Second Generation of Eurocode 8

Assessment and retrofit of masonry buildings

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Seismic behaviour of masonry buildings

- Masonry buildings are complex and vulnerable to earthquakes
- Non-engineered structures
- They form a large part of the existing building stock in Europe

Seismic analysis in codes and distinctive features of the seismic assessment

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

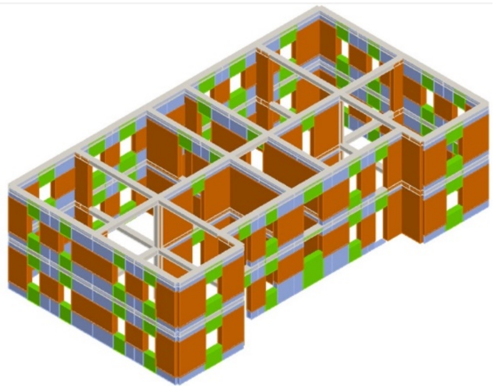
- **DESIGN (EC8 1.2)**
  I conceive the structure by a capacity design and use details that guarantee the assumed ductility level. I don't need nonlinear models to do that.

- **ASSESSMENT (EC8 3)**
  I evaluate the actual building performance by a model as close as possible to the real behaviour. Nonlinear models are needed as they don't assume a predefined capacity. Linear model makes assumptions largely cautionary.

Linear Static Analysis (force-based approach)

Application Check of EC8 (ReLUIS RINTC project)

- 2 storeys building, regular in plan and elevation, placed in Milan (Italy)
- Lateral force method with q-factor approach
- Verification of all masonry panels:
  - mainly vertical loads with eccentricity
  - in-plane shear resistance of piers and spandrels
  - flexural resistance of piers (bending and compression)



	Top section of piers				Lower section of piers			
	+100%Fx -30%Fy	-100%Fx +30%Fy	+100%Fy -30%Fx	-100%Fy +30%Fx	+100%Fx -30%Fy	-100%Fx +30%Fy	+100%Fy -30%Fx	-100%Fy +30%Fx
Verified piers	95%	94%	95%	91%	100%	100%	100%	100%
M_{Rd}/M_{Ed} min	0.33	0.36	0.36	0.15	5.01	4.93	4.73	4.18

FLEXURE - The top section of piers at the upper level are not verified because of the low compression

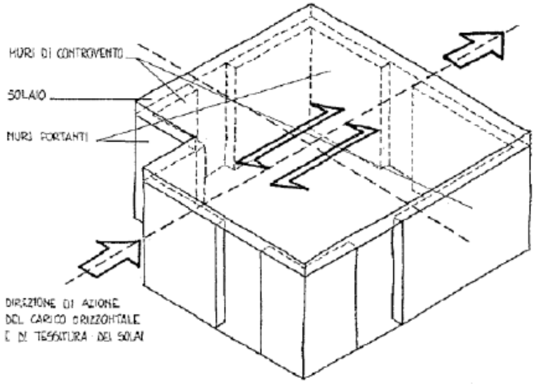
«Surgical» changes of the structural model

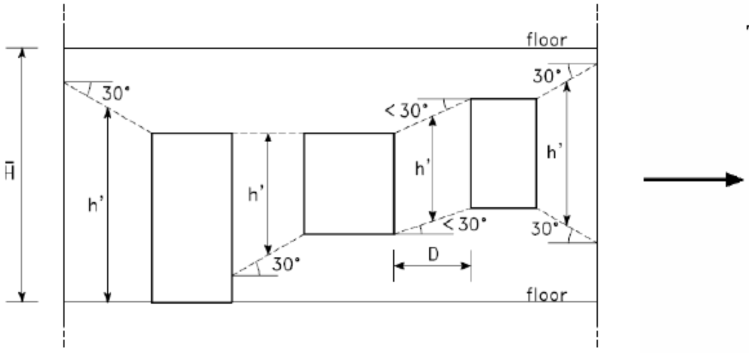
- Transformation of spandrels at the top level into connecting rods
  - Bending moment at the top level becomes zero
  - Shear force is verified because the section is fully compressed
- The verification is not satisfied in medium to high seismicity areas

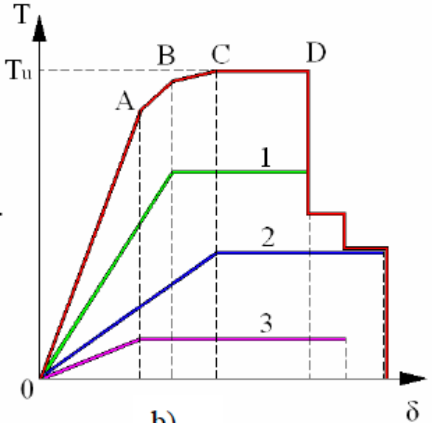
NonLinear Static Analysis of masonry buildings in codes

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.







NONLINEAR APPROACH CURRENTLY IMPLEMENTED IN EC8-Part 3

Equivalent Frame Model (if also spandrels are considered).
Pushover analysis, with strength degradation and displacement verification.

Differences between first and second generation of Eurocode 8 – Part 3

EN1998-3 – June 2005

- Synthetic directions on knowledge levels, methods of analysis, safety verifications
- Informative **Annex C** on masonry buildings (only 8 pages): equivalent frame model, pushover analysis (when conditions for linear analysis are not met), strength degradation and ultimate capacity in terms of global roof displacement.

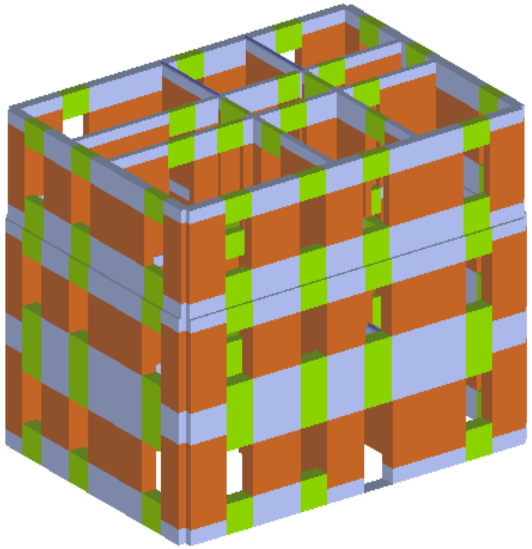
CEN/TC250/SC8 N1236 – November 2022

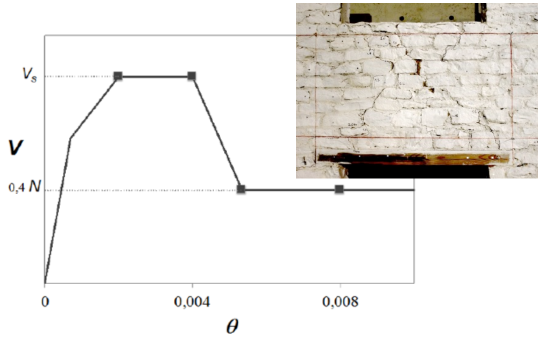
In the new generation, in addition to a detailed description of knowledge, modelling, analysis and verification procedures, specific directions for masonry buildings are provided in clause **11** (40 pages) and in the informative **Annex D** (9 pages).

