Seismic behavior of RC buildings with large lightly reinforced walls

Marisa Pecce
University of Sannio - Engineering Department
Benevento, Italy

Fabio Antonio Bibbò
University of Sannio - Engineering Department
Benevento, Italy

Francesca Ceroni
University of Sannio - Engineering Department
Benevento, Italy

SUMMARY:
The use of large lightly reinforced walls in buildings founded a relevant diffusion in 50-70's because of their good performances under seismic action with few damage in comparison with Reinforced Concrete (RC) framed buildings. Nevertheless, experimental information about this type of buildings is lacking as well as specific design indications in the technical codes. Moreover, it is important to study the behaviour of such a structural system under seismic actions because some innovative technologies for thermal insulation of external walls of buildings are leading to design structures made with large lightly reinforced walls on their perimeter.
This paper illustrates European code requirements on large lightly reinforced walls; then, an experimental test on a RC wall available in literature is studied in detail and is modelled by means of a non-linear Finite Element (FE) analysis. Finally, a non linear analysis of a RC building designed with large lightly reinforced walls on the perimeter and inner frames has been performed.

Keywords: lightly reinforced walls, seismic behaviour, non linear analysis

1. INTRODUCTION

Structural Reinforced Concrete (RC) walls are an efficient system for buildings that have to sustain elevate seismic actions, limit displacements, and, at the same time, be equipped with concentrated resisting elements with large dimensions in spite of widespread ones such as frames.
Structural RC walls have been diffusely used in tall buildings in order to have resistance and ductility against lateral actions, as earthquakes and wind, and, at the same time, a significant stiffness. Most of tall buildings were constructed with structural RC walls for sustaining the seismic actions and RC frames for the vertical loads; the most recent example is the Burj Khalifa building in Dubai that is 828m tall and presents inner structural RC walls aimed to provide lateral stiffness and resistance to the lateral actions represented, in this case, by the wind.
Generally, in the last decades, buildings with large lightly reinforced walls and coupled with RC frames, able to sustain the vertical loads, are spreading in a lot of country as Kirghisistan, Canada, Romania, Turkey, Colombia and Chile (Moroni, 2002). Recent analysis of some of these buildings (Pentangelo et al., 2010, Wood et al., 1991) after the earthquake of 1985 have showed a lower damage level in comparison with the RC framed ones. Moreover, these analyses have shown that the structural performances under seismic actions are function of the walls density, which is defined as the walls-to-floor surface ratio; in particular, buildings with walls density of 3-4% have shown a good performance. Also the buildings having structural walls located along the perimeter and inner RC frames fall in the category of RC buildings made with large lightly reinforced walls; this particular distribution allows to obtain high resistance and stiffness to lateral actions, and, at the same time, to have a large flexibility in the organization of the internal spaces. This is possible thanks to RC frames made of columns characterized by small sections because they have to support the only vertical loads. Many examples of such a type of buildings were built in a lot of countries especially in the years '50-

70; one of the most famous is the Santa Monica Hospital in California, which was damaged by the Northridge earthquake of 1994 and was studied in detail regarding the behavior of the outer walls (Orakcal et al., 2009).
Currently, the use of large lightly reinforced walls located along the perimeter of the building has being rediscovered again and, in particular, is becoming diffuse in Italy for small and medium-sized low-rise buildings not for structural requirements, but in order to improve the performances in terms of thermal insulation and to reduce the construction time. These constructive systems consist of formworks made of insulating materials that improve the overall thermal resistance of the building as well as serving for the construction of the walls. The market offers a wide variety of these systems, which can be divided into three main categories:

1. formwork systems made of insulating material and internal steel reinforcement partially prepared where concrete can be casted;
2. sandwich systems in which the insulating material is located between two RC panels; this solution can be achieved with prefabricated concrete panels or by spraying concrete on both sides of an insulating panel prepared with steel bars;
3. formwork blocks made of material with good thermal insulation properties, in which the steel bars are located before the concrete casting; the blocks are shaped to ensure continuity to concrete and steel reinforcement bars both in the horizontal and vertical direction;

The above solutions do not have the same problems and the same structural behavior, but the system adopting formwork made of insulating materials (case 1) surely results in RC large lightly reinforced walls and, thus, knowledge and code requirements already available in this area can be used. This system, indeed, provides the construction of RC walls, even if suitable detailing and specific controls during construction are required; conversely, in the case of sandwiches systems and formwork blocks, some aspects still need to be investigated, because the final element is not homogeneously made of reinforced concrete.
In this paper, firstly, the characteristics of the large lightly reinforced walls are synthesized in order to emphasize the differences with the so-called 'ductile walls' in terms of mechanical behavior and code requirements (Italian code NTC2008, Eurocode 8 - EC8 2004) for the seismic design. In particular, for the latter ones, reinforcement percentages and construction details much more expensive are required.
Then, some considerations about the non-linear behavior of large lightly reinforced walls are developed by means of a numerical Finite Element (FE) model using the software SAP2000.
Finally, the case study of a very simple and regular RC building, made with large lightly reinforced walls on the perimeter and inner frames, is analyzed by means of a FE model that allowed highlighting some aspects regarding both the linear dynamic analysis and the non-linear static one.

