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PUCP

Equivalent frame modelling of URM buildings: validation examples on monitored structures and TRICKY issues

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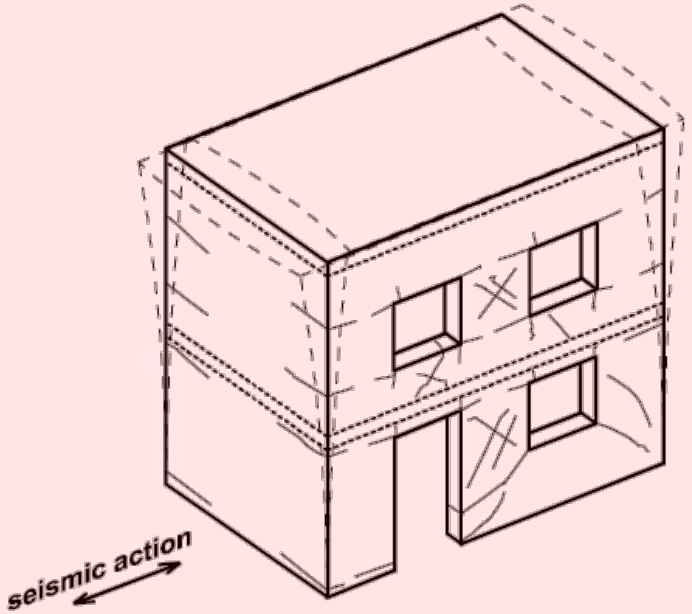
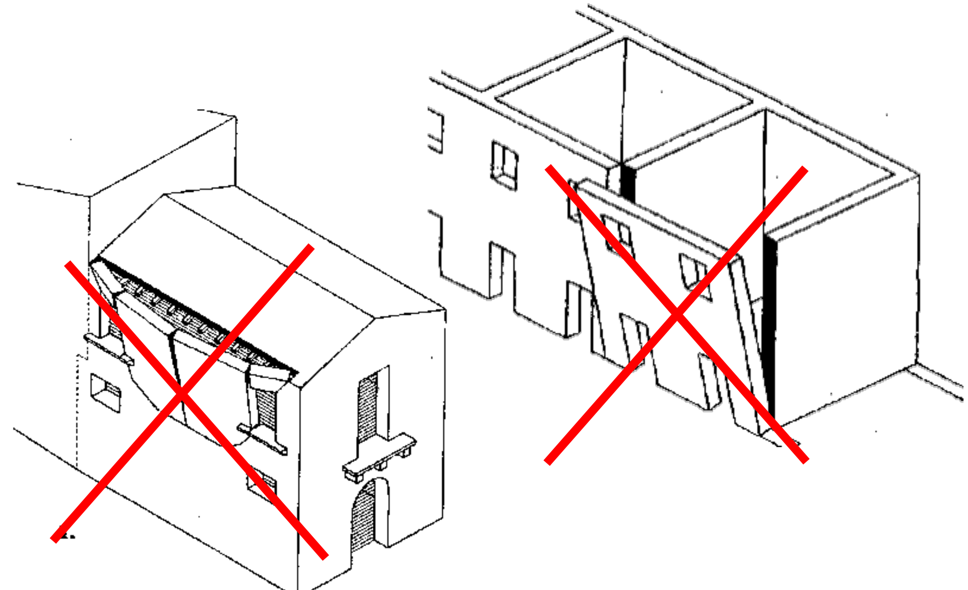
INTRODUCTION - *CONTEXT*

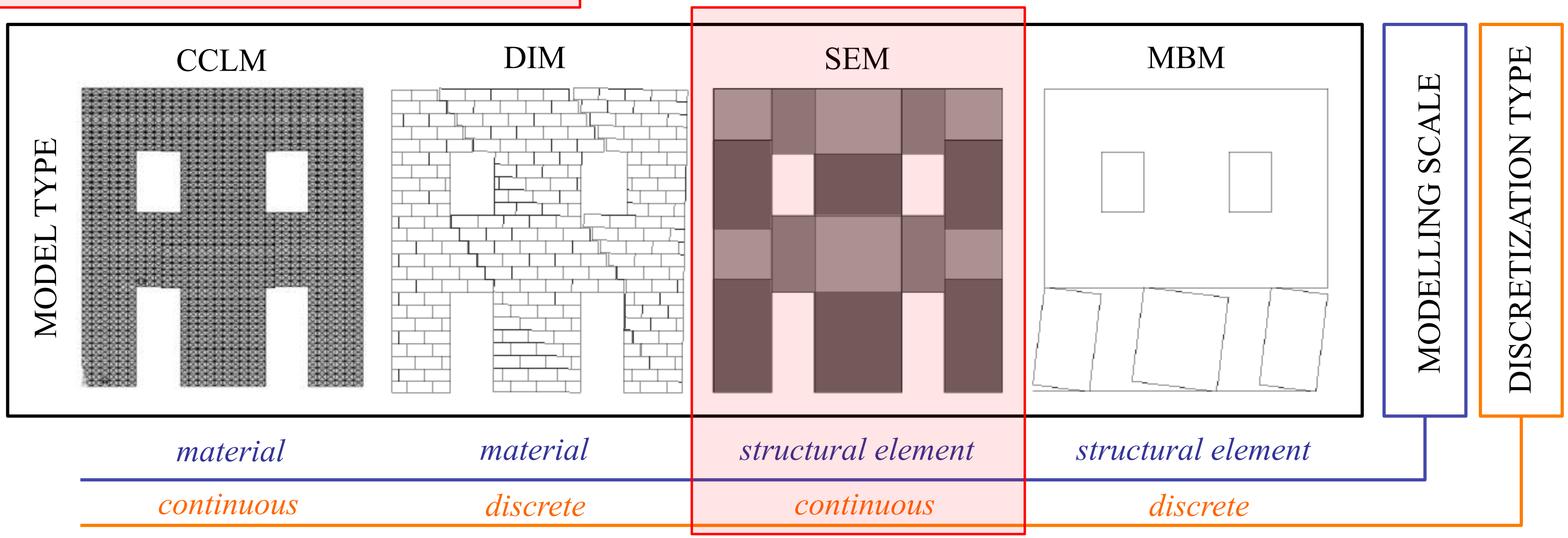
In-plane failure mode:
Global behaviour of the building



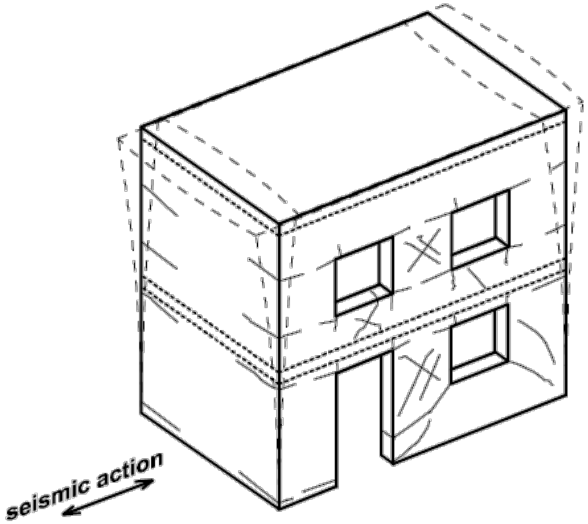
Out-of- plane mechanisms

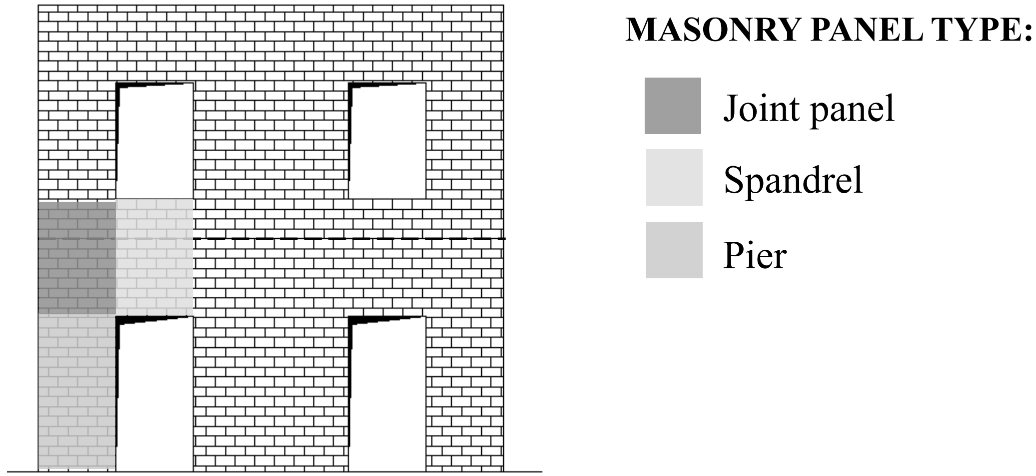


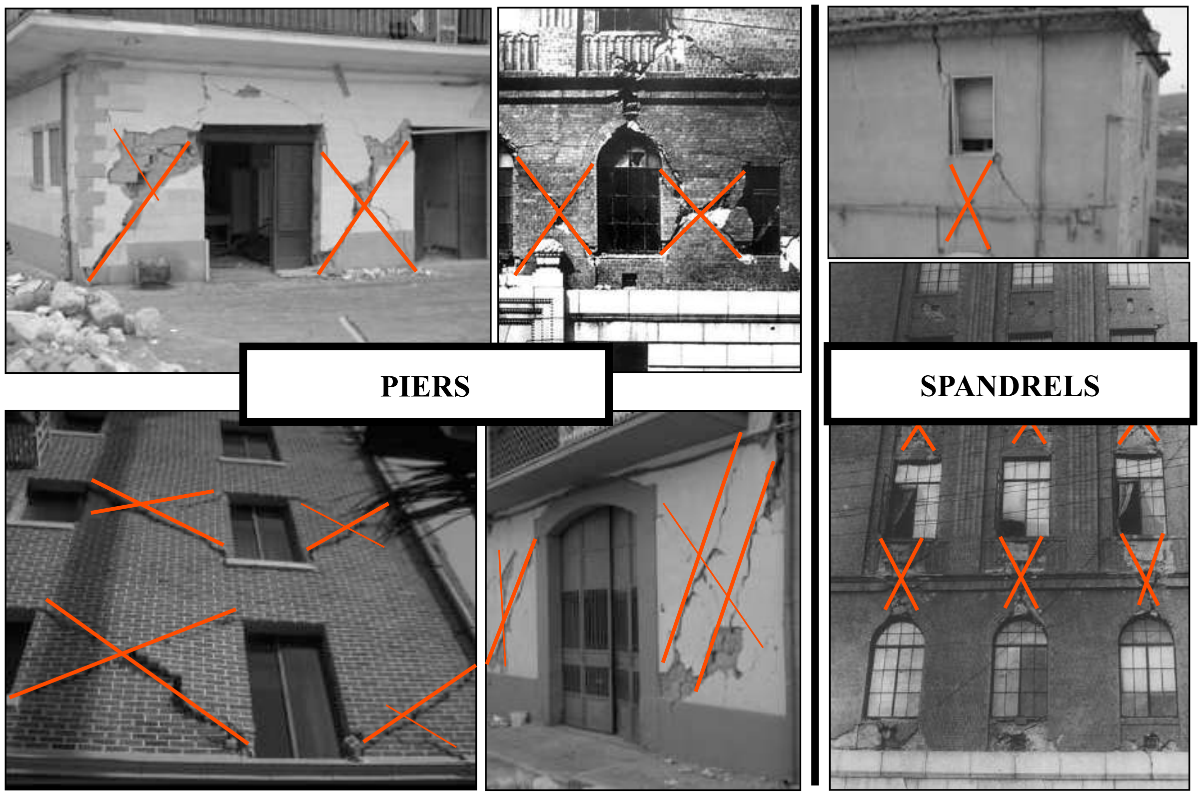
Out-of-plane mechanisms have to be verified separately through suitable analytical methods



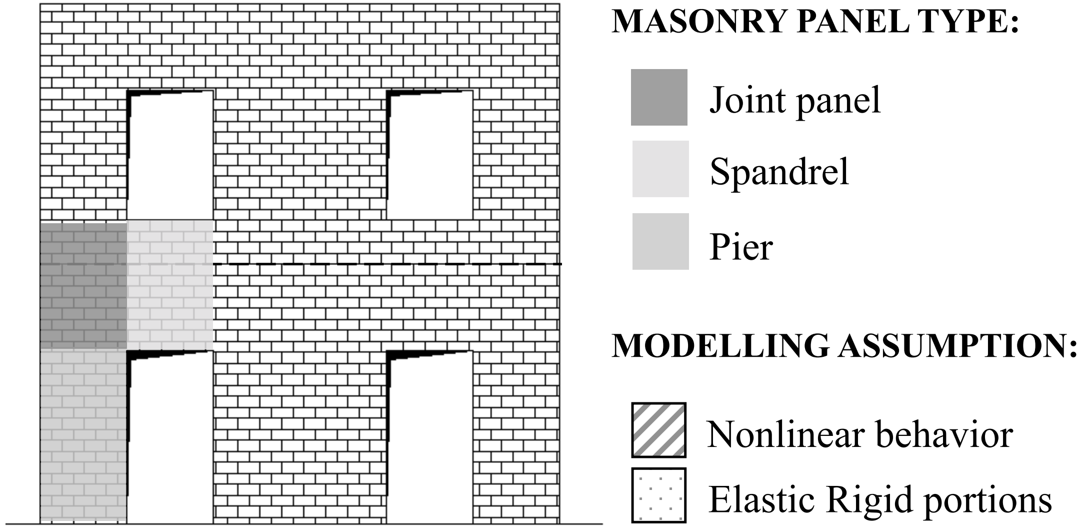
Structural Element Model – *Basics*







Structural Element Model – *Basics*



SIMPLIFIED MODELS

	Joint	*Pier*	*Spandrel*
Stiffness	☐	☑	☐
Non linearity	☐	☑	☐

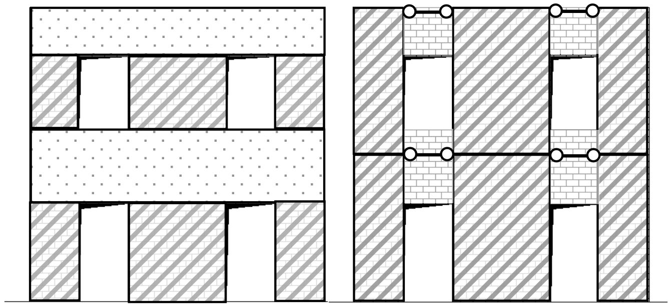
EQUIVALENT FRAME MODELS

	Joint	*Pier*	*Spandrel*
Stiffness	☐	☑	☑
Non linearity	☐	☑	☑

HYBRID MODELS

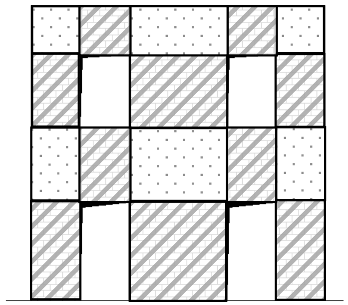
	Joint	*Pier*	*Spandrel*
Stiffness	☑	☑	☑
Non linearity	☑	☑	☑

SSWP **WSSP**

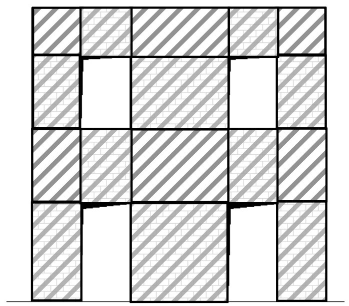


STRONG SPANDREL- WEAK PIER / WEAK SPANDREL-STRONG PIER

FEMA 306 – ASCE 41/13 – POR METHOD (Tomazevic 1978)



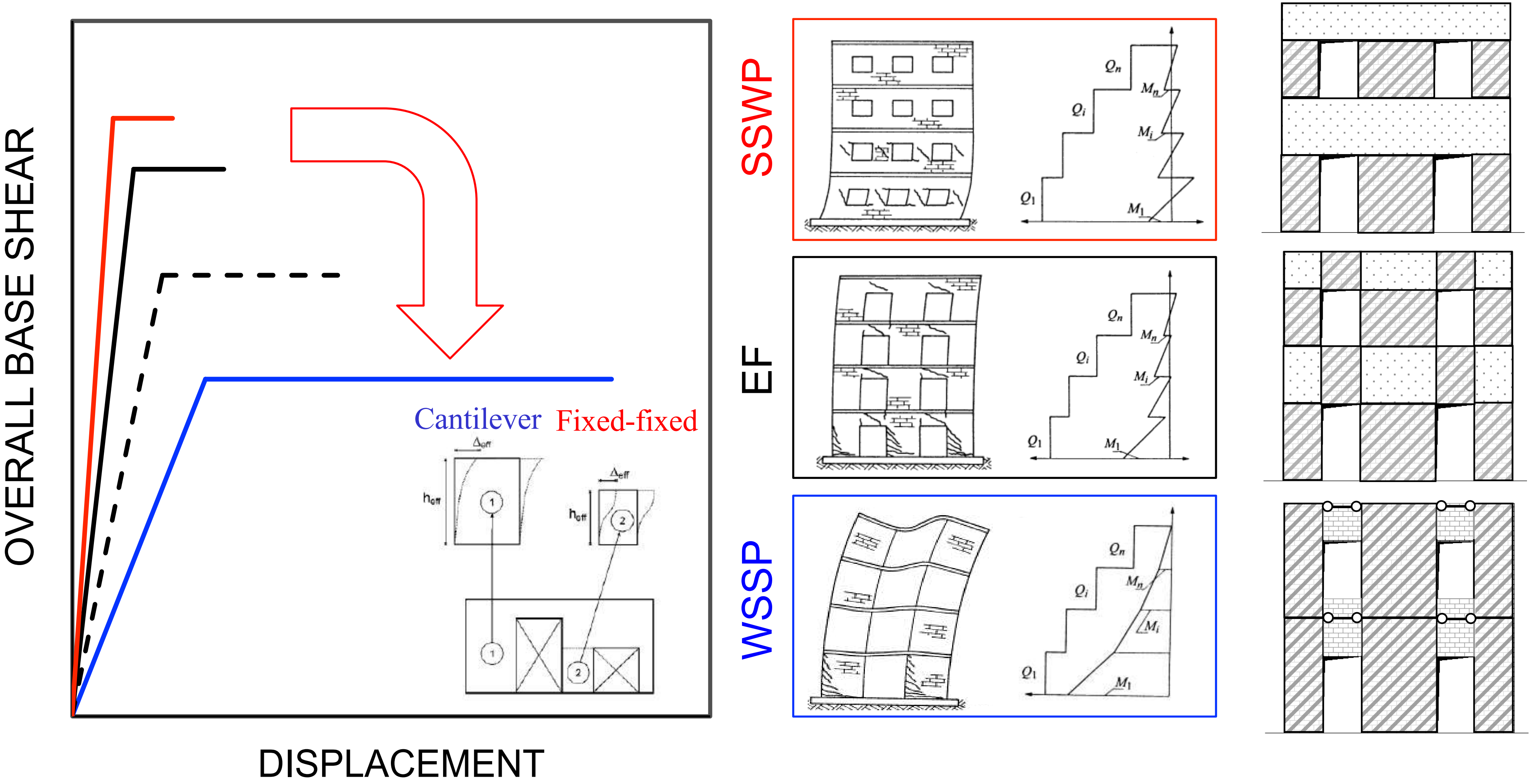
e.g. Tremuri (Lagomarsino et al. 2013) , SAM (Magenes and Della Fontana 1998),easly implementable in purposes software packpages



e.g. 3D Macro (Caliò et al. 2012), D’Asdia and Viskovic (1995), Vanin and Foraboschi (2009), Braga et al. (1998)

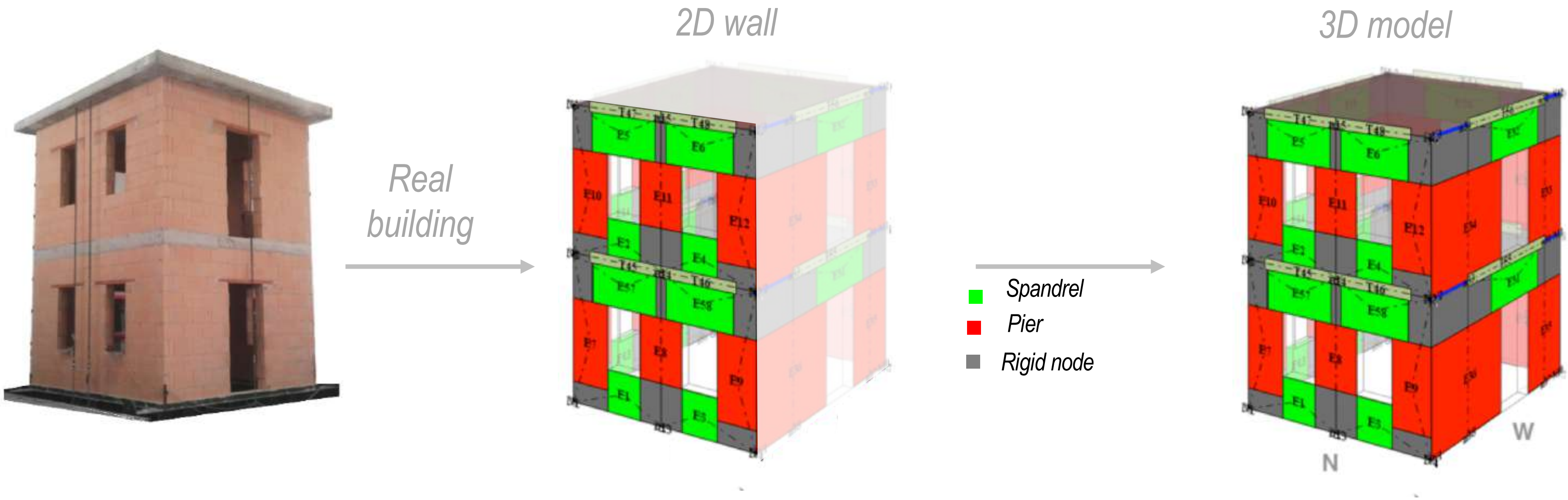
Equivalent Frame Model – *Basics*

Equivalent frame model *versus* simplified analytical mechanical-based models



Equivalent Frame Model – *Basics*

From the ***2D modelling of walls*** to the ***3D modelling of buildings***



The walls are the bearing elements, while diaphragms are the elements governing the sharing of horizontal actions among the walls.

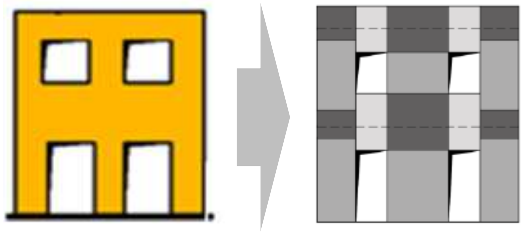
In **MOST** cases the flexural behaviour of the diaphragms and **OFTEN** the wall out-of-plane response are not computed because they are considered negligible with respect to the global building response, which is governed by their in-plane behaviour.

The **MAIN ROLE of diaphrams** is to redistribute and transfer the horizontal force among the walls, both in linear and nonlinear phases

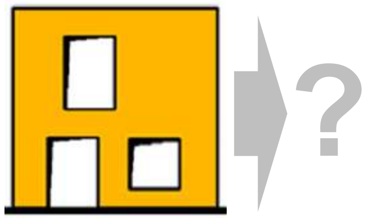
Equivalent Frame Model – *Tricky issues*

2D

Equivalent frame idealization of walls



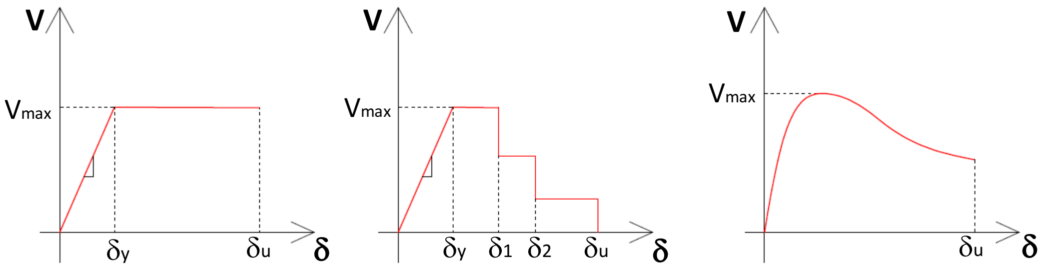
Regular wall



Irregular wall

Constitutive models adopted for masonry panels

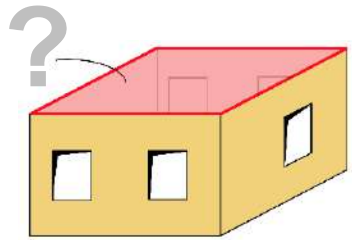
- Strenght criteria to simulate failure criteria
- Backbone curve
- Flexibility in differenting the behavior of spandrels and piers



Possibility of modelling other NON linear elements

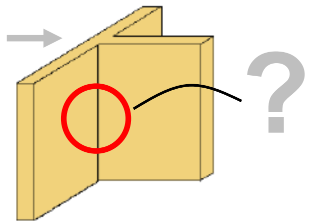
3D

Modelling of diapragms



- Rigid / Stiff /flexible
- Equivalent stiffness

Connection between orthogonal walls

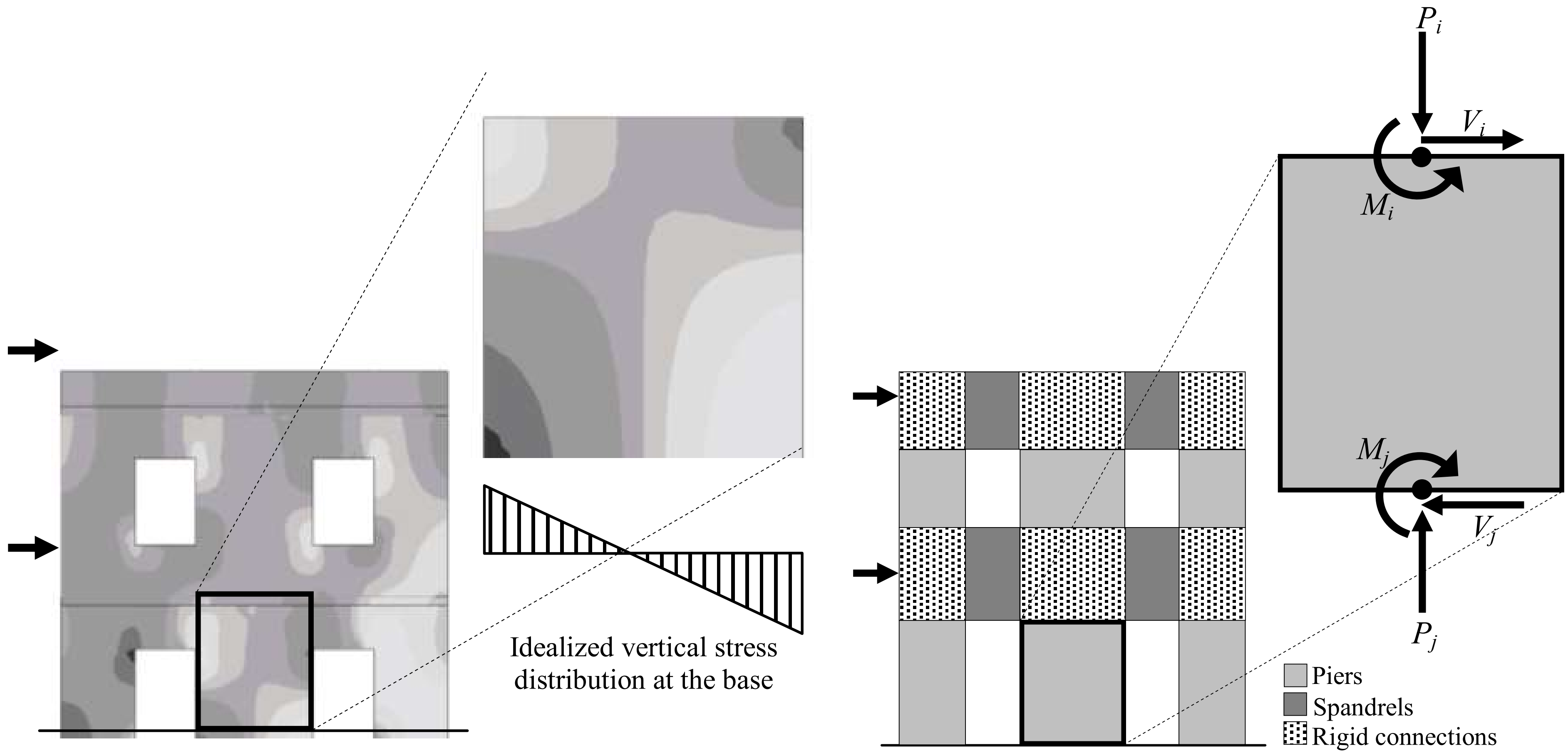


Flange effect

It consists in properly simulating both:

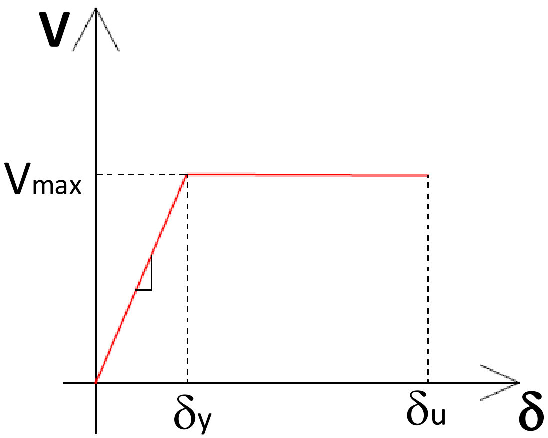
- the wall-to-wall connection quality
- the actual length of the orthogonal pier where it can occur a redistribution of the stress

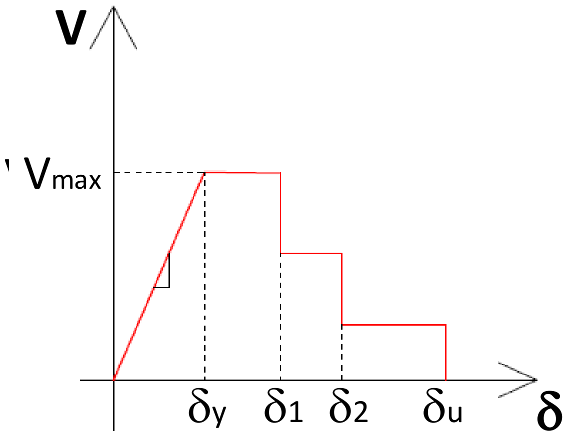
2D Issues: Constitutive models adopted for URM panels

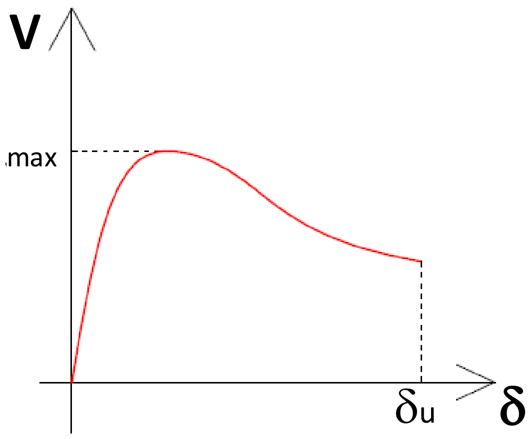


REF: Calderini C., Cattari S., Lagomarsino S. (2009) In-plane strength of unreinforced masonry piers, *Earthquake Engineering and Structural Dynamics*, 38(2), pp. 243-267

2D Issues: Constitutive models adopted for URM panels





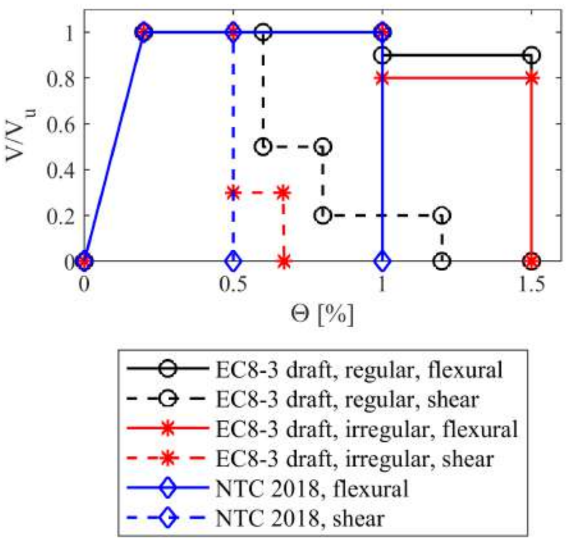


- Italian Structural Code (NTC 2018; MIT 2019)
- Eurocode 8-3

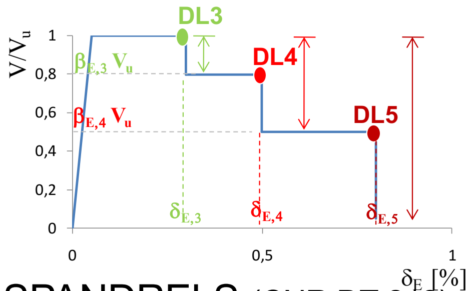
- CNR DT 212 (2013)
- Eurocode 8-3 (updating under review)

- More accurate macroelements

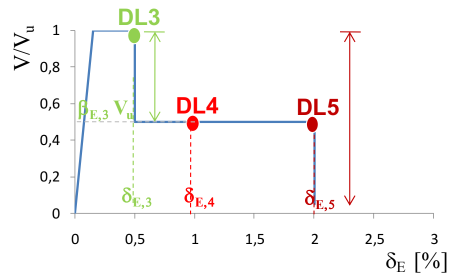
PIERS



PIERS (CNR DT 212)



SPANDRELS (CNR DT 212)



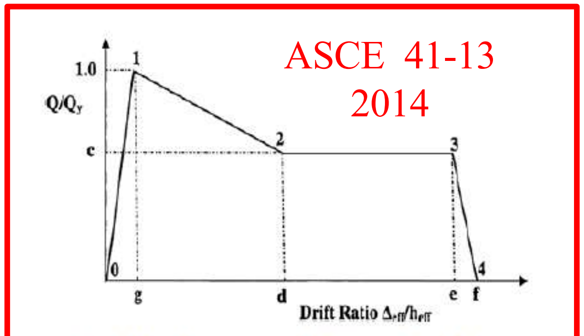
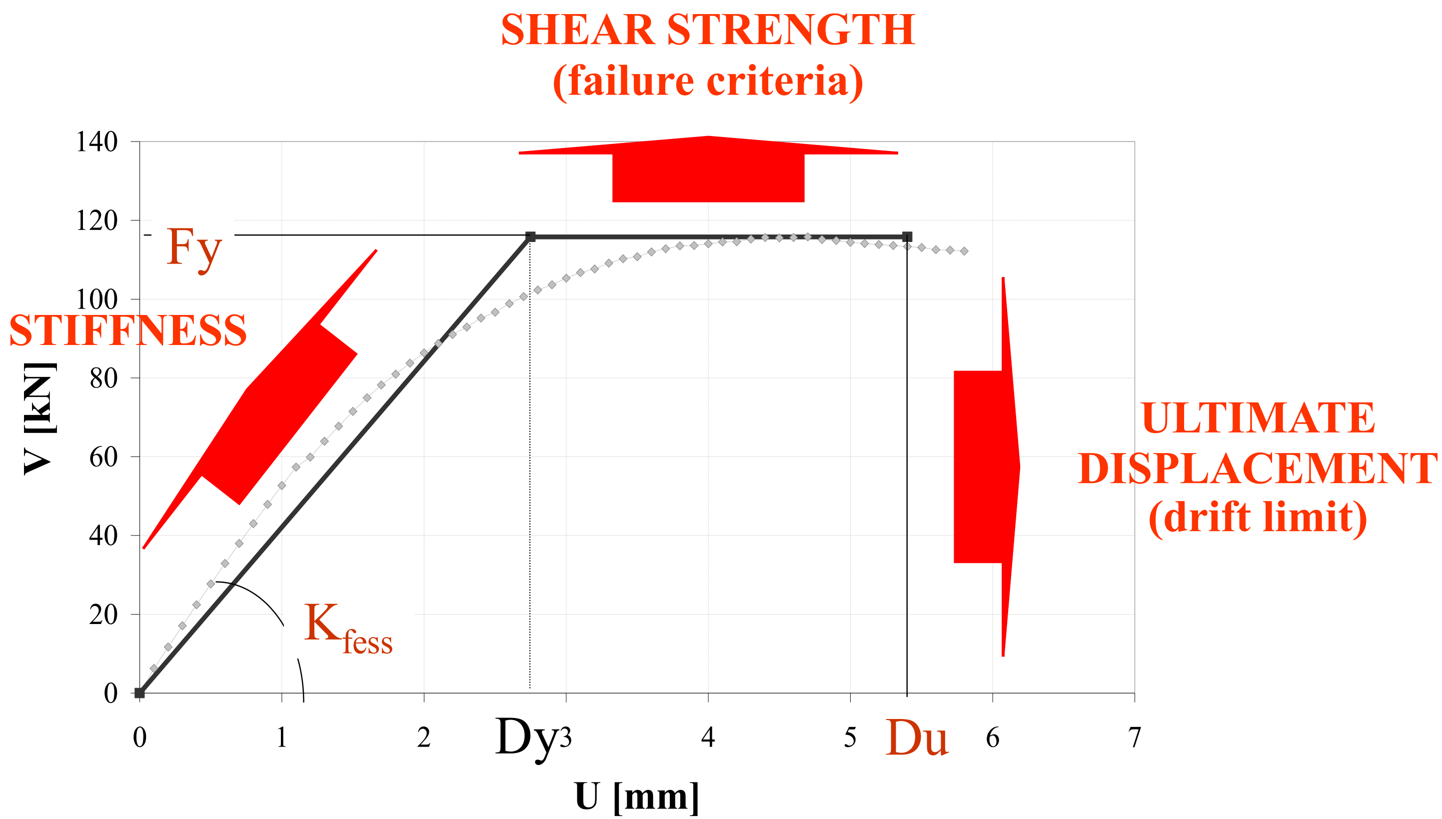


FIG. 11-1(a). Typical Generalized Force–Deformation Relationship for Deformation-Controlled Actions for Reinforced or Unreinforced Masonry

2D Issues: Constitutive models adopted for URM panels

Bilinear relationship

- The force-displacement curve in shear is modelled by a bilinear model, defined by three parameters.
- Different failure modes can occur: ***Rocking*** and ***Crushing*** (flexural behaviour); ***Sliding*** and ***Diagonal Cracking*** (shear behaviour).
- After collapse, the element contribution to the equivalent frame is only related to its capacity of bearing axial loads.



