# Experimental tests on post-tensioned rocking concrete walls with end columns and steel dissipative devices

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## *Abstract*

In many countries, the current seismic design practice for concrete structures is based on the exploitation of the ductility capacity of some structural elements, such as beams in frame structures. This approach, even if theoretically able to guarantee life safety for the occupants of buildings, has recently been criticized because it might lead to extensive damage in structural elements. Damage that in many cases cannot be repaired. Therefore, this design practice might lead to limited resilience; as highlighted by many recent earthquakes (e.g. Christchurch earthquakes) the recovery process after strong seismic events may take years and have significant social and economic negative consequences. For these reasons different structural systems and technical solutions have been developed in the literature in order to limit structural damage, i.e. base isolation, viscous dampers, etc. Some of these technologies are very effective but might be unpractical for some types of buildings, such as precast concrete structures, in particular for economic constrains. For these buildings researchers have proposed specific solutions based on post-tensioned concrete elements free to rock at their base, with easy-to-replace external dissipative devices. These systems aim at solving the limitations of ductility-based design though self-cantering capabilities and easy reparability.

The present paper presents the results of an experimental campaign aimed at studying the behaviour under cyclic horizontal loads of post-tensioned rocking concrete walls with end-columns and steel dissipative devices. The structural system tested comprises a concrete wall with two bundles of post-tensioning cables, two post-tensioned concrete end-columns (one at each end of the wall) connected by beams and hysteretic dissipaters between the wall and each column. Columns are used in order to support beams because, even if base rocking occurs, their uplift is limited given their small cross section size. The paper discusses the quasi-static cyclic behaviour of two of these systems featuring different details at the wall-base. Furthermore, it compares the experimental results with the predictions of design-oriented analytical models.

*Keywords: post tension; concrete walls; rocking walls; hysteretic dissipaters; low-damage design.*

## 1. Introduction

Structural Reinforced Concrete (RC) walls are one of the most effective lateral loads resisting systems, in fact they feature high stiffness and strength. Current seismic design approaches mainly aim at obtaining a ductile behaviour of structures, through the definition of capacity design criteria and detailing rules. In RC walls only a single plastic hinge at base can form, thus providing less ductility capacity than other structural systems, such as moment resisting frames. In the current design approach, ductility, and therefore damage, is used to dissipate part of the seismic energy, with the aim of achieving both life safety for the occupants of buildings and collapse prevention. After strong ground-motions it is therefore possible to have high levels of damage structural elements. The subsequent recovery process could take a very long time and have negative social and economic consequences. Furthermore, based on the lesson from Christchurch earthquakes [1], the costs associated with business interruption, damage to equipment and structural rehabilitation are, often, comparable to the entire building cost.

For these reasons, alternative resisting systems have been developed, defined as "low-damage systems", with the aim of limiting damage or isolating damage in easily replaceable elements. To date, different effective technologies already exist, like base isolation or viscous dampers, but their application might be unpractical for some types of buildings, such as low-rise precast concrete structures, mainly because of economics restraints. Therefore, researchers proposed innovative solutions based on precast concrete elements free to rock at their ends with unbonded pre-stressing cables. The objective of these structural systems, which feature self-centering capabilities, is to offer resilient solutions against structural damage caused by earthquakes.

The concept of connecting precast concrete elements with unbonded post-tensioned cables was introduced, for the first time, during the PreSSS project research program in the 1990s [2][3][4]. In this circumstance, a jointed precast unbonded wall system was developed. It comprised two or more walls free to rotate or "rock" at their base and connected at the foundation with post-tensioned cables. In the vertical joints between each wall, there were special U-shaped connectors. In the final stage of this project a full-scale precast concrete building composed by frame resisting systems and jointed walls, was tested under a cyclic lateral load. Further studies were carried out by Restrepo and Rahman [5] about unbonded precast panels able to dissipate seismic energy through the yielding of mild steel bars at the wall-foundation joint. A variation of the system, which is an alternative approach to jointed walls, was obtained by reducing the dimensions of the panels at the ends, up to assume them as columns. It provided necessary support to slab thanks to the limited uplift due to small section dimensions. The study by Henry, Sritharan and Aaleti [6] [7] [8] was carried-out in the same context. Henry's resisting system, called PreWEC, designed and validated experimentally, consisted in one precast unbonded concrete wall and two End Columns in steel and concrete at both sides. In this system, hysteretic dissipative devices were placed in wall-column joints and they had O-shape.

