16TH EUROPEAN CONFERENCE ON
EARTHQUAKE ENGINEERING THESSALONIKI
18 - 21 JUNE 2018

SEISMIC ASSESSMENT OF EXISTING MASONRY BUILDINGS: MODELING, ANALYSIS AND VERIFICATION PROCEDURES

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LAYOUT OF THE PRESENTATION

- Introduction
  - Observed vulnerability of existing masonry buildings
  - Issues on models for the seismic assessment
- The revision of the Eurocode 8, Part 3
- Global Response of URM buildings
  - Modelling: Force-Deformation relationships of masonry panels
  - Analysis / Verification: Load Patterns, Limit States, Target Displacement
- Local Mechanisms:
  - Use of Limit Analysis
  - Displacement-Based Assessment
  - Floor Response Spectra

ACKNOWLEDGMENTS

D. Camilletti, S. Marino, S. Degli Abbati, D. Ottonelli and S. Cattari

The observed vulnerability of masonry buildings

- Poor performance of pre-modern masonry buildings due to
    - low quality of masonry (rouble stones with earth mortar)
    - lack of aseismic construction details

Amatrice, *after 30 October 2016*

THE CASE OF NORCIA

- LOCAL SEISMIC CULTURE (in high seismic hazard areas)

	1639	1646	1703	1719	1730	1859	1879	1950	1979
Amatrice	IX	VIII	IX	V/VI	VII/VIII	VII	VI	VII	VI/VII
Arquata del Tronto	VIII/IX	< V	IX	VI	VII/VIII	VII/VIII	VI	VI/VII	VI
Accumoli	VIII/IX	VI/VII	X	VI	VII	VII	VI/VII	VIII	VII
Castelsantangelo sul Nera	VII/VIII	VII	IX/X	VII/VIII	VII	VIII	VI/VII	< V	VI/VII
Norcia	< V	IX	X	VIII	IX	VIII/IX	VIII	VI	VIII

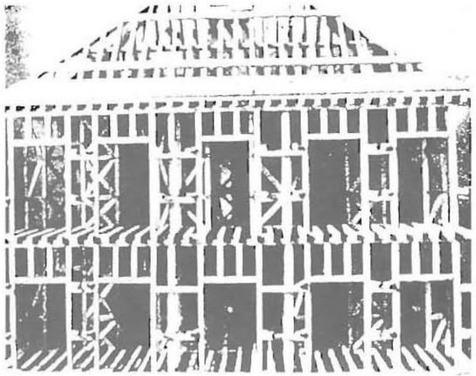
EFFECT OF MODERN «STRENGTHENING» INTERVENTIONS

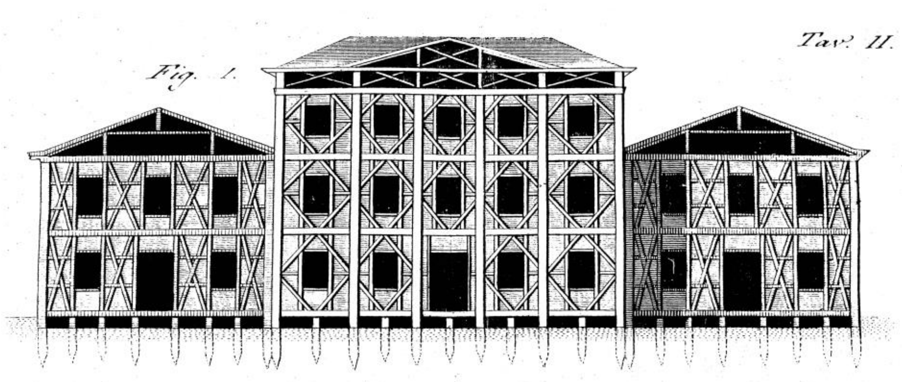
- HEAVY R.C. ROOF: vulnerability increased by inappropriate «modern» interventions

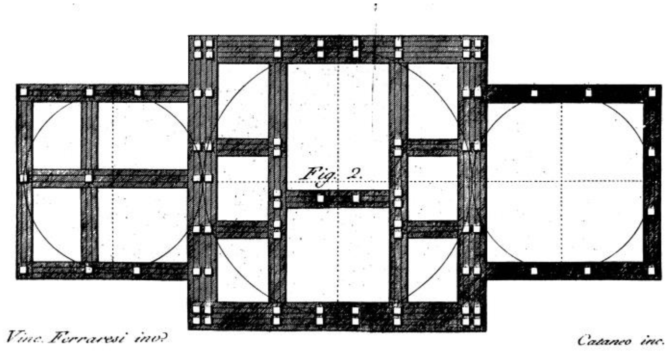
Accumuli, *after 30 October 2016*

ASEISMIC STRUCTURAL SYSTEMS FOR MASONRY BUILDINGS

- TIMBER-FRAMED MASONRY BUILDINGS IN TURKEY AND THE BALKAN AREA
- «CASA BARACCATA» AFTER THE 1624 EARTHQUAKE IN CAMPANIA
- «GAIOLA POMBALINA» FOR THE RECONSTRUCTION OF BAIXA IN LISBON (1755)
- «SEISMIC RESISTANT HOUSE» BY VIVENZIO (1783 MESSINA EARTHQUAKE)







AT THE BEGINNING OF THE XX CENTURY

- 1908 MESSINA EARTHQUAKE – CONFINED MASONRY IN THE 1ST SEISMIC CODE
- 1931 NAPIER EARTHQUAKE, NEW ZEALAND – TIMBER INSTEAD OF MASONRY

WE NEED PROCEDURES FOR THE ASSESSMENT AND RETROFITTING

- MASONRY BUILDINGS ARE A RELEVANT PIECE OF BUILDING STOCK IN EUROPE
- CULTURAL HERITAGE BUILDINGS AND HISTORICAL CENTRE
- GOOD PERFORMANCE OF MODERN MASONRY BUILDINS

  L'Aquila (2009) and Emilia (2012) earthquakes in Italy: very few buildings experienced damage, even in the epicentral area.

- STATISTICAL ANALYSIS OF DAMAGE DATA

  DPC-ReLUIS-EUCENTRE project: Seismic Risk Map of Italy
  Development of fragility curves for different building types from damage data.

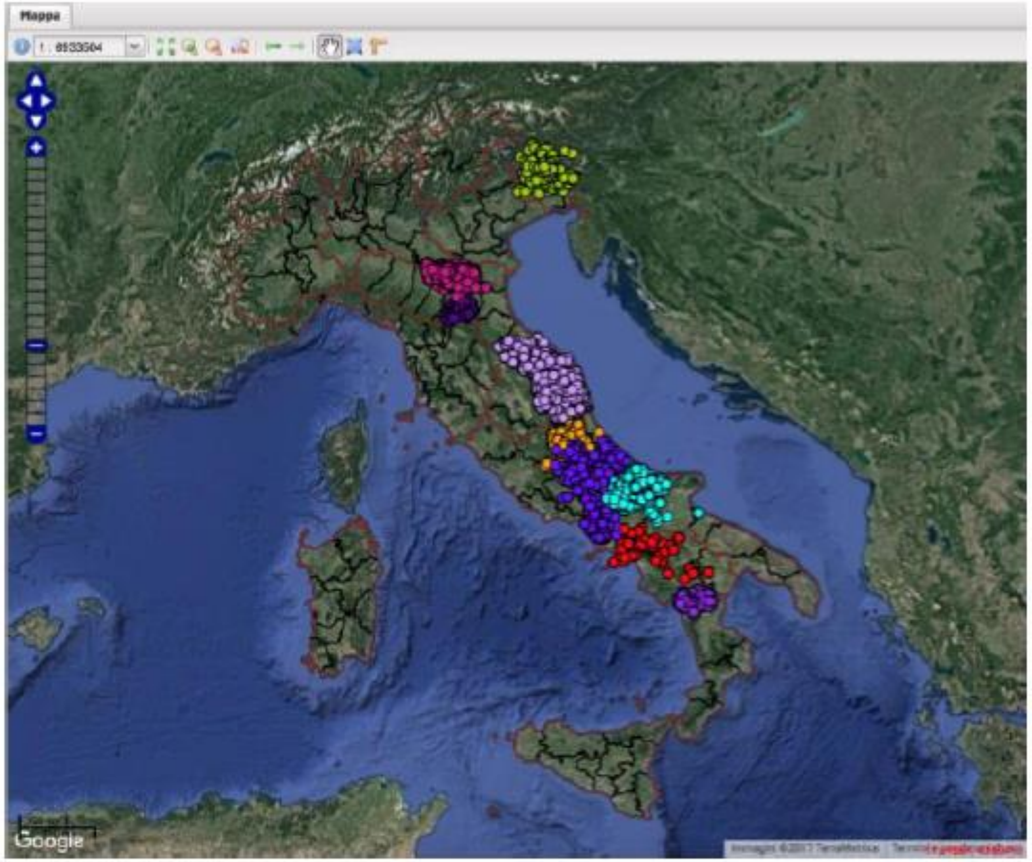
DaDO – A web-GIS Database of Observed Damage

DaDO database: more then 300.000 buildings surveyed after 9 different earthquakes occurred in Italy since the one in Friuli (1976).

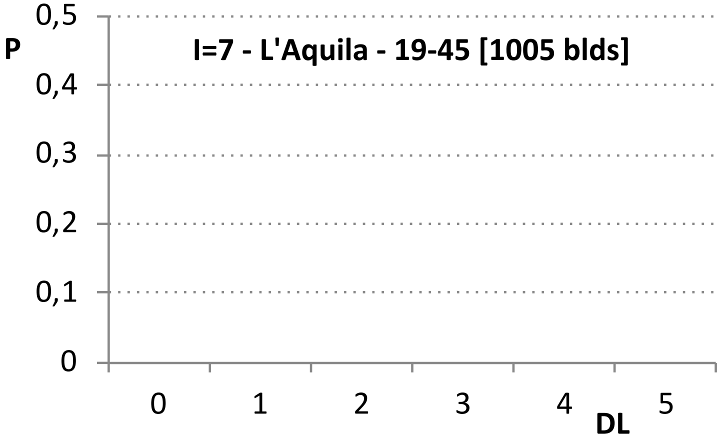
PROTEZIONE CIVILE
Presidenza del Consiglio dei Ministri
Dipartimento della Protezione Civile

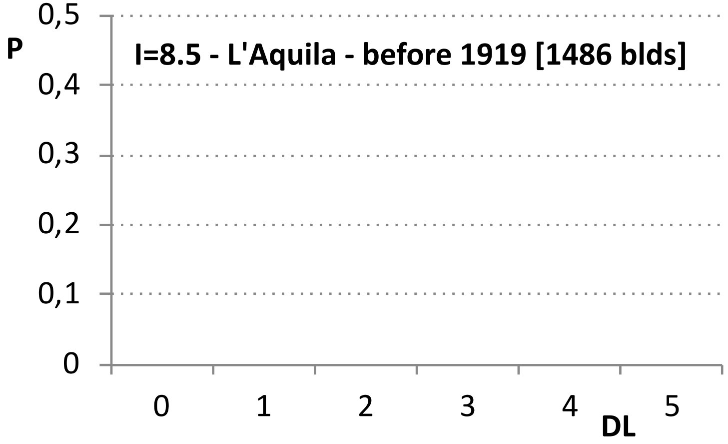
EUCENTRE
FOR YOUR SAFETY.

Evento	Anno	Record	Vers.scheda
Friuli '76	1976	41.852	Friuli '76
Irpinia '80	1980	38.079	Irpinia '80
Abruzzo '84	1984	51.817	Abruzzo '84
Umbria Marche '97	1997	48.525	AeDES 09/97
Pollino '98	1998	17.442	AeDES 06/98
Molise Puglia 2002	2002	24.141	AeDES 05/2000
Emilia 2003	2003	1011	AeDES 05/2000
L'aquila 2009	2009	74.049	AeDES 06/2008
Emilia 2012	2012	22.554	AeDES 06/2008
Totale		**319.470**	



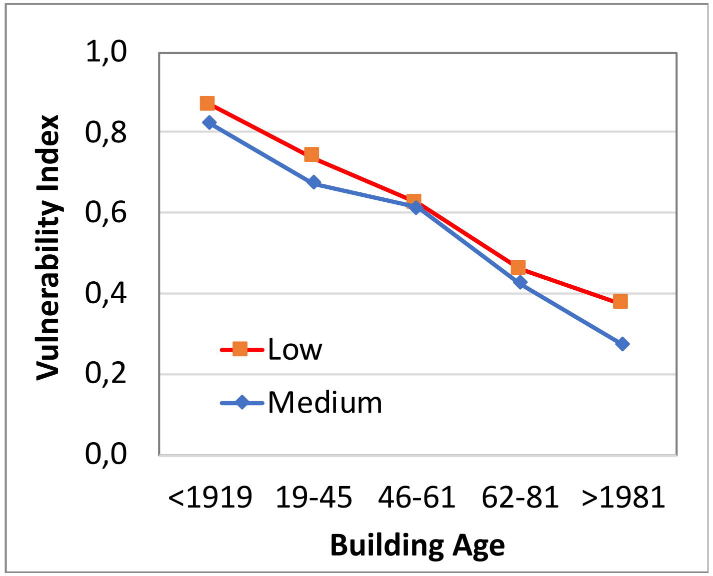
- Ottonelli D. – Validation of the macroseismic vulnerability model (Lagomarsino and Giovinazzi, 2006) and derivation of fragility curves for masonry buildings

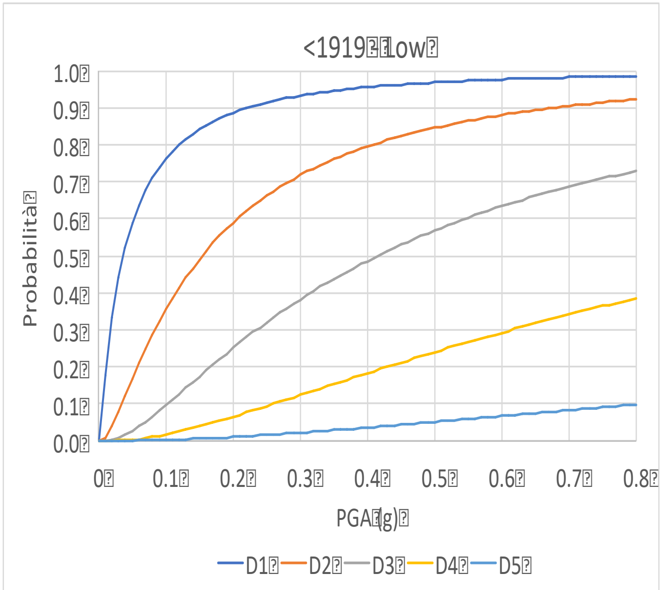


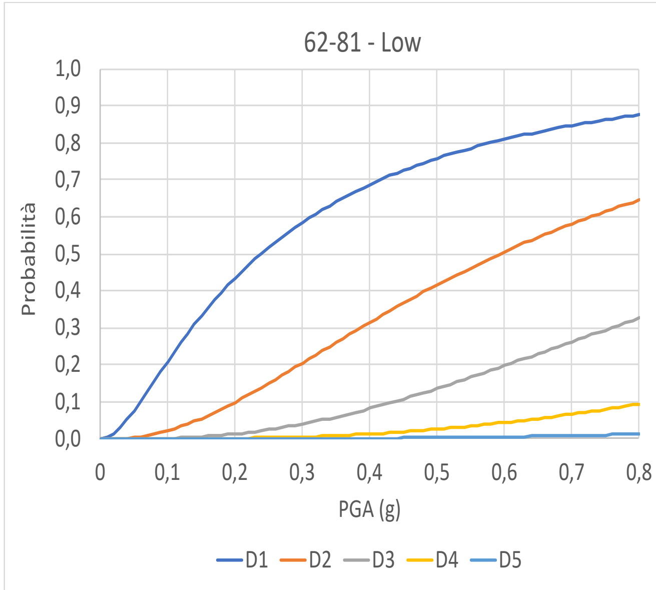


Mean Damage Grade, function of the intensity I, given the Vulnerability Index V

$$\mu_D = 2.5\left[1+\tanh\left(\frac{I+6.25V-13.1}{Q}\right)\right]$$







SEISMIC ASSESSMENT VS SEISMIC DESIGN

- DESIGN

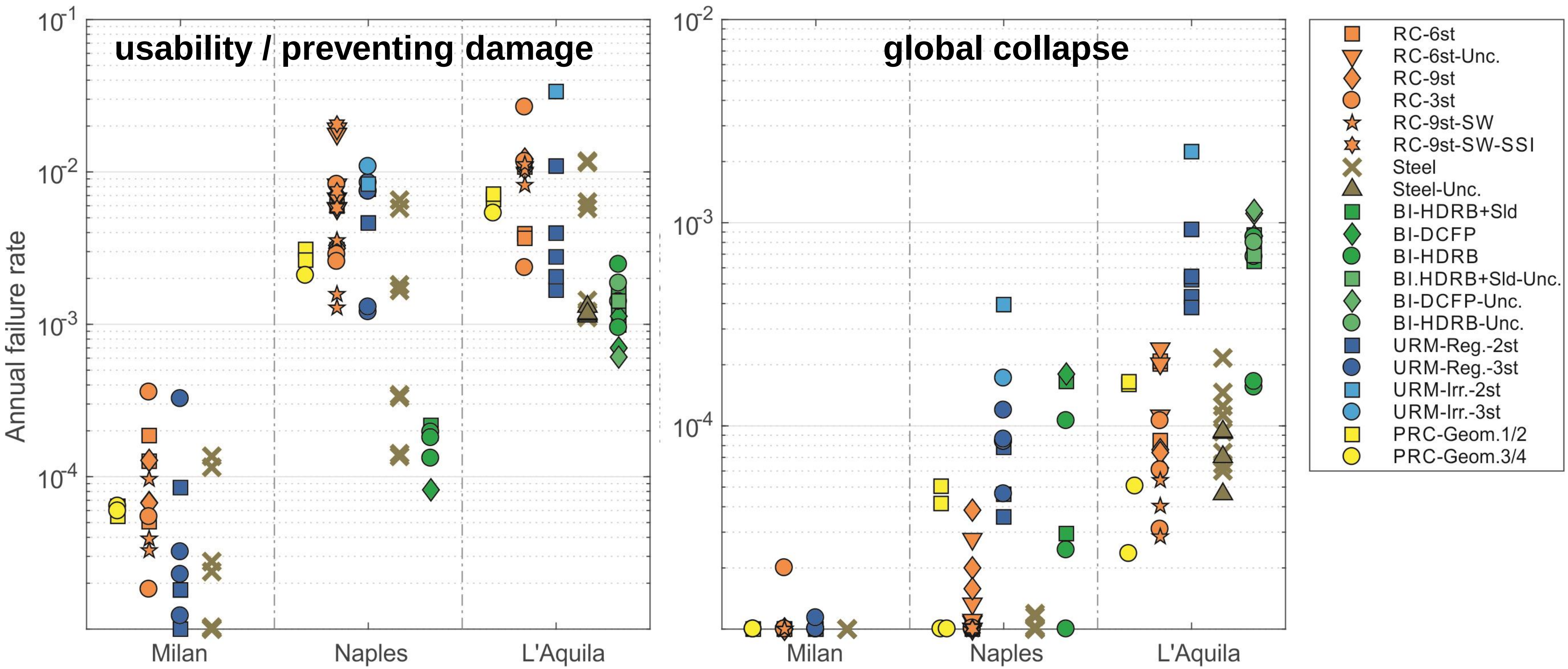
  I conceive the structure by a capacity design and use details that guarantee the assumed ductility level. I don't need nonlinear models to do that.

- ASSESSMENT

  I evaluate the actual performance of the building by using a model as close as possible to the real behaviour. Nonlinear models are necessary because they don't assume a predefined capacity. Linear model makes assumptions, usually largely cautionary.

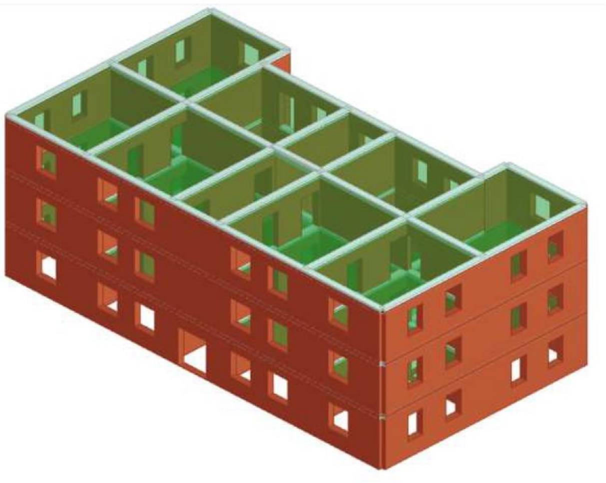
	Methods	Static	Dynamic
DESIGN (strength)	Linear	Equivalent forces	Modal analysis
ASSESSMENT (deform.)	Nonlinear	Pushover analysis	Time-history analysis

➢ Annual failure rates for different structural typologies and sites (soil C)

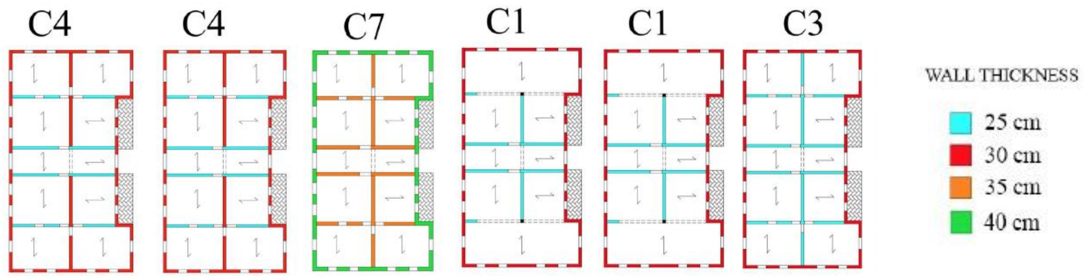


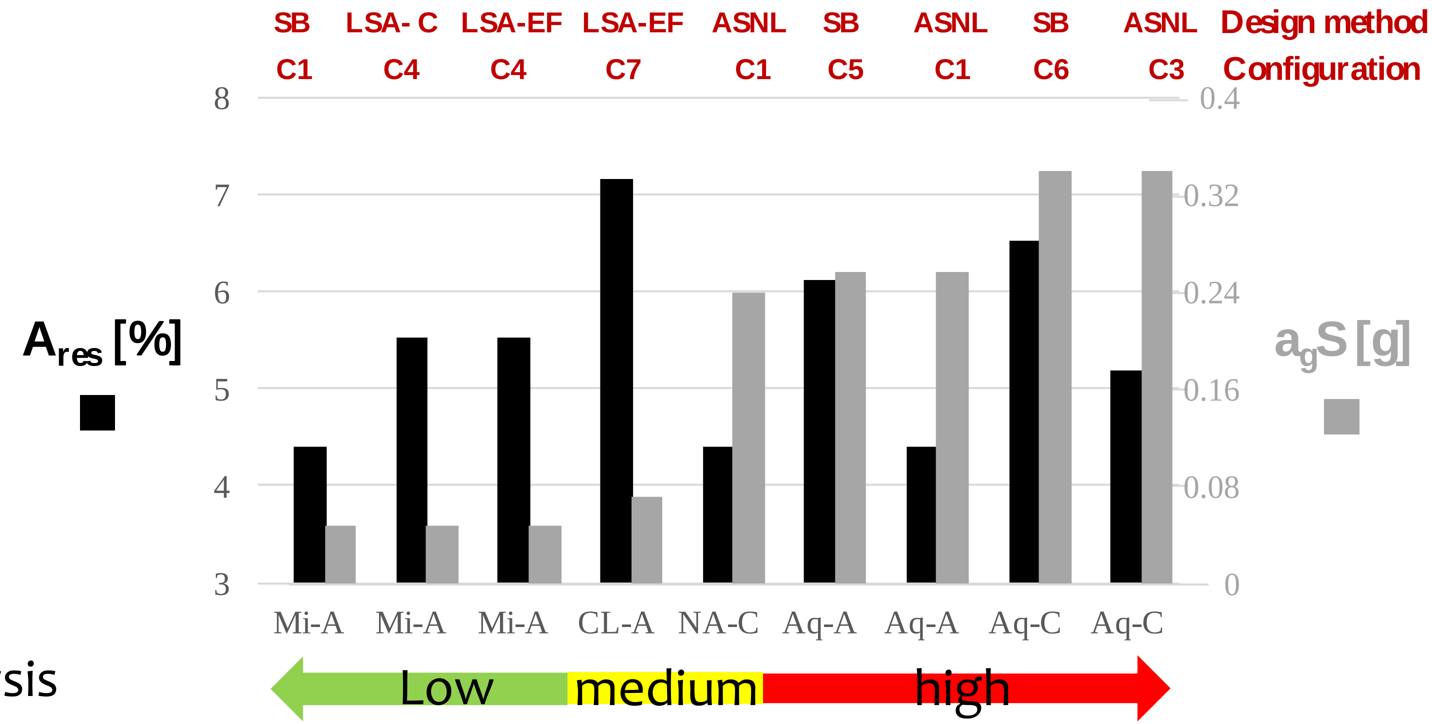
RINTC Workgroup (2018) Results of the 2015-2017 Implicit seismic risk of code conforming structures in Italy (RINTC) project. Coordinator: Iunio Iervolino. ReLUIS report, available at http://www.reluis.it/