2D Issues: Constitutive models adopted for URM panels

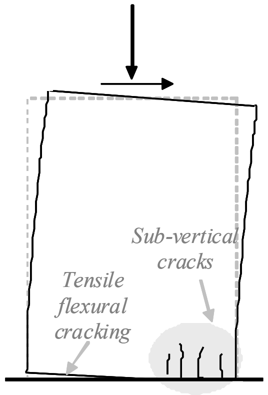
- Shear strength for masonry piers

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*

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

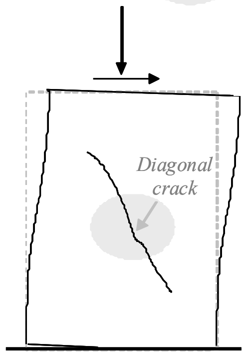


- **SHEAR BEHAVIOUR – PRINCIPAL STRESS MODELS**

Failure modes: *Diagonal Cracking*

(Turnsek and Cacovic, 1971) $$T_u = lt\frac{f_t}{b}\sqrt{1 + \frac{N}{f_t lt}}$$

b = *f* (slenderness of the panel)

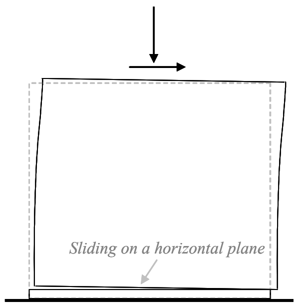


- **SHEAR BEHAVIOUR – COULOMB TYPE MODELS**

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

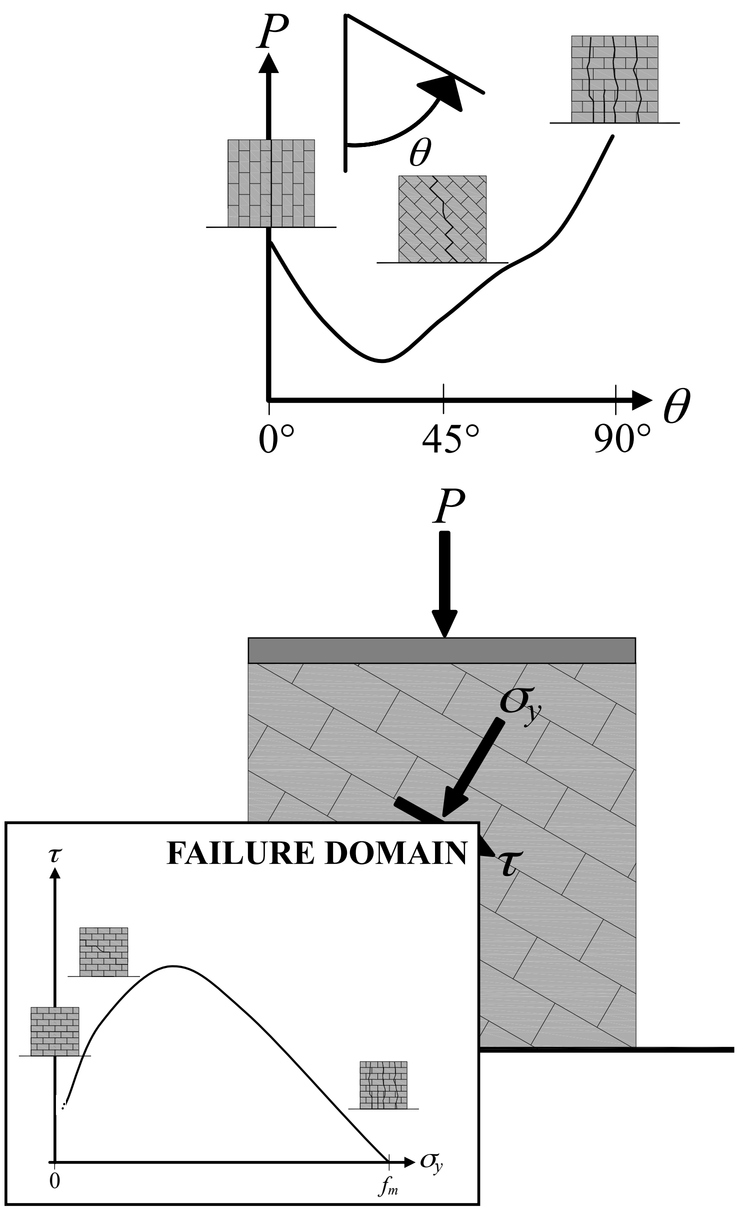
$$\boxed{T_u = l'tc + \mu N}$$

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

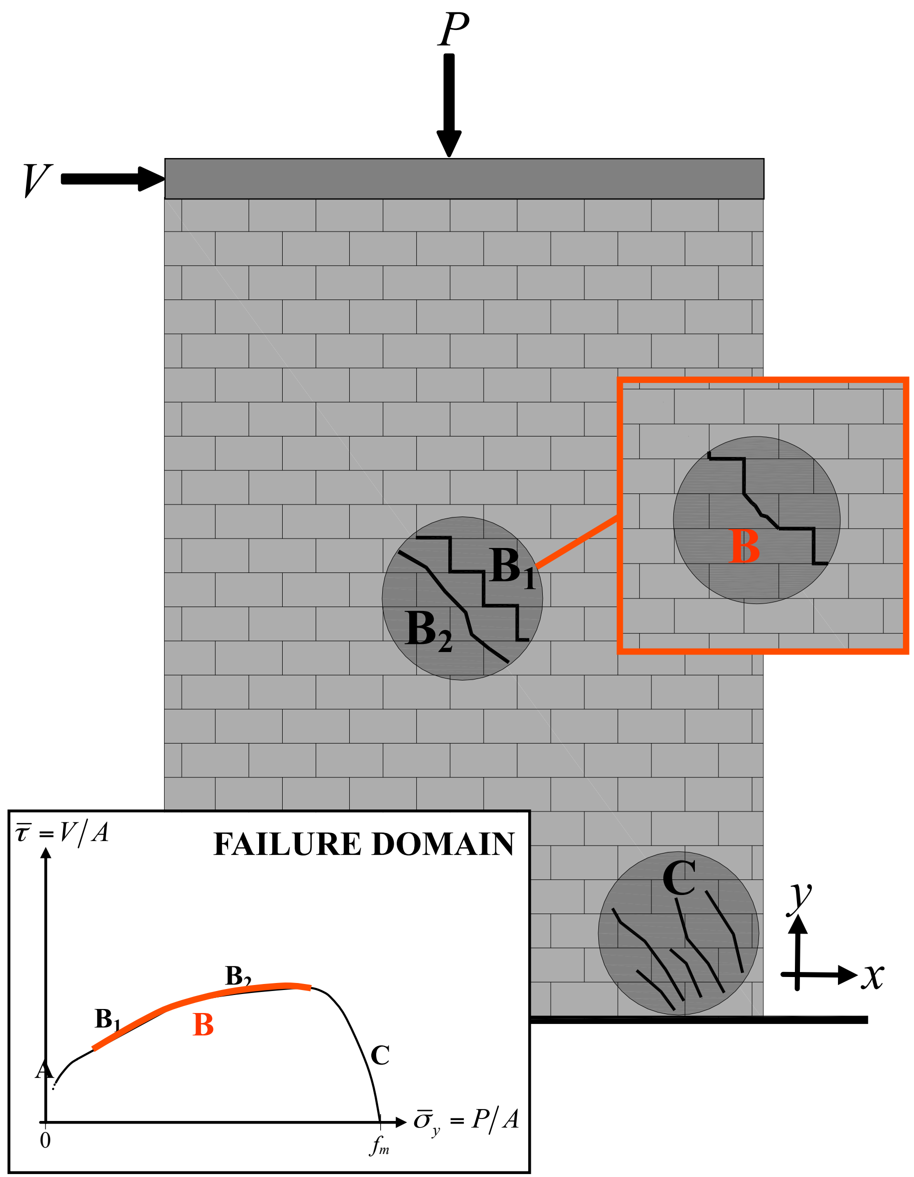


2D Issues: Constitutive models adopted for URM panels

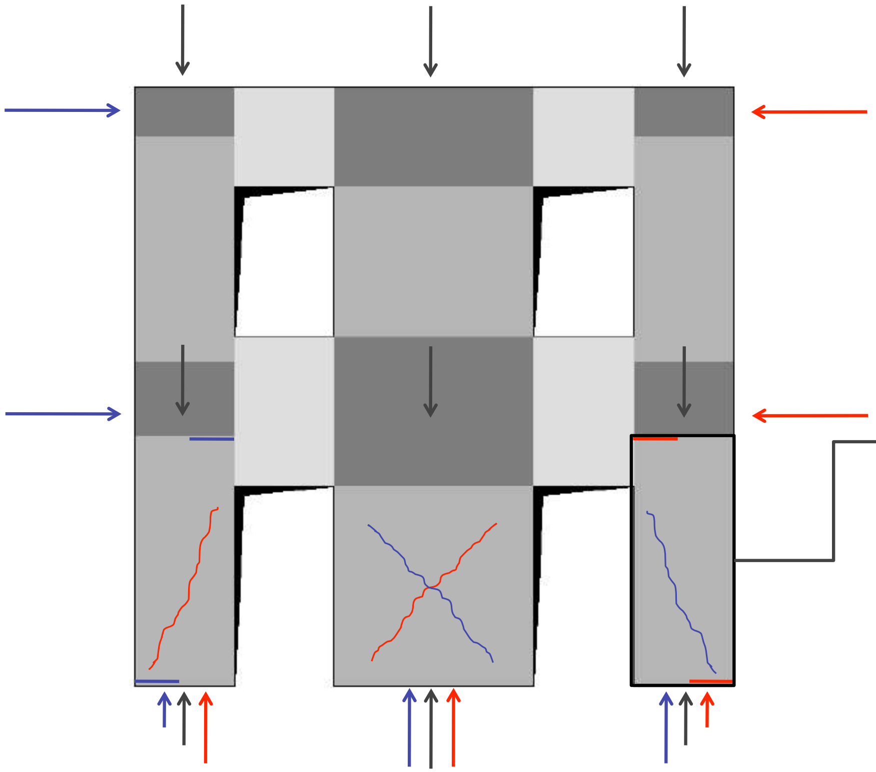
HOMOGENEOUS STRESS STATES

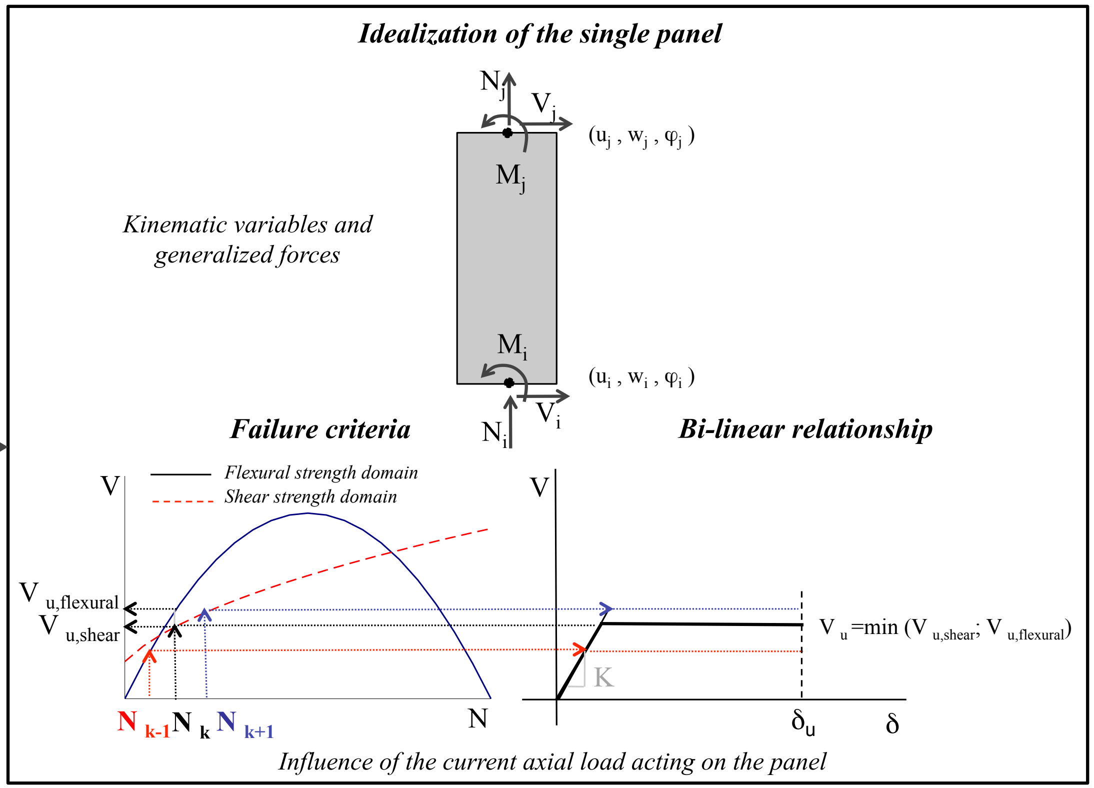


NON-HOMOGENEOUS STRESS STATES



2D Issues: Constitutive models adopted for URM panels





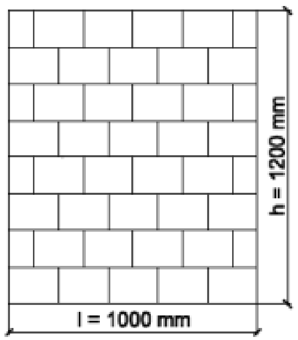
REF:Lagomarsino S, Penna A, Galasco A, Cattari S. (2013) TREMURI program: an equivalent frame model for the nonlinear seismic analysis of masonry buildings, *Engineering Structures*, 56, pp. 1787-1799, Elsevier, ISSN: 0141-0296

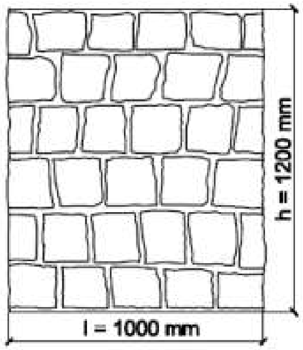
2D Issues: Constitutive models adopted for URM panels

Strength criteria for piers – Trends of recent codes

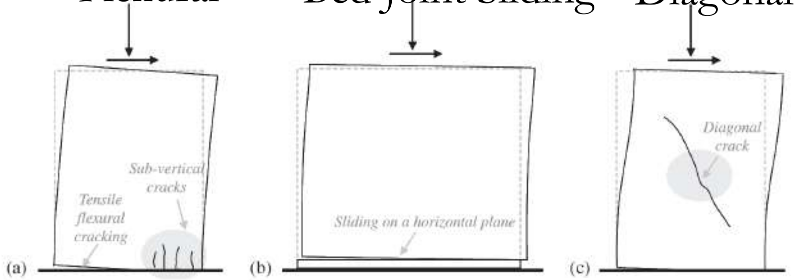
Distinction between REGULAR and IRREGULAR masonry

REGULAR masonry

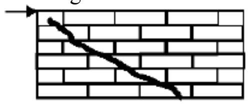




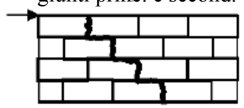
Flexural   Bed Joint Sliding   Diagonal cracking



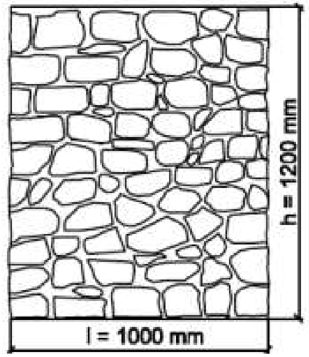
1) Lesione passante tra giunti e blocchi



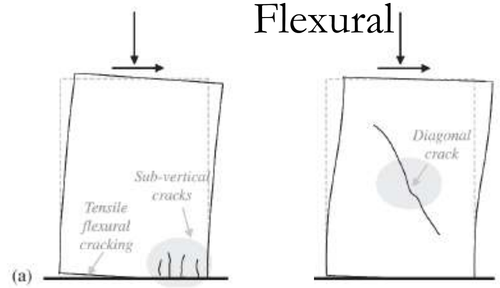
2) Lesione a scaletta sui giunti princ. e second.



IRREGULAR masonry



Flexural



Masonry assumed as isotropic material

2D Issues: Constitutive models adopted for URM panels

Strength criteria for piers

	NTC 2018	Eurocode 8 (draft)
	Rocking	
Regular and irregular masonry	$M_u = \frac{l^2 t \sigma_0}{2}\left(1 - \frac{\sigma_0}{0.85 f_d}\right)$	$V_f = \frac{lN}{2h_0}(1 - 1.15\nu)$
	Diagonal shear[1]	
Regular masonry (Mann & Müller)	$V_d = \frac{lt}{b}\left(\frac{f_{v0}}{1+\mu_j\phi} + \frac{\mu_j}{1+\mu_j\phi}\sigma_0\right) \leq V_{d,lim}$; $V_{d,lim} = \frac{lt}{b}\frac{f_{bt}}{2.3}\sqrt{1 + \frac{\sigma_0}{f_{bt}}}$	
Irregular masonry (Turnsěk & Căcŏvic)	$V_d = \frac{lt}{b} f_t \sqrt{1 + \frac{\sigma_0}{f_t}}$	
	Shear sliding	
Regular masonry	$V_s = l' t f_{vd} = l' t (f_{v0d} + \mu\sigma_0)$	$V_s = l' t \left(f_{v0} + \frac{\mu N}{l' t}\right) \leq V_{s,units}$

(1) for the Eurocode 8, the Italian code uses the design parameters f_{btd}, f_{td}, f_{v0d} rather than f_{bt}, f_t, f_{v0}.

2D Issues: Constitutive models adopted for URM panels

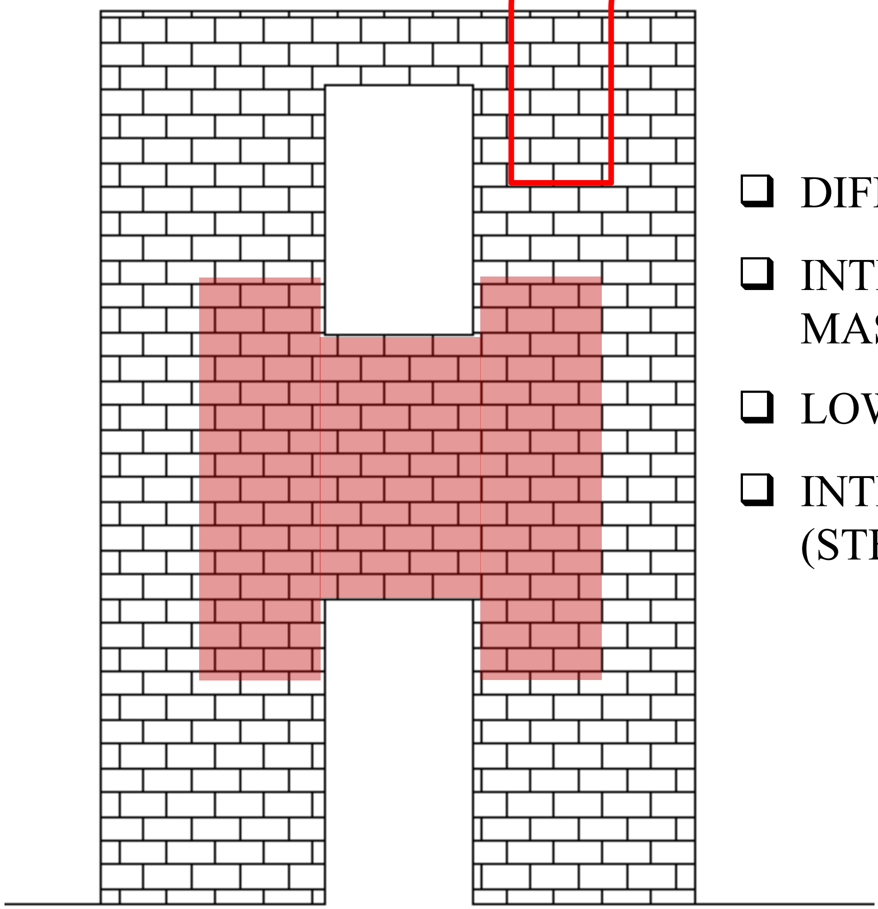
Strength criteria for spandrels

	NTC 2018	Eurocode 8 (draft)
	Flexural mechanism[1]	
Regular and irregular masonry	$M_u = N_{sd}\frac{h}{2}\left[1 - \frac{N_{sd}}{0.85 f_{hd} h t}\right]$ $V_f = \frac{2M_u}{l}$	$V_{f,u} = 1.15\frac{h^2 t}{6h_0} f_{ht}$ $V_{f,s} = \frac{h^2 t}{2h_0(1 + f_{ht}/f_h)} f_{ht}$ $V_{f,d} = \frac{hN_s}{h}\left(1 - 1.15\frac{N_s}{ltf_h}\right)$
	Shear mechanism[2]	
Regular masonry	$V_d = \frac{ht}{b}\left(\frac{f_{v0}}{1+\mu_j\phi} + \frac{\mu_j}{1+\mu_j\phi}\sigma_0\right) \leq V_{d,lim}$ $V_{d,lim} = \frac{ht}{b}\frac{f_{bt}}{2.3}\sqrt{1 + \frac{\sigma_0}{f_{bt}}}$	
Irregular masonry	$V_d = \frac{ht}{b} f_t \sqrt{1 + \frac{\sigma_0}{f_t}}$	

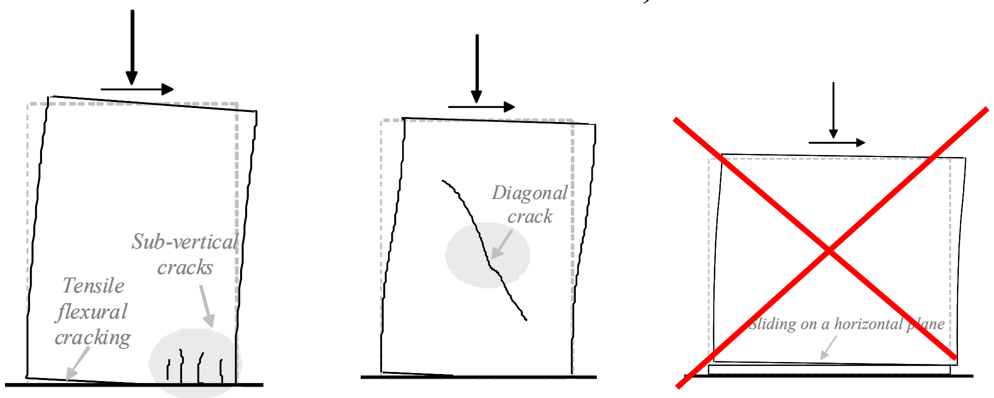
[1] also Comm (2019) allows to take into account the horizontal tensile strength of masonry, however no close form equation to evaluate the strength are provided;
[2] for the Eurocode 8, the Italian code uses the design parameters f_{btd}, f_{td}, f_{v0d} rather than f_{bt}, f_t, f_{v0}.

2D Issues: Constitutive models adopted for URM panels

- Shear strength for masonry spandrels



- ❑ DIFFERENT ORIENTATION OF MAIN JOINTS ACTIVATED
- ❑ INTERLOCKING AT THE END SECTION WITH ADIACENT MASONRY PORTIONS
- ❑ LOW AXIAL FORCES
- ❑ INTERACTION WITH LINTEL/ARCHITRAVE TYPE (STEEL/TIMBER BEAMS – FLAT ARCH)



2D Issues: Constitutive models adopted for URM panels

Diagonal cracking failure mode

L'Aquila Earthquake – 6/04/2009

Existing buildings: Due to the moderate value of axial load acting on spandrels or to the lack of tie-rods, if the same criterion proposed for piers for the flexural response is used, ***Rocking*** tends to prevail on ***Diagonal Cracking*** much more frequently than that observed in existing buildings after an earthquake

This means that masonry spandrels have a higher flexural strength

2D Issues: Constitutive models adopted for URM panels

✓ **Experimental campaign carried out at the University of Trieste – Gattesco et al. 2008**

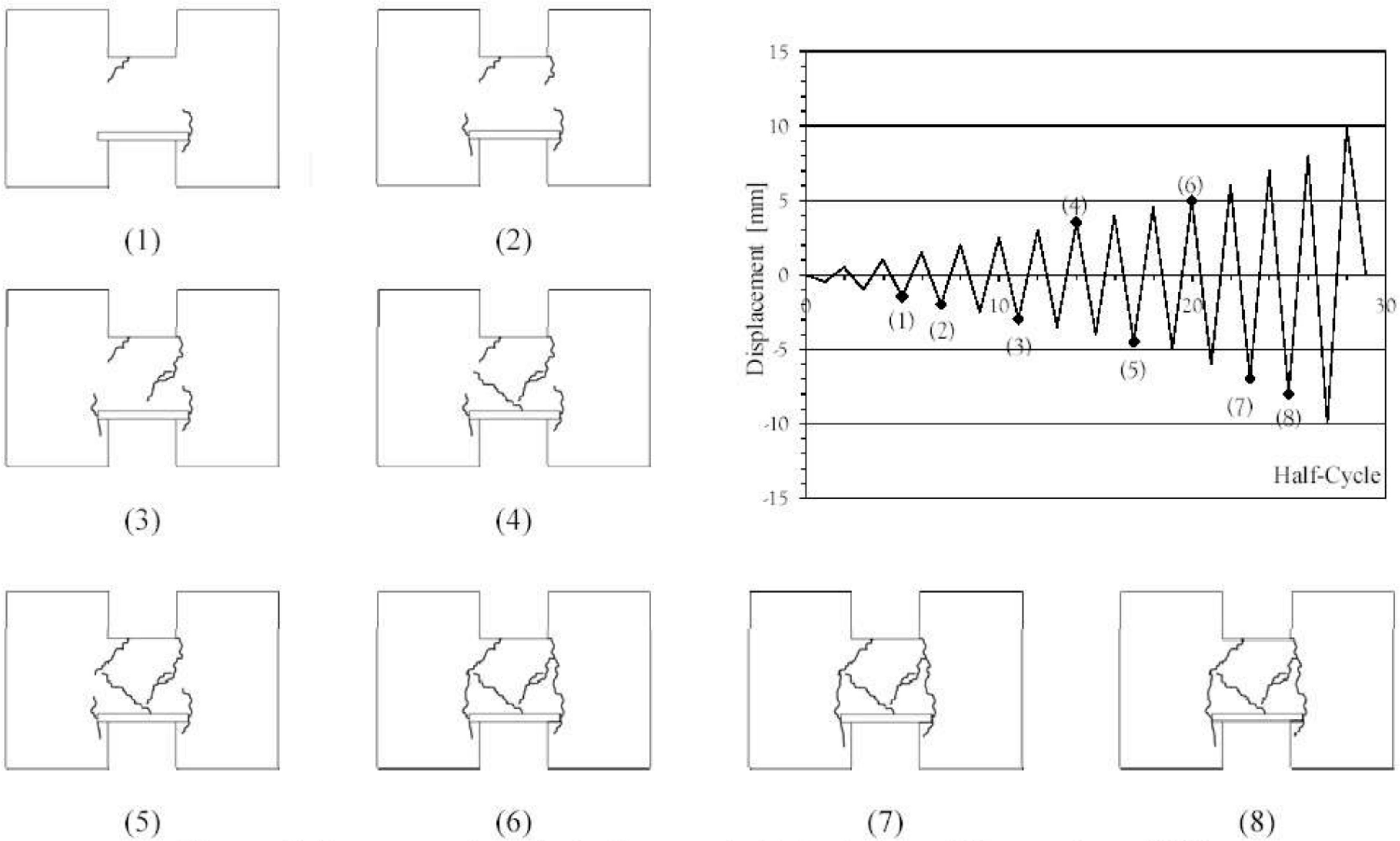
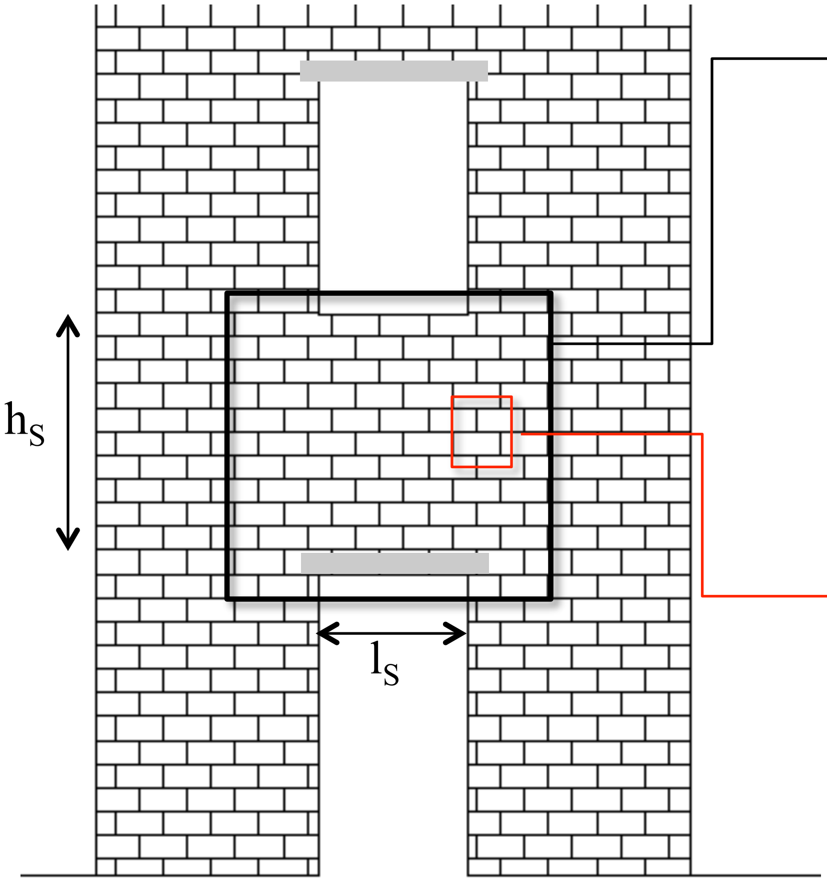


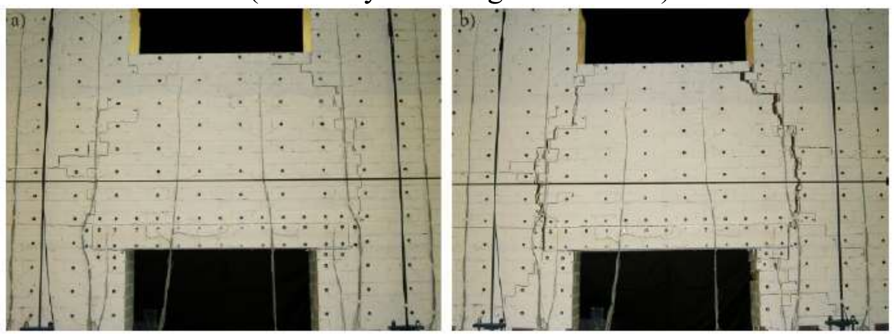
Figure. 7. Sequence of cracks in the spandrel's back face of the specimen MS1.