- Need to consider both in-plane and out-of-plane behaviour (**local mechanisms**)
- Consideration of **rigid, stiff and flexible horizontal diaphragms**
- Classification of **regular or irregular masonry**, with related resistance criteria
- Specific models for **spandrels** (failure criteria, consideration of axial force)
- Deformation capacities of panels and **reference values for material properties**

BUILDING KNOWLEDGE
5.3 - 5.4

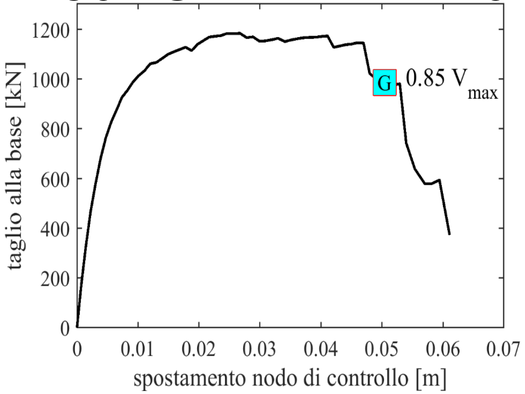
MODELLING
11.3



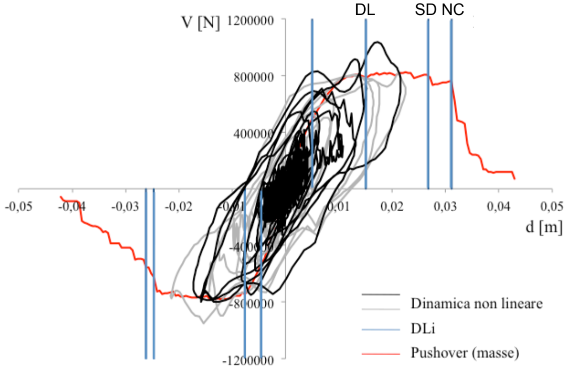


ANALYSIS
11.4

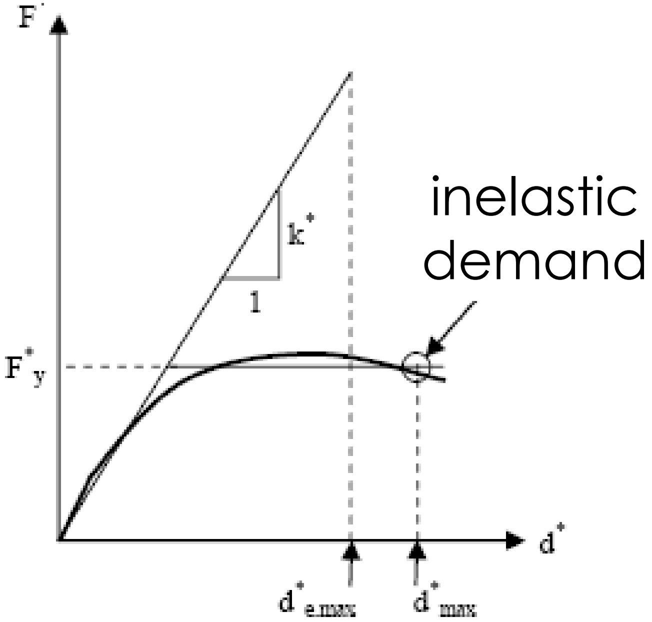
PUSHOVER ANALYSIS



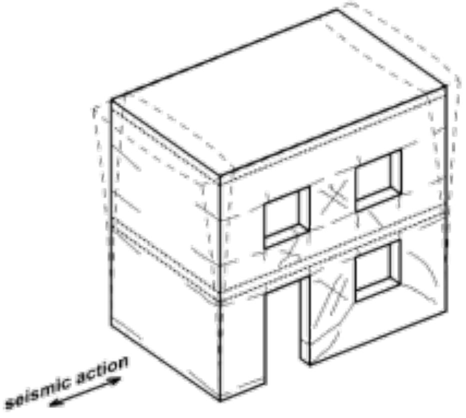
TIME-HISTORY ANALYSIS



VERIFICATION
11.5

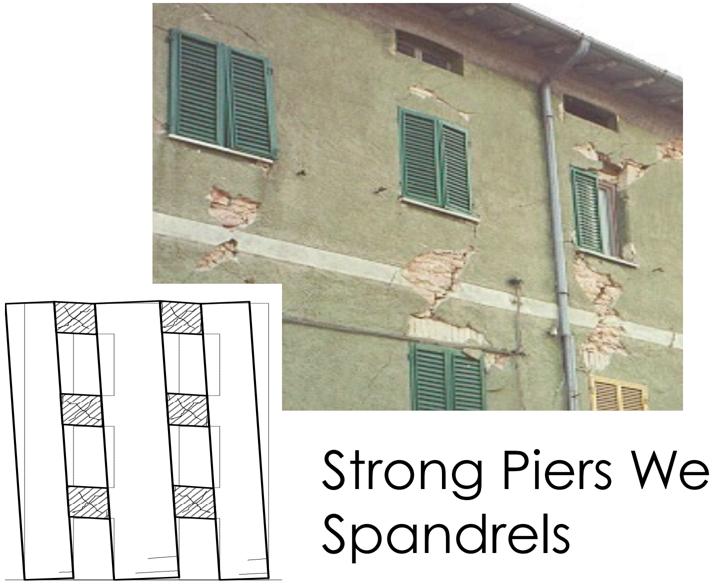


Modelling of the **seismic** behaviour



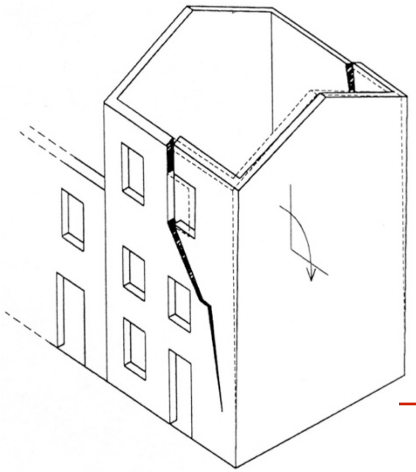
IN-PLANE RESPONSE OF WALLS

Strong Spandrels Weak Piers



Strong Piers Weak Spandrels

OUT-OF-PLANE

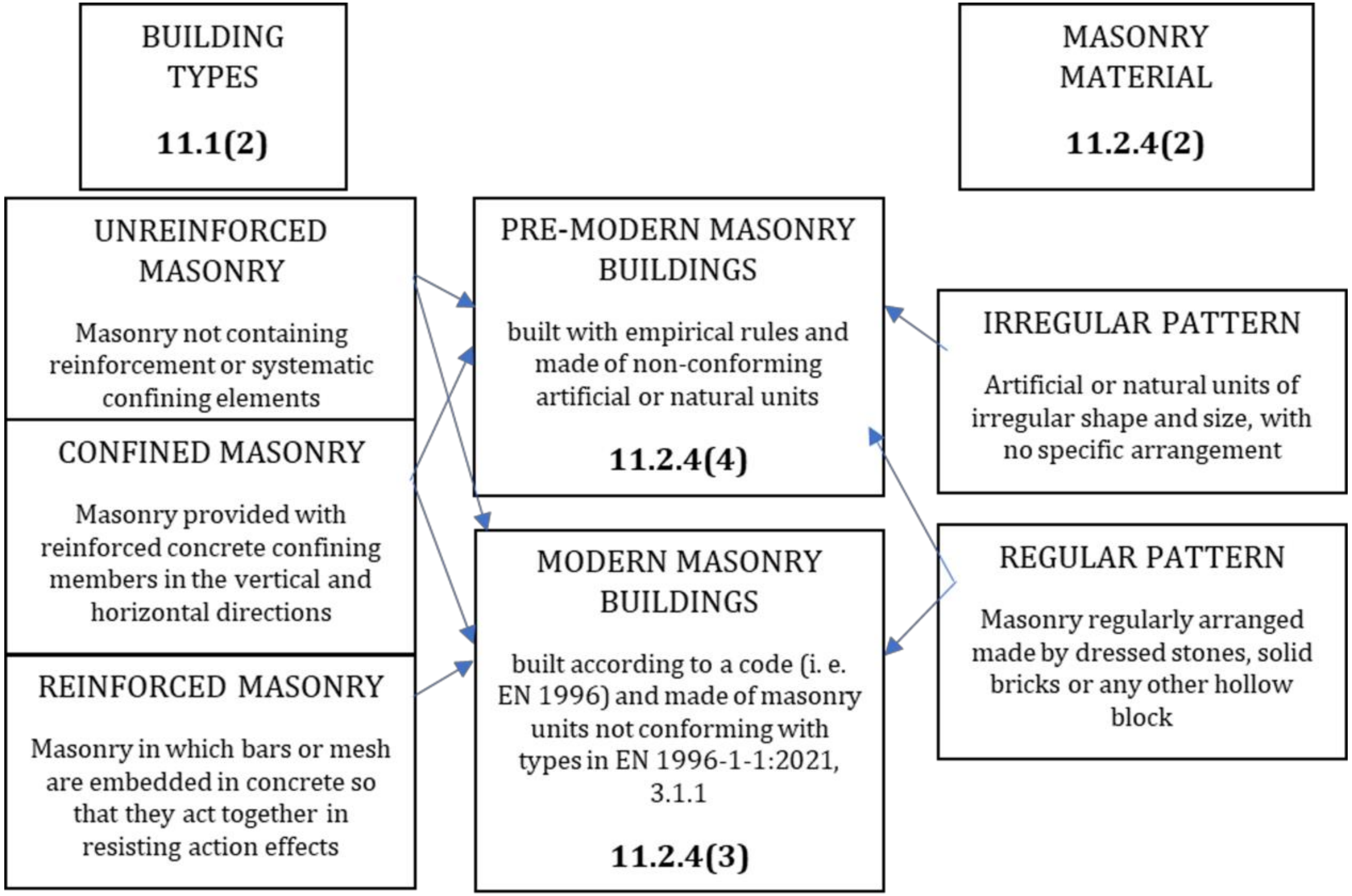


Masonry strength and deformation / drift capacity (material nonlinearity)

