EC8-2G

Il nuovo standard europeo per la progettazione sismica

EUCENTRE FOR YOUR SAFETY.

reluis

Strutture esistenti

EN1998-3. Conoscenza delle costruzioni e approcci dipendenti dal materiale

Sergio Lagomarsino, Università di Genova

Pavia - 5 Giugno 2025

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 building performance by a model as close as possible to the actual behaviour. Nonlinear models are needed as they don’t assume a predefined capacity. Linear model makes assumptions largely cautionary.

Contents

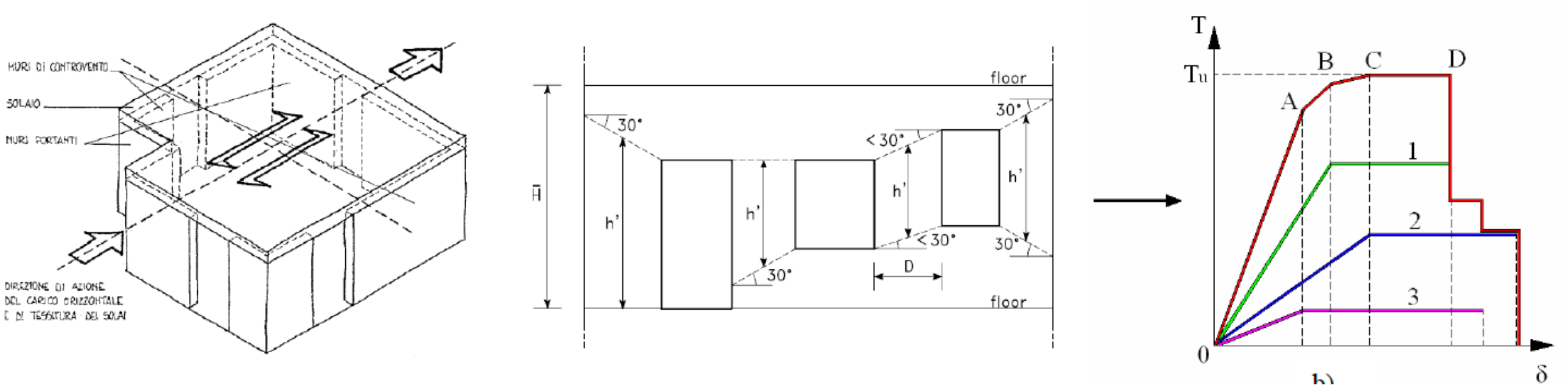
Seismic behaviour of existing masonry buildings

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

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 EC8 – 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 N1362 – September 2024

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 C** (11 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**

