ABSTRACT
In recent years, the seismic behavior of reinforced concrete bearing squat walls structures has been the object of several research works. This paper presents a summary of the results obtained in a wide experimental/analytical/numerical correlation campaign carried out as a joint effort between the University of Bologna and the EUCENTRE labs in Pavia. This effort was devoted at the assessment of the seismic performances of structures composed of (lightly reinforced) concrete/polystyrene sandwich bearing squat walls. In this paper: (1) the results of a number of pseudo-static tests with cyclic horizontal load have been briefly recalled; (2) extensive analytical developments have been carried out to evaluate the mechanical characteristics and the seismic behavior of lightly reinforced concrete panels; (3) a comparison between the analytical and the experimental results has been performed. The comparison shows a good agreement between the experimental results and their analytical counterparts.

KEYWORDS
Seismic behavior; Cellular Structures, Lightly Reinforced Concrete/Polystyrene Sandwich Squat Walls, Experimental tests.

1 INTRODUCTION
The study of the seismic behavior of cellular structures composed of squat shear walls has been developed only recently (Hidalgo et al. 2002, Salonikios 2002, and Chai and Anderson 2005) and lies mainly in the study of the in-plane behavior of single traditional reinforced concrete squat walls under cyclic lateral loading. Therefore, in order to comprehend the actual seismic behavior of structures realized with one of these new constructive technologies, a wide experimental campaign carried out as a joint effort between the University of Bologna and the EUCENTRE labs in Pavia was developed and was widely described by the authors in a previous paper (Trombetti et al. 2010). In more detail, the experimental campaign was conducted on a number of panels made with the “Nidyon” construction systems developed by Nidyon Costruzioni will be shown. Such construction system, described in the following sections, is based on the possibility of combining structural requirements and the need for heat insulation with the use of a lightweight prefabricated (modular)
panel composed of a sheet of structural expanded polystyrene with a thickness of 60÷140 mm inserted between two grids of galvanized and electrowelded steel wire completed, in situ, by spraying concrete to form a structure composed of lightly reinforced concrete/polystyrene sandwich bearing squat walls. In this paper, on the base of the experimental tests previously mentioned, a number of experimental/analytical correlations were developed; in particular, this paper presents (a) the development of a theoretical framework capable of capturing the post-yield in-plane behavior of the single elements which composed such structural systems and (b) the interpretation and validation of the wide experimental campaign performed upon real scale bearing walls and bearing walls systems subjected to simultaneous vertical load and cyclic (in-plane) horizontal loads (to simulate the effects of the earthquake induced actions).

2 THE CONSTRUCTION TECNIQUE

The Nidyon Panel is composed of two external 40 mm reinforced concrete walls, cast in situ, and a central 60÷140 mm expanded polystyrene waved layer. This is why such panel is called “sandwich bearing panel/wall”. The internal expanded polystyrene layer just provides high levels of acoustic and thermal insulation.

![Figure 1](image)

(a) The Nidyon Panel - Central polystyrene waved layer completed with two electrowelded meshes linked together with ties. (b) The concrete/polystyrene sandwich bearing Nidyon Panel.

3 THE PECULIARITIES OF THE SANDWICH BEARING WALL AND THE REFERENCE PROVISIONS

Structures realized with the constructive system at hand are characterized by low reinforcement percentage. In fact, the standard horizontal and vertical reinforcements, electrowelded meshes, lead to low reinforcement ratio. For this reason, such structures are classified as “lightly reinforced concrete wall systems”, LLRCW systems, in according to Eurocode Provisions or as “strutture a pareti estese debolmente armate”, PEDA structures, in according to Italian Provisions (“Nuove Norme Tecniche per le Costruzioni, D.M. 14/01/2008”).