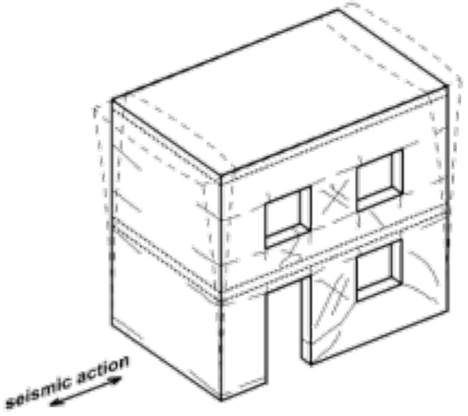
Loss of equilibrium / shape and constraints (geometric nonlinearity)

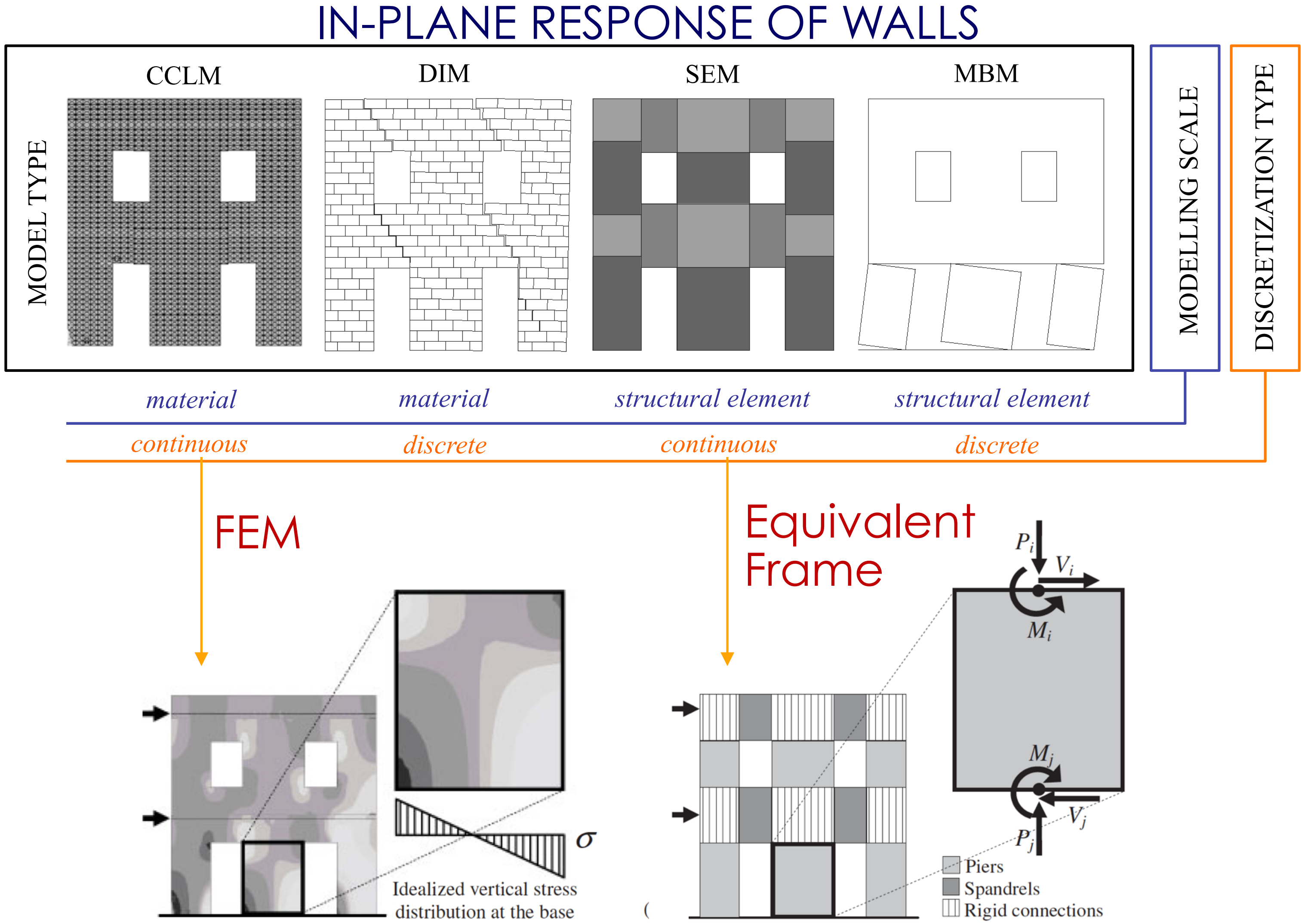
11 Specific rules for masonry buildings

- 11.1 Scope: clarifies ambit of application, with the aim of covering 80% of the existing building stock
- Reference to EC8 Part 1-2 and EC6, when relevant
- Buildings made of mixed materials, when masonry is the prevalent one, may be verified with these rules
  - Masonry + RC frames inside
  - Building expansion in RC
  - Elevation of the building in RC



In-Plane Response of Masonry Walls (11.3.2)





Modelling by FEM:

No need to a-priori defining masonry piers and spandrels

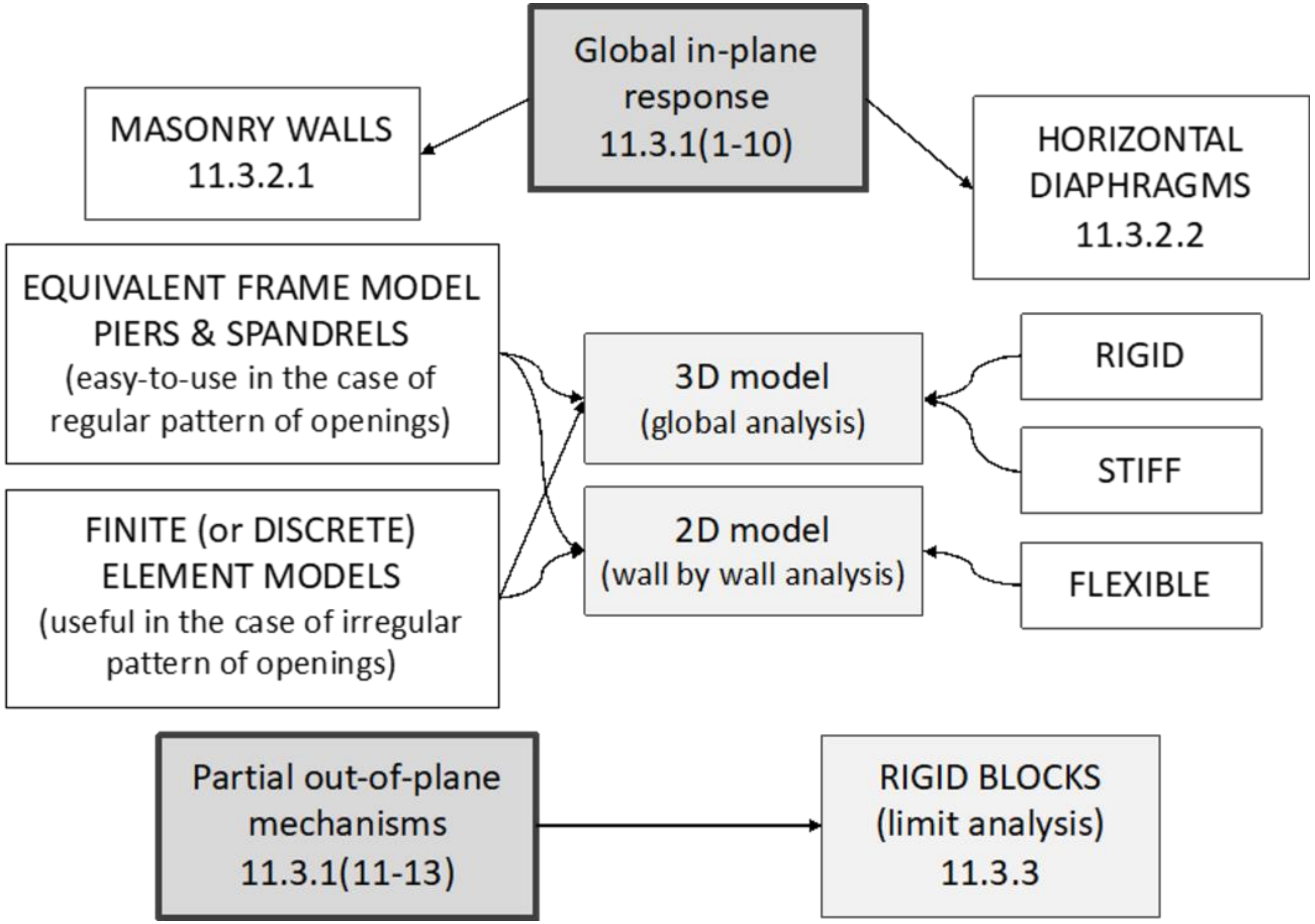
- Elastic analysis: verification in terms of strength, by ex-post stress integration on sections
- Nonlinear analysis: drift check on panels defined ex-post or calibration of softening laws

