# Full Scale Experimental Investigation on Seismic Structural Walls

**Marco Preti and Ezio Giuriani**
*Department of Civil Engineering, Architecture, Land and Environment (DICATA) - University of Brescia. Via Branze 43, 25123 Brescia, Italy.*

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**SUMMARY:**
In historical earthquakes reinforced concrete structural walls showed to be very effective in preventing collapse and damage of buildings under strong and frequent events respectively. The large deformations imposed by strong earthquakes require for the walls large ductility capacity, which may be achieved by means large deformation in the plastic hinge at their base. The paper presents a full-scale experimental test which compares the performance of two analogous structural walls, proportioned for a five story building, reinforced with traditional and un-bonded reinforcement, respectively. A specific detailing solution to ensure shear dowel resistance at the wall base was adopted for each test wall, whose good performances were confirmed by the test results. The two walls reached large deformation (up to 2,5% drift) without collapse or shear sliding. Detailed measurements of the deformation in the critical region were collected to offer a comparison against the results of analytical and numerical modelling.

*Keywords: Structural walls, Full scale, Experimental test, Shear dowels, Rocking*

## 1. INTRODUCTION

In historical earthquakes rc structural walls showed to be very effective in preventing collapse and damage of ordinary buildings under strong and frequent events respectively [Fintel, 1995, EERI, 2012]. The large deformations imposed by strong earthquakes require for the walls large ductility capacity, which may be achieved by means of extensive inelastic deformation in the plastic hinge at their base. A number of experimental tests on structural walls reinforced with traditional steel rebars showed adequate flexural ductility [Pilakoutas and Elnashai, 1995], even if they are characterized by significant damage in the plastic hinge and the risk of shear sliding [Paulay et al. 1982, Salonikios et al., 2000]. As an alternative, rocking walls obtained by replacing the reinforcing longitudinal rebars at the base cross-section with un-bonded post-tensioned tendons, offer the possibility to reduce the damage and nullify the residual drift, thanks to their self-centring properties.

In slender walls, the shear sliding mechanism can particularly take place when poor or none axial compression is applied at the crack location at the base of the wall [Riva et al., 2003]. In a real building such condition may occur in several cases, for example in nearby walls, due to the coupling action of floor diaphragms, in walls with small floor tributary area or in general instantaneously due to vertical seismic acceleration. Sliding may occur also in rocking wall due to impact and dynamic rebound of the walls. In this particular case sliding has to be prevented in order to protect the un-bonded tendons crossing the rocking cross-section.

In the paper, the focus is on the detailing of structural walls against sliding. The results of an experimental test on two full-scale structural walls is presented, which allowed to evaluate the response of two unusual detailing solution to offer adequate shear over-strength against the sliding mechanisms.

## 2. DETAILING FOR SHEAR SLIDING

When proportioning the shear resistance of a structural wall, according to the capacity design approach, particular attention must be devoted to the dynamic response of the structure which typically involves shear amplification with respect to the design action obtained by static analysis [Rutenberg and Nsieri, 2006]. The need for shear over-strength concerns both the diagonal strut mechanism and sliding mechanism and requires solutions to offer adequate and certain resistance. Both for monolithic and rocking walls, the triggering of the sliding mechanism typically occurs at load reversals, when the resistance associated to friction at the crack location is shifting due to the transition of the compressed area from one wall toe end to the other [Paulay et al, 1982]. In this phase the dowel action of the reinforcement crossing the sub-horizontal cracks is particularly important to avoid localized sliding at the crack.
In the experimental work here presented, the design sliding resistance was completely assigned to dowel action. The proposed design solution was to locate the dowel resistance in the central portion of the cross-section, outside the compressed region, where concrete do not suffer damage due to compression stresses and deformation induced by flexure. According to this choice the detailing of the two walls for shear sliding is described in the following.

### 2.1. Detailing against Shear sliding in the rocking wall

In the rocking wall, the sliding shear resistance at the base was assigned to eight unbounded steel tubular shown in Figure 1.

