EARTHQUAKE PLANNING AND PROTECTION ORGANIZATION (E.P.P.O.)

COUNCIL OF EUROPE
EUROPEAN CENTRE ON PREVENTION AND FORECASTING OF EARTHQUAKES (E.C.P.F.E.)

TRAINING COURSE ON SEISMIC RISK ASSESSMENT IN SPECIFIC AREAS WITH MONUMENTAL STRUCTURES

Athens - December 6-10, 2010

Mechanical models for the seismic performance based assessment of monuments and the design of interventions

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Performance Based Assessment

Earthquake is a rare environmental action, that must by defined through a probabilistic approach (**hazard scenario**)

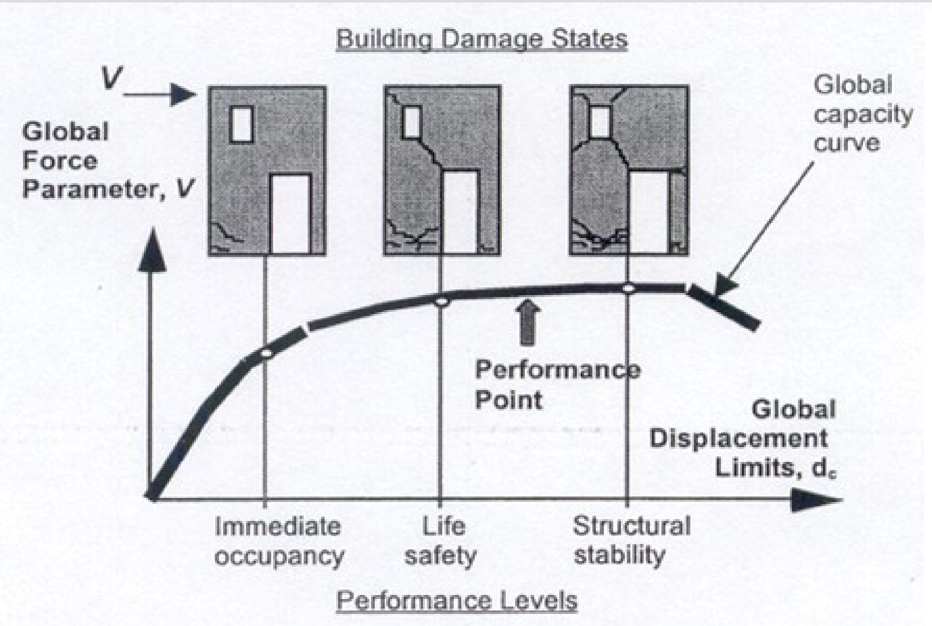
It is impossible to require that an ancient masonry construction withstand without any damage to strong ground motions. Usually we ask that buildings survive to a strong earthquake and suffer few damage for a low intensity earthquake (**performance levels**)

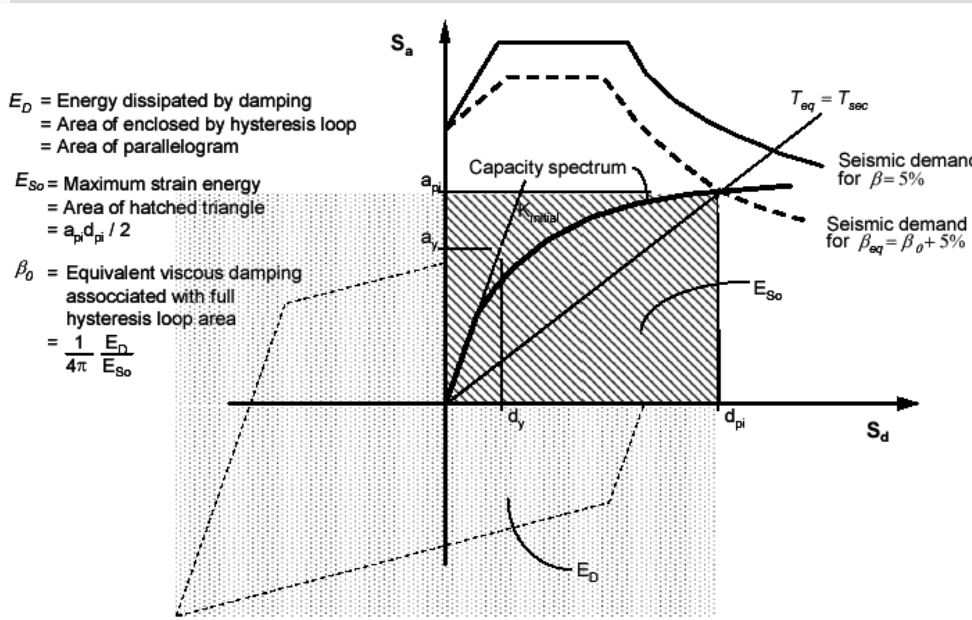
↓

PERFORMANCE BASED ASSESSMENT AND DESIGN

- FEMA 356 (U.S. Federal Emergency Management Agency), Nov. 2000 – Prestandard and commentary for the seismic rehabilitation of buildings
- Eurocode 8: Design of structures for earthquake resistance - Part 3: Assessment and retrofitting of buildings (June 2005)
- Italian Technical Code for the design of constructions (January 2008)

Performance Based Assessment





- Earthquake induces horizontal actions on structures.
- Pushover analysis evaluates the behaviour till to collapse. Increasing the displacement: the stiffness decreases, the maximum strength occurs, some constructions may profit by ductility.
- **Performance levels** (or limit states) can be defined on the pushover curve.
- Once the **earthquake hazard level** is defined, displacement demand may be evaluated by CSM (**capacity spectrum method**), using an acceleration-displacement response spectrum, reduced by considering hysteretic damping.

Performance Based Assessment

REHABILITATION OBJECTIVES

Building performance level		Earthquake hazard levels
FEMA 356		
LS1 Operational performance		p=50% in 50 years (T_R=72 years)
LS2 Immediate occupancy		p=20% in 50 years (T_R=225 years)
LS3 Life safety	→	p=10% in 50 years (T_R=475 years)
LS4 Collapse prevention	→	p=2% in 50 years (T_R=2475 years)
Eurocode 8 – Part 3		
LS-DL Damage limitation		p=20% in 50 years (T_R=225 years)
LS-SD Significant damage		p=10% in 50 years (T_R=475 years)
LS-NC Near collapse		p=2% in 50 years (T_R=2475 years)
Eurocode 8 – Part 1		
LS-DL Damage limitation		p=10% in 10 years (T_R=95 years)
LS-NC No-collapse		p=10% in 50 years (T_R=475 years)
NTC 2008 (Italian Technical Code)		
LS-O Operational performance		p=81% in V_R years
LS-D Damage limitation	→	p=63% in V_R years
LS-V Life safety	→	p=10% in V_R years
LS-C Collapse prevention		p=5% in V_R years

V_R reference life

REHABILITATION OBJECTIVES

Building performance level	Earthquake hazard levels
FEMA 356	
LS1 Operational performance	p=50% in 50 years (T_R=72 years)
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Eurocode 8 – Part 3	
LS-DL Damage limitation	p=20% in 50 years (T_R=225 years)
LS-SD Significant damage	p=10% in 50 years (T_R=475 years)
LS-NC Near collapse	p=2% in 50 years (T_R=2475 years)
Eurocode 8 – Part 1	
LS-DL Damage limitation	p=10% in 10 years (T_R=95 years)
LS-NC No-collapse	p=10% in 50 years (T_R=475 years)
NTC 2008 (Italian Technical Code)	
LS-O Operational performance	p=81% in 50 years (T_R=30 years)
LS-D Damage limitation →	p=63% in 50 years (T_R=50 years)
LS-V Life safety →	p=10% in 50 years (T_R=475 years)
LS-C Collapse prevention	p=5% in 50 years (T_R=975 years)

Italian Guidelines for Cultural Heritage

Italian Guidelines
for evaluation and mitigation of seismic risk to cultural heritage

Ministry of Cultural Heritage – Civil Protection Department – Ministry of Public Works
(NTC 2008 - Italian Technical Code)

Building performance level		Earthquake hazard levels
LS-DL	Damage limitation	p=63% in V_R years
LS-V	Life safety	p=10% in V_R years
LS-A	Artistic assets	p=63% in years

REFERENCE LIFE - $V_R = V_N C_U$

NOMINAL LIFE - Is the time in which the building can be considered safe (serviceable period), only assuring structural health monitoring and proper maintenance ($V_N \leq 50$ years)

USE COEFFICIENT C_U - a) occasional 0.7; b) frequent 1.0;
c) frequent with crowding 1.5

Probability of exceedance and return periods

Tabella 3.2.I – *Probabilità di superamento* P_{V_R} *al variare dello stato limite considerato*

Stati Limite		P_{V_R} : **Probabilità di superamento nel periodo di riferimento V_R**
Stati limite di esercizio	SLO	81%
	SLD	63%
Stati limite ultimi	SLV	10%
	SLC	5%

$$T_R = \frac{1}{1 - e^{\frac{\ln(1-P_{V_R})}{V_R}}} \cong -\frac{V_R}{\ln(1-P_{V_R})}$$

	P_{Vr}				
SLD	**63%**	**35**	**50**	**75**	**101**

$$P_{V_R} = 1 - e^{-V_R/T_R}$$

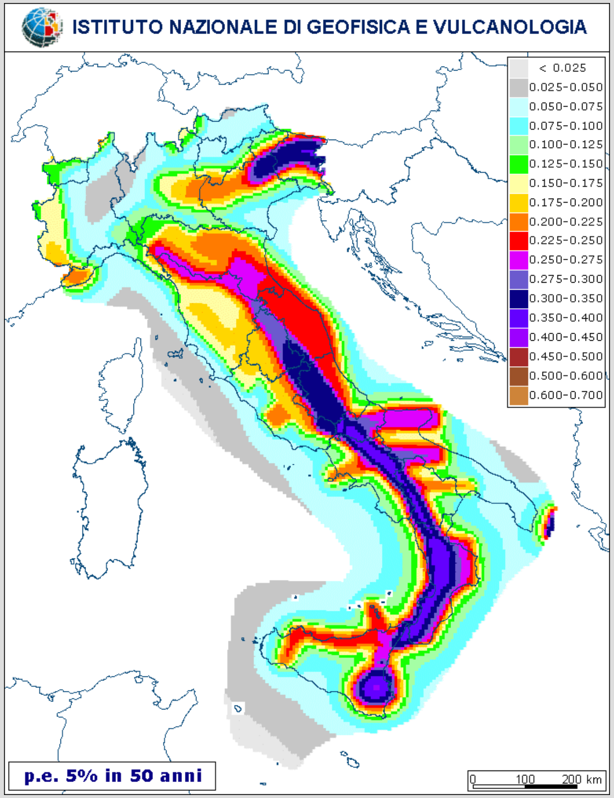
$$P_{annuale} = 1 - e^{-1/T_R}$$

	C_U	**0.7**	**1**	**1.5**	**2**
$V_N = 50$	V_R	**35**	**50**	**75**	**100**
	P_{Vr}	**Annual probability of exceedance**			
SLD	**63%**	**2.8%**	**2%**	**1.3%**	**1%**
SLV	**10%**				

INGV-DPC Framework project 2004-2006
S1 Project
Seismic Hazard in Italy: PGA for different occurrence probabilities in 50 years

C. Meletti, V. Montaldo

Occurence probability in 50 years	Return period	Annual rate of excedance	File (download)
81%	30	0.0332	(file zip - 631 kb)
63%	50	0.0199	(file zip - 642 kb)
50%	72	0.0139	(file zip - 651 kb)
39%	101	0.0099	(file zip - 657 kb)
30%	140	0.0071	(file zip - 650 kb)
22%	201	0.0050	(file zip - 666 kb)
10%	475	0.0021	**MPS04**
5%	975	0.0010	(file zip - 682 kb)
2%	2475	0.0004	(file zip - 686 kb)



http://esse1.mi.ingv.it/d2.html

INGV – National Institute of Geophysics and Vulcanology

DPC – Civil Protection Department

INGV-DPC Framework project 2004-2006
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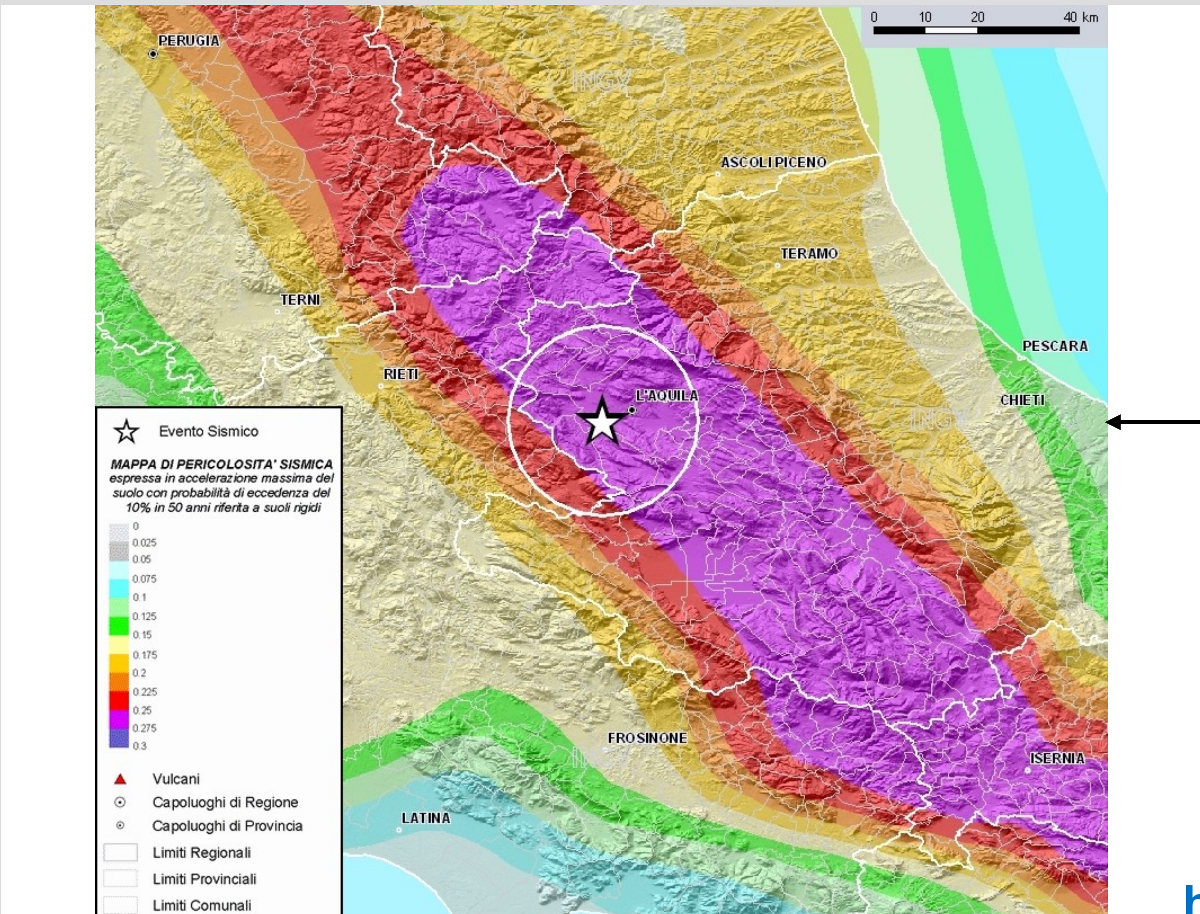

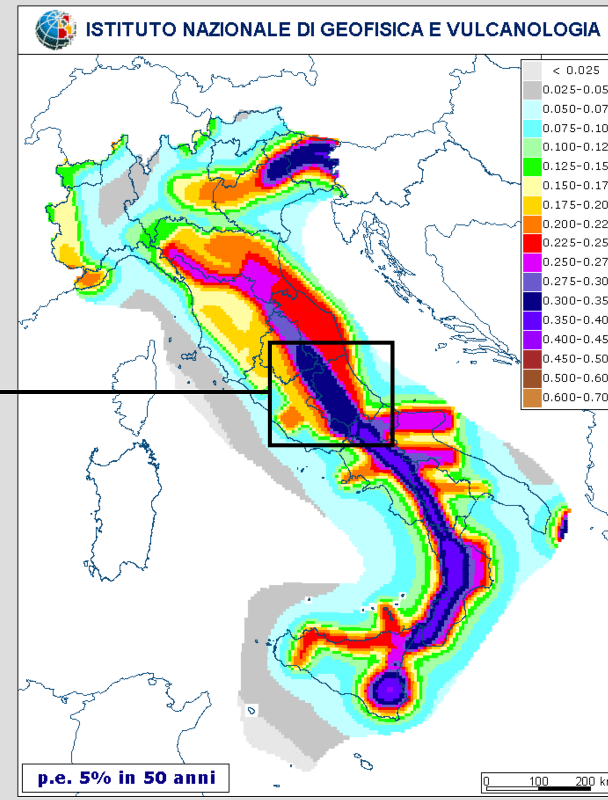

http://esse1.mi.ingv.it/d2.html

Performance based assessment

Tabella 3.2.I – *Probabilità di superamento* P_{V_R} *al variare dello stato limite considerato*

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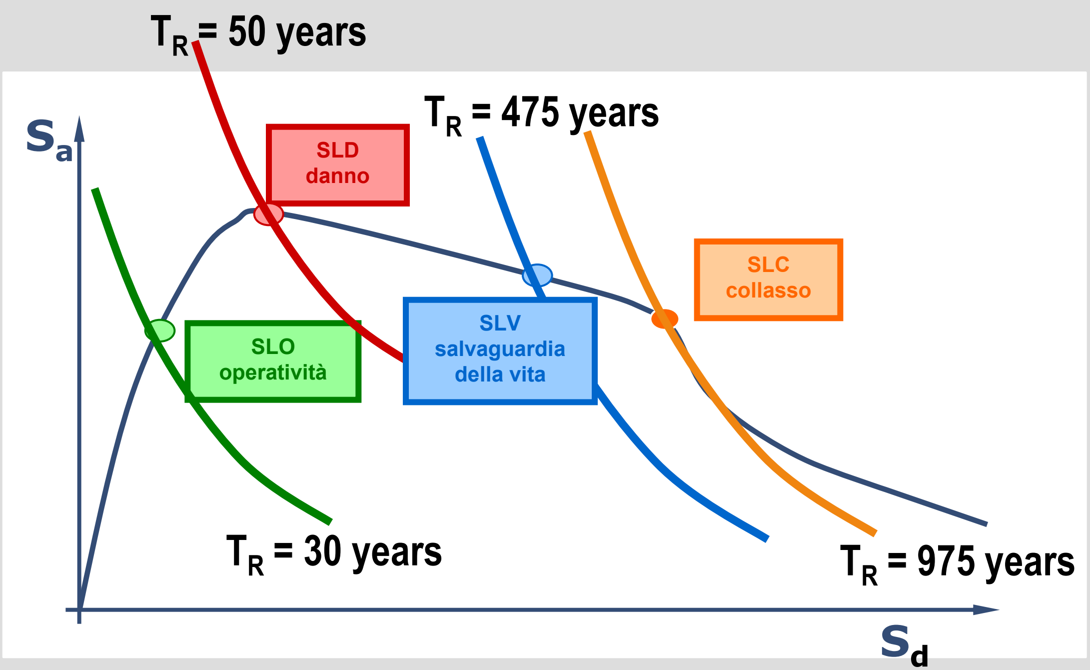
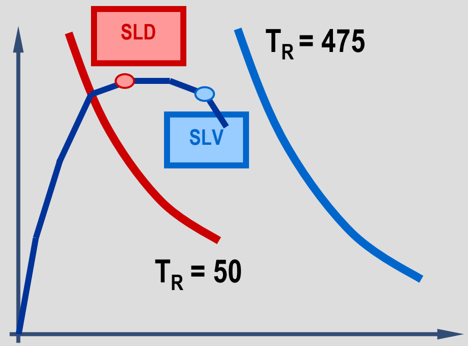


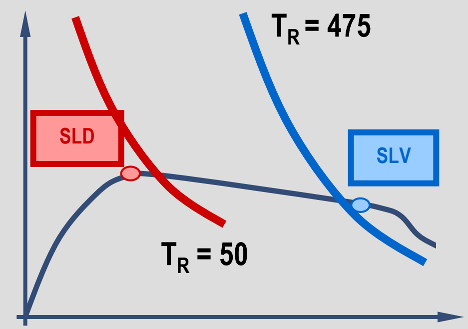
V_N = 50 years

C_U = 1

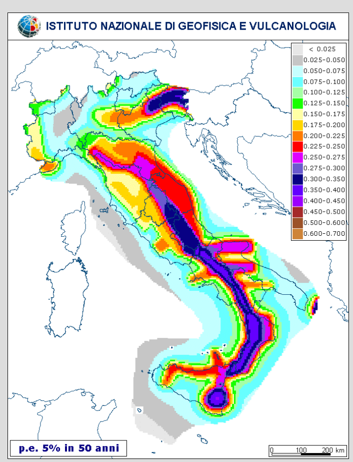
Why it is necessary to consider different limit state?

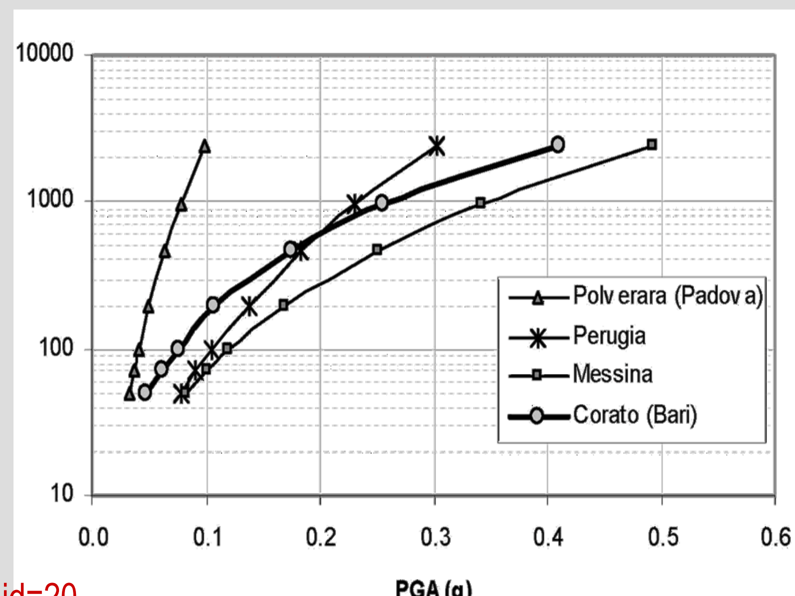
STRENGTH and DUCTILITY





SEISMIC HAZARD





Sito da dove scaricare il programma “Spettri di risposta”

http://www.cslp.it/cslp/index.php?option=com_content&task=view&id=75&Itemid=20

Guidelines for Cultural Heritage

NOMINAL LIFE V_N - Is the time in which the building can be considered safe (serviceable period), only assuring structural health monitoring and proper maintenance ($V_N \leq 50$ years)

What does it means?

If I made a safety check for a monumental building considering V_N=50 years, does it mean that the monument will stand only for 50 years?

Is the age of the monument important in this evaluation?

What can I do after 50 years from this evaluation?

Some short and preliminary answers:

The age of the monument is not important, as I diagnose the actual state of the building after proper investigations.

50 years from now I will evaluate again the seismic safety, considering an updated hazard scenario (possibly obtained by time-dependent models).

THE USE OF NOMINAL LIFE

Nominale life: Is the time in which the building can be considered safe, only assuring structural health monitoring and proper maintenance .

It is not the residual life (LCA – life cycle assessment), evaluated considering the decay of materials, but only the period in which the building can be considered safe (from a probabilistic point of view).

The nominal life of a monument should be very long, because we would like to preserve the building forever; to this end we should consider the earthquake action related to a long return period (i.e. the maximum possible earthquake in the site).

However, this would lead to th design of invasive interventions, not compatible with the conservation. So the interventions can refer to a reduced value of the nominal life.

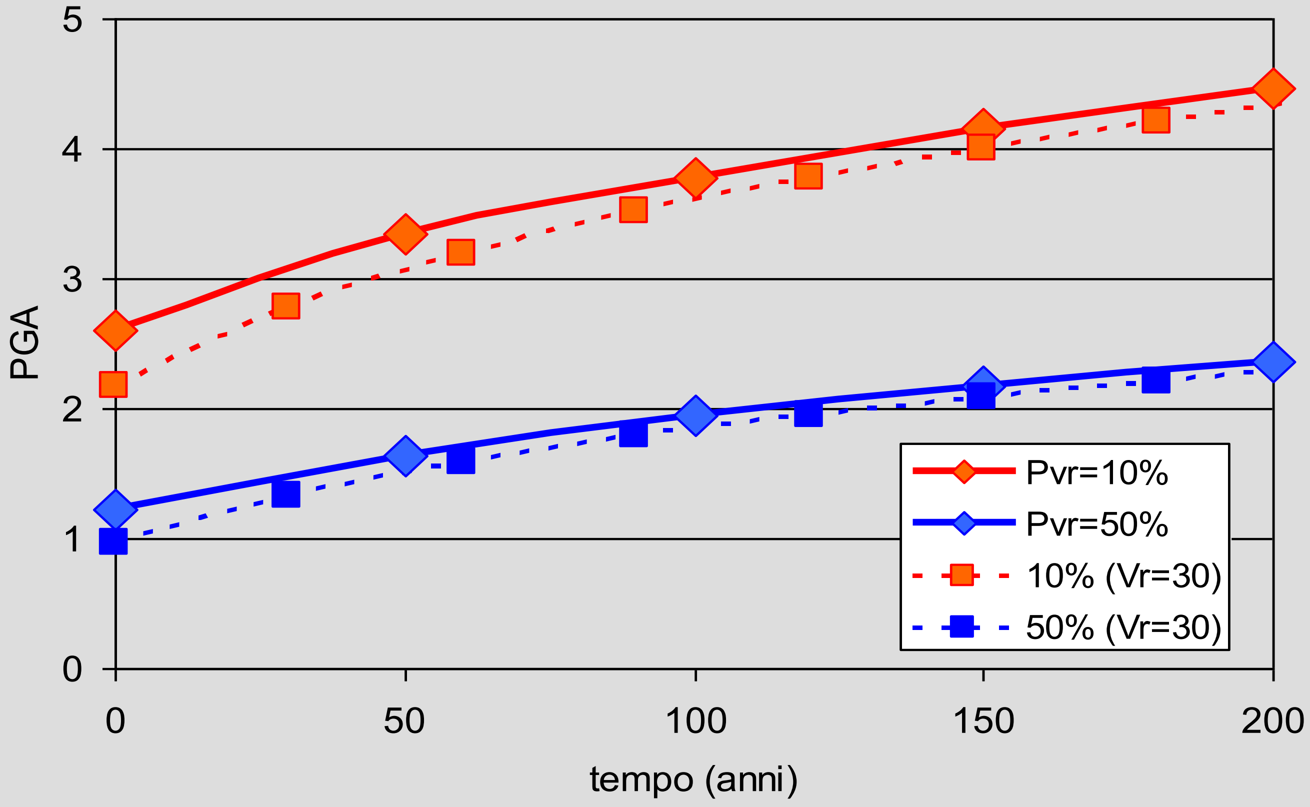
Further interventions will be eventually adopted in the future, at the end of the nominal life assumed today. At that time further information will be available on:
1) seismic hazard; 2) building capacity (new more detailed mechanical models);
3) strengthening interventions (more effective and less invasive).

When we design a restoration intervention, we decide which are the effective strengthening solutions that reduce vulnerability and are compatible with the conservation criteria. Then, we evaluate VN, the time of validity of this check.

What will be necessary at the end of the nominal life?

The seismic hazard to be used for the next check will have to consider the time that has gone by (time-dependent hazard maps).

If the available hazard will still be time-independent (Poisson model), the reference period to be considered for the next check will must take into account the time that has passed.



PERformance-based aPproach to Earthquake proTection of cUlturAl heriTage in European and mediterranean countries

- Development of European Guidelines for the evaluation and mitigation of seismic risk to cultural heritage assets.
- Both architectonic assets (historic buildings; macroelements) and artistic assets (frescos, stucco-works, statues, pinnacles, battlements, banisters, balconies …) will be considered. Only masonry structures will be considered.
- Two different scales will be considered:
  a) assessment at the territorial scale including simplified vulnerability and risk analysis and policy issues for seismic risk mitigation
  b) assessment of a single cultural heritage asset and design of interventions

PERformance-based aPproach to Earthquake proTection of cUlturAl heriTage in European and mediterranean countries

PERPETUATE means “to preserve from extinction”

In the case of cultural heritage assets this means to extend their survival towards **infinity**.

The symbol of infinity is used as a conceptual key for the logo, together with the propagation of seismic waves.

PERPETUATE MANIFESTO

Time is the keyword for the conservation of cultural heritage.

Periodic maintenance protects against material decay, ensuring endless life to buildings.

Structural health monitoring controls the occurrence of damage or instability due to anthropic or environmental transformations.

Preventive actions must be adopted in order to PERPETUATE the life of monuments in seismic areas, in due time, before an earthquake interrupts their life forever.