11.3 Structural modelling and analysis

- 11.3.1 General
  - 11.3.1.2 In-plane behaviour
  - 11.3.1.3 Out-of-plane behaviour
- 11.3.2 Global in-plane response
  - *11.3.2.1 Force-deformation relationships*
  - *11.3.2.2 Horizontal diaphragms*
- 11.3.3 Partial out-of-plane mechanisms

- **11.4 Resistance models for assessment**
- 11.4.1 In-plane loaded masonry members
  - *11.4.1.1 Shear resistance of piers & spandrels*
  - *11.4.1.2 Deformation capacity of members*
- 11.4.2 Partial out-of-plane mechanisms

- **11.5 Verification of Limit States**
- 11.5.1 Global in-plane response of walls
- 11.5.2 Partial out-of-plane mechanisms



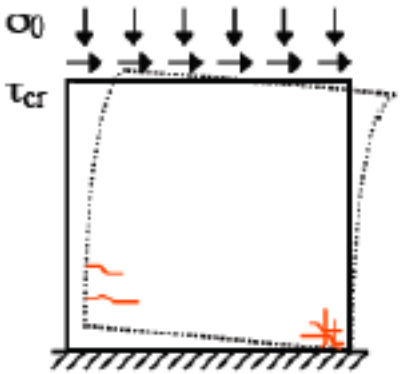
11.4.1 Resistance models for in-plane loaded masonry members

Force-deformation relationships (in terms of generalized force *V* and deformation θ), depends on stiffness, failure criteria and drift limits

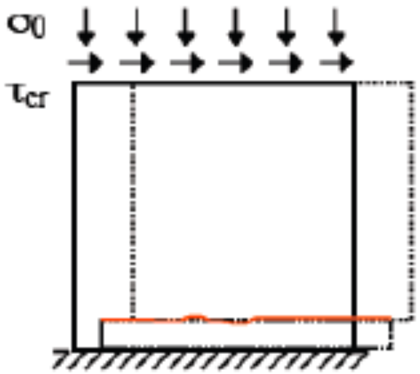
3 failure criteria:
- Flexure
- Shear sliding
- Diagonal cracking

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

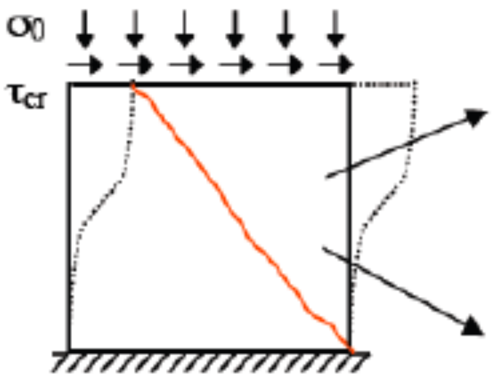
2 masonry elements:
- Piers
- Spandrels

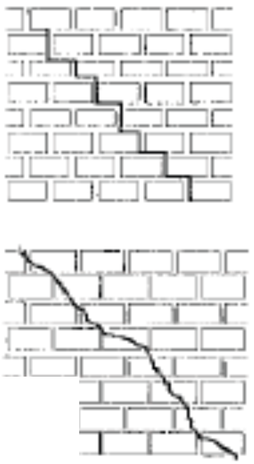


flexure cracking



shear sliding





diagonal

11.4.1.1 In-plane shear resistance of masonry members

Strength criteria for piers

Based on many experimental tests

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

MASONRY	WALL MEMBERS	FLEXURAL	SHEAR SLIDING	DIAGONAL CRACKING (pre-modern only)
REGULAR (modern & pre-modern)	PIERS	11.4.1.1.2(1)	11.4.1.1.3(1-3)	11.4.1.1.4(3)
	SPANDRELS	11.4.1.1.2(4-6)	-	11.4.1.1.4(3)
IRREGULAR (pre-modern)	PIERS	11.4.1.1.2(1)	-	11.4.1.1.4(2)
	SPANDRELS	11.4.1.1.2(4-6) (f_{ht} = 0)	-	11.4.1.1.4(2)

Strength criteria for spandrels

Evidences from experimental campaigns in the last 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
- …

Annex D.3 Masonry parameters

- Reference values of material parameters for masonry types not conforming with EC6
- Correction coefficients as a function of structural details of masonry
- Coefficients for strengthening (D.6)

Table D.5 — Correction coefficients for increasing material properties after retrofitting

Type of masonry	Lime mortar grouting (*)	Reinforced jacketing (**)	Reinforced repointing and transversal bars (**)	Maximum combined factor

Table D.2 — Correction (multiplier) coefficients for strength properties

Type of masonry	Good mortar (*)	Regular alignments	Transversal connection
Irregular stone masonry	1,5	1,3	1,3
Roughly cut stone masonry, with wythes of irregular thickness	1,3	1,2	1,5
Uncut stonework with good texture	1,4	1,1	1,3
Masonry of irregular soft stone blocks	1,5	1,2	1,3
Regular masonry of soft stone blocks	1,6	-	1,2
Squared stone masonry	1,2	-	1,2
Solid brick masonry and lime mortar	1,5	-	1,3

Table D.1 — Reference values for mechanical properties of different masonry types: mean values and coefficient of variation

Type of masonry		f [MPa]	f_t [MPa]	f_{v0} [MPa]	E [MPa]	G [MPa]	w [kN/m³]
Irregular stone masonry, rubble masonry	mean	1,5	0,039	-	870	290	19
	c.o.v.	0,29	0,24	-	0,21	0,21	
Roughly dressed stone masonry, with wythes of irregular thickness	mean	2,5	0,065	-	1230	410	20
	c.o.v.	0,20	0,19	-	0,17	0,17	
Split hard stone masonry with good texture	mean	3,2	0,097	-	1740	580	21
	c.o.v.	0,19	0,14	-	0,14	0,14	
Masonry of irregular soft stone (e.g. tuff, calcarenite)	mean	1,8	0,052	-	1080	360	13 to 16
	c.o.v.	0,23	0,14	-	0,17	0,17	
Regular masonry of cut, soft stone (e.g. tuff, calcarenite)	mean	2,6	-	0,145	1410	470	
	c.o.v.	0,23	-	0,31	0,15	0,15	
Squared hard stone masonry, ashlar masonry	mean	7,0	-	0,220	2800	860	22
	c.o.v.	0,14	-	0,14	0,14	0,09	
Solid clay brick masonry and lime mortar	mean	3,4	0,114	0,160	1500	500	18
	c.o.v.	0,26	0,21	0,21	0,20	0,20	
Lightly perforated clay brick masonry (volume of all holes ≤ 40%) with cement-lime mortar	mean	6,5	-	0,280	4550	1138	15
	c.o.v.	0,24	-	0,14	0,24	0,24	

f: compressive strength of masonry; f_t: diagonal tensile strength of masonry; f_{v0}: initial shear strength of masonry; E: modulus of elasticity; G: shear modulus; w: weight density of masonry