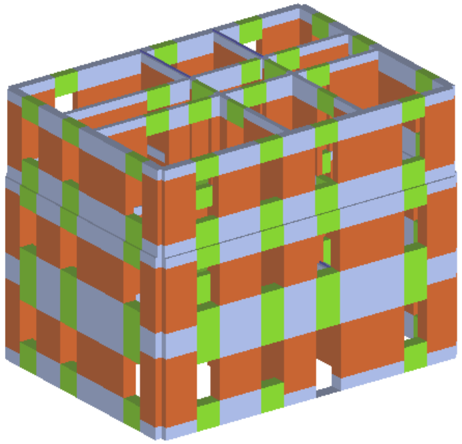
The seismic assessment procedure

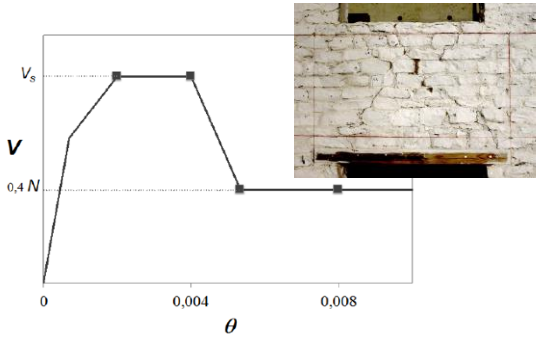
BUILDING KNOWLEDGE
5.3 – 5.4 – 11.2

MODELLING
6.2 – 11.3 – 11.4

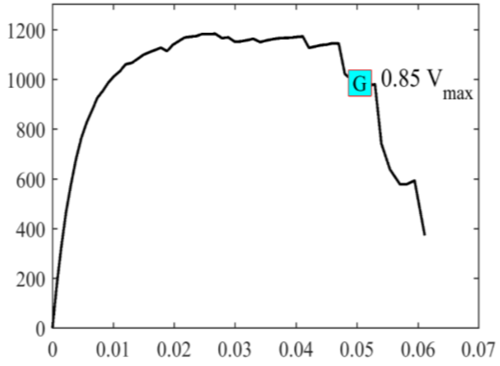
ANALYSIS
6.3 – 6.4 – 11.3

VERIFICATION
6.5 - 11.5

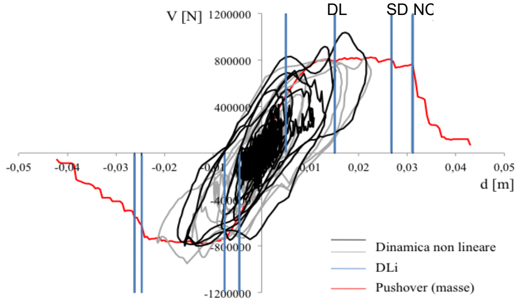


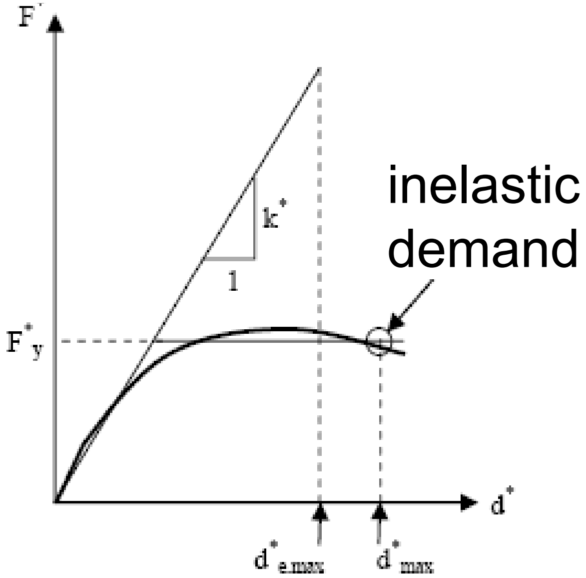


PUSHOVER ANALYSIS



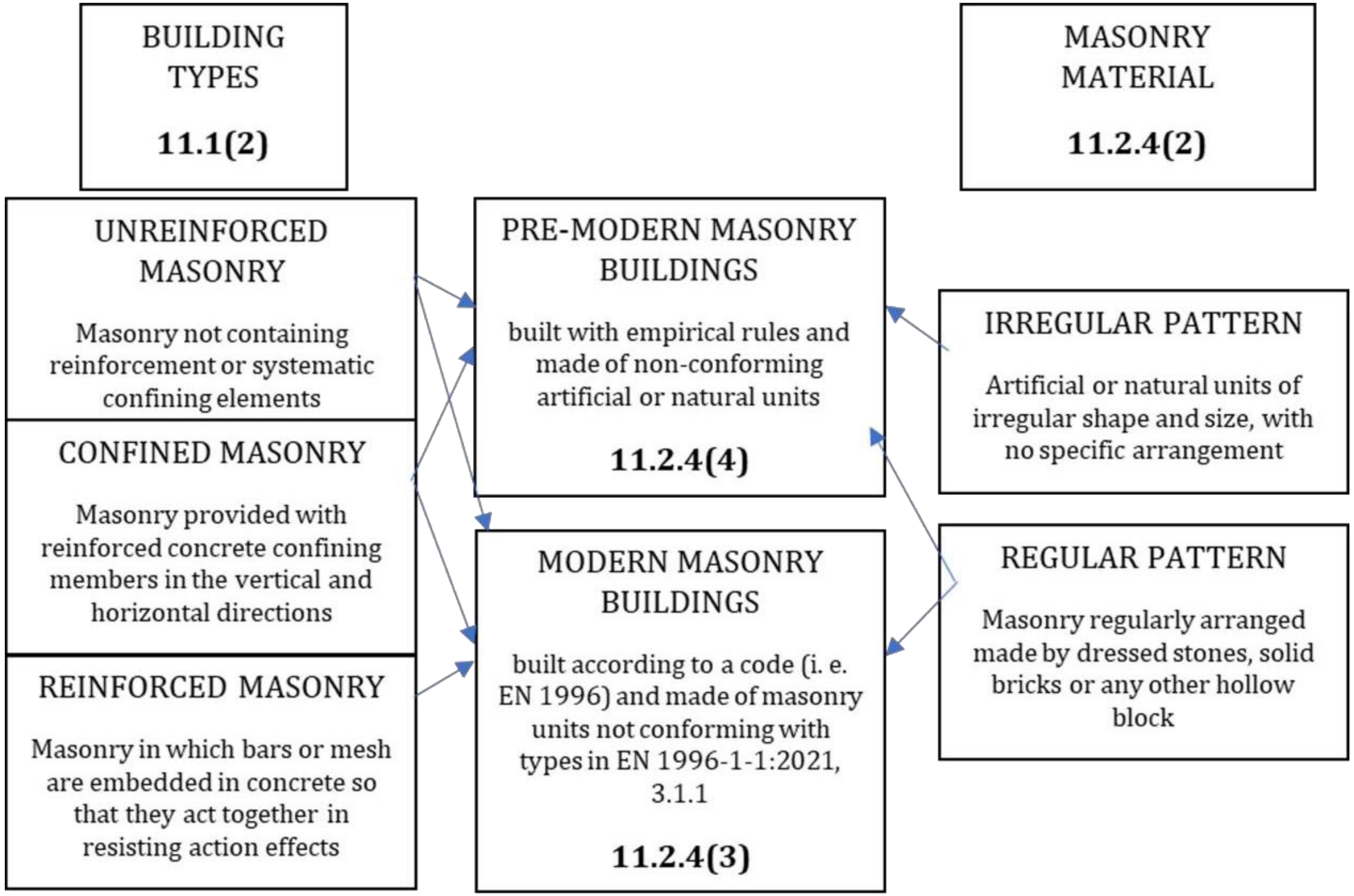
TIME-HISTORY ANALYSIS



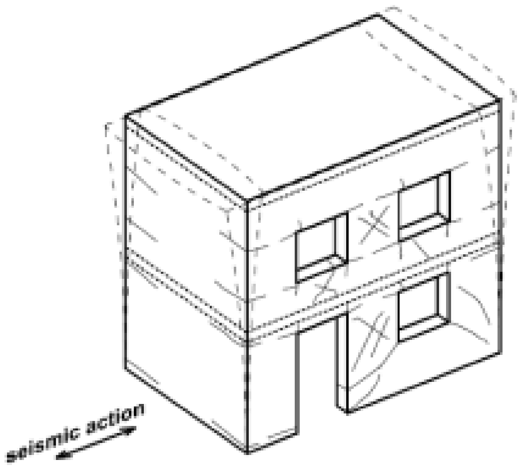


11 Specific rules for masonry buildings

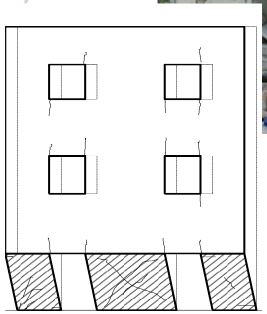
- The aim is covering about 80% of the existing building stock
- Reference to EC8 Part 1-2 and EC6, when relevant
- Buildings made of mixed materials, when masonry is the prevalent one, may be verified with these rules
  - Masonry + RC frames inside
  - Building expansion in RC
  - Elevation of the building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



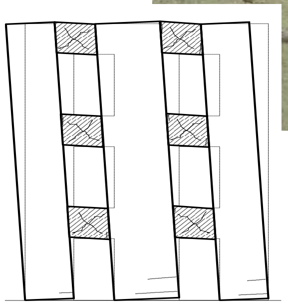
Modelling of the seismic behaviour



IN-PLANE RESPONSE OF WALLS



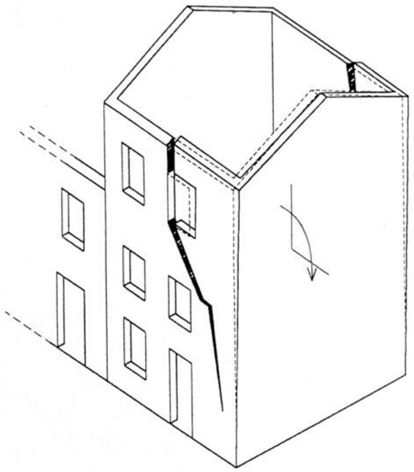
Strong Spandrels Weak Piers



Strong Piers Weak Spandrels

Masonry strength and deformation / drift capacity (material nonlinearity)

