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NON-LINEAR STRUCTURAL ANALYSIS OF AN INNOVATIVE PRECAST BRACING WALL

B. Dal Lago(1), M. Muhaxheri(2), L. Ferrara(3)

(1) *Post-Doc researcher, Politecnico di Milano, Italy, brunoalberto.dallago@polimi.it*
(2) *Researcher, University of Prishtina, Kosovo, milot.muhaxheri@uni-pr.edu*
(3) *Associate Professor, Politecnico di Milano, Italy, liberato.ferrara@polimi.it*

Abstract

The paper presents the results of a campaign of numerical studies that have been carried out on an innovative precast concrete wall system conceived to work as bracing in complex multi-storey structural arrangements for different use, including residential. The dry-assembled wall is provided with lightening vertical hollow cores, while the connection between stacked walls or between the bottom wall and its foundation is made through non-lightened portions of the wall provided with mechanical connections. This technology leaves thin horizontal slots within the wall at each storey. The behaviour of a 3-storey isolated wall under both gravity and lateral forces is studied through non-linear static analyses with a damage-sensitive plate modelling. A comparison is made with reference to the equivalent cast-in-situ wall and to different reinforcement distribution within the precast wall. Furthermore, the results from dynamic non-linear analysis performed on a complete 3-storey office building provided with the investigated precast walls are presented, based on a simplified beam model of the wall. The results are compared with the experimental data from the same prototype building, tested at full scale with the pseudo-dynamic technique at the European Laboratory of Structural Assessment of the Joint Research Centre of the European Commission at Ispra (Italy) within the Safecast project.

Keywords: Precast walls; Seismic performance; Non-linear Analysis; Damage modelling; Full scale testing.

1. Introduction

The paper presents a campaign of numerical studies that have been carried out on an innovative precast concrete wall named Master®, developed within a collaboration between DLC Consulting and Politecnico di Milano. The Master® wall has been conceived to work as a brace in complex structural arrangements isolated or combined in cores by jointing adjacent walls through mechanical connections. The paper focuses on the lateral load behaviour of an isolated wall.

The conception of this structural element follows the worldwide efforts to develop smart dry-assembled precast concrete systems in accordance with the concept to maximise the industrialisation of the whole construction for better quality, construction speed, economy and reliability [1]. Seismically enhanced precast frame innovative solutions are proposed in [2].

Dry-assembled precast concrete structural walls have been and still are largely used as braces in many parts of the world. However, the typical large panel systems predominantly used in Asiatic and Eastern Europe countries, which is the actual main market for this kind of elements, are based on an assemblage performed with brittle connections, such as weld joints, which jeopardise their potential ductility and energy dissipation capacity [3,4].

Full-scale seismic experimental investigation on precast walls is scarcely reported in literature, mainly due to the need of large testing facilities. Full-scale post-tensioned precast walls have been tested by Schoettler et al. [5] and by Perez et al. [6]. A reduced-scale similar experimental test is reported in Marriott et al. [7]. Examples of full-scale testing of ordinary reinforced concrete walls are available in [8-13].

There are several techniques to numerically model the lateral behaviour of reinforced concrete walls, including macro-element lumped plasticity [14-15] and distributed plasticity [16] for beam elements, and more sophisticated techniques using shell elements. Damage-sensitive shell elements [17-19] are suitable to model reinforced concrete and can be used to predict the crack pattern and the strain distribution of a wall when subject to combined axial and lateral loading.

A full-scale prototype of three-storey multi-bay precast structure with externally placed Master® bracing walls has been assembled at the European Laboratory of Structural Assessment of the Joint Research Centre of the European Commission, located in Ispra, Italy, and subjected to seismic pseudo-dynamic testing [20], in the framework of the Safecast project (European programme FP7-SME-2007-2, Grant agreement No. 218417, 2009) [21].

A campaign of non-linear numerical analysis using damage-sensitive shell elements has been carried out at both Politecnico di Milano and University of Prishtina using a wall from the experimental prototype as a case study, with the aim to understand the behaviour of the bracing wall under lateral loading.

Further numerical analyses using non-linear beam modelling have been performed at Politecnico di Milano with the aim to match the dynamic non-linear behaviour of the experimental prototype building.

2. The bearing wall

The investigated wall is an innovative structure under several points of view: (a) the wall is strongly lightened by means of vertical voids left by steel profiles extracted from the bottom after casting, leaving hollow cores, (b) selected cavities (usually the outer) are filled with concrete and thus realises passing "columns" through the whole height of stacked wall moduli, (c) a thin horizontal construction slot is left at each wall joint, below the lightened area (Fig. 1a).

The resulting structural element is an optimised bracing wall that contains much less concrete with respect to completely filled walls having the same dimensions. Its reduced weight eases its handling and lifting. The wall optimisation, in addition to economic, lightening and processing matters, is in fact performed with reference to its combined axial-bending behaviour. As a matter of fact, its typical behaviour resembles that of an equivalent cast-in-situ wall due to the fact that a large area is available on the compression side, while the ductile reinforcement is placed at the tension side, thus maximising the inner lever arm. The flexural behaviour of the innovative and traditional walls are compared in Figg. 1a and 1b.

The purely axial behaviour of the precast wall is obviously reduced with respect to the equivalent cast-in-situ due to the lower cross-section area of the joint. The shear resistance is also lower due to the fact that the central thickness of the wall, on which its shear resistance mainly depends, is reduced from the full section to the two external concrete screeds. In general, the ductile bending behaviour may develop only if pure shear or sliding shear failures are avoided in accordance with capacity design [22].

