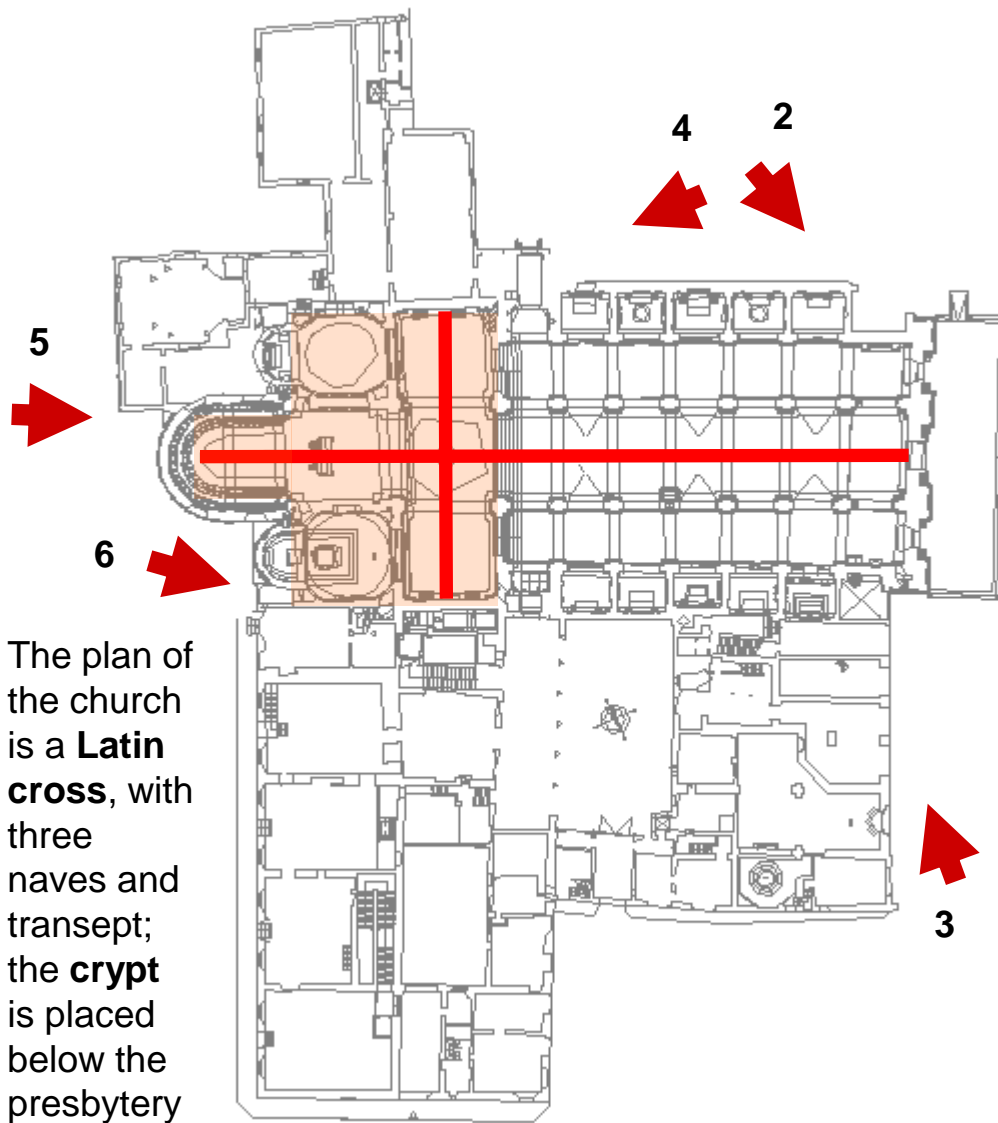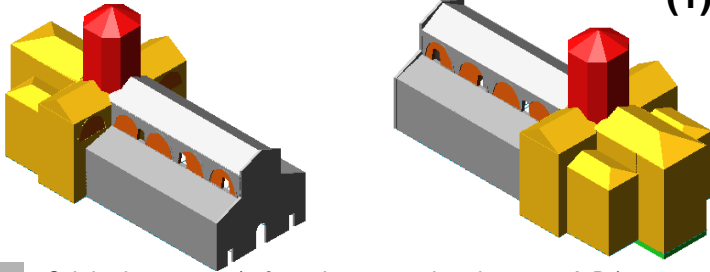

Roman mosaic under the crypt

Main façade

Main nave

Cupola from below

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
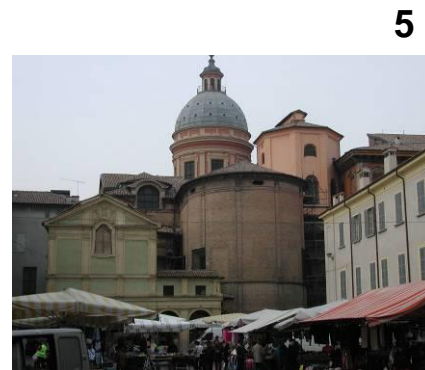
Historical notes

(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.
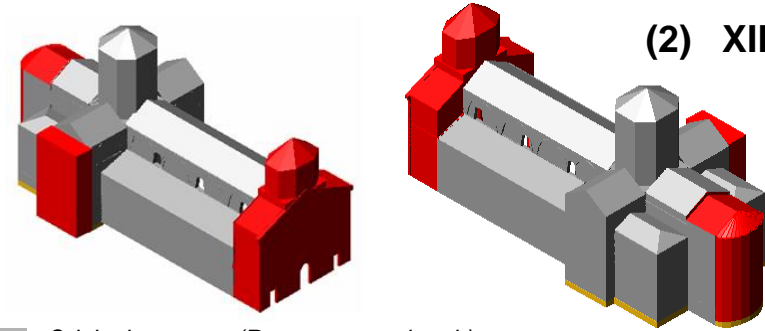
Original structure (Romanesque church)

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.
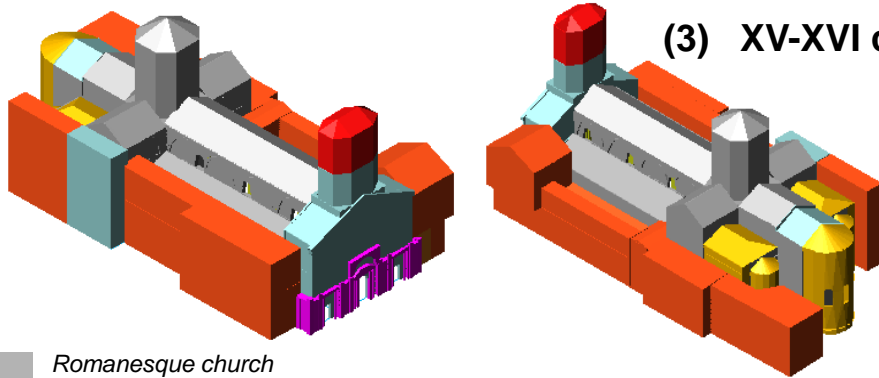
Romanesque church

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.
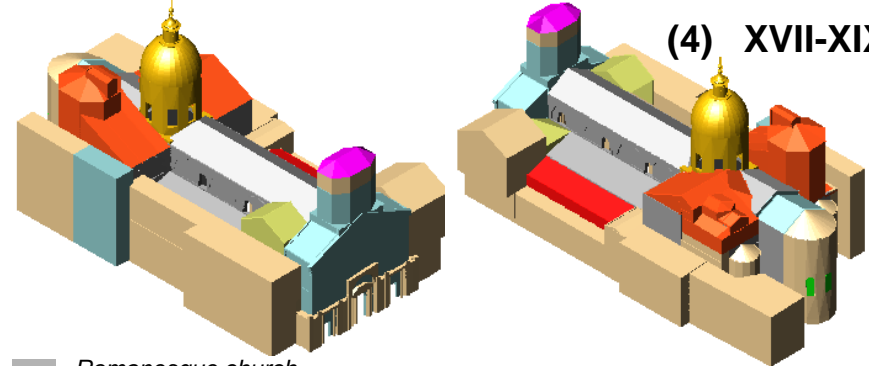
Romanesque church

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

Claudio Modena

Analysis Methods for Unreinforced Heritage Masonry

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
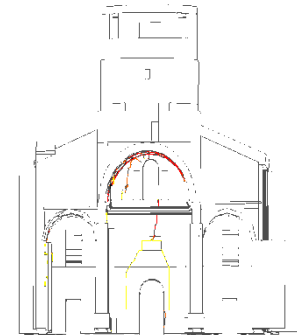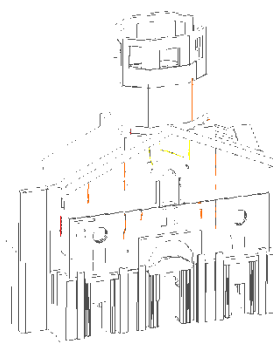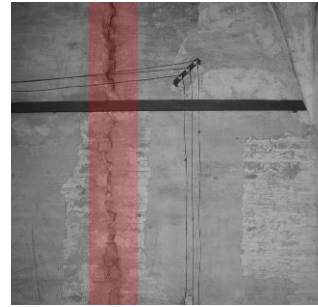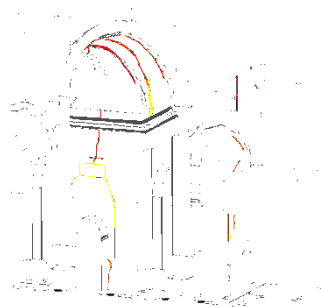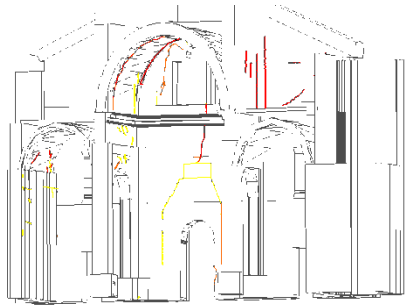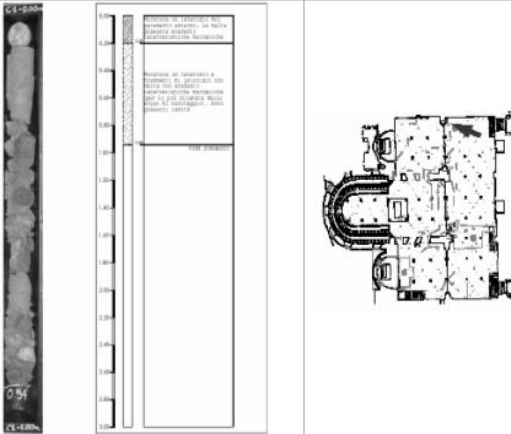

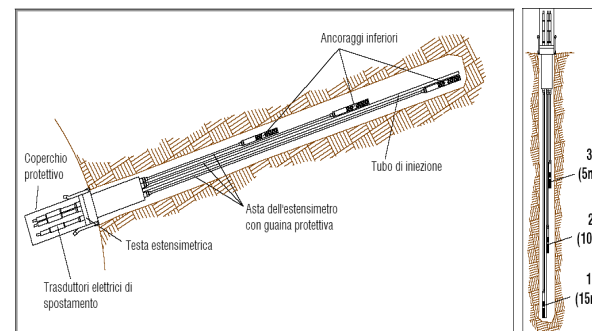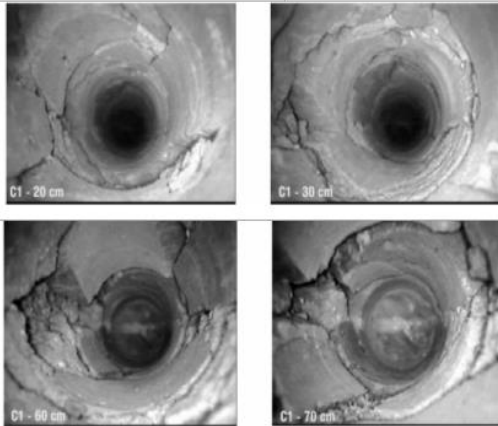Façade area
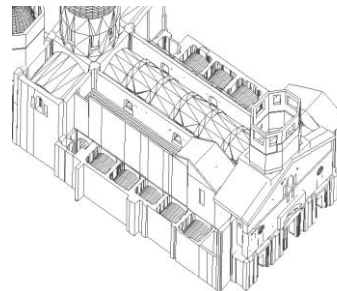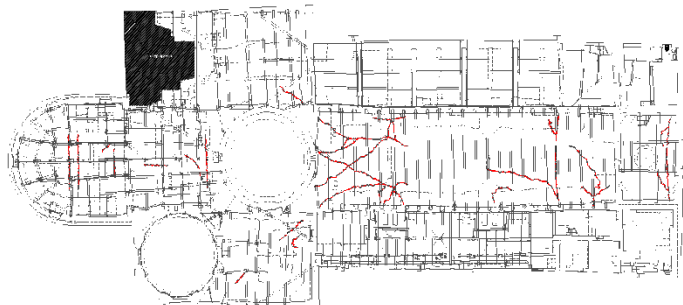
Nave vaults

EXPERIMENTAL INVESTIGATIONS

- ❑ **Core samples;**
- ❑ **endoscopy.**

Static monitoring:
- ❑ **Relative displacements between the vertical structures;**
- ❑ **eventual variations in the openings on the main cracks;**
- ❑ **settlements of the foundation soils below the crypt's two main pillars;**
- ❑ **air temperature control.**

Core sample C1
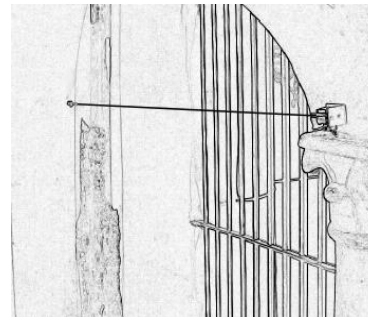
Extensometer cables

Multi-base extensometer

Electrical extensometer

EXPERIMENTAL INVESTIGATIONS

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
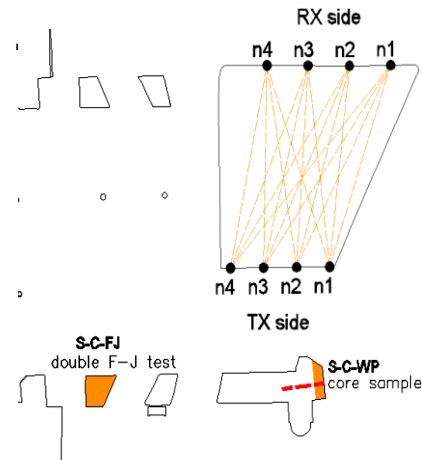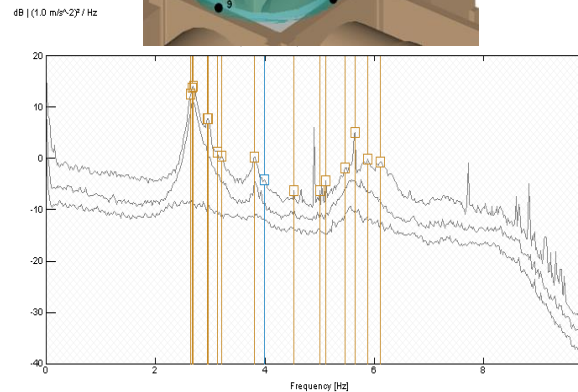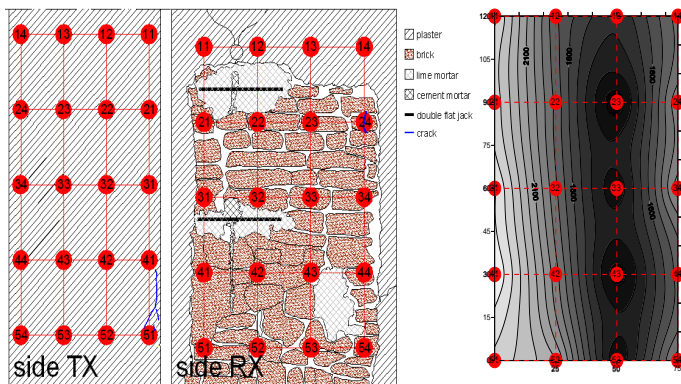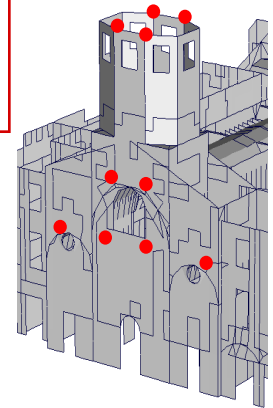

Sonic testing

**dome**
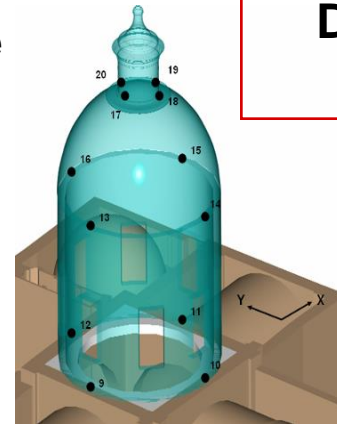
Dynamic testing

façade
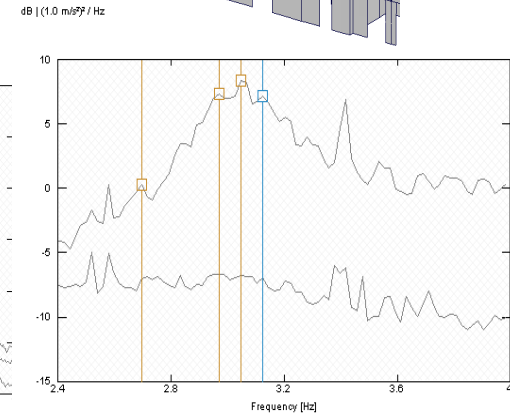
Dynamic tests

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
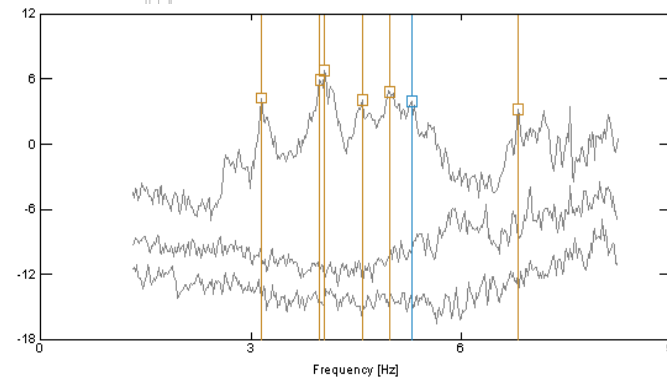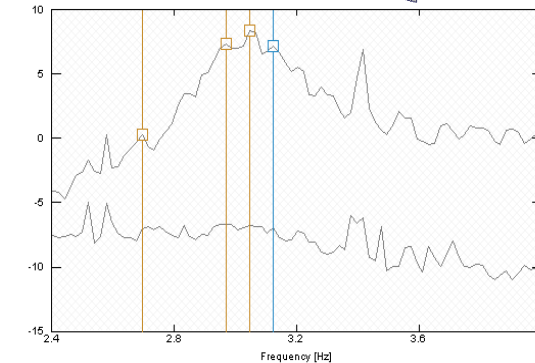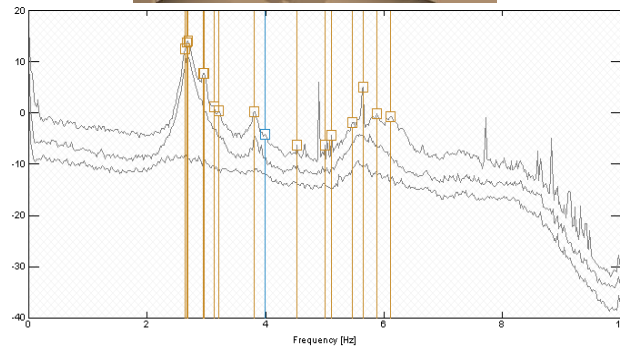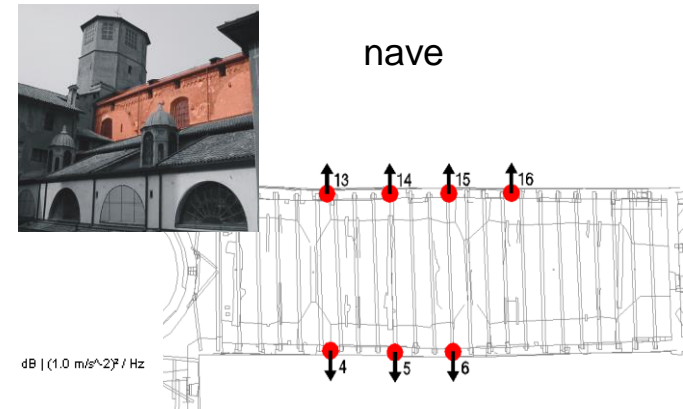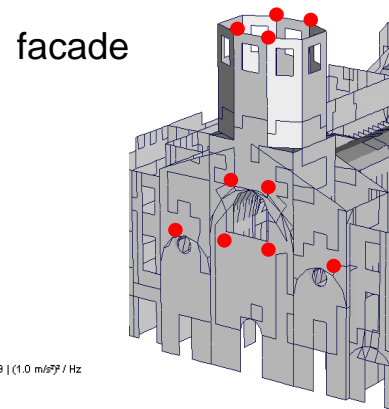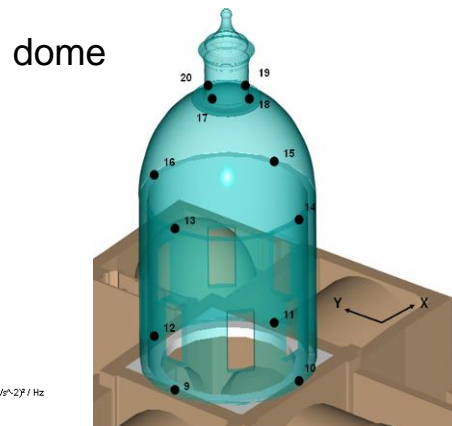
Structural modelling: FEM

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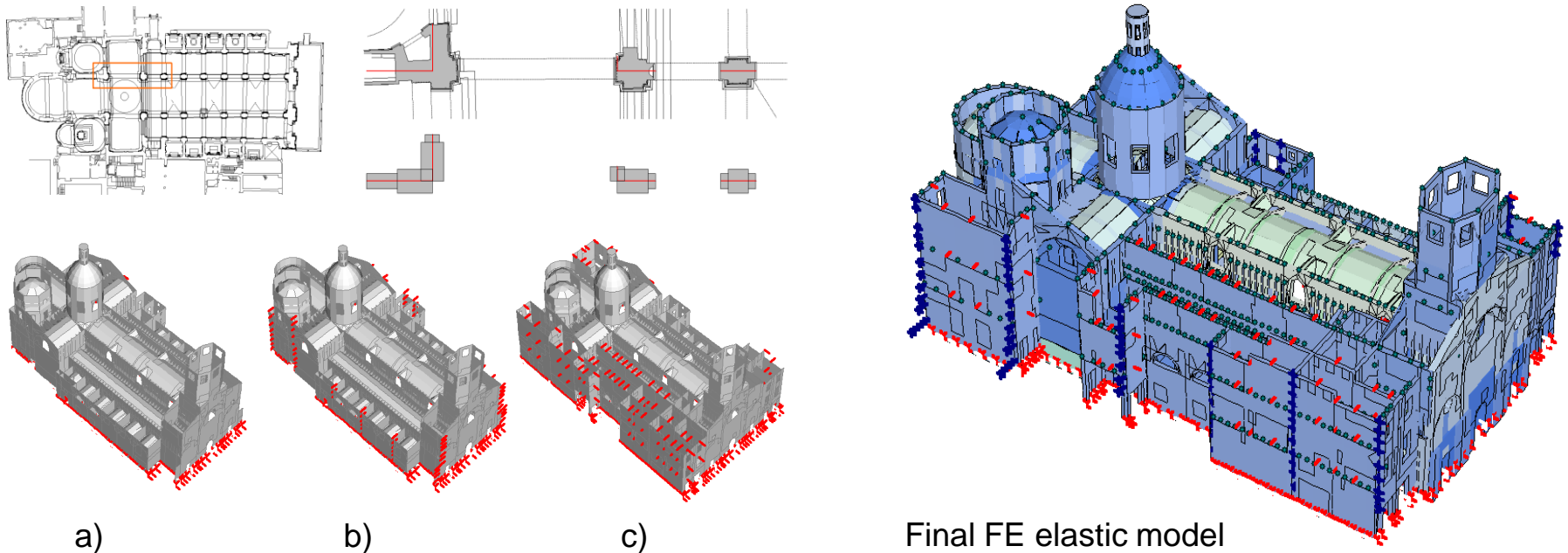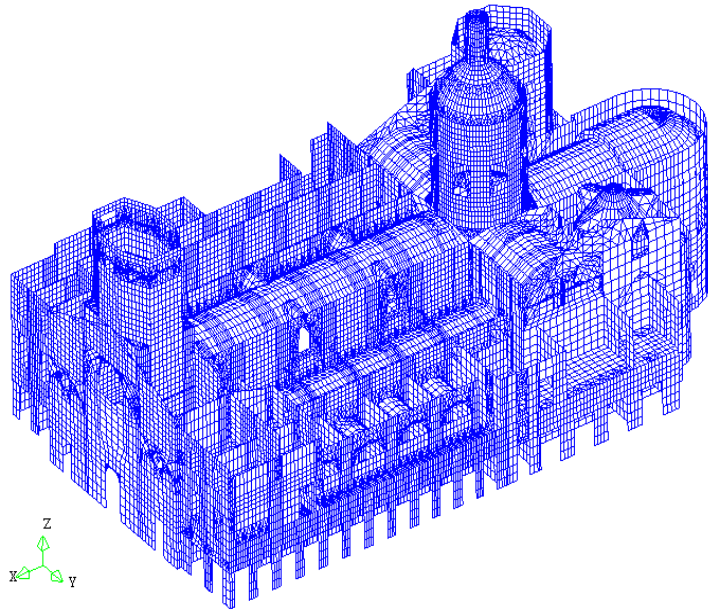
The **full model** comprises 60,158 elements with 161,218 nodes: besides the 2D rectangular curved shell quadratic elements, it was necessary to use triangular elements because of the complex model's geometry.

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.
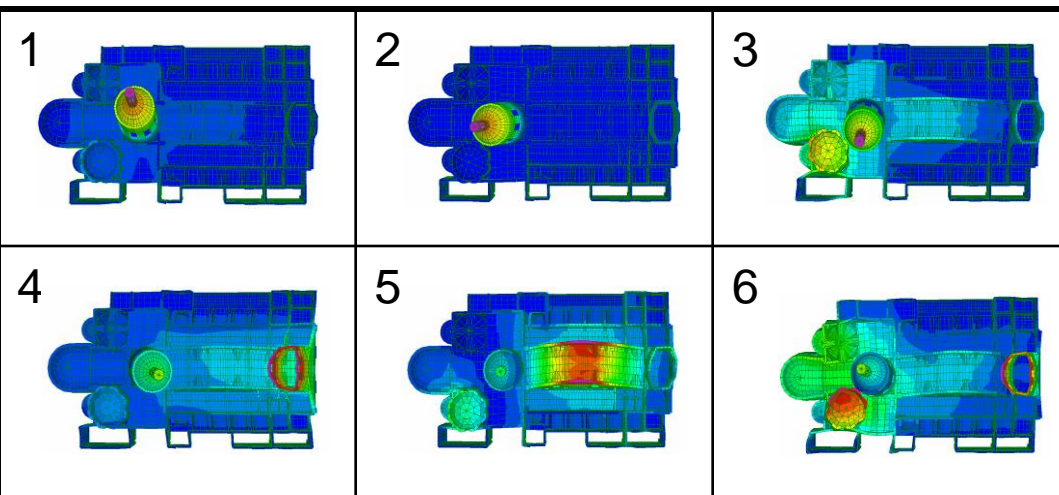

Mode nr.	Frequency	mass participation (%)		Mode description
	(Hz)	Longit.	Transv.	
1	2,65	0,21	16,24	Transverse bending, dome
2	2,66	7,76	0,28	Longitudinal bending, dome
3	2,90	1,36	20,05	Transverse bending, dome
4	3,02	27,82	1,20	Longitudinal bending, façade
5	3,33	0,02	9,05	Transverse bending, nave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

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.
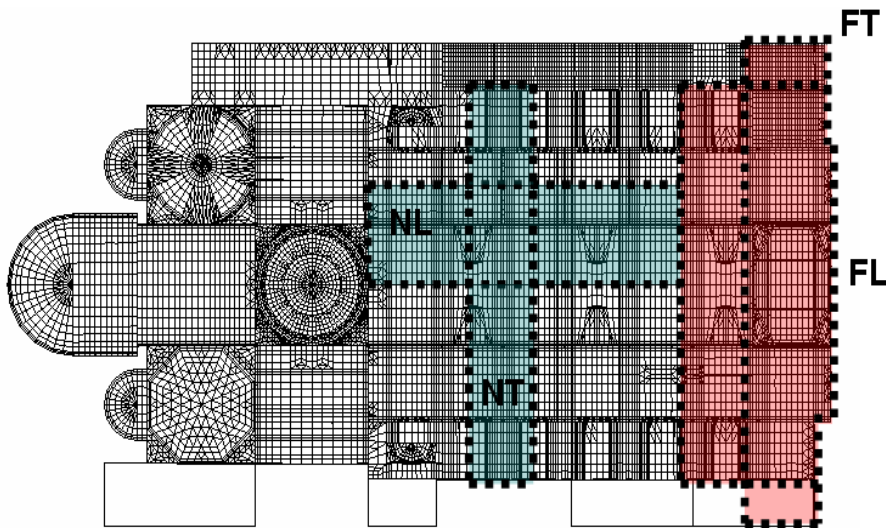
MODEL	FL	FT	NL	NT
MASS DENSITY - STRUCTURAL (kg/m³)	1800	1800	1800	1800
MASS DENSITY – VAULTS' FILLING (kg/m³)	1500	1500	1500	1500
YOUNG MODULUS (MPa)	2100	2100	2100	2100
POISSON'S COEFF.	0.2	0.2	0.2	0.2
TENSILE STRENGTH (MPa)	0.10	0.10	0.10	0.10
CRACK BANDWIDTH (m)	-	-	0,5	0,5
Gf^l (J/m²)			100	100
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.

(a) (b) (c) (d)


(a) (b) (c) (d)
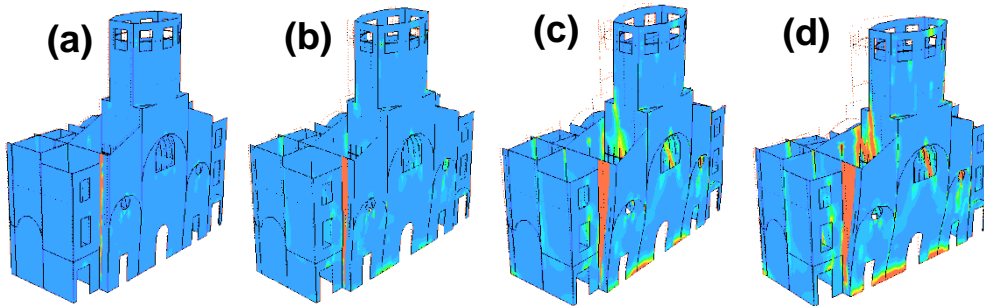
FACADE OUT OF PLANE
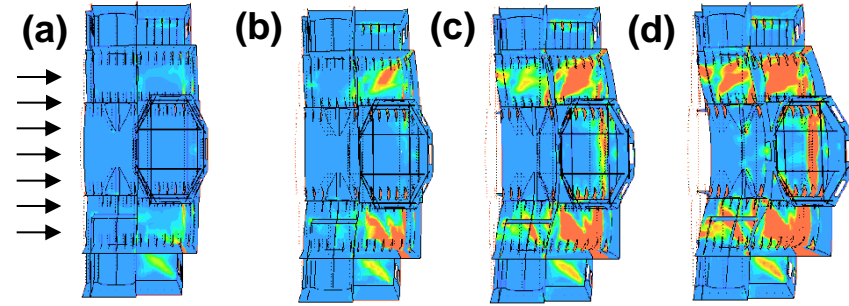
Progression of the damage mechanism; deformation is magnified (x100)

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.