11.4.1.2 In-plane deformation capacity of masonry members

- Force-deformation relationships are provided in terms of member drift ratio:

$$\theta_e = u_j - u_i / h + r_j + r_i / 2$$

- In the case of flexural and shear sliding failure, limit values are referred to the chord rotation at the end where failure occurs:

$$\theta_i = r_i + u_0 - u_i / h_i \cong r_i + u_j - u_i / h$$
$$\theta_j = r_j + u_j - u_0 / h_j \cong r_j + u_j - u_i / h$$

- Annex D.5 – Drift capacity of masonry panels in hybrid modes

MASONRY	WALL MEMBERS	FLEXURAL	SHEAR SLIDING **11.4.1.2.3(1)**	DIAGONAL CRACKING (pre-modern only)
REGULAR (modern & pre-modern)	PIERS	0,01(1-ν) **11.4.1.2.2(1)**	modern: 0,004 pre-modern: 0,008 (sliding) 0,005 (unit failure)	0,006 **11.4.1.2.4(1)**
	SPANDRELS	0,016 (good lintel) 0,012 (other cases) **11.4.1.2.2(2)**	-	0,006 **11.4.1.2.4(2)**
IRREGULAR (pre-modern)	PIERS	0,01(1-ν) **11.4.1.2.2(1)**	-	0,005 **11.4.1.2.4(1)**
	SPANDRELS	0,016 (good lintel) 0,012 (other cases) **11.4.1.2.2(2)**	-	0,005 **11.4.1.2.4(2)**

6.4.2 Non-linear static analysis

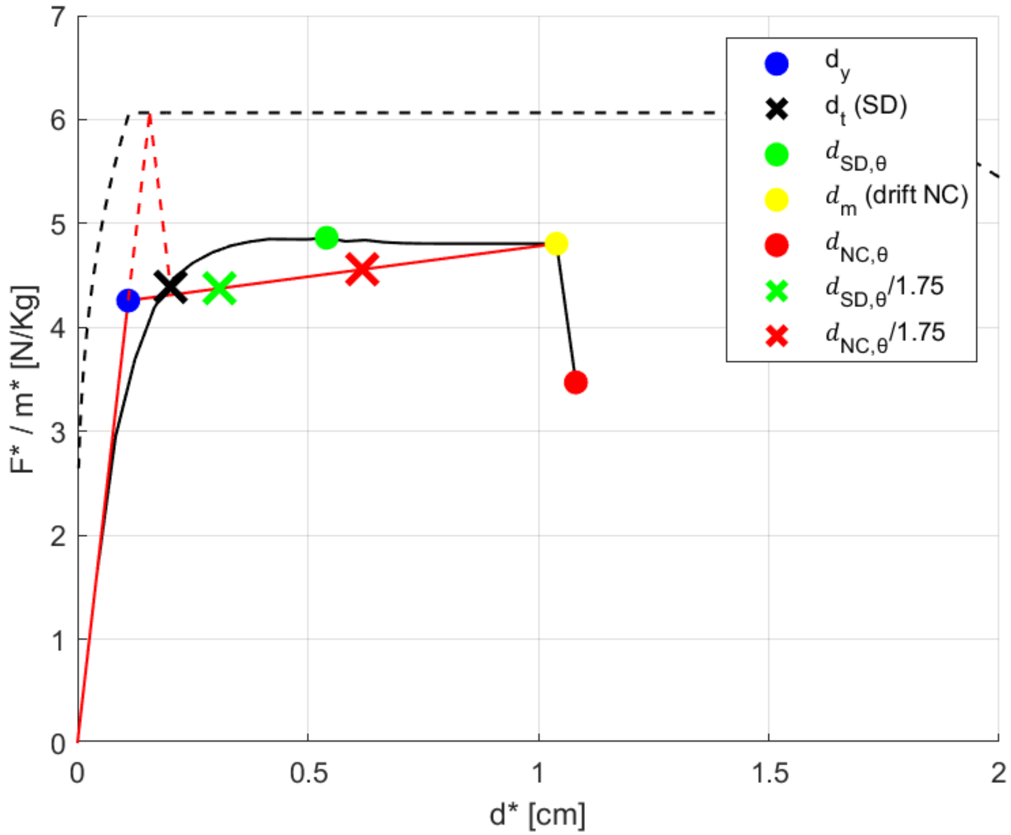
- General rules from EC8-1.1 (6.5 and Annex D), with additional provisions in EC8-1.2 (5.3.5)
- Pushover analysis with "modal" load pattern, based on the displacement corresponding to the horizontal forces used in the lateral force method EC8-1.2 (5.3.5.2(3))
  - 4 analysis only (X and Y, positive and negative direction), with an additional eccentricity if the natural one is lower than a minimum value
  - In addition, "uniform" pattern if in the building a soft story mechanism is expected
- In buildings without rigid diaphragms, lateral loads are calculated and applied in each node
- In the case of stiff diaphragms, the control displacement should be the average top displacement among those of different walls, weighted by the corresponding seismic masses
- At NC limit state, the displacement demand should be lower than the capacity:

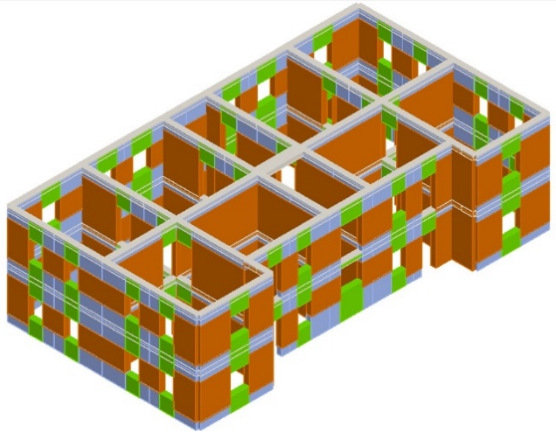
$$d_t^* \leq d_{NC}^* = \max\left(d_y^*, \frac{1}{\gamma_{Rd}} d_{NC,\theta}^*\right)$$

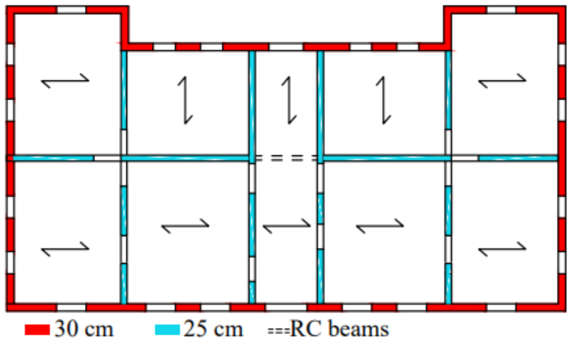
  - $d_{NC,\theta}^*$ is the minimum between 3 conditions: a) 20% drop of total base shear, b) ultimate drift in all piers of one wall at a specific level; c) compressive failure in one pier (1.5 times the ultimate drift)

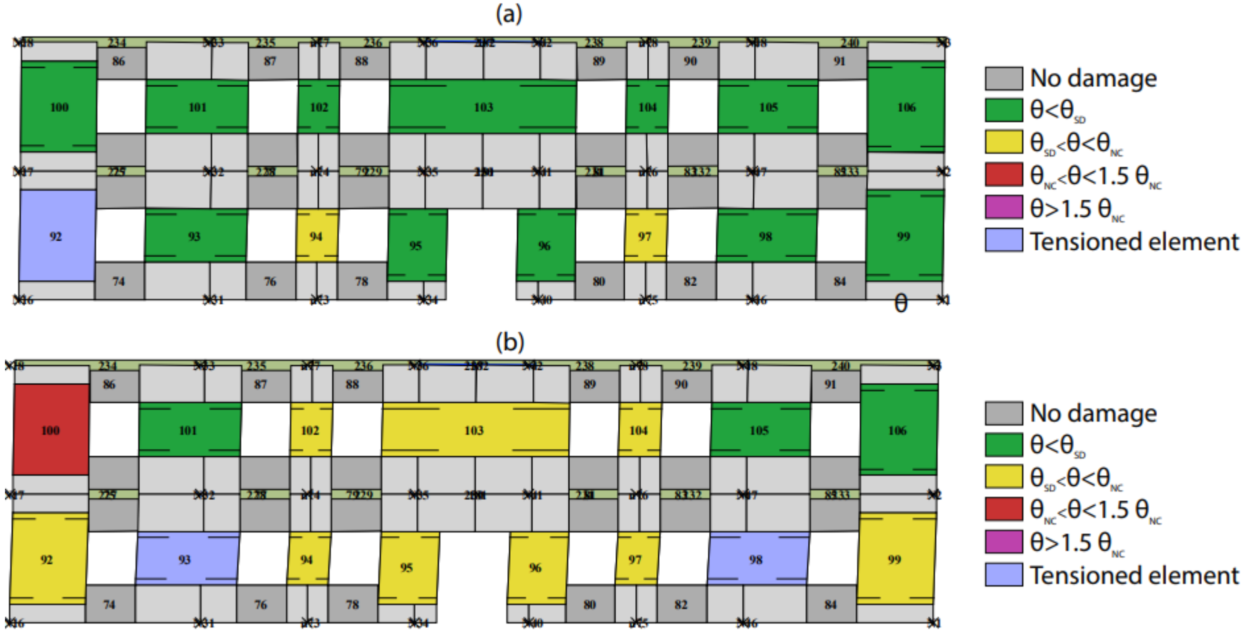
Example

- 2-storeys masonry building in L'Aquila (Italy)
- S_δ=6.065 m/s^2 - F_α=1 (soil A) - F_β=1 (flat ground)



