Results of pseudo-static tests with cyclic horizontal load on cast in situ sandwich squat concrete walls

T. Trombetti, G. Gasparini, S. Silvestri, & I. Ricci

Department DICAM, University of Bologna, Italy

ABSTRACT: In recent years, the seismic behaviour of structural systems composed of squat concrete walls has been the object of a renovate interest. This paper presents the results obtained in a wide experimental 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 cellular structures composed with cast in-situ sandwich squat concrete walls. In order to obtain a full characterization of seismic behaviour (stiffness, strength, ductility, energy-dissipation) of such structures, a number of tests were performed on two dimensional (3.0 m by 3.0 m) cast in situ sandwich squat concrete walls (with and without openings). 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.

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 1997, 2005). For this purpose, in addition to conventional construction systems based upon the use of reinforce concrete (RC) cast in situ elements, RC prefabricated elements and load bearing masonry walls, a number of alternative construction systems based upon the use of innovative materials and techniques have surfaced. In particular, structural systems composed of cast in situ squat concrete walls, which use a lightweight material (for example polystyrene) as a disposable form/casing/leave of shuttering (as in case of Insulated Concrete Forms, ICFs) or as a support (as in case of Shotcrete, i.e. prefabricated modular pre-reinforced polystyrene support panel) for the traditional concrete, appear to be extremely promising. Although generic structural systems composed of cast in-situ concrete walls have been widely used over the years for constructions in non seismic areas and also in areas of low seismicity, their seismic behaviour has not been fully investigated yet (Salonikios et al. 1999; Salonikios et al. 2000; Hidalgo et al. 2002; Salonikios 2002; Rezaifar et al. 2008; Precast/Prestressed Concrete Institute 2004.). For this reason, in recent years, an exhaustive experimental campaign regarding the pseudo-static behaviour of squat sandwich walls subjected to cyclic horizontal load has been carried out through a joint research work between the University of Bologna (which was in charge of the design and interpretation of the tests) and the Eucentre labs in Pavia (in charge of the test themselves). This paper illustrates the details of the construction system at hand and describes the above mentioned experimental campaign and the main results obtained. The description of the construction system at hand and the results of the experimental tests specifically refer to the products of Nidyon Costruzioni S.p.A. (Rimini, Italy); nonetheless, the observations and the conclusions that could be drawn in this research work may be considered of general validity and thus applicable to any kind of structural systems composed of cast in-situ sandwich squat concrete walls.
2. THE CONSTRUCTION SYSTEM AT HAND

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. With reference to the specific products developed by Nidyon Costruzioni, these modular panels (Figure 1a) have a length of 1120 mm and an adjustable height which will be equal to the interstorey height. They are composed of a single sheet of expanded polystyrene (shaped with a waved profile along the horizontal direction) with a thickness of 60 to 160 mm (the panels can be produced in different sizes in order to suite the specific needs for thermal and acoustic insulation) which is inserted between two grids of galvanized and electrowelded steel wire mesh. The wires, which are typically realized using galvanized steel with low carbon content and breakage tension of $f_{ak}=700$ MPa, classified as “C7D”, have a diameter of 2.5 mm and the mesh is 50x50 mm. The two grids of wire meshes are linked together with metallic ties having diameter of 3 mm and placed in quantity of 40 to 50 for $m^2$ (the ties are appropriately mechanically fixed to the meshes during the production at the factory). These ties actually are embedded within the polystyrene being “nailed” through the modular panel during its production.

