COMMISSION OF THE EUROPEAN COMMUNITIES
FP7- INFRASTRUCTURES-2008-1
SP4-Capacities

S E R I E S

SEISMIC ENGINEERING RESEARCH INFRASTRUCTURES FOR EUROPEAN SYNERGIES

SEVENTH FRAMEWORK PROGRAMME

EUCENTRE Centro Europeo di Formazione e Ricerca in Ingegneria Sismica

"Shaking table test design to evaluate earthquake capacity of a 3-storey building specimen composed of cast-in-situ concrete walls"

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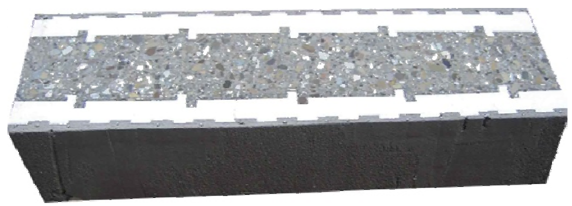
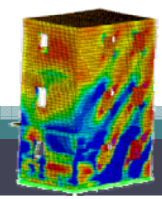
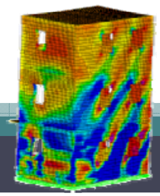


PRESENTATION LAYOUT

- **Construction system**
- **Scientific background** related to sandwich panels
- **Experimental tests** performed during the years and their interpretation
- **Shaking table tests**
  - **Design**
  - **Transportation phase**
  - **Tests**
  - **Preliminary interpretation of the results of the shaking-table tests**

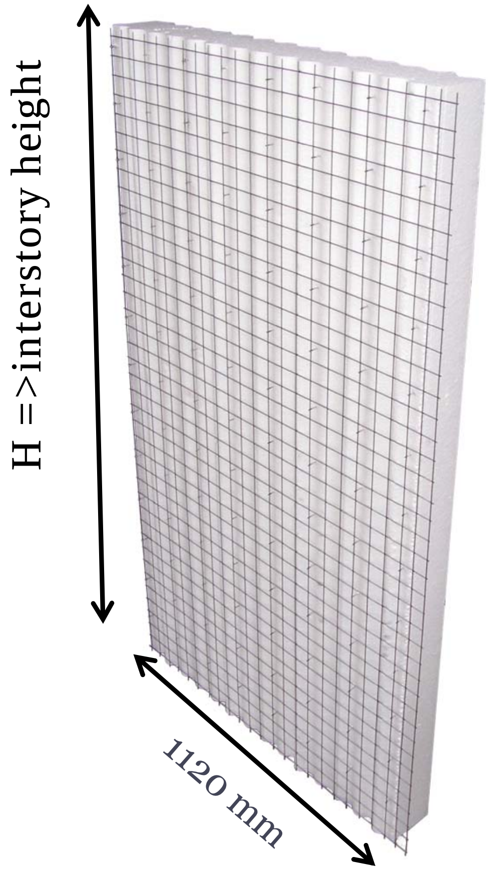


THE CONSTRUCTION SYSTEM




THE CONSTRUCTION SYSTEM

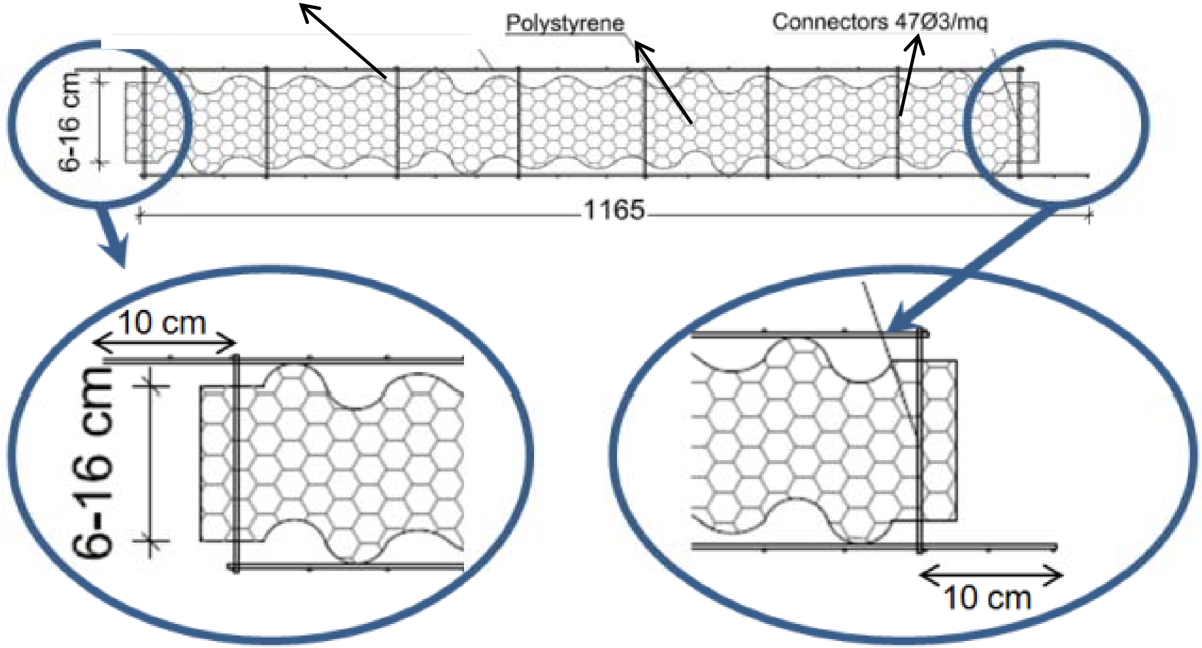
THE MODULAR PANELS



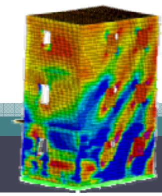
Steel Grids ϕ2.5 mm with a mesh of 100 mm x 100 mm

Polystyrene 60 mm

Metallic Ties ϕ3 mm, quantity 47 for m^2

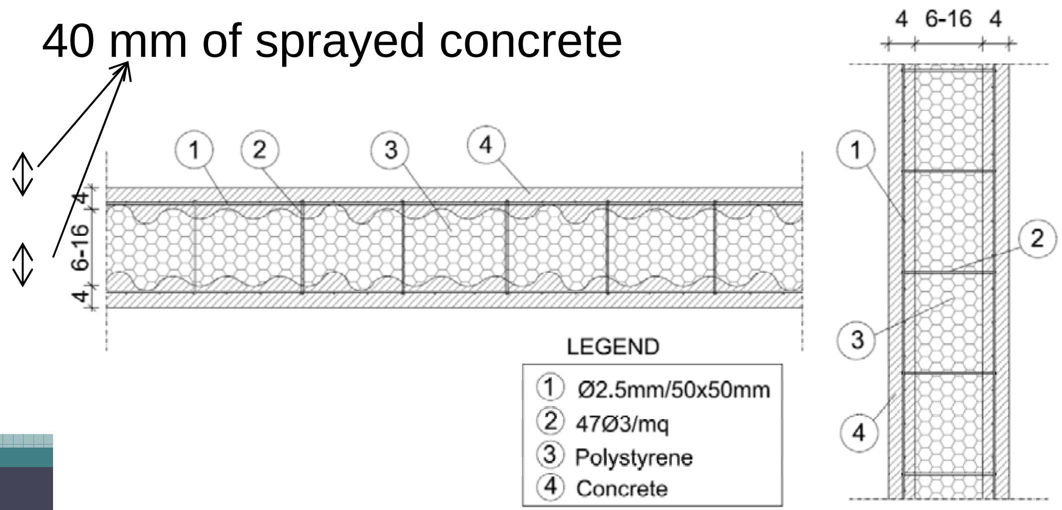


peculiar design of the edges
to allow the continuity of the horizontal reinforcement



THE CONSTRUCTION SYSTEM

THE CAST IN SITU SANDWICH CONCRETE WALLS

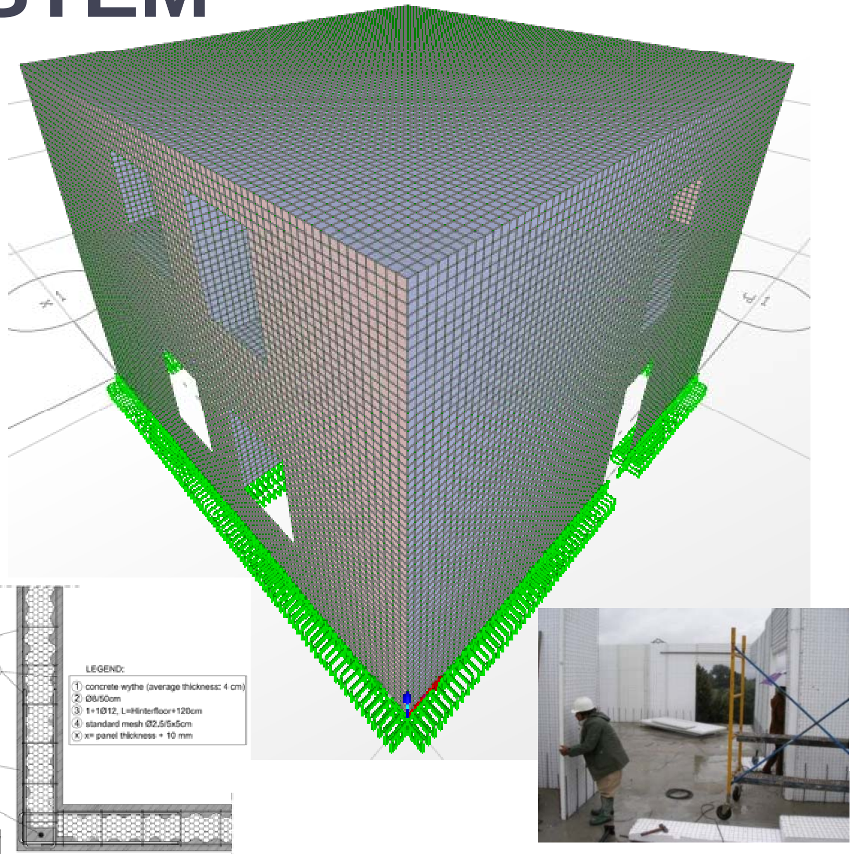


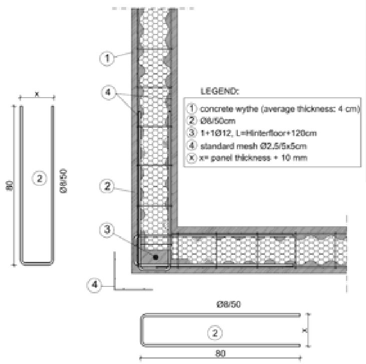
THE PECULARITIES OF THE STRUCTURAL SYSTEM

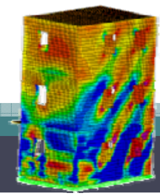
1. **Squat Wall**

2. **Cellular Behaviour**

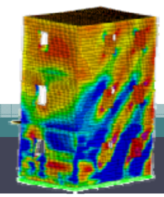
3. **Sandwich wall**







PREVIOUS EXPERIMENTAL TESTS TO SERIES PROJECT




- Uniaxial compression tests
- Diagonal compression tests
- Slip tests between the two r.c. layers
- Out-of-plane bending test

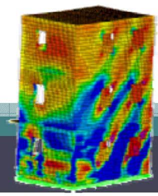
UNIBO in LAPS lab (BOLOGNA) 2002-2003

- **<u>Pseudo-static tests with horizontal loads</u>**

UNIBO in EUCENTRE lab (PAVIA) 2005-2008

- **<u>Shaking table test (december 2011)</u>**

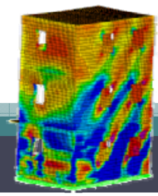
SERIES PROJECT



UNIAXIAL TESTS

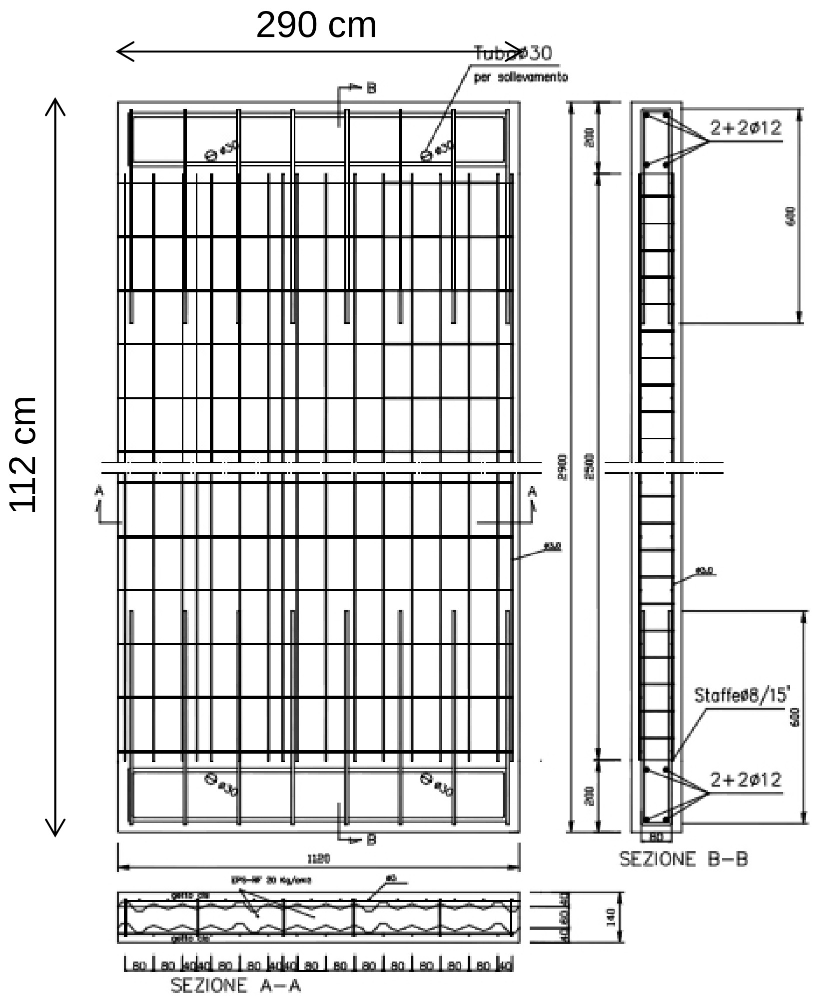
GOAL

To study the **uniaxial behaviour** of single cast in situ sandwich squat concrete wall and to evaluate **the effect of a prescribed eccentricity**

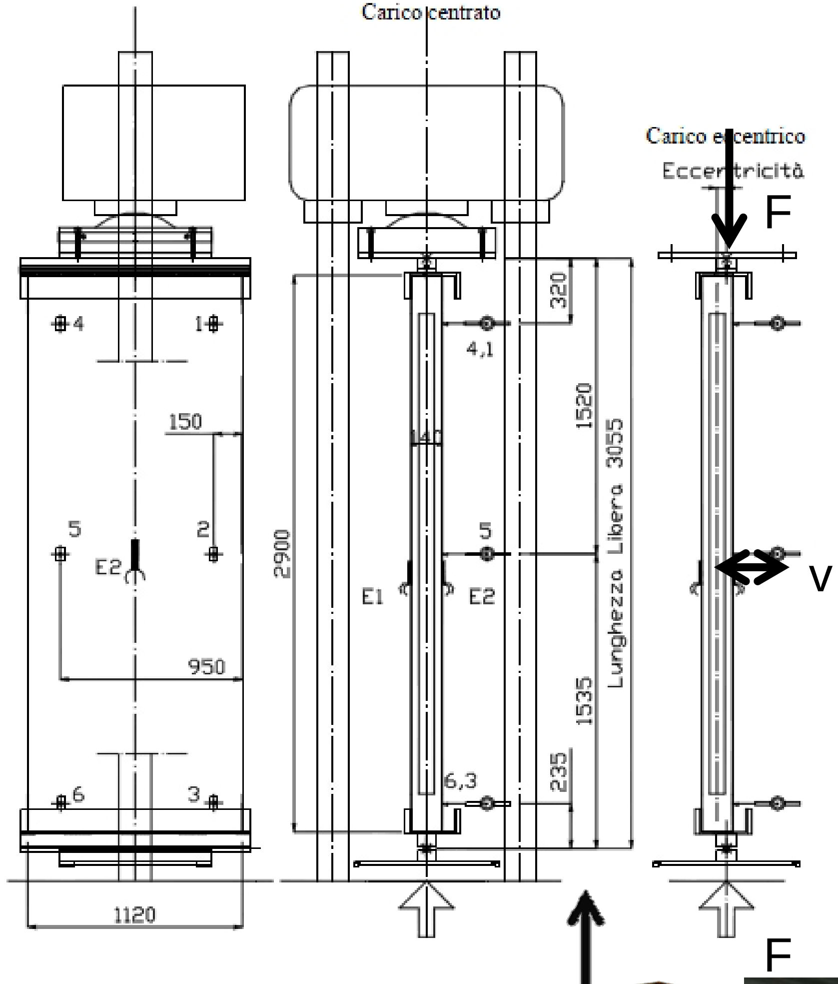


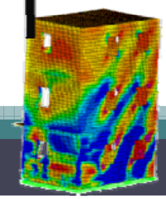
UNIAXIAL TESTS

Panel reinforcement



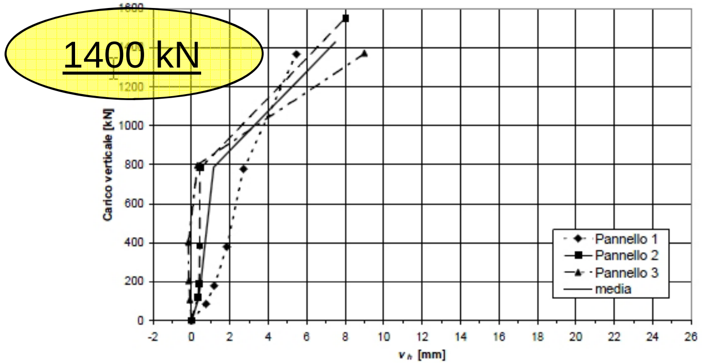
Test layout



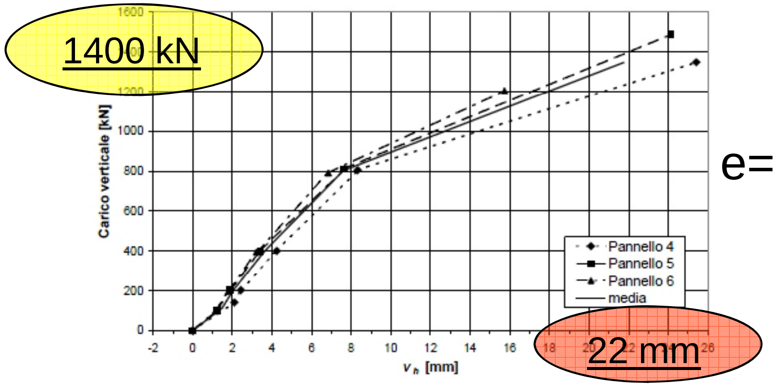


UNIAXIAL TESTS: COMPARISON OF THE RESULTS

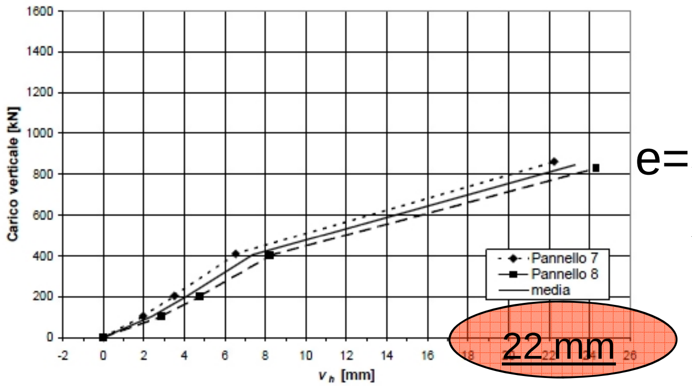
Panel 2, e=0



e=0



e=25 mm



e=50 mm

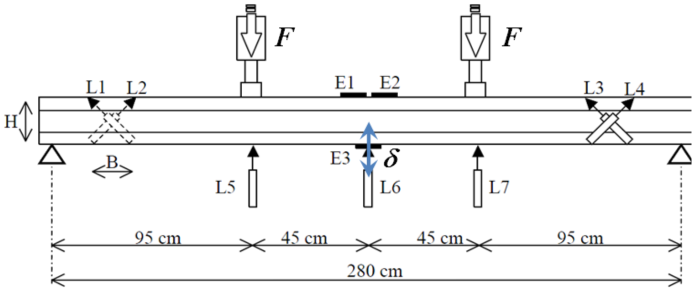
The eccentricity strongly influences both:

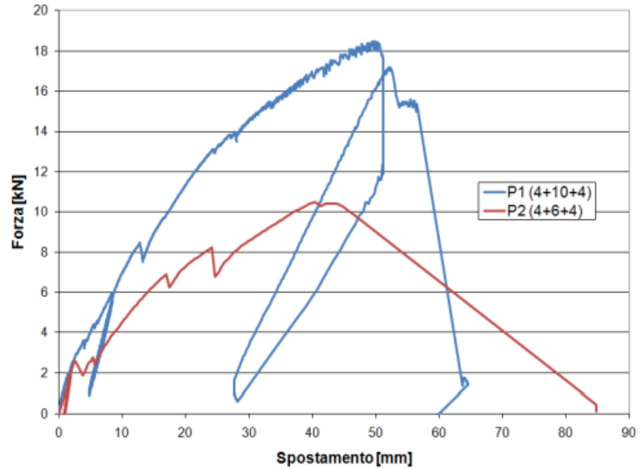
F_{max} → the failure load

V_{max} → the maximum deflection of the panel at the middle length section

Out-of plane bending test

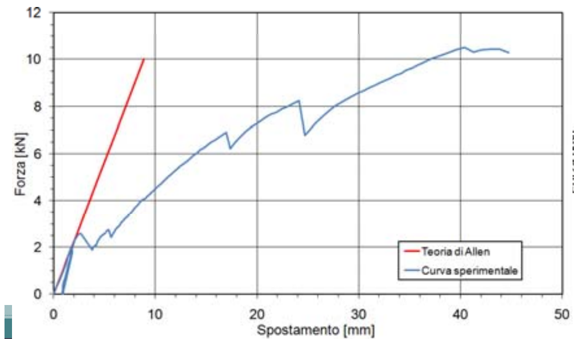
Novembr 2003

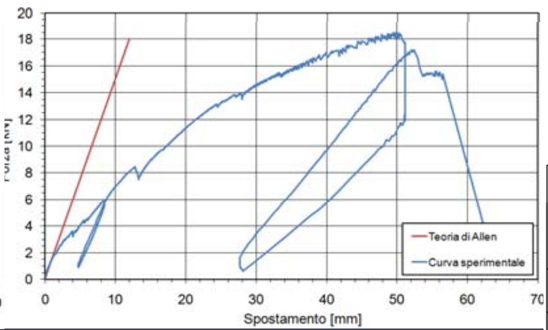




Analtyical-experimental correlations

- Ec = 300000 kg/cm^2
- Ultimate strengths





	M_{Rd} [kNm]	$M_{R,act}$ [kNm]	$M_{D,exp}$ [kNm]
Parete P1 (4+10+4)	7.44	11.50	19.45
Parete P2 (4+6+4)	5.58	8.62	11.85

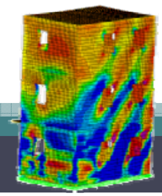
	Taglio resistente teorico, $V_{R,act}$ [kN]	Taglio resistente sperimentale, V_{exp} [kN]
Parete P1 (4+10+4)	20.59	21.25
Parete P2 (4+6+4)	17.36	13.25

PSEUDO-STATIC TESTS WITH CYCLIC HORIZONTAL LOADS

SINGLE WALLS

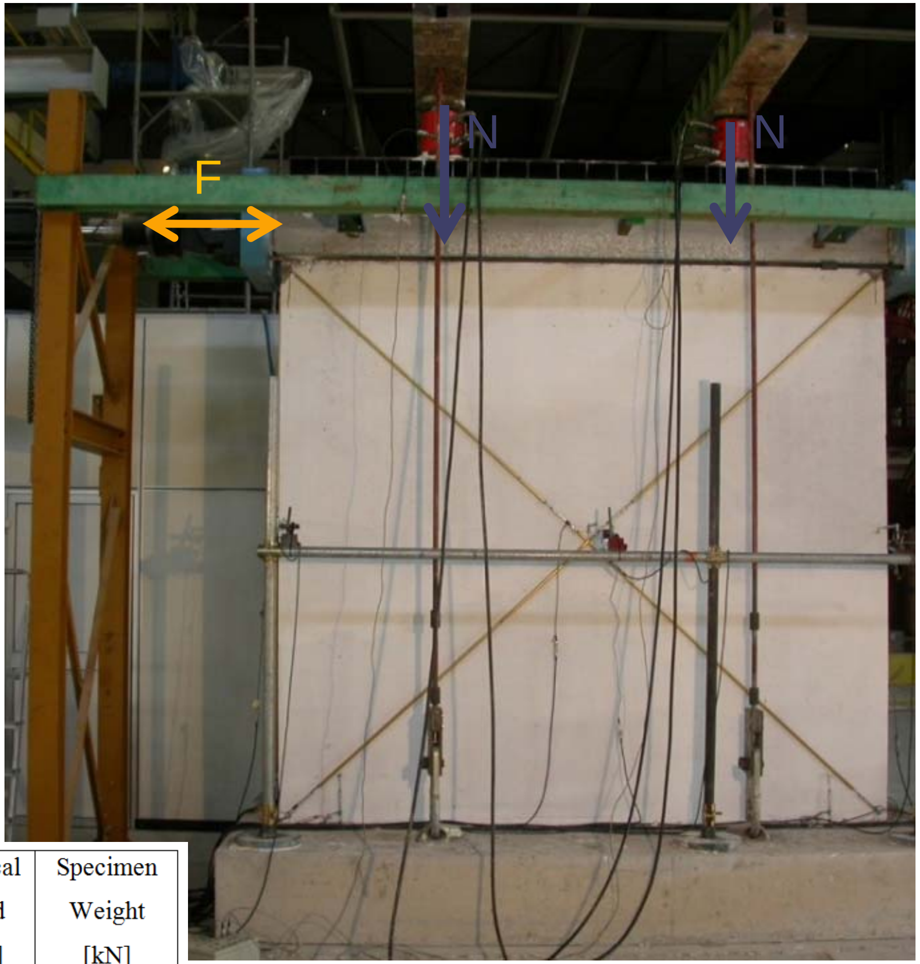
GOAL

Obtaining a **full characterization** of the **pseudo-static behaviour under cyclic horizontal loads** of single cast in situ sandwich squat concrete wall.