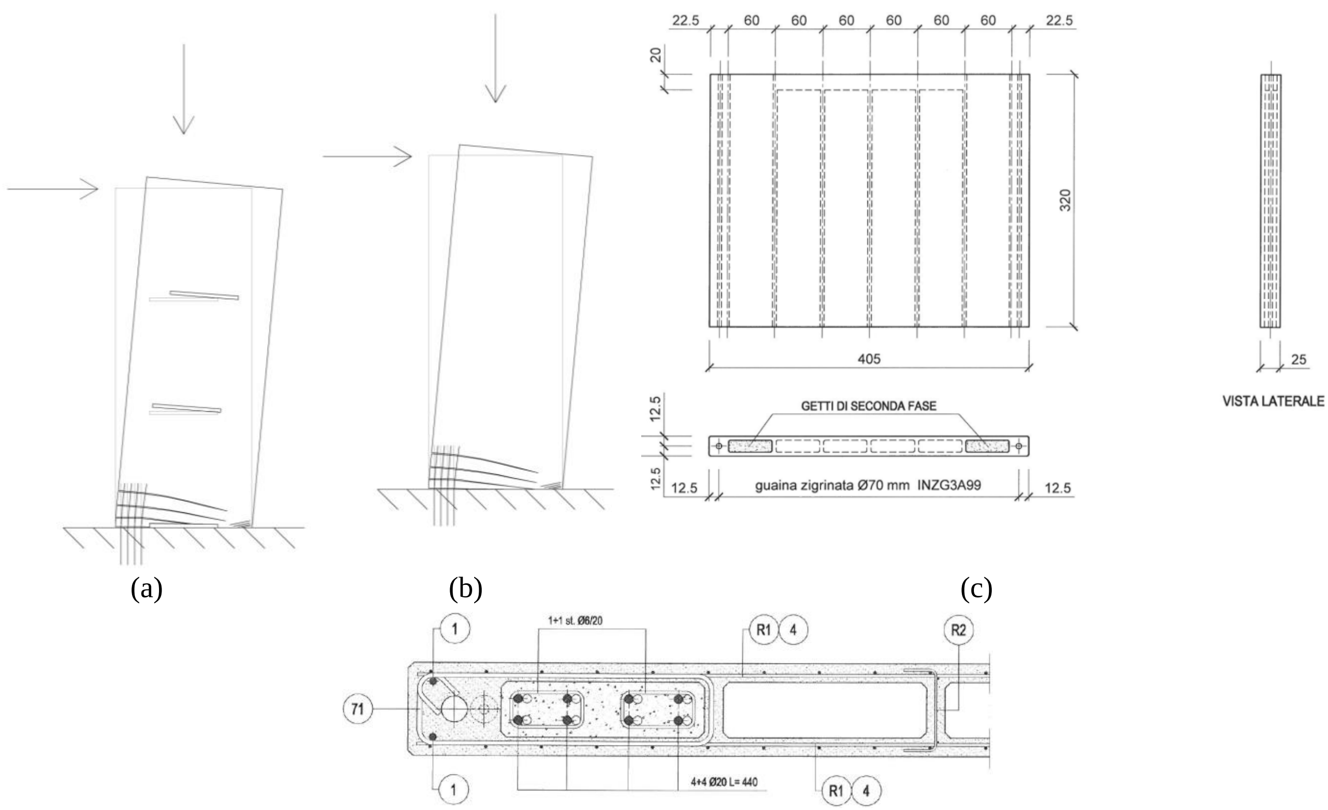


Fig. 1 – Master wall: combined axial-bending behaviour of (a) the precast wall and (b) the equivalent cast-in-situ wall, (c) shop drawings of a single modulus of the case study wall and (d) reinforcement details (courtesy of DLC consulting)

The connection between stacked walls or between the bottom wall and its foundation is made through the non-lightened portions of the wall (the “passing columns”). Two different technological solutions are hereafter highlighted: (1) the production of a fully lightened wall with the subsequent in-situ concrete pouring of those portions, after steel cage insertion that splices the longitudinal reinforcement at each storey and (2) the production of a partially lightened wall where the connection is made through pre-inserted mechanical devices, such as rebar couplers [23]. The latter solution allows for a faster construction process.

Fig. 1c shows the structural sections and front view of a wall modulus from the full-scale precast prototype tested at the ELSA laboratory within the Safecast project [21]. The technological solution (1) has been adopted for this specimen. Details of the reinforcement in the edge area are provided in Fig. 1d.

3. Static non-linear analysis on the isolated wall

Static analyses (pushover) have been carried out with the use of a structural analysis code (CAST3M [24]) able to catch the damage of concrete with a non-linear plate Finite Element Model. The mean resistance values for concrete class C45/55 have been considered.

The “crush crack” damage model was used to better capture the behaviour of concrete under multi-axial loading. It considers as a starting point the damage model developed by Mazars, maintaining the damage as scalar isotropic [17]. Later on, to improve the algorithm in order to have a stable numerical solution [18], a modified ‘crush-crack’ model has been developed by implementing the total energies dissipated in tension and in compression as stabilisers for the damage evolution functions. This model has been shown to well capture concrete behaviour under complex loading histories [18]. In later studies it has been demonstrated that this approach is promising to model also R/C members, even if their failure is shear-dominated [19]. Since its development, this model has been used to study the local behaviour of specific details of structural members. The present study explores the use of this model at structural level, through the modelling of R/C walls with different configurations.