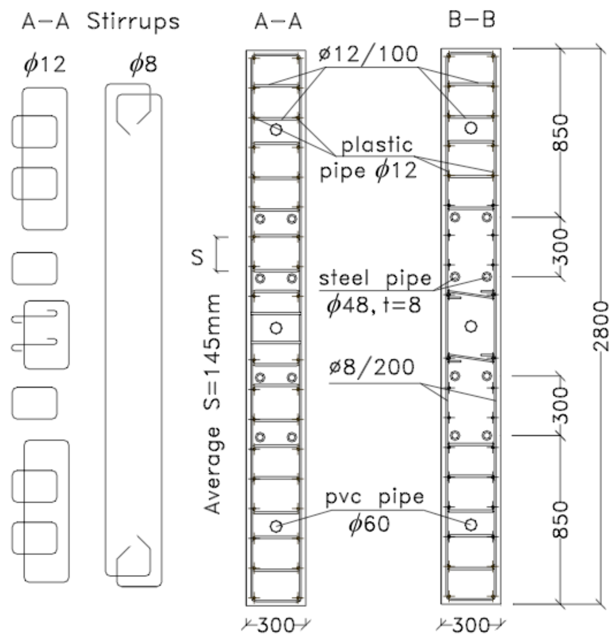



**Figure 1.** Detail of the rocking wall cross-section with the position of the post-tensioned tendons and un-bonded dowels.

The bond drop was obtained with thin polythene wrapping on the tubular that allowed the sliding of the tubular inside the wall cast. The bond drop is necessary to limit the axial load in the tubular, protecting it from yielding, and to uncouple the bending and shear resistance of the wall. As evidenced during the test (Fig.2), the transfer of the shear action from the rocking wall to the dowels requires adequate horizontal reinforcement to connect the dowels to cross-section ends, where the compression strut of the wall is oriented during rocking.

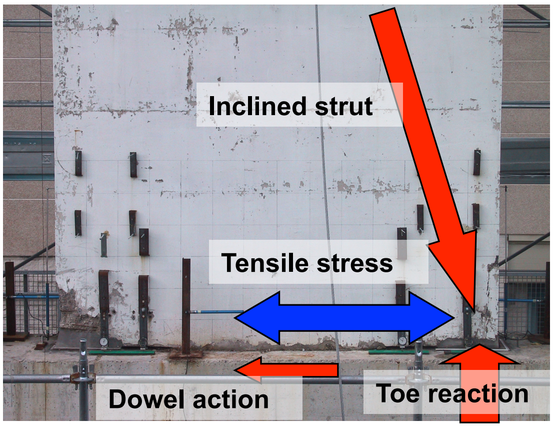



**Figure 2.** Stress flow scheme after the activation of the un-bonded dowels against sliding.

The proportioning of the dowels aimed at ensuring the elastic behavior of the tubular subjected to both shear action and axial load deriving from the residual friction of the tubular in the cast.

### 2.2 Detailing against shear sliding in the monolithic wall

In the monolithic wall, the task of resisting sliding shear action was entirely assigned to the longitudinal rebars, of a large diameter (20mm), working as dowels. In order to limit buckling for these rebars at load reversals due to large axial strain, the vertical rebars were distributed mainly in the web of the wall (Fig.3), ending with a uniform distribution of rebars in the cross-section length [Priestley, 2003]. These rebars where proportioned for resisting both flexure and sliding shear, adopting for sliding resistance the dowels strength formulation of Eurocode 8 [2004]. An amplified design shear resistance was proportioned for the dowels in order to avoid a shear failure in the plastic hinge, 50% larger than the shear action that developed when the design moment capacity was reached. The longitudinal rebars were without lap splices from the foundation to the top of the wall. No inclined reinforcement was inserted (further details in Preti and Giuriani, 2011). Table 1 compares the measured shear action and the calculated probable sliding strength for the wall in the XIV cycle, at maximum drift (2.5%) and load reversals (0% drift).

