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RESULTS OF "PSEUDO-STATIC" TESTS WITH CYCLIC HORIZONTAL LOAD ON R.C. PANELS MADE WITH WOOD-CONCRETE CAISSON BLOCKS

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ABSTRACT :

In recent years, the seismic behaviour of reinforced concrete bearing panels structures has been the object of several research works. This paper presents a summary of the results obtained in a wide experimental/analytical correlation campaign carried out as a joint effort between the Università di Bologna and the EUCENTRE labs in Pavia. This effort was devoted at the assessment of the seismic performances of lightly reinforced concrete bearing panels. The panels are made using leave in place wood-concrete caisson blocks. In order to obtain a correct characterization of seismic behavior (stiffness, strength, ductility) of such panels, a number of tests were performed both upon two dimensional (3.0 m by 3.0 m) panels and upon 6 meter tall H shaped substructure. In the experimental tests a number of horizontal loading cycles were imposed to the structures, while the vertical load was kept constant The results obtained show a good agreement between the experimental data and their analytical counterparts. Also a high (cinematic) ductile behavior of the panel was observed, with vertical re-bars loaded up to yielding (it was never observed a failure in shear). The maximum horizontal load loads applied to the structural systems were found to be always comparable in magnitude (and often wide superior) to the applied vertical loads. All systems tested showed a residual bearing capacity w.r.t. the vertical loads, also when large lateral deformations were developed and the lateral stiffness had undergone substantial reductions. In all test the panels were capable to sustain repeated cycles at maximum deformation.

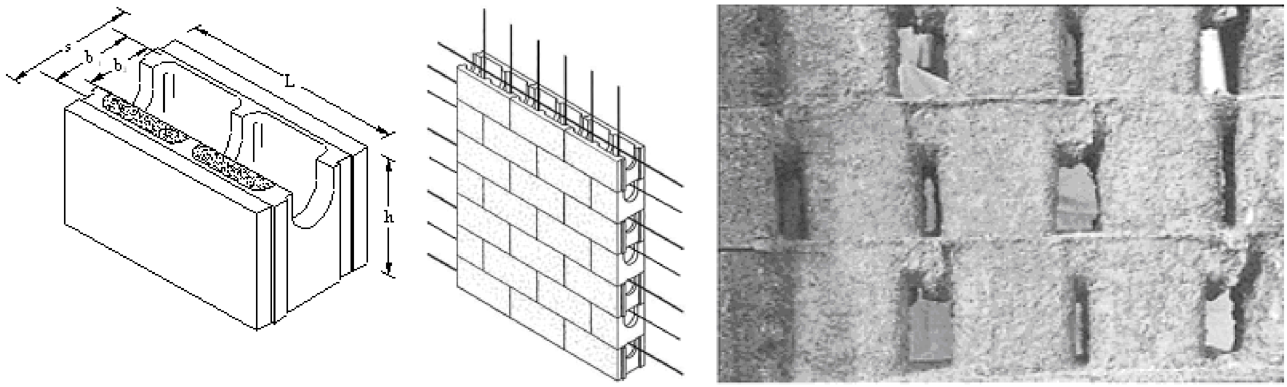
KEYWORDS: pseudo-static tests, r.c. panels, wood-concrete caisson blocks, seismic behavior, ductility

1. INTRODUCTION

Building structures which make use of structural systems obtained filling with concrete hollow bricks (which act as a leave in formwork) have been made for decades in northern Europe (mainly in Germany, Austria and Belgium). This method of construction leads to the realization of "Large Lightly Reinforced Concrete Walls" (LLRCW, as defined in both Eurocode 2 and Eurocode 8). Overall, as far as it is concerned the use of this construction technique to obtain concrete walls structural systems subjected mainly to vertical loads (i.e. horizontal loads of limited magnitude), a wide theoretical, experimental and applicative knowledge is available.
Buildings made using LLRCW have shown good behavior and high strength capabilities under strong seismic events, for example in Montenegro (Fajfar et. al 1981) and in Cile (Wallace and Moehle 1992). However, the issue of earthquake resistant design for building structures constructed following the above technique has not been fully investigated yet.
This paper presents (a) the development of a theoretical framework capable of capturing the post-yeld behavior of such structural systems and (b) the interpretation and validation of a 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 TECHNIQUE

The wall elements tested in the experimental campaign, were obtained using the specific construction techniques described in the following. The constructive system is composed of formwork blocks (or, “shuttering”) in mineralized wood (a mix of compressed crushed mineralized wood with water and cement slurry) which are currently produced in conformity with EU standard PrEN 15498:2006 (see Figure 1). This material provide high levels of acoustic and thermal insulation.