Claudio Modena

Analysis Methods for Unreinforced Heritage Masonry
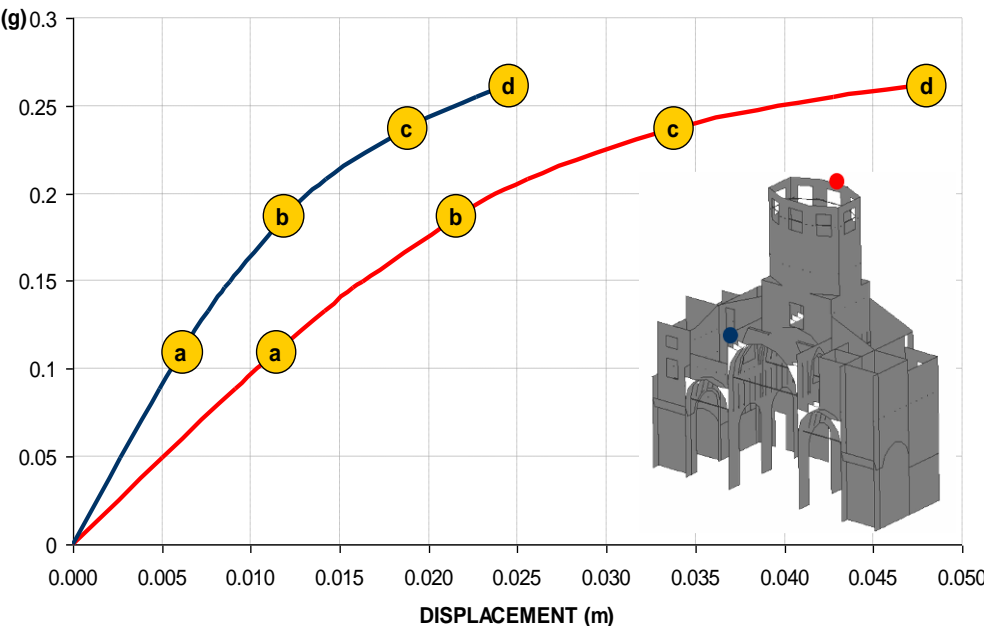
Model FL

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)
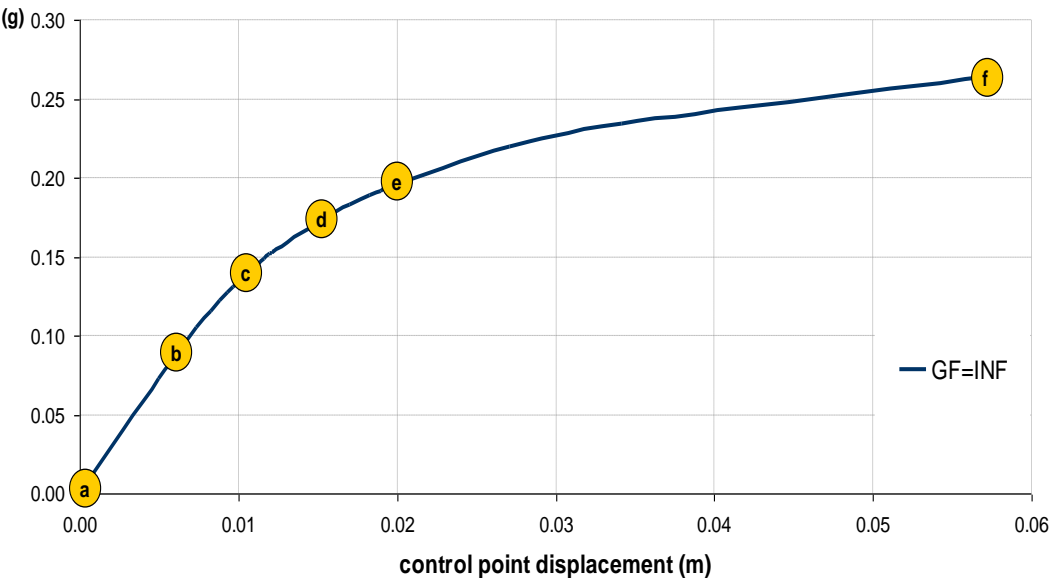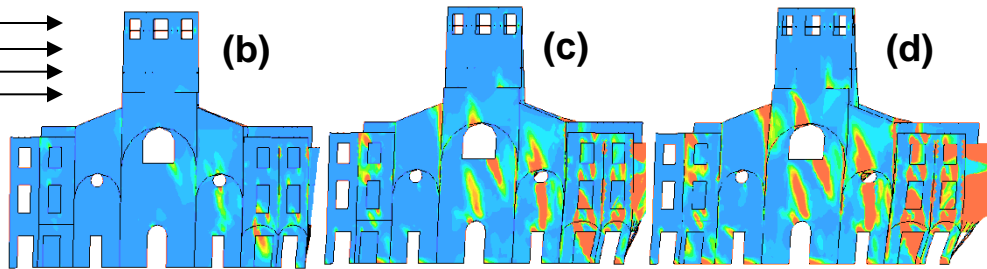
Model NL

(c) (d) (e) (f)
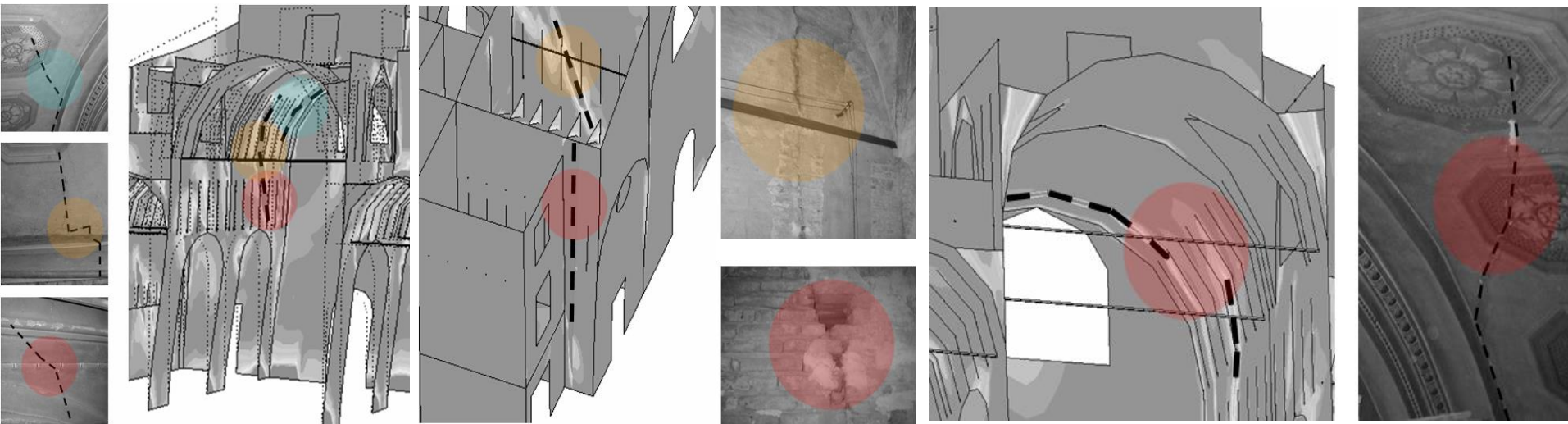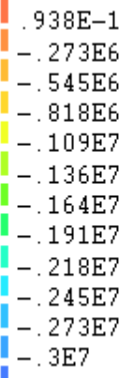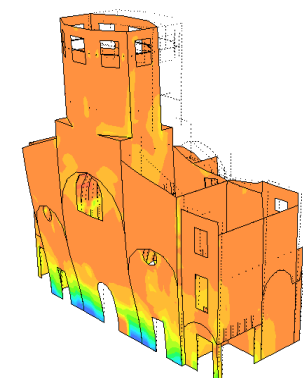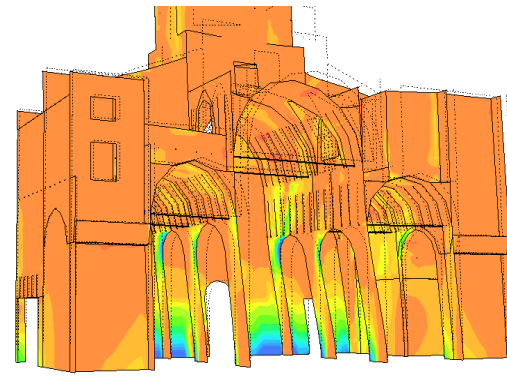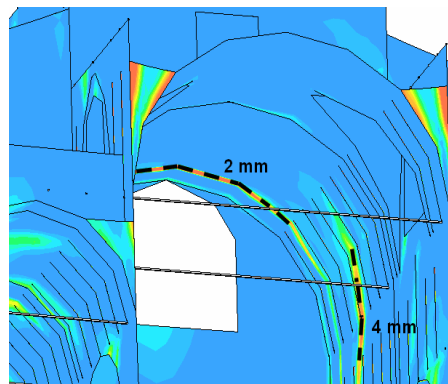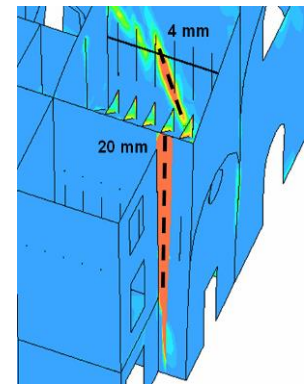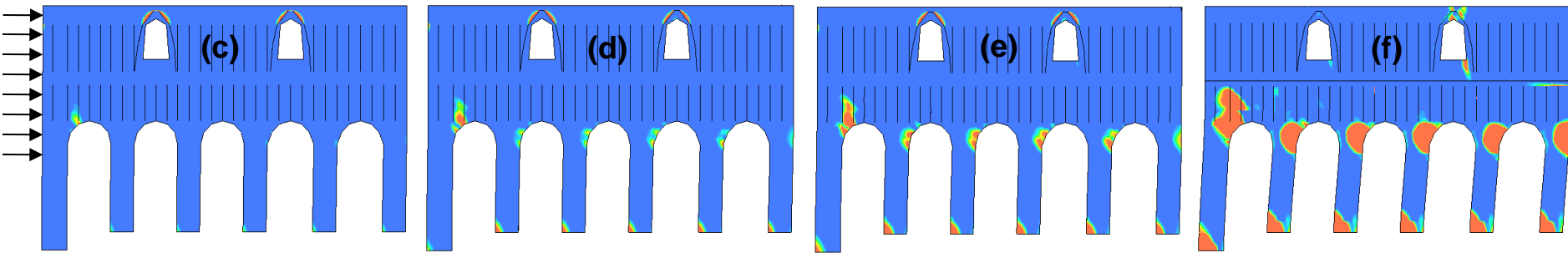
LONGITUDINAL RESPONSE - NAVE

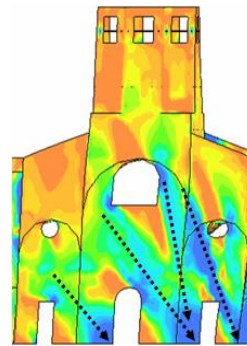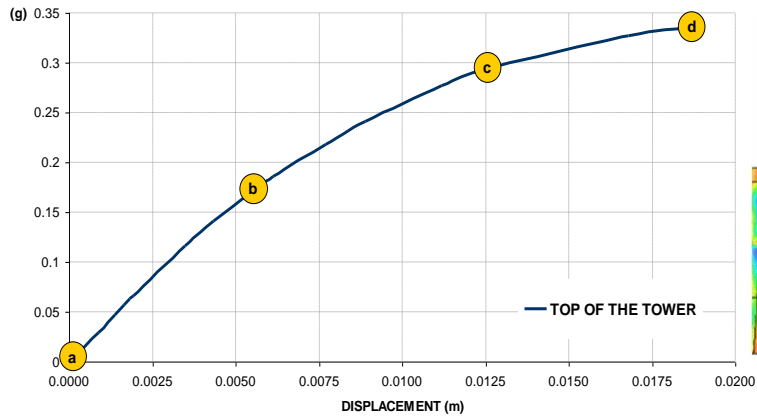**The failure mode, following the application of growing horizontal loads, is determined by the overturning of the pillars**

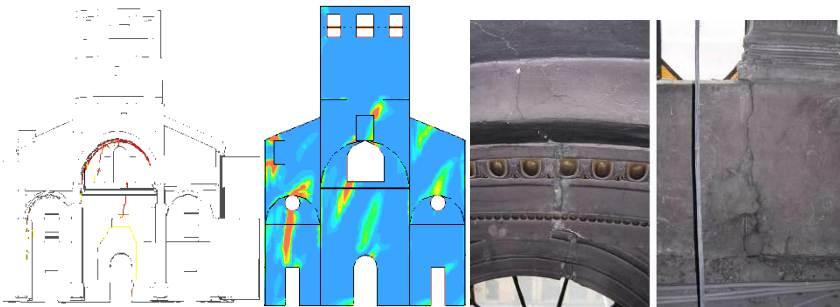**Damage identification – model prediction experimental verification**

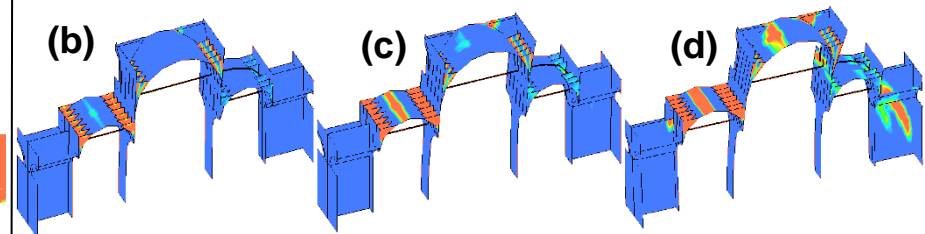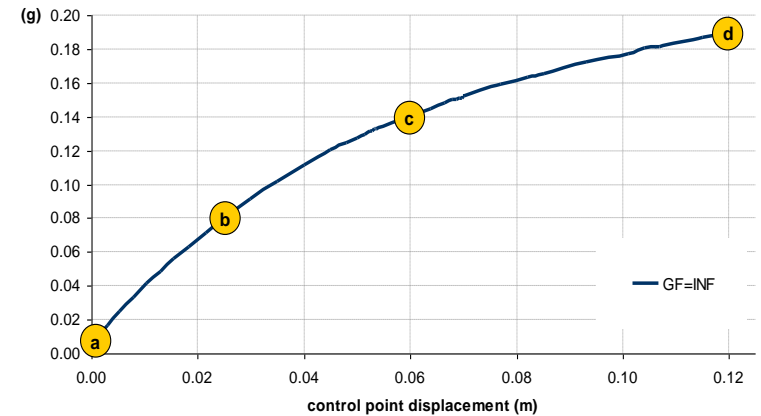# Model FT

(b) (c) (d)

FACADE IN PLANE

Model NT

(b) (c) (d)

**TRANSVERSE RESPONSE - NAVE**

Damage identification

Damage identification
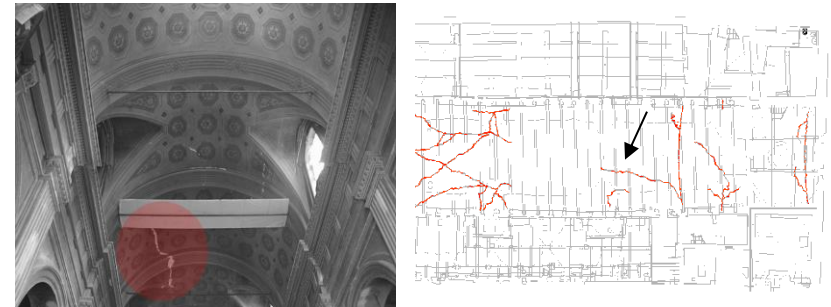
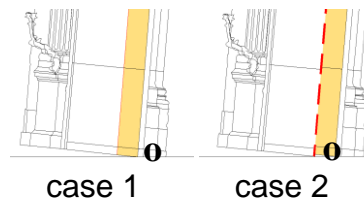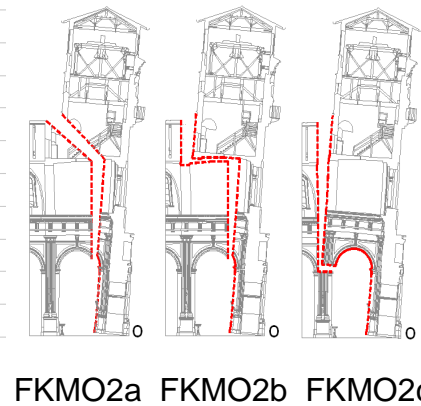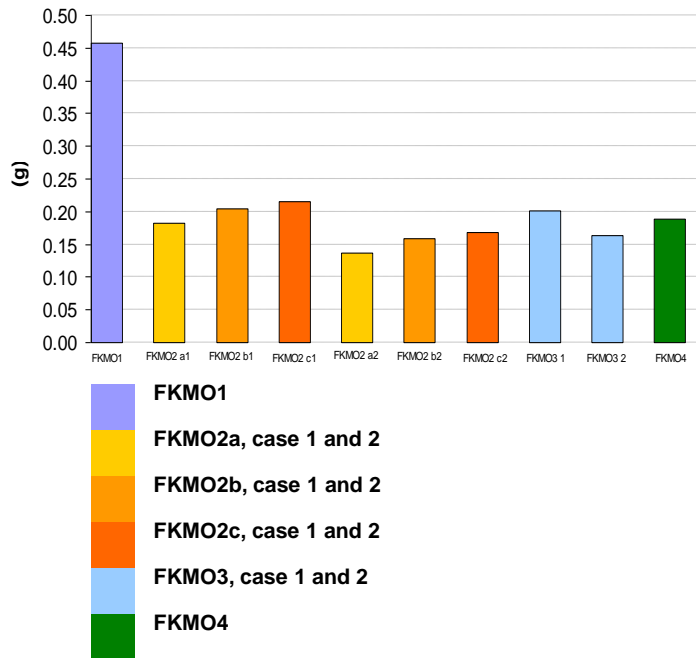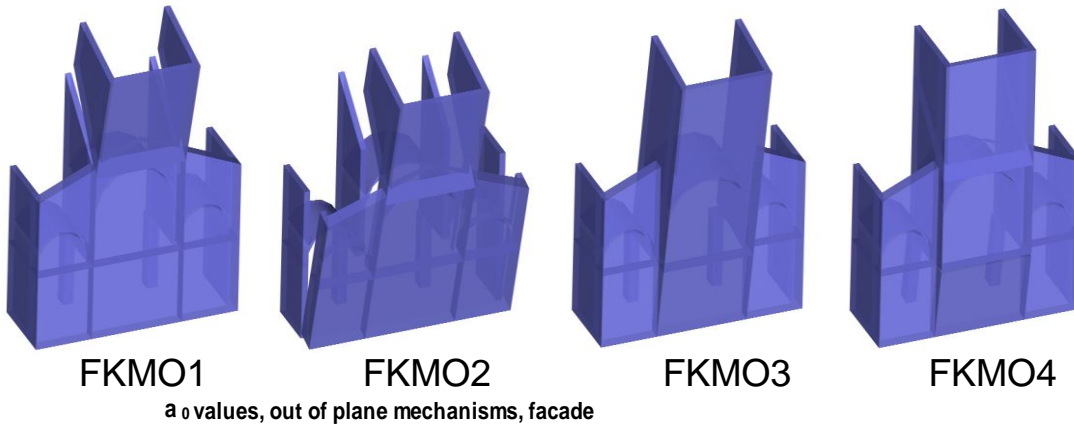## Structural modelling: Limit analysis

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 ($\alpha_0 = c$) 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.
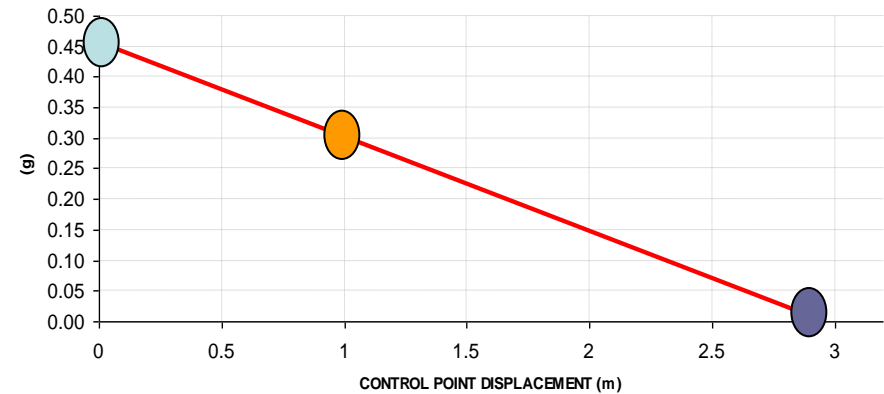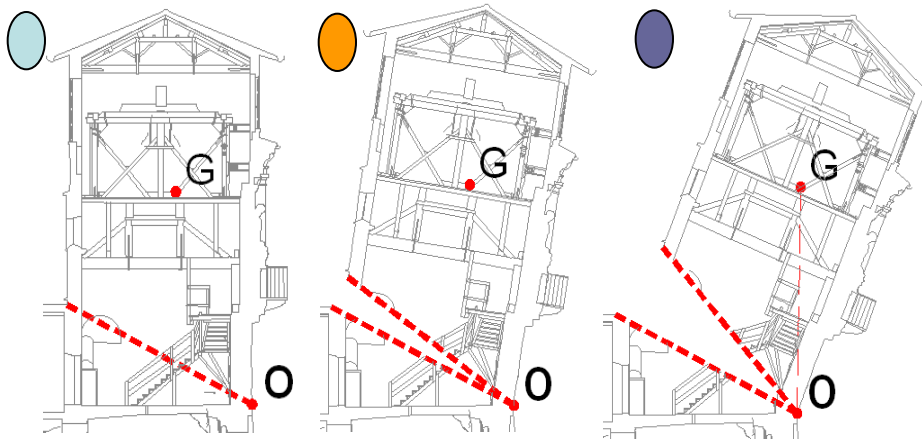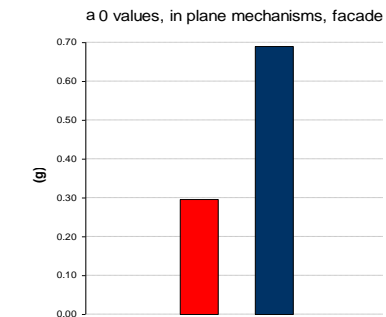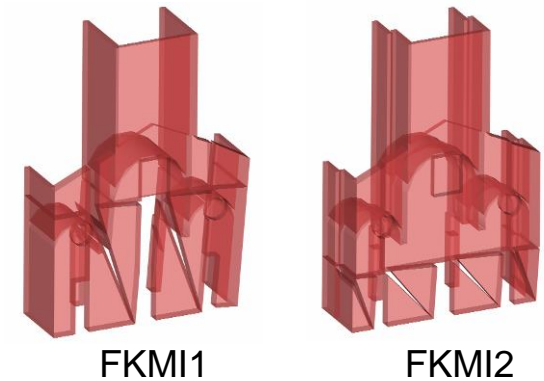

$$\alpha_0 \left[\sum_{i=1}^{n} P_i \delta_{x,i} + \sum_{j=n+1}^{n+m} P_j \delta_{x,j} \right] - \sum_{i=1}^{n} P_i \delta_{y,i} - \sum_{h=1}^{o} F_h \delta_h = L_{fi}$$
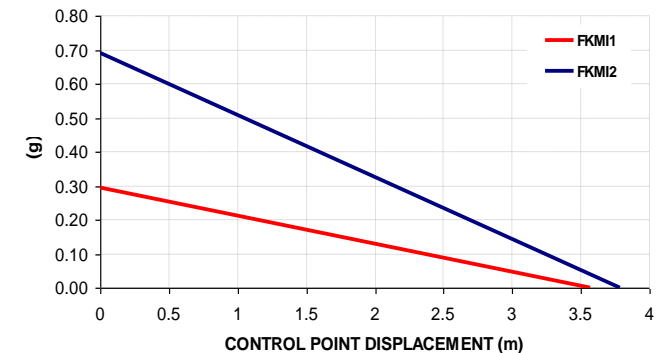
Out of plane façade failure

FKMO1   FKMO2   FKMO3   FKMO4

a₀ values, out of plane mechanisms, facade

- **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

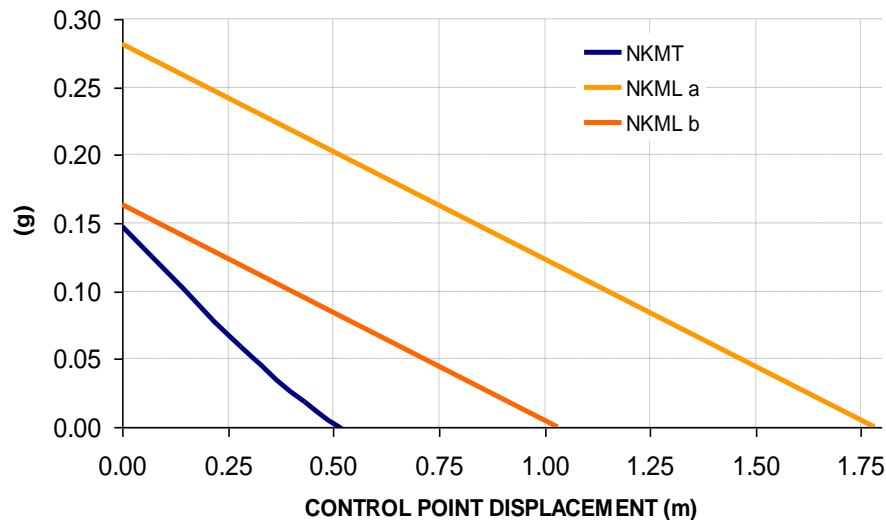a 0 values, in plane mechanisms, facade

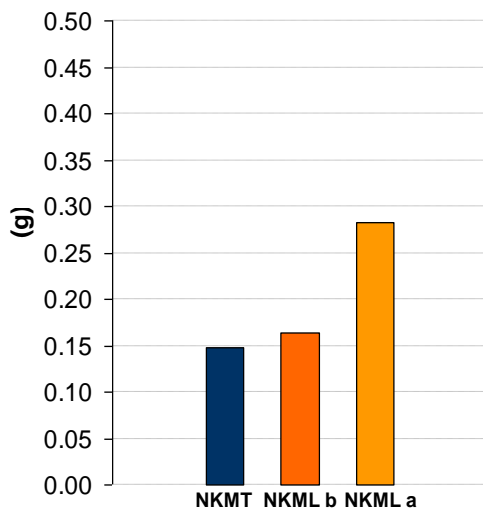The mechanisms' centre of masses was considered as reference point.

Nave longitudinal and transversal failure mechanisms
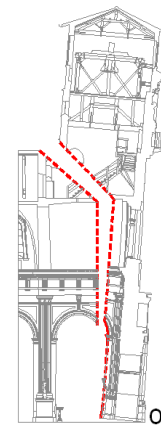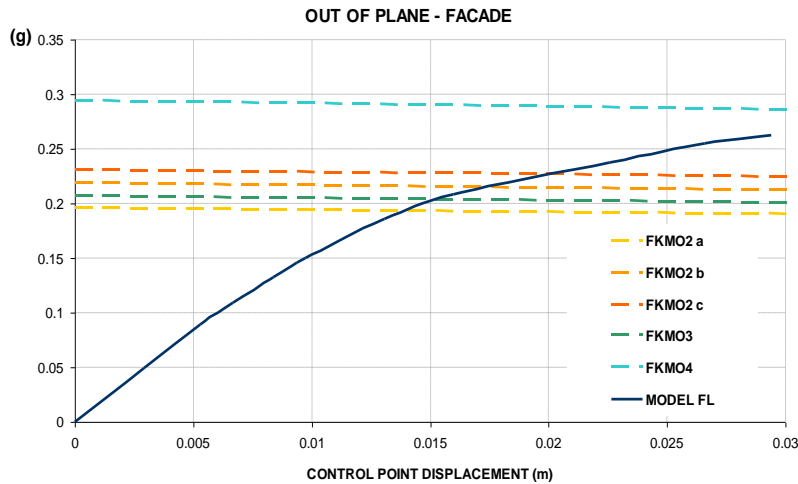
NKML

NKMT
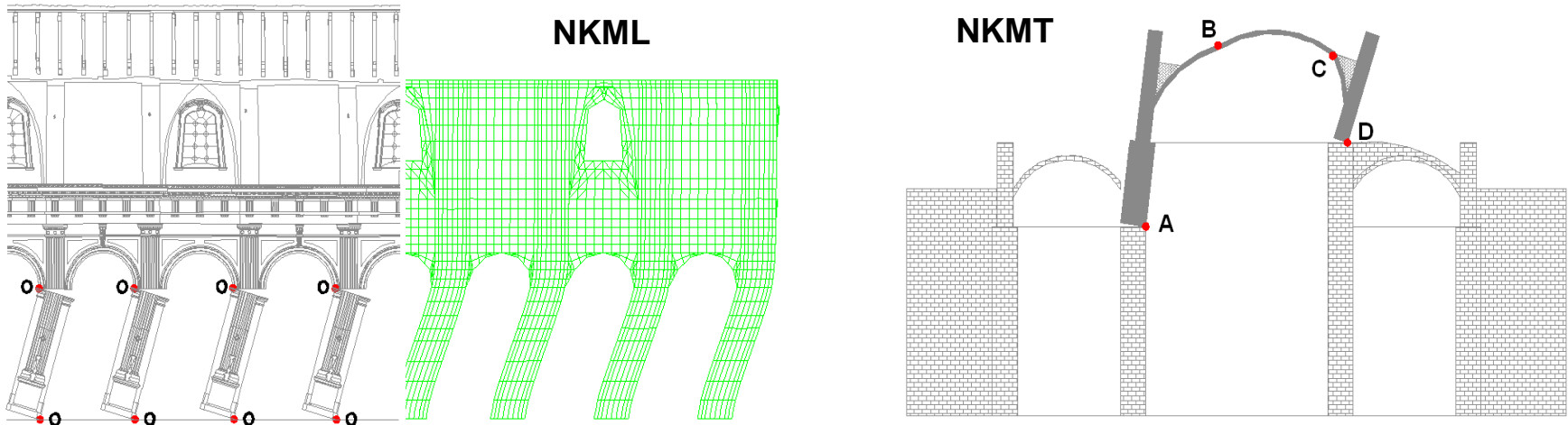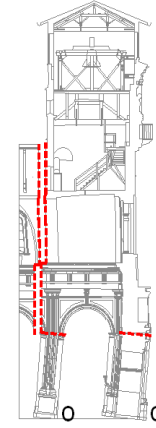

a 0 values, nave mechanisms

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

Both the limit analysis **kinematic approach** and the **FE modelling** strategy provided fairly **satisfactory results in terms of comparative evaluation**.
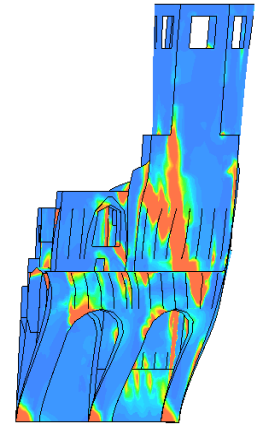
From the comparison between models, it is evident that, according to the numerical outcomes, the closer failure mode implemented with the limit analysis corresponds to model **FKMI1**

OUT OF PLANE - FACADE

FKMO2 a
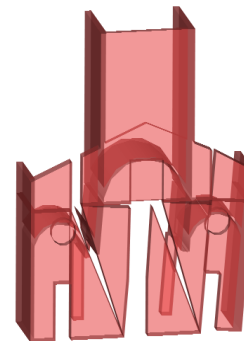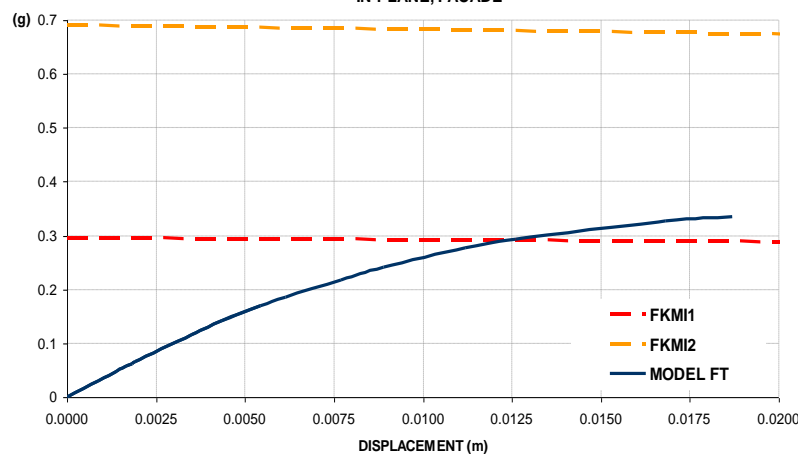FKMO2 b
FKMO2 c
FKMO3
FKMO4
MODEL FL

IN-PLANE, FACADE

FKMI1
FKMI2
MODEL FT

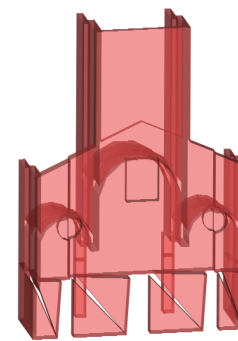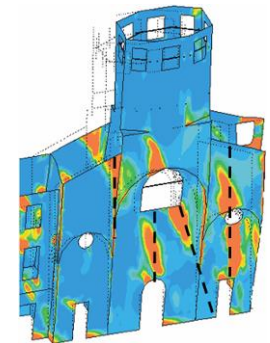FKMO2   FKMO4   FL MODEL

FKMI1   FKMI2   FT MODEL

General interventions were carried out by using traditional techniques, devoted to give back to the structure its integrity and continuity, to reestablish connections between structural elements and to substitute damaged structural elements. The typology of these interventions are:

- ❑ the traditional "scuci-cuci" (substitution of damaged elements with new ones);
- ❑ mortar repointing and grout injections, especially in the cracked vaulted system.

Other strengthening interventions were carried out at the roof level:

- ❑ restoration of the wooden truss structure;
- ❑ reestablishment of the connections between wooden structural elements.
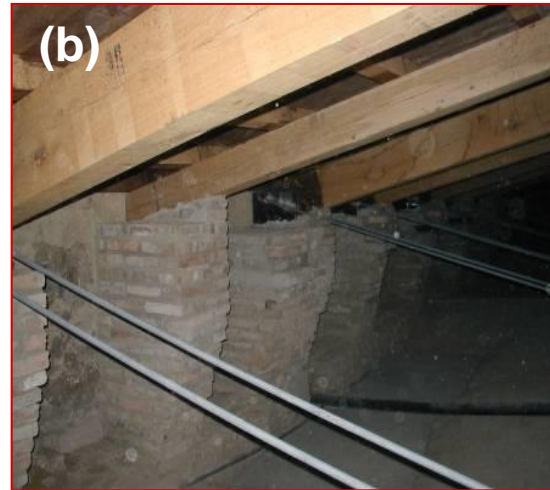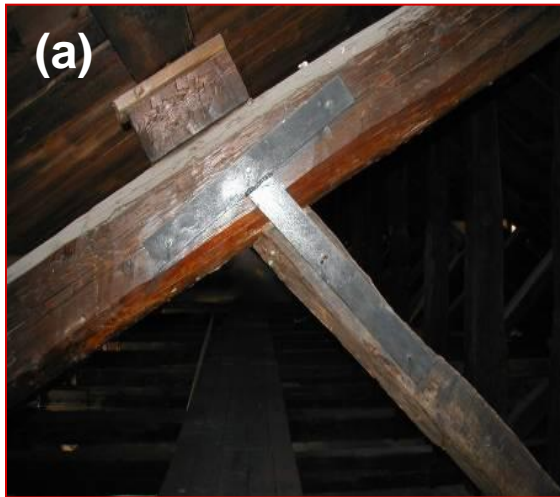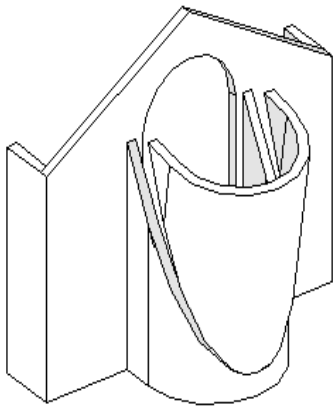
Roof wooden trusses: (a) metallic "T" shaped connection elements; (b) substitution of the old wooden trusses of the left nave with new elements, with stainless steel tie beams. Roof level: (c) a "scuci-cuci" intervention.

Specific strengthening interventions are conceived to improve the seismic behavior of the portions of the complex manifesting higher vulnerability rather than to modify the overall structural response of the structure:

- ❑ **walls of the lateral apses;**
- ❑ **lantern at the top of the dome;**
- ❑ **vaults of the central apse;**
- ❑ **vaults of the main nave;**
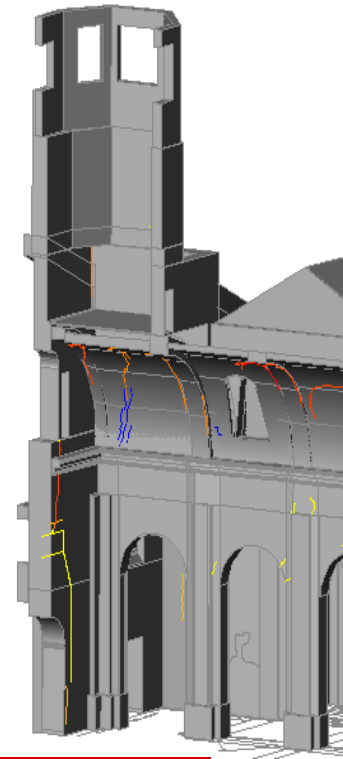- ❑ **façade area.**

Lateral apses

Lantern

Façade

Vaults of the main nave

Central apses

INTERNATIONAL SYMPOSIUM - JANUARY 13 – 14, 2016
SEISMIC RETROFIT OF UNREINFORCED MASONRY HERITAGE CHURCHES IN THE PHILIPPINES
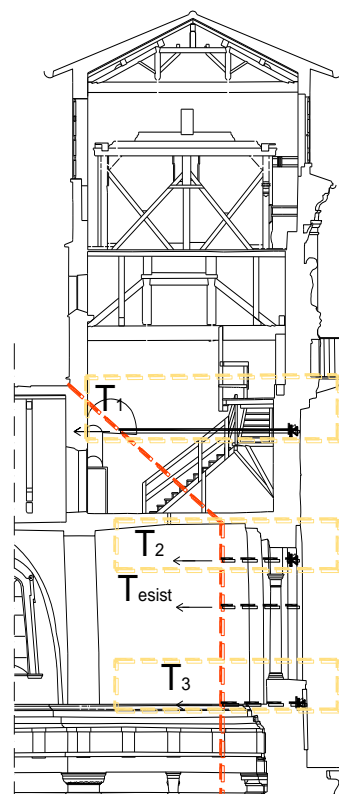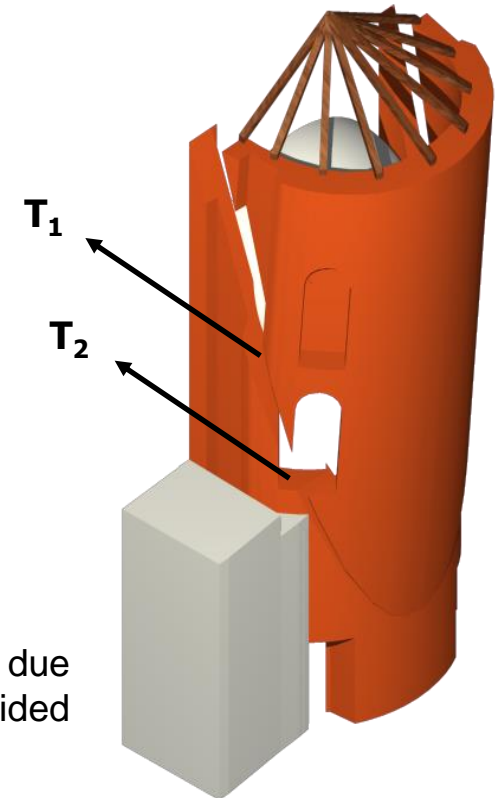
52
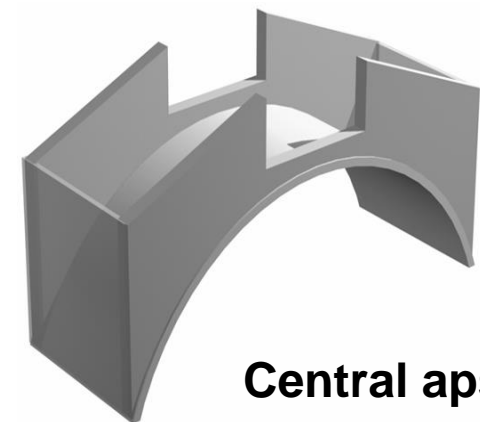
Effectiveness evaluation

The seismic verification according to the national seismic standard (OPCM 3431/2005) is obtained for almost all the considered kinematic mechanism, even before the strengthening interventions: to appreciate the extra stabilizing effect of the structural interventions, the kinematic mechanisms implemented were re-evaluated considering the contribution of the newly installed seismic devices (stainless steel tie beams and confinement rings).
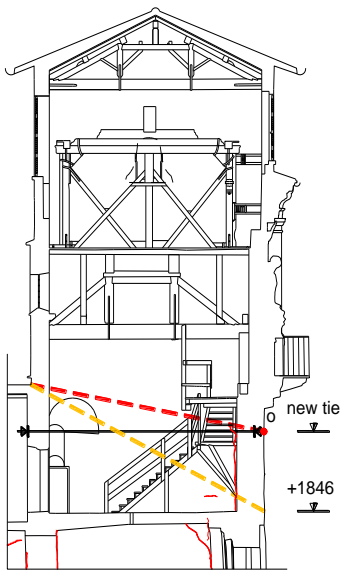
Mechanism's modification

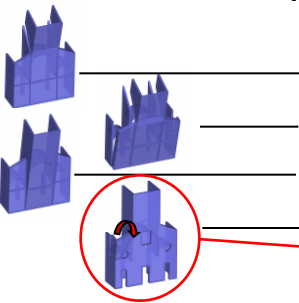Stabilizing effect due to the newly provided seismic devices

INTERNATIONAL SYMPOSIUM - JANUARY 13 – 14, 2016
SEISMIC RETROFIT OF UNREINFORCED MASONRY HERITAGE CHURCHES IN THE PHILIPPINES

53

Results obtained indicate a considerable α_0 value increase; the stainless steel tie beams positioning is particularly effective for the overturning mechanism of the façade corner part, presenting high seismic vulnerability in its previous state (not verified according to the national seismic standards).

