Design of a shaking table test on a 3-storey building composed of cast-in-situ concrete walls

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SUMMARY:
Structural systems composed of cast in situ sandwich squat concrete walls, which make use of a lightweight material (for example polystyrene) as a support for the concrete, are widely used for construction in non seismic areas or in areas of low seismicity, and appreciated for their limited constructions costs, limited installation times, great constructions flexibility and high energy and acoustic efficiency. However their seismic behaviour has not been fully investigated. In recent years, an exhaustive experimental campaign, carried out by the University of Bologna and the Eucentre labs in Pavia, was devoted to the assessment of the seismic performances of single walls and of a portion of structure through cyclic tests under horizontal loads. To validate the theoretically and partially-experimentally anticipated (through cyclic tests under horizontal loads) good seismic behaviour of cellular structures composed of cast in situ squat sandwich concrete walls, shaking table tests were performed, at the laboratory facilities of the Eucentre in Pavia, on a full-scale 3-storey structural system composed of cast-in-situ squat sandwich concrete walls (characterized by 5.50 x 4.10 meters in plan and 8.25 meters in height). This paper describes the design process of the model as it should: (i) be representative of common real built structures; (ii) be easily transportable from the construction site to the table; (iii) lead to significant results; (iv) not damage the shaking table.

Keywords: cast in situ sandwich squat concrete walls, shaking table tests, full-scale 3-storey structural system.

1. INTRODUCTION

Several different construction techniques characterized by low costs, limited installation times, great flexibility and high energy/acoustic efficiency have been proposed in the years for the accomplishment of intensive large-scale programs for low-rise residential buildings at a worldwide level (Vanderwerf *et al*. 2005). Among these, the structural systems composed of cast in situ squat concrete walls, which make use of lightweight material (e.g. polystyrene) as a support (as in case of Shotcrete, i.e. prefabricated modular pre-reinforced polystyrene support panel) for structural concrete, appear to be extremely promising. Indeed these systems allow to obtain high structural, thermal and acoustic efficiency, since the traditional concrete guarantees the loadbearing capacity, while the lightweight material is left in place (once the construction is completed) to ensure thermal and acoustic insulation. When applied to low-rise residential buildings these innovative technologies allows to obtain a cellular structure (i.e. structure characterized by bundled-tube behaviour) composed of cast in situ sandwich squat concrete walls. As far as the seismic behaviour of concrete walls are concerned, most of the research work accomplished up to date is focused upon slender cantilever walls (for sake of conciseness, let's refer to Paulay and Priestley, 1992 and to Coull and Stafford Smith, 1991) with little research works developed for squat walls despite they have already shown valuable strength resources towards earthquake of high intensity (as for example, in Cile, Wood 1991). Squat sandwich walls need of specific studies not only due to their global geometrical squat configuration, but also because they have basic characteristics - like thickness, amount of vertical reinforcement, constructive details, etc - which completely differs from those of the traditional ones.

For all of the above, in recent years, an exhaustive experimental campaign was carried out by the University of Bologna and the Eucentre labs in Pavia (Trombetti *et al.* 2010). The effort was devoted to the assessment of the structural performances of single panels composed of cast-in-situ sandwich squat concrete walls. In order to obtain a correct characterization of the seismic behavior (stiffness, strength, ductility, energy dissipation) of such structural elements, a number of tests were performed upon two-dimensional (3.0 m by 3.0 m) cast-in-situ sandwich squat concrete walls (with and without openings) and upon a 2-story H-shaped portion of structure. In the experimental tests, a number of horizontal in-plane loading cycles were imposed to the specimens, while the vertical load was kept constant. The results obtained have shown that the tested elements are characterized by: (i) absence of a real failure; (ii) high values of the maximum horizontal load applied to the structural systems (higher than the applied vertical load); (iii) residual bearing capacity with respect to the vertical loads, also when large lateral deformations were developed; (iv) a good degree of kinematic ductility.