2D Issues: Constitutive models adopted for URM panels

$$f_{ftd} = min\left(\frac{f_{btd}}{2}; f_{v0d} + \frac{\mu\sigma_y}{\Phi}\right)$$

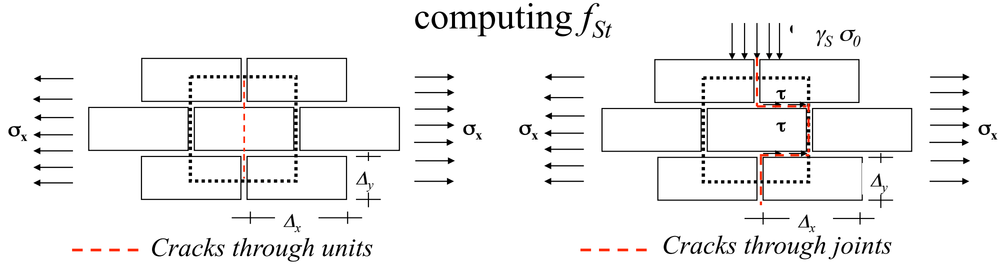


Example of flexural craking of spandrels
(from Beyer&Mangalathu 2013)



Cracks through joints

Basic failure modes assumed in Cattari and Lagomarsino (2008) for computing f_{St}

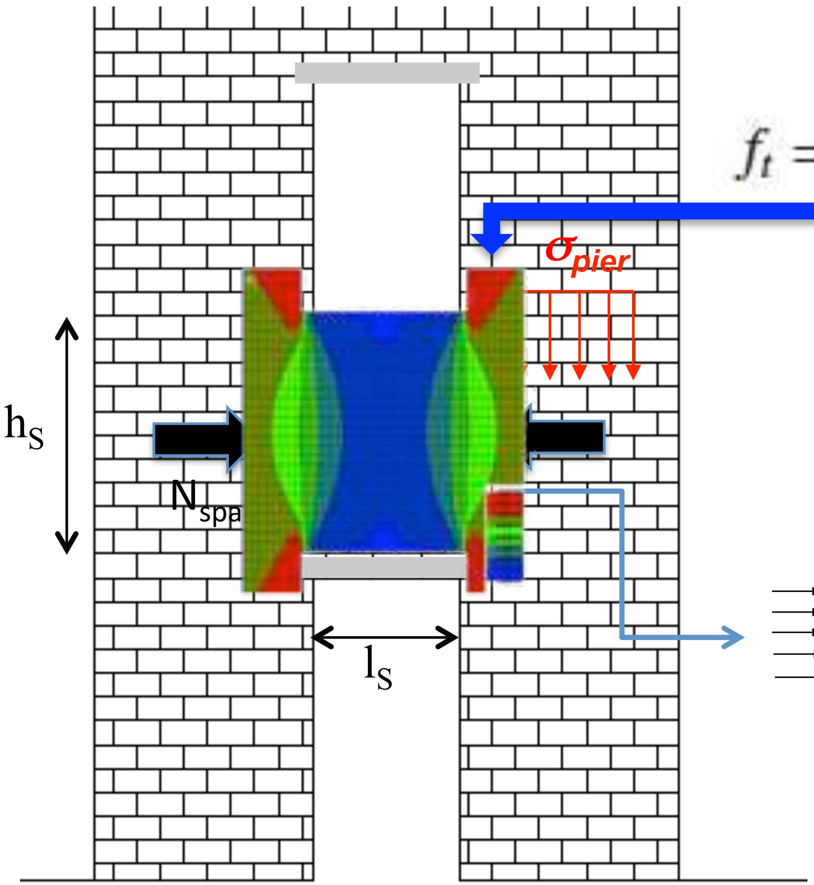


$f_{St} = f_{bt}/2$ $\qquad$ $f_{St} = \gamma_S\ \sigma_0\ \mu\ \Delta_x / (2\Delta_y)$

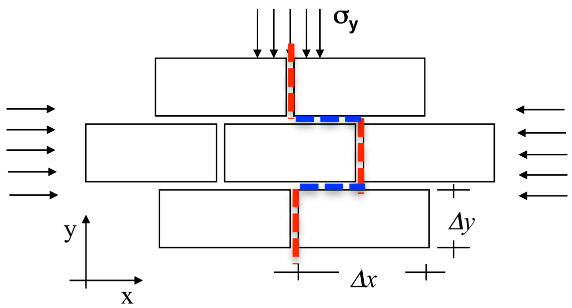
REF : Beyer K., Mangalathu S. 2013. Review of strength models for masonry spandrels. *Bull Earth Eng*,11. 521–542.

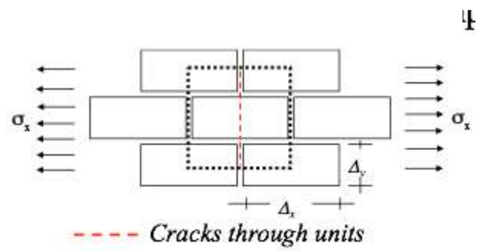
2D Issues: Constitutive models adopted for URM panels

Thanks to the interlocking phenomena, spandrels may rely on the contribution of an "equivalent tensile strength" $f_{t,spandrel}$



$$f_t = \underbrace{(\mu \overline{\gamma}_{0.5}\sigma_p + c)\frac{l_b}{2(h_b+h_j)}}_{\text{Horizontal mortar joints}} + \underbrace{\frac{c}{2\mu}}_{\text{Vertical mortar joints}}$$





Also the failure of block has to be considered !!

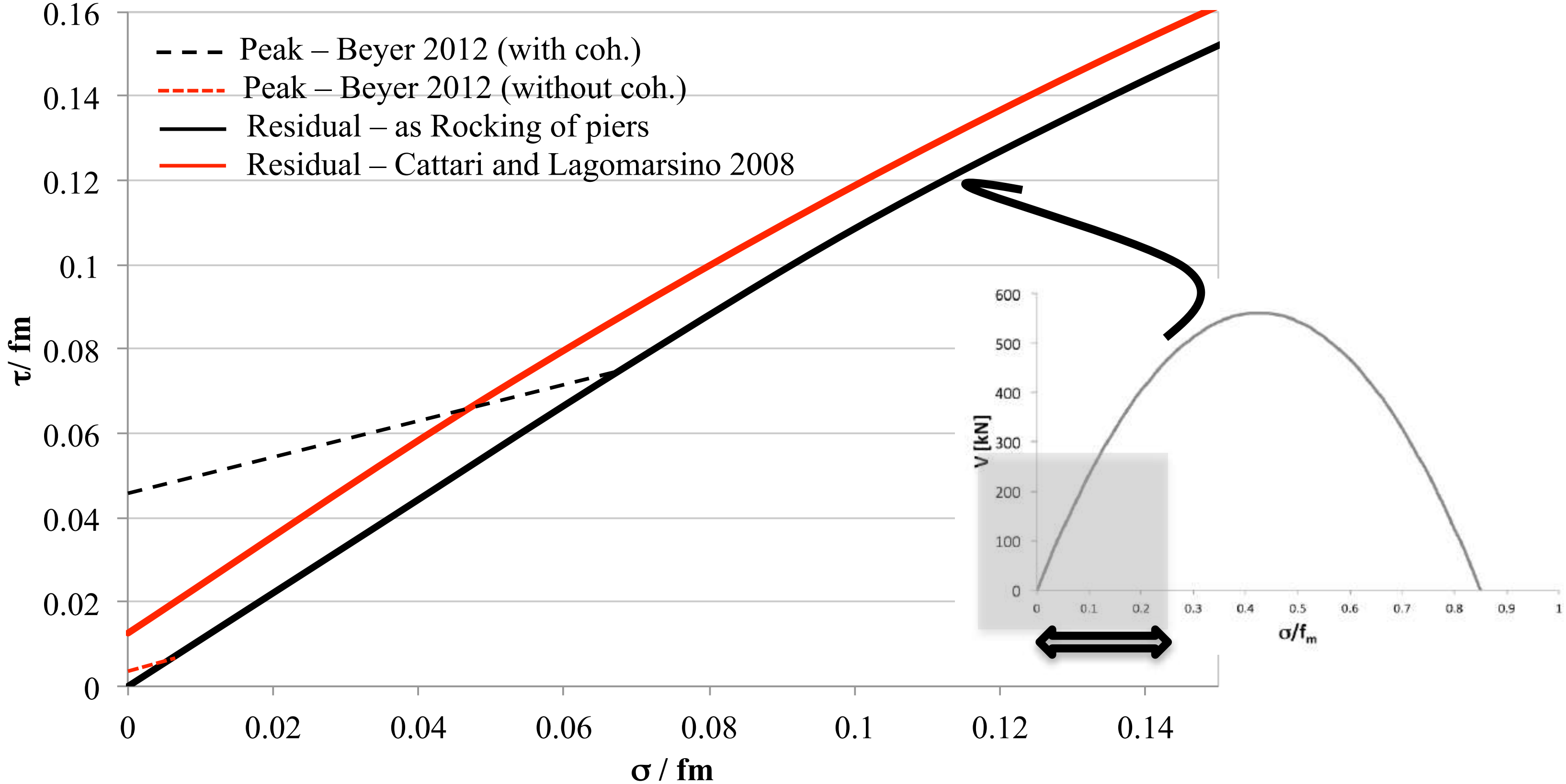
REF : Beyer K., Mangalathu S. 2013. Review of strength models for masonry spandrels. *Bull Earth Eng*,11. 521–542.

REF. Cattari S., Beyer K. (2015) Influence of spandrel modelling on the seismic assessment of existing masonry buildings, in: Proc. of Tenth Pacific Conference on Earthquake Engineering: Building an Earthquake-Resilient Pacific, 6-8 November 2015, Sydney, Australia, 10 pages.

2D Issues: Constitutive models adopted for URM panels

Different proposals in literature: FEMA 306 1998, Cattari and Lagomarsino 2009, Beyer 2012.

	FEMA 306	Cattari and Lagomarsino2009	Beyer 2012
$f_{t,sp}$	$f_{p,sj} = 0,5\,(0,75c)$ $f_{p,bj} = 0,5(0,75c + \gamma_{sp}\sigma_p)$	$f_{tu} = \min\left(\mu\gamma_{sp}\sigma_p \frac{l_b}{2(h_j + h_b)}; \frac{f_{bt}}{2}\right)$	$f_{bj} = (\mu\ \gamma_{sp}\sigma_p + c)\frac{l_b}{2(h_b + h_j)}$ $f_{hj} = \frac{c}{2\mu}$; $f_{t,sp} = f_{bj} + f_{hj}$

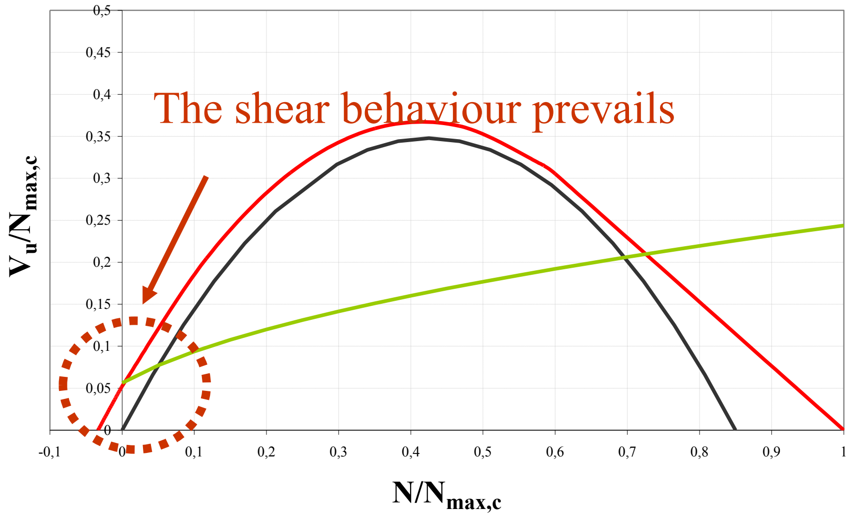


NOW INTRODUCED IN THE ITALIAN STRUCTURAL CODE 2018 &MIT 2019 AND IN EUROCODE 8- PART (DRAFT UNDER REVIEW)

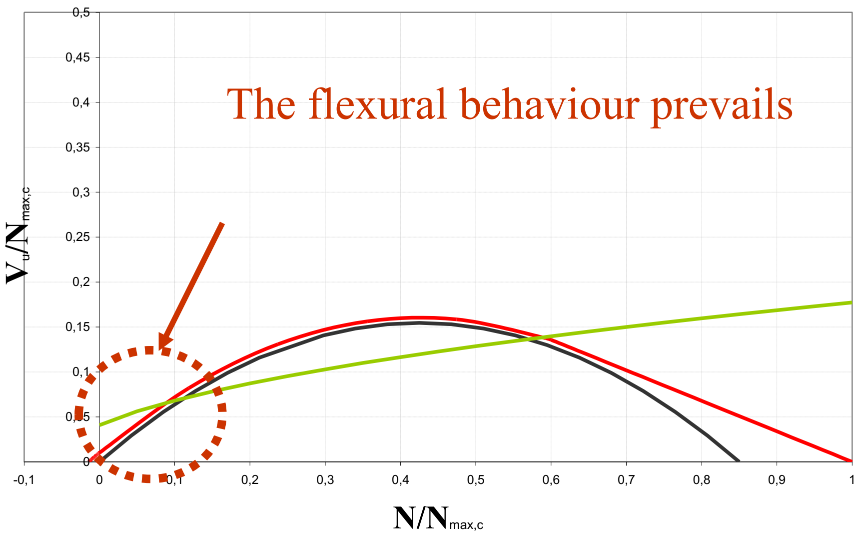
2D Issues: Constitutive models adopted for URM panels

(*Cattari and Lagomarsino 2008, in proceeding of 14th WCEE, Beijing*).

SPANDREL – INTERMEDIATE FLOOR



SPANDREL – TOP FLOOR

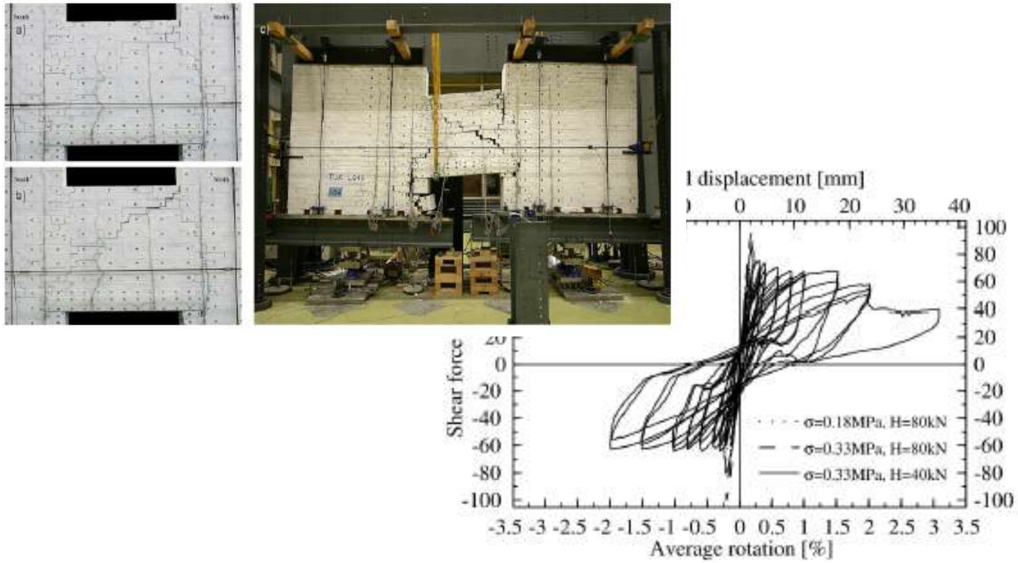


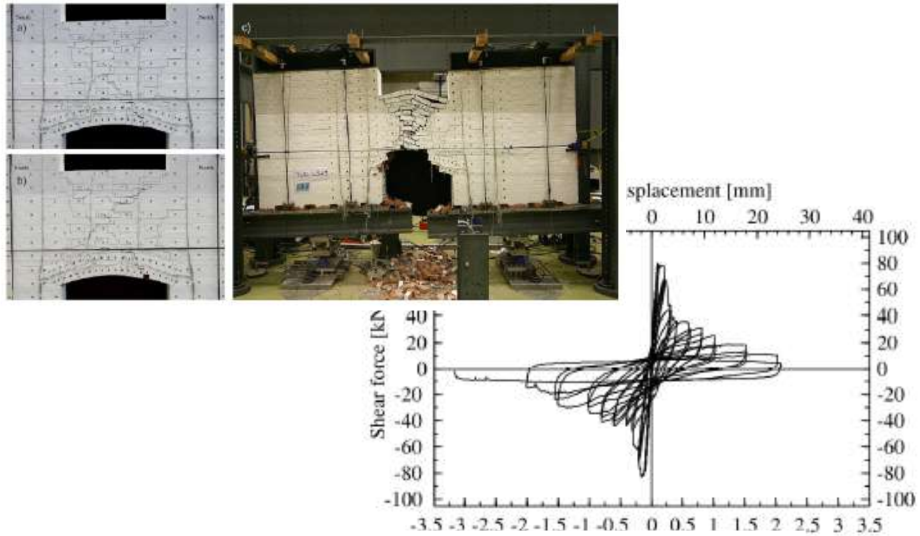
San Giuliano di Puglia 2002

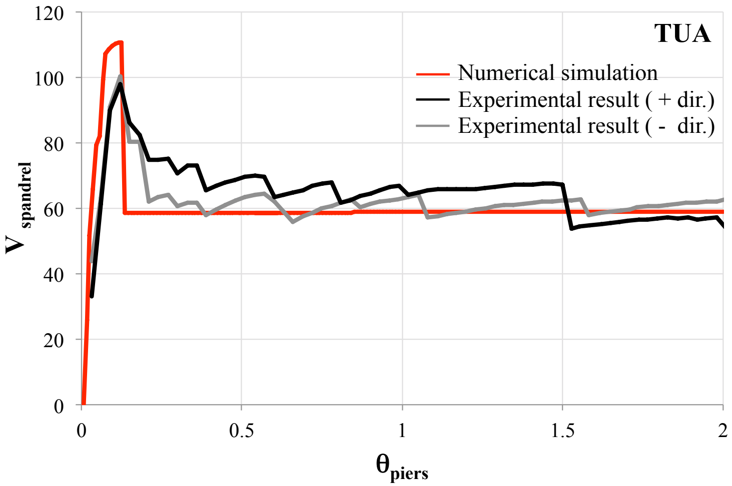
2D Issues: Constitutive models adopted for URM panels

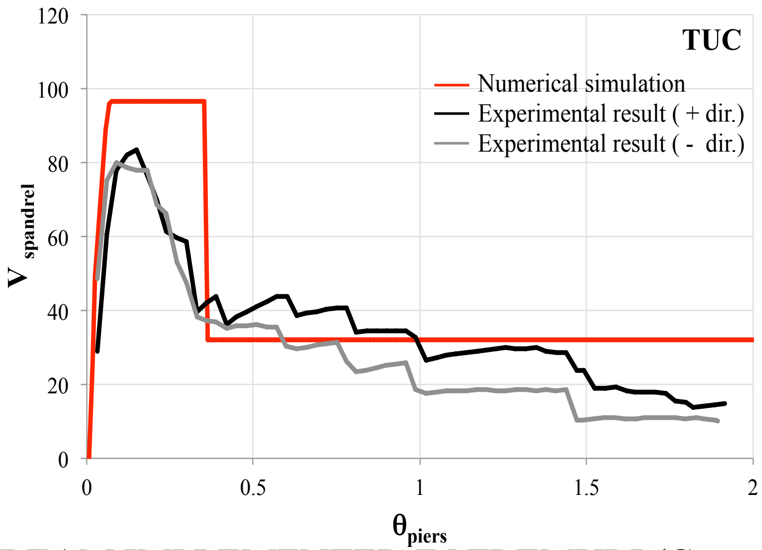
- RESIDUAL STRENGTH OF SPANDREL AS A FUNCTION ALSO OF THE ARCHITRAVE TYPE

EXPERIMENTAL RESULTS FROM BEYER&DAZIO 2012 & STRENGTH CRITERIA PROPOSED IN BEYER 2012 (Engineering Structures)







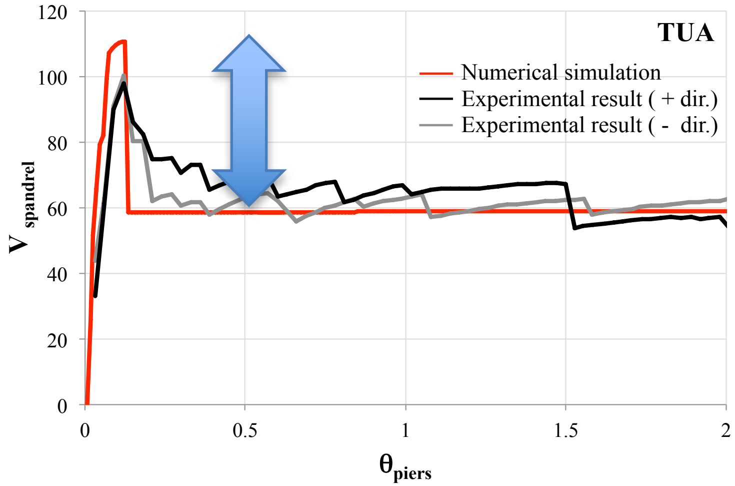


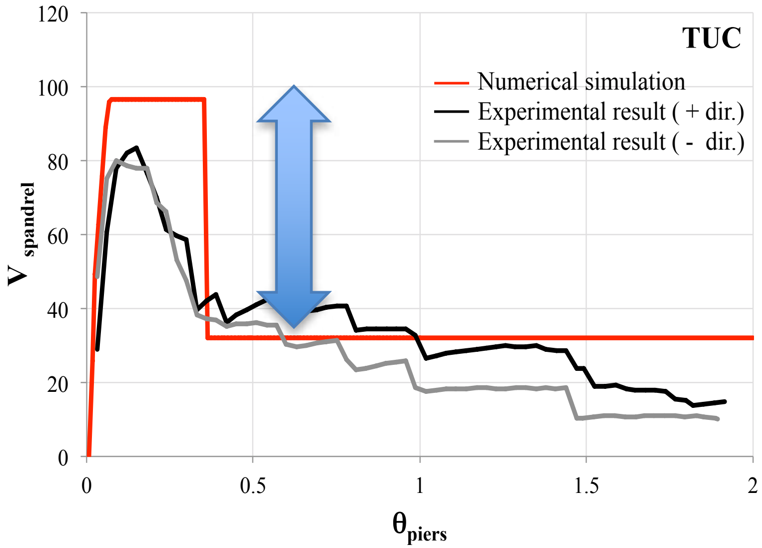
NUMERICAL SIMULATION PROVIDED BY MULTILINEAR BEAM IMPLEMENTED IN TREMURI (Cattari and Beyer 2015, PCEE conference)

2D Issues: Constitutive models adopted for URM panels

SPANDRELS

ITALIAN STRUCTURAL CODE – NTC 2018 & INSTRUCTIONS ISSUED IN 2019





RESIDUAL STRENGTH AS A FUNCTION OF THE LINTEL TYPE

60% when the spandrel is coupled to a r.c. beam or steel lintel

40% when the spandrel is coupled to a timber lintel

10% in presence of a flat arch as lintel

HIGHER DRIFT LIMITS THAN PIERS

Ultimate drift equal to 0.015 (increased until to 0.02 when a tensile resistant element is coupled to the spandrel)

2D Issues: Constitutive models adopted for URM panels

DEFINITION OF DRIFT THRESOLDS

REF: CNR DT 212 (2013)

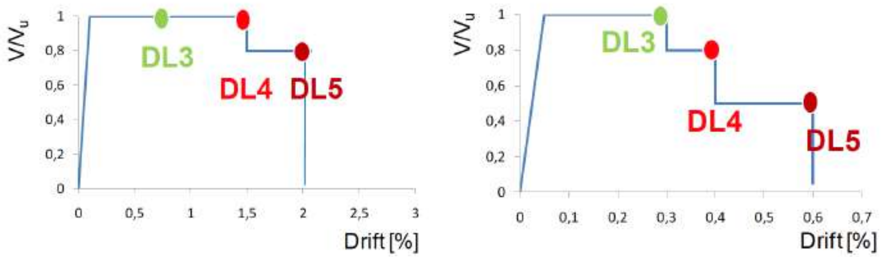


Table 3.2. Indicative intervals for drift values and for the residual resistance for the different states of damage.

	Drift (%)			Residual resistance	
damage	3	4	5	3→4	4→5
normal force and bending	0.4 ÷ 0.8	0.8 ÷ 1.2	1.2 ÷ 1.8	1.0	0.8 ÷ 0.9
shear	0.25 ÷ 0.4	0.4 ÷ 0.6	0.6 ÷ 0.9	0.6 ÷ 0.8	0.25 ÷ 0.6

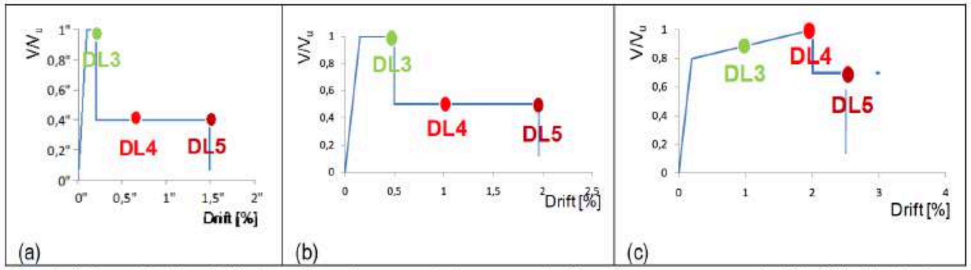


Fig. 3-4. Constitutive drift-shear laws for spandrels supported by: a) a masonry arch; b) a lintel in timber, steel or reinforced concrete; c) a lintel coupled to a tensile-resistant element.

damage	Drift (%)			Residual resistance	
	3	4	5	3→4	4→5
Lintel without tie rod or tie beam	0.40 ÷ 0.60	0.80 ÷ 1.20	1.80 ÷ 2.20	0.40 ÷ 0.60	
Lintel with tie rod or tie beam	0.80 ÷ 1.20	1.60 ÷ 2.00	2.40 ÷ 2.60	1.00	0.60 ÷ 0.80
Arch	0.15 ÷ 0.25	0.45 ÷ 0.75	1.20 ÷ 2.00	0.30 ÷ 0.50	
Arch with Tie rod	0.20 ÷ 0.40	0.50 ÷ 0.80	1.40 ÷ 2.40	1.00	0.40 ÷ 0.60

DATABASE OF EXPERIMENTAL TESTS ONLY FOR PIERS

REF: Vanin et al. (2017, *Bulletin of Earthquake Engineering*)

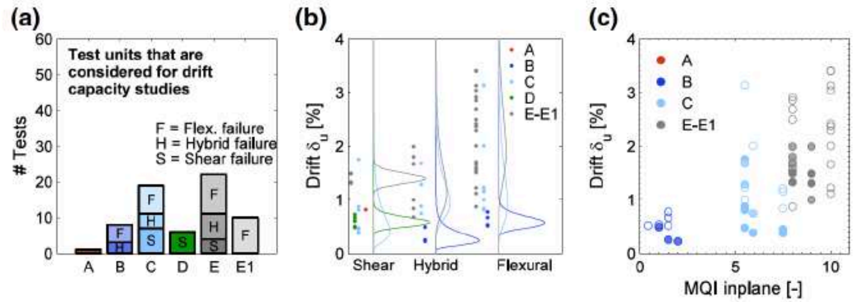
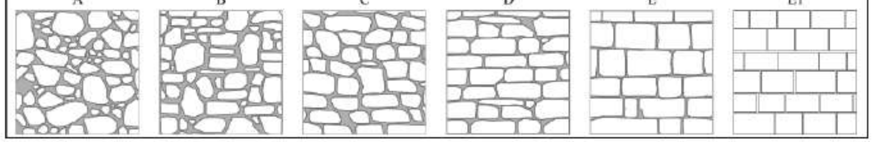


Fig. 15 Distribution of test units considered for drift capacity studies (**a**). Ultimate drift capacity δ_u as a function of the failure mode (**b**), and the Masonry Quality Index for in-plane loading (**c**). *Solid markers* represent walls failing in shear of a hybrid mode; *empty markers* walls failing in flexure

Table 15 Summary of proposed median values and coefficients of variations for stone masonry assessment

MASONRY TYPOLOGIES



DISPLACEMENT CAPACITY

Drift at cracking: $\delta_{cr} = 0.20\%$ — Drift at SD limit state $\delta_{SD} = 0.50 \cdot \delta_u$

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

- shear $\delta_y = 1/4 \cdot \delta_u$ — Drift at collapse $\delta_c = 1.15 \cdot \delta_u$
- flexure $\delta_y = 1/6.5 \cdot \delta_u$

	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

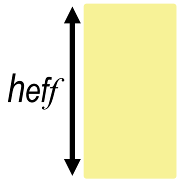
Ultimate drift:
- Model 1: reference values from table
- Model 2: $\delta_u = \max(1.5\% - 4\% \cdot \frac{\sigma_{0,tot}}{f_c}, 0.3\%) \cdot \frac{H_0}{\min(H,L)}$ (typologies A-B-C-D)

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

ALEATORIC VARIABILITY	Suggested coefficients of variation							
	K_{eff}	V_u	δ_{cr}	δ_y	δ_{SD}	δ_{max}	δ_u	δ_c
A-E1	0.20	0.10	0.10	0.20	0.30	0.30	0.30	-

2D Issues: Equivalent frame idealization of URM walls

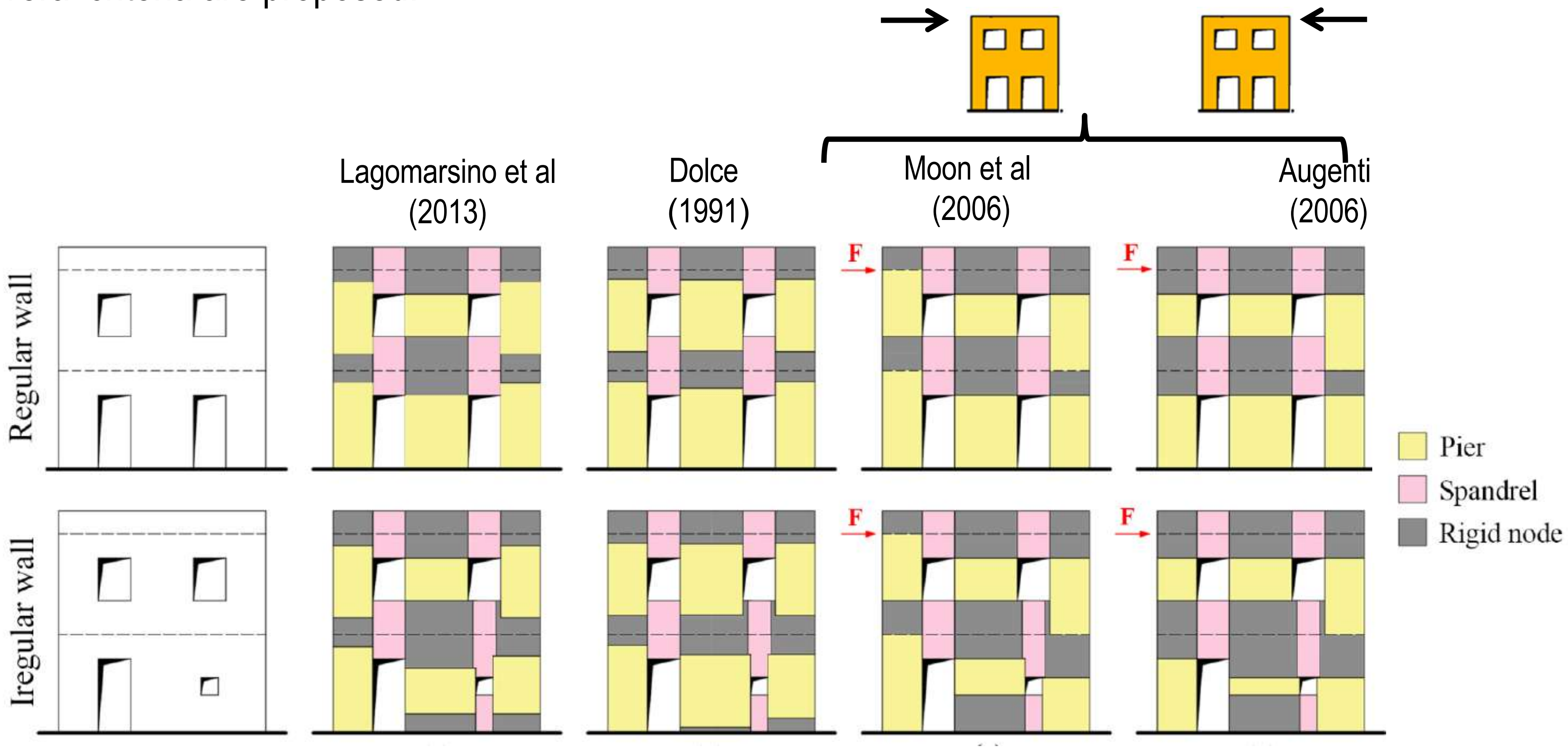
Rules for the **identification of the structural elements**



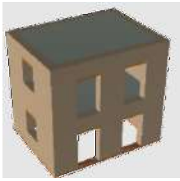
PIERS – Effective height

Several criteria are proposed:

- empirical
- based on limited experimentations
- based on few numerical simulations
- mainly developed for almost regular walls

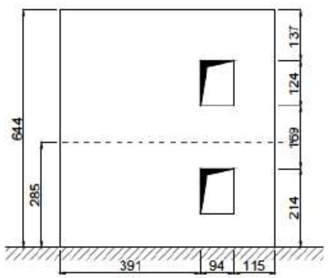


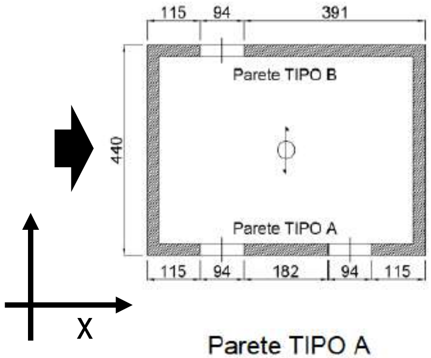
2D Issues: Equivalent frame idealization of URM walls