2. THE LIGHTLY REINFORCED WALLS

2.1 Code indications for design

The large lightly reinforced walls are identified by EC8 basing on some geometric requirements and on their dynamic behavior, as follows:
"*A wall system shall be classified as large lightly reinforced walls system, if, in the horizontal direction of interest, it comprises at least two walls with a horizontal dimension of not less than 4,0m or $2h_w/3$, whichever is less, which collectively support at least 20% of the total gravity load from above in the seismic design situation, and has a fundamental period T_1, for assumed fixity at the base against rotation, less than or equal to 0,5s. It is sufficient to have only one wall meeting the above conditions in one of the two directions, provided that: (a) the basic value of the behaviour factor, q_0, in that direction is divided by a factor of 1,5 over the value given in Table 5.1 and (b) that there are at least two walls meeting the above conditions in the orthogonal direction*".
In addition, in EC8 a note clarifies that for this type of wall the seismic energy is transformed into potential energy (through a temporary lifting of the structural mass) and that this energy is dissipated through the rocking of the walls.
For these walls, due to the large dimensions, the absence of connection at the base or with large transverse walls, the rotation and the formation of plastic hinges do not occur; therefore they cannot be

designed for dissipating energy in plastic hinges at the base.
The EC8 (the same is for NTC2008) gives the same behavior factor associated to uncoupled wall systems having a medium ductility class (MDC), that is 3. However, it should be noted that the behavior factor has to be corrected by a factor k_w defined as follows:

$$k_w = \begin{cases} 1{,}00 \text{ for frame and frame} - \text{equivalent dual systems} \\ 0{,}5 \le (1+\alpha_0)/3 \le 1 \text{ for wall, wall} - \text{equivalent and torsionally flexible systems} \end{cases} \quad (1)$$

$$\alpha_0 = \sum h_{wi} / \sum l_{wi} \quad (2)$$

where α_0 is the more common value of the height-to-length ratio, h_{wi}/l_{wi}, within the walls of the examined structural systems.
With regard to the hierarchy of resistance, both the Italian and the European codes provide the amplification of the shear in analogy with the walls designed under the assumption of MDC.
Finally, about the construction details, EC8 provides the following specific requirements for the steel reinforcement:
– if the acting shear is lower than the shear strength of the section without shear reinforcement, the minimum shear reinforcement ratio in the web is not required; if this condition is not satisfied, the shear reinforcement has to be calculated by a variable inclination truss model or a strut-and-tie model;
– the anchorage length of clamping bars connecting the horizontal zones should be increased;
– the vertical bars, calculated for the flexural strength, should be concentrated at the ends; moreover in these zones stirrups and hoops have to be provided according to certain limitations. In addition, at the first floor the diameter of the vertical bars should be not lower than 12mm and not lower than 10mm for the upper storey;
– the vertical reinforcement should not exceed the amount calculated for the flexural strength;
– continuous steel bars, horizontal and vertical, should be provided: (a) along all the intersections between walls and at the web-flange connections of each wall; (b) at each floor levels; (c) around the openings in the walls.
The Italian code does not provide steel reinforcement requirements for this type of walls; in conclusion, it seems to distinguish the lightly reinforced wall from the ductile ones, but, while suggests that the requirements provided for seismic actions might be not applied, conversely, suggests to adopt the same behavior factor of ductile walls with MDC.

2.2 Experimental behaviour

Numerous results of experimental tests on RC walls (Vallenas et al. 1979, Birely et al. 2008, Warashina et al, 2008) are available in the technical literature, but, generally, they are referred to specimens equipped of additional longitudinal steel reinforcement and confinement at the ends of the cross section. Conversely, there is little information about RC walls with a low percentage of reinforcement uniformly distributed; among these, the tests carried out by (Orakcal et al., 2009) are surely significant. Indeed, they were specifically designed to simulate the behavior of horizontal and vertical parts of perimeter lightly reinforced walls of buildings such as some hospitals built in California in 60 years'.
Herein the main characteristics and results of the only vertical walls tested by (Orakcal et al., 2009) have been examined. The specimens were realized in 3:4 scale compared to the walls of the actual building (width 152mm, length 1370mm, height 1220mm); the materials used in the tests had properties similar to those used at the time of building construction (approximately 30MPa for the mean compressive strength of concrete and 240MPa for the yielding strength of the steel bars); a single layer of reinforcement was used without hooks of the horizontal reinforcements at the end. The vertical walls specimens are 6 divided in 3 different types with two equal samples for each type; the 3 types differ in the value of the axial force, which was 0, 5% or 10% of the compressive strength of the section ($A_g f'_c$). The steel reinforcement is the same for all specimens and consists of longitudinal bars with diameter 13mm spaced of 330mm that are doubled at the ends of the elements where transversal bars with diameter 13mm spaced of 305mm are added too (Figure 2.1a). Therefore, the percentage of

longitudinal reinforcement is 0.23%, although at the end of the element the local percentage is slightly greater. No hooks were provided for the transversal reinforcement.