Architectural configuration



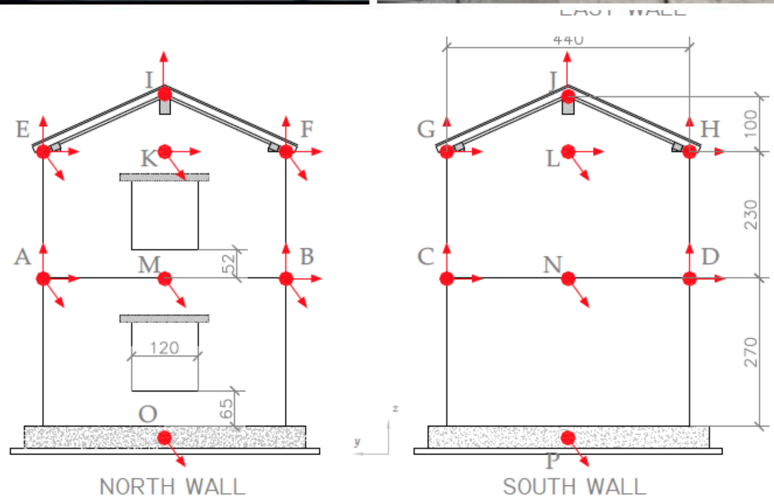
Structural layout of walls that comply with code rules according to different analysis methods

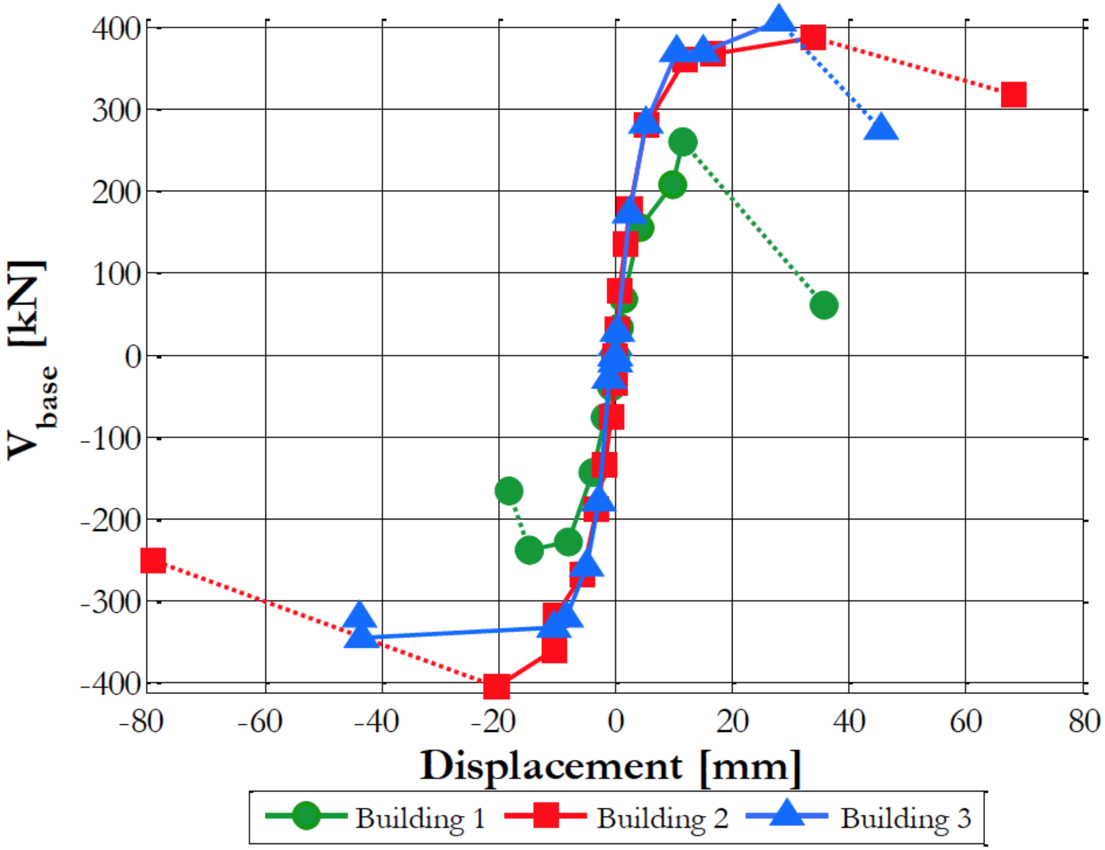




SB = Simple Building Rules
LSA = Linear Static Analysis
ASNL = Nonlinear Static Analysis

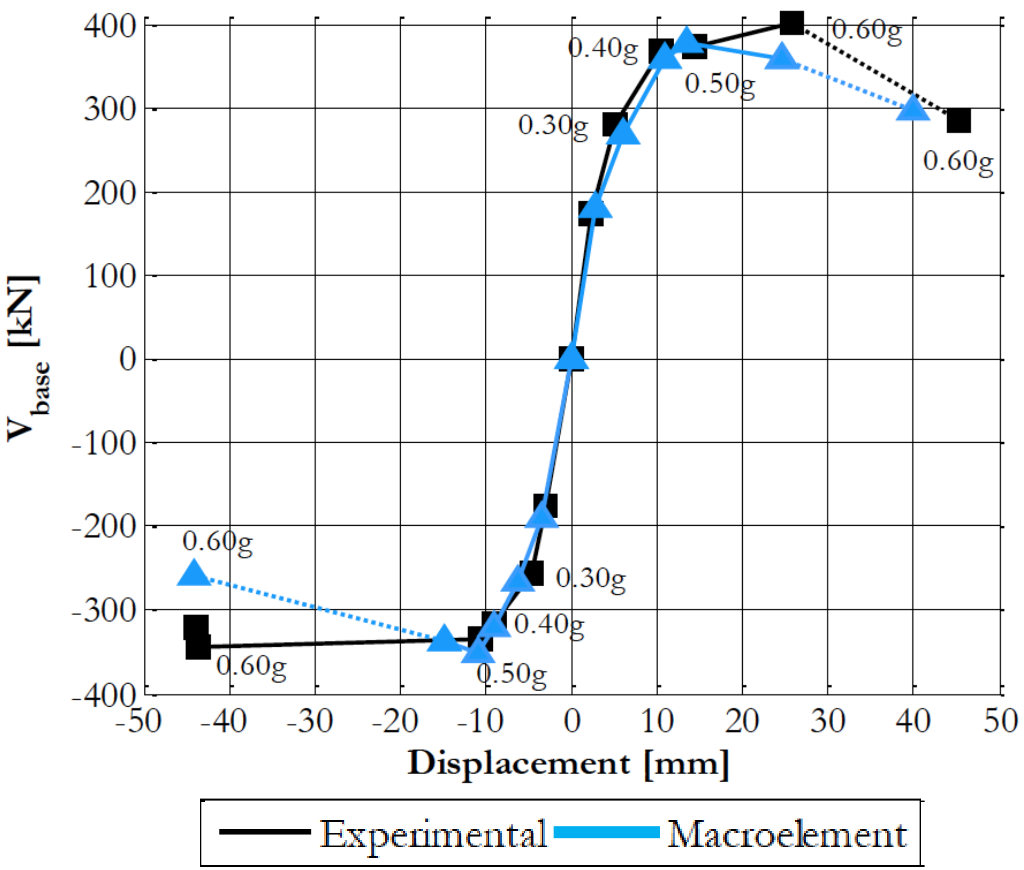
VALIDATION OF MODELS – SHAKING TABLE TEST

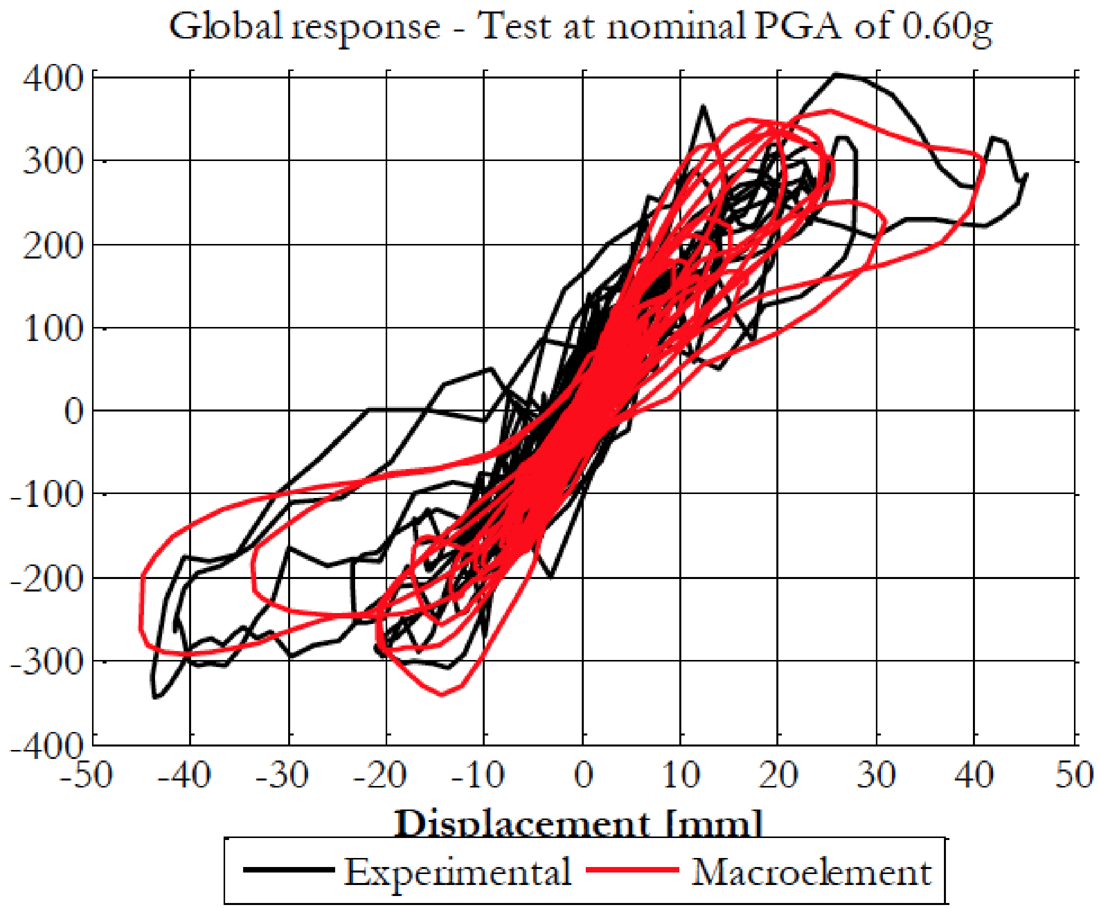




Magenes G, Penna A, Senaldi IE, Rota M, Galasco A, (2015) Shaking table test of a strengthened full-scale stone masonry building with flexible diaphragms, Int. Journal of Architectural Heritage, 10(2-3)

➤ Numerical simulation of Building 3 (rigid diaphragms)



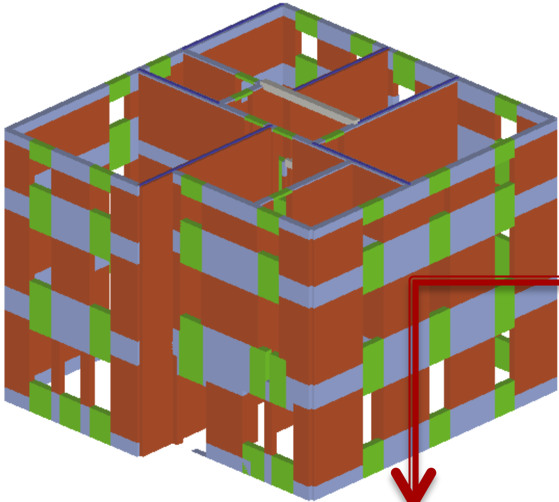


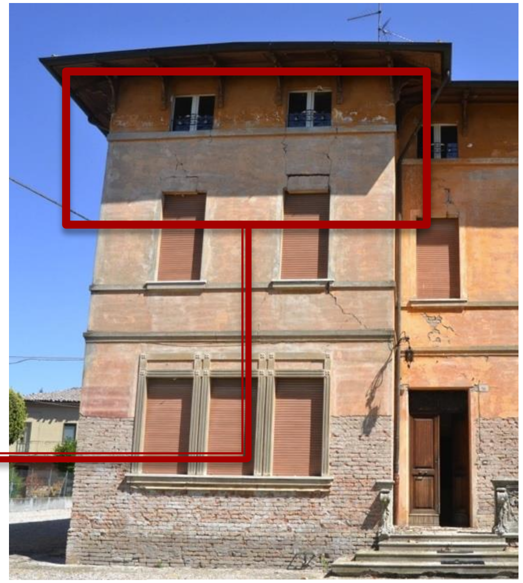
Penna A, Senaldi IE, Galasco A, Magenes G (2015) Numerical simulation of shaking table tests on full scale stone masonry buildings, International Journal of Architectural Heritage, 10 (2-3)

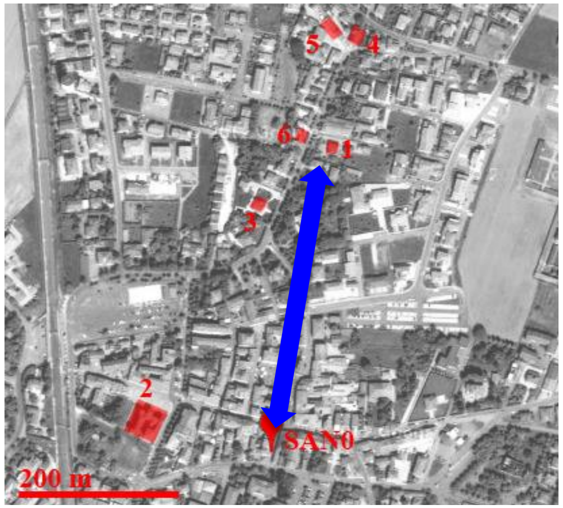
VALIDATION OF MODELS – REAL DAMAGED BUILDINGS

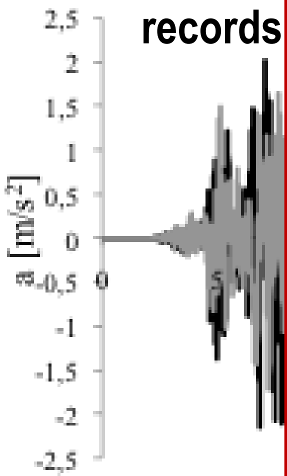
- San Felice sul Panaro, Emilia earthquake 2012

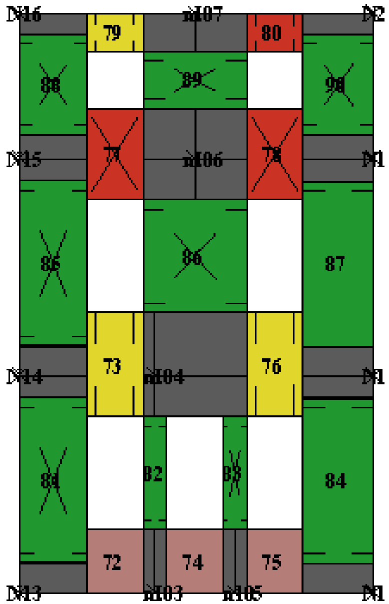
Equivalent Frame Model with Tremuri

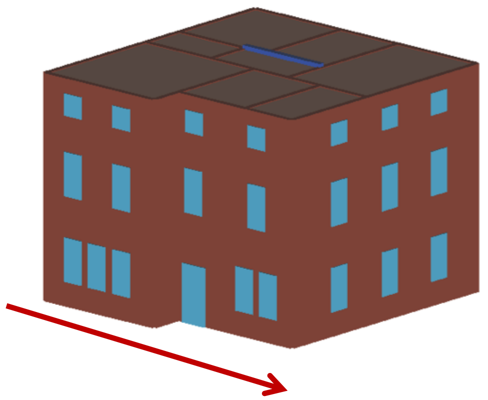


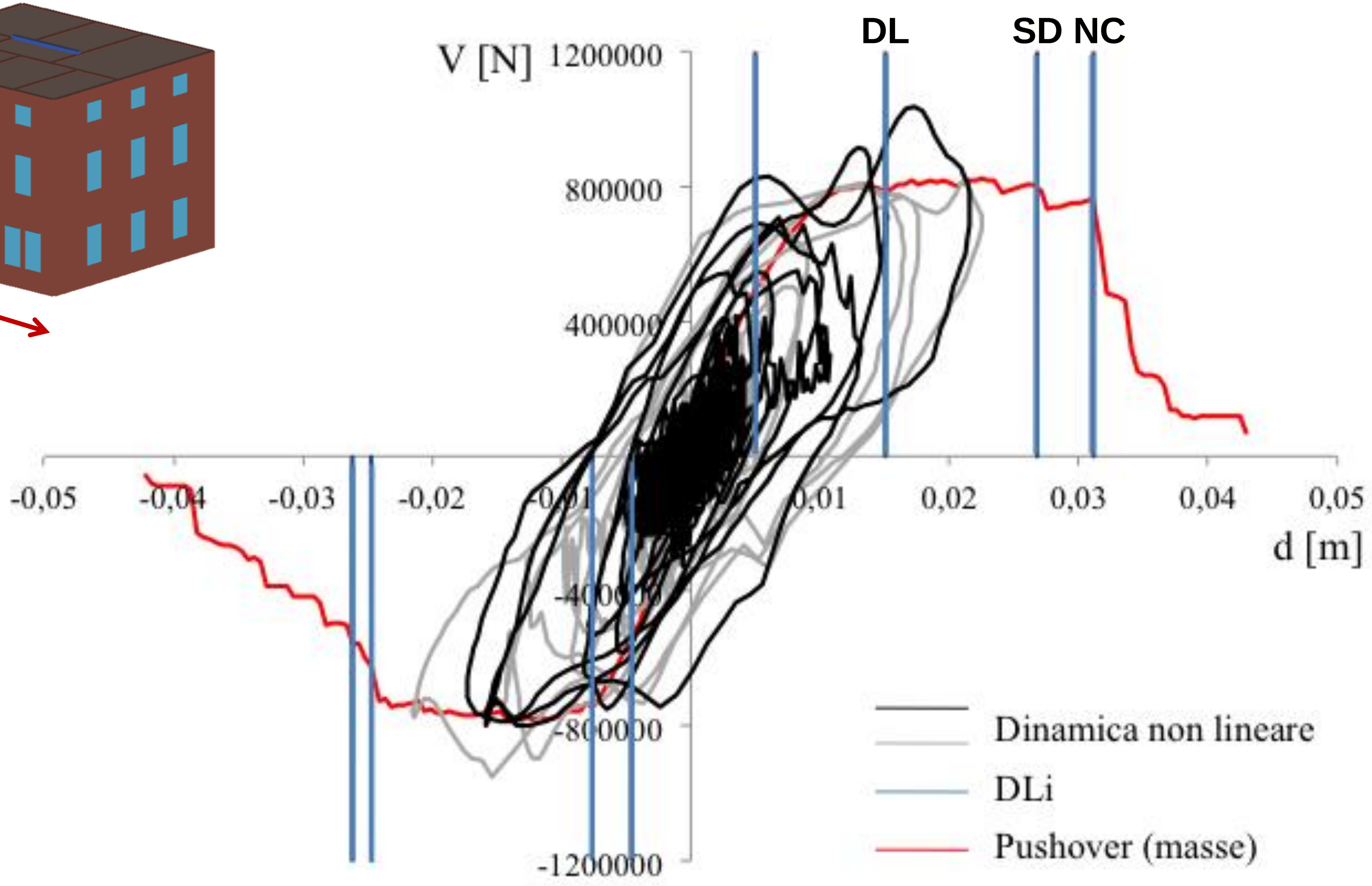












Damage from nonlinear dynamic analysis is between DL2 (Damage Limitation – Immediate Occupancy) and DL3 (Significant Damage – Life Safety)

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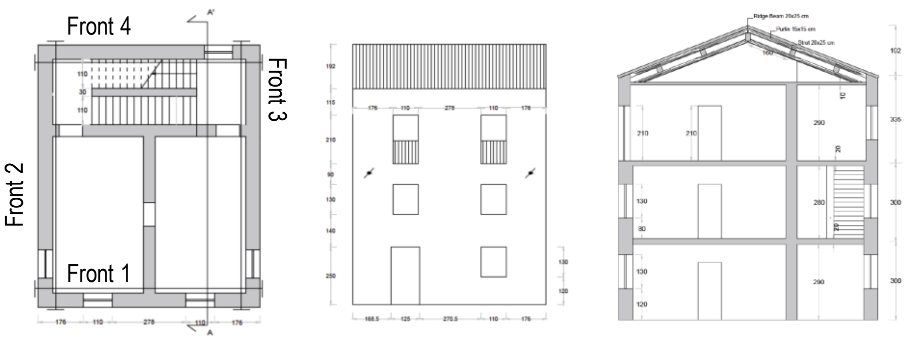
SpSe 18. Seismic modelling of masonry buildings: present knowledge and open challenges for research and practice