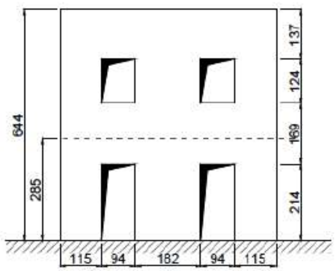
2-STOREY SIMPLE MASONRY BUILDING

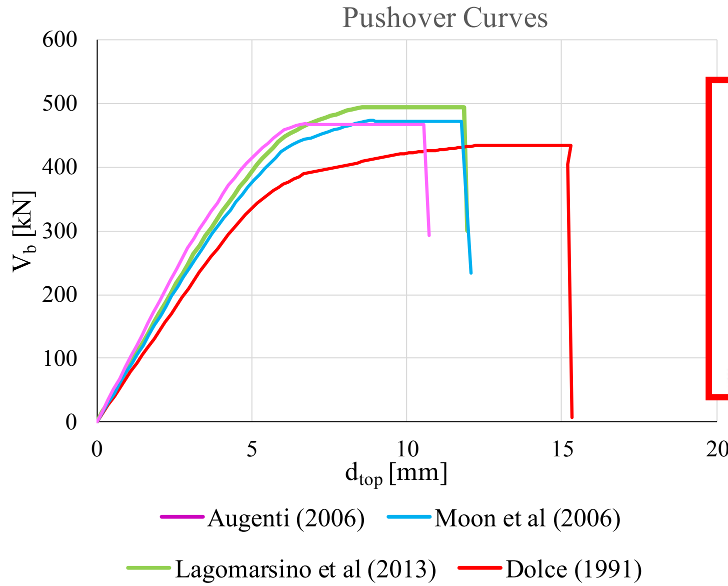
Parete TIPO B

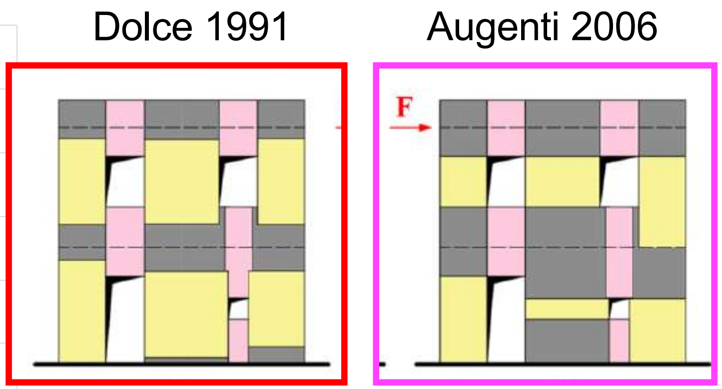


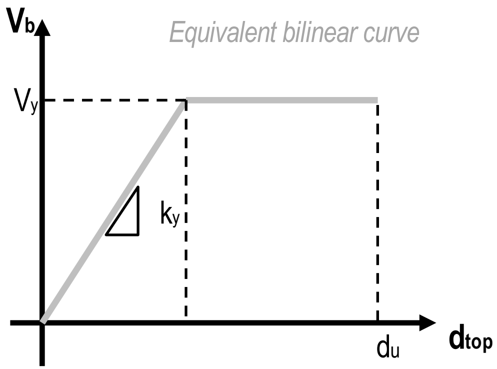


Parete TIPO A



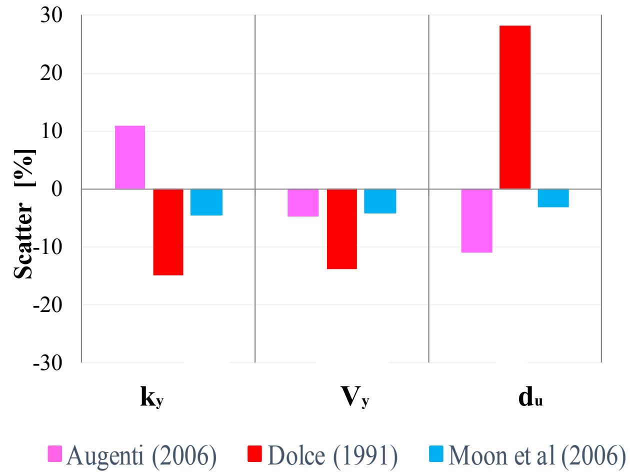




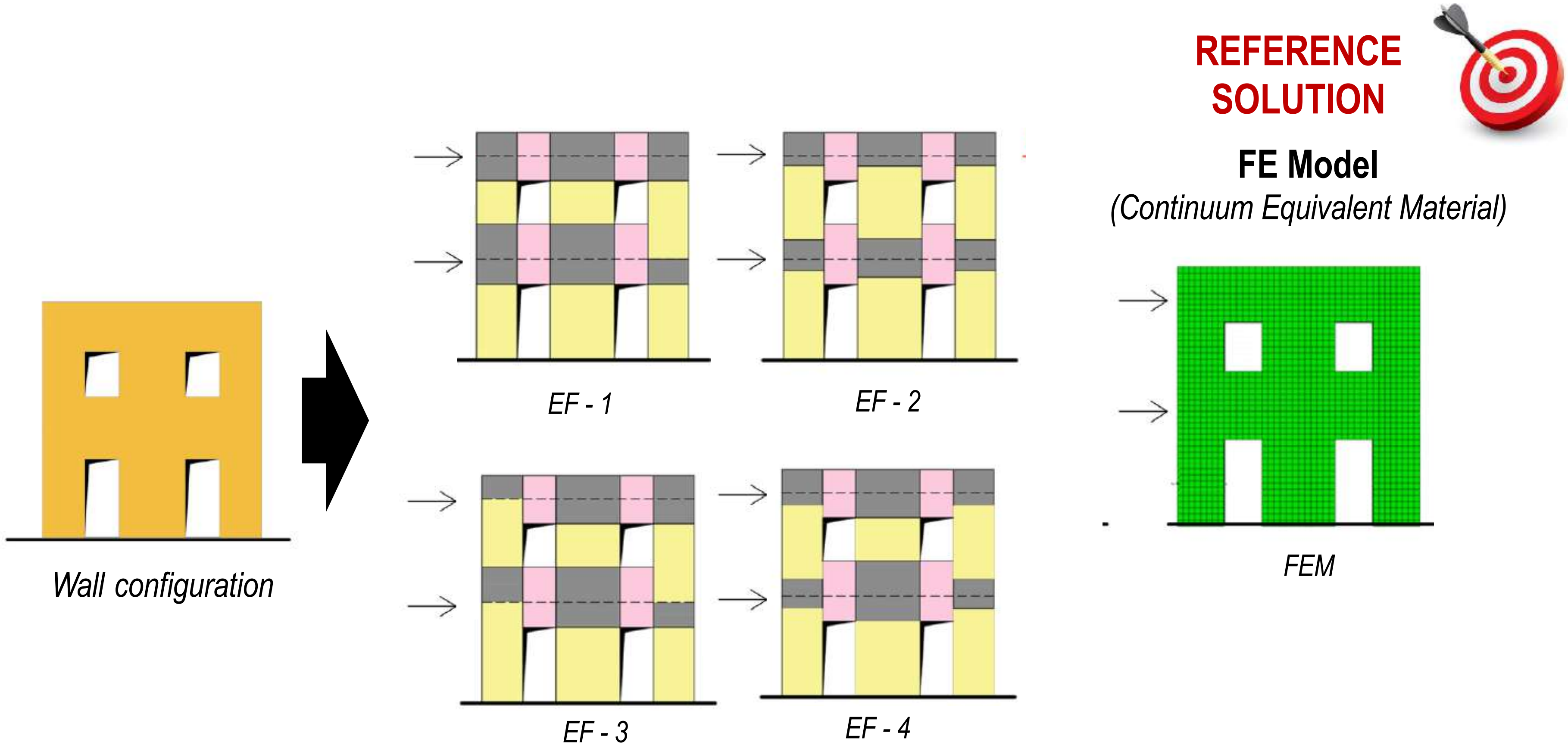


Scatter on V_y, d_u, k_y

(same software, different mesh)



2D Issues: Equivalent frame idealization of URM walls

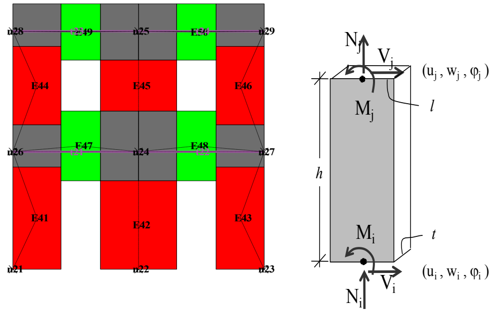


- ❑ PROPER CALIBRATION OF MECHANICAL PARAMETER TO GUARANTEE THE CONSISTENCY
- ❑ COMPARISON OF RESULTS OF STRUCTURAL RESPONSE PARAMETERS AT GLOBAL and LOCAL SCALE

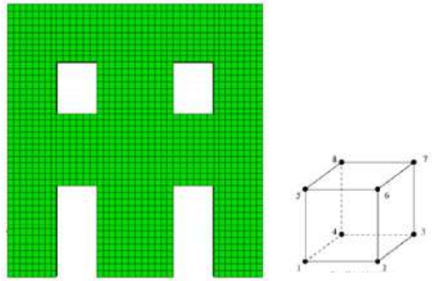
Reference: Camilletti 2019 PhD thesis, University of Genoa

2D Issues: Equivalent frame idealization of URM walls

EF Model: Tremuri Research Version

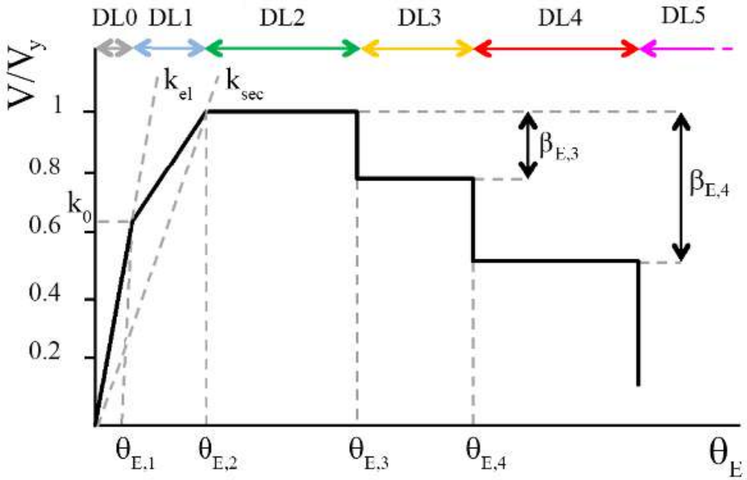


FE Model: Abaqus v.6.14

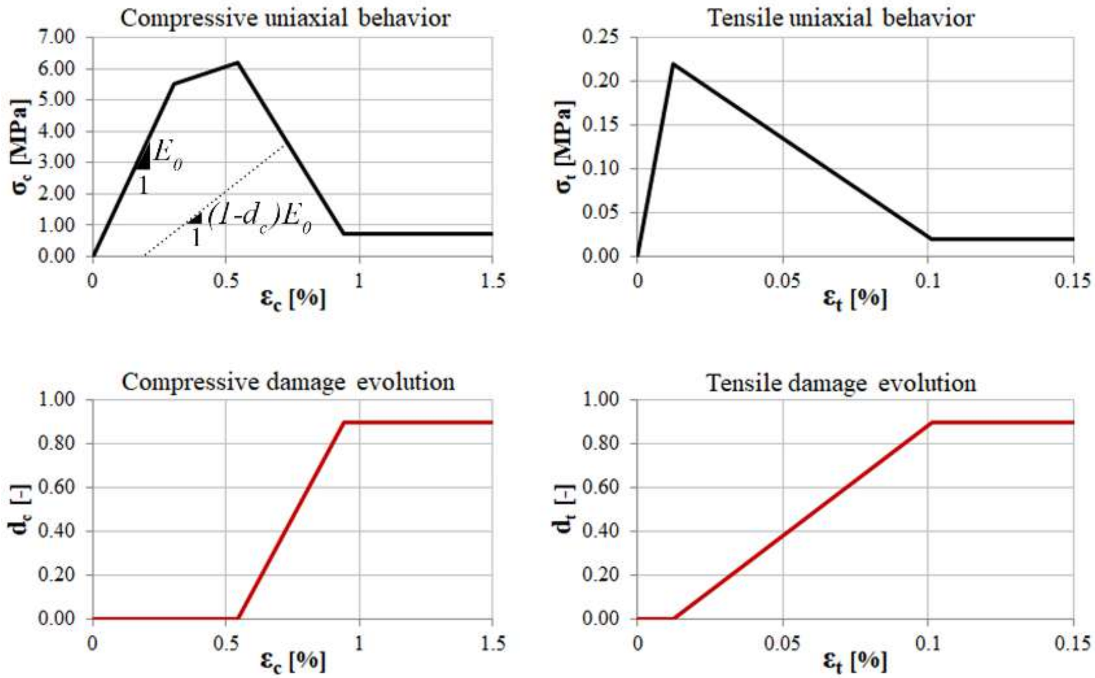


CONCRETE DAMAGED PLASTICITY Model

(Lubliner et al 1989, Lee and Fenves 1998)

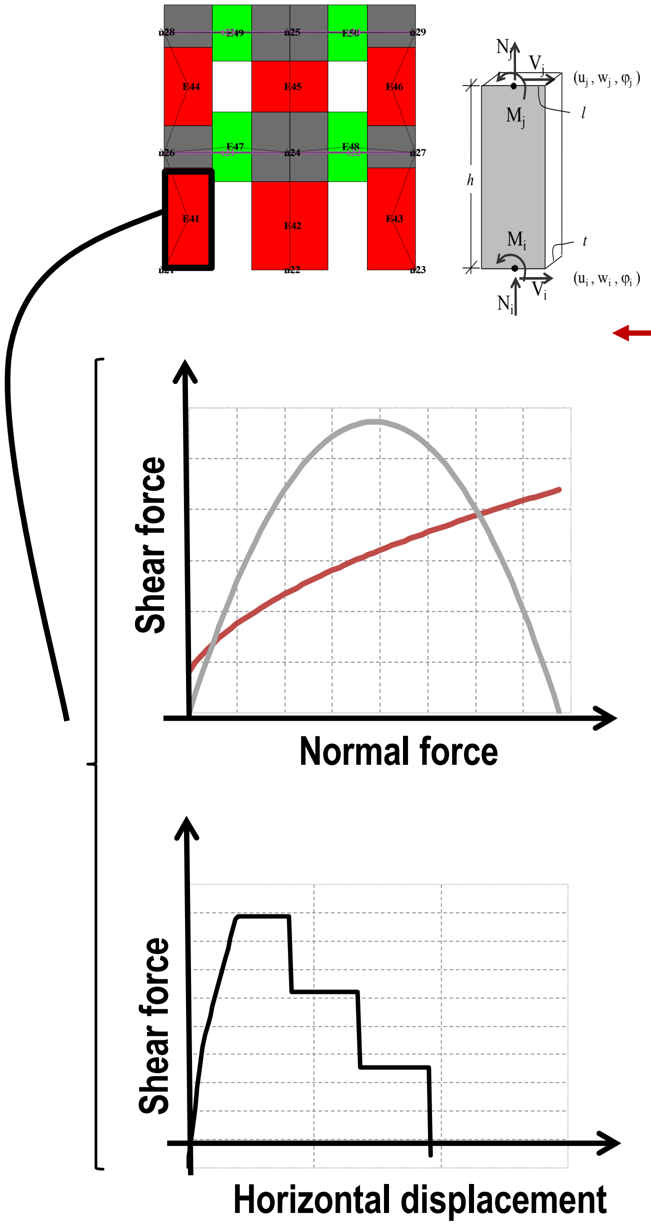


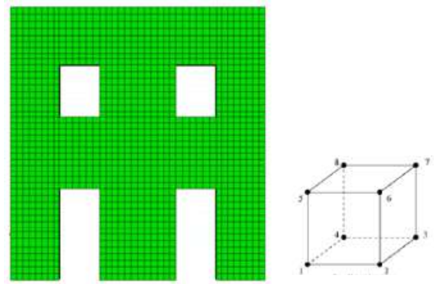
(Cattari and Lagomarsino 2013)



Reference: Camilletti 2019 PhD thesis, University of Genoa

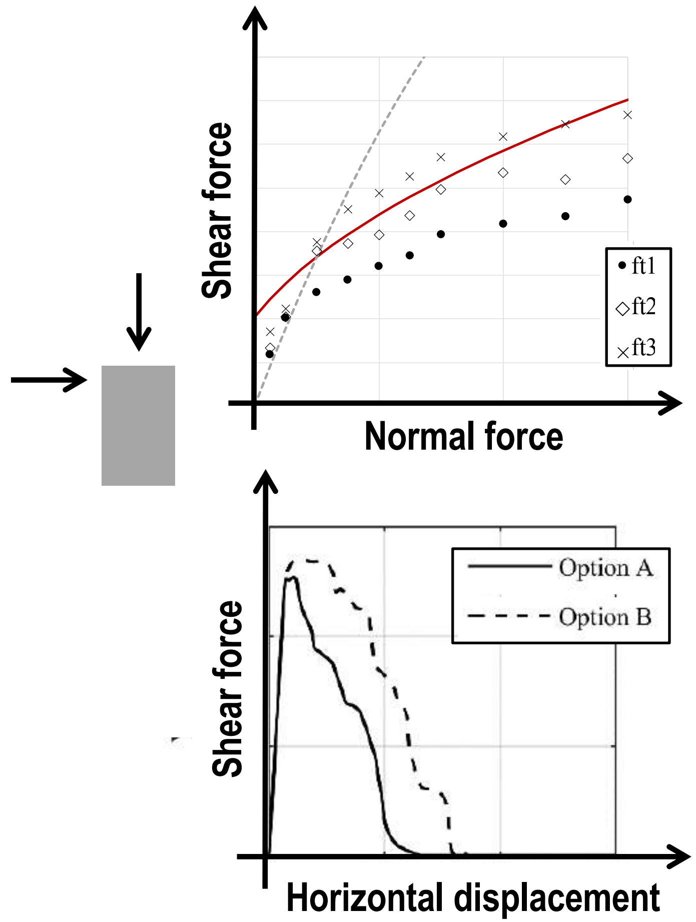
2D Issues: Equivalent frame idealization of URM walls





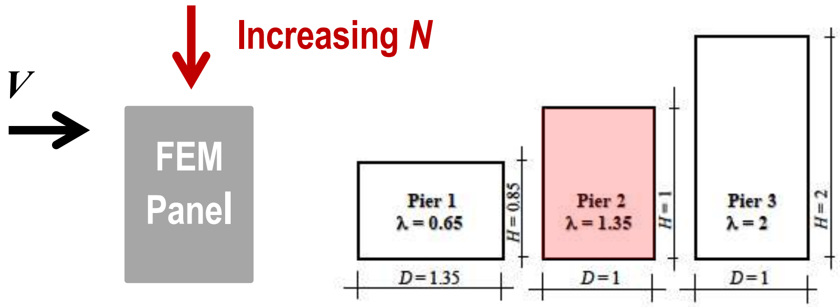
CALIBRATION Of PARAMETERS

- **STRENGTH**
- **POST-PEAK RESPONSE**

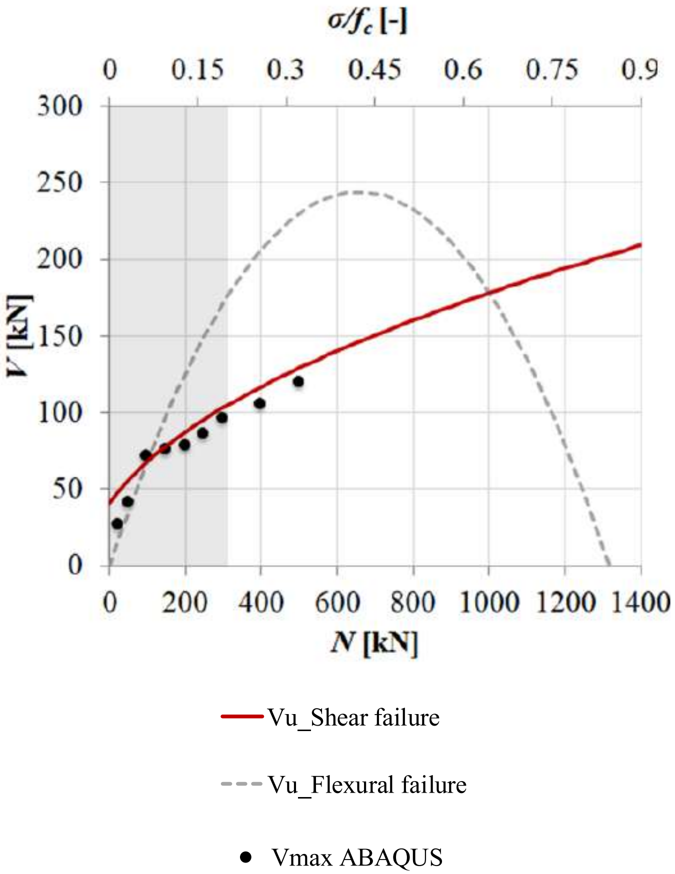


Reference: Camilletti 2019 PhD thesis, University of Genoa

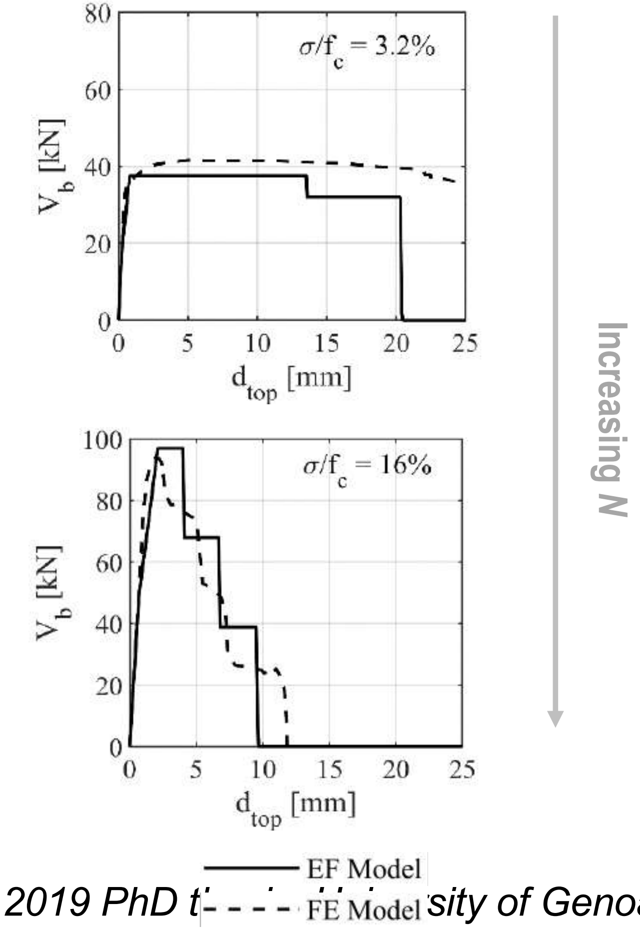
2D Issues: Equivalent frame idealization of URM walls



STRENGTH



POST-PEAK RESPONSE

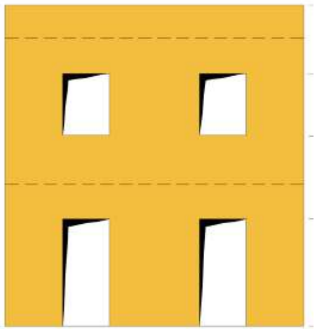


Reference: Camilletti 2019 PhD thesis, University of Genoa

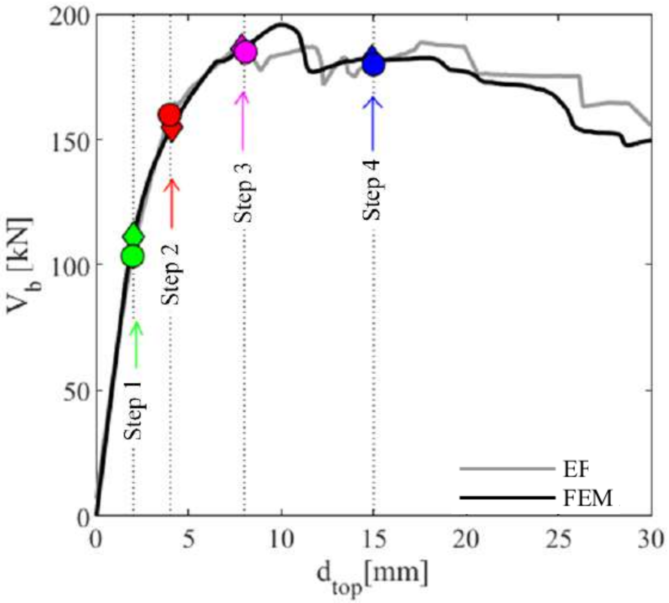
2D Issues: Equivalent frame idealization of URM walls

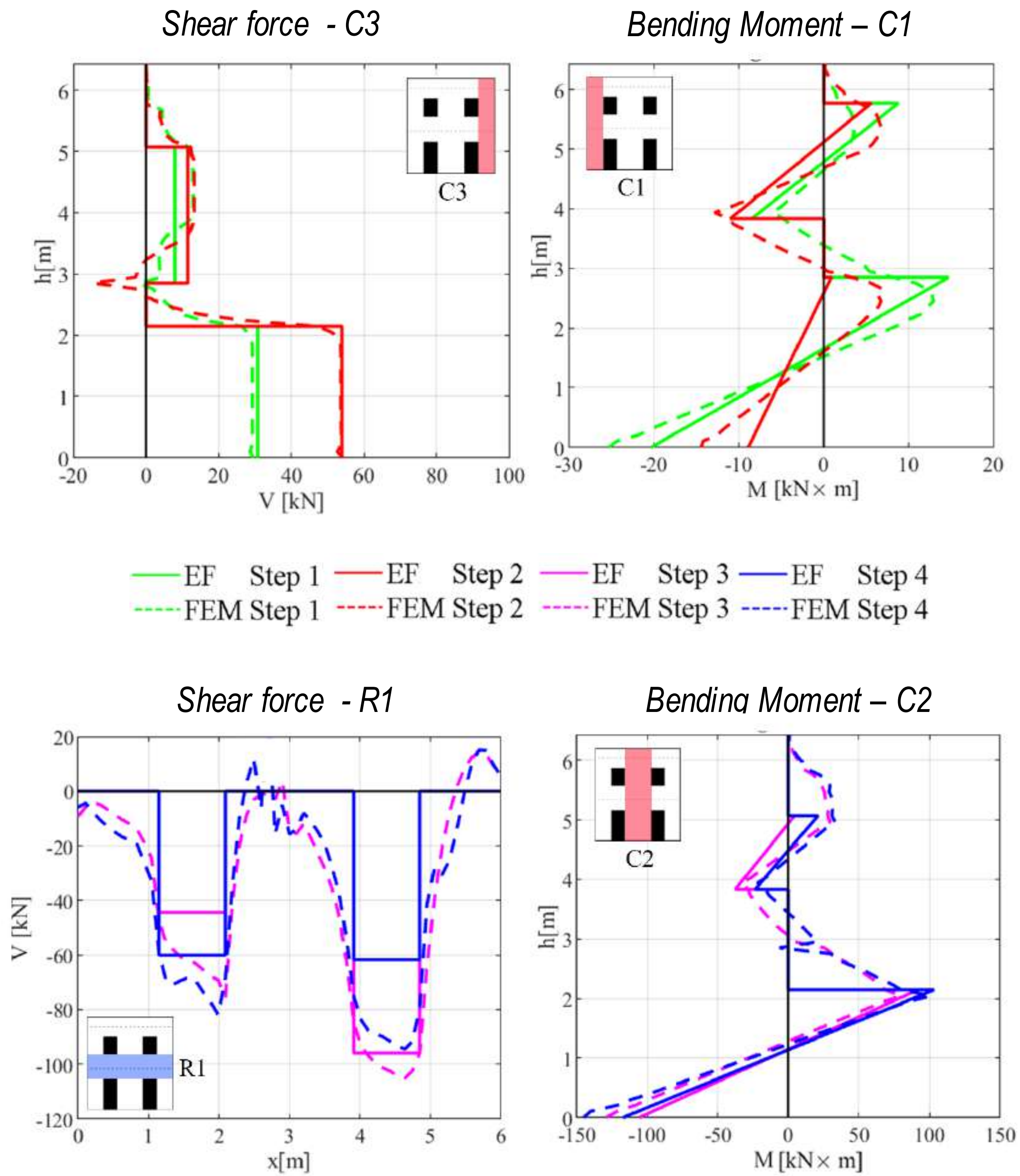
LOCAL RESPONSE

Regular wall



Step	1	2	3	4
d_{top}	2 mm	4 mm	8 mm	15 mm

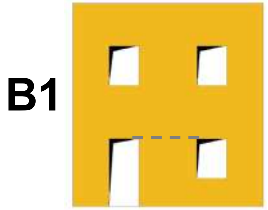


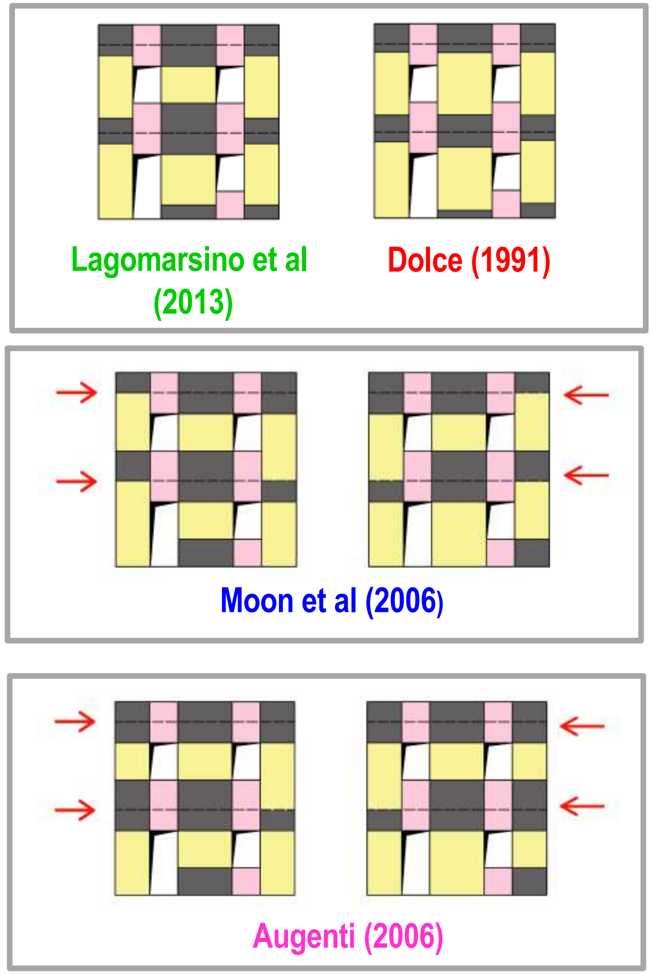


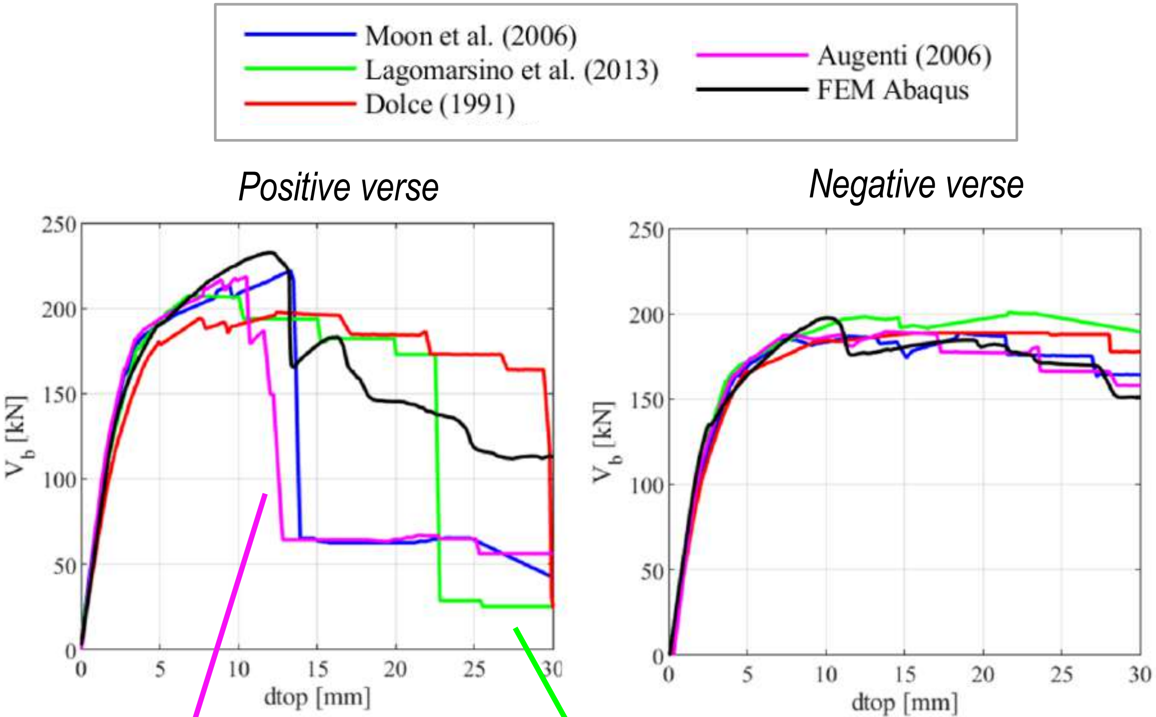
Reference: Camilletti 2019 PhD thesis, University of Genoa

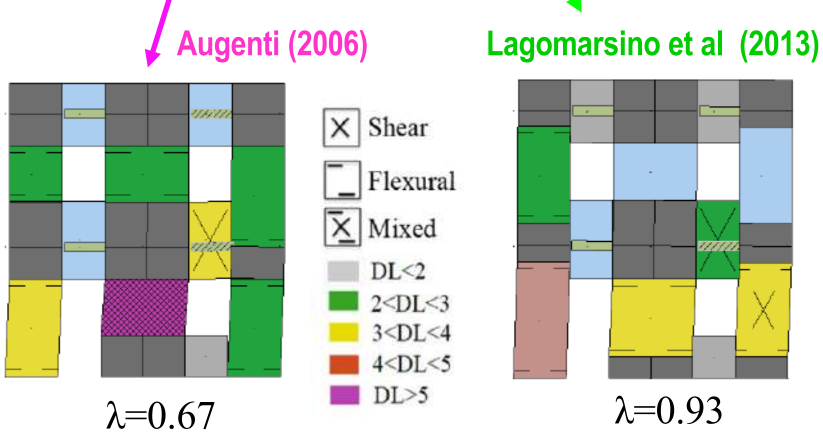
2D Issues: Equivalent frame idealization of URM walls

PROBLEM 1 - Identification of pier's effective height *(horizontal irregularity)*



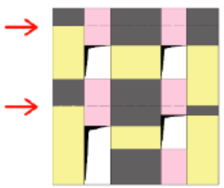






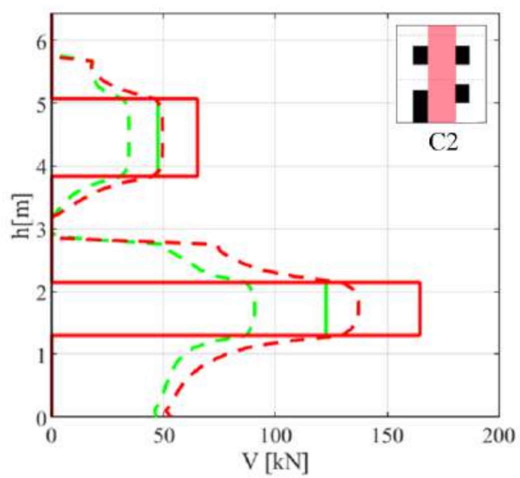
Reference: Camilletti 2019 PhD thesis, University of Genoa

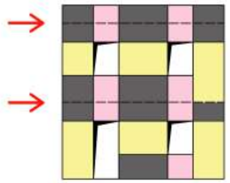
2D Issues: Equivalent frame idealization of URM walls



Moon et al (2006)
(COMPRESSION STRUT)