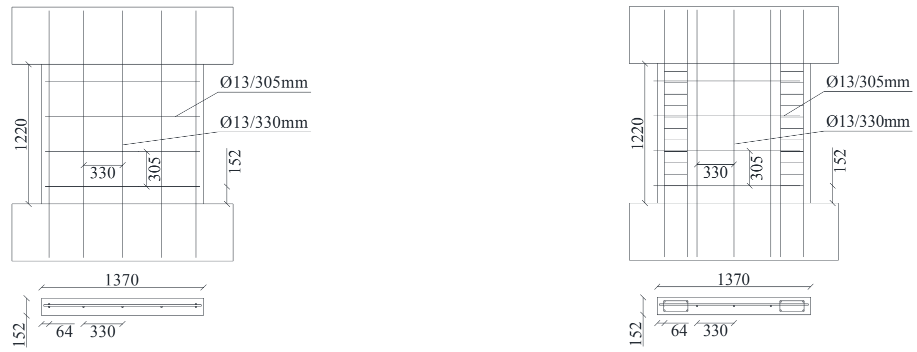


Figure 2.1. Steel reinforcement of RC wall a) tested by (Orakcal et al. 2009) b) ductile wall (measures in mm).

The tests were conducted under displacement control by applying a constant axial load with two actuators that prevented the rotation of the top of the specimen and horizontal cyclic loads with drift levels equal to 0.2, 0.3, 0.4, 0.6, 0.8, 1.2, 1.6, 2.0, and 2.4%.
The experimental measures allowed distinguishing the shear from the flexural deformation and identifying a negligible contribution of the latter one; the main aliquot of deformation is, indeed, due to sliding of the shear diagonal cracks. The final crisis was caused by the failure of the compressed concrete in the central part of the inclined strut.
The authors investigated the influence of various parameters on the shear strength of the walls through the analysis of a database of experimental tests carried out by others (Sugano, 1973, Hidalgo et al., 2002, Bard et al., 1977, Cardenas et al. 1980): percentage of longitudinal steel reinforcement, one or two layers of longitudinal reinforcement, presence of 90° hooks at the ends of the transversal steel reinforcement, percentage of steel reinforcement at the ends of the cross section, level of the normal stress. Furthermore, it was noticed that the absence of hooks for the transversal steel reinforcement at the end of the cross section did not affect the shear strength, while the presence of axial stresses caused a reduction in the lateral drift capacity of the wall. In (Wallace et al., 2008) new formulations for evaluating the residual vertical resistant load in RC walls damaged by shear are analyzed; these formulations take into account the resistant contributions to the vertical normal load given by the sliding mechanisms developed along the interfaces of the inclined shear cracks.

3. NUMERICAL MODEL AND COMPARISONS WITH EXPERIMENTAL RESULTS

3.1 The numerical model

The numerical model was developed using the software SAP2000 and implementing bi-dimensional finite elements able to take into account several layers of different materials, which are represented for this application by steel reinforcement and concrete.
The stress-strain relationship was assigned uncoupled for each main direction for both materials. The cross section of the walls was modeled as consisting of three layers that represent the concrete, the longitudinal, and the transversal reinforcement; the two layers of steel were supposed perfectly bonded to the concrete. The thickness of the three layers was defined as follows:

- the concrete layer has a thickness equal to the total thickness of the section (152mm) without subtracting the steel thickness;
- the thickness of the layer of longitudinal reinforcement is 0.35mm and has been calculated by dividing the area of steel by the central length (990mm) of the panel; at the ends of the cross section the thickness is 2.47mm according to the same procedure, evaluated for a length of 229mm;
- the thickness of the layer of transverse reinforcement is 0.44 mm.

The two-dimensional element used for modeling the concrete layer is provided with out-of-plane flexural stiffness, while the element used for modeling the steel layer is a membrane type.
A multi-axial non-linear behavior was considered for concrete, while an uniaxial non-linear behavior