PSEUDO-STATIC TESTS WITH CYCLIC HORIZONTAL LOADS

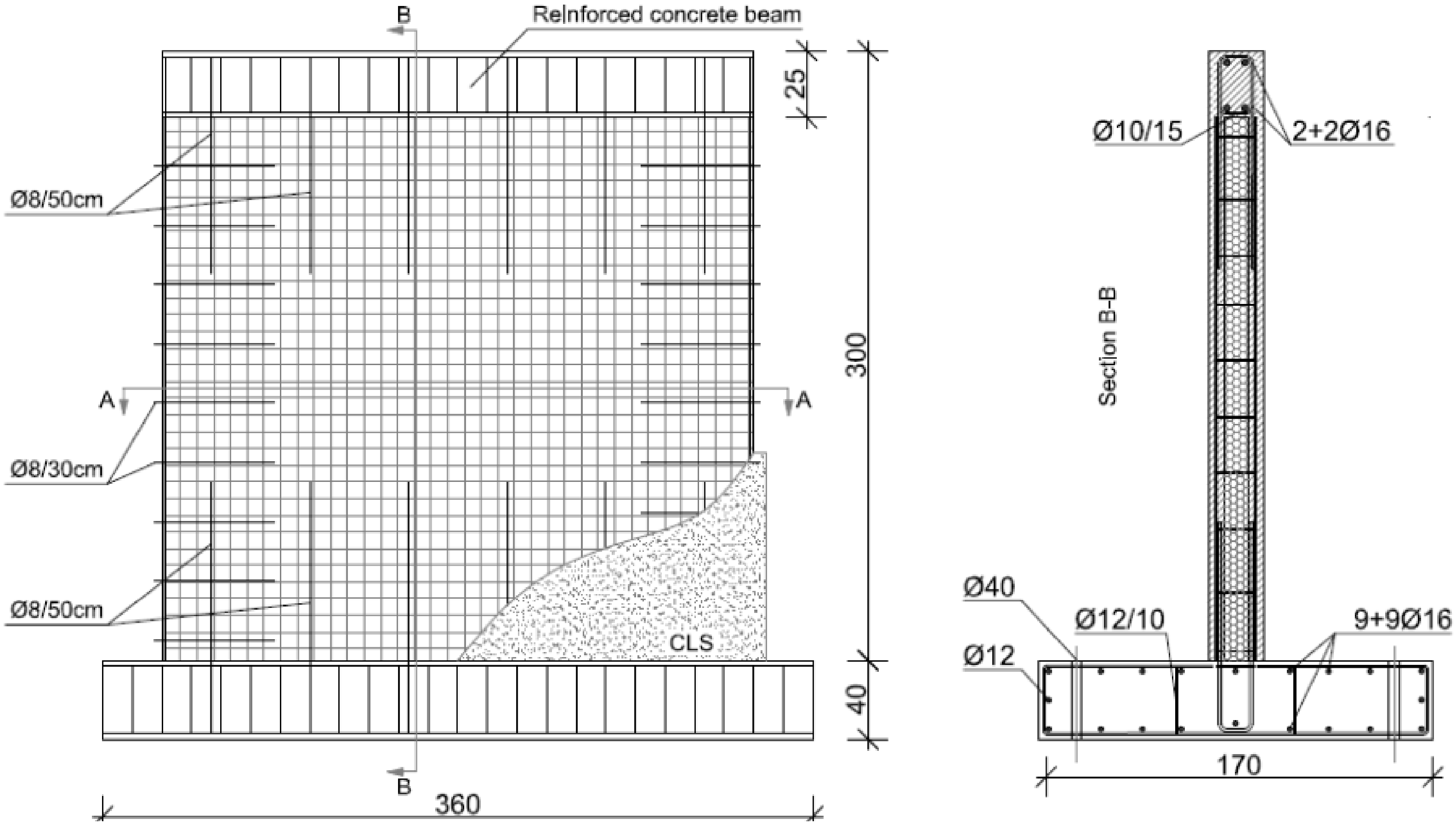
- A total of 6 tests, on two different typology of walls, were performed:
  - **4 tests for wall type A**: 3 m x 3 m square wall with no openings;
  - **2 tests for wall type B**: 3 m x 3 m square wall with a 1 m x 1m square central opening;
- Three different values of the vertical loads applied:
  - 50 kN;
  - 100 kN;
  - 250 kN;
- **3 complete cycles** applied at each step, **increasing levels of imposed horizontal deformations** for a given **constant vertical load**, have been applied



Test	Date	Specimen Typology	Vertical Load [kN]	Specimen Weight [kN]
1	22-12-05	A	50	20
2	20-01-06	A	100	20
3	02-02-06	B	50	20
4	08-02-06	B	100	20
5	09-02-07	A	100	20
6	15-02-07	A	250	20

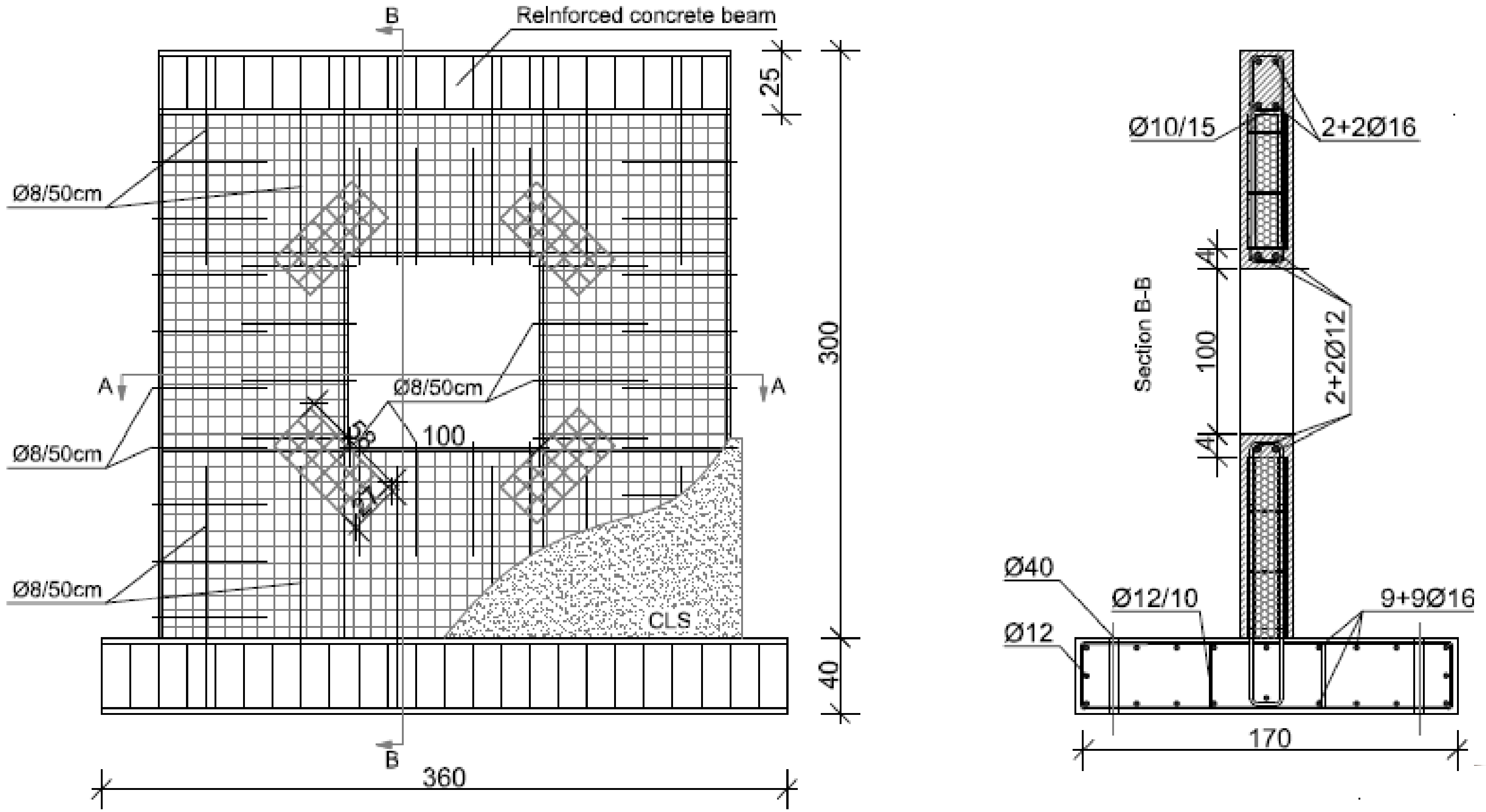
PSEUDO-STATIC TESTS WITH CYCLIC HORIZONTAL LOADS

Reinforcement for Wall **Type A**- Wall without opening



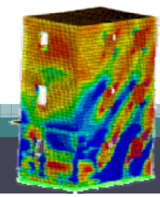
PSEUDO-STATIC TESTS WITH CYCLIC HORIZONTAL LOADS

Reinforcement for Wall **Type B**- Wall with opening



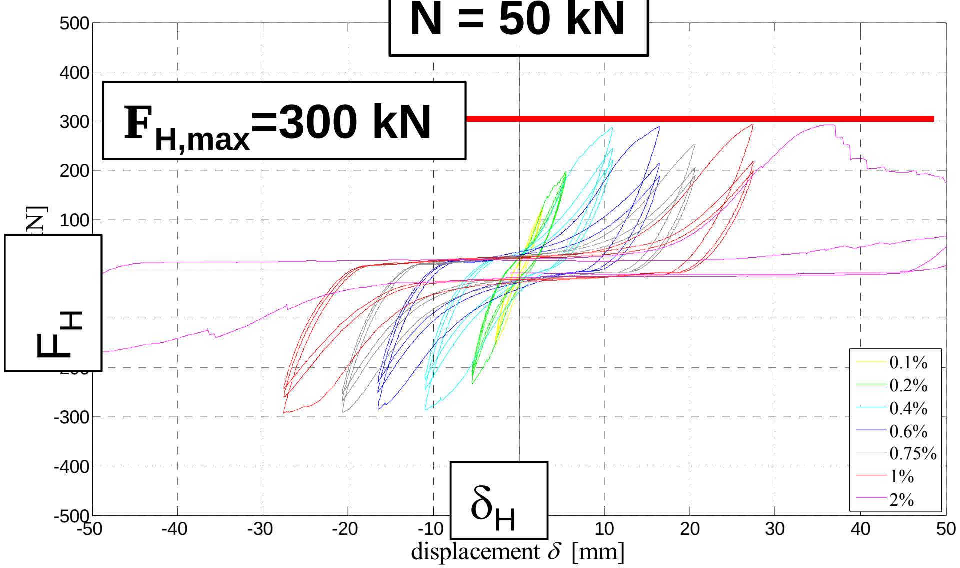
TYPE A:

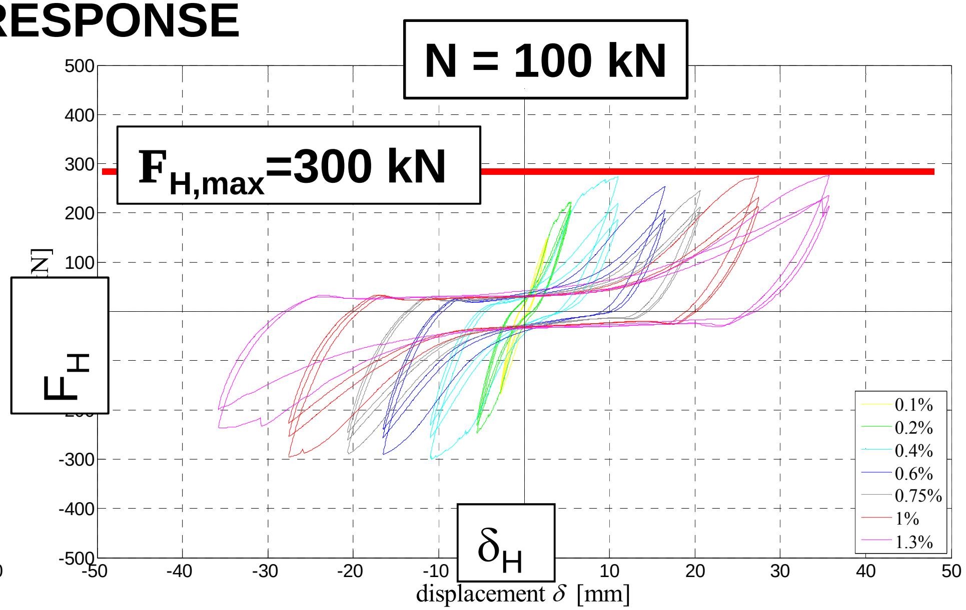
WALLS WITHOUT OPENINGS

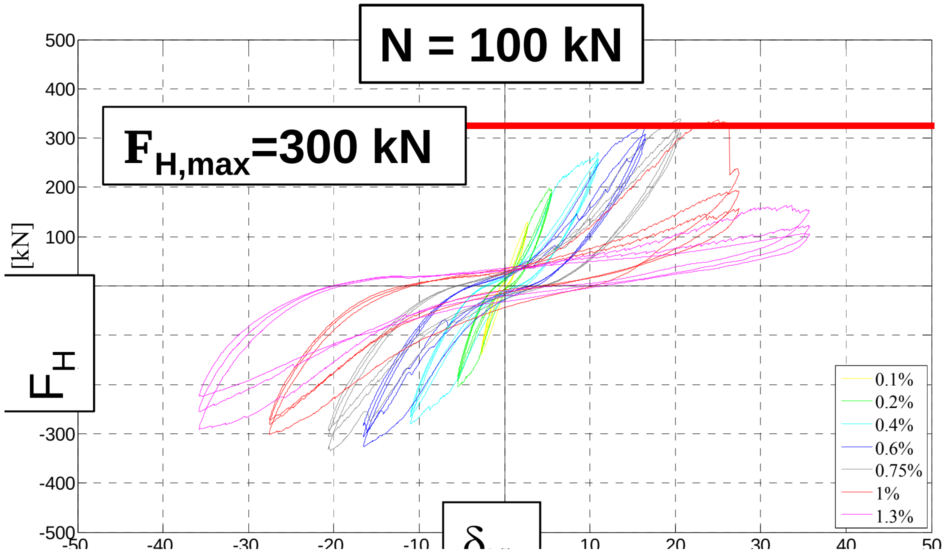


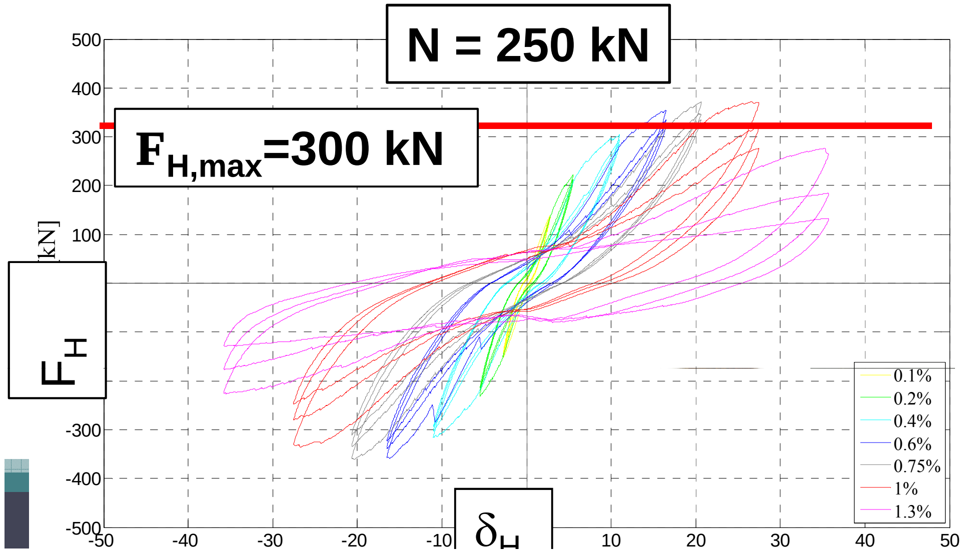

WALL TYPE A: RESULTS

CYCLIC RESPONSE



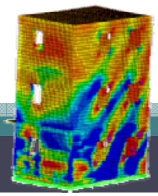






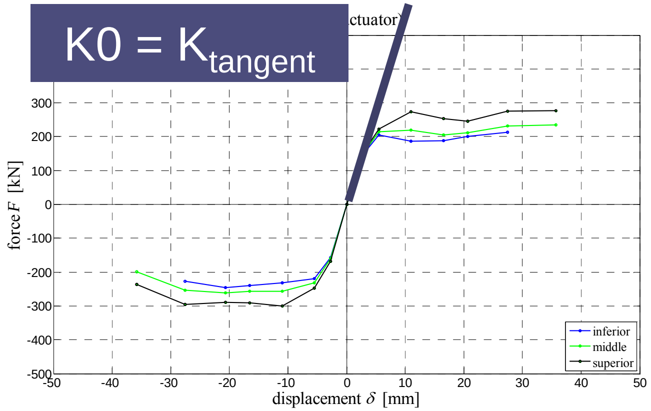
WALL TYPE A: RESULTS

	TEST 1 N=50 kN			TEST 2 N=100 kN			TEST 3 N=250 kN		
Drift [%]	F_{Tmax} [kN]	F_{Cmax} [kN]	F_{Mmax} [kN]	F_{Tmax} [kN]	F_{Cmax} [kN]	F_{Mmax} [kN]	F_{Tmax} [kN]	F_{Cmax} [kN]	F_{Mmax} [kN]
0.10	125.6	154.3	139.9	128.5	141.2	134.9	138.1	152.3	145.1
0.20	197.1	232.8	214.9	199.1	204.5	201.8	221.8	231.2	226.5
0.40	288.4	286.6	287.4	270.2	279.6	274.9	304.5	316.1	310.3
0.60	289	285.3	287.1	327.2	326.1	326.7	354.2	359.6	356.9
0.75	253.7	291.4	272.5	339.2	334.1	336.7	371.5	360.9	366.2
1.00	294.5	291.7	293.1	336.7	301.2	319.0	371.7	335.5	353.6

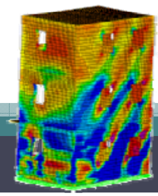


WALL TYPE A: STIFFNESS

N (kN)	K theory, gross section	K theory, uncracked	K theory, fully cracked	K0 experimental, tangent (initial)
50	1	1.04	0.11	0.14
100	1	1.04	0.12	0.16
100	1	1.04	0.12	0.14
200	1	1.04	0.12	0.15

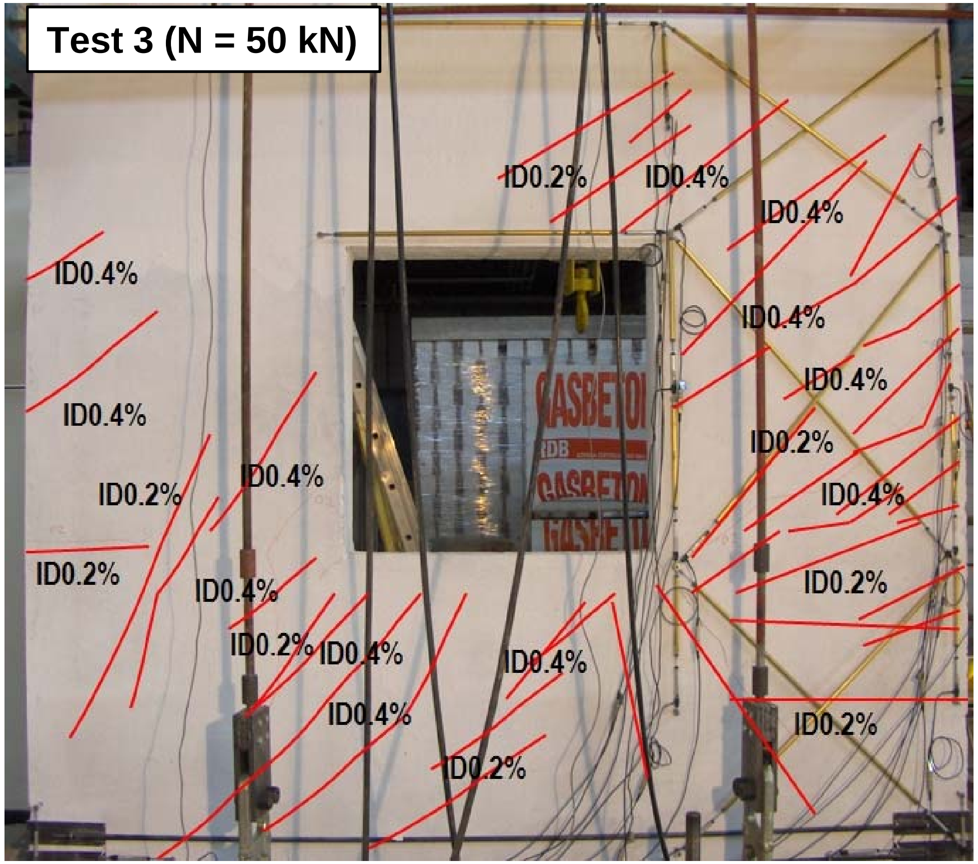


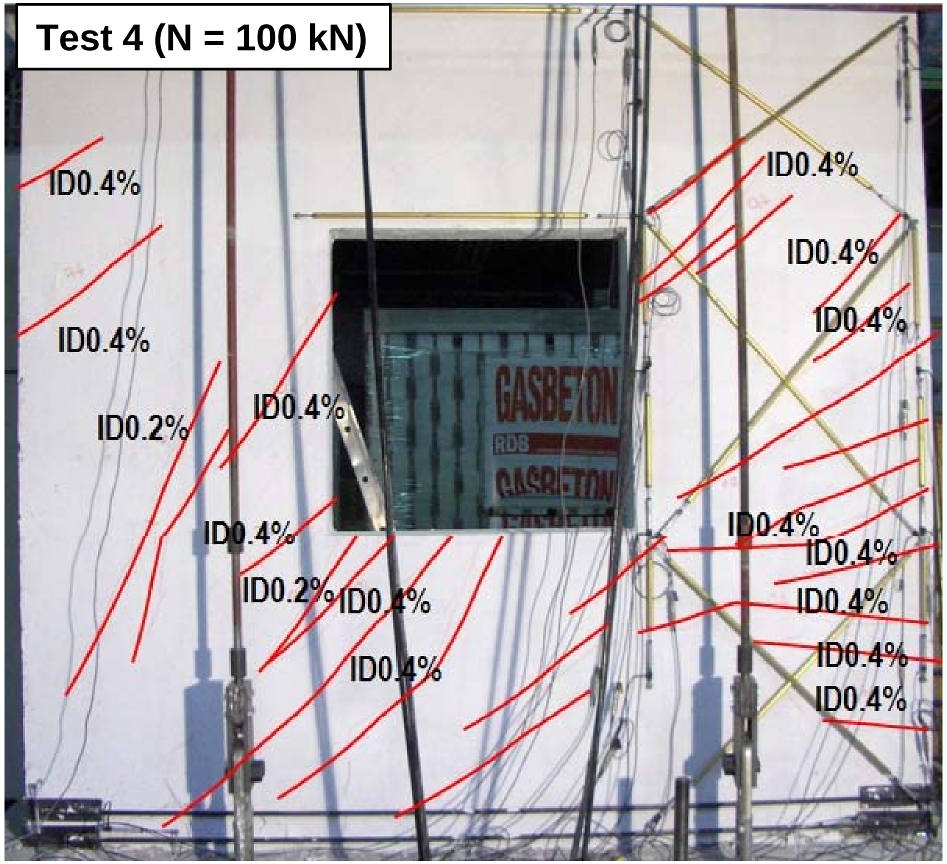
- K_0 is completely different from (much lower than) the $K_{\text{theory,uncracked}}$
- K_0 is closer to the $K_{\text{theory,fully cracked}}$ rather than to the $K_{\text{theory,uncracked}}$
- K_0 is larger than $K_{\text{theory,fully cracked}}$

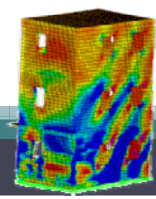


WALL TYPE B:RESULTS

CRACKING PATTERNS



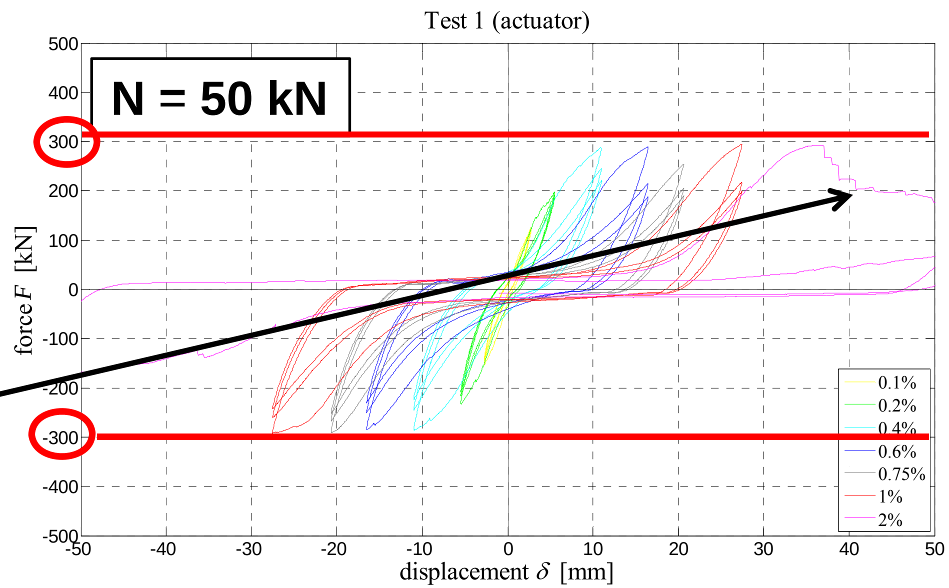


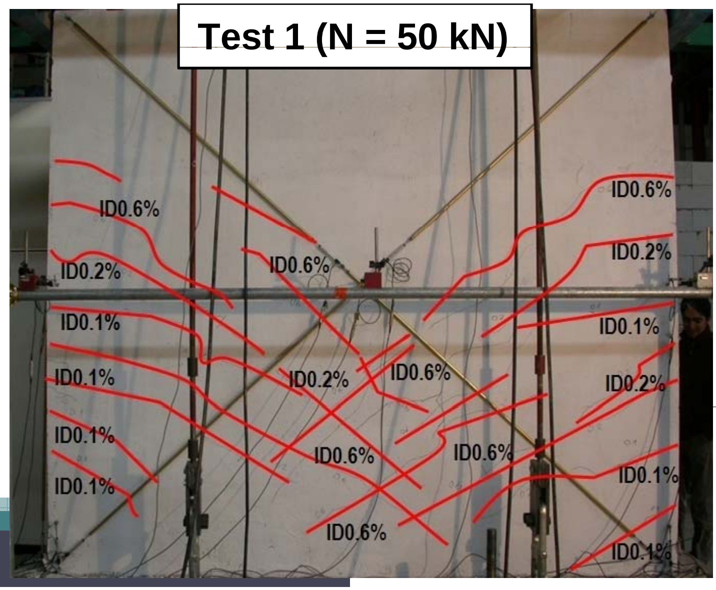


OBSERVATIONS DESUMED FORM THE RESULTS

The results obtained from the pseudo-static tests with cyclic horizontal load upon **six 2-dimensional (3.0 m b 3.0 m) elements with and without opening**, have shown that the tested walls are characterized by:

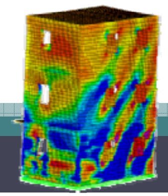
- **absence of a real and authentic failure**:**"virtual collapse"=>**no real collapse of the specimen has been reached, but a **visible lateral strength reduction** of the specimen has been observed;
- **residual bearing capacity** with respect to the **vertical loads**;
- high values (about **300 kN**) of **the maximum horizontal load applied** to the specimens;
- cracking patterns indicating a typical **"bending" mode of failure**;
- a **maximum lateral force** which is **not significantly influenced by the vertical load applied**;
- **no significant differences** between the walls **with and without opening.**