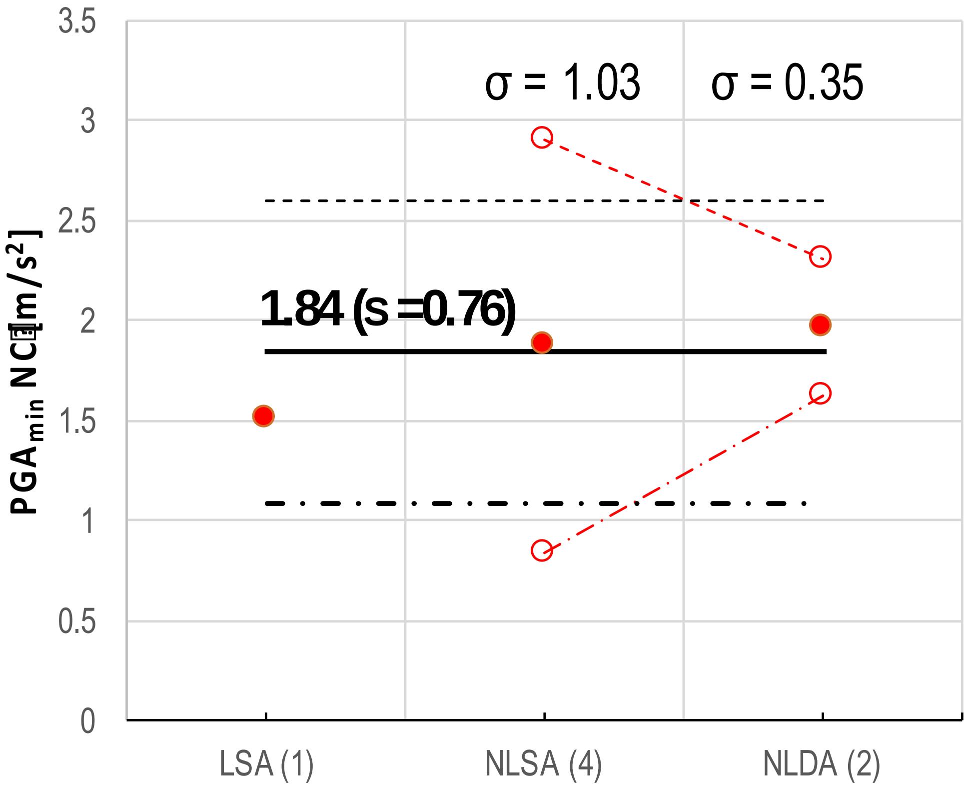
Organizers: S. Cattari, P.B. Lourenco, G. Magenes

Numerical predictions of two case-studies

GEOMETRY



CASE A	CASE B
• Stone masonry • Flexible diaphragms • Tie rods at the first floor • Timber roof	• Brick masonry • Rigid diaphragms • RC beams • Timber roof

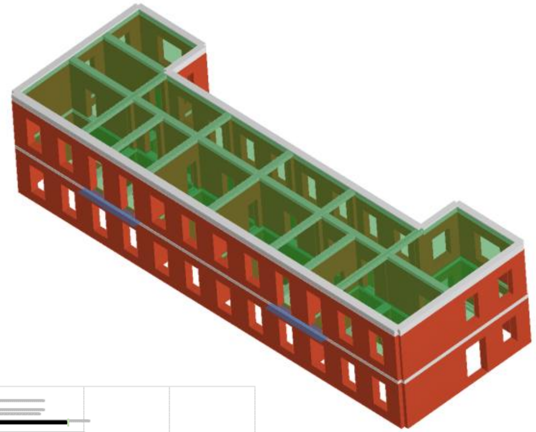


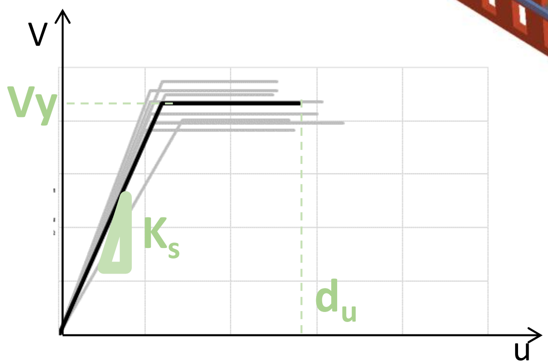
Participants from:

ÉCOLE POLYTECHNIQUE FÉDÉRALE DE LAUSANNE
TU Delft, Delft University of Technology
Ud'A Università degli Studi "G. d'Annunzio"
METU
BOĞAZIÇI ÜNIVERSITESI 1863
EDİTEPE UNIVERSITY
SICILIAE STVDIVM GENERALE 1434

TASK 4.3 - Seismic Assessment of Benchmark Case studies
ReLUIS-DPC Project 2014 -2018 - Masonry Structures

THE PIZZOLI CITY HALL Dispersion of predictions			
CONFIGURATION	V_y	K	D_u
A - weak spandrel	0.18	0.15	0.24
B - tie rods	0.09	0.10	0.15
C - r.c. tie beam	0.11	0.09	0.16





Blind test: G. Magenes, G.M. Calvi, G.R. Kingsley (1995) Seismic Testing of a Full-Scale, Two-Story Masonry Building. Report Univ. of Pavia

Eurocode 8 – Design of structures for earthquake resistance
Part 3: Assessment and retrofitting of buildings and bridges

- Project Team 3: Andreas Kappos (leader), Christis Chrisostomous, Paolo Franchin, Tatjana Isakovic, Sergio Lagomarsino, Telemakos Panagiotakos

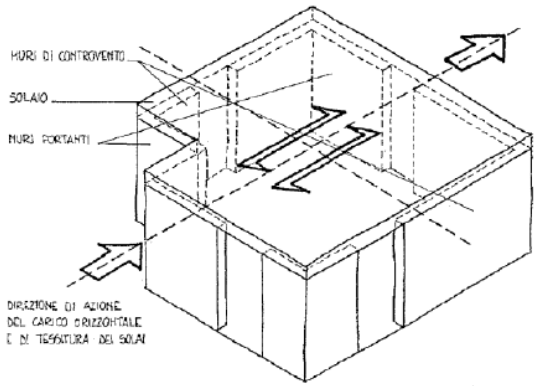
Mandate by CEN TC250/SC8 (chairman : Philippe Bisch)

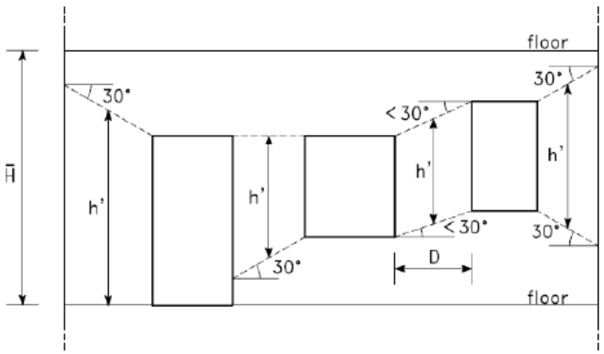
- Update modelling, analysis and verification procedures taking into account the recent advances of knowledge from the research.
- Promote the use of nonlinear methods for the seismic assessment.

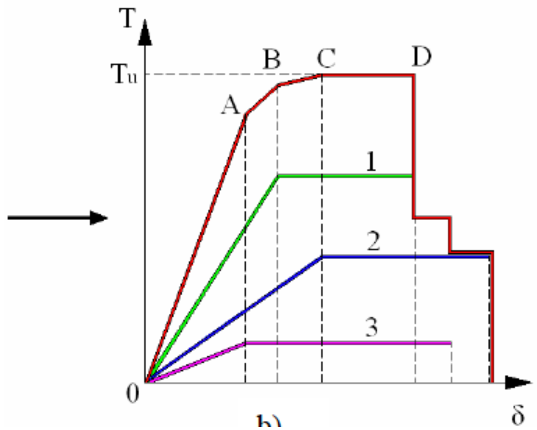
	Methods	Static	Dynamic
	Linear	Equivalent forces	Modal analysis
ASSESSMENT (deform.)	Nonlinear	Pushover analysis	Time-history analysis
			REFERENCE

- POR METHOD (Tomazevic 1978)

  NonLinear Static Analysis (NLSA) is used in Italy since 1981 (code for the reconstruction after the Irpinia earthquake, 1980). The shear behaviour of masonry panels is assumed bilinear with limited ductility. Only piers were considered (strong spandrels). Incremental analysis until reaching the maximum base shear. Verification in terms of strength.



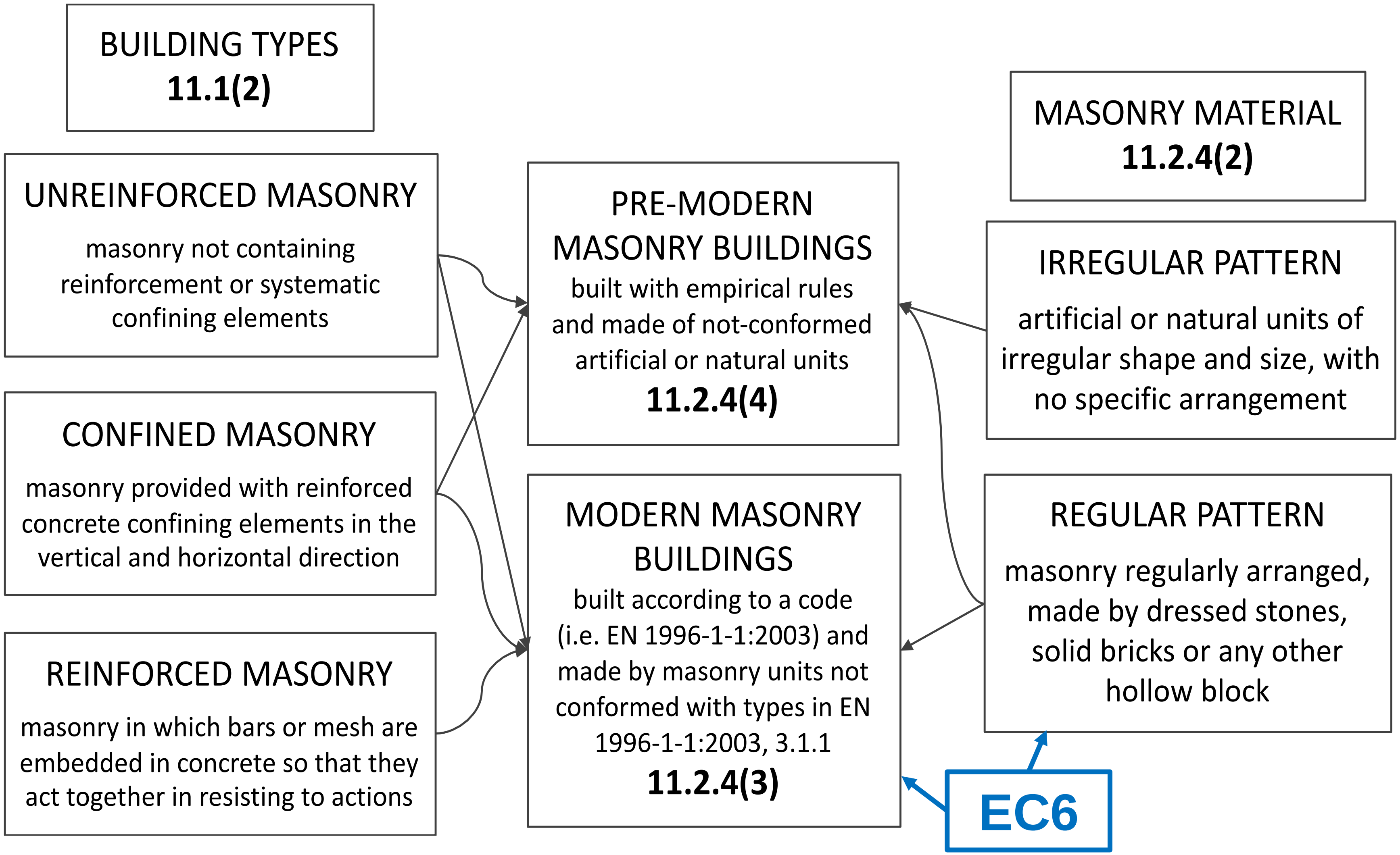




- Current version of Eurocode 8 – Part 3

  Equivalent Frame Model (if spandrels are considered). Bilinear force-deformation model with limited drift ratio. Pushover analysis, modelling strength degradation. Verification in terms of displacement (N2 method) with identification of Near Collapse Limit State by 20% strength decay.

- A CODE SHOULD COVER THE GREAT MAJORITY OF BUILDINGS (80%)



- EC8-Part3 CONSIDERS STRUCTURES MADE BY MASONRY WALLS

Bidimensional elements: the thickness is small compared to the other two sizes of the wall. Two main independent behaviours:

- in-plane ⟹ strength/drift (material nonlinearity)
- out-of-plane ⟹ shape (geometric nonlinearity)

- GLOBAL ANALYSIS

The building is made by a set of walls, in different directions, connected at the intersections and through horizontal diaphragms.

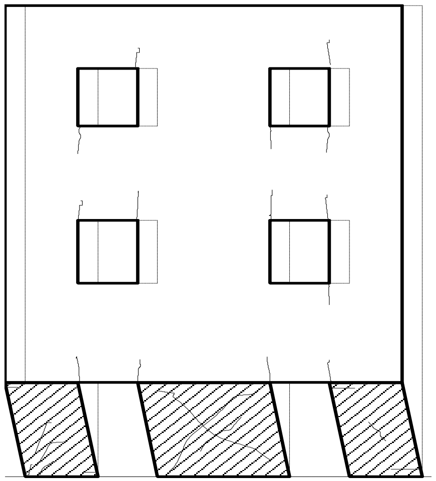
⟹ seismic actions are supported by the in-plane behaviour (the contribution of out-of-plane is neglected)

- PREVENTION OF LOCAL MECHANISMS

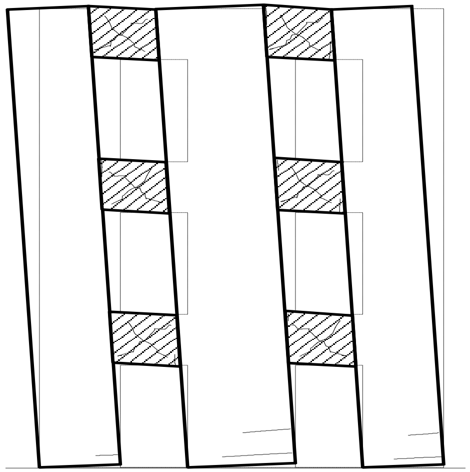
Wall portions should not fail out-of-plane.

⟹ need of local verifications, because this behaviour is not considered in the global in-plane model.

- MODELLING OF MASONRY WALLS
  - damage observation ⟹ piers and spandrels

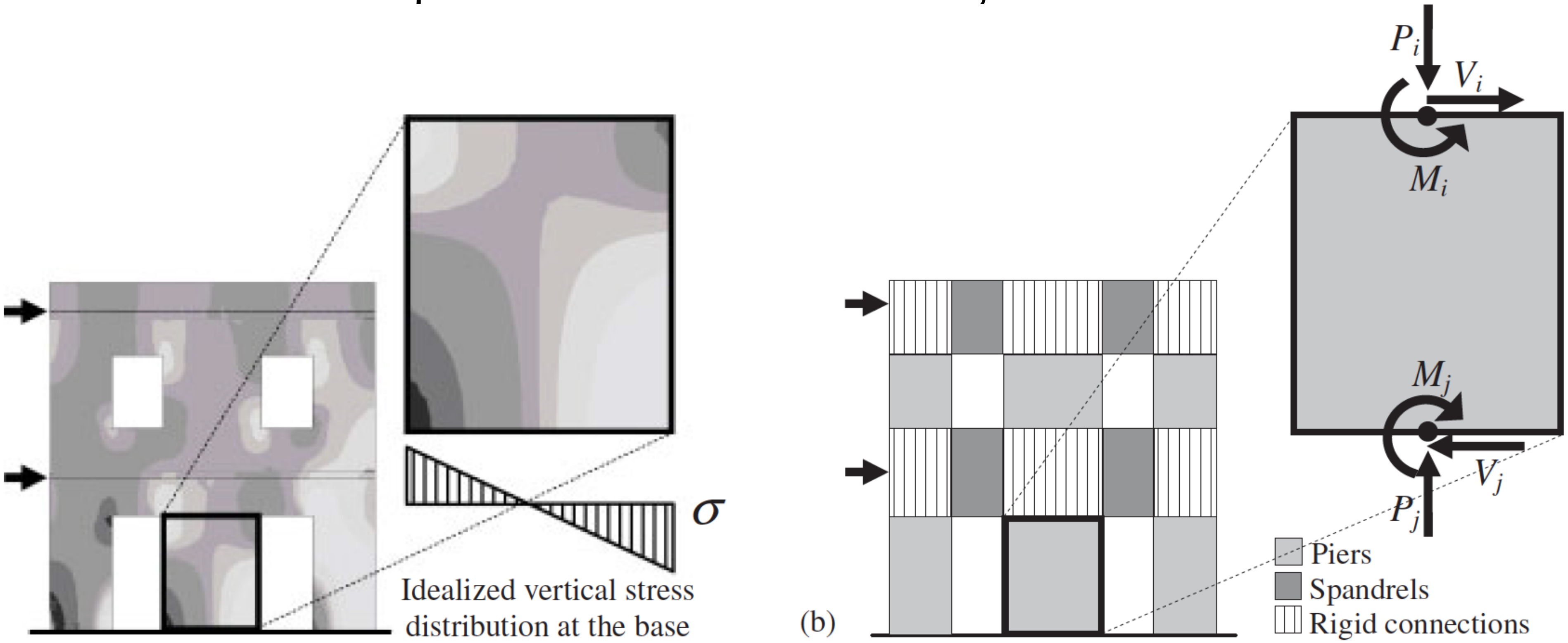


Strong Spandrels
Weak Piers



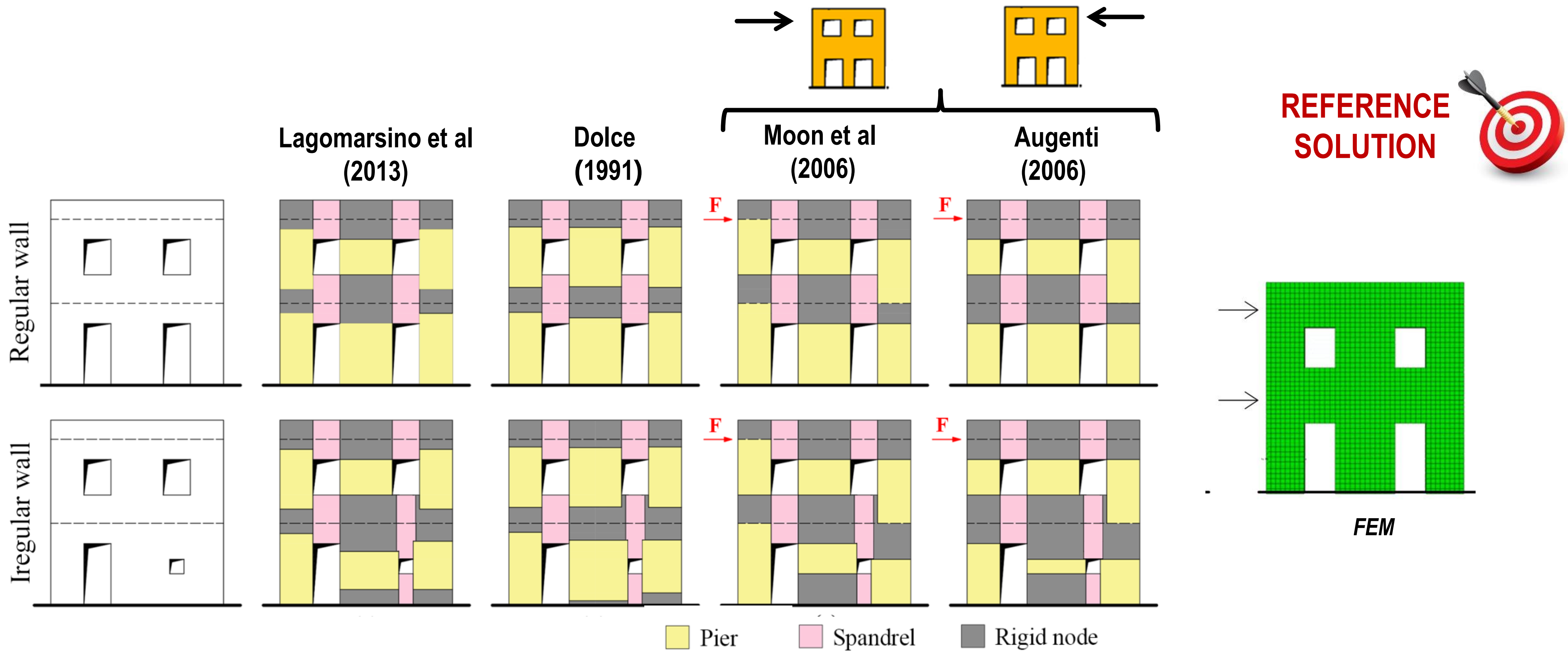
Strong Piers
Weak Spandrels

- MODELLING OF MASONRY WALLS
  - damage observation ⟹ piers and spandrels
  - Continuous Finite Element Model Vs. Equivalent Frame Model
    - ❑ F.E.M. – identification ex-post of sections for strength verification or of panels for checking drift limits
    - ❑ E.F.M. – a-priori definition of masonry elements



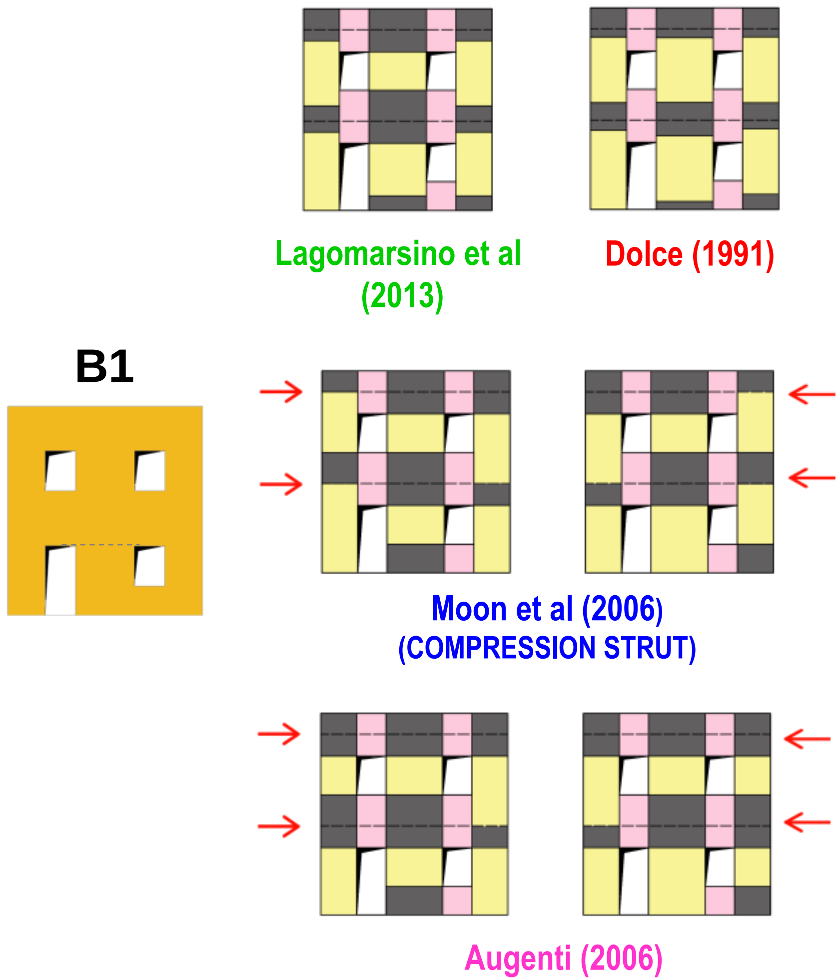
➢ IDENTIFICATION OF THE E.F.M. IN THE CASE OF IRREGULAR WALLS

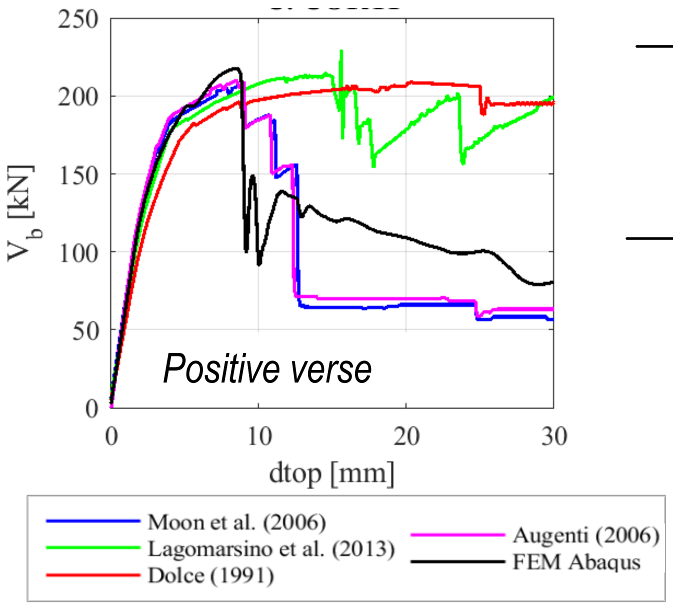
Several criteria are proposed in the literture.