Façade

Out-of-plane damage mechanisms	Collapse coefficient (g)		
	Pre strengthening intervention	Post strengthening intervention	increment
	0,457	0,670	+47%
	0,143	0,190	+33%
	0,151	0,191	+26%
	0,115	0,299	+160%

Horizontal multiplier α_0
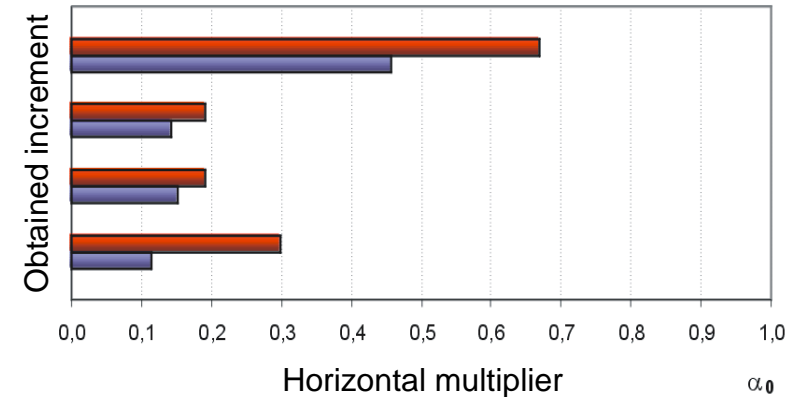
Apses

Out-of-plane damage mechanisms	Collapse coefficient (g)		
	Pre strengthening intervention	Post strengthening intervention	increment
North side apse	0,118	0,196	+66%
South side apse	0,102	0,157	+54%
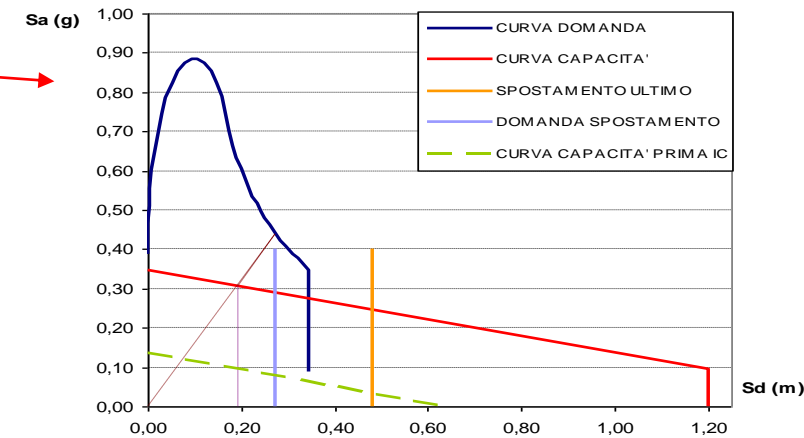
capacity curves of the mechanism in un-strengthened and strengthened state
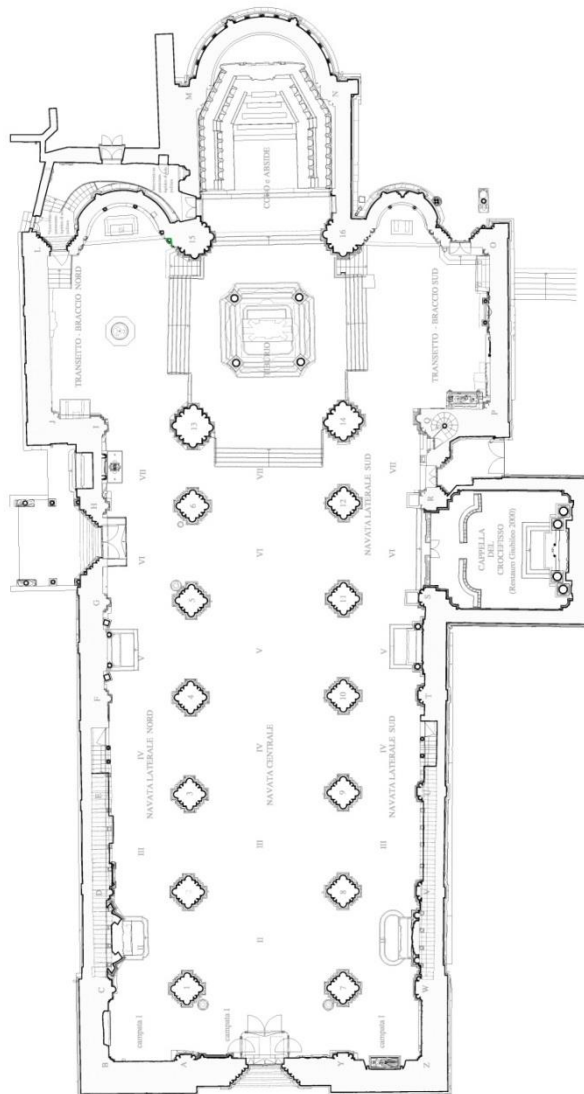
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  - Frari Bell Tower in Venice

  - Aggregate building in L'Aquila
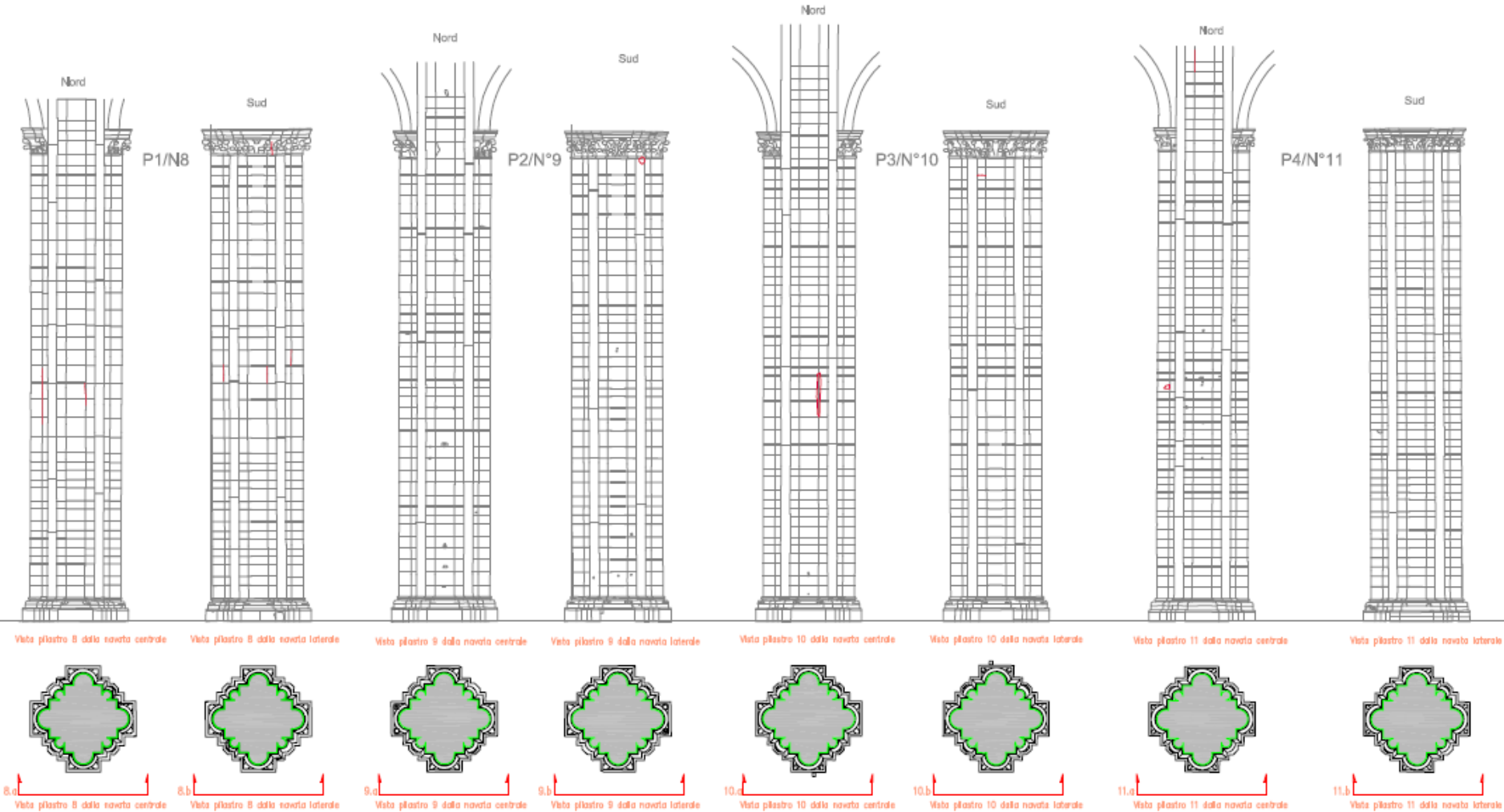
SAN VIGILIO CATHEDRAL - TRENTO

SAN VIGILIO CATHEDRAL - TRENTO
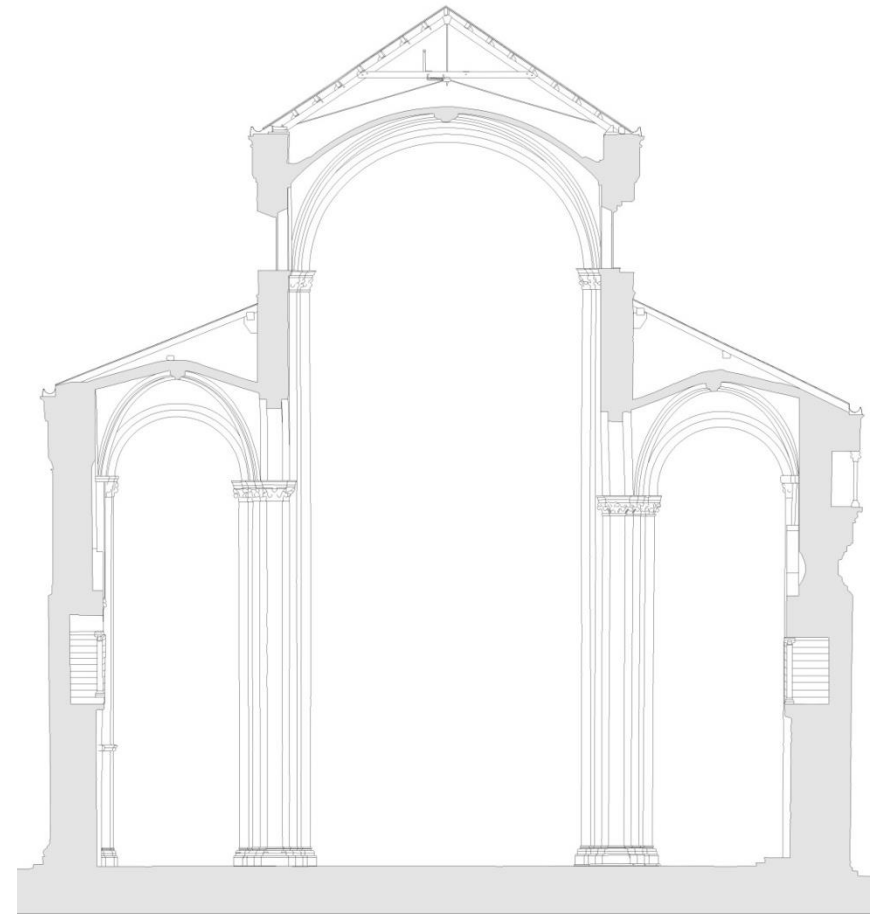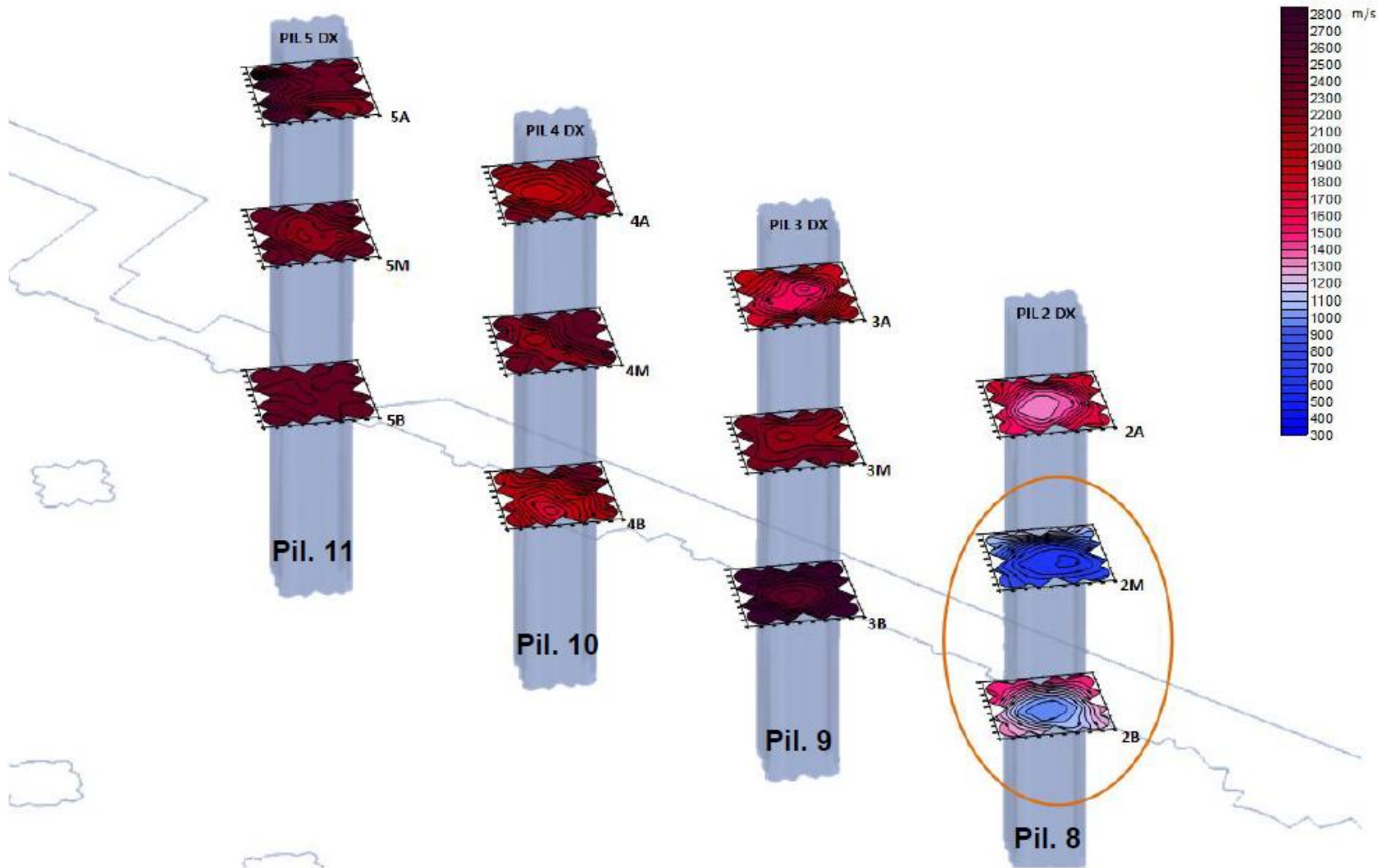
SAN VIGILIO CATHEDRAL - TRENTO
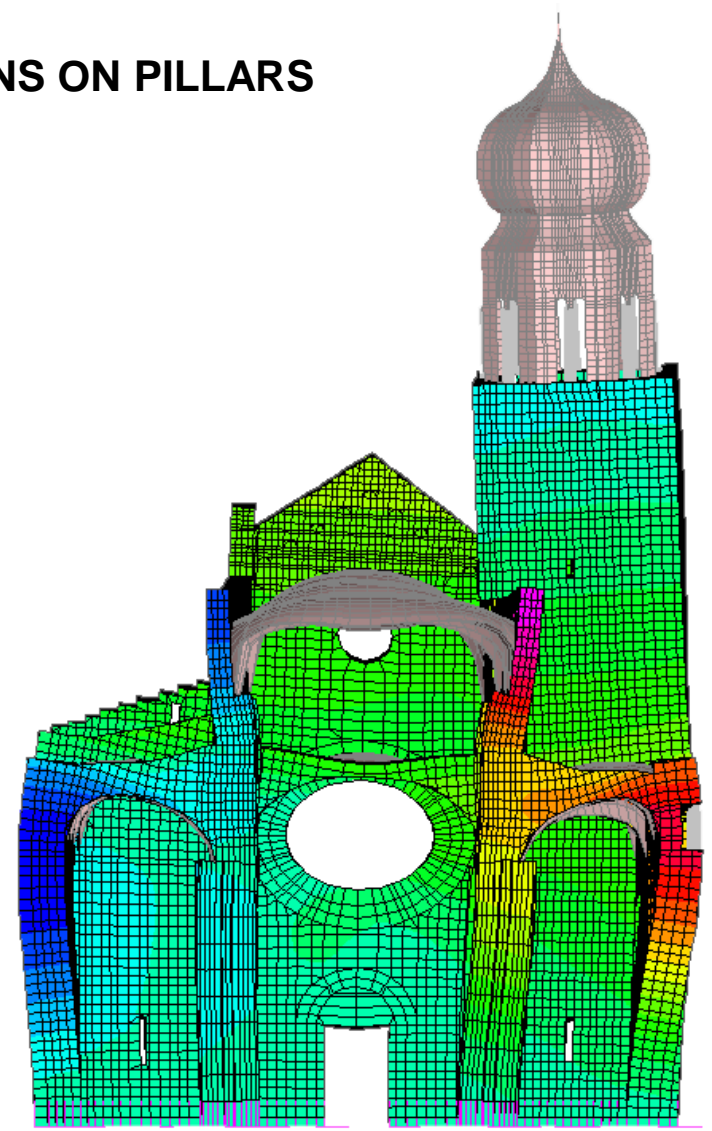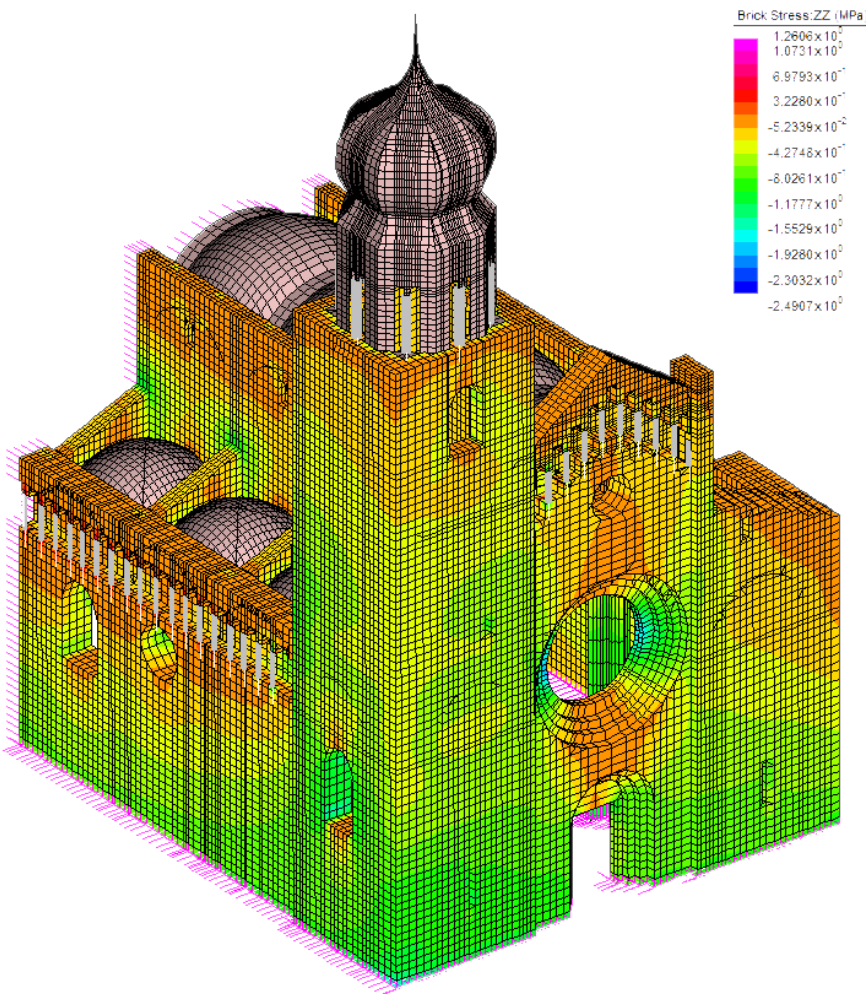
PILLARS CRACK SURVEY

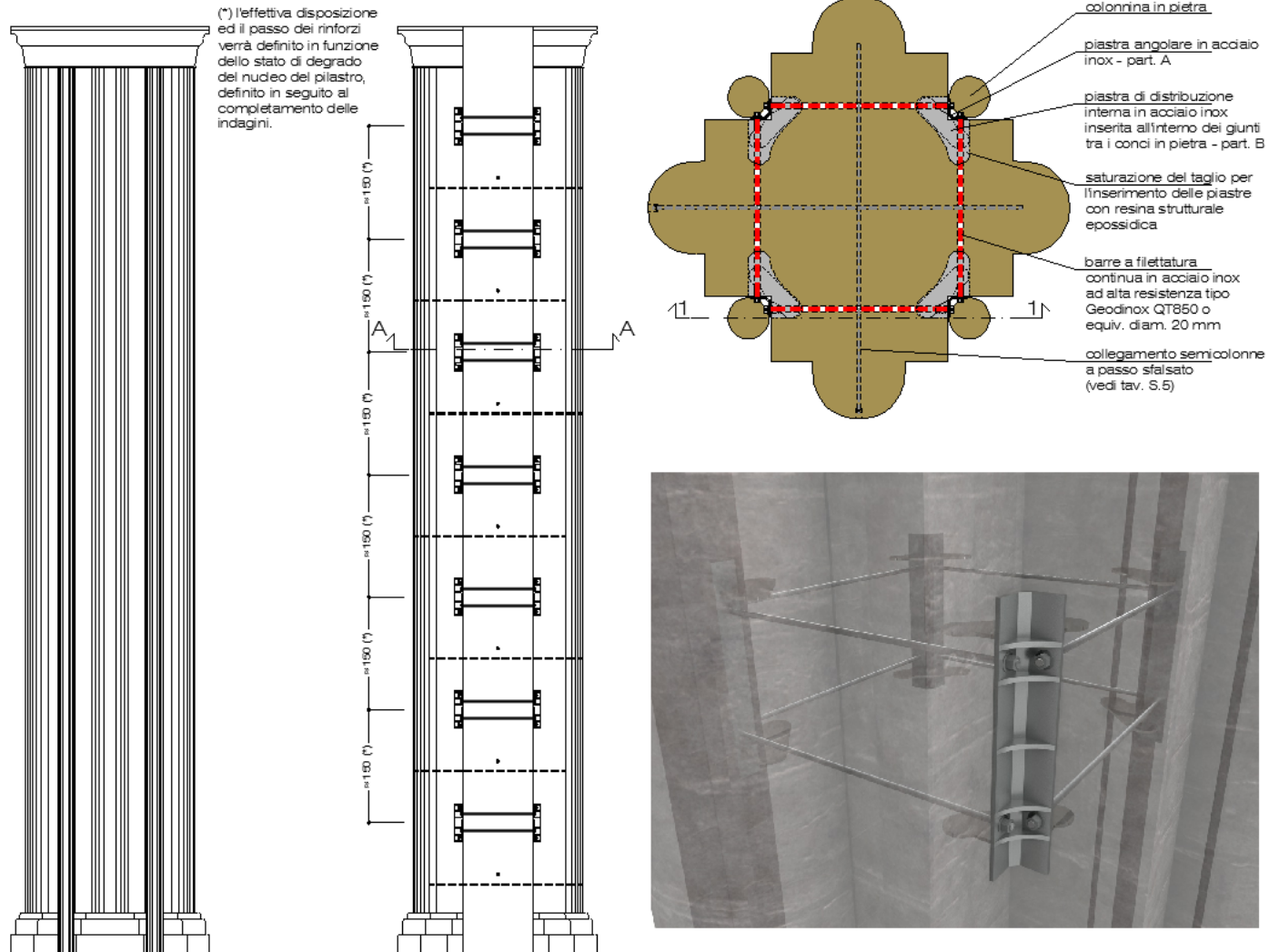# SAN VIGILIO CATHEDRAL - TRENTO

PILLARS SONIC TESTS

SAN VIGILIO CATHEDRAL - TRENTO

FEM MODEL FOR THE DESIGN OF THE INTERVENTIONS ON PILLARS

Brick Stress:ZZ (MPa)

SAN VIGILIO CATHEDRAL - TRENTO

PILLARS CONFINING SYSTEM

(*) l'effettiva disposizione ed il passo dei rinforzi verrà definito in funzione dello stato di degrado del nucleo del pilastro, definito in seguito al completamento delle indagini.

colonnina in pietra

piastra angolare in acciaio inox – part. A

piastra di distribuzione interna in acciaio inox inserita all'interno dei giunti tra i conci in pietra – part. B

saturazione del taglio per l'inserimento delle piastre con resina strutturale epossidica

barre a filettatura continua in acciaio inox ad alta resistenza tipo Geodinox QT850 o equiv. diam. 20 mm

collegamento semicolonne a passo sfalsato (vedi tav. S.5)

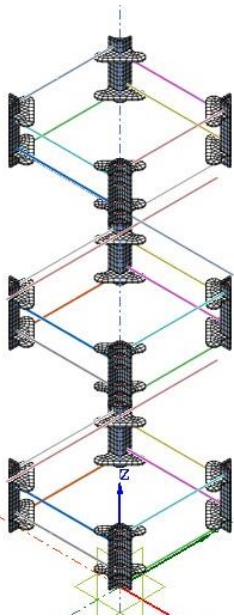**Claudio Modena**

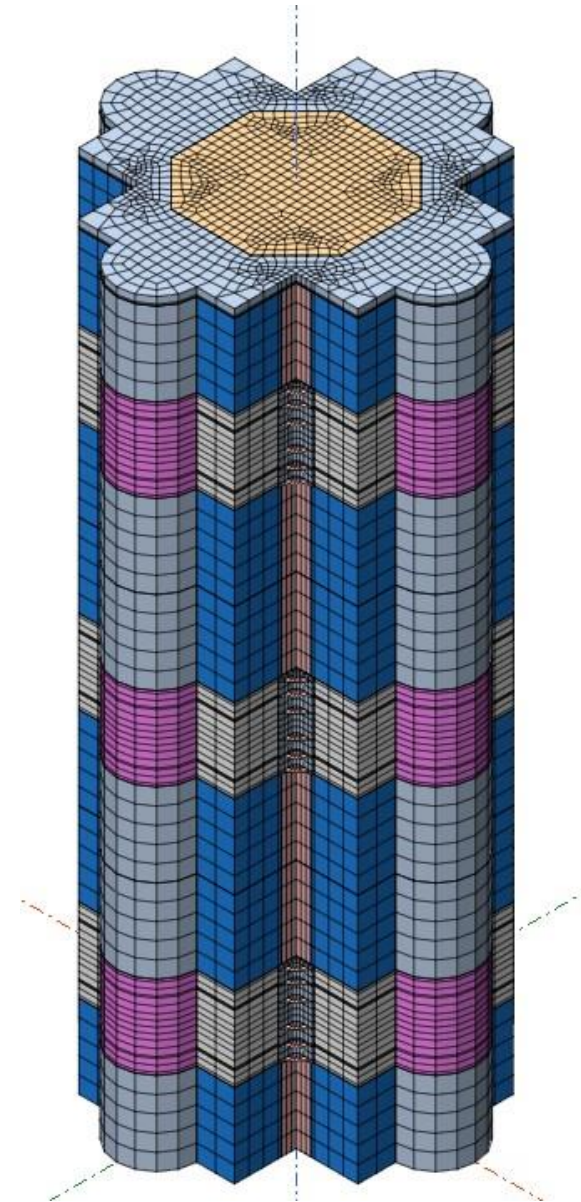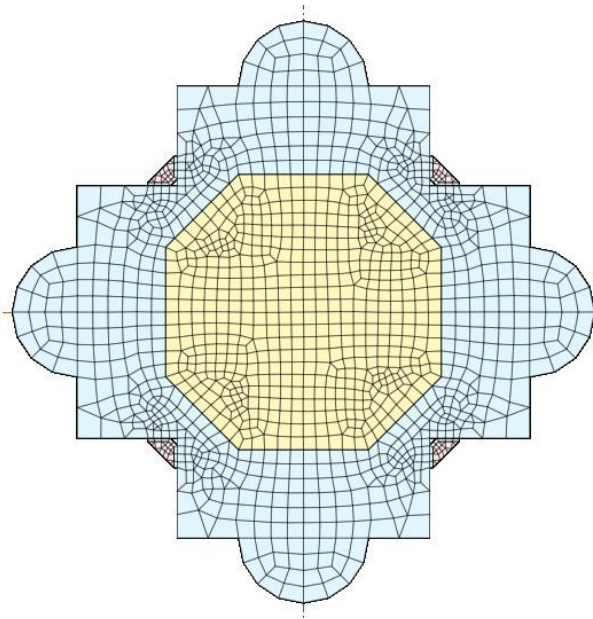**Analysis Methods for Unreinforced Heritage Masonry**

SAN VIGILIO CATHEDRAL - TRENTO

PILLARS FEM MODEL
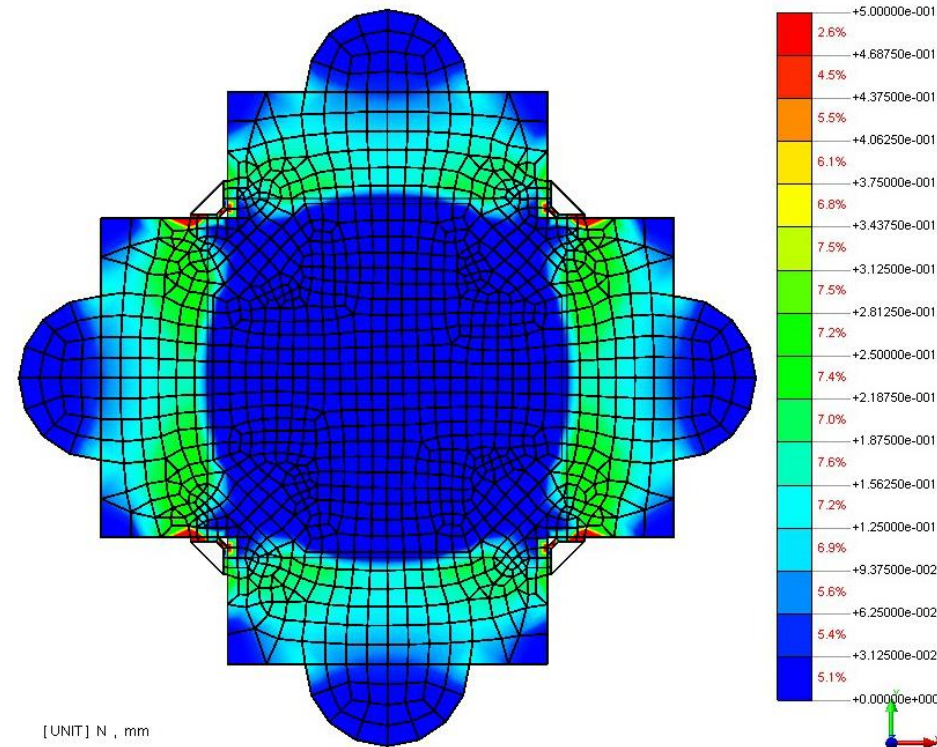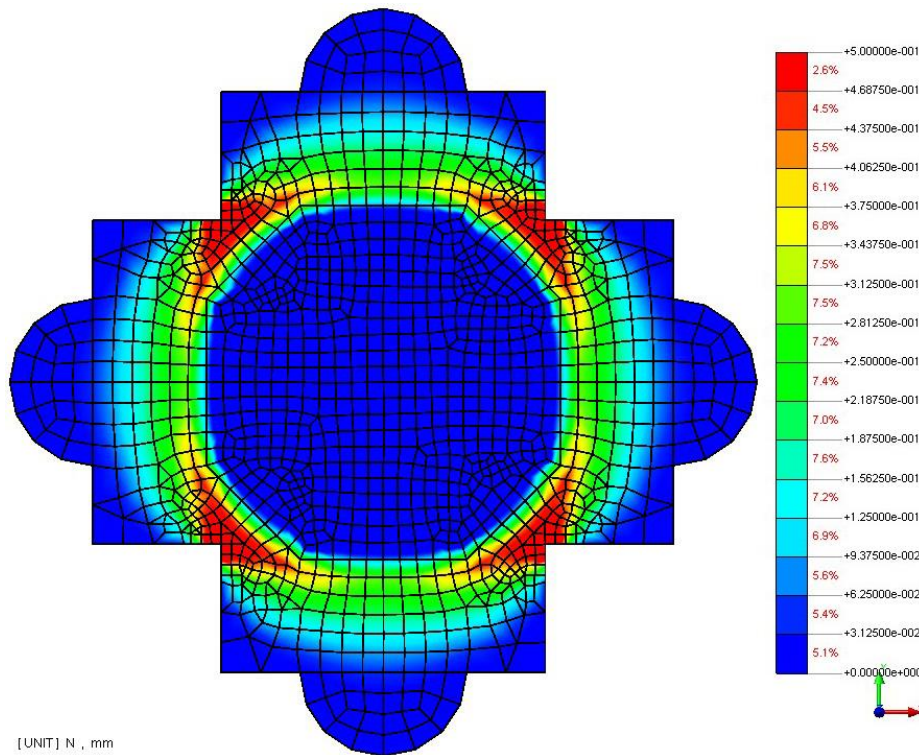
CONFINING SYSTEM