As verified by the experimental tests, the sandwich bearing walls which composed the structures in exam are characterized by:

- an in-plane behavior which is the same of that of a unique (not sandwich) solid 8 cm thickened wall (the thickness value of 8 cm comes from the sum of the 4 cm thickness of each lightly reinforced concrete layer which composed the sandwich wall) reinforced with the two electrowelded meshes effectively inserted in each 8 cm thickened lightly reinforced concrete layer;
- an out-of-plane behavior, not considered in this paper, which is characterized by high values of inertia, as described in other research works (Ceccoli and Dallavalle 2003).
4 THE THEORETICAL FRAMEWORK

4.1 Hypothesis for the development of the analytical prediction of the resistance of the concrete formation

The theoretical framework of the mechanical behavior (bending resistance, axial resistance, shear resistance and their combinations) of structural elements realized with the construction technique in exam were developed under the common (for r.c. structures) hypothesis summarized below:
1. plane sections remain plane;
2. null tensile strength of concrete (in cracked conditions);
3. perfect bonding between concrete and reinforcement;
4. stress strain relationship of the concrete modeled according to the parabola/rectangle schematization, (crushing strength \( f_c \), strain at max compressive stress \( \varepsilon_{c2} = 2^{\circ}/00 \) and strain at crushing \( \varepsilon_{cu} = 3^{\circ}/00 \), no effect of confinement upon concrete are taken into account);
5. stress strain relationship of the steel modeled according to Prandtl schematization (yield stress \( f_y \), Young modulus \( E_s \), strain at yielding \( \varepsilon_{sy} = f_y/E_s \) and ultimate strain \( \varepsilon_{su} = \varepsilon_{su,m} \)).

On the base of such hypothesis, the mechanical characteristics of the concrete formation obtainable using the above mentioned sandwich bearing panel, can be considered equal to those of an “equivalent” large lightly reinforced concrete wall, LLRCW, having an overall thickness, \( b \), of 8 cm and spread reinforcements. Such assumption is possible thanks to the effectiveness of the metallic ties which are able to avoid the buckling of the two lightly reinforced concrete layers and (ii) to guarantee a monolithic (as a unique element) behavior of the sandwich wall.

4.2 Evaluation of N-M strength domain taking into account of vertical re-bars

The ultimate bending moment \( M_u \) (for a given axial force \( N \)) of the LLRCW characterized by spread reinforcement and \( b \times h \) cross-section, can be evaluated as:
\[
\bar{M}_u = \left( f_y \cdot \rho \cdot b \cdot y_u \right) \left( \frac{h}{2} - \frac{y_u}{2} \right) + \left( f_c \cdot b \cdot 0.8(h - y_u) \right) \cdot (0.1h + 0.4y_u) + A_{s,add} \cdot f_y \cdot d (h - 2c)
\]  

(1)

where:
- \( f_y \) in the yield steel strength;
- \( f_c \) is the concrete compressive strength;
- \( \rho \) is the geometric ratio of vertical reinforcing steel;
- \( y_u \) is the position of neutral axis in ultimate conditions, which is valuable with the following relation:
\[
y_u = \left( \frac{0.8 - \frac{N}{f_c bh}}{0.8 + \frac{f_y}{f_c} \cdot \rho} \right) h
\]  

(2)

- \( \nu \) is the normalized axial force, which is valuable with the following relation:
\[
\nu = \frac{N}{f_c bh}
\]  

(3)

- \( A_{s,add} \) cross sectional area of the additional bars placed at the walls ends;
- \( c \) is the re-bar cover.
4.3 Evaluation of ultimate shear strength $V_u$ according to the Eurocode and Italian provisions

The ultimate shear strength $V_u$ (for a given axial force $N$) for a LLRCW, characterized by spread reinforcement and $b \times h$ cross-section is evaluated, according to the EC8 and EC2 provisions and to D.M. 14/01/2008, as the smaller value between following two values:

**Shear horizontal reinforcement resistance**

$$V_u = \frac{A_{hw}}{s} \cdot z \cdot f_y \cdot (\cot \theta + \cot \alpha) \cdot \sin \alpha$$

(4)

where:

- $A_{hw}$ is the cross-sectional area of the horizontal shear reinforcement;
- $s$ is the spacing of the horizontal shear reinforcement;
- $z$ is the inner lever arm;
- $\theta$ is the angle between concrete compression struts and the main tension chord;
- $\alpha$ is the angle between shear reinforcement and the main tension chord.