was assumed for steel. The mechanical properties indicated by the authors (Wallace et al., 2009) were assumed in the model: compressive strength of concrete f_{cm}=31.4MPa and yielding strength of steel f_y=424MPa. The constitutive law of concrete in compression was implemented according to the model of (Mander et al., 1984), which allows to take into account the behavior of the confined concrete too.
The constitutive law of concrete in tension takes into account the effect of tension stiffening after cracking through the establishment of the softening branch of the σ-ε relationship in according with the model of (Vecchio et al., 1986). For the steel reinforcement, an elastic-plastic law up to failure with an ultimate strain ε_u= 12% was assumed.
Further numerical analyses were performed aimed at defining the role of the confining reinforcement at the end of the cross section of the walls; this parameter is, indeed, indicated by the technical codes as a requirement to obtain a 'ductile wall'. Therefore, the same wall previously analyzed was modified by adding vertical steel reinforcement and confining stirrups according to the indications of EC8 and NTC2008.
In particular (Figure 2.1b), in the central part of the wall, the same reinforcement percentage of the experimental test was assumed (ρ_s=0.23%), while at the ends of the cross section a higher reinforcement percentage ρ_s=1.12%, which consists of 3ϕ14, was used. The length of this over-reinforced part of the cross section, calculated in according to EC8, has an extension in plan (critical length) of 274mm; the confinement reinforcement was designed with ϕ8/100mm stirrups. Due to the presence of confining reinforcement at the ends, the constitutive law of confined concrete was introduced according to the model of (Mander et al., 1988).

3.2 The numerical results

Figure 3.1a shows the comparison between the experimental load-displacement curve and the numerical one obtained using the SAP2000 model. The comparison highlights that the numerical model approximates in a good way the initial stiffness of the experimental one and, successively, overestimates the strength in the plastic range of about 10%; however, after the peak, the numerical curve shows a resistance decay similar to that observed experimentally. Moreover, the model, even if is able to predict reliably the beginning of yielding, presents a decay of stiffness much less evident than the experimental results.

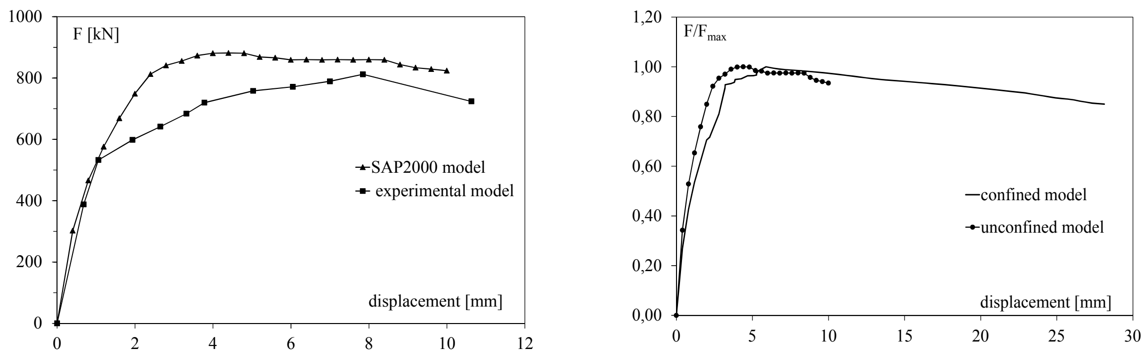


Figure 3.1. Theoretical and experimental load-displacement curves: a) wall tested by (Orakcal et al. 2006); b) comparison between confined and not confined wall.

However, the analysis of these results should take into account that the experimental tests were cyclic; therefore the experimental curve was obtained as a monotonic envelope of the effective loading cycles and, thus, is surely affected by some cyclic degradation. In addition, it should be considered that in the model the effect of the shear deformation and of the shear strength after cracking was not properly introduced. Despite these simplified assumptions, the model seems to be able to estimate reliably the overall ductility of the panel.
Figure 3.1b shows the comparison between the numerical load-displacement curves of the wall with and without the additional reinforcement at the end; the curves are normalized to their maximum load in order to evaluate only the effect of the structural details on the overall ductility of the wall. The additional reinforcement at the ends of the cross section of the wall allows increasing the maximum

displacement, which is evaluated at 85% of the maximum load, of about 3 times compared with the not confined wall. Moreover, the ductility, assessed as the ratio of the ultimate displacement to the displacement at the elastic limit, increased from 4.2 to 8.7 thanks to the presence of details at the ends.

4. NUMERICAL ANALYSIS OF BUILDINGS

4.1 The case study

In the following a RC building equipped with large lightly reinforced walls placed along the outer perimeter and interior frames is analysed. This arrangement comes from technological solutions aimed to both give a high thermal insulation capacity to the building and, simultaneously, allow the construction of RC walls. The building has a rectangular plan with dimensions of 20m x 30m and has 3 floors with height of 3m. The structure consists of a perimeter RC wall 150mm thick and of RC columns with section 300mm x 300mm at all levels and spaced of 5m. The perimeter walls have openings that determine panels with dimensions of 1m and 2m in both directions. The structure was designed considering the spectral parameters listed in Table 4.1 and following the indications given by EC8 for buildings with walls, as the columns absorb a negligible rate of the seismic actions. Due to the use of large lightly reinforced walls, the medium ductility class and the design behaviour factor 1.50 were assumed; the shape factor of the walls, k_w, was calculated with reference to the dimensions of the perimeter walls without openings. However the vertical reinforcement of the walls was realized without the ductility details, but only with bars uniformly distributed. For the walls, steel bars with diameter 12mm, in accordance with the minimum diameter suggested by EC8 of 12mm at the first storey, spaced of 200mm were used. For the columns the construction details suggested by Eurocode2 (EC2, 2004) for RC buildings not subjected to seismic actions were applied. For the same seismic actions, another RC building was designed in order to have the same dimensions, but a different structural type made of RC frames. For this second building the design was carried out in MDC with a behaviour factor of 3.12.