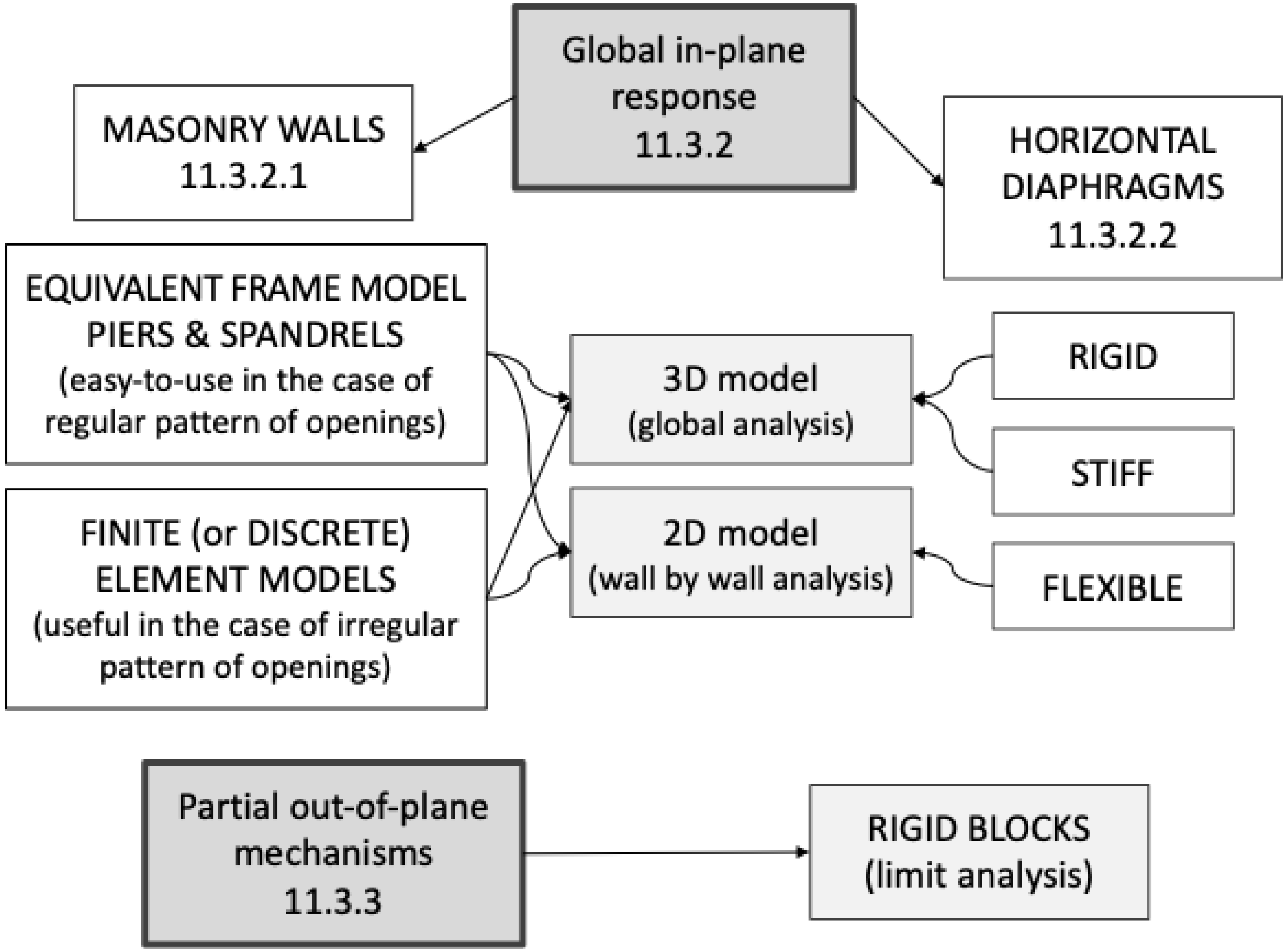
OUT-OF-PLANE



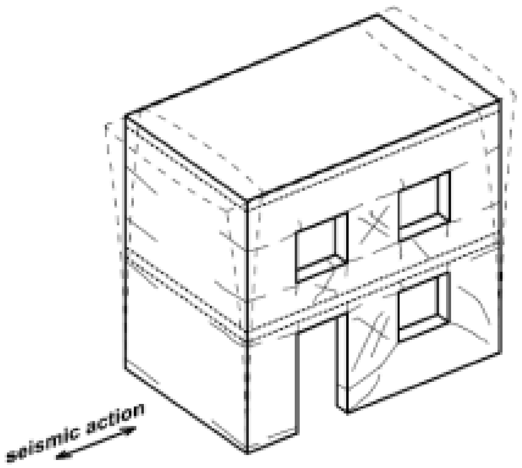
Loss of equilibrium / shape and constraints (geometric nonlinearity)