SECTION OF THE FEM MODEL

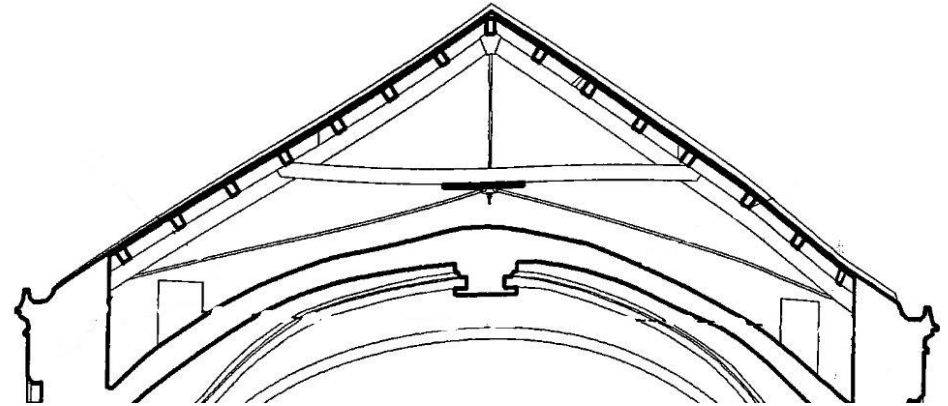# SAN VIGILIO CATHEDRAL - TRENTO

PILLARS FEM MODEL

BEFORE INTERVENTION

AFTER INTERVENTION

SAN VIGILIO CATHEDRAL - TRENTO

FEM MODEL FOR GLOBAL SEISMIC IMPROVEMENT INTERVENTIONS DESIGN
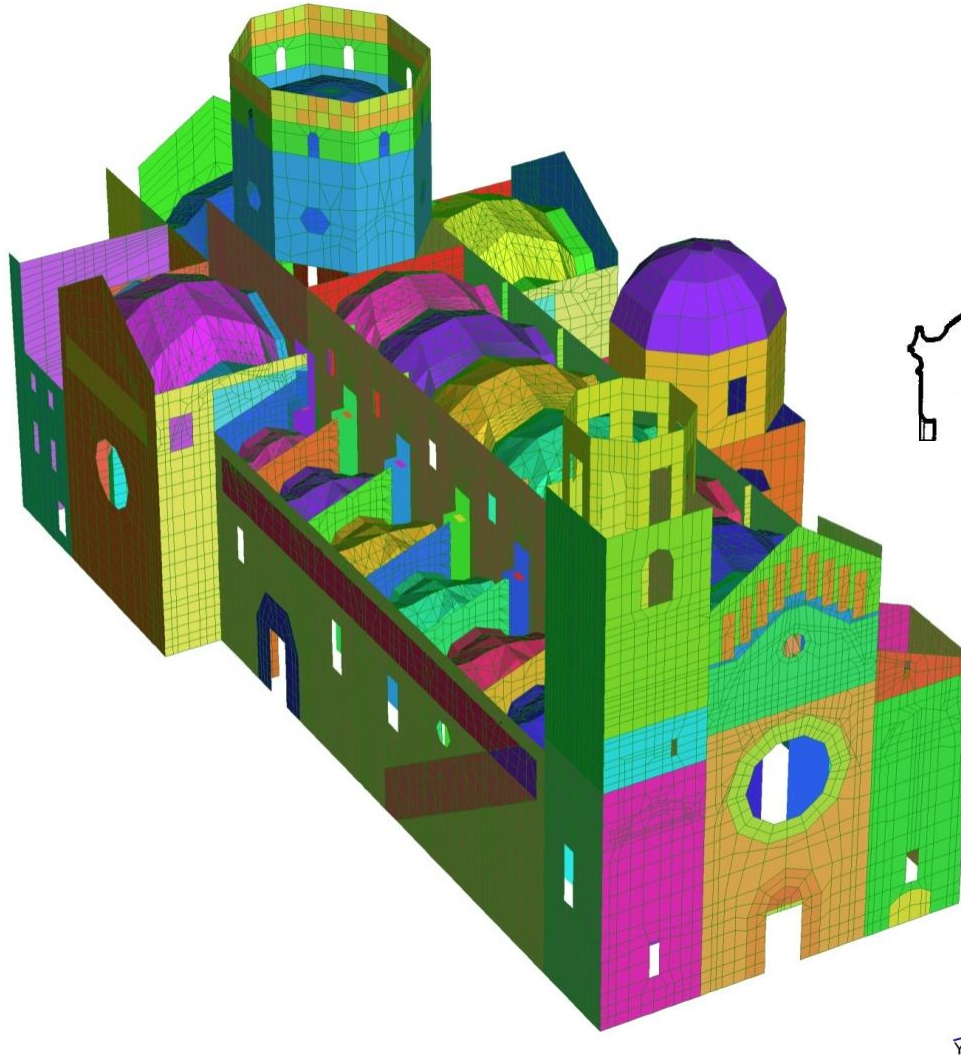
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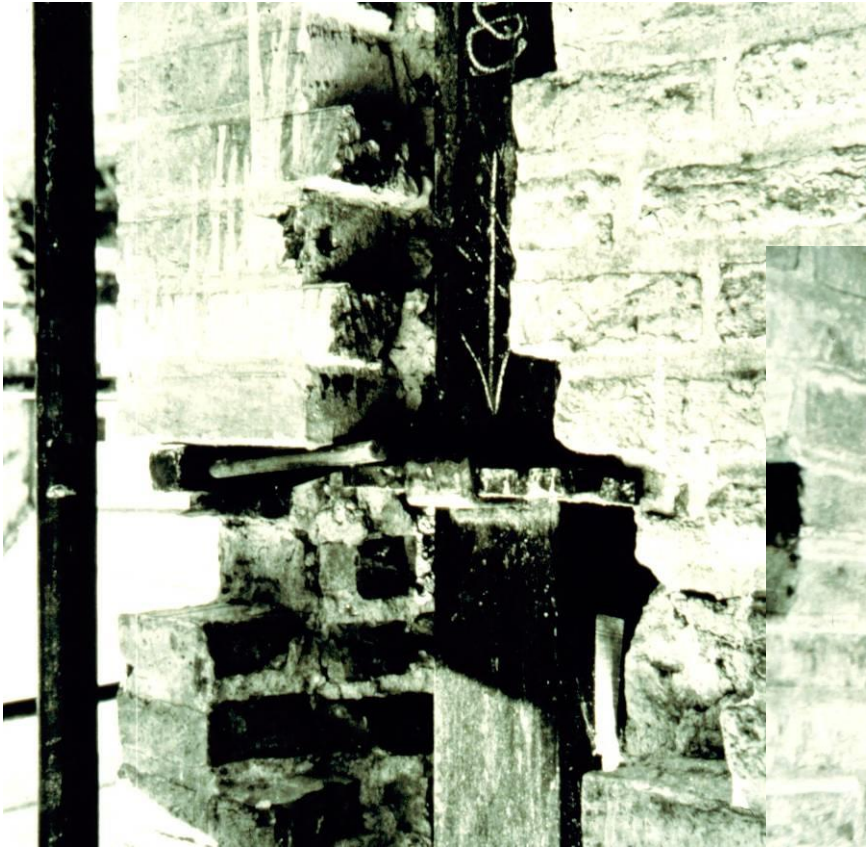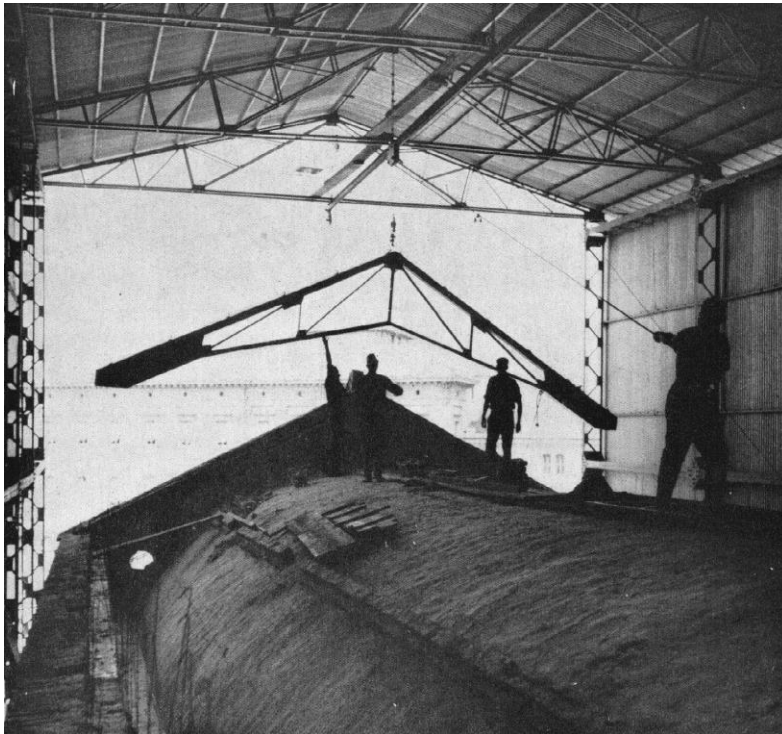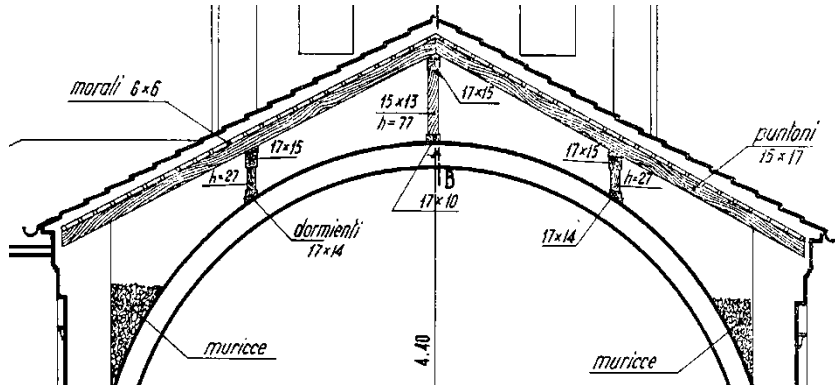## The Scrovegni chapel, Padova

THE "GIOTTO'S CHAPEL" IN PADOVA

THE "GIOTTO'S CHAPEL" IN PADOVA

THE "GIOTTO'S CHAPEL" IN PADOVA

CRACK PATTERN SURVEY

Sezione longitudinale, parete SUD
Quadro fessurativo al, 13/02/98
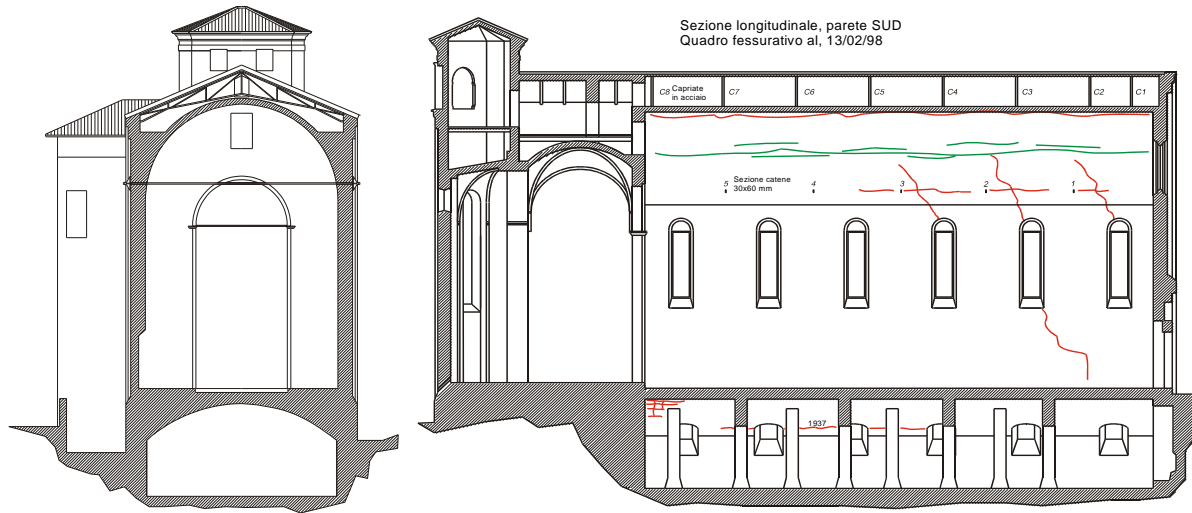
DYNAMIC TESTS ON STEEL TIES
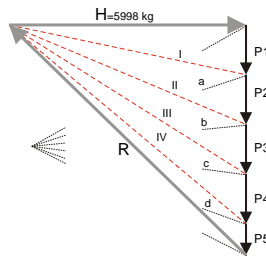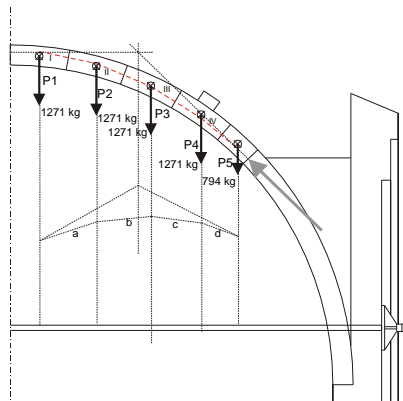
THE "GIOTTO'S CHAPEL" IN PADOVA
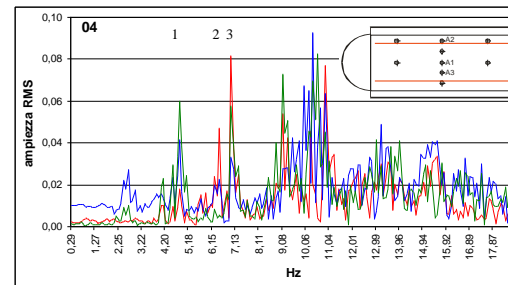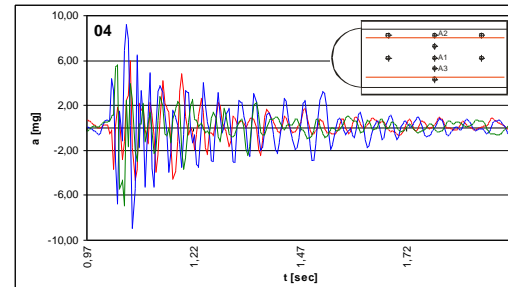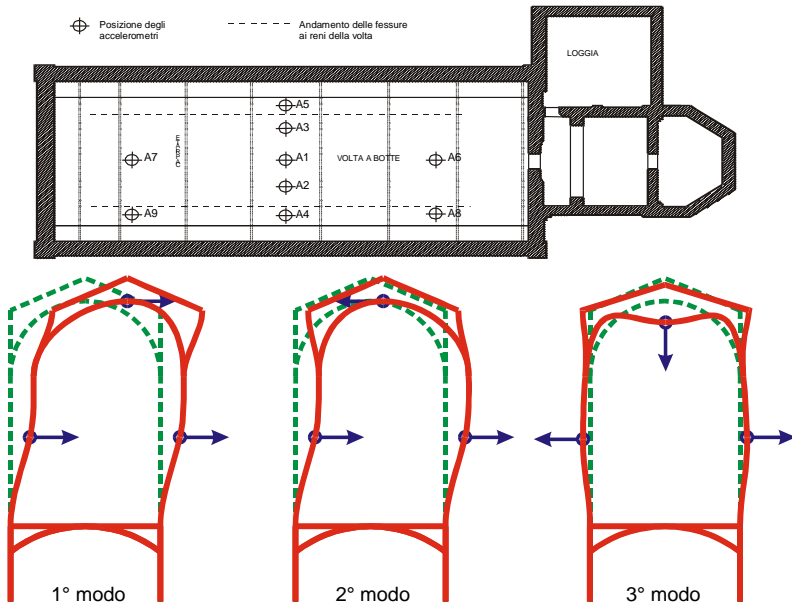
DYNAMIC IDENTIFICATION TESTS
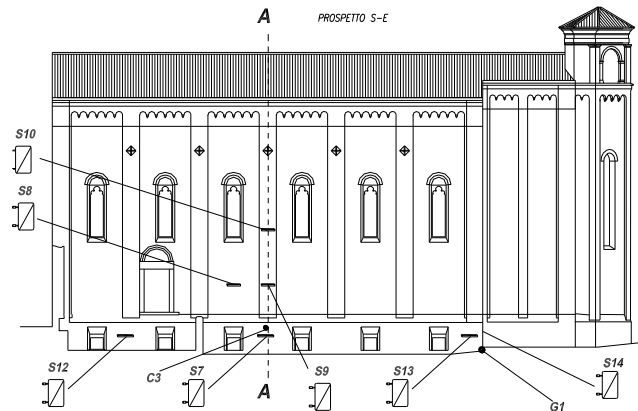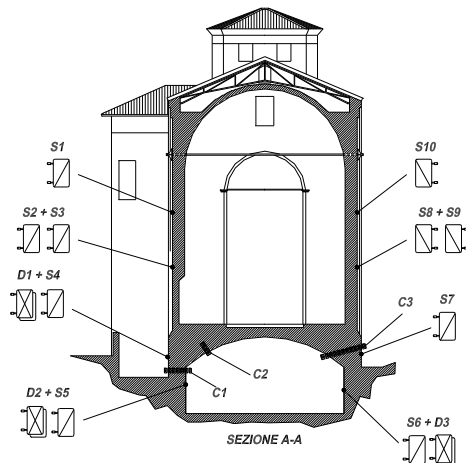
FLAT JACK TESTS

CORING ON MASONRY WALLS

GEOTECHNICAL CORING

THE "GIOTTO'S CHAPEL" IN PADOVA
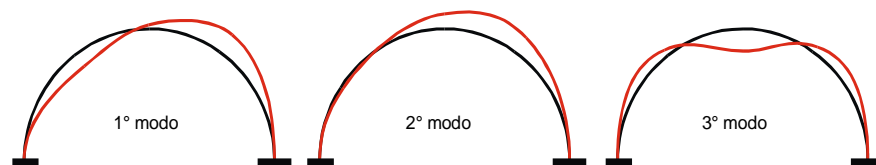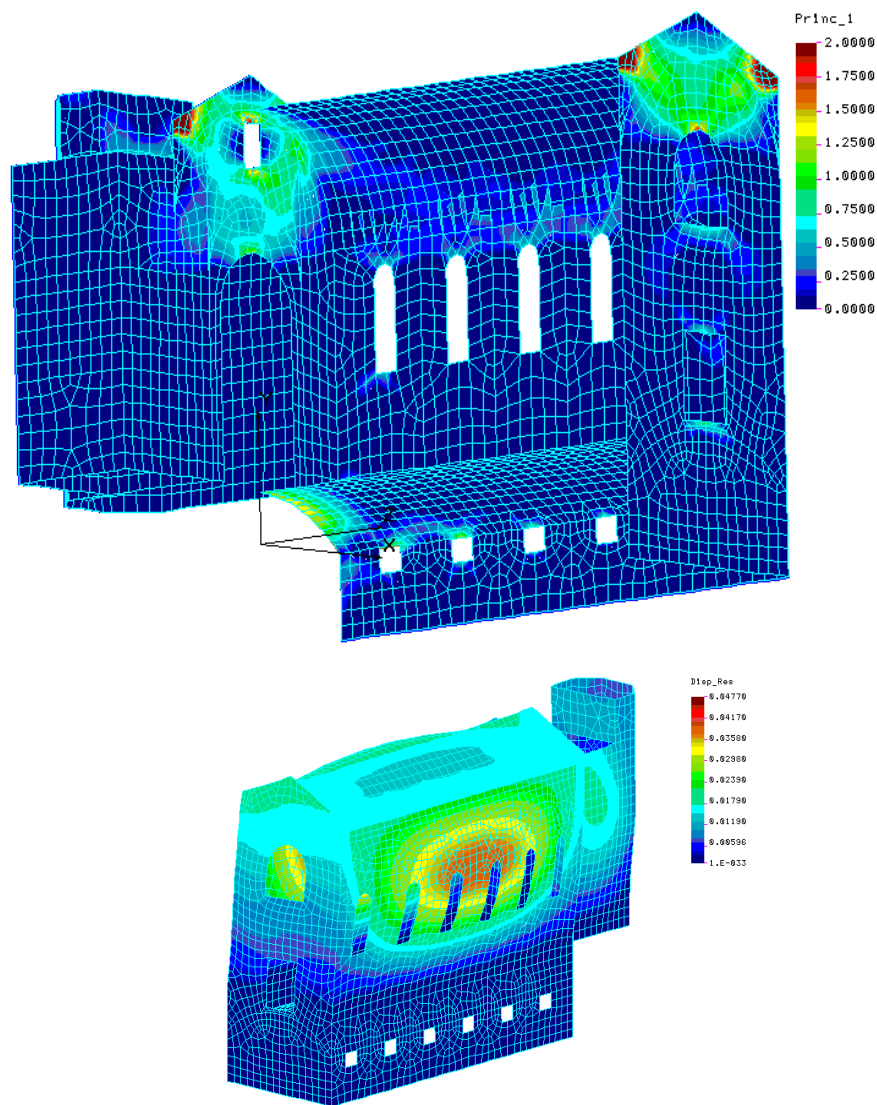
	Model (tension plane)	3D model	Complete model	Experimental
1st mode	5.13	6.32	4.99	4.9
2nd mode	8.53	8.38	6.74	6.5
3rd mode	12.32	9.63	7.28	6.9
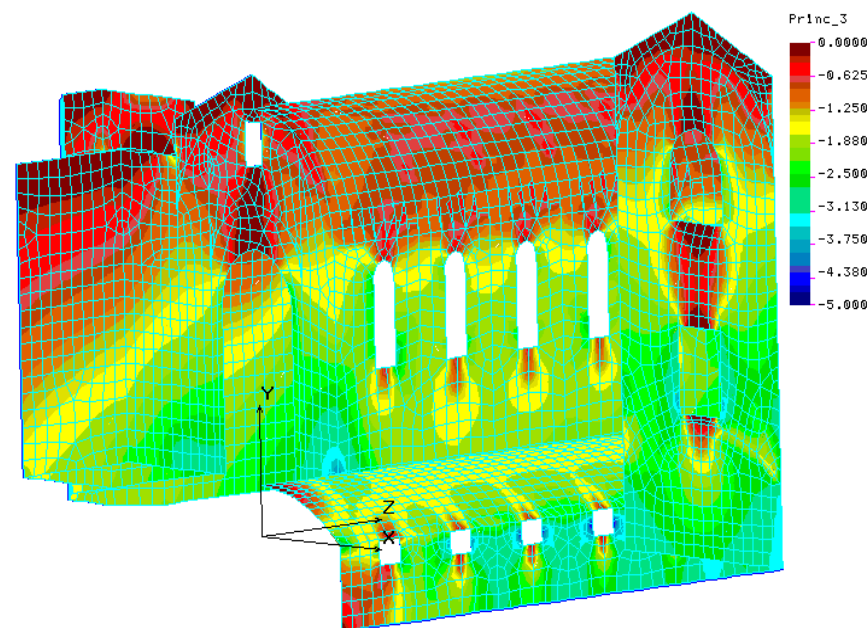
THE "GIOTTO'S CHAPEL" IN PADOVA

MONITORING SYSTEM 1997-1998 → CRACK PATTERN CONTROLS
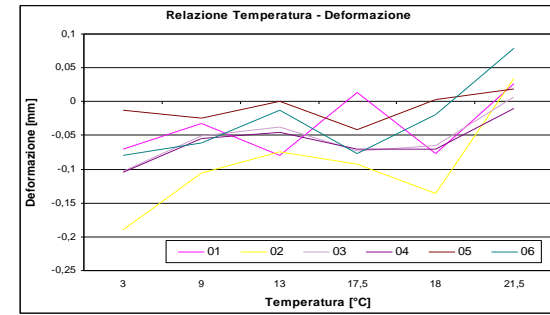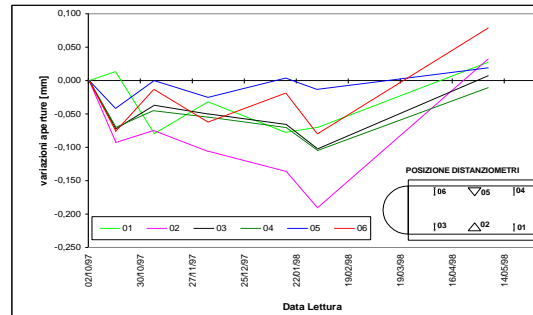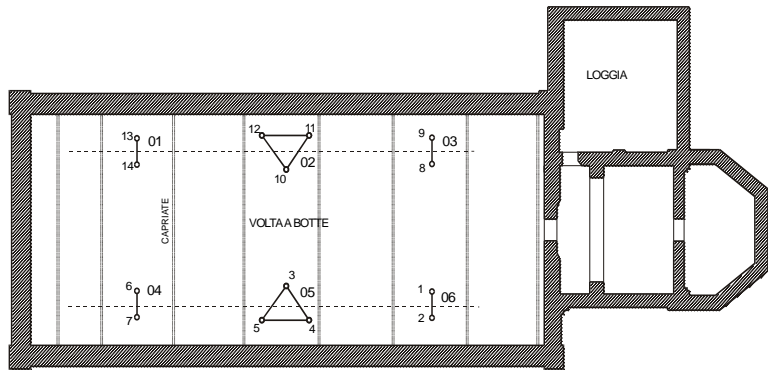
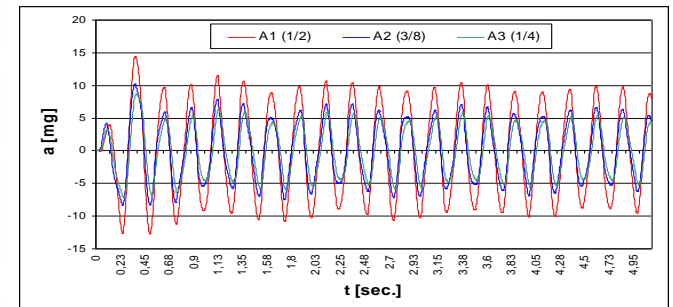## MONITORING SYSTEM MAY-JULY 2011 → CRACK PATTERN CONTROLS
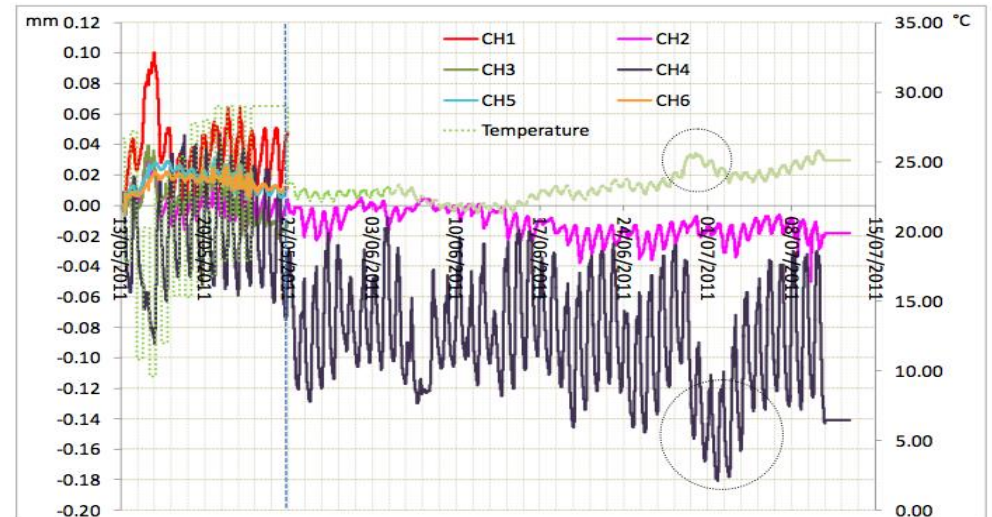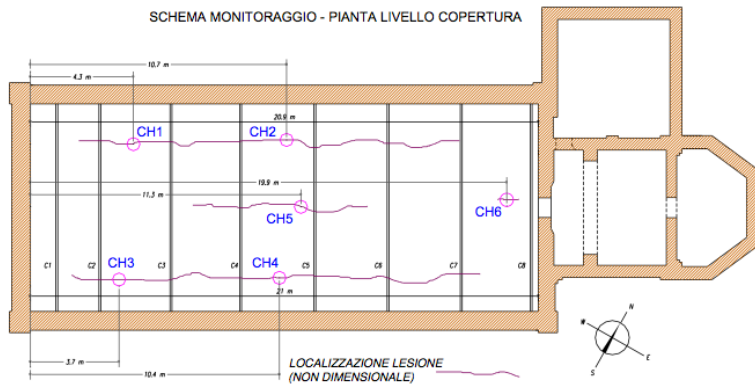
THE "GIOTTO'S CHAPEL" IN PADOVA
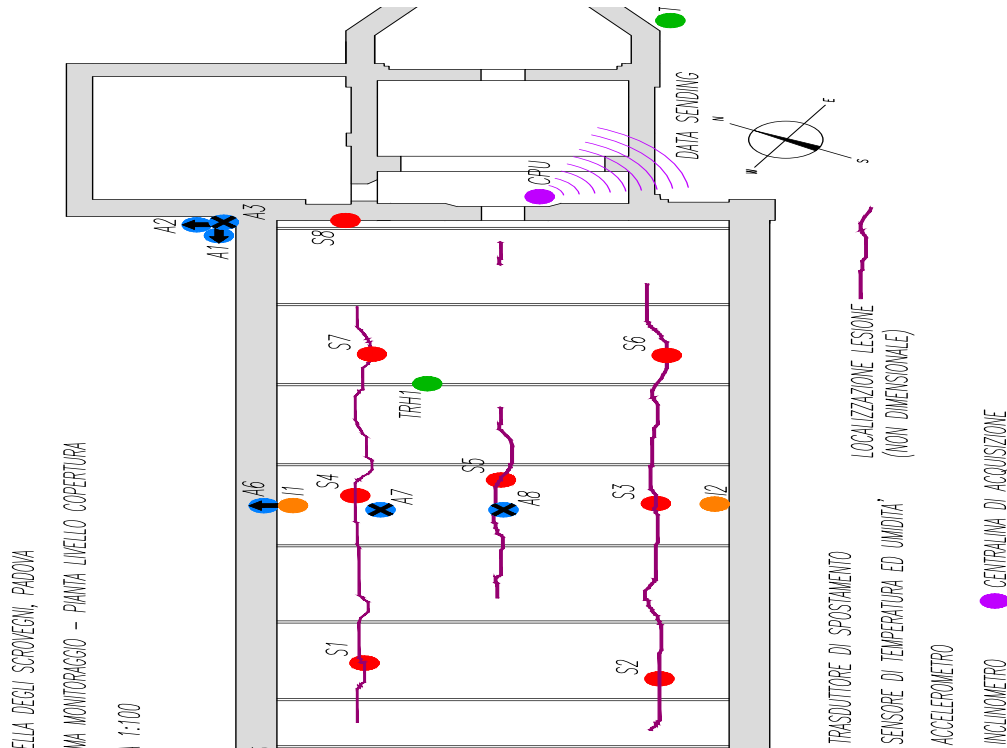
PERMANENT STRUCTURAL HEALTH MONITORING SYSTEM (INSTALLED OCT. 2013)

DYNAMIC MONITORING
8 SINGLE-AXIS ACCELEROMETERS

STATIC MONITORING
8 DISPLACEMENT TRANSDUCERS
2 INCLINOMETERS

ENVIRONMENTAL MONITORING
2 TEMPERATURE/RELATIVE HUMIDITY SENSORS

THE "GIOTTO'S CHAPEL" IN PADOVA

CRACK OPENING (OCT-DEC 2013)
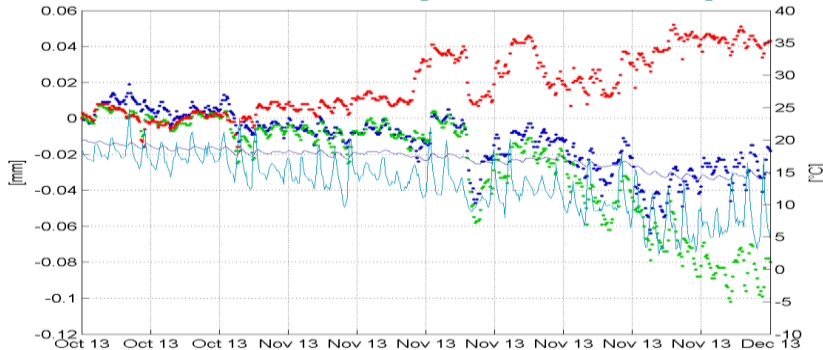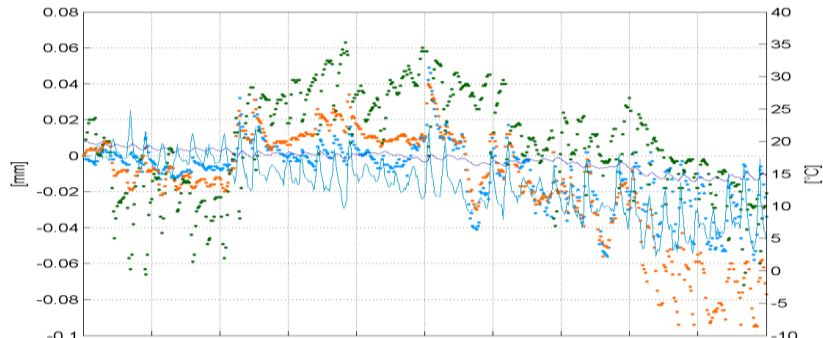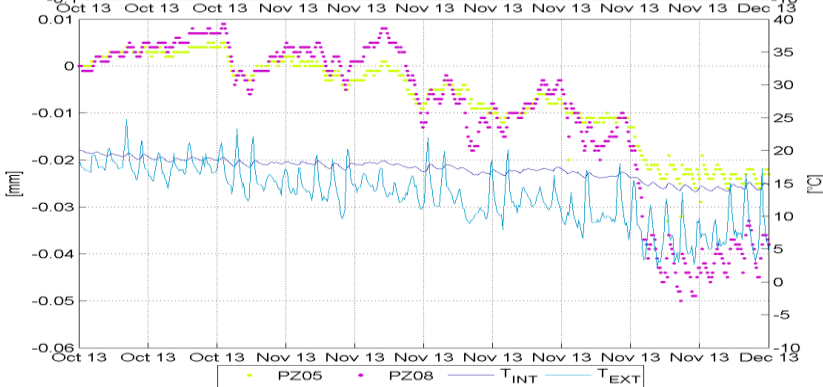
LONG. WALLS HORIZ. DISPLACEMENT (OCT-DEC 2013) OUT-OF-PLANE
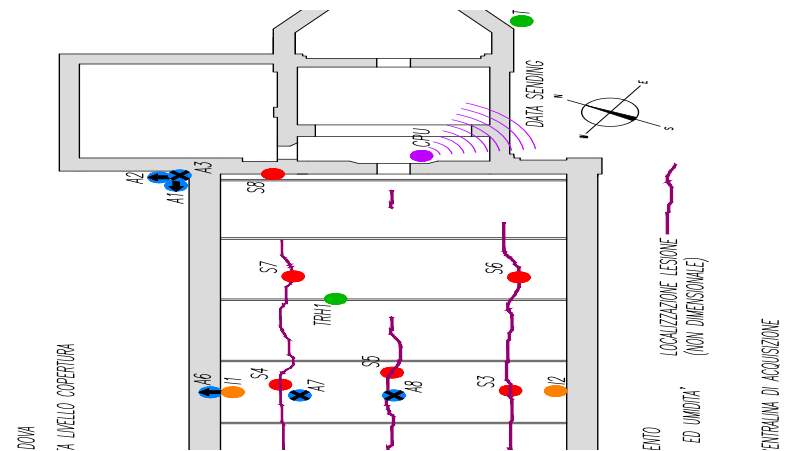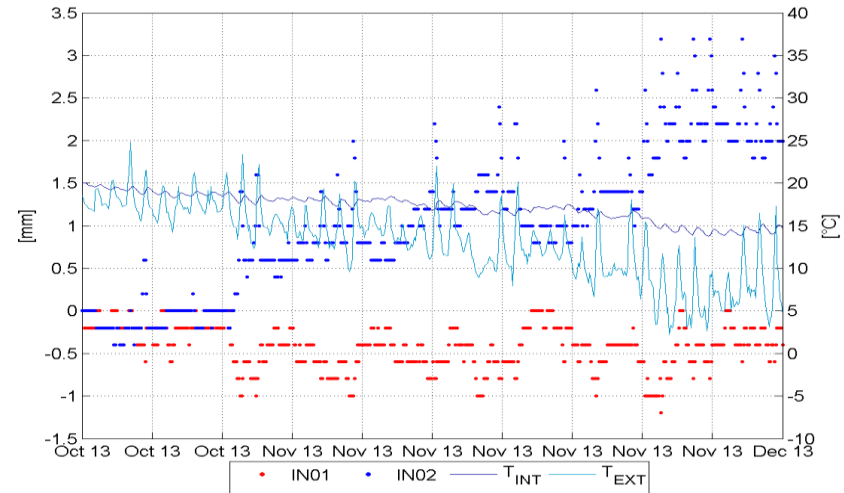
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  - **Frari Bell Tower in Venice**

  - Aggregate building in L'Aquila

BASILICA DEI FRARI - VENICE
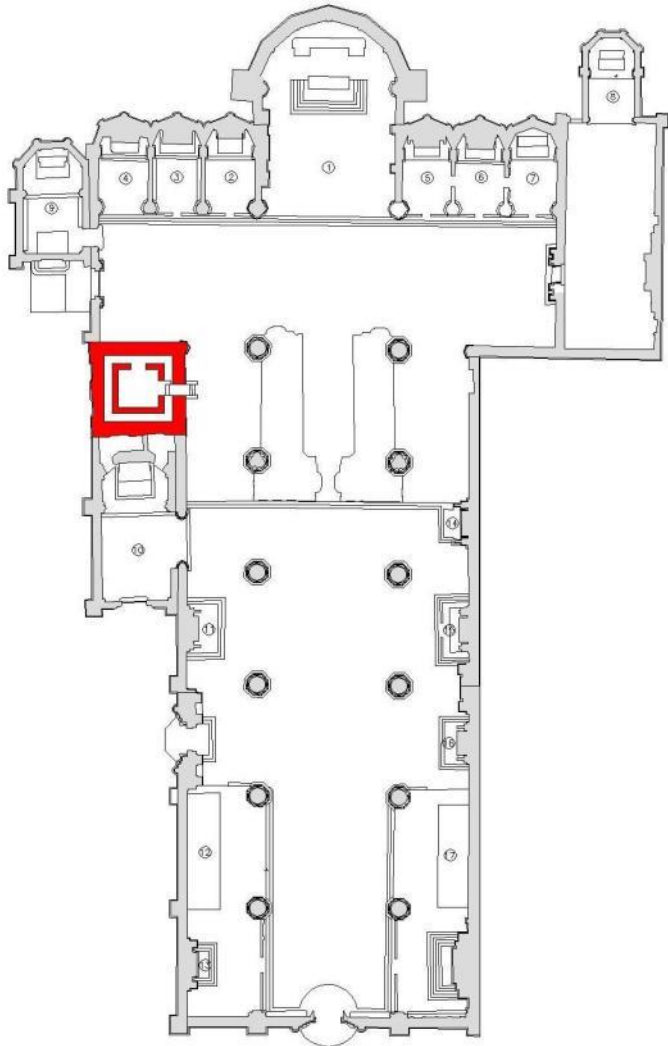
1st PHASE (1361-1396): CONSTRUCTION OF THE BELL TOWER

The tower, built next to the church between 1361 and 1396, was originally conceived as a completely independent structure

It is 65 m tall, has a square base of about 9.5m and shows a double pipe brick masonry structure, supporting the internal staircase.

BASILICA DEI FRARI - VENICE
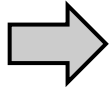
1902: COLLAPSE OF THE BELL TOWER OF THE ST. MARK BASILICA

STRUCTURAL INTERVENTION IN THE 20TH CENTURY
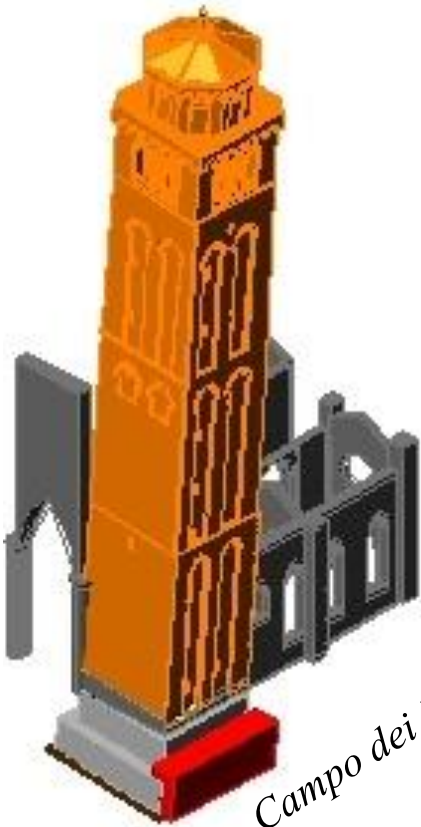
Monitoring of the venetian towers, including the "Frari" bell tower

RESULTS OF THE MONITORING:

- **DIFFERENTIAL SETTLEMENT BETWEEN CHURCH AND TOWER: 30 cm**
- **OUT-OF-PLUMB: 76cm ON A HEIGHT OF 42.5m**

1903: Intervention on tower's foundations

CONSEQUENCES:

- **Reverse of tower's rotation toward the church**
- **The new structural configuration caused the formation of widespread cracks and extensive damages on structural elements of the church directly connected to the bell tower**

Campo dei Frari

Pavimentazione esterna
Cordolo in calcestruzzo
Fondazione aggiuntiva

SECTION A-A

BASILICA DEI FRARI - VENICE

settlements
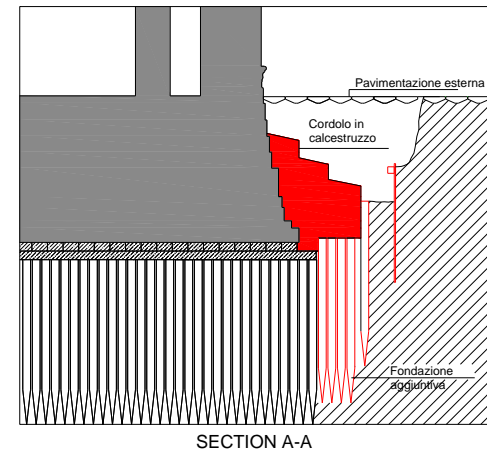
Experimental investigations and monitoring

1990   - fotogrammetric survey;
- geotechnical investigations on the foundation's soil;
- endoscopies;
- single and double flat-jack tests on the masonry elevation structures;
- sonic tests on steel ties;
- monitoring of the main cracks, by means of extensometers;
- positioning of clinometers (detection of rotations of the bell-tower).