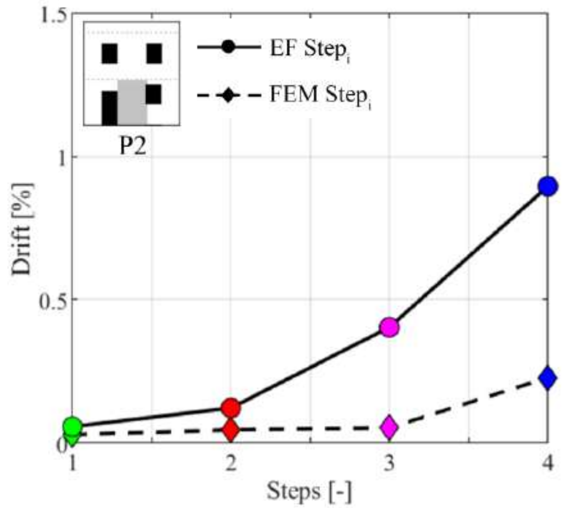
Shear force – C2

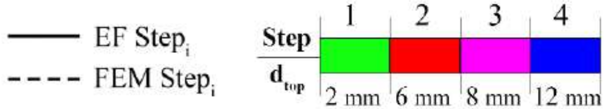


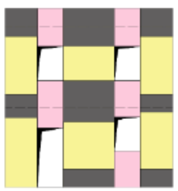


Augenti (2006)

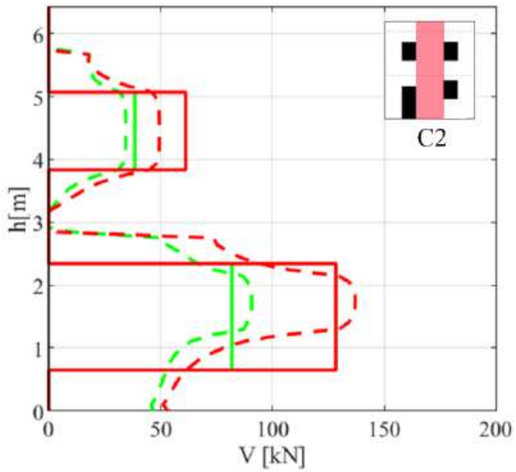
Drift – P2



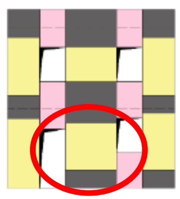




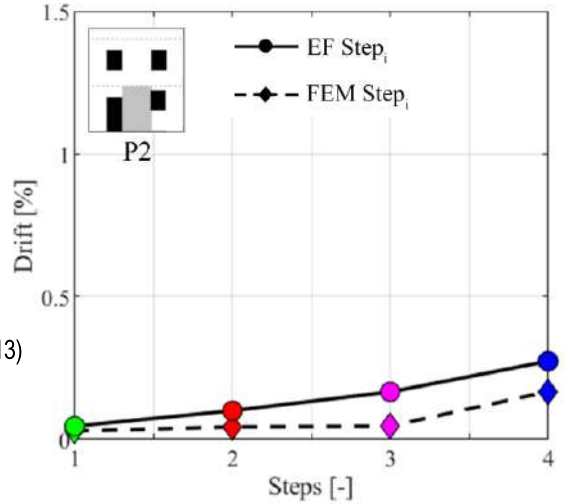
Lagomarsino et al (2013)



Drift – P2

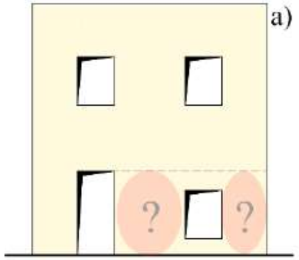


Lagomarsino et al (2013)



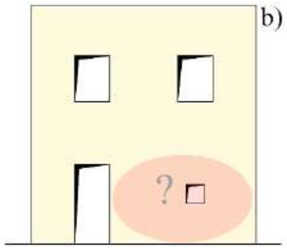
Reference: Camilletti 2019 PhD thesis, University of Genoa

2D Issues: Equivalent frame idealization of URM walls

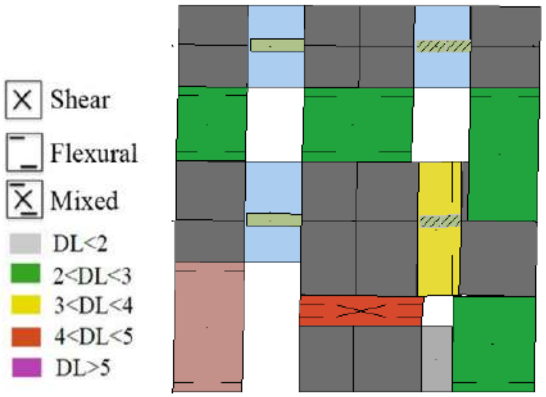


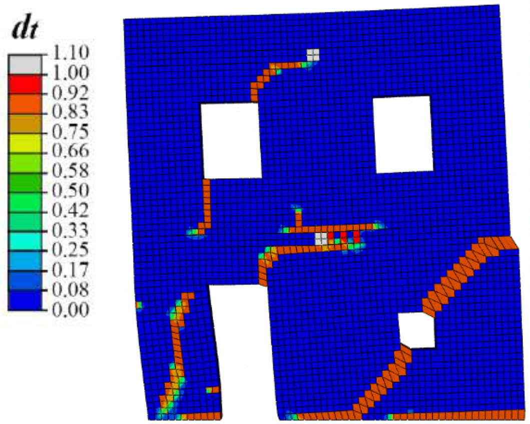
PROBLEM 1 - Identification of pier's effective height *(horizontal irregularity)*

- **AUGENTI (2006) and MOON et al (2006) NOT RECOMMENDED**
- **DOLCE (1991) and LAGOMARSINO et al (2013) WITHOUTH CORRECTIVE MEASURES**



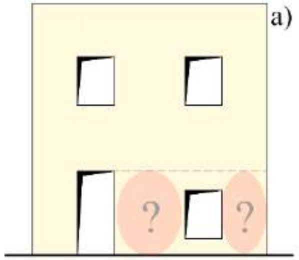
PROBLEM 2 – Presence of little openings





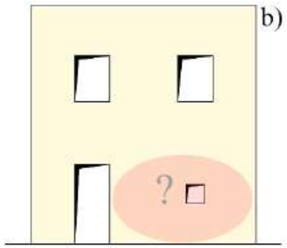
Reference: Camilletti 2019 PhD thesis, University of Genoa

2D Issues: Equivalent frame idealization of URM walls



PROBLEM 1 - Identification of pier's effective height *(horizontal irregularity)*

- **AUGENTI (2006) and MOON et al (2006) NOT RECOMMENDED**
- **DOLCE (1991) and LAGOMARSINO et al (2013) WITHOUTH CORRECTIVE MEASURES**

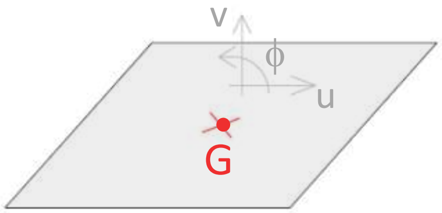


PROBLEM 2 – Presence of little openings

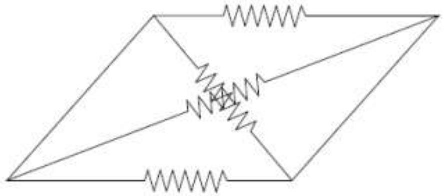
NEGLECT the OPENING in the STRUCTURAL MODEL

Reference: Camilletti 2019 PhD thesis, University of Genoa

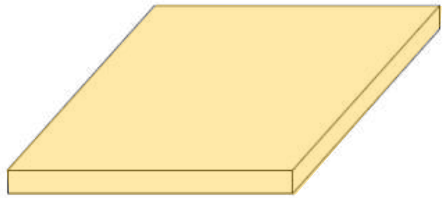
3D Issues: Diaphragms modelling



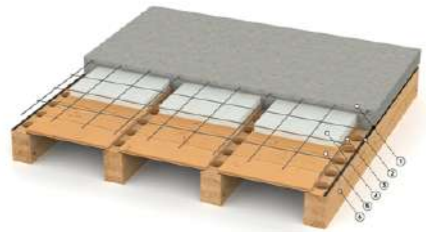
Rigid assumption



Set of equivalent springs



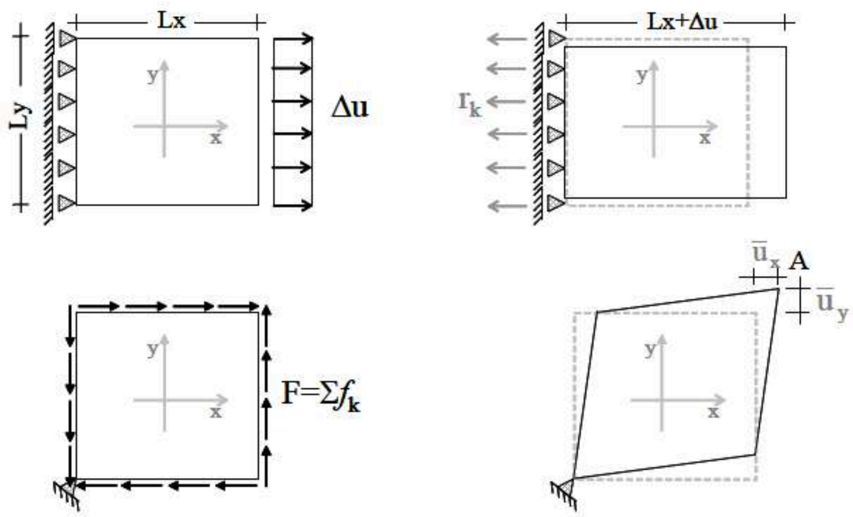
equivalent membranes



Accurate modelling

OFTEN ADOPTED AS DEFAULT BY SOFTWARE, IN MANY CASES IS THE ONLY OPTION

MORE ADEQUATE FOR DESCRIBING THE EXISTING BUILDINGS

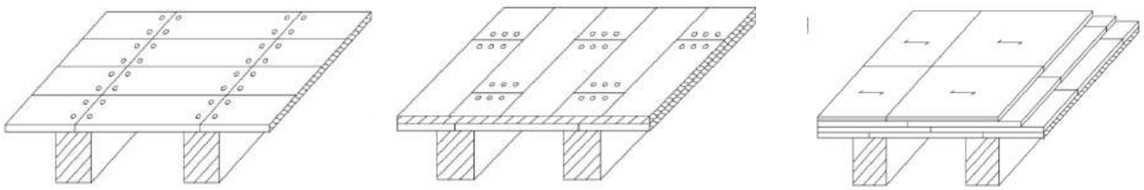


3D Issues: Diaphragms modelling

TIMBER FLOORS

FEMA Guidelines – ASCE 41-13

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.

Table 10.8: Shear stiffness values† for straight sheathed vintage flexible timber floor diaphragms (Giongo et al., 2014)

Direction of loading	Joist continuity	Condition rating	Shear stiffness†, G_d (kN/m)
Parallel to joists	Continuous or discontinuous joists	Good	350
		Fair	285
		Poor	225
Perpendicular to joists††	Continuous joist, or discontinuous joist with reliable mechanical anchorage	Good	265
		Fair	215
		Poor	170
	Discontinuous joist without reliable mechanical anchorage	Good	210
		Fair	170
		Poor	135

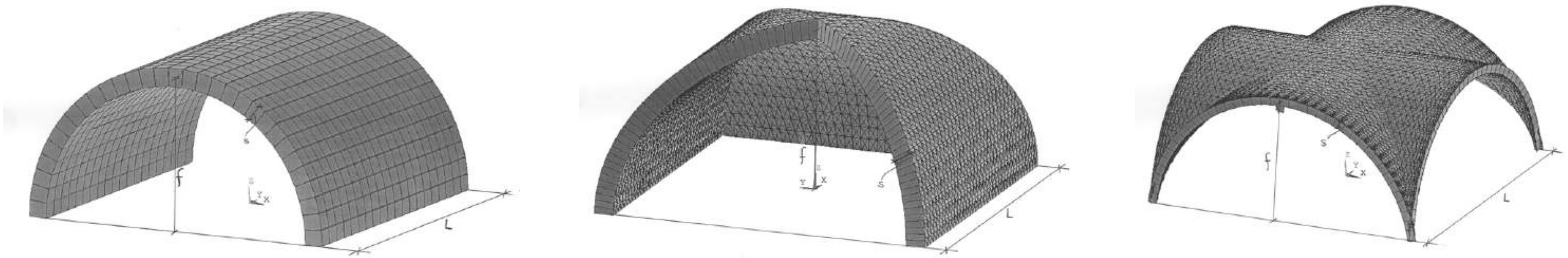
Note:
† Values may be amplified by 20% when the diaphragm has been renailed using modern nails and nail guns
†† Values should be interpolated when there is mixed continuity of joists or to account for continuous sheathing at joist splice

Table 10.9: Stiffness multipliers for other forms of flexible timber diaphragms (derived from ASCE, 2013)

Type of diaphragm sheathing		Multipliers to account for other sheathing types
Single straight sheathing		x 1.0
Double straight sheathing	Chorded	x 7.5
	Unchorded	x 3.5
Single diagonal sheathing	Chorded	x 4.0
	Unchorded	x 2.0
Double diagonal sheathing or straight sheathing above diagonal sheathing	Chorded	x 9.0
	Unchorded	x 4.5

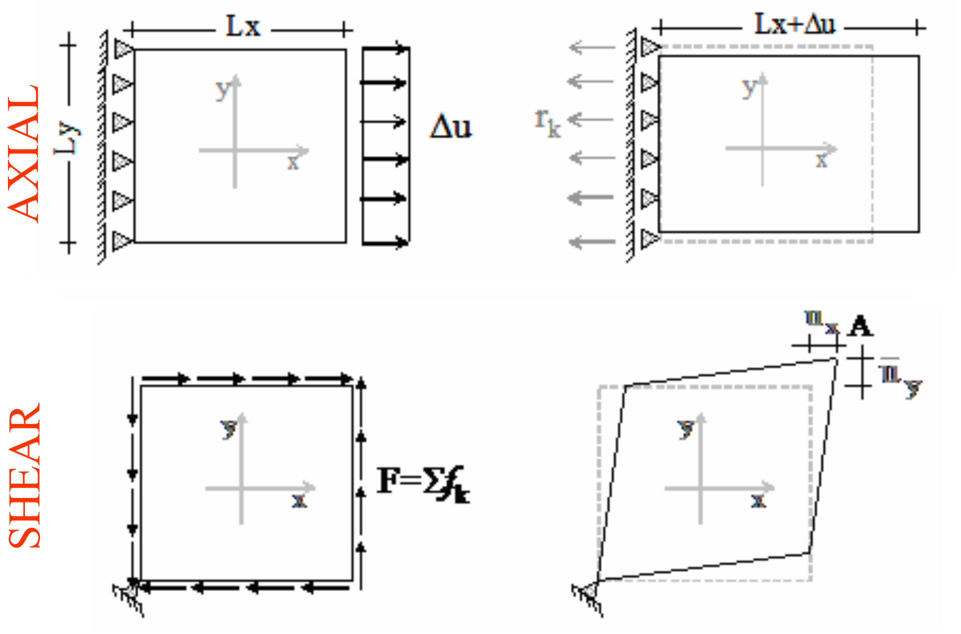
- MASONRY VAULTS

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

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

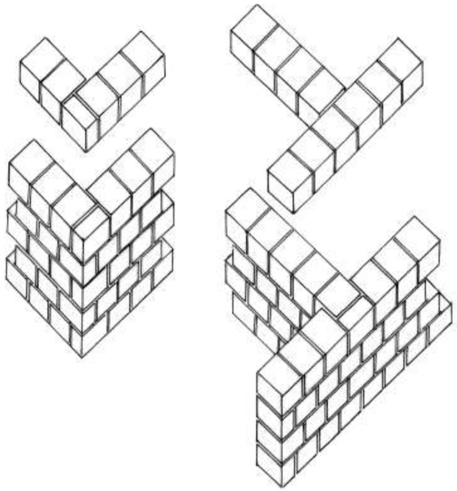
	Vault tipology		
	Barrel	Cloister	Cross
b_1	1.6433	-0.1633	-1.12
b_2	-1.8191	-1.3051	-4.1766
b_3	0.1133	1.9733	8.9033
b_4	1.1189	1.0339	0.7273
b_5	-	-	-34.323
b_6	-	-	5.6202

3D Issues: Connection between orthogonal walls

Quality of wall to wall connection

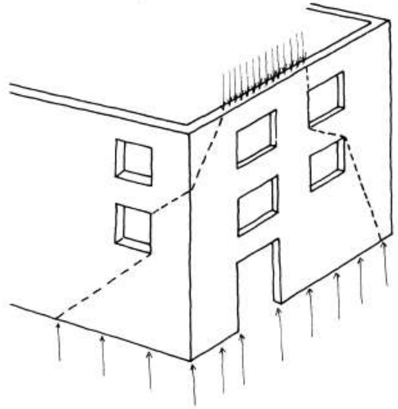
Poor quality

Good quality

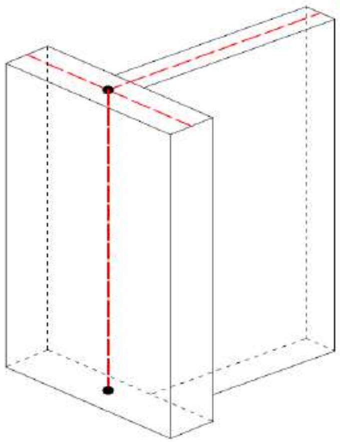


&

Redistribution of vertical actions among well connected walls

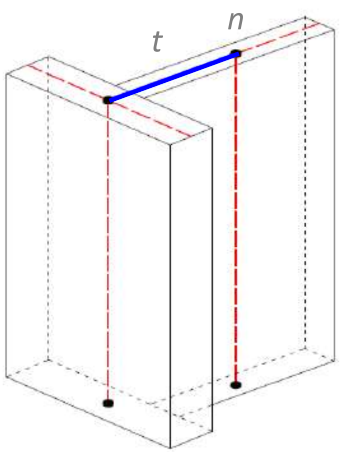


POSSIBLE MODELLING STRATEGY ADOPTED BY SOFTWARE

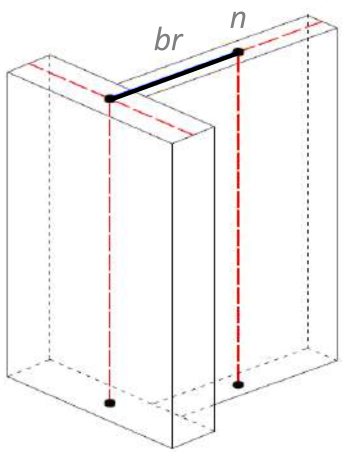


(A)
Full kinematic coupling

FULL COUPLING

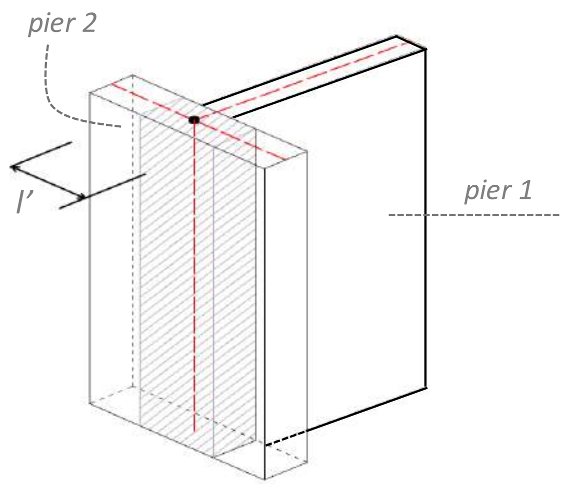


(B)
Equivalent beams of calibrated stiffness



(B')
Rigid beam

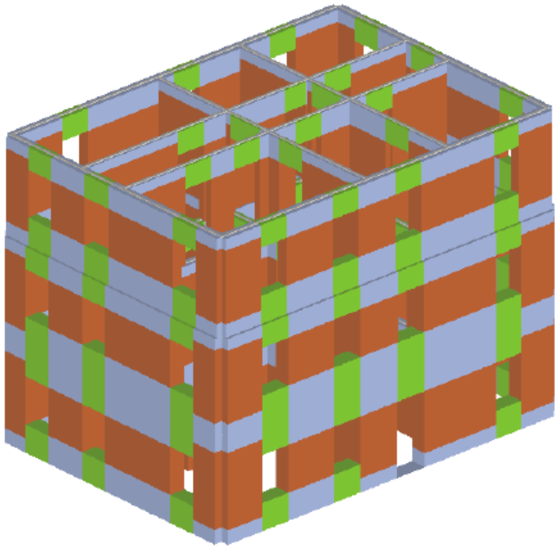
FULL COUPLING



(C)
The stiffness contribution of pier 2 is included in that of pier 1

3D NONLINEAR STATIC AND DYNAMIC ANALYSIS BY EQUIVALENT FRAME MODEL

TREMURI



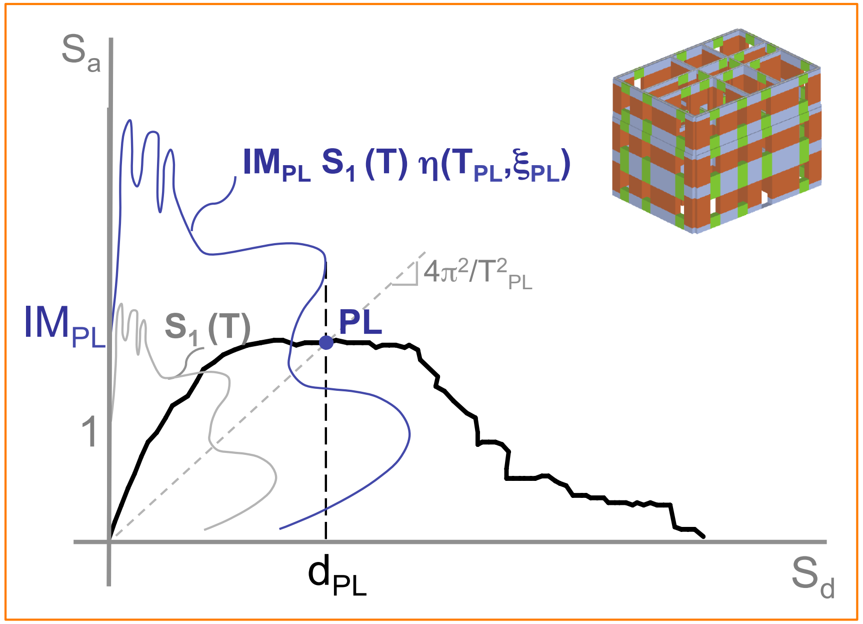
by Lagomarsino S., Penna A., Galasco A., Cattari S.

ask to: **tremuri@gmail.com**

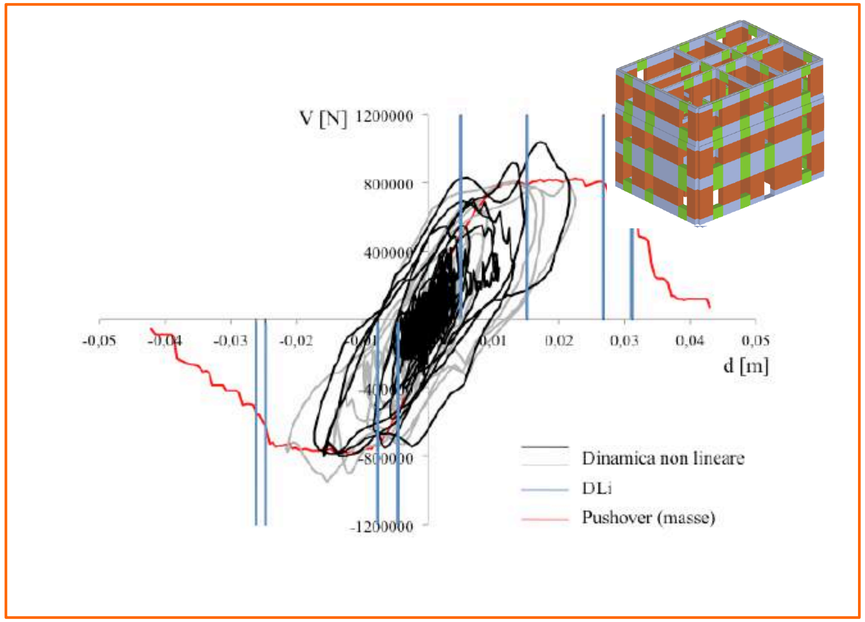
3MURI

ask to: **STADATA s.r.l. (www.stadata.com)**

NON LINEAR DYNAMIC ANALYSIS

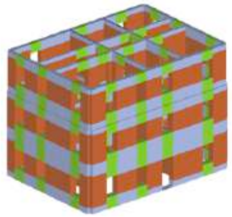


NON LINEAR DYNAMIC ANALYSIS



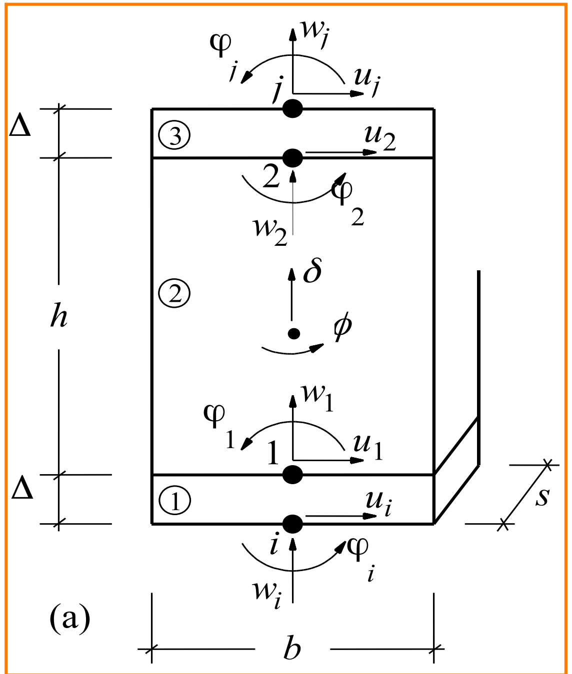
Constitutive models for masonry panels

✓ **CONSTITUTIVE MODELS IMPLEMENTED IN TREMURI FOR MASONRY PANELS:**



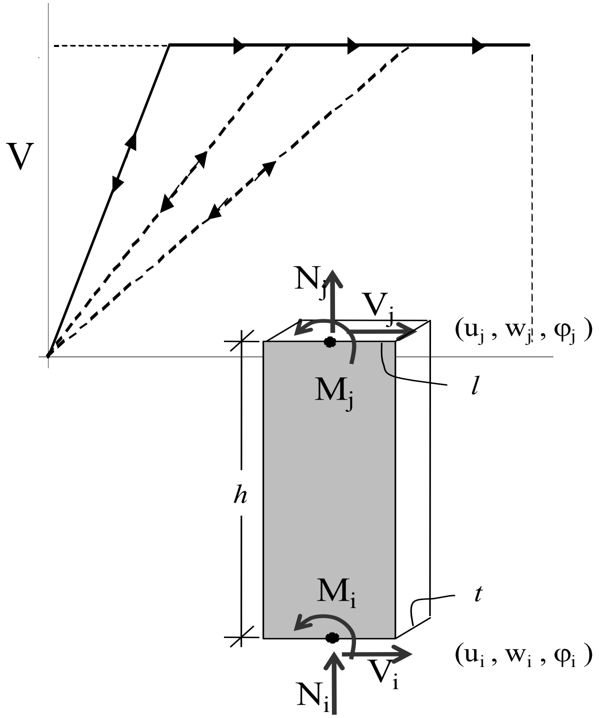
NON LINEAR MACROELEMENT

Gambarotta & Lagomarsino 1997;
Penna 2002;
Penna, Lagomarsino & Galasco 2013



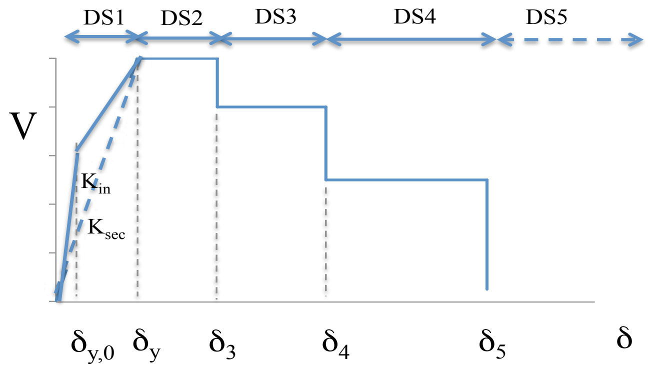
NON LINEAR BEAM
Bi-linear relation

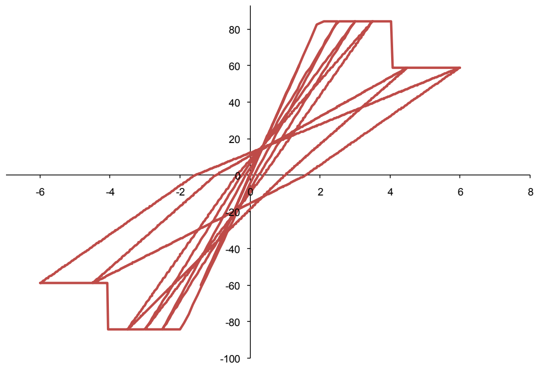
Galasco 2006
Lagomarsino, Penna, Galasco & Cattari 2013



NON LINEAR BEAM
Piecewise linear costitutive law

Cattari & Lagomarsino 2013



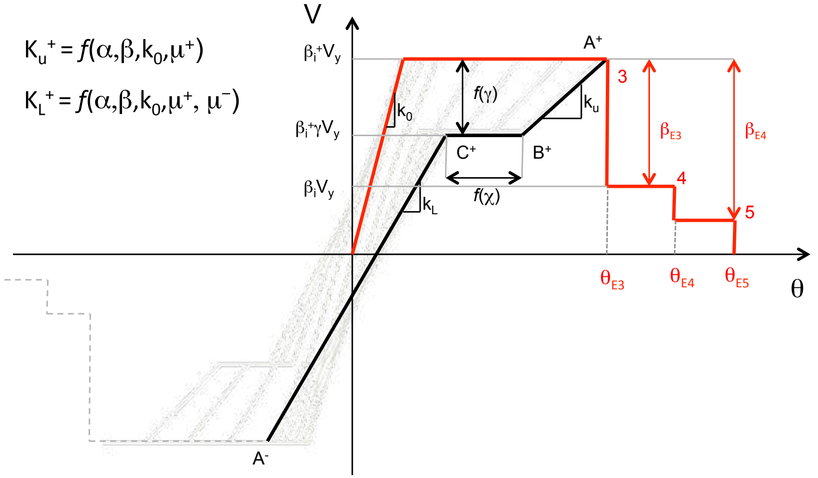


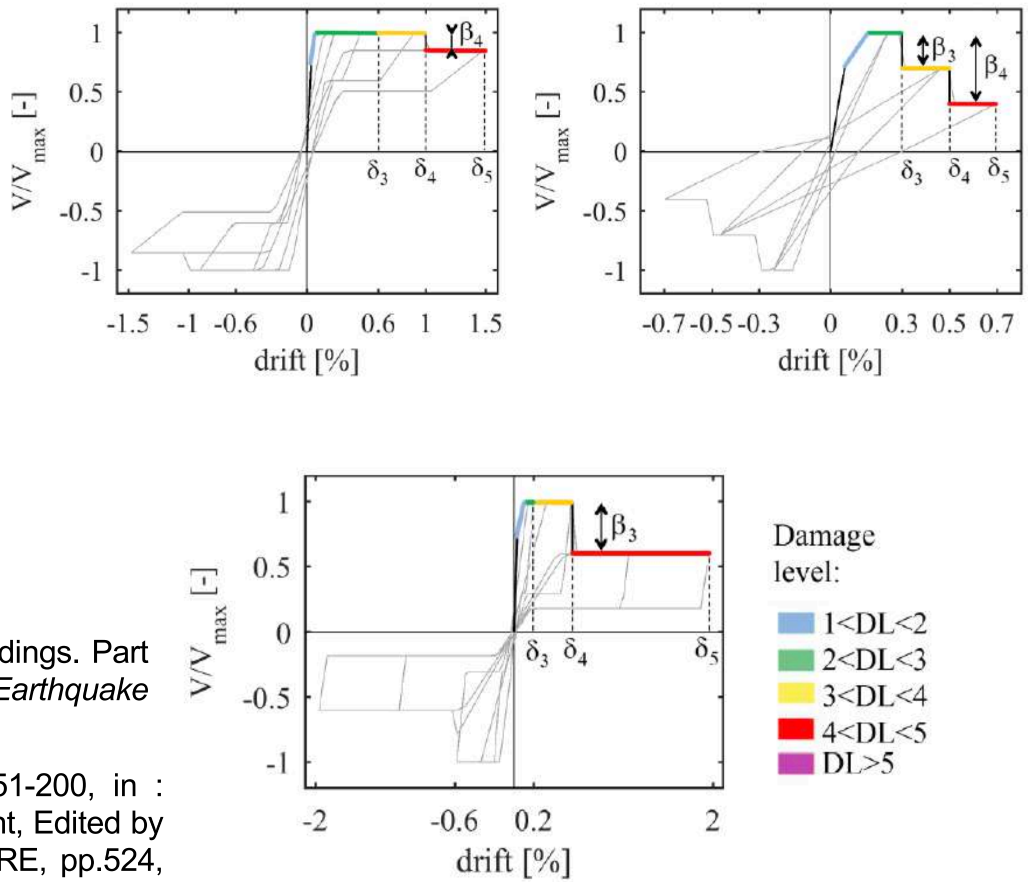
REF:Lagomarsino S, Penna A, Galasco A, Cattari S. (2013) TREMURI program: an equivalent frame model for the nonlinear seismic analysis of masonry buildings, *Engineering Structures*, 56, pp. 1787-1799, Elsevier, ISSN: 0141-0296

Non linear beam – *piecewise linear constitutive law*

Based on a phenomenological approach and allow to describe:

- ❑ the non linear response until very severe damage levels (from 1 to 5) through progressing strength decay (β_{Ei}) in correspondence of assigned values of drift (θ_{Ei});
- ❑ a quite accurate hysteretic response.



