METHODS AND OBJECTIVES OF THE STRUCTURAL ASSESSMENT OF HISTORIC CONSTRUCTIONS GENERAL ASPECTS AND CASE STUDIES

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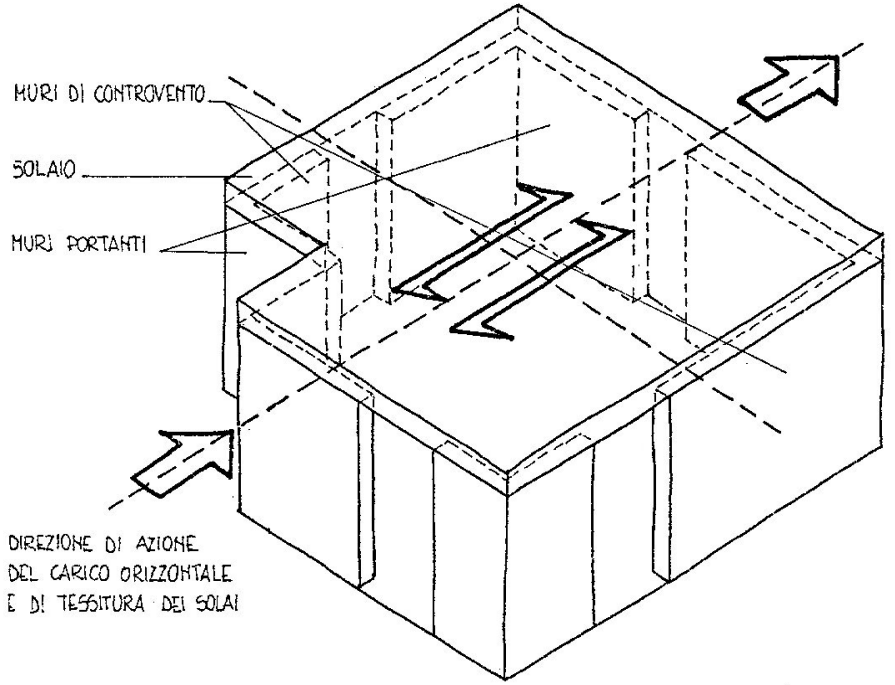
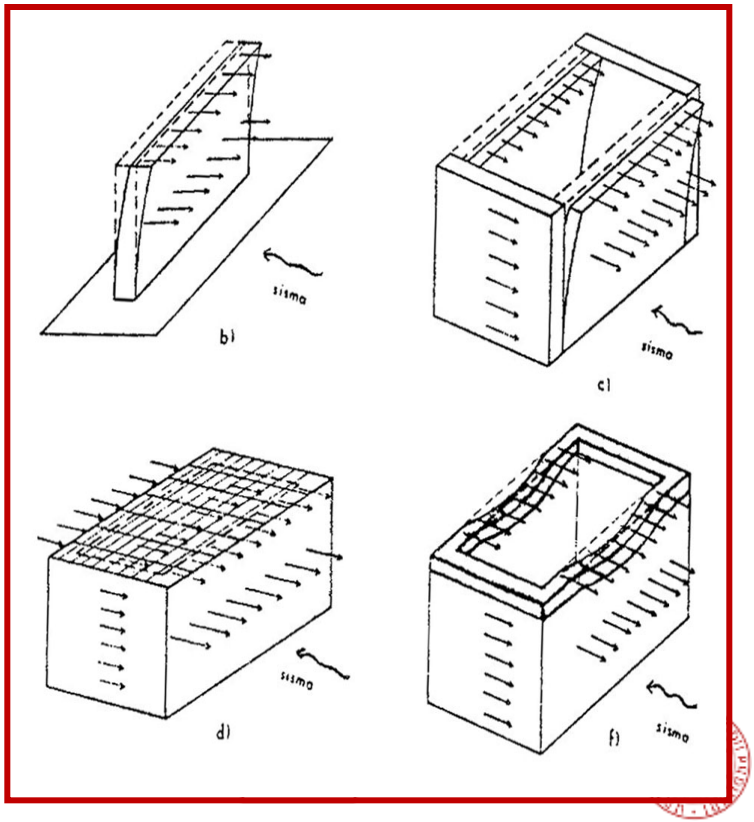
SEISMIC BEHAVIOR OF MASONRY BUILDINGS



- Disconnected walls (isolated wall)
- High stiffness floor
- Link wall-wall and wall-floor with tie-beam



BOX BEHAVIOR:

Horizontal forces absorbed by the walls in their plane

- Sufficiently rigid deck
- Adequate connection between walls
- Link wall-floor and wall-roof

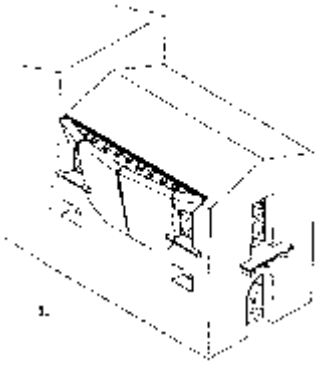
TEST SU TAVOLA VIBRANTE

(ZAG Slovenia, M. Tomazevic)

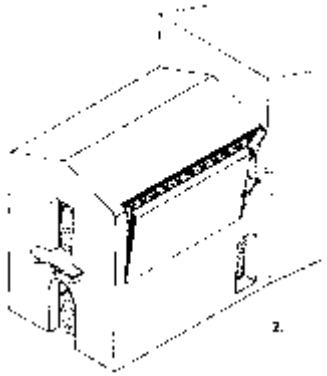
Provided that adequate masonry quality is ensured - in case using appropriate repair/strengthening techniques –**most of the stability problems are connected to equilibrium considerations**.

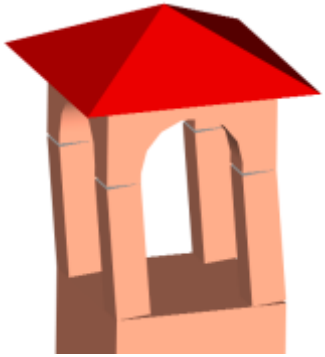
Main causes:

- Lack of connection between walls
- Lack of connection between walls and floors
- Reduced in plane stiffness of floors
- Masonry composition
- Existing crack pattern



Giuffrè, 1993

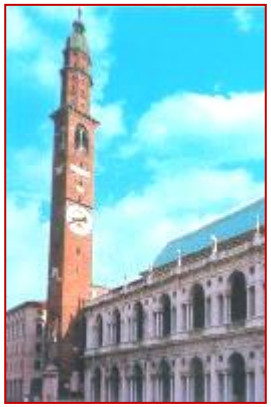




Salò-Garda lake earthquake (24/11/2004)

Case studies

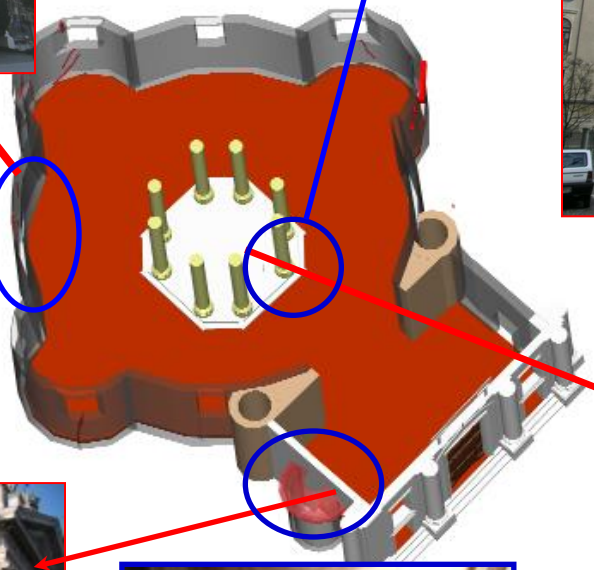




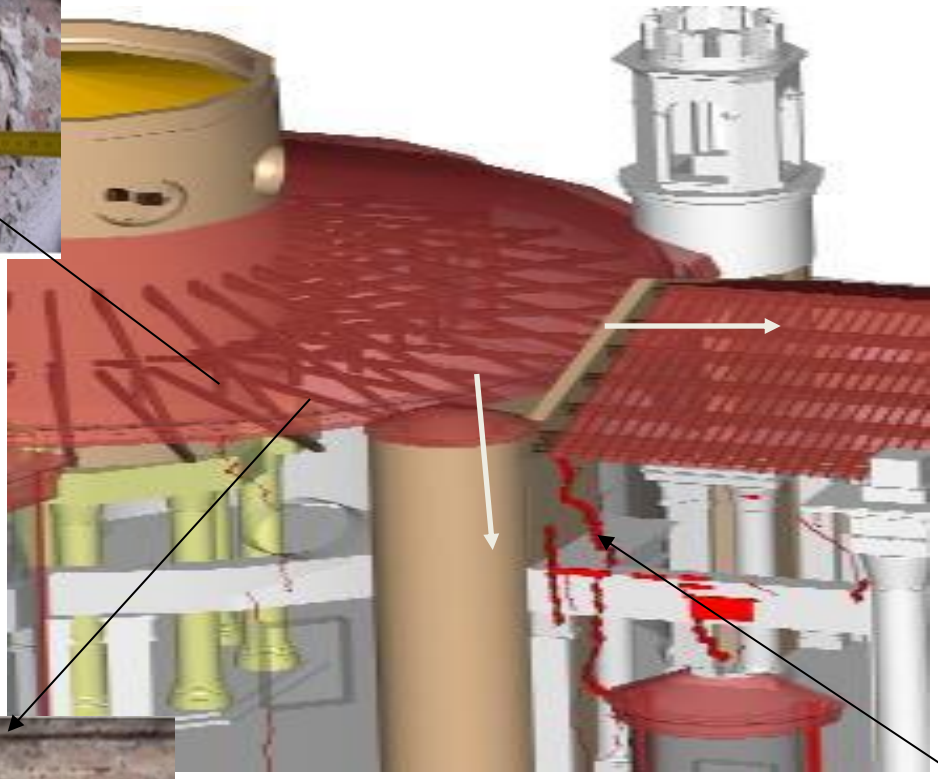

THE CHURCH OF 'MADONNA DEL PIANTO' (Padova)