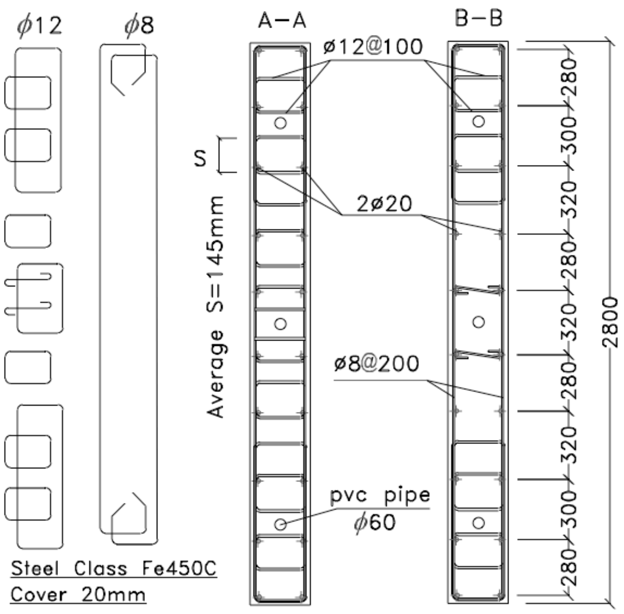



**Figure 3.** Monolithic wall cross-section details.

**Table 1.** Comparison of calculated strength and applied shear sliding for the monolithic test wall at maximum drift and at load reversals, respectively.

| **Shear Action at wall base, kN** | Applied at XIV cycle | Calc. Probable Strength* |
|---|---|---|
| 2.5% drift (Maximum displacement) | 636 | 1794** |
| 0% drift (Load reversal) | 210 | 704* |
| Strength according to [EC8, 2004] * Vn,d =total shear dowel strength; **Vn,(d+f) = dowel + friction shear strength. | | |

## 3. TEST SET-UP

### 3.1. Geometry and mechanical properties of the test walls

The full scale experimental structural walls (Fig. 4) were proportioned and designed referring to a hypothetical ordinary five storey building, subjected to a strong earthquake. The lateral force on the walls was assumed as 30% of the weight of the building tributary area, which was assumed equal to $50m^2$ per floor.


(a) (b) (c)

**Figure 4.** Full scale test walls (a); detail of the rocking wall (b) and position of the post-tensioned un-bonded tendons and dowels (c).

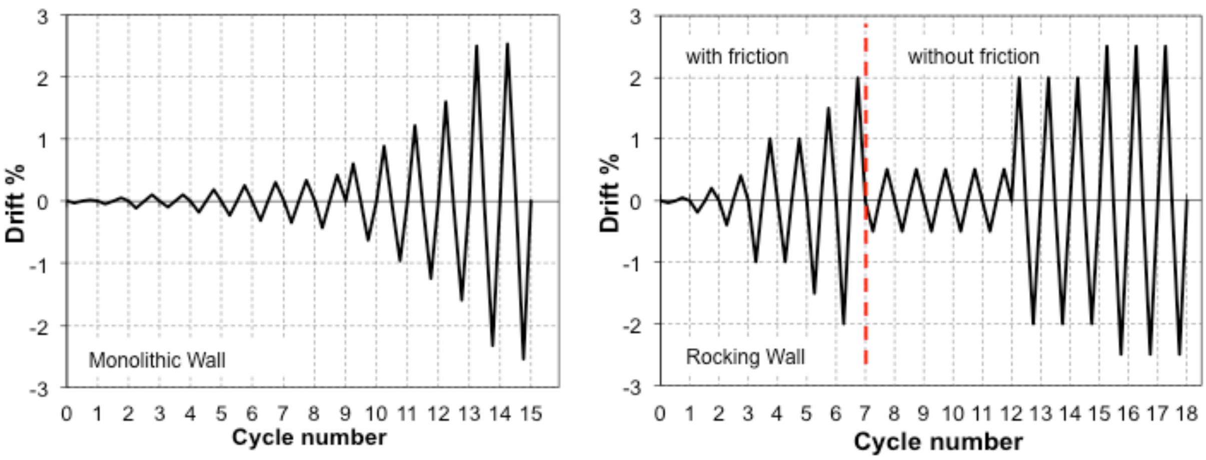
The test walls height, $h_w$=*10m,* was equal to two thirds of the case study height $h_d$. A constant section *2800×300mm* was adopted. So the wall geometric aspect ratio in the case study building was $h_d/l_w=5.4$, where $l_w$ was the overall section depth of the walls.