For the angle between concrete compression struts and the main tension chord, $\theta$, it was assumed the value of $22^\circ$, because as represented in Figure 1 such value is that which maximize the shear resistance of the specimens at hand. For the angle between shear reinforcement and the main tension chord, $\theta$, it was assumed the value of $90^\circ$, because the shear reinforcement are the horizontal bars.

**Concrete struts resistance**

$$V_u = 0.6 \cdot b \cdot z \cdot f_y \cdot \alpha_c \cdot (\cot \theta + \tan \theta)$$

(5)

In the relation above it and $\alpha_c = 1$ as recommended by EC2 and by D.M. 14/01/2008 for non-prestressed structures.

![Figure 1. Shear horizontal reinforcement resistance and concrete struts resistance in function of the value of the angle between concrete compression struts and the main tension chord, $\theta$.](image)

4.4 Ultimate sliding shear resistance according to the Eurocodes

The shear stress at interface of two concrete members filled in different times must satisfy the following expressions according to the EC8 and EC2 provisions:

$$\begin{align*}
\nu_{Ed} \leq \nu_{Rd} & \leq 0.5 \cdot V \cdot f_c \\
\nu_{Ed} & = \beta \cdot V_E / (z \cdot b) \\
\nu_{Rd} & = c \cdot f_{tot} + \mu \left( \frac{N_E}{A_c} + \rho \cdot f_y \right)
\end{align*}$$

(6)

where:

- $N_E$ and $V_E$ are the axial force and shear stress in members;
- $\mu$ and $c$ are parameters to be determined according to the roughness of the surfaces at contact (in such specific case $\mu = 0.7$ and $c = 0.45$).
\( \beta \) is the ratio of the longitudinal force in the new concrete area and the total longitudinal force either in the compression or tension zone, both calculated for the section considered;
\( \nu \) is the effectively factor which is valuable with the following relation:
\[
\nu = 0.6 \left[ 1 - \frac{f_{ck}}{250} \right] 
\]  
\( f_{ck} \) is the tensile strength of the concrete
\( A \) is the concrete cross section.

In the relations above it was assumed \( \alpha = 1 \) (where \( \alpha \) is the angle between shear reinforcement and the main tension chord).

### 4.5 Analytical evaluation of the kinematic ductility

The kinematic ductility of the LLRCW, characterized by spread reinforcement and \( b \times h \) cross-section is evaluated with the following empirical relation (Priestley et al. 2007):

\[
\mu_h = 1 + \frac{(\phi_b - \phi_u) \ell_p \left( \ell - \frac{\ell_p}{2} + \ell_{sp} \right)}{2f_u \left( \ell + \ell_{sp} \right)^2} 
\]  

Where:
\( \ell \) and \( h \) are, respectively, the length of the cantilever reinforced concrete walls, and the height of its cross section;
\( \phi_b \) is the yield curvature of the section of the wall;
\( \phi_u \) is the ultimate curvature of the section of the wall;
\( E_s \) is the Young Modulus of steel;
\( \ell_p \) is the length of the plastic hinge which is valuable with the following relation (Priestley et al. 2007):
\[
\ell_p = k \cdot \ell + 0.1 \cdot h + \ell_{sp} 
\]  
\( k = 0.2 \left( \frac{f_u}{f_y} - 1 \right) \leq 0.08 \)  

\( f_u / f_y \) is the ratio between the failure tension and the yield tension of the longitudinal reinforcements;
\( \ell_{sp} \) is the strain penetration length, that is the length of yield penetration into the foundation, which is valuable with the following relation:
\[
\ell_{sp} = 0.022 d_{bl} f_y [\text{MPa}] 
\]  
\( d_{bl} \) is the diameter of the longitudinal reinforcements expressed in mm.