Sergio Lagomarsino

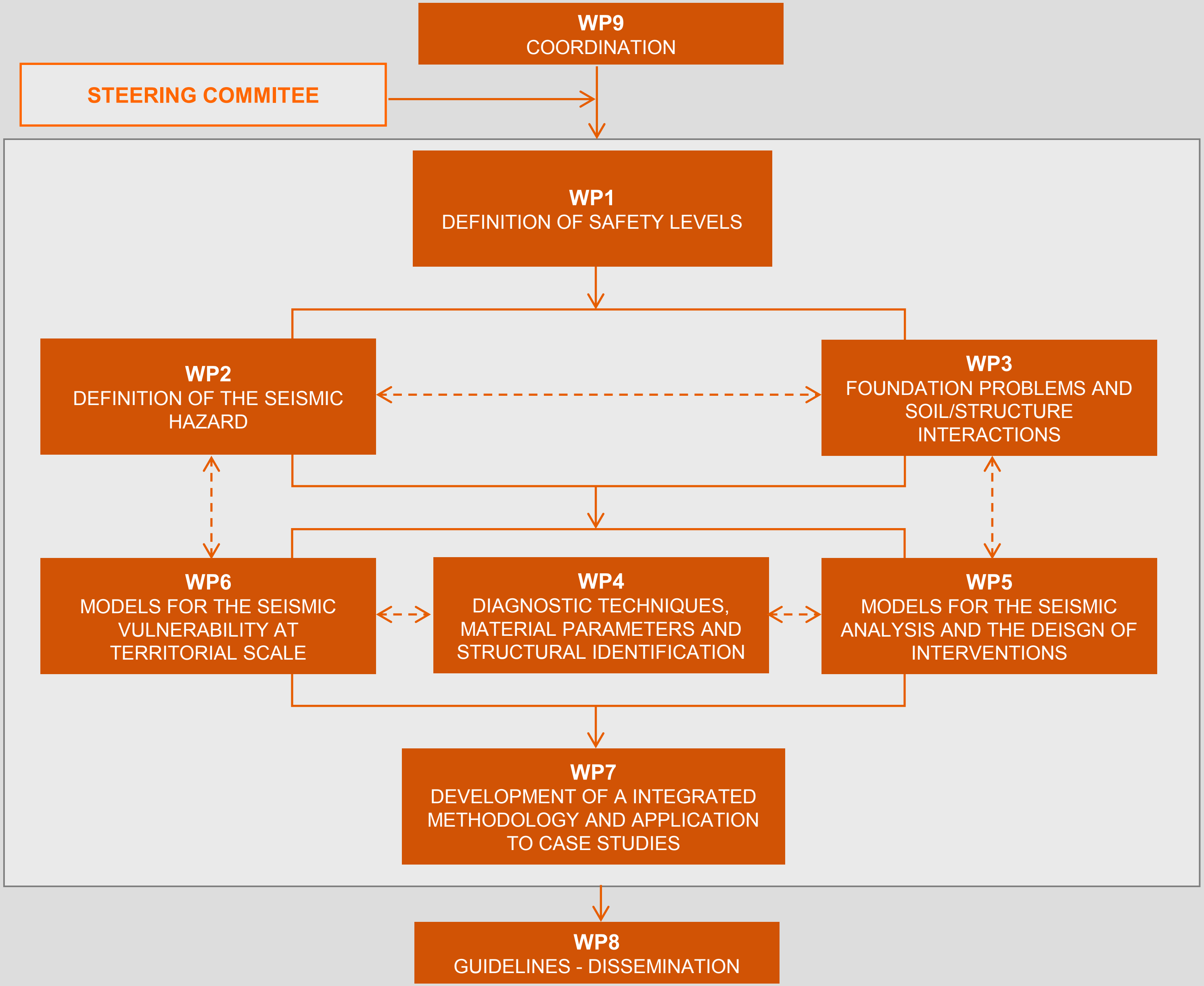
Coordinator of PERPETUATE Project

Santa Maria Church in Tempera, L'Aquila (earthquake on April 6, 2009 - h. 3:30)

PARTNERS

The Consortium consists of:
- **6 Universities** (Genoa, Thessaloniki, Athens, Ljubljana, Bath, Algiers)
- **2 Public/Research Institutions** (ENEA, Italy; BRGM, France)
- **3 SMEs** from Slovenia (ZMRK) and Italy (CENACOLO, PHASE).

PROJECT STRUCTURE

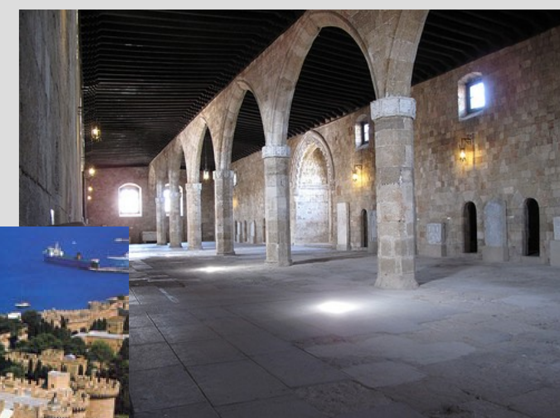


WP7 – APPLICATION TO CASE STUDIES

perpetuate

TASK 8.2: Application to the Citadel and the Great Mosque of Algiers (Algeria)

TASK 8.3: Application to the historical centre of Rhodes (Greece)



WP7 – APPLICATION TO CASE STUDIES

TASK 8.4: Application to the case studies selected in the Abruzzo region (Italy)

TASK 8.5: Application to the St. Pardo Cathedral in Larino (Molise Region, Italy)

TASK 8.6: Application to the Cathedral St. Nicholas in Ljubljana

SHAKING TABLE TESTS AT ENEA, ROME


Perpetuate | Subline description - Windows Internet Explorer
http://www.perpetuate.eu/
File Modifica Visualizza Preferiti Strumenti ?
Google
Effettua la ricerca
Entra
Preferiti
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PERFORMANCE-BASED APPROACH TO EARTHQUAKE PROTECTION OF CULTURAL HERITAGE IN EUROPEAN AND MEDITERRANEAN COUNTRIES
European Research Project funded by FP7
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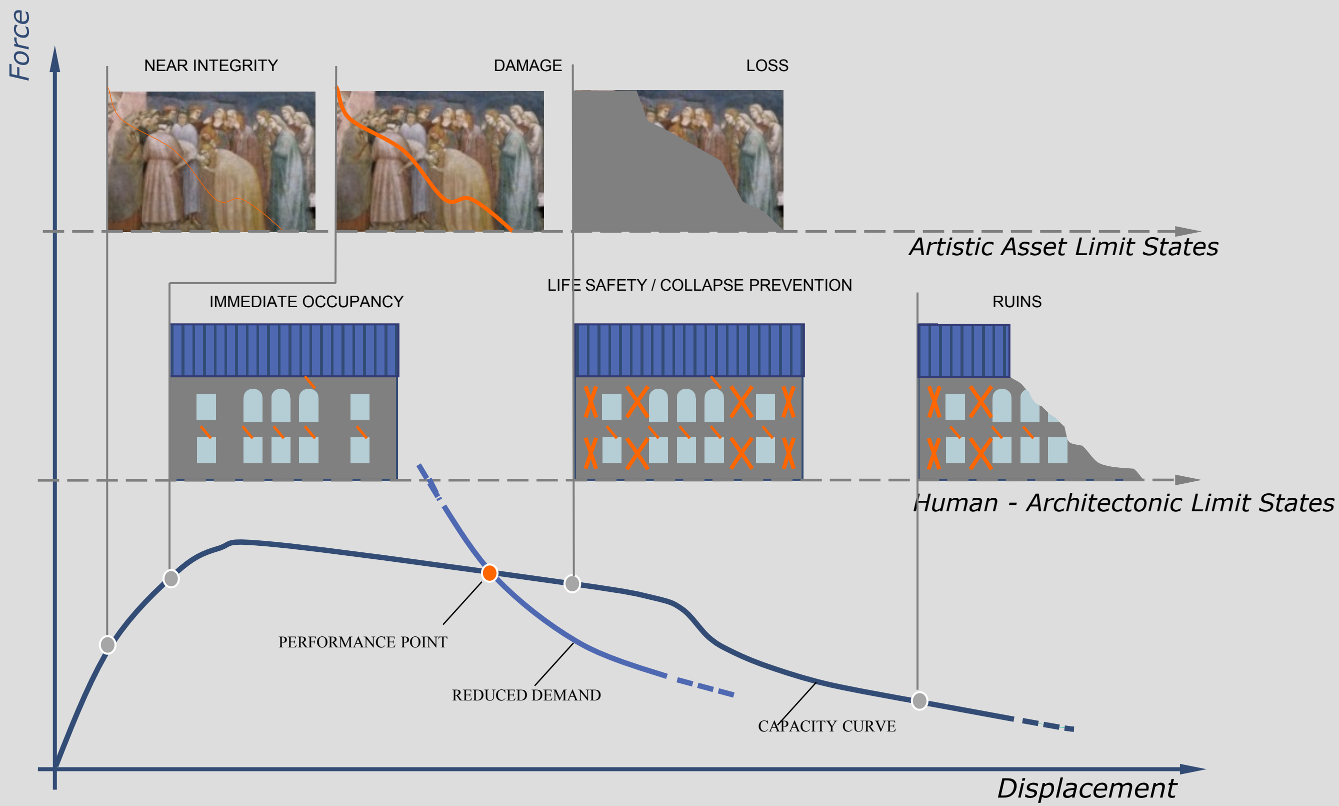
PERFORMANCE-BASED ASSESSMENT

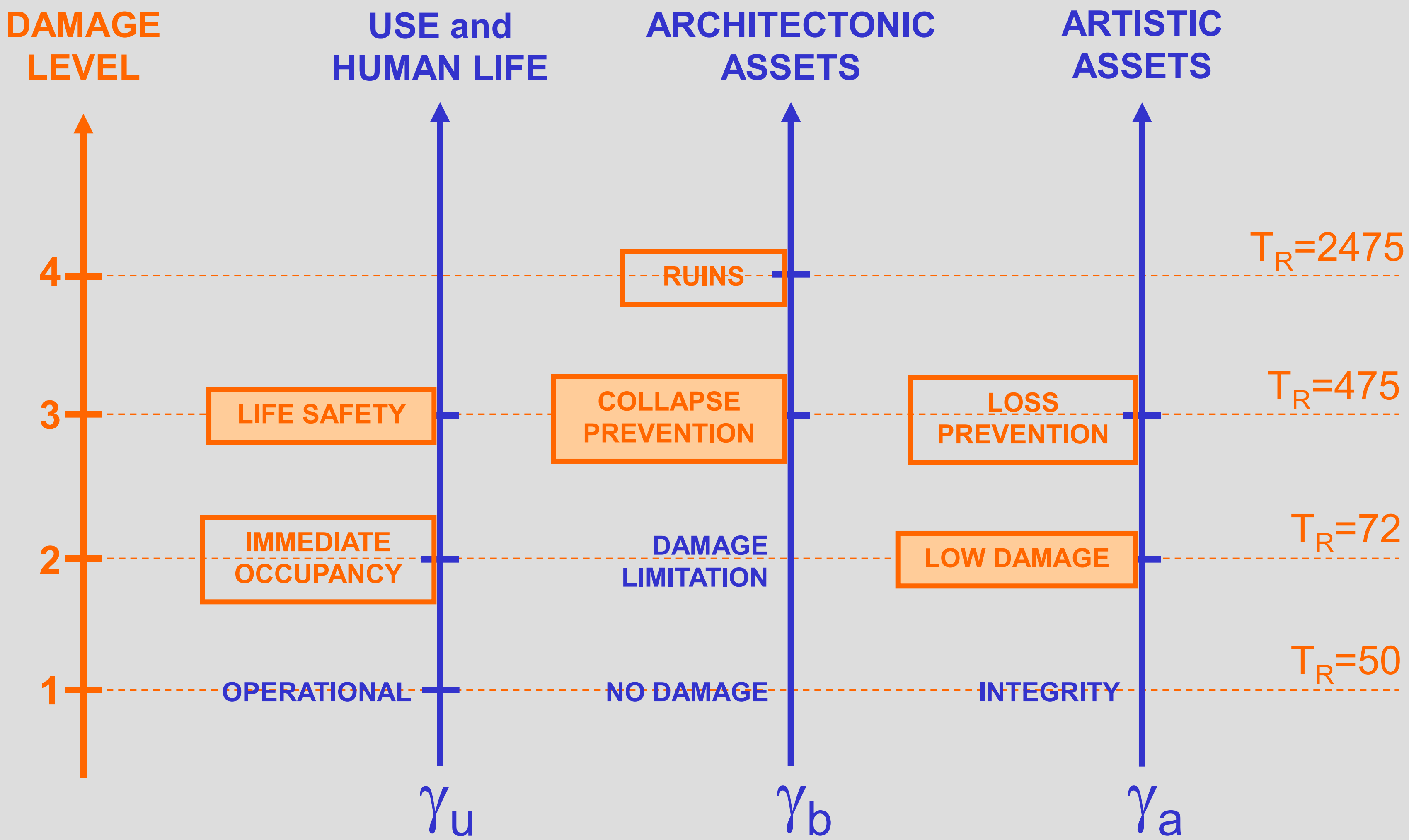
Displacement-based approach for the seismic assessment of architectonic and artistic assets and for the design of strengthening interventions:

1) evaluation of the capacity curve by a non linear static analysis;
2) identification of performance limit states;
3) evaluation of the performance point by capacity spectrum method



BASIC PERFORMANCE REQUIREMENTS

Main and secondary limit states



BASIC PERFORMANCE REQUIREMENTS

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Introduction of coefficients modifying the reference return period

1. USE AND HUMAN LIFE

USE (γ_u)

FUNCTION OF: BUILDING USE; CROWDING LEVEL.

$\gamma_u < 1$ WETHER THE BUILDING IS RARELY USED. IF $\gamma_u < 1$, THE ASSESSMENT OF PERFORMANCE LEVEL “IO” IS NOT REQUIRED.

2. ARCHITECTONIC ASSETS

ARCHITECTONIC RELEVANCE (γ_b)

FUNCTION OF: CULTURAL VALUE OF THE BUILDING ITSELF.

$\gamma_b > 1$ WETHER THE BUILDING HAS A PARTICULAR CULTURAL RELEVANCE. $\gamma_b > 1$, THE ASSESSMENT OF PERFORMANCE LEVEL “RU” IS REQUIRED

3. ARTISTIC ASSETS

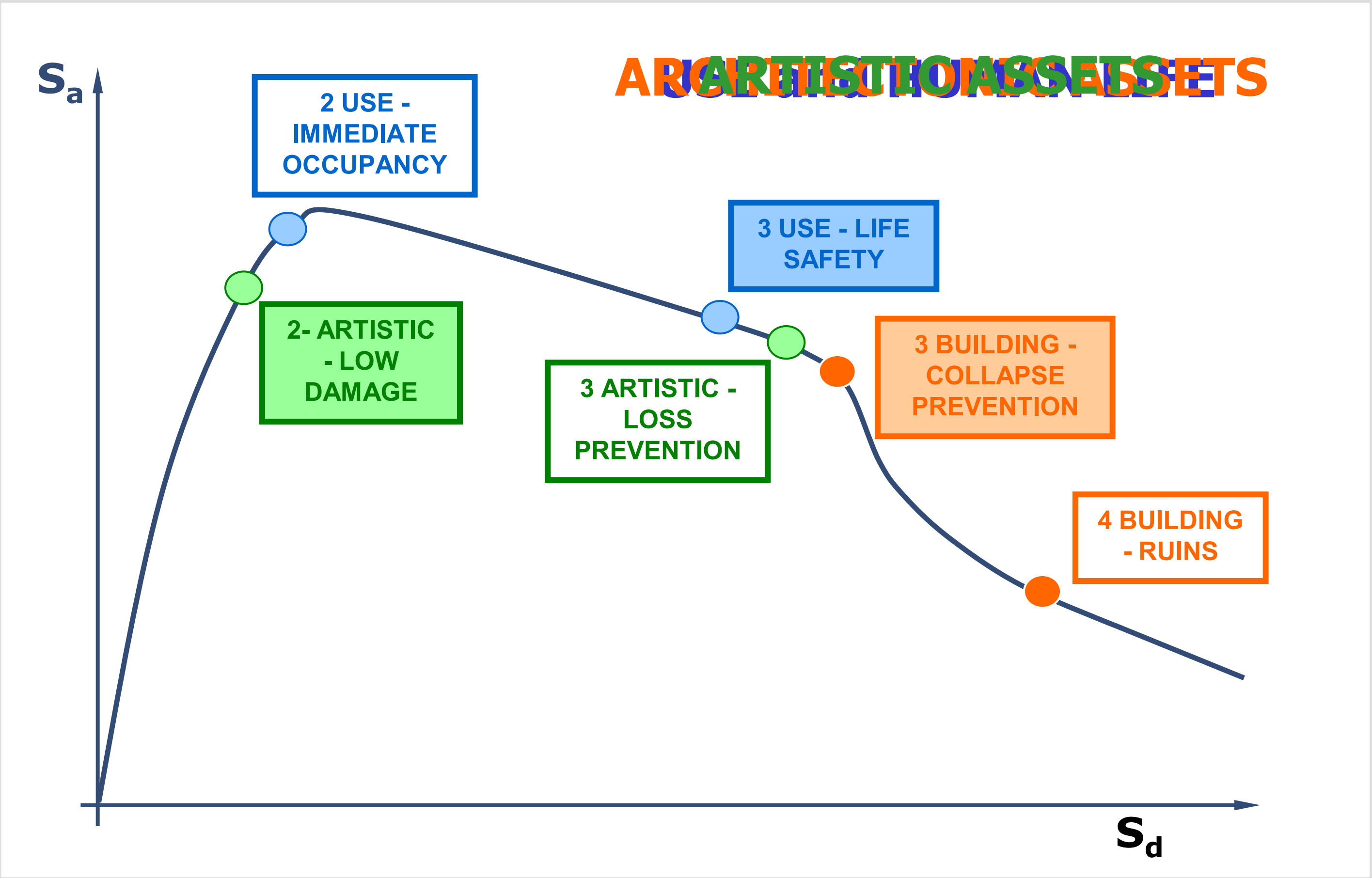
ARTISTIC RELEVANCE (γ_a)

FUNCTION OF: CULTURAL VALUE OF THE ARTISTIC ASSETS PRESENT IN THE BUILDING.

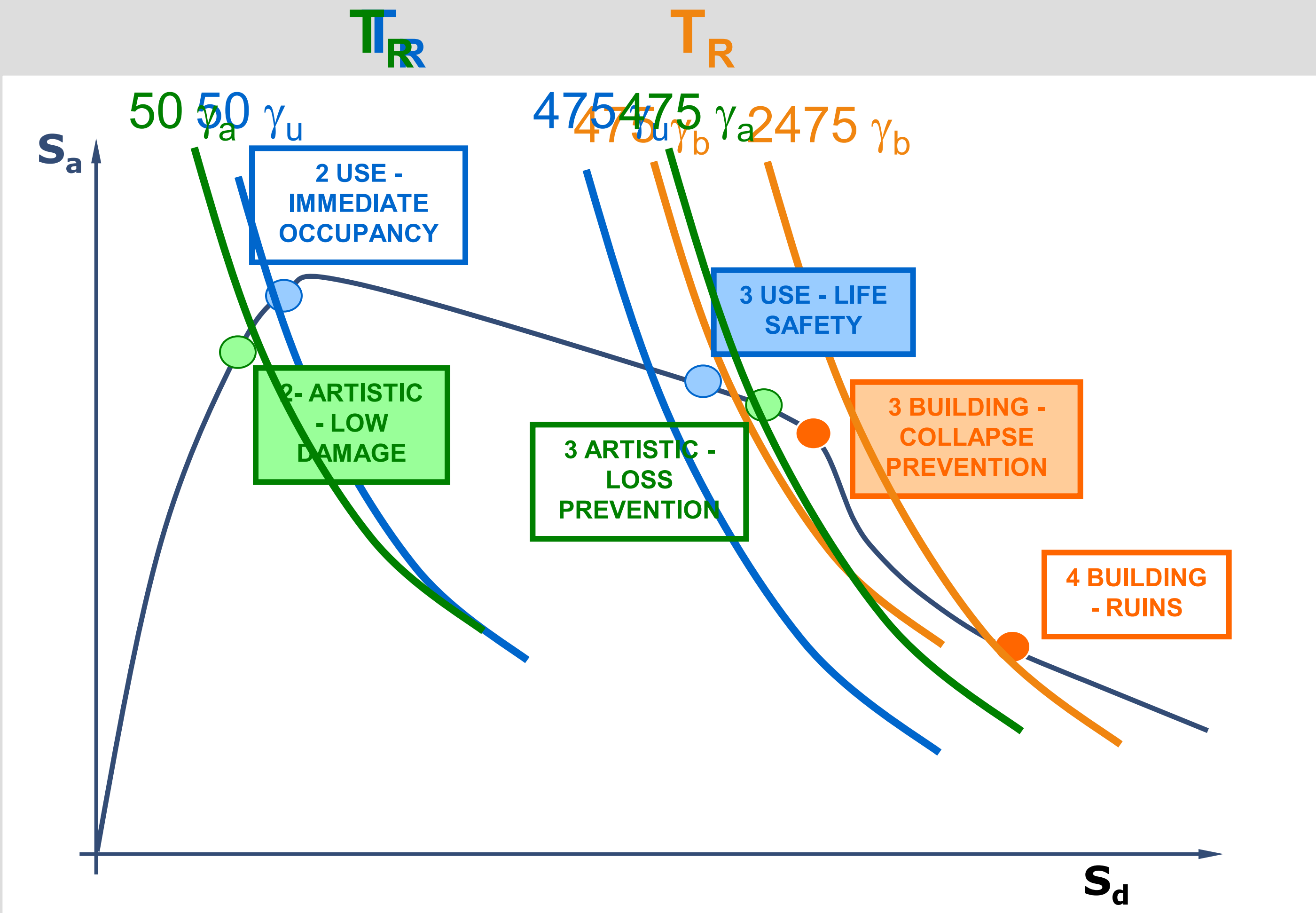
$\gamma_a > 1$ WETHER THE ASSETS HAVE A PARTICULAR CULTURAL RELEVANCE. $\gamma_a > 1$, THE ASSESSMENT OF PERFORMANCE LEVEL “LP” IS REQUIRED

PERFORMANCE-BASED ASSESSMENT

Multicriteria probabilistic approach for the definition of performance limit states: 1) global behaviour; 2) macroelement; 3) local damage



PERFORMANCE-BASED ASSESSMENT



CLASSIFICATION OF ARCHITECTONIC ASSETS

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It is functional to model main seismic behaviour of buildings

Classes	Description	List of assets
A	This class collects architectonic assets with two main bearing structural elements: **vertical walls** and **horizontal floors**. If they are properly connected, mutual cooperation between the structural elements allows the building to **behave as a box**.	A1 *palaces*, A2 *castles*, A3 *religious houses*, A4 *caravansaries*, A5 *madrasas*
B	This class collects architectonic assets which are characterized by **wide spaces without intermediate floors** and few inner walls. Independent damage mechanisms occurs in the different parts of the building, and it is often possible to recognize **specific structural macroelements** (façade, triumphal arch, apse, dome, transept,…).	B1 *churches*, B2 *mosques*, B3 *temples*, B4 *baptisteries,* B5 *mausoleum*, B6 *hammam* B7 *theatres*
C	This class collects architectonic assets in which the **vertical dimension prevails** on the other ones. Since usually, these buildings are characterized by significant slenderness, their seismic response may be assumed as a **global flexural behavior**.	C1 *towers*, C2 *bell towers*, C3 *minarets*, C4 *lighthouses*, C5 *chimneys*
D	This class collects architectonic assets in which the main structural element is an **arch** or a **vault**. Both single arches or much more complex constructions based on this basic structural element are included.	D1 *triumphal arches*, D2 *aqueducts*, D3 *bridges*, D4 *cloisters*
E	This class collects **massive constructions** in which the wide thickness of walls, if compared to other dimensions, doesn't allow the idealization as plane structural element. **Local failure occurs** as, for example, the detachment of external leaf.	E1 *fortresses*, E2 *defensive city walls*
F	This class collects **single isolated architectonic assets**, which does not delimit an interior space.	F1 *columns*, F2 *trilithes*, F3 *obelisks*, F4 *archaeological ruins*
G	This class refers to **historical centers**, made of ordinary buildings' aggregates, which assume the relevance of cultural heritage asset as whole in the urban context. The seismic response must consider the interaction among adjacent buildings.	

BOX-TYPE STRUCTURES (vertical walls and horizontal floors)

Classes	Description	List of assets
A	This class collects architectonic assets with two main bearing structural elements: **vertical walls** and **horizontal floors**. If they are properly connected, mutual cooperation between the structural elements allows the building **to behave as a box**.	A1 *palaces*, A2 *castles*, A3 *religious houses*, A4 *caravansaries*, A5 *madrasas*

A1 Palaces

A2 Castles

A3 Religious houses

A4 Caravansaries

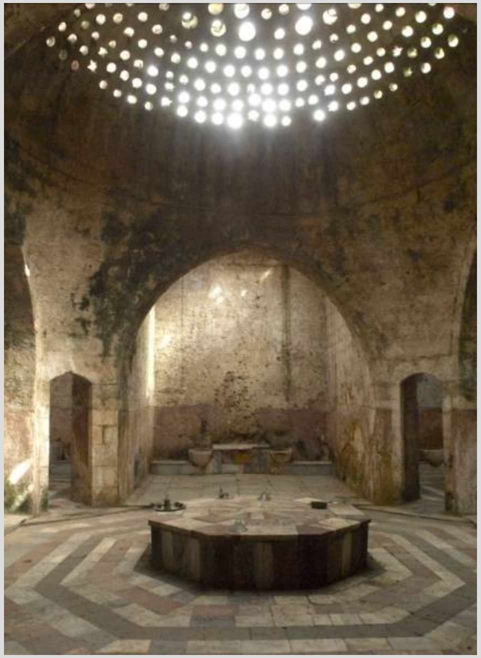
WIDE HALLS WITHOUT INTERMEDIATE FLOORS (macroelements)

Classes	Description	List of assets
B	This class collects architectonic assets which are characterized by **wide spaces without intermediate floors** and few inner walls. Independent damage mechanisms occurs in the different parts of the building, and it is often possible to recognize **specific structural macroelements** (façade, triumphal arch, apse, dome, transept,…).	B1 *churches*, B2 *mosques*, B3 *templ*es, B4 *baptisteries,* B5 *mausoleum*, B6 *hammam* B7 *theatres*

B1 Churches

B2 Mosques

B6 Hammam



SLENDER MASONRY STRUCTURES

Classes	Description	List of assets
C	This class collects architectonic assets in which the **vertical dimension prevails** on the other ones. Since usually, these buildings are characterized by significant slenderness, their seismic response may be assumed as a **global flexural behavior**.	C1 *towers*, C2 *bell towers*, C3 *minarets*, C4 *lighthouses*, C5 *chimneys*

C1 Towers

C2 Bell Towers

C3 Minarets

C4 Lighthouses

ARCHED AND VAULTED STRUCTURES

Classes	Description	List of assets
D	This class collects architectonic assets in which the main structural element is an **arch** or a **vault**. Both single arches or much more complex constructions based on this basic structural element are included.	D1 *triumphal arches*, D2 *aqueducts*, D3 *bridges*, D4 *cloisters*

D1 Triumphal arches

D4 Cloisters

D2 Acqueducts

MASSIVE MASONRY CONSTRUCTIONS

Classes	Description	List of assets
E	This class collects **massive constructions** in which the wide thickness of walls, if compared to other dimensions, doesn't allow the idealization as plane structural element. **Local failure occurs** as, for example, the detachment of external leaf.	E1 *fortresses*, E2 *defensive city walls*

E1 Fortress

E2 Defensive city walls

DRY BLOCKS SIMPLE STRUCTURES

Classes	Description	List of assets
F	This class collects **single isolated architectonic assets**, which does not delimit an interior space.	F1 *columns*, F2 *trilithes*, F3 *obelisks*, F4 *archaeological ruins*

F1 Columns

F2 Trilithes

F3 Obelisks

AGGREGATED BUILDINGS IN HISTORICAL CENTRES

Classes	Description	List of assets
G	This class refers to **historical centers**, made of ordinary buildings' aggregates, which assume the relevance of cultural heritage asset as whole in the urban context. The seismic response must consider the interaction among adjacent buildings.	