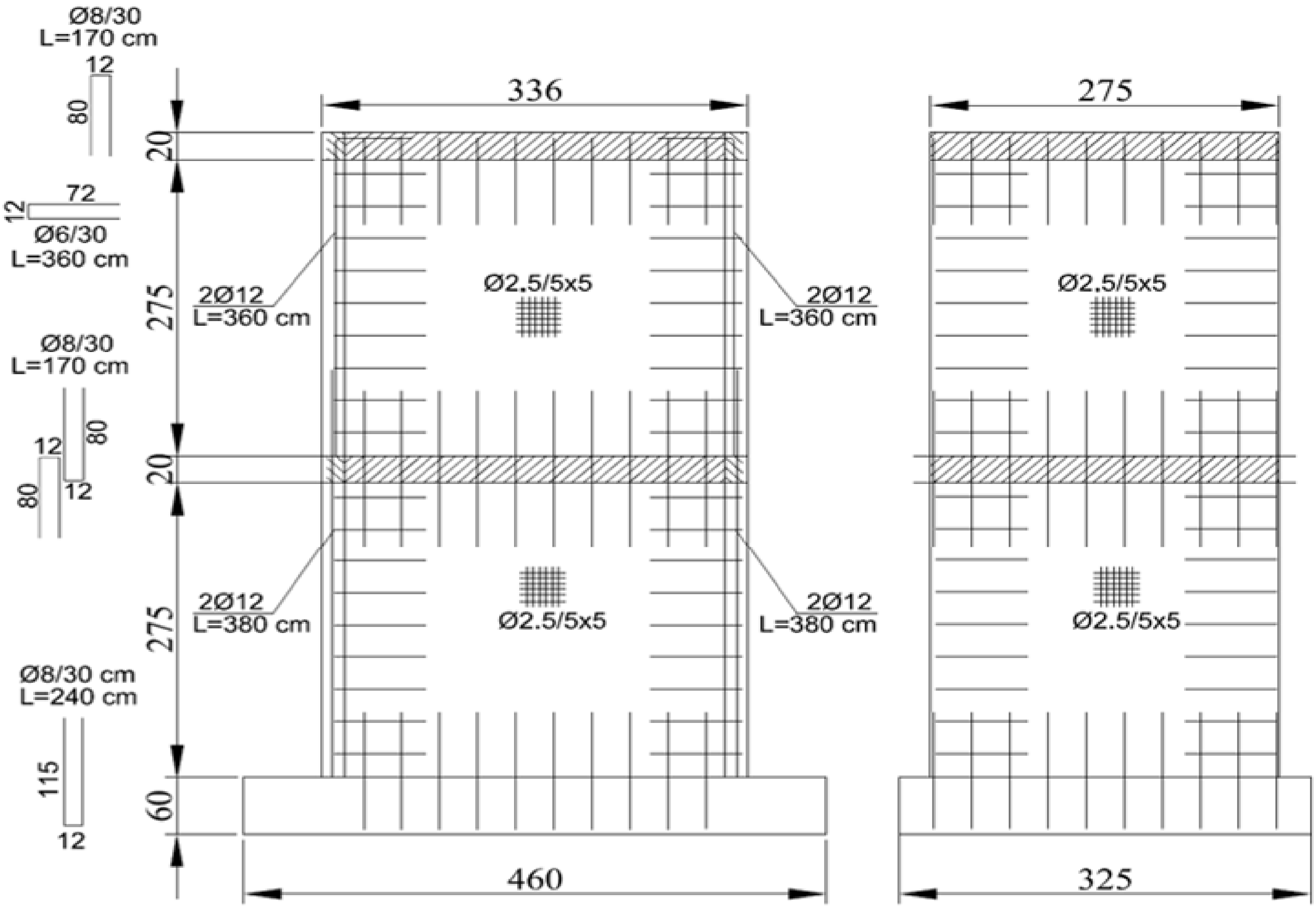


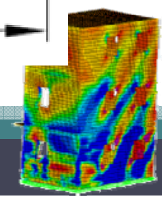
PSEUDO-STATIC TESTS WITH CYCLIC HORIZONTAL LOADS:

H-SHAPED STRUCTURE

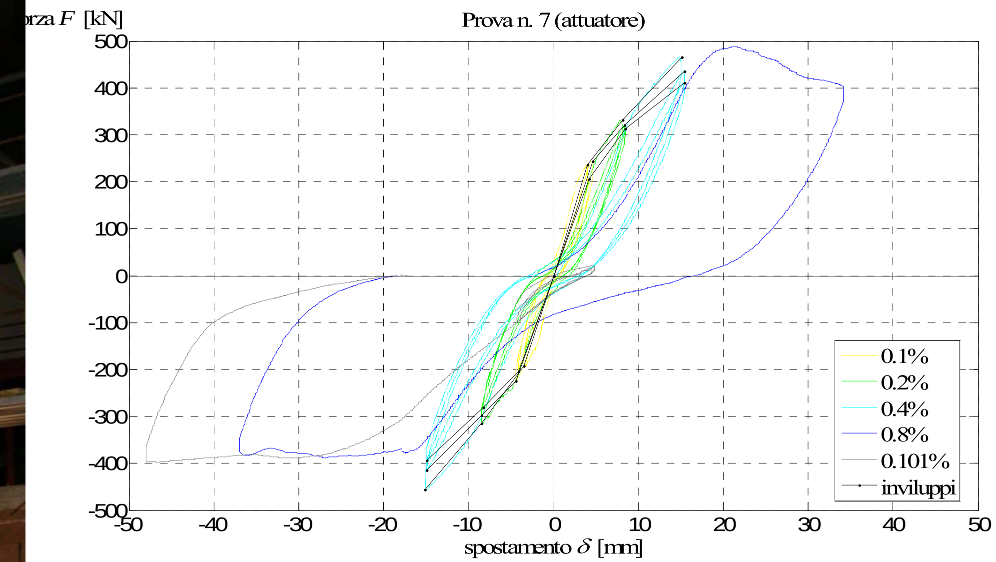


THE H-SHAPED STRUCTURE



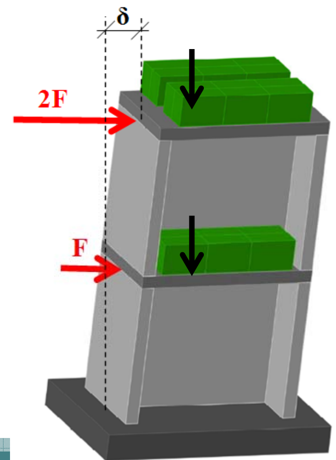


THE H-SHAPED STRUCTURE



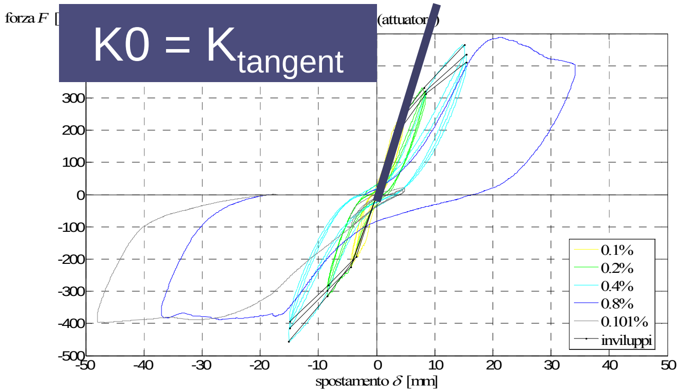
Vertical load
30 t

Ciclic horizontal load:
50 t



THE H-SHAPED STRUCTURE: STIFFNESS

K theory, gross section	K theory, uncracked	K theory, fully cracked	K0 experimental, tangent (initial)
kN/m	kN/m	kN/m	kN/m
324820	342376	20338	36242
1	1.05	0.06	0.11



- K_0 is completely different from (much lower than) the $K_{theory,uncracked}$
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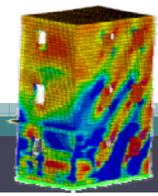
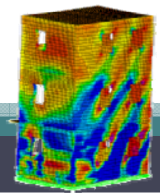


THE H-SHAPED STRUCTURE: STRENGTHS

	$R_{analytical,d}$ [kN] design values	$R_{analytical,mean}$ [kN] mean values	$R_{experimental}$ [kN]
Forza di primo snervamento	$F_{1yd} = 174$ kN	$F_{1y,act} = 236$ kN	$F_{1y,D,\exp} = 217$ kN
Forza ultima	$F_{Rd} = 367$ kN	$F_{R,act} = 471$ kN	$F_{u,D,\exp} = 465.8$ kN
Flessione	$M_{Rd} = 1679$ kN m	$M_{R,act} = 2159$ kN m	$M_{D,\exp} = 2273$ kN m
Taglio	$V_{Rd} = 448.1$ kN	$V_{R,act} = 607.0$ kN	$V_{D,\exp} = 465.8$ kN
Scorrimento alla base	$S^{*}_{Rd} = 442.6$ kN	$S^{*}_{R,act} = 535.1$ kN	$S_{D,\exp} = 465.8$ kN
Scorrimento delle connessioni	$S_{Rd,\ connessioni} = 556$ kN	$S_{R,act,\ connessioni} = 740$ kN	$S_{D,\exp} = 465.8$ kN

detalis given in next slide



THE H-SHAPED STRUCTURE: ANALITYCAL STRENGTHS OF THE SINGLE WALLS

Parallel wall – First yielding for bending in the floor

$$M_{y1} = \left(\frac{\rho b y_{y1}}{2} f_{ym}\right)\cdot\left(\frac{h}{2}-\frac{y_{y1}}{3}\right)+\left(\frac{b\left(h-y_{y1}\right)^2}{2y_{y1}}\frac{f_{ym}}{n}\right)\cdot\left(\frac{h}{6}+\frac{y_{y1}}{3}\right)+A_{s,catena} f_{ym}\left(h-2c\right)=125 \text{ t m}$$

Parallel wall – Ultimate strength for bending in the floor

$$M_{Rd,\,\text{parete}\,//} = \left(f_{ym}\cdot\rho\cdot b\cdot y_{u,sb}\right)\cdot\left(\frac{h}{2}-\frac{y_{u,sb}}{2}\right)+\left(f_{cm}\cdot b\cdot 0{,}8\left(h-y_{u,sb}\right)\right)\cdot\left(0{,}1h+0{,}4y_{u,sb}\right)+A_{s,catena} f_{ym}\left(h-2c\right)=153 \text{ t m}$$

Parallel wall – Shear strength in the wall

$$T_{Rd} = \min\left(T_{Rcd},T_{Scd}\right)=61 \text{ t} \qquad T_{Rsd}=0{,}9\cdot d\cdot\frac{A_{sw}}{s}\cdot f_{ym}\cdot\left(\cot\theta+\cot\alpha\right)\cdot\sin\alpha \qquad T_{Rcd}=0{,}9\cdot d\cdot b\cdot\alpha_c\cdot f'_{cm}\cdot\frac{\left(\cot\theta+\cot\alpha\right)}{\left(1+\cot^2\theta\right)}$$

Perpendicular wall – Shear strength

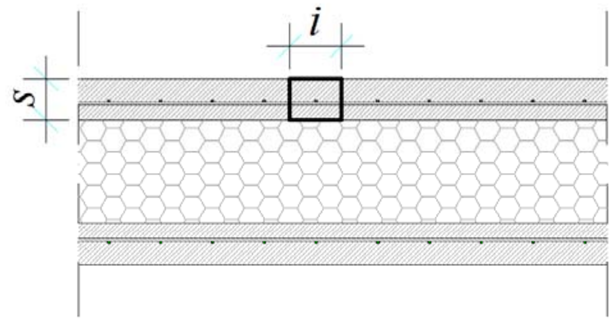
$$N_{Rd,\,\text{parete}\,\perp} = \sigma_{\max}\cdot b\cdot \ell_{\perp} \cong 20 \text{ t}$$

σ_{max} = 12 kg/cm²

Maximum admissible strength (for traction) for "steel-concrete" material considering a diffuse reinforcement of 1+1ϕ2.5/5cm

Parallel wall – Base shear strength

$$S_{Rd,\,parete//} = \mu\cdot N_{Ed}+A_{s,riprese\,//}\cdot\frac{f_{ym}}{\sqrt{3}}=75 \text{ t}$$



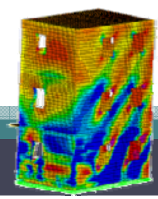
THE H-SHAPED STRUCTURE: ANALITYCAL STRENGTHS OF THE SINGLE WALLS

$$M_{y1,structure} = N_{Rd,\ \mathrm{parete}\ \perp} \cdot h_{//} = 64\ \mathrm{tm}$$

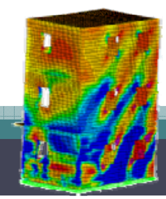
$$M_{u,structure} = N_{Rd,\ \mathrm{parete}\ \perp} \cdot h_{//} + M_{u,\mathrm{parete}\ //} = 217\ \mathrm{tm}$$

$$T_{u,structure} = T_{Rd,parete//} = 61\ \mathrm{t}$$

$$S_{u,structure} = S_{Rd,parete//} = 75\ \mathrm{t}$$

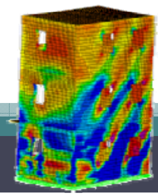


THE 3-STOREY BUILDING
AND
THE SHAKING-TABLE TESTS

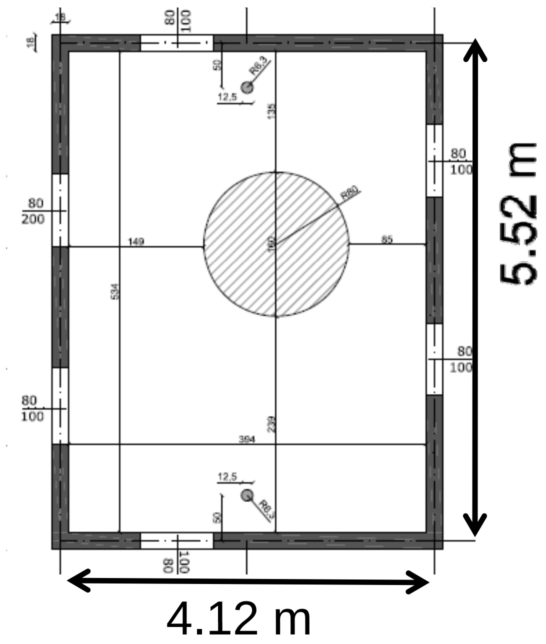


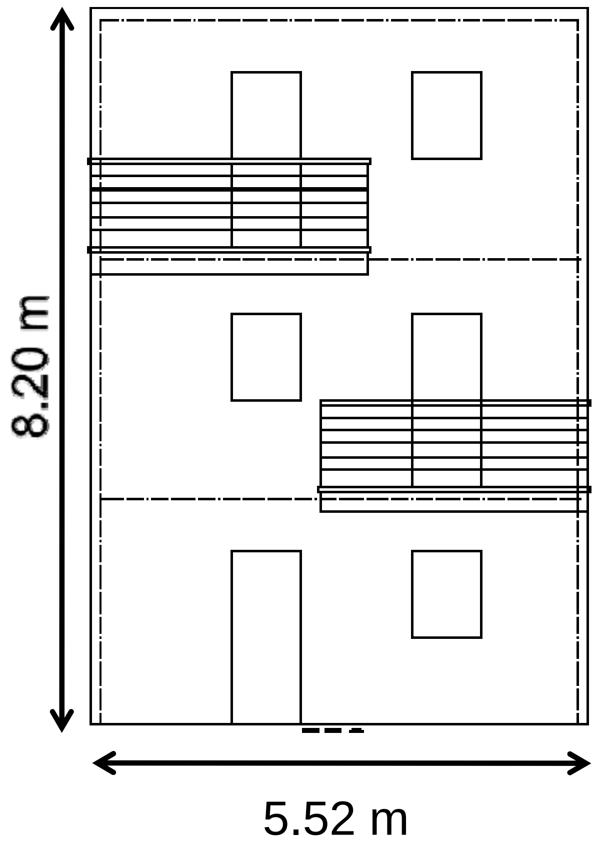

DESIGN PHASE

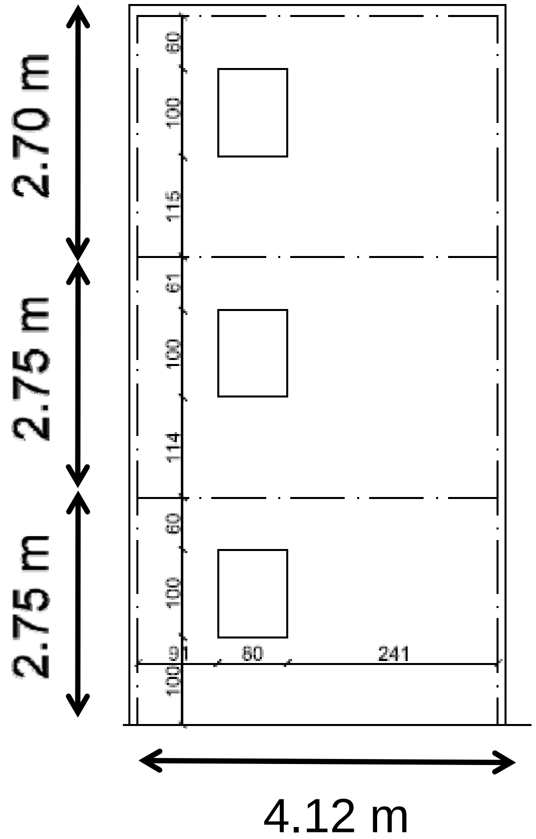
- Shaking table: a single degree-of-freedom
- Rigid platform: 5.6mx7.0 m
- Payload range between 700 to 1400 kN
- Peak acceleration with a maximum payload: 1.8g.
- Maximum force is 2100 kN and the
- Maximum overturning moment: 4000 kNm.
- Maximum admissible height: 9 m.

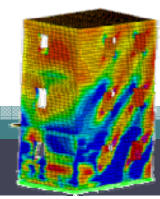


DIMENSIONS OF THE 3-STOREY BUILDING









SERIES

Loads

Solaio di Copertura					
Permanenti	235	kg/m^2	$A_{copertura}$	21	m^2
Extra	285	kg/m^2	$W_{copertura}$	10.9	t

Solaio di Piano Secondo					
Permanenti	329	kg/m^2	$A_{piano\ 2}$	19	m^2
Balcone	212	kg/m^2	$A_{balcone\ piano\ 2}$	1.93	m^2
Extra	224	kg/m^2	$W_{piano\ 2}$	10.9	t

Solaio di Piano Primo					
Permanenti	329	kg/m^2	$A_{piano\ 1}$	19	m^2
Balcone	215	kg/m^2	$A_{balcone\ piano\ 2}$	1.86	m^2
Extra	224	kg/m^2	$W_{piano\ 1}$	10.9	t

Structure weight

- Elevation weight during the transportation phase (only elevation without extra):

$$W_{\text{elevazione nuda}} = 51\ \text{t}$$

- Total weight of the structure **during the transportation phase** (elevation without extra + foundation):

$$W_{\text{struttura nuda}} = 51 + 14 = 65\ \text{t}$$

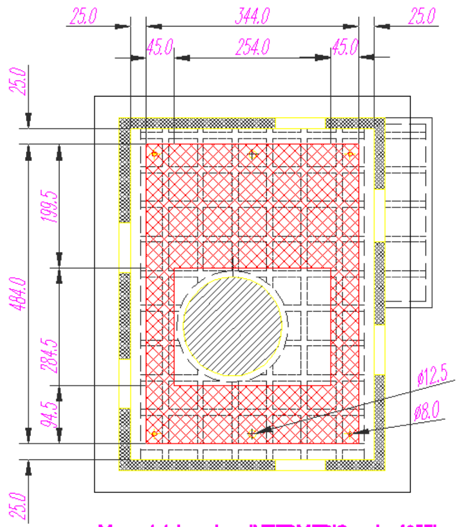
- Elevation weight during the test (elevation with extra):

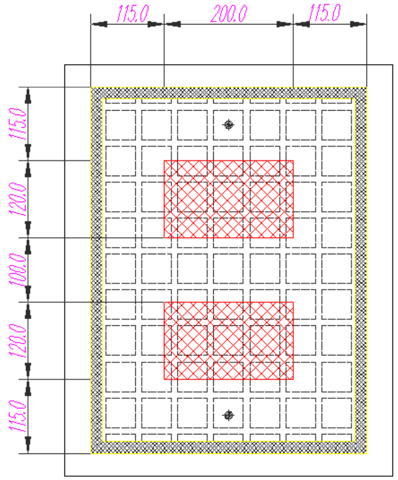
$$W_{\text{elevazione in fase di prova}} = 66\ \text{t}$$

- Total weight of the structure **during the test** (elevation with extra + foundation):

$$W_{\text{struttura in fase di prova}} = 66 + 14 = 80\ \text{t}$$

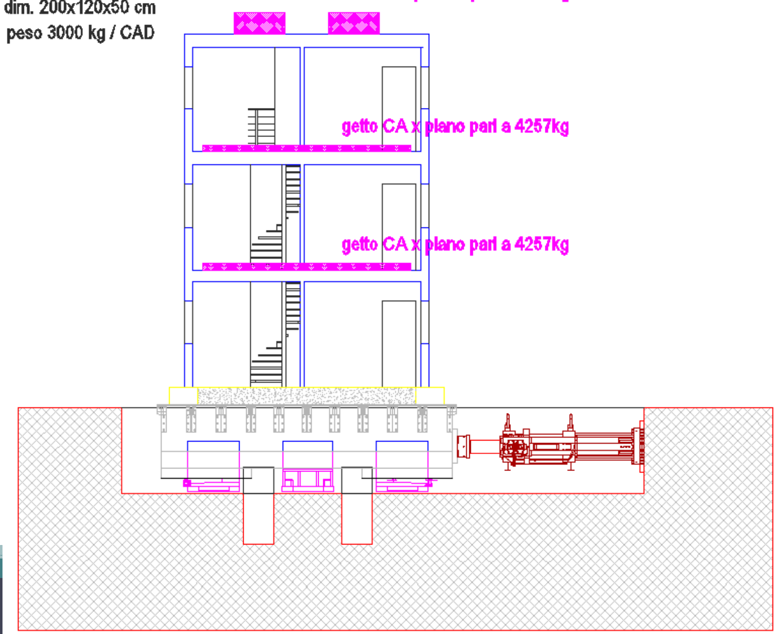
Additional loads

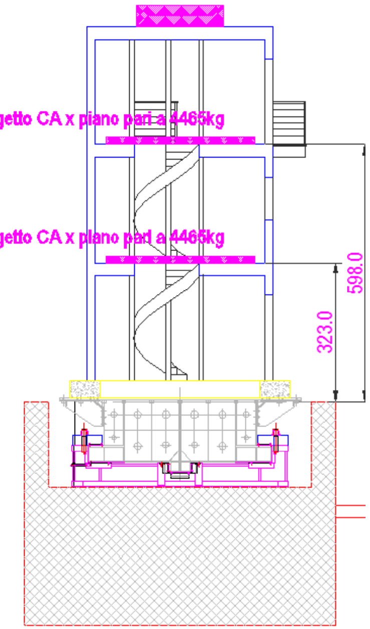




Top floor:
Two **3 t-mass** in r.c.

Intermediate floors:
Shot-crete in concrete
(s=**15 cm**)





Material parameters adopted for the design phase

- ✓ **WALLS**: **C25/30** concrete applied as "**spritz beton**"; (shotcrete)
- ✓ **FLOORS**: **C25/30** concrete applied with a traditional concreting;
- ✓ **INTEGRATIVE REINFORCEMENT**: **B450C** steel;
- ✓ **REINFORCEMENT IN THE PANELS** : **zinc-plated steel** with the same characteristics of **B450C.**

Strength in the design phase:

- Average compression stength in concrete : $f_{cm} = 30 \text{ MPa}$
- Average yielding strength in steel: $f_{ym} = 500 \text{ MPa}$
- Average yielding strength in zinc-plated steel: $f_{ym} = 500 \text{ MPa}$

Analytical evaluation of the accelerations corresponding to possible collapse mechanisms of the structure

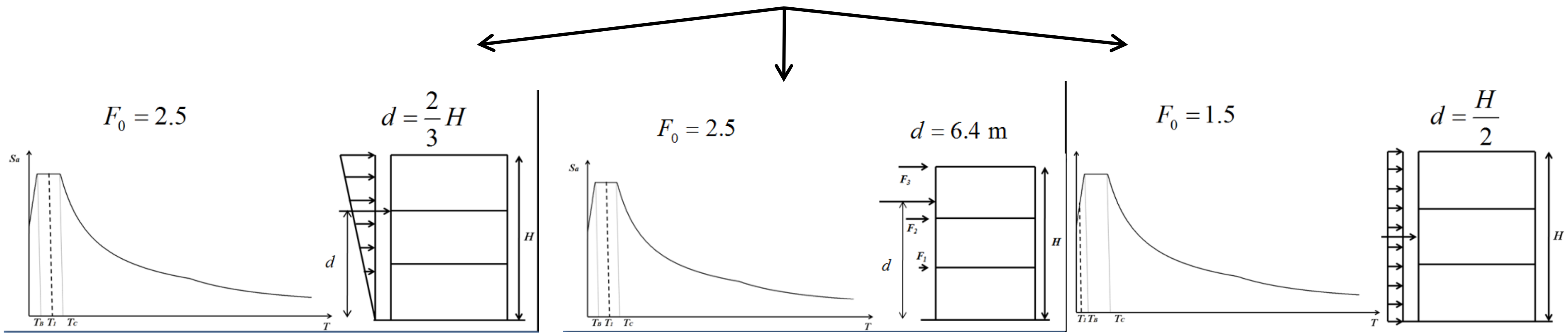
To evaluate the **spectral accelerations** of the different **collapse mechanisms** of the model building it has been determined:

•The **actions** (i.e. demand) **in the walls** (**parallel and perpendicular**) following the application of a spectral acceleration equal to $S_a = 1g$ and

•The corresponding **rstrength** (i.e. capacity).

Comparing the **actions** due to 1g with the corresponding **strength**, it has been possible to find the sequence of all the possible collapse mechanisms of the structure.