The modified “crush-crack” damage model is implemented in the CAST3M [24] computer program and requires the constitutive relationships of the materials to be assigned as input. The stress strain relationship for concrete in compression has been adopted from the prescriptions of the fib Model Code 2010 [25], (Fig. 2a).

The tensile behaviour of C45/55 concrete (Fig. 2b) has been modelled through a stress-strain curve for the plain matrix recommended in [25] for plane modelling of concrete. The tensile strength and fracture energy are evaluated from experimentally measured compressive strength. The constitutive relationship consists of four branches; firstly the linear elastic branch up to 90% of tensile strength is defined. Afterwards, in order to consider the micro-crack formation, a further ascending branch up to tensile strength and a strain equal to 0,015% is considered. At this level crack localisation occurs. After crack localisation there is a linear softening descending down to a residual strength equal to the 15% of the tensile strength. The slope is estimated based on fracture energy release by definition of first crack opening displacement w_1. The softening continues linearly with reduced slope until zero residual tensile strength, corresponding to the ultimate crack opening displacement w_c (Fig. 2c). The non-linear stress-strain diagram for steel class B450C has been considered bi-linear elastic-hardening with nominal resistance values.

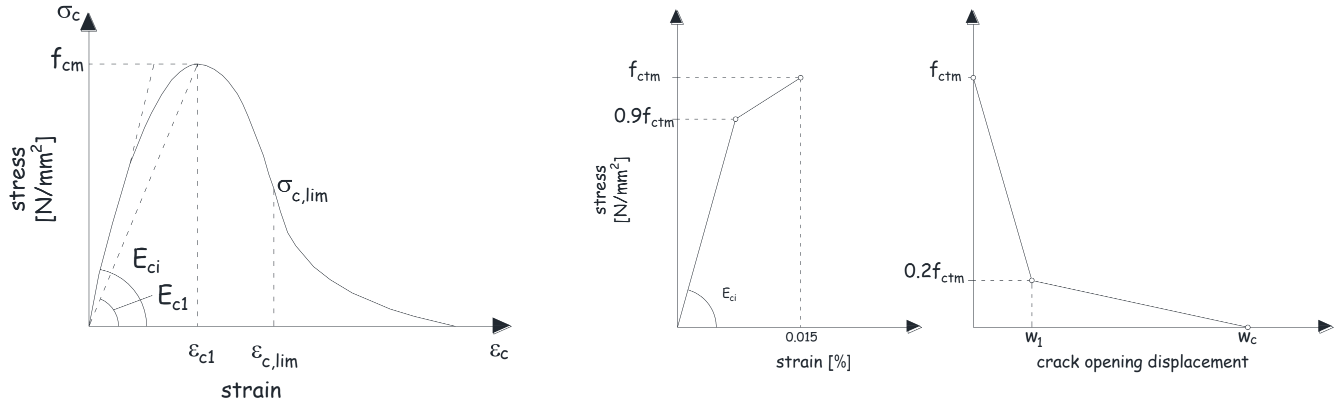
The lateral load behaviour of the precast wall was investigated. The walls are made by 3 stacked moduli (details of a single modulus are available in Fig. 1c). All walls are modelled with an I section, considering the central width equal to the sum of that of the two outer screeds (100 mm). Four types of wall are considered:

- Plain wall (cast-in-situ equivalent, Fig. 3a)
- Precast wall without shear reinforcement (Fig. 3b)
- Precast wall with shear reinforcement (Fig. 3c)
- Precast wall with shear reinforcement and de-bonding sleeves (Fig. 3d)

The plain wall and the precast wall are provided with longitudinal reinforcement only consisting of 8 Φ20 and 2 Φ16 rebars per side (Fig. 1d). The precast wall with shear reinforcement is provided with a strong Φ8@150 mm mesh per screed, in addition to 4 Φ20 rebars placed horizontally at the top and at the bottom of each wall storey modulus working as chain reinforcement. The precast wall with shear reinforcement and de-bonding sleeves is reinforced similarly to the previous, but 600 mm de-bonding sleeves are added at the bottom portion of the longitudinal rebars at each wall storey modulus. Only the plain wall is restrained in the foundation along its full depth, while all the precast walls are only restrained in correspondence of the outer passing columns. Table 1 reports the details of the longitudinal reinforcement of an edge area of the wall.

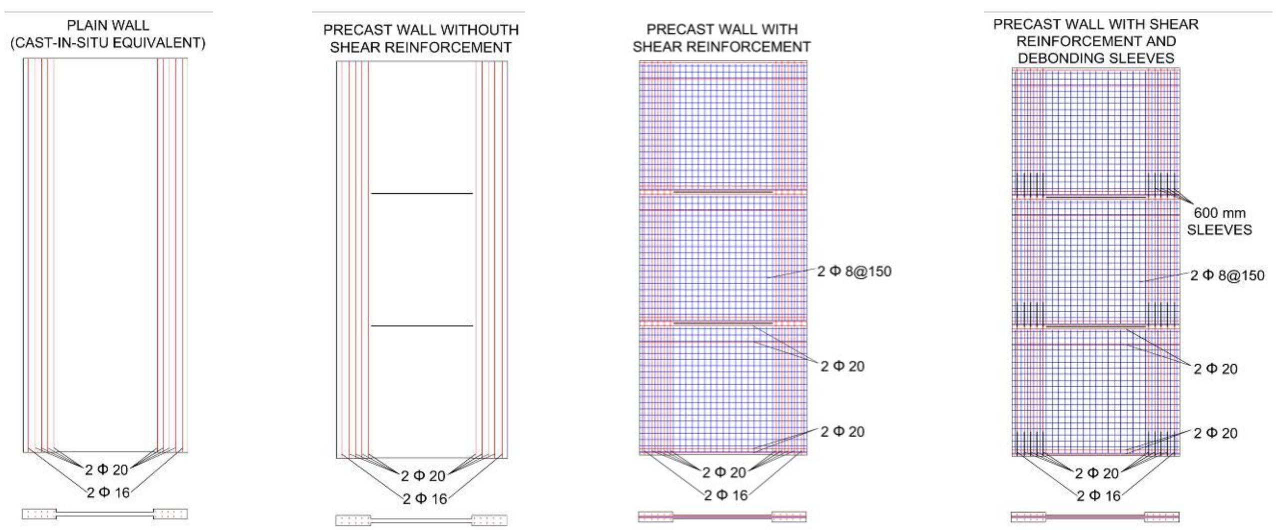
Table 1 – Longitudinal reinforcement details of each edge of the wall