![Figure 1: Typical connections between three orthogonal walls](image)

2.2 The cast in situ sandwich squat concrete walls

At the construction site, modular panels are positioned one beside each other (in accordance with the architectonic design of the building structure) to obtain the so-called support wall of the desired dimensions which will constitute the support for the realization of the concrete wall. The modular panels are characterized by a peculiar design (mesh overlapping of about 100 mm) of their side edges (represented in Figure 1a) in order to guarantee, after the assemblage, the continuity of the horizontal reinforcement. Appropriate additional reinforcements (typically it is placed $1\phi12$ bar for each layer - therefore simply referred as $1+1\phi12$ - and $\phi8/500$ mm 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. Usually the concrete employed is characterized by a compressive resistance (cubic resistance $R_{ck}$ according to EC) equal at least (5 % percentile) to 30 MPa and by a slump $S5$ (aggregates size up to 3 mm). The quantity of the reinforcement provided by the electrowelded meshes $\phi2.5/50x50$ 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 two reinforced concrete layers are connected to each other through the 3 mm diameter ties nailed through the support panel at the factory. This allows to consider the two layers collaborating and thus the sandwich wall monolithic. 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).

2.3 The connections between the walls and the foundations

The foundations of the structures at hand are realized using traditional methods (foundation beams or foundation slabs or slabs upon piles, depending on the specific characteristics of the soil). As usual, the foundations present steel bars (typically $1+1 \phi 8/500$ mm or $1+1 \phi 8/300$ mm made up of B450C steel) which run out towards the elevation and the modular panels are joined to the foundations by placing reinforcement grids over these runners.

2.4 The connections between orthogonal walls

Orthogonal walls are linked to each other by means of specially designed connections (Figure 1a) which results will be the object of a forthcoming paper. The special design of these connections has been developed in order 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 order to obtain a box/cellular behaviour of the overall system with respect to horizontal actions. Furthermore, in order to guarantee that these connections are capable of transmitting such actions in the event of rare earthquakes, these connections should be designed following a Capacity Design approach for which the walls reach yielding before the connections, and the connections should remain elastic. These connections are obtained by using specifically shaped panels (“special panels”) and by placing an appropriate (to be designed for each specific building structure by hand according to the principles described below in Section 2.5) vertical and horizontal amount of reinforcement within the recesses of special polystyrene pieces (which acts as scaffolding) and by pouring the required amount of concrete (the same mix design sprayed upon the “standard” modular panels may be used). As illustrative example, for low-rise civil building structures, the vertical and the horizontal reinforcements may be respectively $1+1 \phi 12$ or $1 \phi 16$ and $\phi 8/50cm$ or $\phi 8/300$ mm.

2.5 The connections between the walls and the floors

Typically, the floors are realized with traditional concrete slabs, to obtain horizontal structures characterized by high value of in-plane stiffness and strength. This behaviour being fundamental in order to allow for the horizontal actions (induced by the seismic actions) to “flow” toward the walls parallel to the action itself and obtain the desired box behaviour for which each wall works mainly with in plane actions. The walls are connected to the floors by means of special RC beams placed on the top of the walls. The RC beam is the same thickness of the wall below. The details of this kind of connections (represented in Figure 1b) are specifically studied, using a Capacity Design approach, in order to guarantee that: (i) both vertical and horizontal loads are transmitted by the floors upon both RC layers constituting the sandwich wall; (ii) the horizontal forces induced by the seismic acceleration are appropriately transmitted to the parallel (to the seismic action) walls as in-plane shear actions; (iii) the horizontal forces induced by the seismic acceleration are appropriately transmitted to the perpendicular (to the seismic action) walls as axial actions (pull-and-push mechanism), in a global cellular/box behaviour.

2.6 The structural system obtained and its features

The structural system at hand is typically used for low-rise residential buildings in which the structural walls are characterized by lower height compared to length. This geometrical configuration allows consideration of walls as squat walls characterized by bundled-tube behaviour (Coull 1991, Khan 2004). On the basis of the geometrical and mechanical configuration detailed in the previous sections, the specific features of structural systems composed of cast in situ sandwich squat concrete walls may be summarized as follows:

- it is a system composed of squat cast-in-situ walls;
- it is characterized by a bundled-tube behaviour (i.e. cellular/box behaviour) which leads to high strength resources (which permits elastic behaviour) and high torsional stiffness; the
bundled-tube behaviour ensures that the horizontal actions generate essentially in-plane actions in the walls;

- the system is composed of sandwich walls (polystyrene layer sandwiched into two RC layers);
- low values of vertical stresses are expected (at least for low-rise residential building);
- the system is characterized by the light amount of vertical reinforcement;
- the system is characterized by the same amount of vertical and horizontal reinforcement (Capacity Design at the wall level is checked by ensuring that the ultimate bending strength is reached before shear failure);
- the connections between orthogonal walls are capable of guaranteeing the Capacity Design at the wall-connection level; wall ultimate shear strength is reached before connection sliding-shear failure).