Table 4.1. Design spectral parameters

a_g/g [-]	F_0 [-]	T_c^* [s]	S_s [-]	C_c [-]	S_t [-]
0.197	2.389	0.374	1.417	1.452	1.000

For the second building, the dimensions of beams and columns resulted larger than those of the first one and for all the elements the construction details given by the codes for buildings in seismic areas were considered. Table 4.2 reports the main information concerning the dimensions of columns, beams, and reinforcement percentages. For both buildings the concrete is C25/30 and the reinforcing steel is B450C. In figure 4.1 the schemes of the two buildings are shown.

Table 4.2. Dimensions of elements and reinforcement percentage

	Building with wall		**Frame building**	
	columns [cmxcm]	**ρ_s (%)**	**Columns [cmxcm]**	**ρ_s (%)**
I° floor	30x30	1.40	30x40	2.24
II° floor	30x30	1.40	30x35	2.24
III° floor	30x30	1.40	30x30	2.01
	Beams [cmxcm]	**ρ_s (%)**	**Beams [cmxcm]**	**ρ_s (%)**
I° floor in x	30x25	1.26	40x25	0.75
I° floor in y	50x25	0.75	50x25 – 40x25	0.94 – 0.75
II° floor in x	30x25	0,69 - 1.26	35x25	1.07
II° floor in y	40x25	0.52 - 0.94	40x25 – 35x25	0.75 – 1.07
III° floor in x	30x20	1.57	35x20	1.35
III° floor in y	50x20	0.94	45x20 – 35x20	1.05 – 1.35

4.2 Linear dynamic behaviour

The dynamic behavior of the two RC buildings was examined in terms of:

– vibration modes;
– periods;
– participant masses.

Both structures are regular in plan and height according to the definitions of EC8. In particular, the two structures have two axes of symmetry both for the distribution of masses and the elements stiffness; thus, the mass centroid coincides with the stiffness centroid; however, in the design the accidental eccentricity was considered according to indications of EC8.
In Table 4.3 the participant masses corresponding to the fundamental vibration modes of the two structures are reported.

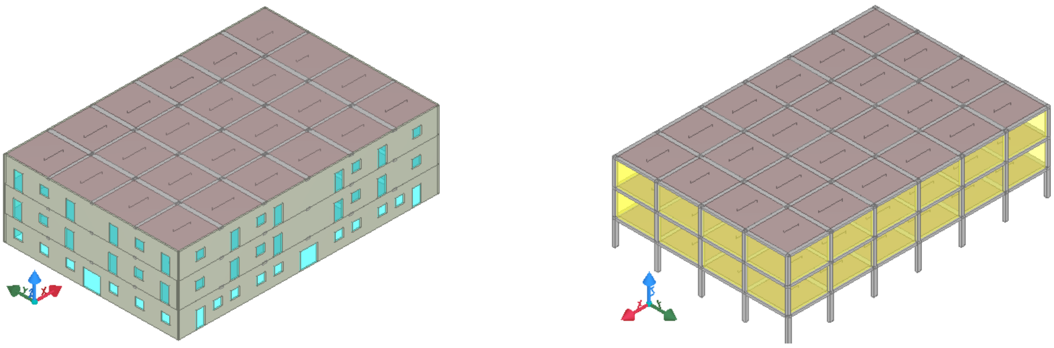


Figure 4.1. 3D model of the building: a) with walls; b)with frames.

Table 4.3. Fundamental period and participant masses

	Walls structure	Framed structure
Fundamental period dir. x	0.057s	0.624s
Participant mass dir. x	89.2%	80.8%
Fundamental period dir. y	0.071s	0.597s
Participant mass dir. y	87.6%	80.0%
Total mass	1248010 kg	1134235 kg

It is worth to notice that the periods of vibration of the walls building is significantly lower than the one of the frame building (0.06-0.07s vs. 0.6s), because the walls building has a much higher stiffness compared to the mass that is only 10% greater. Since both structures are regular, the period of vibration can be evaluated also with the approximate formulas suggested by EC8 for RC frames and walls buildings:

$$\text{frames} \quad T_1 = C_1 \cdot H^{3/4} \tag{3}$$

$$\text{walls} \quad T_1 = C_t \cdot H^{3/4} \qquad C_t = 0{,}075/\sqrt{A_c} \qquad A_c = \Sigma \cdot \left[A_i \cdot (0{,}2 + (l_{wi}/H))^2\right] \tag{4}$$