The research developed in this paper is focused on a precast concrete wall with two bundles of post-tensioning cables and concrete end-columns connected to concrete beams. Beam-column connection was made with steel dowels; while wall-column connection is made with special metallic hysteretic dissipaters. Two different walls were tested: one with a steel plate at the base and no shear-key, and one with shear keys. The paper discusses the quasi-static cyclic behaviour observed experimentally for one of the two walls and compares the experimental results with the design procedure proposed by Aaleti and Sritharan [9].

## 2. Experimental program

The tested setup was designed to ensure three performance levels at the life-safety limit state, which are (Restrepo [5]) i) ensuring self-centering capability; ii) maintaining the post-tensioning elements in elastic conditions; iii) avoiding concrete spalling at the element toes during rocking. The Self-centering capability is guaranteed by calibrating the initial post-tensioning force and by limiting the stress-loss due permanent deformations in compression. The analytical procedure developed by Aaleti [9] was applied to define the initial post-tension value and to estimate the stress in the cables during the test. Concrete cracking at the wall base was controlled by means of confinement.



According to ACI_ITG5, for jointed wall systems the life safety limit state is reached for a maximum drift ratio of 3%. For limitations of the experimental setup the maximum drift level imposed was 2.5% in one direction, while in the other direction the maximum drift ratio imposed was 2%.

Fig. 1 – One of the structural systems tested under quasi-static lateral loads.

### 2.1 Description of the structural system

The structural systems tested consisted in a precast concrete wall post-tensioned with unbonded tendons (Fig. 1), two precast concrete columns, with unbonded post-tensioned steel bars, and two precast concrete beams. In an actual building these beams would support slabs. As discussed by Henry and Aaleti [6], being connected to the end columns, their vertical movement during rocking is limited thus decreasing the possible damage to slabs. The whole system was positioned on a prefabricated concrete foundation. Specially designed dissipators, connected the columns to the wall, transferring lateral loads among those elements and dissipating energy. Further details about the dissipators cannot be disclosed.

Two variations of the wall were tested: one had a 10mm thick steel plate at the base (Wall 1) and the other had shear keys at the ends (Wall 2). These two different details were considered in order to assess their influence on the global behaviour. Both wall panels had a width of 1300 mm a thickness of 250 mm and a height of 4000 mm. Their basic reinforcement comprised ϕ10 mm longitudinal steel bars and ϕ10/150 mm stirrups. Confining reinforcement was placed at the wall bottom corner, in order to improve the concrete compressive capacity (Fig. 2a, 2b). Columns had a 400x400 mm rectangular section and were 4000 m high. At 2420 mm from the base they featured corbels for supporting the beams. They were reinforced with 4ϕ14 mild steel bars and ϕ10/150 mm stirrups. As for the wall, confining reinforcement was added at the base. Each corbel had 3 ϕ30 mm dowels for connecting the beams. These latter, had a 300x500 mm rectangular section and were 4000 mm long. They were reinforced with 4ϕ20 mm bars in the longitudinal direction and ϕ10 mm stirrups with variable spacing. During the casting process two rectangular holes were left for the connection with the columns, these holes were filled with high strength mortar after the installation of the beams on the corbels.



a) b) c)

Fig. 2 – Detail of the reinforcement of Wall #2: a) confining reinforcement and shear key at the toe, b) reinforcement in the upper part and steel plates for connecting the dissipators. c) pockets at the top of the foundation.

The foundation had a rectangular 1400x550 mm cross section and was 4500 mm long. It was connected to the laboratory strong floor using 8 Dywidag steel bars. On the upper face, the foundation was equipped with three 40 mm deep pockets, for installing the wall and the columns; the two pockets for columns had in-plan dimensions of 410x410 mm, that for the wall was 500x1310 mm (Fig. 2c). Before assembling the elements for the test of W1, a 20 mm thick layer of high strength mortar was cast in order to obtain an even surface. During this first test the short edges of the wall pocked acted as shear key, to this aim two vertical steel plates were placed at each side (short side). In the second test (T2), performed on the wall W2, two 20 mm steel plates were placed at the ends of the pocket for the wall, with the purpose of installing a shear key, consisting in one half of a steel cylinder that was welded on the aforementioned plates. All the pockets were then filled with further 10mm of high-strength mortar.