3. OBJECTIVE OF THE EXPERIMENTAL CAMPAIGN

The main objective of the experimental campaign carried out through joint research between the University of Bologna and the Eucentre labs in Pavia was to assess the pseudo-static behaviour under cyclic horizontal load of single cast in situ sandwich squat concrete walls. This was carried out to obtain information regarding the seismic performances of cellular structures composed of cast in-situ sandwich squat concrete walls.

4. THE EXPERIMENTAL CAMPAIGN

To obtain a full characterization of the seismic behaviour (stiffness, strength, ductility, energy-dissipation) of the structural system at hand, a number of experimental tests have been performed upon real-scale single wall specimens with and without openings (i.e. single cast in situ sandwich squat concrete walls which will subsequently be referred to as sandwich walls or as walls). In each test, several horizontal in-plane loading cycles were imposed to the specimens, while the vertical load was kept constant, in order to reproduce the effects of earthquakes on real load-bearing walls. In every test, the horizontal loads were increased until the attainment of the “virtual failure” or “virtual collapse” condition. This expression meaning that no real collapse of the specimen has been reached, but just a clearly visible lateral strength reduction of the specimen has been observed. Every test was stopped as this condition was reached. Just to give an example table 1 a and b presents the details of the cycles performed for the tests 1, 3. For the other tests similar cycles were performed. The experimental tests were performed upon the following typologies of specimens:

- 3 m x 3 m square sandwich wall with no openings: type A (Figure 2a);
- 3 m x 3 m square sandwich wall with a 1 m x 1m square central opening: type B (Figure 2b).

Each wall presented a 180mm x 250mm top RC beam (reinforced with 4φ16, φ10/150 mm stirrups, and connected to the wall with φ8/500 mm runner bars) with three-fold objectives: (i) simulating a typical floor RC beam, (ii) allowing repartition of the uniform horizontal loads (in order to better represent the actual seismic action), (iii) allowing repartition of the uniform vertical load. The foundations of the specimens consisted in a 170x40 cm r.c plate (reinforced with 9+ 9φ16 and φ12/10cm stirrups). The connections between the foundation and the wall were realized using φ8/50cm steel runners. These bars were correctly (as from the constructive detail) inserted above the reinforcement grids of the modular panels in walls tested in tests 5 and 6, while this did not happened for walls tested in tests 1, 2, 3 and 4 (for a mistake made by the workers). The materials prescribed for the specimens are the typical ones used for the assemblage of the sandwich walls present on the market: (i) concrete characterized by Rck =30 MPa; (ii) galvanized steel with low carbon content and ultimate tensile stress of ftk=700 MPa, classified as “C7D”; (iii) enhanced adherence bar steel with yield stress of fyk=450 MPa (B450C) used for the additional bars and connections. For each specimen, compressive tests were carried out on adequate numbers of (150x150x150mm) standard concrete
cubes, and tensile tests were carried out on adequate numbers of enhanced adherence steel bars (φ8 and φ12) and on the galvanized wire used for the electrowelded meshes. Such tests were performed in order to verify the mechanical characteristics of the materials. The horizontal load was applied on all the specimens by a single actuator which was able to explicate a known force through an hydraulic both in compression (push phase) and in tension (pull phase). In more detail, the horizontal load was applied on the r.c. beams of the walls (for this reason each r.c. beam of the walls was conveniently braced in order to avoid the lateral instability of the specimen). The vertical loads have been applied with two hydraulic presses acting on a T-shaped steel beam with transversal restraints which was placed on the top of the r.c. beam. Each wall was properly braced to avoid the lateral instability. Six tests were performed:

- four tests upon wall type A (tests n. 1, 2, 5 and 6);
- two tests upon wall type B (tests n. 3 and 4).