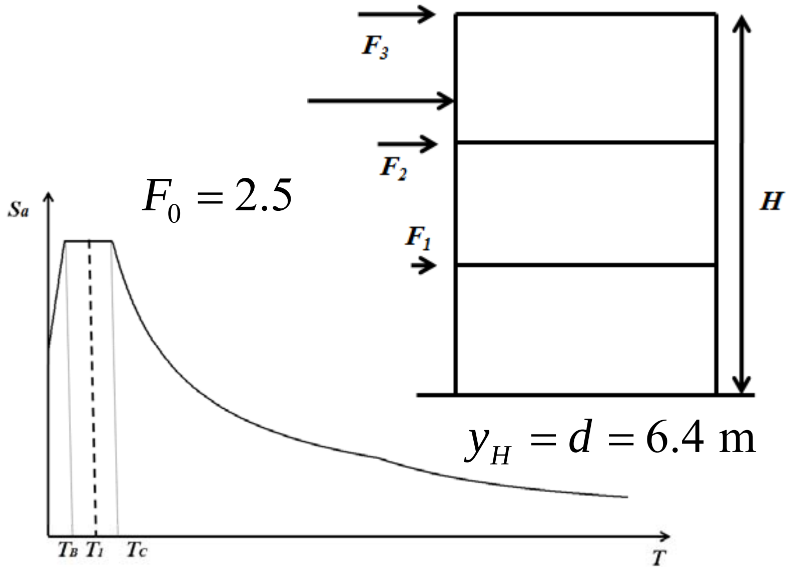
Hypotheses



Seismic loads due to Sa = 1g

$$T_{\text{Tot, base}} = T_{\text{Ed}} = m_{\text{struttura}} \cdot 1g = 66 \cdot 1 = 66 \text{ t}$$

$$M_{\text{Tot, base}} = M_{\text{Ed}} = T_{\text{Tot, base}} \cdot y_H = 66 \cdot 6.4 = 420 \text{ t m}$$



$$F_0 = 2.5$$

$$y_H = d = 6.4 \text{ m}$$

In the hypothesis of:

- Linear-elastic behavior
- Plane sections after the deformation
- Orthigonal walls are perfectly connected

$$\rho_{//} = \frac{J_{//}}{J_{Tot}} = \frac{2.22}{7.22} = 0.31 \quad \rightarrow \quad 30\%$$

$$\rho_{\perp} = \frac{J_{\perp}}{J_{Tot}} = \frac{4.99}{7.22} = 0.69 \quad \rightarrow \quad 70\%$$

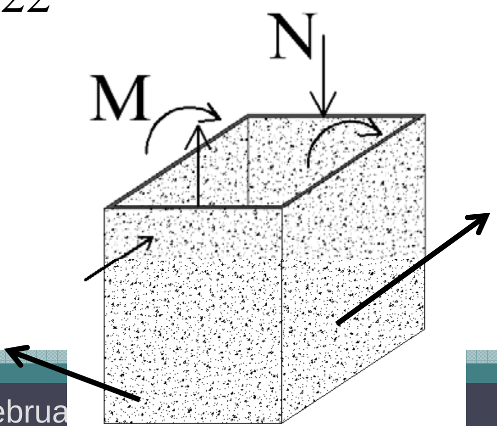
$$M_{Ed,\,//} = 0.30 \cdot M_{\text{Ed}} = 0.30 \cdot 420 = 130 \text{ t m}$$

$$M_{Ed,\,\perp} = 0.70 \cdot M_{\text{Ed}} = 0.70 \cdot 420 = 291 \text{ t m}$$

$$M_{\text{Ed, parete }//} = \frac{0.30 \cdot M_{\text{Ed}}}{2} = \frac{0.30 \cdot 420}{2} = 65 \text{ t m}$$

$$N_{\text{Ed, sismico, parete }\perp} = \frac{M_{Ed,\,\perp}}{\ell_{//}} = \frac{291}{5.52} = 53 \text{ t}$$

$$T_{\text{Ed, parete }//} = \frac{T_{\text{Ed}}}{2} = \frac{66}{2} = 33 \text{ t}$$



Parallel walls

Perpendicular walls

Strength

Parallel wall – Strength of first yielding for bending in the floor

$$M_{y1} = \left(\frac{\rho b y_{y1}}{2} f_{ym} \right) \cdot \left(\frac{h}{2} - \frac{y_{y1}}{3} \right) + \left(\frac{b\left(h - y_{y1}\right)^2}{2 y_{y1}} \frac{f_{ym}}{n} \right) \cdot \left(\frac{h}{6} + \frac{y_{y1}}{3} \right) + A_{s,catena} f_{ym} \left(h - 2c \right) = 149 \text{ t m}$$

Parallel wall – Ultimate strength for bending in the floor

$$M_{Rd,\text{ parete}//} = \left(f_{ym} \cdot \rho \cdot b \cdot y_{u,sb} \right) \cdot \left(\frac{h}{2} - \frac{y_{u,sb}}{2} \right) + \left(f_{cm} \cdot b \cdot 0{,}8 \left(h - y_{u,sb} \right) \right) \cdot \left(0{,}1h + 0{,}4 y_{u,sb} \right) + A_{s,catena} f_{ym} \left(h - 2c \right) = 181 \text{ t m}$$

Parallel wall – Shear strength in the floor

$$T_{Rd} = \min\left(T_{Rcd}, T_{Scd} \right) = 60 \text{ t} \qquad T_{Rsd} = 0{,}9 \cdot d \cdot \frac{A_{sw}}{s} \cdot f_{ym} \cdot \left(\cot\theta + \cot\alpha \right) \cdot \sin\alpha \qquad T_{Rcd} = 0{,}9 \cdot d \cdot b \cdot \alpha_c \cdot f'_{cm} \cdot \frac{\left(\cot\theta + \cot\alpha \right)}{\left(1 + \cot^2\theta \right)}$$

Perpendicular wall – Tensile strength

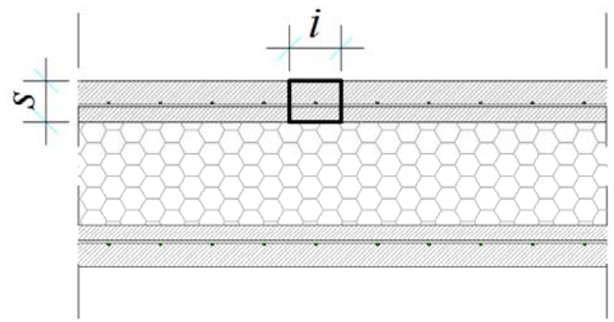
$$N_{Rd,\text{ parete}\perp} = \sigma_{max} \cdot b \cdot \ell_{\perp} = 6 \frac{\text{kg}}{\text{cm}^2} \cdot 8 \text{ cm} \cdot 412 \text{ cm} \cong 20 \text{ t}$$

σ_{max} = 6 kg/cm^2

Maximum admissible strength (in traction) for “steel-concrete” material considering a diffuse reinforcement of 1+1ϕ2.5/10cm

Parallel wall – Shear strength at the base

$$S_{Rd,\text{ parete}//} = \mu \cdot N_{Ed} + A_{s,riprese\,//} \cdot \frac{f_{ym}}{\sqrt{3}} = 75 \text{ t}$$



Comparison of the actions due to 1g and corresponding strength

Perpendicular wall – Tensile strength vs. Tensile action

$$\frac{N_{\text{Rd, parete}\perp} + N_{\text{Ed, statico, parete}\perp}}{N_{\text{Ed, sismico, parete}\perp}} = \frac{20+12}{53} = 0.61$$

Parallel wall – First yielding strength for bending in the floor vs. bending action in the floor

$$\frac{M_{Rd\text{, parete //}}}{M_{Ed\text{, parete //}}} = \frac{149}{65} = 2.30$$

Parallel wall – Ultimate strength for bending in the floor vs. bending action in the floor

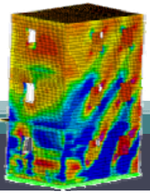
$$\frac{M_{Rd\text{, parete //}}}{M_{Ed\text{, parete //}}} = \frac{181}{65} = 2.79$$

Parallel wall – Shear strength in the floor vs. shear action in the floor

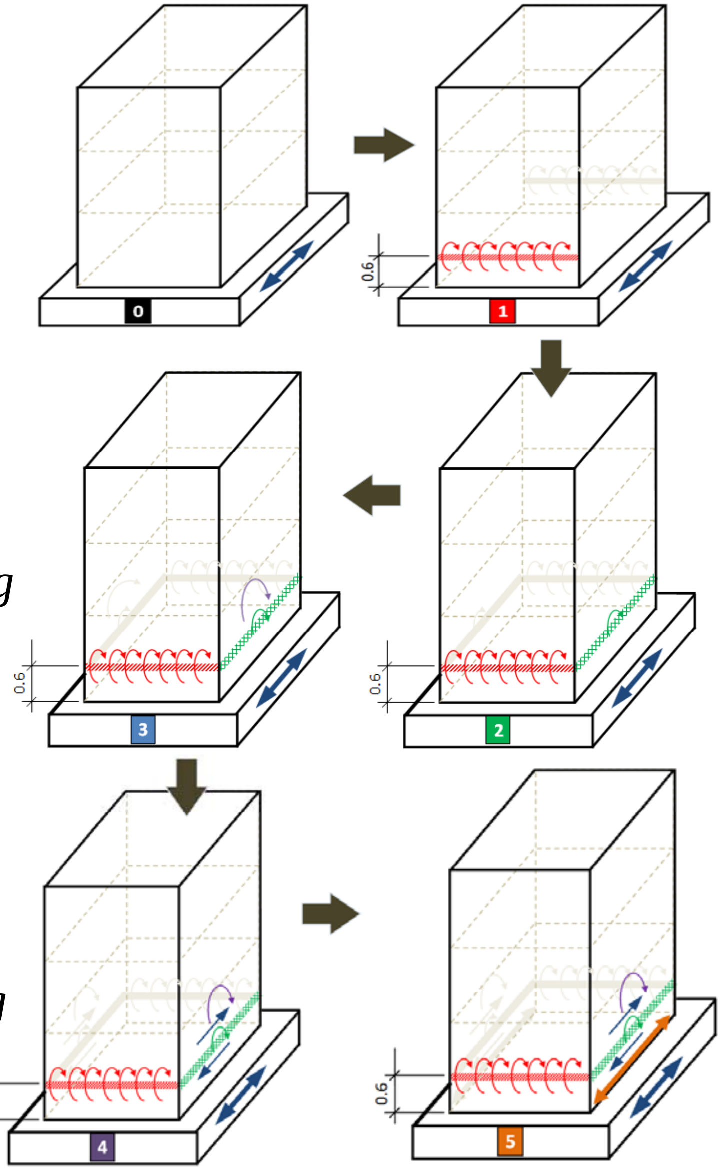
$$\text{Se } \theta = 22\,^{\circ} \rightarrow \frac{T_{Rd\text{, parete //}}}{T_{Ed\text{, parete //}}} = \frac{60\text{ t}}{33\text{ t}} = 1.82$$

Parallel wall – Shear strength at the base vs. shear action

$$\frac{T_{Rd\text{, parete //}}}{T_{Ed\text{, parete //}}} = \frac{75\text{ t}}{33\text{ t}} = 2.28$$



POSSIBLE COLLAPSE MECHANISMS



1. Tensile **yielding** of the **perpendicular** walls

$S_a = 0.61g \quad PGA = 0.24g$

$FS_F = 3.2 \quad FS_M = 7.4$

2. **Yielding** in the plane of the **parallel** walls

$S_a = 1.13g \quad PGA = 0.45g$

$FS_F = 1.8 \quad FS_M = 1.4$

3. Ultimate **bending** conditions (in the plane) of the **parallel** walls

$S_a = 1.28g \quad PGA = 0.51g$

$FS_F = 1.5 \quad FS_M = 1.1$

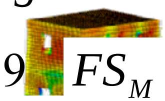
4. **Shear** collapse (in the plane) of the **parallel** walls

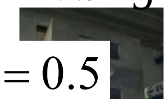
$S_a = 1.82g \quad PGA = 0.73g$

$FS_F = 1.1 \quad FS_M = 0.7$

5. Base **displacement** of the **parallel** walls

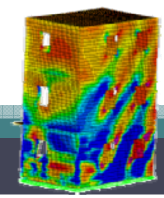
$S_a = 2.28g \quad PGA = 0.91g$





$FS_F = 0.9 \quad FS_M = 0.5$

CONSTRUCTION PHASE



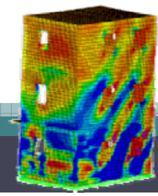
CONSTRUCTION PHASES












ACTUAL STRENGTHS OF THE MATERIALS USED FOR THE CONSTRUCTION OF THE BUILDING

Zinc-plated steel

Tabella 1: parametri caratteristici tratti dai test di trazione dei fili di diametro 2.5 mm

Campione	ϕ_{nom} mm	A_0 mm²	F_y N	f_y Mpa	F_t N	f_t Mpa	l_0 mm	l mm	A_{gt} %	f_t/f_y
Trafilcoop -1.1	2.5	4.91	2182.50	444.5	2746.2	542.3	100	118.5	18.5	1.22
Trafilcoop -1.2	2.5	4.91	2218.34	451.8	2662.2	542.2	100	118.0	18	1.20
Trafilcoop -1.3	2.5	4.91	2249.76	458.2	2722.10	554.4	100	118.5	18.5	1.21
Trafilcoop -1 medie			**2216.87**	**451.5**	**2710.16**	**546.3**			**18.3**	**1.21**
Trafilcoop -2.1	2.5	4.91	2186.17	445.3	2778.1	565.9	100	119.5	19.5	1.27
Trafilcoop -2.2	2.5	4.91	2288.06	466.0	2791.33	568.5	100	118.5	18.5	1.22
Trafilcoop -2.3	2.5	4.91	2247.80	457.8	2764.82	563.1	100	120.0		
Trafilcoop -2 medie			**2240.67**	**456.4**	**2778.08**	**565.8**				
Trafilcoop -3.1	2.5	4.91	2168.89	441.7	2758.25	561.9	100	120.0		
Trafilcoop -3.2	2.5	4.91	2215.56	451.35	2661.7	542.2	100	118.0		
Trafilcoop -3.3	2.5	4.91	2310.84	470.76	2701.7	550.4	100	118.5		
Trafilcoop -3 medie			**2281.83**	**464.85**	**2685.0**	**547.0**				

Test: 550 MPa

Desingned: 500 MPa

(UNI EN ISO 15630-1)

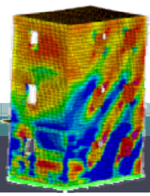
N.	Diametro nominale (mm)	Lunghezza barra (mm)	Diametro barra equipesante (mm)	Sezione (mm²)	Tolleranza sezione (%)	Massa per unità di lungh. (kg/m)	TRAZIONE Snervamento f_y (N/mm²)	TRAZIONE Rottura f_t (N/mm²)	TRAZIONE f_t/f_y	ALLUNG. A_{gt} (%)	Piegamento e Raddrizz.	Marchio Identificazione e Scheda
1	8	500	8,04	50,75	1,01	0,398	561	696	1,24	11,7	SF	a: Stefana 042/08-CA
2	8	500	8,05	50,90	1,31	0,400	559	694	1,24	11,3	SF	a: Stefana 042/08-CA
3	8	500	8,05	50,85	1,21	0,399	540	695	1,29	12,3	SF	a: Stefana 042/08-CA
4	10	500	9,98	78,11	-0,49	0,613	540	653	1,21	6,8	SF	a: Feralpi Sid. 022/10-CA
5	10	500	9,95	77,73	-0,99	0,610	543	644	1,19	6,9	SF	a: Feralpi Sid. 022/10-CA
6	10	500	9,98	78,14	-0,46	0,613	540	640	1,19	7,1	SF	a: Feralpi Sid. 022/10-CA
7	12	500	11,99	112,84	-0,18	0,886	548	661	1,21	8,3	SF	a: Feralpi Sid. 022/10-CA
8	12	500	12,02	113,32	0,25	0,890	554	641	1,16	8,5	SF	a: Feralpi Sid. 022/10-CA
9	12	500	12,00	112,99	-0,04	0,887	564	660	1,17	7,8	SF	a: Feralpi Sid. 022/10-CA
10	14	500	14,01	154,17	0,20	1,210	522	617	1,18	10,4	SF	a: Feralpi Sid. 022/10-CA
11	14	500	14,00	153,91	0,03	1,208	510	606	1,19	8,9	SF	a: Feralpi Sid. 022/10-CA
12	14	500	14,02	154,19	0,21	1,210	528	617	1,17	9,3	SF	a: Feralpi Sid. 022/10-CA
13	16	500	16,00	200,99	0,02	1,578	547	620	1,13	12,0	SF	a: Alfa Acciai 007/08-CA
14	16	500	16,00	201,04	0,04	1,578	551	625	1,13	11,1	SF	a: Alfa Acciai 007/08-CA
15	16	500	16,00	200,99	0,02	1,578	542	615	1,14	12,6	SF	a: Alfa Acciai 007/08-CA

Concrete cubic specimens

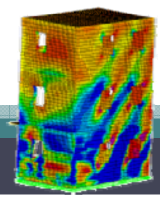
N°	Contrassegno del provino	Dimensioni (mm) Lung.	Larg.	Altez.	Massa provino kg	Massa volumica kg/m³	Car mass... kN	N/mm²	... rottura
1	intonaco p.terra 06/05/11	150	150	150	7,150	2.119	638	28,34	s
2	intonaco p.terra 06/05/11	150	150	150	7,200	2.133	618	27,47	s
3	getto solaio 11/05/11	150	150	150	7,890	2.338	687	30,52	s
4	getto solaio 11/05/11	150	150	150	7,820	2.317	687	30,52	s
5	tavola vib. 1° passata 13/05/11	150	150	150	7,410	2.196	755	33,57	s
6	tavola vib. 2° passata 14/05/11	150	150	150	7,290	2.160	667	29,65	s
7	tavola vib. 1° passata 28/05/11	150	150	150	7,300	2.163	677	30,08	s
8	tavola vib. 1° passata 29/05/11	150	150	150	7,340	2.175	736	32,70	s
9	intonaco 2° mano P.T. 09/06/11	150	150	150	7,160	2.121	657	29,21	s
10	intonaco 2° mano P.T. 09/06/11	150	150	150	7,230	2.142	657	29,21	s

Test: 25 MPa (cilyndrical)

Desingned: 30 MPa



TRANSPORTATION PHASE




TRANSPORTATION PHASES

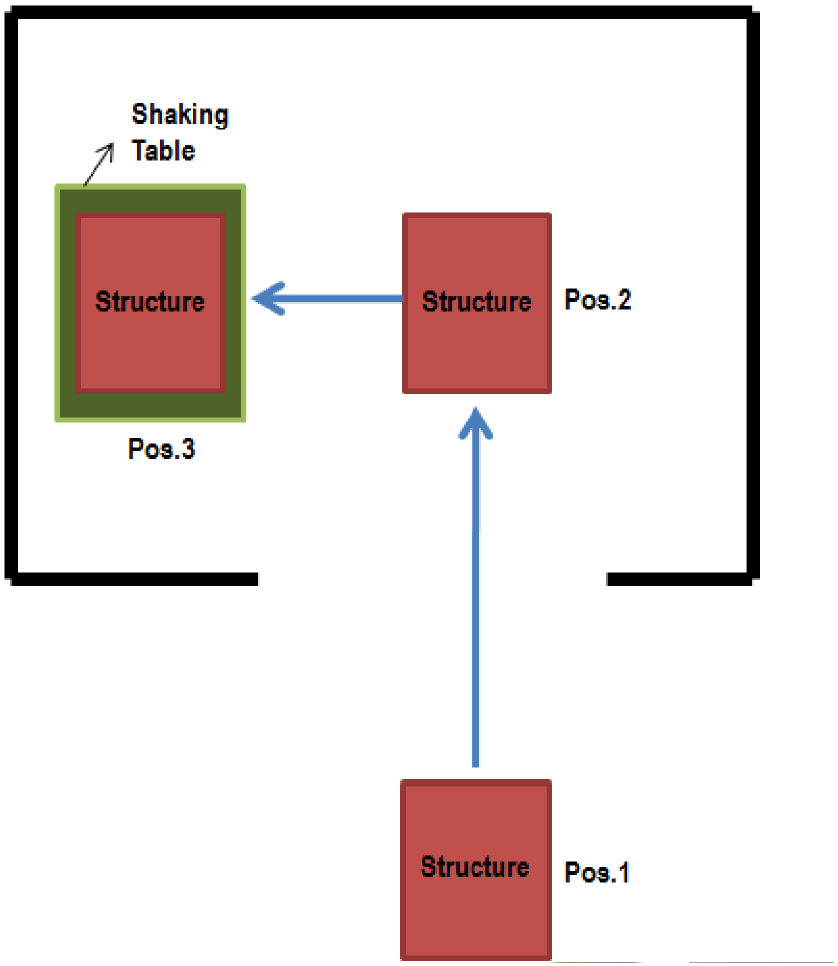
The transport of the complex structure-foundation is as in the following:

1.The complex structure-foundation is **uploaded in position 1** with four actuator;

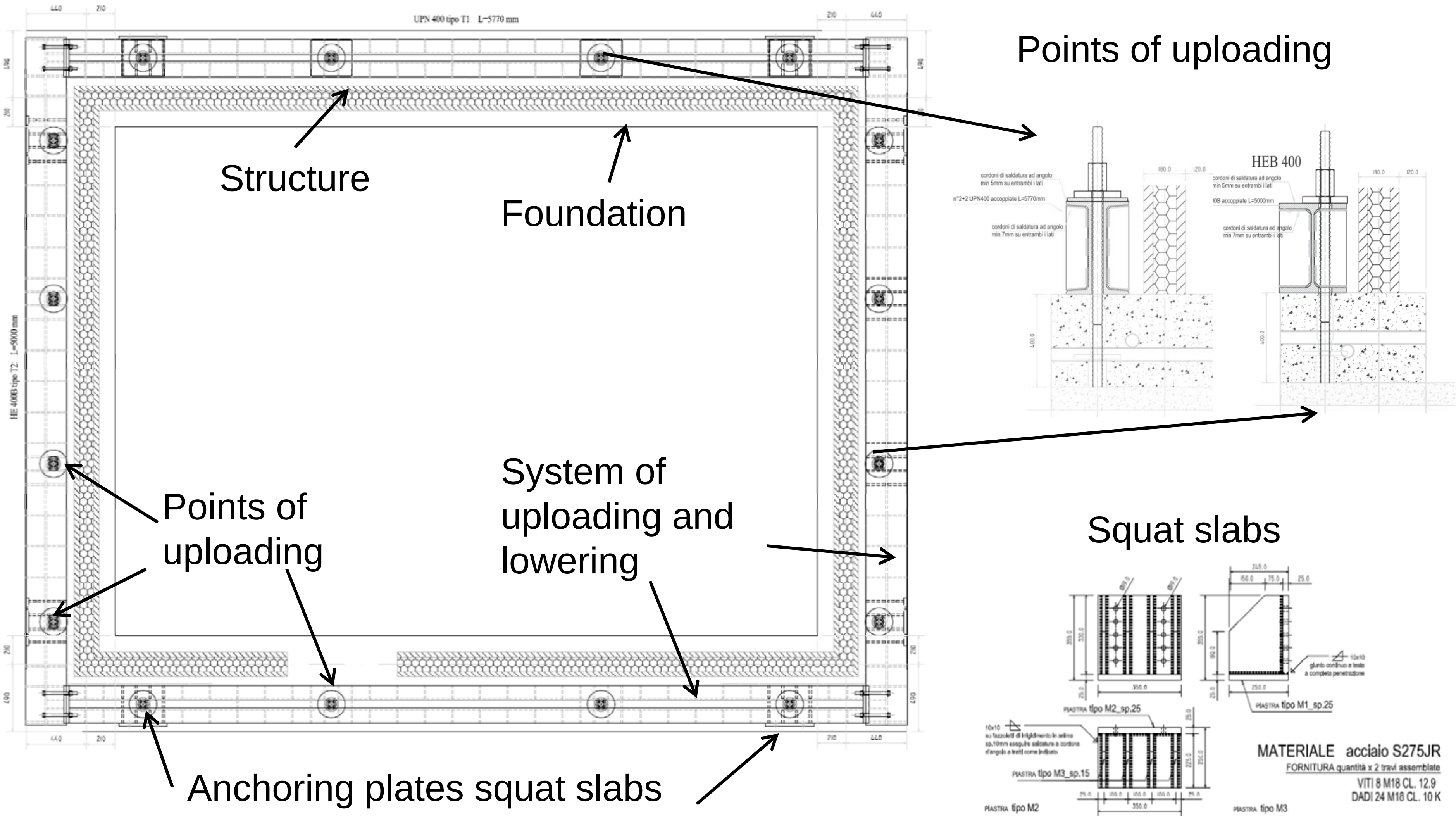
2.The complex structure-foundation is positioned on some **sliders** and pulled with chains up to **position 2**;

3.in **position 2** the complex structure-foundation is **lowered** and then **re-uploaded**;

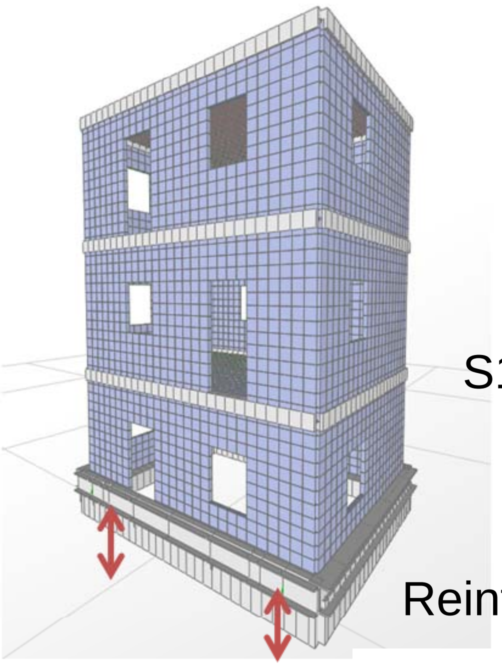
4.The complex structure-foundation is positioned on some **sliders** and pulled with chains up to the **shaking table** (position 3).