To validate the theoretically and partially-experimentally anticipated (through cyclic tests under horizontal loads) good seismic behaviour of cellular structures composed of cast in situ squat sandwich concrete walls, shaking table tests were designed and performed, at the laboratory facilities of the Eucentre in Pavia, on a full-scale 3-storey structural system composed of cast-in-situ squat sandwich concrete walls (characterized by 5.50 x 4.10 meters in plan and 8.25 meters in height). The shaking table test, performed on December 6th – 7th 2011, represent the main objective of the research project "*SEismic behaviour of structural SYstems composed of cast in situ COncrete WAlls*" (SE.SY.CO.WA) and it is part of the SERIES Project (*Seismic Engineering Research Infrastructures for European Synergies*). This paper focuses on the design phase and synthetically anticipates (as they will be accurately discussed in following research works) some preliminary results obtained from the shaking table tests.

2. THE CONSTRUCTION SYSTEM AT HAND: IDENTIFICATION OF ITS PECULIARITIES

2.1. The modular panels

The construction system at hand is based on the production and use of *prefabricated modular pre-reinforced polystyrene panels* (which therefore will be simply referred as *modular panels*) which act as support for the placing of the structural concrete. These *modular panels* (Fig. 1(a)) have a length of 1120 mm and an adjustable height which will be equal to the interstorey height. They are composed of a single expanded polystyrene sheet (this sheet can be produced with thickness varying between 60÷160 mm in order to suit the specific needs for thermal and acoustic insulation) which feature two grids of galvanized and electrowelded steel wire mesh on the two external faces. Notice that the expanded polystyrene sheet is shaped with a waved profile along the horizontal direction. The two grids of wire meshes are mechanically linked together with metallic ties (having diameter of 3 mm in quantity of 40÷50 for m^2) which are placed during the production at the factory and which are thus embedded within the polystyrene. The modular panels display a peculiar design of the polystyrene edges and of wire meshes (represented in Fig. 1 (a)) so that when the modular panels are positioned one beside each other (the reason of this positioning will be provided in the following paragraph) the meshes are overlapped of about 100 mm to guarantee the continuity of the horizontal reinforcement. The polystyrene used for the panel is typically characterized by a density of 15 kg/m^3. Both the meshes and the ties are typically realized using galvanized steel with low carbon content and breakage tension of f_{tk}=700 MPa, classified as "C7D", have a diameter of 2.5 mm and the mesh is 50 mm x50 mm (ϕ2.5/50x50 mm).

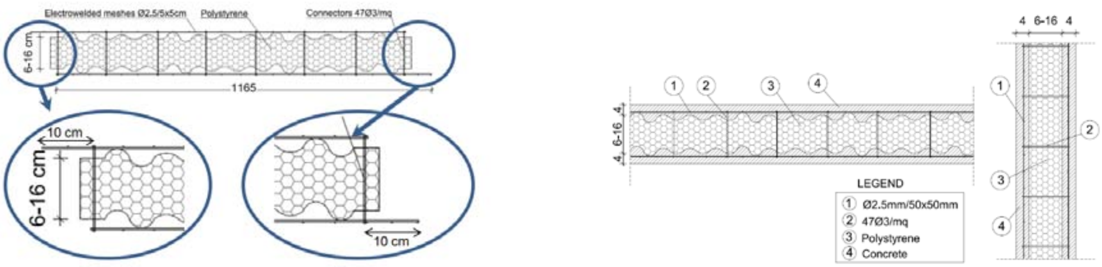
2.2. The cast in situ sandwich concrete walls

At the construction site, the *modular panels* are positioned one beside each other to obtain the so-called *support wall* of the desired dimensions (in accordance with the architectonic design of the building structure). Fig. 2 shows the positioning of the modular panels to obtain the *support walls*.