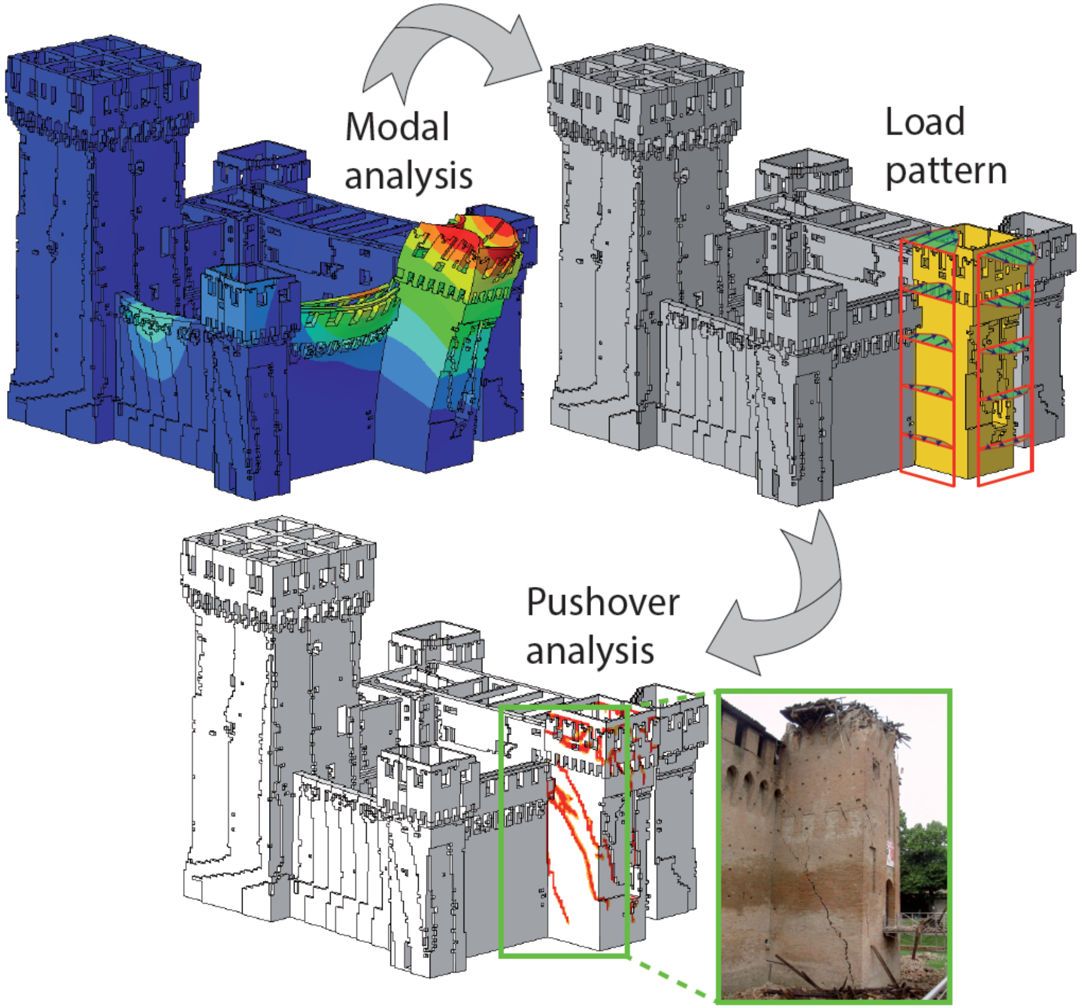




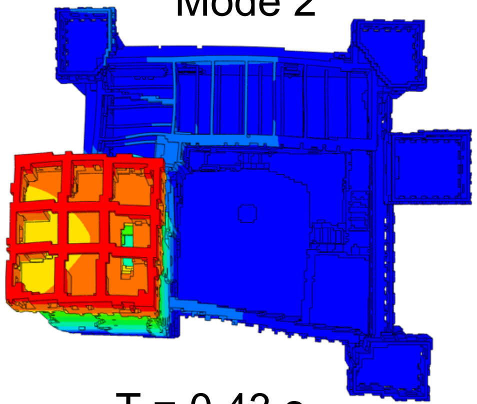


San Felice sul Panaro Fortress (Emilia 2012)