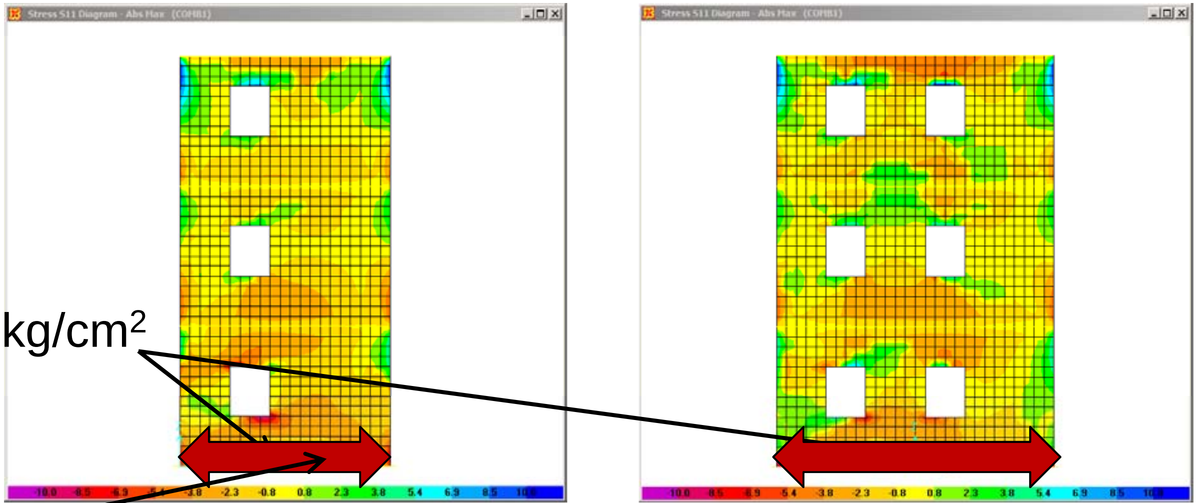
The uploading and lowering system



The uploading and lowering system



Horizontal stresses (S11)

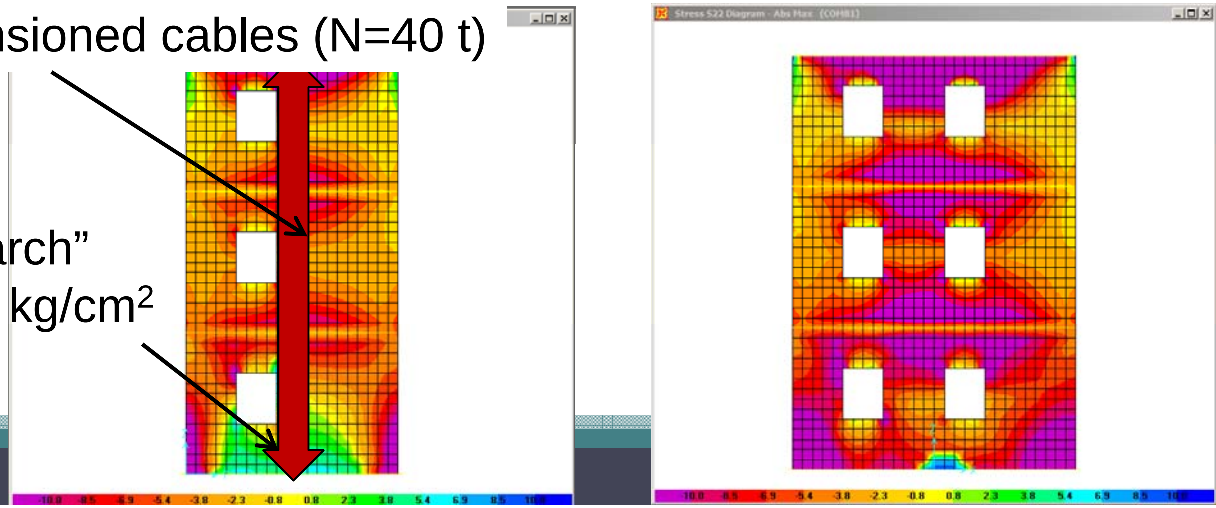


S11 = 3 kg/cm^2

Reinforcement of the base

Vertical stresses (S22)

1+1 post-tensioned cables (N=40 t)

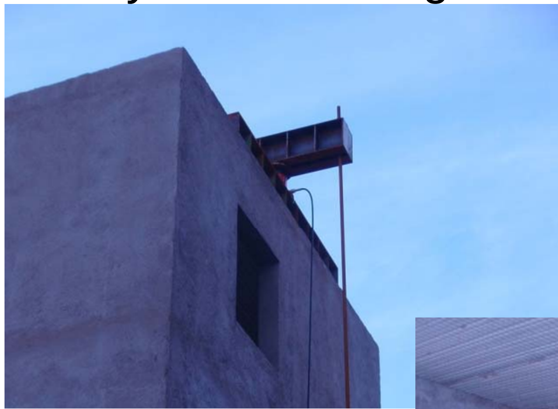


σ_{max} = **6 kg/cm^2**
Maximum admissible strenght (in tension) for the “steel-concrete” material”

“Effect arch”
S22 = 5 kg/cm^2

The uploading and lowering system

System for fixing the cables

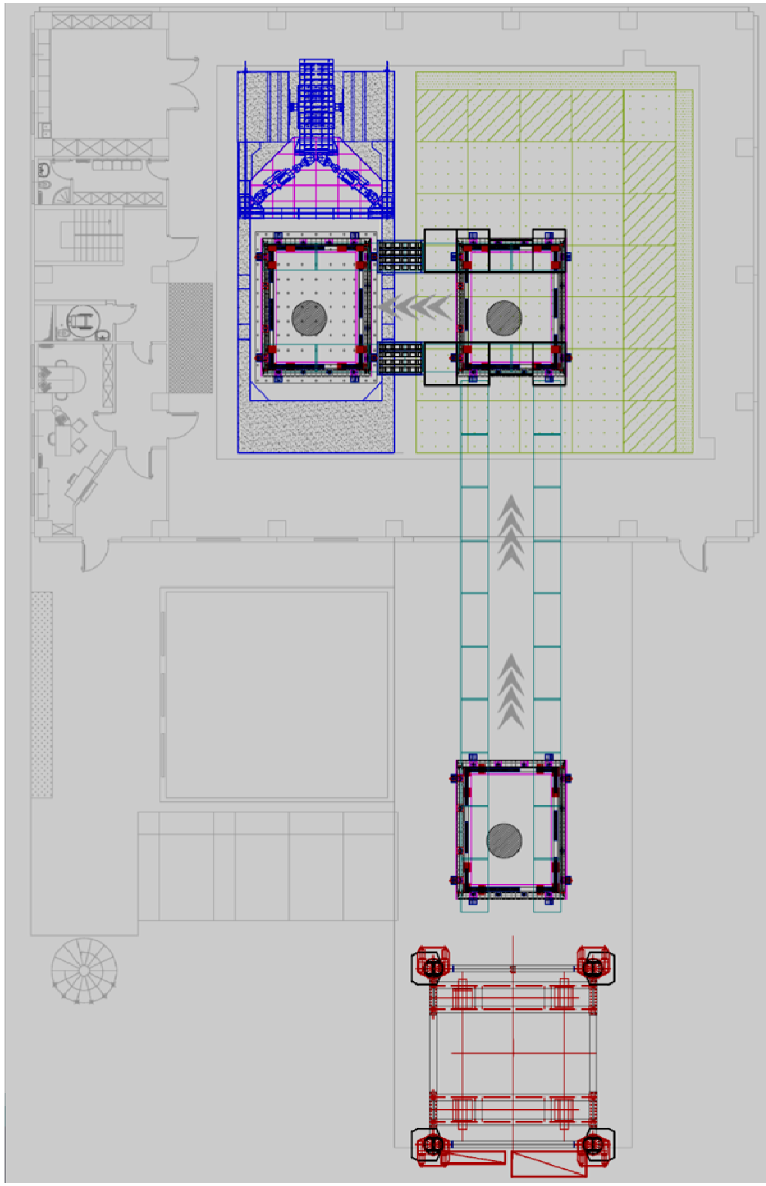


Cables to avoid the arch effect

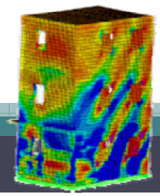
Reinforcement at the base

Hydraulic Actuator

TRANSPORTATION PHASES

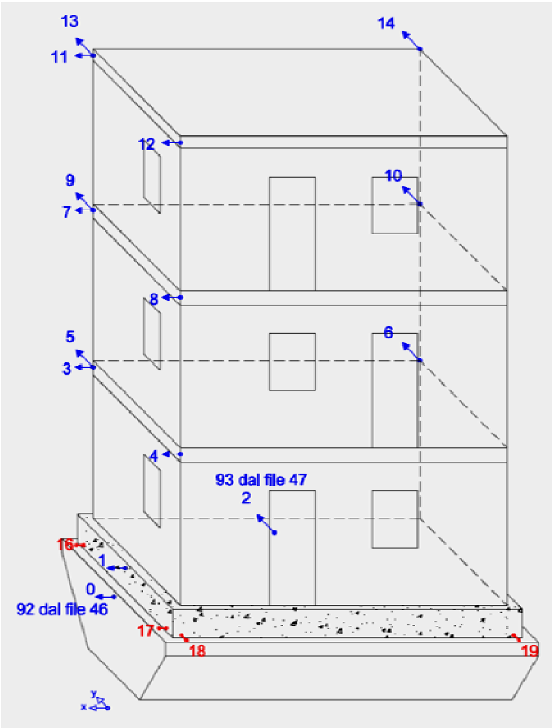


TESTING PHASE



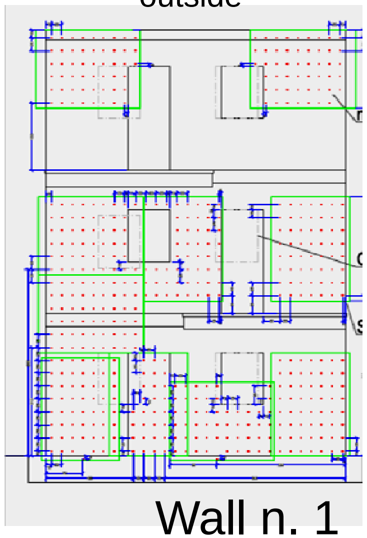
INSTRUMENTATION

Accelerometers



Wall n. 4

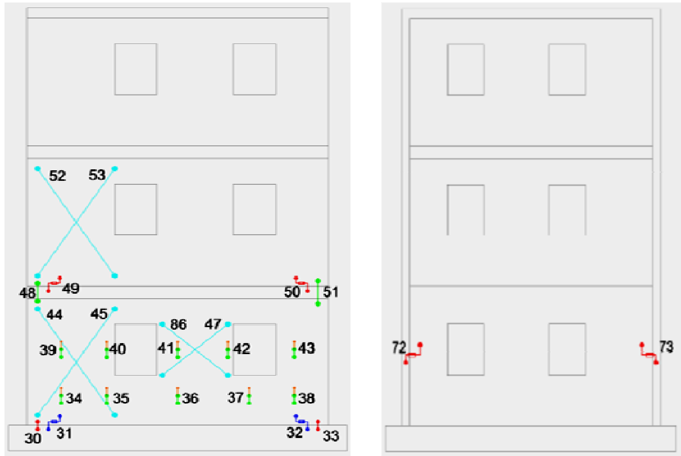
outside



Wall n. 2

outside

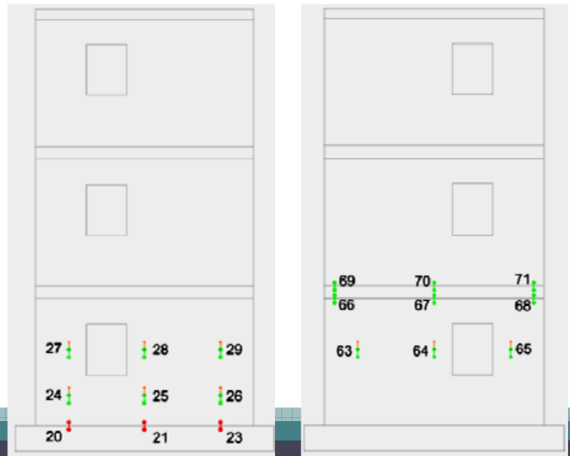
inside



Wall n. 1

outside

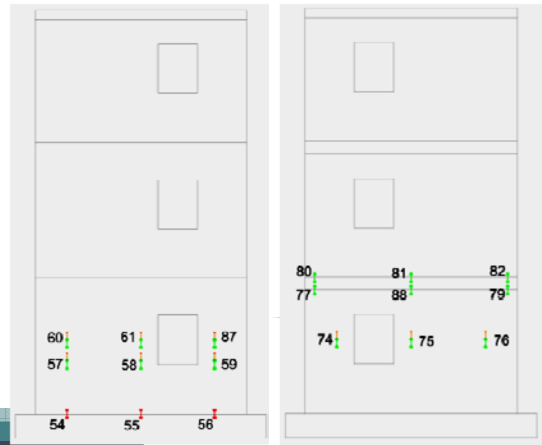
inside



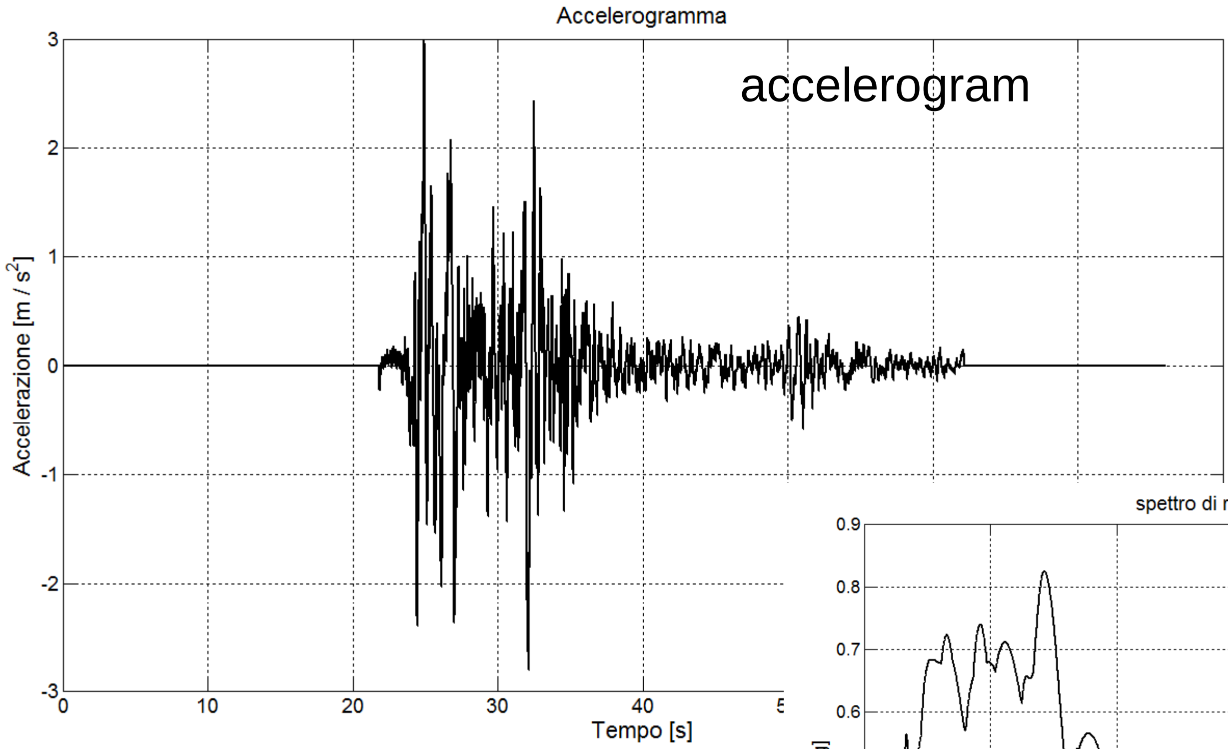
Wall n. 3

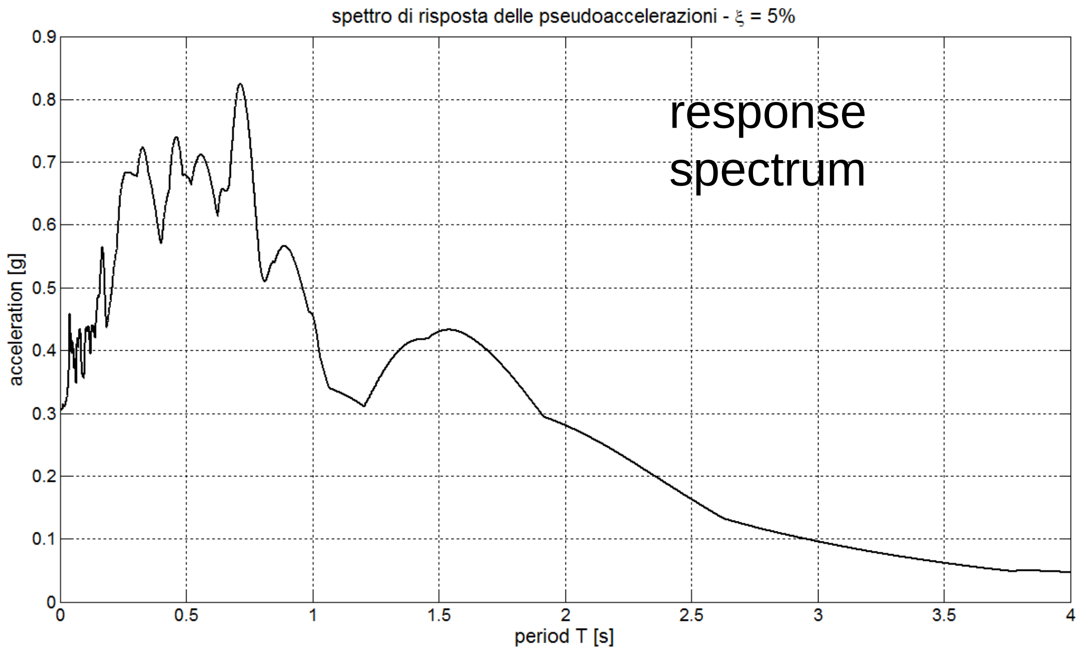
outside

inside



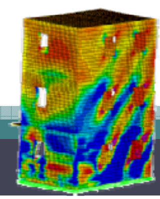
INPUT: Montenegro recorded ground motion (1979)
original PGA = 0.305g





TEST PROGRAM

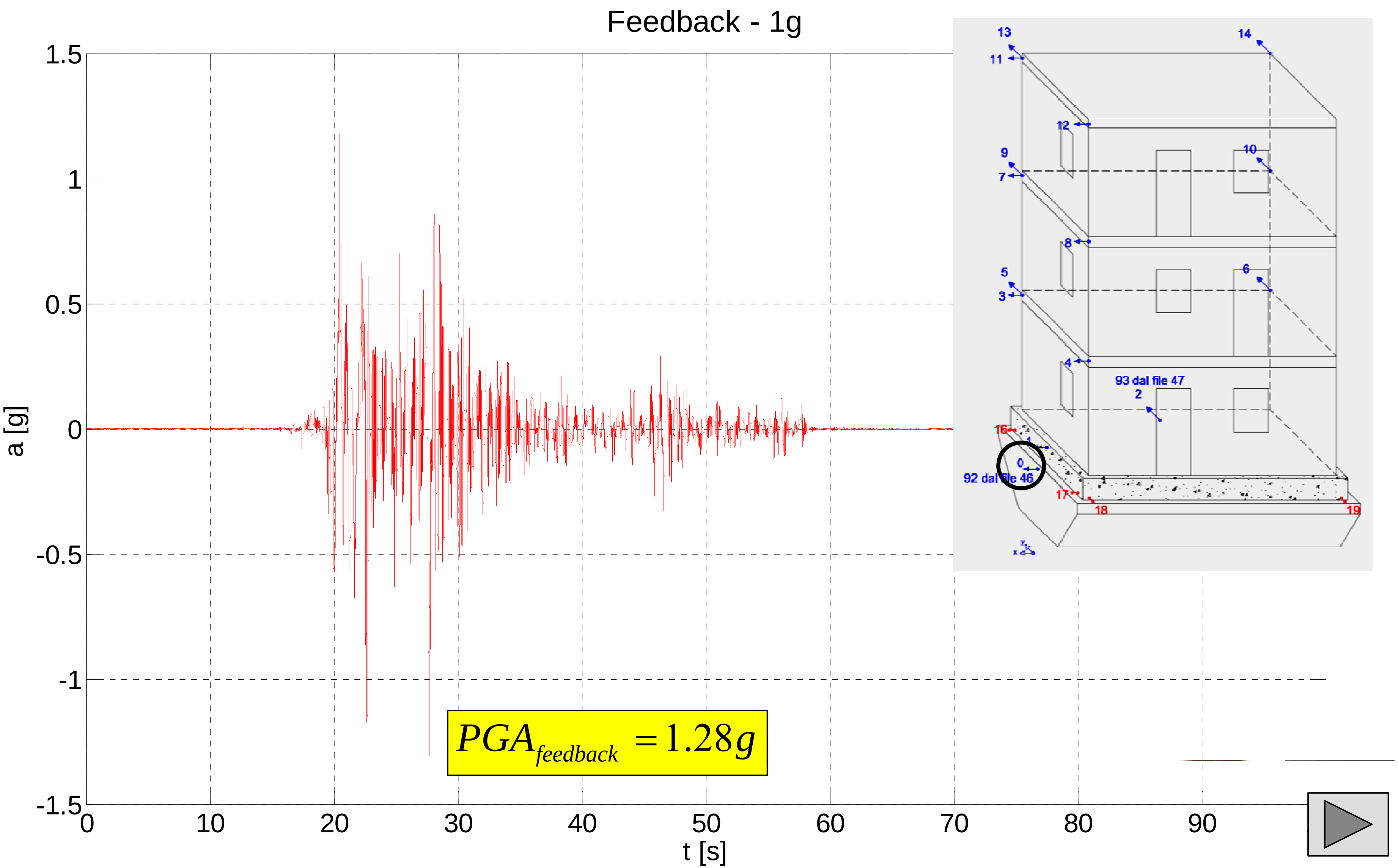
n.	Test
1	0.05 g test
2	0.15 g test
3	0.50 g test
4	1.00 g test
5	1.20 g first test
6	1.20 g second test



EXPERIMENTAL - FREQUENCIES

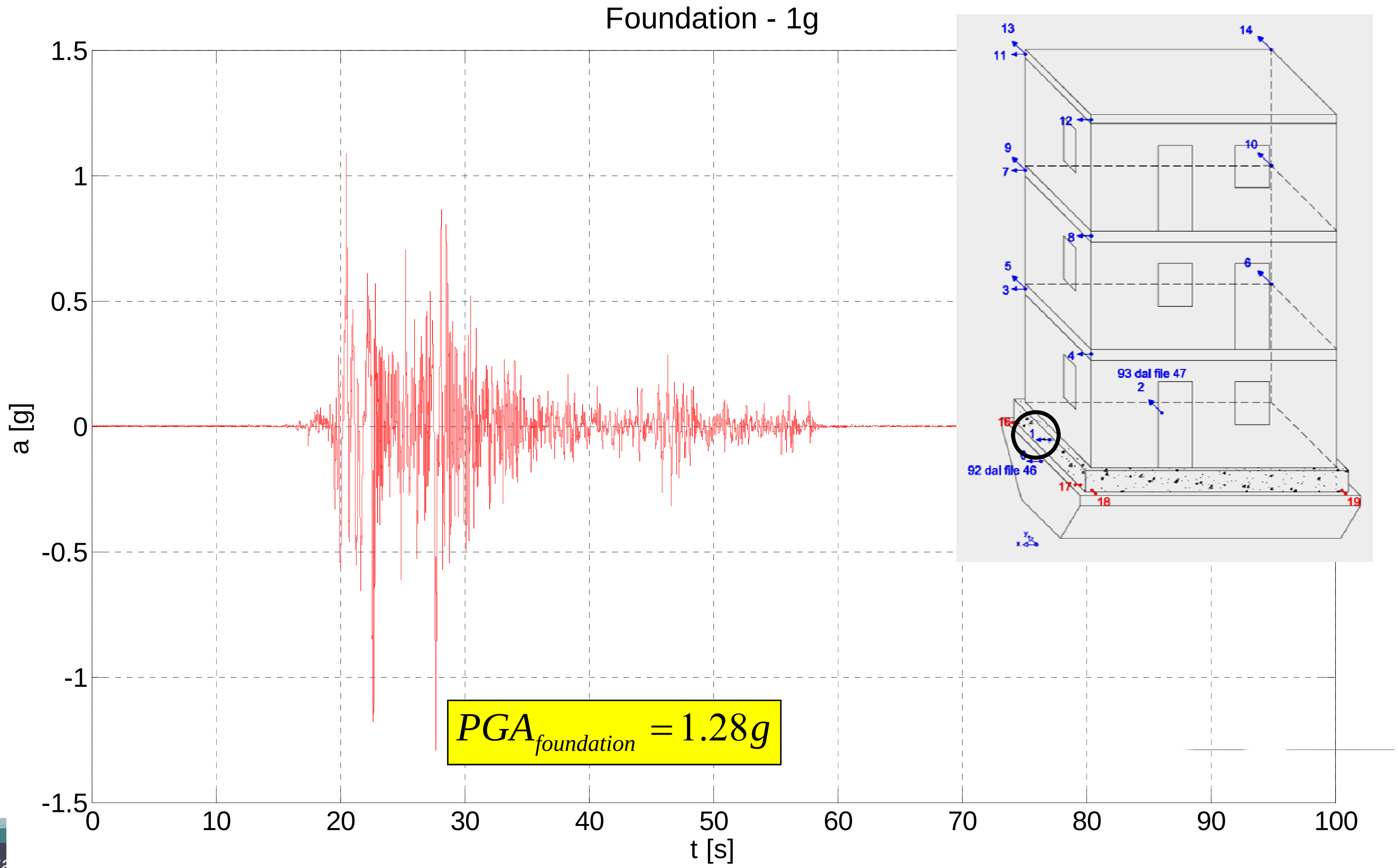
	Freq. Hz	Period . s
Before 0.05 g test	10 11.7	0.100 0.085
Between 0.05 g and 0.15 g tests	10 11.7	0.100 0.085
Between 0.15 g and 0.50 g tests	10 11.7	0.100 0.085
Between 0.50 g and 1.00 g tests	- 11	0.091
Between 1.00 g and the first 0.30 g white noises	- 10.4	0.096
Between the 0.30 g white noises and the first 1.2 g test	- 8.6	0.116
Between the first 1.20 g test and the 0.50 g white noises	-	
Between the 0.50 g white noises and the second 1.20 g test	- 8.2	0.122
After the last 1.20 g test	-	