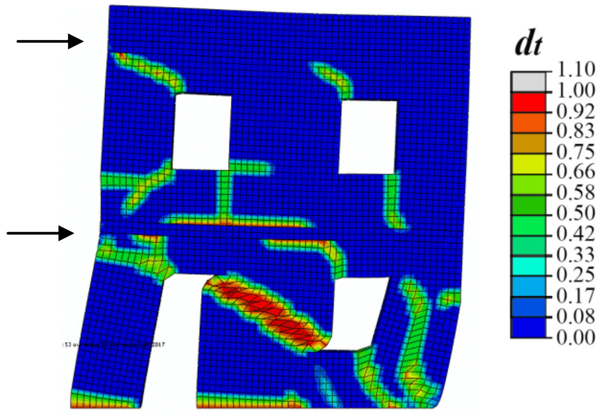


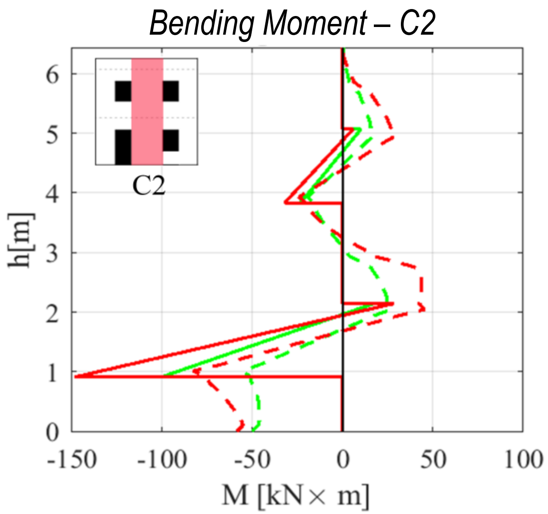
Camilletti et al. (2018) "In Plane Seismic Response of Irregular URM Walls through Equivalent Frame and Finite Element Models", Proc. of 16ECEE, paper ID 11593

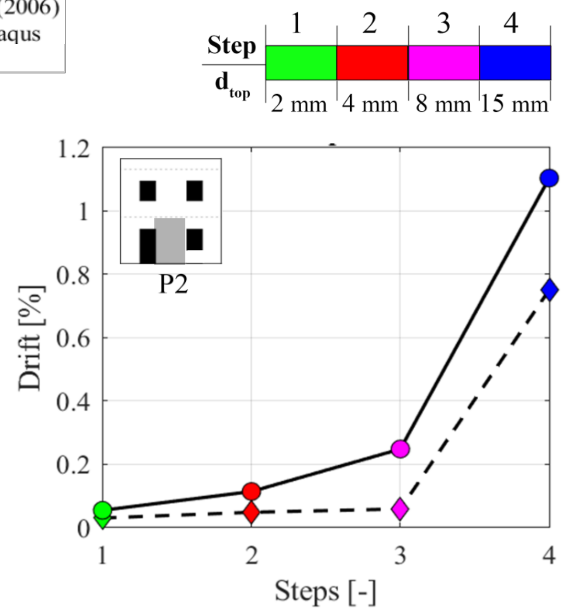
➢ IDENTIFICATION OF THE E.F.M. IN THE CASE OF IRREGULAR WALLS







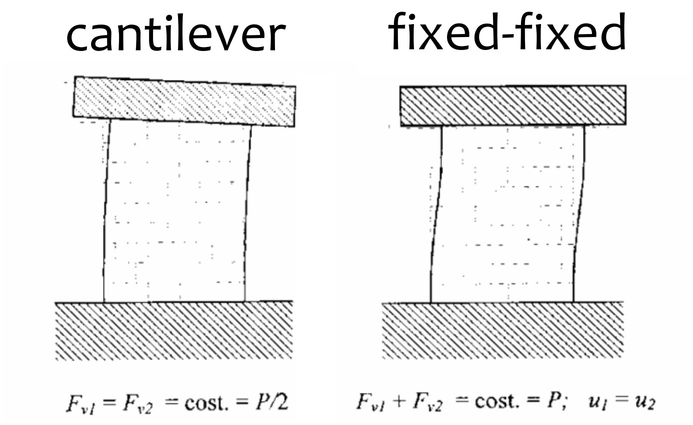




Strength criteria for piers

Based on experimental tests **from more than 50 years**

- ❑ Turnsek and Cacovic, 1970
- ❑ Mann and Muller, 1980
- ❑ …..



Strength criteria for spandrels

Evidences from experimental campaigns **from less than 20 years :**
Gattesco et al. 2008, Beyer and Dazio 2012, Graziotti et al. 2012, Knox 2012, Parisi et al. 2014 , …

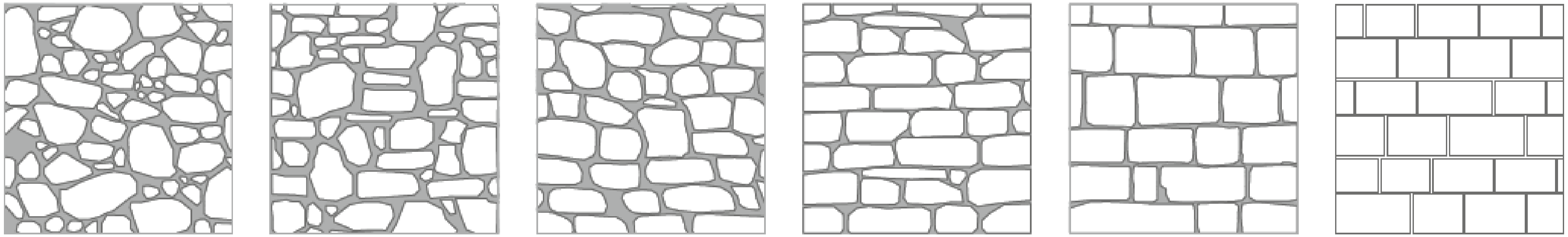
- ❑ Cattari and Lagomarsino, 2008
- ❑ Beyer, 2012
- ❑ Beyer and Mangalathu, 2013
- ❑ …

Drift limits for piers - STONE MASONRY

Vanin, Zaganelli, Penna, Beyer (2017) *Estimates for the stiffness, strength and drift capacity of stone masonry walls based on 123 quasi-static cyclic tests reported in the literature*, BEE

MASONRY TYPOLOGIES

A | B | C | D | E | E1



DISPLACEMENT CAPACITY

Drift at cracking: $\delta_{cr} = 0.20\%$

Yielding drift:
- shear $\delta_y = 1/4 \cdot \delta_u$
- flexure $\delta_y = 1/6.5 \cdot \delta_u$

Drift at SD limit state $\delta_{SD} = 0.50 \cdot \delta_u$

Drift at max. force: $\delta_{max} = 0.70 \cdot \delta_u$

Drift at collapse $\delta_c = 1.15 \cdot \delta_u$

	A-B-C-D		E-E1	
	Shear failure	Flexural failure	Shear failure	Flexural failure
Model 1: δ_u	0.60	0.90	1.50	2.25
CoV	0.50	0.50	0.50	0.50

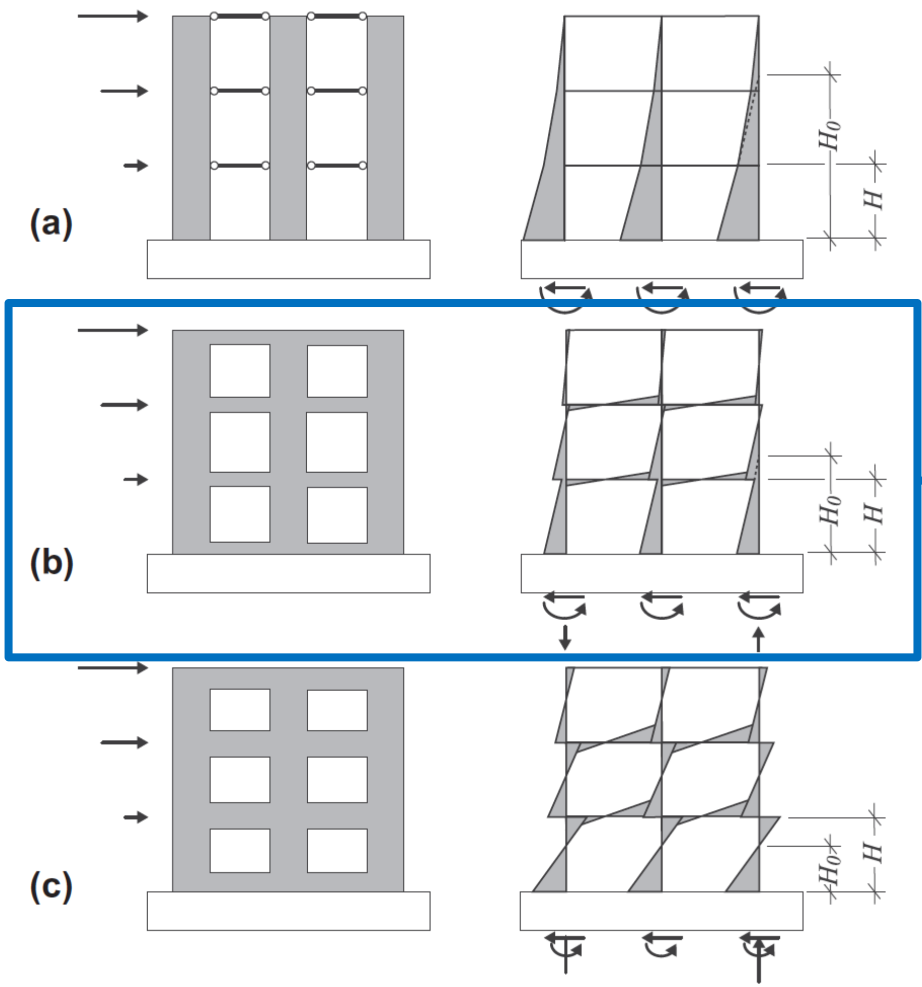
Ultimate drift:
- Model 1: reference values from table
- Model 2:

$$\delta_u = \max\left(1.5\% - 4\% \cdot \frac{\sigma_{0,tot}}{f_c}, 0.3\%\right) \cdot \frac{H_0}{\min(H,L)} \quad \text{(typologies A-B-C-D)}$$

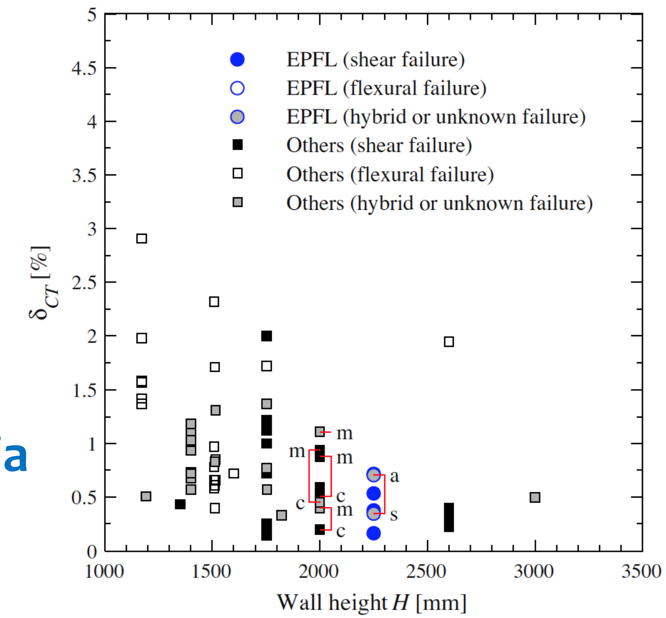
$$\delta_u = \max\left(2.25\% - 6\% \cdot \frac{\sigma_{0,tot}}{f_c}, 0.3\%\right) \cdot \frac{H_0}{\min(H,L)} \quad \text{(typologies E-E1)}$$

Drift limits for piers - BRICK MASONRY

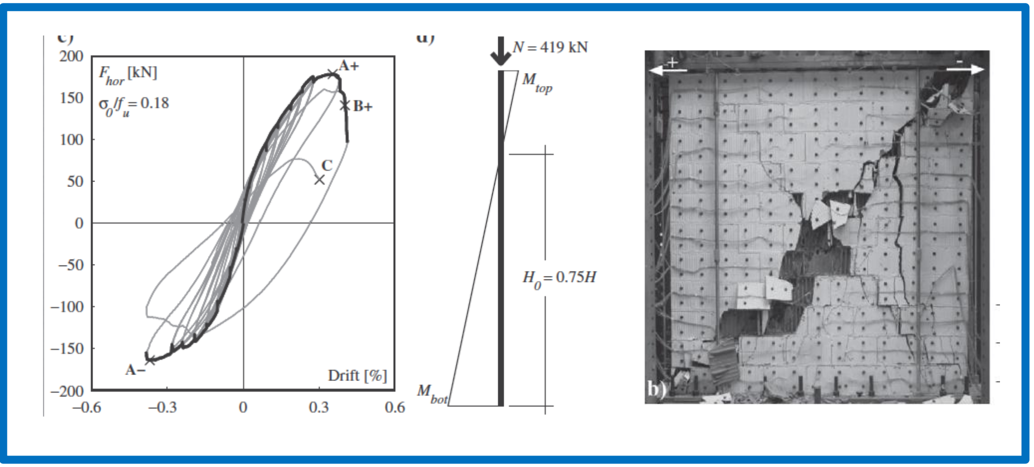
Petry S, Beyer K (2014) Influence of boundary conditions and size effect on the drift capacity of URM walls, Engineering Structures



- ❑ **Correction factor *b* shear failure criteria**
- ❑ **Representative drift measure**



influence of boundary condition

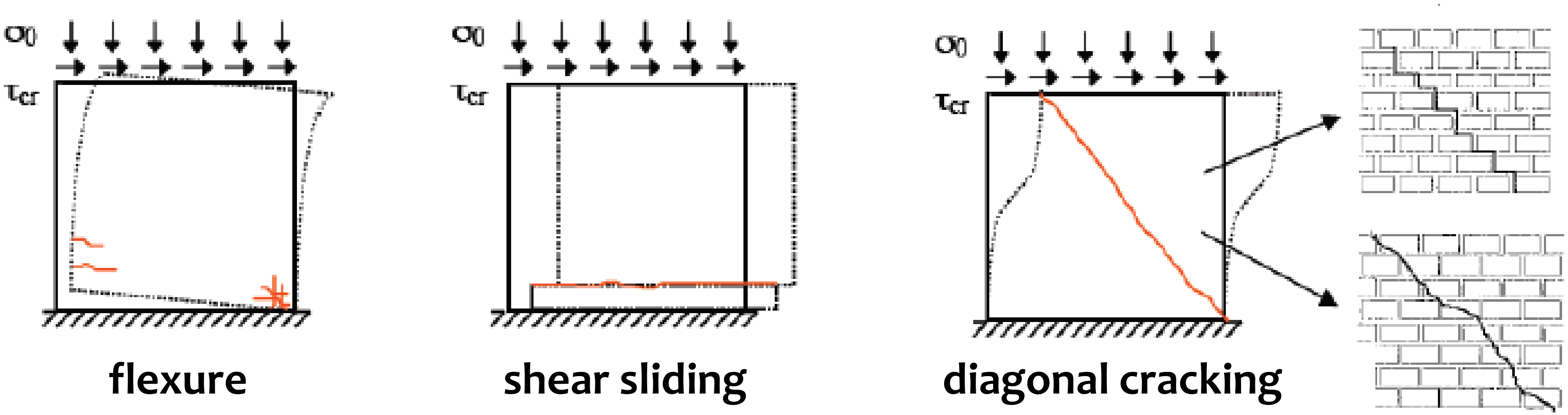


- FORCE-DEFORMATION RELATIONSHIPS (in terms of generalized force *V* and deformation θ), DEPENDS ON STIFFNESS, FAILURE CRITERIA & DRIFT LIMITS
  - a classification is proposed

3 failure criteria:
- Flexure
- Shear sliding
- Diagonal cracking

2 masonry types:
- Regular (horizontal layers and stair-stepped joints)
- Irregular (isotropic behaviour)

2 masonry elements:
- Piers
- Spandrels



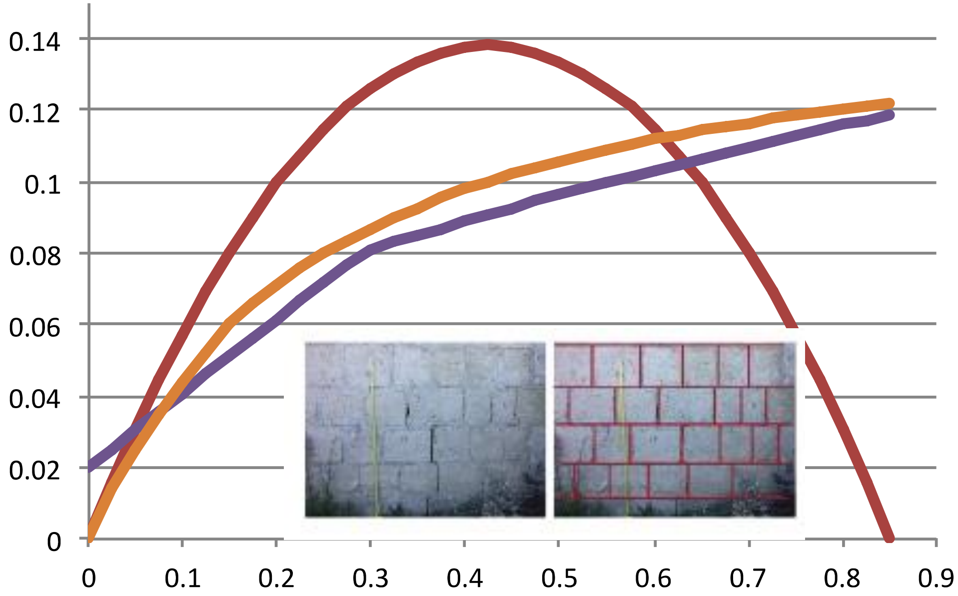
- FAILURE CRITERIA FOR PIERS

	Flexure	Shear Sliding	Diagonal Cracking
REGULAR	$V_f = \frac{D N}{2H_0}\left(1 - 1{,}15 n_d\right)$	*The one proposed in the current version of EC6* $V_s = D' t (f_{v0} + 0{,}4N/D't)$ with $(f_{v0} + 0{,}4N/D't) < 0.065/0.7\ f_b$	$V_d = \frac{D t}{b}\left(\frac{f_{v0}}{1+\mu\phi} + \frac{\mu}{1+\mu\phi}\sigma_0\right) \le V_{d,lim}$ $V_{d,\lim} = \frac{D t}{b}\frac{f_{bt}}{2.3}\sqrt{1+\frac{\sigma_0}{f_{bt}}}$
IRREGULAR		NOT CONSIDERED	$V_d = \frac{D t}{b} f_t \sqrt{1+\frac{\sigma_0}{f_t}}$

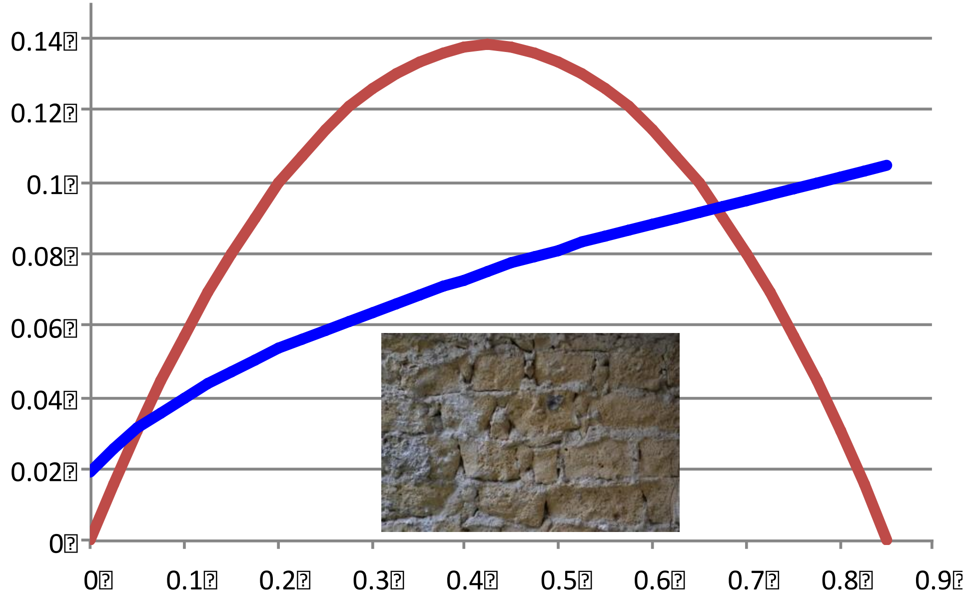
- FAILURE CRITERIA FOR PIERS

V–N domains
(soft stone – squat panel – fixed/fixed condition)

REGULAR



IRREGULAR



$$V_f = \frac{D N}{2H_0}\left(1 - 1{,}15 n_d\right)$$

$$V_d = \frac{D t}{b}\left(\frac{f_{v0}}{1+\mu\phi} + \frac{\mu}{1+\mu\phi}\sigma_0\right) \le V_{d,lim}$$

$$V_s = D' t\left(f_{v0} + 0{,}4 N / D' t\right)$$

$$V_d = \frac{D t}{b} f_t \sqrt{1 + \frac{\sigma_0}{f_t}}$$

MODELLING OF MASONRY PANELS

➢ FAILURE CRITERIA FOR SPANDRELS (no shear sliding, axial force is neglected)

Flexure

If coupled horizontal tensile elements are present (tie rods or ring beams):

$$V_f = \frac{D N_S}{H}\left(1 - 1.15\frac{N_S}{D\,t\,f_{hm}}\right)$$

In other cases, a limited horizontal tensile strength f_{ht} at the end sections is considered

$$f_{ht} = \min\left(\frac{f_{bt}}{2}\ ;\ \ f_{v0} + \frac{\mu\sigma_y}{\phi}\right)$$

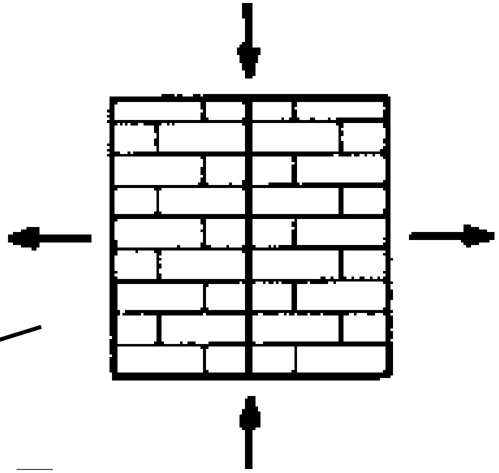
Failure of blocks (brittle)

$$V_f = 1{,}15\,\frac{d^2 t}{6\,h_0}\,f_{ht}$$

Sliding in joints (ductile)

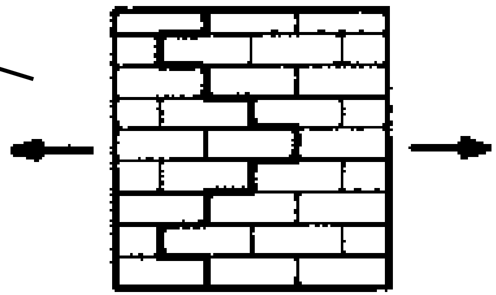
$$V_f = \frac{d^2 t}{2h_0(1+f_{ht}/f_h)}\,f_{ht}$$