Navelli, L'Aquila, Italy

Skofja Loka, Slovenia

CLASSIFICATION OF ARTISTIC ASSETS

Class	Description	Sub-class	Examples
P	**Artistic assets which are structural elements themselves**	P1 – carved or shaped vertical structural assets	Caryatid, carved stone columns, walls with carved blocks or shaped bricks, …
		P2 – carved or decorated horizontal structural assets	Carved stone or wooden lintels, decorated wooden beams, …
		P3 – carved structural arched assets	Carved stone arches, vaults and domes, etc.
		P4 – carved or decorated wooden roof	Decorated wooden roof, etc.
Q	**Artistic assets which are strictly connected to structural elements**	Q1 – assets connected to vertical structural elements	Carved stone plates, frescos, mosaics, stuccoes, …
		Q2 – assets connected to the intrados of horizontal and arched structural elements	**Frescos**, **mosaics**, stuccoes, wooden or plaster false ceiling, light thin plaster vaults, …
		Q3 – assets connected to the extrados of horizontal structural elements	Floor with mosaics, decorated tiles, parquets, …
R	**Artistic assets which has their own seismic response**	R1 – assets leant on horizontal structural elements	**Pinnacles**, altars, sculptures, pulpits, …
		R2 – assets jutting out from vertical structural elements	Balconies, shelves, gargoyles, …
		R3 – assets hanging on horizontal structural elements	Lamps, bells, crosses, …

It is functional to model the seismic behaviour

Q - Artistic assets connected to structural elements

- ROSE WINDOWS

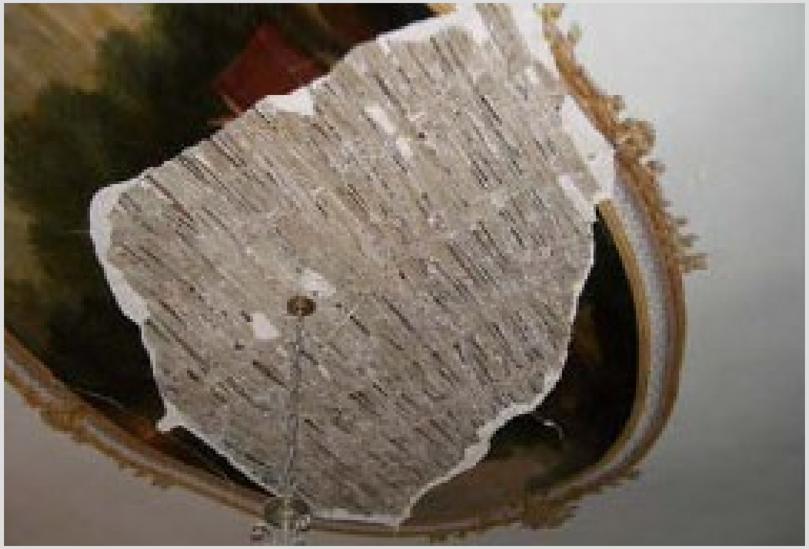
- PORTALS

Q -Artistic assets connected to structural elements

- San Bernardino, L'Aquila

Q - Artistic assets connected to structural elements

- LATHWORK VAULTS



- FRESCOES

R - Artistic assets with their own seismic response

- PINNACLES, STATUES

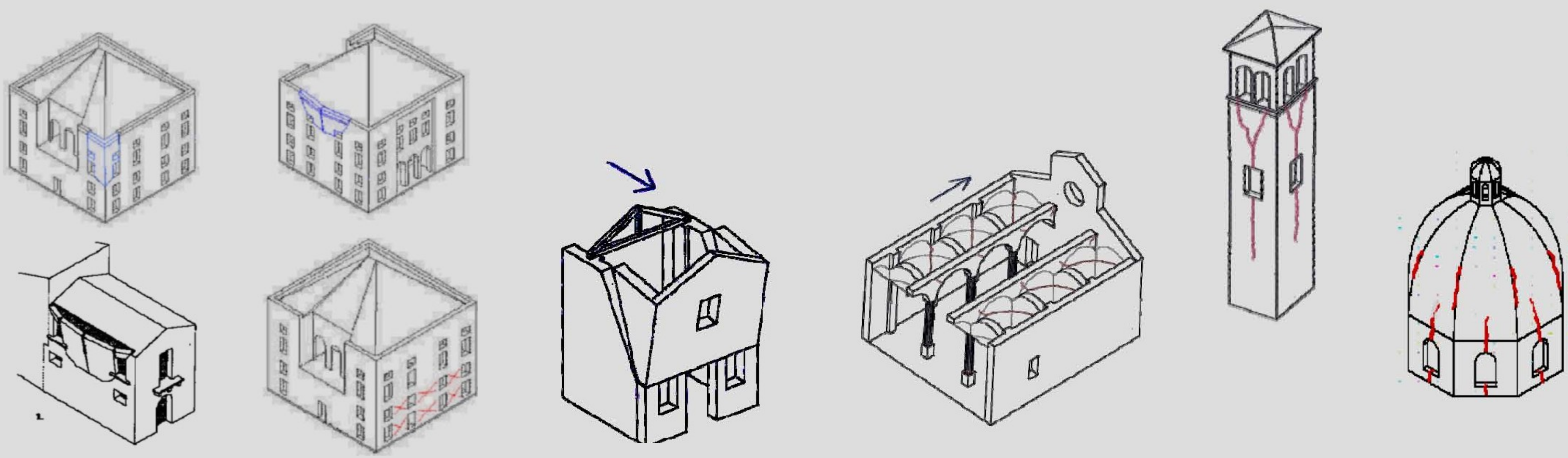
- CORBELS, GARGOYLES

DAMAGE CLASSIFICATION

It is possible to identify different **seismic damage modes** for cultural heritage assets, related to the different classes previously defined.

ARCHITECTONIC AND ARTISTIC ASSETS CLASSIFICATION

DAMAGE CLASSIFICATION



DAMAGE CLASSIFICATION

Correlation between type of building and damage classification

	A	B	C	D	E	F	G
1 – in plane							
2 – out of plane				D2			
3 – flexural/crushing							
4 – arches			C2				
5 – local masonry failure							
6 – blocks							
7 – floors / roofs					E1		
8 – vaults							
9 - domes							

Prevailing behaviour
Possible behaviour
Occasional behaviour

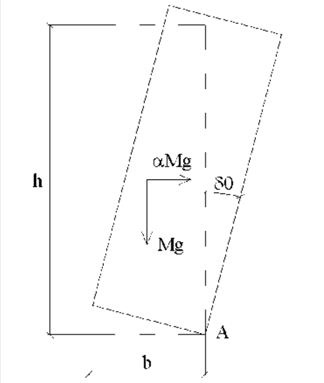
The table above is only qualitative and based most on the presence of macroelements than on frequency of damage.

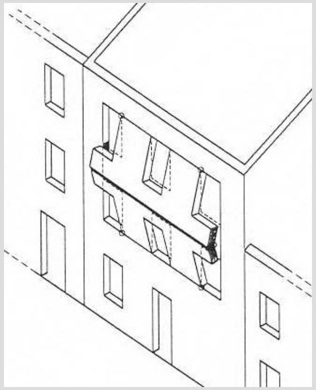
SEISMIC BEHAVIOUR OF MASONRY BUILDINGS

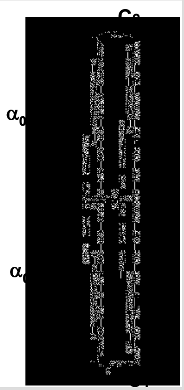
L'AQUILA, ITALY – 2009

• TASK 5.1 – Modelling of local mechanisms of buildings

OUT-OF-PLANE MECHANISMS
(1° failure mode)





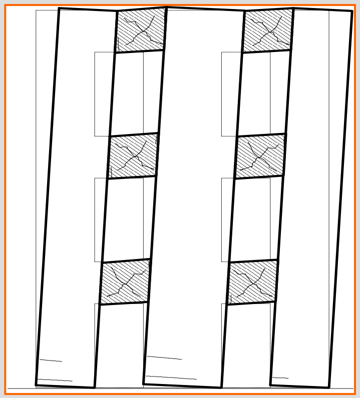
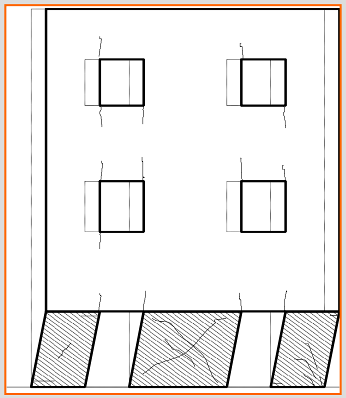


- **LIMIT ANALYSIS – KINEMATIC APPROACH**
- **PUSH-OVER CURVE THROUGH NON LINEAR KINEMATIC ANALYSIS**

$$\lambda = a_{max} / g$$

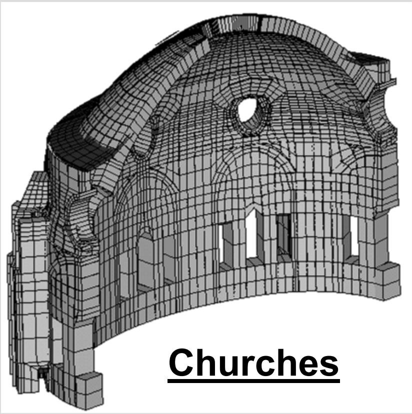
WP5 – Mechanical models for assessment and design

- <u>TASK 5.2 – Modelling of global response of buildings</u>

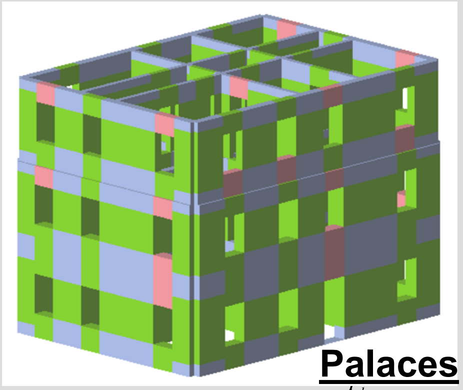



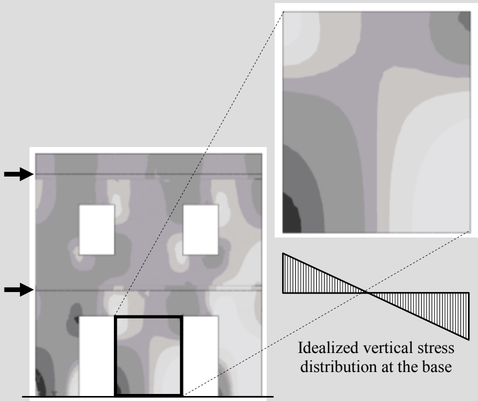
IN-PLANE MECHANISMS
(2° failure mode)

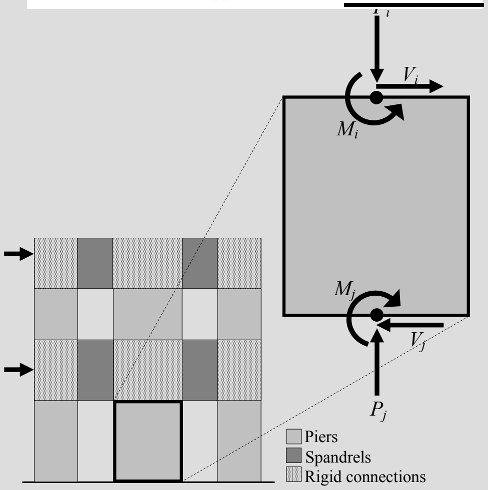
<u>Finite element approach</u>



<u>Structural element approach</u>







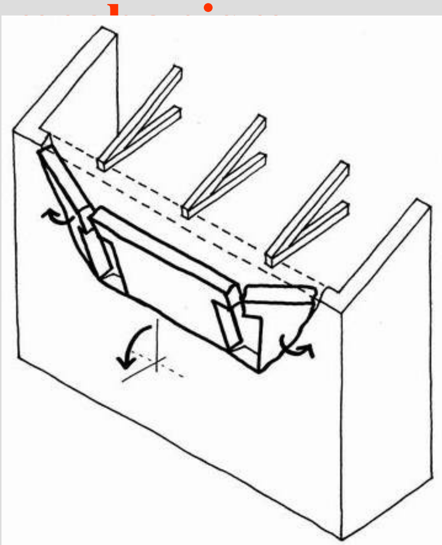
SEISMIC LIMIT ANALYSIS

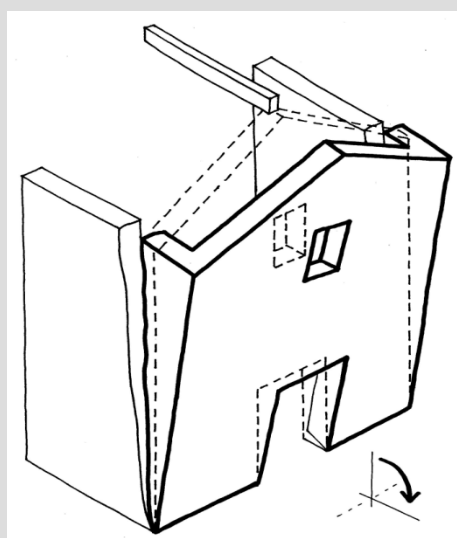
For the seismic evaluation by collapse limit analysis the following actions have to be considered:

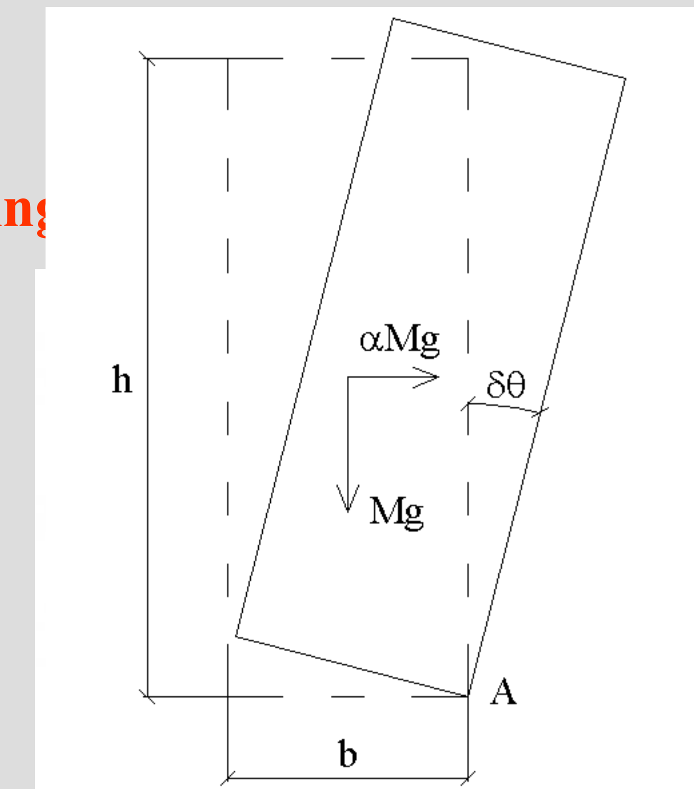
- dead loads
- applied external actions (tie rods, clamping with other structures)
- horizontal actions proportional to dead loads through the multiplier λ, representative of seismic actions

$$\lambda = a_{max} / g$$

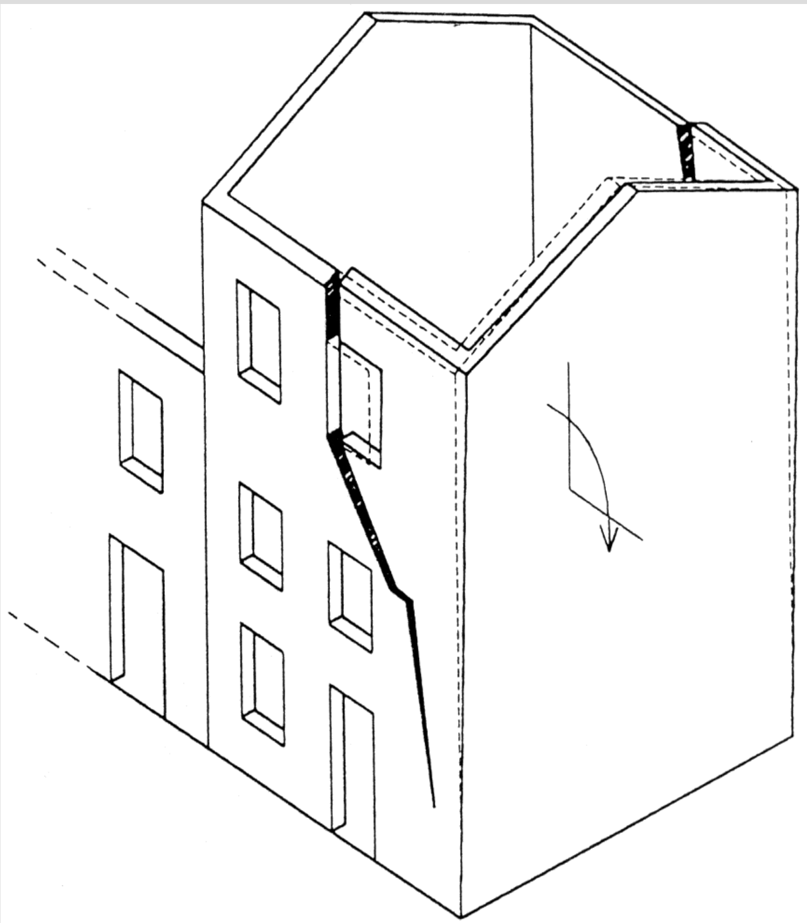
With the unsafe theorem, it is necessary to sing

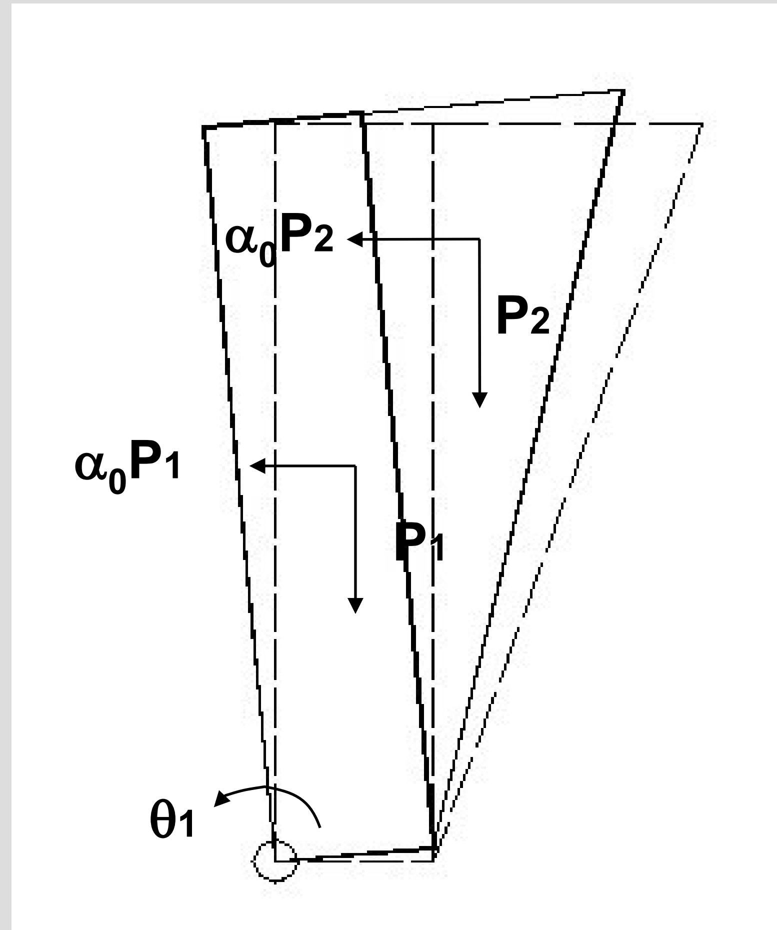






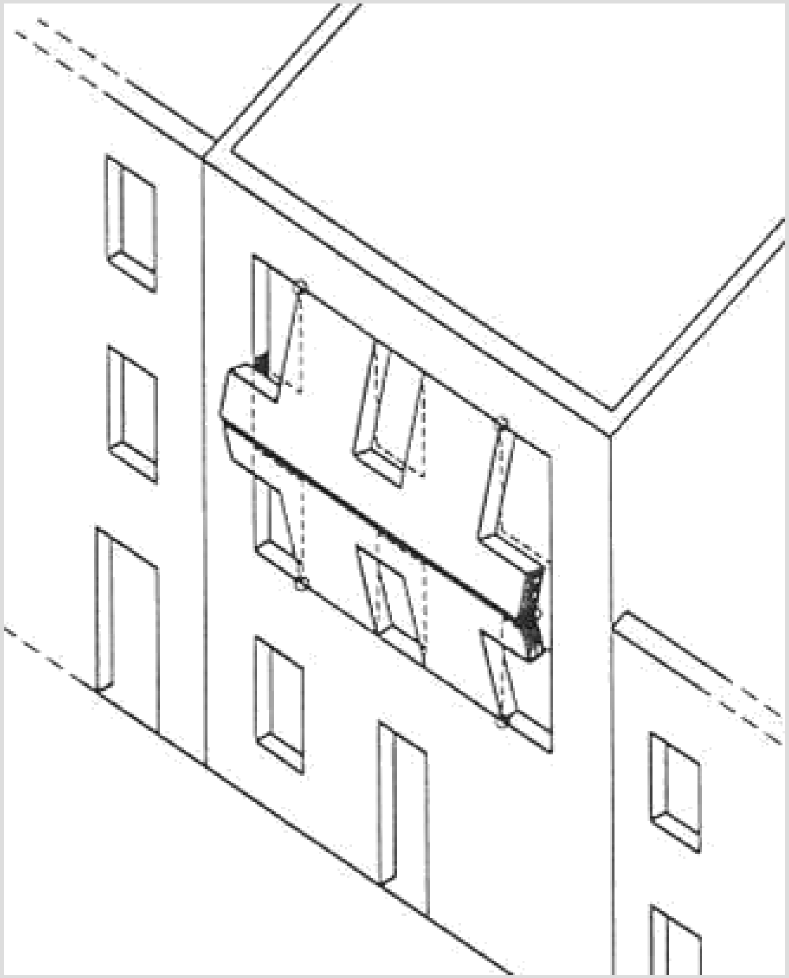
WP5 – Mechanical models for assessment and design

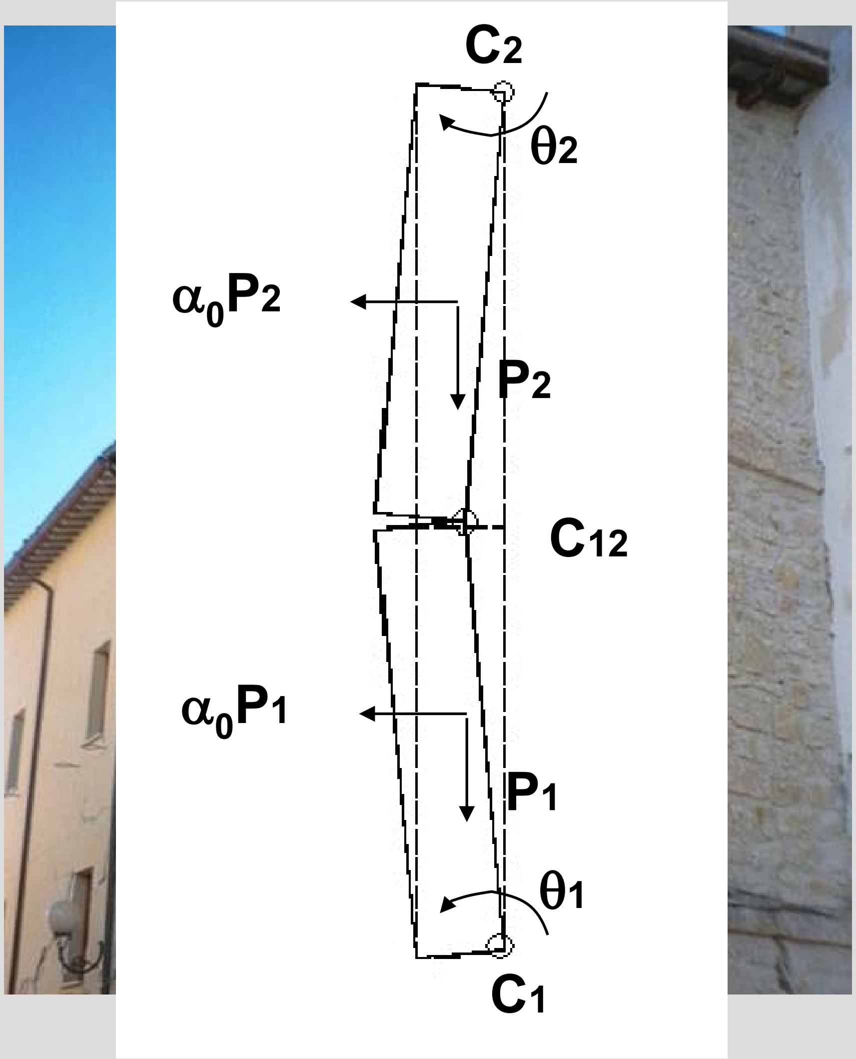




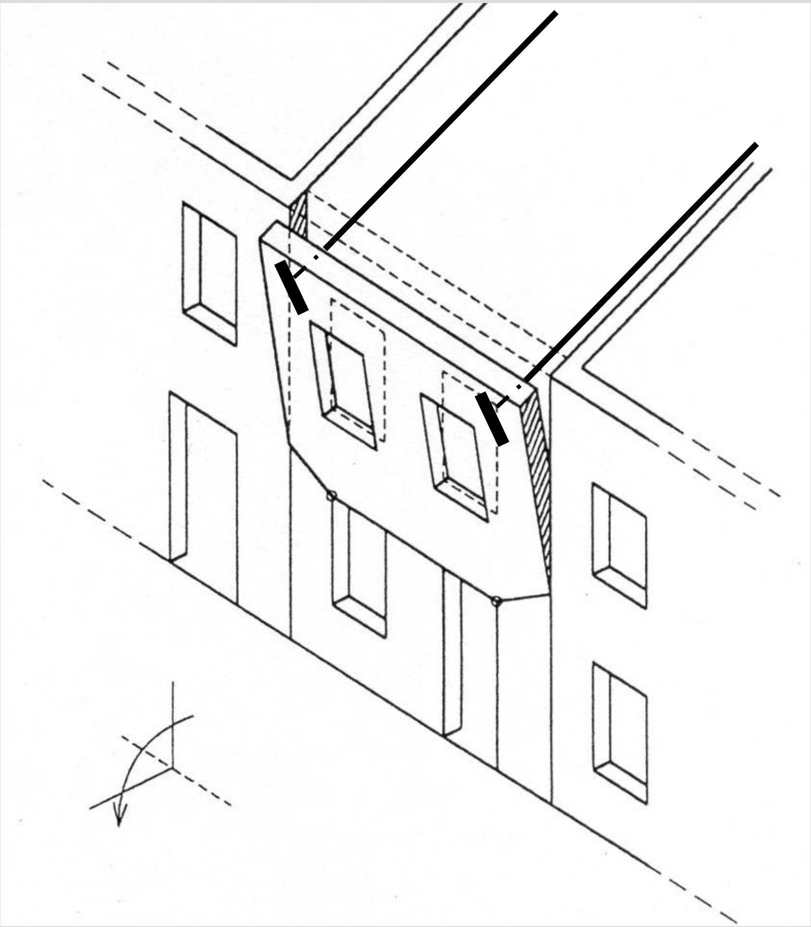
Overturning of a façade with good clamping to orthogonal walls

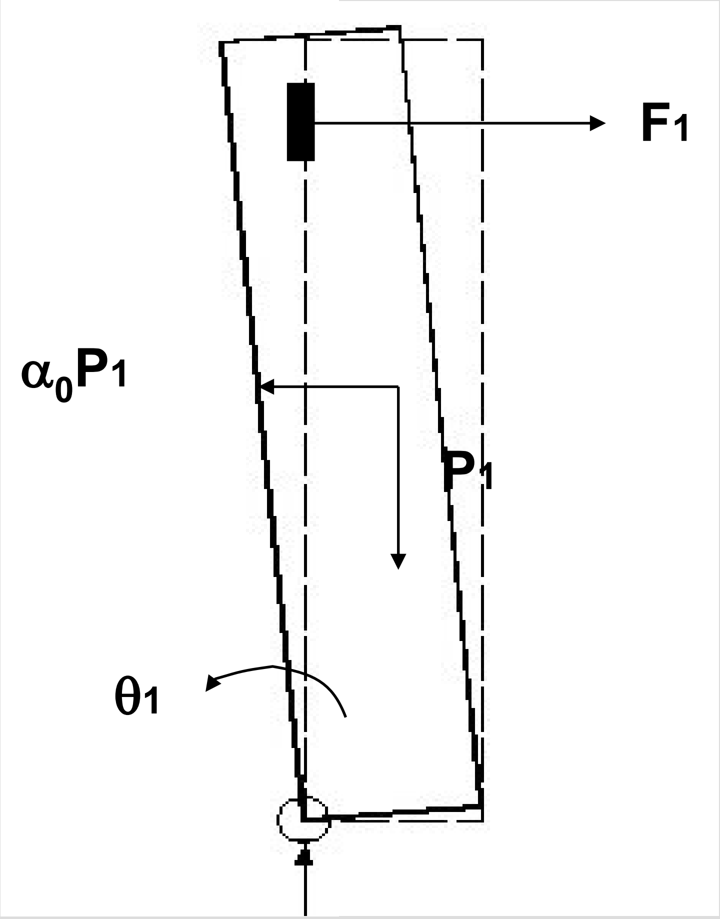
WP5 – Mechanical models for assessment and design





Local collapse mechanism in the case of stiff and heavy r.c. slab





Seismic strengthening by tie rods

WP5 – Mechanical models for assessment and design

$$\theta_2 = \theta_1 \frac{(y_B - y_A)(x_C - x_D) + (y_C - y_D)(x_A - x_B)}{(y_B - y_C)(x_C - x_D) + (y_C - y_D)(x_C - x_B)}$$

$$\theta_3 = \theta_1 \frac{(y_B - y_A)(x_C - x_B) + (y_C - y_B)(x_A - x_B)}{(y_D - y_C)(x_B - x_C) + (y_B - y_C)(x_C - x_D)}$$

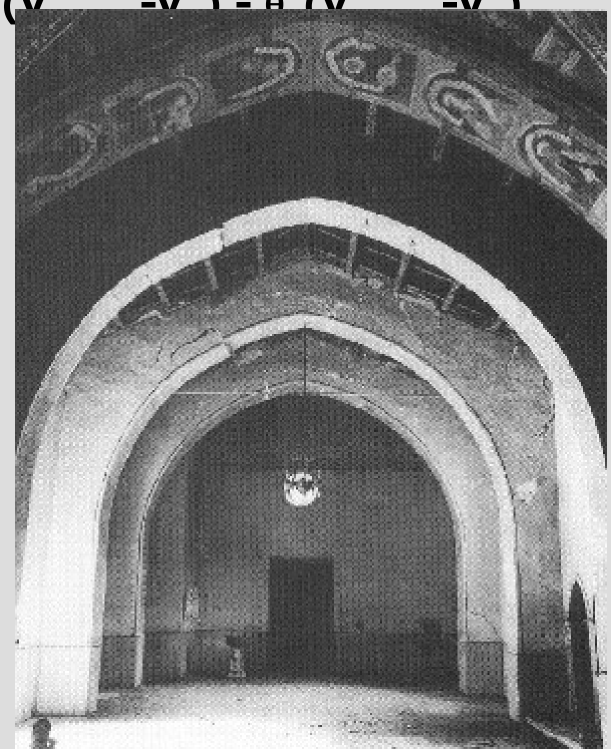
$$\sum_{i=1}^{3} P_i \Delta y_i - \alpha \sum_{i=1}^{3} P_i \Delta x_i = T \Delta l$$