### 4.6 Analytical evaluation of the equivalent damping ratio

The theoretical damping ratio of the LLRCW, characterized by spread reinforcement and \( b \times h \) cross-section is evaluated with the following empirical relation (Priestley et al. 2007):

\[
\xi_{eq} = 0.05 + 0.444 \left( \frac{\mu - 1}{\mu \pi} \right) 
\]  

Where \( \mu \) is the kinematic ductility.
5 THE TEST EXPERIENCES: DESCRIPTION AND MAIN RESULTS

5.1 Description of walls tested

The experimental campaign, carried out as a joint effort between the University of Bologna and the EUCENTRE labs in Pavia, has been performed upon the following typologies of elements (obtained using the before mentioned constructive system):

- 3 m x 3 m square sandwich wall with no openings (Panel Type A, schematized in Figure 2 (a));
- 3 m x 3 m square sandwich wall with a 1 m x 1m square central opening (Panel B, schematized in Figure 2 (b));
- a two storey H shaped substructure, (Substructure C, schematized in Figure 2 (c)).

The choice of these geometrical characteristics of the tested specimens is linked, from one hand to the necessity of investigating the seismic behavior of the single non-conventional lightly reinforced concrete/polystyrene sandwich squat wall with or without opening and, from the other hand to the necessity of investigating the seismic behavior of a representative two-story building portion.

![Figure 2. Tested elements typologies: (a) Panel Type A, (b) Panel B, (c) Substructure C.](image)

All the tested Panels (either type A and B) and the Substructure C are realized with 180 mm overall thickness Nidyon Panels (central 100 mm expanded polystyrene waved layer + two external 40 mm reinforced concrete layers). The materials prescribed for the tested elements are the typical used Nidyon materials, and in particular they are:

- concrete with $R_{ck}=30$ MPa;
- galvanized steel with low carbon content and breakage tension of $f_{tk}=700$ MPa, classified as “C7D”;
- enhanced adherence steel reinforcement with breakage tension of $f_{tk}=450$ MPa (B450C) used for the connections.

All the structures were subjected to cyclic horizontal loading at increasing levels of imposed horizontal deformations (for a given constant vertical load). At each level of deformation three complete cycles (deforming the structure both in the positive and negative direction) were developed (in order to simulate the effect of seismic loads). Table 1 summarizes the vertical loads applied at the different structures tested. The panels were tested under wide range of vertical load in order to simulate the vertical load working rate of the panels under both “standard” and “heavy” conditions.

Table 1 Vertical loads applied in each test.

<table>
<thead>
<tr>
<th>Test</th>
<th>Date</th>
<th>Specimen Typology</th>
<th>Vertical Load [kN]</th>
<th>Specimen Weight [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
5.2 Results obtained

Seven tests were performed, in particular:
- four tests upon Panel Type A;
- two test upon Panel Type B;
- one test upon the Substructure C.

The repeated tests were lead to quite similar results (obviously following the different vertical applied load’s conditions).
In all the tests it was observed that the specimens developed:
- no real collapse, but just a “virtual collapse” (i.e. clearly visible lateral strength reduction of the element tested);
- high values of bearing capacity w.r.t. horizontal load (the maximum horizontal load applied is about 300 kN for Panels Type A and B and about 500 kN for Sub-structure C);
- residual bearing capacity w.r.t. the vertical loads, also when large lateral deformations were achieved;
- a stable cracking pattern which developed at the load cycles corresponding to the first/second ID imposed.
- a good degree of kinematic ductility.