Diagonal Cracking



ut the compressive
t due to adjacent piers

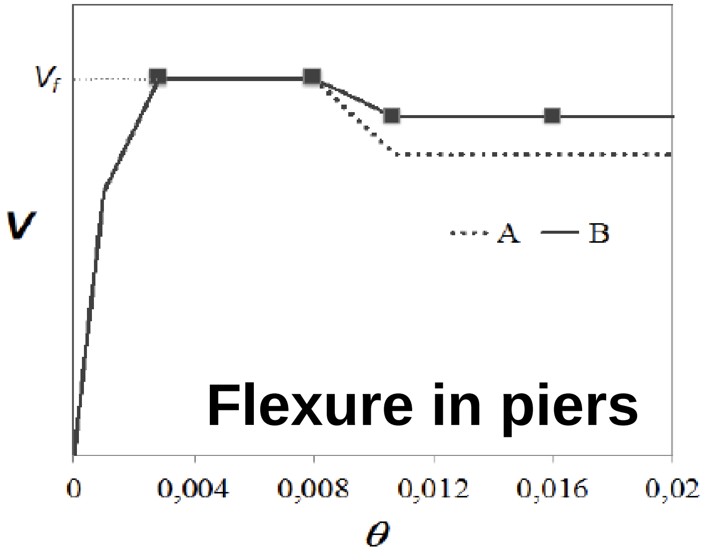
$$\frac{-}{f} + \frac{m}{1+mf}\,s_0 \ \ddot{o}\ \div £ V_{d,lim}\ ø$$

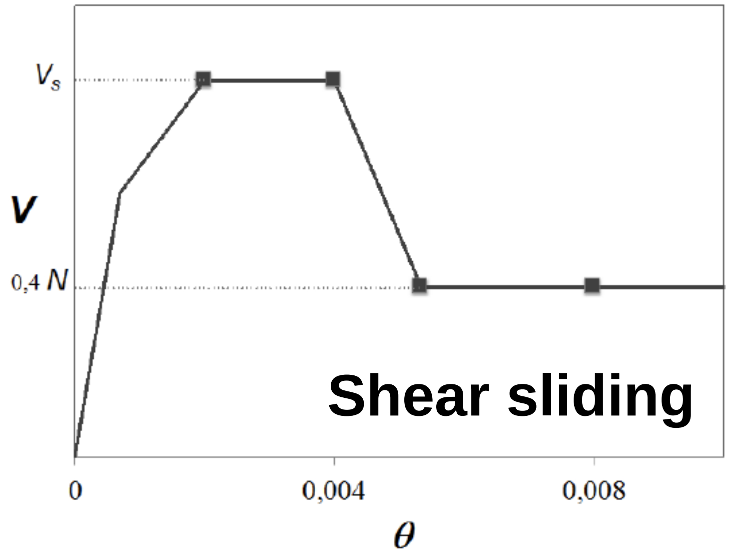


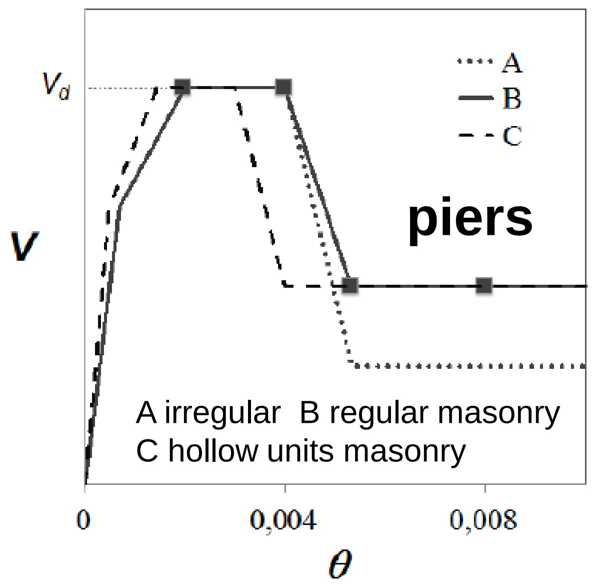
$$f_t\sqrt{1+\frac{s_0}{f_t}}$$

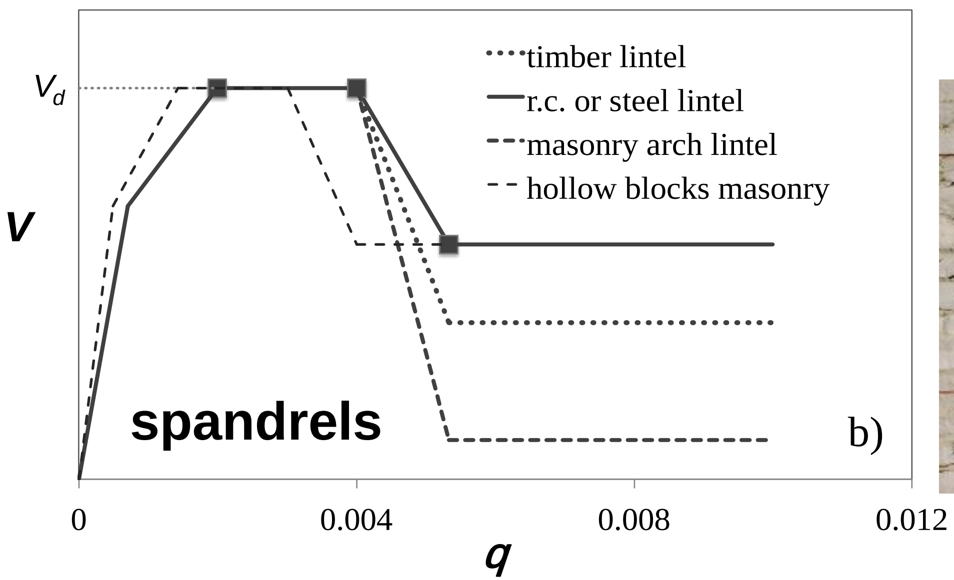
- DRIFT THRESHOLDS AT SD LIMIT STATE, FOR PIERS AND SPANDRELS

MASONRY	WALL ELEMENTS	FLEXURAL	SHEAR SLIDING	DIAGONAL CRACKING (pre-modern only)
REGULAR (modern & pre-modern)	PIERS	**0,01(1-n)**	modern: **0,004** pre-modern: **0,008** (sliding) **0,005** (units failure)	**0,006**
	SPANDRELS	**0,016** (good lintel) **0,012** (other cases)	-	**0,006**
IRREGULAR (pre-modern)	PIERS	**0,01(1-n)**	-	**0,005**
	SPANDRELS	**0,016** (good lintel) **0,012** (other cases)	-	**0,005**









Diagonal cracking

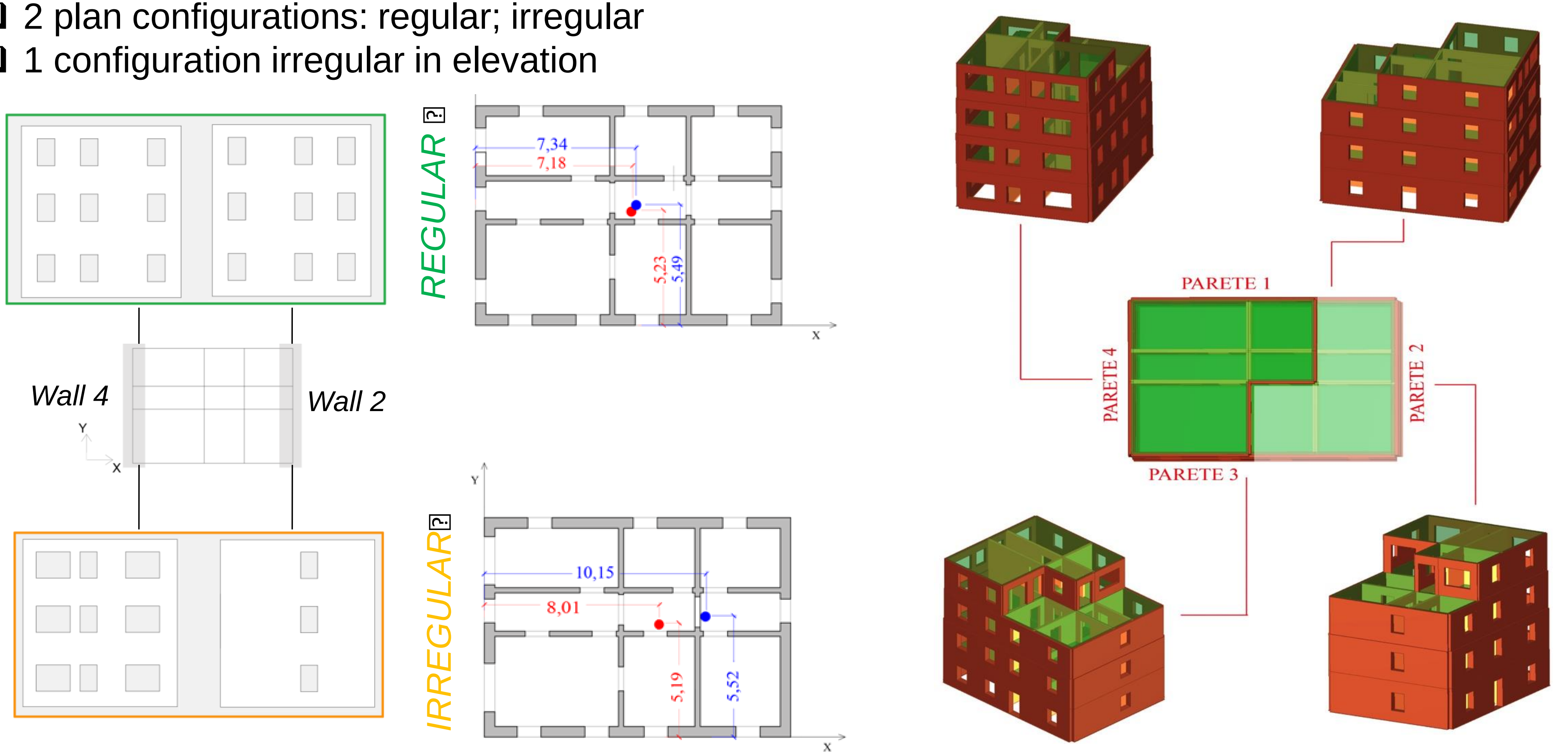
GLOBAL ANALYSIS

- 3D MODELLING OF THE BUILDING
  - wall to wall connection ⟹ flange effect
  - stiffness of horizontal diaphragms:
    - ❑ rigid – seismic actions are applied in the centre of mass ⟹ 3D
    - ❑ stiff – seismic actions are applied to each node of the model
    - ❑ flexible – global model is meaningless ⟹ **wall-by-wall analyses**

IS PUSHOVER ANALYSIS FEASIBLE FOR THESE BUILDINGS?

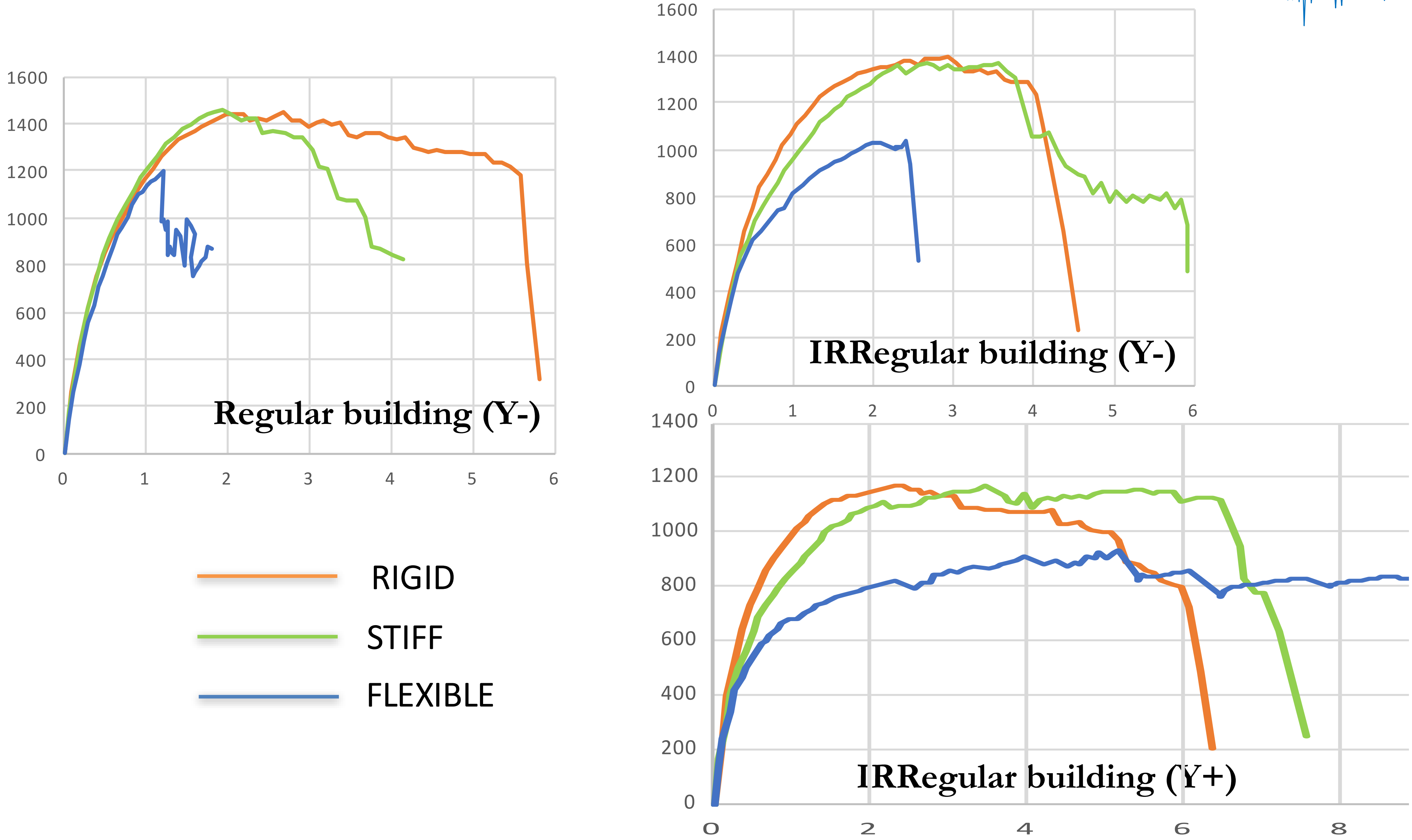
- CRITICAL ISSUES:
  - equivalent SDOF system ⟹ control displacement
  - higher mode effects ⟹ load patterns
  - displacement demand ⟹ applicability of N2 method for low period structures

- 3 stiffness of diaphragms - equivalent G[MPa]: 12500 MPa (rigid); 100 (stiff); 10 (flexible)
- 2 structural details: A) with tie-rods; B) with RC tie-beams
- 2 plan configurations: regular; irregular
- 1 configuration irregular in elevation

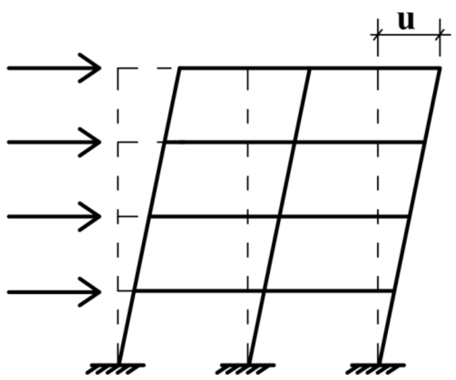


Marino et al. (2018) "Use of nonlinear static procedures for irregular URM buildings in literature and codes", Proc. of 16ECEE, paper ID 11593

INFLUENCE OF STIFFNESS OF HORIZONTAL DIAPHRAGMS



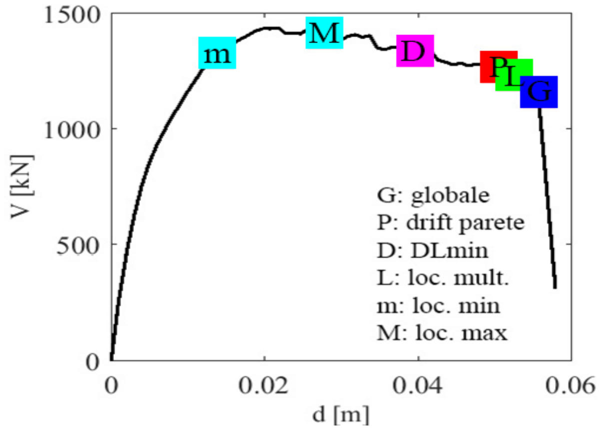
1) Pushover analysis



choice of Load Pattern

- ❖ Uniform
- ❖ Triangular
- ❖ 1^{st} modal
- ❖ Multimodal
- ❖ …

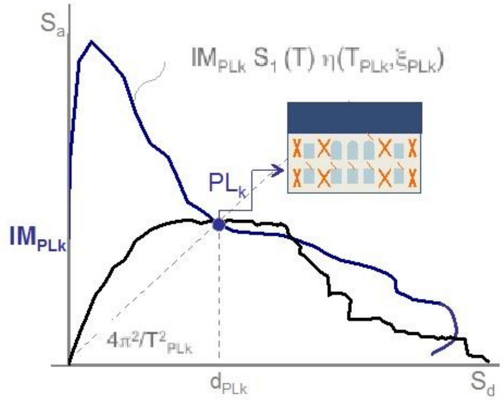
2) Definition of LSs



Possible approaches

- ❖ Local checks
- ❖ Global scale
- ❖ Mixed methods
- ❖ …

3) Displacement demand



SDOF target displacement

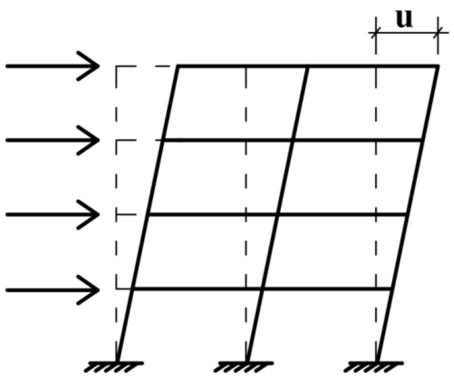
- ❖ N2 method
- ❖ Coeff. Method
- ❖ CSM
- ❖ …

Proposed by EC8 an Italian code

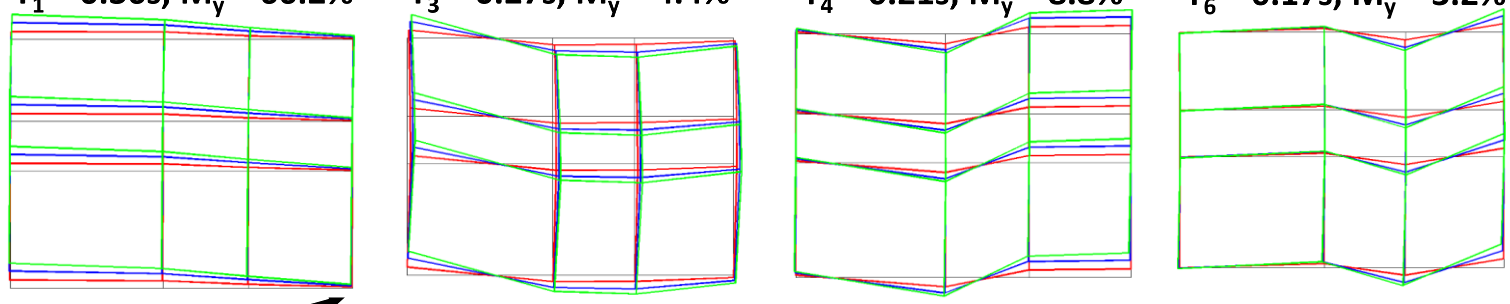
Proposed by ASCE-SEI 41-13

Proposed by all codes above

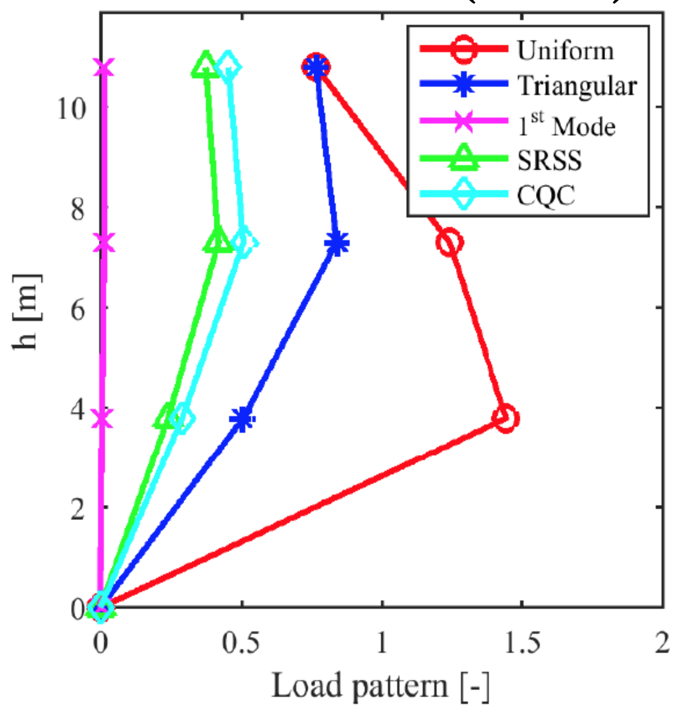
ANALYSIS: DEFINITION OF THE LOAD PATTERN



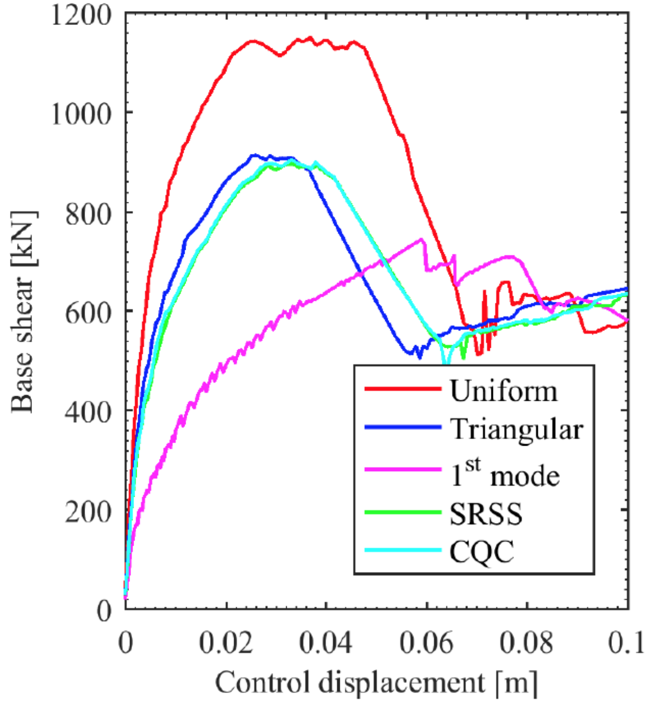
T_1 = 0.36s, M_y = 66.2% | T_3 = 0.27s, M_y = 4.4% | T_4 = 0.21s, M_y = 8.8% | T_6 = 0.17s, M_y = 3.2%