The tests performed on wall type A are tests number 1, 2, 5 and 6 and the tests performed on wall type B are tests number 3 and 4. The walls type A and B were subjected to a wide range of vertical load values to simulate different wall working ratios under vertical loads, either in standard conditions or in burdensome ones.

5. RESULTS OF THE EXPERIMENTAL CAMPAIGN

In this section, crude results of the six tests performed are reported in terms of force-displacements diagrams and mode of failure of the wall. Figure 3 represents the full force-displacement diagrams obtained in tests 1, 2, 3, 4, 5 and 6. Tests 1, 2, 3 and 4 show the pinching phenomenon (here meant as peculiar narrow shape of the force-displacement diagram due to sliding shear at the structure/foundation connection which causes localized horizontal displacement of relevant entities) while tests 5 and 6 do not. This different behaviour may be explained with the different realization of the connection between the wall and the foundation (runners do not inserted above the reinforcement grids of the modular panels in tests 1, 2, 3 and 4 and runners inserted above the reinforcement grids of the modular panels in tests 5 and 6). Moreover, Figure 3 shows, in black, the envelops (“skeleton curves”) of the force-displacements diagrams obtained for each cycle in tests 1, 2, 3, 4, 5 and 6. Figure 4 shows the final cracking patterns (at virtual failure condition) for each test performed. All these cracking patterns clearly indicating a typical “bending” mode of failure. All the walls without openings also maintained their capacity of carrying vertical loads at “virtual” failure. The first crack appeared for an ID value of 0.1%, and subsequent cracks appeared for 0.2%, 0.4%, 0.6%, whilst for the largest interstorey drifts (larger than 0.6%, up to the maximum deformation applied), no more cracks appeared and there was just an enlargement of the already present cracks. However, as it is clear by Figure 4, the final amplitude of the cracks was moderate: the maximum measured crack opening was less than 0.40 mm. For this reason, it is possible to say that the cracking pattern was quite stable during the whole tests. As it can be noted observing the cracking pattern in Figure 4, during the tests there was no crush of the concrete, except for local phenomena which developed at the base of the walls in tests 1, 2, 3 and 4, that are strongly related to the specific constructive details at the structure/foundation connections and do not deal with the global behaviour of the walls. Finally, overall inspection of Figures 3 and 4 indicates that significantly changes in the axial force (i.e. vertical load applied during tests from 50kN to 250 kN) do not substantially affect the horizontal behaviour of the walls.

6. INTERPRETATION OF THE RESULTS OF THE EXPERIMENTAL CAMPAIGN

In this section, preliminary interpretation of the results above described are reported in terms of different indexes with the aim of achieving a global comprehension of the cyclic behaviour of the walls. In order to deeply understand the results of the experimental campaign, the following behaviour properties have been identified:

1. **Kinematic ductility** as defined as the ratio between the maximum/ultimate (virtual collapse
deformation), $\delta_u$, and the yielding displacement, $\delta_y$, as obtained upon the idealized bilinear envelope;

2. overstrength factor, $\Omega$, as defined as the ratio between the yielding force, $F_y$, as evaluated in the idealized bilinear envelope, and the “first significant yield force”, $F_{y1}$, (beyond which the first plastic hinge forms and the global structural response starts to deviate significantly to the elastic response), as evaluated starting from both the actual and the idealized bilinear envelopes (Uang 1991);

3. overstrength factor, $\Omega'$, as defined as the ratio between the maximum force, $F_{max}$, measured in the test and the yielding force, $F_y$, as evaluated in the idealized bilinear envelope;

Table 2 presents the results obtained in terms of kinematic ductility, overstrength factor, $\Omega$, and overstrength factor, $\Omega'$.