⇓

discrete stability of the tower structure; out of plumb of about 0.8 m

2000   - worrying sign of structural deterioration (new crack patterns; widening of
already existing fissures; falling of small portions of plaster and bricks
from the vaults).
- survey of differential settlements in different points of the complex

⇓

disconnectedness of the stone ashlars of the aisle arch adjacent to the bell-tower
(differential settlement of the arch supports) → **installation of a timber prop**

average subsidence of the **structure of the church:**   **- 10 ÷ - 20 mm**
average subsidence of the **area of the bell-tower base:**   **- 49.8 mm East corner**
                                                               - 61.3 mm South corner
                                                               - 93.3 mm West corner

BASILICA DEI FRARI - VENICE

XV   XV-XX   XX-XXI
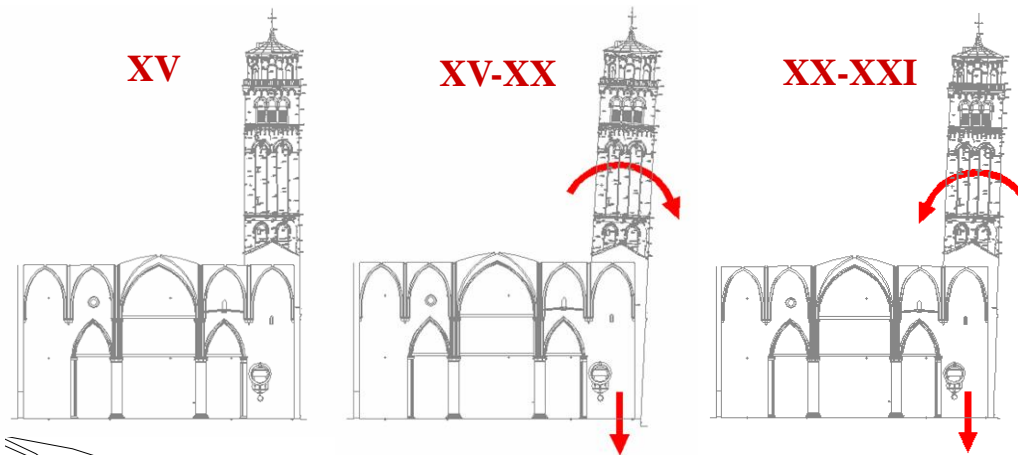
The differential settlements and the comparison between the fotogrammetric survey of 1995 and 2000, indicated that **the bell-tower is tilting in the opposite direction respect the "historical" tendency**, meaning that it is going back towards its vertical.

B4
A1
B3
B2
A2
A5
A3
A4
A6
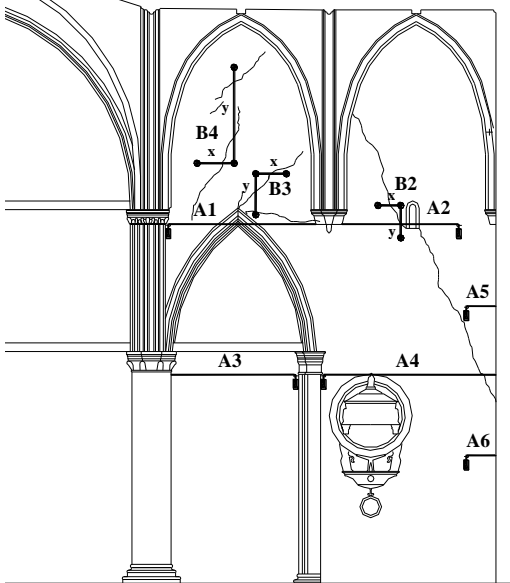
0   2.5   5m

A : long-base extensometers
B : crack-gauges

2001 automatic **monitoring system** (check of the deformations):
- **6 long base extensometers** relative displacements between the walls of the bell-tower and the adjacent structures of the basilica;
- **8 crack-gauges** installed on the main cracks of the South-West side of the bell-tower and of the wall above the stone arch.

⇓

the opening of the cracks is only partly caused by the settlement noticed at the foundations level

BASILICA DEI FRARI - VENICE

2003 investigation campaigns - analysis of the state of stress: flat jack tests

Second Test Campaign

M10   M11

M8   M9

1.76 MPa

M6
M7   M5

3.12 MPa

1.92 MPa

M1
M2
M12

M3
M4

1.44 MPa

Horizontal section of the column

M6
M5
M7

Third Test Campaign

(0.56)

(0.00)
(0.09)
(0.72)

(0.00)

(0.00)
(0.63)   (0.95)

Values in MPa
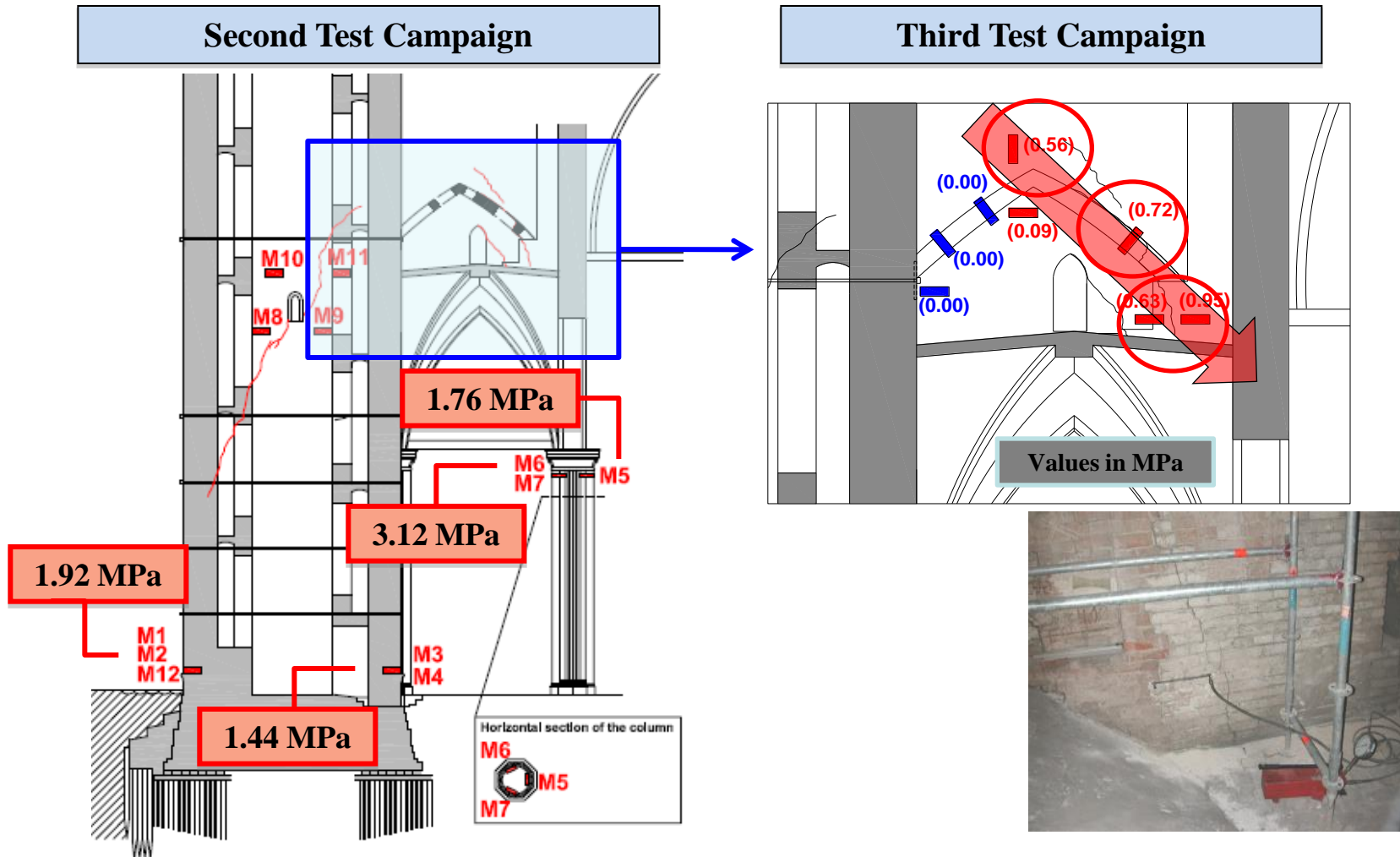
BASILICA DEI FRARI - VENICE
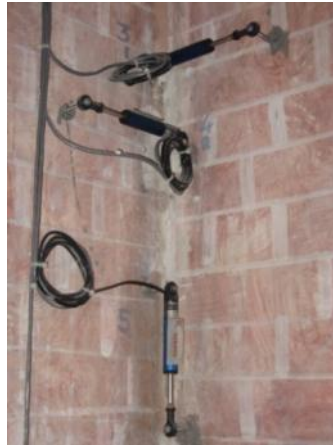
2003 extension of the automatic monitoring system

- ➤ **Direct Pendulum**
- ➤ **Crack-gauges**
- ➤ **Strain Gauges**
- ➤ **Temperature sensors**

SECTION A-A

SECTION B-B
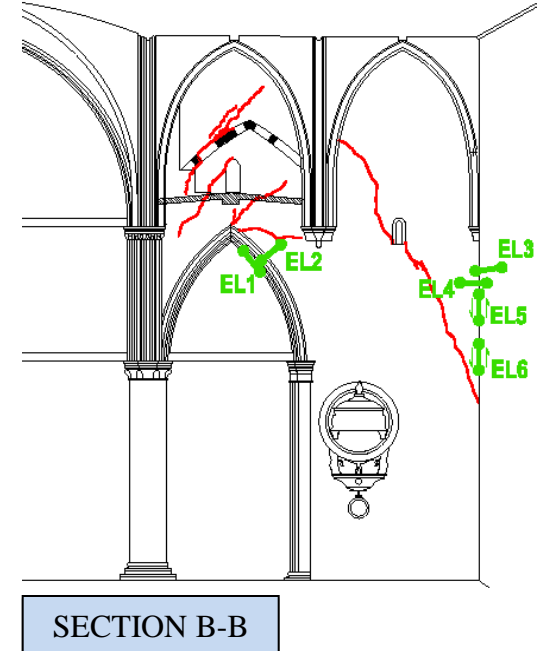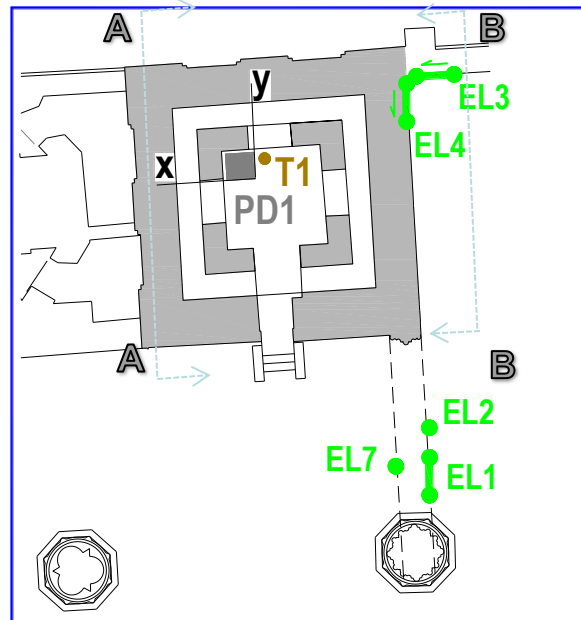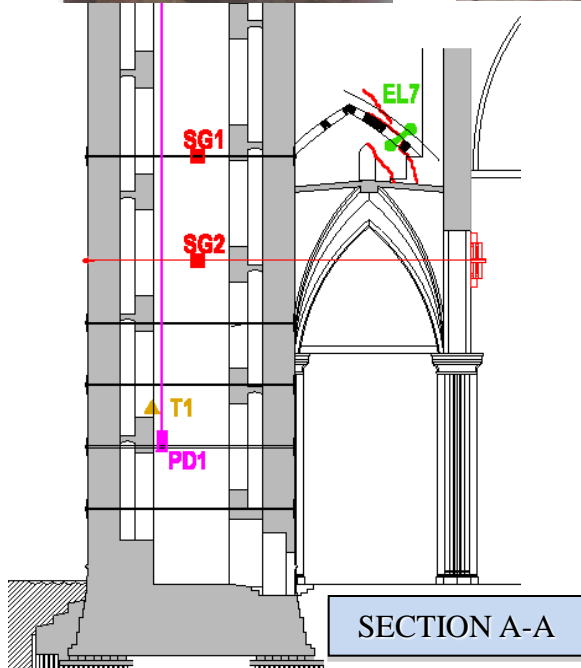
BASILICA DEI FRARI - VENICE

2003 extensive geotechnical investigation campaign

- analysis of the **subsoil stratigraphy and geotechnical properties**;
- exploration of the exact geometry and typology of the **foundation block**;
- definition of an accurate **geotechnical model** of the foundation finally completed;
- **in situ tests**:
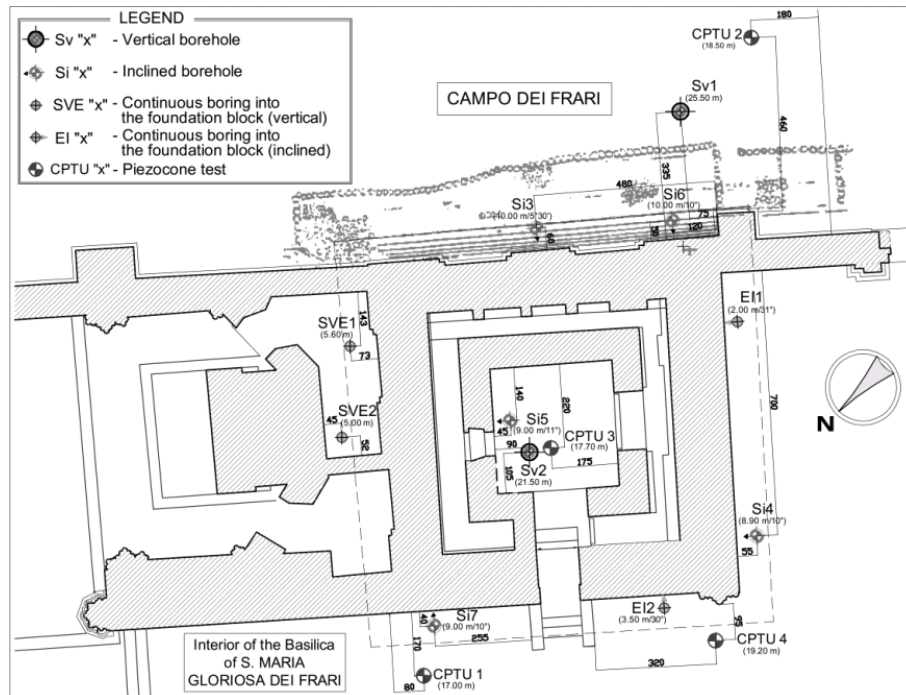
- 2 **vertical boreholes**: outside the church, to a depth of 25.50 m and inside the bell-tower, to a depth of 21.50 m;
- 5 **inclined boreholes**;
- 4 continuous **borings** into the foundation block:
    - EI1 and EI2 inclined of 30°, drilled for length of 4 m and located inside the basilica;
   - SVE1 and SVE2 short vertical borings, carried out on the NE side of the bell-tower;
- 4 **static penetrometer tests** with monitoring of pore water pressure (piezocone tests – CPTU) pushed to variable depths (17.00 ÷ 19.20 m);
- **Standard Penetration Tests** (SPT), in boreholes;
- **Extractions** of soil and foundation samples.

Claudio Modena

Analysis Methods for Unreinforced Heritage Masonry

BASILICA DEI FRARI - VENICE

Geotechnical section and foundation geometry
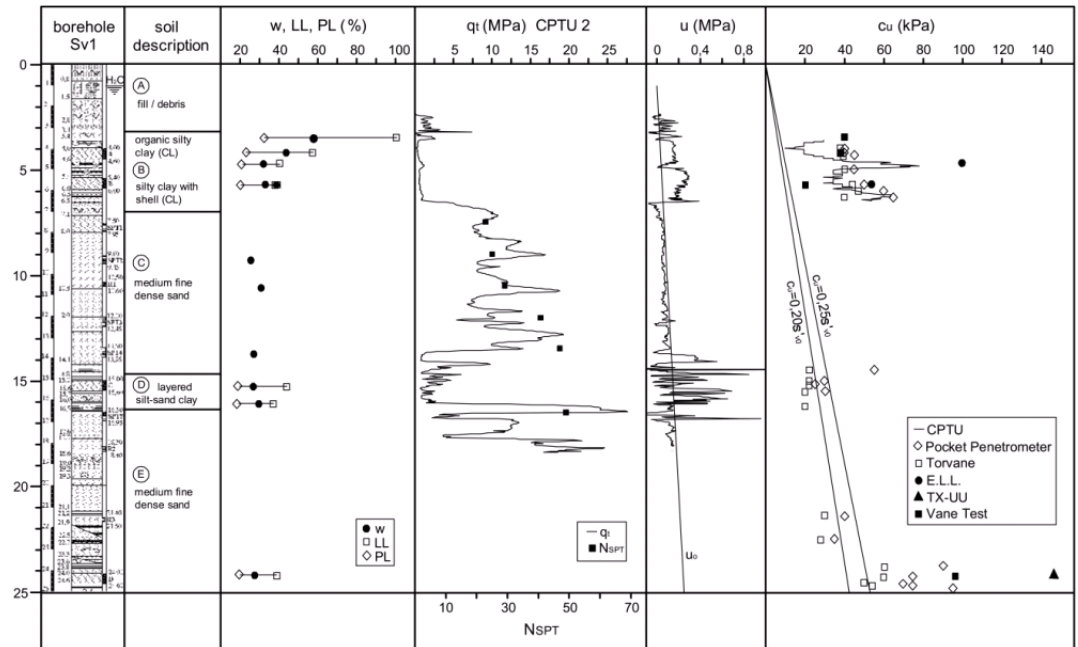
Origin of continuous settlements and stability problems

Some **progressive failure of the soft silty clay layer**, squeezed between the piles end and the unit C, must be taken into account.

In addition, a **possible increasing decay of the mechanical characteristics of the wooden piles** under the foundation block could also be seen as concomitant cause.

BASILICA DEI FRARI - VENICE

Structural modeling

The modelled portion of structure includes the **bell-tower and the adjoining parts of the church** that were mostly affected by the interaction with the tower

The only load condition considered is the **self weight**. The load corresponding to some parts of the real structure not modelled (timber structure roof of the basilica, belfry), was imposed as external forces; the crossed vaults' filling was included as surface load.

The **mechanical properties** chosen to describe the materials arise from the results of previous tests performed on the masonry structures. In particular:

- **elastic modulus** E = 3300 MPa (average of the results of double flat jack tests performed on the bell-tower masonry)
- **density** ρ = 2000 kg/m^3

The material is considered homogeneous and isotropic, and the analyses performed are linear elastic.

BASILICA DEI FRARI - VENICE


1450 model

1903 model
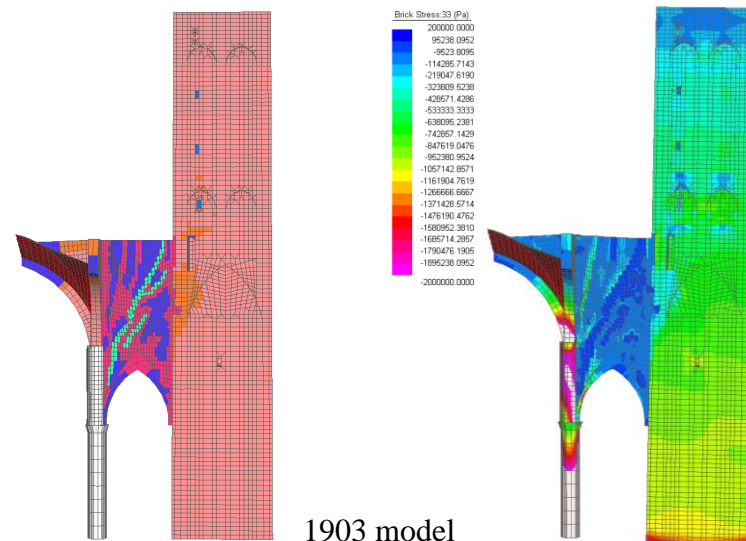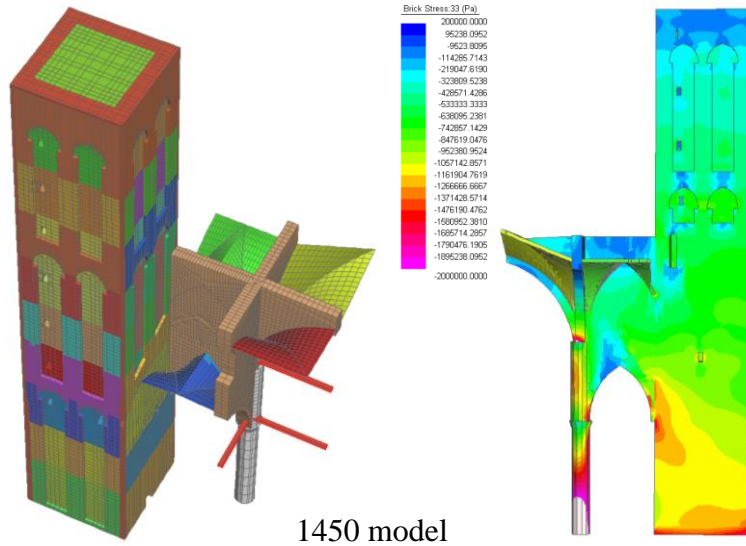
Two **previous models**, calibrated with the available experimental data (historical drawings, surveys, monitoring and on site tests), were analyzed before implementing the actual one, by means of imposed rotations and translations at the base of the bell-tower:

- after the construction (1450);
- before the strengthening intervention on the bell-tower (1903).

In each model, after running the analysis, a higher deformability was assigned to the elements subjected to an excessive tension respect the assumed strength of the material, in the successive analysis.
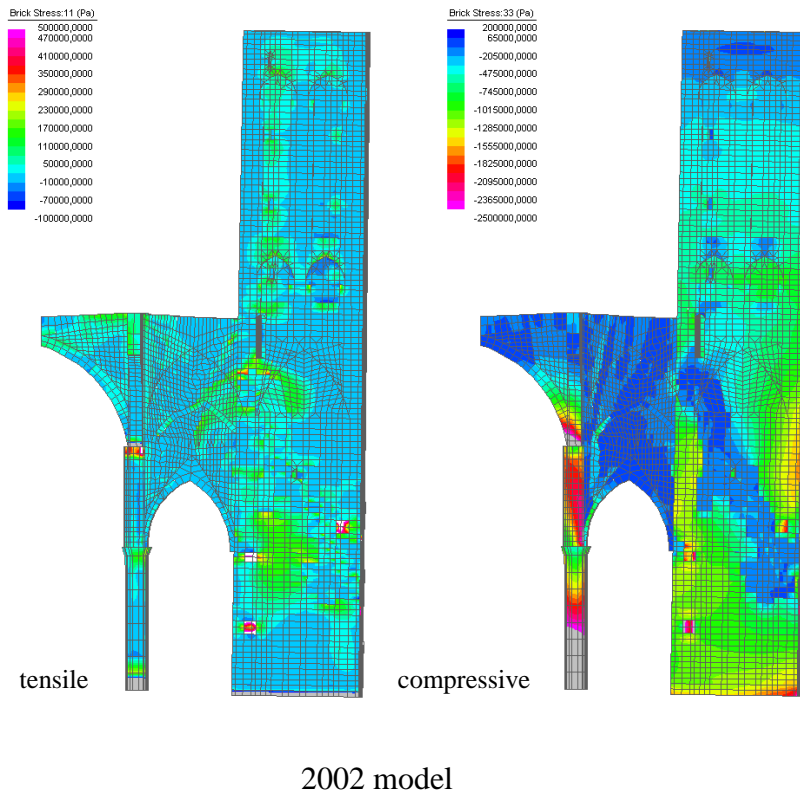
A **tensile stress concentration** appeared in the model where a **real crack pattern** is evident. The propagation of some principal cracks was followed by the subsequent iterative process.
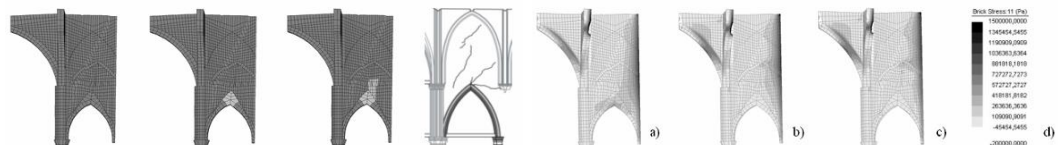
BASILICA DEI FRARI - VENICE

The **final model** reflects the tendency of the XX century. An "inverted" rotation was imposed to the bell-tower, with an average settlement of 84 mm.

Results:

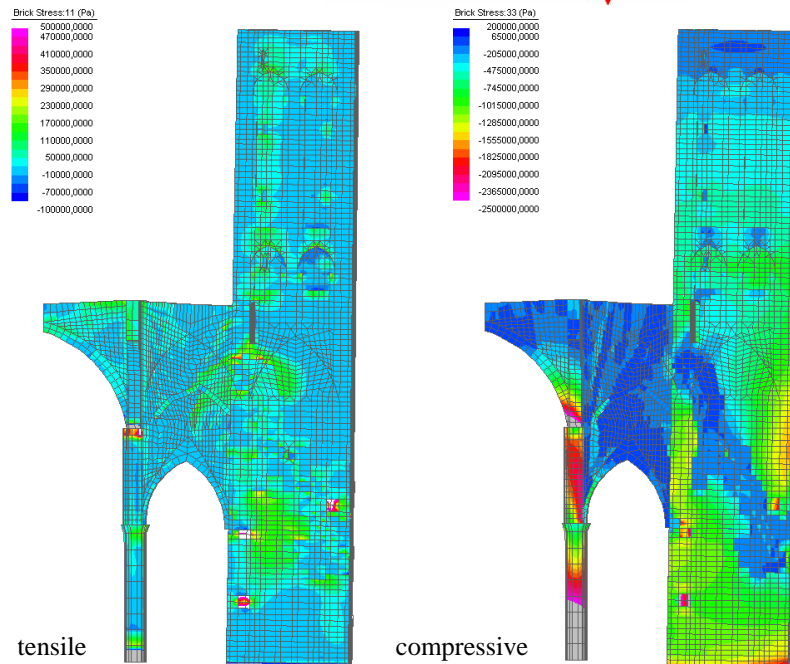tensile   compressive

2002 model

- preferential channels of **compressive stress** localized inside the masonry wall above the propped arch;

- high **tensile stresses** found in the same masonry wall, due to the settlements of the bell-tower → wide crack patterns and loss of shape of the stone arch;

- **high stress** found below the capital of the column → horizontal thrust determined by the movements of the bell-tower;

- **tensile stress** in correspondence of the bell-tower window opening on the transept → presence of the main fissure on the external pipe of the bell-tower.

BASILICA DEI FRARI - VENICE


XV   XV-XX   XX-XXI
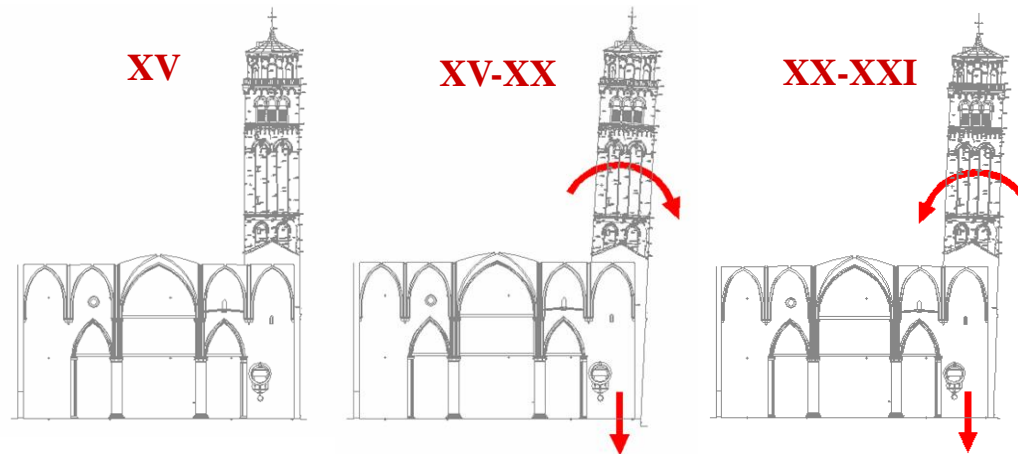
The differential settlements and the comparison between the fotogrammetric surveys, indicated that **the bell-tower is tilting in the opposite direction respect the "historical" tendency**, meaning that it is going back towards its vertical.

tensile   compressive

- preferential channels of **compressive stress** localized inside the masonry wall above the propped arch;
- high **tensile stresses** in the same masonry wall → wide crack patterns and loss of shape of the stone arch;
- **high stress** below the capital of the column → horizontal thrust determined by the movements of the bell-tower;
- **tensile stress** in correspondence of the bell-tower window opening on the transept → presence of the main fissure on the external pipe of the bell-tower.
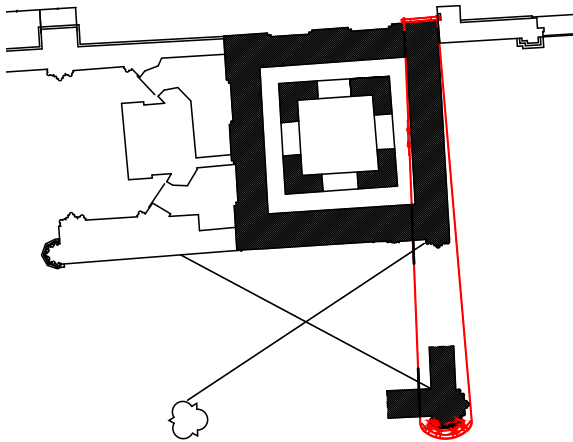
BASILICA DEI FRARI - VENICE
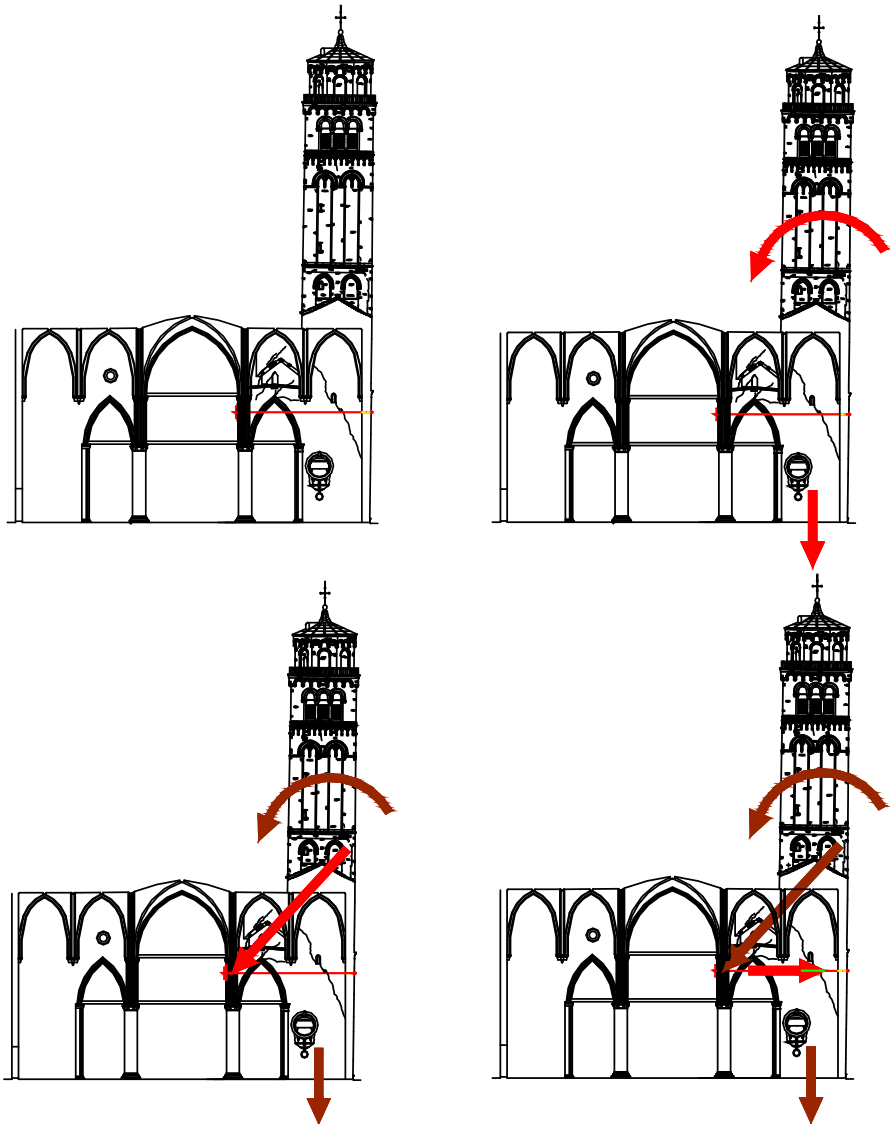
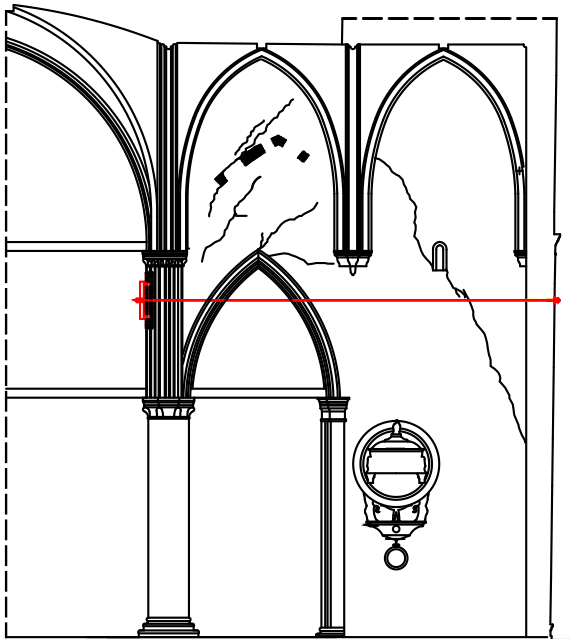**Realization of a provisional intervention for static control**

Structural schemes considered for the design of the intervention
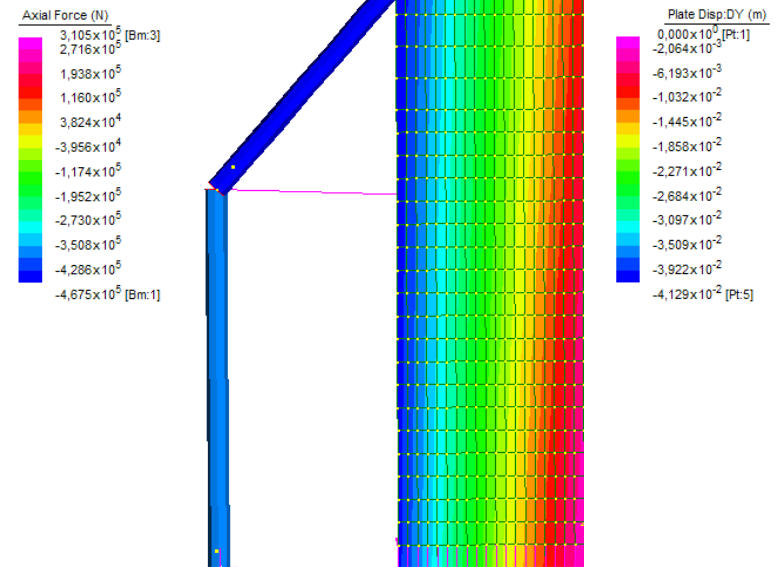
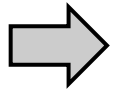# BASILICA DEI FRARI - VENICE