Position	1	2	3	4	5
Bars	2x Φ16	2x Φ20	2x Φ20	2x Φ20	2x Φ20
Distance from the edge [mm]	125	300	450	600	750



(a) (b) (c)

Fig. 2 – Constitutive law for concrete behaviour in (a) compression, (b) tension, (c) post-cracking softening



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

Fig. 3 – Wall configurations: (a) plain wall (cast-in-situ equivalent), (b) precast wall without shear reinforcement, (c) precast wall with shear reinforcement, (d) precast wall with shear reinforcement and de-bonding sleeves

Fig. 4 shows the comparison between the pushover curves of the 4 models corresponding to the different types of wall. It can be observed that the presence of the thin slots within the precast wall only negligibly affects the lateral load behaviour. However, a relevant reduction in ductility is observed when the walls are provided with shear reinforcement. This is due to the fact that the shear reinforcement is installed within the external screeds and is not passing at the wall-to-wall or wall-to-foundation joint. The splicing with the longitudinal reinforcement occurs very soon due to the typically small diameter of the net rebars, increasing the tensile resistance of the wall central portion and concentrating the ductility demand in a small length of the longitudinal reinforcement only. The results also show that the use of de-bonding sleeves in the longitudinal rebars at the base of the wall can avoid this problem, allowing a plastic deformation in longitudinal rebars to spread over the required length, which is not influenced by any splicing with the surrounding additional reinforcement. The sleeve length may be designed in order to get the required ductility of the wall. The damage patterns of the 4 different walls are reported in Fig. 5. All the walls develop horizontal flexural cracks on the left side of the wall, which is in tension due to the bending moment, and diagonal cracks developing on the central area because of shear. The presence of the thin slots concentrates the damage in correspondence of the corners of the central area of each wall modulus. However, the damage develops in an uncontrolled fashion only in the case of the precast wall without shear reinforcement, while the shear reinforced precast walls show that the development of damage is controlled by the presence of the mesh and the chain reinforcement. In all cases, flexural cracking is well distributed along the height of the wall, leading to a better distribution of plasticity in the cases (a) and (b), whilst in the cases (c) and (d) the plastic demand is more strongly concentrated at the base of the wall. However, if for case (c) large base cracks correspond to high strain in the longitudinal steel, for case (d) they correspond to a much lower steel strain, which is distributed within the length of the sleeves. This may also be observed by analysing the stress distribution along the height of the outer tensioned rebar, plotted in Fig. 6.

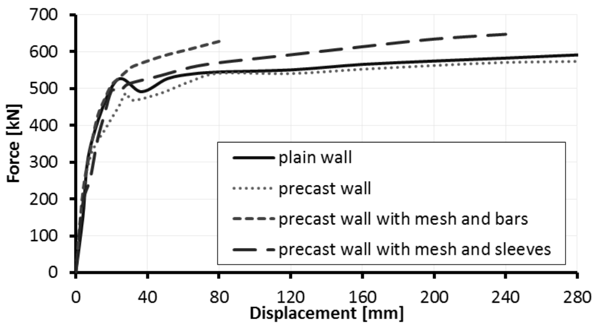
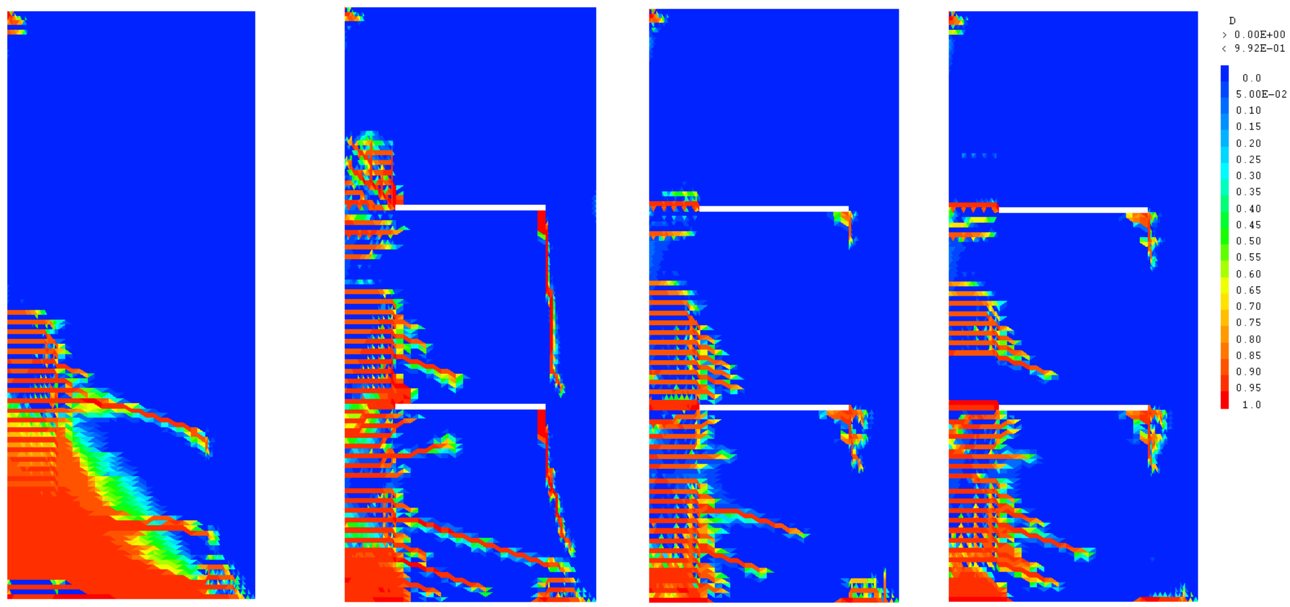