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

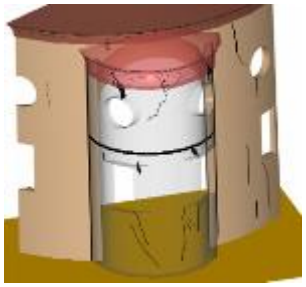
CURRENT CONDITION



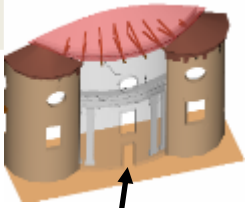
CRITICAL SURVEY



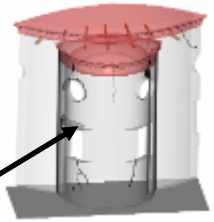
MACROELEMENTS



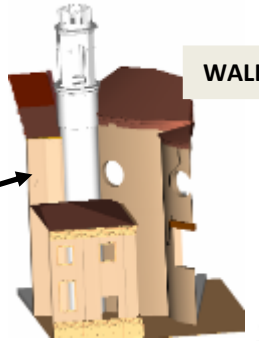
SOUTHERN APSE



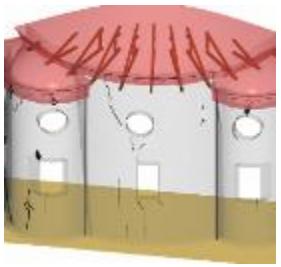
WALL C



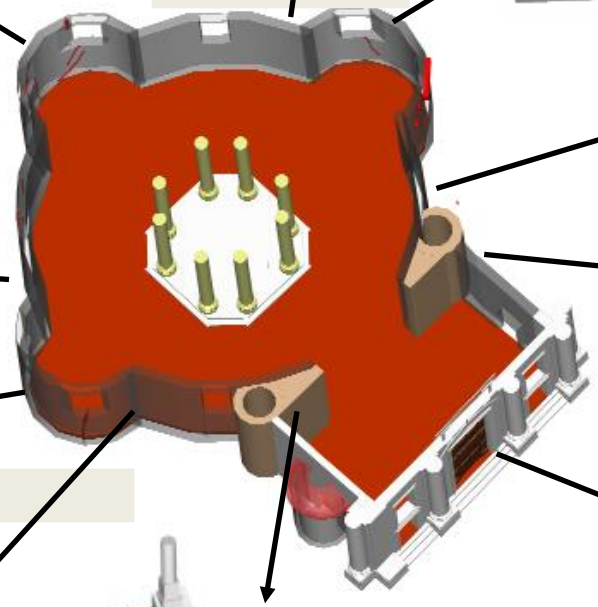
WESTERN APSE

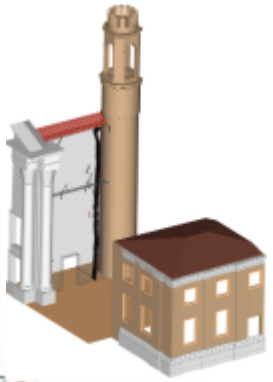


WALL B



WALL D



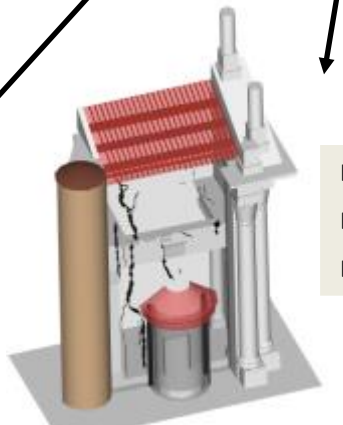


LATERAL WALL A
WESTERN BELL TOWER
SACRISTY

EASTERN APSE

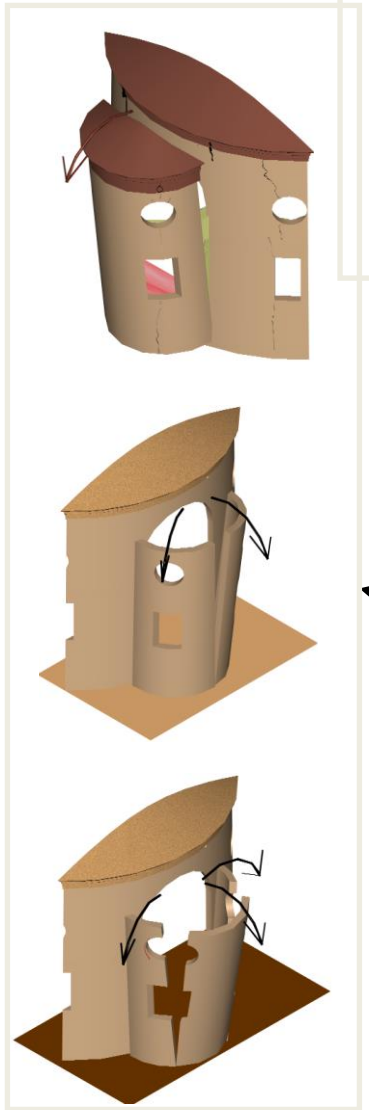
FAÇADE

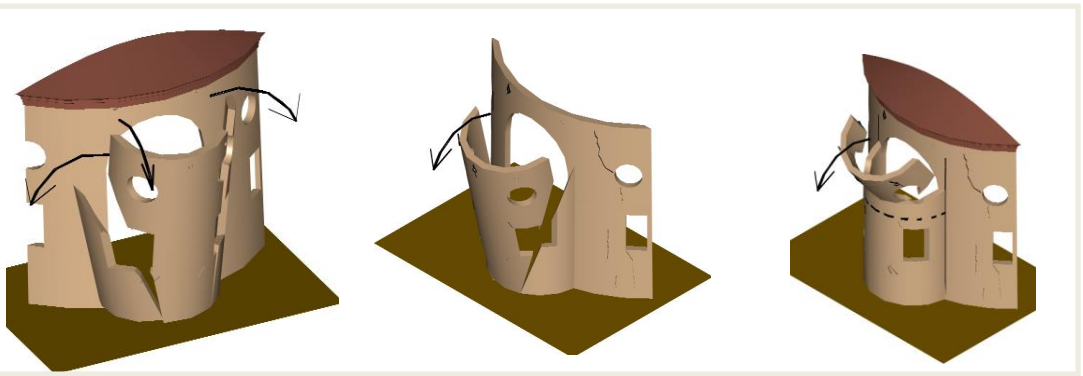
WALL E

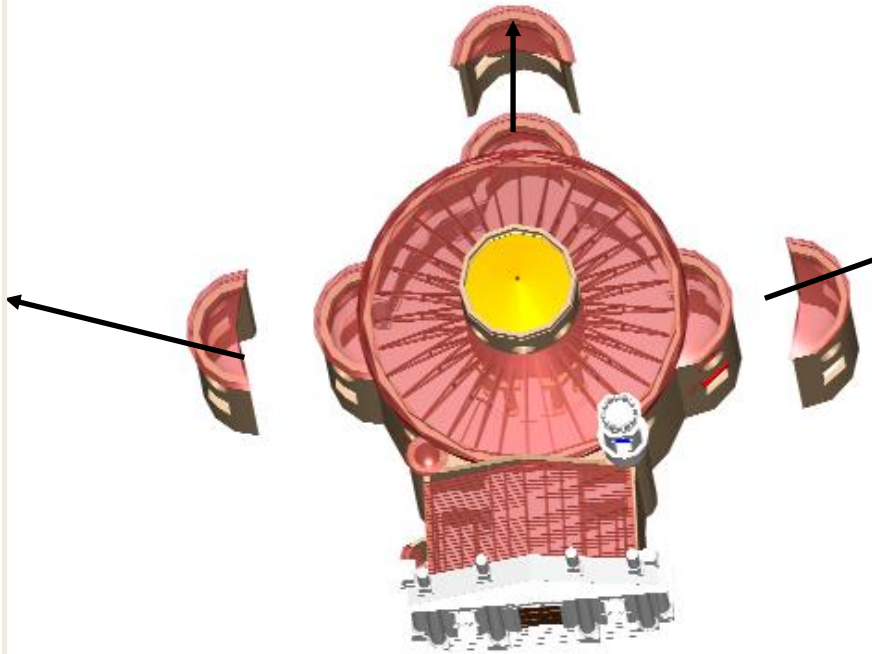


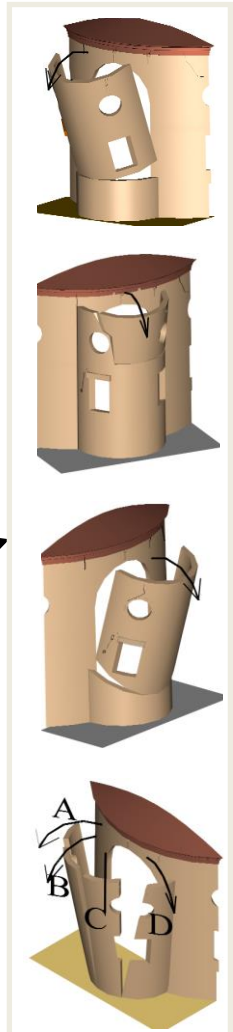
LATERAL WALL F
EASTERN BELL TOWER
BAPTISMAL FONT

LATERAL CHAPELS



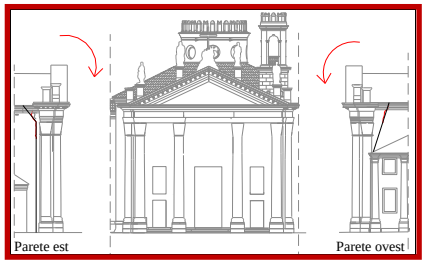






POSSIBLE AND PROBABLE COLLAPSE MECHANISMS

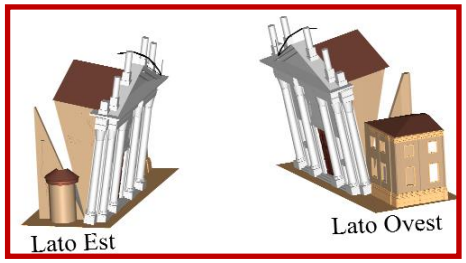
FAÇADE

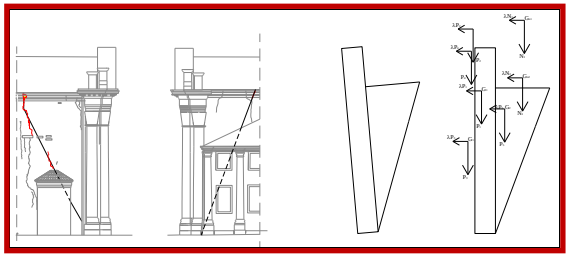


M_{st1} = 52230 daNm

M_{inst1} = 3984900 daNm

λ_1=0,0131

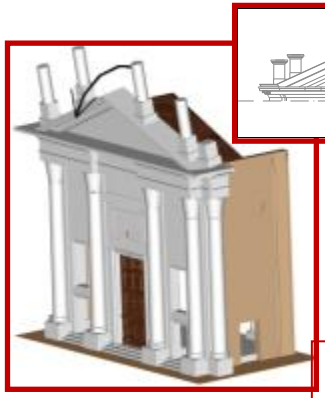


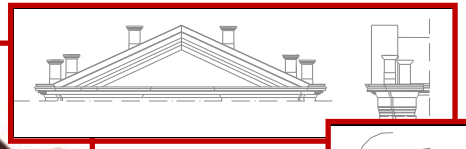


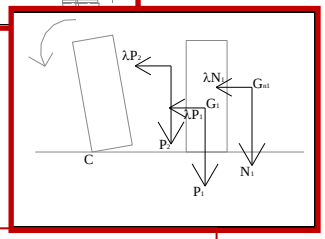
M_{st1} = 236220daNm

M_{inst1} =4441250daNm

λ1=0,05319



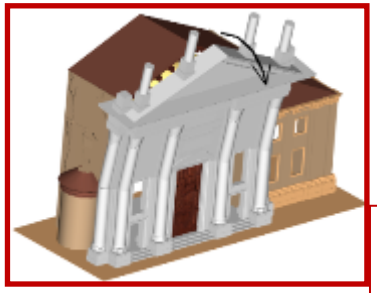


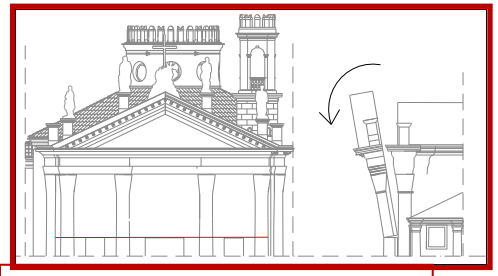


M_{st1} = 41137daNm

M_{inst1} =1473034daNm

λ_1=0,0279





M_{st1} = 2125 daNm

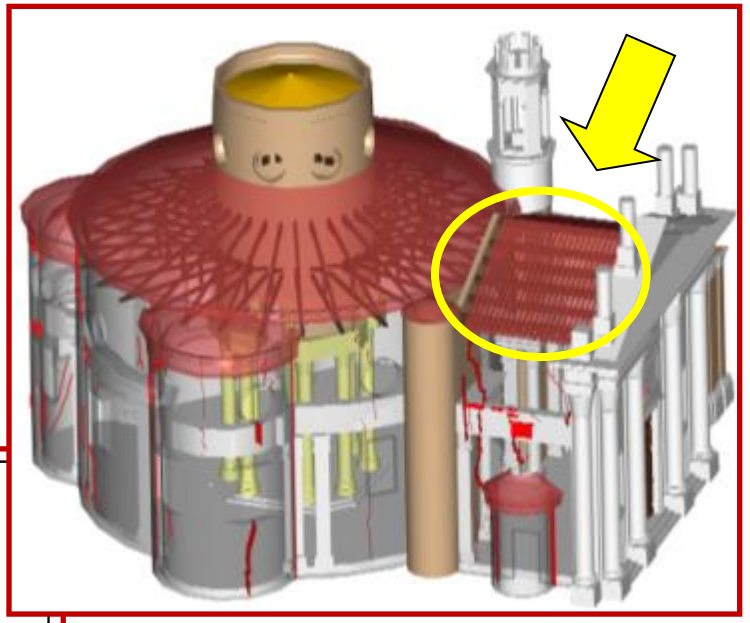
M_{inst1} =148300 daNm

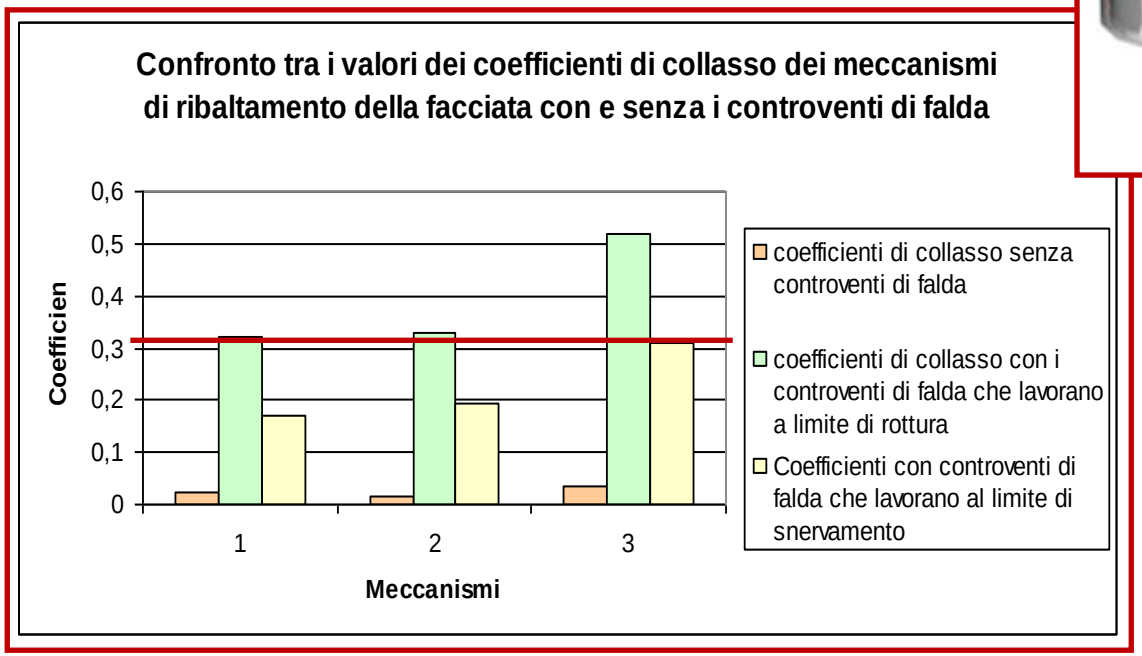
λ_1=0,0143

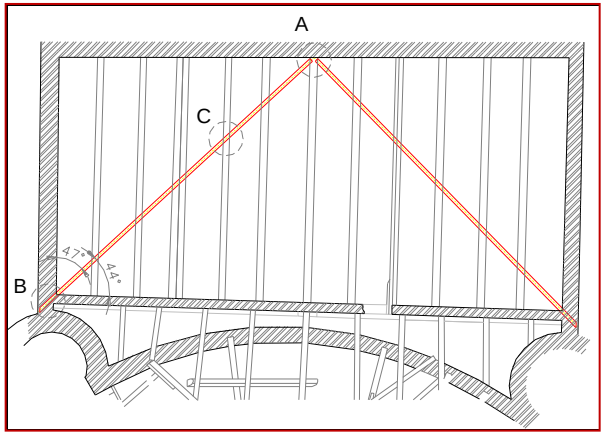
INTERVENTIONS

ROOF BRACING

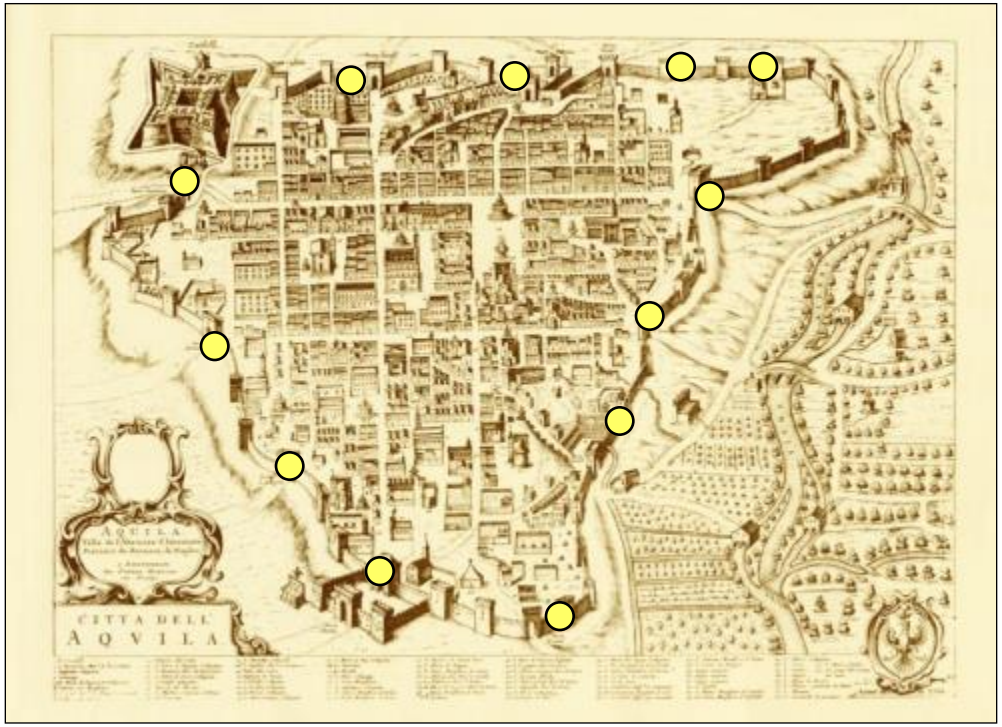
Between 2003 and 2004 some interventions were carried out. Bracings were installed on the roof above the façade. This has significantly improved the seismic response of this element, as shown by the increase of the collapse coefficient for the overturning of the façade.



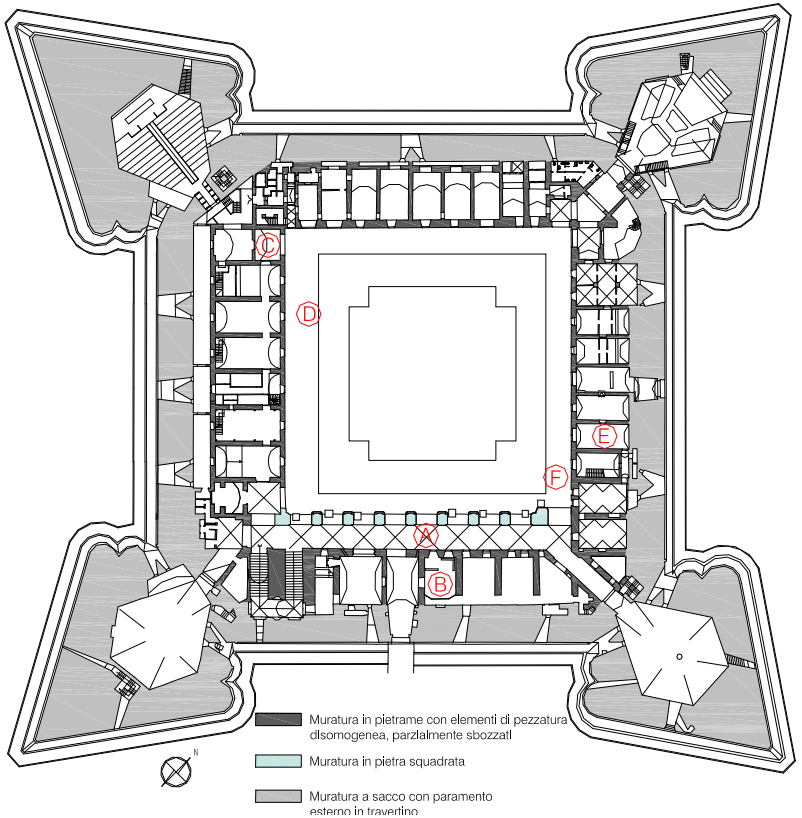




SPANISH FORTRESS – L'AQUILA



- Porta Paganica
- Porta Rivera
- Porta Leoni
- Porta Brinconia
- Porta Roiana
- Porta Castello
- Porta S.Lorenzo
- Porta di Bagno
- Porta Barete
- Porta Tione
- Porta Romana
- Porta Bazzano



Muratura in pietrame con elementi di pezzatura disomogenea, parzialmente sbozzati

Muratura in pietra squadrata

Muratura a sacco con paramento esterno in travertino

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DANNI PRINCIPALI:

- ribaltamento fuori-piano e meccanismi flessionali nelle pareti;
- rotture a taglio nei muri;
- danni ad archi e volte;
- crolli locali di solai e volte.

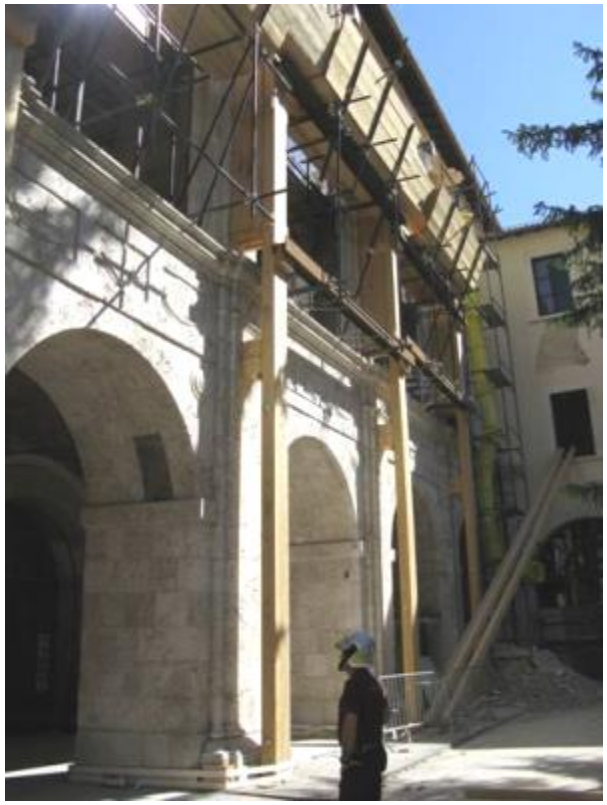
Crollo parziale della sommità della parete laterale e della copertura

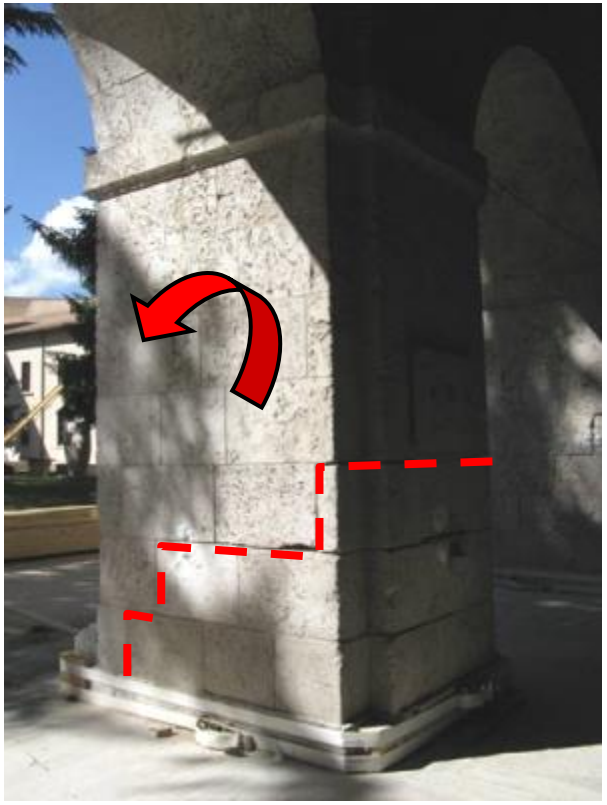
Cattiva qualità della muratura e cattivo ammorsamento tra pareti. Rivestimento esterno in travertino.

Presenza di cordoli in c.a. e solai in latero-cemento, hanno portato ad un appesantimento ed martellamento della muratura

Caratteristiche scadenti delle murature

Ribaltamento del loggiato e dei piloni del piano terra

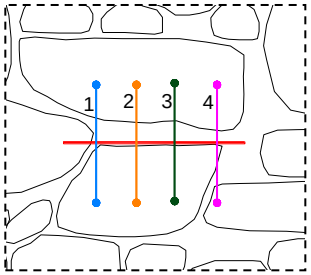




Il porticato a doppia loggia.

Individuazione della lesione.

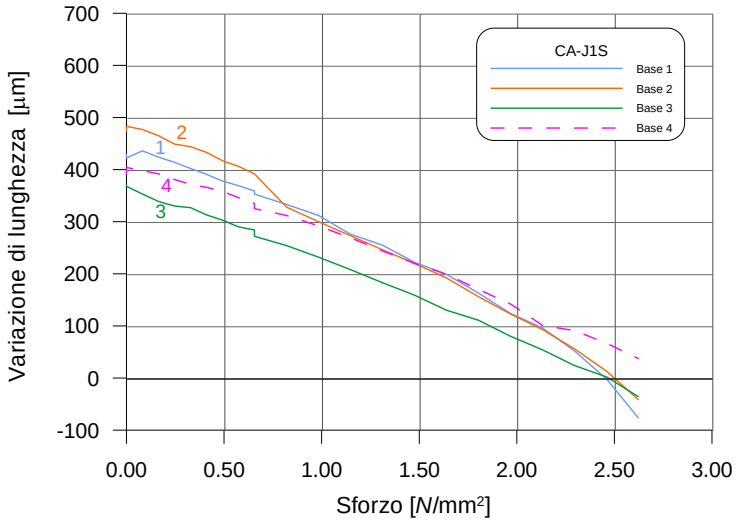

CAPOCHIAVE ESTERNI

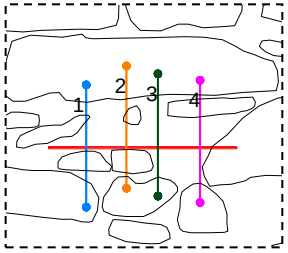


Point 1

State of stress

2.47 *N*/mm²

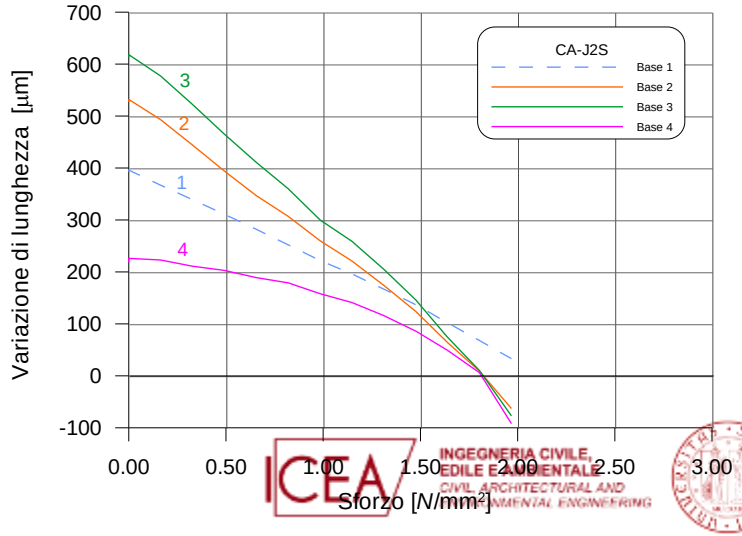




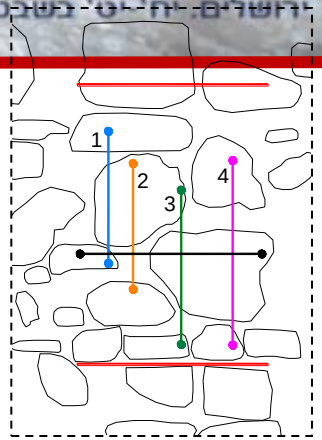
Point 2

State of stress

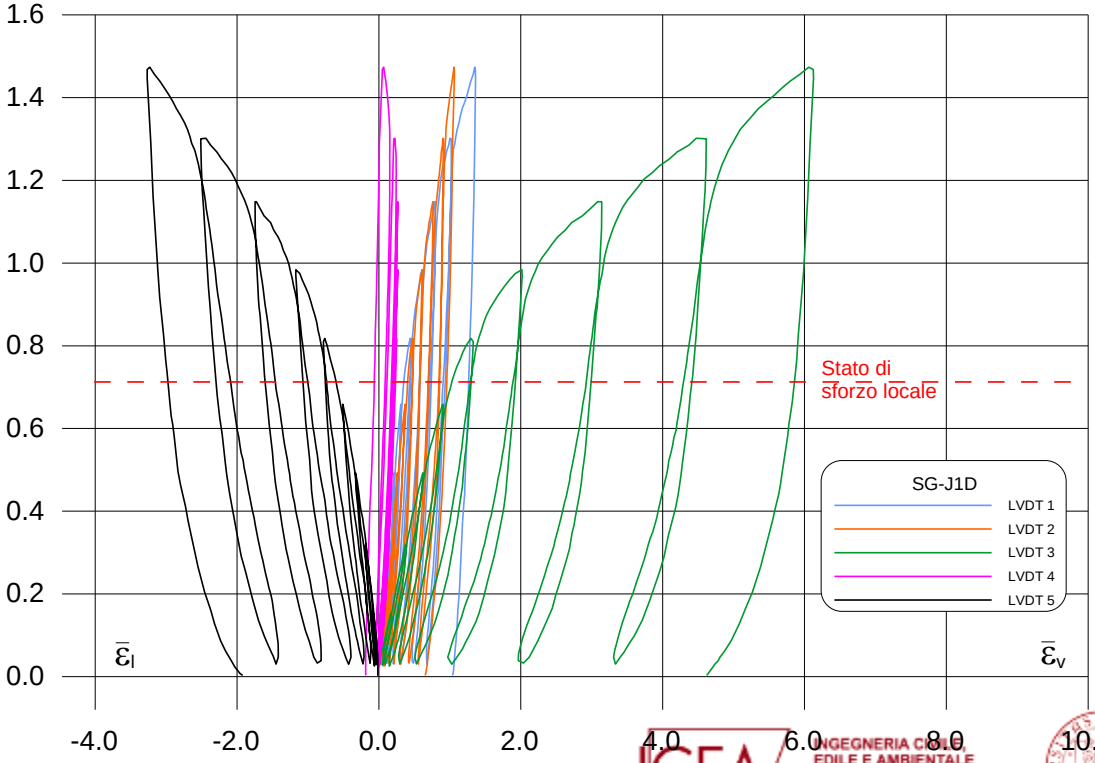
1.82 *N*/mm²



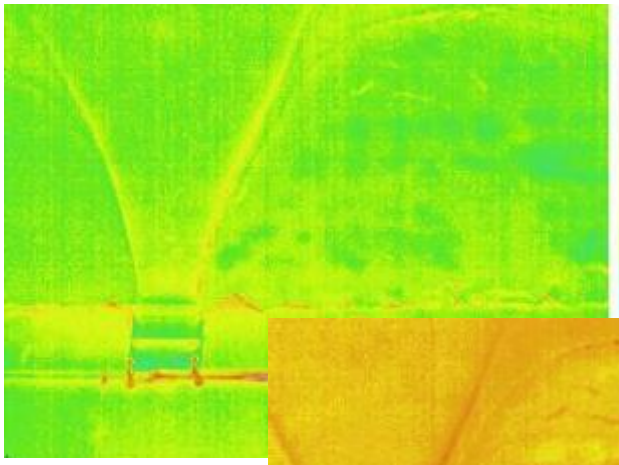
MARTINETTI PIATTI DOPPI



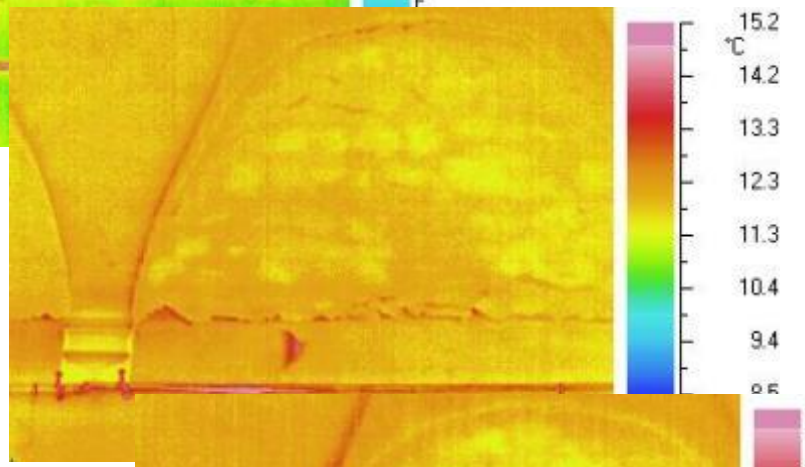
Martinetto piatto doppio – CA-J3D

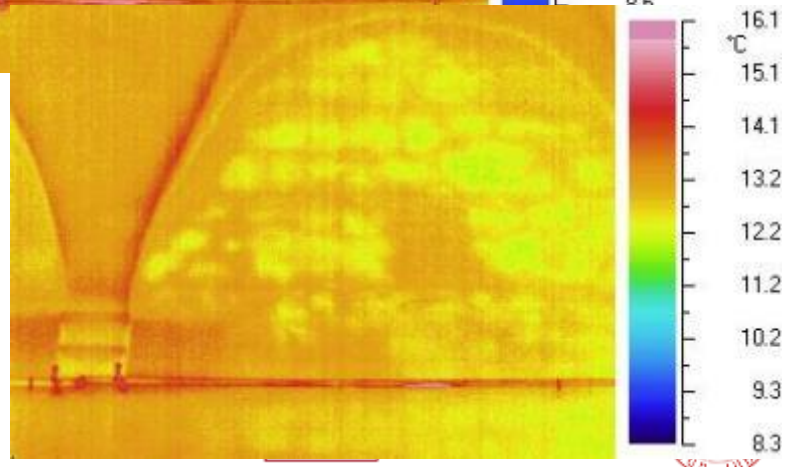


TERMOGRAFIE



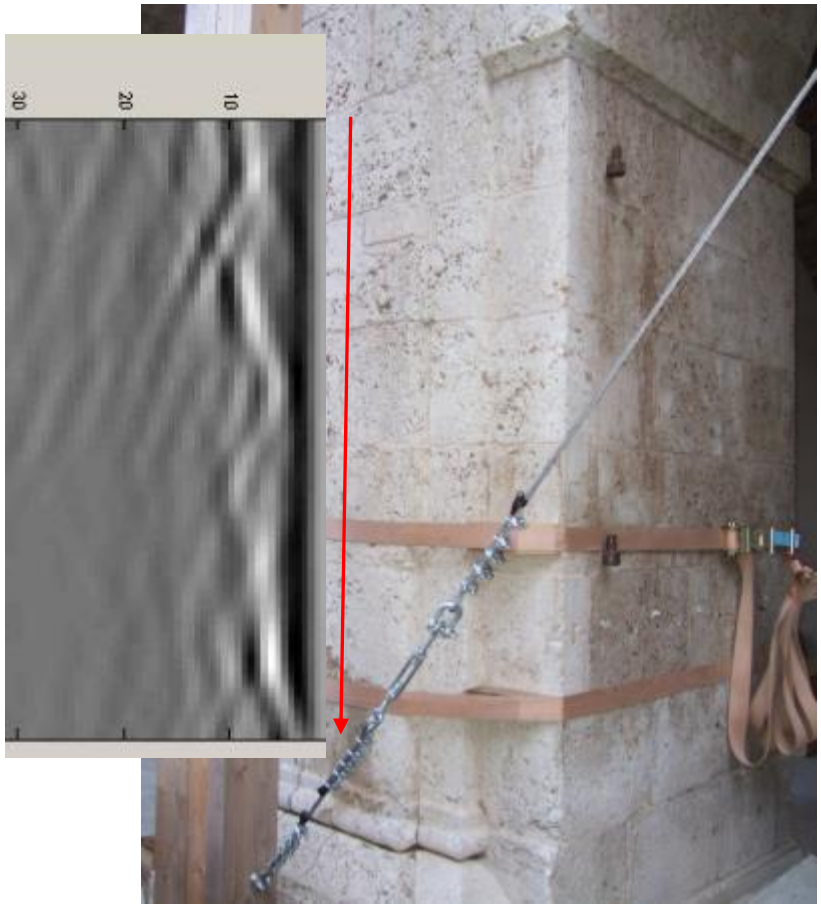
CA-T2





RADAR

Profilo su pilastro 1 con antenna da 200MHz

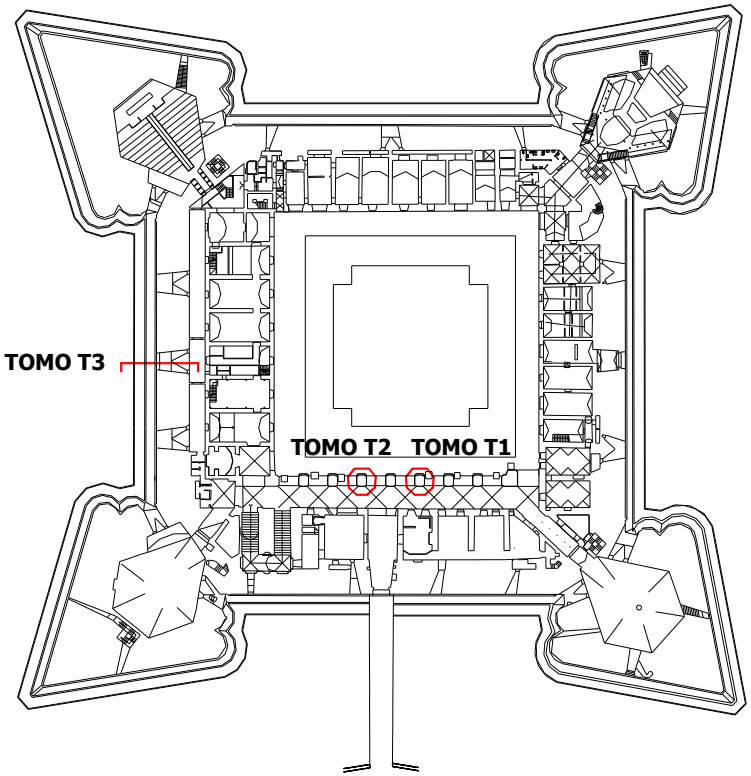


Sono state eseguite prove radar sui pilastri danneggiati per studiare la morfologia della sezione. Si nota il diverso spessore delle pietre.