Realization of a provisional intervention for static control

BASILICA DEI FRARI - VENICE
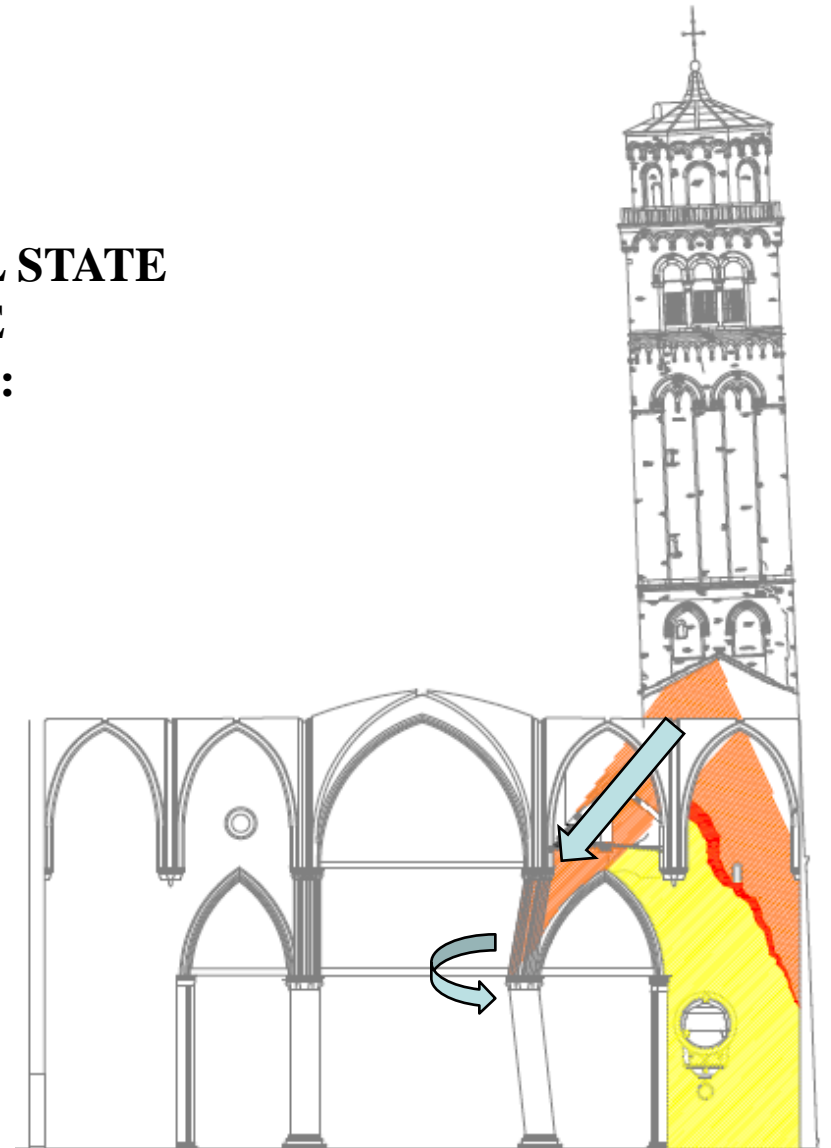
Structural Diagnosis

CONSEQUENCES OF THE INTERNAL STATE OF STRESS CREATED BY THE MECHANICAL INTERACTION:

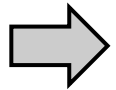increase of the compression load on the column

a strong transverse bending stress on the column, due to both the eccentricity of the vertical load applied to it and the horizontal component of the thrust

decrease of the vertical load (equal to the increase on the column) on the tower
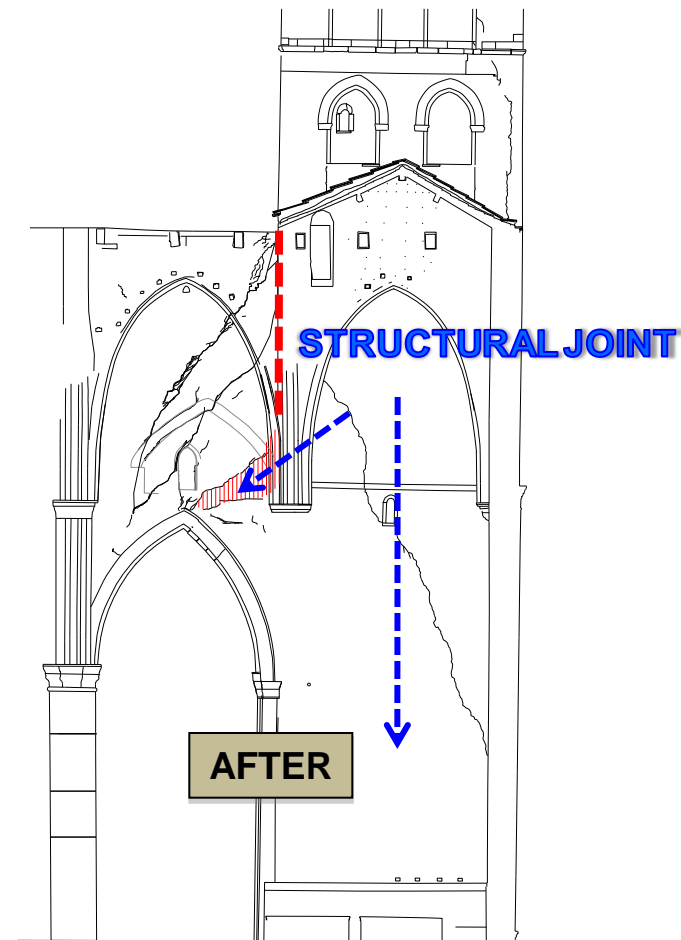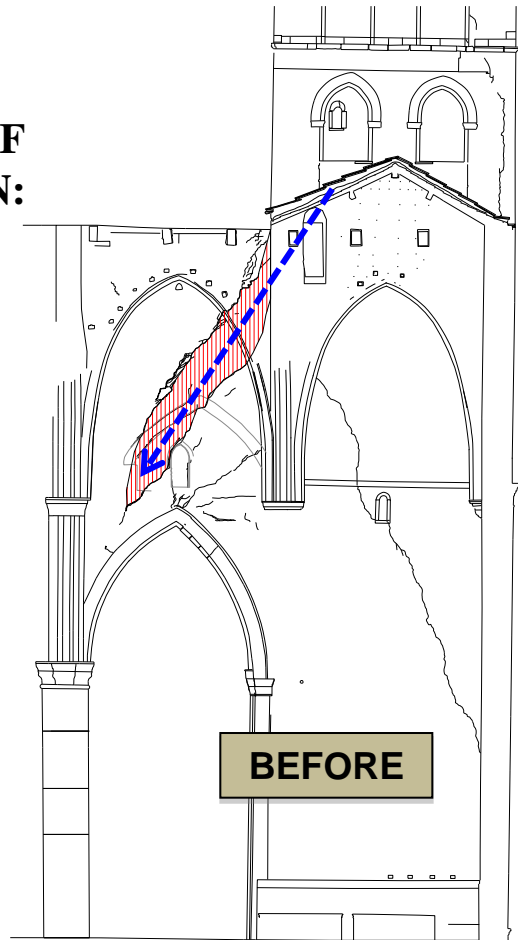
BASILICA DEI FRARI - VENICE
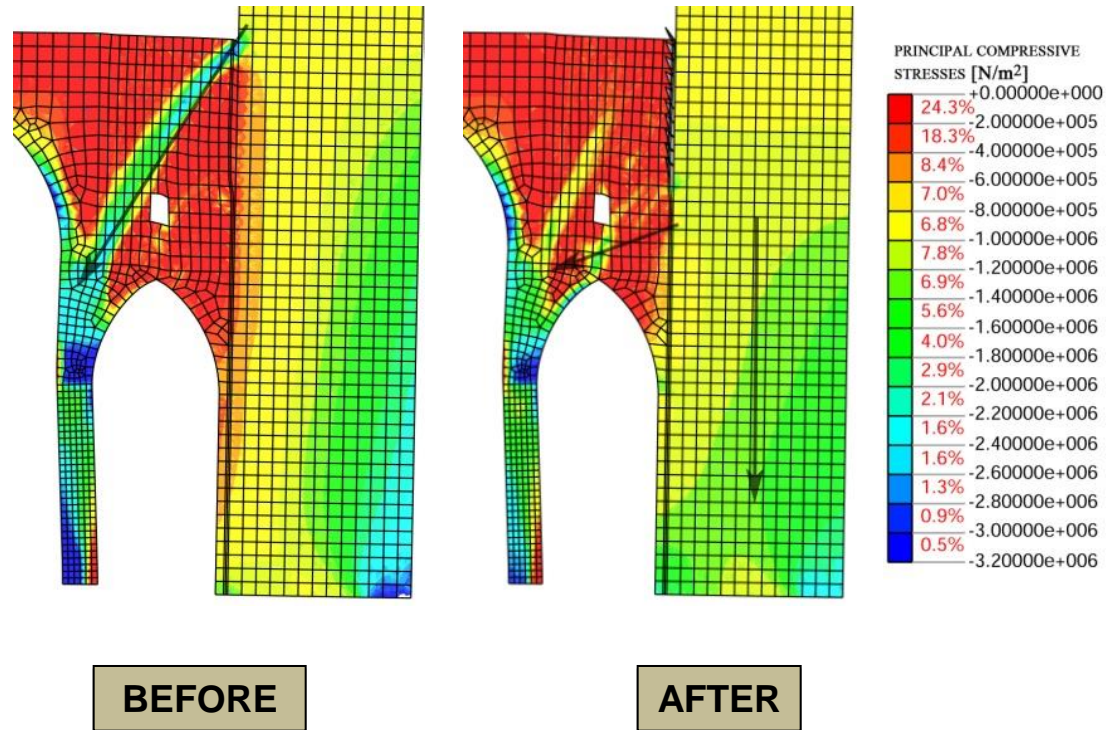
Structural Diagnosis

➡ **BASIC PRINCIPLE OF THE INTERVENTION:**

- Creation of a joint in order to separate the bell tower from the church and make them structurally more independent

- Reduction of the compressive forces that transfer part of the tower's self weight to the column

BEFORE

AFTER

STRUCTURAL JOINT

BASILICA DEI FRARI - VENICE

BEFORE

AFTER

PRINCIPAL COMPRESSIVE STRESSES [N/m^2]

24.3%	+0.00000e+000
18.3%	-2.00000e+005
8.4%	-4.00000e+005
7.0%	-6.00000e+005
6.8%	-8.00000e+005
7.8%	-1.00000e+006
6.9%	-1.20000e+006
5.6%	-1.40000e+006
4.0%	-1.60000e+006
2.9%	-1.80000e+006
2.1%	-2.00000e+006
1.6%	-2.20000e+006
1.6%	-2.40000e+006
1.3%	-2.60000e+006
0.9%	-2.80000e+006
0.5%	-3.00000e+006
	-3.20000e+006

Claudio Modena

Analysis Methods for Unreinforced Heritage Masonry

CONTENTS

- General issues on appropriate modeling strategies

- Cases studies:

    - Church of S. Maria del Pianto in Padova

    - Santa Maria Assunta Cathedral in Reggio Emilia

    - San Vigilio Cathedral in Trento

    - Scrovegni Chapel in Padova

    - Frari Bell Tower in Venice

    - **Aggregate building in L'Aquila**

- Carry out a pilot case study of an aggregate building in L'Aquila historic center, to give an operational methodology;

- The work has been carried out by the "Working group for the evaluation of seismic safety and intervention strategies on masonry buildings in historic centres".

- Department of Civil Defense
- ReLUIS consortium
- Vice-commissioner for cultural heritage

An aggregate building is set of portions that are the result of **an articulated and not unitary genesis**, due to several factors (construction sequence, change in materials, change of needs, change of owners, etc.).

In the analysis of a building that is part of an aggregate building it is necessary to take into account the **possible interactions** due to the close structures, which are interconnected or simply adjacent, with the aim of reducing the fragmentation into single intervention to a minimum.

It has to be preliminarily identified the **structural unit** (US) that is being studied and is formed by **one or more estate units**: it will have unitary structural behaviour against static and dynamic loads and will be delimited by open spaces, or structural joints, or close buildings made of different structural types, or material types, or built in different ages.
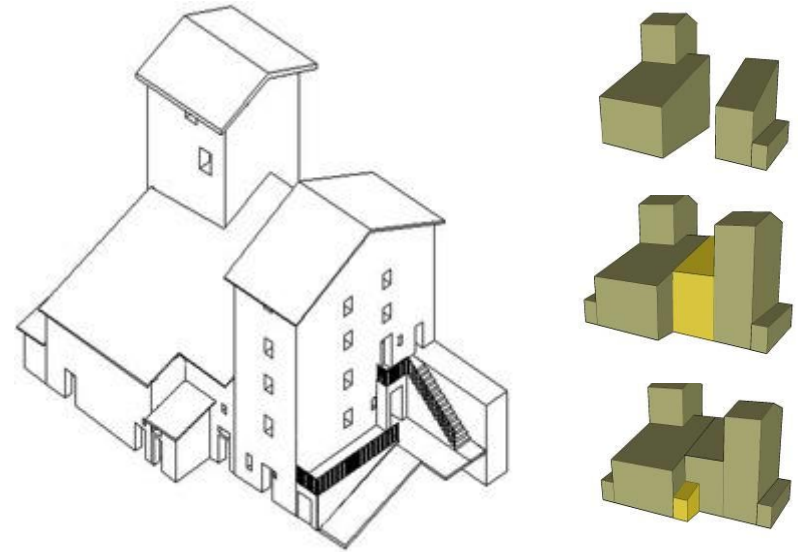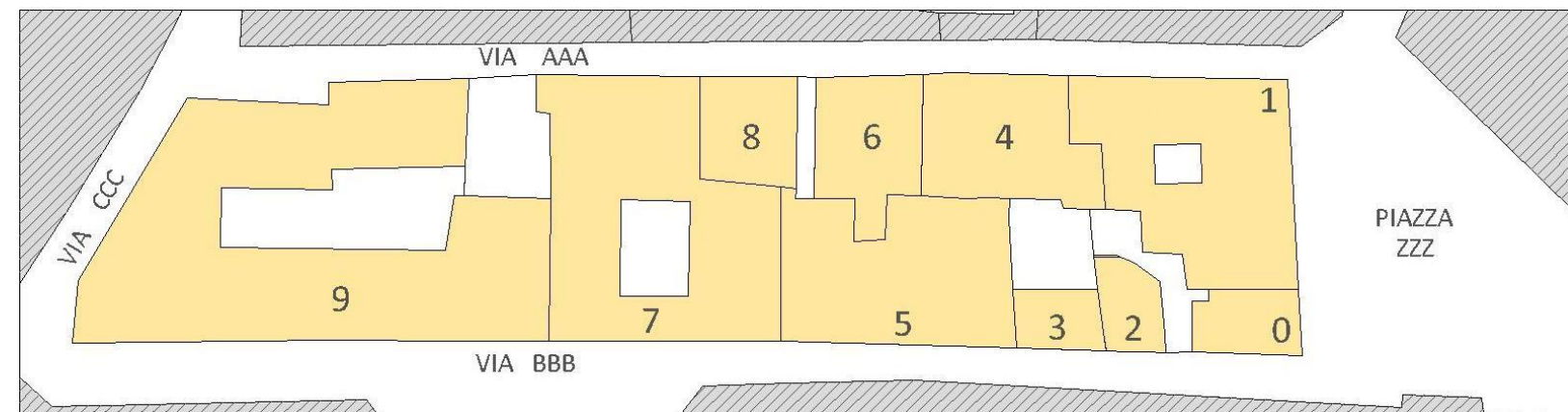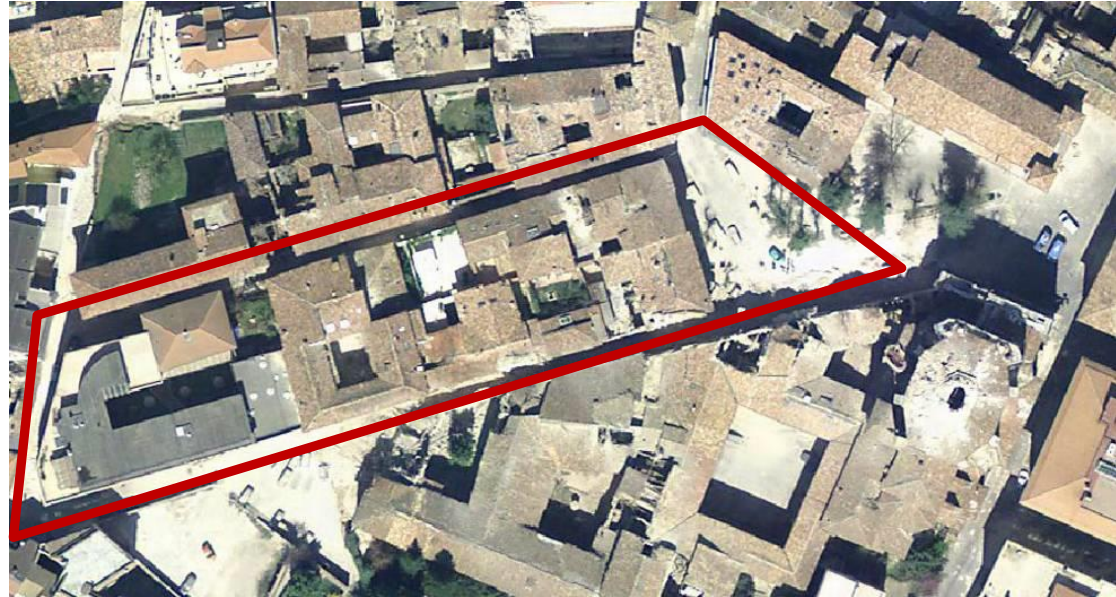
PHASE 1: SURVEY AND KNOWLEDGE

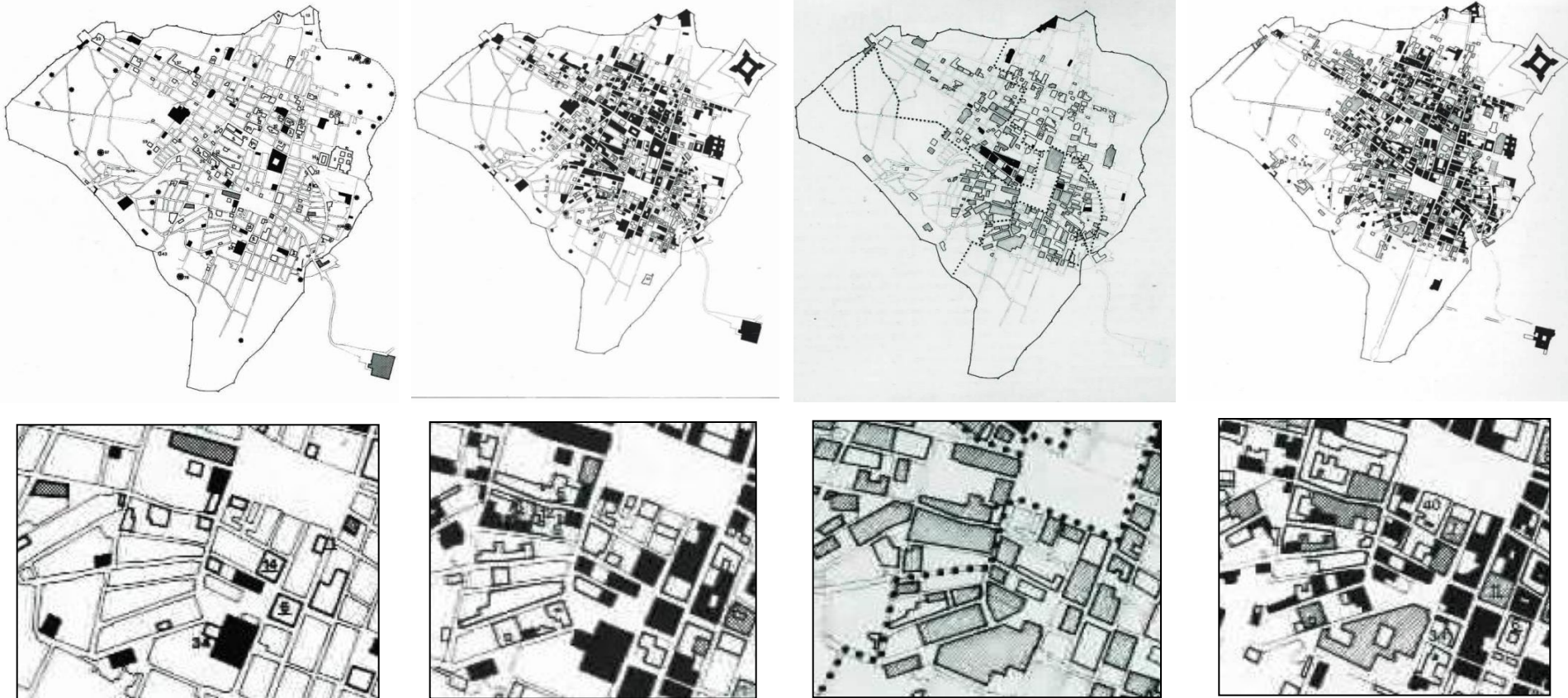## IDENTIFICATION OF THE BUILDING AGGREGATE

The block:
• Independent of neighbouring blocks
• analysed in a unified way, separately from the general context
• slope in both longitudinal and transverse directions
• three empty spaces

STUDY OF THE EVOLUTION OF THE AGGREGATE

ANALYSIS OF HISTORIC SECTIONS AND ARCHIVE MATERIAL

Historic section of **1500.**

Historic section of **1700**.
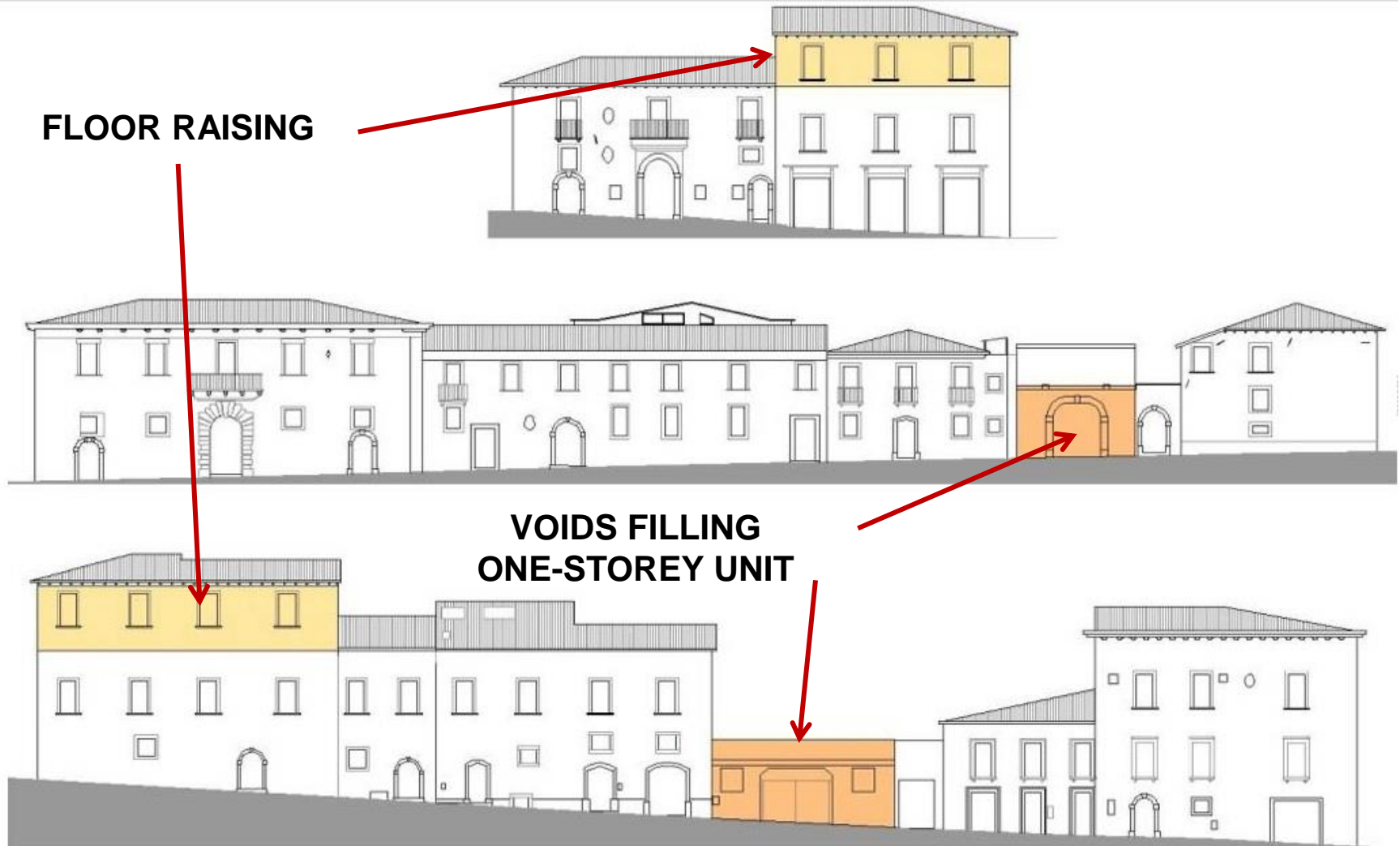
Historic section of **1703**.
The earthquake-induced damages are highlighted

Historic section of **1858**.

SOURCES OF INFORMATION
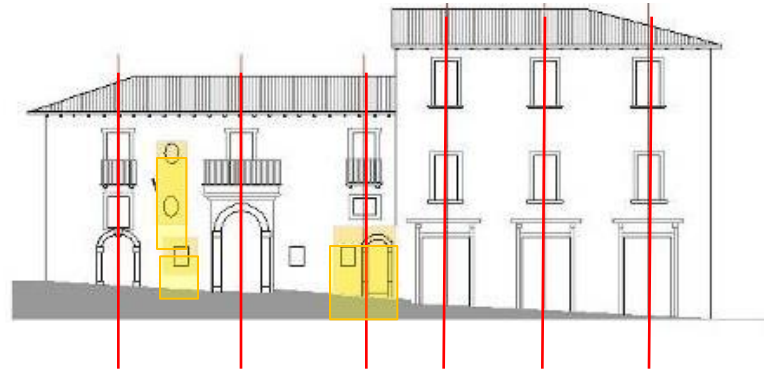
STUDY OF THE EVOLUTION OF THE AGGREGATE

CRITICAL EVALUATION OF THE STRUCTURAL AND ARCHITECTURAL ELEMENTS

FLOOR RAISING

VOIDS FILLING
ONE-STOREY UNIT

CRITICAL EVALUATION OF THE STRUCTURAL AND ARCHITECTURAL ELEMENTS
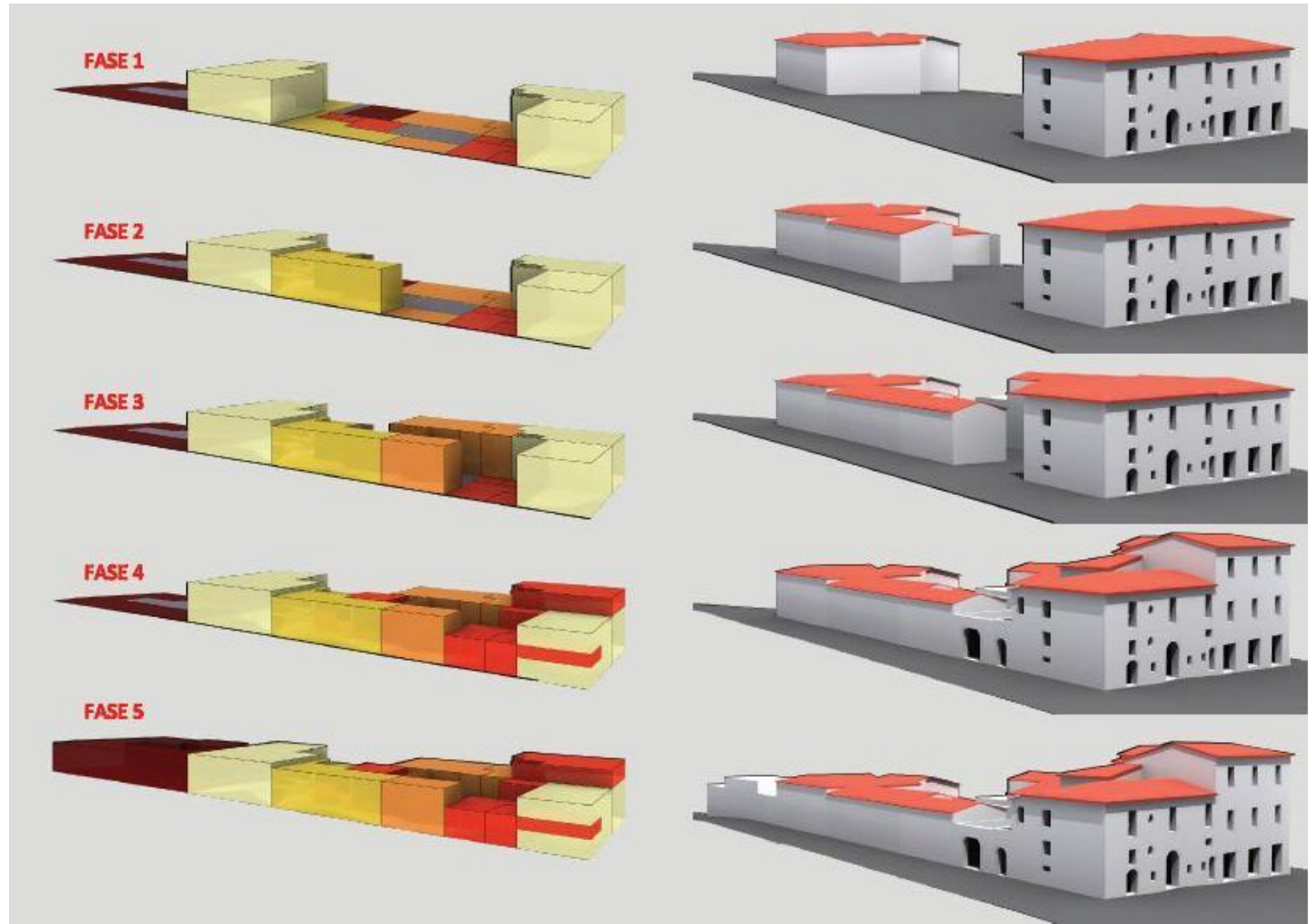
OPENINGS ALIGNMENT

IDENTIFICATION OF NON ORIGINAL OPENINGS
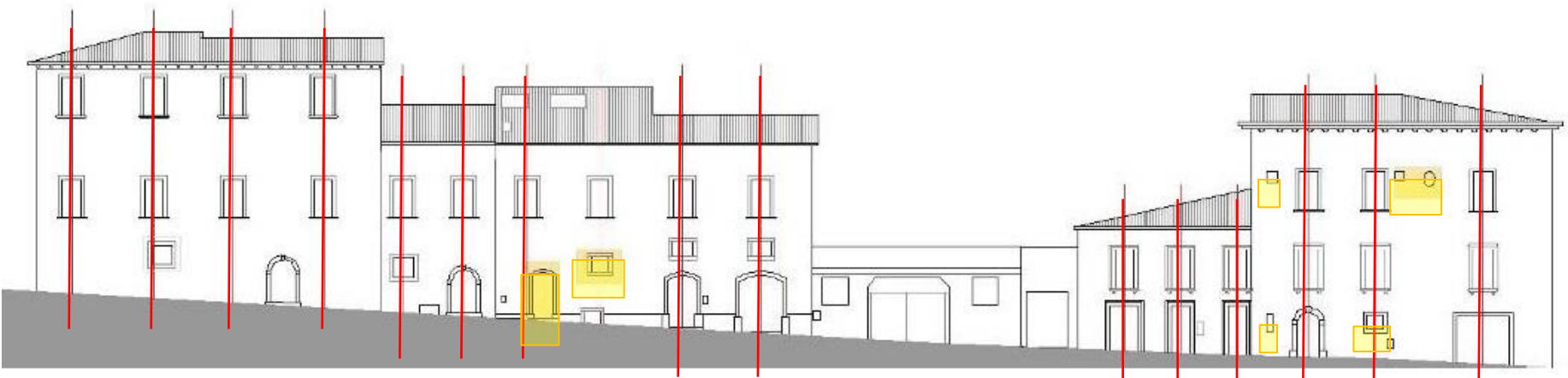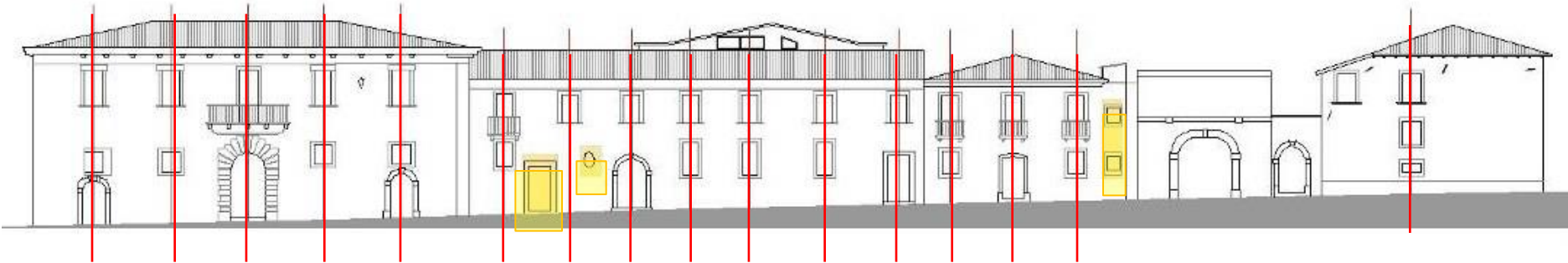
STUDY OF THE EVOLUTION OF THE AGGREGATE

HYPOTHESIS ON THE EVOLUTION OF THE AGGREGATE

Understand the evolution of the buildings, identifying or supposing first nucleus and following additions, in order to determine the structural behaviour

GEOMETRIC SURVEY

## IDENTIFICATION OF THE PLAN AND ELEVATION CHARACTERISTICS OF THE CONSTRUCTION ELEMENTS

PROSPETTO VIA AAA

PROSPETTO PIAZZA ZZZ


VIA AAA

VIA CCC

PIAZZA ZZZ

VIA BBB

PROSPETTO VIA BBB
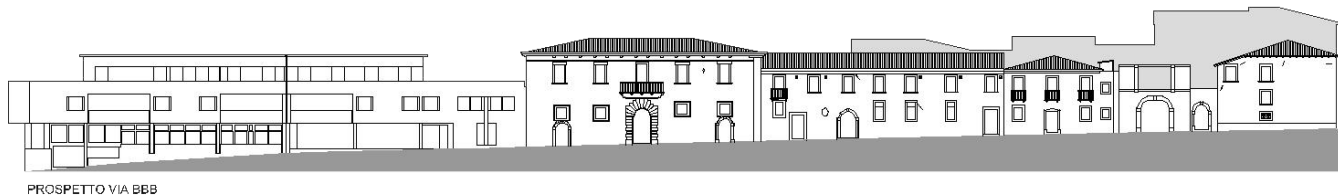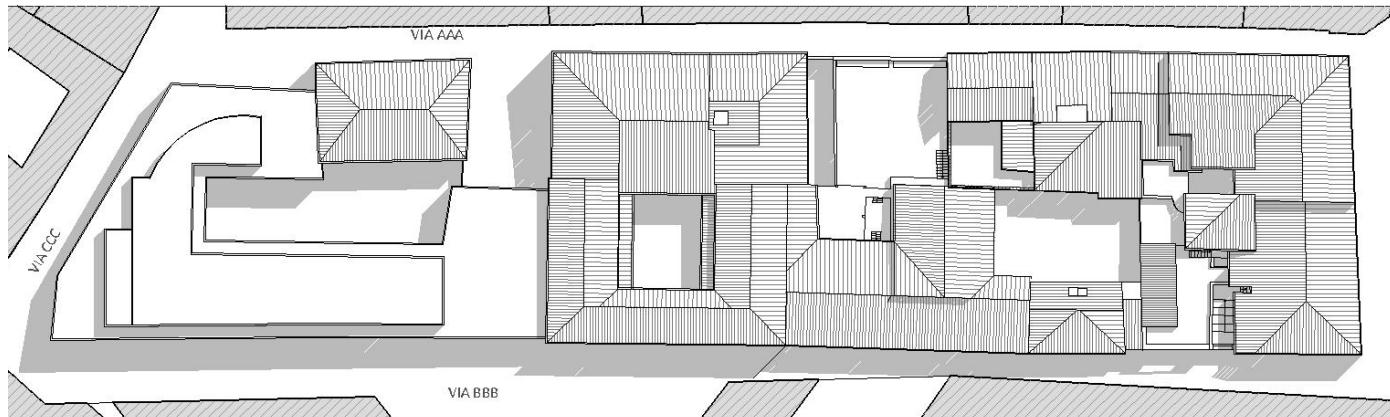
A geometric and structural survey is carried out in order to understand the structural behaviour of the buildings, to discover the local construction role and to identify any potential problem or alteration. It is often possible to find historical seismic protection systems like tie rods, buttresses and contrast arches.
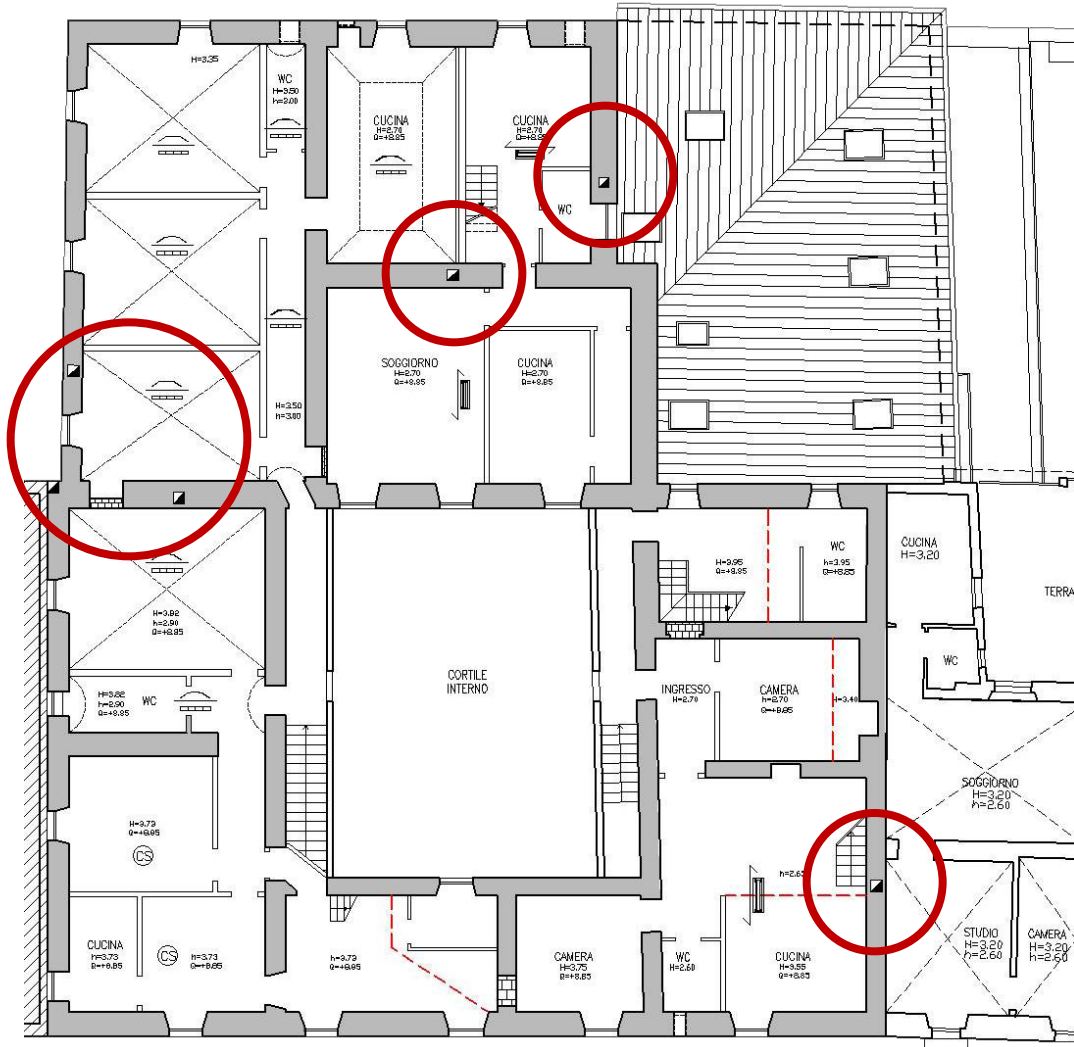
GEOMETRIC SURVEY