The shear span for the monolithic wall (reinforced with threaded bars), defined as the ratio of the resultant lateral force height on the section depth, was $M_b/(V_b \times l_w)=3.4$, being $M_b$ and $V_b$ the bending moment and shear action at the base of the wall. The height of the applied force over the wall base in the experimental test was equal to *9.5m*, *500mm* below the level of *2/3* of the case study building height.

For the rocking wall (reinforced with unbounded tendons), which was tested afterword the monolithic one, the shear span was slightly reduced ($M_h/[V_h \times l_w]=3.15$) to allow the positioning of an enhanced loading system.

### 3.2. Experimental apparatus and loading history

The two test walls in the experimental apparatus, described in detail in Preti and Giuriani [2011], were built in line on a stiff basement element (Fig.4) anchored on an underground pre-stressed RC box structure available at the Pisa Laboratory of the University of Brescia.
The walls were tested in successive times. First the monolithic wall was tested and pushed up to 2,5% total drift, while the rocking one worked as a reaction wall; the rocking wall was made over-resistant by means of doubling the post-tensioning in the un-bonded strands, taking advantage of the capacity of such wall to sustain the design experimental force of the monolithic wall without any damage and with very small deformation.
Afterwards, the monolithic wall was repaired and strengthened with additional unbounded tendons and confinement at the wall toe (see fig.4) to act as reaction wall during the rocking wall testing.
The horizontal load was applied at the top of the wall, by means of pushing and pulling jacks, for cycles of varying amplitude under displacement control. The average loading rate, defined by adopting a displacement-settling criterion at each load step, was equal to about *180mm/h.*
The two loading history applied to the monolithic and rocking wall are shown in Figure 5.



(a) (b)

**Figure 5.** Loading history for the test monolithic wall (a) and for the rocking wall (b) with the two test phases: “with friction” test and “without friction” test.

The loading history for the rocking wall test is shown in Figure 5b: seven cycles were first applied to the original structure (phase 1). The following eleven cycles (phase 2) were applied after the insertion under the wall toes of a sliding device and the consequent drop in the friction resistance against sliding [Preti et al., 2009]. Five of these eleven cycles reached amplitude of *0,5%* drift, corresponding to the design damage limit state for the structure. Afterwards 3 more cycles per each level of amplitude of *2%* and *2,5%* drift were applied in order to evaluate the wall behaviour at the design ultimate limit state.
Instrumentation was placed on the walls to measure the top wall horizontal displacement, the applied lateral load, the base crack opening, the sliding at the wall toes and the concrete deformations at the compressed wall ends near the base. For the rocking wall test, additional measures were done by means of a mechanical gauge instrument (Witmore instrument, *300mm* reference length) at the crack locations to survey the evolution of the mechanism induced by the dowel action of the steel vertical dowels. The displacements and deformations were measured by means of a total of *14* potentiometric transducers. A plumb line system measured the wall horizontal displacement at the applied load; the plumb was immersed in water to damp its oscillation.
The horizontal force was measured indirectly by the jack oil pressure, using a full-bridge resistive pressure transducer. The maximum jack loading capacity was *800kN* and the maximum inaccuracy was about *1%*, measured by means of experimental calibration.