(i) (ii) (iii)

Figure 1 (i) picture and horizontal section of the typical formwork block, (ii) an ensemble of block assembled in order to create a structural wall and (iii) a picture of a 140mm thick wall without the external and internal shuttering (grid type structural pattern).

When the hollows of the blocks are filled with appropriate (a) steel reinforcement and (b) concrete, it is possible to obtain a reinforced concrete bearing walls (also referred to as “concrete formation”) of various characteristics, depending upon the type of blocks, reinforcement and concrete used in the construction. The typical construction sequence can be summarized as follows:

- after the completion of each horizontal layer of blocks, the basic horizontal reinforcement is inserted in the system by placing a single horizontal bar of relatively small diameter ($\phi 8 \div \phi 10$) at the bottom of half moon indent of the blocks (see Figs. 1i and 1ii);
- after all the blocks are assembled together (to create the complete formwork) the basic vertical reinforcement is inserted in the system by placing a single vertical bar of relatively small diameter ($\phi 8 \div \phi 12$) at the center of each vertical hole (each block is characterized by two vertical holes as shown in Figure 1i and 1ii);
- after the insertion of all reinforcing bars (in addition to the “basic” reinforcement, typically additional bars are inserted around the openings and at both panel ends) the formwork thus obtained is filled with concrete of appropriate characteristics (typically concrete with $R_{ck} > 25$ N/mm^2).

After curing, the structure obtained (referred to in the following as the “structural concrete formation”) is a concrete bearing wall with (horizontal and vertical) steel reinforcement mesh positioned in its center plane and characterized by a regular pattern of small horizontal holes (see Figure 1 iii). All bearing walls are joined together with appropriate re-bars in order to obtain a cellular network of structural walls capable of allowing a “box behavior” of the structural system (i.e. all horizontal actions can be taken by each wall through an “in plane” action). For this reason, the theoretical framework presented in the following focuses mainly on in plane bending and shear strength of a single concrete formation panel. Also, the constructive system at hand is characterized by: (1) the insertion of a large amount of horizontal reinforcement to prevent shear failure, and (2) a rigid self imposed limitation upon the maximum vertical stress in the concrete, in order to prevent the brittle failure of the concrete in compression, even under bending. The “standard” vertical and horizontal re-bars (placed in the centre-plane of the walls) lead to an area reinforcement ratio (w.r.t. the effective section of concrete) varying between 0,13 % and 0,3 %, or, in terms of weight, to about $0,25 \div 0,35$ KN of steel per cubic meter of concrete. This classify the concrete formation as “concrete structures with small amount of reinforcement" or "lightly reinforced concrete structures" (according to EC).

3. THE THEORETICAL FRAMEWORK

3.1. The equivalence criterion

The mechanical characteristics of the concrete formation obtainable using the above mentioned shuttering, can be considered equal to those of an "equivalent" LLRCW having the following geometric characteristics:
area (horizontal cross section), $A_{eq-LLRCW}$: $b \cdot L \cdot \varphi$;
area moment of inertia for in-plane bending: $b \cdot L^3 \cdot \varphi / 12$;
area moment of inertia for out of plane bending: $L \cdot b^3 \cdot \varphi / 12$;
Where $\varphi = 0,7$ is the coefficient of equivalence which allows to obtain the geometrical characteristics of the equivalent concrete wall (actual area and moment of inertia of the concrete formation), starting from the nominal dimensions (b and L) of the concrete formation.
L is the physical (nominal) length of the concrete formation; b is the physical (nominal) thickness of the concrete formation.

3.2. Hypothesis for the development of the analytical prediction of the resistance of the concrete formation

The resistance of the "equivalent LLRCW" is 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 modelled according to the parabola/rectangle schematisation, (crushing strength f_c, strain at max compressive stress $\varepsilon_{c2} = 2\,^0\!/_{00}$ and strain at crushing $\varepsilon_{cu} = 3,5\,^0\!/_{00}$, no effect of confinement upon concrete are taken into account);
5. stress strain relationship of the steel modelled according to Prandtl schematisation (yield stress f_y, Young modulus E_s, strain at yielding $\varepsilon_{sy} = \dfrac{f_y}{E_s}$ and ultimate strain $\varepsilon_{su} = \varepsilon_{su,m}$).