6 FLEXURAL AND SHEAR STRENGTH

For each test performed the experimentally determined “virtual collapse condition” (maximum bending moment at the base and applied vertical load) were compared with their analytical counterparts developed according to Eq. (1). In particular the analytical developments were obtained considering (i) as values of the strengths of materials (steel and concrete) those obtained experimentally from tests performed upon specimens of materials employed for the realization of the sandwich bearing walls (such analytical development is called “experimental”, EX Curve) and (ii) uses as values of the strength of materials (steel and concrete) the design values of the strengths calculated following the reference Provisions (EC 2, EC8 and D.M. 14/01/2008) (such analytical development is called “design”, D Curve). All results obtained indicate that the analytical formulations are capable to capture the engineering essence of the panel resistance. Figure 4 shows selected results comparing the axial force/bending moment limit curve (a curve representing all points of collapse, as analytically evaluated) with the experimentally determined “collapse points”. It can be noted that the analytical curve and the collapse point are, in general very close. Also, the analytical previsions generally provide conservative indications.

7 KINEMATIC DUCTILITY

Analytical interpretation of the experimentally determined force/displacement envelope curves indicate that the panels tested are characterized by high level of kinematic ductility (the specimen developed maximum horizontal displacement corresponding to a kinematic ductility in the range between 8 and 10). Also the test results indicate large values of the ratio between maximum and yielding horizontal forces ($F_y/F_1$ in Figure 5). This ratio, typically indicated in literature and in the EC as $\alpha_y/\alpha_1$, was experimentally evaluated for the panels at about 1.5 to 1.7, while for typical wall
systems the EC suggest a value of about 1,2 to 1,3. These “optimal” (in terms of high system ductility) performances can be mainly ascribed to the light reinforcement ratio, the concurrent reduced working vertical loads of the concrete and the good anchorage of the reinforcement guaranteed the its central positioning. Indeed the above characteristics prevents the wall to develop a fragile collapse due to concrete crushing (as it may the case for concrete walls with a higher compressive working rate) and allows almost all the reinforcement in tension to yield (encompassing a wide portion of the panel and not just its ends).

8 EQUIVALENT VISCOUS DAMPING

Evaluation of the equivalent viscous damping, as obtained according to the Jacobsen formulation, indicate that: (a) for imposed deformations which sees the panel remaining in the elastic range (ID up to 0.3 %), an equivalent damping ratio of about $\xi_{eq} \approx 5\%$ seems a reasonable assumption; (b) for imposed deformation which lead the panel to develop inelastic deformations (ID larger than 0.4 %), an equivalent damping ratio of about $\xi_{eq} \approx 10\%$ seems a reasonable one. Figure 6 (a) shows the equivalent viscous damping $\xi_{eq}$ determined on the basis of the experimental results obtained for the seven tests performed as a function of the inter-storey drift. Also, as illustrative example, a load cycle is represented in Fig. 6 (b).
9 CONCLUSIONS

This paper presented the results of an exhaustive experimental campaign performed upon structural systems composed of lightly reinforced concrete panels (LLRCW) obtained using a peculiar construction technique. This technique sees both the insertion of a large amount of horizontal reinforcement to prevent shear failure, as well as a self imposed maximum vertical load, to prevent a brittle failure of the panels in compression under bending. The results obtained indicate that these panels do show large values of kinematic ductility. Also, the comparison between the experimentally determined strength of the panels with their analytical counterparts (specifically developed by the authors, starting from basic hypothesis and principles of the functioning of r.c members, i.e. not following an empirical approach) do show a high level of agreement. This gives confidence that the analytical tools here developed for the seismic design of such elements can be successfully used for the actual seismic design of building structures.

10 REFERENCES

Salonikios, T. N. [2002], “Shear strength and deformation patterns of R/C walls with aspect ratio 1.0 and 1.5 designed to Eurocode 8 (EC8)”, Engineering Structures 24, 39-49.
Ceccoli C., Dallavalle G., 2003. “Metodo costruttivo antisismico NIDYON per la realizzazione in opera di strutture a pareti portanti anche in combinazione con elementi strutturali tradizionali, Relazione tecnica illustrativa”.