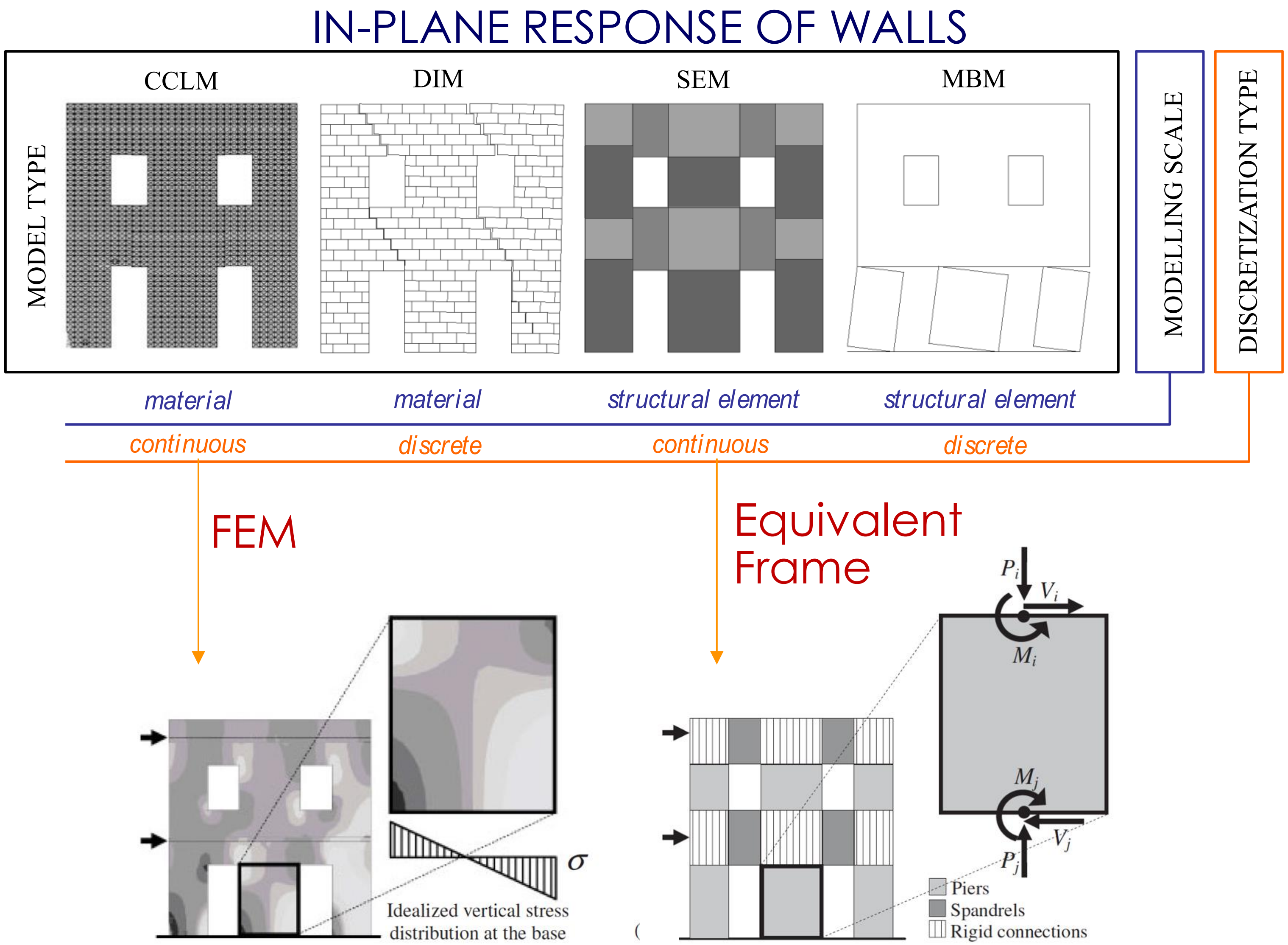
11.3 Structural modelling and analysis

- 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 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



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

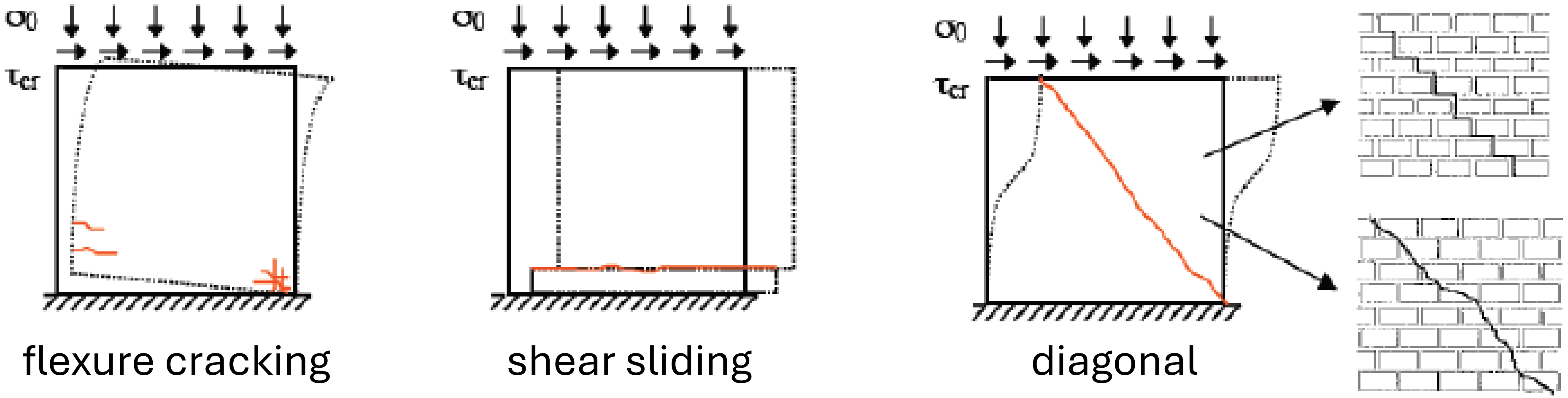
11.4.1 Resistance models for in-plane loaded masonry members

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 C.3 Masonry parameters

- Reference values of material properties for masonry types not conforming with EC6
- Correction coefficients as a function of structural details of masonry
- Coefficients for strengthening

Type of masonry	Lime mortar grouting (*)	Reinforced jacketing (**)	Reinforced repointing and transversal bars (**)	Maximum combined factor
Irregular stone masonry	2	2,5	1,6	3,5
(perforations < 40%)				

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

11.4.1.2 In-plane deformation capacity of masonry members

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

$$\theta_e = \frac{u_j - u_i}{h} + \frac{r_j + r_i}{2}$$

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

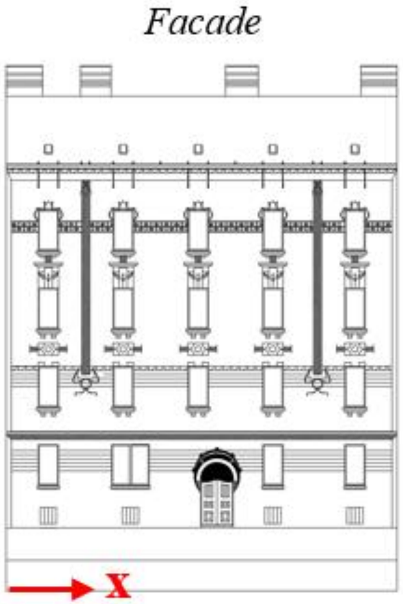
$$\theta_i = r_i + \frac{u_0 - u_i}{h_i} \cong r_i + \frac{u_j - u_i}{h}$$