3.3. 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 "equivalent" LLRC wall, can be evaluated as:

$$M_{u,sb} \cong \left(f_y \cdot \rho \cdot b \cdot y_{u,sb} \right) \cdot \left(\frac{L}{2} - \frac{y_{u,sb}}{2} \right) + \left(f_c \cdot b \cdot 0.8 \left(L - y_{u,sb} \right) \right) \cdot \left(0.1L + 0.4 y_{u,sb} \right) + A_{s,add} f_y \left(L - 2c \right) \tag{1}$$

where:

ρ geometric ratio of vertical reinforcing steel (as computed w.r.t. the area of the equivalent LLRW),

$y_{u,sb} = \left(\dfrac{1 - 1.25 \cdot \nu}{1 + 1.25 \cdot \rho_m} \right) L$ position of neutral axis,

$\nu = \dfrac{N}{f_c b L \cdot \varphi}$ normalized axial force,

$\rho_m = \frac{f_y}{f_c} \cdot \rho$ ratio of mechanical reinforcing steel,

$A_{s,add}$ cross sectional area of the additional bars placed at the walls ends,

c rebar cover.

3.4. Evaluation of ultimate shear strength Vu

The ultimate shear strength V_u (for a given axial force N) for a LLRC wall, is evaluated (according to the EC8 and EC2 provisions) as the smaller value between following two values :
3.4.1 shear reinforcement resistance

$$V_u = \frac{A_{sw}}{s} \cdot z \cdot f_y \tag{2}$$

where: A_{sw} is the cross-sectional area of the shear reinforcement; s is the spacing of the stirrups; z is the inner lever arm

3.4.2 concrete struts resistance

$$V_u = 0.6 \cdot b \cdot z \cdot f_c \tag{3}$$

3.5. Evaluation of ultimate sliding shear

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):

$$\begin{cases} v_{Edi} \leq v_{Rdi} \leq 0,5 \cdot v \cdot f_c \\ v_{Edi} = \beta \cdot V_E / \left(z \cdot b\right) \\ v_{Rdi} = c \cdot f_{ctd} + \mu \cdot \left(\frac{N_E}{A_c} + \rho \cdot f_y\right) \end{cases} \tag{4}$$

where: N_E and V_E are the axial force and shear stress in members; μ e c are parameters to be determined according to the roughness of the surfaces at contact; v is a force reduction factor, depending upon f_c value.

4. THE TEST EXPERIENCES: DESCRIPTION AND MAIN RESULTS

4.1 Description of walls tested

The experimental campaign performed at Eucentre encompassed:
- The test of five 3m by 3m walls under in-plane horizontal loads (and constant vertical load).
- The test of one 3m by 3m walls with an opening (window) under in-plane horizontal loads (and constant vertical load).
- The test of a two storey H shaped structure under horizontal loads applied in the plane of the web (and constant vertical load).

Figure 2 shows the geometrical characteristics of concrete formations tested in the Eucentre lab.