The initial tension in the pre-stressing elements was set to 475 kN for the wall and 270 kN for the columns. Two bundles of four strands with nominal diameter 15.7 mm and cross-section area 600 mm$^2$, were used in the wall, while in the columns ϕ36 mm Dywidag bars were used (one per column). Strands and bars were anchored at the top of the relative element and in the foundation.

## 2.2 Material properties

All the RC elements were produced using the same concrete mix. Its compressive strength was determined by performing cubic compression tests in accordance with UNI EN206-2014, while the elastic modulus was measured according to UNI EN12390-2013. The average values of compressive strength and elastic modulus were 80 MPa 33400 MPa, respectively. Concrete rebars were tested according to UNI EN15630-2019, obtaining a yielding strength of 545 MPa and a tensile strength of 640 MPa. The steel used for the steel plates, was tested according to UNI EN10002-2001. It had a yielding stress 295 MPa and a tensile strength of 379 MPa.

The strands used were made of high-strength steel with nominal yielding strength of 1670 MPa and tensile strength of 1860 MPa. The steel of the Dywidag bars has a yielding stress of 963 MPa and a strength of 1050 MPa.



### 2.3 Loading protocol

Test was executed using a servo-hydraulic actuator (maximum force capacity 1000 kN; maximum stroke ± 125 mm), supported by a reaction wall. The actuator applied the load on the beams, at a 2950 mm height from the foundation, to simulate the force path in actual structures, where most of the mass is at the floor levels. Two steel beams were placed at the ends of the concrete beams and connected by means of four pre-tensioned ϕ26 mm Dywidag bars, in order to transfer the load from the actuator to the structural system to be tested.

The load protocol adopted for the test was defined in accordance with ACI ITG-5.1, for the acceptance criteria for unbonded concrete walls. It consists of two force limited cycles in the elastic range, followed by a series of cycles at different drift values, in displacement control. In particular (Fig. 3), one cycle at 50 kN, one cycle at 100 kN, 3 cycles at 0.1% drift, 3 cycles at 0.25% drift, 3 cycles at 0.5% drift, 3 cycles at 1% drift, 3 cycles at 2% drift and *3* cycles at 2.5% in one direction only. All the drift values were referred to the wall.

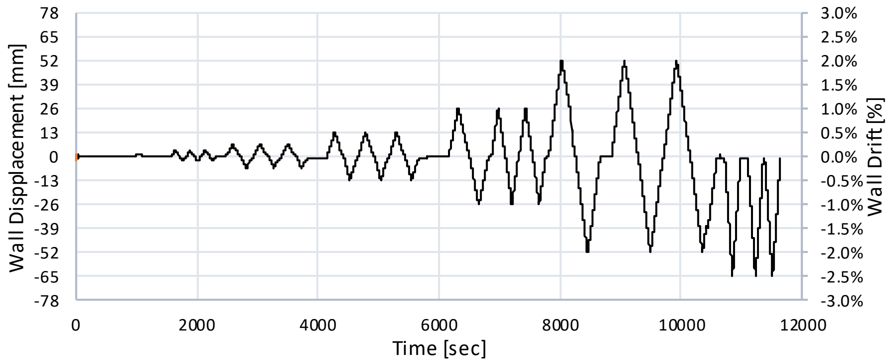



Fig. 3 – Loading protocol used for displacement controlled cycles of the test.

### 2.4 Instrumentation

The horizontal displacement (drift) of the wall was measured using one LVDT, for small displacements, and one Wire Displacement Transducers (WDT) on each side of the wall. All these four transducers were located at a height of 2600 mm from the base of the wall. The wall base sliding was measured using a LVDT, while wall uplift was measured with 7 potentiometric displacement transducers at the wall-foundation interface (Fig. 4a). An additional LVDT was fixed at the wall top for monitoring the out-of-plane displacement. At the base of each column the horizontal sliding was measured using a LVDT while each column uplift was monitored by means of two LVDTs (Fig. 4b). The relative vertical movements between the wall and each column were also measured using LVTDs. The beam sliding on each corbel was measured using a LVDT. At the actuator level, the horizontal displacements of the wall, the beams and the columns were measured with potentiometric transducers. Furthermore, a LVDT was used to measure any foundation sliding with respect to the strong floor. Strain gauges were installed on the dissipators connecting the wall to the columns. A load cell was used to measure the lateral load in the actuator, while hollow hydraulic jacks were used to measure the force in the post tensioning elements (Fig. 5).



a) b)

Fig. 4 – Displacement transducers at the base of the wall (a) and at the base of one of the two columns (b).