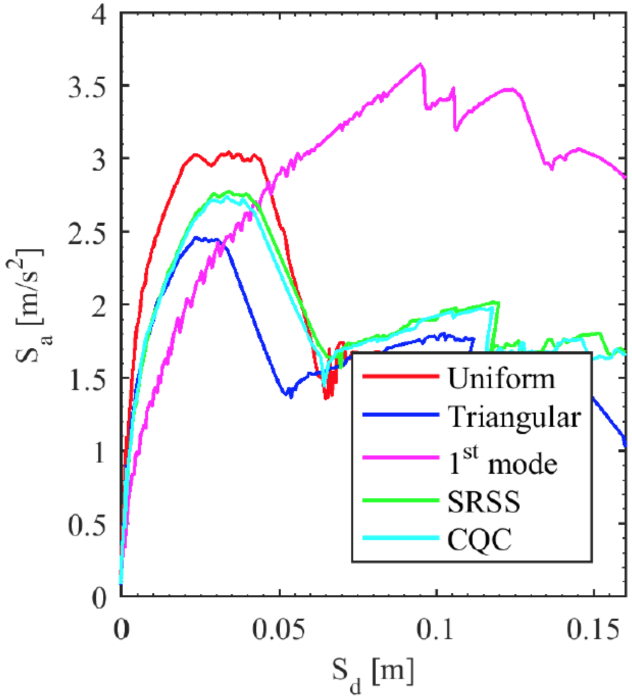
Load Patterns (Wall 2)



Pushover curve



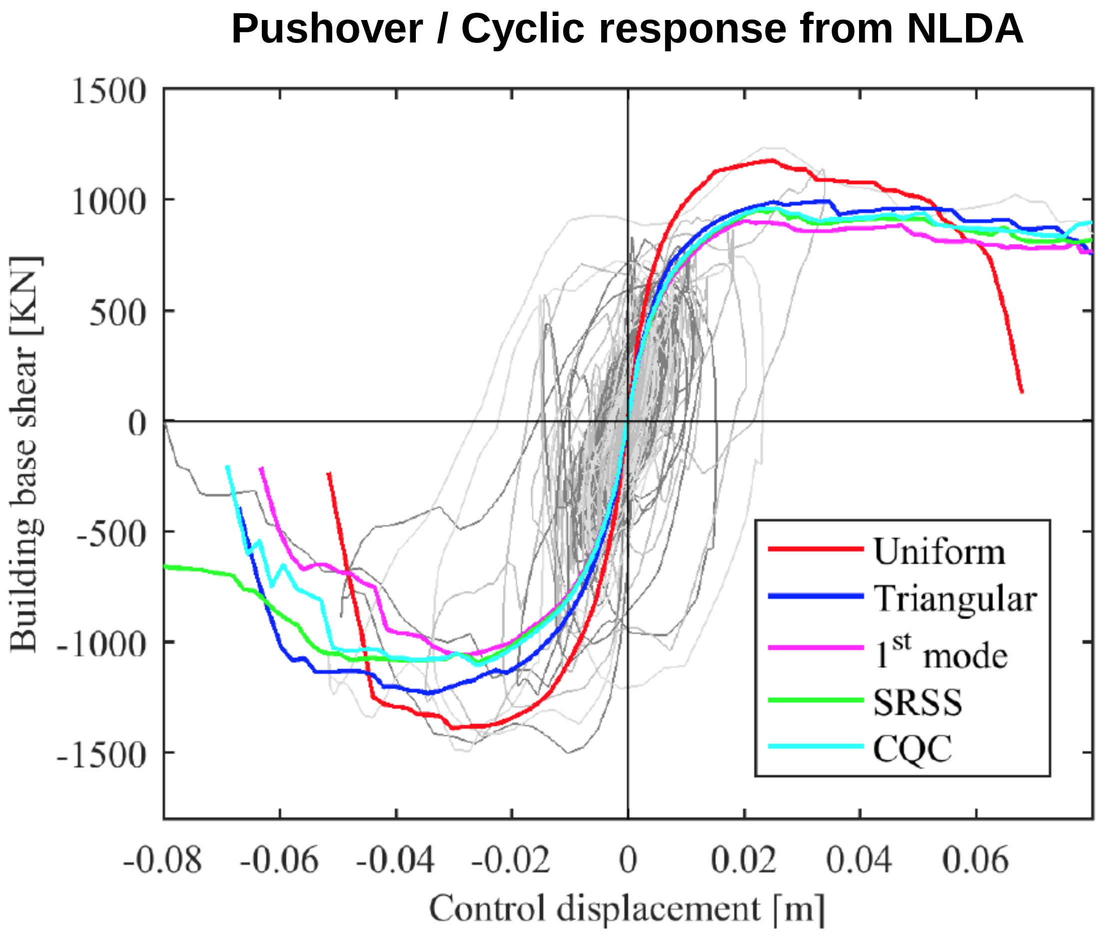
Capacity curve

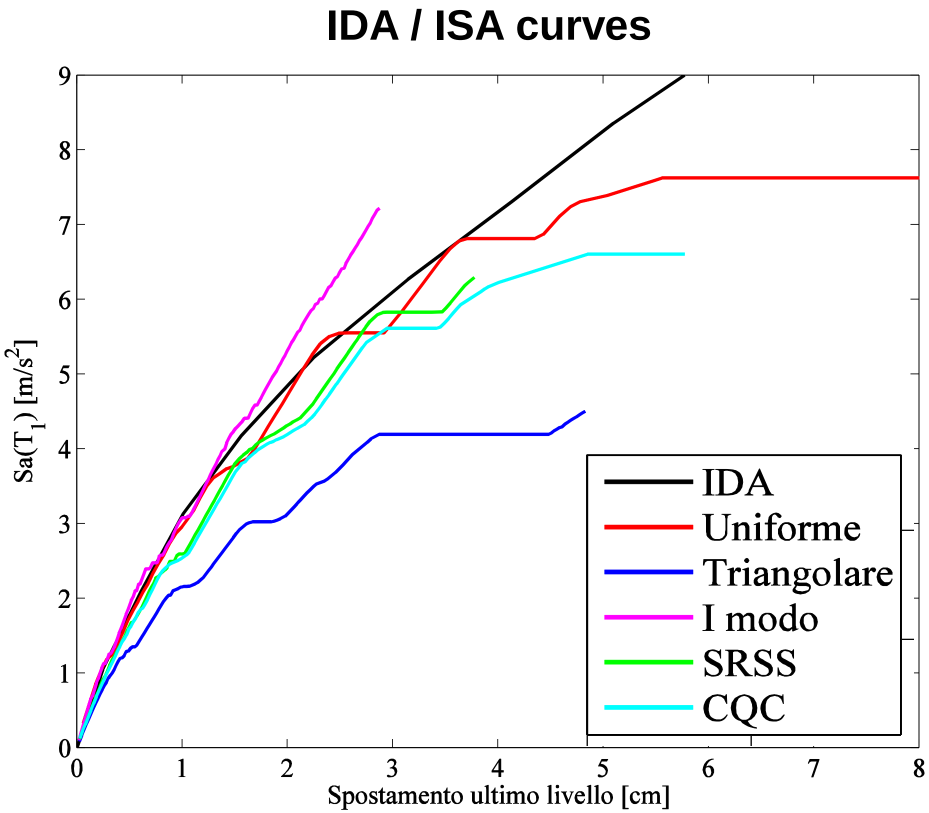


e* (1st mode) ≈ 50% **e* (other LPs) ≈ 80%**

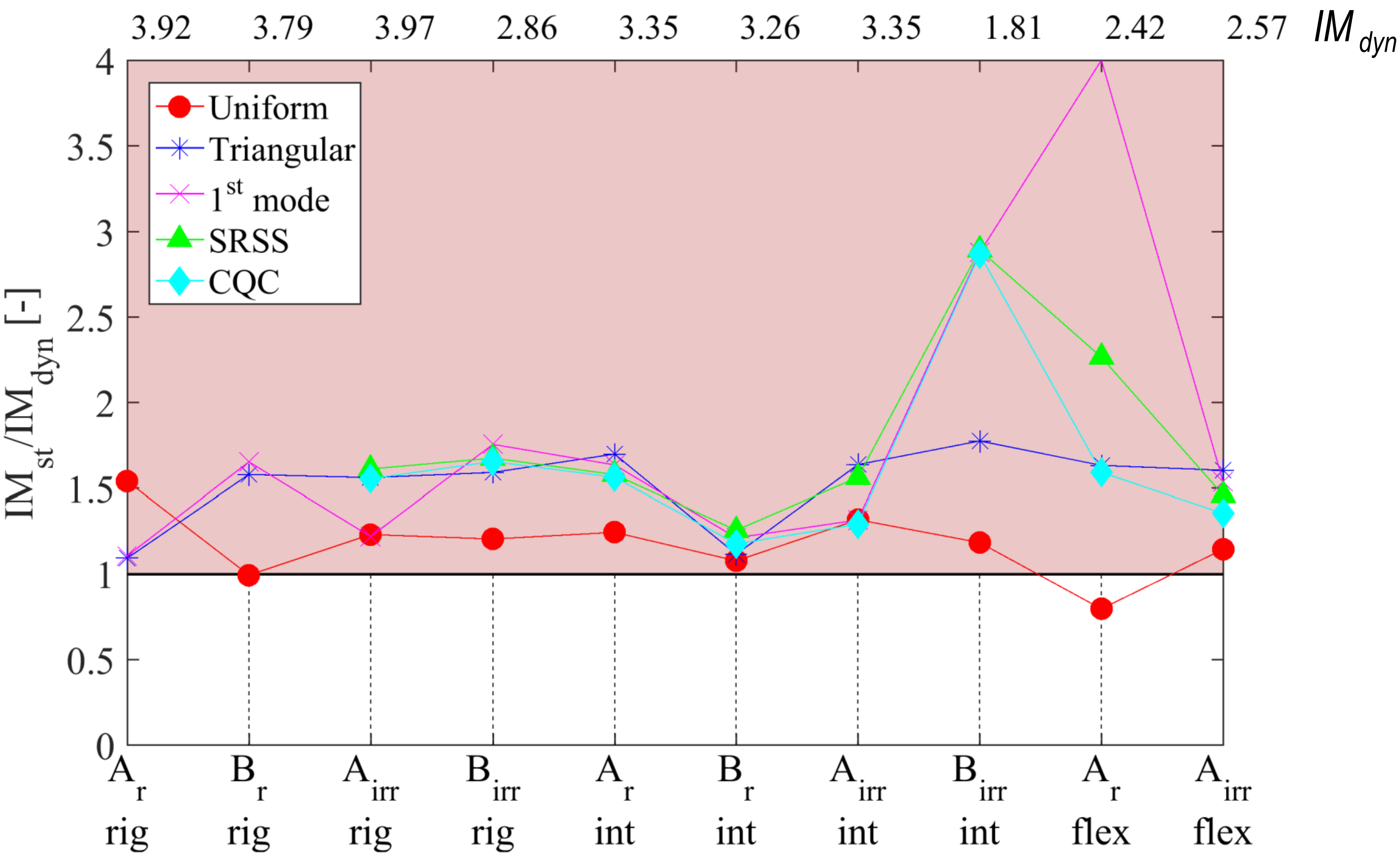
VALIDATION OF NLSA BY NONLINEAR DYNAMIC ANALYSES

- Incremental Dynamic Analysis (IDA) with real records, as reference solution
- Evaluation by NLSA of the IM_{NC} using the fractile response spectra





$IM_{NC,NLSA}$ / $IM_{NC,NLDA}$

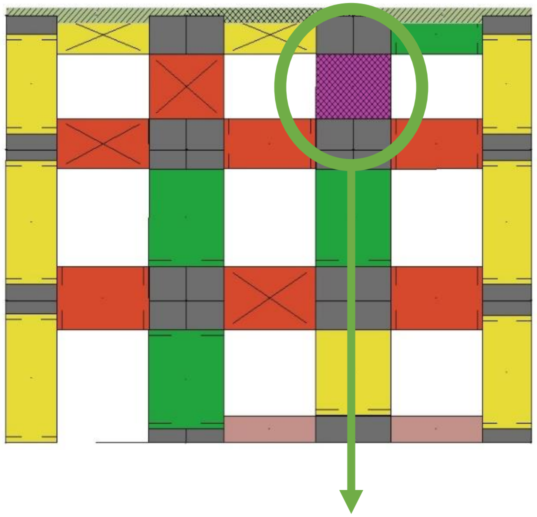


LS of NC: in Global terms

TD: computed by using N2 method

VERIFICATION OF LIMIT STATES

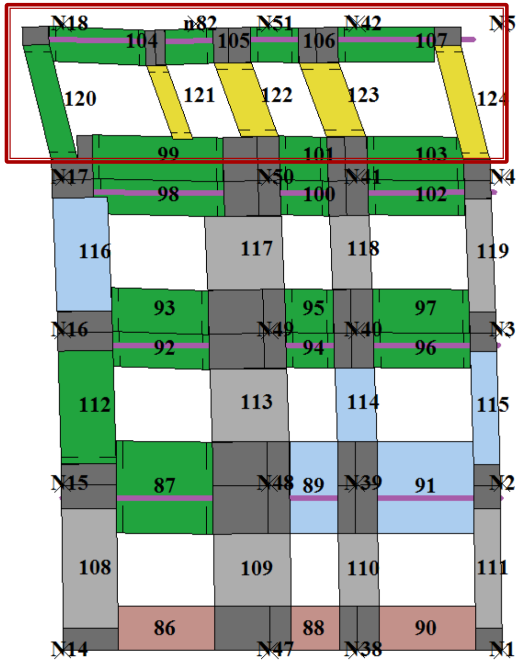
a) in Local terms (element level)



Limit State is attained when the corresponding damage level is achieved in the first element

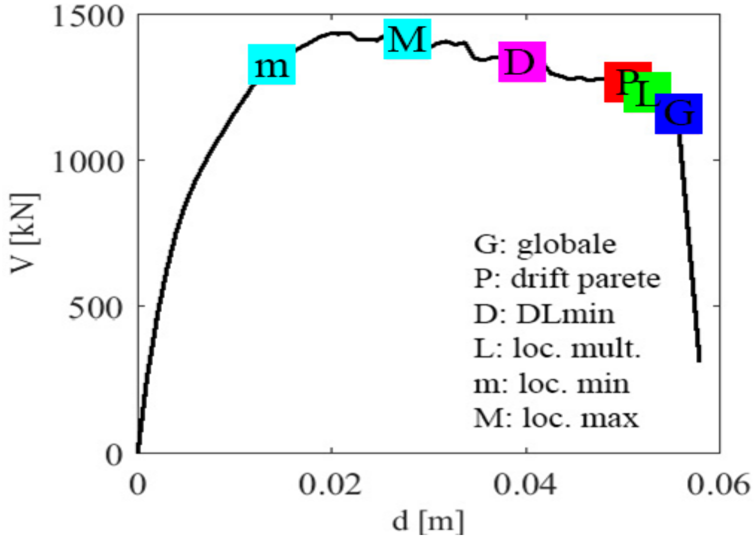
ASCE/SEI 41-13

b) at Wall scale



Interstorey drift limitation or check of damage level in all piers of the same wall/storey

c) in Global terms

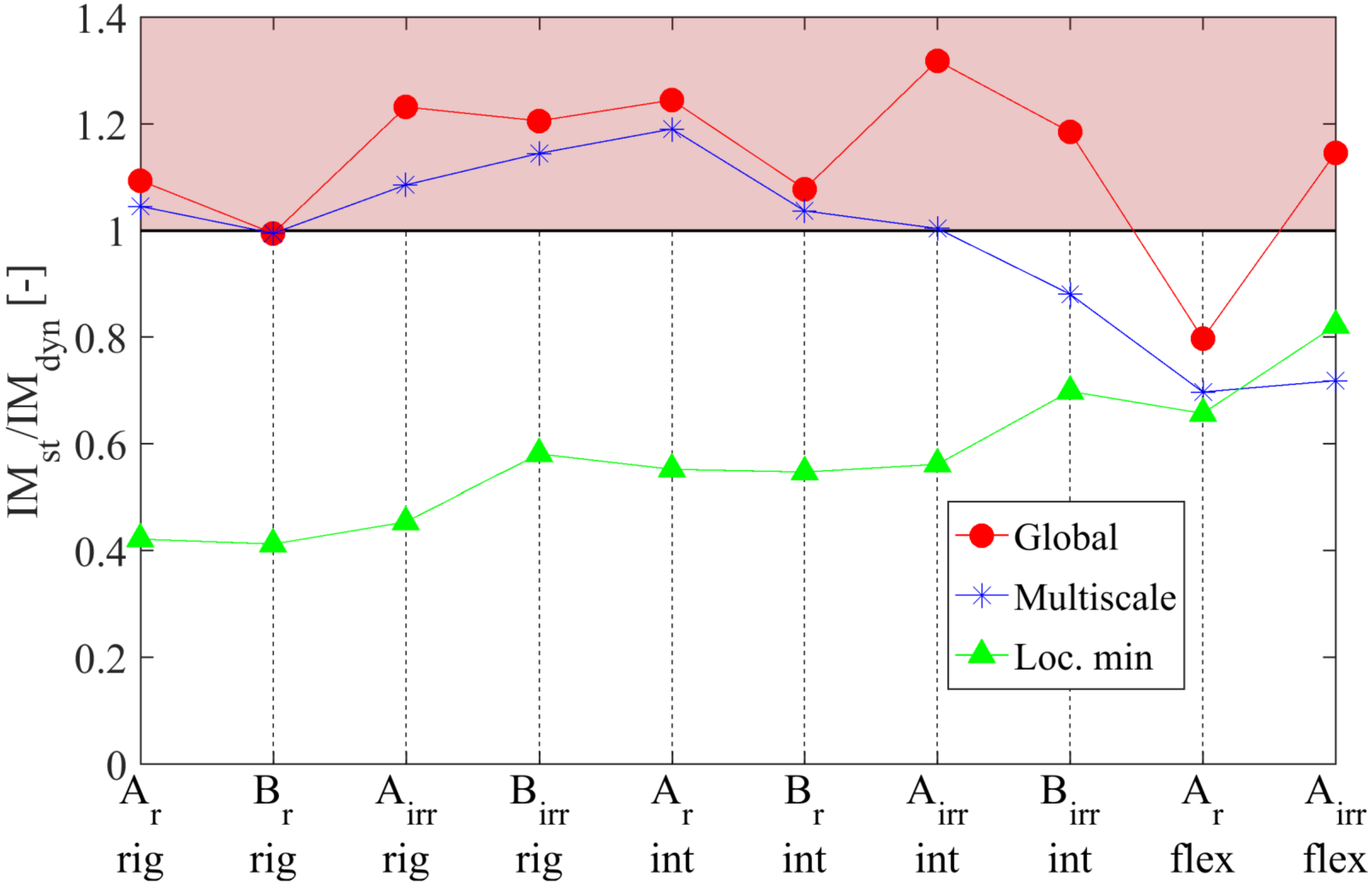


Limit Scale is attained when the total base shear drop down of 20%

EC8 - NTC 2008

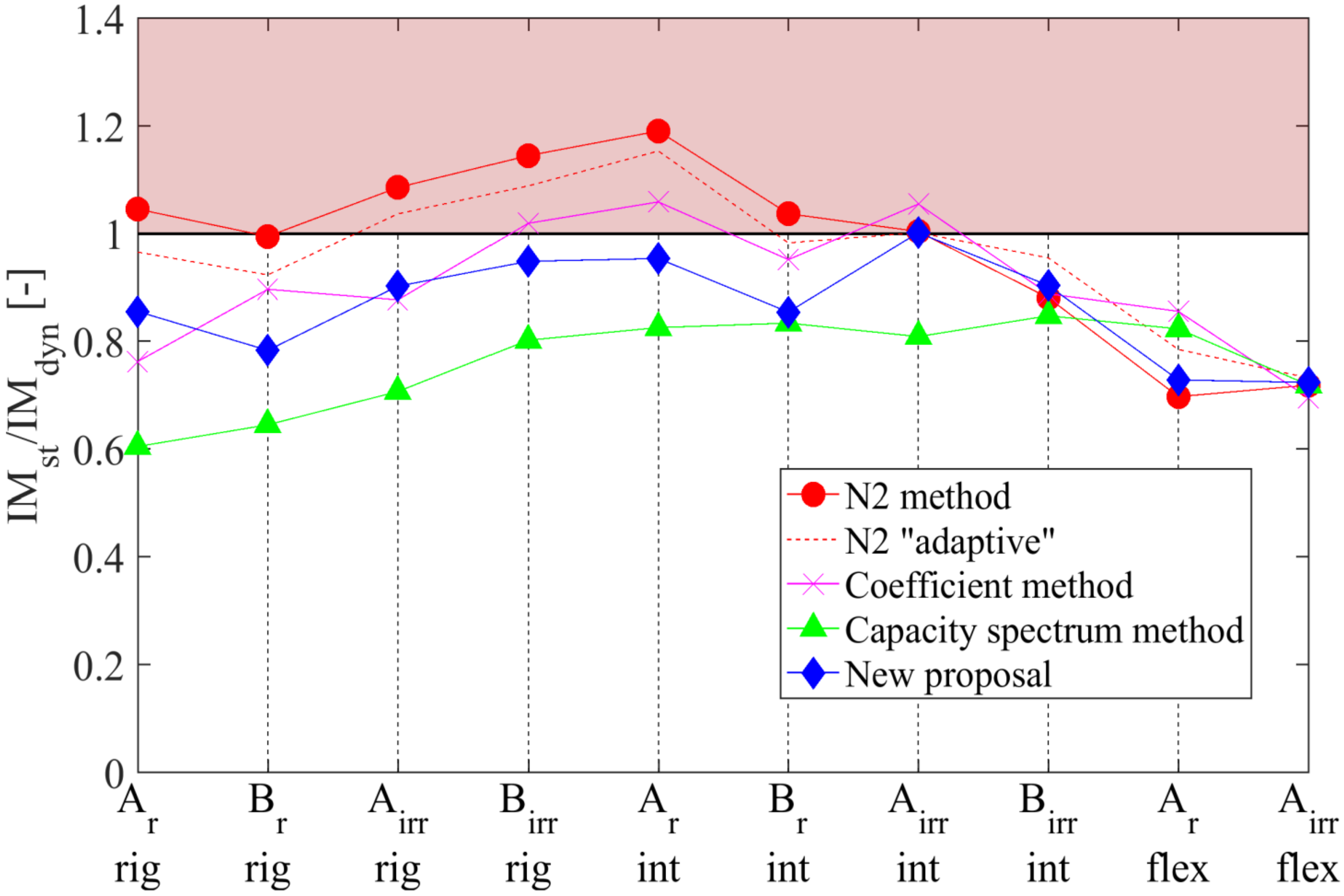
- Verification in Local terms is too much cautionary for masonry buildings
- Drawbacks of Global terms: a) torsional effect; b) stiff diaphragms

➔ D_{NC} is the minimum between: i) global, ii) mechanism in a wall, iii) crushing of pier