after Emilia earthquake, 2012

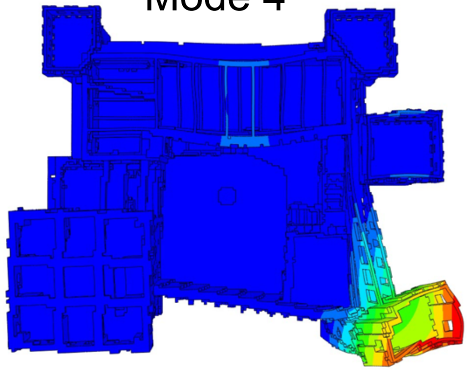


Mode 2



T = 0.43 s
m_y= 34%

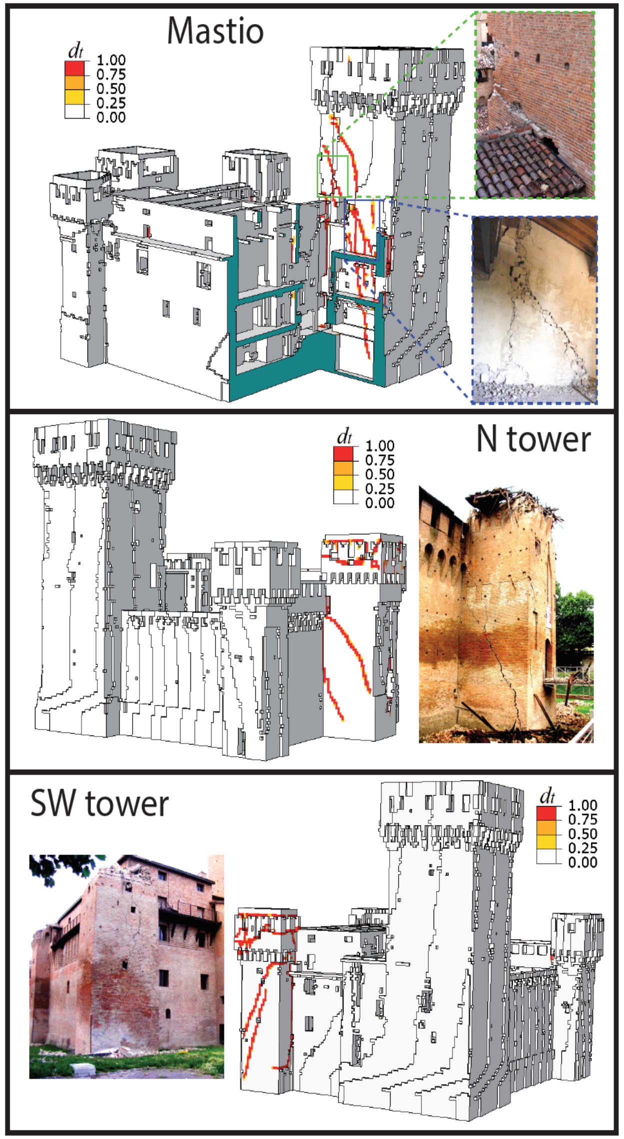
Mode 4



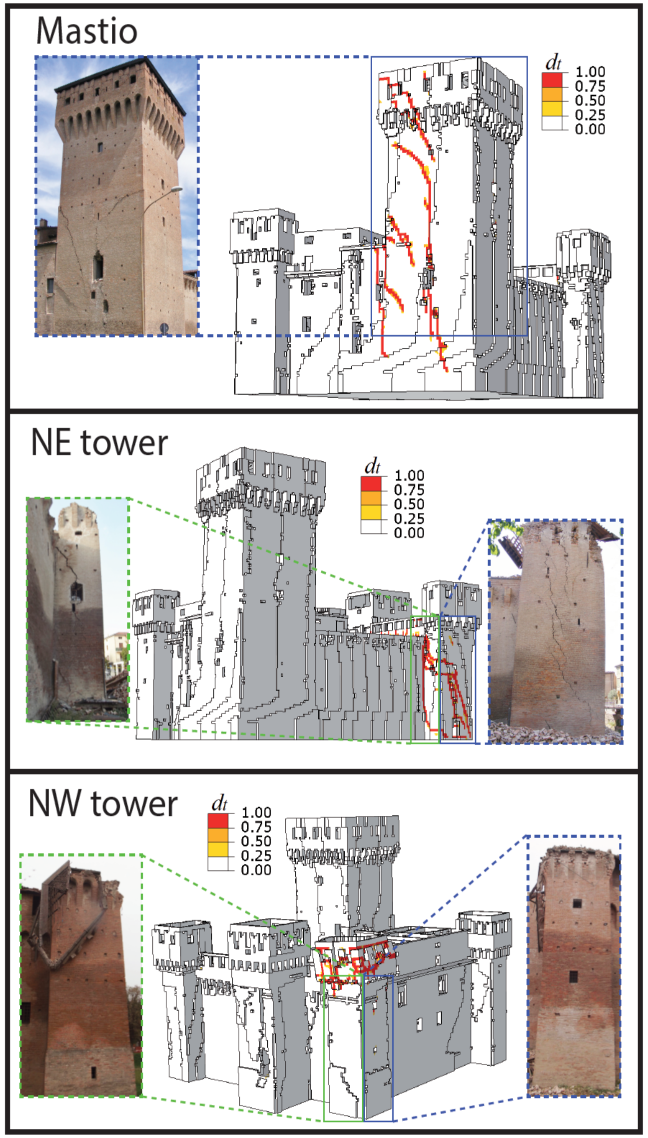
T = 0.25 s
m_x= 6.1%

Degli Abbati et al. (2019) Seismic assessment of interacting structural units in complex historic masonry constructions by nonlinear static analyses, Computers and Structures, 213

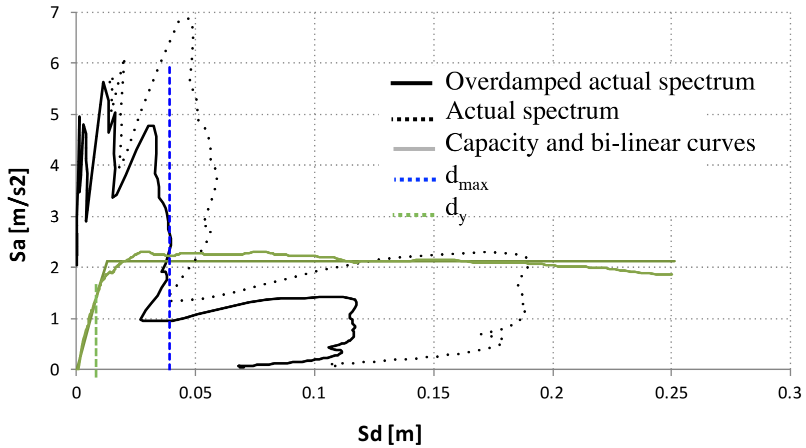
North-South direction

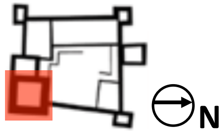


East-West direction



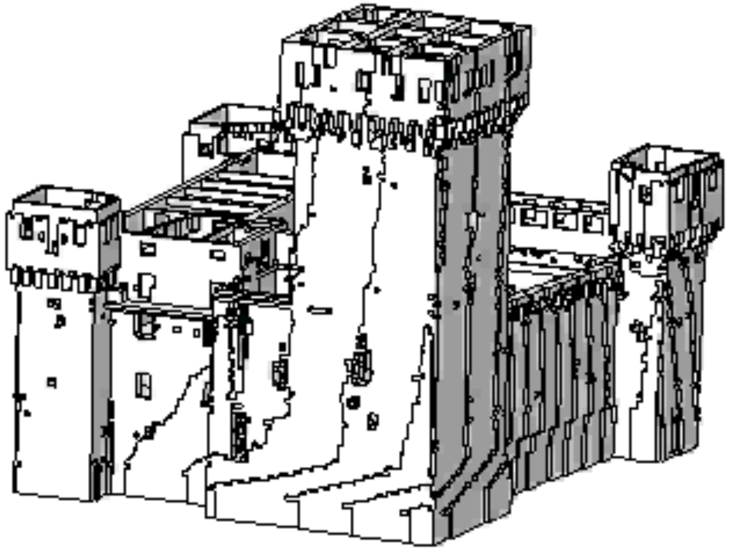
$\mu_{max} = 3.10$





$\Gamma = 1.6054$

$\Gamma\ M^* = 1838986$



11.3.3 Modelling and analysis of partial out-of-plane mechanisms

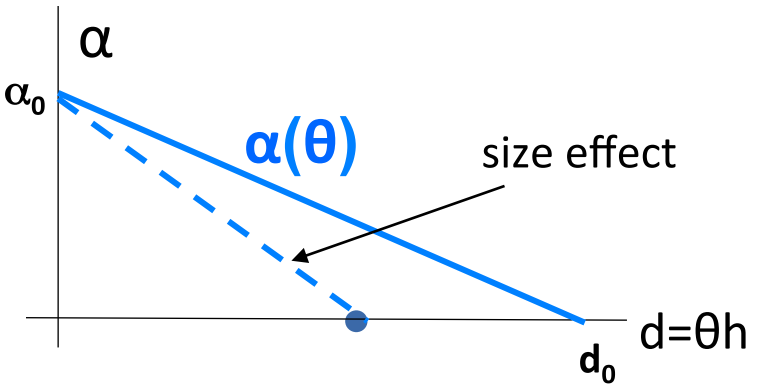
- MODERN MASONRY BUILDINGS ⟹ possible only at interstorey level
- PRE-MODERN MASONRY BUILDINGS ⟹ connections are poor

- a-priori identification of rigid blocks mechanisms
- Limit analysis to calculate the horizontal seismic action that activates
- Non-linear kinematic analysis to identify the displacement capacity