$\Delta x_1 = \theta_1(y_A - y_1)$ $\Delta y_1 = \theta_1(x_1 - x_A)$

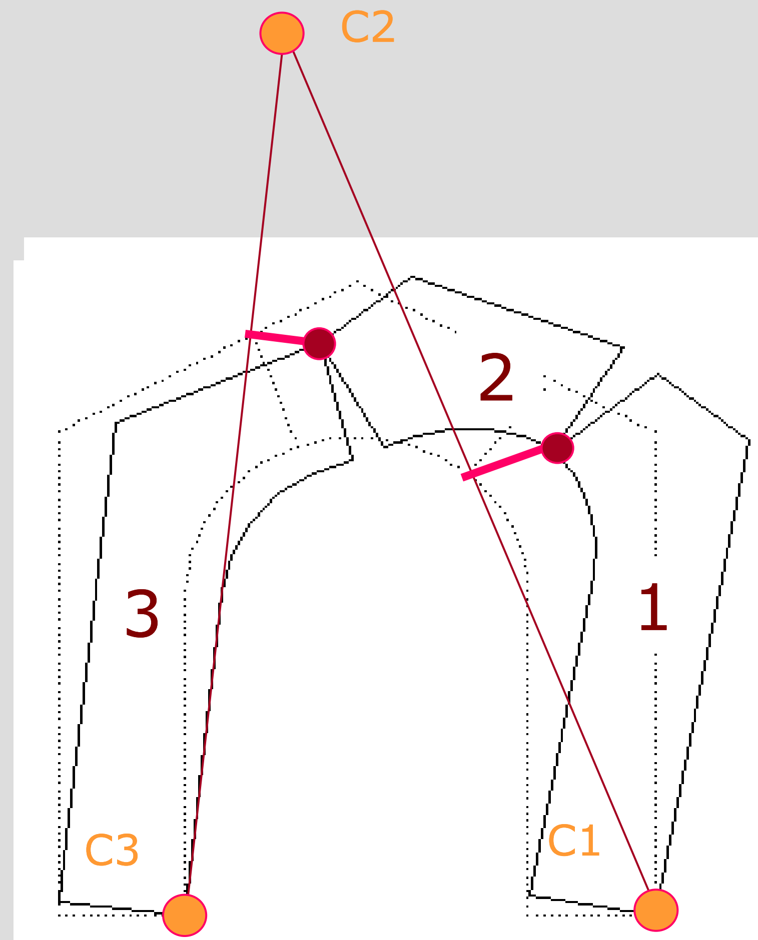
$\Delta x_2 = \theta_1(y_A - y_B) + \theta_2(y_B - y_2)$ $\Delta y_2 = \theta_1(x_B - x_A) + \theta_2(x_2 - x_B)$

$\Delta x_3 = \theta_3(y_D - y_3)$ $\Delta y_3 = \theta_3(x_3 - x_D)$

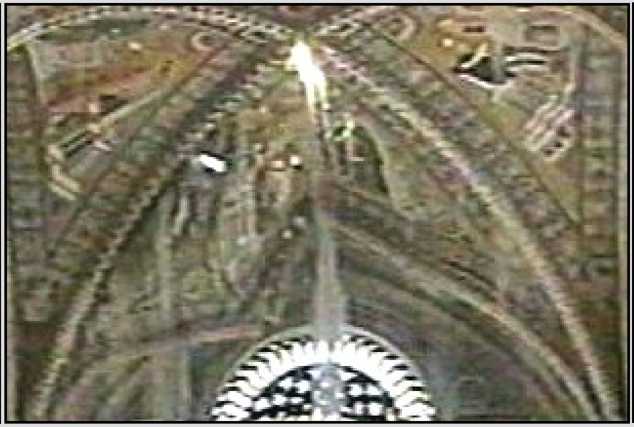
$\Delta l = \theta_3(y \quad -y \quad) - \theta \ (y \quad -y \quad)$



α

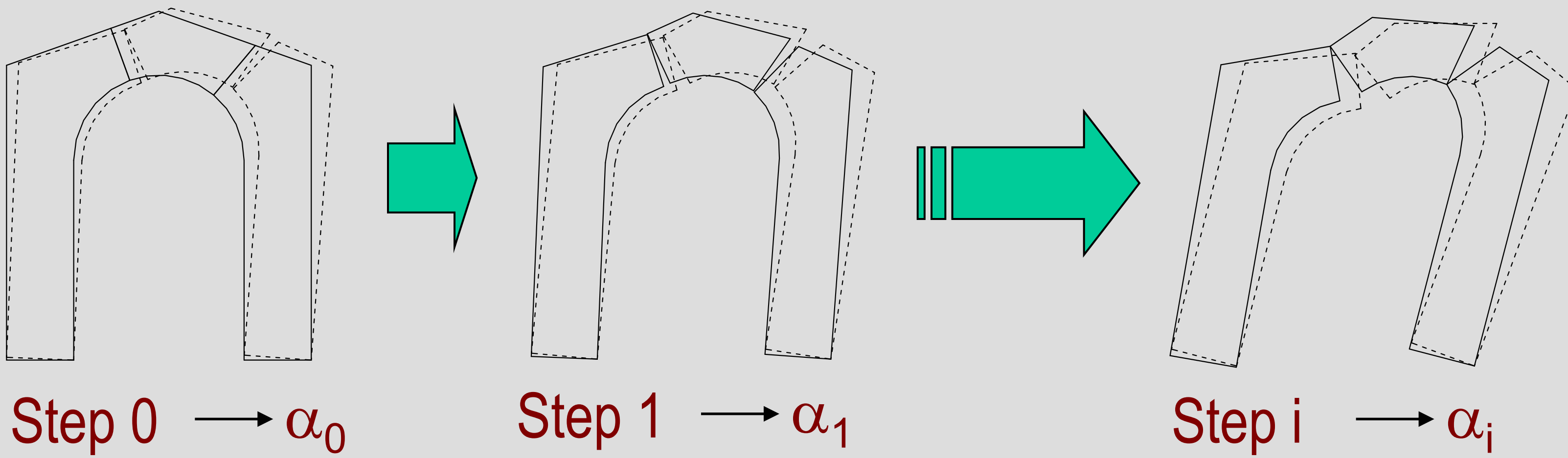


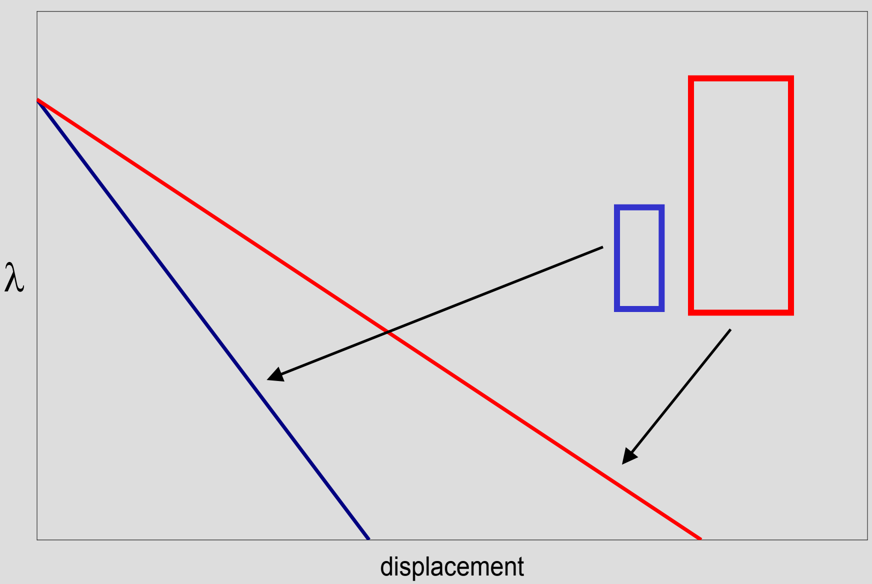
The Basilica of St. Francesco d'Assisi – September 26th, 1997

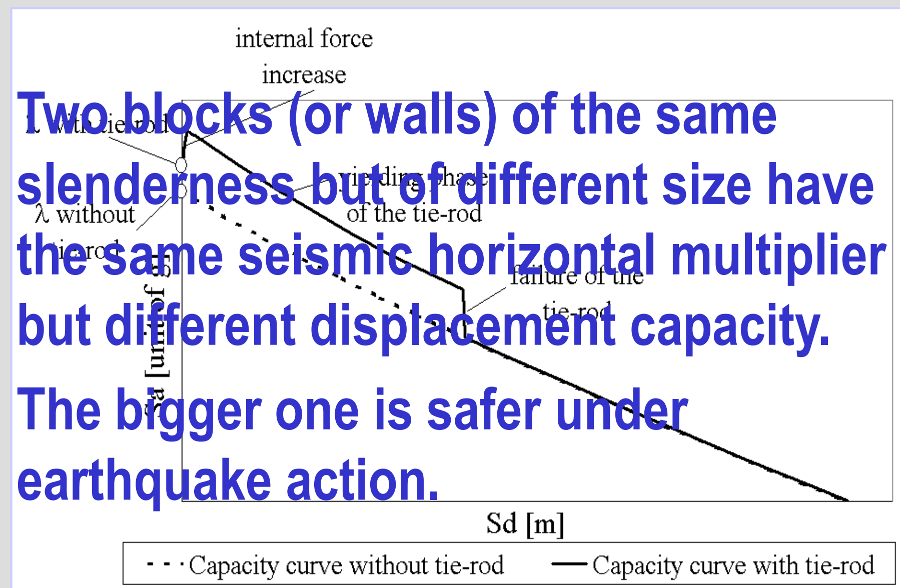




TREMA Project – ENEA, DPC, University of Basilicata, ReLUIS

Pushover curve by equilibrium limit analysis





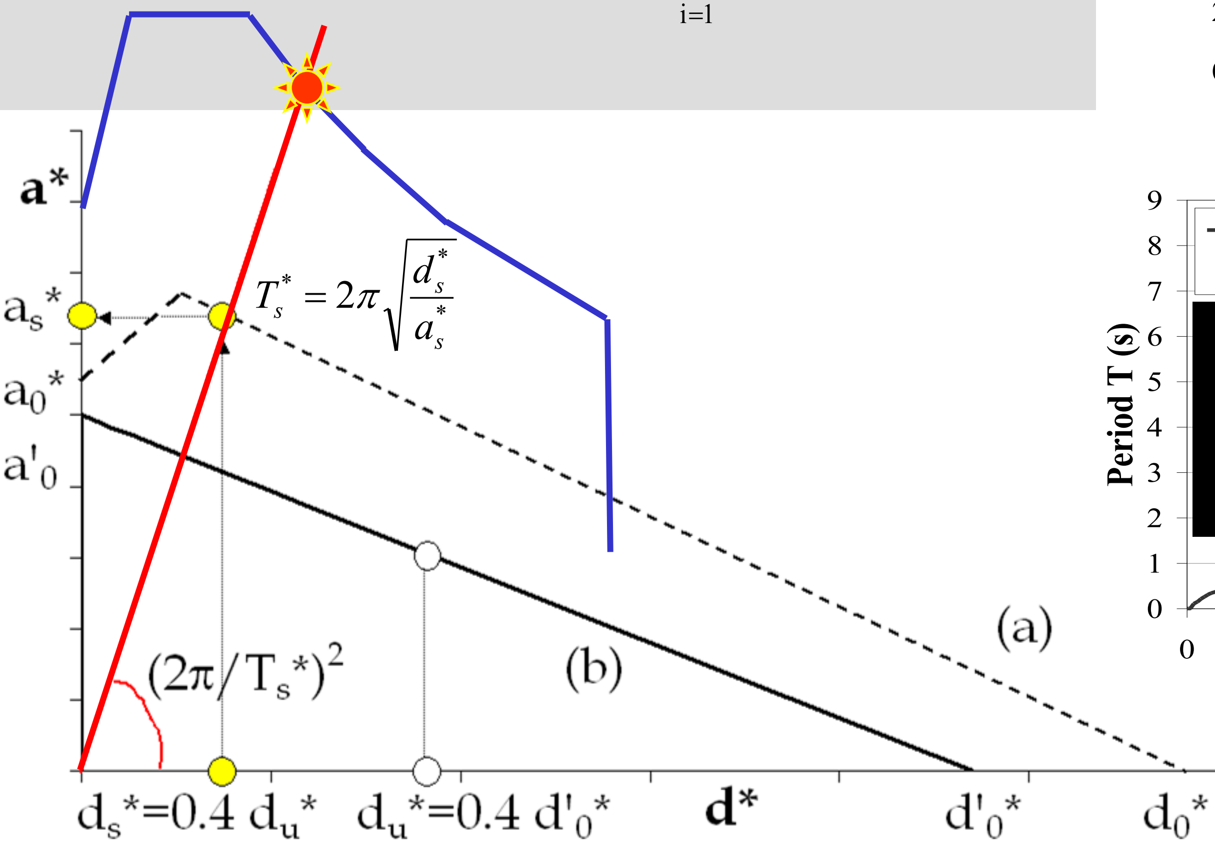


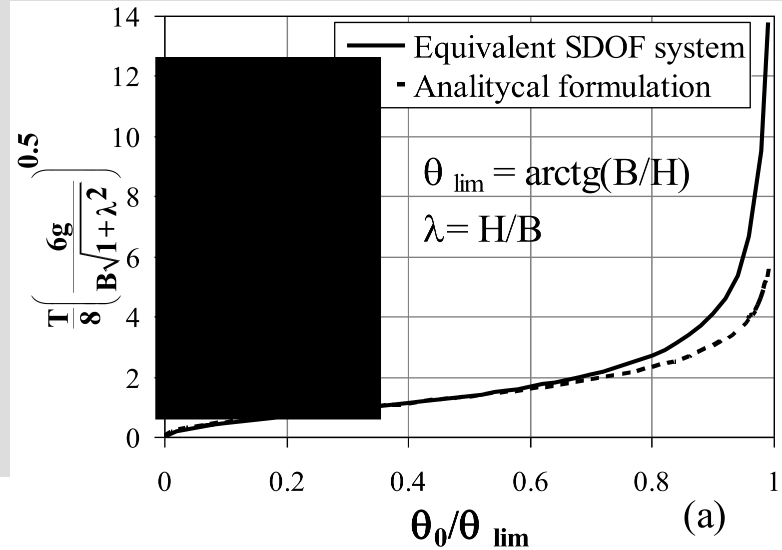
Two blocks (or walls) of the same slenderness but of different size have the same seismic horizontal multiplier but different displacement capacity.

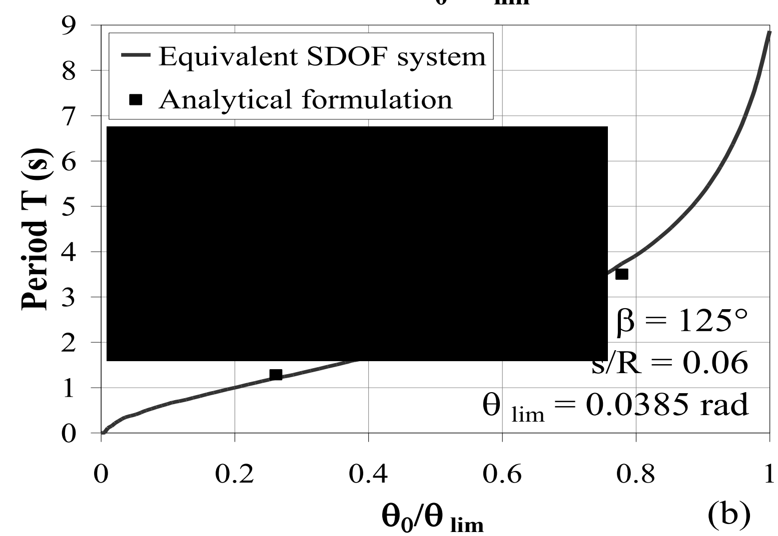
The bigger one is safer under earthquake action.

Conversion of pushover curve into a capacity spectrum

$$a^* = \frac{\alpha \sum_{i=1}^{n+m} P_i}{M^* FC} = \frac{\alpha g}{FC \cdot e^*} \qquad d^* = d_k \frac{\sum_{i=1}^{n+m} P_i \delta^2_{x,i}}{\delta_{x,k} \sum_{i=1}^{n+m} P_i \delta_{x,i}}$$



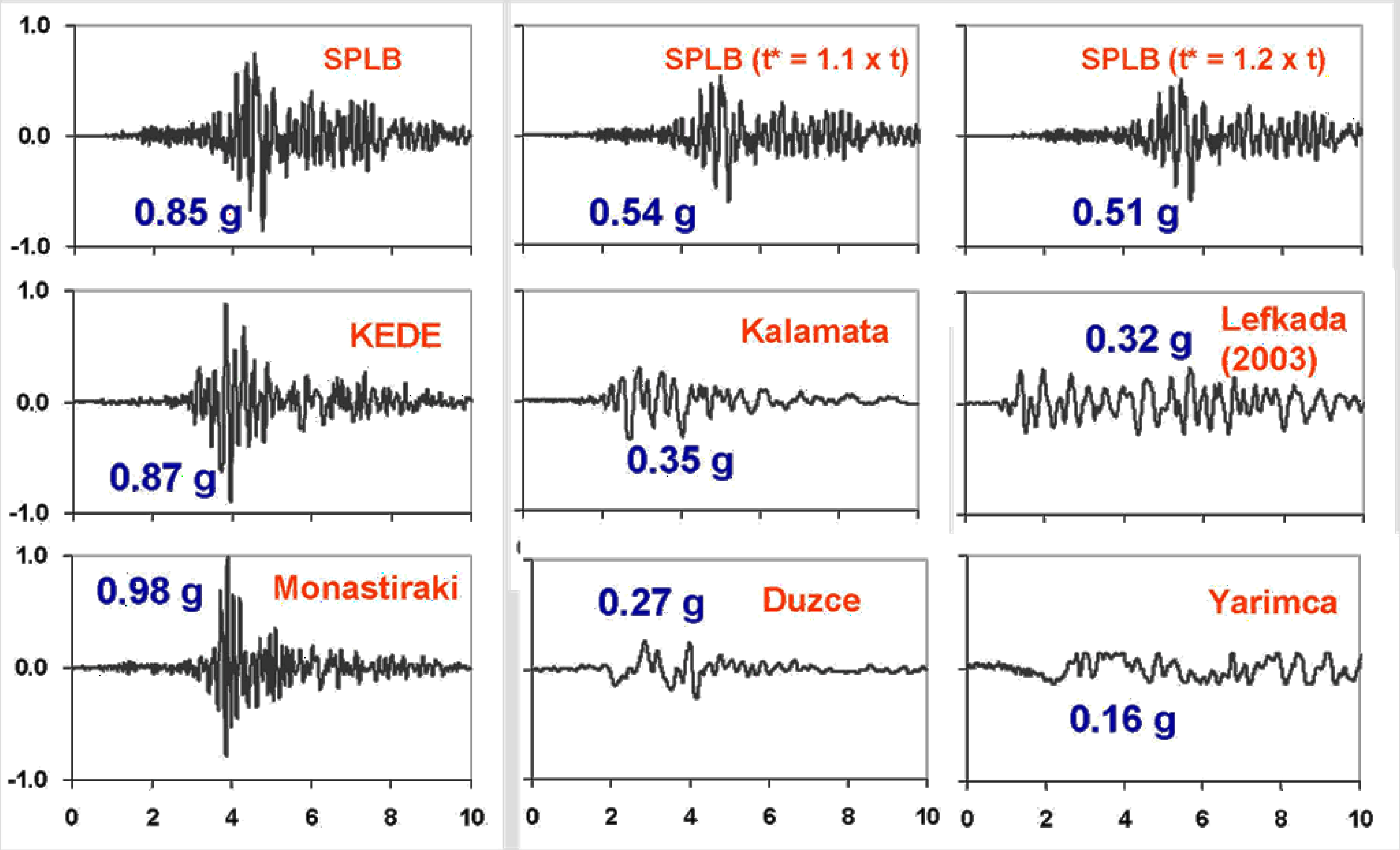




Validation with reference solutions

VALIDATION - Overturning of a block

Non linear dynamic analysis of a rigid block (by Gazetas et al.)



2b = 20 cm
2h = 127 cm

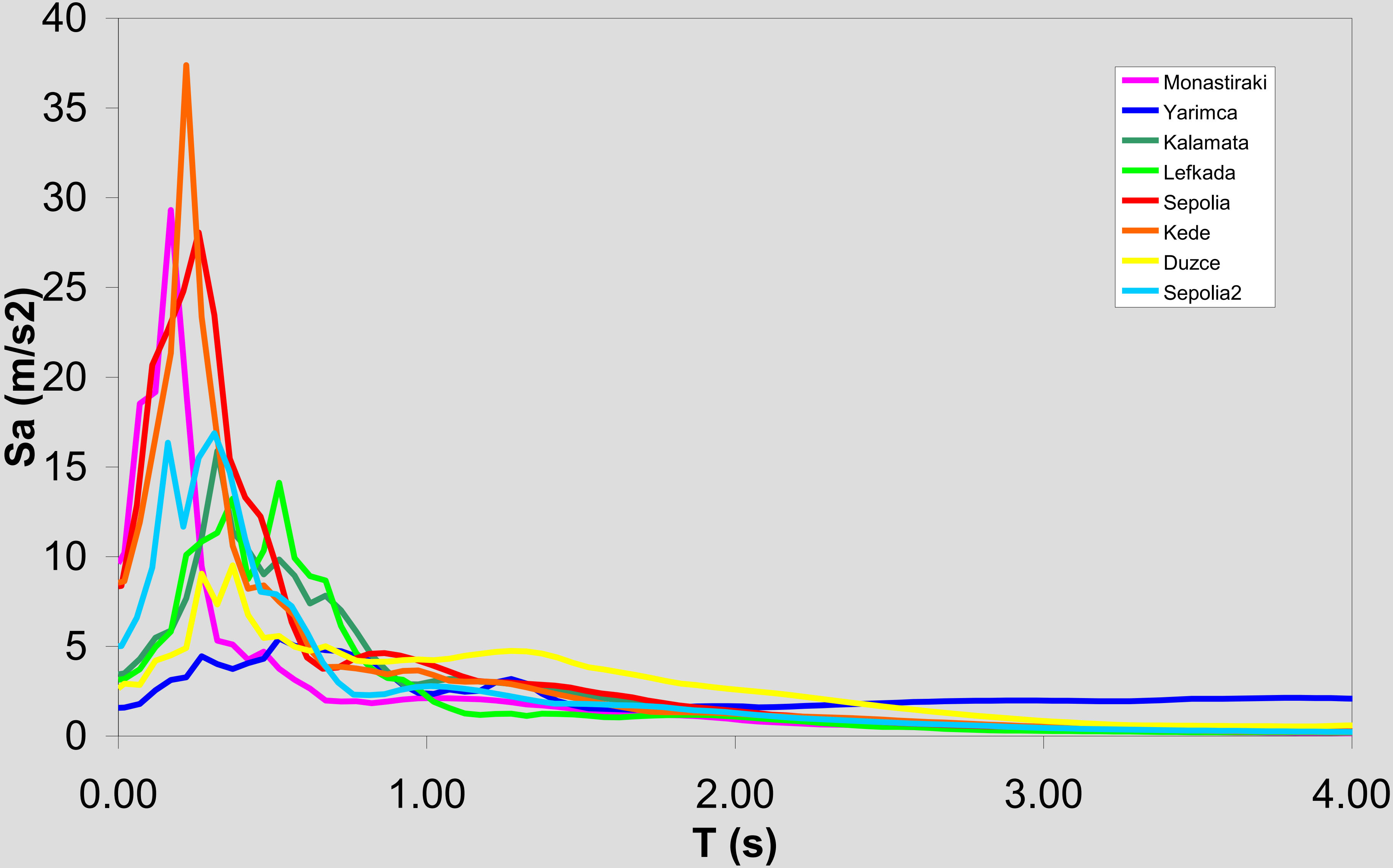
α = b/h = 0.16

$$\delta = \ln\left(\frac{d_n}{d_{n+1}}\right) \qquad \xi = \frac{\delta}{2\pi} = \frac{1}{2\pi}\ln\left(\frac{1}{c^2}\right)$$

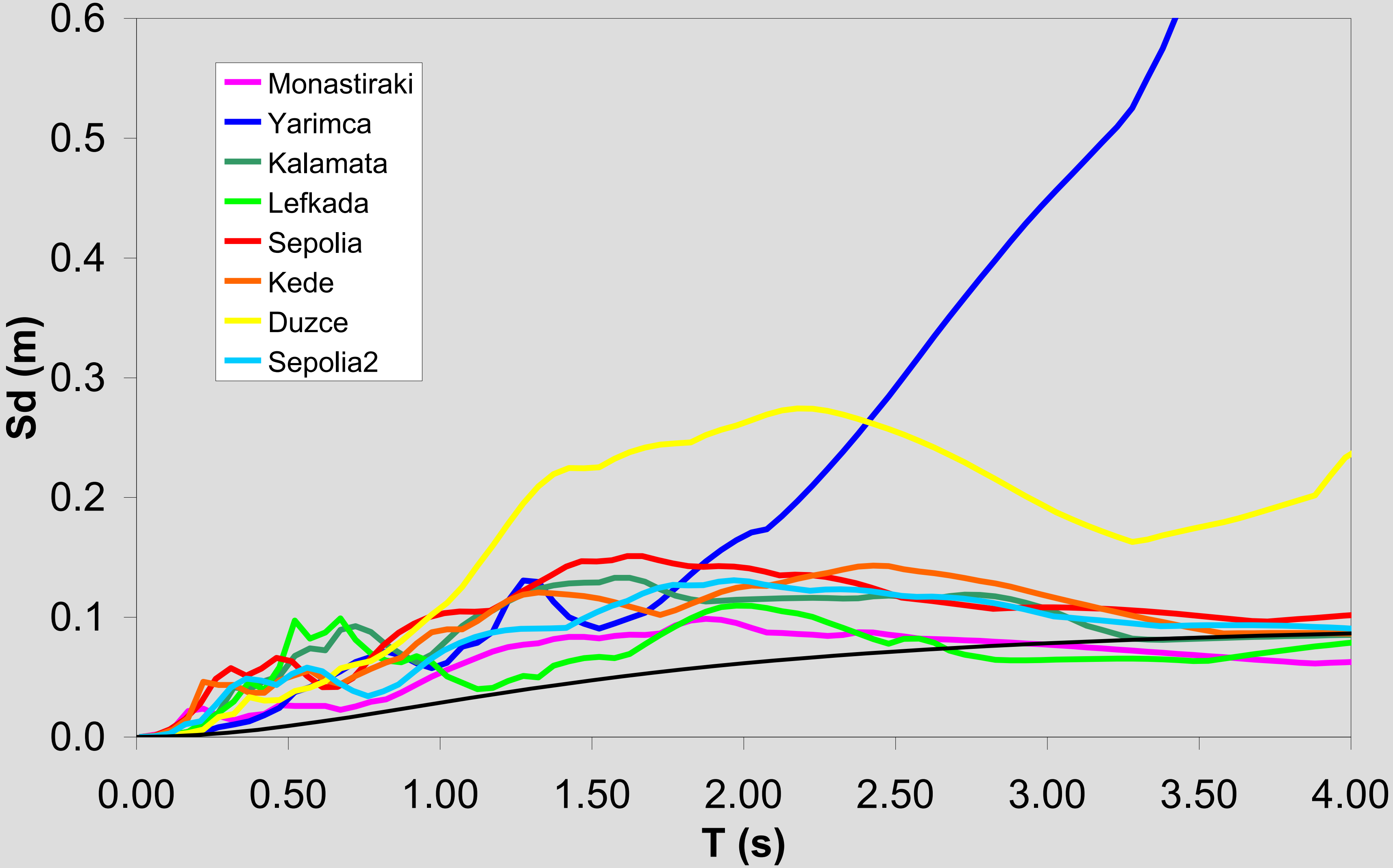
Elastic response spectra (5% damping)