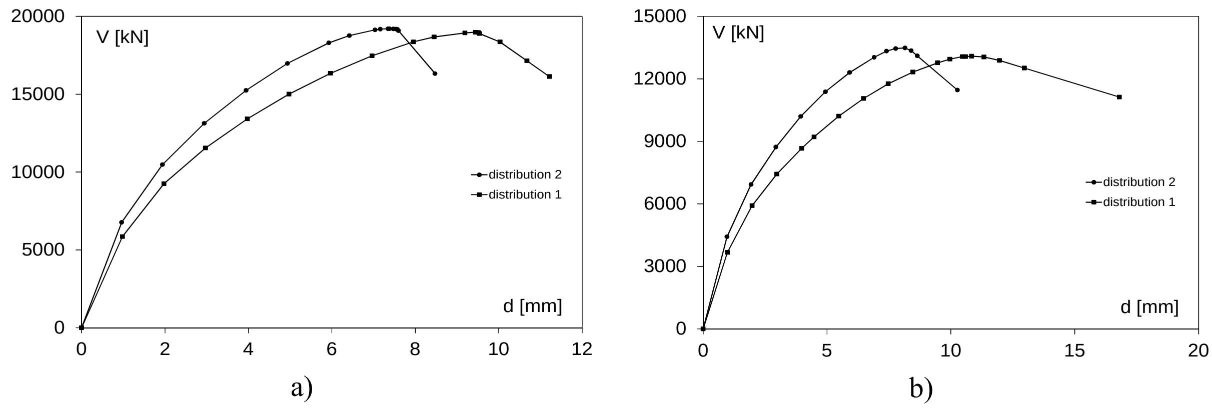
where C_1 for RC structures is equal to 0.075, A_c is the total effective area of shear walls at the first floor of the building, A_i is the effective area of the i-th shear wall at the first floor of the building, H is the total height of the building measured from the foundation or from the rigid basement, and l_{wi} is the length of the i-th wall shear at the first floor in the direction parallel to the applied forces, with the limitation that l_{wi}/H must be less than 0.9. Note that in the calculation of the areas, the openings have been excluded and l_{wi} was calculated for the entire wall with the openings.
According to Eq. (3) the period results T=0.39s, which is about 20% smaller for the framed building and approximately 6 times lower for the walls building. On the contrary, Eq. (4) gives a period T=0.127s in x direction and T=0.152s in y direction: these values are about twice the values identified by the dynamic analysis for the walls building.
In addition, the walls building was modeled also as a cantilever hollow section neglecting the presence of openings due to doors and windows distributed along the perimeter. At each floor, concentrated masses were applied in order to simulate the acting loads. The dynamic analysis of such a cantilever furnished a period of 0.071s and 0.058s for the y and x direction, respectively. It can be observed as the periods of the building given by the 2D FE model are practically coincident with those of the

cantilever in both directions; this means that the period of a RC building made of walls extended only along the perimeter could be reliably estimated with the simple model of a cantilever with hollow section.

4.3 Static non linear analysis

For the walls building designed in the previous section, a non-linear static analysis was performed too using the same model implemented with the software SAP2000 for simulating the behavior of the experimental panel described in section 3.1. The materials used are concrete C25/30 and steel B450C. The design values of strength were assumed; in particular, for steel, the yield stress is f_y=391.3MPa and the ultimate strain ε_u=6.75%, while for concrete the compressive strength is f_{cd}=14.1MPa. The RC walls were modeled as a multi-layers section made of three layers perfectly bonded representing the concrete, the longitudinal, and the transversal steel reinforcement. The thickness of the three layers was defined under the same assumptions adopted in section 3.1, as follows:
- the concrete layer has a thickness equal to the effective total thickness of the section (150mm) neglecting the presence of the steel reinforcements;
- the thickness of the two layers simulating the steel reinforcement is obtained by dividing the area of the steel bars by the length of the wall; thus, the thickness is 0.78mm for both the longitudinal and transversal reinforcement.

In the model, the non-linear behavior was considered only for the walls, while for the columns and beams an indefinite elastic behavior was assumed.
Two separate distributions of forces were considered for each direction as indicated in EC8. The first distribution of forces corresponds to a distribution of accelerations proportional to the fundamental modal shape for each direction and is applicable only if the modal shape in the considered direction has a mass participation equal, at least, to 75%.
Conversely, the second distribution of forces corresponds to a uniform distribution of forces for each direction corresponding to an uniform distribution of accelerations along the height of the building.
The results of the non-linear analyses are usually represented by load-displacement curves (capacity curves), where the load is the base total shear, V, and the displacement, d, is measured at the top of the building. In Figures 4.2 the four curves, V-d, obtained for the two principal directions and for the two forces distributions, are shown. All curves were stopped when V=0.85V_{max} and the corresponding displacement was assumed as the maximum one (EC8, 2004).



a) b)

Figure 4.2. Capacity curves for the wall building for two force distributions: a) x direction; b) y direction.