Fig. 5 – Hydraulic jacks at the top of the wall, used to mesure the load on the cables.

## 3. Experimental Results

The hysteretic response of a resisting system can be assessed based on the amount of self-centring force and its dissipative capacities. Rocking behaviour is characterized by the opening of a joint at the element's base, a phenomenon that occurs when the external moment due to lateral force imposed is larger than the system decompression moment. The gap opening at the base leads to a reduction of the stiffness which is restored when the unbonded cables bring the elements back in their initial position. Energy dissipation is possible thanks to the presence of the steel dissipators. The system has shown an excellent cyclic behaviour, for all drift levels. Wall and columns have rotated independently and relative vertical sliding in wall-column joints was generated. This movement has caused a yielding deformative state in the connectors, producing high energy dissipation. No concrete cracking was visible at the base elements, which is a positive result of compared with the traditional cracking pattern in RC elements (Fig. 6). Although, even if constraints have not been set up, the out-of-plane behaviour was neglectable.



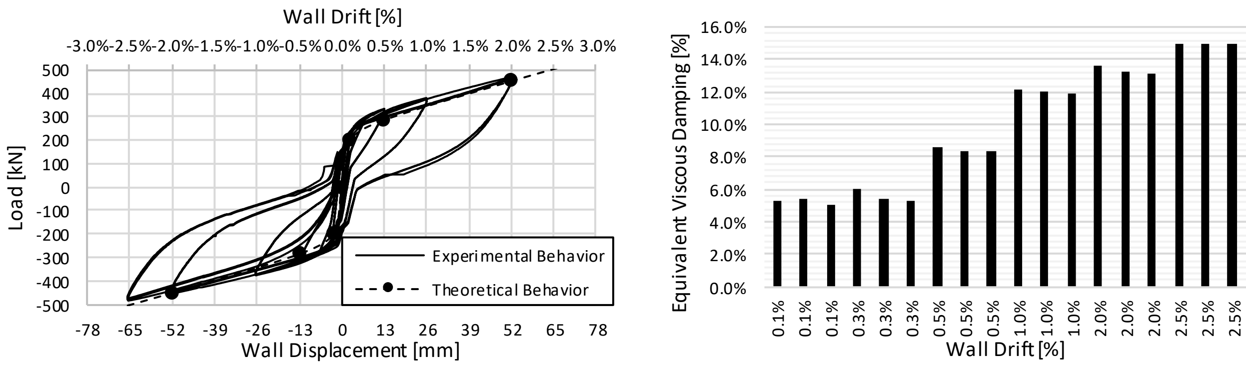
 

a) b)

Fig. 6 – Base of the wall (a) and of one of the two colums (b) at the end of the test.

### 3.1 Force displacement response

The Force-Displacement plot in Fig. 7a illustrates the global behaviour of the system. The wall displacement was computed as average of the displacement measures obtained from the transducers at both sides of the wall. The shape of the hysteretic cycles observed is consistent the typical flag-shape behaviour observed on similar systems tested in the literature. It is easy to notice that the tested system satisfied the requirements discussed in Section 2. In general, the response of the system is stable and symmetrical. Furthermore, the structure tested has good self-centring capabilities. Fig. 7a also shows a comparison with the theoretical response given by the model proposed by Aaleti [9], highlighting an excellent correlation.




a) b)

Fig. 7 – Force displacement curve for the system tested (a) and equivalent viscous damping ratio (b).