Fig. 4 – Base shear vs top displacement pushover curves



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

Fig. 5 – Damage pattern evolution of the walls at 65 mm of displacement

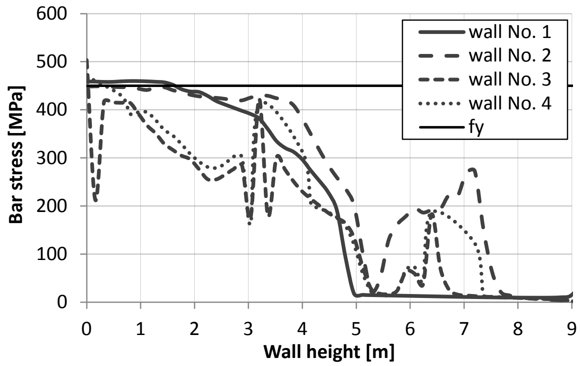


Fig. 6 – Stress distribution in the outer tensioned rebar for 65 mm of displacement

4. Dynamic non-linear analysis of a precast building with innovative bracing walls

A full-scale precast 3-storey precast building prototype has been subjected to pseudo-dynamic testing at the European Laboratory of Structural Assessment (ELSA) of the Joint Research Centre (JRC) of the European Commission located in Ispra, Italy. The mock-up is shown in Fig. 7, together with a detail of the loading apparatus. A detailed description of the mock-up and of the test results can be found in Negro et al. [20]. This experimental activity is part of the wider Safecast research project [21], a 3-year research project aimed at studying the seismic behaviour of the connections and the joints of precast structures and to develop innovative solutions for enhanced seismic resistant precast structures. The project involved, in addition to Politecnico di Milano and the JRC, University of Ljubljana, National Technical University of Athens, Istanbul Technical University, Laboratorio Nacional de Engenharia Civil of Lisbon, the national precast associations of Italy (Assobeton), Portugal (ANIPB), Turkey (TPCA) and Spain (ANDECE). Also industry and consulting companies participated to the consortium, including DLC (Milan, Italy), Halfen Italy, Yapi Merkezi and AFA Prefabrik from Turkey. The structure has been designed by DLC in accordance with the rules provided by EC8 [27] with an ULS PGA equal to 0,30g and a behaviour factor equal to 3,0. The mock-up was produced, assembled and dismantled by Pizzarotti (Parma, Italy).

Non-linear numerical finite element models have been developed using Timoshenko beam elements with distributed plasticity for the wall. The models have been built in STRAUS7 [28] environment. The code requires as input the non-linear moment vs curvature diagram of the cross section, and then, when associated to the beam element, automatically distributes it to several Gaussian points along the element length according to the level of forces and to the specific displacement time history of a seismic event. Non-linear dynamic analyses simulating the full-scale prototype structure seismic behaviour under the Tolmezzo accelerogram (see [20, 23, 26]) have been carried out with the aim to compare the numerical and experimental results.

Since the beam and floor members may be considered as simply supported, due to the spaced dowel connections which may be simulated with hinges, those members do not belong to the main lateral load resisting system of the structure. Anyway, they have been modelled with elastic elements clamped in the in-plane direction in order to simulate the diaphragm effect. A perfect clamp is also provided in correspondence of the foundation of all vertical members. The columns have also been modelled with distributed plasticity beam members, according to the same procedure as per the walls. Even if the cross-section is the same for all of them, as previously described, the non-linear moment vs curvature diagram is evaluated according to different axial loads. Even if the input masses to solve the hybrid experimental-numerical procedure consider the dead and live loads of an office building in seismic combination, the axial loads effectively acting on the mock-up columns during the test are related to the structural weights only.

The walls have the same geometry of the one numerically studied shown in Fig. 3(b) and have been reinforced with the thereby indicated longitudinal rebars. A $\Phi5/200$ mm steel net has been added in each screed.

The non-linear moment vs curvature diagrams have been calculated based on the rotational and translational equilibrium of the cross section. The non-linear material properties have been inserted considering the characteristic resistance values without safety coefficients. A tri-linear elastic-hardening-softening relationship has been defined for the reinforcing steel. Concrete is divided into unconfined for the portions outside the columns external stirrup rectangle and confined for the inner. The tensile resistance of concrete is neglected. Sargin-Saenz model [29] has been used for the unconfined concrete. For confined concrete, a modified Sargin-Saenz model has been adopted taking into account the increase of strain due to confinement of the section and considering a constant after-peak stress up to that value. The calculation of the ultimate deformation including the confinement effect has been carried out based on the formula proposed by CEB Model Code [25].