These curves are representative of the system with more degrees of freedom (MDOF) and have to be transformed in order to represent the response of a single degree of freedom system (SDOF). The transformation was performed by dividing both values of shear and displacement by the modal participation factor Γ (EC8, 2004). Then, the bilinear curves, characterized by an elastic-plastic behavior of the equivalent SDOF system were constructed. In particular, the linear branch was fixed imposing the point where V=0.6V_u, while the plastic range is characterized by the same ultimate displacement, d_u, of the effective curve.
In Figure 4.3, an example of construction of the equivalent bilinear curve is shown. A summary of the

main properties of the SDOF systems corresponding to the capacity curves obtained for each direction and for each force distribution is reported in Table 4.4

The results show that each bilinear curve furnishes a displacement capacity of the structure higher than the demand ($d^*_u \geq d^*_{max}$) with seismic safety factors ranging between 2.05 and 2.44. Moreover, the behavior factor q is due to various contributions:

$$q = R_\mu \cdot R_s \cdot R_\omega = \frac{V_e}{V_y} \cdot \frac{V_y}{V_1} \cdot \frac{V_1}{V_d} = \frac{V_e}{V_d} \tag{5}$$

where V_e is the base shear required by the seismic action if the structure remains in the elastic field, V_y is the base shear at the formation of mechanism, V_1 is the base shear at the first plasticization, and V_d is the design resistance obtained by the design spectrum (i.e. elastic spectrum reduced by the design behavior factor). Therefore, R_μ represents the ductility of the structure and assumes values ranging between 2.36 and 3.28; R_s represents the over-strength of the structure due to the energy dissipation by plasticization of materials and assumes values ranging between 1.53 and 1.57; R_ω represents the over-strength of the structure due to the design approach and assumes values ranging between 1.77 and 2.67.

The behavior factor assumes values not lower than 7.17 and, thus, it is largely greater than the value assumed in the design phase (1.5) especially for the contribution of high ductility (R_μ) and design over-strength (R_ω). Thus, the structure shows a high capacity to dissipate energy both in terms of ductility and resistance that are greater than those permitted by the European code.

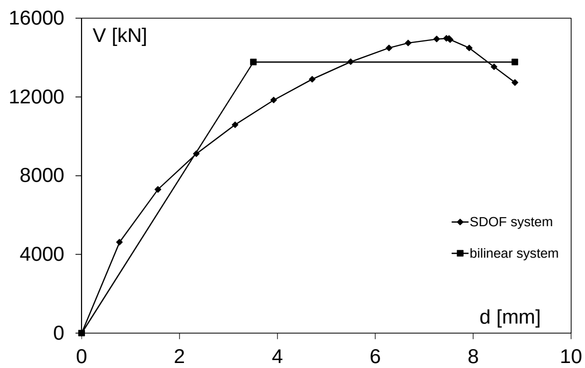


Figure 4.3. Curves V-d for the SDOF system in x direction for distribution 1.

Table 4.4. Main properties of the SDOF systems.

	direction x		direction y	
	distr. 1	distr. 2	distr. 1	distr. 2
k^* [kN/m]	$3,9 \cdot 10^6$	$4,9 \cdot 10^6$	$2,4 \cdot 10^6$	$3,1 \cdot 10^6$
F_y [kN]	13774	13975	9462	9714
m^* [t]	864	864	839	839
T_C [s]	0.543	0.543	0.543	0.543
T^* [s]	0.093	0.083	0.118	0.103
$d^*_{e.max}$ [mm]	0.926	0.737	1.492	1.127
R_μ [/]	2.52	2.36	3.28	2.59
R_s [/]	1.53	1.55	1.53	1.57
R_ω [/]	2.67	2.67	1.77	1.77
q	10.30	9.79	8.87	7.17
d^*_{max} [mm]	3.63	3.09	5.22	4.09
d^*_u [mm]	8.85	6.69	13.13	8.02

5. CONCLUSION

The analysis of the large lightly reinforced walls presented in this paper is a first step of the authors in the study of the seismic performance of such a structural type. Indeed, the FE model used to simulate the behavior of the walls does not take into account the contribution to the ductility provided by the rocking at the base, which typically occurs in the walls, and overestimates the shear stiffness and strength. However the numerical-experimental comparison of the single panel examined is quite effective in terms of stiffness, strength, and ductility. Furthermore, the FE modeling of a large lightly reinforced wall and of a ductile wall showed that the absence of the details at the ends of the cross section, which is explicitly provided for the 'ductile' walls by the codes, considerably reduces the ductility; for the examined case, the ductility is reduced to about 1/2.