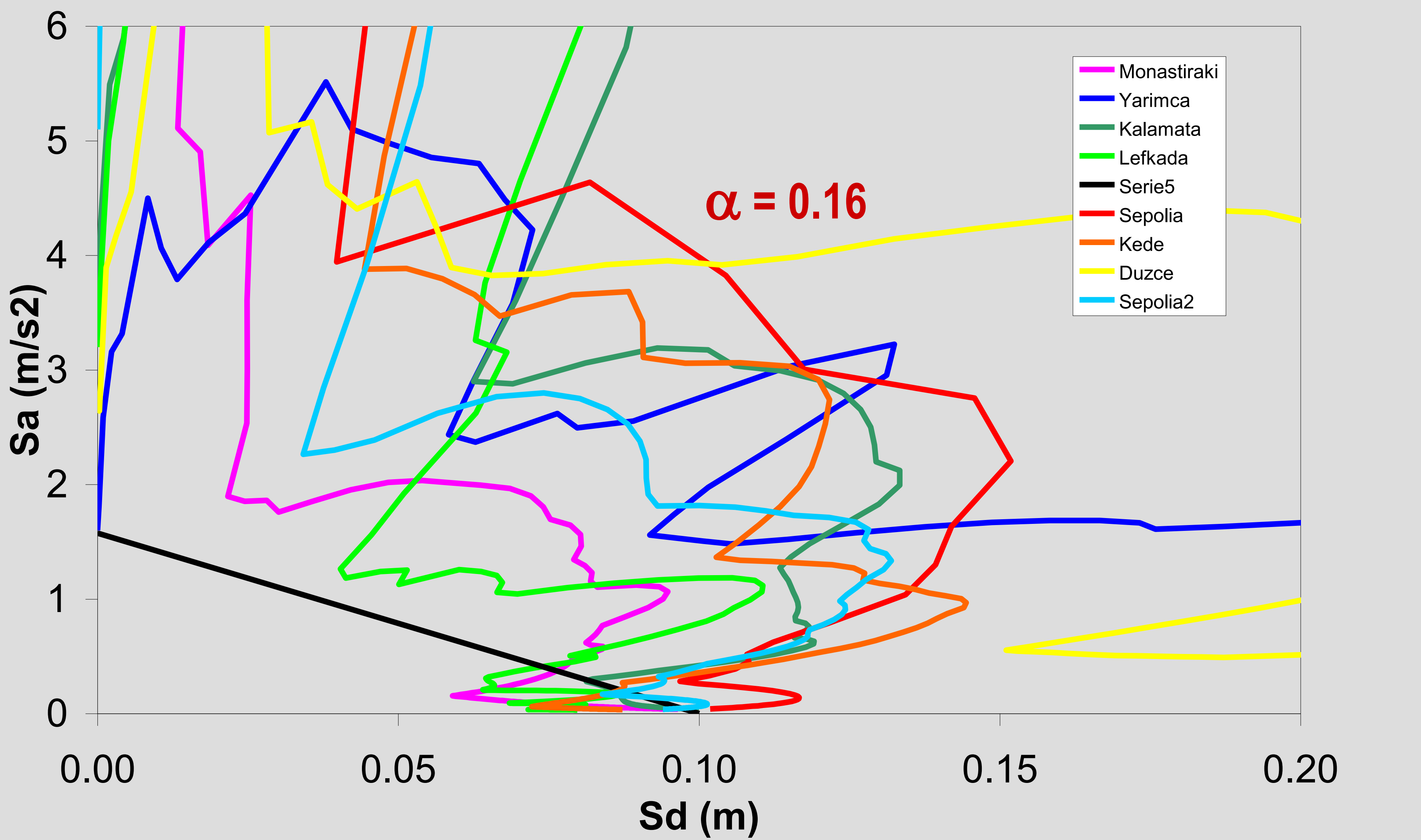
$$c = 0.85 \quad \xi = 5.2\%$$

Acceleration response spectra



Displacement response spectra

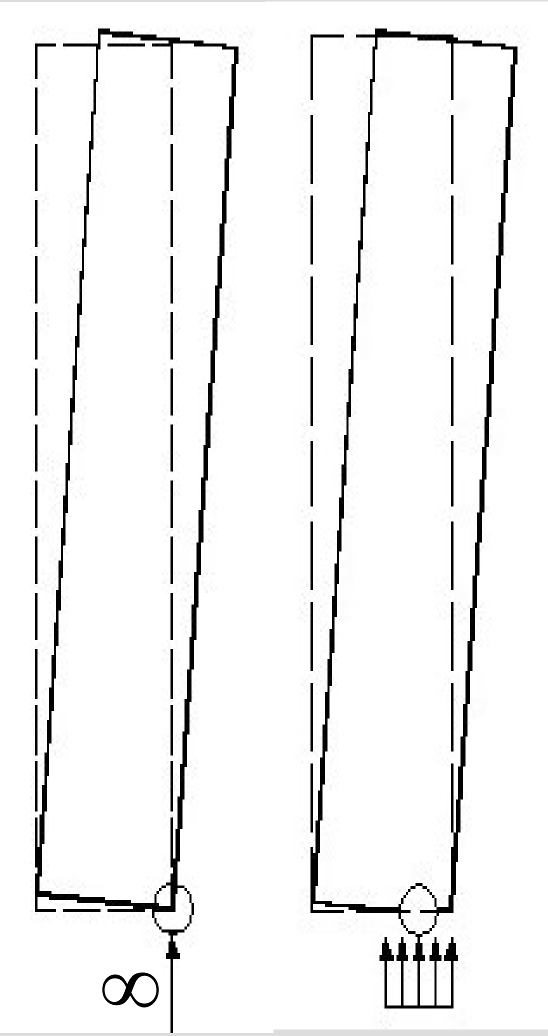


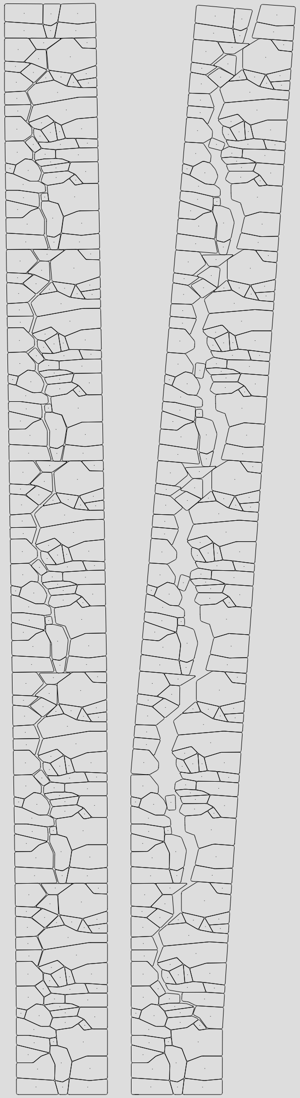


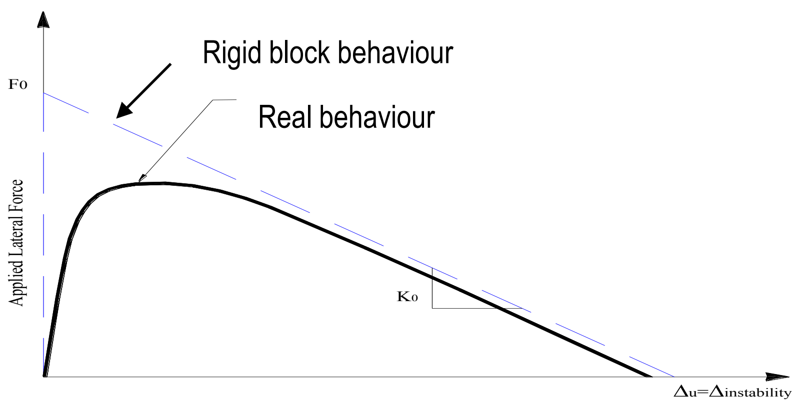
All response spectra (in acceleration-displacement form) intersect the capacity curve for displacements values which are close to the ultimate displacement of the block. In the case of Yarimca earthquake, the acceleration is that producing the activation of rocking.

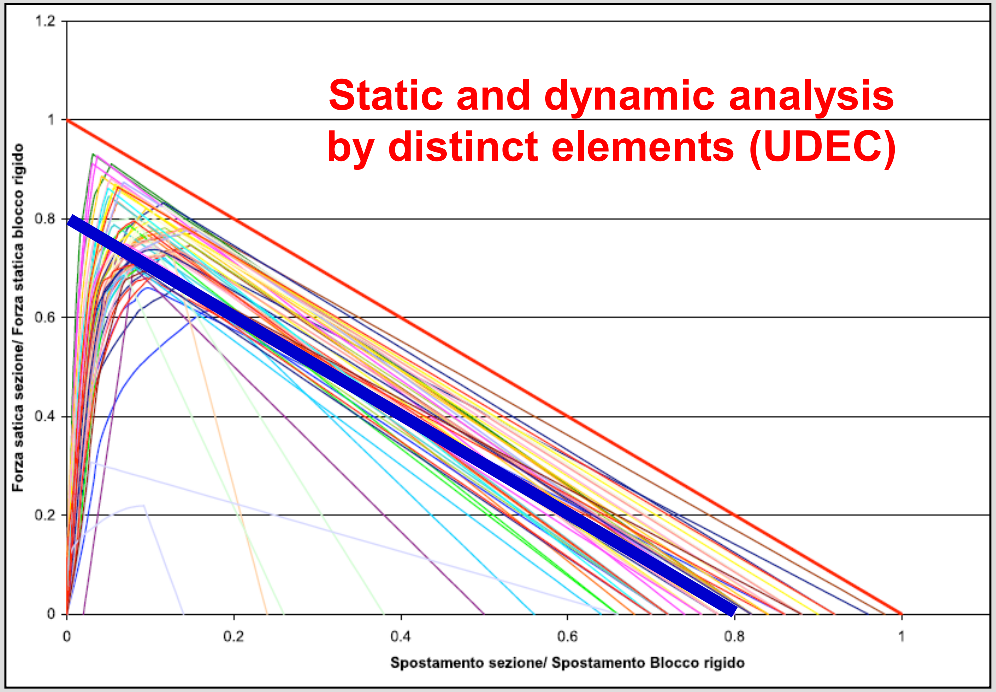
WP5 – Mechanical models for assessment and design

INFLUENCE OF THE QUALITY OF MASONRY



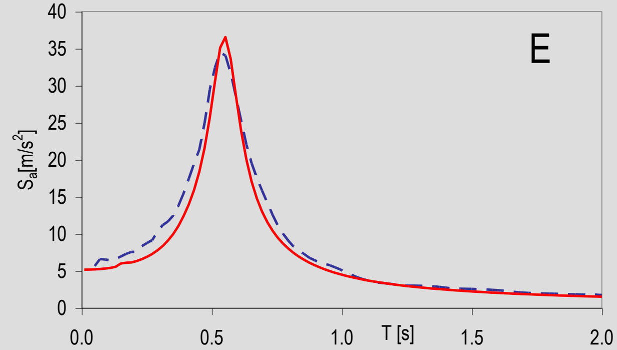


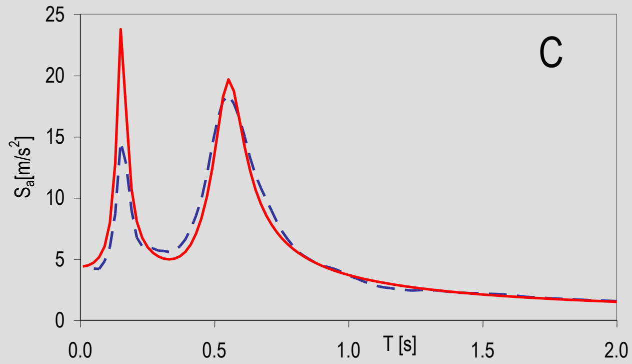


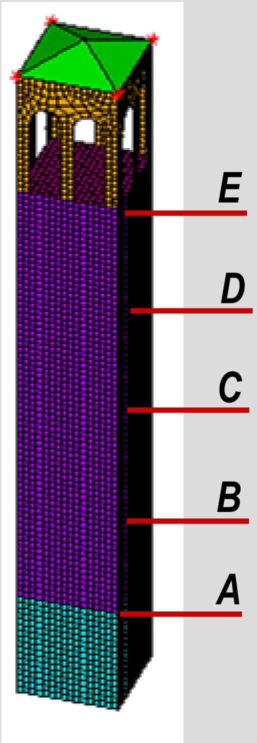


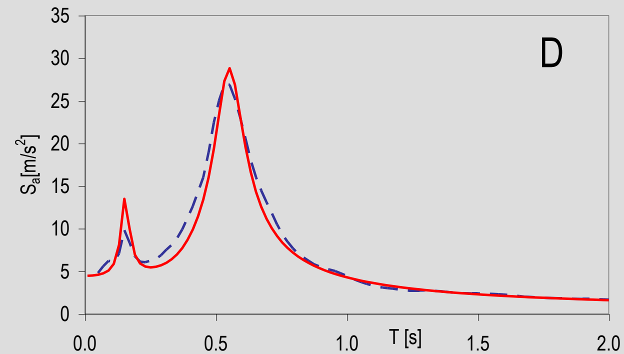
WP5 – Mechanical models for assessment and design

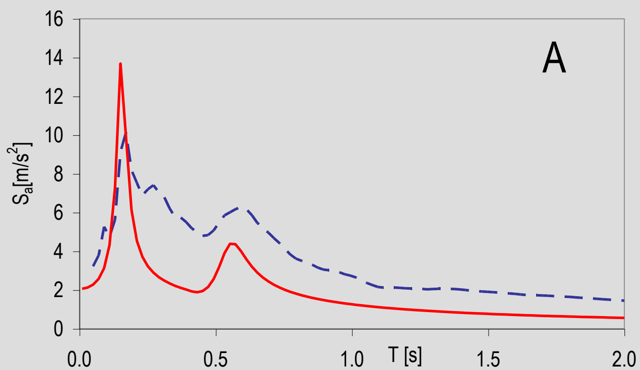
$$\Delta(T)=\sum_{r=1}^{n}\begin{cases}\dfrac{T^2}{T_r^2}\dfrac{\Delta_{h=0}(T_r)|\gamma_r\psi_r(z)|}{\sqrt{\left[1-\left(\frac{T}{T_r}\right)\right]^2+\frac{0.02}{\eta_D{}^2(\xi_s)}\left(\frac{T}{T_r}\right)}} & T\le T_r\\[2ex] \dfrac{T^2}{T_r^2}\dfrac{\Delta_{h=0}(T_r)|\gamma_r\psi_r(z)|}{\sqrt{\left[1-\left(\frac{T}{T_r}\right)\right]^2+0.02\left(\frac{T}{T_r}\right)}}\cdot\eta_D(\xi_s) & T>T_r\end{cases}$$

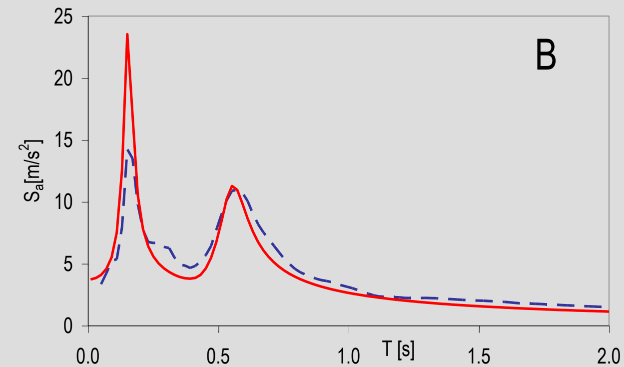






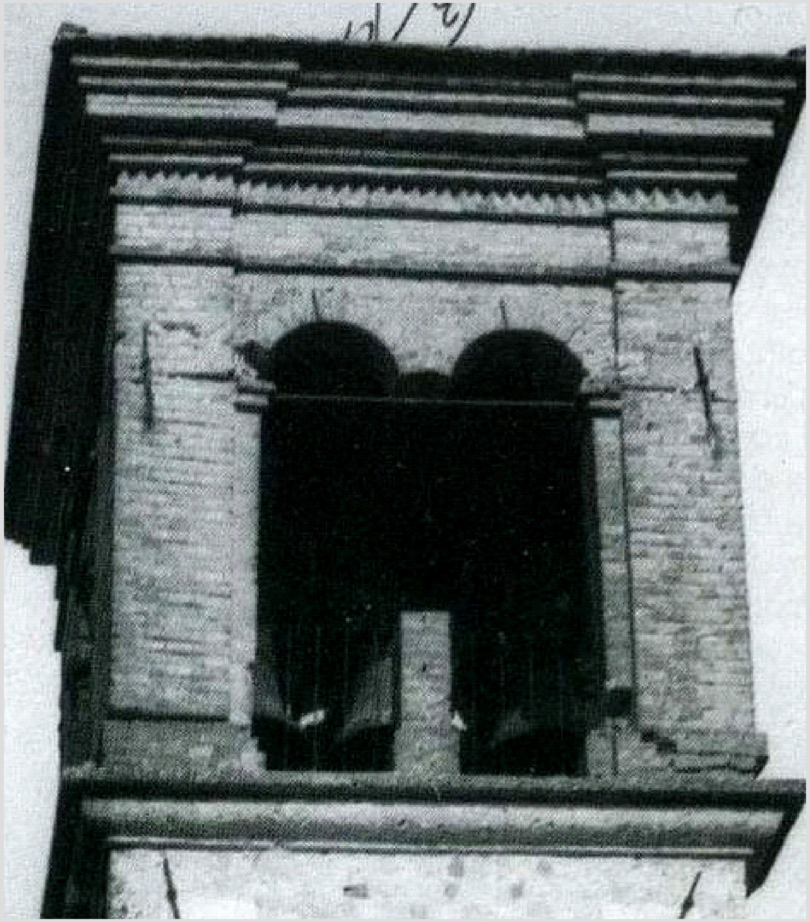




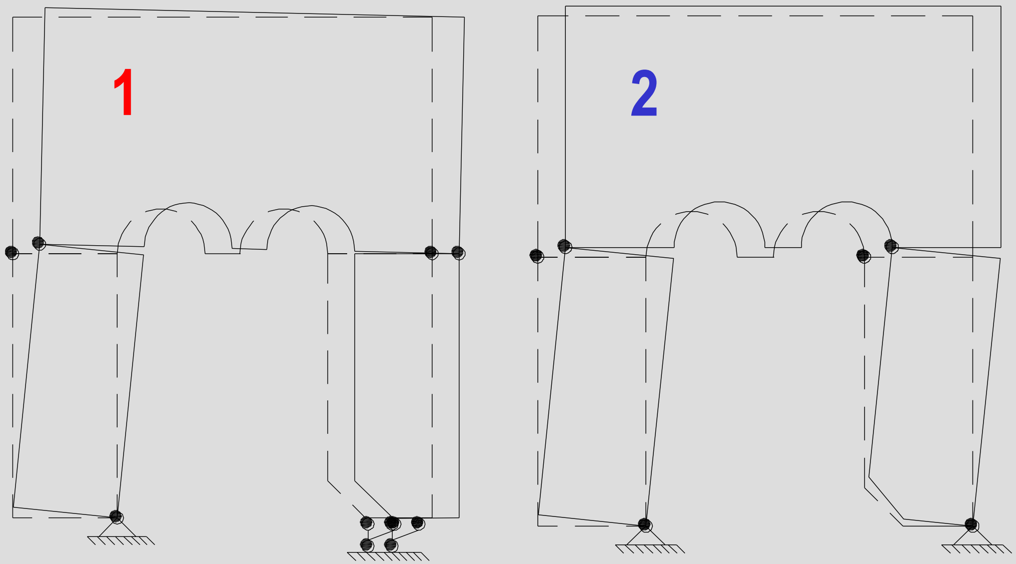


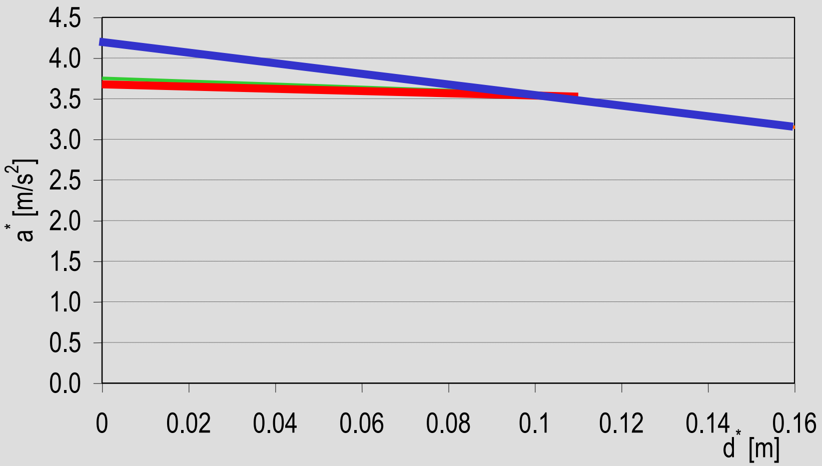
Numerical response spectra — Simplified response spectra

WP5 – Mechanical models for assessment and design



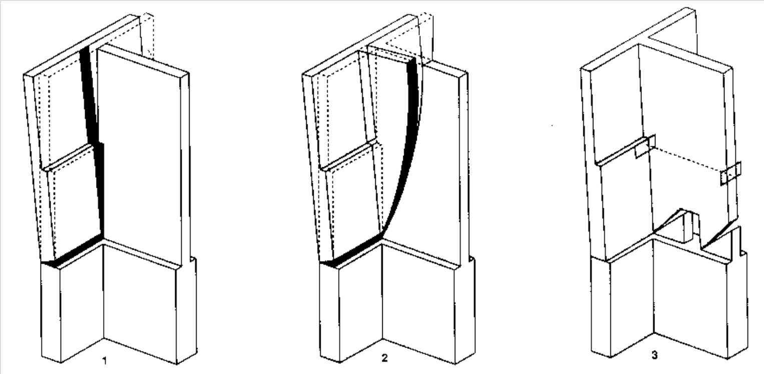
Spilimbergo bell tower
Friuli earthquake, 1976



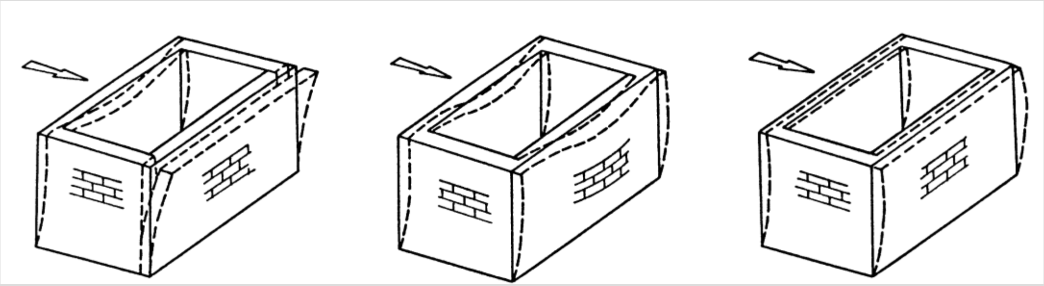


SEISMIC BEHAVIOUR OF MASONRY BUILDINGS

The occurrence of out-of-plane mechanisms may be prevented by improving wall-wall and floor-wall connections (insertion of tie-rods, stiffening of floors, ..)

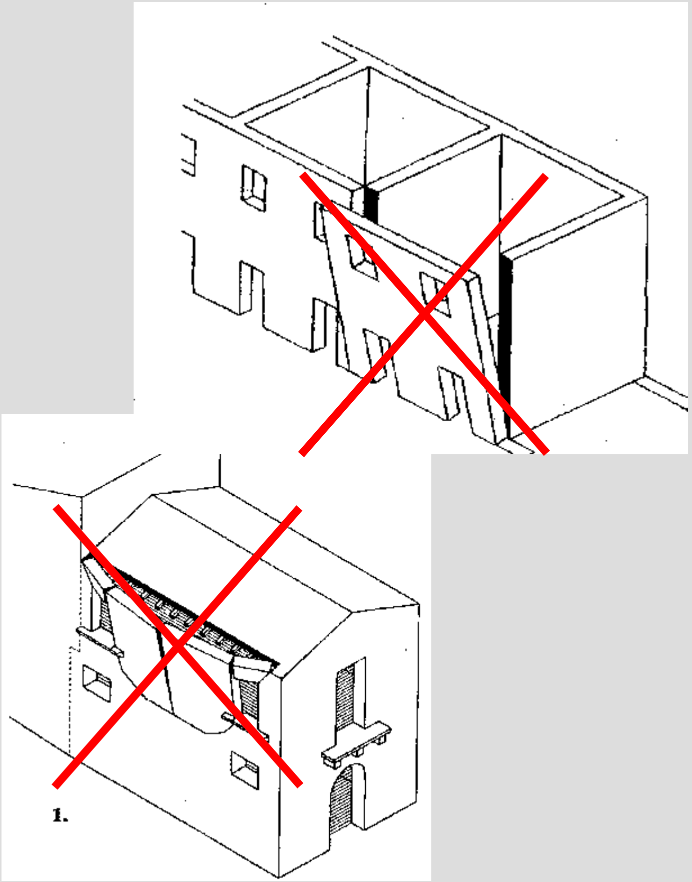


Effect of increasing quality of walls interlocking and insertion of tie-rods



Effect of increasing quality of wall-floor connection and floor stiffening

Examples of out-of-plane mechanisms



SEISMIC BEHAVIOUR OF MASONRY BUILDINGS

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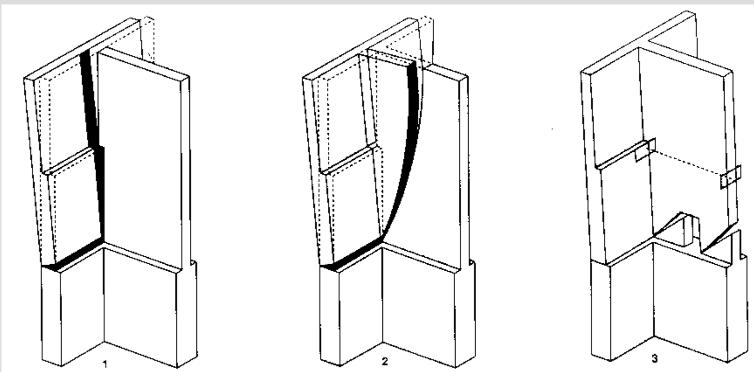
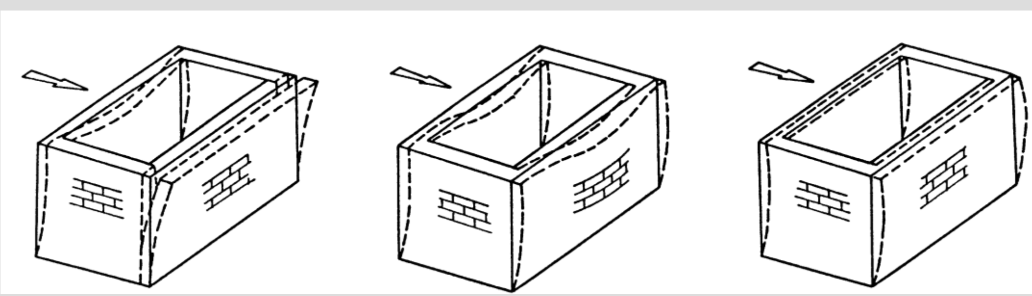
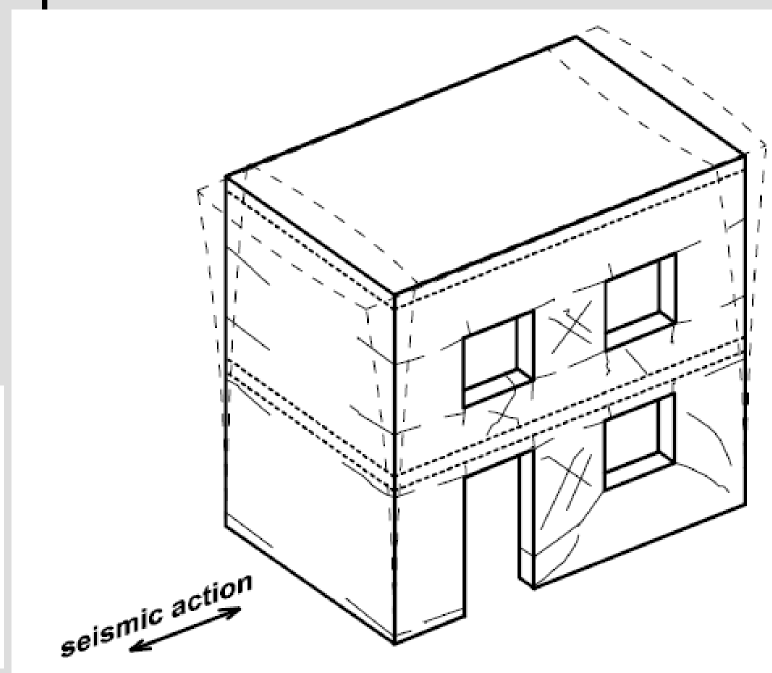


Effect of increasing quality of walls interlocking and insertion of tie-rods



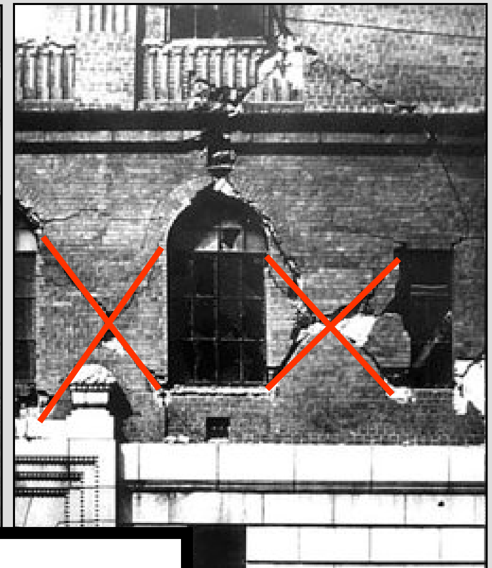
Effect of increasing quality of wall-floor connection and floor stiffening

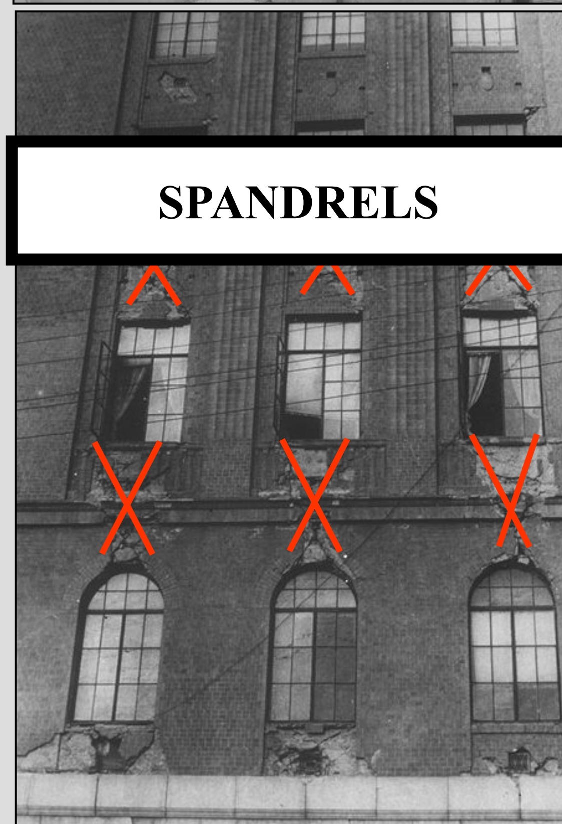
In-plane mechanisms:
Global behaviour of the building



from Magenes 2007 (Proc. of 1th ECEES)