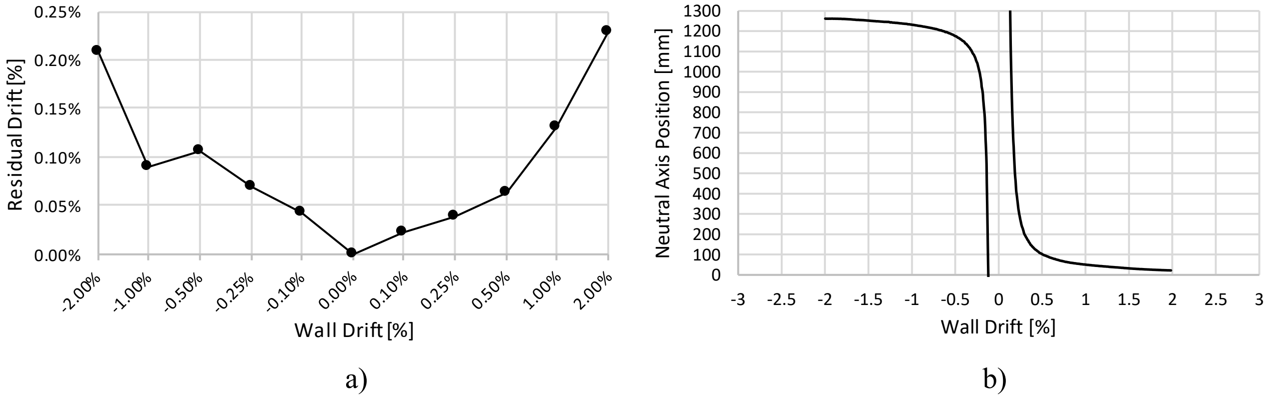



Fig. 8 – Maximum residual drift measured during the loading cyles at different drift values (a) and neutral axis position at the base of the wall (b).

### 3.2 Equivalent Viscous Damping

Equivalent Viscous Damping (EVD) is used as a measure of energy dissipation capacity of structural systems, and is essential for displacement-based design procedures, commonly used for unbonded post-tensioned systems. Based on Jacobsen's formulation EVD [10] can be computed as

$$\xi = \frac{1}{4\pi} \cdot \frac{E_{DISS}}{E_{STO}} \tag{1}$$

where $E_{DISS}$ is the dissipated energy and is equal to the sum of the area enclosed inside each entire loop, and $E_{STO}$ is the stored energy given by an area inside a triangle of the first quadrant obtained knowing the maximum force and the relative displacement in each loop.

Fig. 7b shows the values of EVD obtained from the different cycles. As mentioned, at 2.5% of drift it was possible to perform only cycles in one direction, but based on the symmetry observed in previous cycles, the EVD was computed by assuming symmetrical behaviour. The energy dissipation capacity of the system is in general good and consistent with similar structural systems tested in the literature. It should be noticed that EVD values are stable among the different repetitions of the cycles at the same drift level, indicating very limited deterioration due to damage. In particular, excluding the first repetition of each cycle, where the dissipate energy slightly higher, no difference can be notice between the EVD values for the second and third repetition.

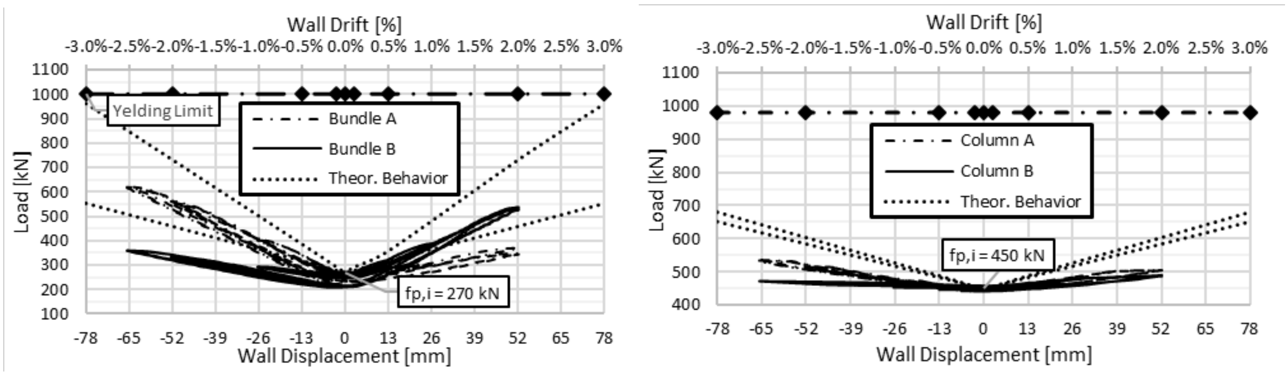
### 3.3 Residual drift

Residual drift is a critical aspect of lateral load resisting systems in general, since it is an indicator of their repairability after seismic events. In case of rocking systems, repairability is an intrinsic property of the system, but residual drift an indicator of the self-centring capability. This parameter is defined as the displacement at zero lateral force. The maximum value obtained among the three repetitions of the loading cycles in displacement control are reported in Fig. 8a. The values obtained are consistent with those of similar systems in the literature and allow to conclude that they systems tested fulfils the self-cantering requirement.