Appropriate additional reinforcements (typically 1+1ϕ12 bars and ϕ8/50 cm U-shaped bars, made up of B450C steel) are added: (i) above, below and on the sides of the (window/door) openings in order to obtain a reinforcing square, (ii) at the lateral edges of the support walls in order to provide additional strength in areas where the seismic action generally induces high levels of stresses. Once the polystyrene support walls are set in place, two layers of concrete (each one of about 40 mm in thickness) are sprayed on each side to obtain a sandwich wall. These layers are obtained in two distinct phases: (i) a first layer is sprayed so that the mesh is cover, then, after the first one is hardened, (ii) a second layer is sprayed, in order to reach the desired thickness. Finally, the walls are smoothed with surface filler. Usually the concrete employed is C25/30 in accordance with EC2 (cubic compressive resistance is 30MPa) with slump S5 and aggregates size up to 3 mm. The quantity of the reinforcement provided by the electrowelded meshes (ϕ2.5/50x50 mm), together with the typical total thickness of the two layers constituting the final concrete wall (40+40 mm), leads to an amount of vertical reinforcement of 0.00245% (without any additional bars). The cast in situ concrete walls thus obtained are sandwich walls composed of two 40 mm thickened walls each reinforced with electrowelded meshes (and eventually also with additional bars) connected to each other with the polystyrene and the ties. In the following sections of the paper, we will refer to this specific concrete formation as *cast in situ sandwich squat concrete wall* (or, for sake of conciseness, simply as the *wall*). The out-of-plane behaviour (i.e. the effectiveness of the polystyrene and the ties to ensure the transmission of the shear stresses guaranteeing the conservation of the plane section) of these walls is not the object of this paper. In this research the in-plane behaviour of the *sandwich wall* will be investigated. For this reason, the *sandwich wall* (despite its name) will be considered as composed by a single wall of thickness equal to the sum of the thicknesses of the 40 mm concrete layers.

2.3. The connections

The connections between (i) walls and foundation, (ii) orthogonal walls (iii) walls and floors and (ii) are designed to ensure the complete transmission of the actions (i.e. shear, bending, and eventually any axial force) which are exerted through adjacent orthogonal walls. In this way it is possible to realize a *cellular behaviour* with respect to horizontal loads. Moreover, to ensure the complete transmission of the actions even during rare seismic events, the connections are designed following a *Capacity Design* approach. These connections are created using special panels and placing the appropriate amount of horizontal and vertical reinforcement (Fig. 3).



(a) (b)

Figure 1. (a) Horizontal section of the modular panel and the special design of the ends. (b) The *sandwich cast in situ squat concrete wall.*

2.4. The structural system obtained and its features

The construction system, described in the previous sections, allows to obtain a structural system characterized by the following specific features:

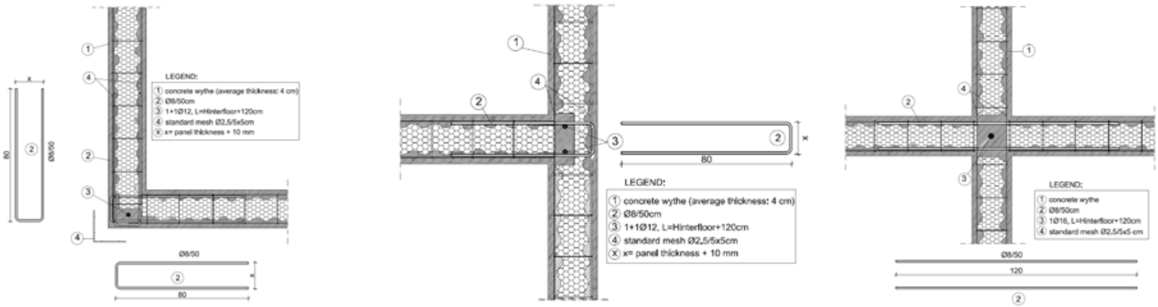
1. the structural system results composed of *sandwich walls*, characterized by:
   - *light amount* of vertical reinforcement;
   - the *same amount of vertical and horizontal reinforcement*;
2. the system is characterized by a *bundled-tube behaviour* (i.e. *cellular behaviour*) (Coull and Stafford Smith 1991; Khan 2004).

Moreover, given that this structural system is typically used for the realization of low-rise residential buildings, the sandwich walls are characterized by:

3. a *squat* configuration;
4. low values of *vertical stresses* ($\nu \leq 0.03$).

(a) (b) (c)

Figure 2. Illustrative example of the typical assemblage of the *modular panel* to create *the pre-reinforced polystyrene support squat panel*, to be completed further on with the spraying of the double concrete layers.



(a) (b) (c)

Figure 3. "*Special panels*" used to obtain the connections between (a) two, (b) three and (c) four orthogonal walls.