Moreover, in the non-linear analyses of the walls building, which the same model for the wall behavior was assumed for, the contribution to the non-linearity of the columns and beams was not considered, since they were assumed indefinitely elastic. However, the examined case study of a RC building with large lightly reinforced walls on the outer perimeter shows a good performance in terms of behavior factor, since it resulted, at least, 7 according to the non-linear analysis.
In conclusion, RC buildings with large lightly reinforced walls on the outer perimeter seem to be a structural type characterized by a high global ductility and over-strength, though the single structural element, i.e. the wall, if constructive details at the end of the cross section are lack, shows a limited deformation capacity in the plastic field. Thus, the structural solution examined is interesting and promising, but, evidently, needs a more accurate modeling with deeper and wider numerical analyses.

REFERENCES

ACI Committee 318, 2005. Building Code Requirements for Structural Concrete (ACI 318-05) and Commentary (318R-05). *American Concrete Institute, Farmington Hills, MI".*

Birely, A., Lehman, D., Lowes, L., Kuchma D., Hart, C., Marley, K., 2008. Investigation of the seismic behavior and analysis of reinforced concrete structural walls, *The 14th World Conference on Earthquake Engineering,* Beijing, China.

Circolare 2 febbraio n.617, 2009. *Istruzioni per l'applicazione delle "Nuove norme tecniche per le costruzioni"*

Eurocode 8, 2004. Design of structures for earthquake resistance-part 1: General rules, seismic actions and rules for buildings.

FEMA 450, 2003. National Earthquake Hazards Reduction Program Reccomended Provisions for Seismic Regulations for New Building and Other Structures.

Hidalgo, P. A.; Ledezma, C. A.; and Jordan, R. M., 2002. Seismic Behavior of Squat Reinforced Concrete Shear Walls, *Earthquake Spectra*, **V. 18,** pp.287-308.

Mander, J. B., Priestley, M. J. N., and Park, R., 1984. Seismic design of bridge piers. ***Research Report No. 84-2,*** *Univ. of Canterbury, New Zealand.*

Mander J. B., Priestley M. J. N, Park R.,1988. Theoretical Stress-Strain Model for Confined Concrete, *Journal of Structural Engineering,* ***Vol.114, No.8,*** *pp. 1804-1826.*

Massone, L. M., 2006. RC Wall Shear-Flexure Interaction: Analytical and Experimental Responses, *PhD dissertation, University of California Los Angeles, Los Angeles, CA.*

Moroni M. O., 2002. Concrete shear wall construction. *University of Chile, Santiago, Chil*e.

Min.LL.PP, DM 14 gennaio 2008. *Norme Tecniche per le Costruzioni (NTC)*, Gazzetta Ufficiale della Repubblica Italiana, n.29.

Orakcal, K., Massone, L., Wallace, J., 2009. Shear strength of lightly reinforced wall piers and spandrels, *ACI Structural Journal,* ***V. 106, No. 4,*** pp. 455-465.

Pentangelo V., Magliulo G., Cosenza E, 2010 Analysis of buildings with large lightly reinforced walls, *The 14th European Conference on Earthquake Engineering,* Ohrid Macedonia .

Vallenas, Bertero, Popov , 1979. Hysteretic behaviour of reinforced concrete structural walls, ***Report no. UCB/EERC-79/20,*** *E*arthquake Engineering Research Center, University of California, Berkeley.

Vecchio, F. J., Collins, M. P., 1982. The Response of Reinforced Concrete to in-a plane shear and normal stresses, **Pubblication No 82-03,** *department of Civil Engineering, University of Toronto, Canada.*

Vecchio, F. J., Collins, M. P., 1986. The Modified Compression-Field Theory for reinforced Concrete element subjected to shear, *ACI Journal, pp. 219-231.*

Wallace, J. W., Orakcal, K., Massone, L. M., and Kang, T. H. K., 2007. St. Jude Medical Center, Fullerton, California, Horizontal Wall Segment Component Test Program—Final Report, **Report No. UCLA SEERL 2007/1**,*University of California Los Angeles, Los Angeles, CA.*

Wallace, J. W., Elwood, K. J., Massone, L. M., 2008. Investigation of the Axial load capacity for lightly reinforced wall piers, *Journal of Structural Engineering,* **Vol. 134,** pp. 1548-1557.

Warashina, M., Kono, S., Sakashita, M., Tanaka, H., 2008. Shear behavior of multi-story rc structural walls with eccentric openings, *The 14th World Conference on Earthquake Engineering,* Beijing, China.

Wood, S., 1990. Shear Strength of Low-Rise Reinforced Concrete Walls, *ACI Structural Journal,* **V. 87, No. 1,** pp.99-107.

Wood, S; Greer, W 1991. Collapse of Eight-Story RC Building During 1985 Chile Earthquake, *Journal of Structural Engineering,* **Vol. 117, No 2,** pp.600-619.