$$\theta_j = r_j + \frac{u_j - u_0}{h_j} \cong r_j + \frac{u_j - u_i}{h}$$

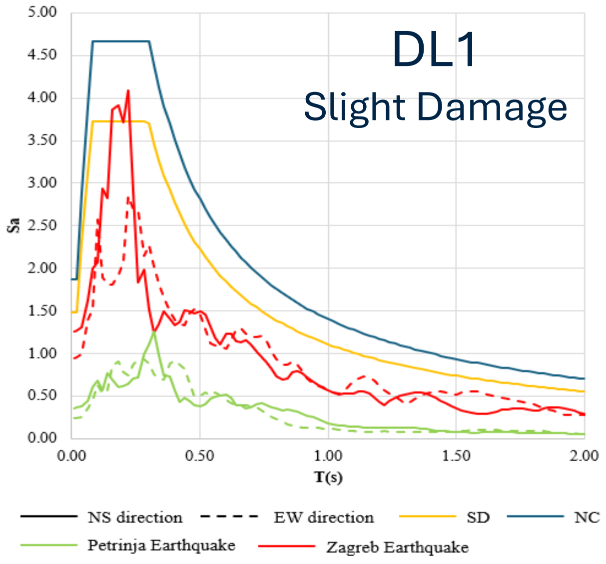
- Annex D.5 – Drift capacity of masonry panels in hybrid modes

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)**

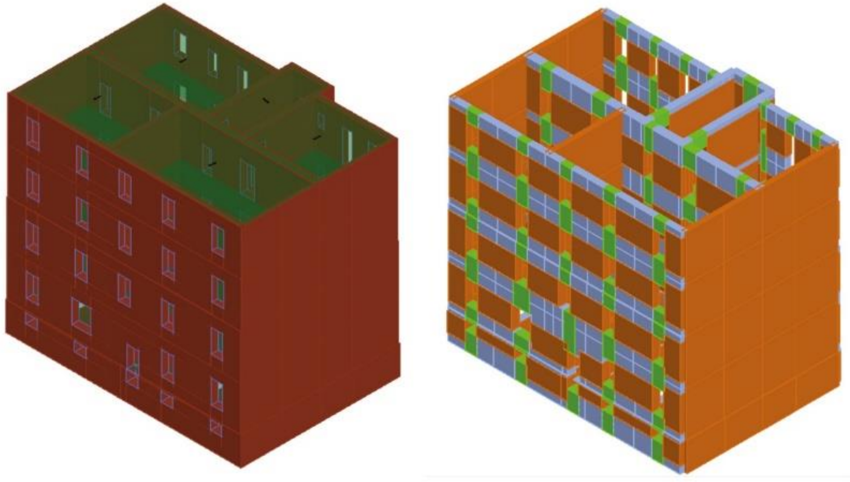
Case study - Zagreb's Lower Town

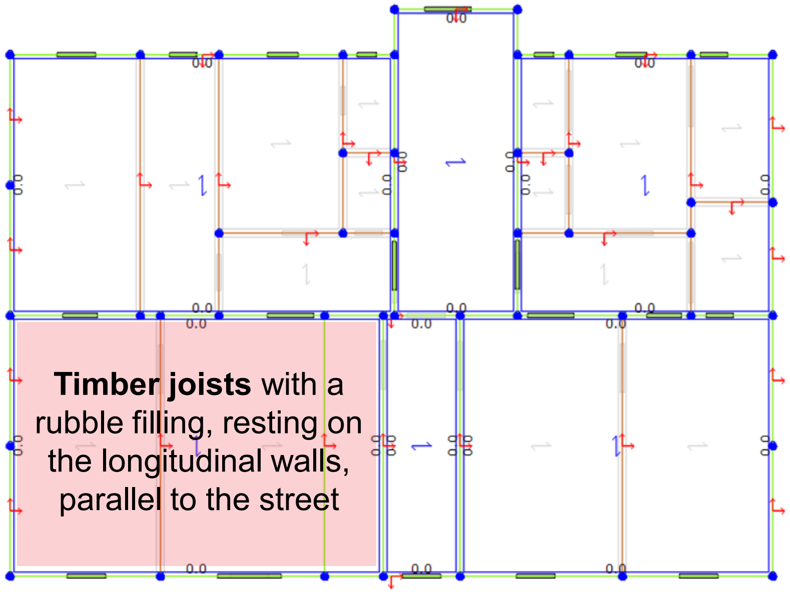


- 5-storey URM unit in aggregate, built in 1908.
- Rectangular plan: 19,20x12,35 m.
- Solid bricks walls, with wall thickness 75 cm at the base, and 30 cm at higher levels.
- Almost flexible diaphragms.
- No reinforced concrete ring beams.
- Knowledge Level = 2



Equivalent Frame model

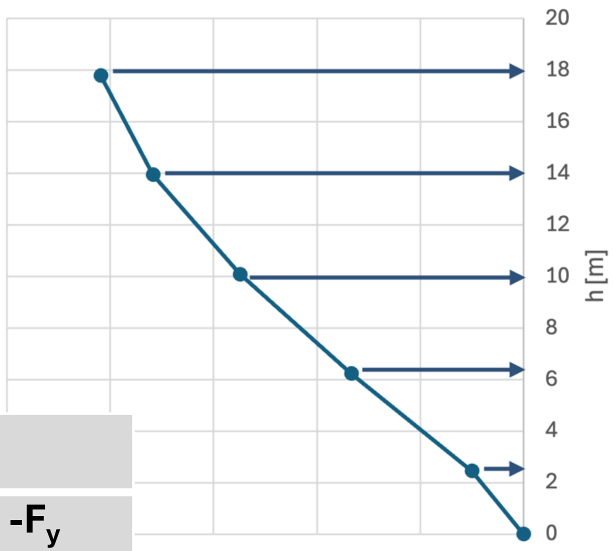




Linear static analysis with q-factor (force-based approach)