FEEDBACK ACCELERATION AS FUNCTION OF TIME



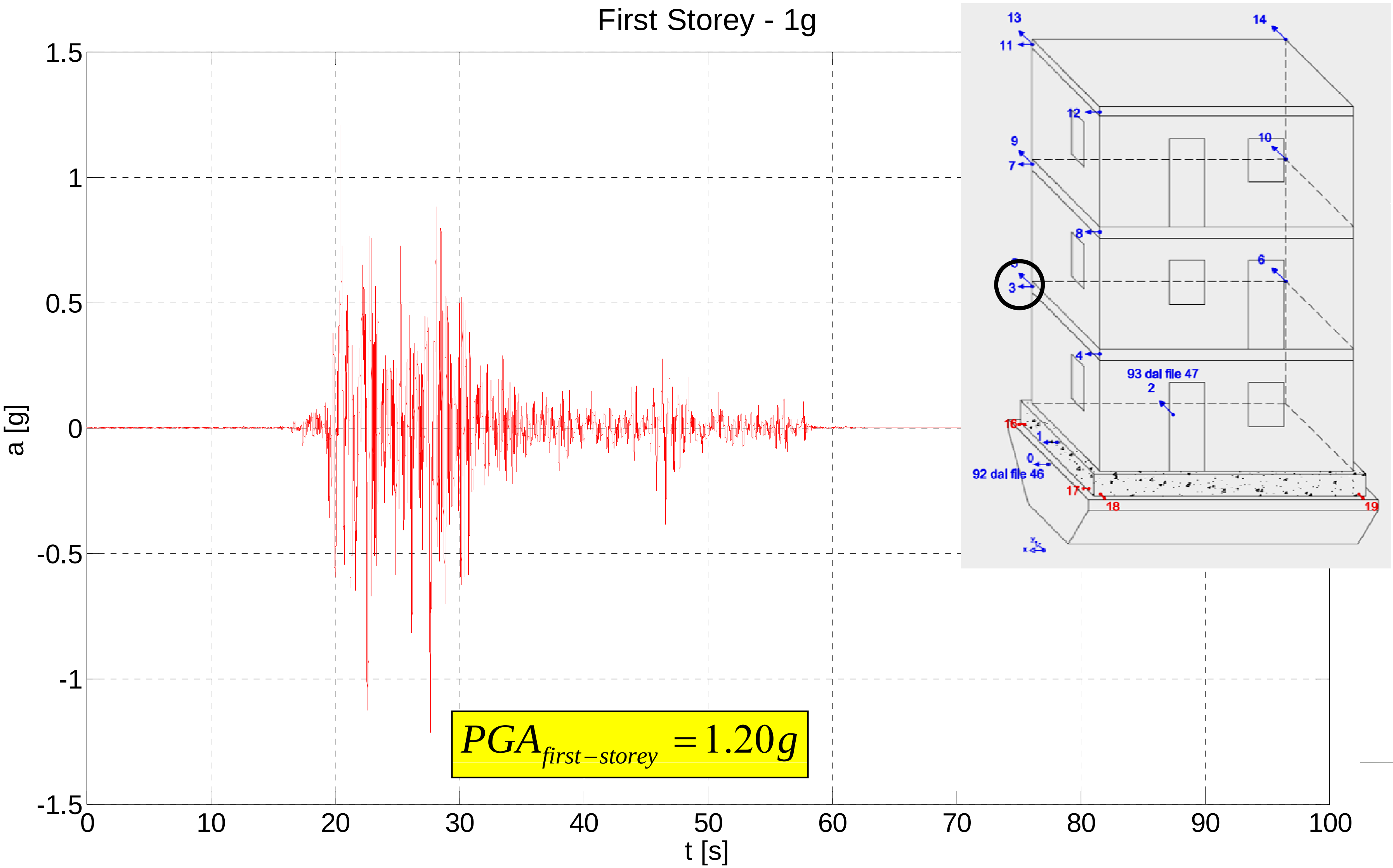
$$PGA_{feedback} = 1.28g$$

FOUNDATION ACCELERATION AS FUNCTION OF TIME

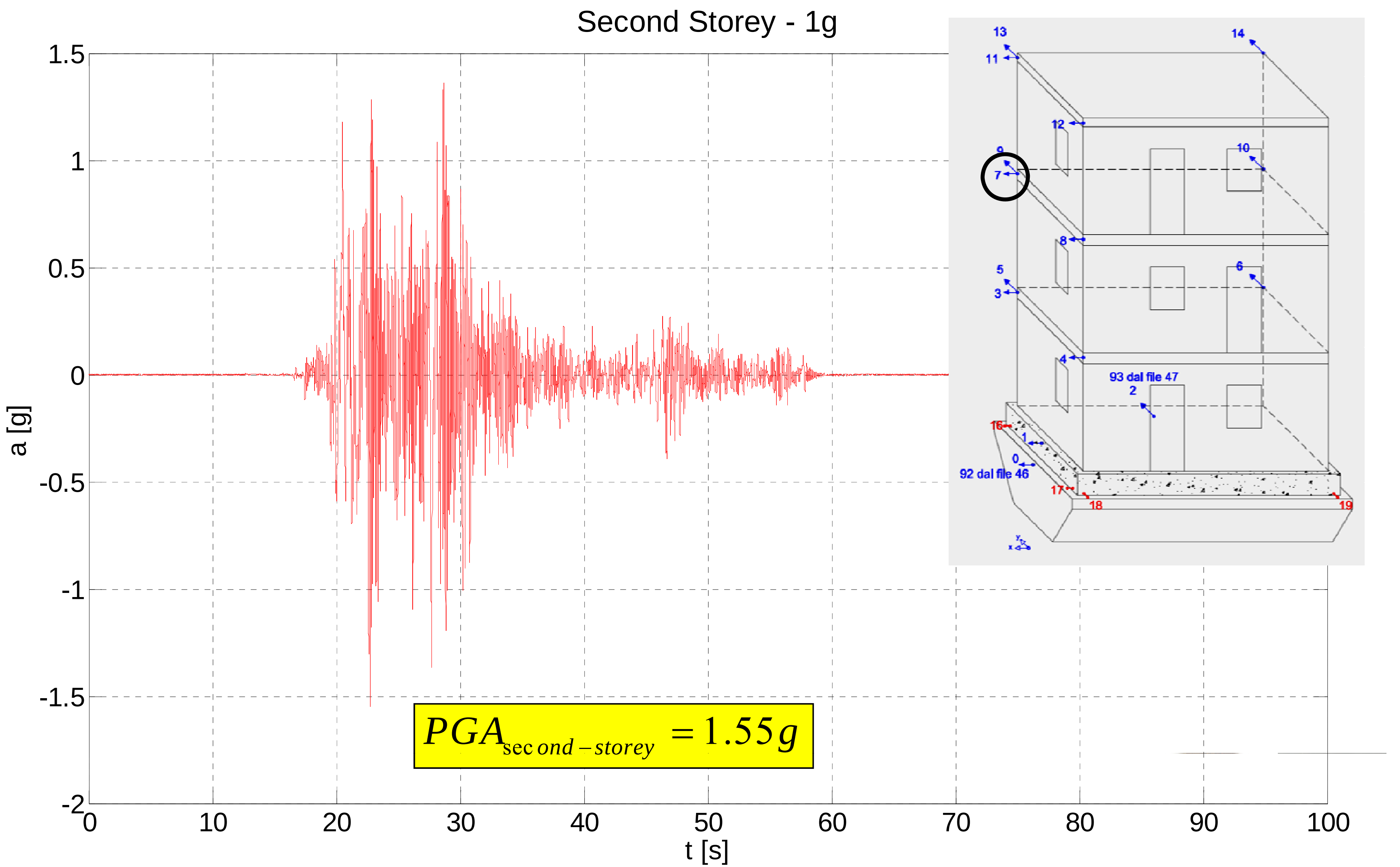


$$PGA_{foundation} = 1.28g$$

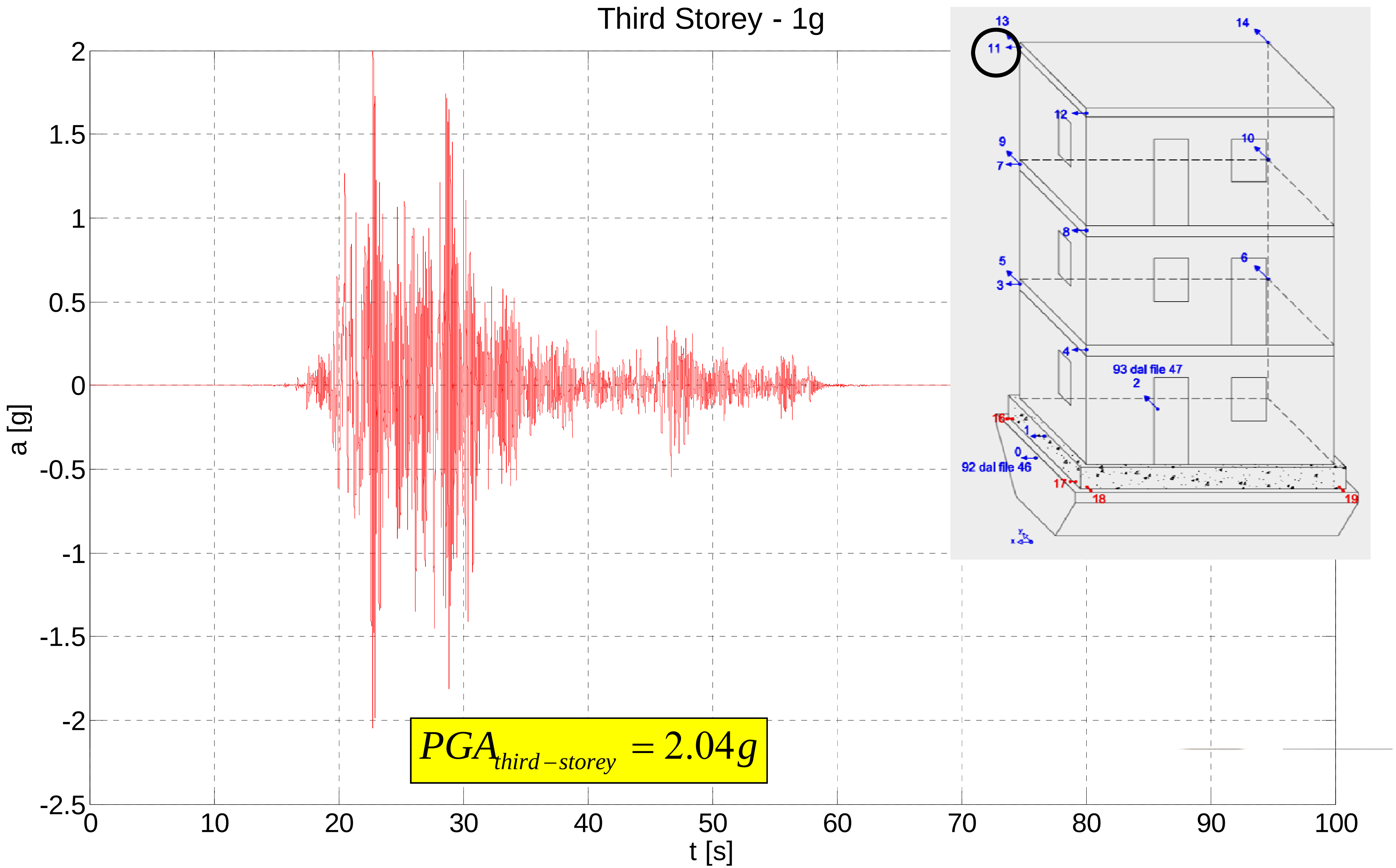
1ST STOREY ACCELERATION AS FUNCTION OF TIME



2ND STOREY ACCELERATION AS FUNCTION OF TIME

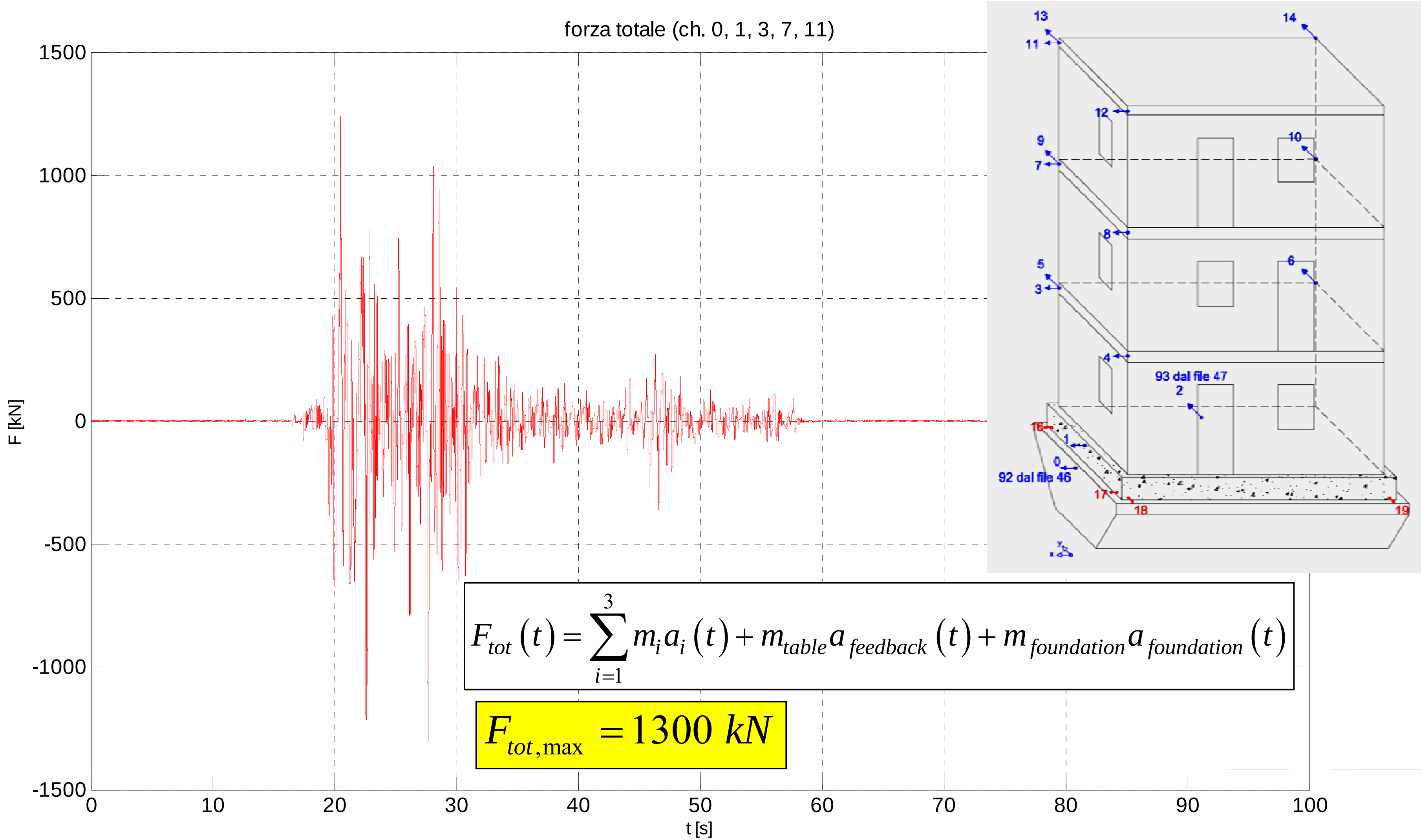


3RD STOREY ACCELERATION AS FUNCTION OF TIME



$$PGA_{third-storey} = 2.04g$$

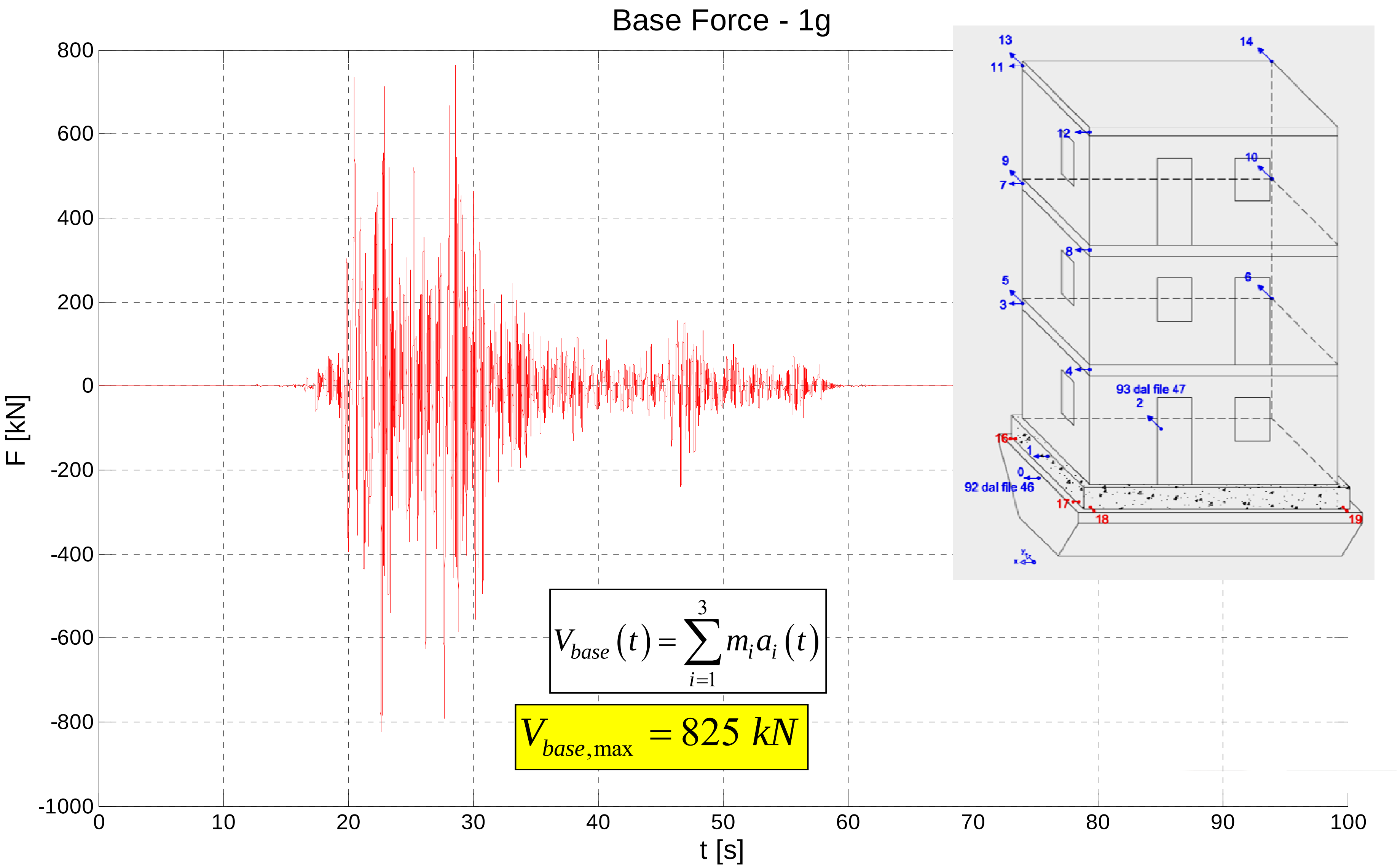
TOTAL FORCE AS FUNCTION OF TIME



$$F_{tot}(t) = \sum_{i=1}^{3} m_i a_i(t) + m_{table} a_{feedback}(t) + m_{foundation} a_{foundation}(t)$$

$$F_{tot,\max} = 1300\ kN$$

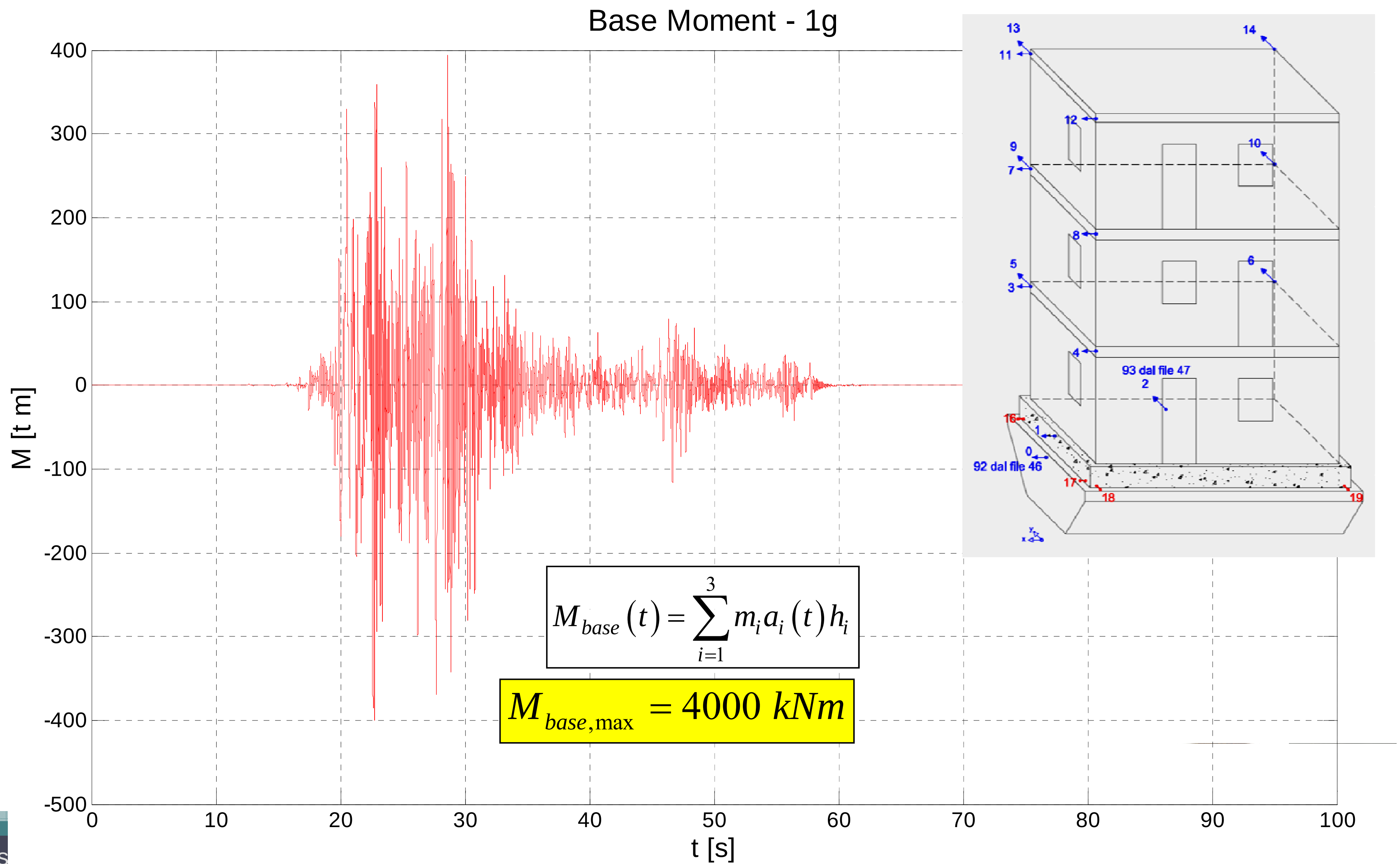
BASE SHEAR AS FUNCTION OF TIME



$$V_{base}(t) = \sum_{i=1}^{3} m_i a_i(t)$$

$$V_{base,\max} = 825\ kN$$

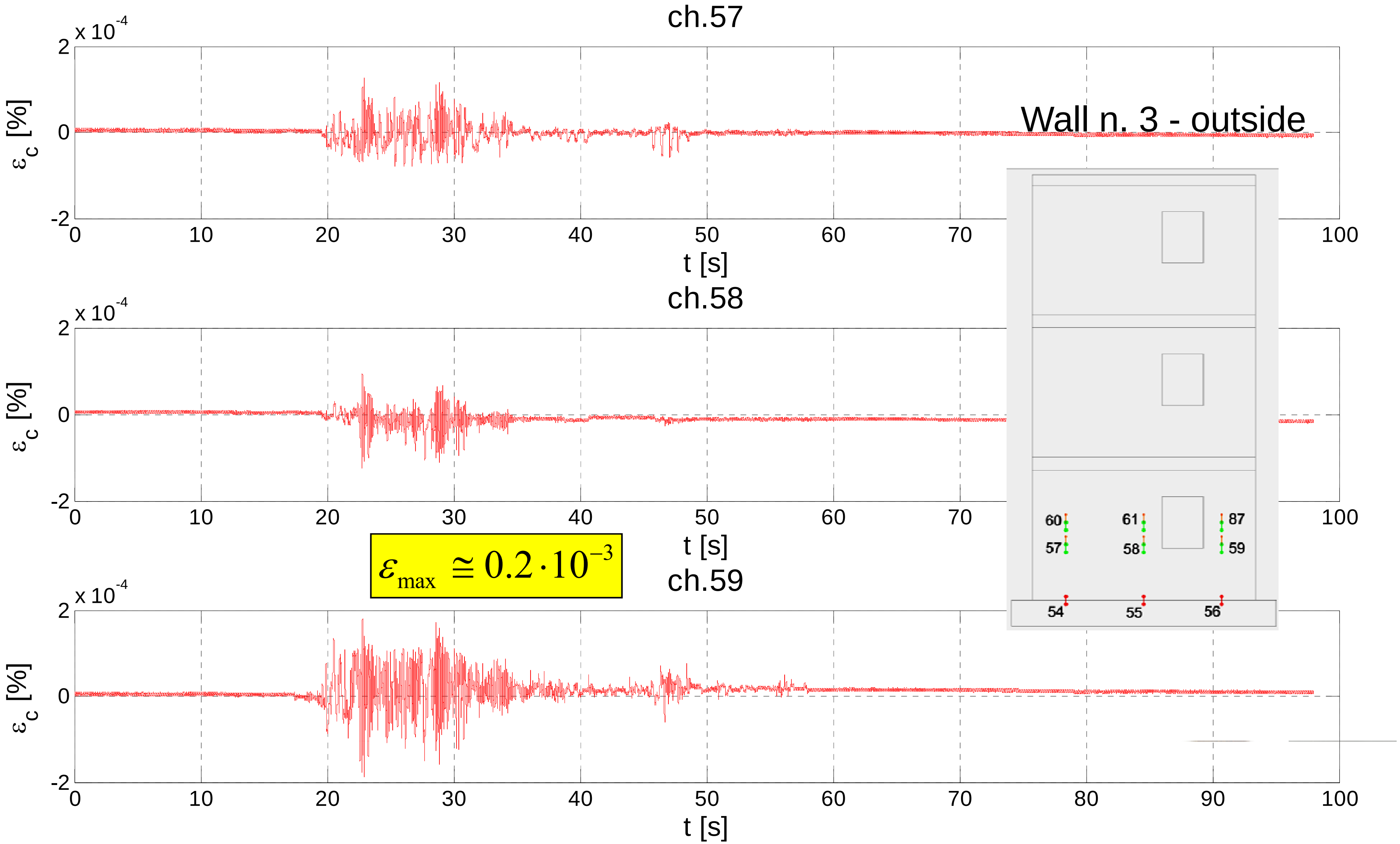
BASE MOMENT AS FUNCTION OF TIME



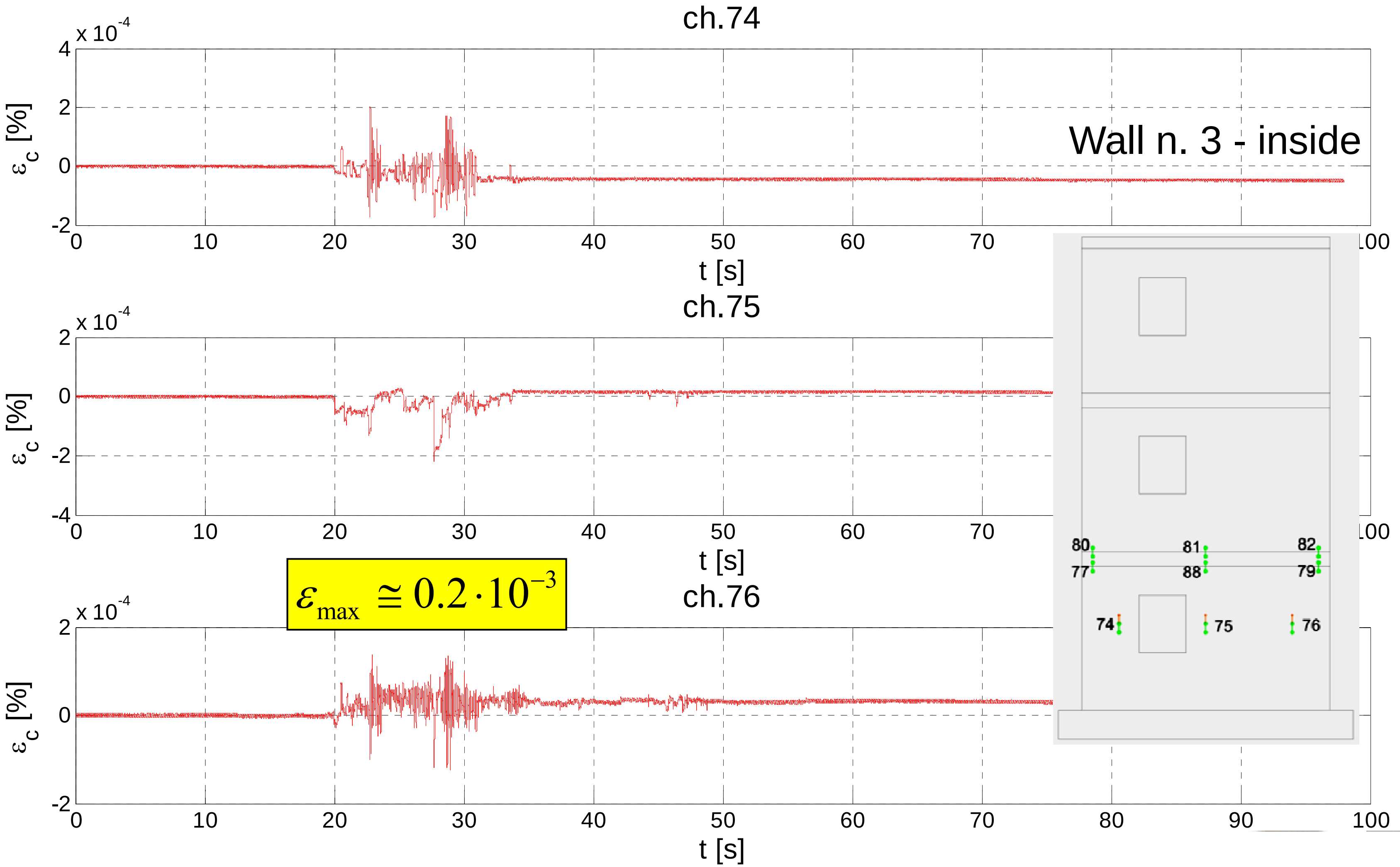
$$M_{base}(t) = \sum_{i=1}^{3} m_i a_i(t) h_i$$