Si stanno ancora elaborando I risultati

TOMOGRAFIE SONICHE

Le prove soniche sono state eseguite su due pilastri della fortezza: uno danneggiato dal terremoto, l'altro invece intatto.



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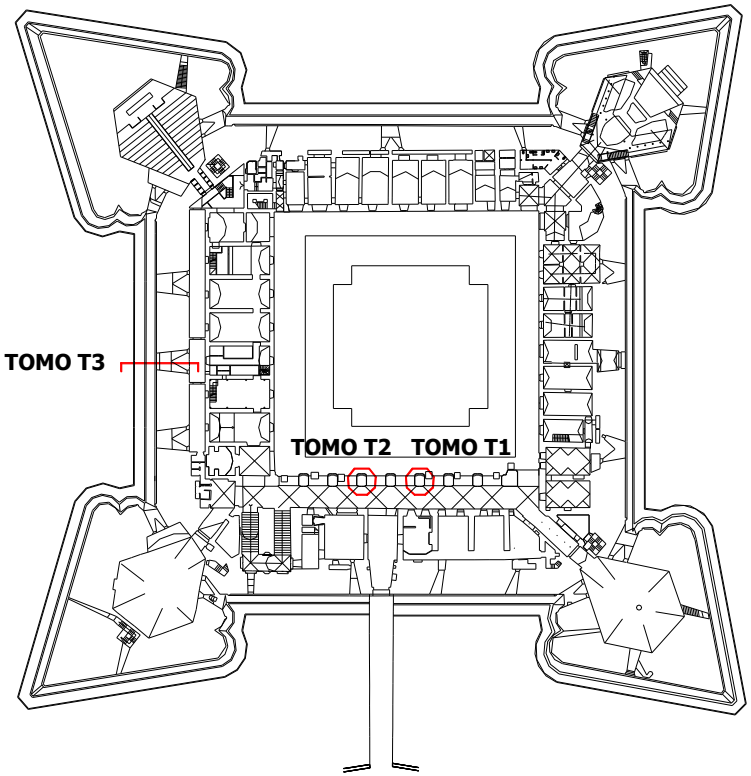
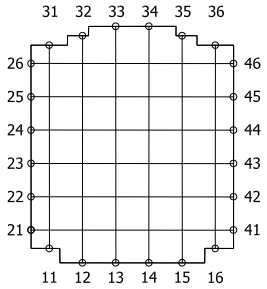
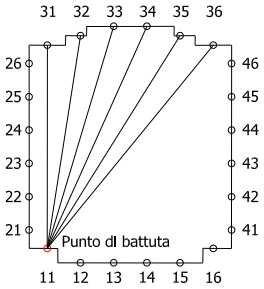




a)



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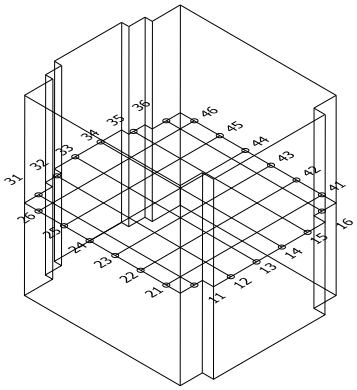
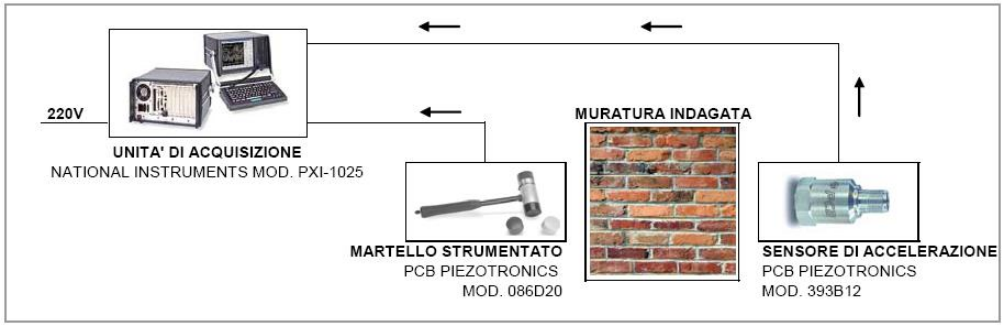
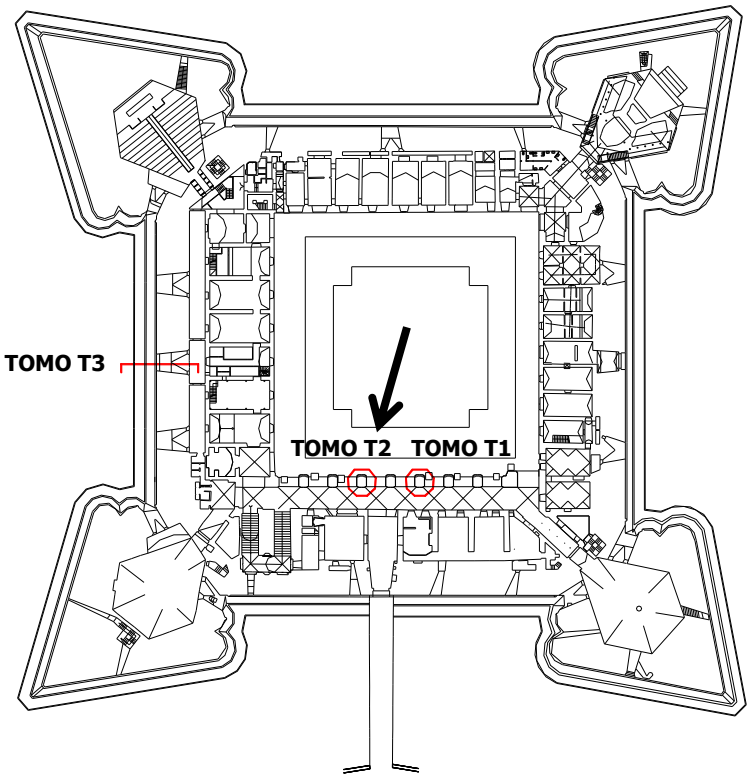


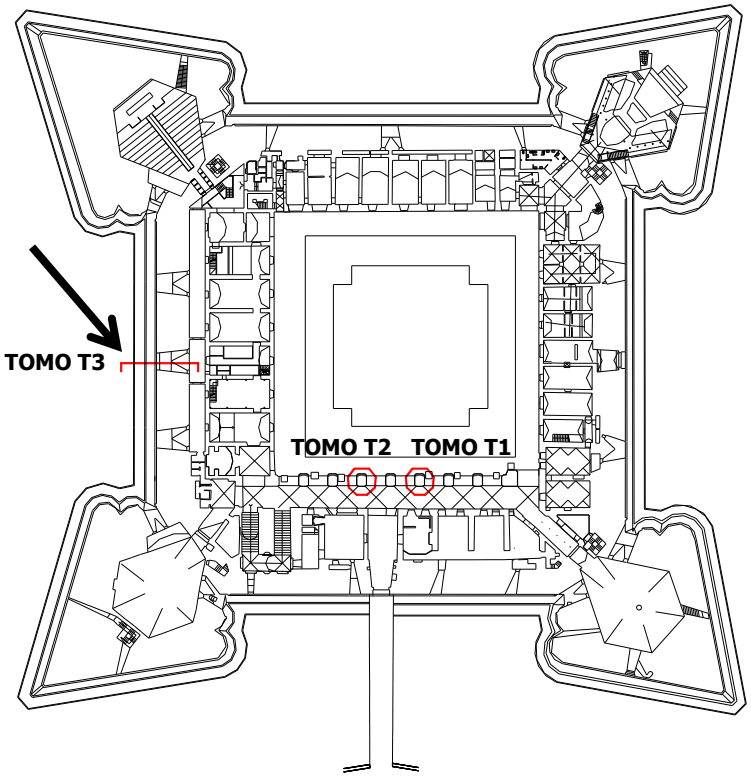
Griglia dei punti di acquisizione



SONIC TOMOGRAPHY

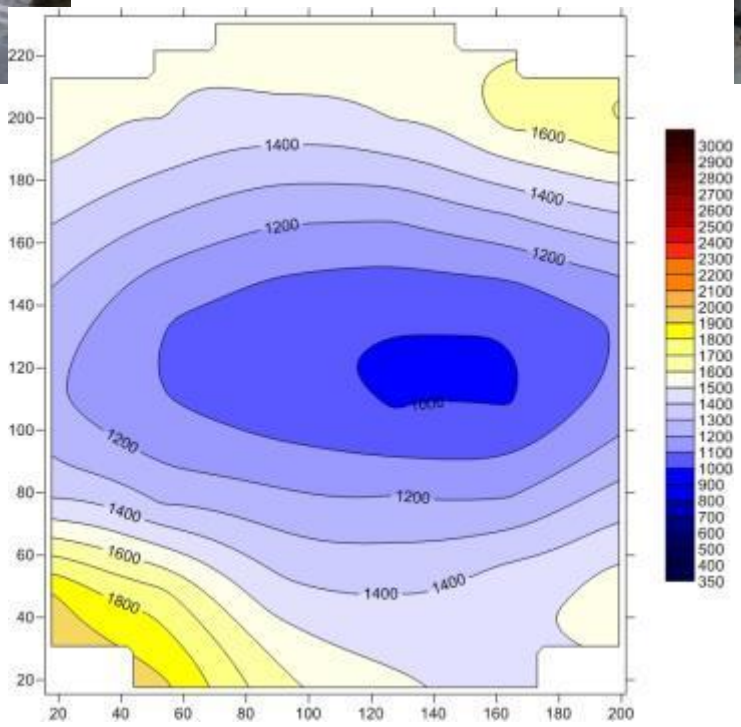


La terza tomografia è stata condotta su una parete muraria nell'ala sud-est del castello. Lo scopo prefisso è stato quello di raccogliere dati qualitativi sulla muratura di questa area della fortezza pesantemente danneggiata dal sisma.

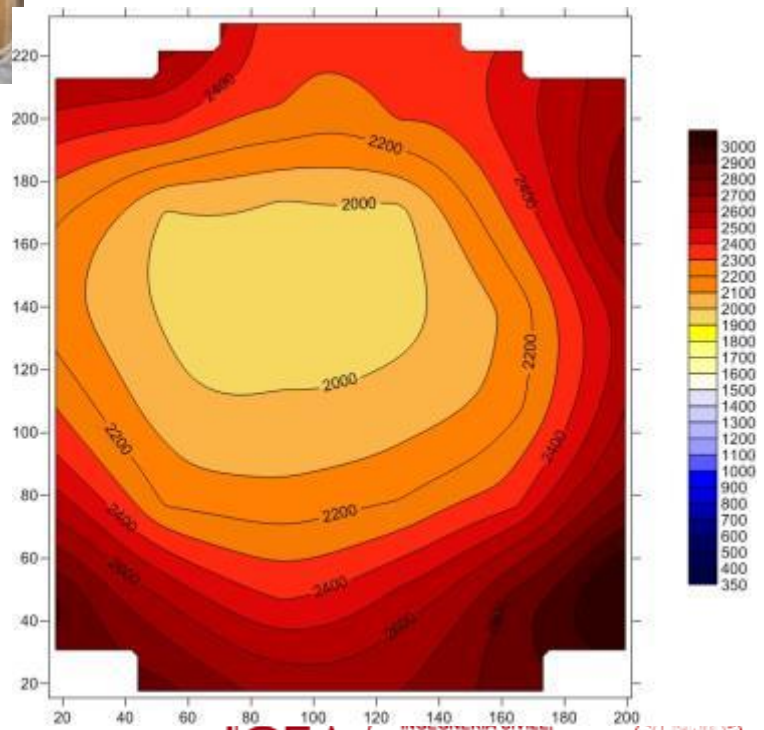


PILASTRO DANNEGGIATO

PILASTRO INTEGRO

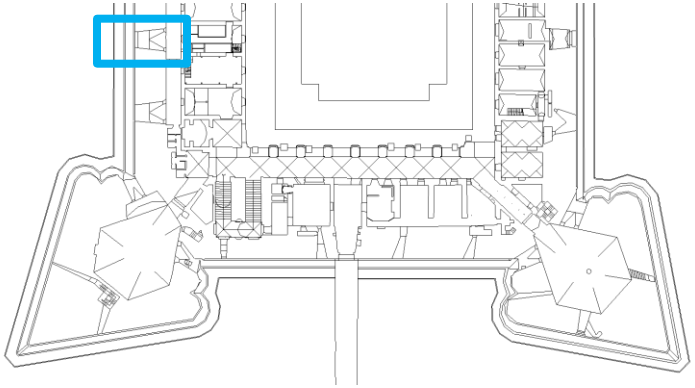


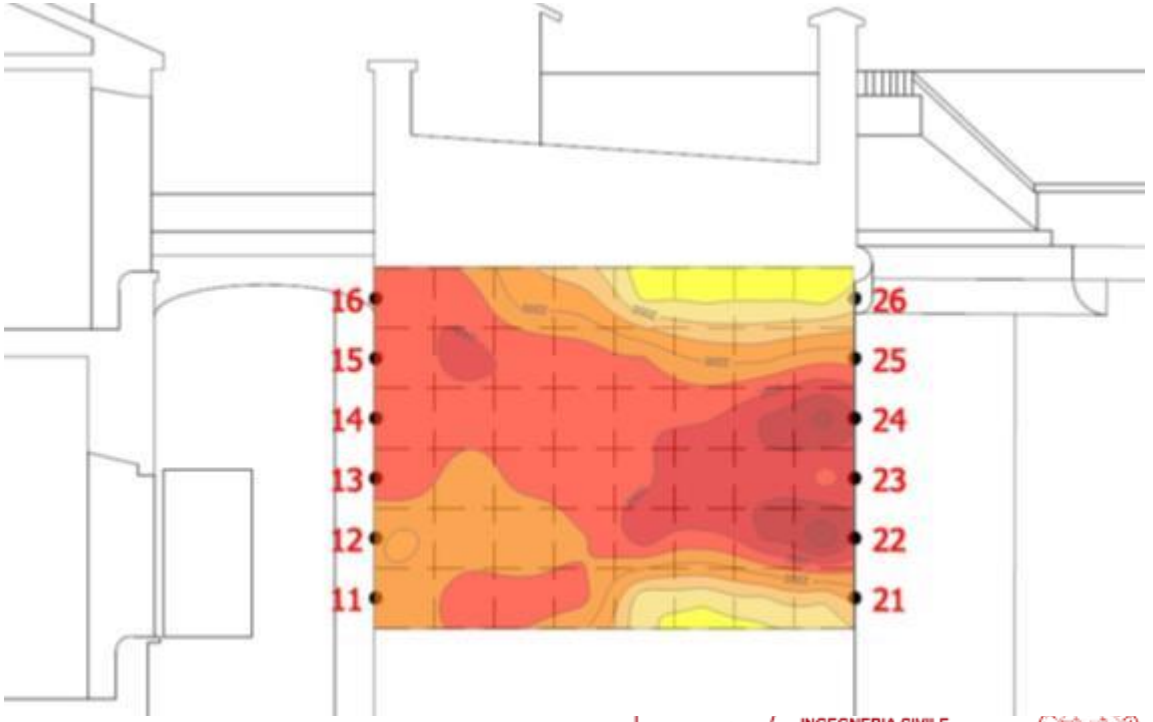
TOMO 2



TOMO 1

La tomografia della parete spessa mostra una distribuzione omogenea delle velocità soniche attraverso lo spessore. La muratura, anche all'interno del muro, è di buona qualità



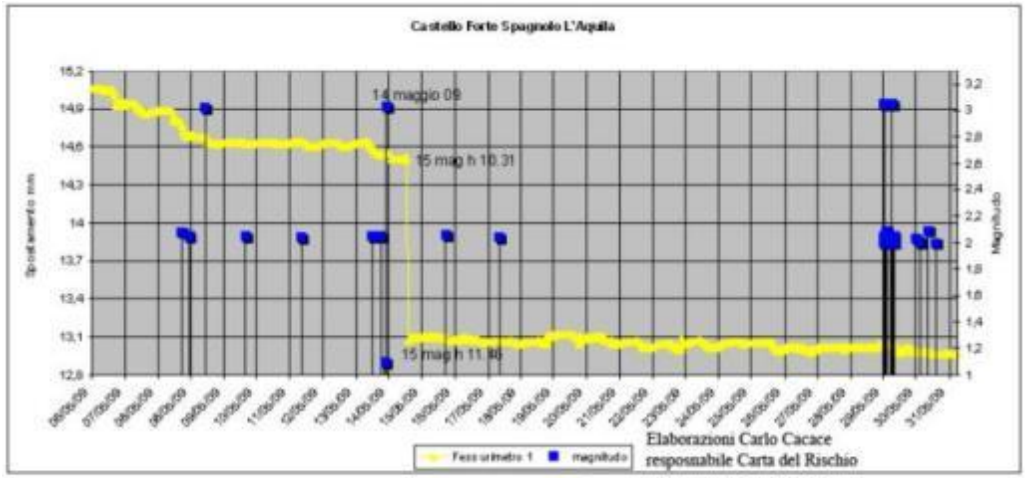


L'AQUILA: LA FORTEZZA SPAGNOLA

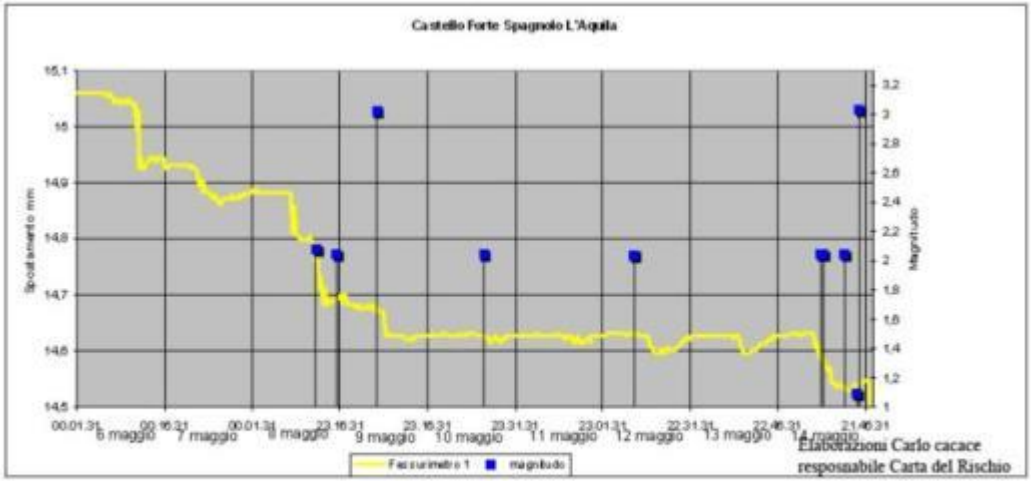
SISTEMA DI MONITORAGGIO STATICO

L'AQUILA: LA FORTEZZA SPAGNOLA

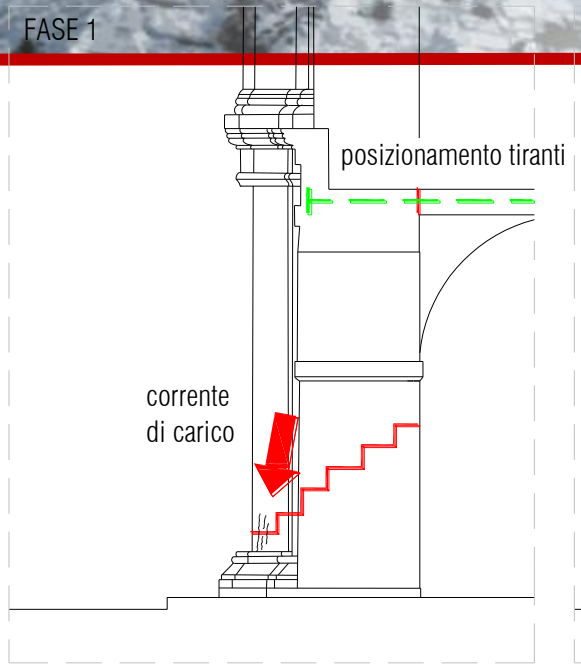
SISTEMA DI MONITORAGGIO STATICO

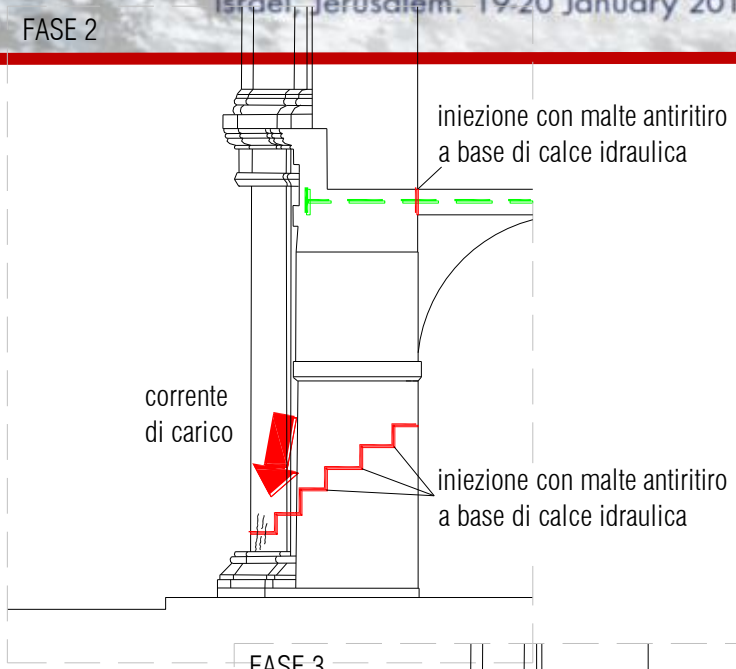


Maggio 2009 rappresentazione delle magnitudo (IGNV) e i rilevamenti dei fessurimetri Il giorno 15 maggio Sono stati applicati i tiranti e si giustifica il salto (chiusura della fessura).