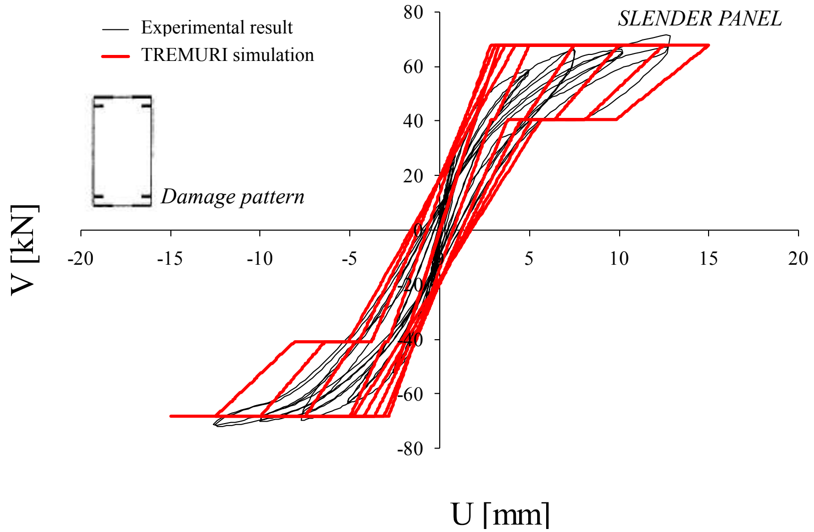


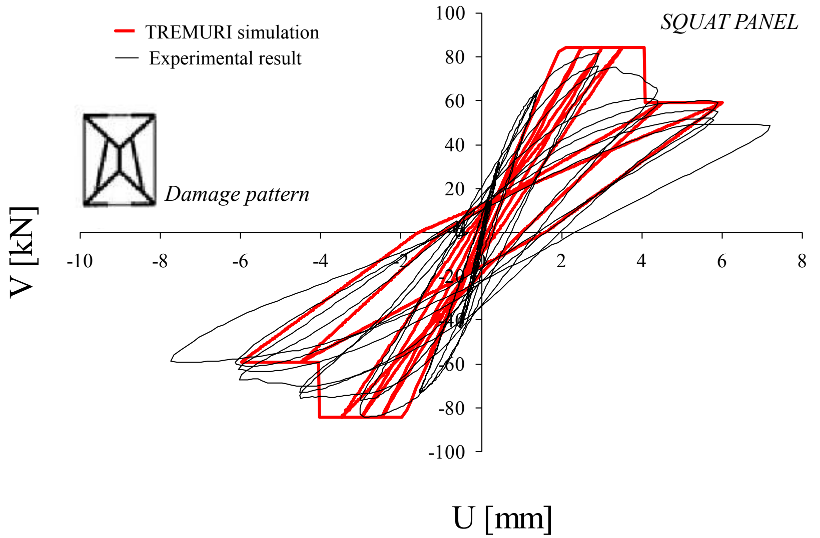
REF Cattari S., et al. (2018) Masonry Italian Code-Conforming Buildings. Part 2: Nonlinear Modelling and Time-History Analysis, *Journal of Earthquake Engineering*, Taylor and Francis, 22(sup2), 2010-2040

REFCattari S., Lagomarsino S. (2013) Masonry structures, pp.151-200, in : Developments in the field of displacement based seismic assessment, Edited by T. Sullivan and G.M.Calvi, Ed. IUSS Press (PAVIA) and EUCENTRE, pp.524, ISBN:978-88-6198-090-7.

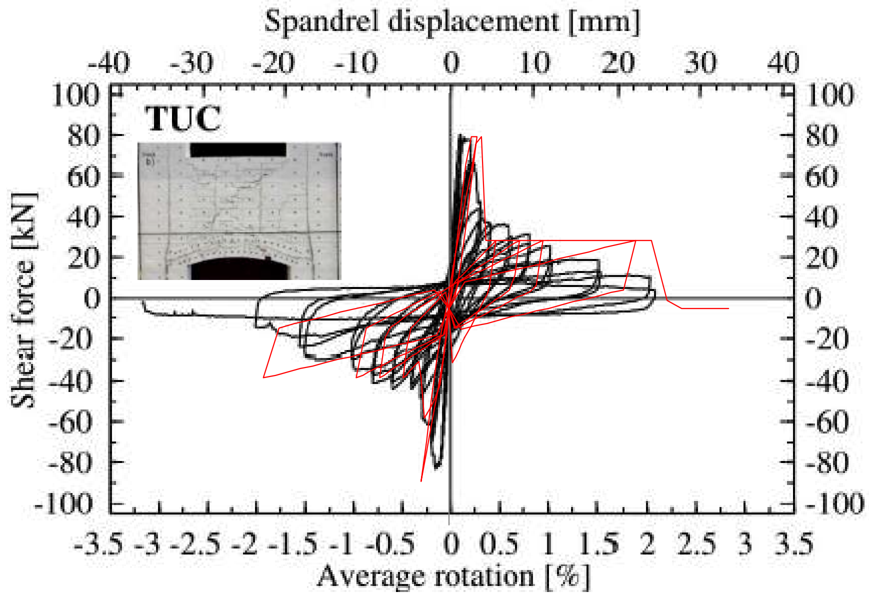
Non linear beam – *piecewise linear constitutive law*

PIERS – Anthoine et al. 1995





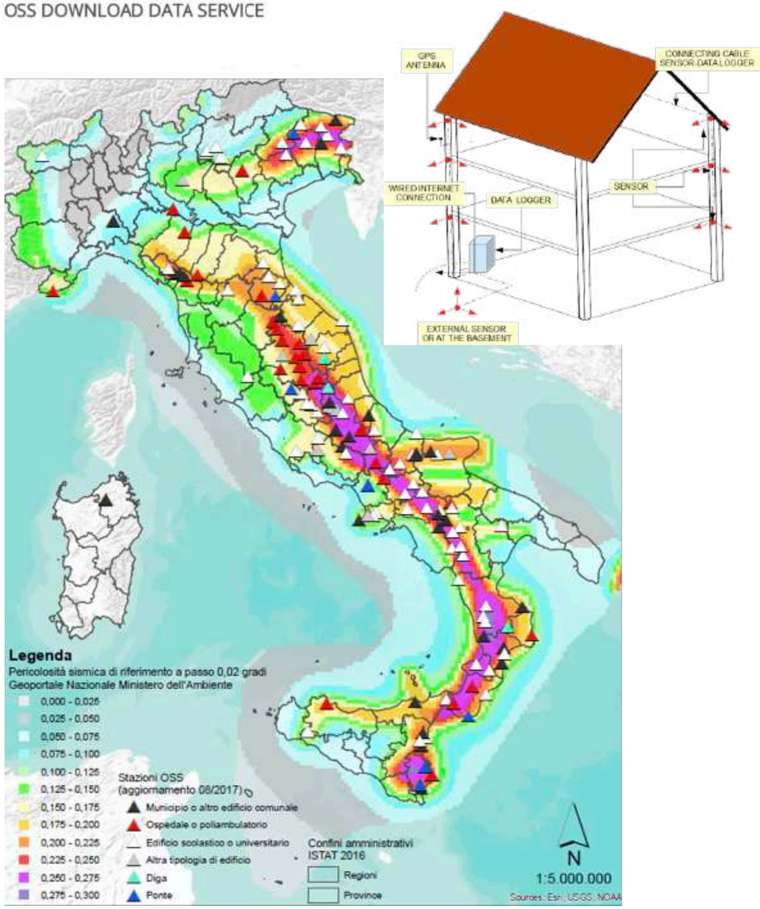
SPANDRELS – Beyer & Dazio 2012



VALIDATION ON MONITORED URM BUILDINGS

OSS
Osservatorio Sismico delle Strutture
OSS DOWNLOAD DATA SERVICE

PROTEZIONE CIVILE NAZIONALE



reluis

WORKGROUP -Task 4.1

Analysis of masonry buildings monitored by "OSS"

REFERENCE: S. Cattari et al. 2019 Discussion on data recorded by the Italian structural seismic monitoring network on three masonry structures hit by the 2016-2017 Central Italy earthquake, *Proc. of COMPDYN 2019,* Crete 24-26 June 2019.

VALIDATION ON MONITORED URM BUILDINGS

Fabriano Courthouse

Built in 1940 – 4-story building - interstory heigth 4.00 m - gross area: 1220 m^2

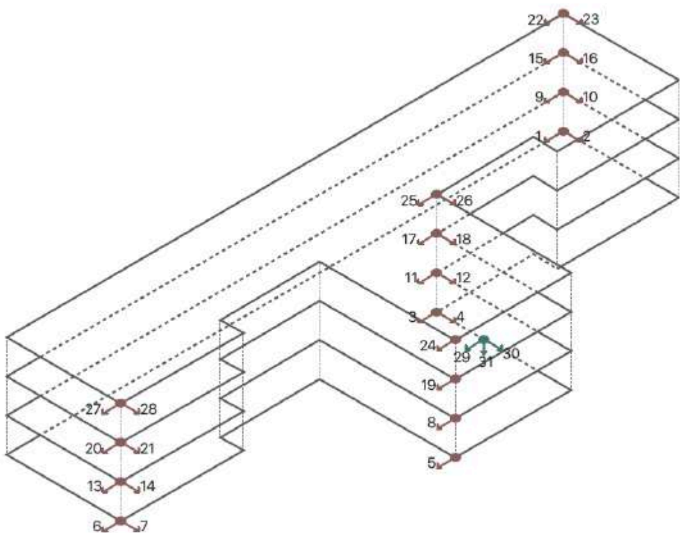
First level plan

East facade

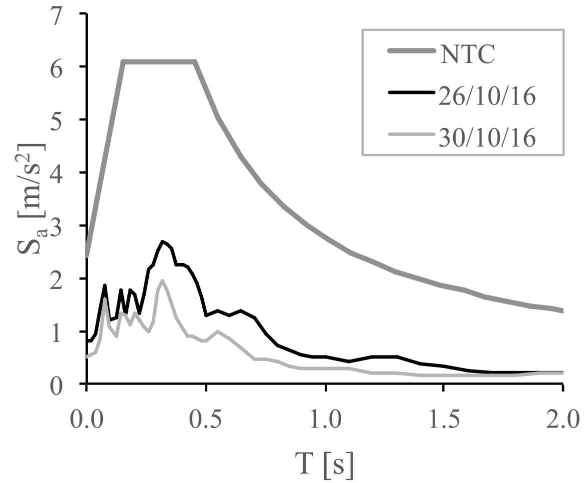
VALIDATION ON MONITORED URM BUILDINGS

Fabriano Courthouse

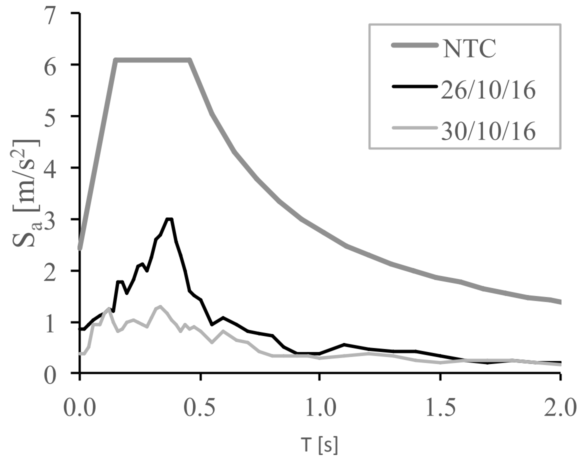
MONITORING SISTEM & DATA AVAILABLE (RECORDINGS OF MAIN and SECONDARY EVENTS & AMBIENT VIBRATION NOISE)



X direction



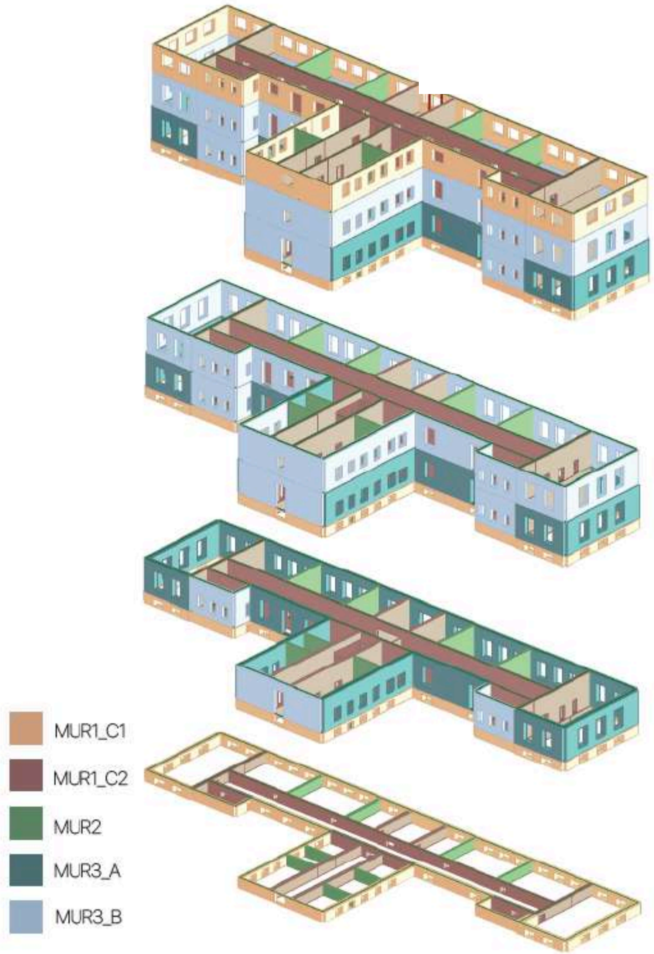
Y direction

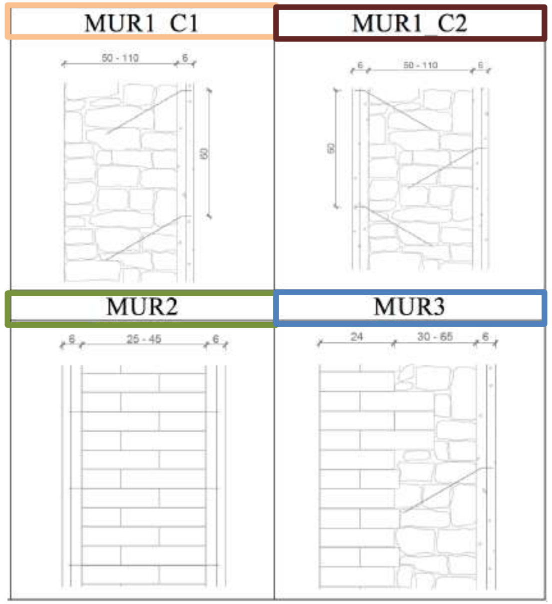


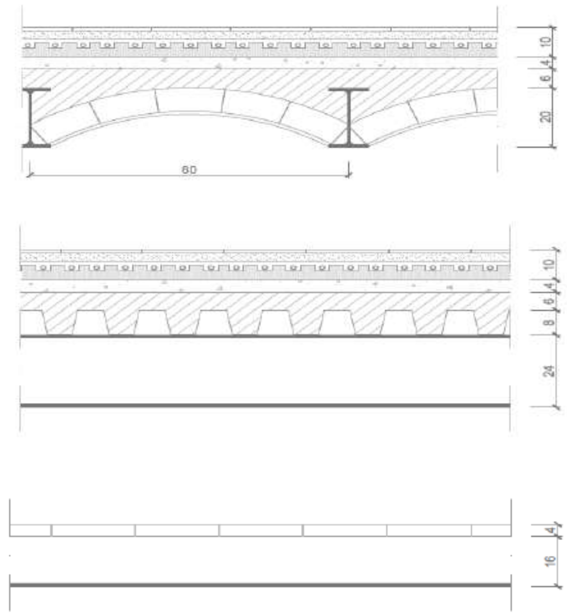
T_R=712 years – Soil B

Fabriano Courthouse

- Many kinds of strengthening interventions (date back to 1999) to masonry walls and to improve the wall-to-wall quality connection
- Many masonry types
- Many kinds of floors





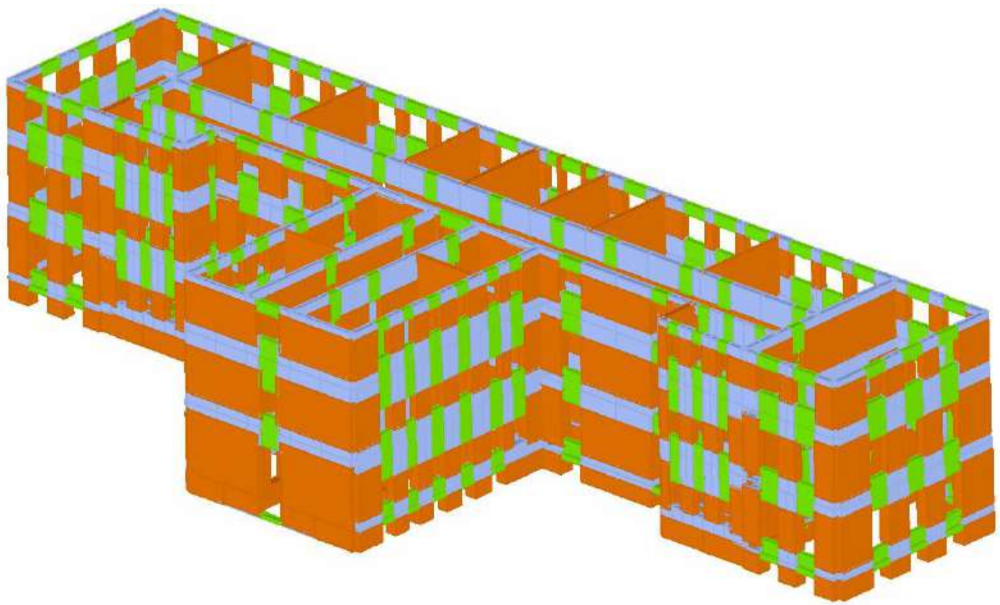


Many uncertainties!
Interesting case for model calibration!!!

VALIDATION ON MONITORED URM BUILDINGS

Fabriano Courthouse

CALIBRATION OF THE MODEL IN ELASTIC FIELD



UNCERTAINTIES CONSIDERED IN THE CALIBRATION PROCESS

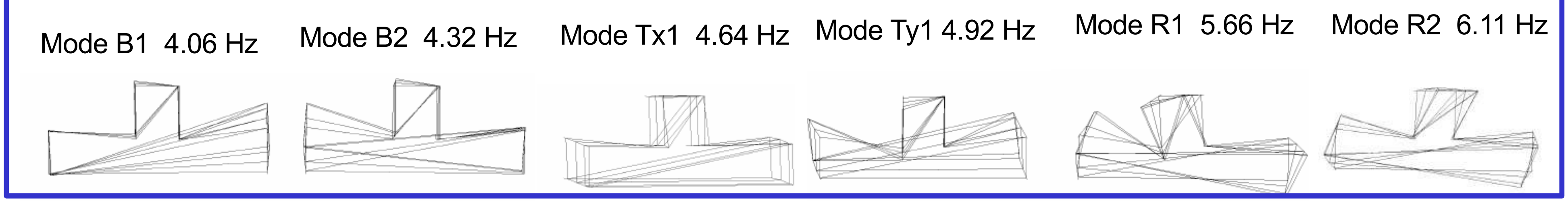
16 aleatory uncertainties

5 type of different masonry types differentiated between piers & spandrels – raising up of last floor

Various epistemic uncertainties

- Flange effect
- Interaction with external stair (actual effectiveness of seismic joint)
- Role of basement
- Stiffness diaphragms

TARGET FOR THE CALIBRATION (BY UNIPD)

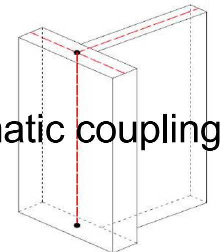


VALIDATION ON MONITORED URM BUILDINGS

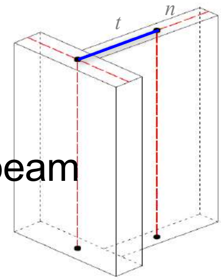
Fabriano Courthouse

CALIBRATION OF THE MODEL IN ELASTIC FIELD

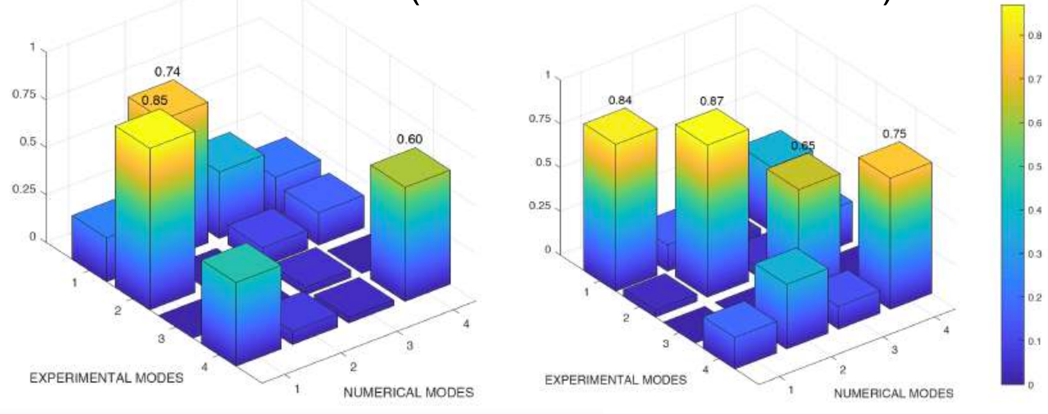
Full kinematic coupling



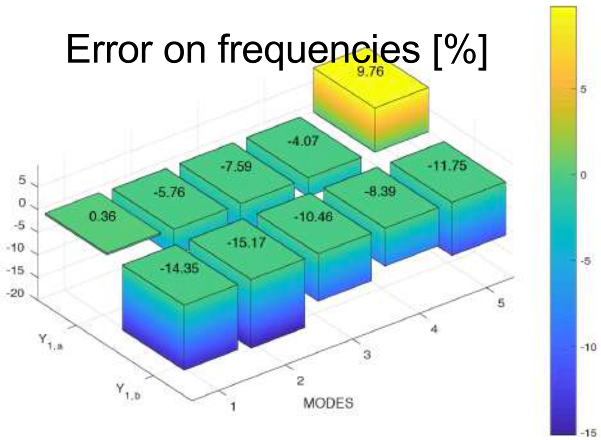
Calibrated beam



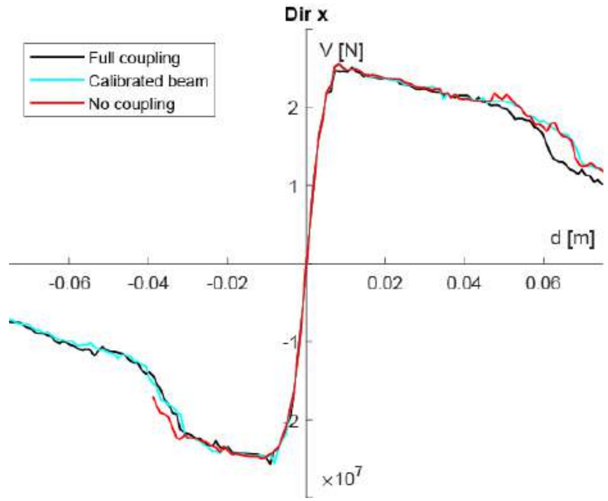
MAC (Modal Assurance Criterion) Index

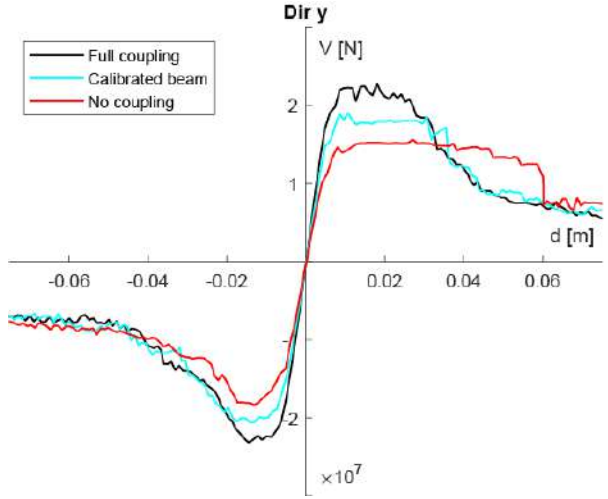


Error on frequencies [%]



Repercussion of the correct assumption on NONLINEAR FIELD



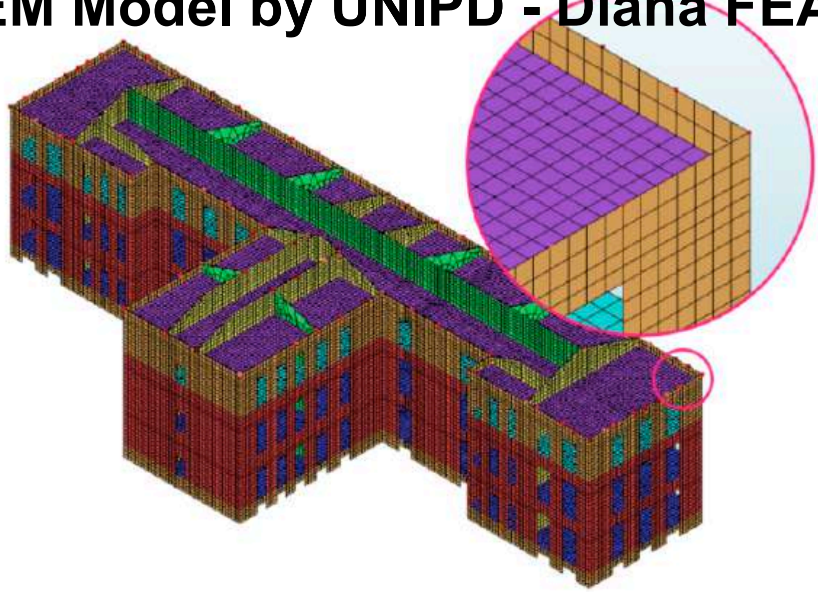


VALIDATION ON MONITORED URM BUILDINGS

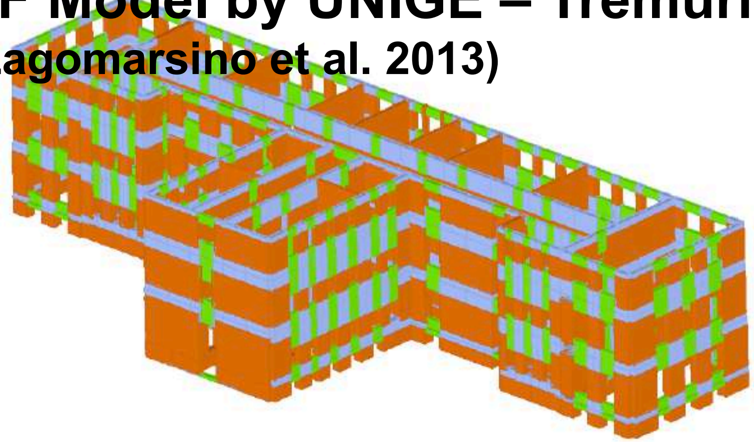
Fabriano Courthouse

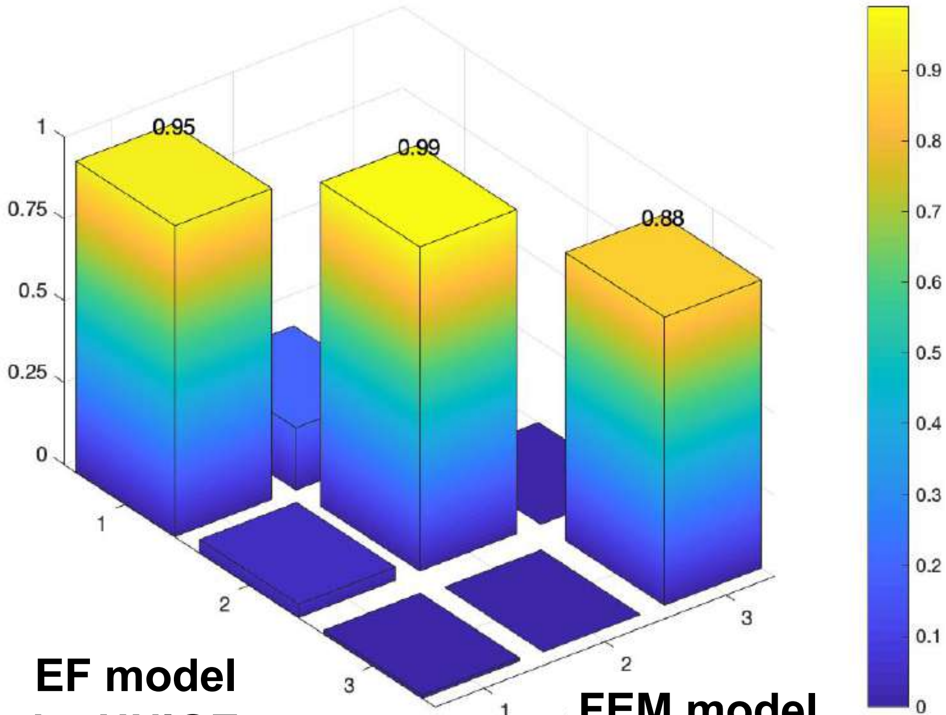
CALIBRATION OF THE MODEL IN ELASTIC FIELD

FEM Model by UNIPD - Diana FEA



EF Model by UNIGE – Tremuri
(Lagomarsino et al. 2013)

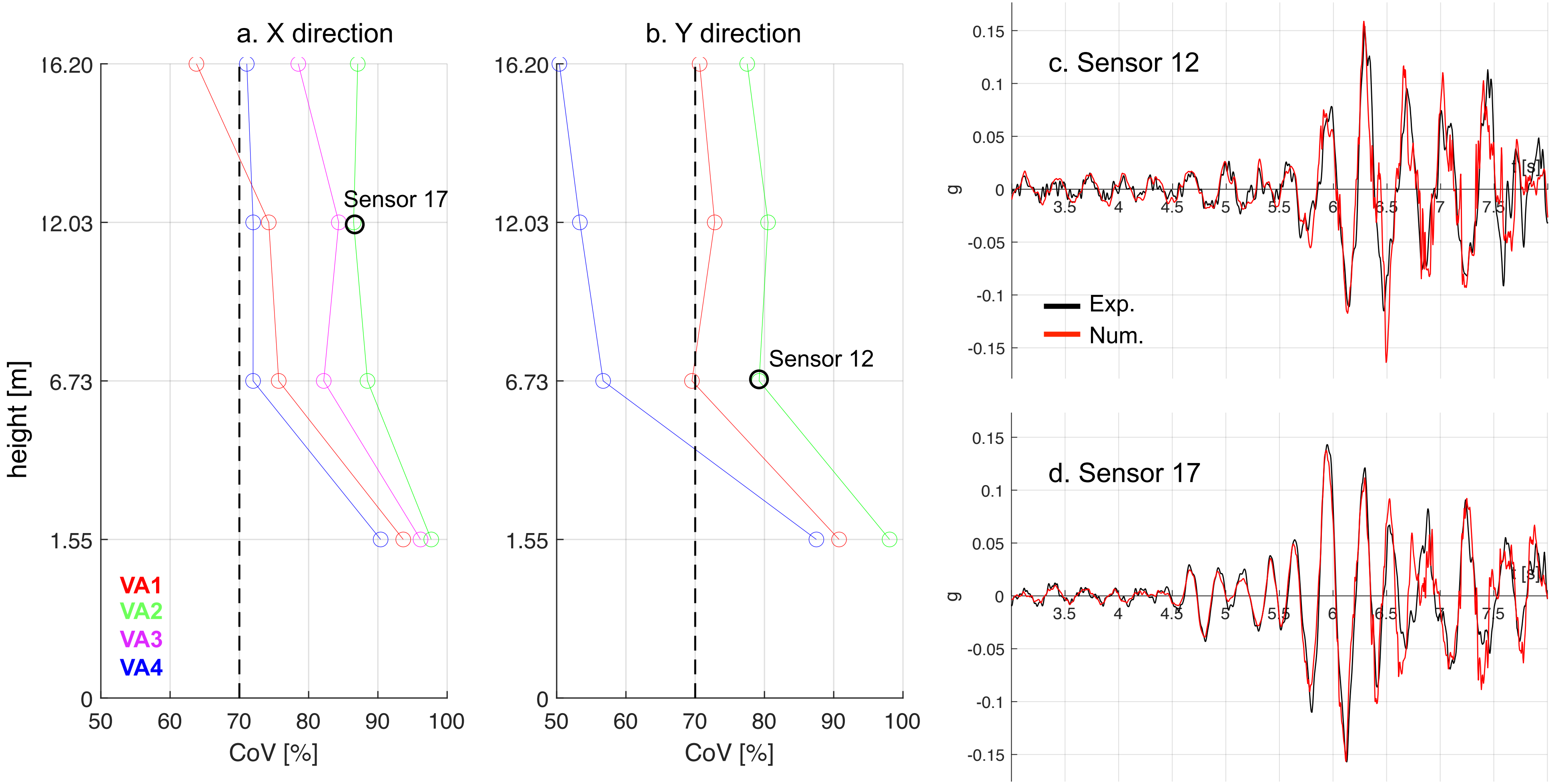




With analogous hypotheses and by adopting mechanical parameters with a maximum difference of 20%

Fabriano Courthouse

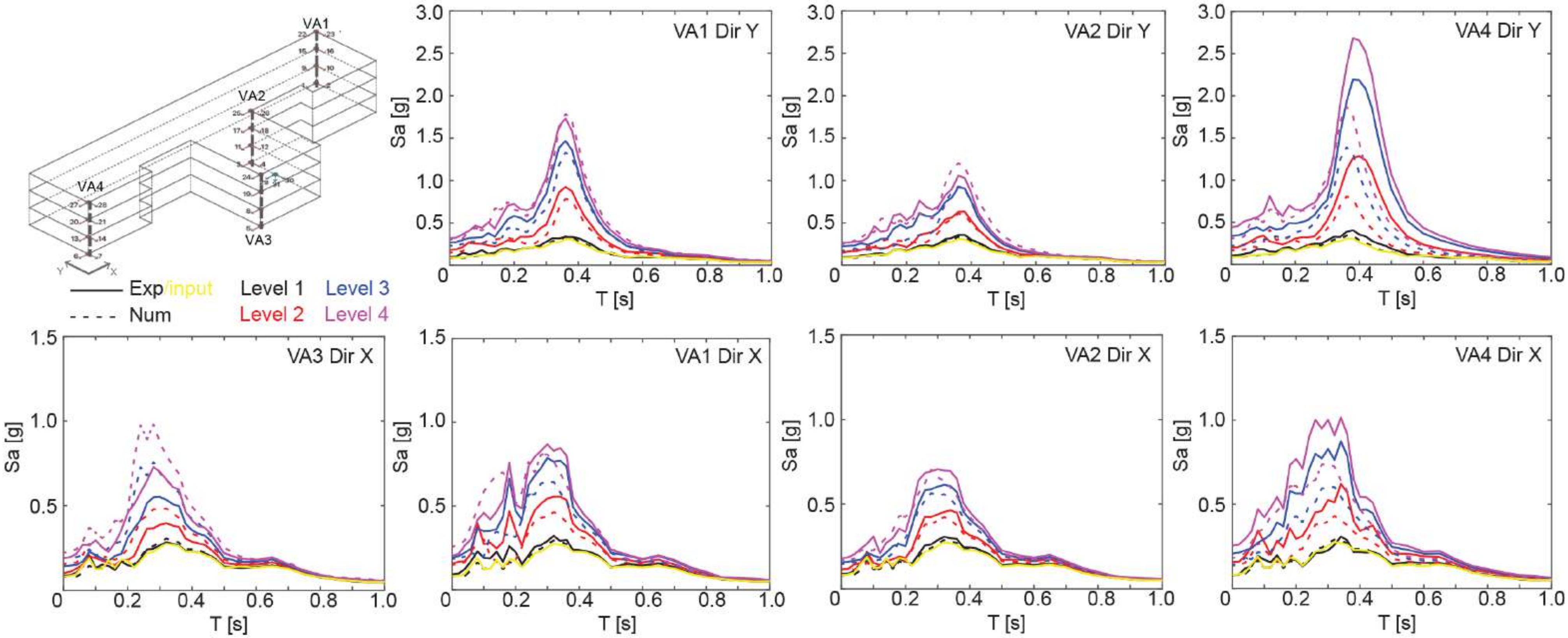
VALIDATION OF THE MODEL IN NONLINEAR FIELD – simulation of the actual response by performing nonlinear dynamic analyses



REF: Cattari et al. (2019) Calibrazione e validazione di modelli numerici da dati di monitoraggio permanente: il caso studio del Tribunale di Fabriano monitorato dall'Osservatorio Sismico delle Strutture, XVIII ANIDIS Conference, Ascoli Piceno.