STRUCTURAL ELEMENTS IN MASONRY WALLS

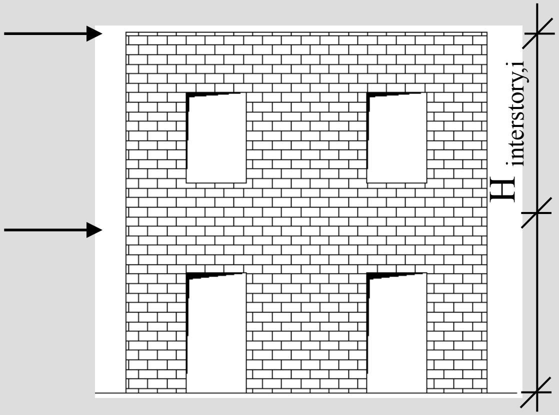


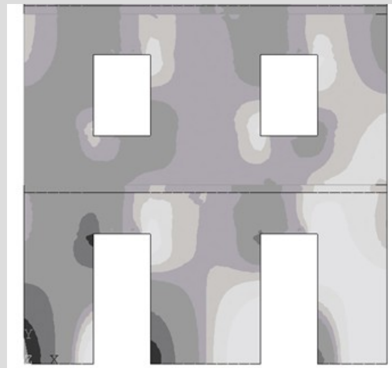


PIERS

SPANDRELS

MODELLING STRATEGIES





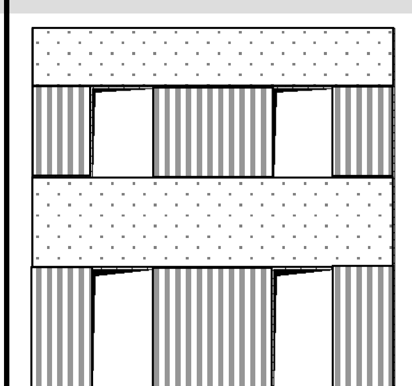
DISCRETIZION OF THE WALL BY FINITE ELEMENTS

Lofti and Shing 1991, Gambarotta and Lagomarsino 1997, Lourenço et al. 1997, Lourenço and Rots 1997, Luciano and Sacco 1997, Zhuge et al. 1998, Pietruszczak and Ushaksaraei 2003, Massart 2003, Schlegel 2004, Calderini and Lagomarsino 2008

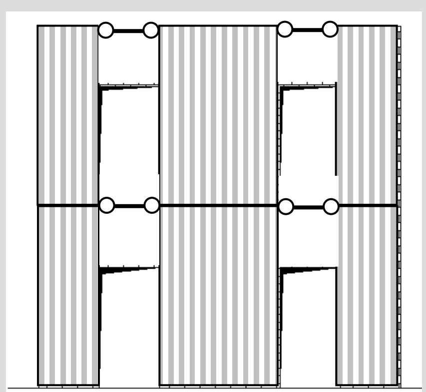
SIMPLIFIED MODELS

(as proposed in FEMA 356 – FEMA 306 – POR Method)

STRONG SPANDREL – WEAK PIER

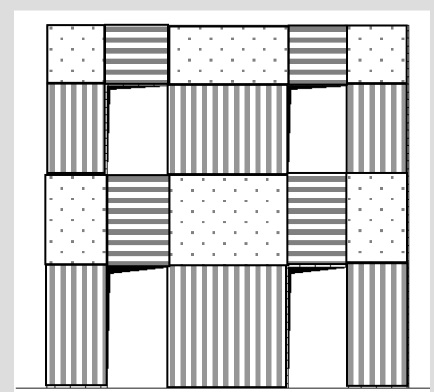


WEAK SPANDREL – STRONG PIER

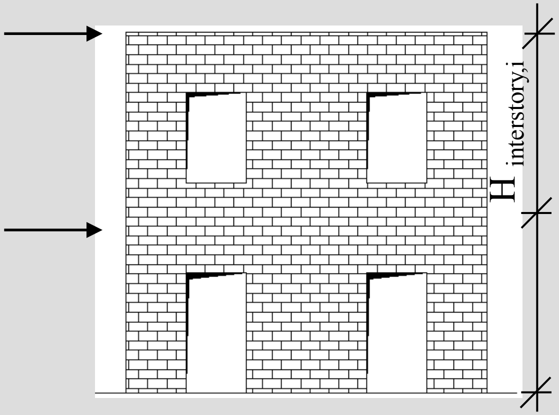


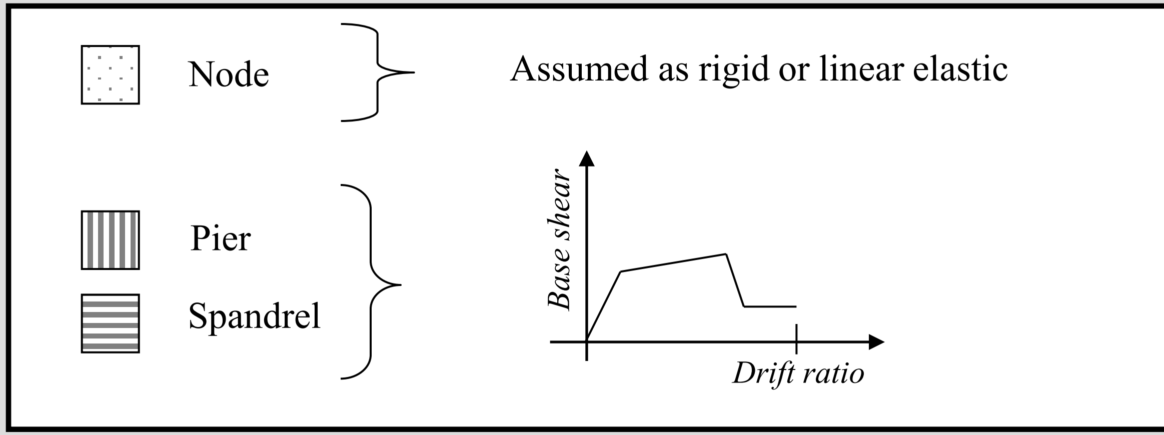
EQUIVALENT FRAME MODEL

(Tomaževič and Weiss 1990, D'Asdia and Viskovic 1995, Brencich and Lagomarsino 1998, Magenes and Della Fontana 1998, Galasco et al. 2004)



MODELLING STRATEGIES

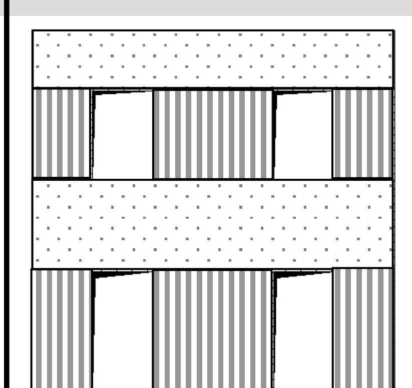




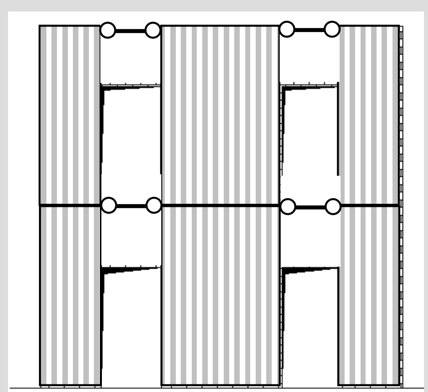
SIMPLIFIED MODELS

(as proposed in FEMA 356 – FEMA 306 – POR Method)

STRONG SPANDREL – WEAK PIER

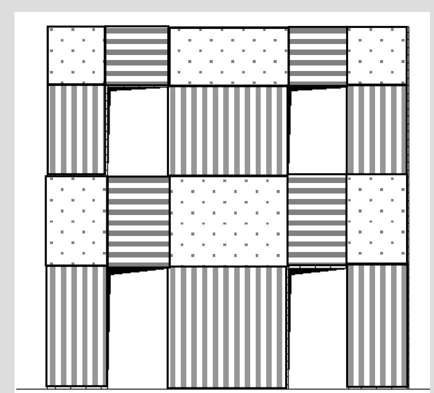


WEAK SPANDREL – STRONG PIER

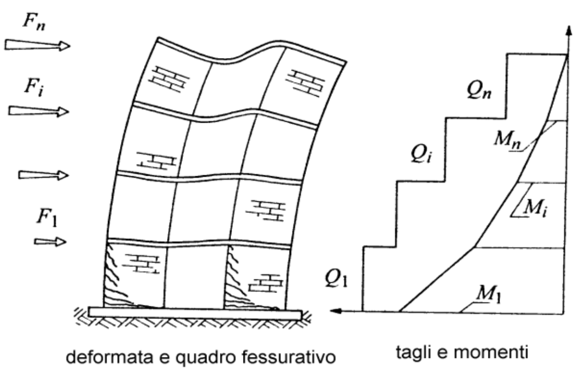


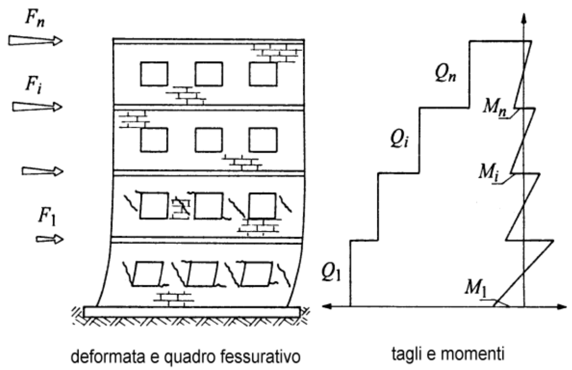
EQUIVALENT FRAME MODEL

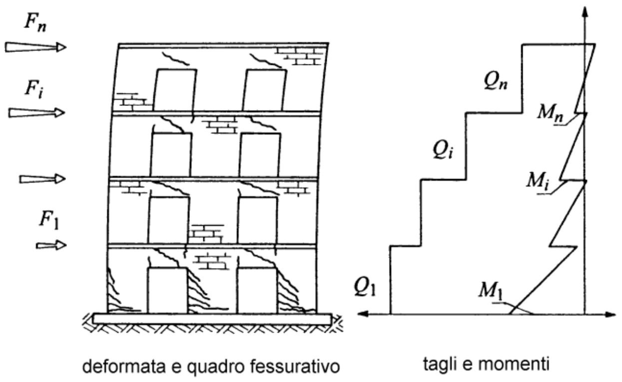
(Tomaževič and Weiss 1990, D'Asdia and Viskovic 1995, Brencich and Lagomarsino 1998, Magenes and Della Fontana 1998, Galasco et al. 2004)

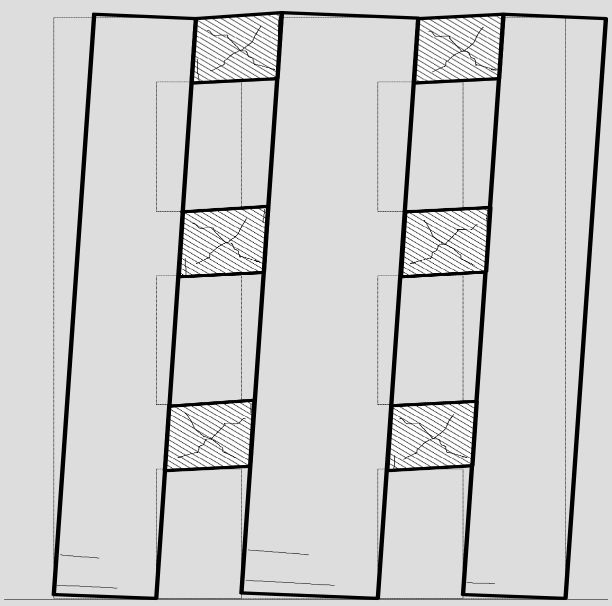


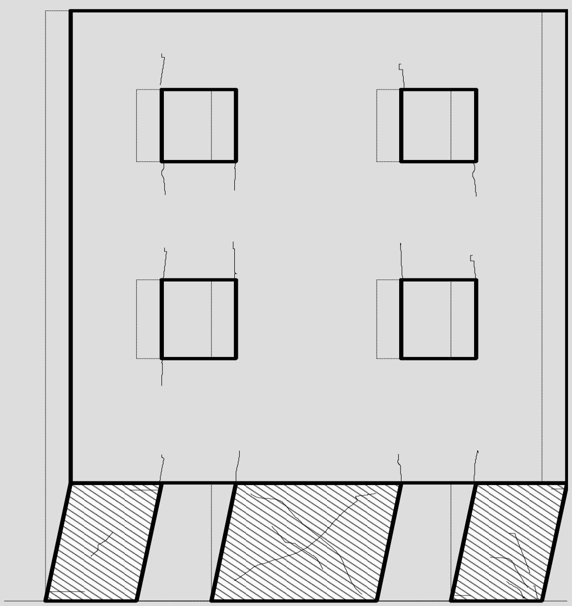
NON LINEAR PUSHOVER ANALYSIS



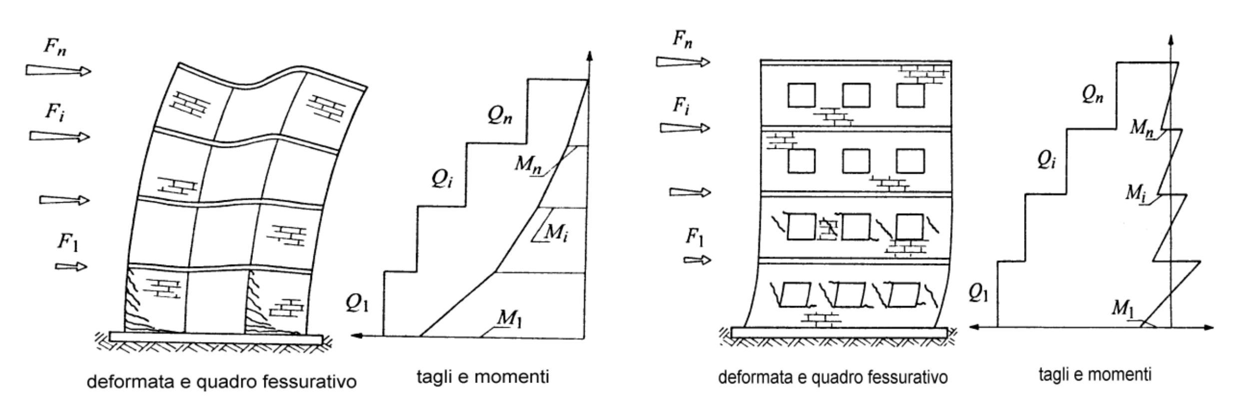






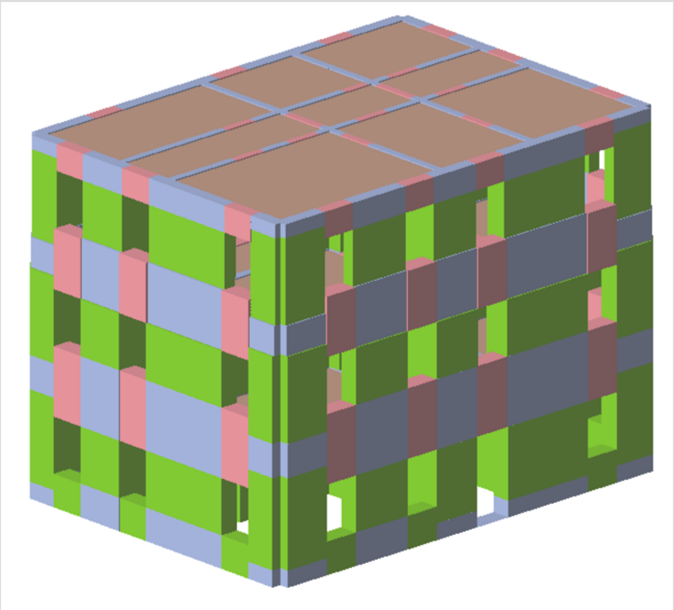


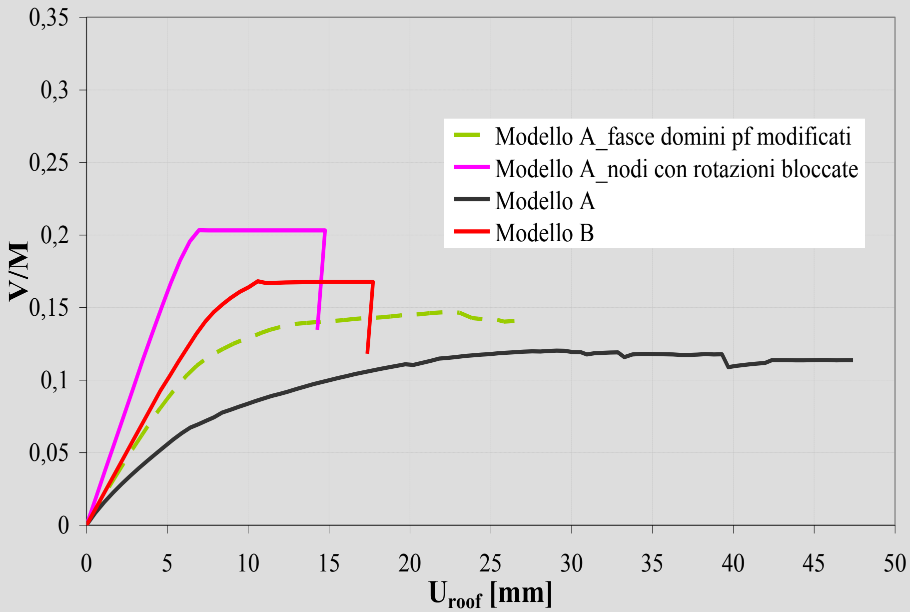
NON LINEAR PUSHOVER ANALYSIS



TREMURI Program (ask to tremuri@gmail.com – commercial release: www.stadata.com)

3D modelling of masonry buildings by the equivalent frame model.





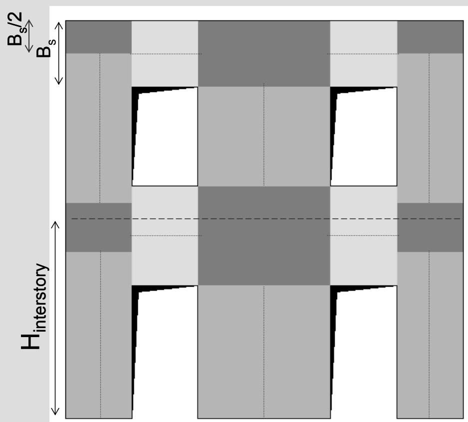
DISCRETISATION OF THE WALL IN AN EQUIVALENT FRAME MODEL

Identification of the geometry of piers, spandrels and rigid nodes

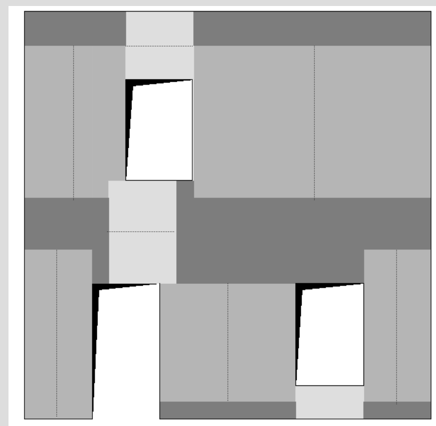
✓ Conventional criteria are assumed in the literature, validated by damage observation after earthquakes and by experimental campaigns (e.g. Benedetti et al., 1998; Calvi and Magenes, 1994; Yi et al., 2006).

✓ These criteria are based on: empirical rules based on opening distribution; conventional stresses diffusion; pattern of existing cracks (in case of existing buildings)

Regular openings



Irregular openings



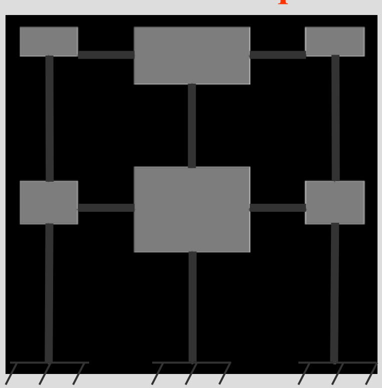
Element axis

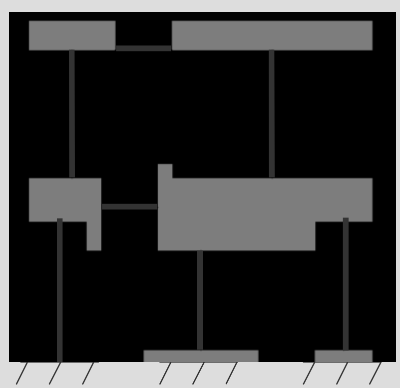
Pier

Spandrel

Rigid node

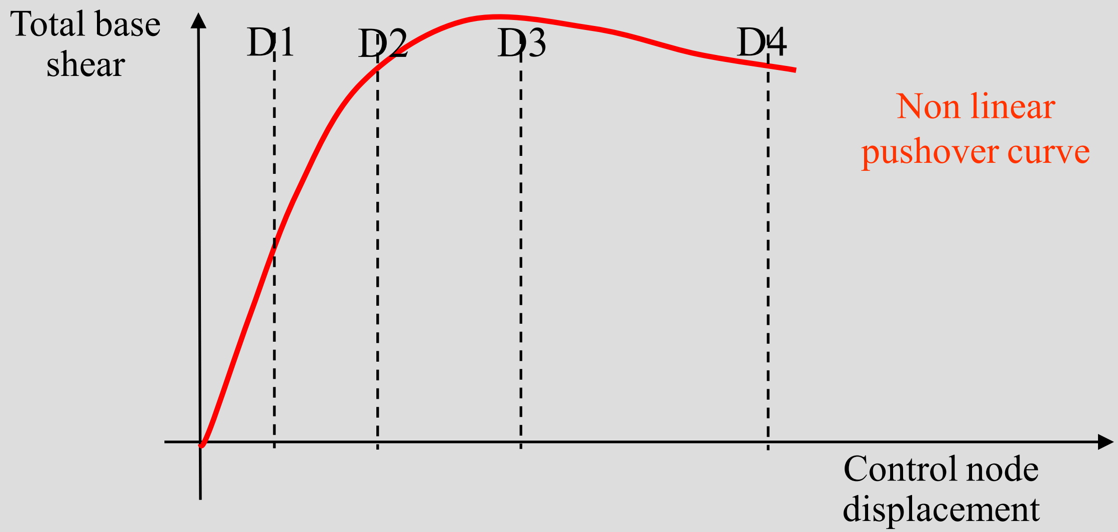
Equivalent frame disretisation

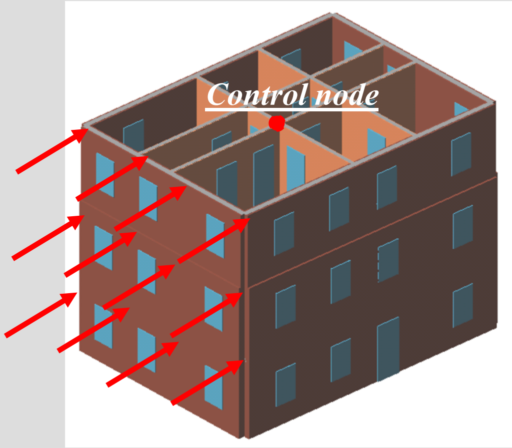


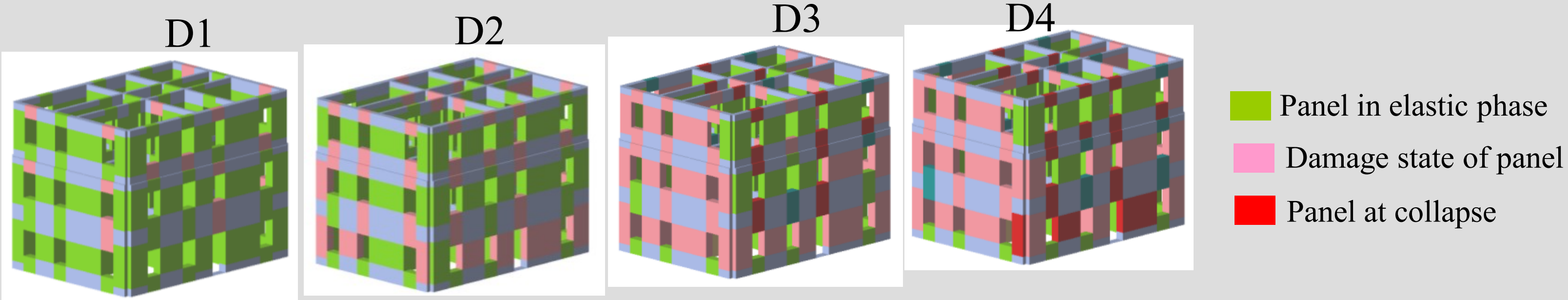


Nonlinear beam or macroelement

NON LINEAR STATIC ANALYSIS



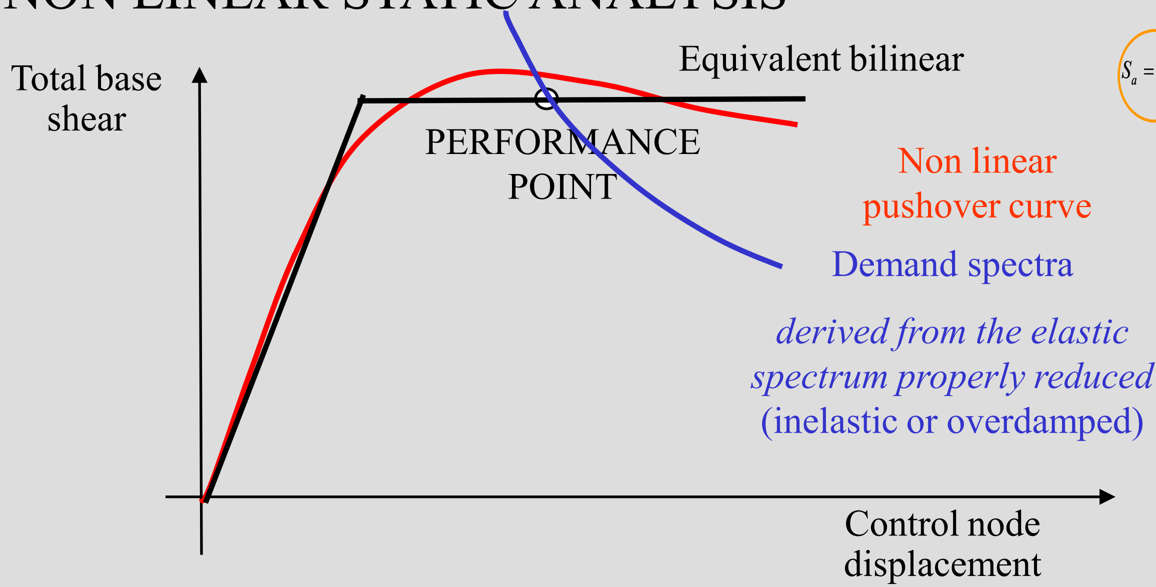




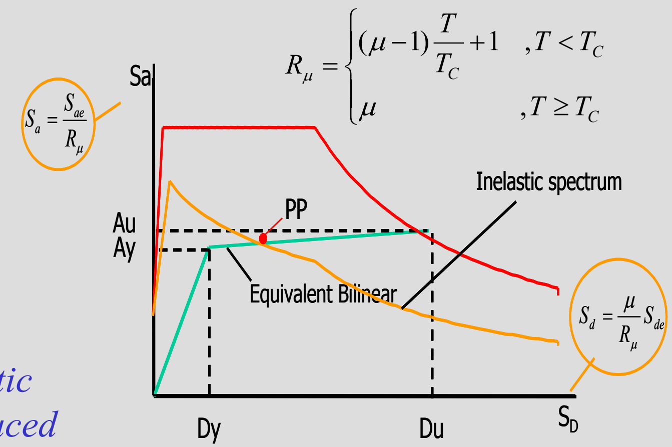
Progressing of non linear response and the damage state of building

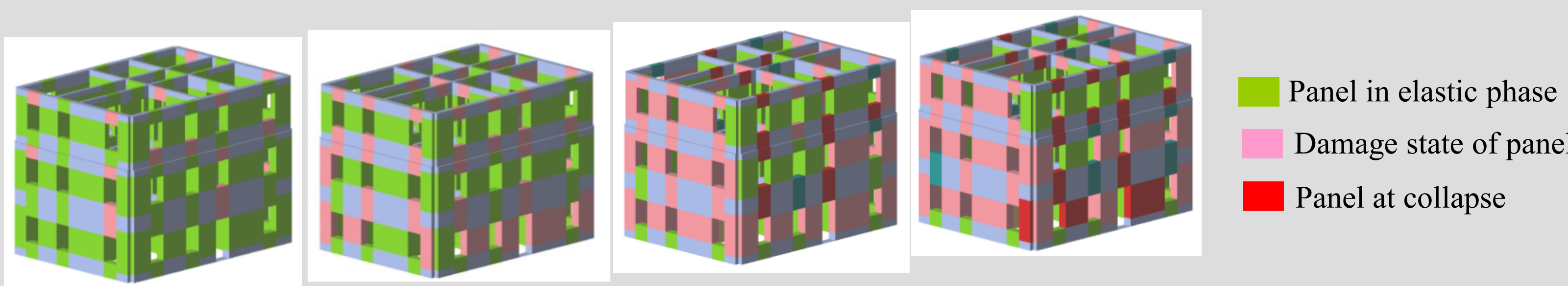
Usually just from the beginning of incremental analysis a non linear response occurs (with damage concentration in spandrels)

NON LINEAR STATIC ANALYSIS



(Fajfar 2000)



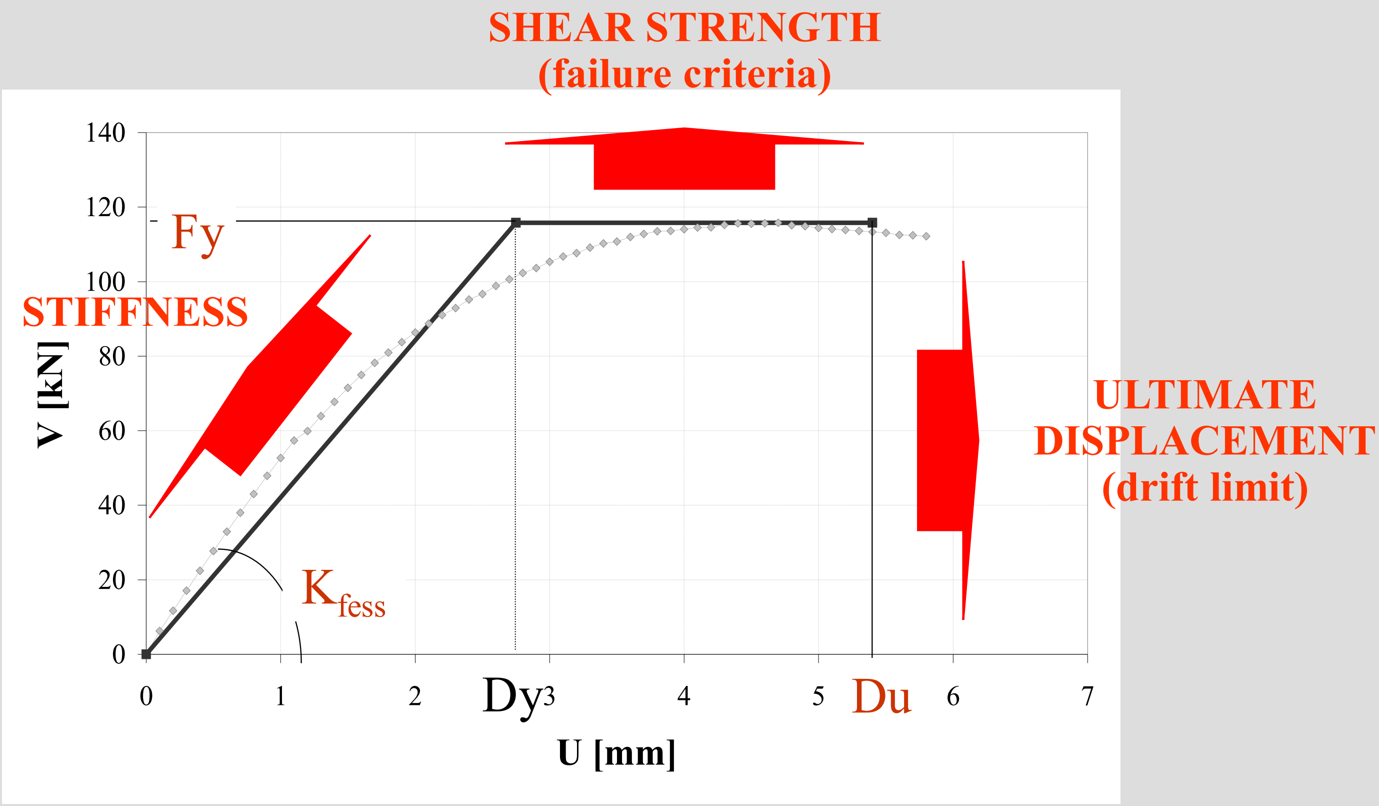


Progressing of non linear response and the damage state of building

Usually just from the beginning of incremental analysis a non linear response occurs (with damage concentration in spandrels)

FORCE – DISPLACEMENT RELATION IN SHEAR

- Laboratory tests on masonry panels (cyclic raking tests) are well simulated by TREMURI macroelement. The force-displacement curve in shear presents a curvilinear trend, with an ultimate displacement (drift limit) depending on softening or brittle failure.
- Different failure modes can occur: ***Rocking*** and ***Crushing*** (flexural behaviour); ***Sliding*** and **Diagonal Cracking** (shear behaviour).
- The curve may be approximated by a bilinear curve, defined by three parameters.