STRUCTURAL JOINTS

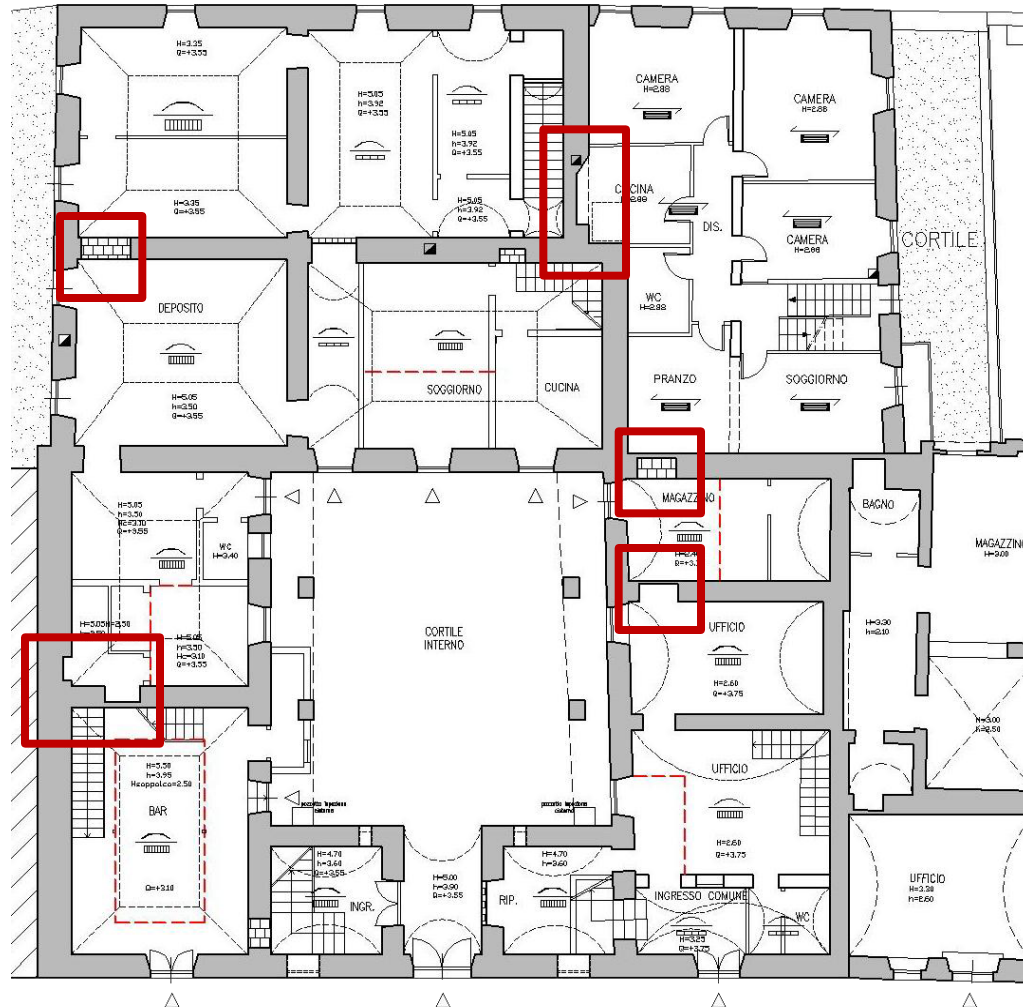## STUDY OF THE BUILDINGS CONNECTIONS
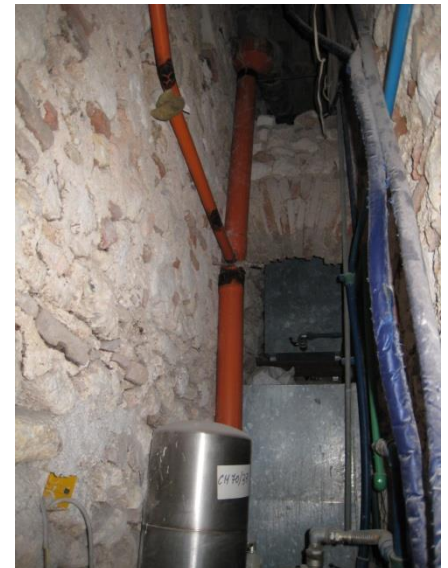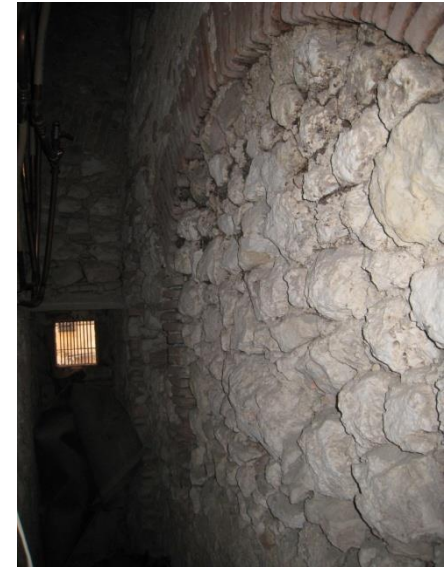
GEOMETRIC SURVEY

SPECIAL ELEMENTS OF THE SURVEY

PRESENCE OF CHIMNEY IN THE WALL THICKNESS

GEOMETRIC SURVEY
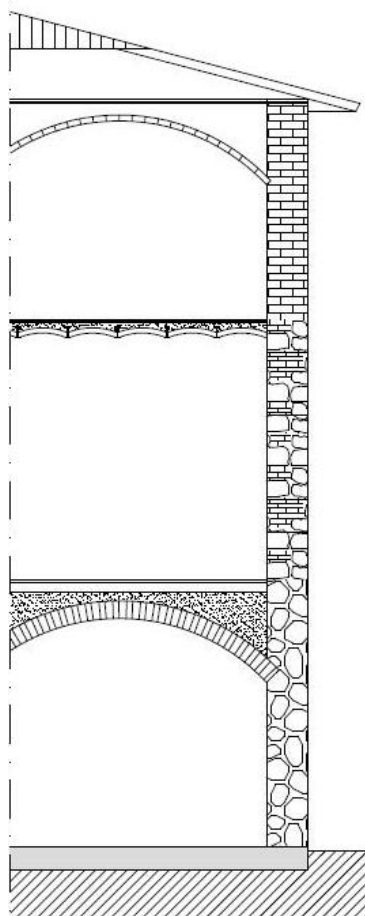## SPECIAL ELEMENTS OF THE SURVEY

PRESENCE OF NICHES AND CLOSED OPENINGS

STRUCTURAL SURVEY

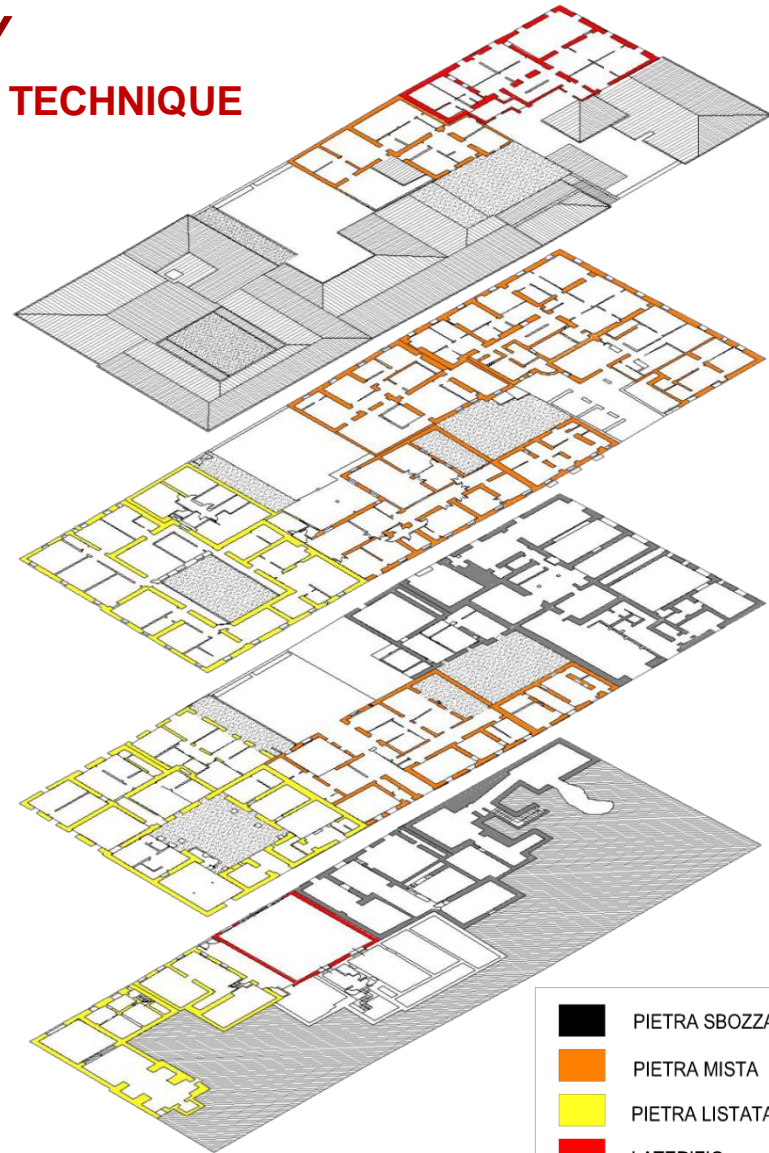## ANALYSIS OF THE LOCAL CONSTRUCTION TECHNIQUE

VERTICAL ELEMENTS

BRICK MASONRY

irregular texture and some waste materials such as small bricks or stones

irregularly hewn limestone walls, with mortar of poor quality;

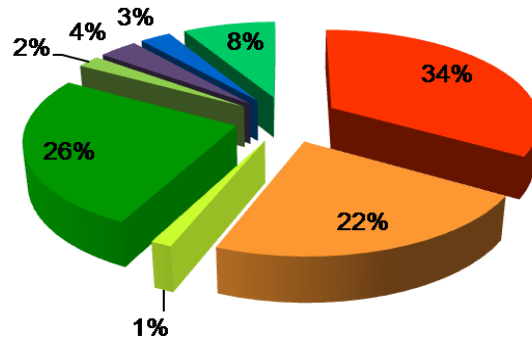|	
⬛	PIETRA SBOZZATA E IRREGOLARE
🟧	PIETRA MISTA
🟨	PIETRA LISTATA
🟥	LATERIZIO

STRUCTURAL SURVEY

ANALYSIS OF THE LOCAL CONSTRUCTION TECHNIQUE

HORIZONTAL ELEMENTS

34%
22%
1%
26%
2%
4%
3%
8%

- vaults with bricks laid edgewise or vaults with stones
- vaults with bricks laid flatwise
- steel beams and brick small tiles
- steel beams and brick tiles
- steel beams and brick vaults
- concrete and masonry
- wooden diaphragms
- light diaphragms

The horizontal diaphragms play an important role in the seismic response of the building as:
- transferring the horizontal actions to the walls which are parallel to the seismic action;
- a constraint to the walls stressed by orthogonal actions to their plane and a connection between walls.

MIXED FLOOR: STEEL BEAMS AND BRICK HALLOW ELEMENTS

CRITICAL SURVEY OF DAMAGES
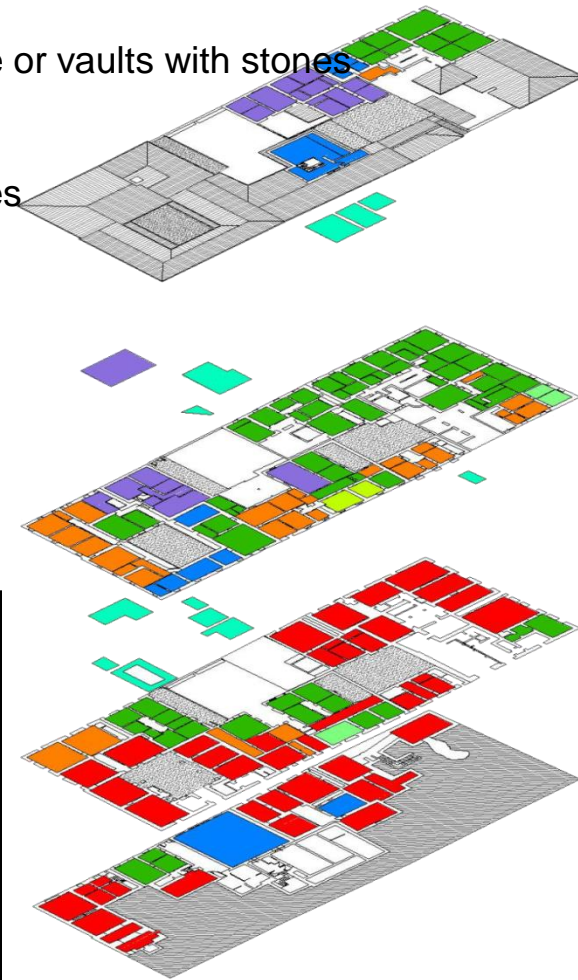
IDENTIFICATION OF THE VULNERABILITIES

COLLAPSE OF THIN VAULTS
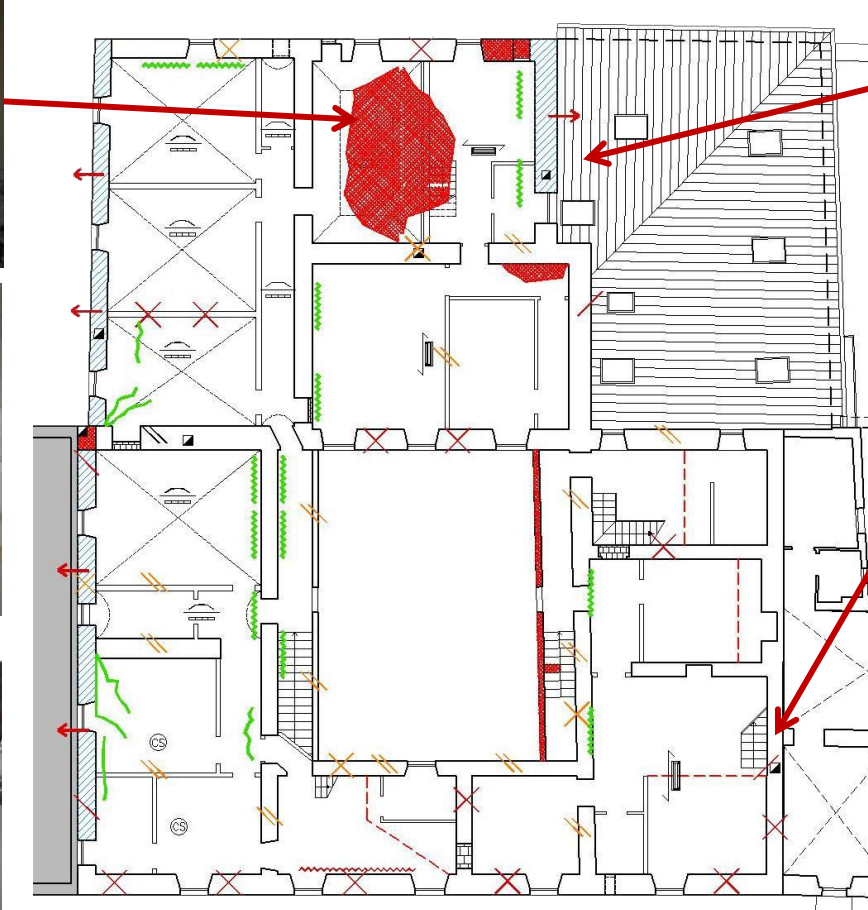
WEAKENING OF MASONRY

R.C TIE BEAM
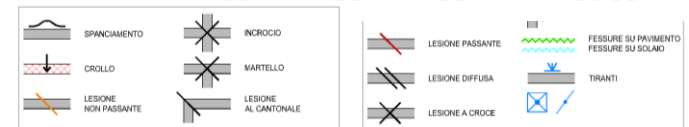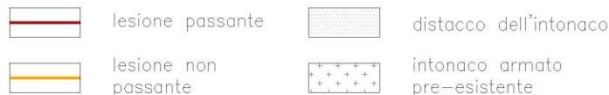
CRITICAL SURVEY OF DAMAGES

ANALYSIS OF THE CRACK PATTERN

IDENTIFICATION OF THE ACTIVATED MECHANISMS

RECOGNITION OF THE CAUSES OF DAMAGE AND OF THE MAIN VULNERABILITIES

QUALITATIVE EVALUATION OF THE GLOBAL BEHAVIOUR

The critical survey of damage is necessary to identify critical elements, toward static or dynamic actions, of the structural system, including deterioration. The analysis and systematic description of the cracks pattern allows to understand the reasons of their existence:

Identification of significant mechanisms of collapse

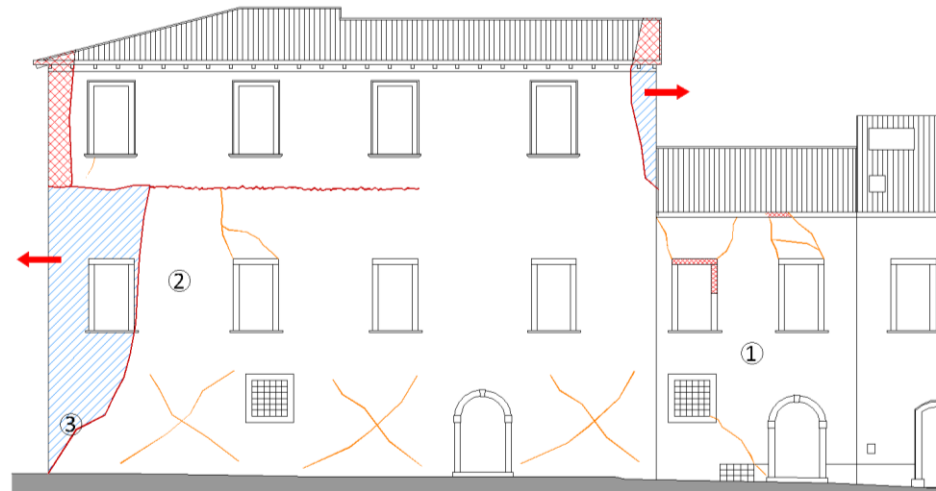# CRITICAL SURVEY OF DAMAGES

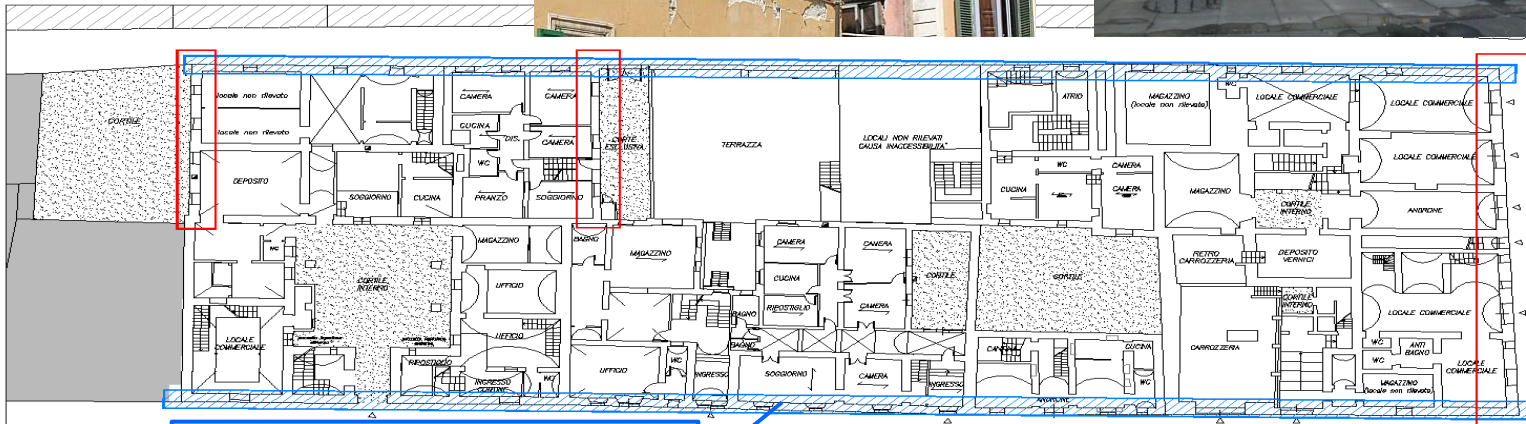## IDENTIFICATION OF THE ACTIVATED MECHANISMS

SIMPLE OVERTURNING
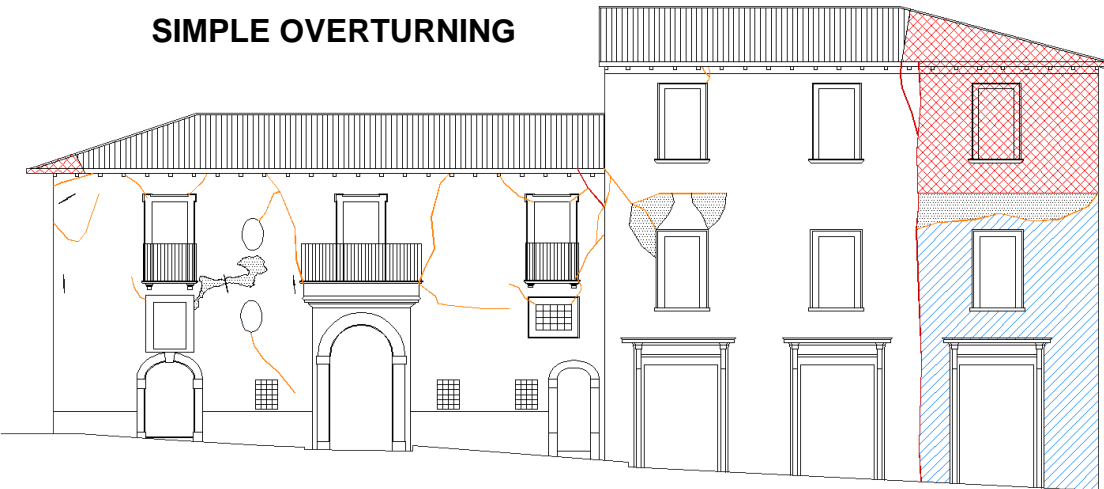
OVERTURNING WITH INVOLVEMENT OF THE WEDGE

CRITICAL SURVEY OF DAMAGES

GLOBAL BEHAVIOUR OF THE AGGREAGATE

TRANSVERSAL DIRECTION: **OUT-OF-PLANE OVERTURNING**

LONGITUDINAL DIRECTION: **IN PLANE FAILURE**

In existing masonry buildings under horizontal (seismic) loads, **local** and **global mechanisms** can develop.

The **safety level** of buildings must be evaluated against both types of mechanisms.

Existing masonry buildings

Global analysis and assessment

Analysis & assessment of local mechanisms
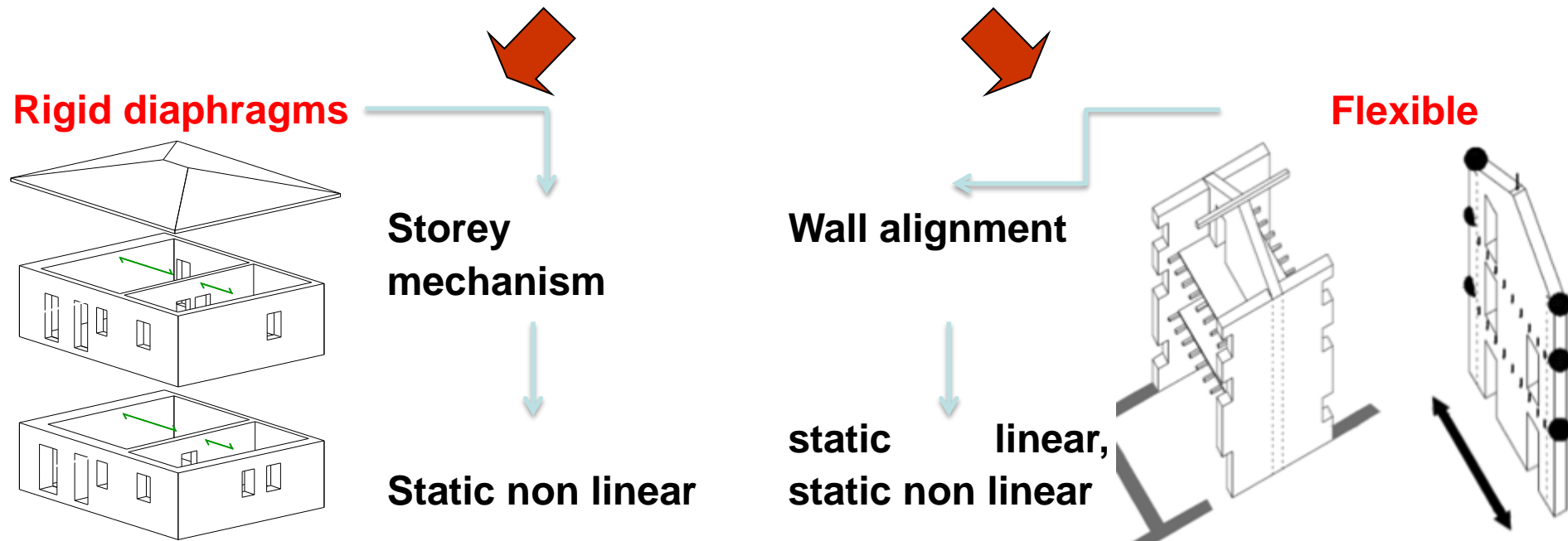
Global models

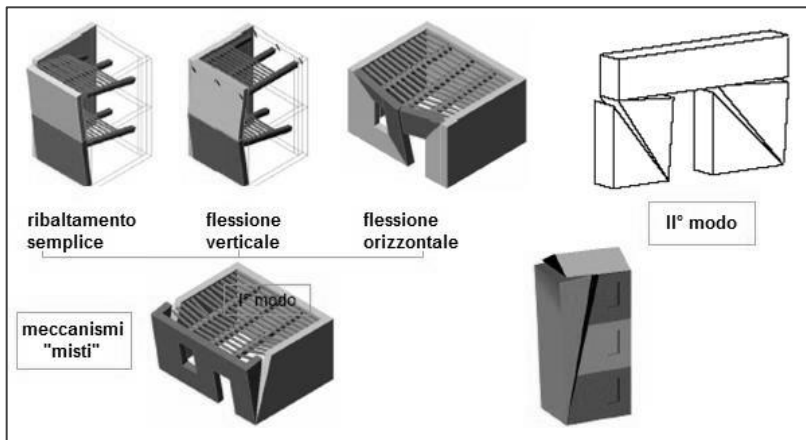**Comprehensive set of local verifications**

Kinematic approach

AGGREGATE BUILDINGS: SIMPLIFIED GLOBAL ANALISYS

When analysing a construction inside an **aggregate building,** we must take into account possible **interactions** […] we must identify the **structural unit** (US) under study, and evidence the actions deriving from adjacent structural units. We can use
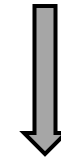
Simplified global methodologies

Rigid diaphragms

Flexible

Storey mechanism

Wall alignment

Static non linear

static   linear, static non linear

ANALYSIS THROUGH SIMPLIFIED MODELS: VULNUS

VULNUS (v.3.2)

EVALUATION OF THE SEISMIC VULNERABILITY OF ISOLATED MASONRY BUILDINGS OR BUILDINGS IN AGGREGATE

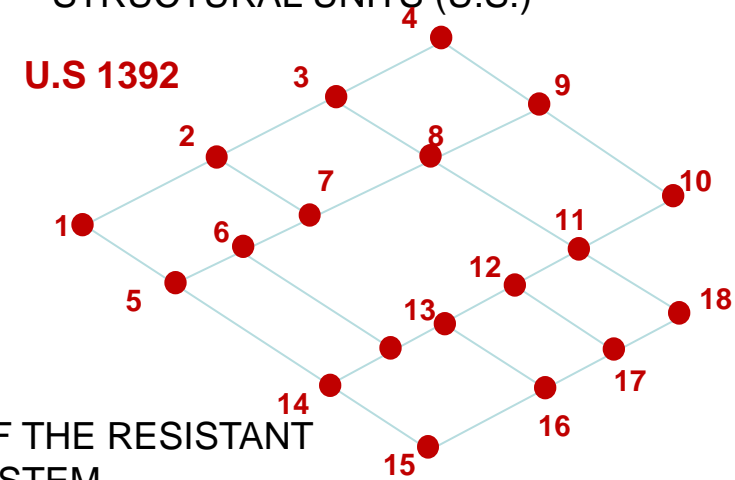**1-** STUDY OF THE ACTIVATION OF COLLAPSE MECHANISMS IN PLANE AND OUT OF PLANE
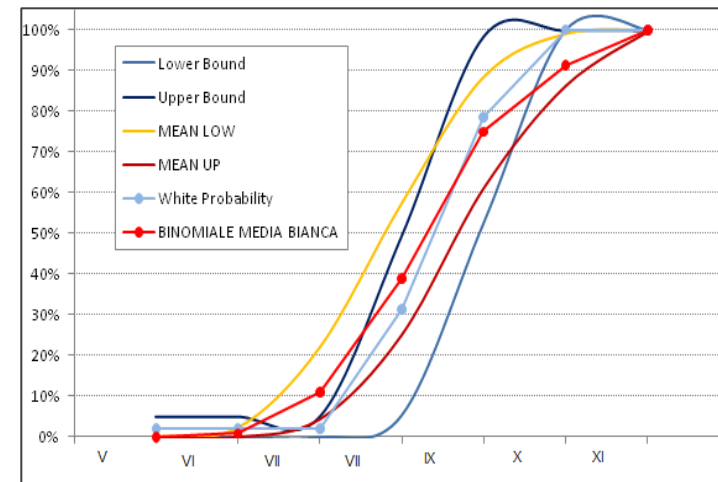
2- VULNERABILITY ANALYSIS OF SINGLE BUILDINGS AND OF THE AGGREGATE AND EVALUATION OF FORESEEN DAMAGES

ANALYSIS THROUGH SIMPLIFIED MODELS: VULNUS

SCHEME OF THE AGGREGATE
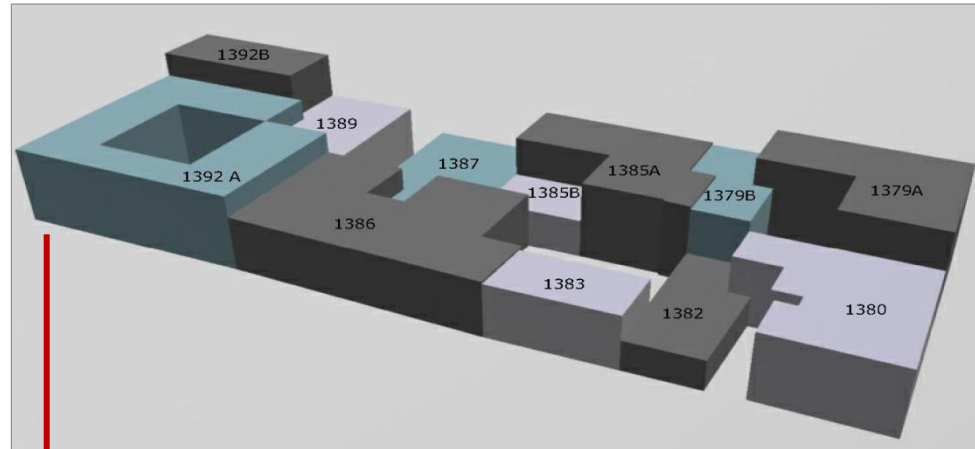
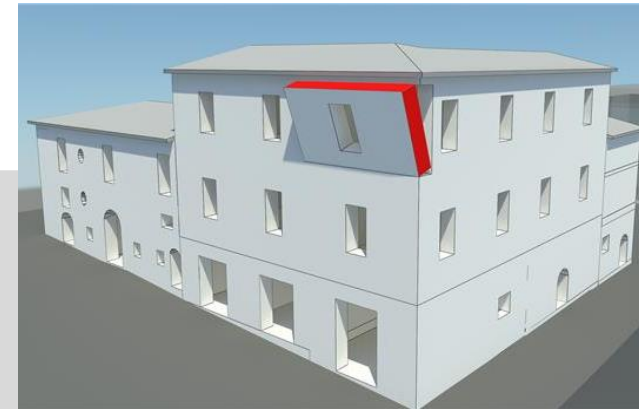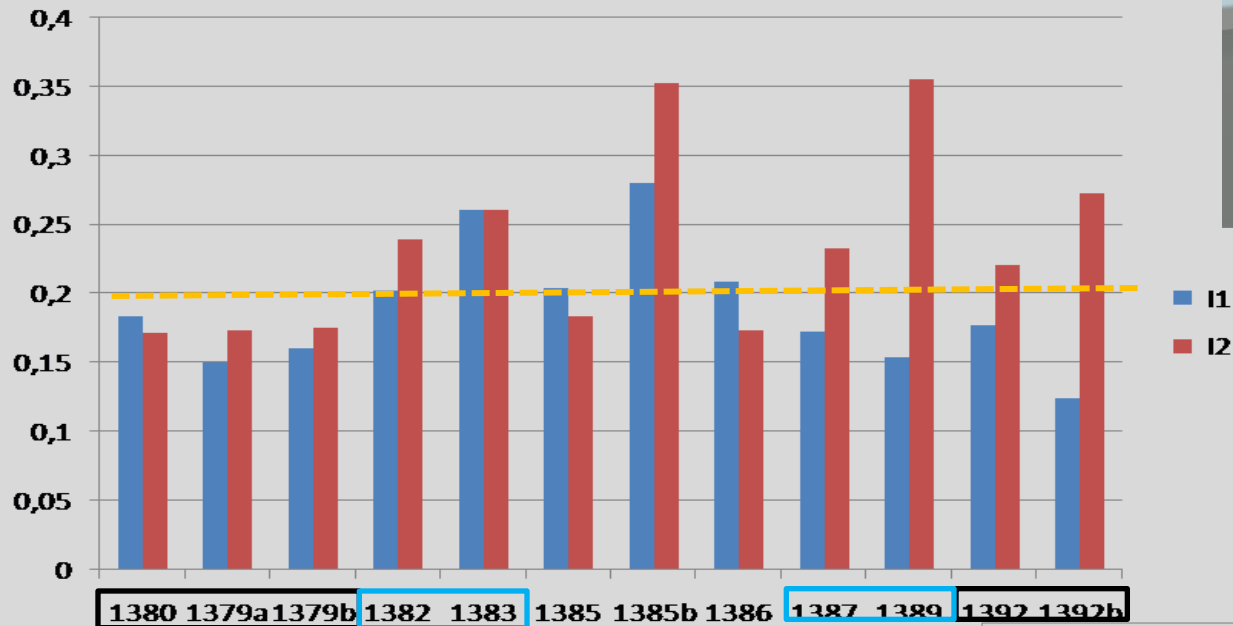1) SUBDIVISION OF THE AGGREGATE INTO 12 STRUCTURAL UNITS (U.S.)