3. SHAKING TABLE TEST: DESIGN OF THE STRUCTURE

The objective of the shaking table tests is to validate the theoretically and partially-experimentally anticipated (as obtained with pseudo-static cyclic loading) good seismic behaviour of cellular structures composed of cast in situ squat sandwich concrete walls, in case of actual seismic loading. In more detail, the test are aimed at identifying the possible differences in the behaviour of the structural system at hand between the cases of (i) pseudo-static cyclic loading and (ii) dynamic seismic loading. For this reason the shaking table test represents the completion of the wide experimental campaign started, in the first decade of the year 2000, with pseudo-static cyclic tests performed upon two-dimensional (3.0 m by 3.0 m) cast-in-situ sandwich squat concrete walls (with and without openings) and upon a 2-story H-shaped portion of structure, and carried on with the development of analytical formulas for the theoretical evaluation of the behavior under horizontal loads verified on the basis of the experimental results (Trombetti *et al.* 2010).

To satisfy the above mentioned objectives, the specimen tested on the shaking table test was designed to: (i) be representative of common real built structures (squat configuration of the walls); (ii) be easily transportable from the construction site to the table (this involves the design of specific lifting and lowering systems); (iii) lead to significant results (carried out up to the collapse of the walls, in order to capture also the post-yielding behavior and the ductility resources); (iv) not damage the shaking table (analytical and numerical prediction of the specimen behavior should be developed). Various solutions have been proposed for both the specimen and the foundation systems, before reaching the final setup which consist in a full-scale 3-storey structural system composed of cast-in-situ squat sandwich concrete walls characterized by 5.50 x 4.10 meters in plan and 8.25 meters in height created by adopting the construction system previously described but halving the amount of vertical and horizontal reinforcement in the walls (ϕ2.5/100x100 mm instead of ϕ2.5/50x50 mm). the foundation is composed of four beams and it characterized by the following dimensions: 6.40 m x 5.10 m. The two long sides of the foundation are characterized by a 70 cm x 40 cm rectangular section whereas the two short sides are characterized by a 650 cm x 40 cm rectangular section. The shaking table of the Eucentre Lab in Pavia is characterized by one degree of freedom and for this reason the seismic input was applied in the direction parallel to the long walls (called in the following “parallel walls”, characterized by a length of $\ell_{//} = 5.52$ m). The other two walls, the short ones, characterized by a length of $\ell_{\perp} = 4.12$ m , will be indicated as “perpendicular walls”. In the following (i) the stresses (i.e. demand) in the walls (perpendicular and parallel) due to a spectral acceleration, S_a, equal to 1g will be evaluated together with (ii) the corresponding strength (i.e. capacity). The strength of the walls will be evaluated applying the analytical formulas developed by the authors on the basis of the classical hypothesis for traditional r.c. elements (Trombetti et al. 2010). The validity of these formulas has been verified comparing the analytical results so obtained with the corresponding experimental ones as obtained from pseudo-static cyclic tests performed upon two-dimensional (3.0 m by 3.0 m) cast-in-situ sandwich squat concrete walls (with and without openings) and upon a 2-story H-shaped portion of structure. Comparing the stresses due to 1g spectral acceleration with the corresponding strength, the sequence of the possible collapse mechanism of the structure has been identified.

Figure 4. Specimen structure.

3.1. Demand due to 1g spectral acceleration

The total shear, T_{Ed} , and the total bending moment, M_{Ed} , at the base of the structure due to a spectral acceleration S_{a} = 1g are equal to: $T_{\text{Ed}} = m_{\text{structure}} \cdot 1g = 66$ t and $M_{\text{Ed}} = T_{\text{Tot, base}} \cdot (2/3) H = 360$ t m . In the previous equations, $W_{\text{structure}}$ is the total weight of the structure (i.e. 66 t) and H is the height of the building. For the evaluation of the total bending moment at the base of the structure a triangular distribution of the horizontal forces has been supposed. During the design phase a number of distribution have been considered, but for sake of conciseness they have been omitted. Under the hypothesis of (i)linear elastic behavior and of (ii) conservation of plane sections the moment repartition under the walls has been evaluated. In detail the bending moment of the parallel and perpendicular walls results in: $M_{Ed,\,//} = \rho_{//} \cdot M_{\text{Ed}} = 0.31 \cdot 360 = 111$ t m and $M_{Ed,\,\perp} = \rho_{\perp} \cdot M_{\text{Ed}} = 0.69 \cdot 360 = 249$ t m ; where $\rho_{//}$ and $\rho_{\perp}$ are respectively the repartition coefficient of parallel and perpendicular walls (i.e. $\rho_{//} = J_{//} / (J_{//} + J_{\perp}) = 0.31$ and $\rho_{\perp} = J_{\perp} / (J_{//} + J_{\perp}) = 0.69$ with $J_{//}$ and $J_{\perp}$ respectively the moments of inertia of parallel and perpendicular walls). The parallel bending moment on the single parallel wall (ignoring the presence of the openings) is: $M_{\text{Ed, single // wall}} = M_{Ed,\,//} / 2 = 56$ t m . The perpendicular bending moment results in stresses of compression and traction for the perpendicular walls: $N_{\text{Ed, }\perp\text{ wall}} = M_{Ed,\,\perp} / \ell_{//} = 45$ t . The total shear at the base of the structure is taken by the single parallel walls ($T_{\text{Ed, single // wall}}$) and ignoring the presence of the openings, it is equal to: $T_{\text{Ed, single //wall}} = T_{\text{Ed}}/2 = 33\text{ t}$.