LPs: minimum IM between values obtained from Uniform & SRSS

TD: computed by using N2 method



- N2 method underestimates the demand for low period structures (Fajfar keynote - Guerrini et al. 2017, BEE)
- A new correction should be adopted, depending on the dissipation and softening:

$$d^*_{max} = d^*_y R_i^a$$

$$R = \frac{d_e^*}{d_y^*} = \frac{F_e^*}{F_y^*} = \frac{m^* S_{ae}(T^*)}{F_y^*}$$

$$a_i = \frac{1}{\ln 4} \ln\left(1 + 3b\frac{T_C}{T_i^*}\right) \geq 1$$

LP: minimum IM between values obtained from Uniform & SRSS

LS of NC: multiscale approach

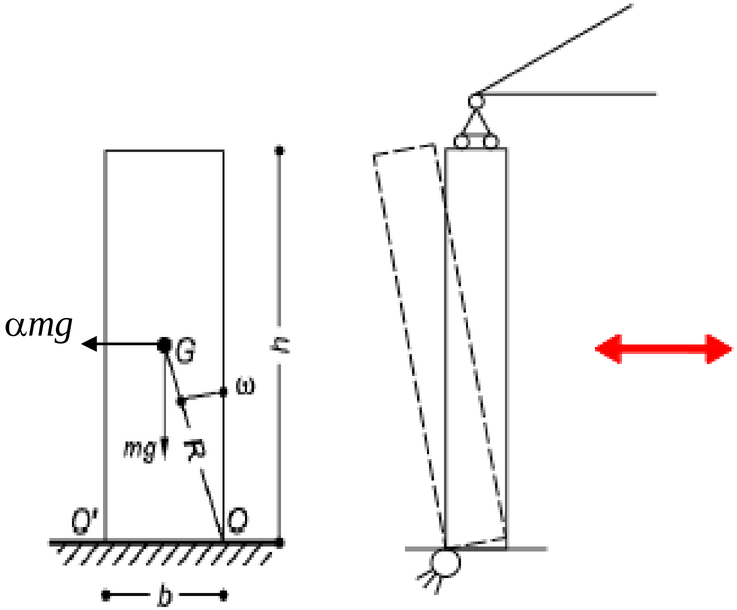
PREVENTION OF LOCAL MECHANISMS

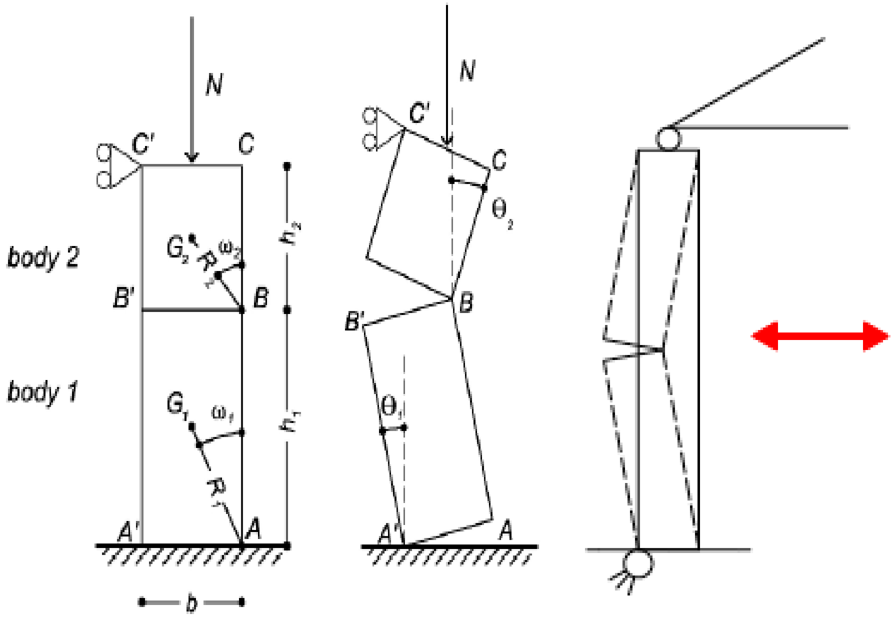
- MODERN MASONRY BUILDINGS ⟹ possible only at interstorey level
- PRE-MODERN MASONRY BUILDINGS ⟹ connections are not continuous

Limit Analysis of Masonry Structures

- Heyman (1966) "The stone skeleton", Int. J. Solids Structures

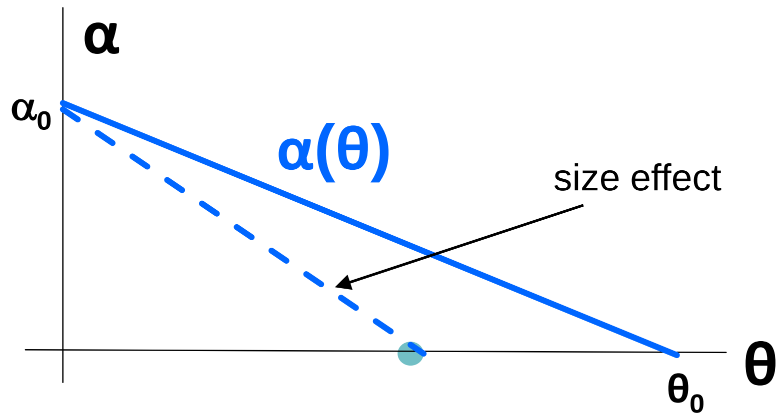
  mechanism of rigid blocks (no tensile strength) that rotates and slide





Linear Kinematic Analysis → α_0

NonLinear Kinematic Analysis → $\alpha(\theta)$



Rocking under seismic excitation

- Housner (1963) "The behavior of inverted pendulum structures during earthquakes"
  - overturning under seismic excitation presents dynamic instability

✓ Ishihama (1982) "Motion of rigid bodies and criteria for overturning by earthquake excitations", Earthq Eng Struct Dyn
  - onset of rocking and overturning as a function of PGA, PGD and PGV

✓ Makris and Konstantinidis (2003) "The rocking spectrum and the limitation of practical design methodologies", Earthq Eng Struct Dyn
  - elastic acceleration response spectrum is not useful for rocking

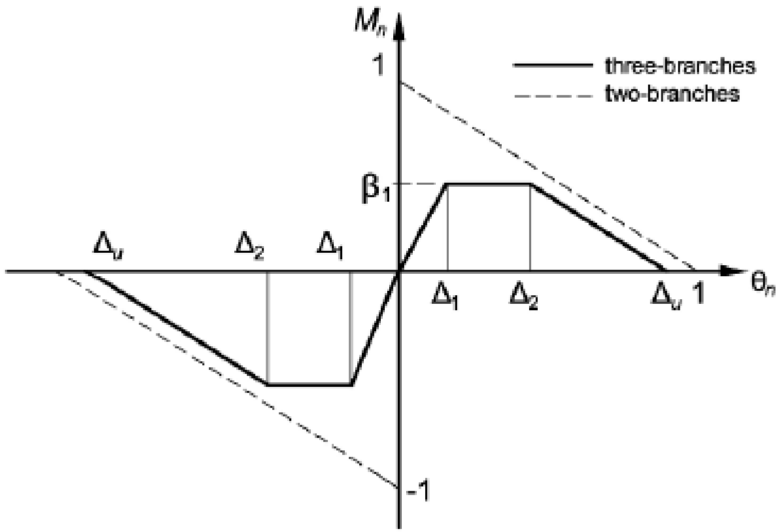
✓ DeJong and Dimitrakopoulos (2014) "Dynamically equivalent rocking structures", Earthq Eng Struct Dyn
  - identification of equivalent pulse from earthquake records

→ Masonry walls are not rigid blocks (deformation and dissipation)

Displacement-Based Assessment of Rocking

- Priestley et al. (1978) "Seismic response of structures free to rock on their foundation", Bull NZ Nat Soc Earthq Eng 11(3)

  - calculation of the demand by the displacement response spectrum on a linear equivalent structure



- ✓ Doherty, Griffith, Lam and Wilson (2002) "Displacement-Based analysis for out-of-plane bending of seismically loaded unreinforced masonry walls", Earthq Eng Struct Dyn

  - definition of a 3-linear model

- ✓ Lagomarsino (2015) "Seismic assessment of rocking masonry structures", BEE 13

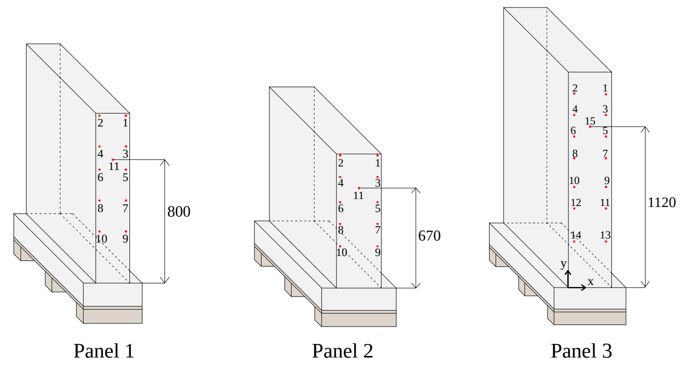
  - generalization to other local out-of-plane mechanisms

→ Implementation in codes for engineering-practice use

OUT-OF-PLANE TESTS ON STONE MASONRY PANELS

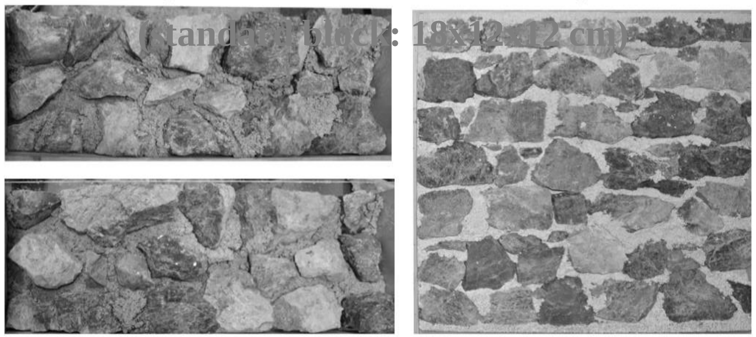
DIMENSIONS:

Real size – bigger structures – smaller elements

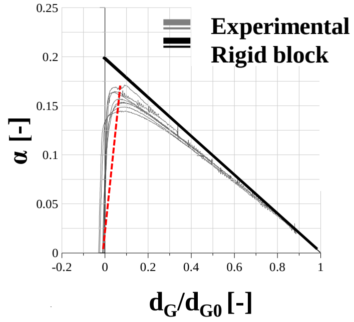


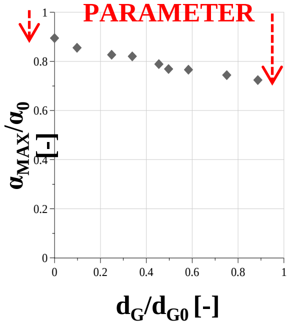
PANEL 1:	**90x110x22 cm³**	**λ=5**	**M=479 kg**
PANEL 2:	**90x90x30 cm³**	**λ=3**	**M=535 kg**
PANEL 3:	**90x150x30 cm³**	**λ=5**	**M=891 kg**

TYPICAL SECTION AND FACE OF THE PANELS

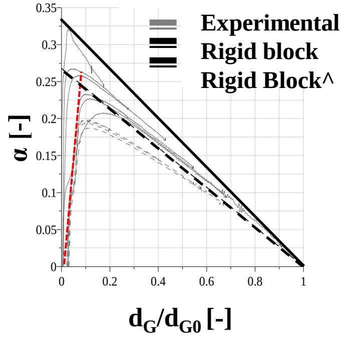


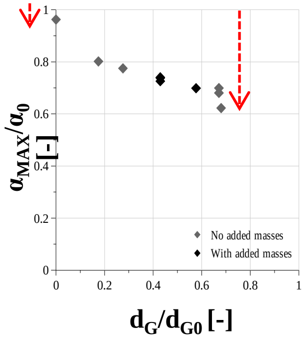
PANEL 1



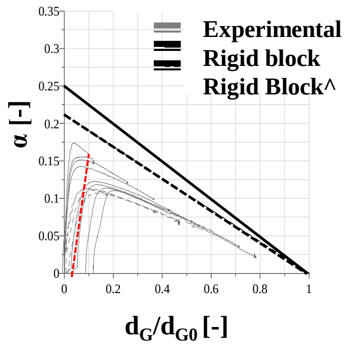


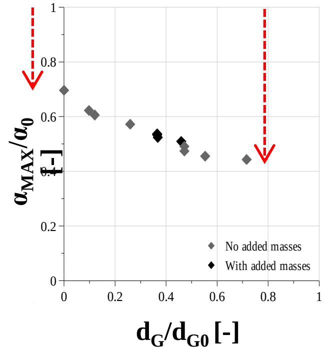
PANEL 2





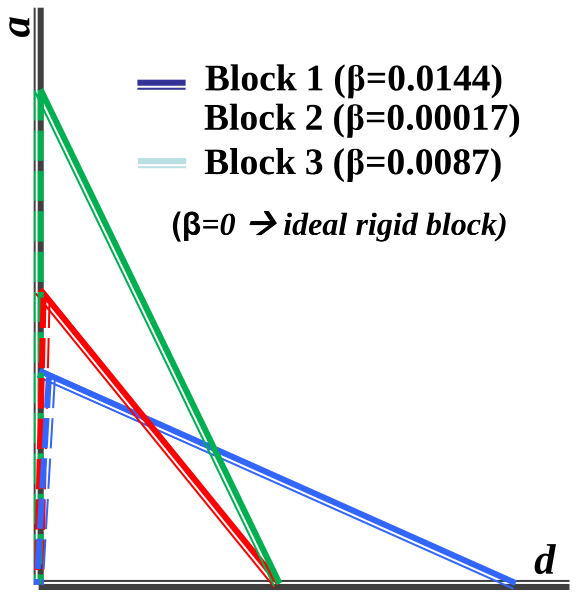
PANEL 3





Degli Abbati and Lagomarsino (2017) "Out-of-plane static and dynamic response of masonry panels ", Engineering Structures 150

- Validation by Nonlinear Dynamic Analyses (Degli Abbati S., PhD thesis, 2016)

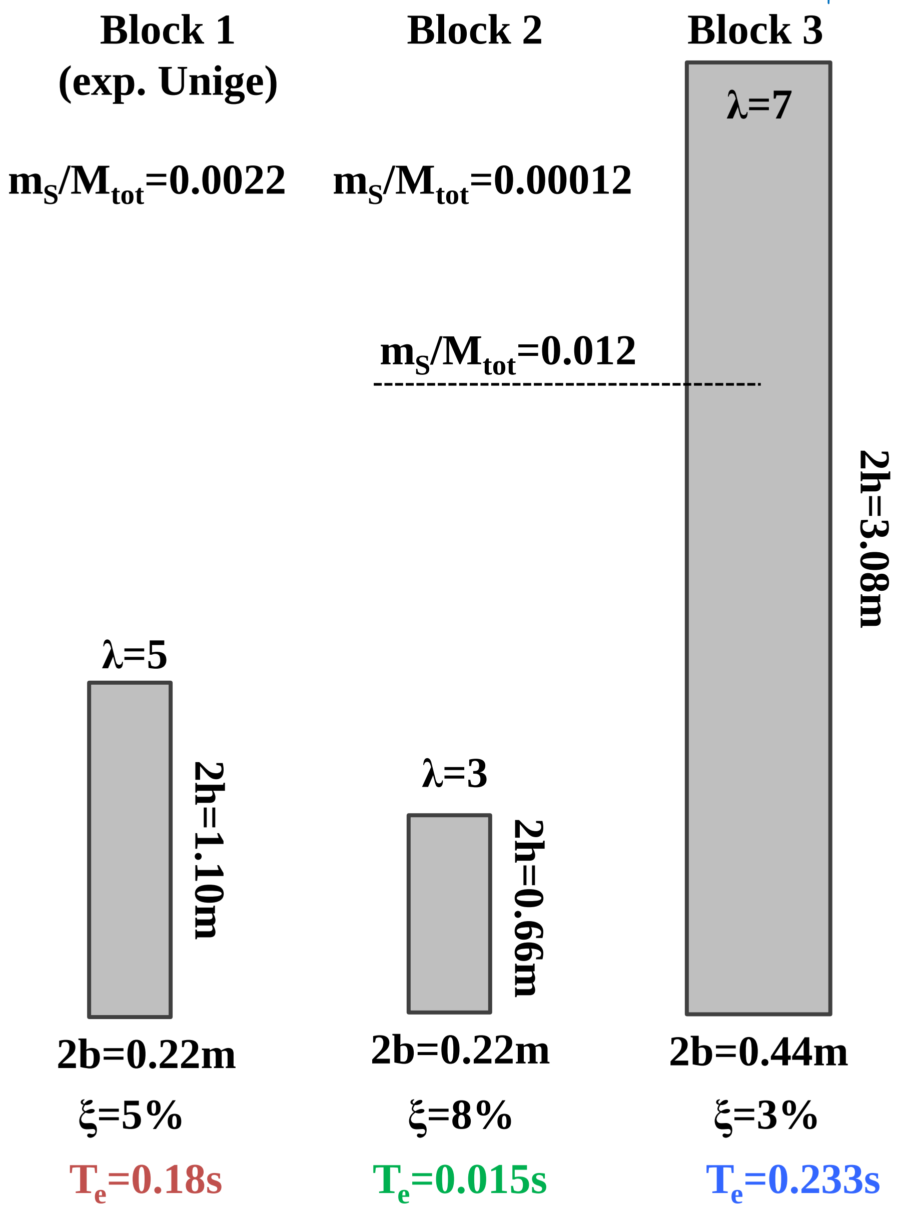


Examples of possible rocking elements

Parapets

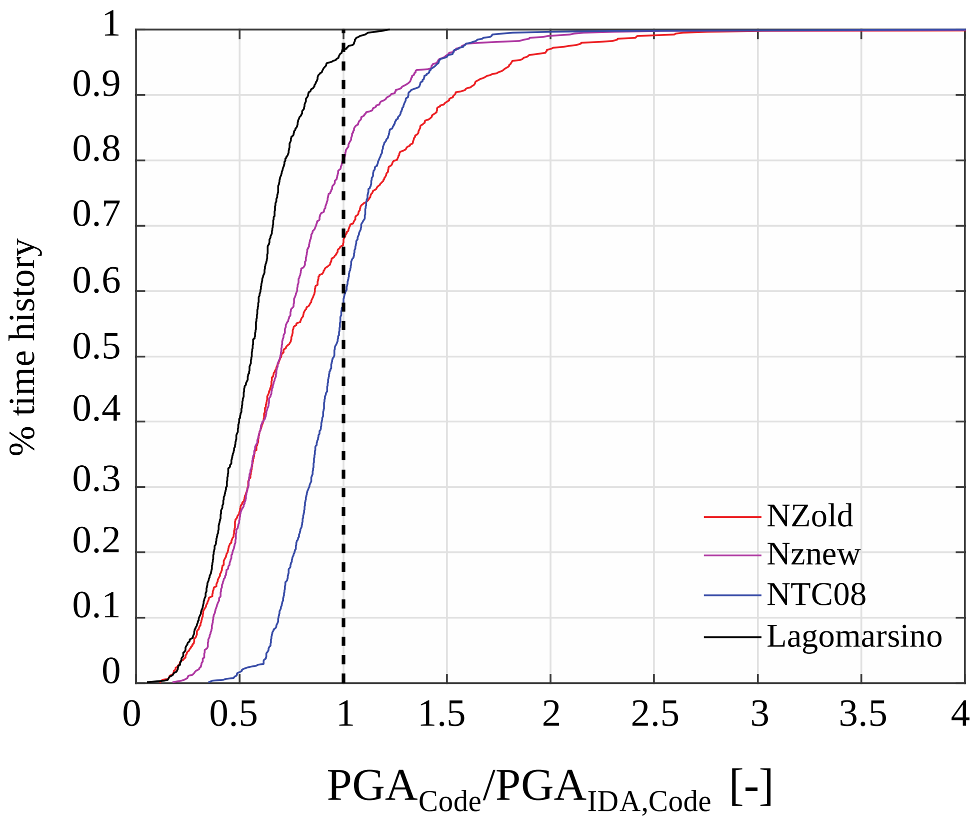
Statues - Pinnacles

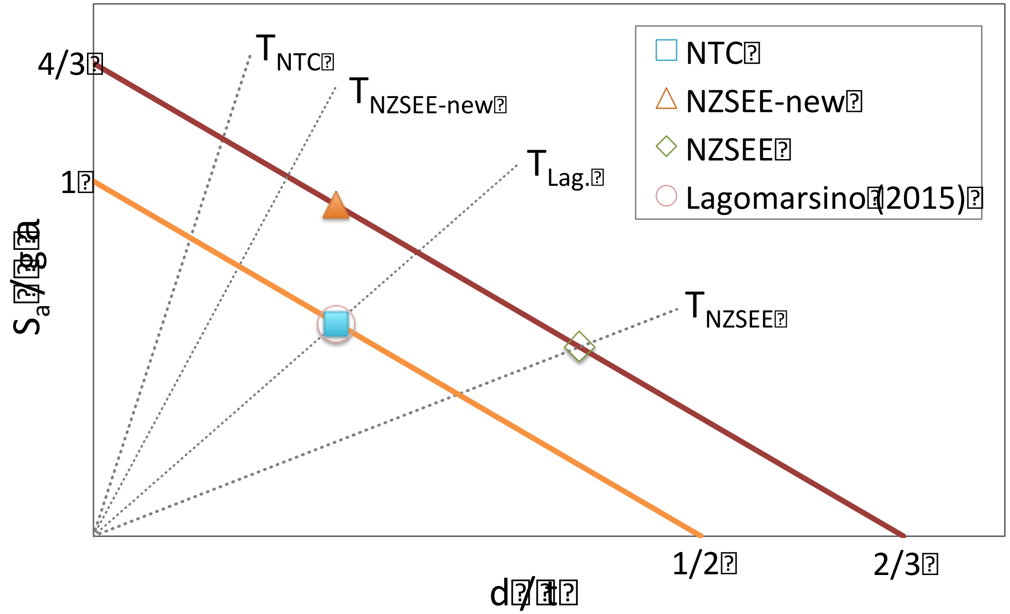
Belfry



- Validation by Nonlinear Dynamic Analyses (Degli Abbati S., PhD thesis, 2016)

Block 1 – on the ground floor





Secant periods for the evaluation of the displacement demand

$$T_{SLV} = 1.68\pi \sqrt{\frac{d_{SLV}}{a(d_{SLV})}}$$

$$T_{SLC} = 1.56\pi \sqrt{\frac{d_{SLC}}{a(d_{SLC})}}$$

- Recent proposals: Menon and Magenes 2011; Calvi and Sullivan 2014; Petrone et al. 2015; Vukobratovic and Fajfar 2015/16/17; Lucchini, Franchin and Mollaioli 2017
- Theoretical-based formulation (Degli Abbati S., PhD thesis, 2016)

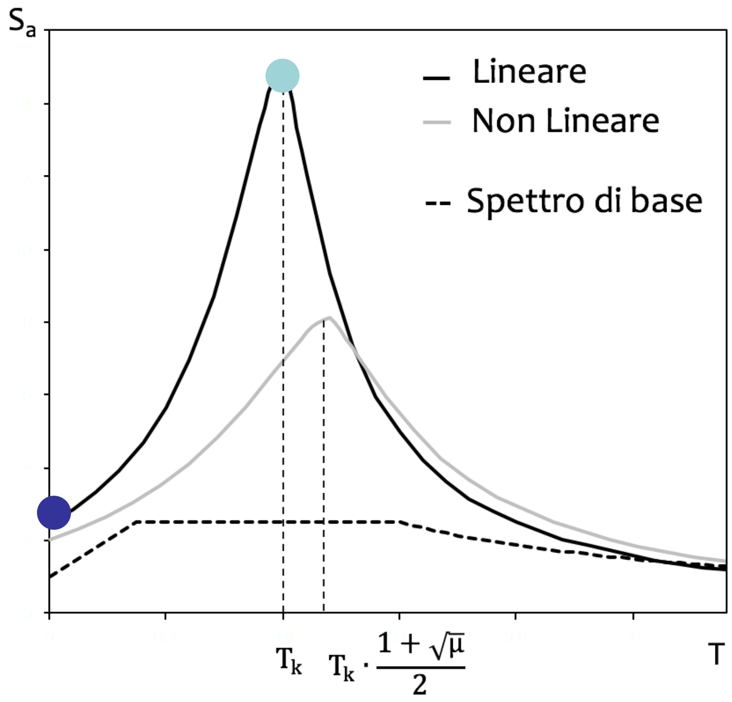
- Relevant points from closed-form solution (Burdisso Singh 1987):

$$\mathrm{PFA}_{Z,k} = S_a(T_k)\eta(\xi_k)|\gamma_k\psi_k|\sqrt{1+4\xi_k^2} \qquad \mathrm{T}=0$$

$$S_{aZ,k}(T_k) = \mathrm{AMP_k}\cdot\mathrm{PFA}_{Z,k} = f_k\cdot f_s\cdot PFA_{Z,k} \qquad \mathrm{T}=\mathrm{T_k}$$