U.S 1392

2) OUTLINE OF THE RESISTANT SYSTEM
  -WALLS
  -NODES

ANALYSIS THROUGH SIMPLIFIED MODELS: VULNUS

I1 :SHEAR STRENGTH OF MASONRY WALLS PARALLEL TO SEISMIC ACTION

I2: OUT-OF-PLANE STRENGTH OF MASONRY WALLS PERPENDICULAR TO SEISMIC ACTION
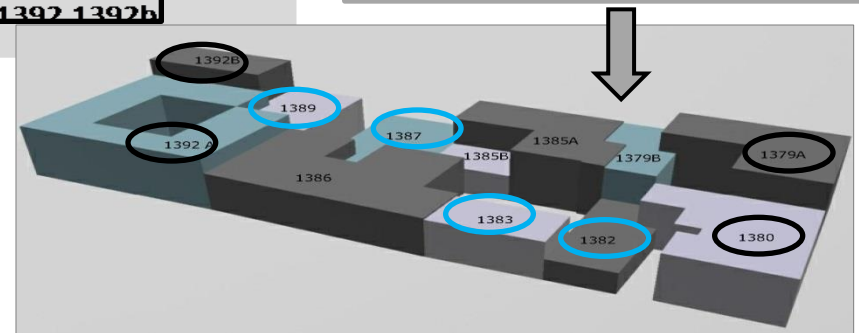

OBJECTIVE
IDENTIFICATION OF MOST VULNERABLE MECHANISMS FOR EACH U.S.

I1<I2

HIGHER IN-PLANE VULNERABILITY

- LOW VALUES OF I1 AND I2 FOR TALL BUILDINGS OR BUILDINGS AT THE ENDS OF THE AGGREGATE

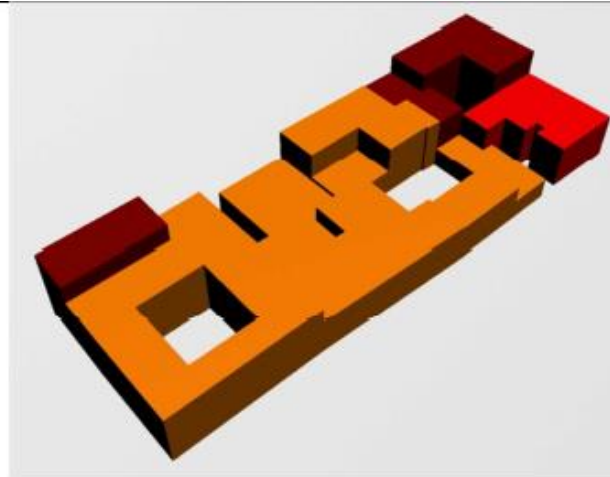- LOW VULNERABILITY FOR SQUAT OR INTERNAL BUILDINGS
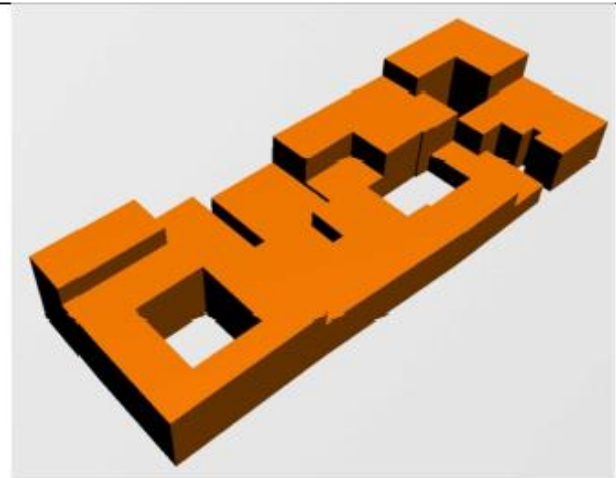
ANALYSIS THROUGH SIMPLIFIED MODELS: VULNUS

According to fuzzy set theory, Vulnus provides linguistic vulnerability evaluations of single buildings and entire blocks, at five different levels
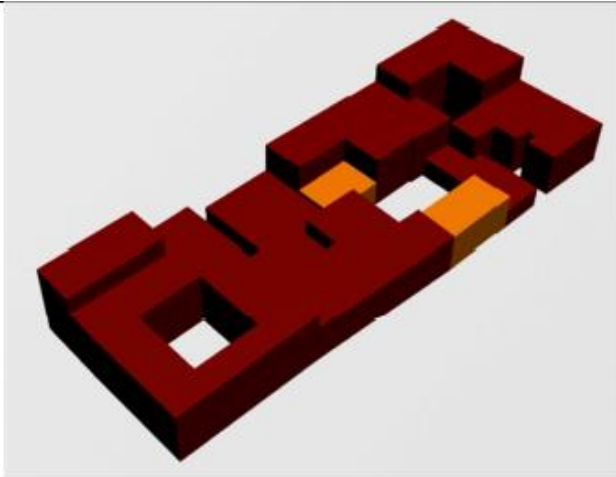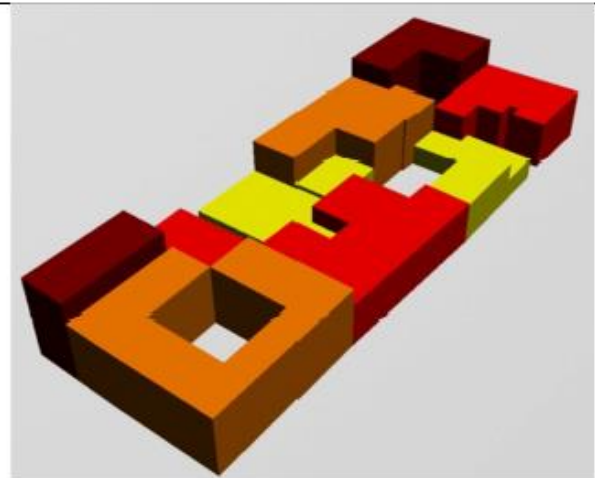
ANALYSIS THROUGH SIMPLIFIED MODELS: VULNUS
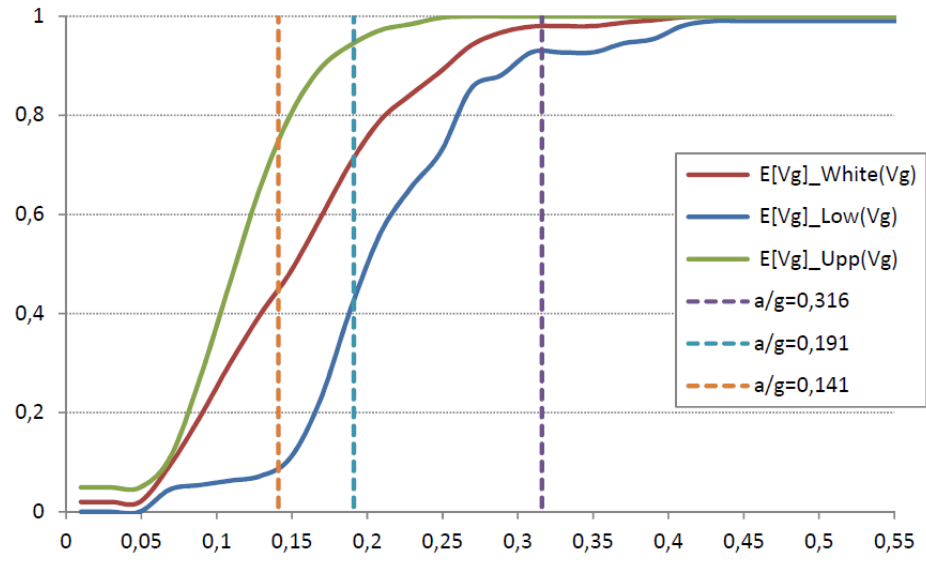

fragility curves obtained by considering all the buildings in the block, which yielded estimated overall vulnerability, and the three considered reference values of PGA/g

Another procedure used by Vulnus to define vulnerability determines the expected values of severe damage E[Vg], depending on the PGA/g ratio of peak ground acceleration and gravity acceleration.

Vulnus provides diagrams (fragility curves) with three curves representing lower, upper and average values, which allow estimation of the expected frequency of serious damage at any value of PGA/g and the relative uncertainty of that value. The area between the upper and lower curves thus represents the range of most probable values of expected frequency of severe damage

VERIFICATION OF THE LOCAL MECHANISMS OF COLLAPSE

GLOBAL OVERTURNING

ANALISI LINEARE								
t [m]	M_S [kNm]	M_R [kNm]	α_0	M^* [t]	e^*	a_0^* [m/s²]	$\overline{a_0^*}$ [m/s²]	$a_0^* \geq \overline{a_0^*}$
0,15	46,24	11072,63	0,004	154,65	0,7495	0,040	1,313	NO

ANALISI NON LINEARE				
θ [rad]	d_{k0}[m]	d_0^*[m]	d_u^*[m]	$\overline{d_u^*}$ [m]	$d_u^* \geq \overline{d_u^*}$ [m]
0,0204	0,1152	0,1536	0,0615	0,2209	NO

NOT VERIFIED
ULTIMATE DISPLACEMENT CAPACITY **27%**

PARTIAL OVERTURNING

ANALISI LINEARE								
t [m]	M_S [kNm]	M_R [kNm]	α_0	M^* [t]	e^*	a_0^* [m/s²]	$\overline{a_0^*}$ [m/s²]	$a_0^* \geq \overline{a_0^*}$
0,11	342,91	5146,99	0,067	96,18	0,8217	0,589	1,31	NO

ANALISI NON LINEARE				
θ [rad]	d_{k0}[m]	d_0^*[m]	d_u^*[m]	$\overline{d_u^*}$ [m]	$d_u^* \geq \overline{d_u^*}$ [m]
0,067	0,300	0,365	0,1461	0,1804	NO

NOT VERIFIED
ULTIMATE DISPLACEMENT CAPACITY **80%**

ANALISI LINEARE								
t [m]	M_S [kNm]	M_R [kNm]	α_0	M^* [t]	e^*	a_0^* [m/s²]	$\overline{a_0^*}$ [m/s²]	$a_0^* \geq \overline{a_0^*}$
0,041	179,54	1080,87	0,166	48,04	0,947	1,274	2,64	NO

ANALISI NON LINEARE				
θ [rad]	d_{k0}[m]	d_0^*[m]	d_u^*[m]	$\overline{d_u^*}$ [m]	$d_u^* \geq \overline{d_u^*}$ [m]
0,1648	0,3570	0,3764	0,1506	0,1245	SI

HIGH HORIZONTAL THRUSTS DUE TO THRUSTING ELEMENTS

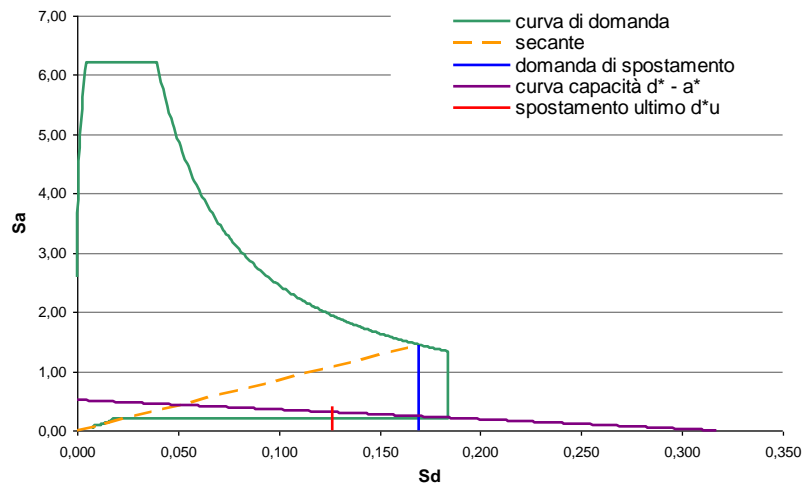**VERIFIED**
ULTIMATE DISPLACEMENT CAPACITY **120%**
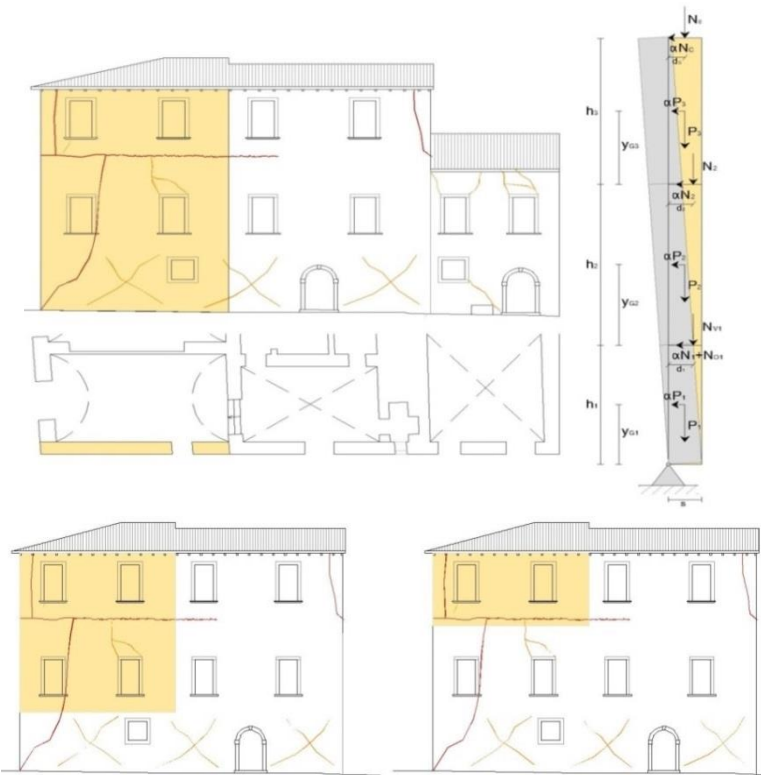
ANALYSIS THROUGH SIMPLIFIED MODELS: VULNUS

B.2 Schede per il calcolo del coefficiente di attivazione di tipici meccanismi cinematici

B.2.1 Ribaltamento di parete ad 1 piano monolitica

Descrizione del meccanismo: Il meccanismo si manifesta attraverso la rotazione rigida di intere facciate o porzioni di pareti rispetto ad assi in prevalenza orizzontali alla base di esse e che percorrono la struttura muraria sollecitata da azioni fuori dal piano. Si consideri l'azione sismica come forza statica equivalente data dal prodotto della massa della parete per l'accelerazione sismica, ipotizzata costante lungo l'altezza della parete. Questa sarà allora soggetta all'azione di un momento ribaltante (dato dall'azione sismica applicata a livello del baricentro della muratura in questione per il braccio pari ad d_1) che tenderà a farla ruotare attorno alla cerniera cilindrica che si forma alla sua base (punto A). A contrastare tale azione vi sarà un momento di segno opposto generato dalla forza peso della parete e dei carichi che su di essa gravano, e dall'azione di eventuali tiranti, diminuita dalla spinta di eventuali volte. Si determina quindi il coefficiente moltiplicativo dei pesi per cui si ha l'attivazione del meccanismo, ovvero il coefficiente c, attraverso l'imposizione delle condizioni di equilibrio del corpo alla rotazione.

- curva di domanda
- secante
- domanda di spostamento
- curva capacità d* - a*
- spostamento ultimo d*u

Schema meccanismo
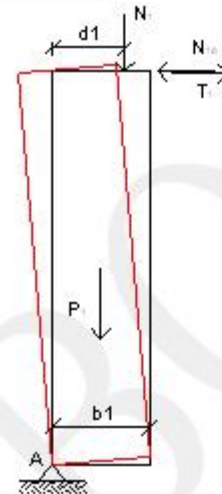
Immagine rappresentativa

Variabili

Variabili	
P_i	Peso proprio parete
h_i	Altezza
b_i	Spessore medio
N_i	Carico agente sulla parete
d_i	Braccio del carico
N_{io}	Componente orizzontale spinta volta
T_i	Tensione nel tirante

Calcolo coefficiente di attivazione

$$c = \frac{P_i \frac{b_i}{2} + N_i d_i + (T_i - N_{io})h_i}{P_i \frac{h_i}{2} + N_i h_i}$$

Altro – Calcolo della tensione nel tirante

$$T_i = a \cdot \left(\frac{P_i}{N_i} + N_i\right) - \left[\frac{P_i \frac{b_i}{2} + N_i d_i}{h_i}\right]$$

Claudio Modena

Analysis Methods for Unreinforced Heritage Masonry
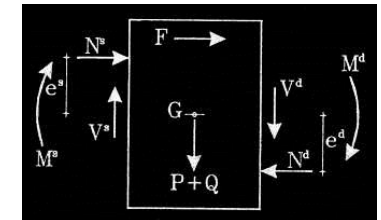
GLOBAL MECHANIMS: IN-PLANE VERIFICATIONS

The strength and stiffness of floors in each direction must be evaluated and taken into account in the model.

FLEXIBLE FLOORS: analysis of single walls
and alignment of walls

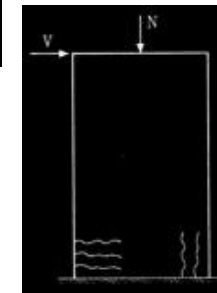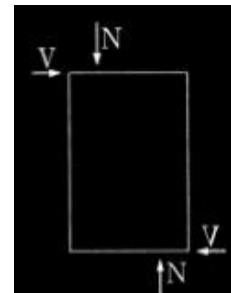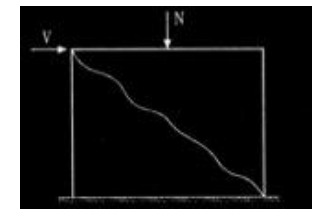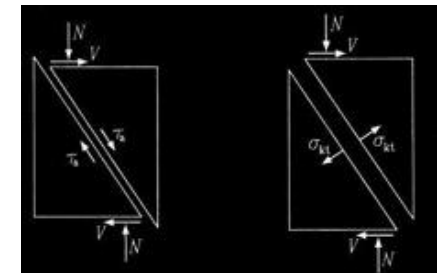IDENTIFICATION OF WALLS AND WALL ALIGNMENTS
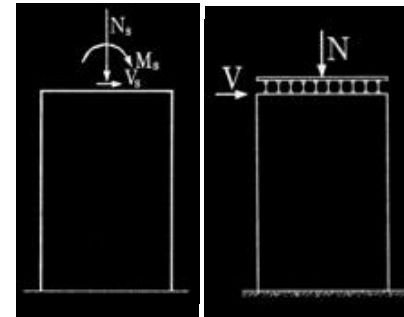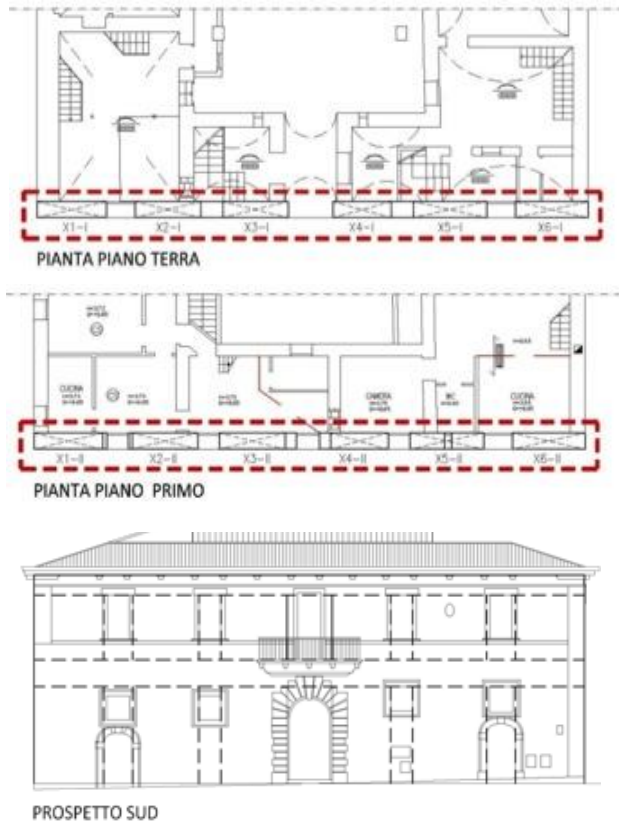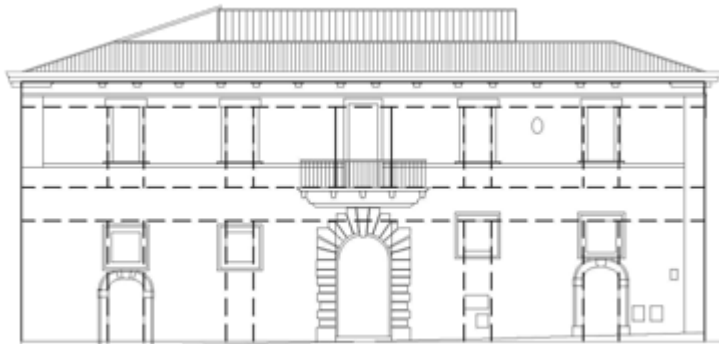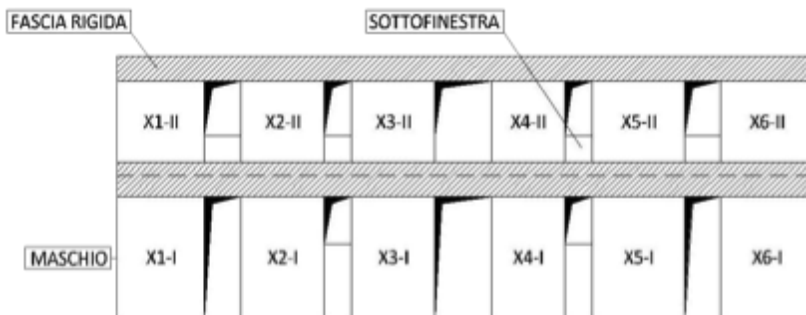

piers

spandrels

Flexure

Shear

GLOBAL MECHANIMS: IN-PLANE VERIFICATIONS

LINEAR STATIC ANALISYS

PROSPETTO SUD

VERIFICA LINEARE LIVELLO I

SETTO	$V_{Sd,x}$	$M_{Sd,x}$	V_t	$\eta=V_t/V_{Sd}$	$V_t>V_{xl}$	M_u	$\eta=M_u/M_{Sd}$	$M_u>M_{Sd}$
	kN	kNm	kN			kNm		
X1	165,65	430,70	82,05	0,50	FALSO	137,00	0,32	FALSO
X2	152,41	396,26	77,66	0,51	FALSO	69,76	0,18	FALSO
X3	151,53	393,97	67,69	0,45	FALSO	196,16	0,50	FALSO
X4	121,26	315,27	61,68	0,51	FALSO	126,31	0,40	FALSO
X5	183,47	477,03	89,80	0,49	FALSO	194,68	0,41	FALSO
X6	180,79	470,06	84,74	0,47	FALSO	228,76	0,49	FALSO
TOTALE	955,11		463,63	0,49	FALSO			

VERIFICA LINEARE LIVELLO II

SETTO	$V_{Sd,x}$	$M_{Sd,x}$	V_t	$\eta=V_t/V_{Sd}$	$V_t>V_{xl}$	M_u	$\eta=M_u/M_{Sd}$	$M_u>M_{Sd}$
	kN	kNm	kN			kNm		
X1	102,76	223,50	72,71	0,71	FALSO	224,56	1,00	VERO
X2	96,01	208,83	72,72	0,76	FALSO	214,94	1,03	VERO
X3	95,56	207,85	70,72	0,74	FALSO	208,12	1,00	VERO
X4	79,77	173,50	61,36	0,77	FALSO	176,03	1,01	VERO
X5	111,72	243,00	89,12	0,80	FALSO	287,30	1,18	VERO
X6	110,38	240,08	85,17	0,77	FALSO	276,06	1,15	VERO
TOTALE	592,21		451,81	0,76	FALSO			

Flexure

$$M_u = \frac{l^2 t\, \sigma_0}{2}\left(1 - \frac{\sigma_0}{0.85 \cdot f_d}\right)$$

Shear

$$V_t = l \cdot t \cdot \frac{1.5\tau_{0_d}}{b} \cdot \sqrt{1 + \frac{\sigma_0}{1.5 \cdot \tau_{0_d}}}$$

GLOBAL MECHANIMS: IN-PLANE VERIFICATIONS

STATIC NON LINEAR ANALISYS

MACROMODEL WITH RIGID SPANDRELS
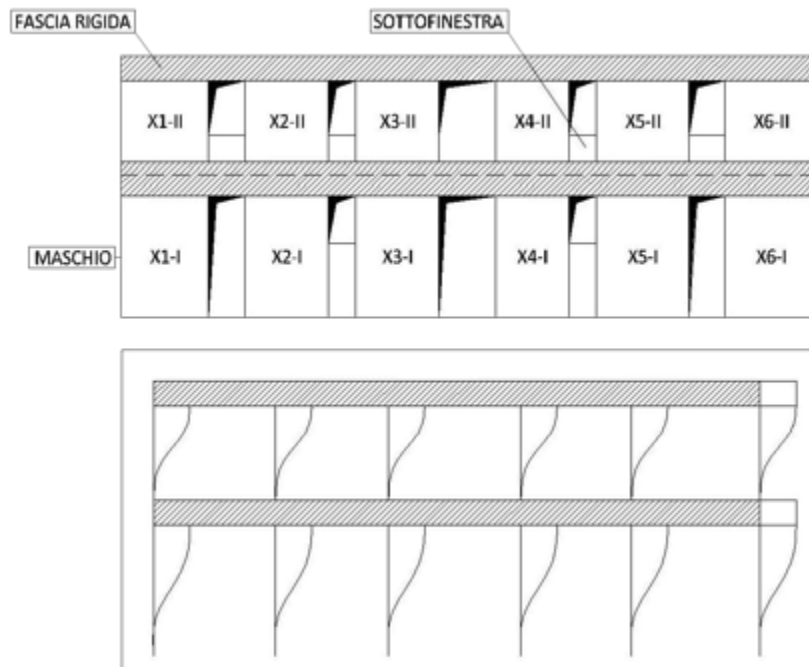
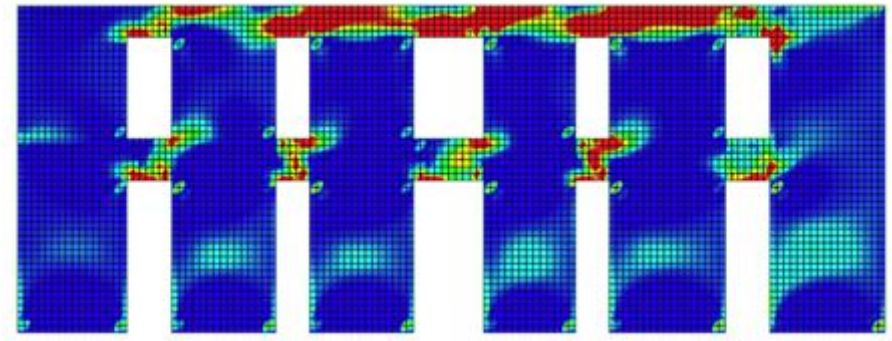MACROMODEL WITH DEFORMABLE SPANDRELS
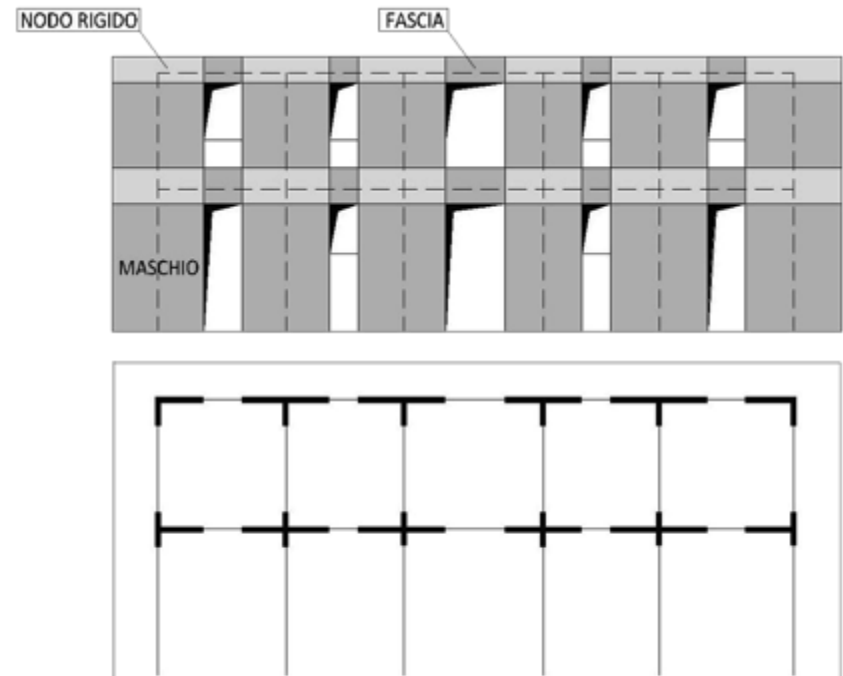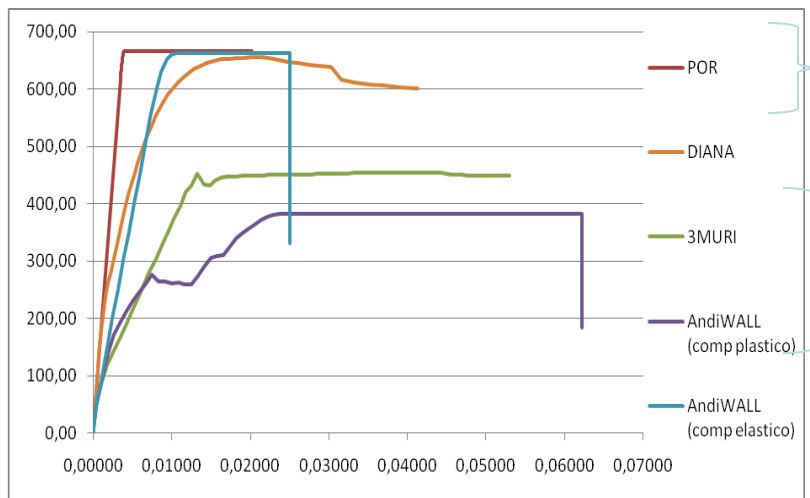
GLOBAL MECHANIMS: IN-PLANE VERIFICATIONS

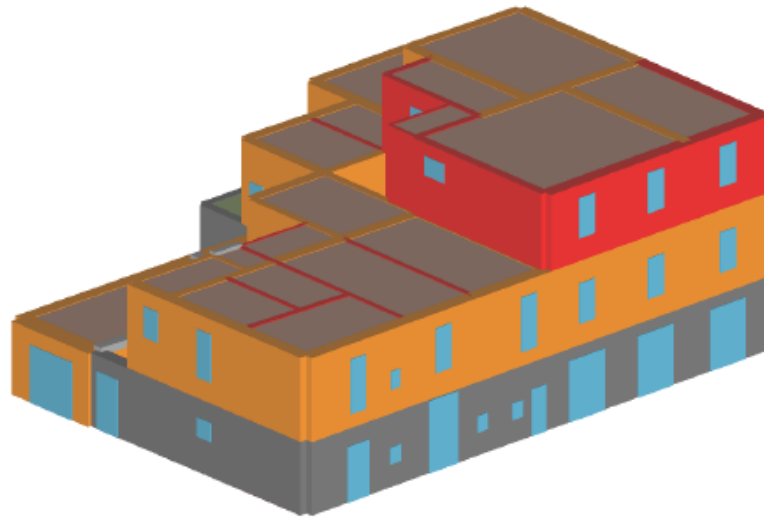## MACROMODEL WITH DEFORMABLE SPANDRELS
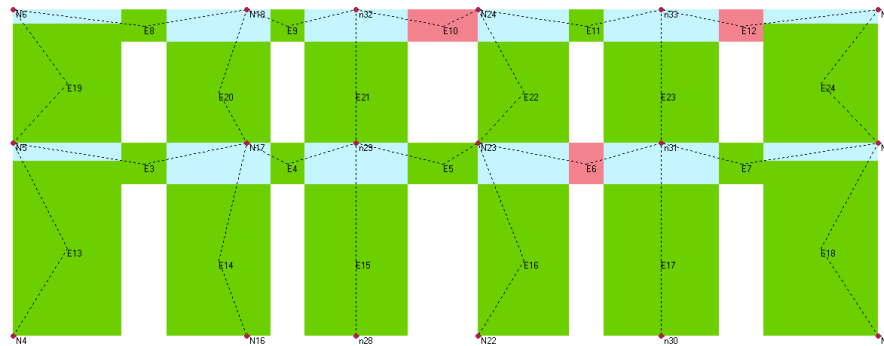
FINITE ELEMENT MODEL

Curves (in terms of maximum wall strength) obtained under the assumption of the infinitely rigid (POR method) or infinitely linear elastic behavior of the spandrel;
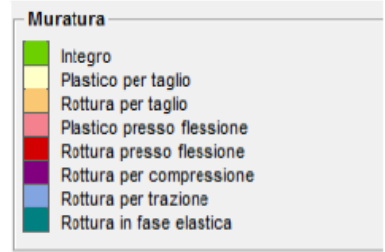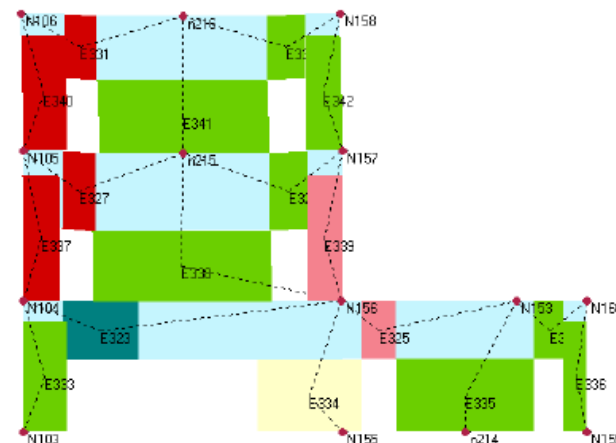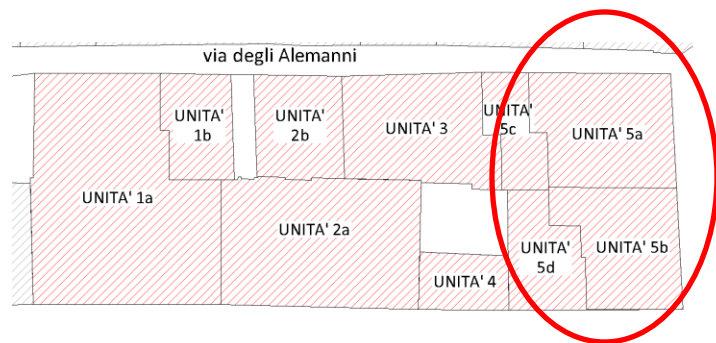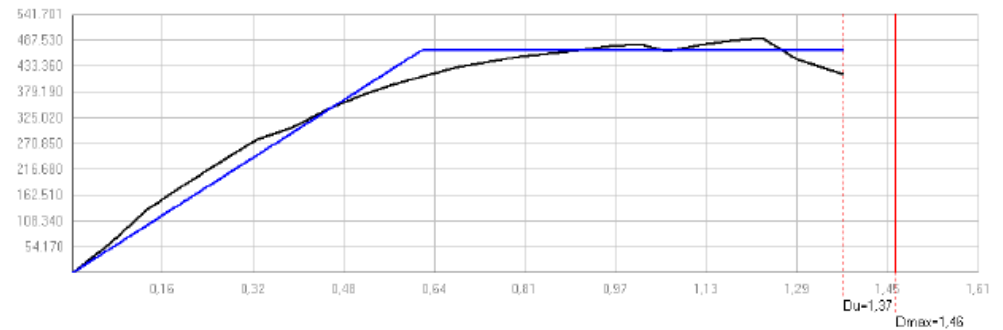
curves presume possible plasticization of spandrel walls. Indeed, the main differences are

GLOBAL MECHANIMS: IN-PLANE VERIFICATIONS

Pre-interventions results

GLOBAL MECHANIMS: IN-PLANE VERIFICATIONS

Post-interventions results

GLOBAL MECHANIMS: IN-PLANE VERIFICATIONS

Pre-interventions results

GLOBAL MECHANIMS: IN-PLANE VERIFICATIONS
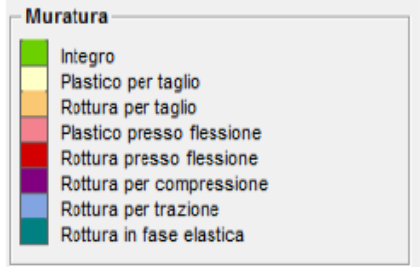
Post-interventions results

National Commission for Culture and the Arts
National Museum, Philippines
Bakas Pilipinas
ICOMOS Philippines
University of Santo Tomas - Center for Conservation of
Cultural Property and Environment in the Tropics

JANUARY 13 – 14, 2016

Thank you for your attention!

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