- Equivalent forces derived from a displacement profile s_i obtained by a uniform load pattern (from Part 1.2)
- F_b is obtained with the spectrum reduced by q=1.5
- Equivalent frame model neglecting spandrels at the top level (in order to get zero moment at the upper section)
- Local verification of piers by three failure criteria

$$\boxed{\gamma_{Sd}\, V \leq \frac{V_R}{\gamma_{Rd}}}$$



$$F_i = F_b \frac{s_i m_i}{\Sigma s_j m_j}$$

	Top section of piers			
	+F_x -30%F_y	-F_x +30%F_y	+F_y -30%F_x	-F_y +30%F_x
Verified cases	**82%**	**88%**	**79%**	**78%**
$min\left(\frac{V_{R,s}}{\gamma_{Rd} V}\right)$	**0.02**	**0.02**	**0.04**	**0.03**

Failure in flexure

$$V_{R,f} = \frac{DN}{2H_0}\left(1 - \frac{N}{Dtf_m\eta_f}\right) \qquad \gamma_{Rd}=1.85$$

Failure by shear sliding

$$V_{R,s} = \min\begin{cases} D't\left(f_{v0} + \frac{\mu N}{D't}\right) \\ 0.065\, f_b D't \end{cases} \qquad \gamma_{Rd}=1.50$$

Failure by diagonal cracking

$$V_{R,d} = \min\begin{cases} \frac{Dt}{b}\left(\frac{f_{v0}}{1+\mu_i\Phi} + \frac{\mu_i N}{(1+\mu_i\Phi)Dt}\right) \\ \frac{Dt}{b}\frac{0.1 f_b}{2.3}\sqrt{1 + \frac{N}{0.1 Dt f_b}} \end{cases}$$

$$\gamma_{Rd}=1.55$$

Linear elastic analysis (displacement-based approach)

- Application of the same load pattern but with an unreduced spectral force (q=1)
- Local verification of masonry piers in terms of deformation (drift), where $\rho = V_{ed}/V_{Rd} > 1$

$$\gamma_{Sd}\, f_{SD}\, \theta \leq \frac{\theta_{SD}}{\gamma_{Rd}}$$

$$f_{SD} = u_{LS}\left[1 + \left(\frac{1 - u_{LS}}{u_{LS}}\right)\frac{T_C}{T_1}\right] \geq 1$$

Flexure		Shear sliding		Diagonal cracking	
θ_{SD} [%]	γ_{Rd}	θ_{SD} [%]	γ_{Rd}	θ_{SD} [%]	γ_{Rd}
1.0(1-ν)	1.85	0.8	1.50	0.6	1.55

with $\nu = \sigma_0/f_m$

- The method is applicable only if $\rho_{max}/\rho_{min} < 2.5$

	+Fx -30%Fy	-Fx +30%Fy	+Fy -30%Fx	-Fy +30%Fx
With shear sliding failure	76.72	33.68	97.19	118.90
Neglecting shear sliding failure	1.83	1.53	2.24	2.46

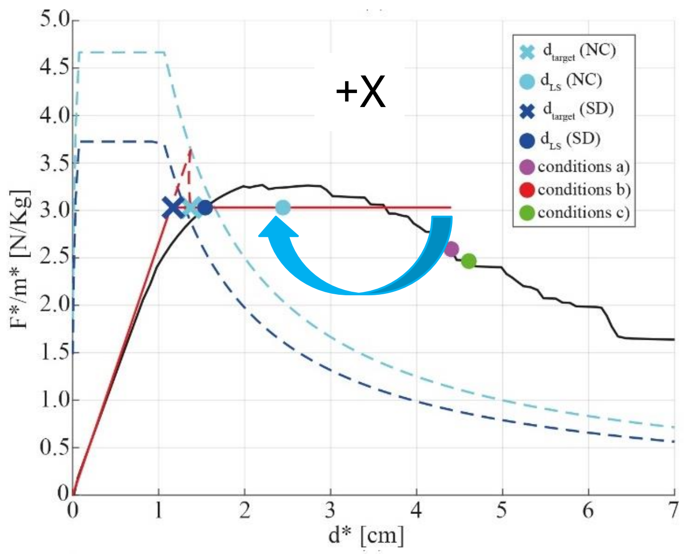
Shear sliding verification is very punitive. If you don't consider it as a limiting condition, you can use this approach and the verification is satisfied

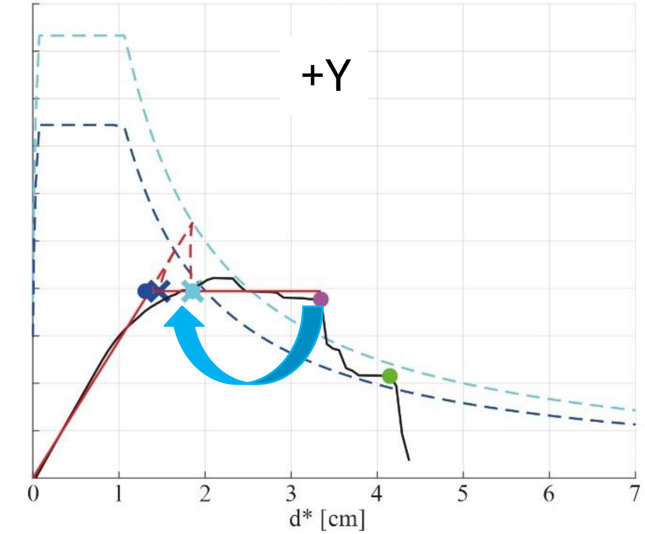
	+Fx -30%Fy	-Fx +30%Fy	+Fy -30%Fx	-Fy +30%Fx
Min θ_{Rd}/θ_{Ed}	1.374	1.438	1.002	1.039