- Analytical fitting:

$$S_{aZ,k}(T,\xi) = \begin{cases} \dfrac{\mathrm{AMP_k}\cdot\mathrm{PFA_{Z,k}}}{1+[\mathrm{AMP_k}-1]\left(1-\dfrac{\mathrm{T}}{\mathrm{T_k}}\right)^{1.6}} & \mathrm{T}\le\mathrm{T_k} \\ \dfrac{\mathrm{AMP_k}\cdot\mathrm{PFA_{Z,k}}}{1+[\mathrm{AMP_k}-1]\left(\dfrac{\mathrm{T}}{\mathrm{T_k}}-1\right)^{1.2}} & \mathrm{T}>\mathrm{T_k} \end{cases}$$



Combination of the contribution from relevant modes combined by SRSS

$$S_{aZ}(T,\xi) = \sqrt{\sum_{k=1}^{N} S_{aZ,k}^2(T,\xi)} \quad \left(\ge S_a(T)\,\eta(\xi) \quad for\ T > T_1 \right)$$

FLOOR RESPONSE SPECTRA

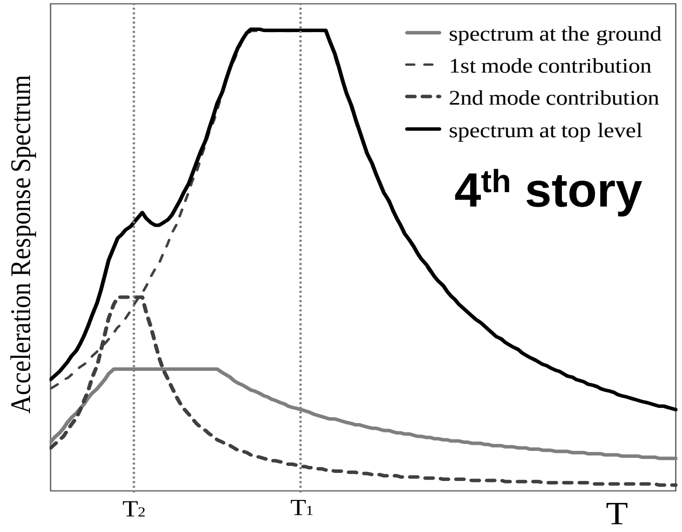
- Practice-oriented formulation

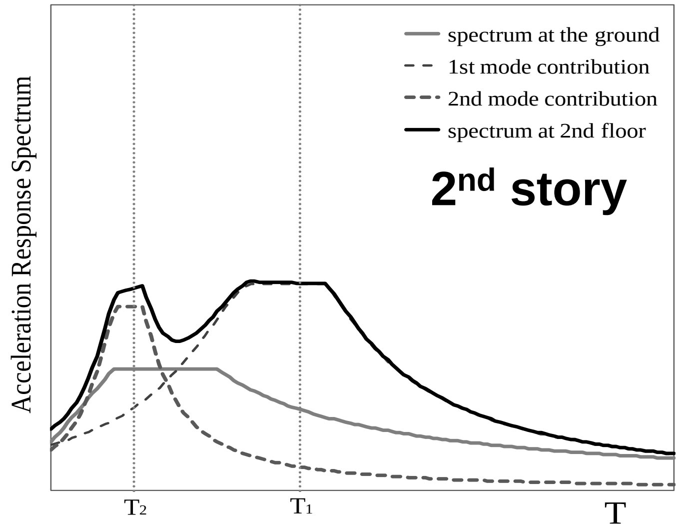
  (it is important to have a reliable displacement demand for long periods)

$$S_{eZ}(T,\xi,z)=\sqrt{\sum S^{2}_{eZ,k}(T,\xi,z)} \quad \left(\geq S_e(T,\xi) \quad per\ T>T_1\right)$$

$$S_{eZ,k}(T,\xi,z)=\begin{cases} \dfrac{1.1\,\xi_k^{-0.5}\eta(\xi)\,a_{Z,k}(z)}{1+\left[\left(1.1\,\xi_k^{-0.5}\eta(\xi)-1\right)\left(1-\dfrac{T}{aT_k}\right)\right]^{1.6}} & T<a\,T_k \\ 1.1\,\xi_k^{-0.5}\eta(\xi)\,a_{Z,k}(z) & a\,T_k\leq T\leq b\,T_k \\ \dfrac{1.1\,\xi_k^{-0.5}\eta(\xi)\,a_{Z,k}(z)}{1+\left[\left(1.1\,\xi_k^{-0.5}\eta(\xi)-1\right)\left(\dfrac{T}{bT_k}-1\right)\right]^{1.2}} & T>b\,T_k \end{cases}$$

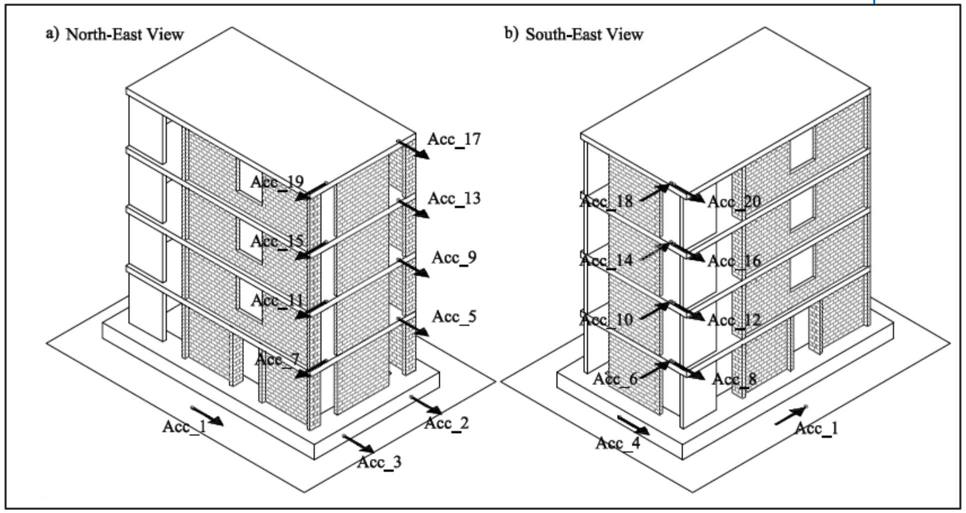
$$a_{Z,k}(z)=S_e(T_k,\xi_k)\left|\gamma_k\psi_k(z)\right|\sqrt{1+0.0004\,\xi_k^{2}}$$



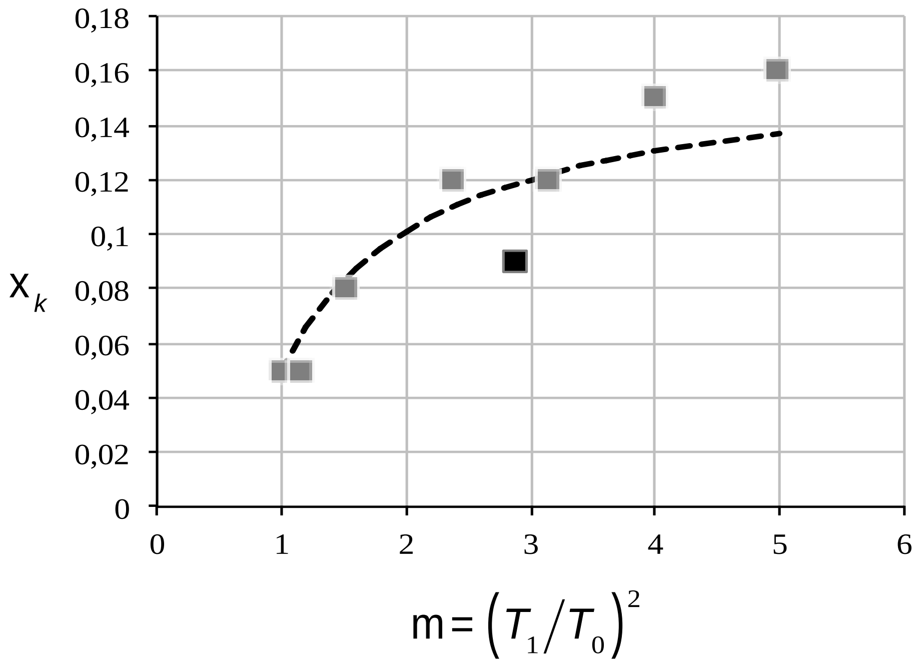


EXPERIMENTAL VALIDATION OF FLOOR SPECTRA

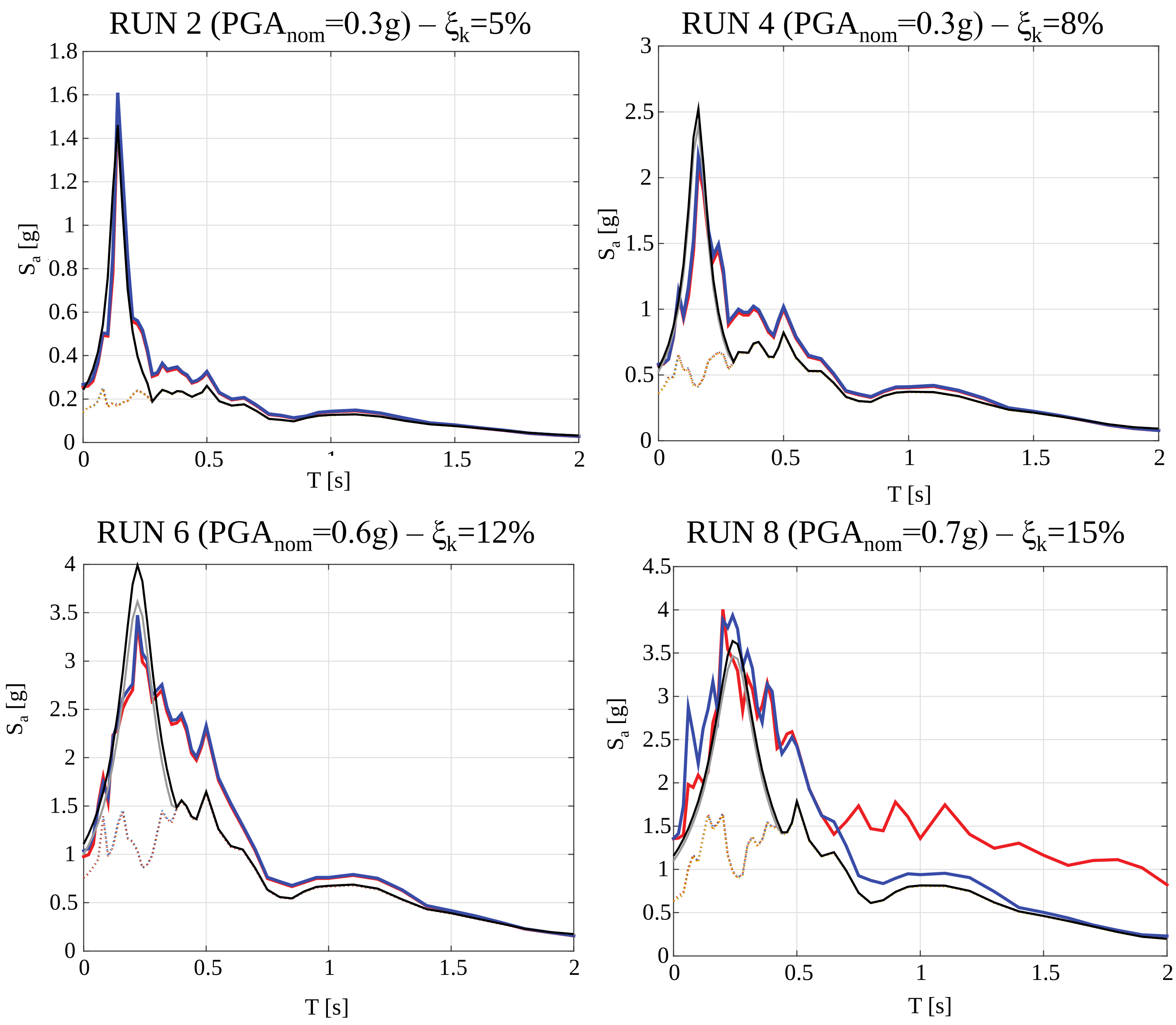
- Data from: BEYER et al. 2015
- Shake-table tests on half-scale 4-storey samples with RC and URM walls coupled by RC slabs
- Input: Montenegro earthquake (1979)
- Record scaled to match PGA from 0.05g to 0.9g



VALIDATION BOTH IN **LINEAR AND NONLINEAR BEHAVIOUR**



Parameters (T_1, γ, ψ) directly obtained by experimental results without calibration.



4th Floor

Analytical Floor Spectra:

- with γ and Ψ from PFA
- with γ and Ψ from $S_{a,zk}(T_k)$

Experimental Floor Spectra:

- Floor Spectra (from sensors at the level)
- Input Response Spectra (from sensors at the foundation)

$$PFA_k = S_a(T_k)\,h(x_k)\left|g_k \gamma_k(z)\right|\sqrt{1+4x_k^2}$$

$$S_{a,zk}(T_k) = \frac{PFA_k}{\sqrt{A}} \qquad A = x_k^{1.2}$$

- Dolce et al. (2017) "Osservatorio sismico delle strutture: the Italian structural seismic monitoring network", Bulletin of Earthquake Engineering 15

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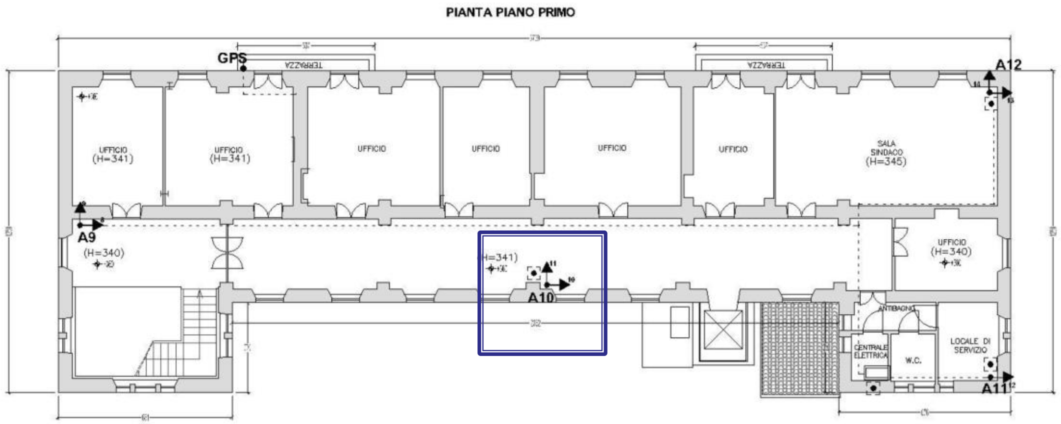
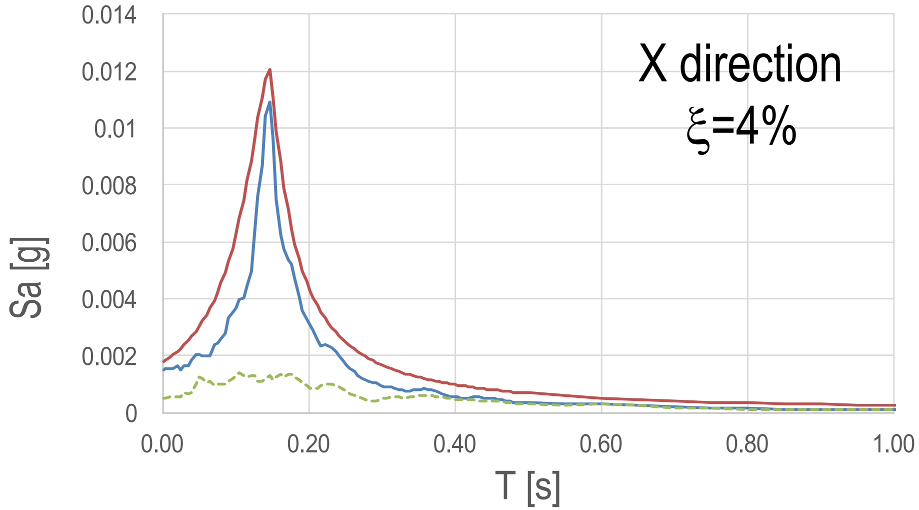
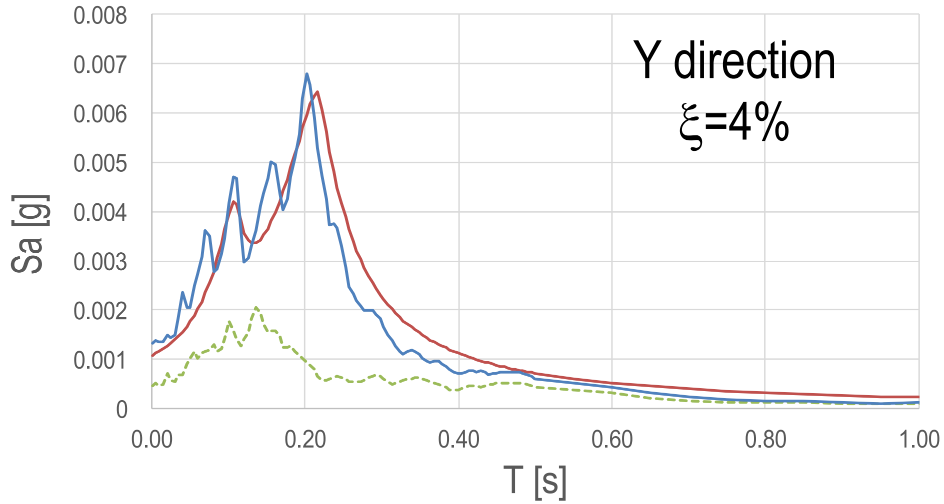


25/07/2015 minor event





Experimental (25/07/2015) Analytical Response spectum at the base

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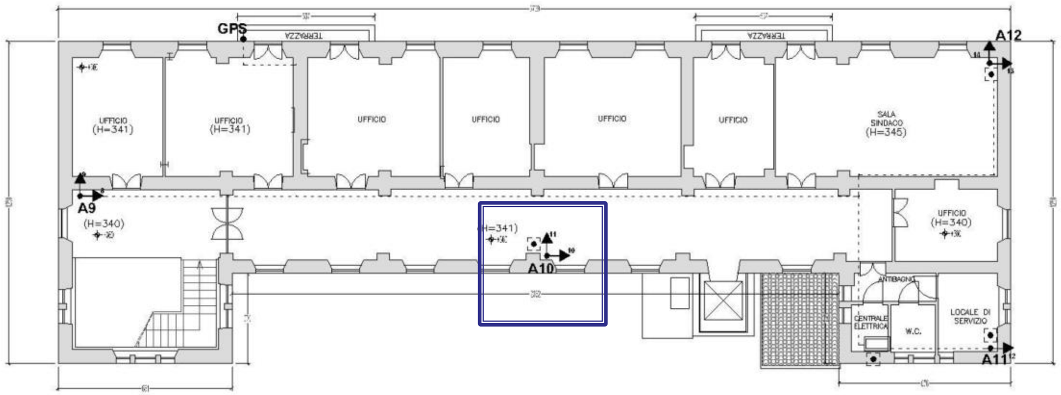
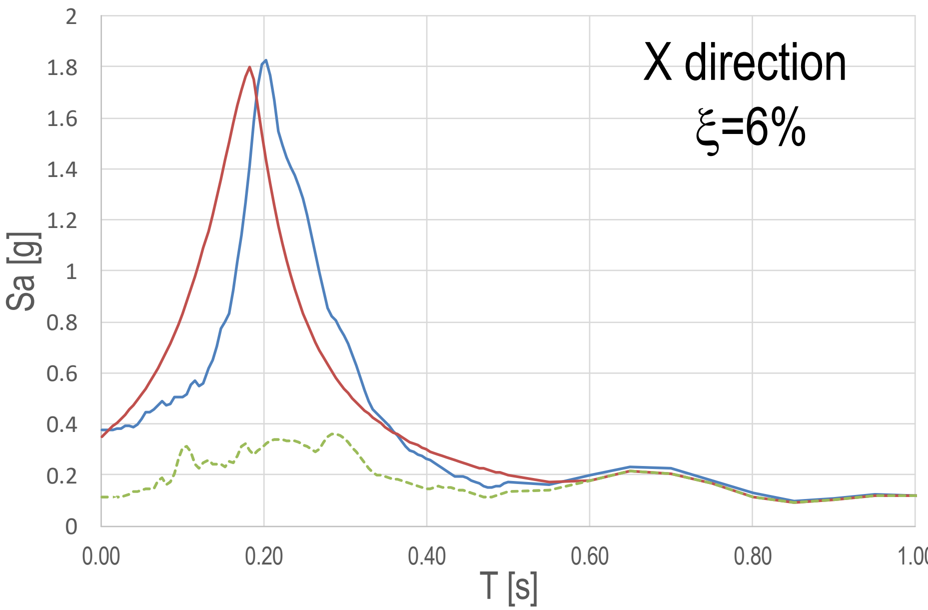
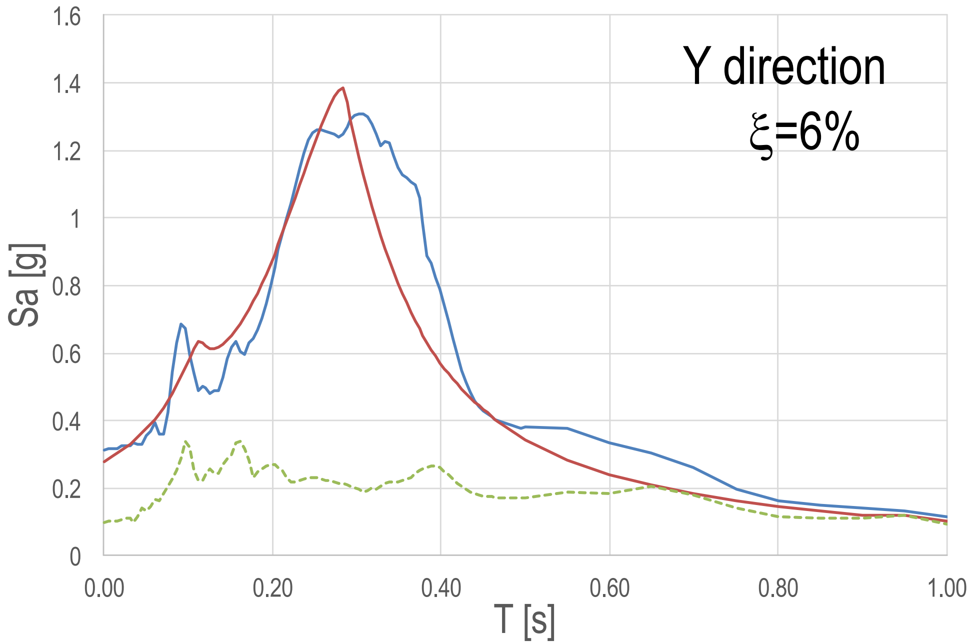
2nd floor – Sensors' localization



18/01/2017 earthquake





Experimental (18/01/2017) — Analytical — Response spectum at the base

CONCLUSIONS

- The seismic assessment of existing URM buildings requires models accurate enough to get the main features of the actual response, but simple enough to be used at engineering-practice level.
- Models developed at research level in the last 20 years have been validated by experimental tests (also full scale, static and dynamic) and by on-field observation.
- The final draft of EC8-Part 3, delivered by SC8-PT3, proposes a general framework for the seismic assessment of existing masonry buildings through the NonLinear Static Analysis.
- More experimental data on ultimate drift capacity, for different masonry typologies, and on the behaviour of spandrels are useful.
- Some issues are still open (interaction between in-plane and out-of-plane, … .)