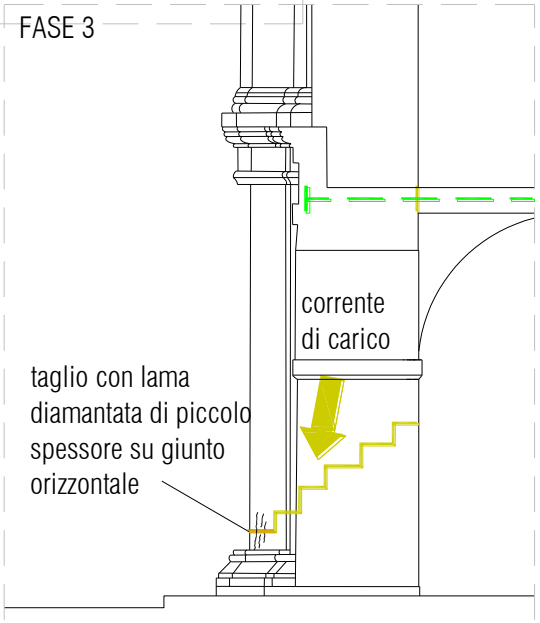


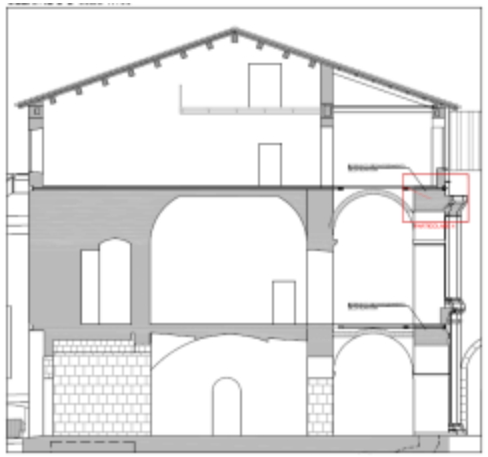
Maggio 2009 rappresentazione delle magnitudo (IGNV) e i rilevamenti dei fessurimetri. Il mese è suddiviso due momenti prima dei tiranti. (06-15/05)





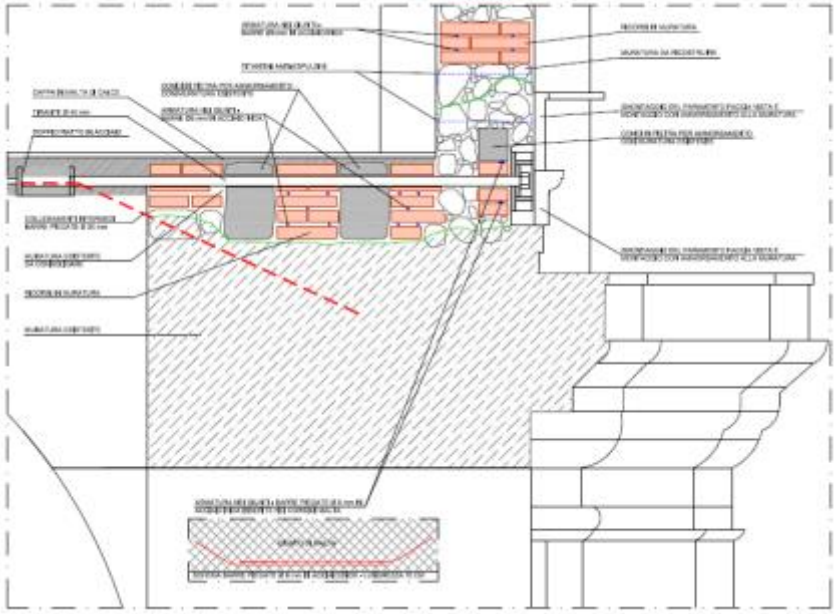
Il progetto di recupero dei pilastri prevede il taglio alla base degli stessi per «ri-centrare» il carico, attualmente eccentrico a causa del meccanismo di ribaltamento della parete, che ha determinato lo schiacciamento della base degli stessi.





<u>Problematiche rispetto al posizionamento dei tiranti:</u>

- Assenza di allineamento tra setti murari e pilastri
- Limitato spessore tra pavimento e estradosso della volta

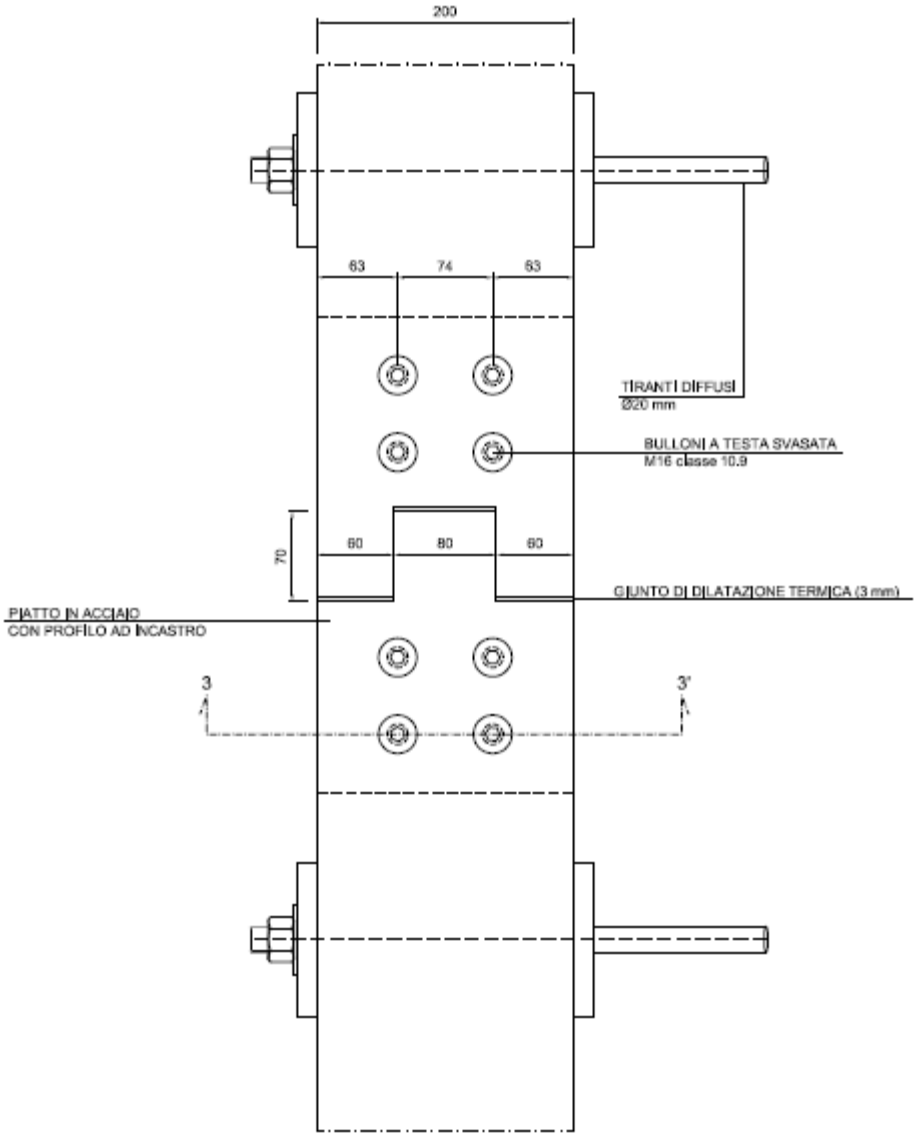


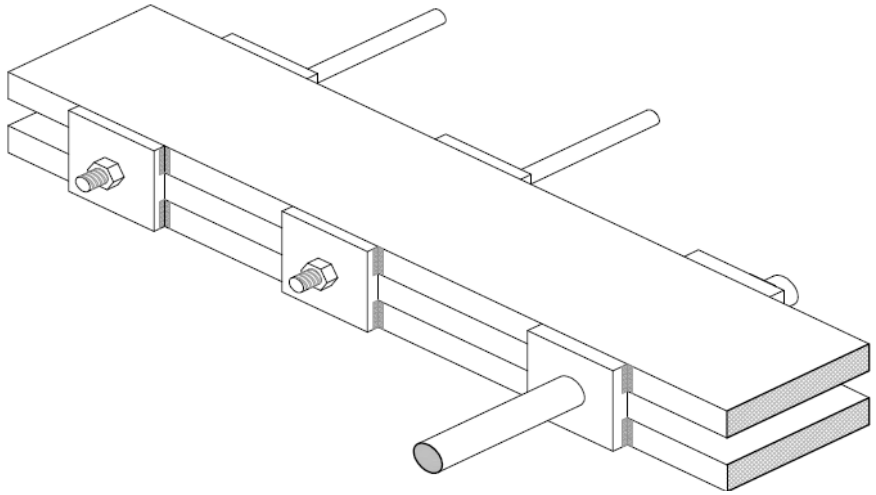
Ala sud-est: progetto d'interventi

POSIZIONAMENTO DEI TIRANTI

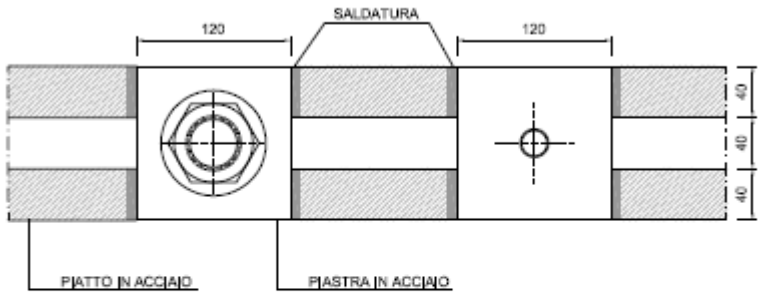
Creazione di un sistema «reticolare» per consentire lo sfalsamento dei tiranti.

VISTA DALL'ALTO GIUNZIONE PIATTI

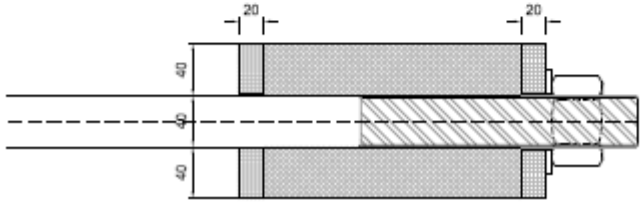




VISTA LATERALE



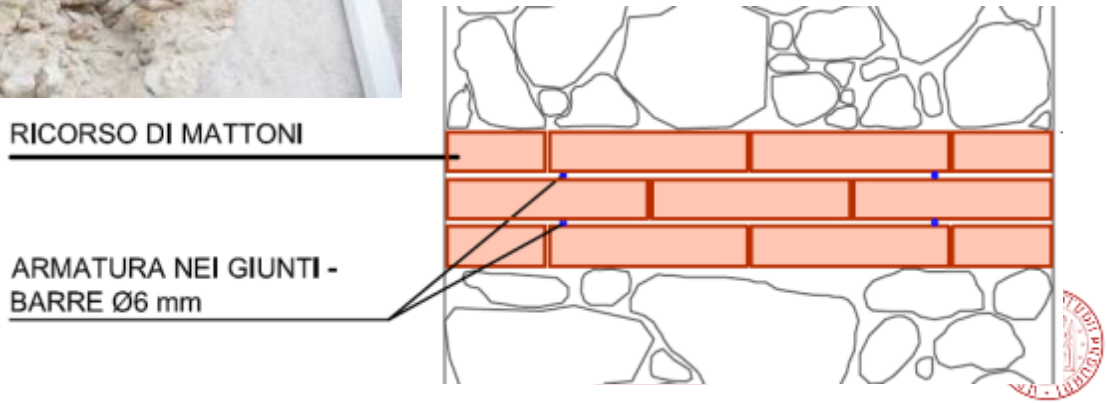
SEZIONE 1-1'

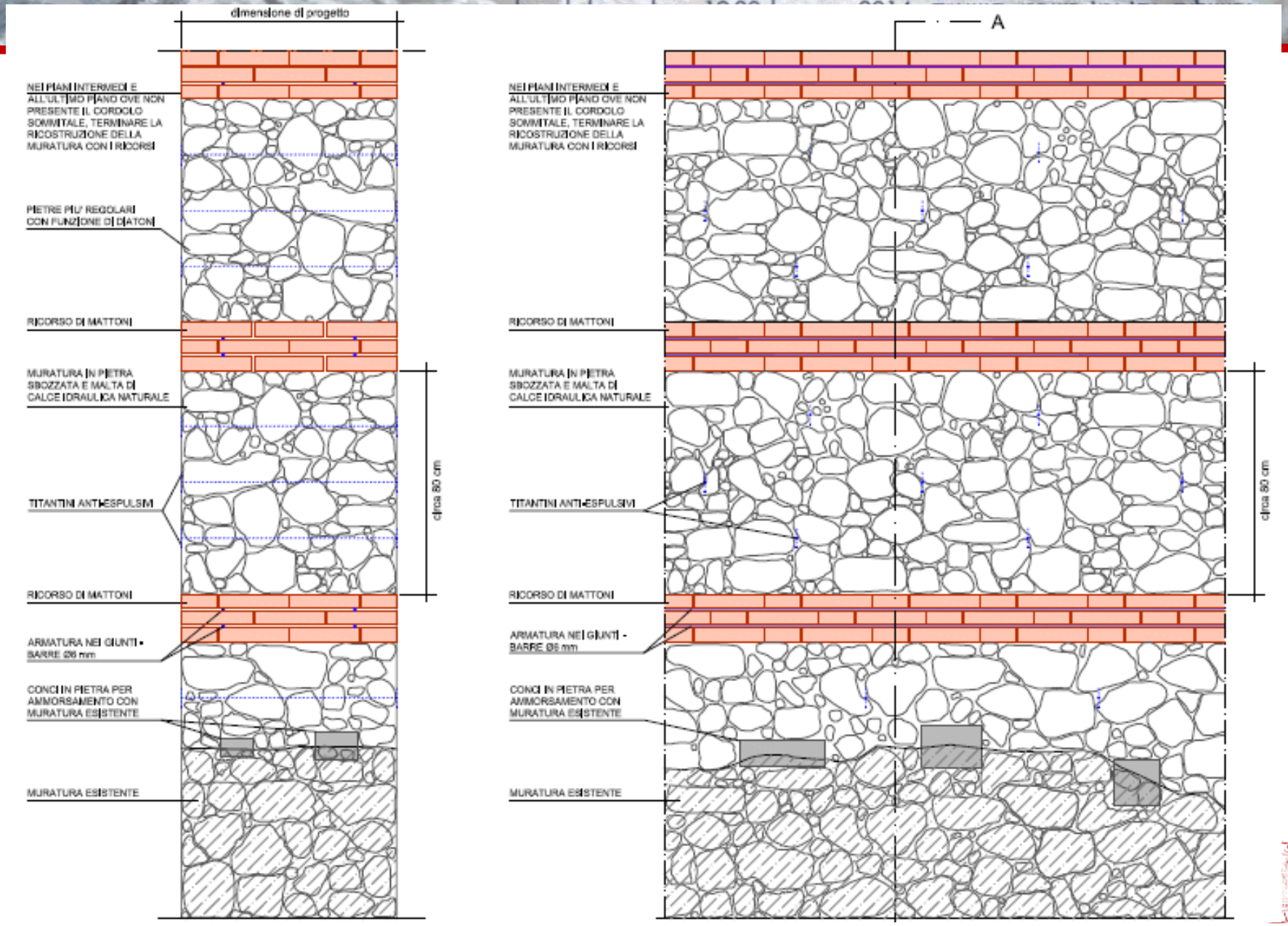


Tra le principali problematiche connesse alla ricostruzione di strutture gravemente danneggiate vi è la presenza di <u>murature con caratteristiche meccaniche scarse o addirittura pessime</u>.

Si pone quindi il problema di trovare una tipologia costruttiva che garantisca delle buone proprietà meccaniche di resistenza e coesione e al tempo stesso rispetti la tradizione costruttiva originaria.

PROGETTO DI RICOSTRUZIONE: utilizzo di ricorsi in mattoni con armature inserite nei giunti



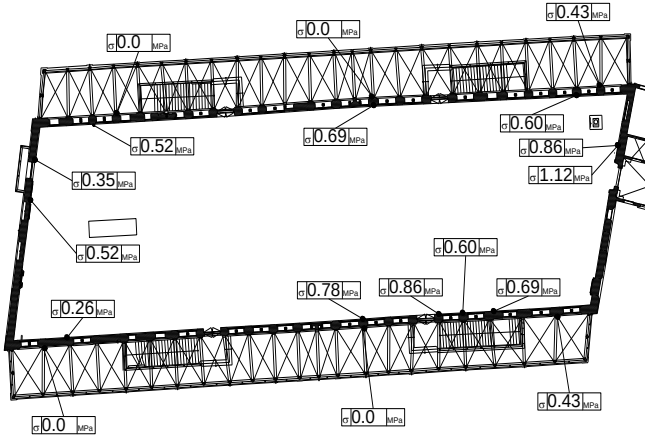

dimensione di progetto
NEI PIANI INTERMEDI E ALL'ULTIMO PIANO OVE NON PRESENTE IL CORDOLO SOMMITALE, TERMINARE LA RICOSTRUZIONE DELLA MURATURA CON I RICORSI
PIETRE PIU' REGOLARI CON FUNZIONE DI DIATONI
RICORSO DI MATTONI
MURATURA IN PIETRA SBOZZATA E MALTA DI CALCE IDRAULICA NATURALE
TITANTINI ANTI-ESPULSIVI
RICORSO DI MATTONI
ARMATURA NEI GIUNTI - BARRE Ø6 mm
CONCI IN PIETRA PER AMMORSAMENTO CON MURATURA ESISTENTE
MURATURA ESISTENTE
circa 80 cm
A
NEI PIANI INTERMEDI E ALL'ULTIMO PIANO OVE NON PRESENTE IL CORDOLO SOMMITALE, TERMINARE LA RICOSTRUZIONE DELLA MURATURA CON I RICORSI
RICORSO DI MATTONI
MURATURA IN PIETRA SBOZZATA E MALTA DI CALCE IDRAULICA NATURALE
TITANTINI ANTI-ESPULSIVI
RICORSO DI MATTONI
ARMATURA NEI GIUNTI - BARRE Ø6 mm
CONCI IN PIETRA PER AMMORSAMENTO CON MURATURA ESISTENTE
MURATURA ESISTENTE
circa 80 cm

PALAZZO DELLA RAGIONE (Padova – Italy)

The building was the City Council Palace of Padova (Italy).
Its construction started in 1218.

PALAZZO DELLA RAGIONE (Padova – Italy)

The roof covers a unique, skewed room, approximately 28 m wide and 80 m long, formed by slender ribs (36x40 cm2, obtained by joining curved boards 12 cm thick, whose length varies from less than one meter to three meters), which 3 cm thick boards are simply nailed to.



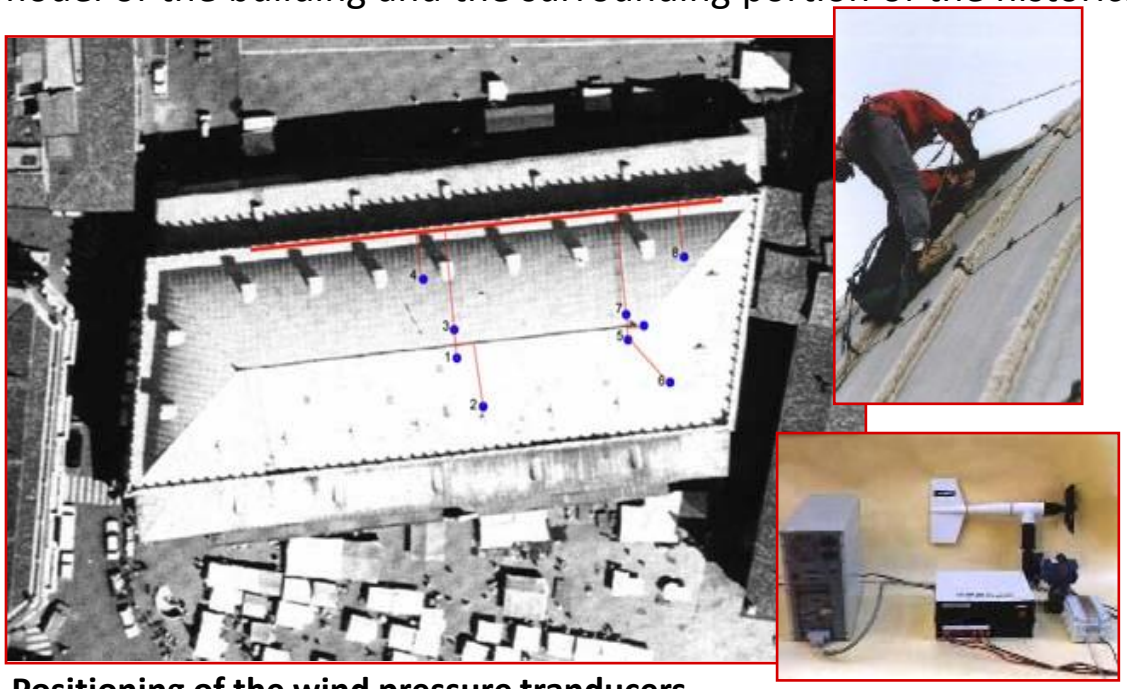
PALAZZO DELLA RAGIONE (Padova – Italy)

OPTIMIZING THE INTERVENTIONS: INVESTIGATIONS AND MONITORING

Two different type of investigations were adopted to analyze the static and dynamic effects of the wind on the roof shell:

- on-site measurement of the wind pressure;
- wind tunnel testing of a scaled-model of the building and the surrounding portion of the historical centre.

Scaled model for the wind tunnel testing

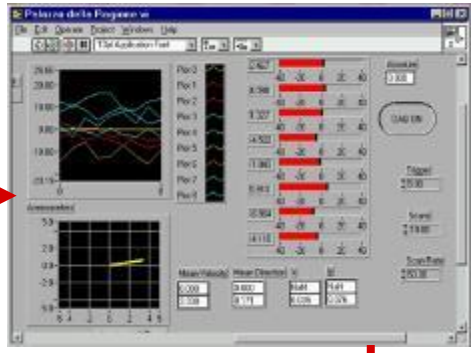


Positioning of the wind pressure tranducers

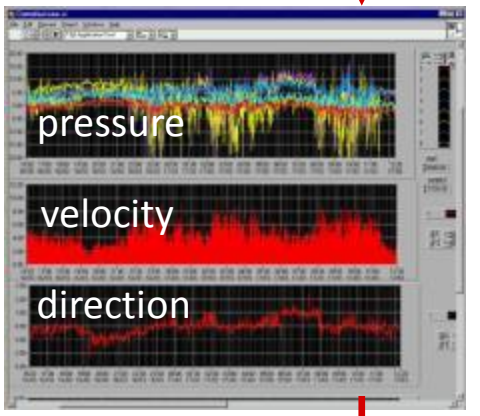
PALAZZO DELLA RAGIONE (Padova – Italy)

Idea:

- ❑ monitoring of the wind effects
- ❑ stepped sine test
- ❑ FE modeling
- ❑ identificazione modale
- ❑ on-line data management

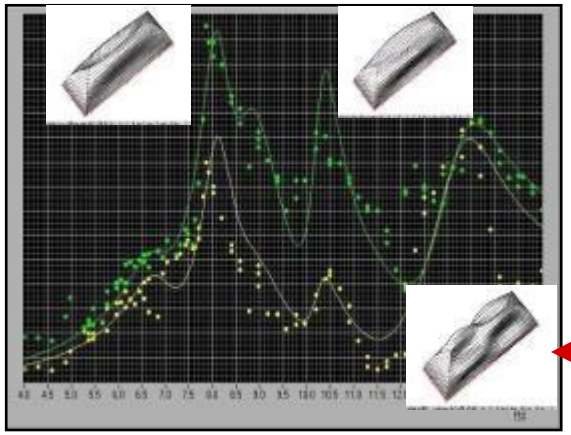


Data acquisition



www database

Web monitoring home page



Modal analysis

PALAZZO DELLA RAGIONE (Padova – Italy)

TYING OF A CRACKED PILLAR

Provisional intervention

Final intervention

PALAZZO DELLA RAGIONE (Padova – Italy)

Deterioration of the internal part of a rib

The design of the local repairing was made by taking simple temporary measures allowing for maintaining the stable 3D behaviour and inhibiting local buckling in any phase of the works, by simply limiting the removal (when needed) of the boards forming the shell very locally, in order to permit local repairing, and through simple lateral supports.