3.2. Capacity of the walls

3.2.1. Parallel wall – In plane first yielding bending strength

The in plane first yielding bending moment of the single parallel wall is given by (Trombetti *et al.*):

$$M_{y1} = \left(\frac{\rho b y_{y1}}{2} f_{ym}\right)\cdot\left(\frac{h}{2}-\frac{y_{y1}}{3}\right)+\left(\frac{b\left(h-y_{y1}\right)^2}{2y_{y1}}\frac{f_{ym}}{n}\right)\cdot\left(\frac{h}{6}+\frac{y_{y1}}{3}\right)+A_{s,c} f_{ym}\left(h-2c\right)=149\text{ t m} \quad (3.1)$$

Where: $b = 8\text{ cm}$ is the thickness of the wall; $h = \ell_{//} = 412\text{ cm}$ is the width of the wall, $N_{Ed} = 25\text{ t}$ is the axial load applied, $n = E_s / E_c = 7$ is the homogenization coefficient, $f_{ym} = 500\text{ MPa}$ is the average yielding strength of steel, $\rho = A_{s,//}/bh$ is the geometrical amount of vertical reinforcement, $A_{s,//}$ is the area of the vertical reinforcements (1+1ϕ2.5mm/10cm), $A_{s,c}$ is the area of the additional tensile bars (1ϕ16mm), $c = 4\text{ cm}$ is the rebar cover, y_{y1} is the position of the neutral axis with respect to the tensile fiber:

$$y_{y1} = \left[\left(h+\frac{N_{Ed}n}{f_{yd}b}\right)-\sqrt{\left(h+\frac{N_{Ed}n}{f_{yd}b}\right)^2-h^2\left(1-n\rho\right)}\right]\Big/\left(1-n\rho\right) \quad (3.2)$$

3.2.2. Parallel wall – In plane ultimate bending strength

The in plane ultimate bending moment of the single parallel wall is given by (Trombetti *et al.*):

$$M_u = \left(f_{ym}\rho b y_{u,sb}\right)\left(\frac{h}{2}-\frac{y_{u,sb}}{2}\right)+\left(f_{cm}b0.8\left(h-y_{u,sb}\right)\right)\left(0.1h+0.4y_{u,sb}\right)+A_{s,c} f_{ym}\left(h-2c\right)=181\text{ t m} \quad (3.3)$$

Where: $\nu = N_{Ed}/\left(f_{cm}bh\right)$ is the dimensionless axial stress, $\rho_m = \left(f_{ym}/f_{cm}\right)\rho$ is the mechanical amount of reinforcement, $f_{cm} = 30\text{ MPa}$ is the average concrete compression strength, $y_{u,sb}$ is the position of the neutral axis with respect to the tensile fiber (evaluated considering the stress block diagram):

$$y_{u,sb} = \left(\frac{0.8-\nu}{0.8+\rho_m}\right)h \quad (3.4)$$

3.2.3. Parallel wall – In plane shear strength

The shear strength of the single parallel wall is given by (Trombetti *et al.*):

$$T_{Rd} = \min\left(T_{Rcd}, T_{Scd}\right) = 60\text{ t};\quad T_{Rsd} = 0.9d\frac{A_{sw}}{s} f_{ym}\left(\cot\theta+\cot\alpha\right)\sin\alpha;\quad T_{Rcd} = 0.9db\alpha_c f'_{cm}\frac{\left(\cot\theta+\cot\alpha\right)}{\left(1+\cot^2\theta\right)} \quad (3.5)$$