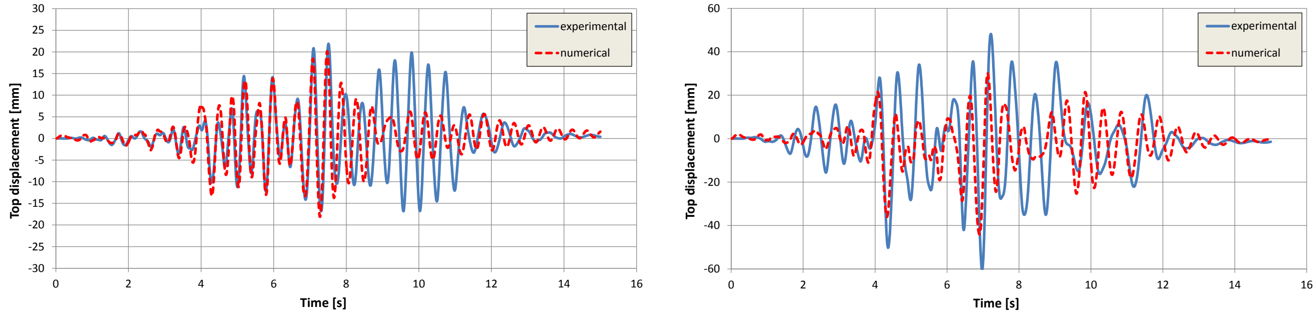
A viscous Rayleigh damping of 2% of the critical value has been applied to the first two natural vibration periods of the structure. The results of the analysis with the modified Tolmezzo accelerogram scaled at PGA equal to 0,15g are reported in Figg. 8(a), 9(a) and 10(a) with reference to top displacement, shear and overturning bending moment, respectively, and compared with the experimental results.

(a) (b)

Fig. 7 – Full-scale precast prototype building assembled in the ELSA laboratory of the Joint Research Centre (Ispra, Italy): (a) loading apparatus, (b) specimen view

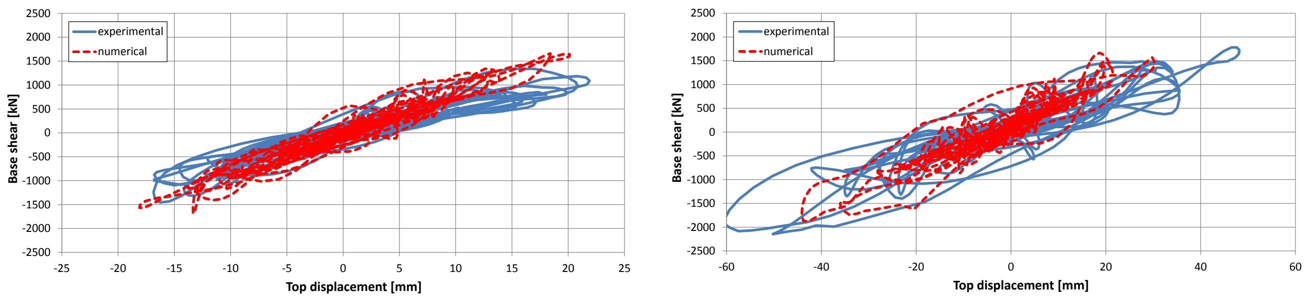
The agreement between numerical and experimental curves is remarkable for the initial part of the earthquake, with a very good matching of the displacement history up to about 8 seconds, after which a slight increase of period and a considerable scatter of the curves occur. This may be due to a weakening of the frame-to-wall connections. It has to be pointed out that the frame-to-wall connections were acting in compression only with direct contact between a recess left in the beam and the wall, ensured by pouring of a few centimetre thickness of mortar. This simple connection has been adopted due to the need to test 4 different structural arrangements with the same prototype (in addition to wall braced, hinged frame, clamped at roof frame and fully clamped frame). An inspection on the prototype showed that many connections suffered from slight damage, mainly spalling of the external concrete (Fig. 11a). The hysteresis of the structure suggests that the structure remained in elastic field and that the numerical model satisfactorily predicts the elastic stiffness of the structure.

The vibratory curves corresponding to the earthquake application scaled at a PGA of 0,30g show a larger scatter between experimental and numerical results (Figg. 8b, 9b and 10b). The trend is basically matching, but relevant differences are detected both for entity of maximum displacements and for the displacement history. The loop tendency of the results for PGA equal to 0,30g looks similar, with a sort of shift in both displacements and forces. This may be explained with the severe damaging and failure that occurred within the frame-wall connections, leading to an evolution of a pinching effect that may be observed in the experimental diagram. This is also reported in [31, 32]. When the gap is formed, around zero displacement the structure temporarily acts according to the much more flexible behaviour of the frame with hinged beam-to-column joints only up to when the gap is closed, and the corresponding increase of drift may be related to the larger shear contribution of the frame columns. The presence of confused peaks in the diagram portion around zero displacement may provide a further confirmation of this hypothesis, since it suggests a contribution of higher vibration modes which may be related to the very flexible frame behaviour. The results of the numerical analysis and the observation of the experimental hysteretic loops indicate that the walls entered the plastic field, ensuring a stable energy dissipation.