Non Linear Static Analysis

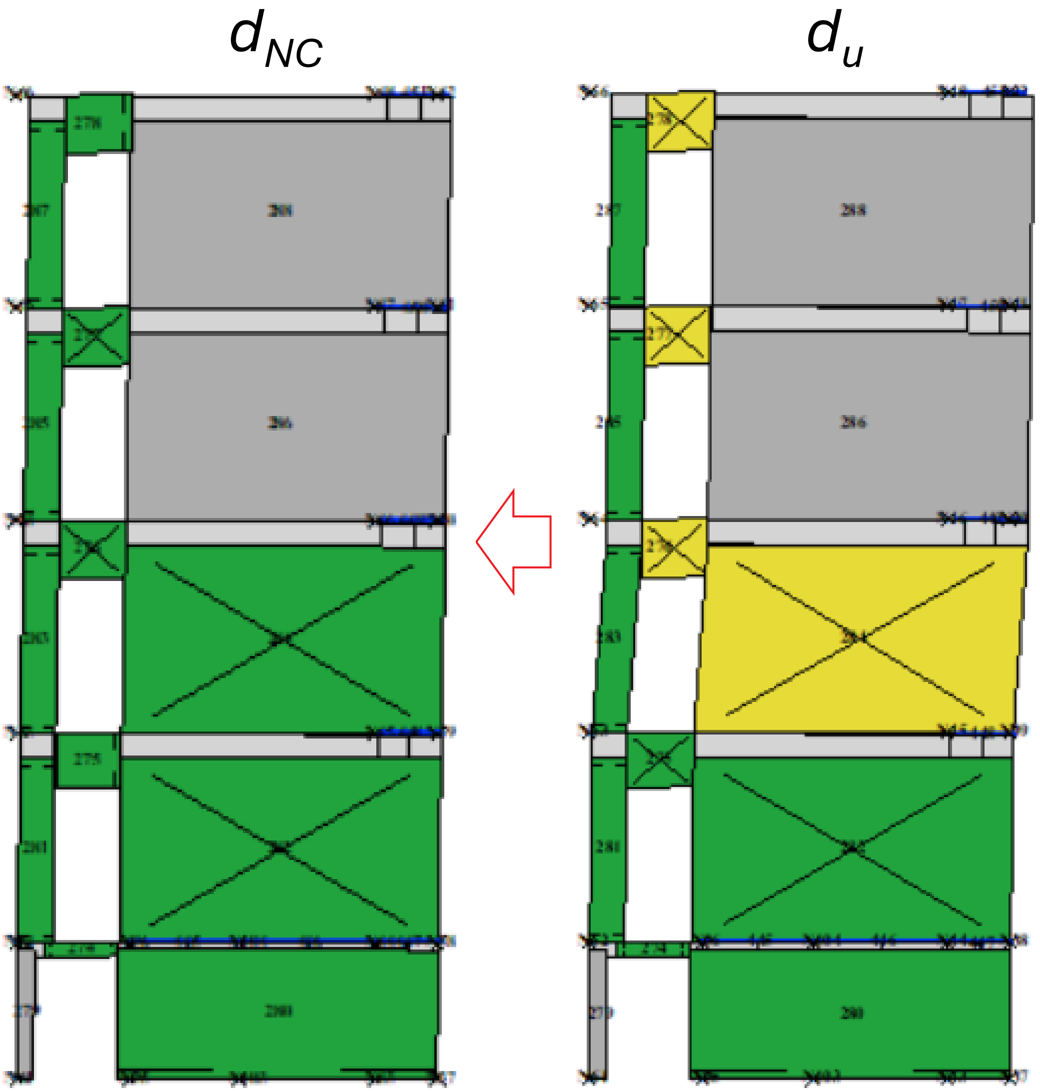
- General rules from EC8-1.1, with additional provisions in EC8-1.2
- Only 4 pushover analyses are requested, with a proper load pattern (additional eccentricity if the natural one is below a minimum value). In addition, “uniform” pattern if a soft story mechanism is expected.
- If diaphragms are not rigid, lateral loads are applied in each node.
- In the case of stiff diaphragms, the control displacement should be the average top displacement of walls, weighted by the masses
- 3 criteria to identify the displacement capacity d_u at NC limit state:
  1. the total base shear has dropped below 80% of the peak
  2. the drift θ_{NC} is reached in all piers at any level of any walls
  3. one pier reached the drift $1.5\,\theta_{NC}$ (loss of bearing capacity)
- A reduction factor γ_{Rd}=1.8 is applied to d_u capacity to obtain d_{NC}
- Displacement at SD is the mean value between d_y and d_{NC}

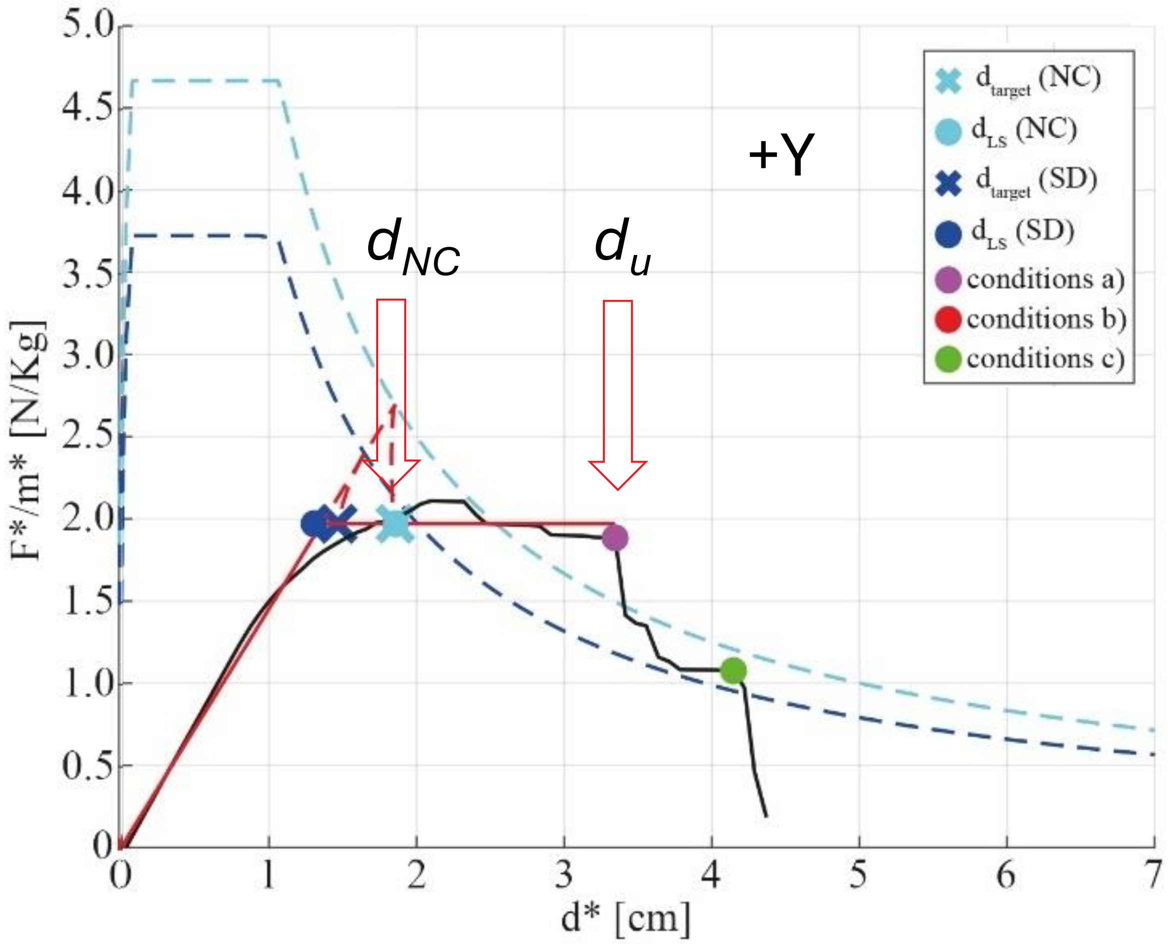
SAFETY FACTOR	+X	-X	+Y	-Y
NC limit state	1.782	1.602	1.001	1.097
SD limit state	1.420	1.296	0.926	0.913