Fabriano Courthouse

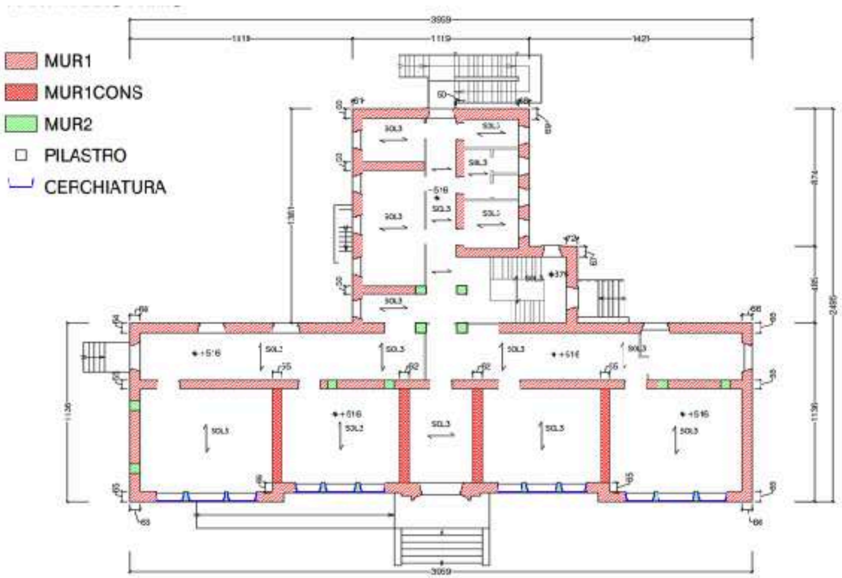
VALIDATION OF THE MODEL IN NONLINEAR FIELD – simulation of the actual response by performing nonlinear dynamic analyses



REF: Cattari et al. (2019) Calibrazione e validazione di modelli numerici da dati di monitoraggio permanente: il caso studio del Tribunale di Fabriano monitorato dall'Osservatorio Sismico delle Strutture, XVIII ANIDIS Conference, Ascoli Piceno.

The school of P. Capuzi in Visso

- Built in 1930
- Interstory heigth: 4.00 m
- Gross area:520 m^2
- T-shaped plan
- 3 story building
- Rigid floors
- Stone masonry



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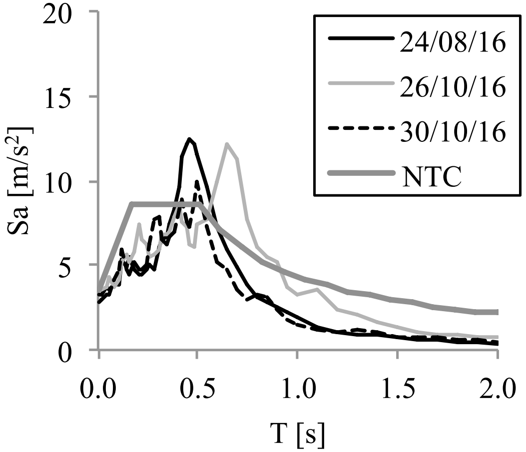
- Significant damage accumulation effects after the subsequent shocks

X direction

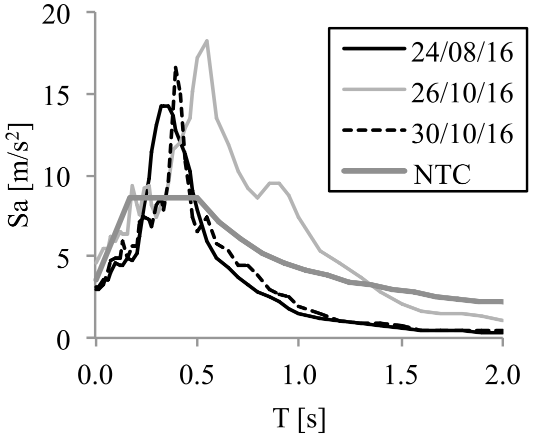


24/8

26/10

30/10

Y direction



T_R=712 years - Soil C

24/8

26/10

30/10

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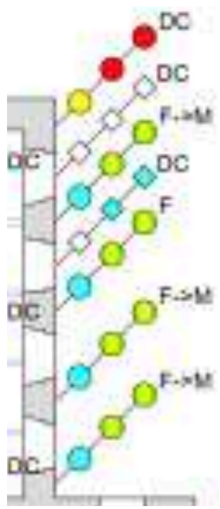
- Reconstruction of damage accumulation effects through the photos available from DPC and the survey made by UNIGE RU

Final damage at 8/12/2016

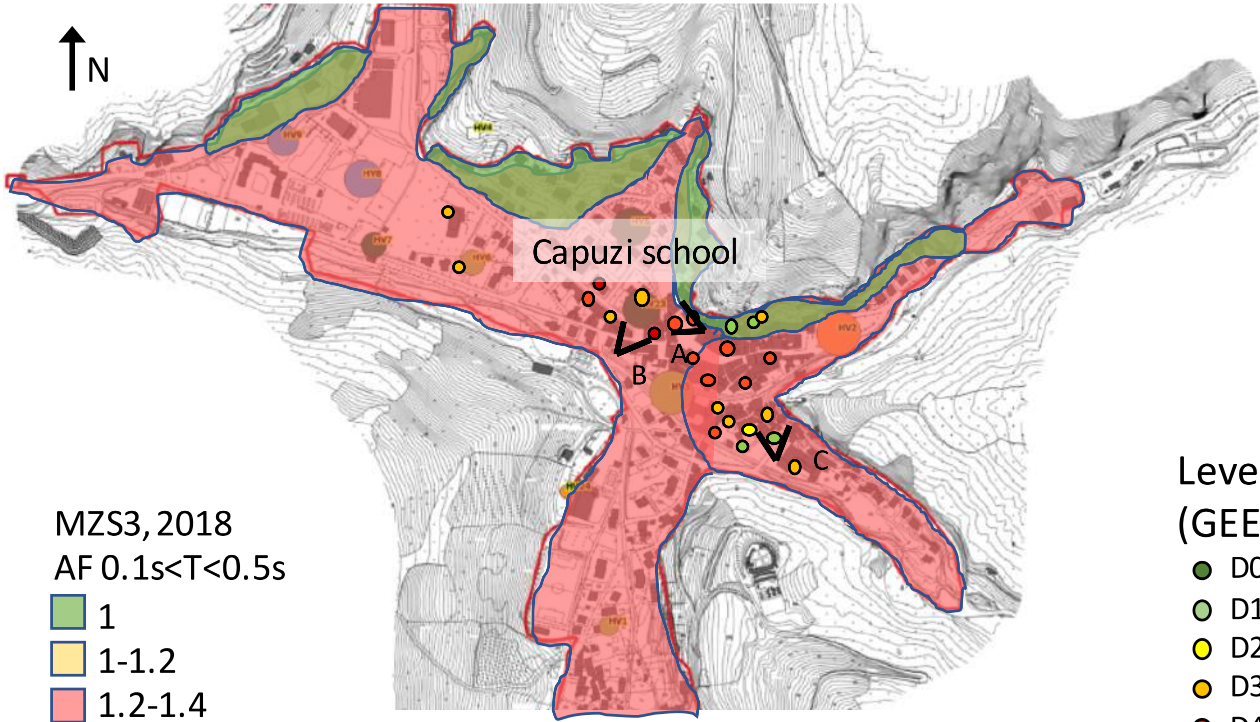
Data on damage accumulation at scale of each element



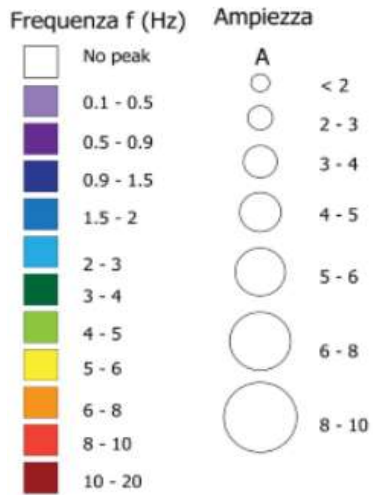
- ✓ The three dots refer to the three main events:
  - 24/08/2016
  - 26/10/2016
  - 30/10/2016
- ✓ The colour refer to the damage level
- ✓ The code (DC –*Diagonal cracking*; F – *flexural*; M – *mixed*) refers to the prevailing failure mode

VALIDATION ON MONITORED URM BUILDINGS

- Topographic & stratigrafic site amplifications were recognized at urban scale



HVSR MZS3, 2018



Level of damage after 24° August 2016 (GEER 1, 2017)

- D0-No damages
- D1-Cracking on non-structural elements
- D2-Major damage to non-structural elements
- D3-Significant damage to load-bearing elements
- D4-Prtial structural collapse
- D5-Full collapse

A

B

C

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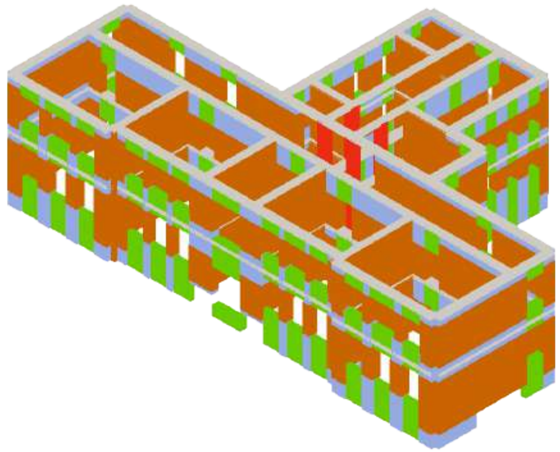
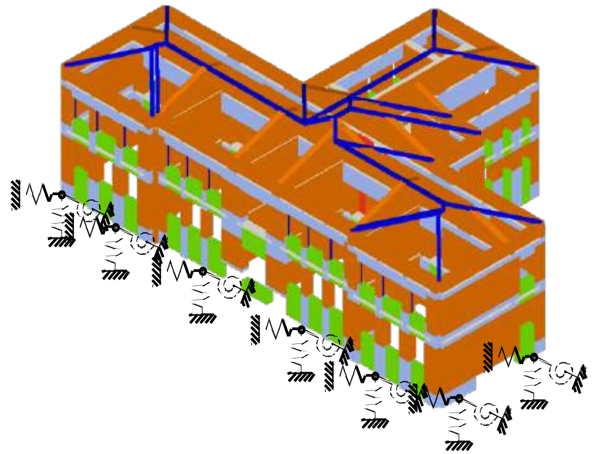
CALIBRATION OF THE MODEL IN ELASTIC FIELD

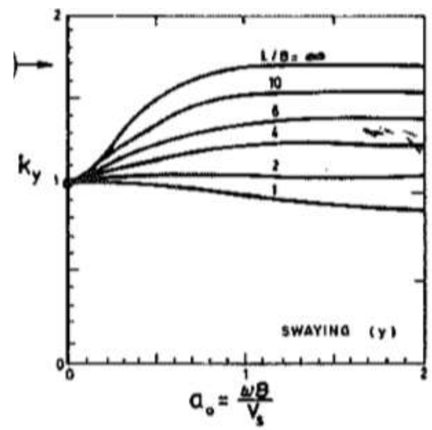


FIXED- BASE MODEL



COMPLIANT- BASE MODEL

	Dynamic identification procedure				Back-analysis			
	Impedances (G_0)				Impedances (G_{deg})			
	Area I		Area II		Area I		Area II	
	$\mu_{area\ I}$	σ_{area_I}	$\mu_{area\ II}$	σ_{area_II}	$\mu_{area\ I}$	σ_{area_I}	$\mu_{area\ II}$	σ_{area_II}
K_x(kN/m)	2.9E+05	0.32	7.9E+05	0.21	2.2E+05	0.36	3.5E+05	0.64
K_y(kN/m)	4.1E+05	0.45	6.7E+05	0.12	3.0E+05	0.44	2.3E+05	0.43
K_z(kN/m)	5.9E+05	0.44	1.4E+06	0.41	3.7E+05	0.44	3.7E+05	0.90
$K_{r,x}$(kNm)	3.5E+05	1.85	2.3E+06	0.08	2.5E+05	1.85	2.7E+05	1.37
$K_{r,y}$(kNm)	2.8E+05	1.38	4.2E+06	0.58	2.0E+05	1.38	6.3E+05	1.97



(Gazetas 1991)

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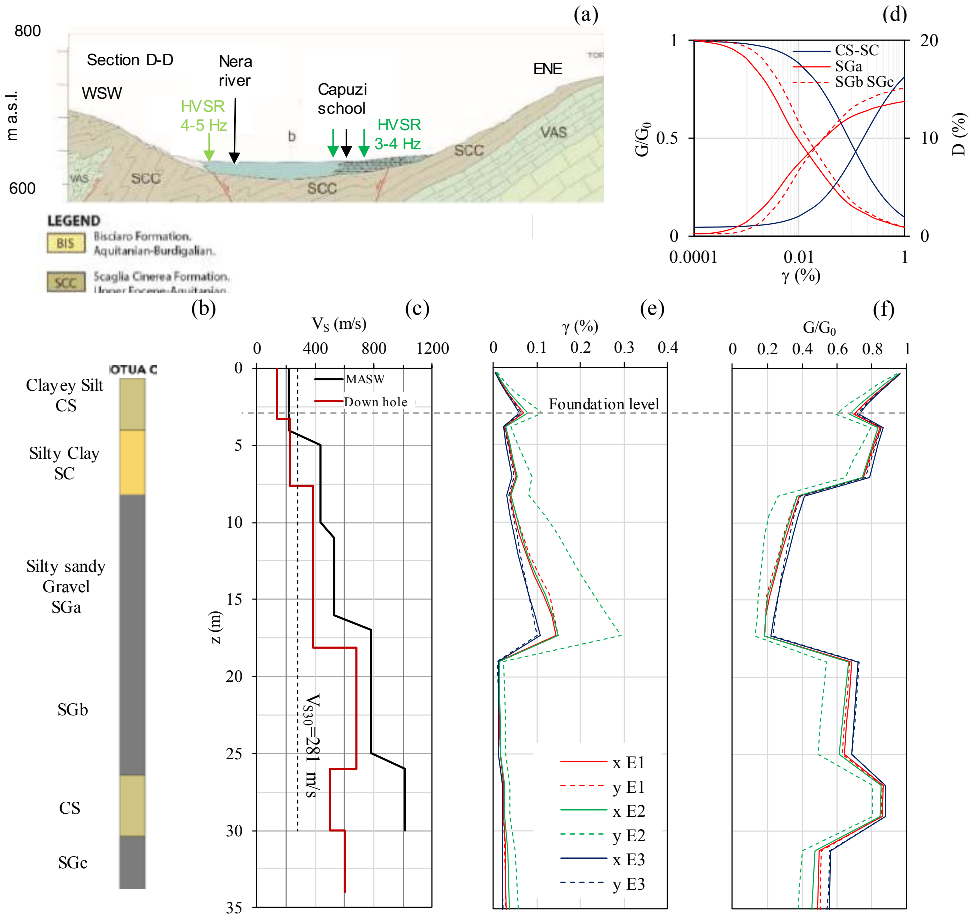
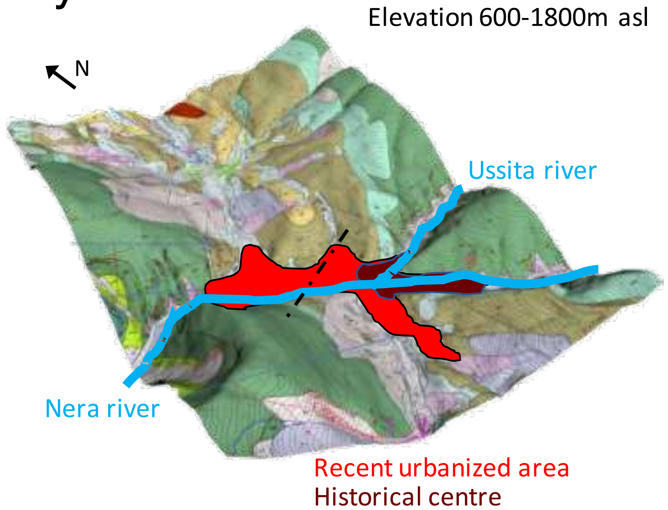
- Soil characterization in both linear and nonlinear field by prof. F. Silvestri & F.de Silva from University of Napoli and S.Sica from Univerisity of Sannio





GC = Gravel & Clay
SCC = Scaglia Cinerea, VAS = Scaglia Variegata

	z_{min} (m)	z_{max} (m)	V_S (m/s)	G (MPa)
CS_a	0	3.2	136	38
SC_b	3.2	8	226	104
SG_a	8	18	383	314
SG_b	18	26	683	999
CS_b	26	30	500	510
SG_c	30	40	602	776
Bedrock	40	–	1300	3790

REF: F. de Silva, A. Piro, A. Brunelli, S. Cattari, F. Parisi , S. Sica, F. Silvestri , On the Soil-structure interaction in the seismic response of a monitored masonry school building struck by the 2016-2017 Central Italy earthquake, Proc. of COMPDYN conference 2019. Crete 24-26 June 2019, 2019.

REF:S. Cattari, D. Sivori, A. Brunelli, S. Sica, A. Piro, F. de Silva, F. Parisi, F. Silvestri, Soil-structure interaction effects on the dynamic behaviour of a masonry school damaged by the 2016-2017 Central Italy earthquake sequence, Proceedings of the 7th International Conference on Earthquake Geotechnical Engineering (VII ICEGE). Rome, Italy, June 17-20, 2019

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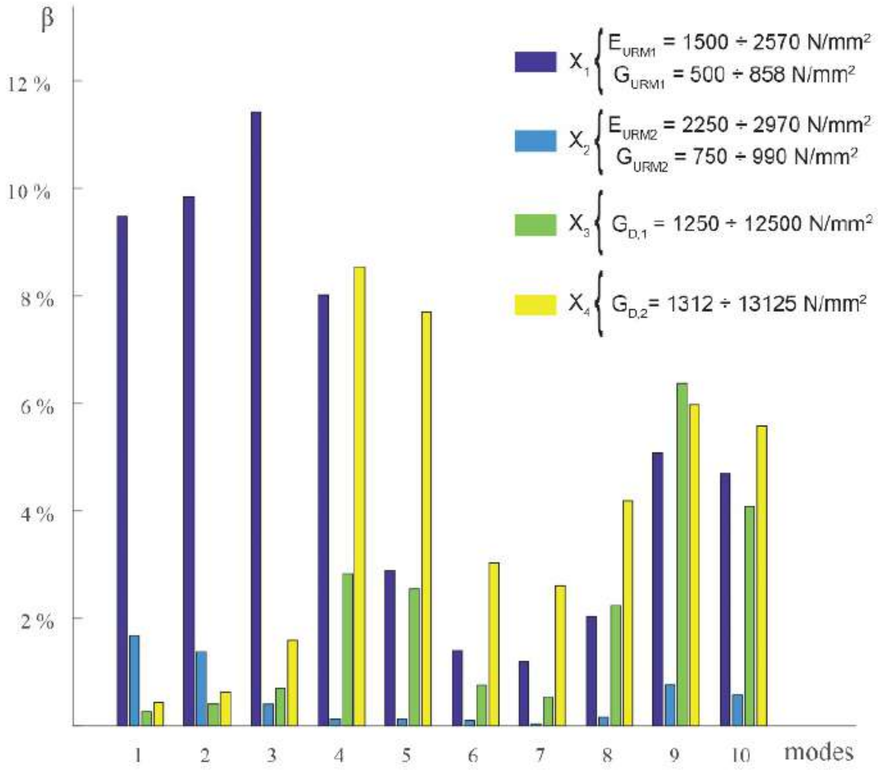
CALIBRATION OF THE MODEL IN ELASTIC FIELD



Random variables :

- elastic and shear moduli of masonry
- shear modulus of the equivalent membrane that simulate the actual stiffness of diaphragms

	URM_1 (strengthened)	URM_2	SOL_1	SOL_2
E [MPa]	2574 (2970)	2701	/	/
G [MPa]	858 (991)	901	12500	26300
τ_0 [MPa]	0.096 (0.111)	0.114	/	/
f_m [MPa]	4.94 (5.7)	4.8	/	/

τ_0: diagonal shear strength of masonry; f_m: Compressive strength of masonry

Epistemic uncertainties:

- Actual height of foundation system
- flange effect (coupling effectiveness between orthogonal walls)
- roof modelling
- stiffness of r.c. tie beams

+ Role of soil on the structural response

VALIDATION ON MONITORED URM BUILDINGS

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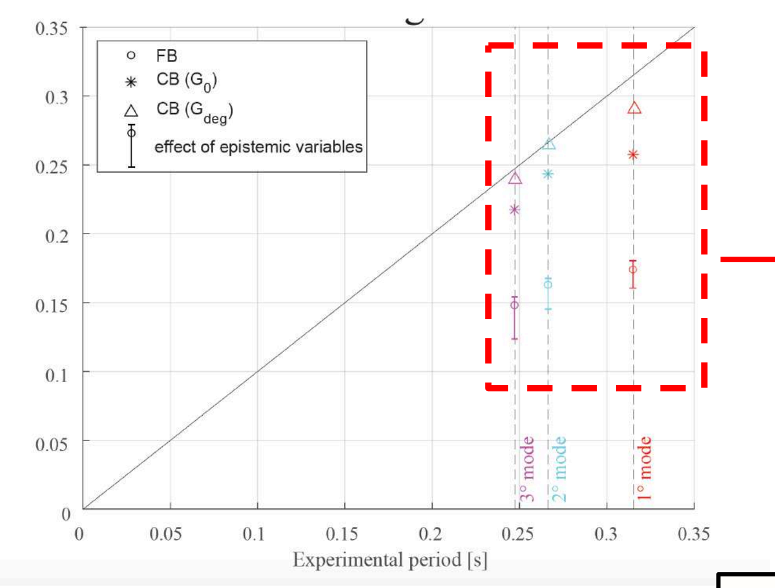
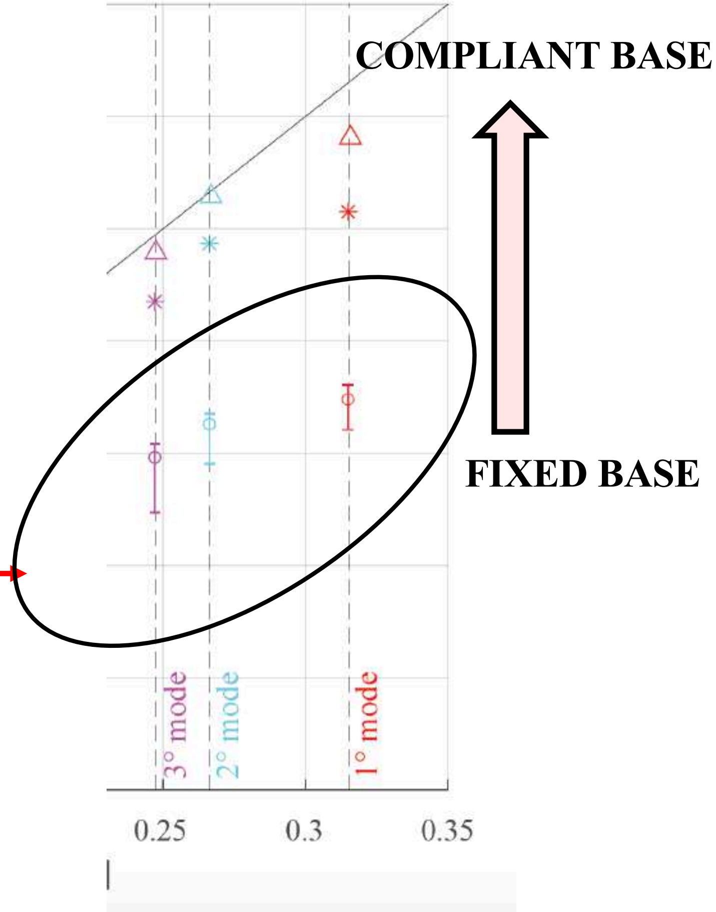
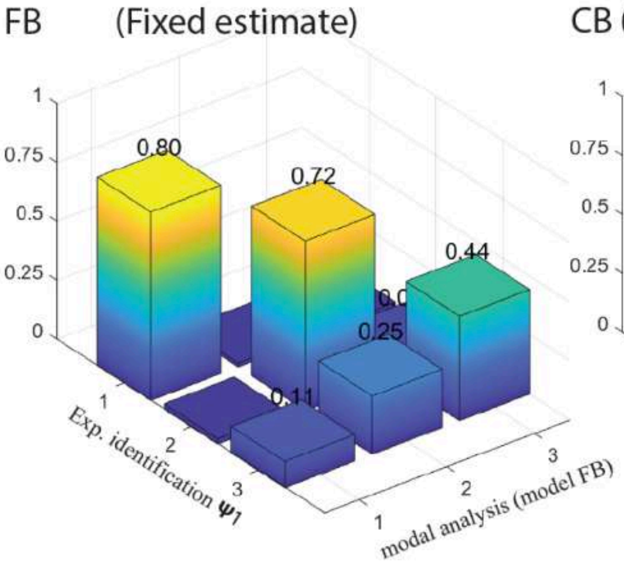
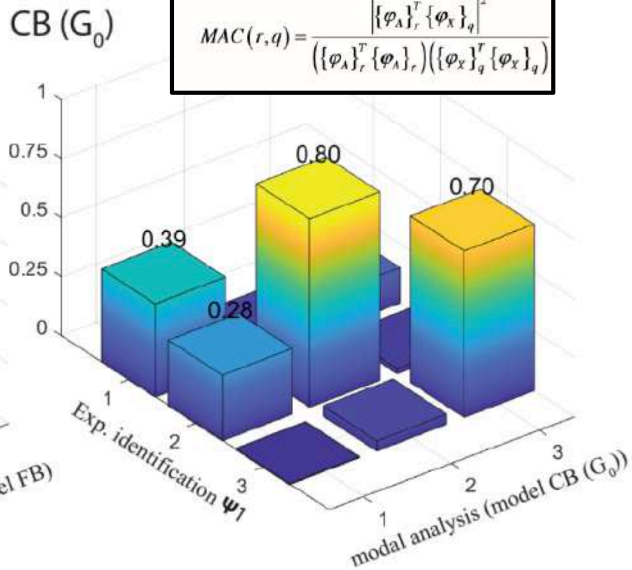
THE MODEL IN ELASTIC FIELD





$$MAC(r,q)=\frac{\left|\{\varphi_A\}_r^T\{\varphi_X\}_q\right|^2}{\left(\{\varphi_A\}_r^T\{\varphi_A\}_r\right)\left(\{\varphi_X\}_q^T\{\varphi_X\}_q\right)}$$





	1° mode T [s]	2° mode T [s]	3° mode T [s]
Ψ2	0.315	0.267	0.247

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VALIDATION OF THE MODEL IN NON LINEAR FIELD

Pier/Spandrel	SHEAR (S)				FLEXURAL (F)			
	Backbone curve		Hysteretic response		Backbone curve		Hysteret response	
	$\theta_{i,S}$ [%]	$\beta_{E,i}$ [%]	c_1	0.8/0.2	$\theta_{I,F}$ [%]	$\beta_{E,i}$ [%]	c_1	0.9/0
DL3	0.45/[*]	0.6/0.7	c_2	0.8/0	0.48/0.35	1	c_2	0.8/
DL4	0.70/1.50	0.2/0.7	c_3	0/0.3	0.8/1.50	0.85/0.70	c_3	0.6/0
DL5	2.23/2.00	0/0			2.72/2.00	0/0	c_4	0.5/0

(*) in case of spandrels, θ_3 has been defined starting from the value of drift corresponding to the yielding point of the element and assuming then a ductility equal to 4, similarly to what suggested in Beyer et al., 2014.

ACCORDING TO EXPERIMENTAL DATA & REFERENCE VALUE OF LITERATURE !!

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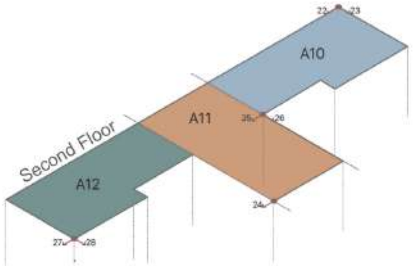
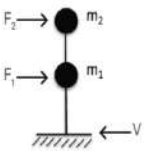
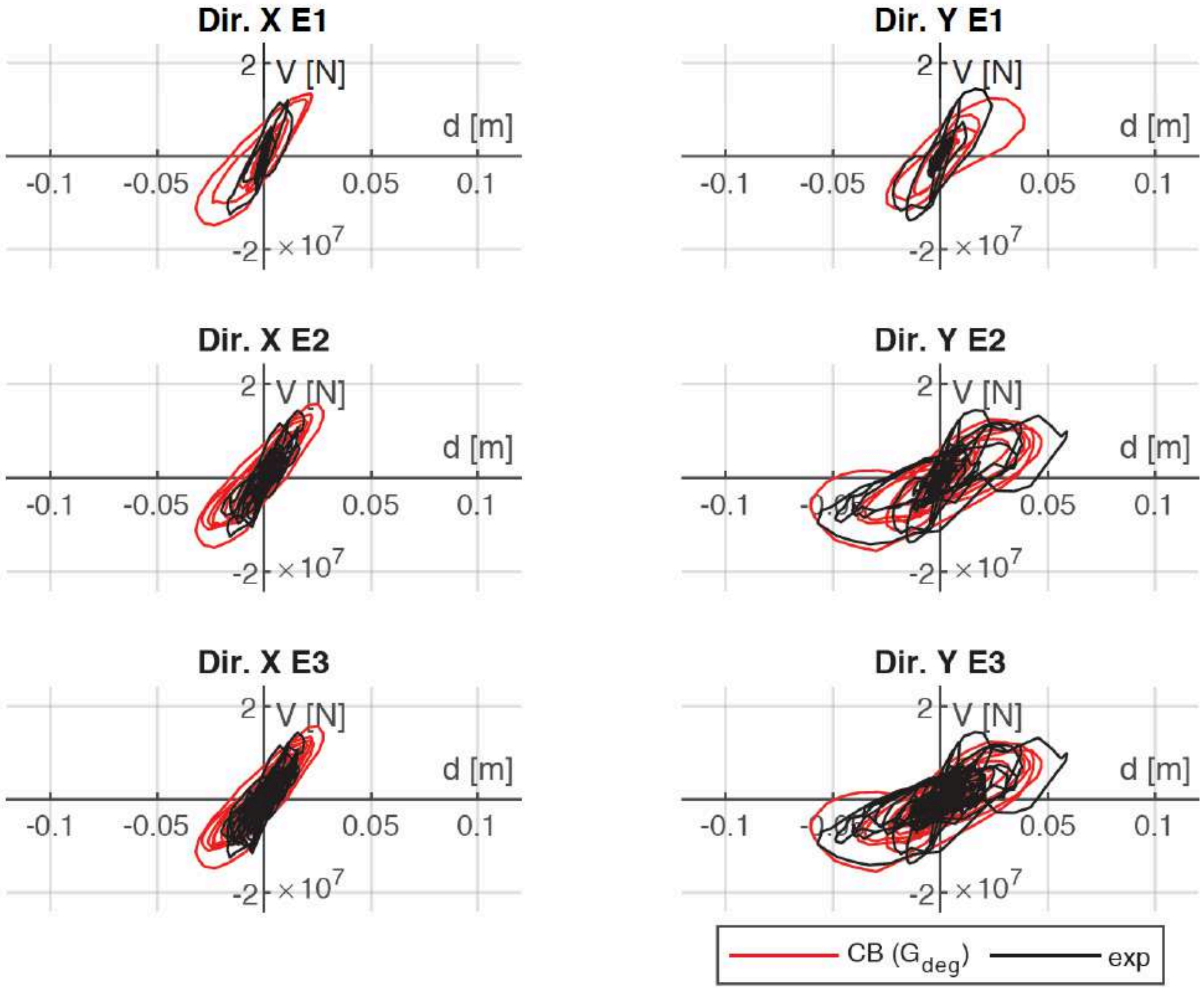
VALIDATION IN NON LINEAR FIELD - INERTIAL FORCES ACTIVATED



$$V = \sum m_i \cdot a_i$$

$$d = \frac{\sum m_{i,4} \cdot d_{i,4}}{\sum m_{i,4}}$$





Brunelli et al. (2020) NUMERICAL SIMULATION OF THE SEISMIC RESPONSE OF A MONITORED MASONRY SCHOOL BUILDING STRUCK BY THE 2016-2017 CENTRAL ITALY EARTHQUAKE INVESTIGATING THE ROLE OF THE SOIL-STRUCTURE INTERACTION, BULLETIN OF EARTHQUAKE ENGINEERING, TO BE SUBMITTED.