where: T_{Rsd} in the steel shear strength, T_{Rcd} is the concrete shear strength, $s = 10\text{ cm}$ is the step of the horizontal reinforcement, $\theta = 22°$ is the angle of the concrete truss (different values of θ have been considered in the design phase, but here omitted), $\alpha = 90°$ is the agle of the horizontal reinforcement, d is the height of the wall section, A_{sw} is the shear area: 1+1ϕ2.5mm/10cm, $f'_{cm} = 0.5 f_{cm}$ is the reduced concrete compressive strength; α_c is a coefficient equal to: $\alpha_c = 1 + \sigma_{cp} / f_{cm}$ and $\sigma_{cp} = N_{\mathrm{Ed}} / b \cdot h$ is the average compressive stress in the section.

3.2.4. Perpendicular wall – Traction strength

The traction strength of the single perpendicular wall (ignoring the concrete tensile strength) is given by:

$$N_{Rd,\perp\mathrm{wall}} = A_{\mathrm{s},\perp} \cdot f_{ym} = \left(2 \frac{412}{10} \frac{\pi \cdot 0.25^2}{4} \right) \cdot 5 \simeq 20\ \mathrm{t} \tag{3.6}$$

Where $A_{\mathrm{s},\perp}$ is the vertical reinforcement area (1+1ϕ2.5mm/10cm).

3.2.5. Parallel wall – Sliding base strength

The sliding base strength of the structure is given by (Trombetti *et al.*):

$$S_{Rd,//\ \mathrm{wall}} = \mu \cdot N_{Ed} + A_{s,runners\ //} \cdot \frac{f_{ym}}{\sqrt{3}} = 75\ \mathrm{t} \tag{3.7}$$

Where: $\mu = 0.5$ is the fiction coefficient, $A_{s,riprese\ //}$ is the area of the runners (1+1ϕ8/30).

3.3. Comparison between capacity and demand

The ratio between the traction strength of the single perpendicular wall and the corresponding demand due to a Sa = 1g (considering the compression due to the vertical loads) has been evaluated as follows:

$$\frac{N_{Rd,\perp\mathrm{wall}} + N_{\mathrm{Ed,\ static},\perp\mathrm{wall}}}{N_{\mathrm{Ed,\ seismic},\perp\mathrm{wall}}} = \frac{20+12}{45} = 0.72 \tag{3.8}$$

In the same way also the other ratios between the different capacities and the corresponding demands have been evaluated as follows:

$$\begin{aligned} &\frac{M_{y1,\ //\mathrm{wall}}}{M_{\mathrm{Ed,\ single\ //\ wall}}} = \frac{149}{56} = 2.68; \qquad &\frac{M_{Rd,//\mathrm{wall}}}{M_{\mathrm{Ed,\ single\ //\ wall}}} = \frac{181}{56} = 3.23; \\ &\frac{T_{Rd,//\mathrm{wall}}}{T_{\mathrm{Ed,\ single\ //wall}}} = \frac{60\ \mathrm{t}}{33\ \mathrm{t}} = 1.82; \qquad &\frac{T_{Rd,//\mathrm{wall}}}{T_{\mathrm{Ed,\ single\ //wall}}} = \frac{75\ \mathrm{t}}{33\ \mathrm{t}} = 2.28 \end{aligned} \tag{3.9}$$

Observing the different ratios it is clear that the weakest mechanism is the yielding for traction of the perpendicular walls.

3.4. Evaluation of the spectral acceleration corresponding to the different crisis mechanism

To yield for traction the perpendicular walls the following equation must be satisfied:

$$N_{\mathrm{Ed,\ sismico,\ parete}\ \perp} = N_{\mathrm{Rd,\ parete}\ \perp} + N_{\mathrm{Ed,\ statico,\ parete}\ \perp} = 20 + 12 = 32\ \mathrm{t} \tag{3.10}$$

The yielding spectral acceleration, $S_{\mathrm{a,y}}$, is given by the ratio between the demand due to the yielding spectral acceleration, $S_{\mathrm{a,y}}$, and the demand due to $S_{\mathrm{a}} = 1g$:

$$S_{\text{a,y}} = \frac{N_{\text{Ed, sismico, parete}\perp}\left(\text{per } S_{\text{a,y}}\right)}{N_{\text{Ed, sismico, parete}\perp}\left(\text{per } S_{\text{a}} = 1g\right)} = \frac{32}{45} = 0.72g \tag{3.11}$$

The corresponding peak ground acceleration (hypnotizing an amplification factor of 2.5) is equal to 0.29g.