$$M_{base,\max} = 4000\ kNm$$

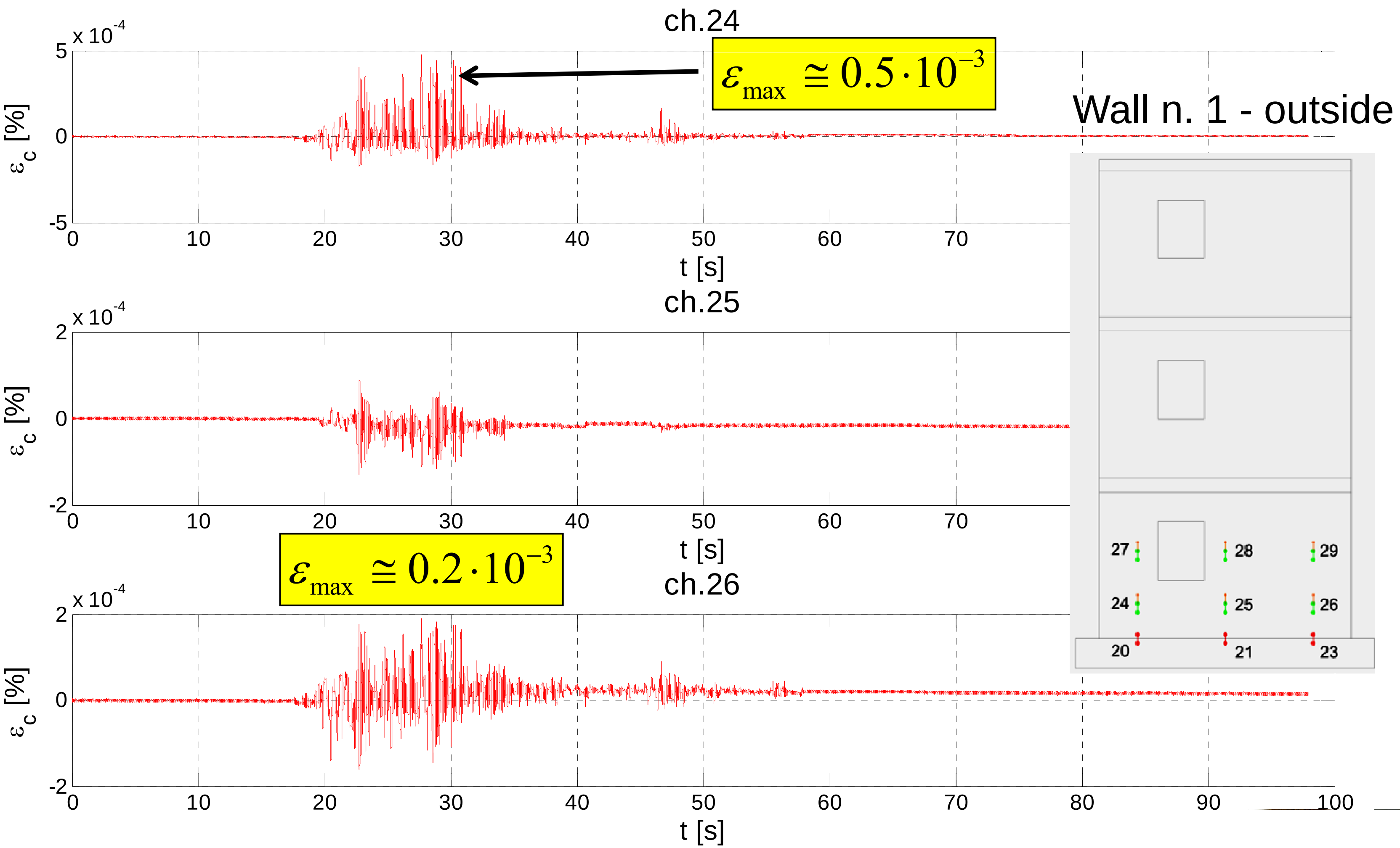
EPSILON AS FUNCTION OF TIME



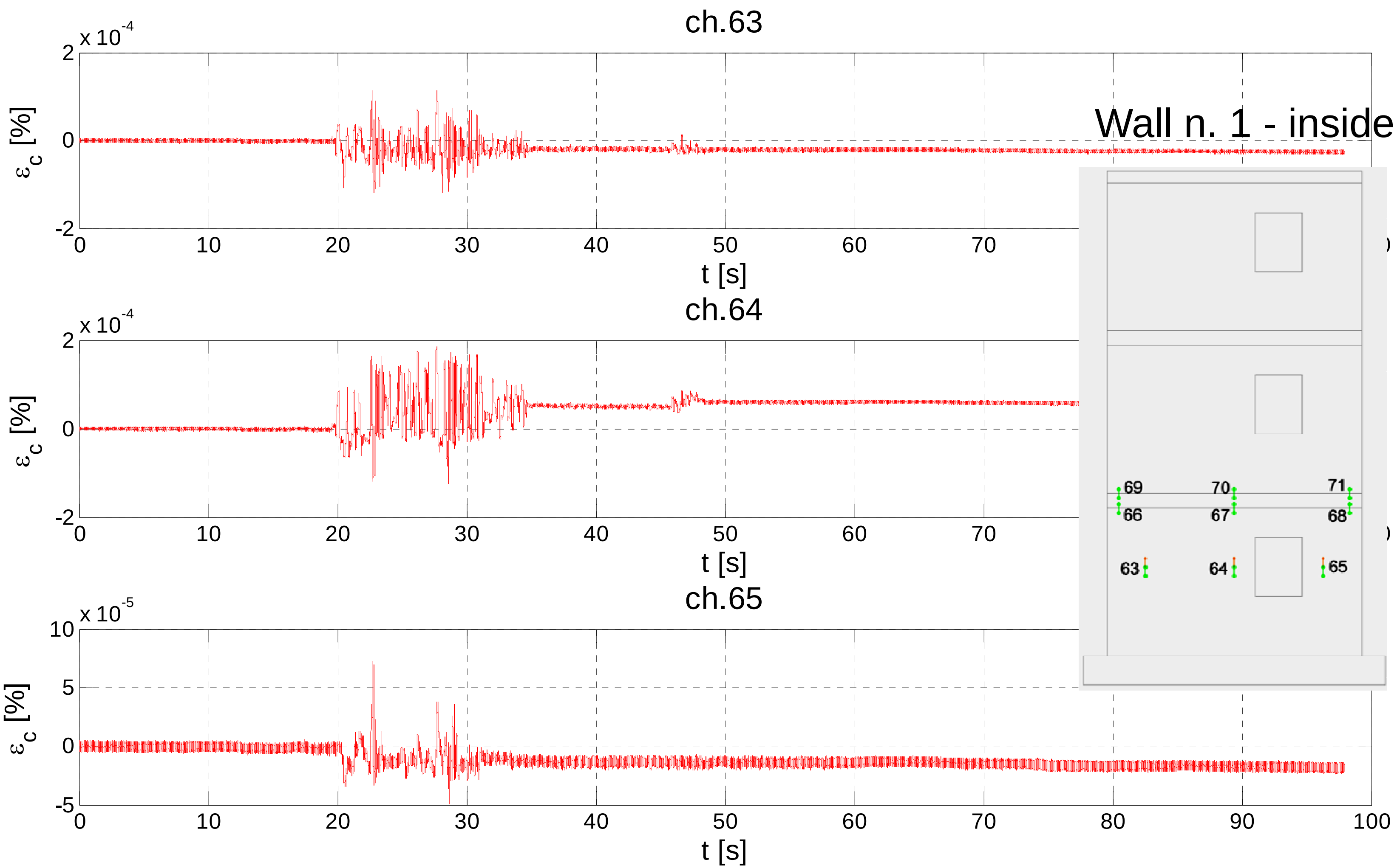
EPSILON AS FUNCTION OF TIME



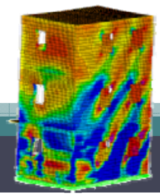
EPSILON AS FUNCTION OF TIME



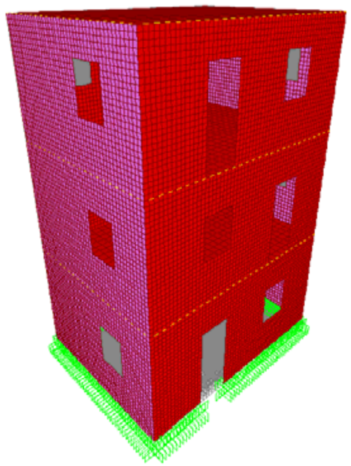
EPSILON AS FUNCTION OF TIME

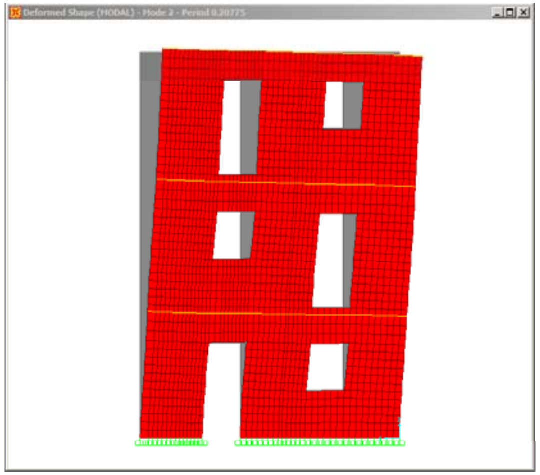


PRELIMINARY INTERPRETATION OF THE RESULTS OF THE SHAKING-TABLE TESTS



PERIODS AND FREQUENCIES





FEM analysis

Elastic Modulus [kg/cm²]	Period [s]	Frequency [Hz]
E **= 30 MPa**	0.07	14
0.5*E* = 15 MPa	0.095	10.5
0.1*E* = 3 MPa	0.21	4.8

$$f = \frac{1}{T} = \frac{1}{2\pi}\sqrt{\frac{k}{m}} = C \cdot \sqrt{E_c}$$

$$\frac{f_{\exp}}{\sqrt{E_{\exp}}} = \frac{f_{SAP}}{\sqrt{E_{SAP}}}$$

$$\frac{E_{\exp}}{E_{SAP}} = \left(\frac{f_{\exp}}{f_{SAP}}\right)^2$$

$$\frac{E_{\exp}}{0.1E_c} = \left(\frac{f_{\exp}}{f_{SAP}}\right)^2$$

$$\frac{E_{\exp}}{E_c} = 0.1\left(\frac{f_{\exp}}{f_{SAP}}\right)^2$$

PERIOD - FREQUENCIES

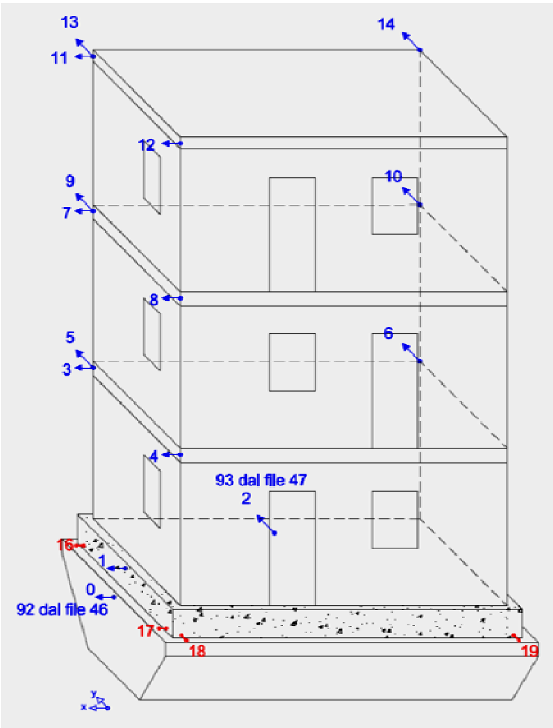
Test	Experimental frequency As given by Simone Girello	$(E_cJ)/(E_cJ_{gross\ section})$ which gives a numerical frequency = experimental frequency
Before 0.05 g test	10.0 Hz 11.7 Hz	**0.43** **0.59**
Between 0.05 g and 0.15 g tests	10.0 Hz 11.7 Hz	**0.43** **0.59**
Between 0.15 g and 0.50 g tests	10.0 Hz 11.7 Hz	**0.43** **0.59**
Between 0.50 g and 1.00 g tests	- 11.0 Hz	- **0.52**
Between 1.00 g and the first 0.30 g white noises	- 10.4 Hz	- **0.47**
Between the 0.30 g white noises and the first 1.2 g test	- 8.6 Hz	- **0.32**
Between the first 1.20 g test and the 0.50 g white noises	-	-
Between the 0.50 g white noises and the second 1.20 g test	- 8.2 Hz	- **0.29**
After the last 1.20 g test	-	-

INDICATION ON GLOBAL STIFFNESS

different from the 0.15 values of the single walls and from the 0.11 values of the H-shaped structure

APPROACH A

Accelerometers



$$a_i(t)$$

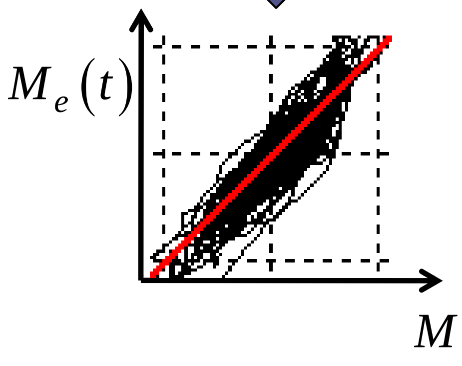
$$\varepsilon_j(t)$$

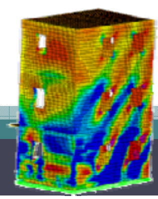
$$HP:\begin{cases} E_c = 30000 \text{ MPa} \\ E_s = 210000 \text{ MPa} \end{cases}$$

$$HP:\begin{cases} W_{\text{gross section}} \\ W_{\text{uncracked}} \\ W_{\text{fully cracked}} \end{cases}$$

$$M_{ext}(t) = \sum_i m_i a_i(t) h_i$$

$$M_{\text{int}}(t) = W \cdot E_c \cdot \varepsilon_j(t)$$





3-STOREY STUCTURE: STIFFNESS

K gross section:

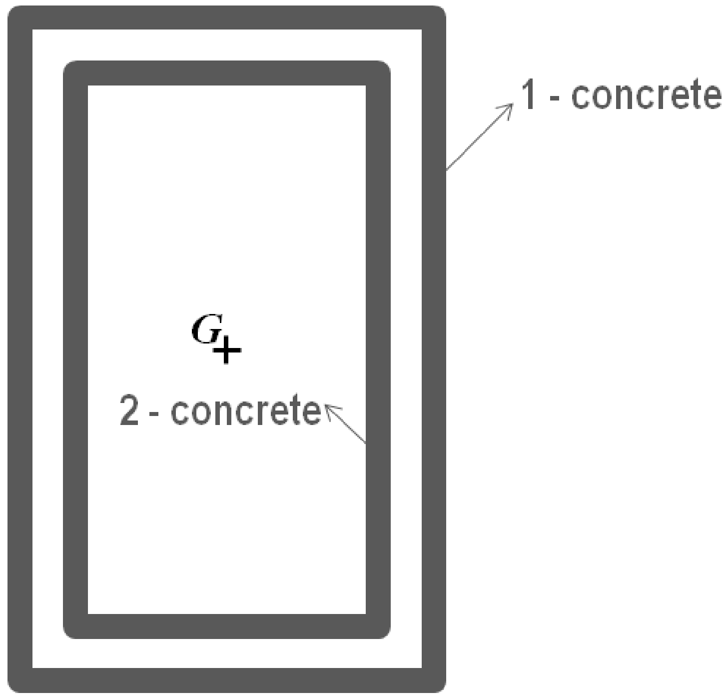
$$K_{\text{gross section}} = \left(\frac{1}{K_{flex}} + \frac{1}{K_{shear}} \right)^{-1}$$

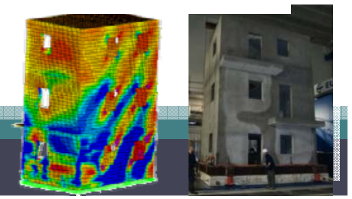
$$K_{flex} = \frac{3E_c J_{\text{gross section}}}{h^3}$$

$$K_{shear} = \frac{G_c A_{\text{gross section}}}{\chi h}$$

$$J_{\text{gross section}} = J_1 + J_2$$

$$A_{\text{gross section}} = A_1 + A_2$$





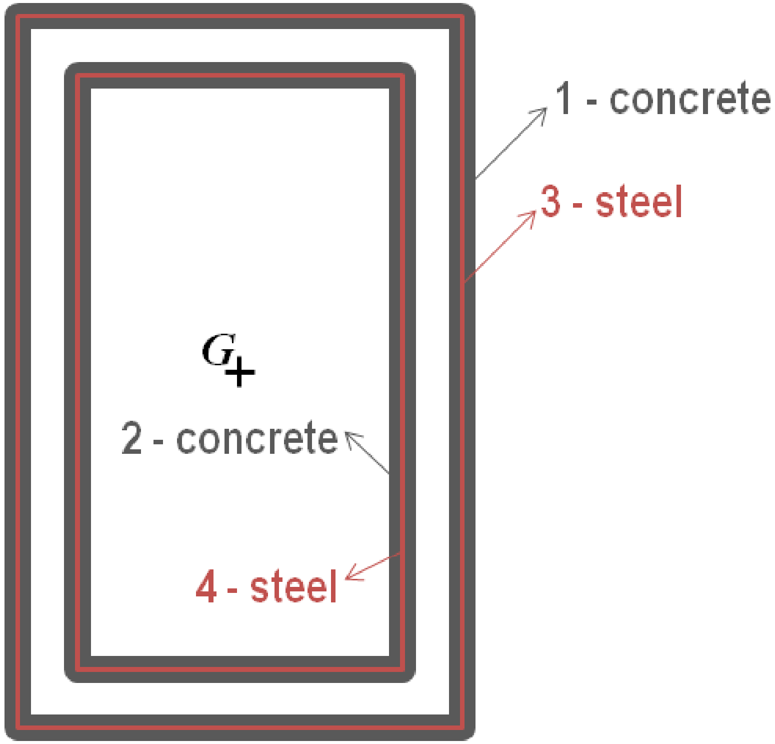
3-STOREY STUCTURE: STIFFNESS

K uncracked:

$$K_{\text{uncracked}} = \left(\frac{1}{K_{flex}} + \frac{1}{K_{shear}} \right)^{-1}$$

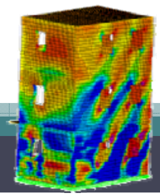
$$K_{flex} = \frac{3E_c J_{\text{uncracked}}}{h^3}$$

$$K_{shear} = \frac{G_c A_{\text{uncracked}}}{\chi h}$$



$$J_{\text{uncracked}} = J_1 + J_2 + n\left(J_3 + J_4\right)$$

$$A_{\text{uncracked}} = A_1 + A_2 + n\left(A_3 + A_4\right)$$



3-STOREY STUCTURE: STIFFNESS

K fully cracked:

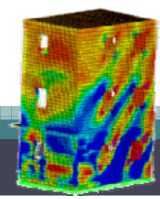
$$K_{\text{fully cracked}} = \left(\frac{1}{K_{flex}} + \frac{1}{K_{shear}} \right)^{-1}$$

$$K_{flex} = \frac{3E_c J_{\text{fully cracked}}}{h^3}$$

$$K_{shear} = \frac{G_c A_{\text{fully cracked}}}{\chi h}$$

$$J_{\text{fully cracked}} = \frac{2b_{//}x^3}{3} + \frac{b_{\perp}{}^3\left(h_{\perp} - 2b_{//}\right)}{12} + b_{\perp}\left(h_{\perp} - 2b_{//}\right)\cdot\left(x - \frac{b_{\perp}}{2}\right)^2 + \frac{2nA_{s,//}h_{//}{}^2}{12} + 2nA_{s,//}\left(\frac{h_{//}}{2} - x\right)^2 + n\left(A_{s,\perp} + A_{\text{catena}}\right)\left(h_{//} - \frac{b_{\perp}}{2} - x\right)^2 + n\left(A_{s,\perp} + A_{\text{catena}}\right)\left(x - \frac{b_{\perp}}{2}\right)^2$$

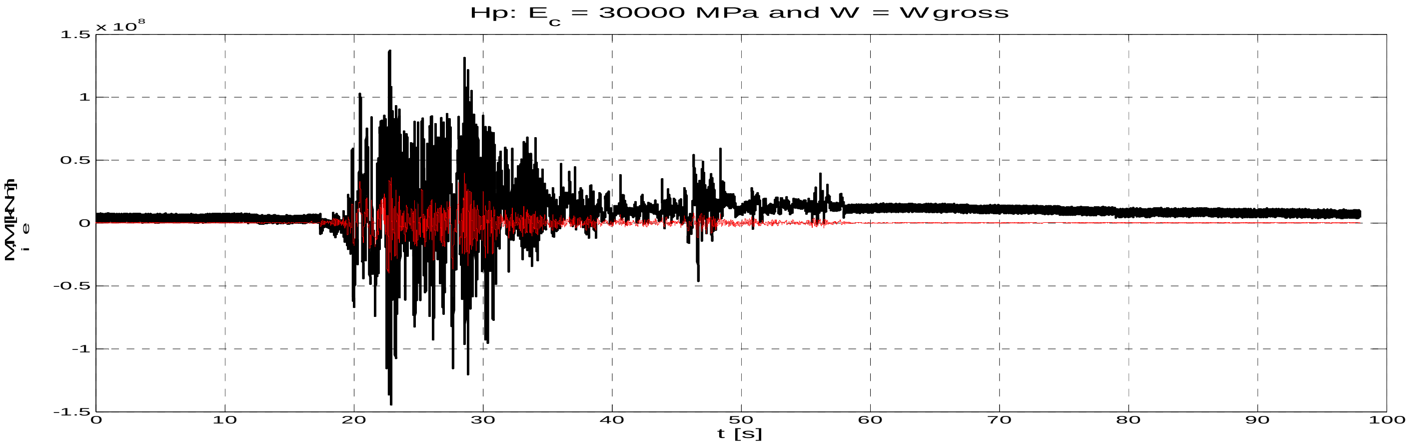
$$A_{\text{fully cracked}} = b_{//}\cdot x + 2nA_{s,//} + 2nA_{s,\perp} + 2nA_{catena}$$



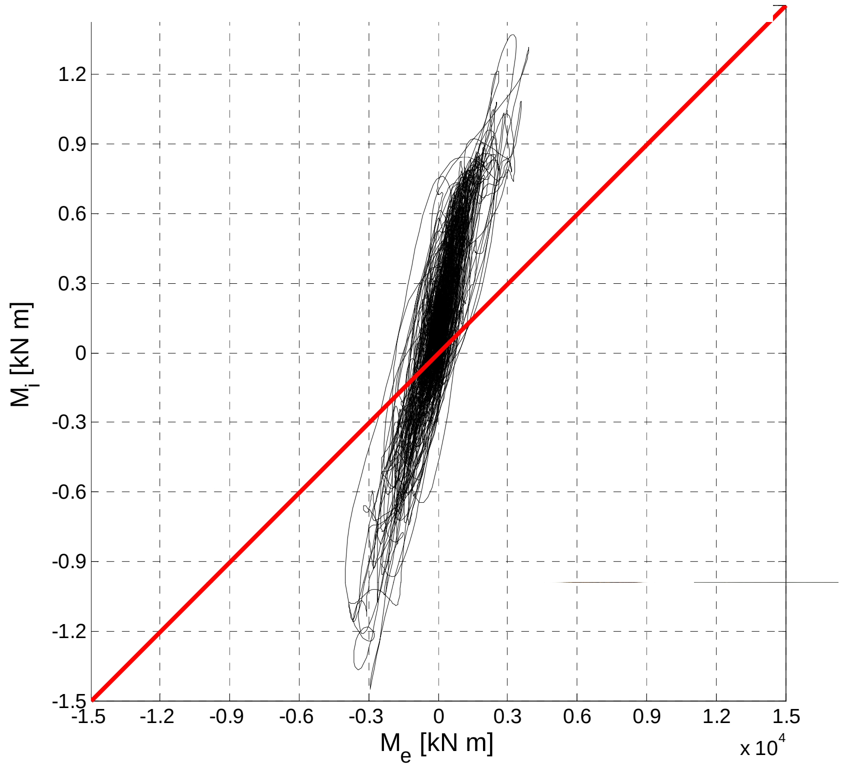
APPROACH A: GROSS SECTION

$M_{ext}(t)$

$M_{int}(t)$



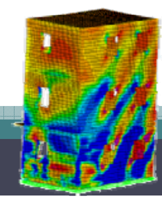
$M_{int}(t)$ vs. $M_{ext}(t)$



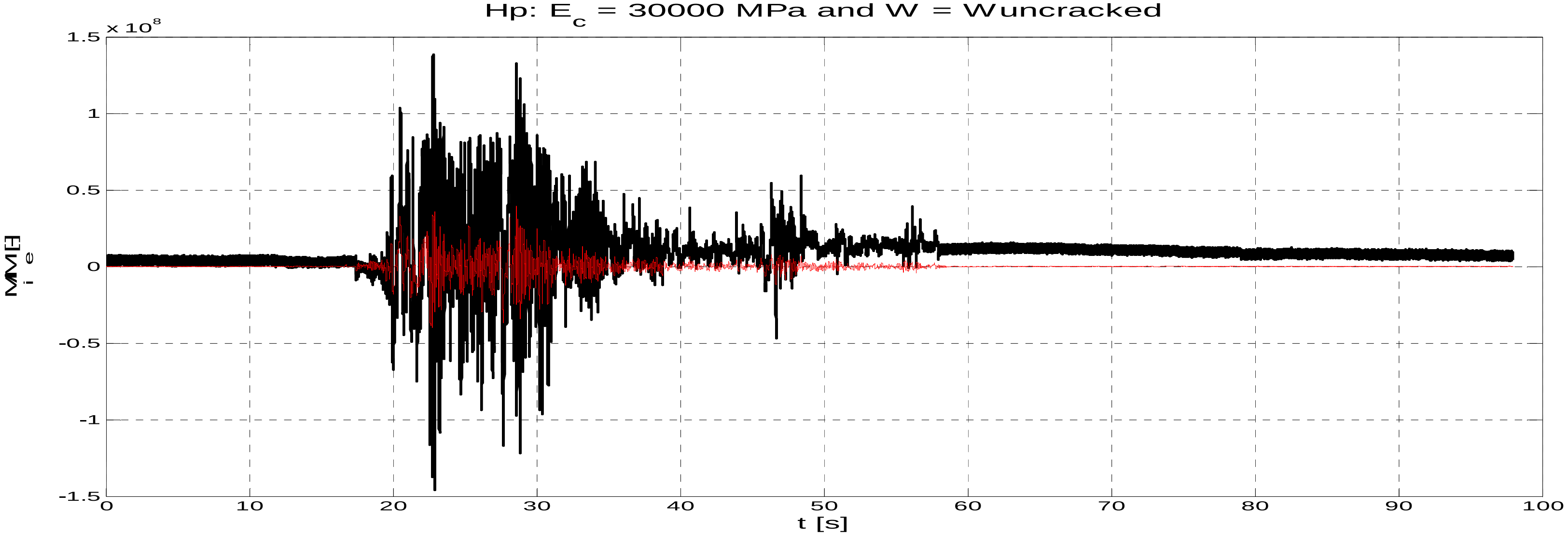
APPROACH A: UNCRACKED SECTION

$$M_{ext}(t)$$

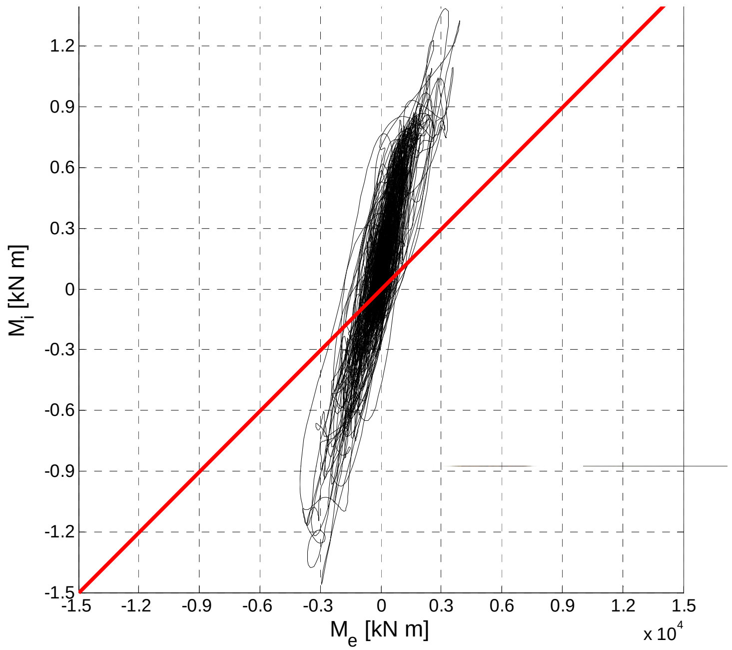
$$M_{\mathrm{int}}(t)$$



APPROACH A: UNCRACKED SECTION

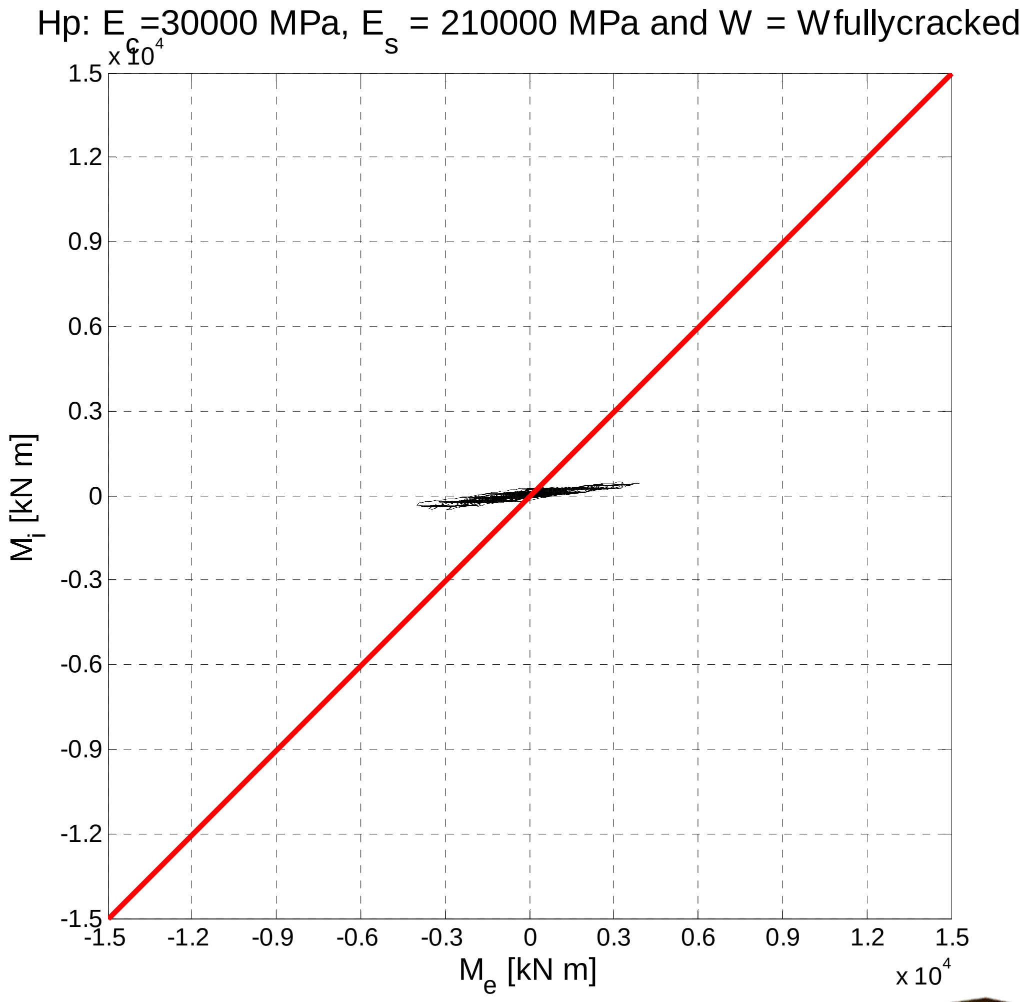


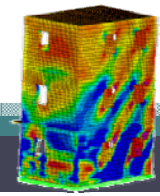
$$M_{\text{int}}(t) \text{ vs. } M_{ext}(t)$$