Displacement-Based Assessment of Rocking

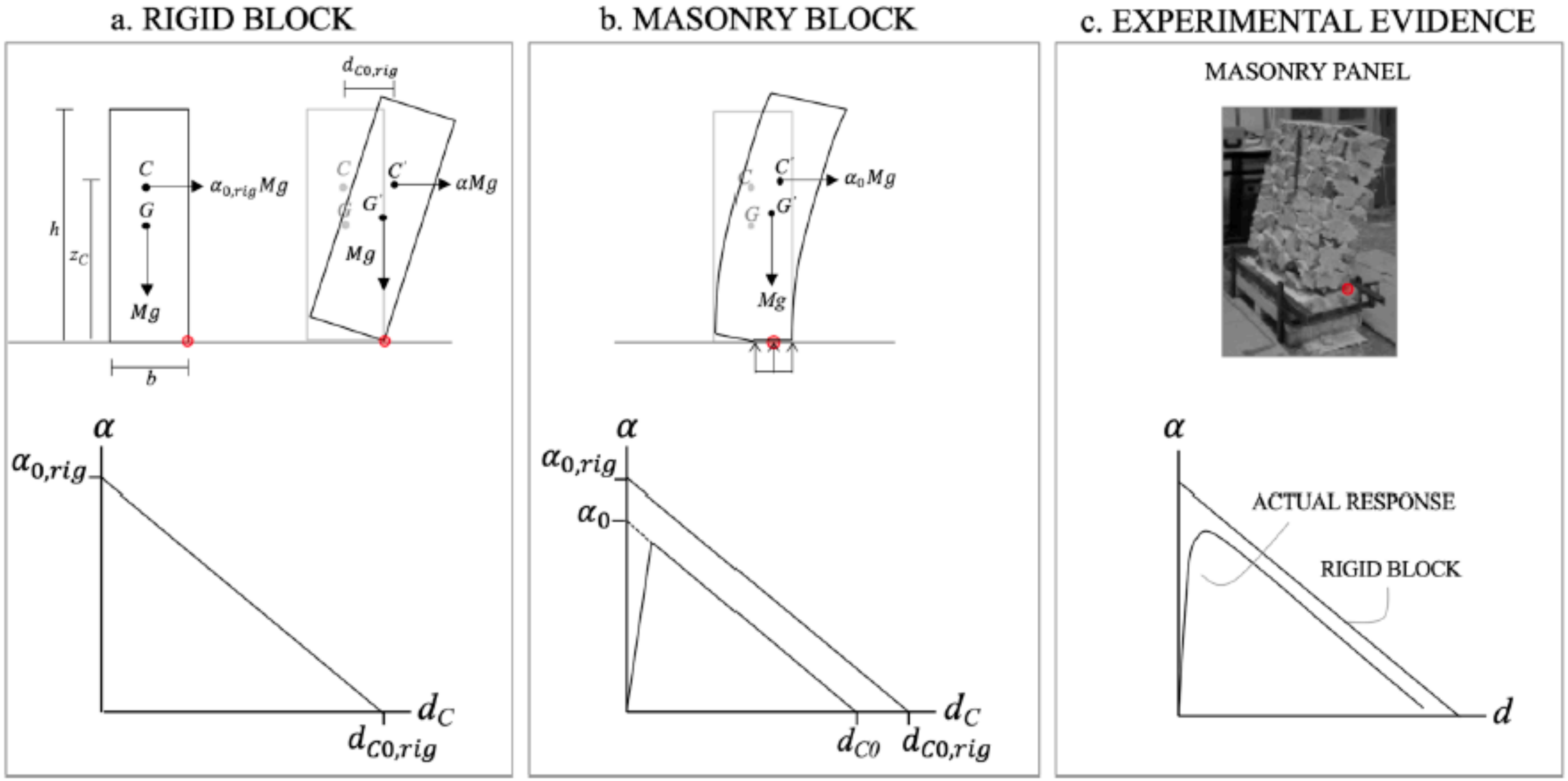
Linear Kinematic Analysis → α_0

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



This verification should be made in addition to the global in-plane shear resistance of masonry members:

- in masonry walls not well connected to orthogonal walls and diaphragms
- for vertically cantilevering members
- for slender masonry walls



capacity curve

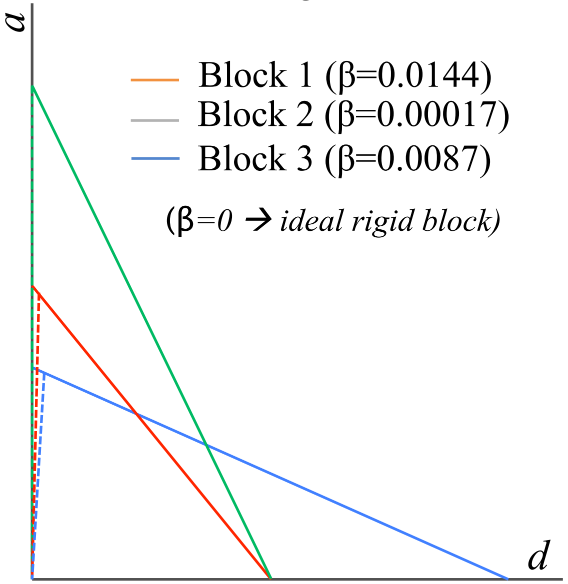
$$a^* = \begin{cases} \dfrac{4\pi^2}{T_e^2} d^* & d^* \leq d_e \\ a_0\left(1 - \dfrac{d^*}{d_0}\right) & d^* > d_e \end{cases}$$

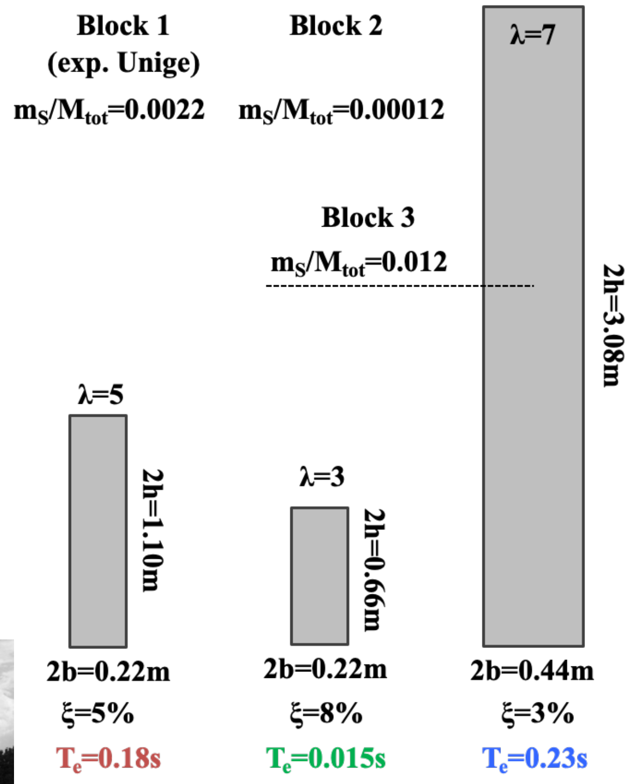
$$d_e = \frac{a_0}{\frac{4\pi^2}{T_e^2} + \frac{a_0}{d_0}}$$

$$a_e = \frac{a_0}{1 + \frac{a_0}{d_0}\frac{T_e^2}{4\pi^2}}$$

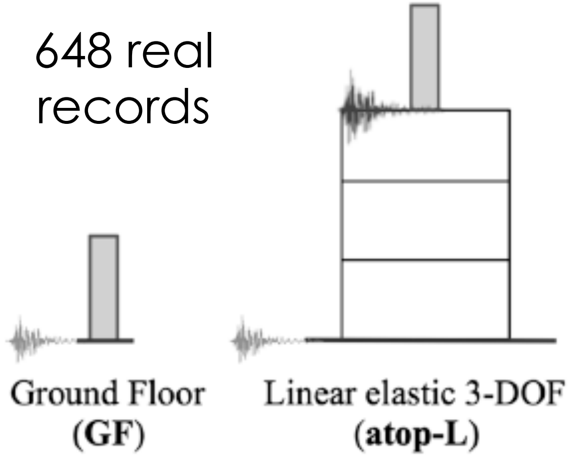
Displacement-Based Assessment of Rocking

Validation by Nonlinear Dynamic Analyses





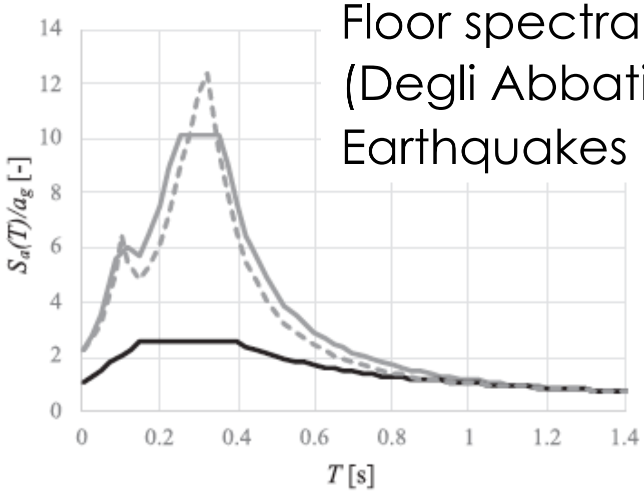
648 real records



Examples of possible rocking elements

Parapets Statues-Pinnacles Belfry

Floor spectra (Degli Abbati et al., 2018, Earthquakes & Structures)



Degli Abbati, Cattari and Lagomarsino (2021) "Validation of displacement-based procedures for rocking assessment of cantilever masonry elements", Structures 33

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 post-earthquake damage observation.
- The final draft of EC8-Part 3 proposes a general framework for the seismic assessment of existing masonry buildings through non-linear models, tailored to a wide variety of complex configurations:
  - global in-plane behaviour and local out-of-plane mechanisms
  - rigid, stiff and flexible horizontal diaphragms