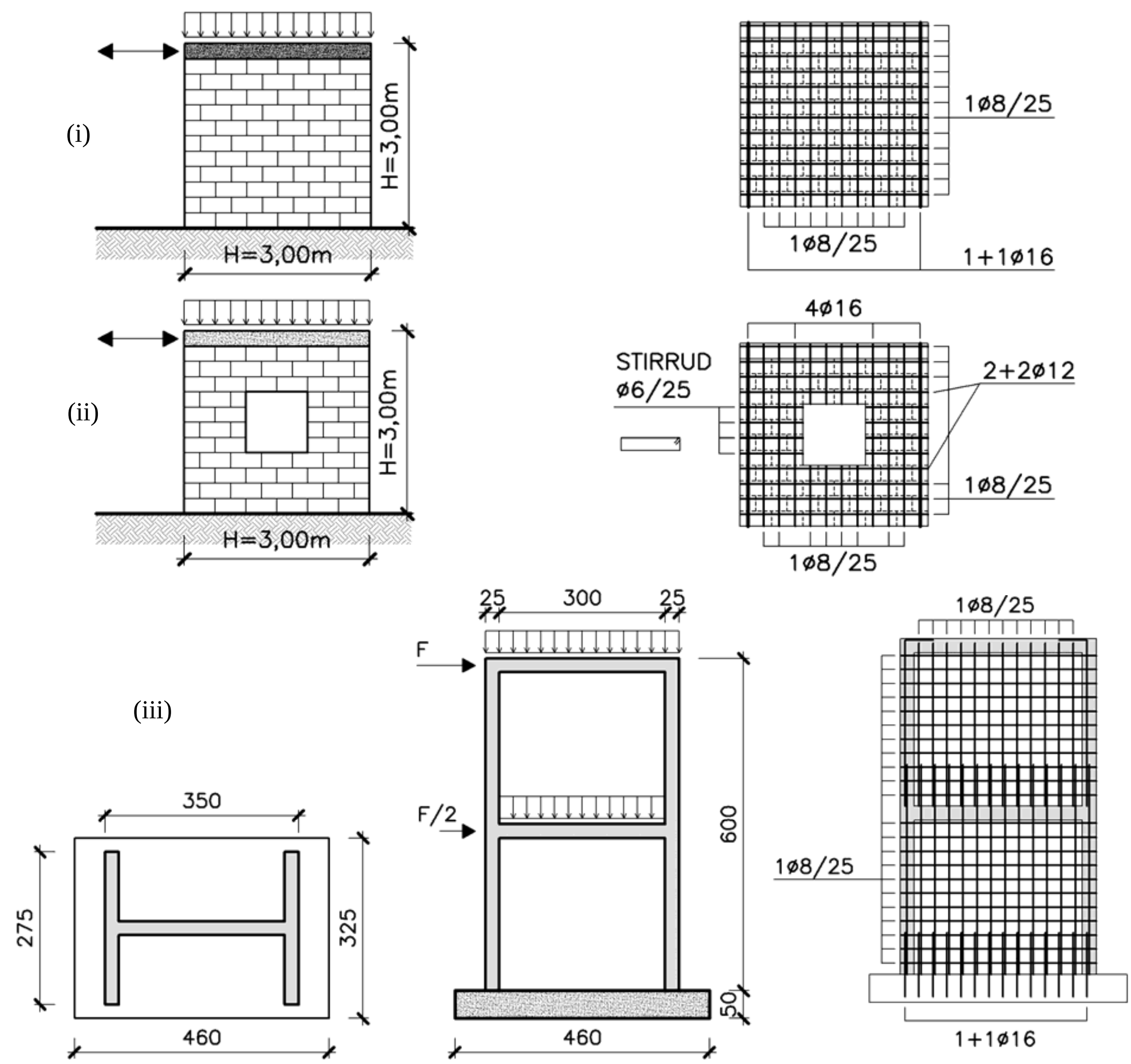


Figure 2 Geometrical and reinforcement characteristics of the tested specimen; (i) wall without opening, (ii) wall with opening and (iii) two storey H shaped structure.

For all the specimens tested, the concrete formations have a physical thickness of 140 mm and were made using concrete of class $C25/30$ ($R_{ck} = 30$ N/mm^2) and steel grade $B450C$ ($f_{yk} = 430$ N/mm^2).

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 4.1. summarizes the vertical loads applied at the different structures tested.

Table 4.1 Vertical loads applied for the different tests.

Test #	Date	Panel Type	Vertical Load [KN]	Structure weight [KN}
1	04-04-06	A	240	22,7
2	12-04-06	A	400	22,7
3	20-07-06	A	0	22,7
4	27-07-06	A	200	22,7

5	12-09-06	C	200	132,0
6	22-02-07	A	400	22,7
7	07-03-07	B	240	20,0

Note that a null vertical load was used in the test. This was selected in order to minimize the shear resistance of the concrete formation and verify the capacity design of the panel (a flexural failure is imposed). 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.

4.2 Results obtained

The lateral load/deformation diagrams obtained experimentally show that the panels are capable of large inelastic deformations. Also the experimental tests indicate that, even under large values of imposed deformations, all panels retained a residual vertical bearing capacity. Consequently, only a "virtual" collapse of the panels, could then be identified and the tests were interrupted when the panels showed a high degradation of their horizontal stiffness.
None of the tested panels showed a failure in shear. In all cases large horizontal cracks (indicating a ductile flexural behavior with extensive yelding of the vertical bars)could be identified. Also, The maximum horizontal load loads applied to the structural systems were found to be always comparable in magnitude (and often wide superior) to the applied vertical loads. Figure 3 shows selected results in terms of experimentally determined force-deformation diagrams.