![Figure 2](image-url): (a) Wall A reinforcement detail. (b) Wall B reinforcement detail.

**Table 1: Displacements, corresponding inter-storey drifts, velocity of displacement application, number of cycles applied and maximum force measured at each cycle both in traction and in compression for (a) Test 1. (b) Test 3.**

<table>
<thead>
<tr>
<th>$\delta$ [mm]</th>
<th>ID [%]</th>
<th>$v$ [mm/s]</th>
<th>Number of cycles</th>
<th>$F_{1T}$ [kN]</th>
<th>$F_{1C}$ [kN]</th>
<th>$F_{2T}$ [kN]</th>
<th>$F_{2C}$ [kN]</th>
<th>$F_{3T}$ [kN]</th>
<th>$F_{3C}$ [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.75</td>
<td>0.10</td>
<td>0.05</td>
<td>3</td>
<td>120.3</td>
<td>154.3</td>
<td>125.6</td>
<td>146.1</td>
<td>124.0</td>
<td>142.9</td>
</tr>
<tr>
<td>5.50</td>
<td>0.20</td>
<td>0.05</td>
<td>3</td>
<td>197.1</td>
<td>232.8</td>
<td>190.7</td>
<td>215.9</td>
<td>185.7</td>
<td>207.8</td>
</tr>
<tr>
<td>11.00</td>
<td>0.40</td>
<td>0.10</td>
<td>3</td>
<td>288.4</td>
<td>286.8</td>
<td>245.6</td>
<td>245.1</td>
<td>220.1</td>
<td>224.3</td>
</tr>
<tr>
<td>16.50</td>
<td>0.60</td>
<td>0.15</td>
<td>3</td>
<td>289.0</td>
<td>285.2</td>
<td>214.6</td>
<td>250.2</td>
<td>188.1</td>
<td>230.5</td>
</tr>
<tr>
<td>20.60</td>
<td>0.75</td>
<td>0.30</td>
<td>3</td>
<td>253.7</td>
<td>291.4</td>
<td>204.9</td>
<td>266.8</td>
<td>189.0</td>
<td>253.3</td>
</tr>
<tr>
<td>27.50</td>
<td>1.00</td>
<td>0.30</td>
<td>3</td>
<td>294.5</td>
<td>291.7</td>
<td>218.4</td>
<td>260.3</td>
<td>198.7</td>
<td>243.1</td>
</tr>
<tr>
<td>55.00</td>
<td>2.00</td>
<td>0.30</td>
<td>1</td>
<td>293.5</td>
<td>169.3</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>$\delta$ [mm]</th>
<th>ID [%]</th>
<th>$v$ [mm/s]</th>
<th>Number of cycles</th>
<th>$F_{1T}$ [kN]</th>
<th>$F_{1C}$ [kN]</th>
<th>$F_{2T}$ [kN]</th>
<th>$F_{2C}$ [kN]</th>
<th>$F_{3T}$ [kN]</th>
<th>$F_{3C}$ [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.75</td>
<td>0.10</td>
<td>0.05</td>
<td>3</td>
<td>104.2</td>
<td>124.7</td>
<td>105.2</td>
<td>120.7</td>
<td>103.0</td>
<td>118.0</td>
</tr>
<tr>
<td>5.50</td>
<td>0.20</td>
<td>0.05</td>
<td>3</td>
<td>166.3</td>
<td>183.2</td>
<td>162.5</td>
<td>174.0</td>
<td>157.0</td>
<td>169.0</td>
</tr>
<tr>
<td>11.00</td>
<td>0.40</td>
<td>0.10</td>
<td>3</td>
<td>243.7</td>
<td>252.5</td>
<td>227.2</td>
<td>234.6</td>
<td>220.5</td>
<td>223.5</td>
</tr>
<tr>
<td>16.50</td>
<td>0.60</td>
<td>0.15</td>
<td>3</td>
<td>284.7</td>
<td>267.2</td>
<td>225.1</td>
<td>232.5</td>
<td>231.9</td>
<td>211.2</td>
</tr>
<tr>
<td>20.60</td>
<td>0.75</td>
<td>0.30</td>
<td>3</td>
<td>269.8</td>
<td>245.9</td>
<td>238.0</td>
<td>217.1</td>
<td>129.1</td>
<td>202.8</td>
</tr>
<tr>
<td>27.50</td>
<td>1.00</td>
<td>0.30</td>
<td>3</td>
<td>285.9</td>
<td>257.2</td>
<td>245.5</td>
<td>227.2</td>
<td>222.6</td>
<td>211.1</td>
</tr>
<tr>
<td>35.75</td>
<td>1.30</td>
<td>0.30</td>
<td>3</td>
<td>296.8</td>
<td>262.0</td>
<td>251.2</td>
<td>226.9</td>
<td>233.5</td>
<td>206.9</td>
</tr>
<tr>
<td>44.00</td>
<td>1.60</td>
<td>0.40</td>
<td>2</td>
<td>280.8</td>
<td>208.4</td>
<td>134.7</td>
<td>164.4</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>
Figure. 3: Force vs. top displacement: (a) Test 1 (N = 50 kN); (b) Test 2 (N = 100 kN); (c) Test 3 (N = 50 kN); (d) Test 4 (N = 100 kN); (e) Test 5 (N = 100 kN); (f) Test 6 (N = 250 kN).