Deterioration of a shelf

Affected top of a rib

PALAZZO DELLA RAGIONE (Padova – Italy)

INTERVENTION ON THE RIBS

1) lateral bracing of the ribs

2) insertion of a temporary steel tie

3) carrying out a metallic stirrup

ICEA
INGEGNERIA CIVILE, EDILE E AMBIENTALE
CIVIL, ARCHITECTURAL AND ENVIRONMENTAL ENGINEERING

PALAZZO DELLA RAGIONE (Padova – Italy)

INTERVENTION ON THE RIBS

Substitution of degraded elements

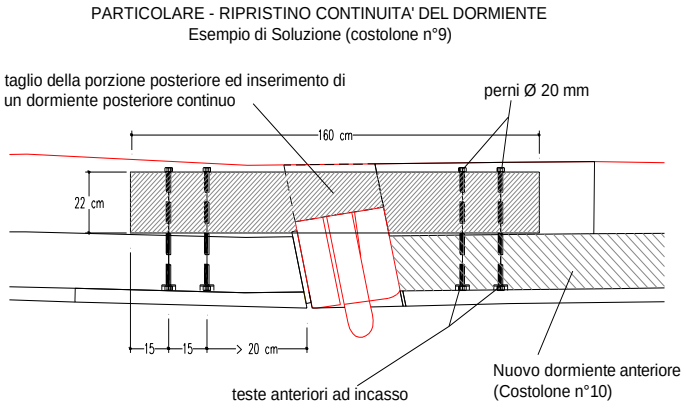
Local stitching of cracks

Restoration and integration of the nails

PALAZZO DELLA RAGIONE (Padova – Italy)

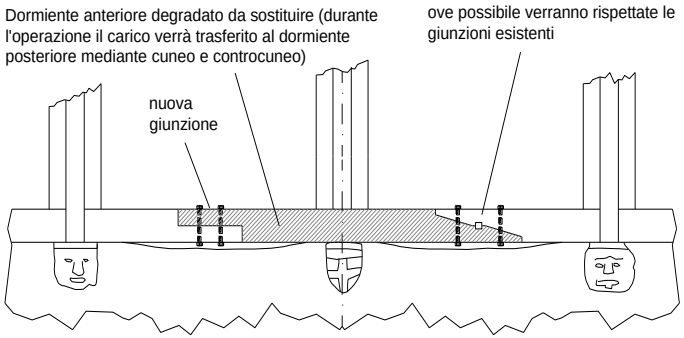
INTERVENTION ON THE SHELFS

Removal of the deteriorated portion of the shelf and back-blocking of the rib



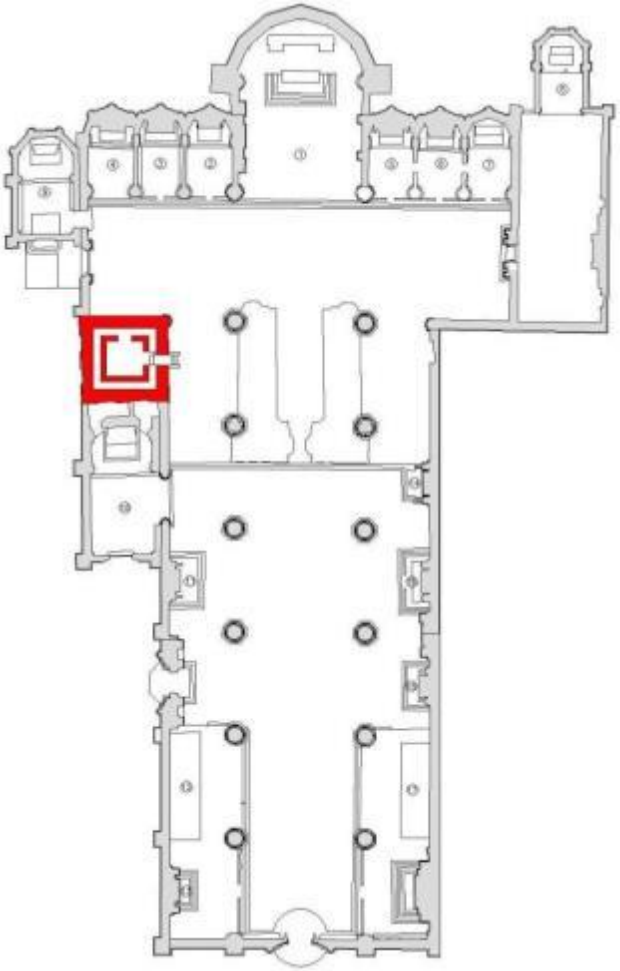
Restoration

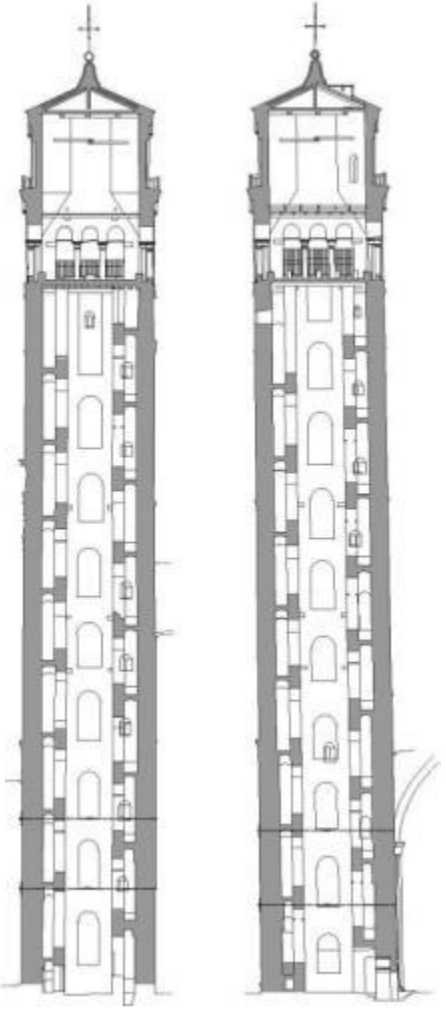
Substitution



Investigation and monitoring for the design of a strengthening intervention on the Basilica dei Frari – Venice

Basilica dei Frari (Venice)





1st PHASE (1361-1396): CONSTRUCTION OF THE BELL TOWER

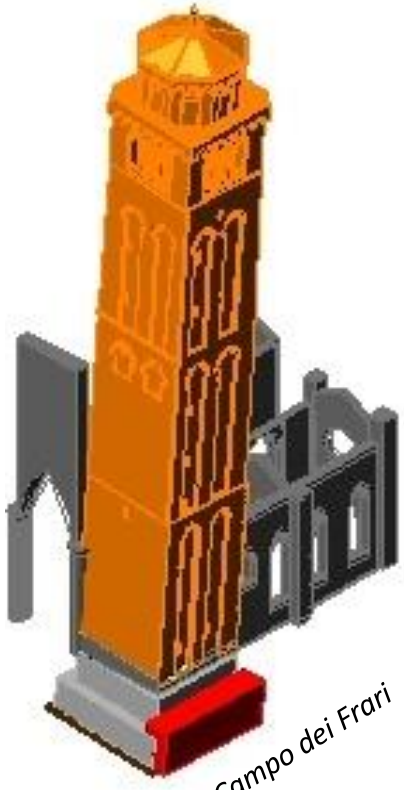
The tower, built next to the church between 1361 and 1396, was originally conceived as a completely independent structure

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



Campo dei Frari

Monitoring of the venetian towers, including the "Frari" bell tower

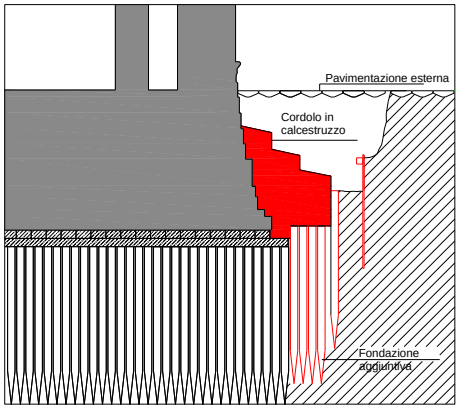
RESULTS OF THE MONITORING:

- **Differential settlement between church and tower: 30 cm**
- **Out-of-plumb: 76cm on a height of 42.5m**

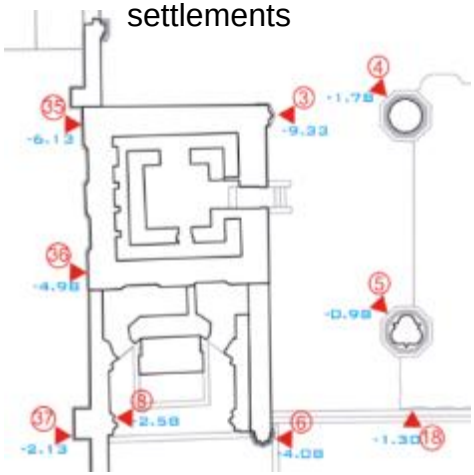
1903: Intervention on tower's foundations

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**



SECTION A-A



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).

⇓

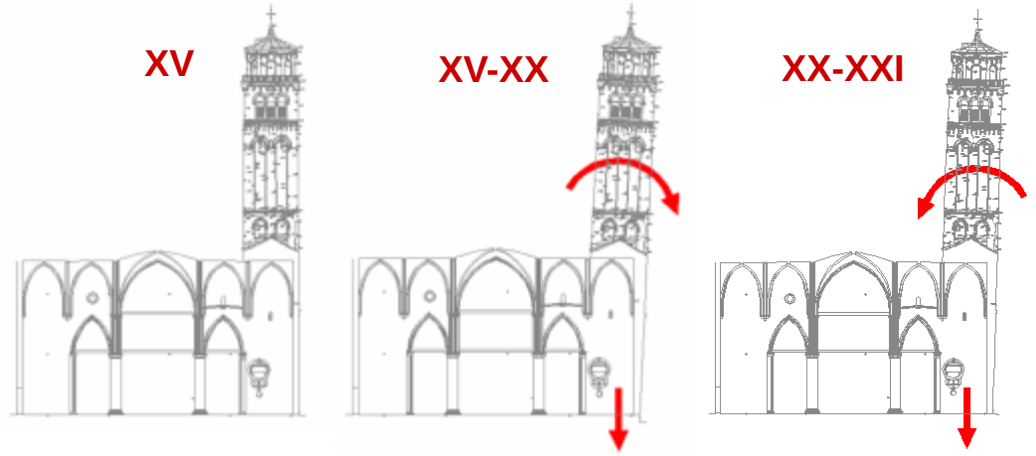
discrete stability of the tower structure; out of plumb of about 0.8 m

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

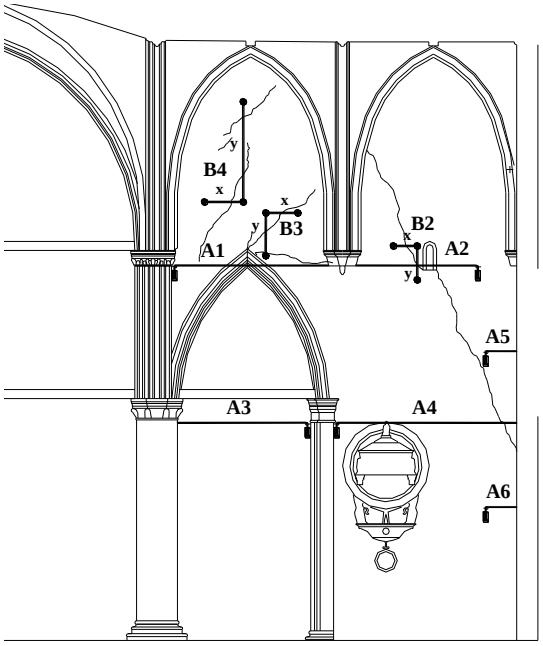
⇓

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



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.



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.

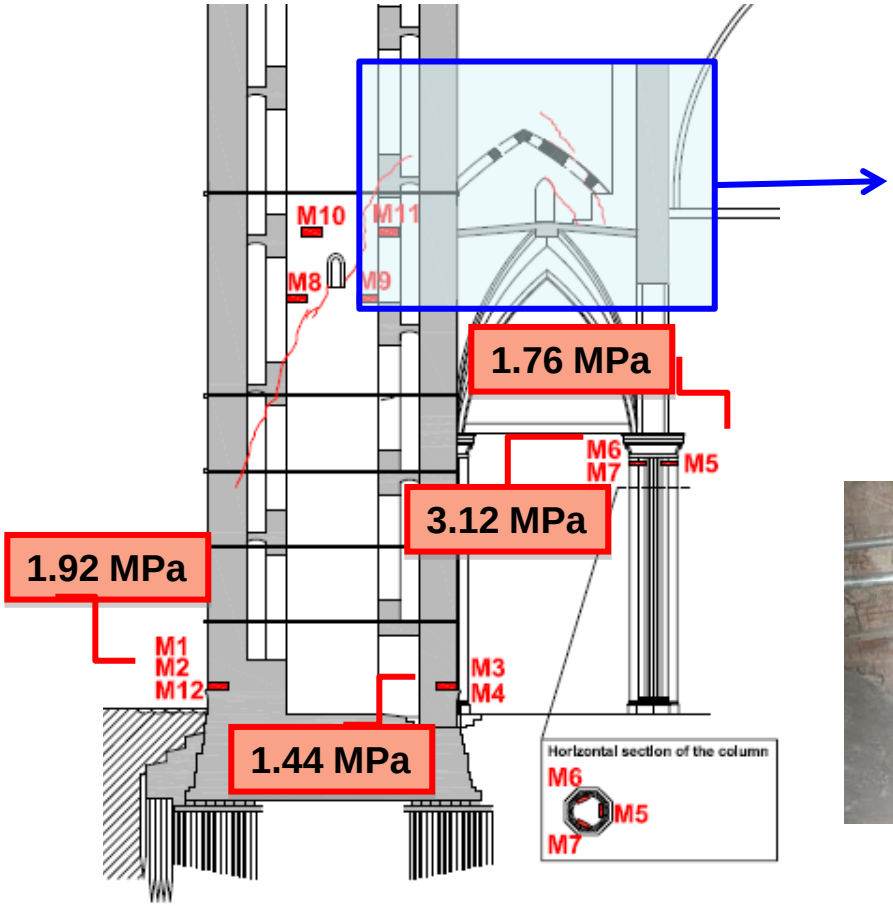
⇓

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

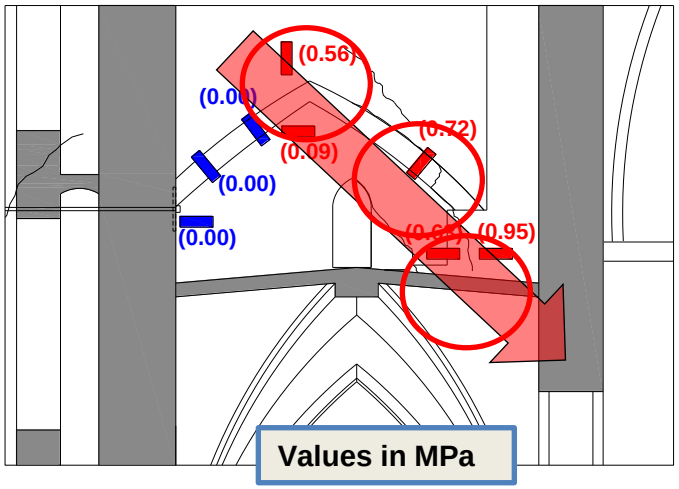
2003 investigation campaigns

ANALYSIS OF THE STATE OF STRESS: FLAT JACK TESTS

Second Test Campaign

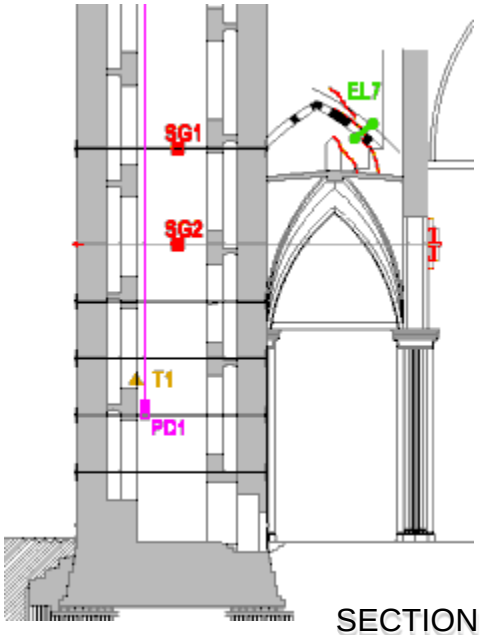


Third Test Campaign

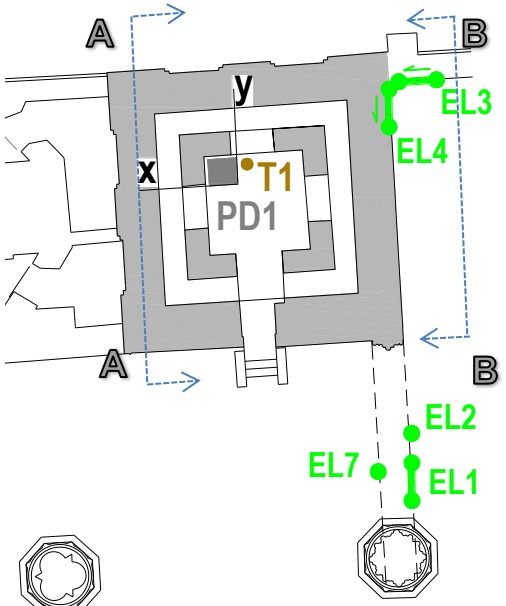


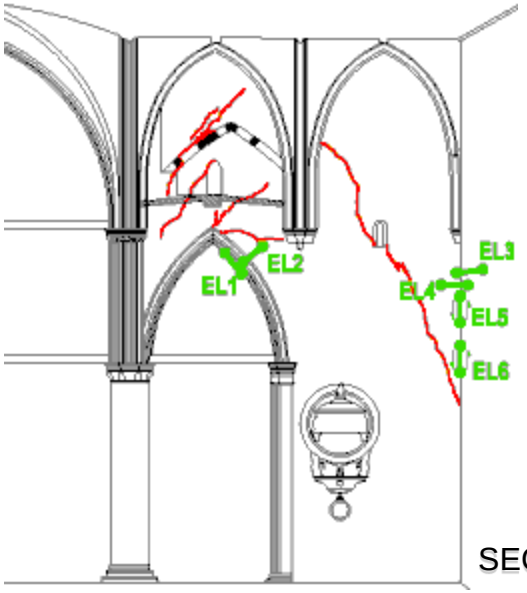
2003 extension of the automatic monitoring system

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



SECTION A-A

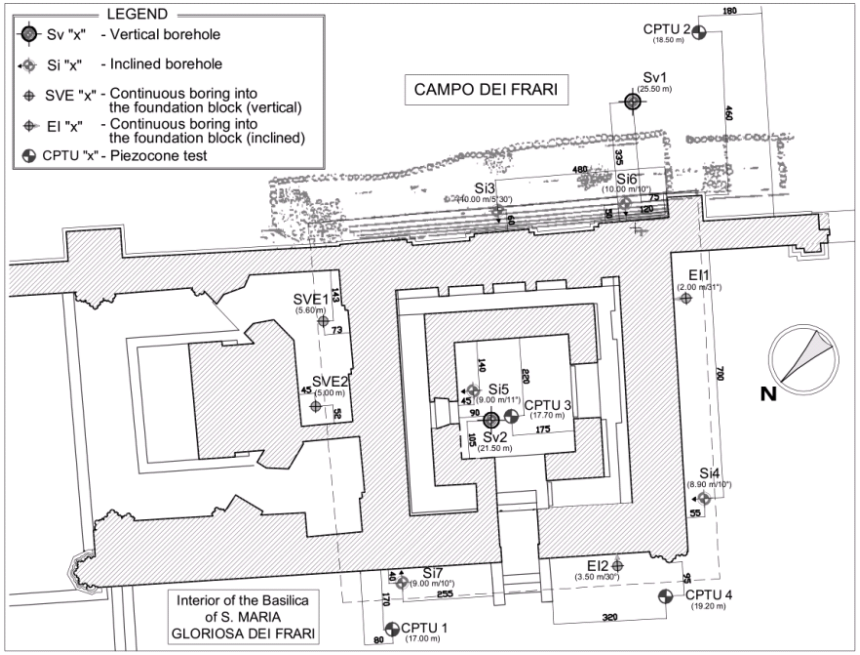




SECTION B-B

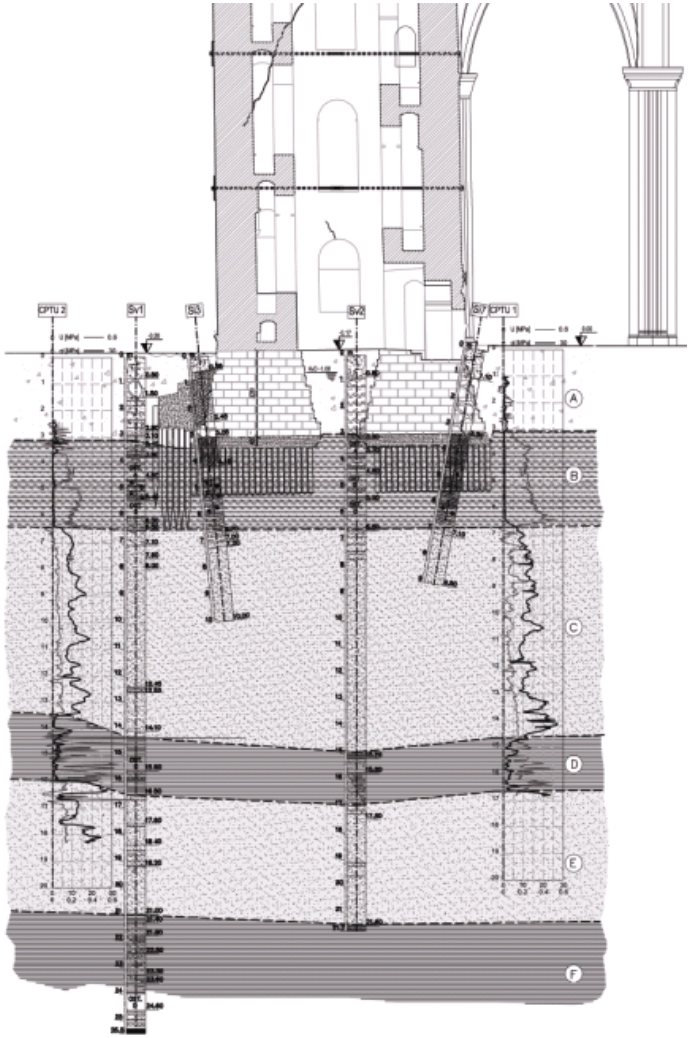
2003 extensive geotechnical investigation campaign

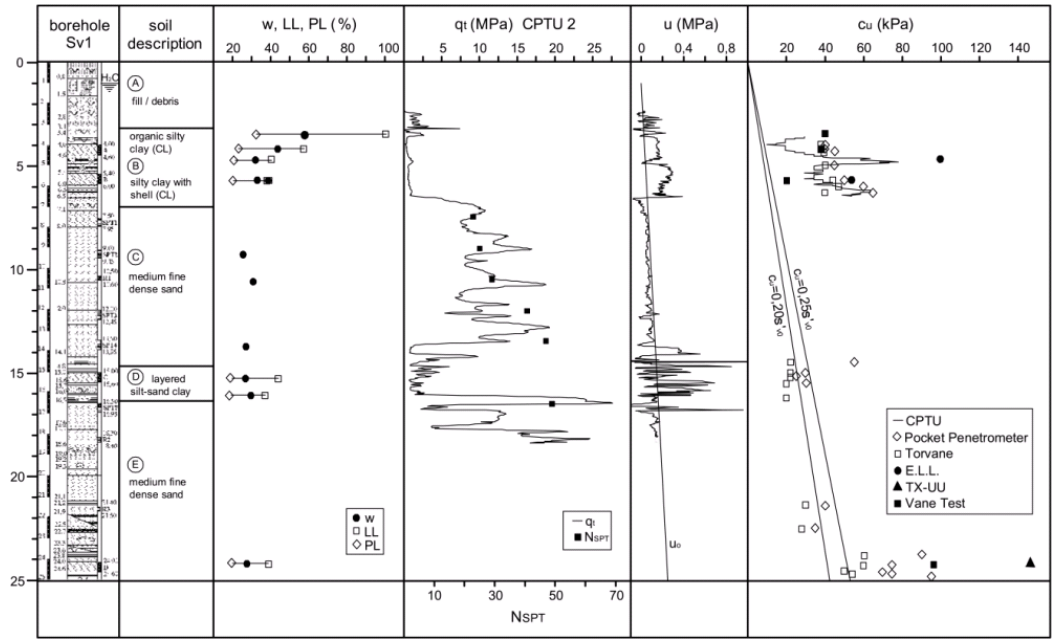
- analysis of the **subsoil stratigraphy and geotechnical properties**;
- exploration of the exact geometry and typology of the **foundation block**;
- definition of an accurate **geotechnical model** of the foundation finally completed;
- **in situ tests**:



- 2 **vertical boreholes**: outside the church, to a depth of 5.50 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.