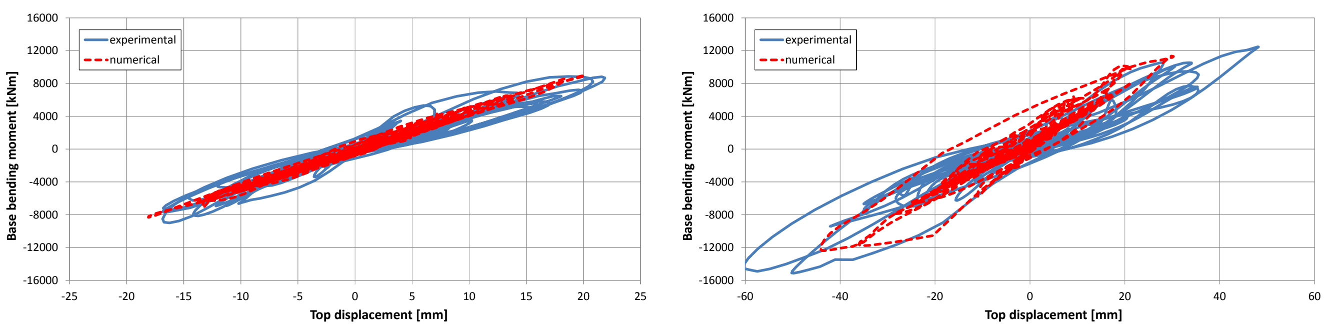
(a) (b)

Fig. 8 – Top floor displacement history obtained from the numerical simulation in comparison with the experimental for PGA equal to (a) 0,15g and (b) 0,30g



(a) (b)

Fig. 9 – Base shear vs top displacement diagram obtained from the numerical simulation in comparison with the experimental for PGA equal to (a) 0,15g and (b) 0,30g



(a) (b)

Fig. 10 – Base overturning moment vs top displacement diagram obtained from the numerical simulation in comparison with the experimental for PGA equal to (a) 0,15g and (b) 0,30g

(a) (b)

Fig. 11 – Detail of one beam-to-wall simply grouted connection (a) spalling at the end of the test with PGA equal to 0,15g and (b) failure at the end of the test with PGA equal to 0,30g

The experimental crack pattern observed on the base modulus of one wall is shown in Fig. 12(a), properly marked to make it visible. The crack pattern clearly shows the formation of distributed horizontal flexural cracks in correspondence of the wall “columns” along the wall height. Those cracks develop with a strong inclination when entering the central portion of the wall, due to the strong shear action, resulting in a classical rhomboidal pattern. The central area of the wall is free from cracks, due to the diffusion of the stress in the screeds. Vertical cracks in correspondence of the change of section have also been observed. Even if the precise measurement of the crack width has not been performed, in general all cracks had small opening. Fig. 12(b) shows a detail of the lightened portions of the wall.

(a) (b)

Fig. 12 – Full scale specimen wall: (a) marked crack pattern at the end of the test at PGA equal to 0,30g, (b) bottom view with lightening detail

5. Conclusions

The structural behaviour of an innovative ligthened precast wall has been studied through numerical and experimental investigation. The results of static non-linear analyses performed with a damage-sensitive shell finite element model show that the technological peculiarities of the wall, including horizontal thin construction slots, do not affect its resistance and ductility potentials when considering the longitudinal reinforcement only. However, the splicing of non-passing shear reinforcement located in correspondence of the external screeds of each wall modulus causes a concentration of ductility demand in short lengths of the wall, located at its bottom and top, strongly reducing the ductility of the full member. The introduction of de-bonding sleeves provides a solution to this problem, allowing for concentrated cracking but distributing the plastic strain of the longitudinal rebars along the length of the sleeves. Two pseudo-dynamic tests performed on a full scale prototype of a 3-storey precast structure provided with two innovative bracing walls items of investigation showed that the walls were efficient in bracing the very flexible precast frame provided with hinged beam-to-column connections. The structure behaved in elastic field when subjected to the accelerogram scaled at a PGA of 0,15g and in elastic-plastic field for the ULS accelerogram, scaled at 0,30g, guaranteeing the required ductility and dissipation of energy. The crack pattern observed at the end of the tests is compatible with the one predicted by the numerical damage-sensitive model. A simplified non-linear beam model of the wall was finally used for the dynamic non-linear simulation of the tests and it provided results in good agreement with the experimental ones, with a certain mismatching in the ULS test due to the progressive damaging of the prototype frame-to-wall connections. However, the simple contact connections used are not representative of the real construction practice, since they have been adopted to be easily removed to perform further tests on different structural configurations of the same structural mock-up. Based on the investigation outcome, it can be concluded that the innovative precast wall herein investigated represents a promising technological solution for the evolution of the precast concrete construction industry, providing safe and robust structural performance if correctly designed and detailed. However, specific issues mainly related to shear strength are planned to be deeper investigated in the near future.

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7. References

[1] Dal Lago B, Dal Lago A, Franceschelli F (2016): Innovation for smart industrial housing. *Concrete Plant International*, **2**, 298-300.

[2] Dal Lago B, Dal Lago A (2012): Precast structures with adaptable restraints. *15th World Conference of Earthquake Engineering (WCEE)*, Lisbon, Portugal, September 24-28.

[3] Simeonov S et al. (1985): *Building Construction Under Seismic Conditions in The Balkan Region – Design and Construction of Prefabricated Reinforced Concrete Building Systems*. Technical Report from the UNDP/UNIDO project RER/79/015.

[4] Sun J, Qiu H, Xu J (2015): Experimental verification of vertical joints in an innovative prefabricated structural wall system. *Advances in Structural Engineering*, **18**(7), 1071-1086.

[5] Schoettler MJ, Belleri A, Zhang D, Restrepo JI, Fleischman RB (2009): Preliminary results of the shake-table testing for the development of a diaphragm seismic design methodology. *PCI Journal*, Winter 2009, 100-124.