### 3.4 Neutral Axis position

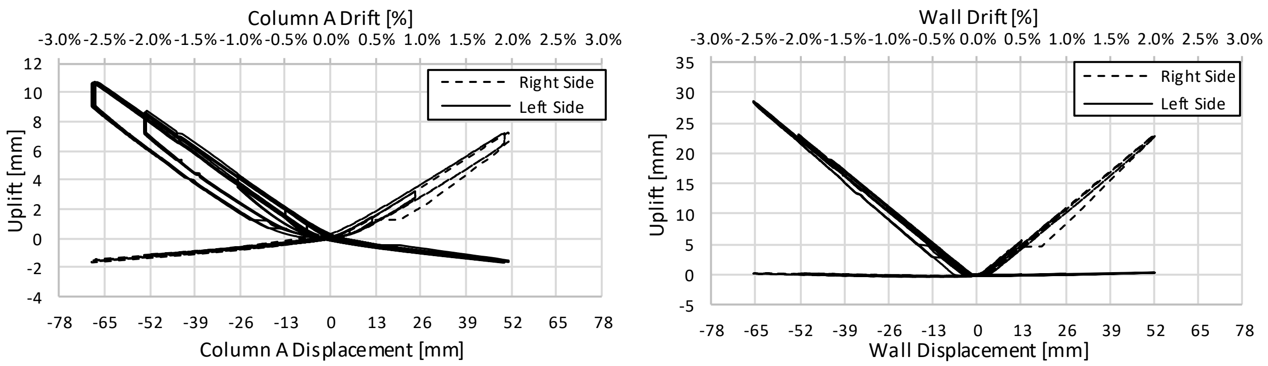
The position of the neutral axis, which defines the extent of the compression zone at the base of the wall, was calculated by assuming that the base of the wall remains plane and performing a linear regression on the uplift measurements given by the displacement transducers at the wall base. In Fig. 8b the typical trend associated to a rocking behaviour is clearly visible. At zero drift the base of the wall is fully compressed, therefore the neutral axis position is at infinity, as the lateral load increase, and thus the bending moment at the base, the neutral axis moves towards the section and, at the decompression condition, corresponds to the edge of the section on the tension side. As the lateral load keeps increasing the neutral axis keeps moving towards the compression side. The overall trend of the neutral axis position is roughly symmetrical. Indicatively, at + 0.5% the compressed portion is about 95 mm while at -0.5% it is about 130 mm; at + 1% it is equal to 50 mm while at -1% it is 70 mm; finally, at 2% it is 20 mm instead at -2% 40 mm.

### 3.4 Post-tension force

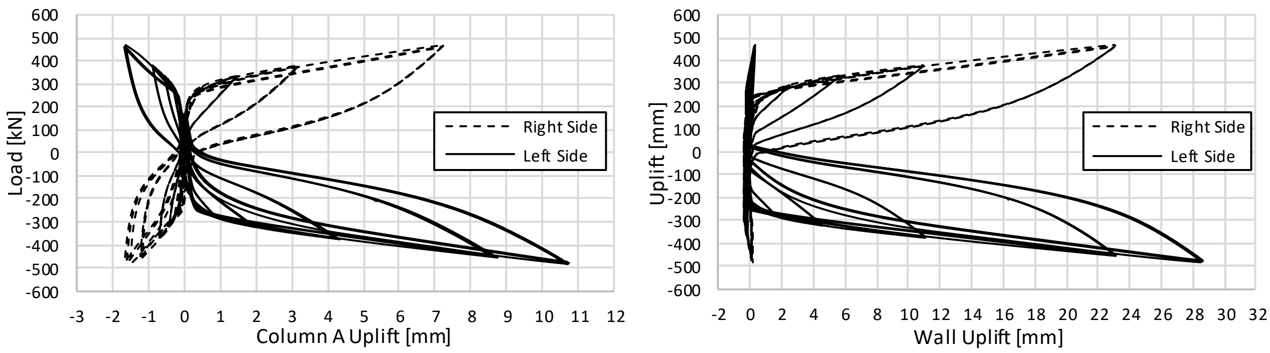



a) b)