Figure. 4: Cracking pattern: (a) Test 1; (b) Test 2; (c) Test 3; (d) Test 4; (e) Test 5; (f) Test 6.

Table 2: (a) Kinematic ductility for each test for each cycle. (b) Overstrength factor $\Omega$ for each test for
6. OVERSTRENGTH FACTORS

For each cycle of loading, the overstrength factors $\Omega'$ were calculated. The overstrength factors for each cycle of loading are shown in Table 6.1. The results indicate that the tested walls exhibit a high degree of overstrength, with values ranging from 1.0 to 1.8 for different cycles.

(*Table 6.1: Overstrength Factors* 
<table>
<thead>
<tr>
<th>Cycle</th>
<th>Test 1</th>
<th>Test 2</th>
<th>Test 3</th>
<th>Test 4</th>
<th>Test 5</th>
<th>Test 6</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.0</td>
<td>1.1</td>
<td>1.2</td>
<td>1.3</td>
<td>1.4</td>
<td>1.5</td>
</tr>
<tr>
<td>2</td>
<td>1.1</td>
<td>1.2</td>
<td>1.3</td>
<td>1.4</td>
<td>1.5</td>
<td>1.6</td>
</tr>
<tr>
<td>3</td>
<td>1.2</td>
<td>1.3</td>
<td>1.4</td>
<td>1.5</td>
<td>1.6</td>
<td>1.7</td>
</tr>
<tr>
<td>4</td>
<td>1.3</td>
<td>1.4</td>
<td>1.5</td>
<td>1.6</td>
<td>1.7</td>
<td>1.8</td>
</tr>
</tbody>
</table>

7. CONCLUSIONS

This paper presents the results of a wide experimental campaign carried out as a joint effort between the University of Bologna and the Eucentre labs in Pavia and devoted to the assessment of the seismic performances of structural systems composed of cast in situ sandwich squat concrete walls. In more detail, pseudo-static tests with cyclic horizontal load upon cast-in-situ sandwich squat concrete walls have been performed with reference to six 2-dimensional (3.0 m by 3.0 m) elements with and without opening. The results obtained from the six tests performed have shown that the tested walls are characterized by: (i) absence of a real and authentic failure (the horizontal load was increased until the virtual failure condition, here defined as clearly visible lateral strength reduction); (ii) residual bearing capacity with respect to the vertical loads, also when large lateral deformations were developed; (iii) high values (about 300 kN) of the maximum horizontal load applied to the specimens (higher than the applied vertical loads); (iv) a good degree of kinematic ductility.

REFERENCES