Case study - Zagreb's Lower Town





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

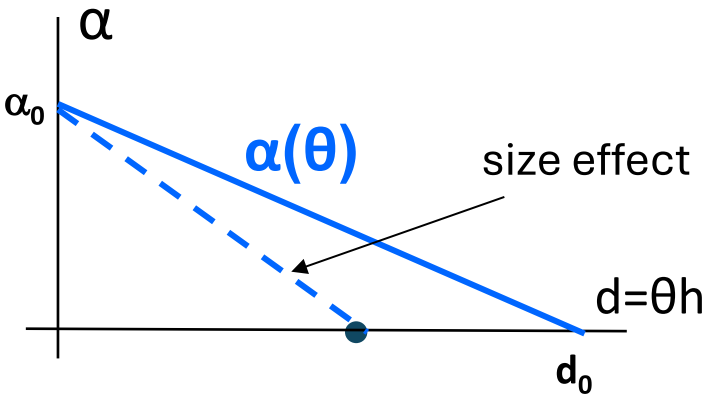
- MODERN MASONRY BUILDINGS $\Longrightarrow$ possible only at interstorey level
- PRE-MODERN MASONRY BUILDINGS $\Longrightarrow$ connections are poor

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

Displacement-Based Assessment of Rocking

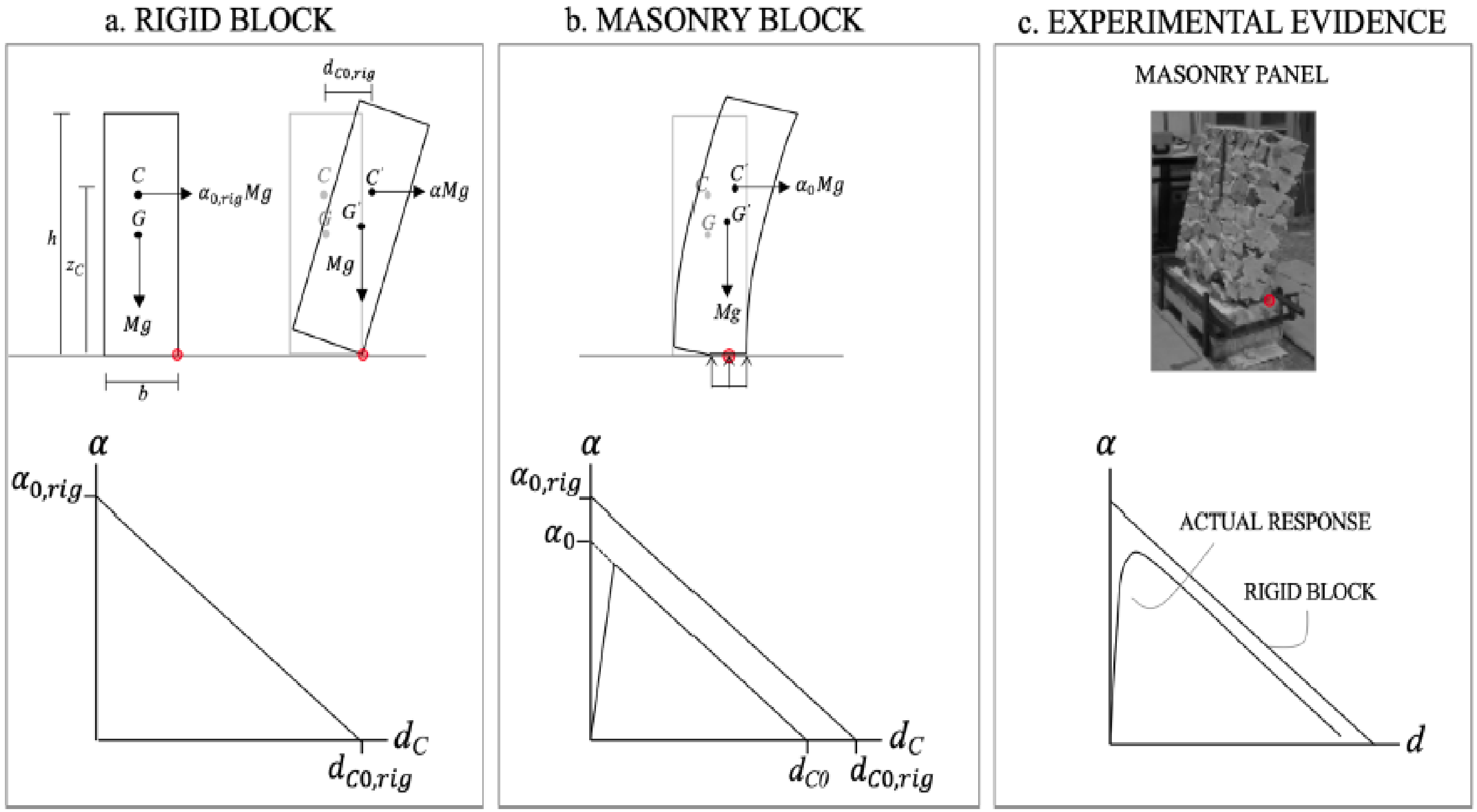
Linear Kinematic Analysis → α_0

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

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

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 second generation of EC8-Part 3 proposes a general framework for the seismic assessment of existing 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