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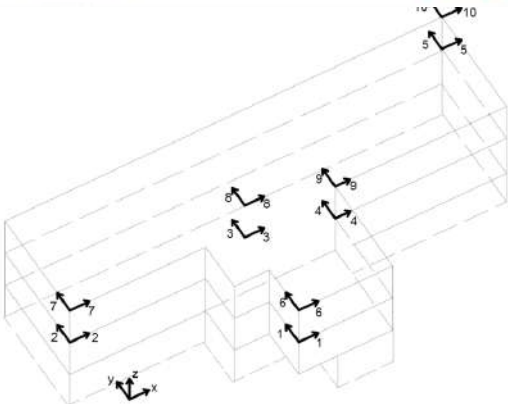
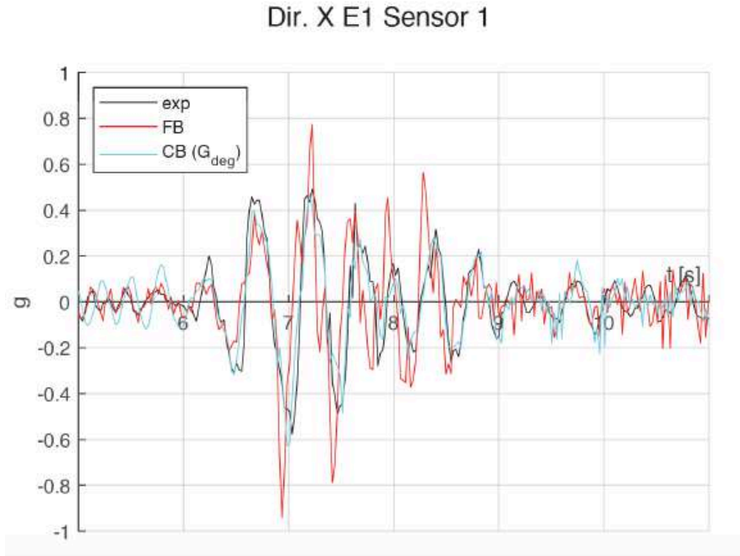
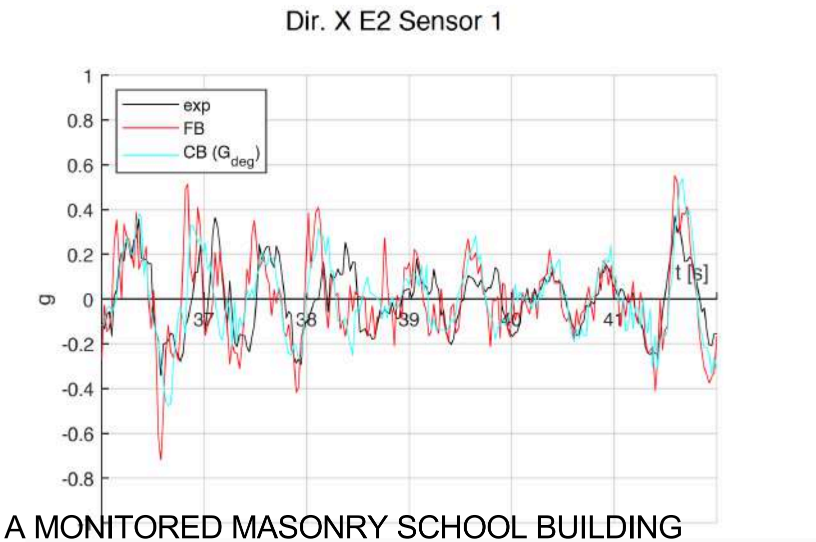
THE MODEL IN NON LINEAR FIELD – SIMULATED ACCELEROGRAMS

	FB								CB (G_{deg})							
	1st floor				2nd floor				1st floor				2nd floor			
	E1	E2	E3		E1	E2	E3		E1	E2	E3		E1	E2	E3	
1	75%	61%	63%	6	84%	47%	56%	1	92%	65%	62%	6	85%	58%	44%	Dir X
2	82%	67%	55%	7	84%	79%	65%	2	83%	89%	81%	7	83%	88%	79%	
3	89%	84%	76%	8	86%	82%	73%	3	85%	92%	88%	8	82%	88%	85%	
4	85%	68%	70%	9	83%	82%	76%	4	86%	78%	87%	9	82%	86%	84%	
5	69%	66%	57%	10	66%	63%	58%	5	71%	76%	71%	10	66%	70%	70%	
1	64%	50%	21%	6	62%	45%	21%	1	80%	76%	36%	6	75%	68%	56%	Dir Y
2	30%	28%	18%	7	37%	31%	18%	2	49%	67%	50%	7	49%	58%	53%	
3	78%	75%	48%	8	79%	71%	53%	3	88%	86%	70%	8	89%	79%	74%	
4	79%	76%	43%	9	84%	66%	43%	4	90%	86%	65%	9	88%	75%	81%	
5	77%	62%	33%	10	78%	65%	34%	5	81%	75%	58%	10	79%	73%	57%	

CoV ≥ 70%   40 % ≤ CoV < 70%   CoV < 40%







Brunelli et al. (2020) NUMERICAL SIMULATION OF THE SEISMIC RESPONSE OF A MONITORED MASONRY SCHOOL BUILDING STRUCK BY THE 2016-2017 CENTRAL ITALY EARTHQUAKE INVESTIGATING THE ROLE OF THE SOIL-STRUCTURE INTERACTION, BULLETIN OF EARTHQUAKE ENGINEERING, TO BE SUBMITTED.

VALIDATION ON MONITORED URM BUILDINGS

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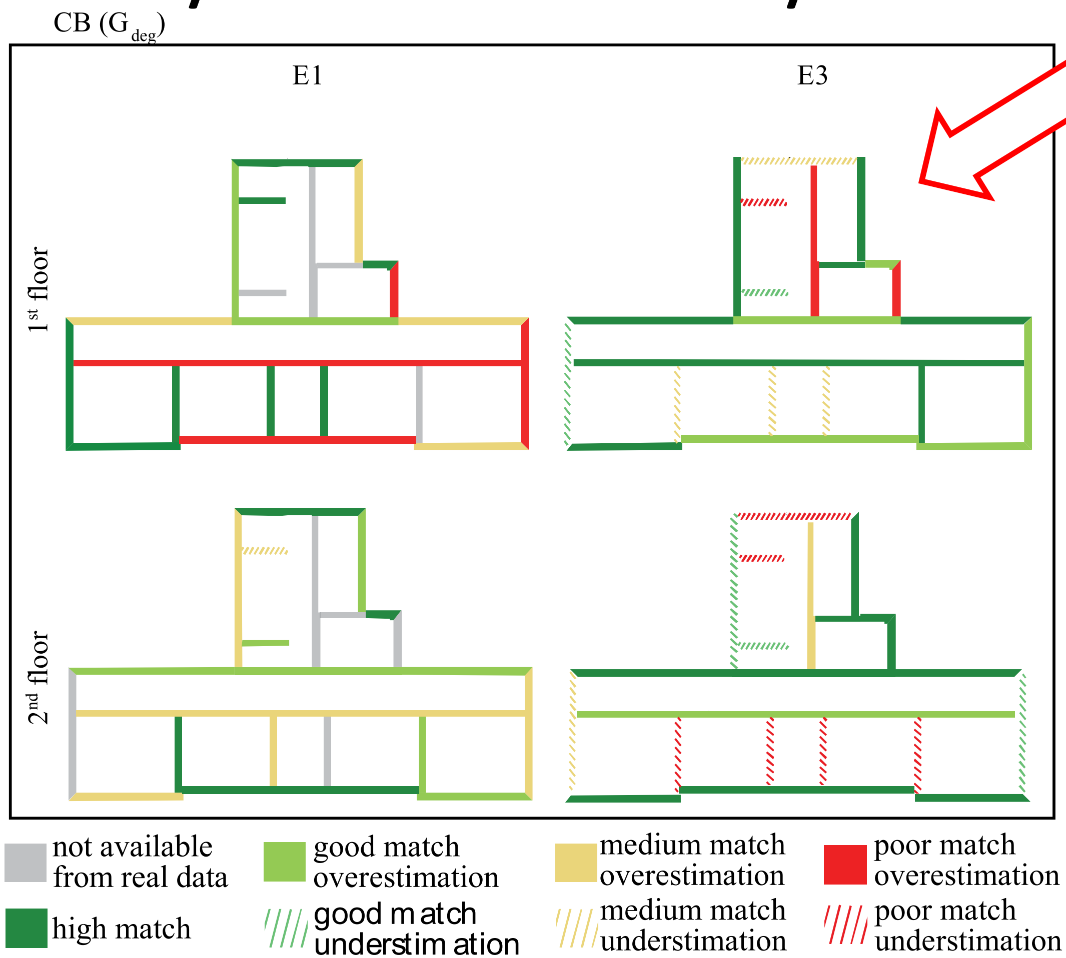
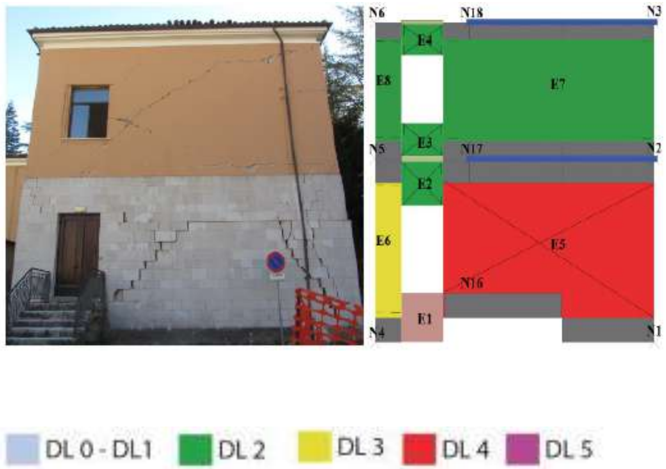
THE MODEL IN NON LINEAR FIELD

24/08 **30/10**



After E3 – 30/10



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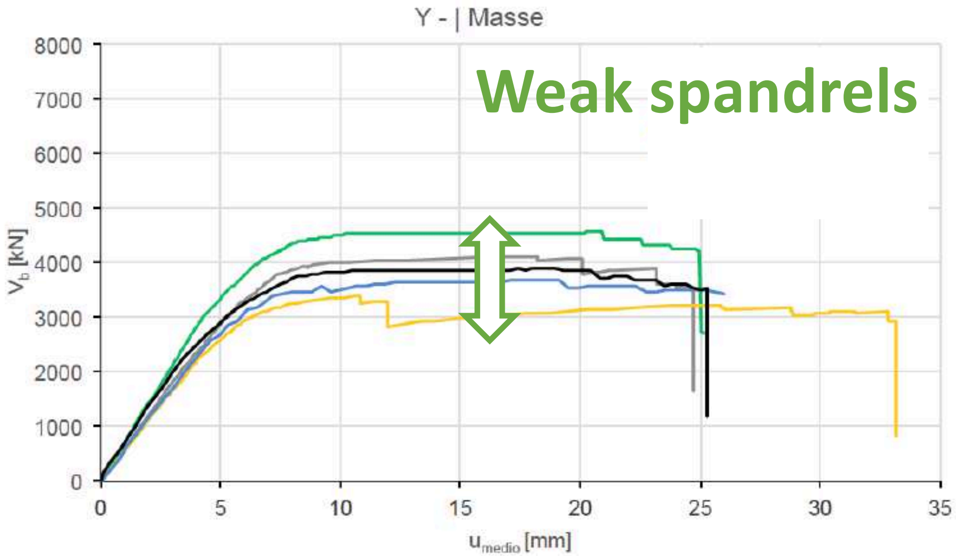
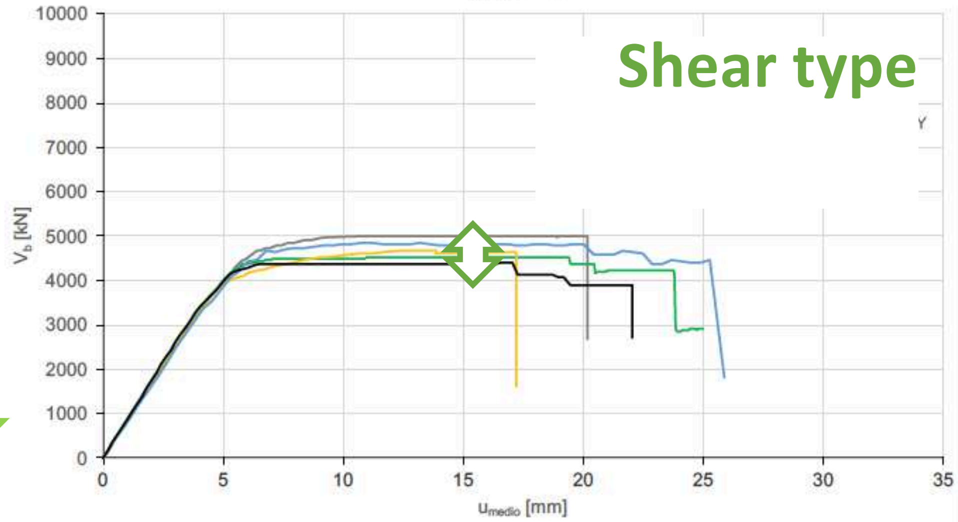
…WHAT ABOUT THE RELIABILITY OF BLIND PREDICTION ?

Task 10.3 of ReLUIS project 2019: Numerical Models **Reliability** through Benchmark Structures:

Pizzoli case-study

Decreasing dispersion on results

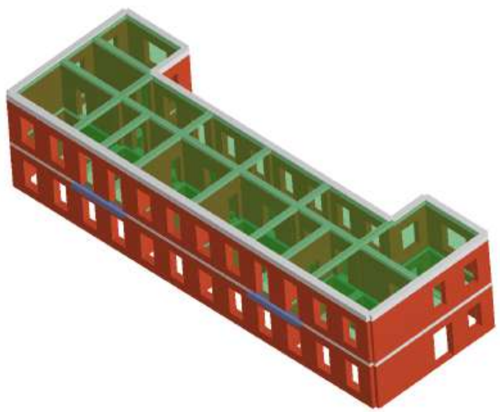




reluis

URM Benchmark Workgroup

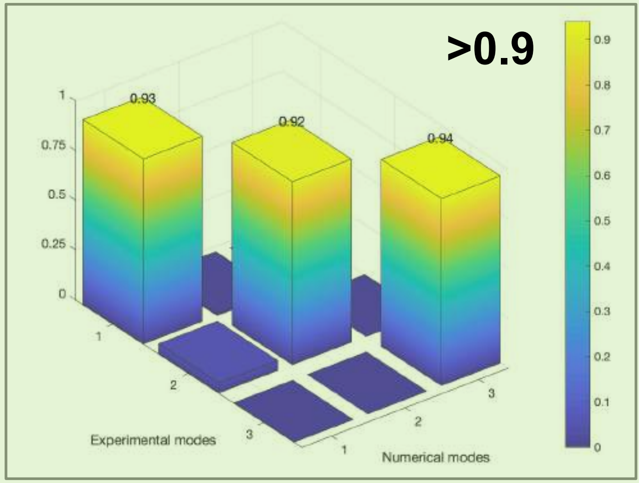
- **Config. C: strong spandrels – spandrels coupled to r.c. ring beam**



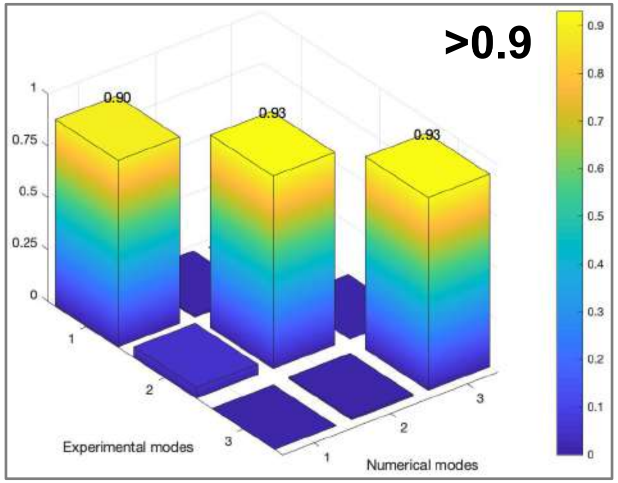
…WHAT ABOUT THE RELIABILITY OF BLIND PREDICTION ?

Equivalent frame models

3Muri **UNIGE**



Andilwall

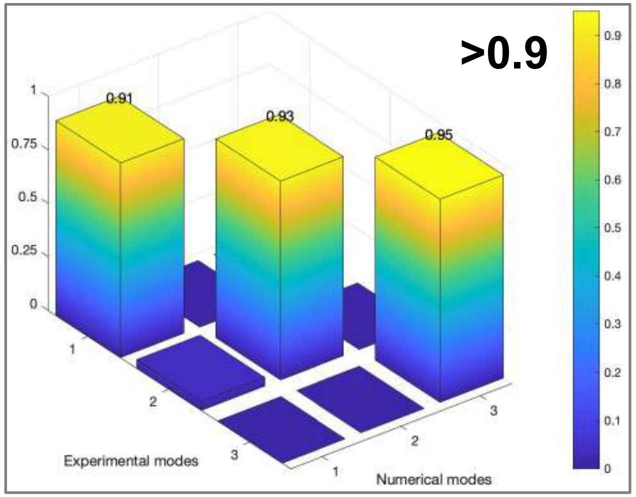


Parameters adopted for the“*blind prediction*” *by the BENCHMARK WORKGROUP*

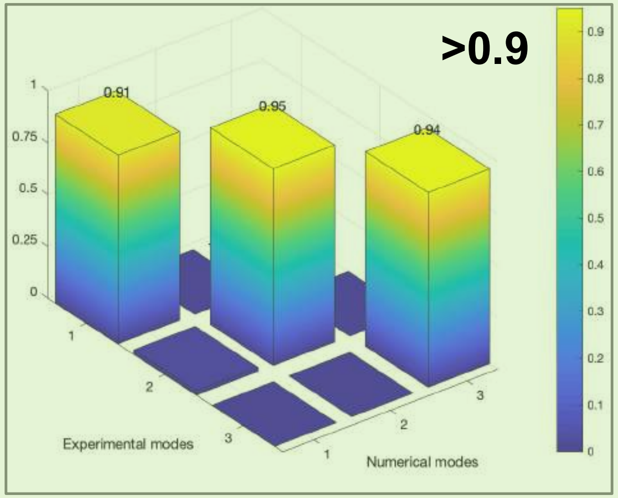
E= 2262 MPa
G=754 MPa

Uncracked Values for stone masonry proposed by NTC 2008 (mean values plus meliorative coefficients)

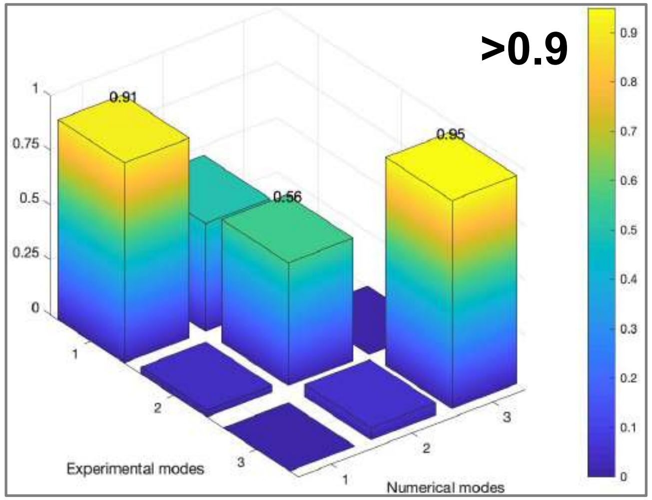
Aedes



Midas **UNICH-PE**



Cds

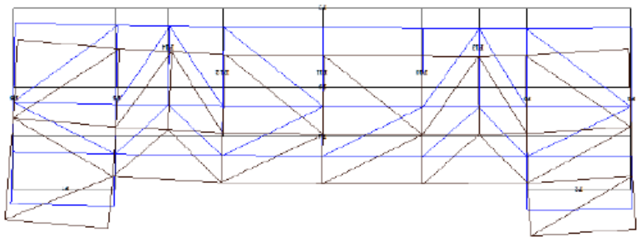


…WHAT ABOUT THE RELIABILITY OF BLIND PREDICTION ?

Parameters	*Blind Prediction*	UNIGE RU	UNICH-PE RU
E [Mpa]	2262	2488	2550
G [Mpa]	754	829	840

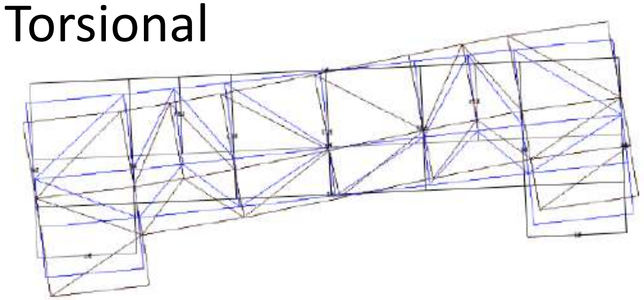
Y-Traslational - **85% Mass in Y**

Mode 1



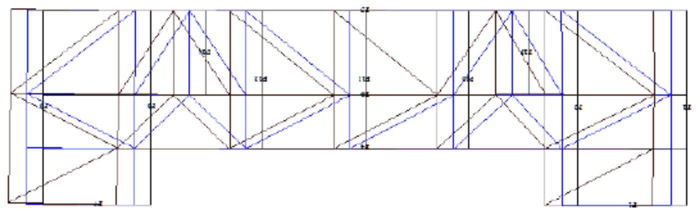
Torsional

Mode 2

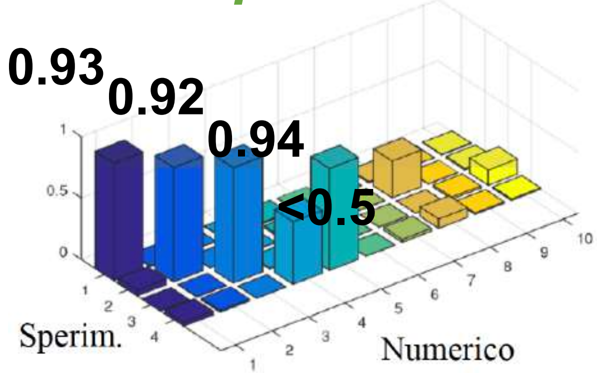


X-Traslational - **89% Mass in X**

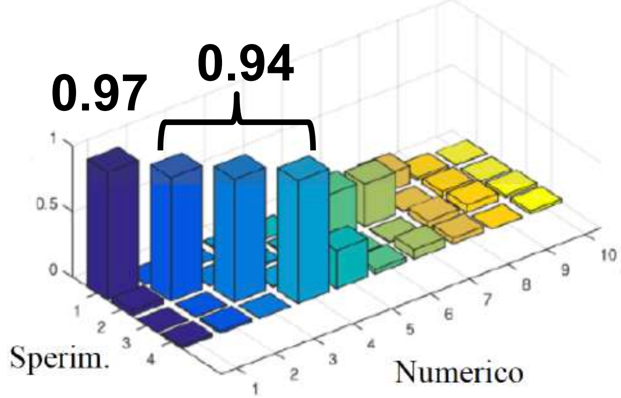
Mode 3



Blind prediction



Using cracked stiffness for rc slabs



…WHAT ABOUT THE RELIABILITY OF BLIND PREDICTION ?

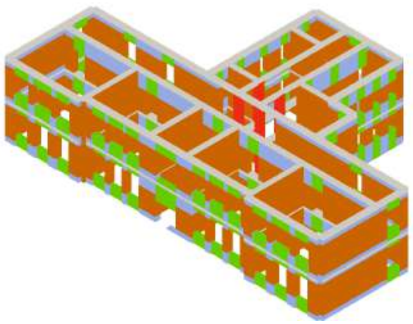
Equivalent frame models (6)

A B C D E F

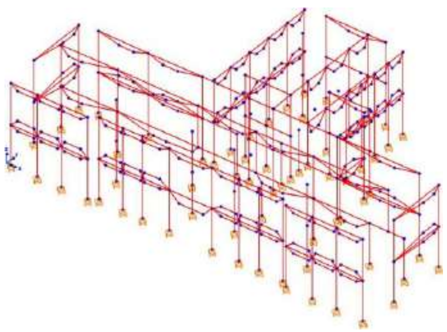
Finite element models (2) & Plane discrete macro models (1)

G H I

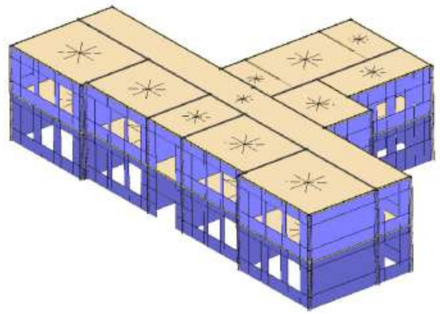
3Muri



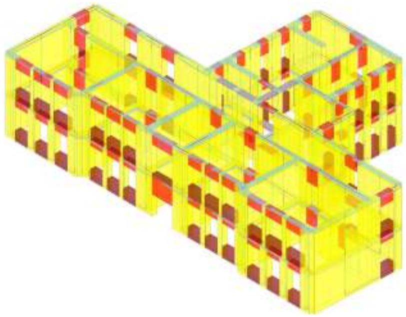
SAP 2000



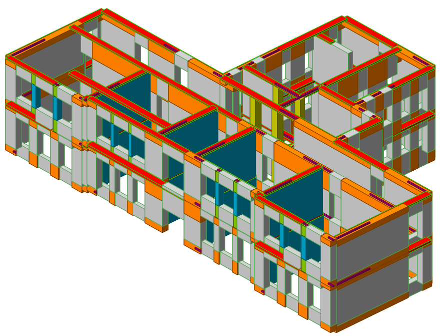
3DMacro



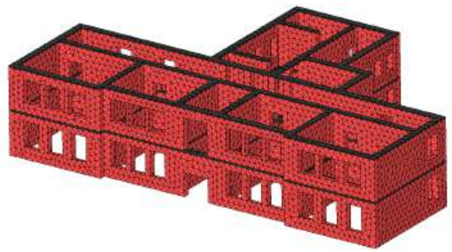
AEDES



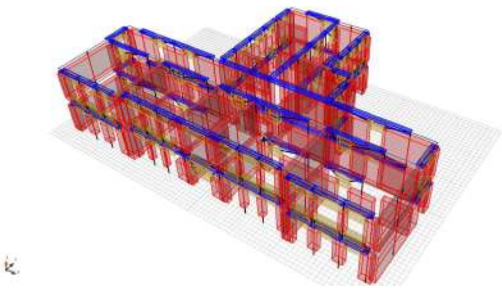
MIDAS-GEN (Plasticità concentrata)



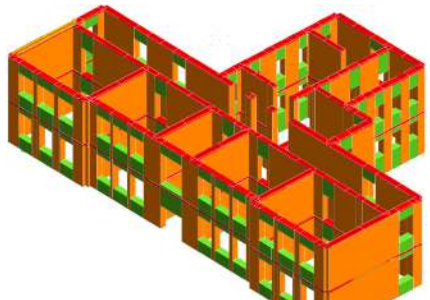
ABAQUS



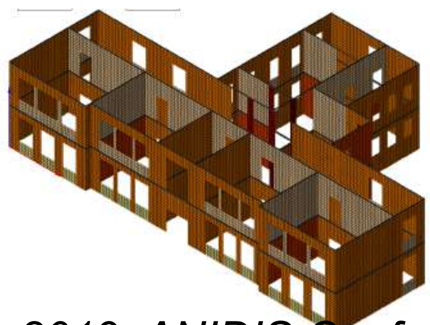
ANDILWALL



CDS

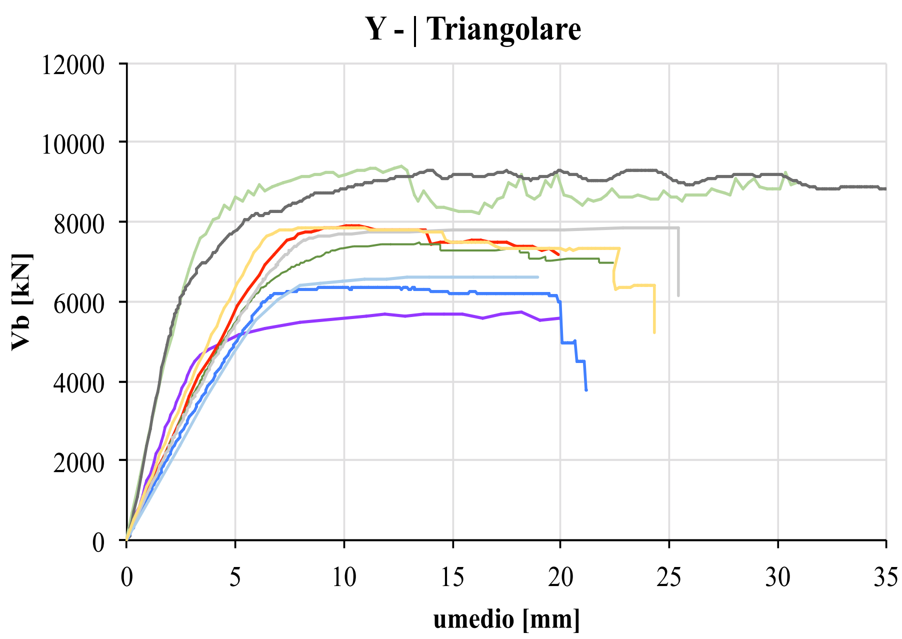


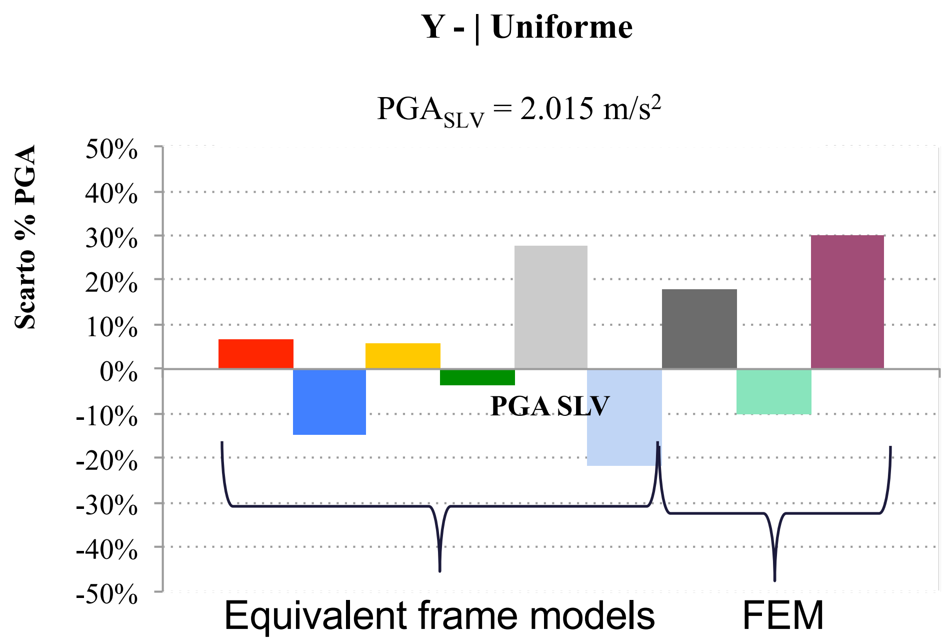
MIDAS-FEA



REF: Cattari et al. 2019, ANIDIS Conference

…WHAT ABOUT THE RELIABILITY OF BLIND PREDICTION ?



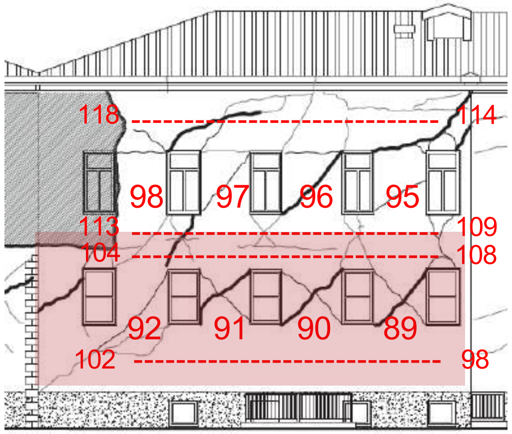


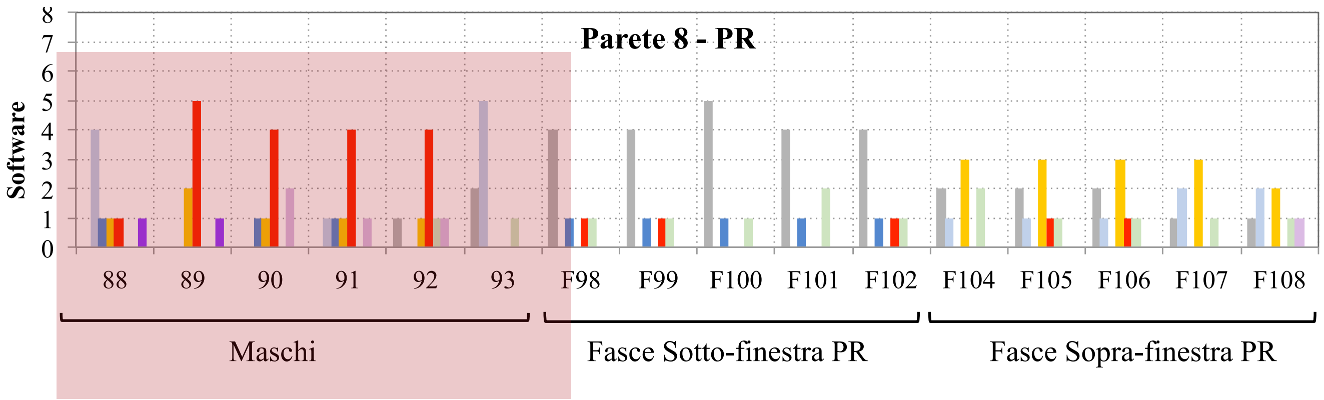
REF: Cattari et al. 2019, Uso dei codici di calcolo per l'analisi sismica nonlineare di edifici in muratura: confronto dei risultati ottenuti con diversi software su un caso studio reale, *ANIDIS Conference*

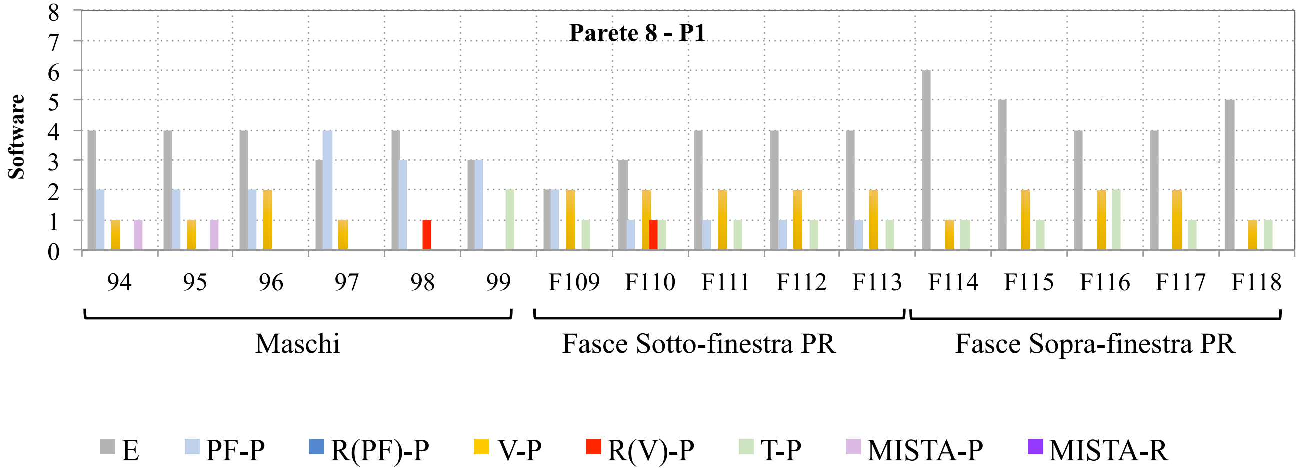
…WHAT ABOUT THE RELIABILITY OF BLIND PREDICTION ?

Figures illustrate the number of software in agreement in predicting the same failure mode at element scale (the colour indicate the prevailing failure mode if flexural or diagonal cracking)

Y+ | UNIFORM







PF= Flexural ; V= diagonal cracking; P= plastic; R=attainment of the ultimate drift

REF: Cattari et al. 2019, ANIDIS Conference

FINAL REMARKS

- IMPORTANCE TO BE AWARE OF THE HYPHOTESES WHICH THE MODEL ADOPTED IS FOUNDED ON IN ORDER TO ASSESS ITS RELIABILITY WITH RESPECT THE BUILDING EXAMINED

- CAPABILITY OF EQUIVALENT FRAME MODEL TO SIMULATE THE NONLINEAR RESPONSE OF URM 3D BUILDINGS WITH A LIMITED COMPUTATIONAL EFFORT (ALSO TO PERFORM NONLINEAR DYNAMIC ANALYSES)

- USE OF NONLINEAR ANALYSIS AS TOOL TO ASSESS THE PLAN OF INVESTIGATION (BY POINTING OUT THE PARAMETERS THAT MOST AFFECT THE RESPONSE ON WHICH CONCENTRATE TESTS AND SURVEY) AND TO ORIENTATE THE CHOICE OF MOST EFFECTIVE STRENGTHENING INTERVENTIONS

THANK YOU FOR YOUR KIND ATTENTION!

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