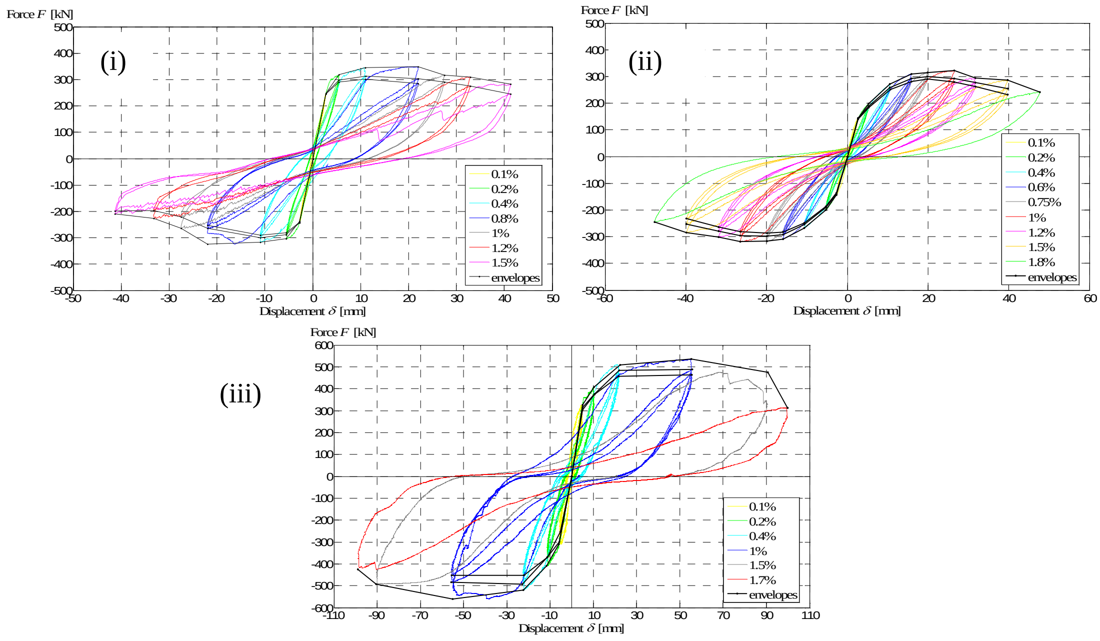


Figure 3 force-deformation relationship for (i) Test # 1 (panel type A), (ii) Test # 7 (panel type B) and (iii) Test # 5 (panel type C).

5. EXPERIMENTAL AND ANALYTICAL TEST COMPARISON STUDY

5.1 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). 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. As previously mentioned, none of tested concrete formations showed failure in shear, and the analytical predictions agree with this.

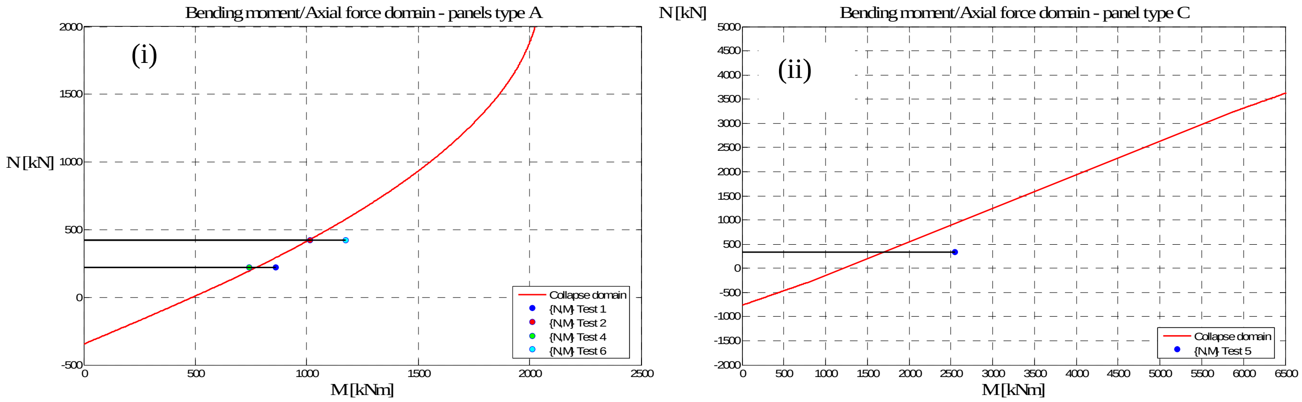


Figure 4 Bending moment/axial force domain of (i) panels type A, and (ii) panel type C

5.2 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 α_u/α_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). These results allows to conservatively suggest to use, as reduction factor (for design purposes) for the structural system at hand, the same reduction factor coefficient (“q = 3”) proposed by the Eurocode 8 for concrete wall buildings. Also, it indicates that, on the basis of appropriate additional research work for validation, suggestions for larger reduction factors may be achieved in the future.