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.

Structural modeling

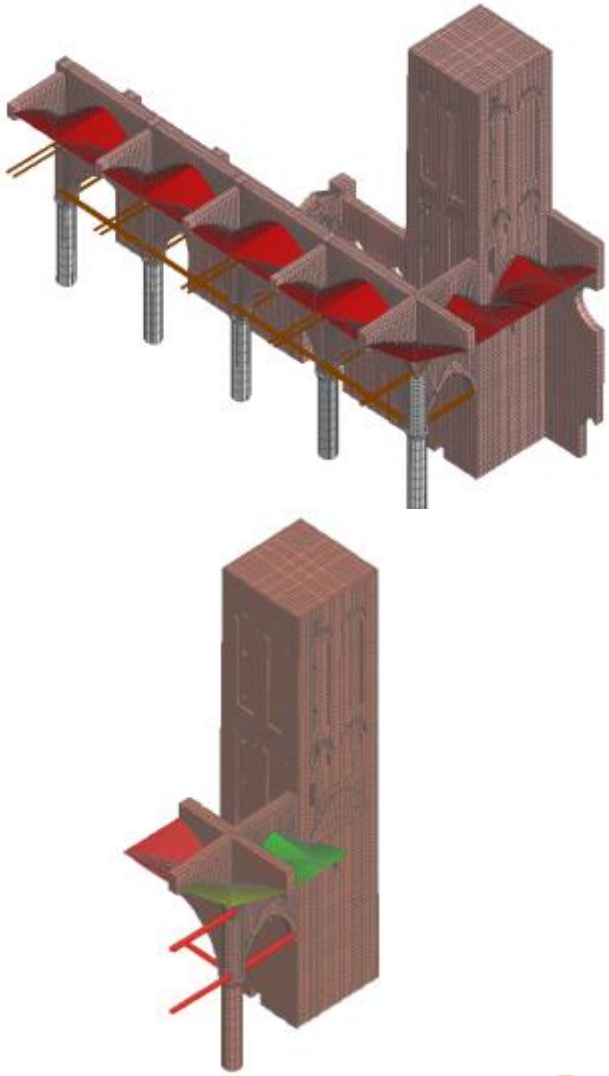
The modelled portion of structure includes the **bell-tower and the adjoining parts of the church** that were mostly affected by the interaction with the tower.

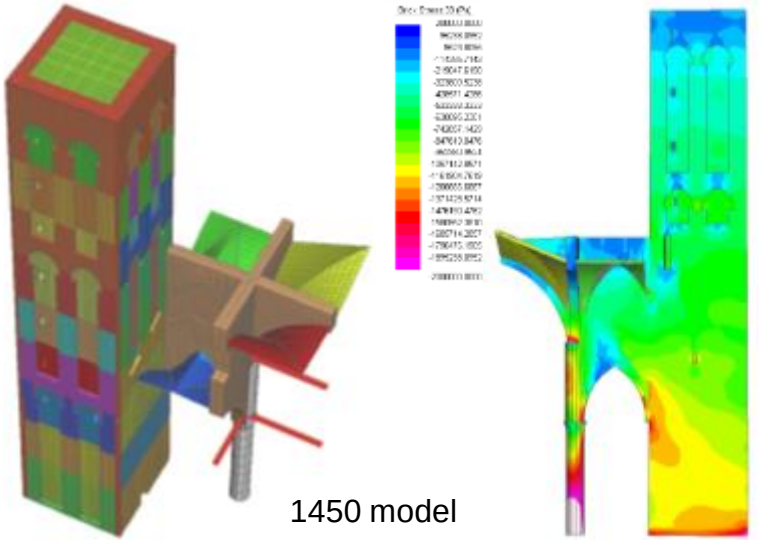
The only load condition considered is the **self weight**. The load corresponding to some parts of the real structure not modelled (timber structure roof of the basilica, belfry), was imposed as external forces; the crossed vaults' filling was included as surface load.

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

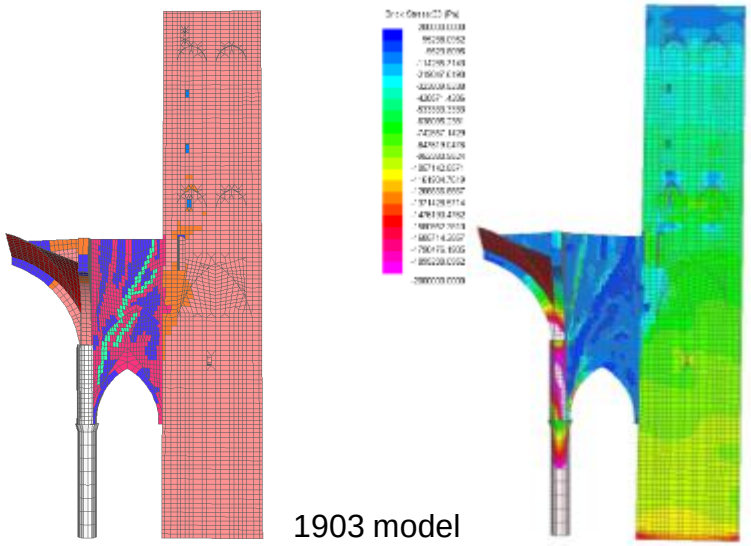
- **elastic modulus** E = 3300 MPa (average of the results of double flat jack tests performed on the bell-tower masonry)
- **density** $\rho = 2000\ kg/m^3$

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





1450 model



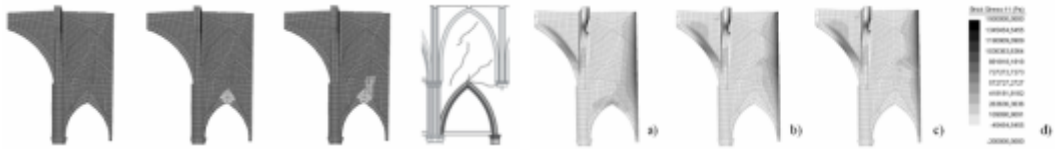
1903 model

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).

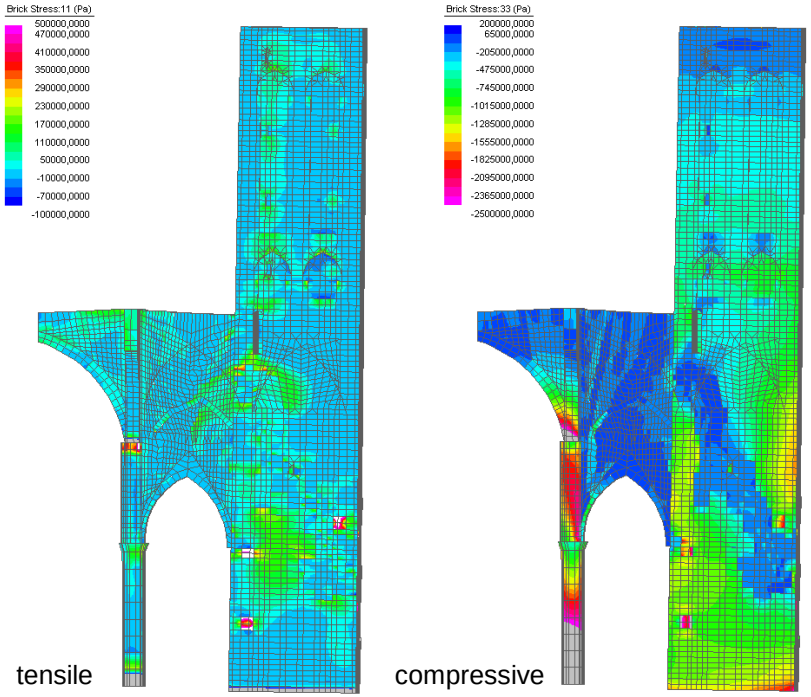
In each model, after running the analysis, a higher deformability was assigned to the elements subjected to an excessive tension respect the assumed strength of the material, in the successive analysis.

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.



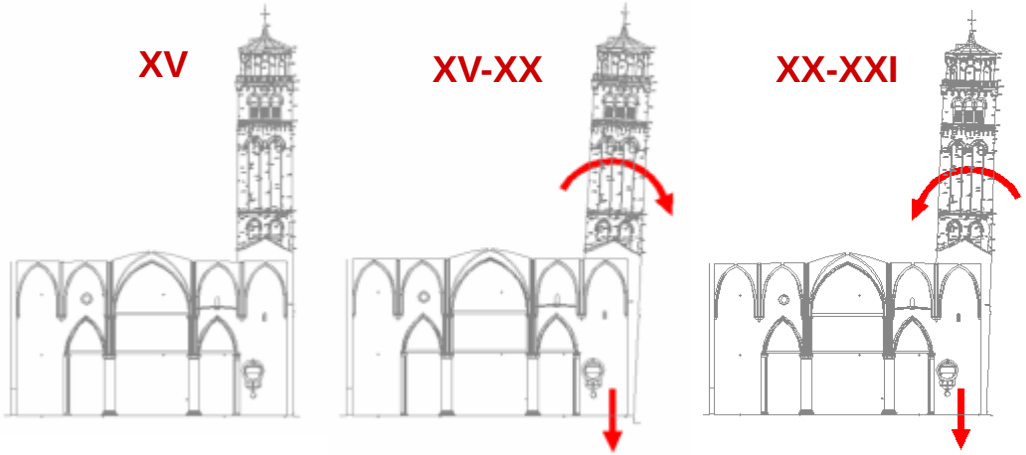
The **final model** reflects the tendency of the XX century. An "inverted" rotation was imposed to the bell-tower, with an average settlement of 84 mm.

Results:

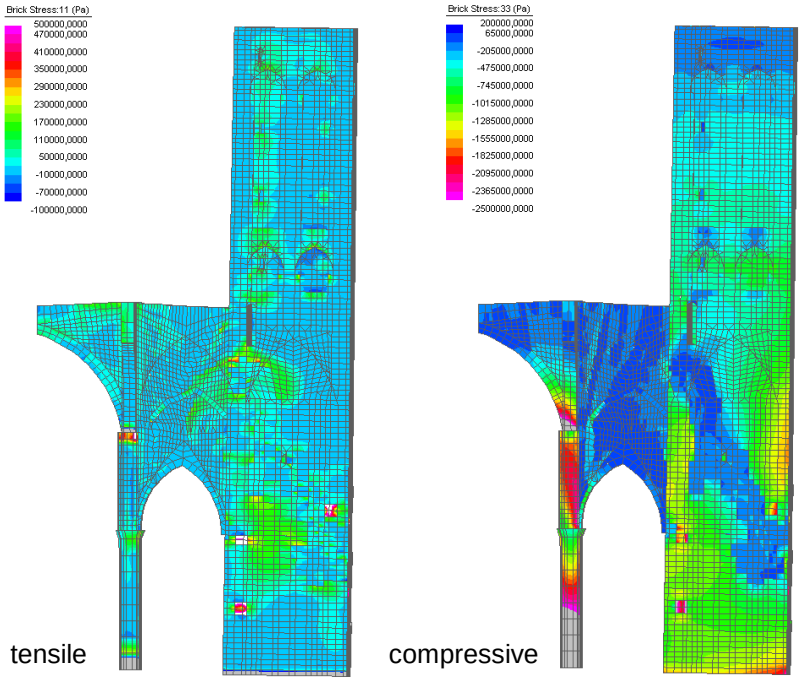


2002 model

- 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.



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.

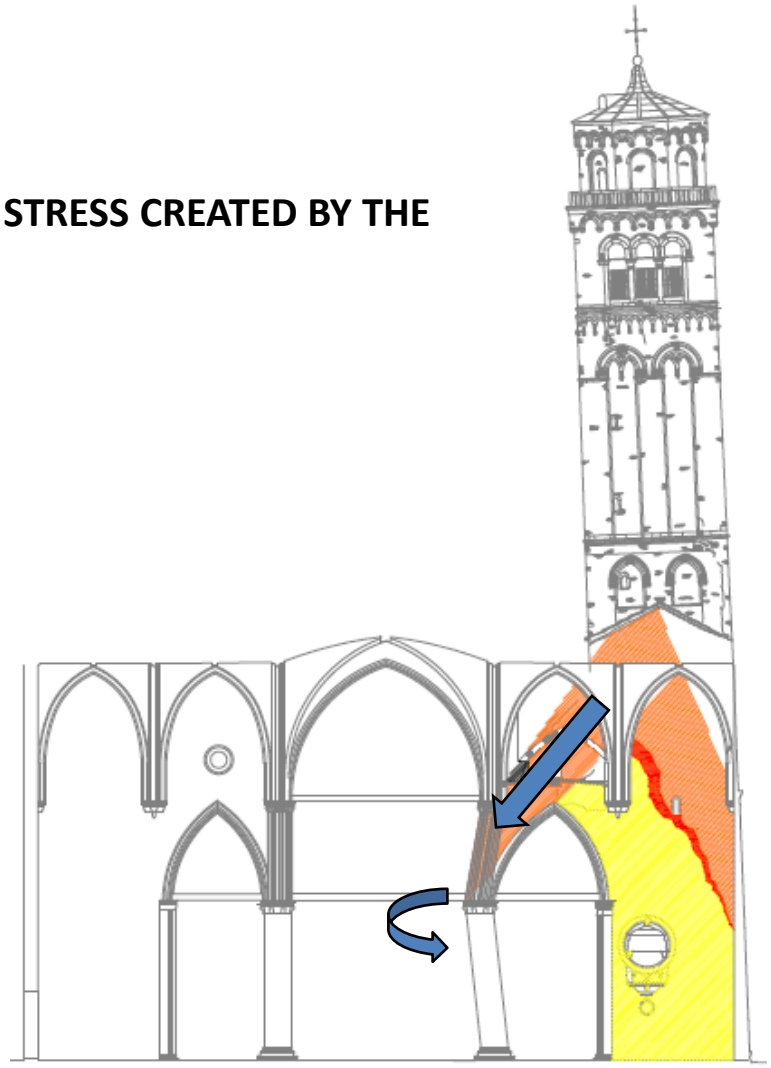


- preferential channels of **compressive stress** localized inside the masonry wall above the propped arch;
- high **tensile stresses** in the same masonry wall → wide crack patterns and loss of shape of the stone arch;
- **high stress** below the capital of the column → horizontal thrust determined 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.

Structural Diagnosis

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

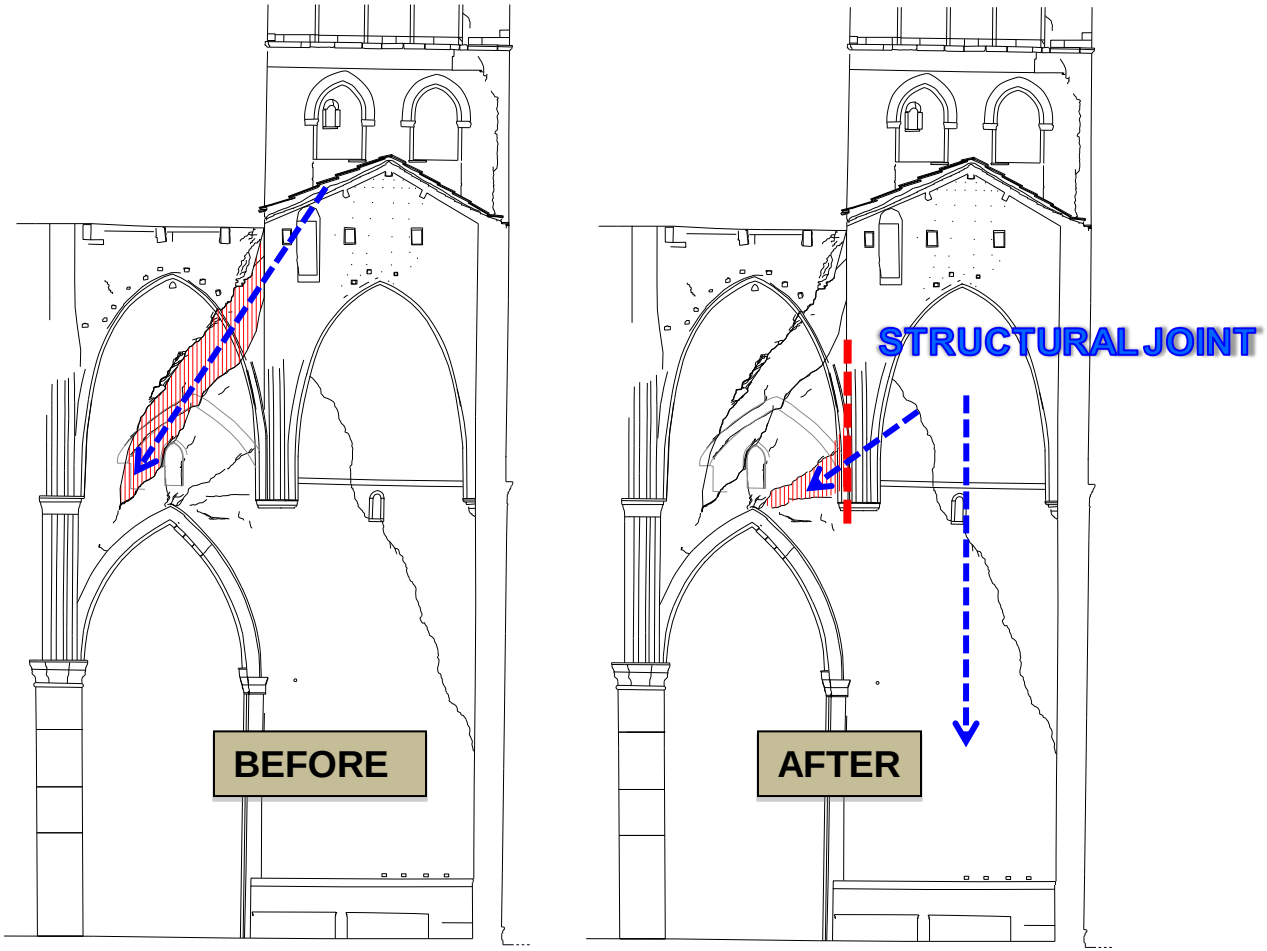
- **increase of the compression load on the column**
- **a strong transverse bending stress on the column, due to both the eccentricity of the vertical load applied to it and the horizontal component of the thrust**
- **decrease of the vertical load (equal to the increase on the column) on the tower**



BASIC PRINCIPLE OF THE INTERVENTION:

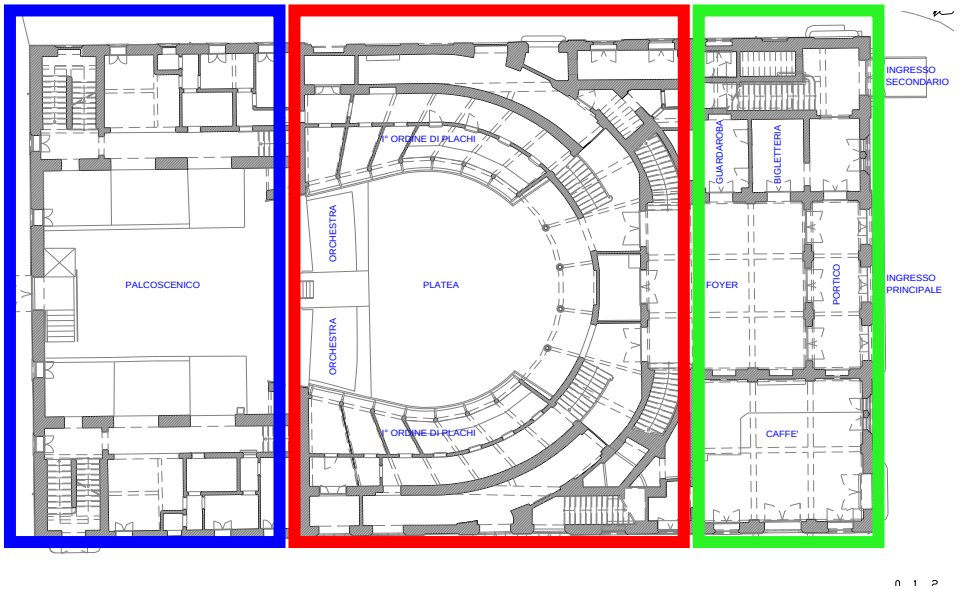
- Creation of a joint in order to separate the bell tower from the church and make them structurally more independent

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



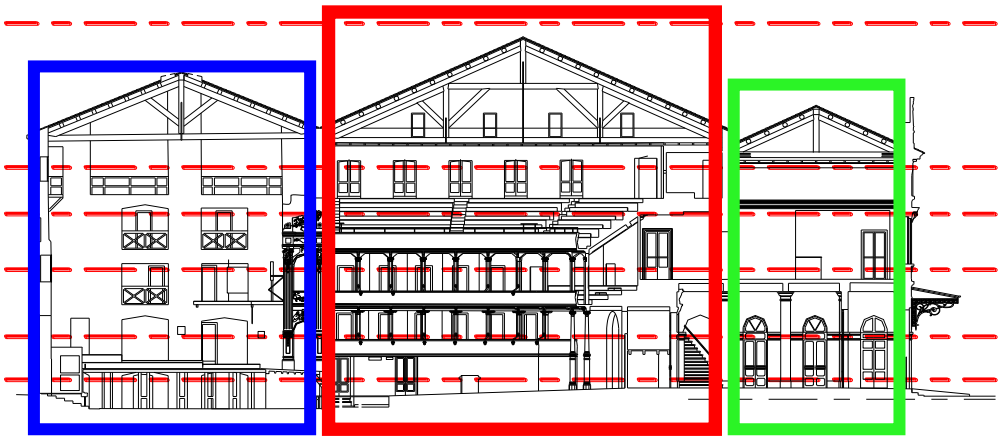
CIVIC THEATRE – SCHIO (VI)

Morphology



Ridotto

Platea



Liv. 04: 215.76 s.l.m.m.

Palcoscenico

Liv. 03: 208.33 s.l.m.m.

Liv. 02: 205.90 s.l.m.m.

Liv. 01: 203.00 s.l.m.m.

Liv. 00: 199.50 s.l.m.m.