Fig. 9 –Tension forces in the post-tensioned strands of the wall (a) and in the Dywidag bars of the columns (b).




a) b)




c) d)

Fig. 10 – Uplift versus horizontal displacement and force uplift curves for column A (a,c) and for the wall (b,d).

Maintaining the post-tensioned elements elastic is a key requirement in self-centering systems, otherwise they would lose their effectiveness. The forces measured in the cables and in the Dywidag bars are reported in Fig.



9, together with the design predictions. In general, the observed behaviour is consistent with literature data. The main elements worth noticing are the limited pre-stress loss due to permanent deformations and the stable behaviour during the different repetitions of the cycles. In general, the analytical prediction of behaviour of the post-tensioned elements, used for design calculations, overestimated the values of force, because of overestimate of the stiffness of the elements. This difference can be explained considering that the analytical model does not take into account the flexibility of the anchors at the end of the cables and of the Dywidag bars.

### 3.5 System uplift

The uplift of the columns was obtained from the measurements of the transducers at their two sides. For the wall the two transducers closer the toes were evaluated. Fig. 10a,c shows the uplift data for column A versus the its drift. The behaviour is symmetrical and consistent with design predictions, furthermore the limited uplift values confirm the effectiveness of PreWEC systems in limiting the vertical movement of slab supporting elements. The behaviour of the wall (Fig. 10b,d) is consistent with design predictions as well.

### 3.6 Beam-column sliding

One of the most critical elements of the structural system tested are the beam-column connections. In fact, lateral loads must pass through these connections. Furthermore, the dowels that provide the connection, and the concrete around them must remain undamaged. Fig. 11 shows the relationship between the total lateral-load and the sliding on the corbels of Column A for the two beams. For both of them, sliding was limited, even if the left beam had more sliding that the right beam. This different behaviour is due to the different level of local damage in the connection zone. In any case, sliding was small compared to the total horizontal displacement of the system.

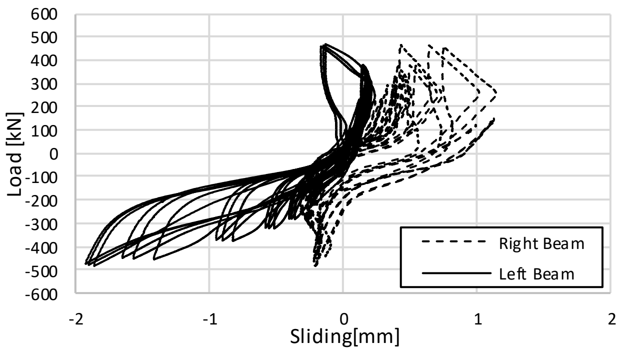



Fig. 11 – Lateral load versus average sliding of the beams.

## 4. Conclusions

The paper describes part of the results of an experimental campaign aimed at testing lateral load resisting systems based on a rocking concrete wall with two lateral columns. The columns and wall were constrained to a concrete foundation through post-tensioned cables, unbonded their entire length. In addition, one steel dissipator has been inserted at each side of the wall, with the dual function of dissipating energy and transferring lateral loads. The main objectives of the experimental test were evaluating the self-centering capacity, the dissipative capacity and the damage level at the end of the test.

The outcomes of the experimental test described in the paper were positive. The hysteretic behaviour was stable and dissipative. Connectors were able to dissipate a significant amount of energy and to fully transfer lateral loads. Regarding the post-test damage level, considering the residual drift values observed and the lack of damage to concrete elements, it can be concluded that the system does not require any repairing intervention to restore its resisting capacity. By simply replacing the dissipative devices, the system can return to its initial performance levels.



The analytical theory employed for design was able to correctly estimate the hysteretic response of the system, even if it overestimates the tension increase in the cables and in the bars.

## 4. Acknowledgements

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