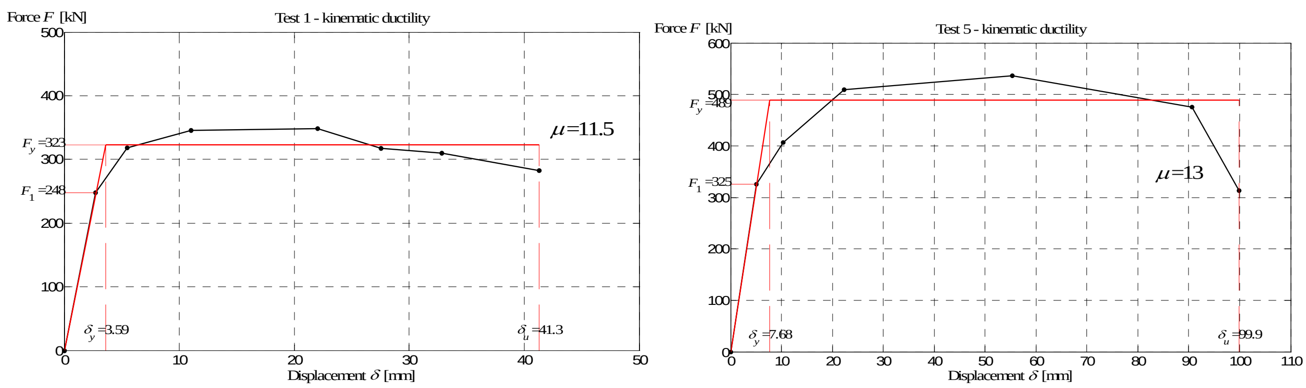


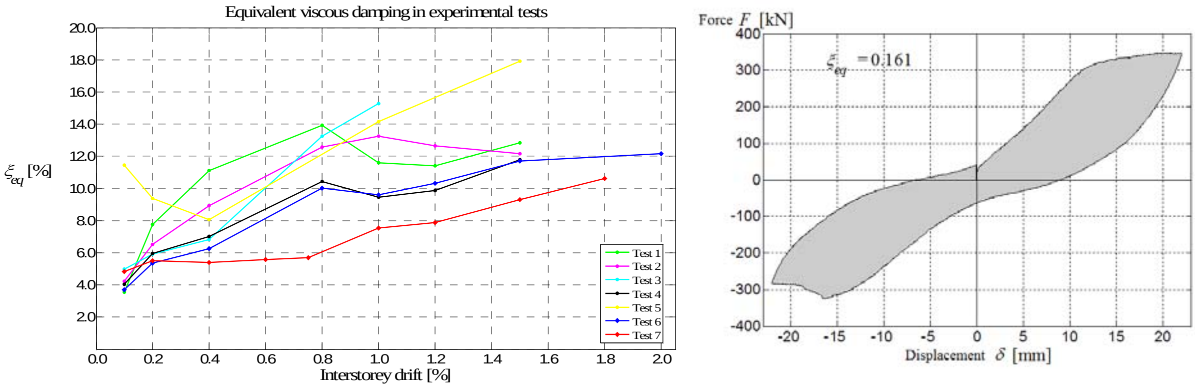
Figure 5 Kinematic ductility of tested concrete formations # 1 and 5.

5.3 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 $\xi_{eq} \cong 4\%$ 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 $\xi_{eq} \cong 12\%$ seems a reasonable one.

Figure 6i shows the equivalent viscous damping ξ_{eq} determined on the basis of the experimental results obtained for the seven tests performed as a function of the interstorey drift. Also, as illustrative example, a load cycle is represented in Fig. 6ii.



(i) (ii)

Figure 6 (i) Equivalent viscous damping of 7 panels tested and (ii) a single load cycle of a panel, showing the dissipated area.

6. CONCLUSIONS

This paper presents the results of an exhaustive experimental campaign performed upon structural systems composed of lightly reinforced concrete panels (LLRW) 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. Also the kinematic ductility observed experimentally (superior to that of equivalent reinforced concrete walls) suggests that the reduction factors (“q” factors according to the EC notation) to be used for seismic design of such structures, can be assumed to be equal or superior to those suggested by the codes for the more common RC walls.

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