Liv. - 01: 197.26 s.l.m.m.
Liv. - 02: 196.26 s.l.m.m.

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CIVIC THEATRE – SCHIO (VI)

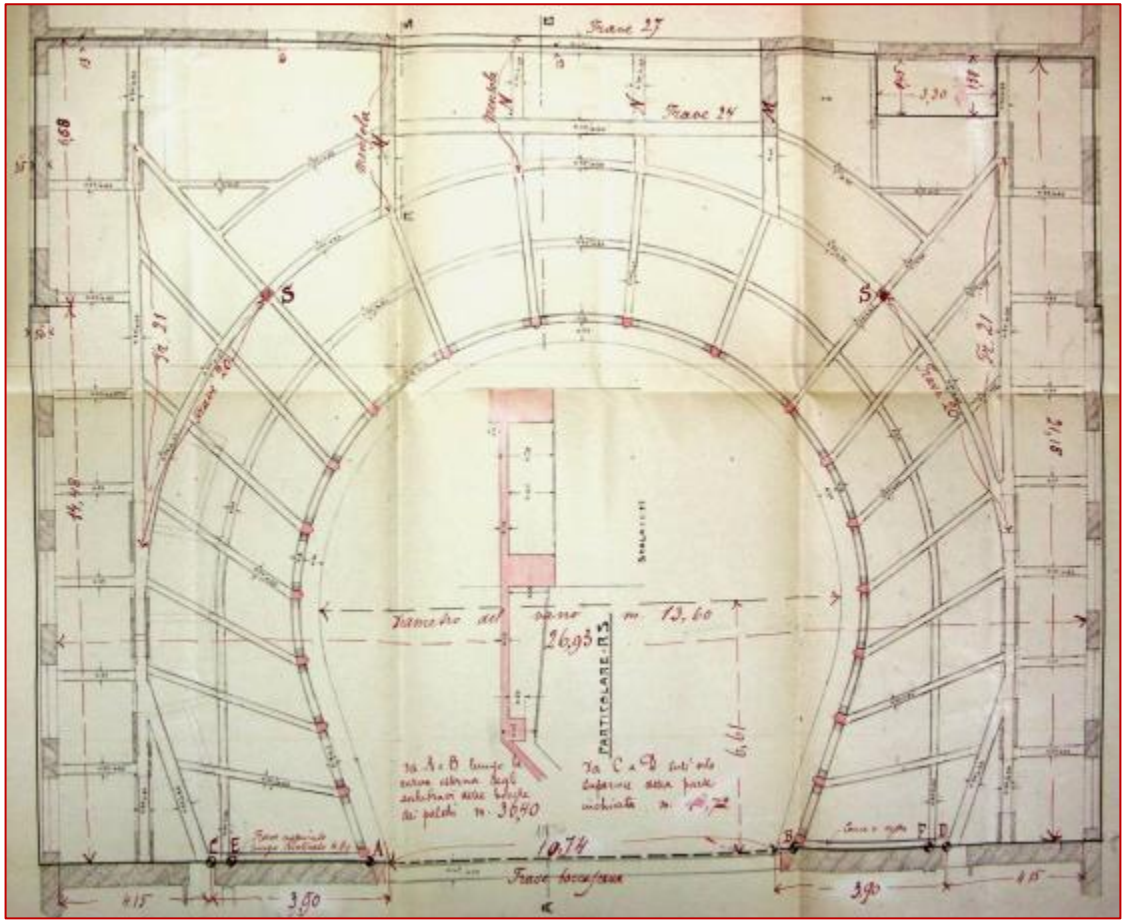
Structures

- foundations:
  - stone, h ≅ 1.50 m
- vertical structures:
  - masonry walls
  - RC columns → Hennebique system
- horizontal structures:
  - beams and floors → Hennebique system
- roof structures:
  - timber trusses

Baice,1993

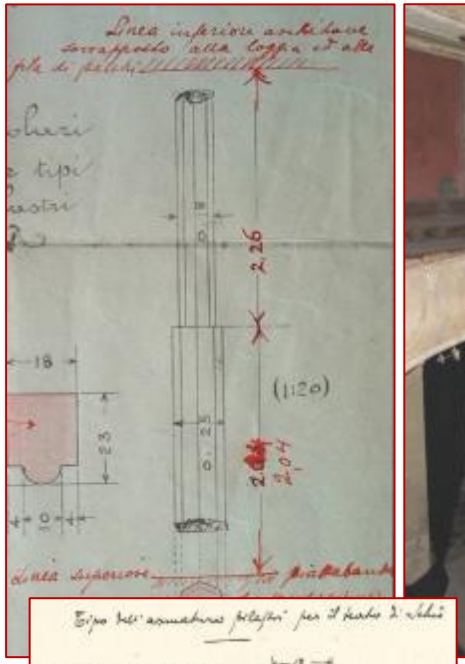
CIVIC THEATRE – SCHIO (VI)

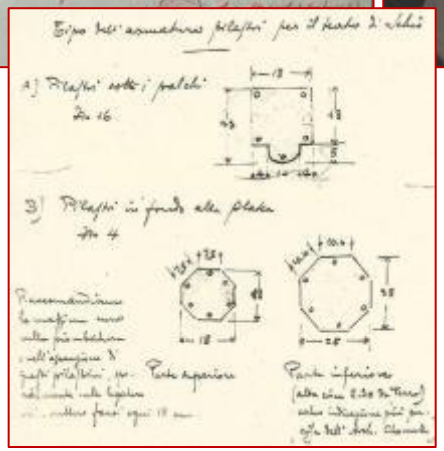
Hennebique system



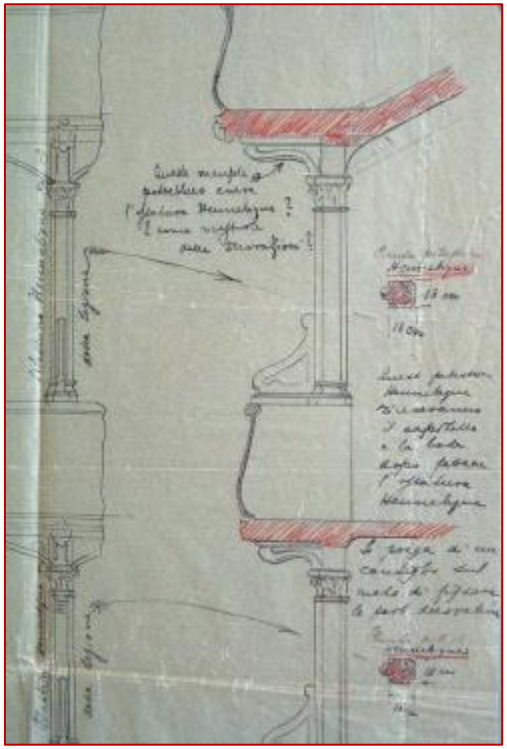
CIVIC THEATRE – SCHIO (VI)

Hennebique system





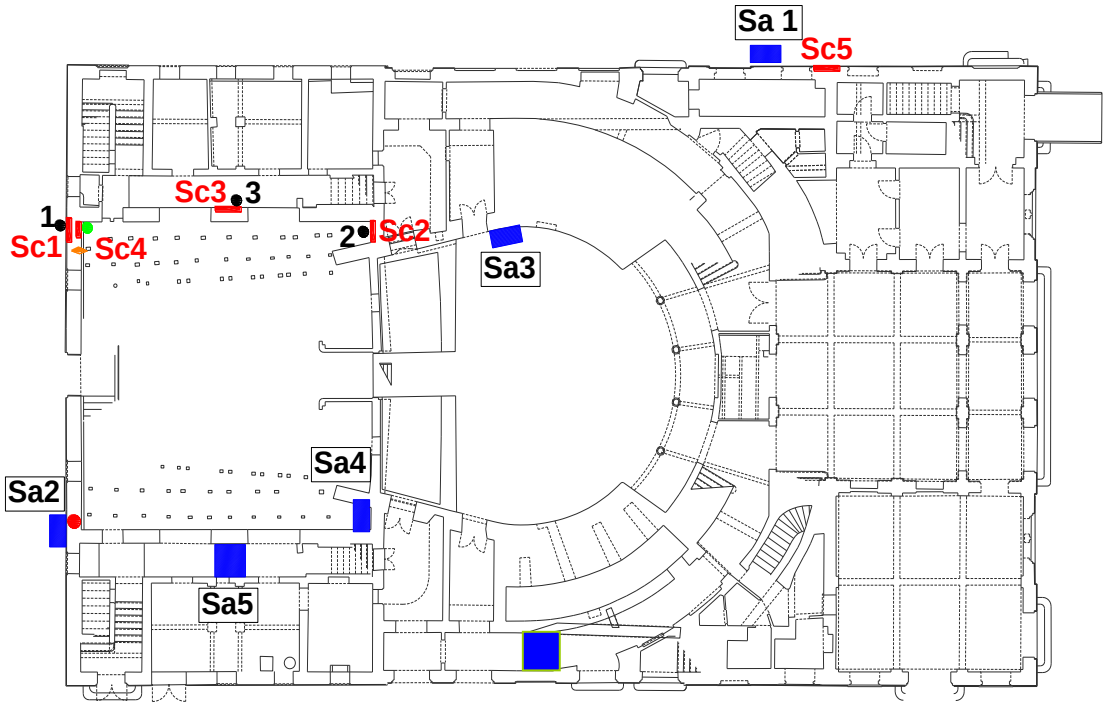
Porcheddu archive, Torino



Porcheddu archive, Torino

CIVIC THEATRE – SCHIO (VI)

Investigation campaign



CIVIC THEATRE – SCHIO (VI)

Geotechnical investigations





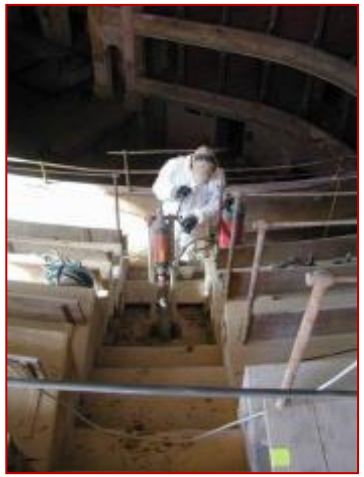



CIVIC THEATRE – SCHIO (VI)

Investigations in situ on masonry walls

CIVIC THEATRE – SCHIO (VI)

Investigations in situ on RC elements



CIVIC THEATRE – SCHIO (VI)

Interventions

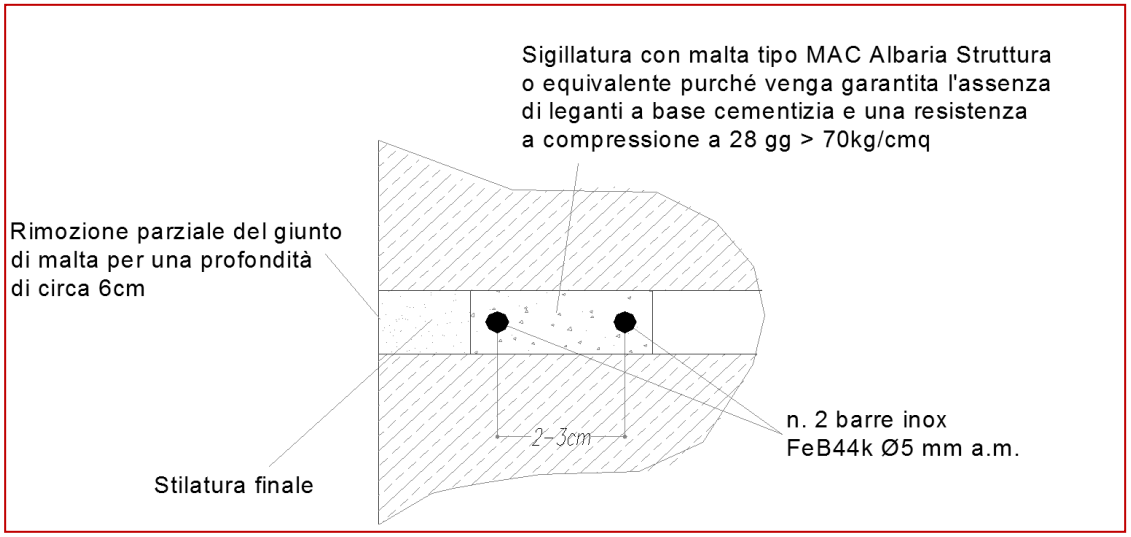
- rehabilitation of vertical masonry structures
- retrofitting of RC structures
- strengthening of existing stairways
- retrofitting of roof structures

CIVIC THEATRE – SCHIO (VI)

Interventions

Rehabilitation of vertical masonry structures:

- 'scuci-cuci' technique
- injections of slurries based on hydraulic lime
- repointing of joints after elimination of degraded mortar
- wall reconstructions in correspondence of vertical cavities



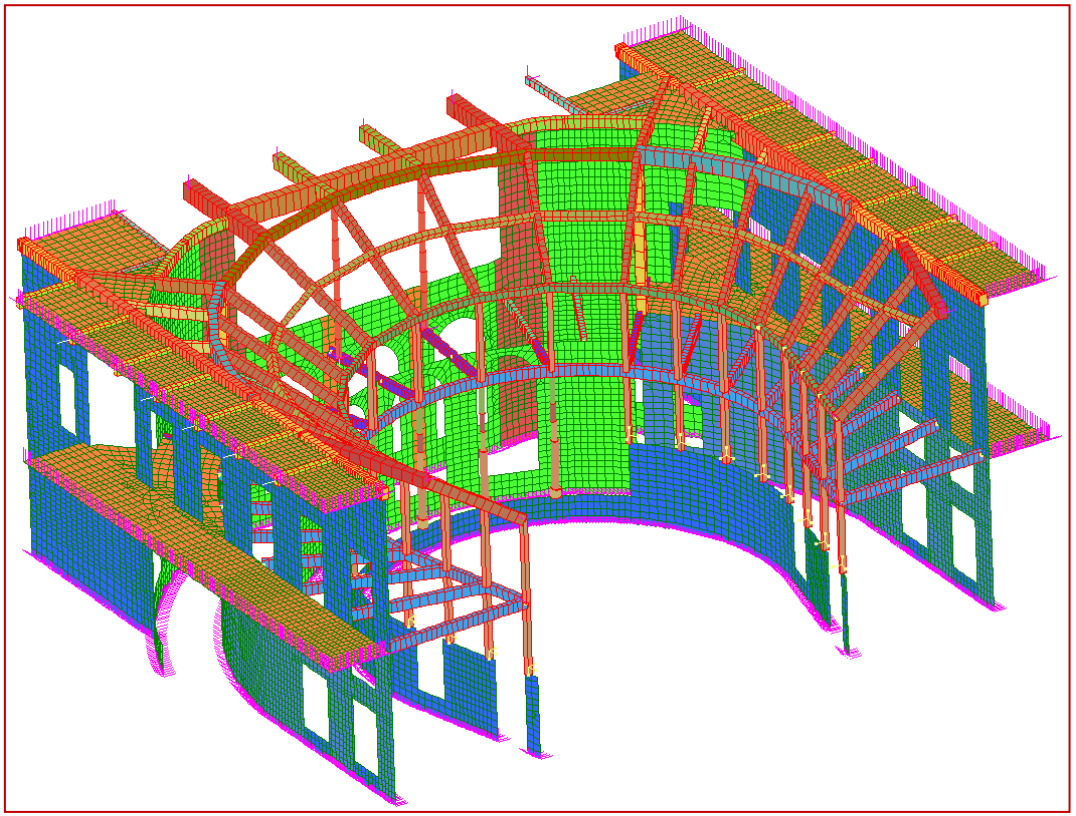
Strengthening intervention on masonry walls using reinforcement bars with a small diameter

CIVIC THEATRE – SCHIO (VI)

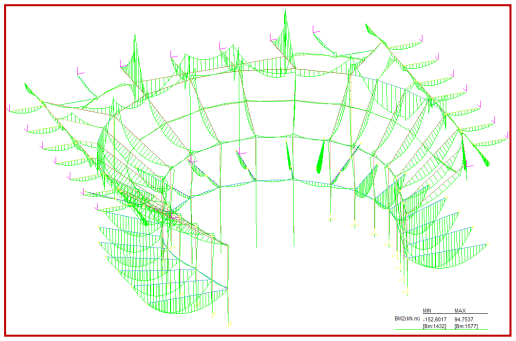
Interventions

Retrofitting of RC structures:

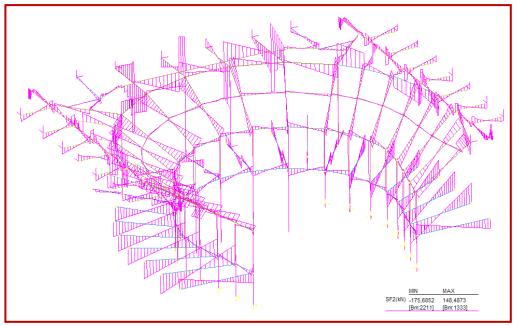
- 3D FEM model



Mesh



Bending moment



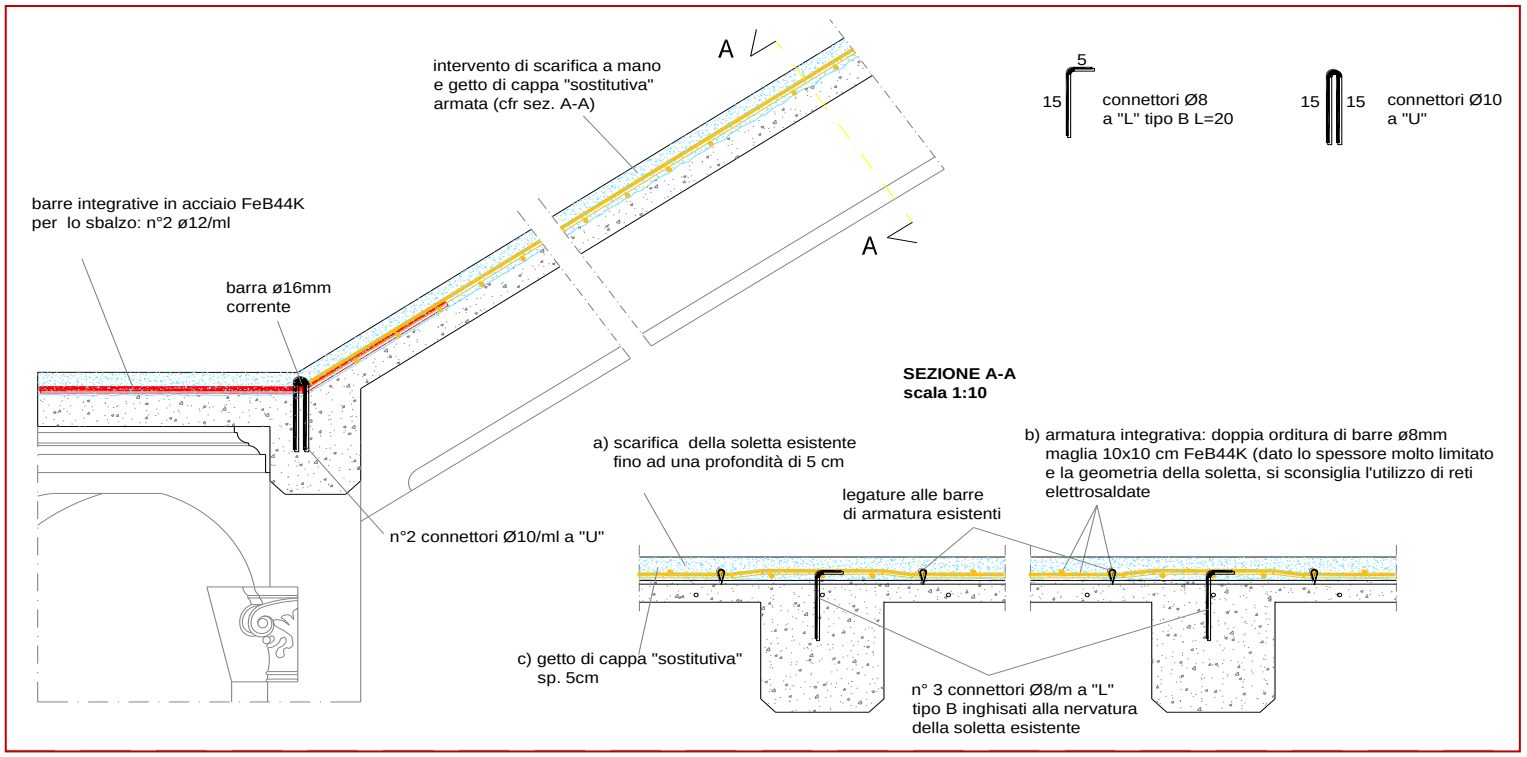
Shear

CIVIC THEATRE – SCHIO (VI)

Interventions

Retrofitting of RC structures:

- strengthening detail with a RC slab

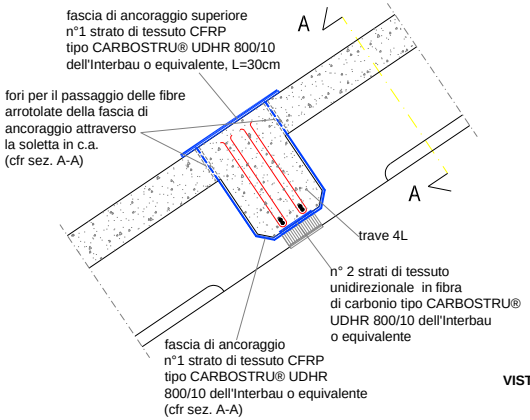


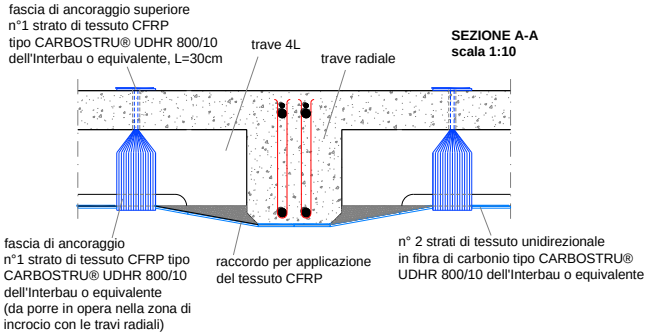
CIVIC THEATRE – SCHIO (VI)

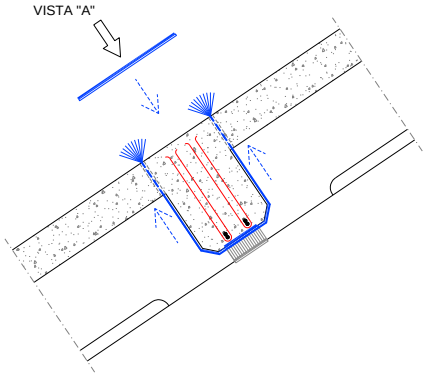
Interventions

Retrofitting of RC structures:

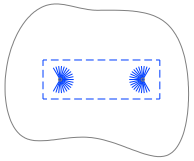
- strengthening detail of beam with FRP



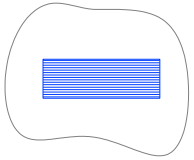




VISTA "A" - DESCRIZIONE DELLE FASI OPERATIVE



a) inserimento dei filamenti di fibra di carbonio arrotolati attraverso due fori di piccolo diametro praticati sulla soletta in c.a. e apertura a ventaglio degli stessi a contatto con l'estradosso della soletta.



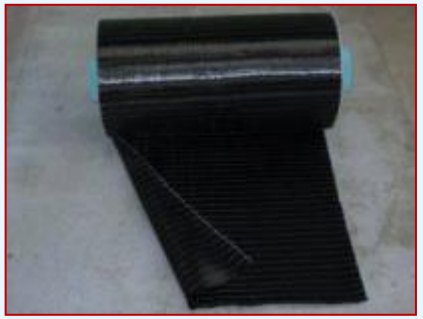
b) incollaggio della fascia superiore (n°1 strato 10x30cm) per l'ancoraggio dei filamenti all'estradosso della soletta in c.a.