[6] Perez F, Sause R, Pessiki S (2007): Analytical and Experimental Lateral Load Behavior of Unbonded Posttensioned Precast Concrete Walls. *Journal of Structural Engineering*, ASCE, 1531-1540.

[7] Marriott D, Pampanin S, Bull D, Palermo A (2008): Dynamic testing of precast, post-tensioned rocking wall systems with alternative dissipating solutions. *NZSEE Conference*, Paper No. 39.

[8] Riva P, Meda A, Giuriani E (2003): Cyclic behaviour of a full scale RC structural wall. *Engineering Structures*, **25**, 835-845.

[9] Fischinger M., Isaković T., Kante P (2006): Shaking Table Response of a Thin H-shaped Coupled Wall. *Managing Risk in Earthquake Country - 100th Anniversary Earthquake Conference*: *Centennial Meeting*, San Francisco, USA. Earthquake Engineering Research Institute, Berkeley, USA.

[10] Kim Y, Kabeyasawa T, Matsumori T, Kabeyasawa T (2011): Numerical study of a full-scale six-storey reinforced concrete wall-frame structure tested at E-Defense. *Earthquake Engineering and Structural Dynamics*. doi:10.1002/eqe.1179.

[11] Paulay T, Priestley MJN (1993): Stability of ductile structural walls. *ACI Structural Journal*, **77** (4), 385–92.

[12] Pilakoutas K, Elnashai AS (1995): Cyclic behaviour of RC cantilever walls, Part I: Experimental results. *ACI Structural Journal*, **92** (3), 271–81.

[13] Tasnimi AA (2000): Strength and deformation of mid-rise shear walls under load reversal. *Engineering Structures*, **22**, 311–322.

[14] Fischinger M, Rejec K, Isaković T (2012): Modeling inelastic shear response of RC walls. *15th World Conference on Earthquake Engineering*, Lisbon, Portugal, Paper No. 2120.

[15] Fischinger M, Kramar M, Isaković T (2008): Using macro elements to predict near-collapse performance of two typical RC building structural systems with lightly reinforced walls and slender precast columns. *14th World Conference on Earthquake Engineering*, Beijing, China.

[16] Orakcal K, Massone LM, Wallace JW (2006): *Analytical modeling of reinforced concrete walls for predicting flexural and coupled shear - flexural Responses*. PEER Report 2006/07. University of California, Berkeley.

[17] di Prisco M, Mazars J (1996): Crush-crack: a non-local damage model for concrete. *Journal of Mechanics of Cohesive and Frictional Materials*, **1**, 321-347.

[18] Ferrara L, di Prisco M (2001): Mode I Fracture behavior in concrete: Nonlocal damage modelling. *ASCE Journal of Engineering Mechanics*, 678-692.

[19] Muhaxheri M (2014): *Behaviour of coupling beams retrofitted with advanced cementitious composites: experiments and modelling*. PhD thesis, Politecnico di Milano, Italy.

[20] Negro P, Bournas DA, Molina J (2013): Pseudodynamic Tests on a full-scale 3-storey precast concrete building: global response. *Engineering Structures*, **57**, 594-608.

[21] Toniolo G (2012): SAFECAST project: European research on seismic behaviour of the connections of precast structures. *15th World Conference of Earthquake Engineering (WCEE)*, Lisbon, Portugal, September 24-28, paper No.1389.

[22] Keintzel E (1992): *Advances in the design of shear for RC structural walls under seismic loading*. Nonlinear seismic analysis and design of reinforced concrete buildings, Elsevier, New York, USA.

[23] Dal Lago B, Toniolo G, Lamperti Tornaghi M (2016): Influence of different mechanical column-foundation connection devices on the seismic behaviour of precast structures. *Bulletin of Earthquake Engineering*, submitted.

[24] CEA (2000): *CAST3M computer program. Education & Research version. Saclay (France)*. http://www-cast3m.cea.fr/cast3m/index.jsp

[25] CEB-*fib* (2010): *Model Code for Concrete Structures*. Fédération Internationale du Béton / International Federation for Structural Concrete, Lausanne, Switzerland.

[26] Biondini F, Titi A, Toniolo G (2012): Pseudodynamic Tests and Numerical Simulations on a Full-Scale Prototype of a Multi-Storey Precast Structure. *15th World Conference of Earthquake Engineering (WCEE)*, Lisbon, Portugal, September 24-28.

[27] EN 1998-1:2004 (2004): *Eurocode 8: Design of structures for earthquake resistance. Part 1: General rules, seismic actions and rules for buildings*. European Committee for Standardization, Brussels, Belgium.

[28] Strand7 Pty Limited (2010): Using Strand7 (Straus7) – introduction to the Strand7 finite element analysis system, edition 3.

[29] Sargin M (1971): *Stress-strain relationship for concrete and analysis of structural concrete sections*. Study n. 4, Solid Mechanics Division, University of Waterloo, Canada.

[30] Takeda T, Sozen MA, Nielsen NN (1970): Reinforced Concrete Response to Simulated Earthquakes. *Journal of the Structural Division*, ASCE, **96** (12), 2557-2573.

[31] Dal Lago A, Dal Lago B (2012): Progetto SAFECAST: problematiche riscontrate confrontando progetto e prove sismiche. *19th CTE Congress*, Bologna, Italy, November 8-10, 553-562.

[32] Bournas DA, Negro P, Molina J (2013): Pseudodynamic Tests on a full-scale 3-storey precast concrete building: behaviour of the mechanical connections and floor diaphragms. *Engineering Structures*, **57**, 609-627.