The crack pattern of the front face of the critical region was recorded at each load step by means of a high-resolution camera. A *200x200mm* grid painted on the wall surface was adopted as reference system for the cracks.

## 4. RESULTS

A complete discussion of the test results can be found in Preti and Giuriani [2011] and Preti et al. [2009]. Here the performance of the two walls against shear sliding is stressed and compared referring in particular to the role of the dowels at the wall base.

### 4.1 Rocking wall response

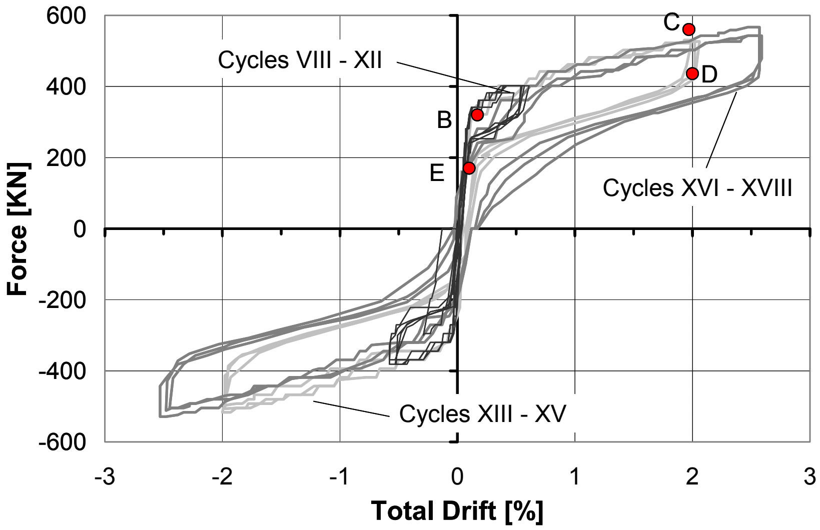



**Figure 6.** Rocking wall: Lateral Force versus Total Drift diagrams of the loading cycles after the drop of shear friction at the wall toes ("without friction" test phase), at 5%, 20% and 25% wall total drift deformation.

Figure 6 shows the Force-Drift response of the ensemble of cycles applied on the rocking wall during phase 2 ("without friction"). Up to 2% drift no stiffness degradation was measured; the curves of subsequent cycles were practically superposed, drawing a flag-shaped behaviour typical of rocking walls with associated energy dissipation. Up to 2% drift such unexpected energy dissipation was interpreted to be mainly due to friction of the tubular extracted from the foundation. Such reading is supported by a rough evaluation of the action necessary to extract the tubular from the foundation during rocking, from which the work done during the cycles by friction in this mechanism varies from 90% to 70% of the energy dissipation measured during the test in each cycle (in cycles at 5% and 2,5% drift, respectively). The evaluation was done taking into account the confining stress on the unbounded dowels in the foundation originating from concrete shrinkage and the wall up-lift during the test.
For increasing displacement and subsequent cycles at 2% drift, such energy dissipation remained stable, as testified by the force amplitude of the "flag" in the response curve, which remained constant for varying displacement. In the following cycles up to 2.5% drift, some stiffness degradation arose from the progressive concrete damage at the compressed toes, which progressively cumulated plastic deformation. For this cycles the vertical amplitude of the "flag" in the response curve slightly increased, showing an additional contribution to energy dissipation arising from the already mentioned damage at the wall toe.
In phase 2 ("without friction") dowels were activated at every cycles when the shear action reached a value of about 200kN. Figure 7 shows the maximum value of the base sliding displacement for the last 6 cycles applied and the corresponding value of shear action reached in the cycle.