The same procedure has been followed to find the spectral acceleration and the corresponding PGA for the other possible crisis mechanisms, but for sake of conciseness they are here omitted.

3.5. Synthesis of the results

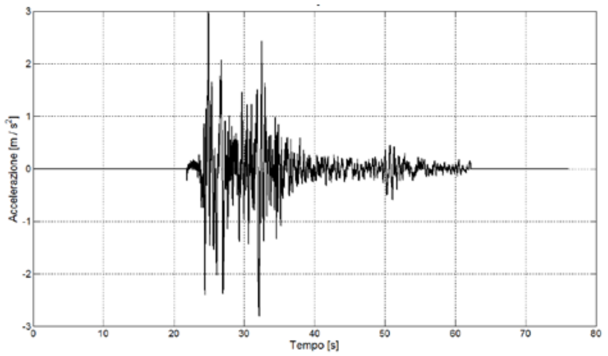
Considering a triangular distribution of the horizontal forces along the height of the structure and amplification factor equal to 2.5, the sequence of the collapse mechanism is hereafter presented:

1. yielding for traction of the perpendicular walls, for Sa = 0.72g and PGA = 0.29;
2. yielding for in-plane bending of the parallel walls, for Sa = 1.32g and PGA = 0.53;
3. reaching of the ultimate bending moment of the parallel walls, for Sa = 1.50g and PGA = 0.60;
4. in-plane shear crisis of parallel walls, for Sa = 1.82g and PGA = 0.73;
5. sliding shear at base, for Sa = 2.28g and PGA = 0.91.

4. SHAKING TABLE TEST: RESULTS

4.1. The reference seismic input and the test program

The reference seismic input (Fig. 5a) was the accelerogram recorded at Ulcinj station (Hotel Albatros, X component) during the Montenegro Earthquake of 1979 (PGA = 0.305g). In Fig. 5b the test program is presented. For the specific characteristics of the control system of the shaking table, a long calibration has been performed. For this reason, the structure has been subjected to a limited number of seismic tests. The reference input has been scaled at increasing levels of PGA and, among the different seismic tests, important sessions of white noise have been performed to keep monitoring the changes in natural frequency (Fig. 5 b).



(a)

n.	Test Type
WN1	White noise
ST1	0.05 g seismic test
WN2	White noise
ST2	0.15 g seismic test
WN3	White noise
ST3	0.50 g seismic test
WN4	White noise
ST4	1.00 g seismic test
WN5	0.3g White noise
ST5	1.20 g first seismic test
WN6	0.5g White noise
WN7	White noise
ST6	1.2 g second seismic test

(b)

Figure 5. (a) Reference seismic input; (b) Test Program

4.2. Results

4.2.1. Experimental frequencies

In Table 4.1, the values of the experimental frequencies observed during the tests are presented. Looking at Table 4.1, it is possible to observe that (i) up to the 1g seismic test:

- the experimental frequencies remain constant and equal to 10 Hz;
- the structure did not show any damage;

whereas (ii) after the 0.30g white noise (performed with the objective of cracking the concrete with high frequencies vibrations) a decrease of frequency (even though modest) has been observed from 10

Hz to 8.6 Hz. Another decrease of frequencies has been observed after the 0.5g white noise. From these results in terms of frequencies, it is possible to note that:

- there was no substantial changes in frequencies with the increasing of the seismic intensity; the structure showed a stable behavior during the whole test;
- this lack of changes in frequencies proved that the structure developed a linear elastic behavior, and for this reason the sequence of the collapse mechanisms was not observed.

Moreover, it is worth to be noted that, to obtain this frequency value with a FEM model which makes use of linear shell elements, a Young Modulus of about $E_{concrete}/2$ (roughly 15000 MPa) should be introduced to capture the experimental results. This indicates that the structure basically remained in the linear elastic field. This result is different from those that could be expected starting from the results of the previously experimental (cyclic) tests. In fact, during the previous experimental (cyclic) tests, the specimens behaved in totally cracked conditions since the beginning of the tests and for this reason the reduction of the Young Modulus to be applied to match the experimental stiffness, was found to be equal to $E_{concrete}/10$ (roughly 3000 MPa) both for simple 3mx3m walls and the two-storey H-shaped structure made of the same identical construction system. At the contrary, during the actual (dynamic) shaking-test, the structure behaved between uncracked and cracked conditions.