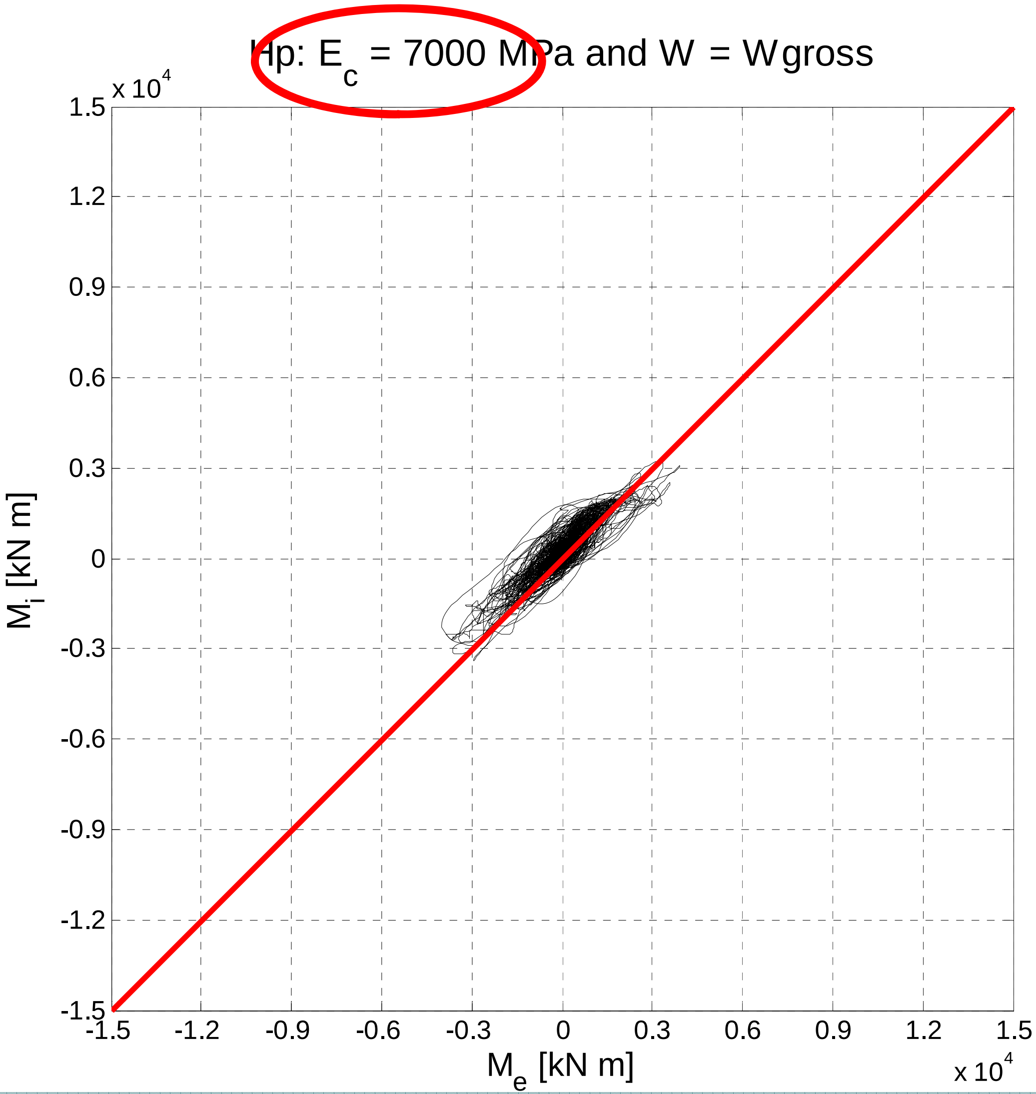
APPROACH A: FULLY CRACKED SECTION

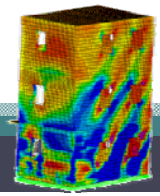
$$M_{\text{int}}(t) \text{ vs. } M_{ext}(t)$$



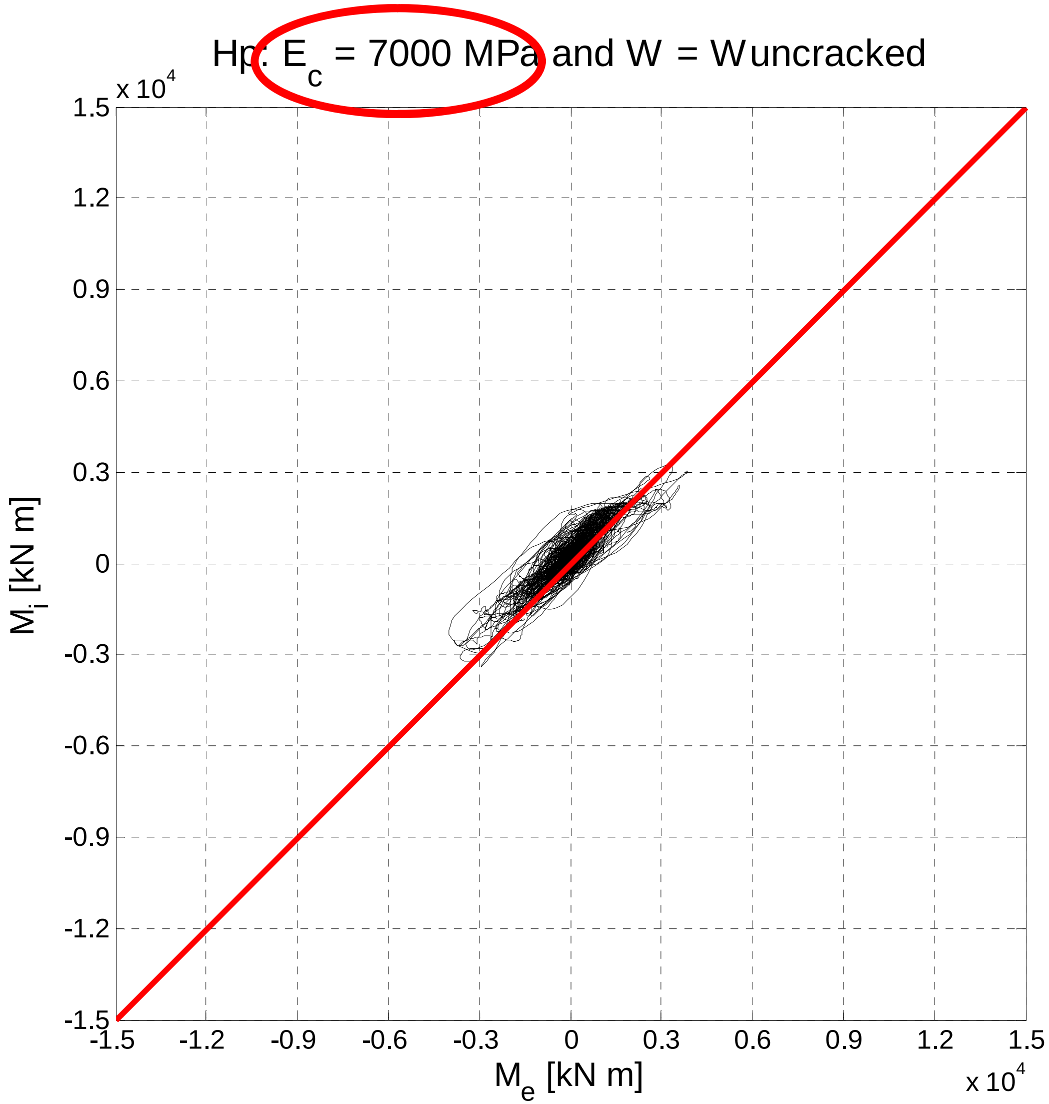


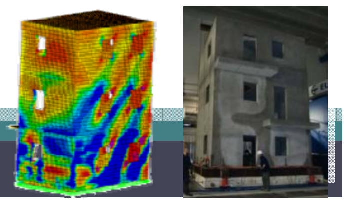
APPROACH A: MODIFIED Ec GROSS SECTION





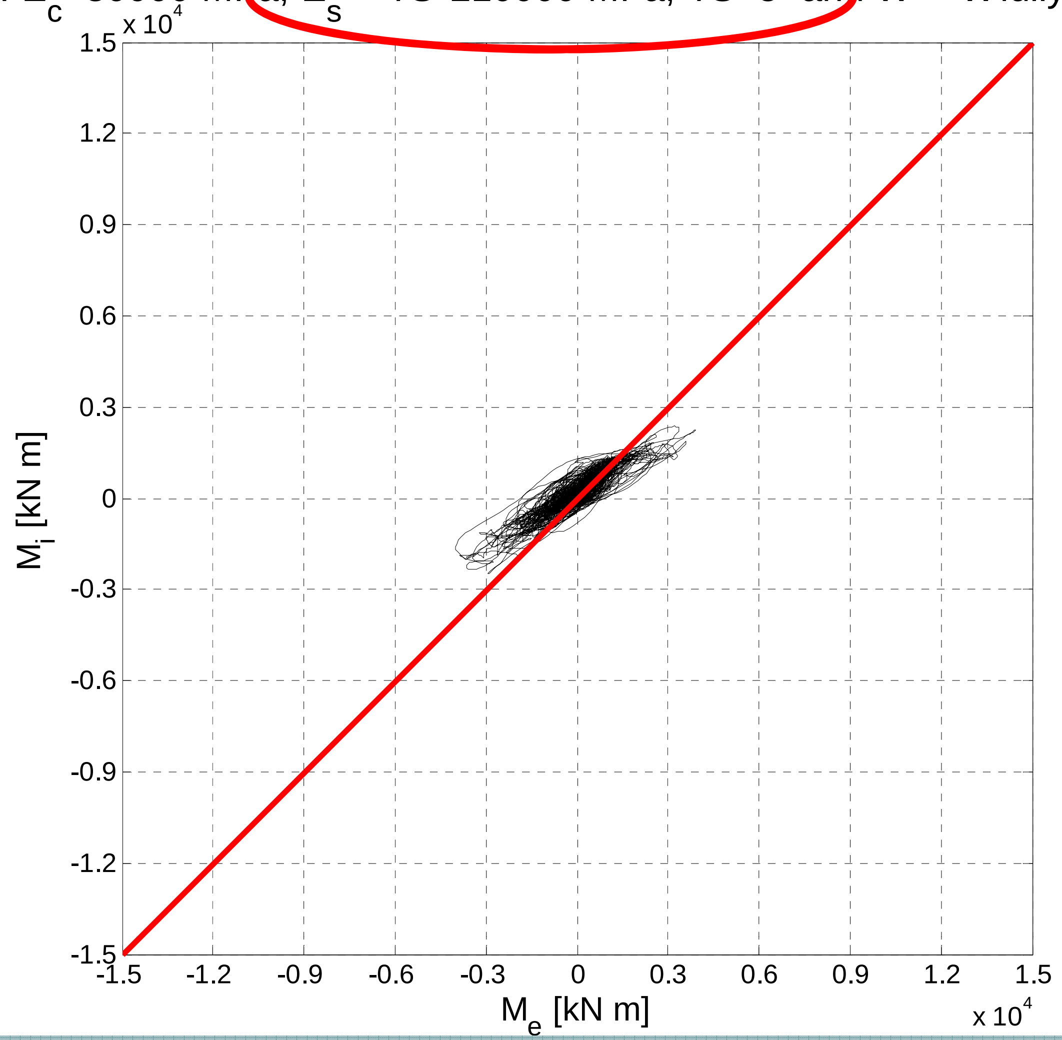
APPROACH A: MODIFIED Ec UNCRACKED SECTION

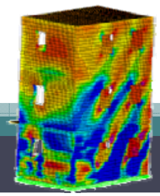




APPROACH A: MODIFIED Es CRACKED SECTION

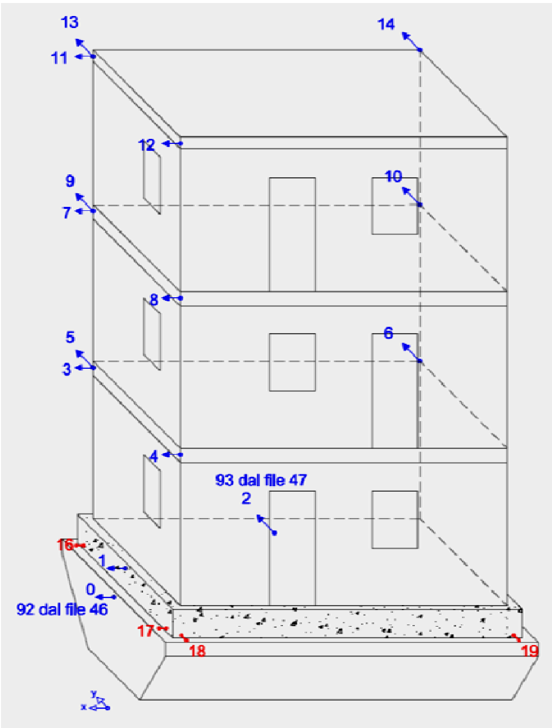
Hp: E_c=30000 MPa, E_s = TS*210000 MPa, TS=5 and W = Wfullycra





APPROACH B

Accelerometers



$$a_i(t)$$

$$M_e(t) = \sum_i m_i a_i(t) h_i$$

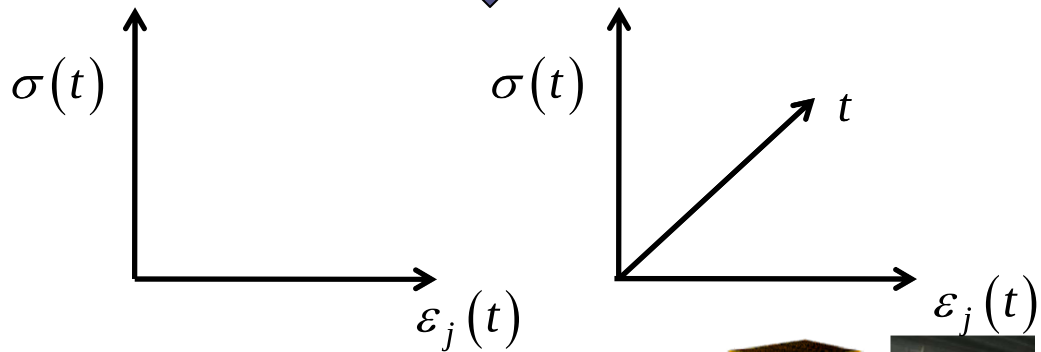
$$HP: \begin{cases} E_c = 30000 \text{ MPa} \\ E_s = 210000 \text{ MPa} \end{cases}$$

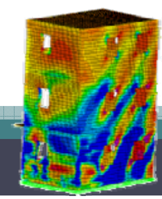
$$HP: \begin{cases} W_{\text{gross section}} \\ W_{\text{uncracked}} \\ W_{\text{fully cracked}} \end{cases}$$

$$\varepsilon_j(t)$$

$$\sigma(t) = \frac{M_e(t)}{W}$$

$$\varepsilon_j(t)$$

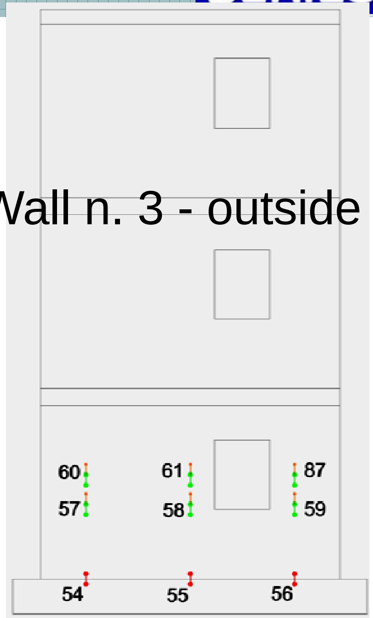


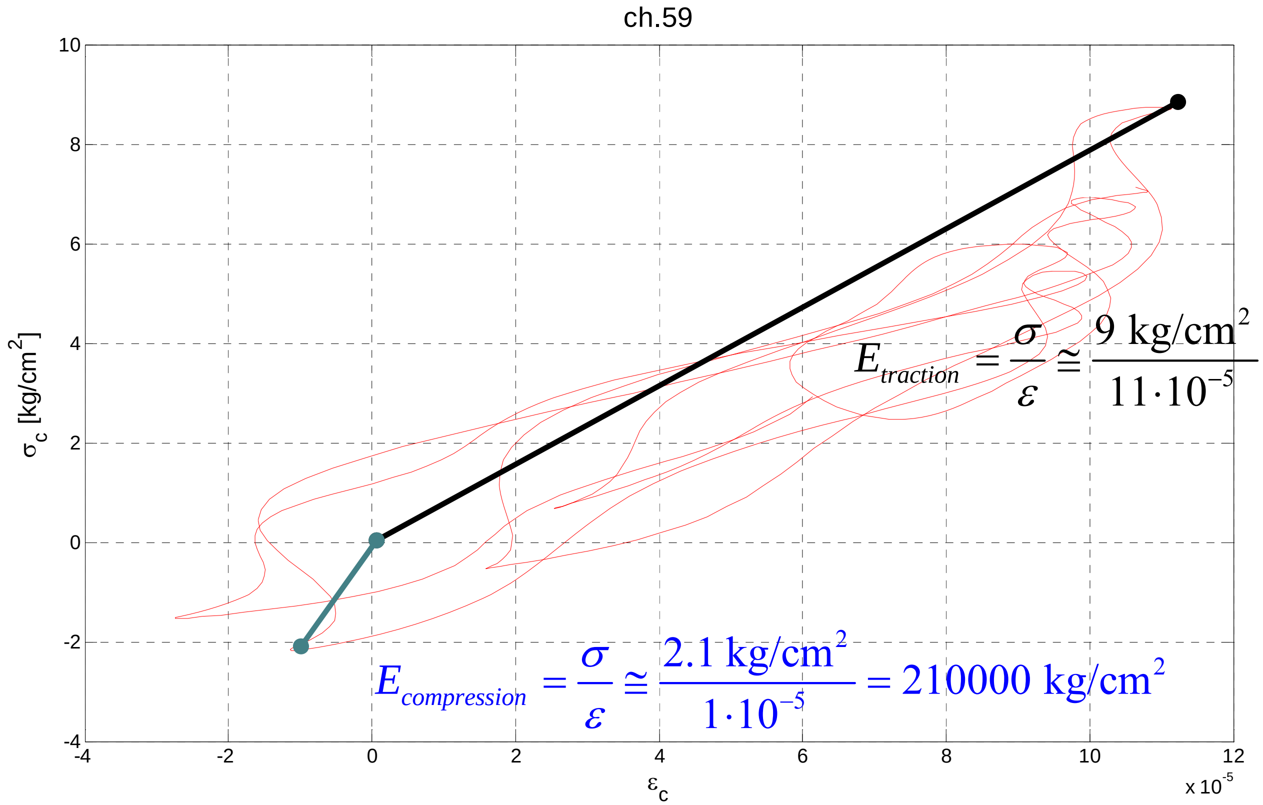


SIGMA-STRAIN CH. 59

HP) gross section
time window = 22.5 s ÷ 23.0 s

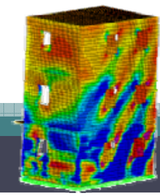
Wall n. 3 - outside





$$E_{traction} = \frac{\sigma}{\varepsilon} \cong \frac{9\ \text{kg/cm}^2}{11 \cdot 10^{-5}} = 82000\ \text{kg/cm}^2$$

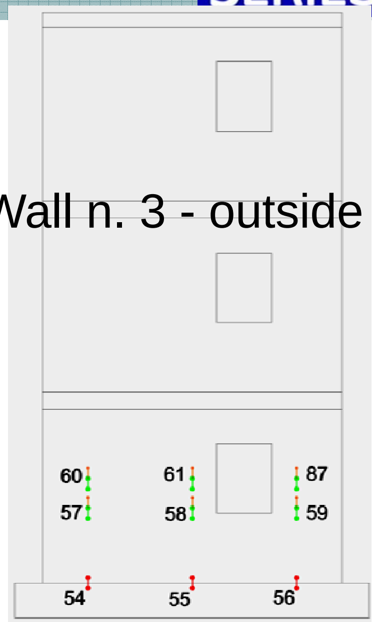
$$E_{compression} = \frac{\sigma}{\varepsilon} \cong \frac{2.1\ \text{kg/cm}^2}{1 \cdot 10^{-5}} = 210000\ \text{kg/cm}^2$$

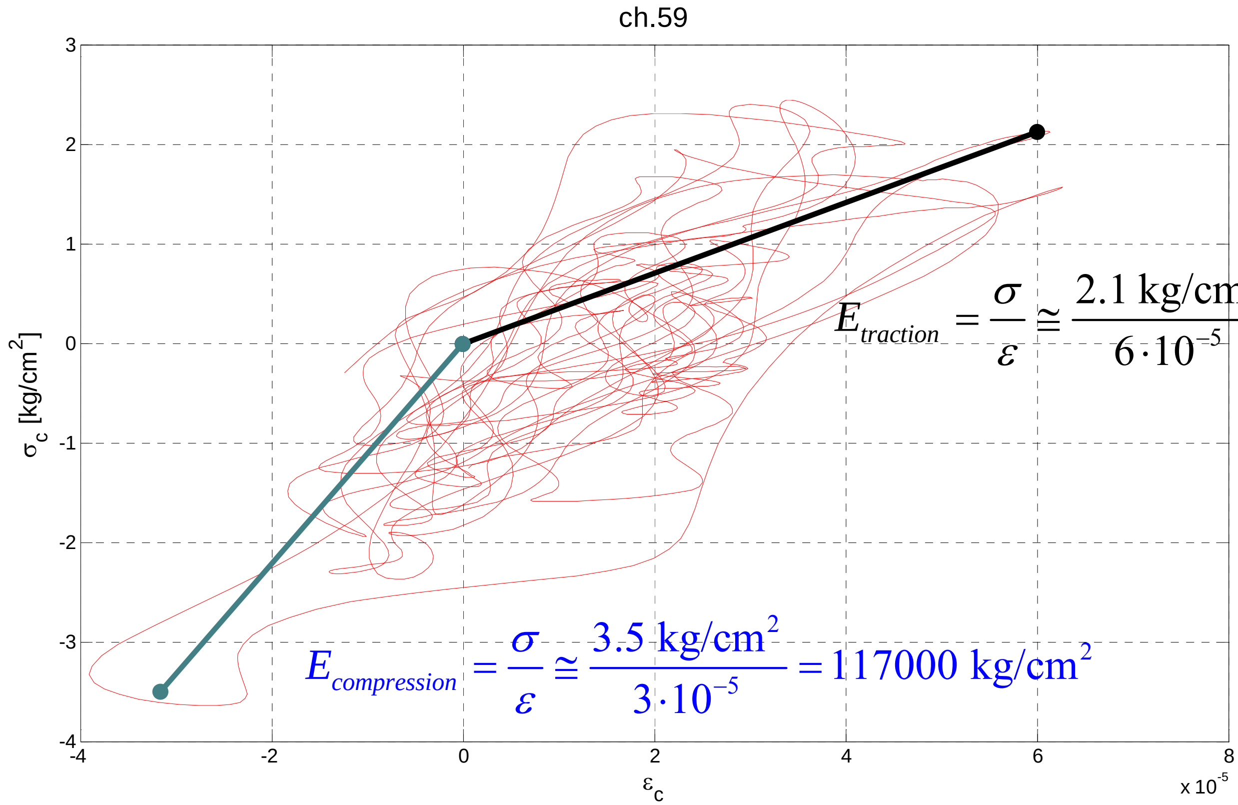


SIGMA-STRAIN CH. 59

HP) gross section
time window = 35.0 s ÷ 37.0 s

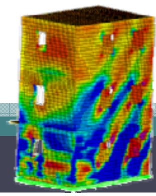
Wall n. 3 - outside





$$E_{traction} = \frac{\sigma}{\varepsilon} \cong \frac{2.1\ \mathrm{kg/cm^2}}{6 \cdot 10^{-5}} = 35000\ \mathrm{kg/cm^2}$$

$$E_{compression} = \frac{\sigma}{\varepsilon} \cong \frac{3.5\ \mathrm{kg/cm^2}}{3 \cdot 10^{-5}} = 117000\ \mathrm{kg/cm^2}$$



PRELIMINARY CONCLUSIONS

- **Solution adopted for brace the structure for the lifting and transport phase is correct.**
- **The wall polystyrene-concrete system works correctly under seismic loads**
- **The 3d building in more rigid and strong than the predicted by the models (analytical and numerical) calibrated with the results of cyclic tests.**

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