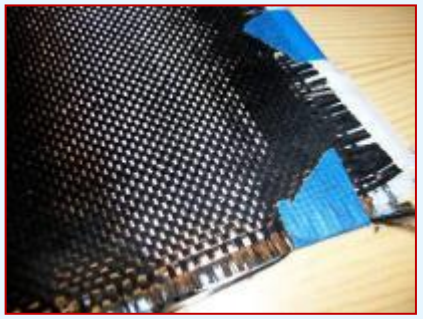
INDICAZIONI OPERATIVE:

- pulizia e regolarizzazione della superficie;
- eventuale applicazione del primer;
- eventuale applicazione della rasatura;
- applicazione dell'adesivo tipo CARBOSTRU® AD;
- applicazione della fascia in tessuto unidirezionale in fibra di carbonio tipo CARBOSTRU® UDHR 800 o equivalente;
- applicazione dello strato finale di adesivo.

NOTA: rispettare in ogni caso le indicazioni riportate nella documentazione tecnica del produttore.

Carbon fiber fabric





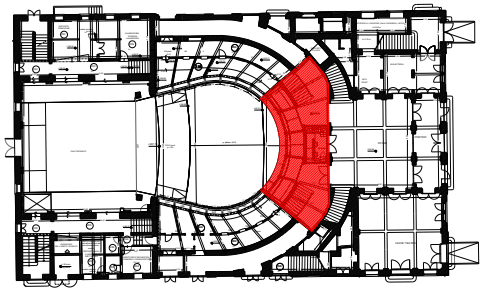
CIVIC THEATRE – SCHIO (VI)

Interventions

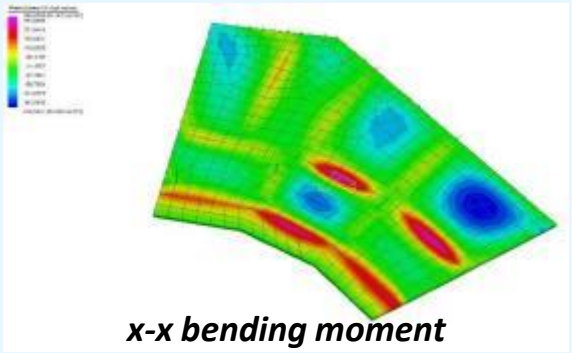
Retrofitting of RC structures:

- local FEM model

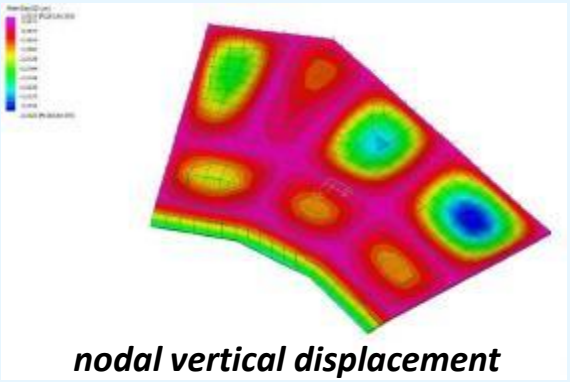
slab thickness = 12 cm



Finite Element Model

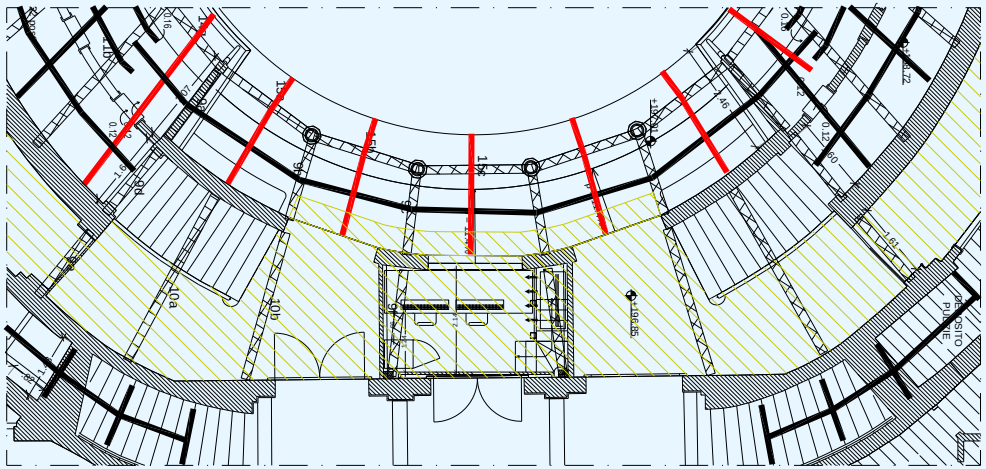


x-x bending moment



nodal vertical displacement

Strengthening interventions



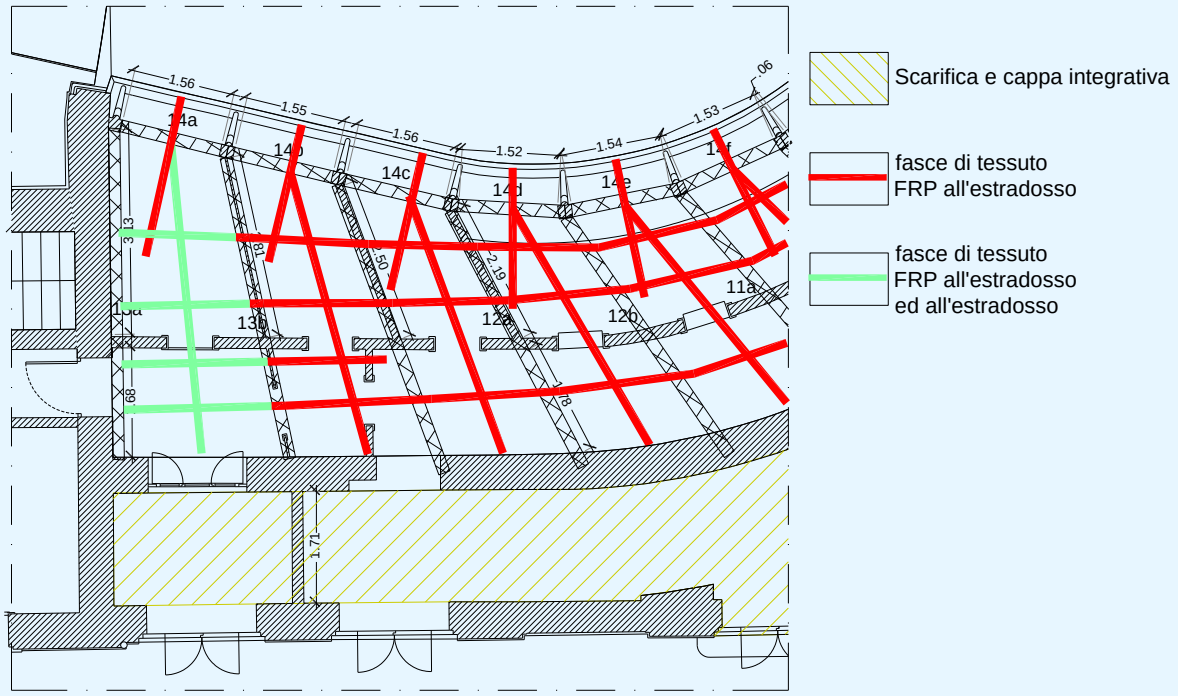
CIVIC THEATRE – SCHIO (VI)

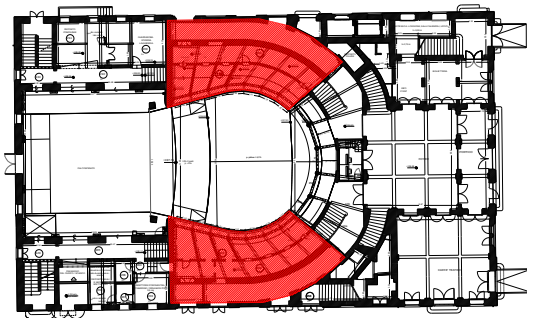
Interventions

Retrofitting of RC structures:

- strengthening detail of slab with FRP

Strengthening interventions





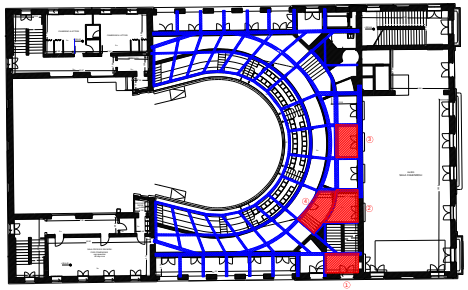
slab thickness = 12 cm

CIVIC THEATRE – SCHIO (VI)

Interventions

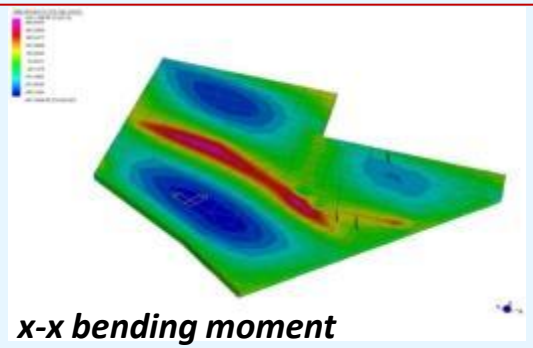
Retrofitting of RC structures:

- local FEM model

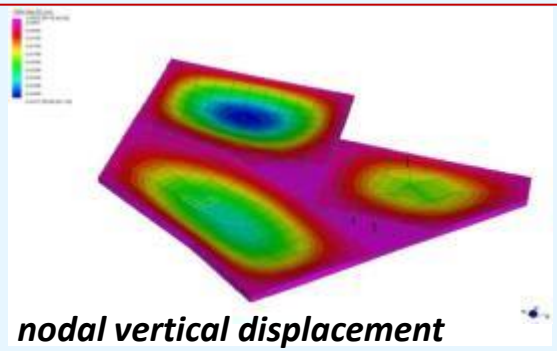


slab thickness = 10 cm

Finite Element Model

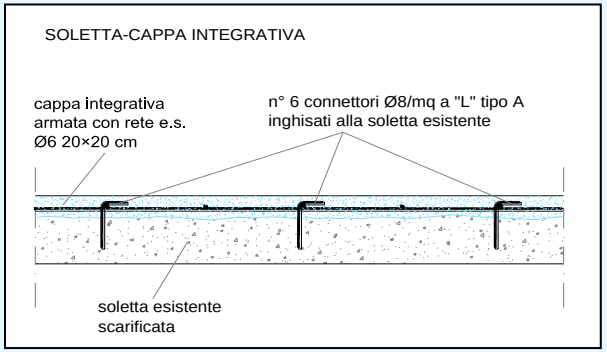


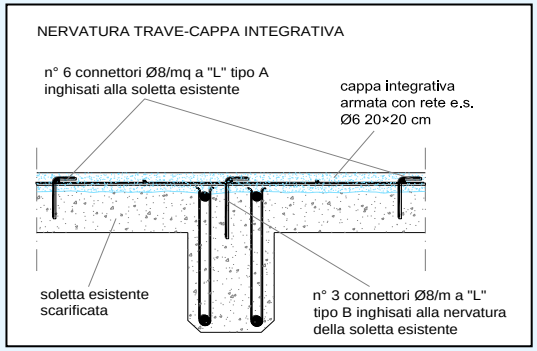
x-x bending moment



nodal vertical displacement

Strengthening details of the added RC slab



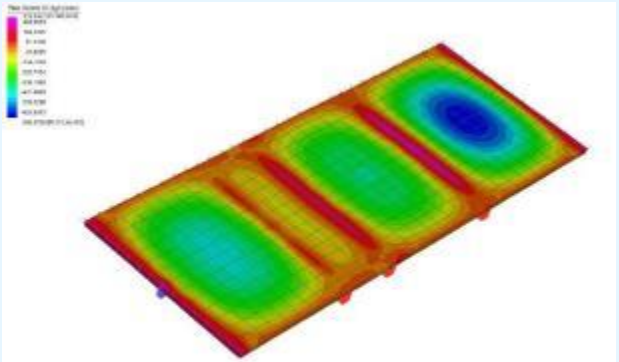


CIVIC THEATRE – SCHIO (VI)

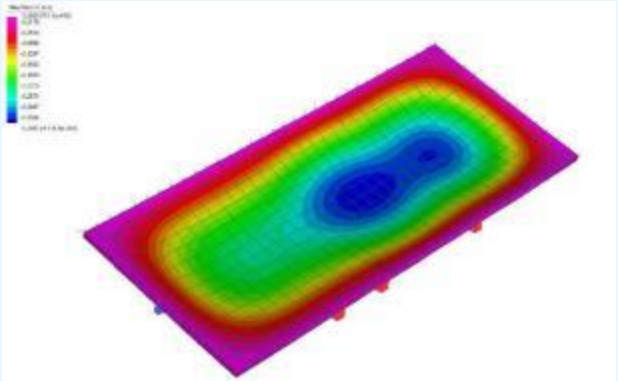
Interventions

Retrofitting of RC structures:

Finite Element Model

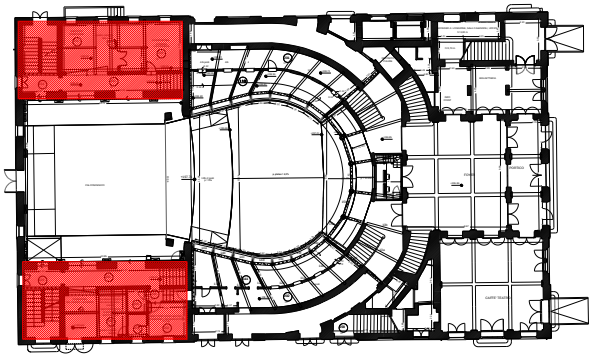


x-x bending moment

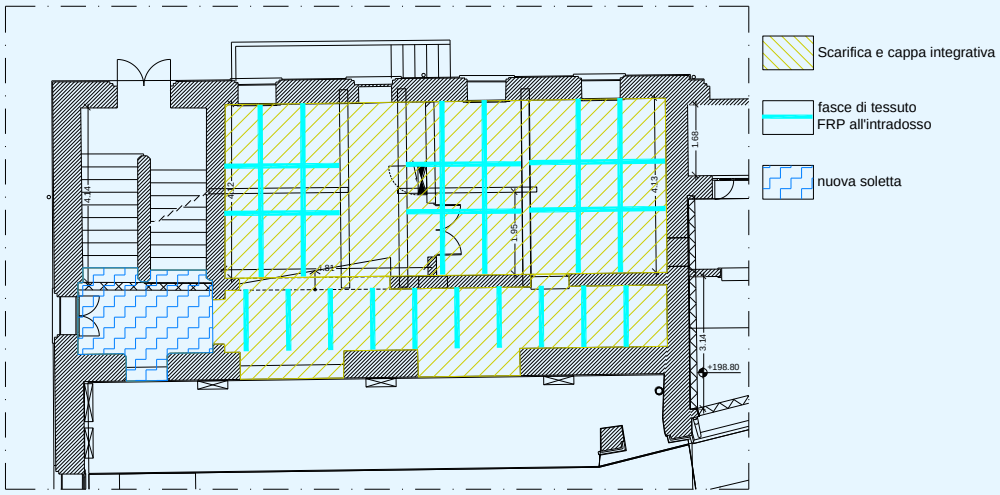


nodal vertical displacement

slab thickness = 10 cm



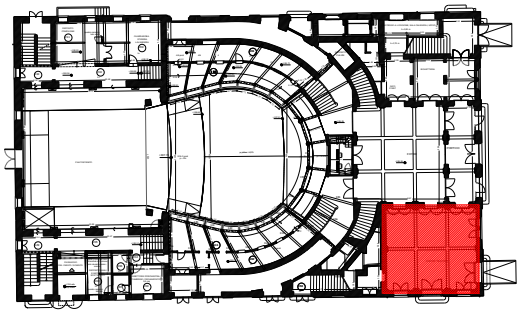
Strengthening interventions



CIVIC THEATRE – SCHIO (VI)

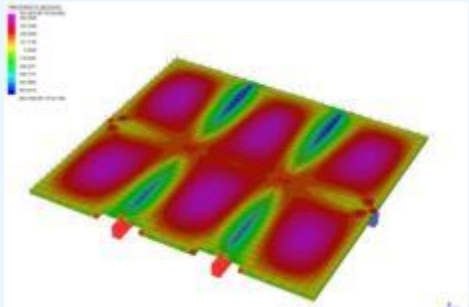
Interventions

Retrofitting of RC structures

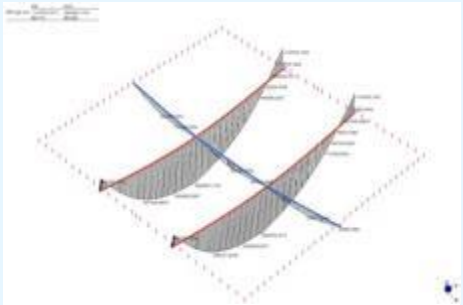


slab thickness = 10 cm

Finite Element Model

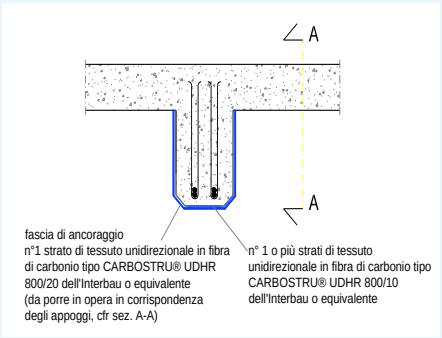


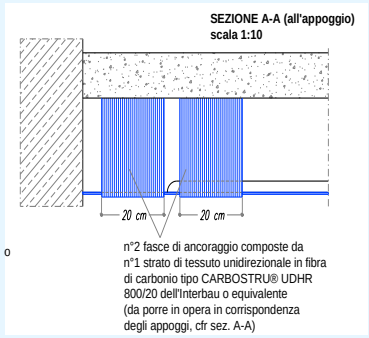
Bending moment on a RC slab



bending moment on beams

Strengthening interventions with an added RC slab



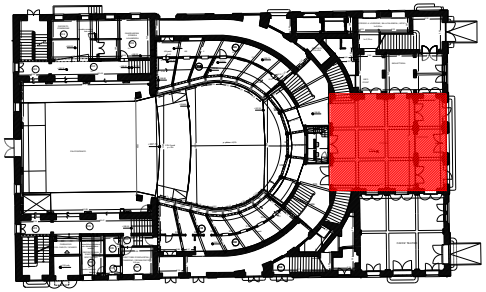


CIVIC THEATRE – SCHIO (VI)

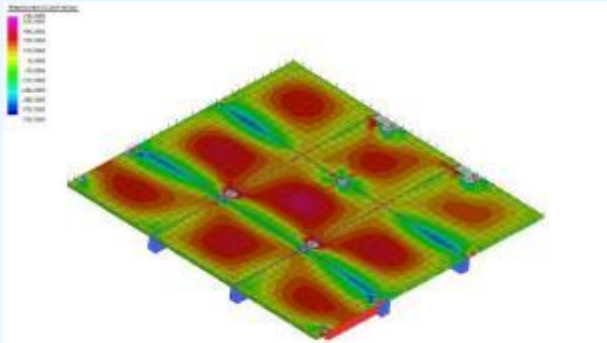
Interventions

Retrofitting of RC structures

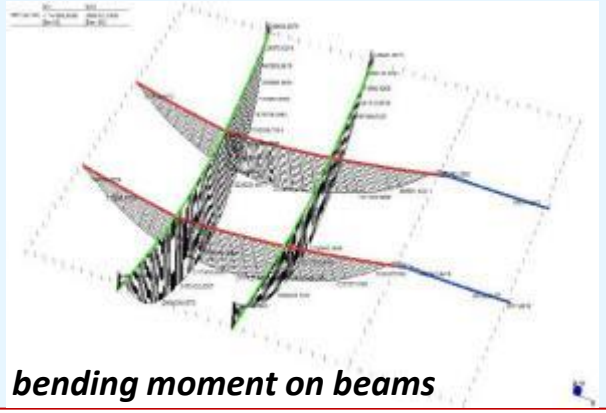
slab thickness = 10 cm



Finite Element Model

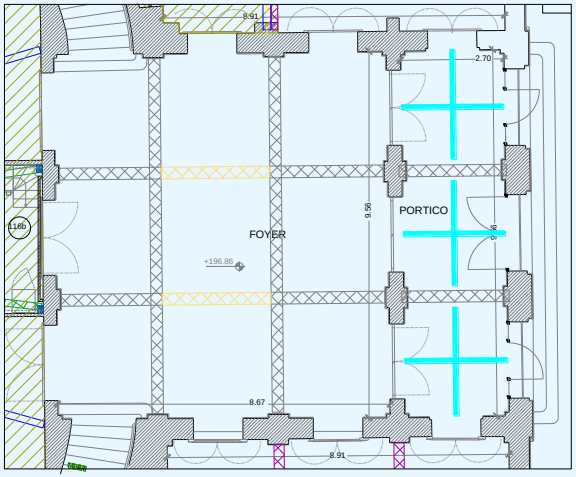


x-x bending moment on a RC slab



bending moment on beams

Strengthening interventions



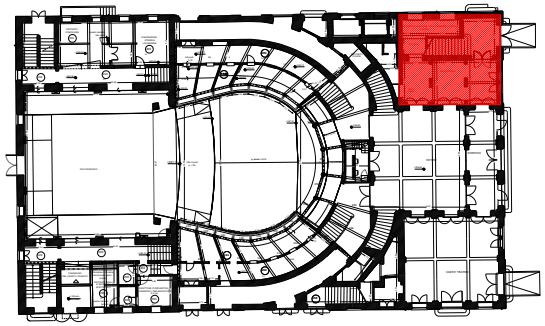
Rinforzo solaio con applicazione di fasce di tessuto FRP (n° 1 strato b=10 cm)

Trave rinforzata con tessuto FRP (n° 2 fasce n° 1 strato b=10 cm)

CIVIC THEATRE – SCHIO (VI)

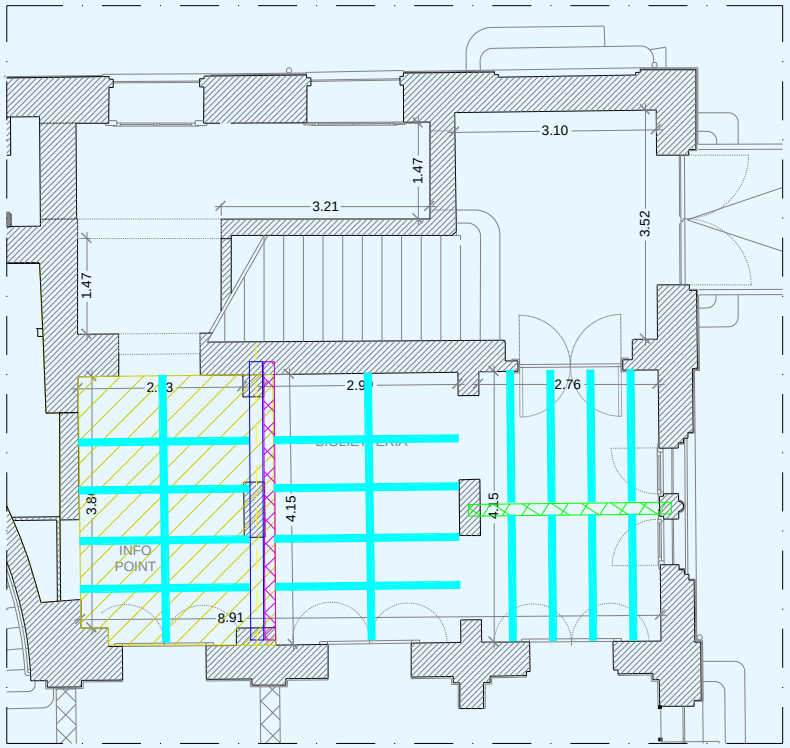
Interventions

Retrofitting of RC structures



slab thickness = 10 cm

Strenghtening interventions



Trave rinforzata con tessuto FRP (n° 1 strato b=10 cm)

Rinforzo solaio con applicazione di fasce di tessuto FRP (n° 1 strato b=10 cm)

THANKS FOR YOUR ATTENTION

Prof. Eng. Claudio Modena