Table 4.1. Experimental frequencies.

Test	Experimental frequencies
Before 0.05 g test	10.0 Hz
Between 0.05 g and 0.15 g tests	10.0 Hz
Between 0.15 g and 0.50 g tests	10.0 Hz
Between 0.50 g and 1.00 g tests	10.0 Hz
Between 1.00 g test and 0.30 g white noise	10.0 Hz
Between 0.30 g white noise and 1.2 g first test	8.6 Hz
Between 1.20 g first test and 0.50 g white noise	-
Between 0.50 g white noise and 1.20 g second test	8.2 Hz
After 1.20 g second test	-

4.2.2. Cracking pattern

As far as the cracking pattern is concerned, it is worth to point out that up to the 1g test no cracks have been developed. Then, after the 0.5g white noise (in conjunction with the decrease of the frequency), very little, hairline and not spread cracks appeared especially on the top floor walls. From the 0.5g white noise to the final 1.2g seismic test the cracking pattern remain stable and almost unchanged. Figs. 6 a and c show the picture of the walls of the top floor after the 0.5g white noise, whereas, Figs. 6 b and d show the same picture but with the cracks underlined.

(a) (b) (c) (d)

Figura 6. (a) and (c)Cracking pattern after the 0.5g white noise (inside third floor) (b) and (d) same walls with cracks underlined in red.

4.2.3. Overstrengths

Form the data obtain thanks to the accelerometer placed on the structure, the following information has been evaluated: (i) the experimental total base shear, $T_{base}(t)=\sum_{i=1}^{3} m_i a_i(t)$, and the experimental total base bending moment, $M_{base}(t)=\sum_{i=1}^{3} m_i a_i(t) h_i$ .In the previous equations m_i, $a_i(t)$ and h_i are respectively the mass, the acceleration and the height of the *i-th* floor, with i = 1, 2, 3. Comparing the maximum experimental value of the total base shear and of the total base bending moment, another important difference regards the maximum strength capacity of the building under seismic loads compared to the previous cyclic tests: the shaking table tests were designed to have the collapse of the building with a lower accelerogram that the actual one utilized in the shaking table tests, but after the tests no damage was observed on the structure.

5. CONCLUSIONS

In this paper the design phase of the shaking table tests on a full-scale 3-storey structural system composed of cast-in-situ squat sandwich concrete walls have been presented. This shaking table test represents the completion of the wide experimental campaign started, in the first decade of the year 2000, with pseudo-static cyclic tests performed upon two-dimensional (3.0 m by 3.0 m) cast-in-situ sandwich squat concrete walls (with and without openings) and upon a 2-story H-shaped portion of structure, and carried on with the development of analytical formulas for the theoretical evaluation of the behavior under horizontal loads verified on the basis of the experimental results (Trombetti *et al*. 2010). In this paper also some of the preliminary results has been synthetically anticipated, as they will be accurately discussed in following research works. In detail, the shaking table test, performed on December 6th – 7th 2011, allows to verify the cellular behavior of this kind of structures but it also highlighted important differences between the results of the previously experimental (cyclic) tests and the results of the actual (dynamic) shaking-tests of the building. One of the main differences regards the stiffness: this parameter presents significant variations from the experimental results carried out previously on both simple 3mx3m walls and the two-storey H-shaped structure made of the same identical construction system. Another important difference regards the maximum strength capacity of the building under seismic loads compared to the previous tests: the tests were designed to have the collapse of the building with a lower accelerogram that the actual one utilized in the shaking table tests, but after the tests no damage was observed on the structure. These difference could be induced by the different loading modalities (pseudo-static cyclic load vs. dynamic seismic ones). For these reasons, in order to fully understand the behavior of the structure, the following additional experimental tests will be performed: (i) quasi-static cyclic horizontal loading test on the same 3-storey building and (ii) shaking table test on two dimensional single walls with the aim to comprehend the eventual influence of the type of load in the response of this kind of structures.

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