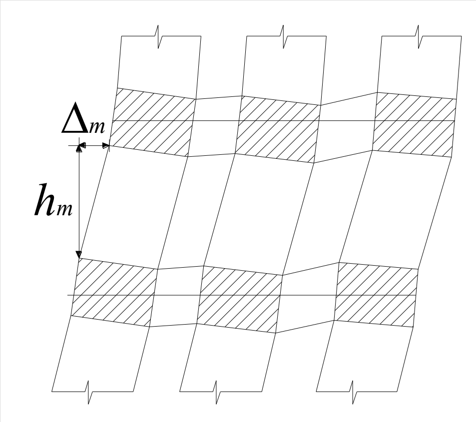
3MURI – NON LINEAR BEAM

- Stiffness properties

A secant stiffness may be defined, but this is an approximation, as it is not possible to know in advance the actual stiffness degradation in each element.

Eurocode 8 and the new Italian code propose to adopt for the panel a reduced initial elastic stiffness of the beam; usually, unless specific information are available, a reduction of 50% is proposed.

- Ductility control in terms of drift limits



Drift limits defined as a function of the failure mode occurred

$$\delta_m^{DL} = \frac{\Delta_m}{h_m} = \delta_u \begin{cases} 0.004 & \text{Shear} \\ 0.006\text{-}0.008 & \text{Rocking} \end{cases}$$

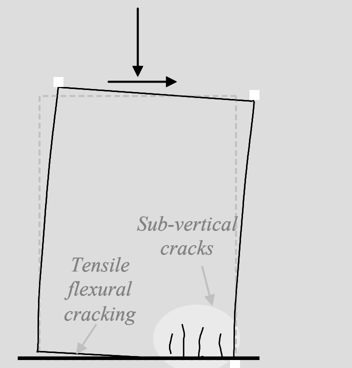
Shear failure

Rocking failure

3MURI – NON LINEAR BEAM – FAILURE CRITERIA FOR PIERS

- Shear strength for masonry piers

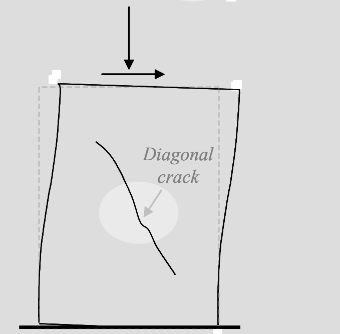
Failure criteria for masonry piers are based on the approximate evaluation of the local (or mean) stress state induced by the applied forces on predefined points (or sections) of the panel, to be compared with proper limit strength domain for the constituent material (and the panel itself)



FLEXURAL BEHAVIOUR

Failure modes: *Rocking* and/or *Crushing*

$$M_u = \frac{Nl}{0.425 f_m}\left(1 - \frac{N}{lt}\right)$$



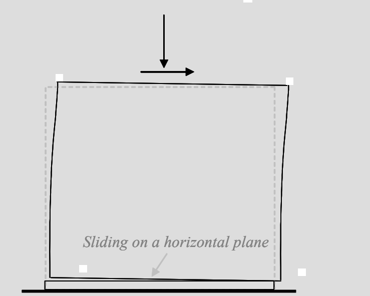
SHEAR BEHAVIOUR – PRINCIPAL STRESS MODELS

Failure modes: *Diagonal Cracking*

(Turnsek and Cacovic Theory, 1971)

$$T_u = lt\frac{f_t}{b}\sqrt{1 + \frac{N}{f_t lt}}$$

b = *f* (slenderness of the panel)



SHEAR BEHAVIOUR – COULOMB TYPE MODELS

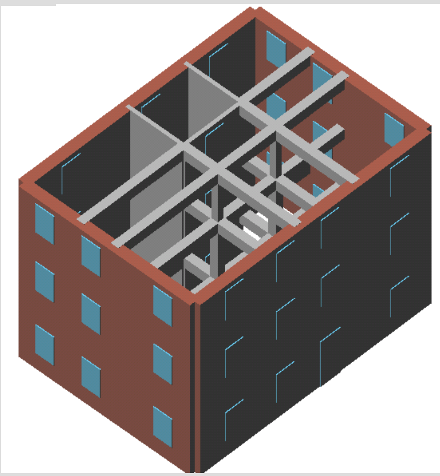
Failure modes: *Bed Joint Sliding* – *Diagonal Cracking through Joints*

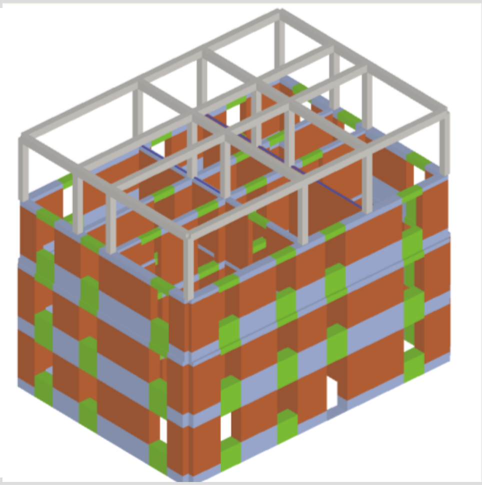
$$T_u = l'tc + \mu N$$

$$T_u = \frac{lt}{b}(\tilde{c} + \tilde{\mu}N)$$

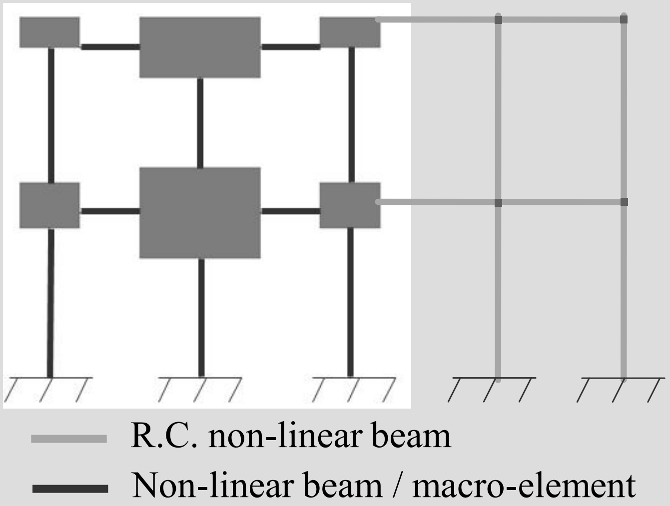
3MURI – REINFORCED CONCRETE BEAM ELEMENTS

✓ Examples A: buildings with external masonry walls and internal r.c .beams, raising-up of masonry building with new r.c. storey, enlargement with a r.c. portion

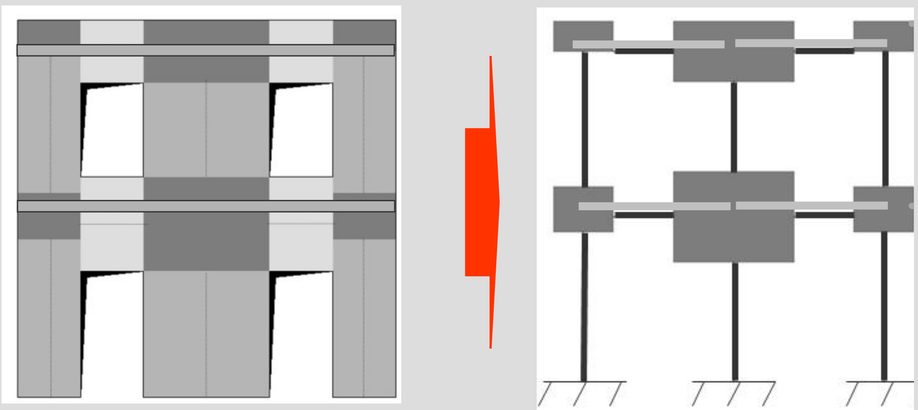


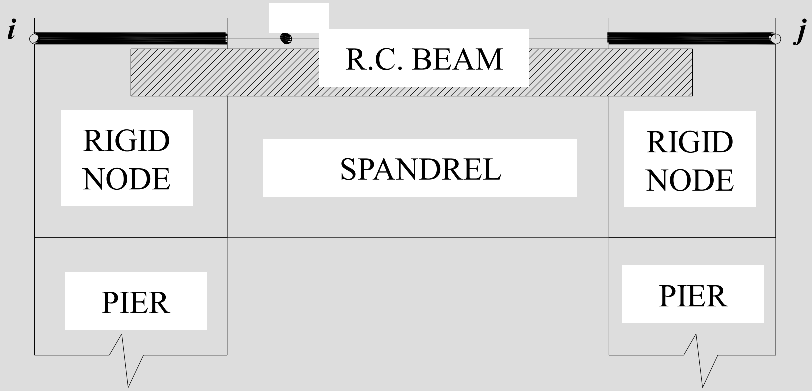


Equivalent frame idealisation



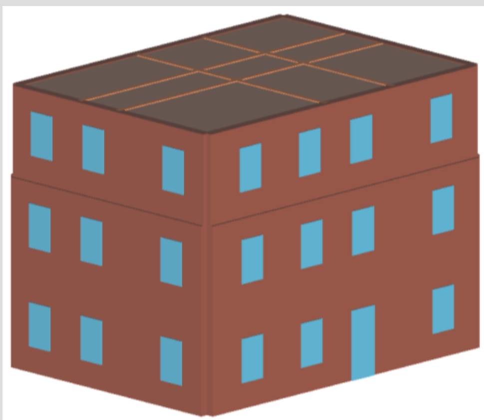
✓ Examples B: prensence of r.c. beam coupled to spandrels

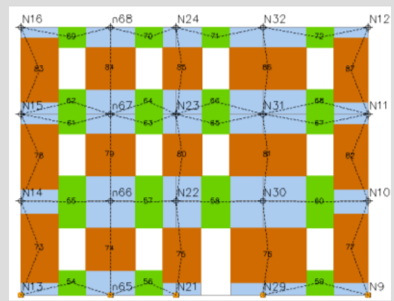




3-D EQUIVALENT FRAME MODEL

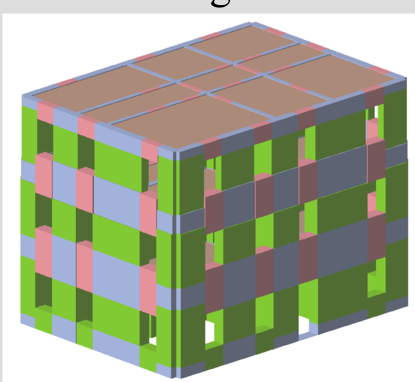
- ✓ 3-D equivalent frame is made of masonry walls, elastic floors, reinforced concrete elements
- ✓ out-of-plane stiffness and strength of masonry walls are neglected
- ✓ floors are planar elements with only in-plane stiffness, for sharing seismic action on walls



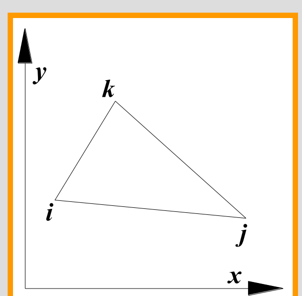


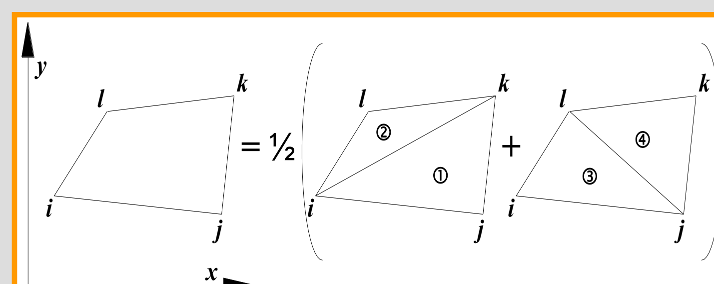
- Equivalent frame idealisation of each structural walls
- Assembling of 2D walls

Modelling of floors

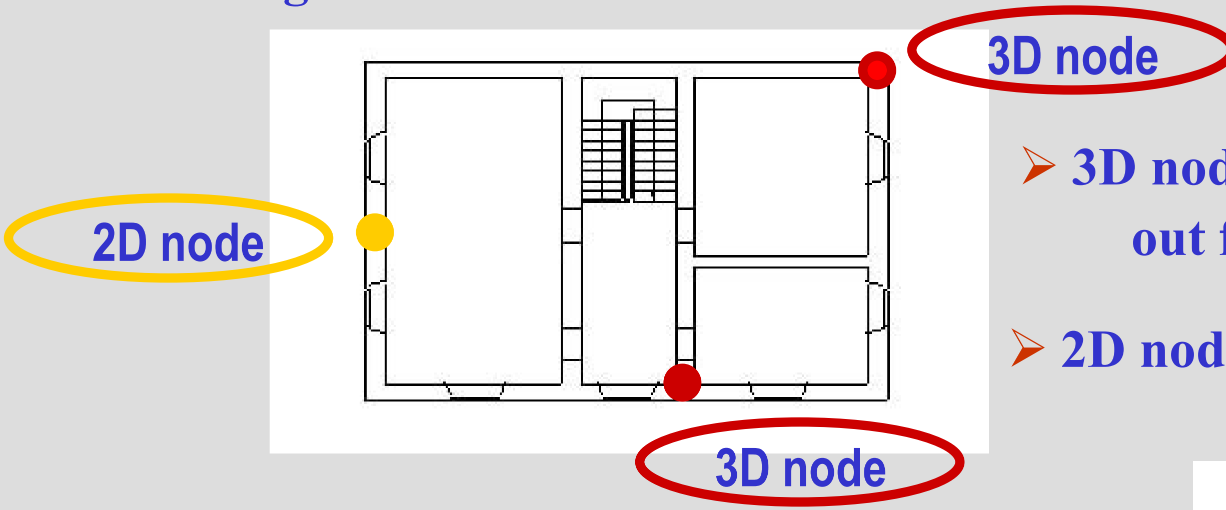


The floor are orthotropic membrane finite elements, with 3 or 4 nodes, defined by two independent axial stiffness and the shear stiffness

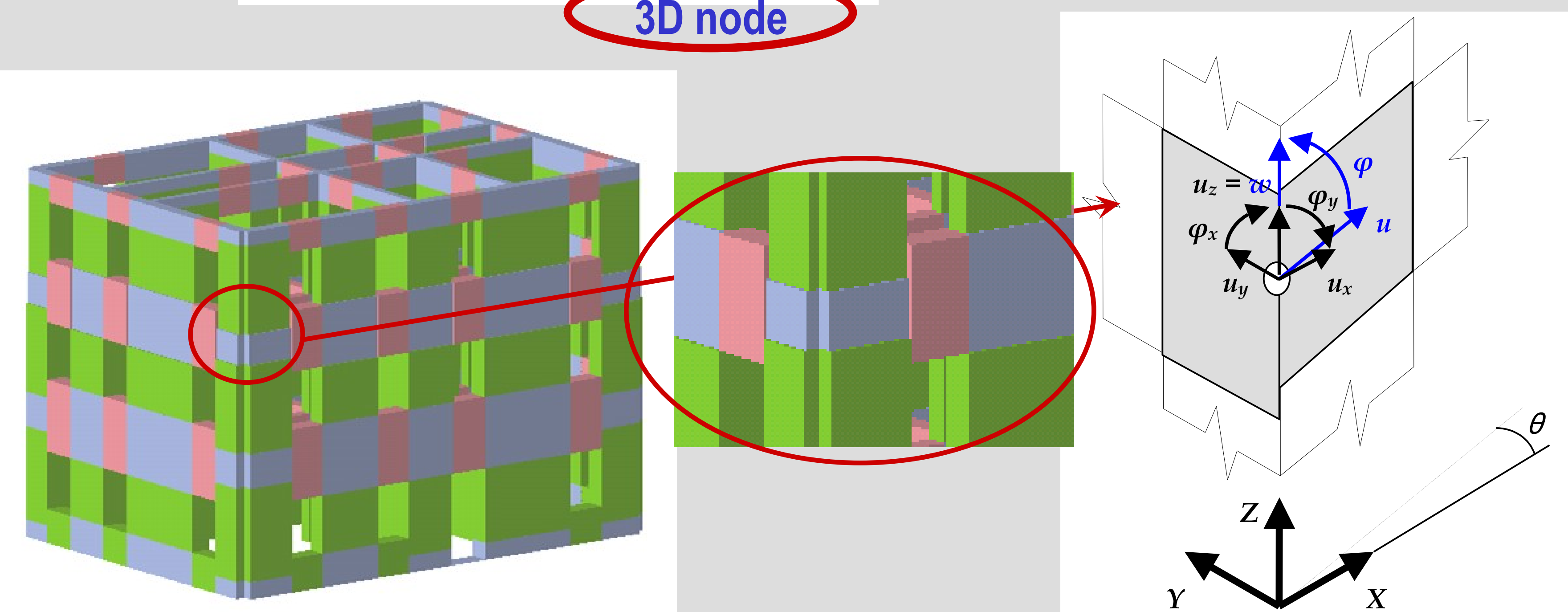




Assembling 2D walls



- 3D nodes: 5 d.o.f → they come out from two 2D nodes
- 2D nodes: 3 d.o.f. in the wall plane

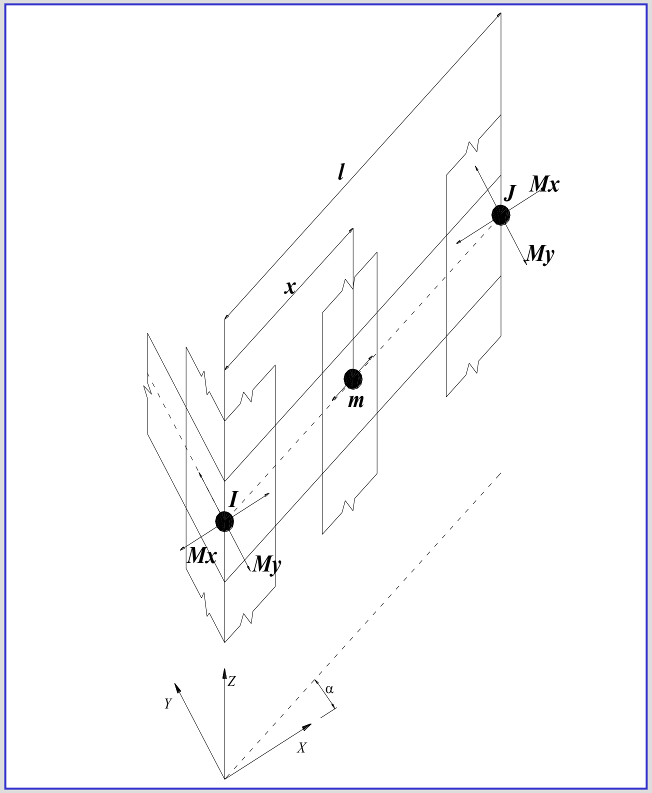


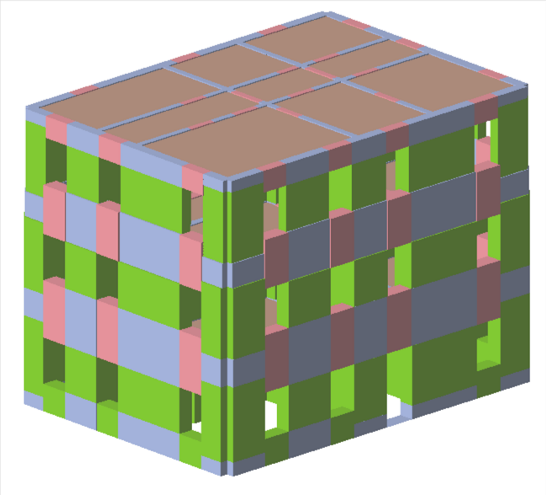
3-D EQUIVALENT FRAME MODEL

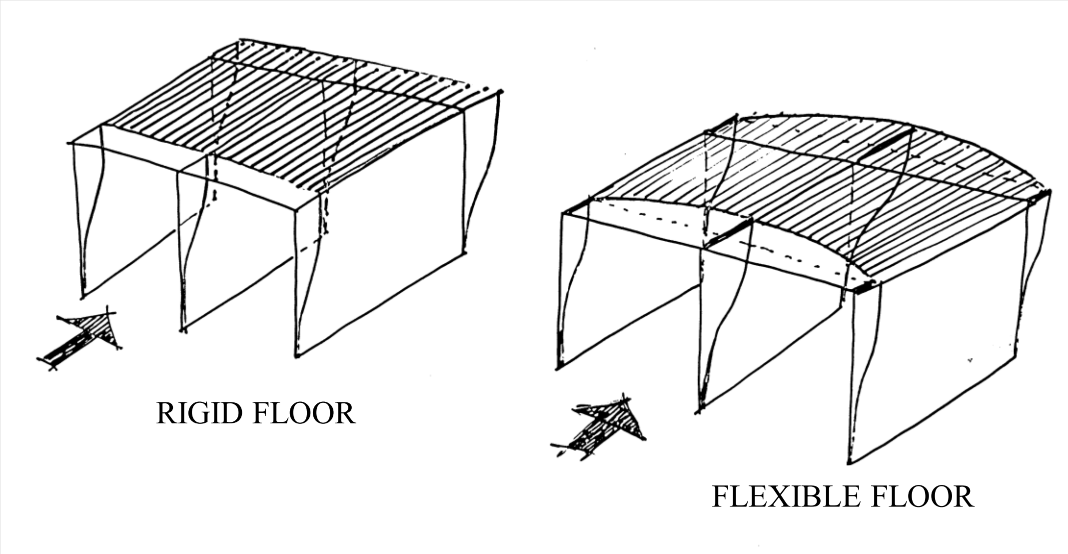
2D nodes: Sharing out of the nodal mass in the out-of-plane direction

$$M_x^I = M_x^I + m(1-|\cos\alpha|)\frac{l-x}{l}$$

$$M_y^I = M_y^I + m(1-|sin\alpha|)\frac{l-x}{l}$$







3-D EQUIVALENT FRAME MODEL – FLOOR STIFFNESS

TIMBER FLOORS

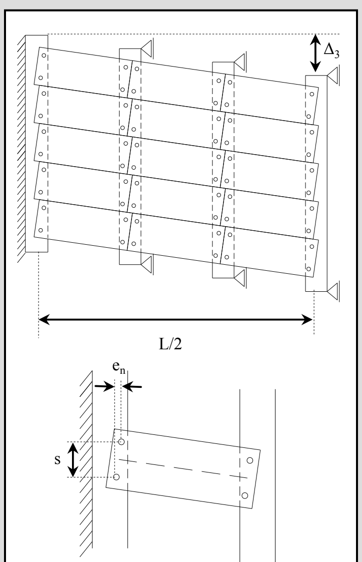
FEMA Guidelines

Default expected values for the in-plane stiffness of different kinds of diaphragm related to existing, enhanced or new configurations

Diaphragm type	G_d [kN/mm]
Single straight sheathing	0.35
Double straight sheathing	
Chorded	2.67
Un-chorded	1.24
Wood structural panel overlay on straight sheathing	
Un-blocked, un-chorded	0.87
Un-blocked , chorded	1.58
Blocked, un-chorded	1.24
Blocked, chorded	3.20

New Zealand guidelines

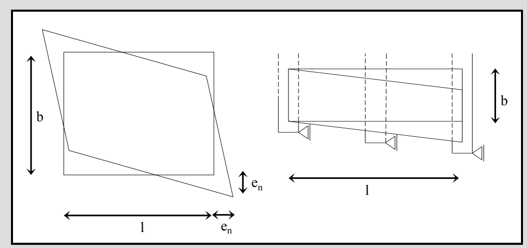
Provide formulae for the evaluation of the diaphragm stiffness depending, for each floor type, on the properties of each component.



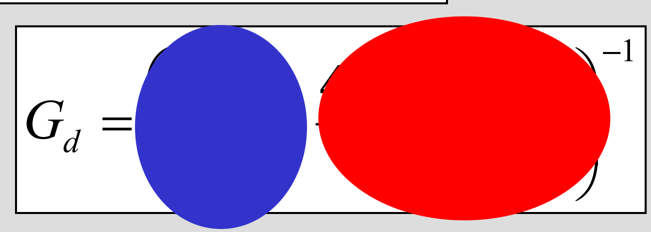
Single straight Sheathing

$$G_d = \frac{s^2 \cdot k_{nail}}{2 \cdot l \cdot b}$$

(only the contribution of nails is considered)



Panels sheathing



Panel contribution

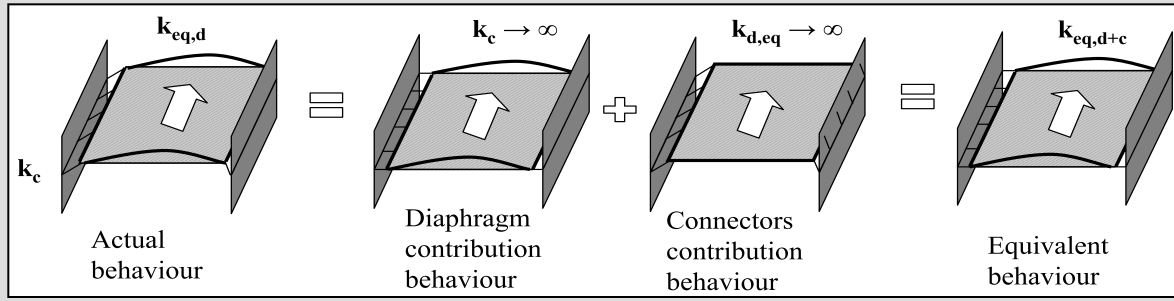
Nails contribution

3-D EQUIVALENT FRAME MODEL – FLOOR STIFFNESS

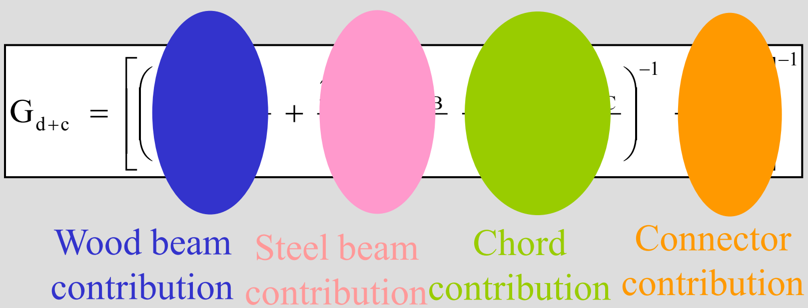
TIMBER FLOORS

Brignola 2009 (PhD Thesis)

Proposal based on the results of an experimental campaign carried out at Canterbury University

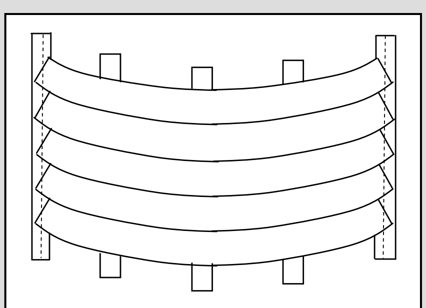


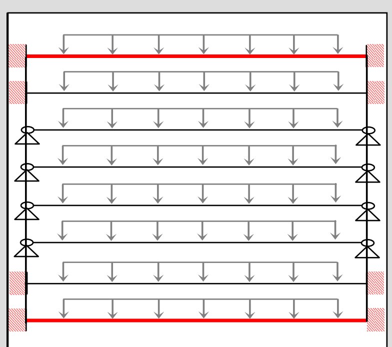
Straight flooring boards



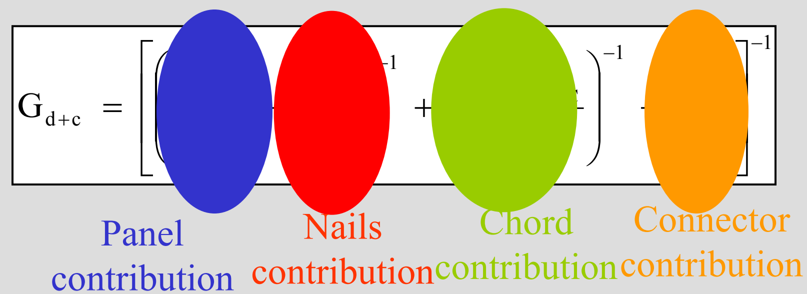
Chord and steel beam contribution

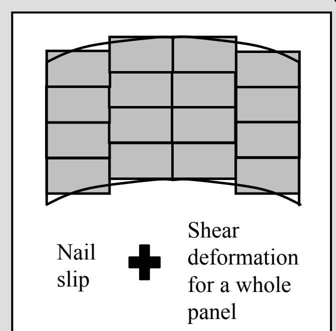
Wood beam contribution

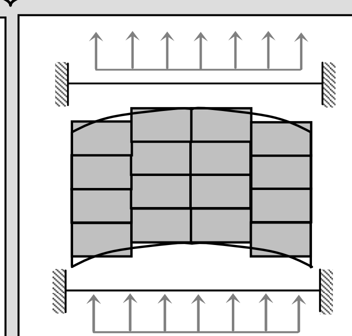




Straight flooring boards and plywood panels overly



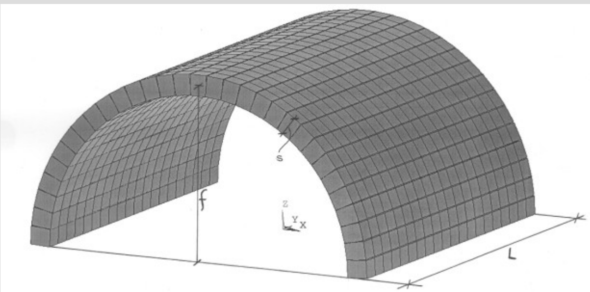




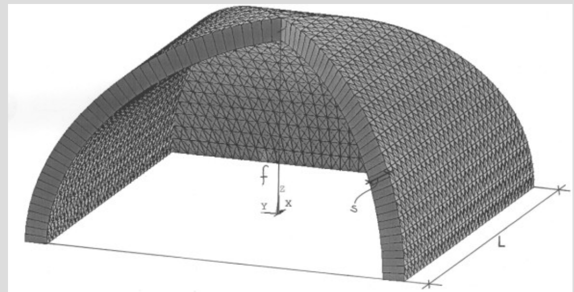
3-D EQUIVALENT FRAME MODEL – FLOOR STIFFNESS

MASONRY VAULTS

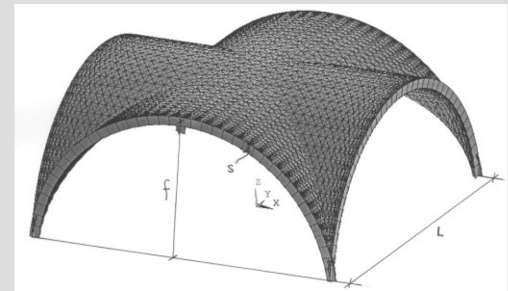
Linear and non linear parametric FEM simulations (Cattari et al. 2008, Proc. of 6th SAHC)



Barrel vault

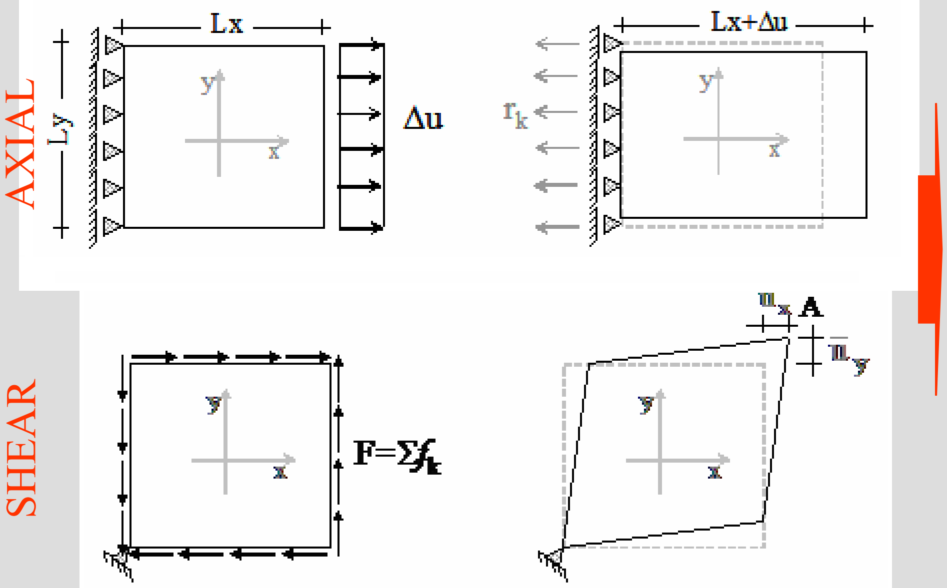


Cloister vault



Cross vault

Boundary configuration and loads



LINEAR NUMERICAL ANALYSES

Shear stiffness G_v/G

Analytical expressions aimed to take into account the shape effect and the boundary conditions as a function of the vault types

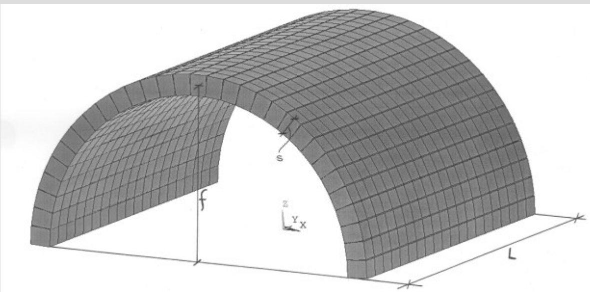
$$\frac{G_v}{G} = \left(b_1 \frac{s}{L} + b_2\right)\left(\frac{f}{L}\right) + b_3 \frac{s}{L} + b_4$$

	Vault tipology		
	Barrel	Cloister	Cross
b_1	1.6433	-0.1633	-1.12
b_2	-1.8191	-1.3051	-4.1766
b_3	0.1133	1.9733	8.9033
b_4	1.1189	1.0339	0.7273
b_5	-	-	-34.323
b_6	-	-	5.6202

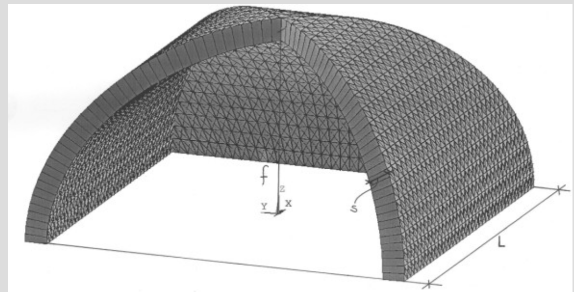
3-D EQUIVALENT FRAME MODEL – FLOOR STIFFNESS

MASONRY VAULTS

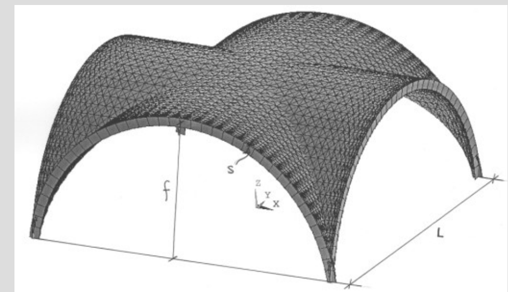
Linear and non linear parametric FEM simulations (Cattari et al. 2008, Proc. of 6th SAHC)



Barrel vault

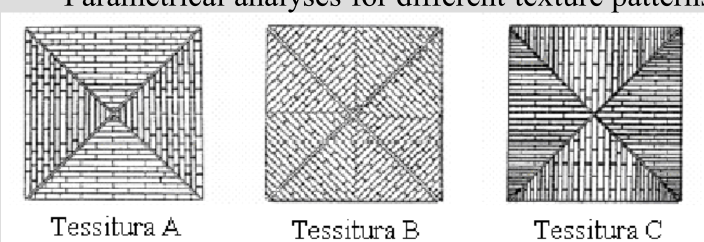


Cloister vault

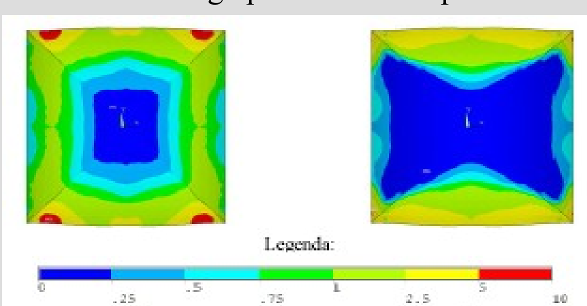


Cross vault

Parametrical analyses for different texture patterns

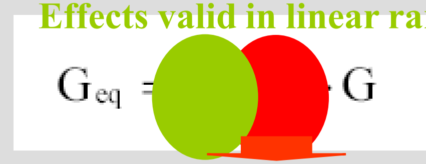


Cloister vaults – damage pattern texture pattern A – shear response

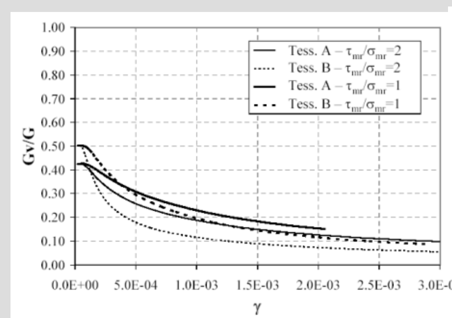


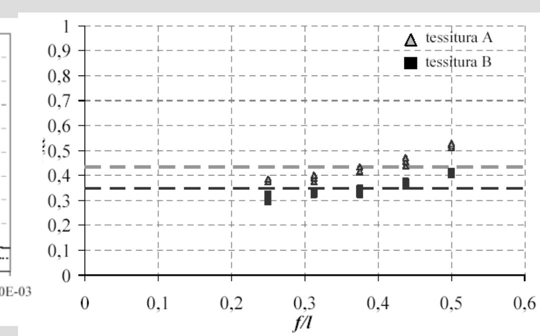
NON LINEAR NUMERICAL ANALYSES

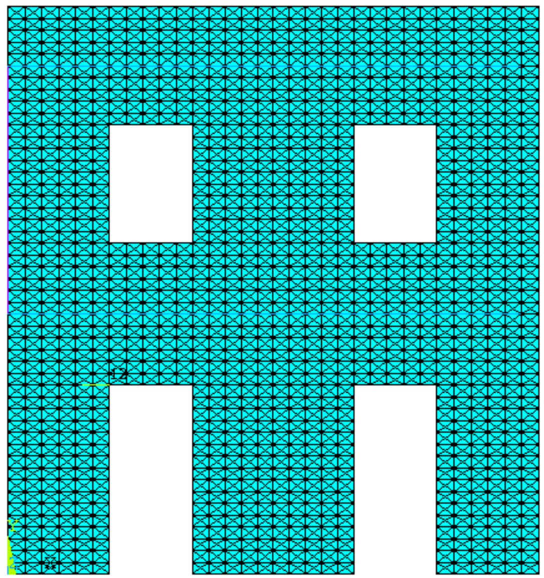
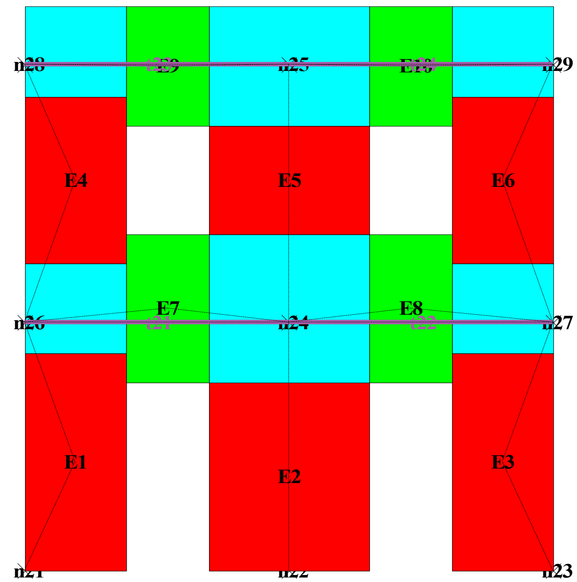
Effects valid in linear range



Effect of stiffness degradation





	Finite Element Model	Equivalent Frame Model
Modelling scale		
Constitutive law	Constitutive law for masonry formulated by Calderini and Lagomarsino	Non-linear beam model - Elasto-plastic constitutive law with secant stiffness degradation. The resistance criteria adopted are: for the *Rocking*, that proposed in the Italian Code ; for the *Diagonal Cracking*, the criterion proposed by Mann and Müller

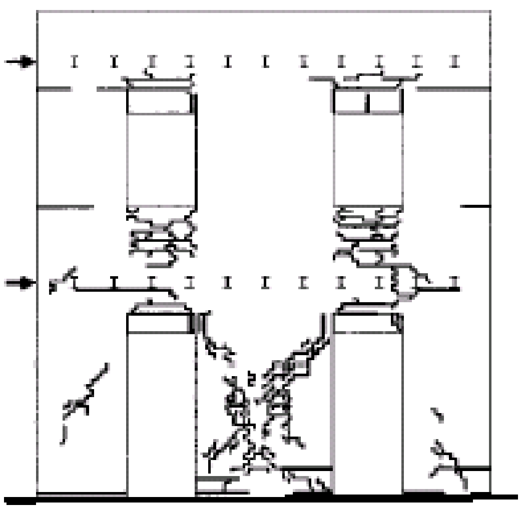
REF.: Calderini C. , Cattari S. , Lagomarsino S. ,IN PLANE SEISMIC RESPONSE OF UNREINFORCED MASONRY WALLS: COMPARISON BETWEEN DETAILED AND EQUIVALENT FRAME MODELS, COMPDYN Conference, Rodhes, 2009

EQUIVALENT FRAME MODEL: VALIDATION AND EXAMPLES OF APPLICATION

	FINITE ELEMENT MODEL	EQUIVALENT FRAME MODEL
Mechanical parameters	*Elastic compliance parameters* Young modulus: 1400 MPa Shear modulus: 480 MPa *Inelastic compliance parameters* Ratio elastic/inelastic shear strain at failure in mortar joints: 4 Ratio elastic/inelastic normal strain of masonry in compression at failure: 2 *Strength parameters* Mortar joints Cohesion: 0.23 MPa Friction coefficient: 0.58 Tensile strength: 0.04 MPa Softening coefficient: 0.4 Blocks Tensile strength: 1.22 MPa Shear strength: 2.1 MPa Softening coefficient: 0.0 Masonry Compressive strength: 6.2 MPa Block aspect ratio (width/height): 4	*Elastic compliance parameters* Young modulus: 700 MPa Shear modulus: 240 MPa *Strength parameters* Mortar joints Cohesion: 0.23 MPa Friction coefficient: 0.58 Blocks Tensile strength: 1.22 MPa Masonry Compressive strength of masonry: 6.2 MPa Block aspect ratio (width/height): 4

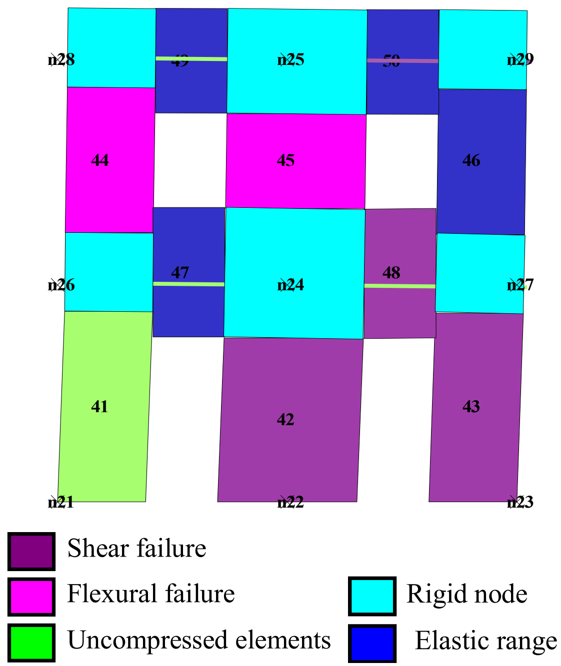
DAMAGE STATE AT COLLAPSE

PROPOTYPE

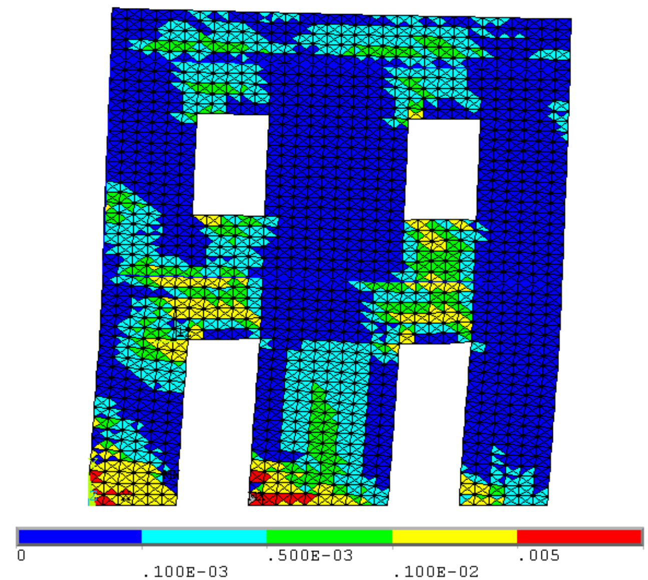


dx = 15 mm

EQUIVALENT FRAME

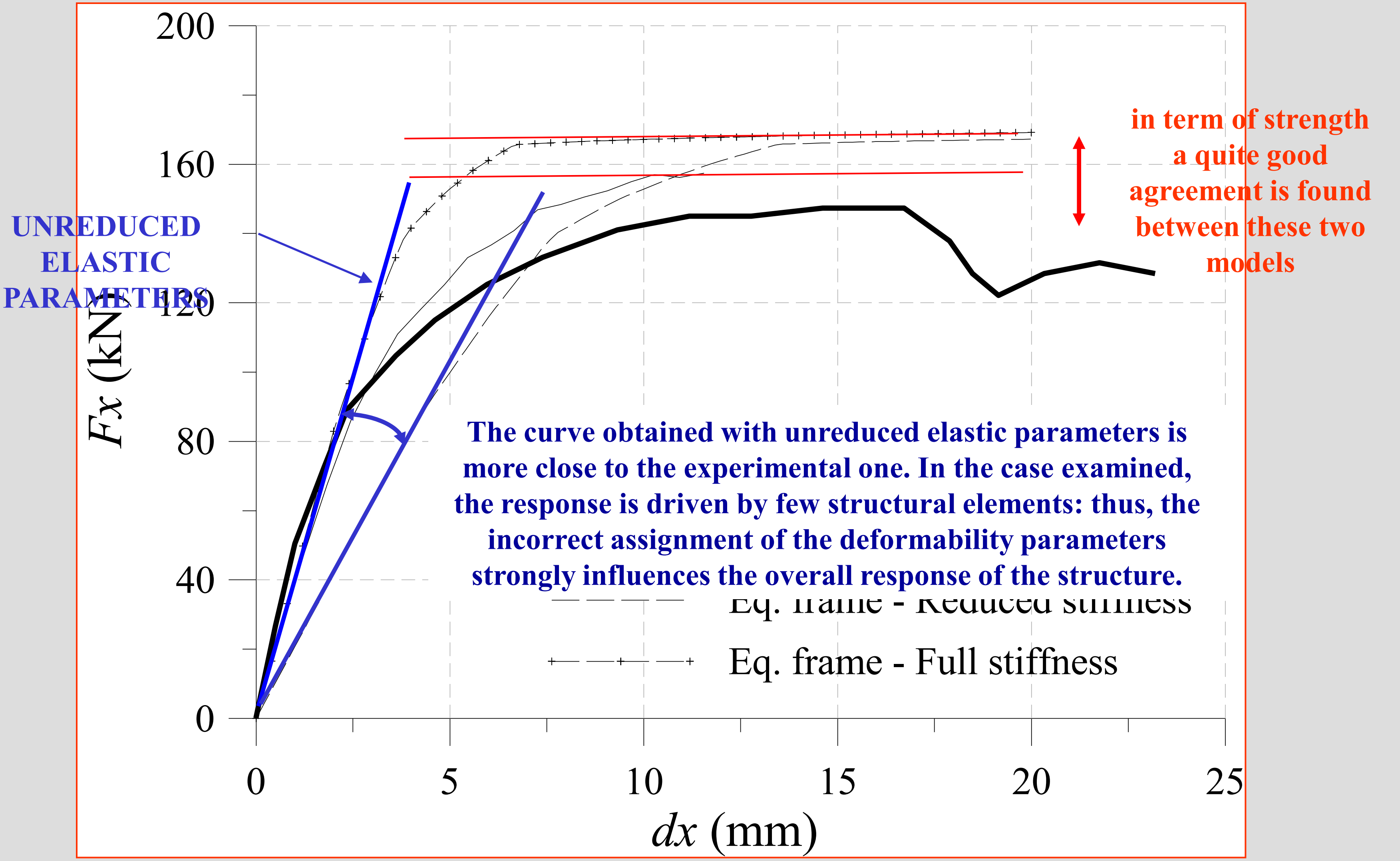


FEM



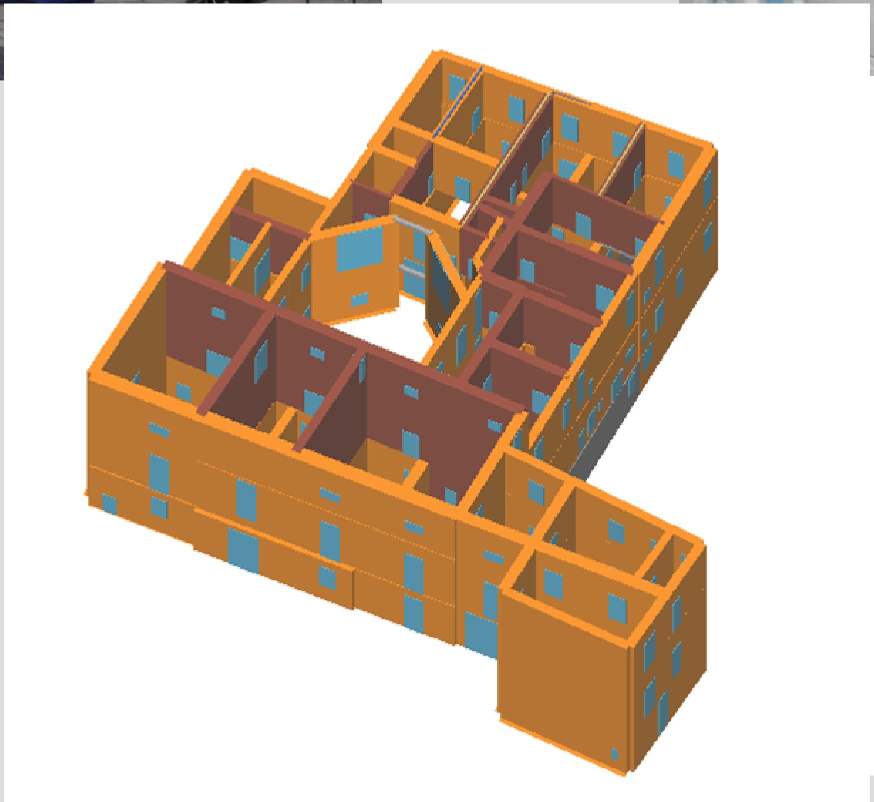
Principal inelastic strains

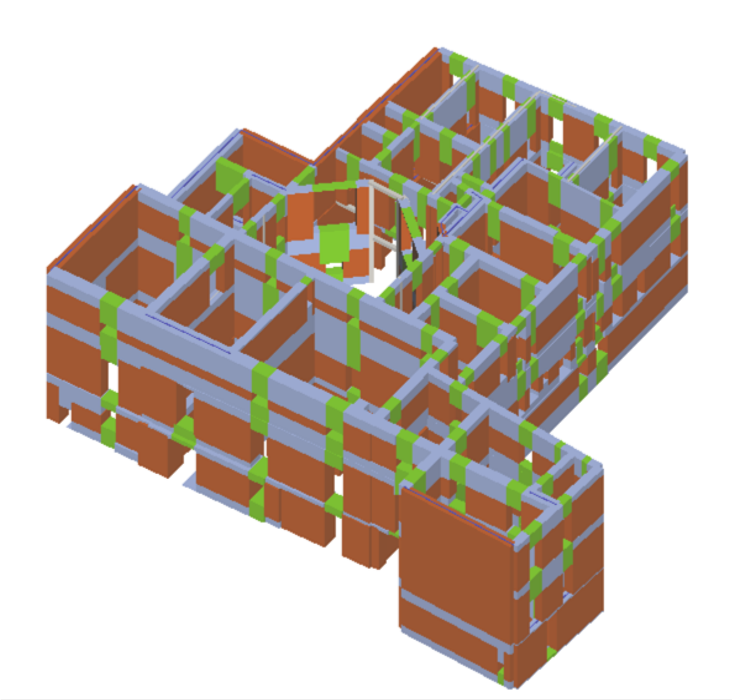
EQUIVALENT FRAME MODEL: VALIDATION AND EXAMPLES OF APPLICATION

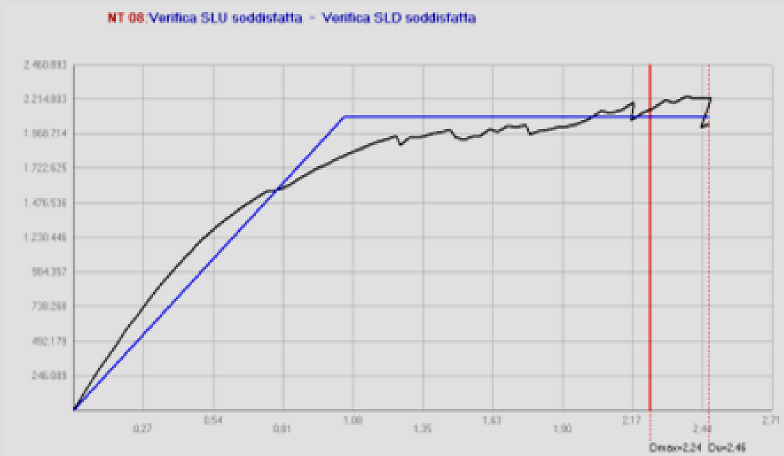


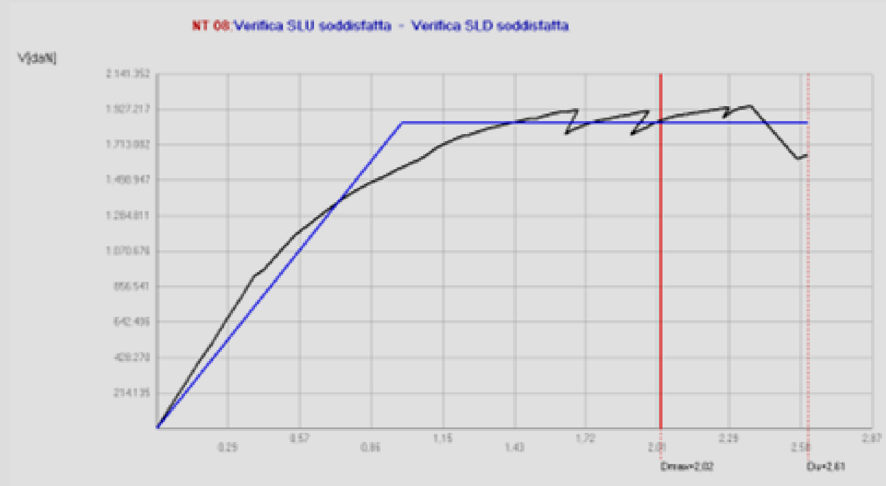
The curve obtained with unreduced elastic parameters is more close to the experimental one. In the case examined, the response is driven by few structural elements: thus, the incorrect assignment of the deformability parameters strongly influences the overall response of the structure.

ARDINGHELLI PALACE, L'AQUILA






NT 08:Verifica SLU soddisfatta - Verifica SLD soddisfatta
2.460.893
2.214.803
1.968.714
1.722.625
1.476.536
1.230.446
984.357
738.268
492.179
246.089
0.27
0.54
0.81
1.08
1.35
1.63
1.90
2.17
2.44
2.71
Dmax=2.24
Du=2.46


NT 08:Verifica SLU soddisfatta - Verifica SLD soddisfatta
V[daN]
2.141.352
1.927.217
1.713.082
1.498.947
1.284.811
1.070.676
856.541
642.406
428.270
214.135
0.29
0.57
0.86
1.15
1.43
1.72
2.01
2.29
2.58
2.87
Dmax=2.02
Du=2.61


Muratura
Integro
Plastico per taglio
Rottura per taglio
Plastico presso flessione
Rottura presso flessione
Rottura per compressione
Rottura per trazione
Rottura in fase elastica