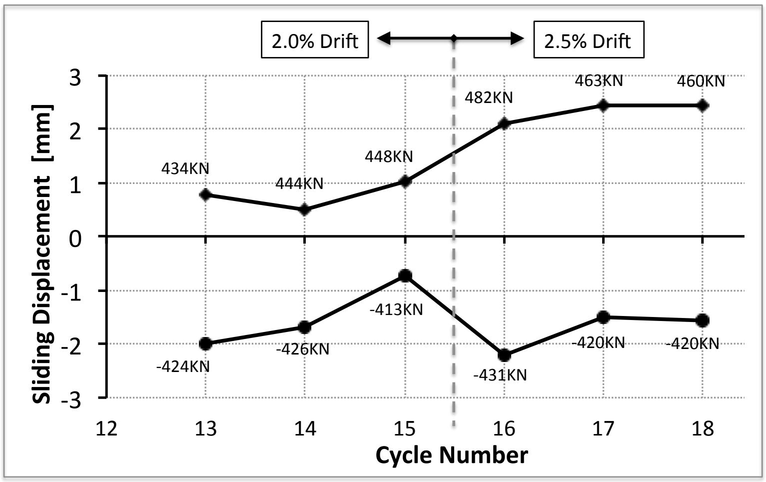



**Figure 7.** Maximum measured value of base sliding during cycles at 2% and 2.5% wall total drift and corresponding value of base shear action.

The base sliding displacement for the eight un-bonded dowel remained below 3mm during the whole test duration. The stability and repeatability of the response suggest that the dowel mechanism remained within the elastic stage, with negligible non-linear behaviour due to dowel yielding or concrete bearing stress plasticity.

### 4.1 Monolithic wall response

Figure 8 shows the Force-Drift response of the ensemble of cycles applied on the monolithic wall. The wall reached up to 2.5% drift without strength degradation. A significant pinching took place at load reversals in cycles of large displacement applied. The associated stiffness degradation was nearly completely due to the delayed cracks closing at load reversals, which was observed to occur at about 200kN of lateral force. In this stage the contribution of sliding to the global displacement was absolutely negligible. In fact the amplitude of the base displacement measured 250mm over the base remained below 2mm, two order of magnitude lower than the wall top displacement.

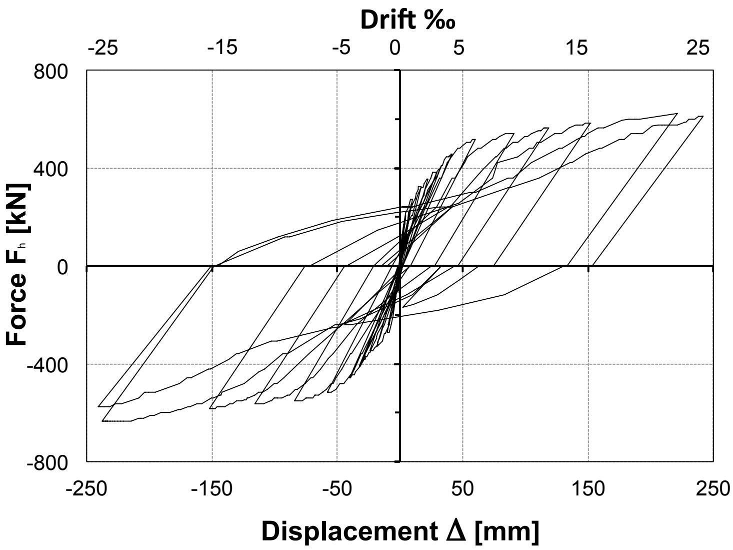



**Figure 8.** Force versus Displacement diagram of the ensemble of the fifteen cycles imposed on the test wall.

The uniform distribution of the vertical rebars along the wall cross-section length enabled a regularly spaced crack pattern and the spread of plasticity in the rebars along the eight of the wall, according to the measured deformation in the region of plasticity at the base of the wall (Fig.9). Only the most external vertical rebars suffered buckling, in particular where concrete spalling was extensive, whilst the remaining rebars, far from the region of maximum compression at the cross-section end, kept their bearing capacity against sliding, thanks to the negligible damage of the surrounding concrete.

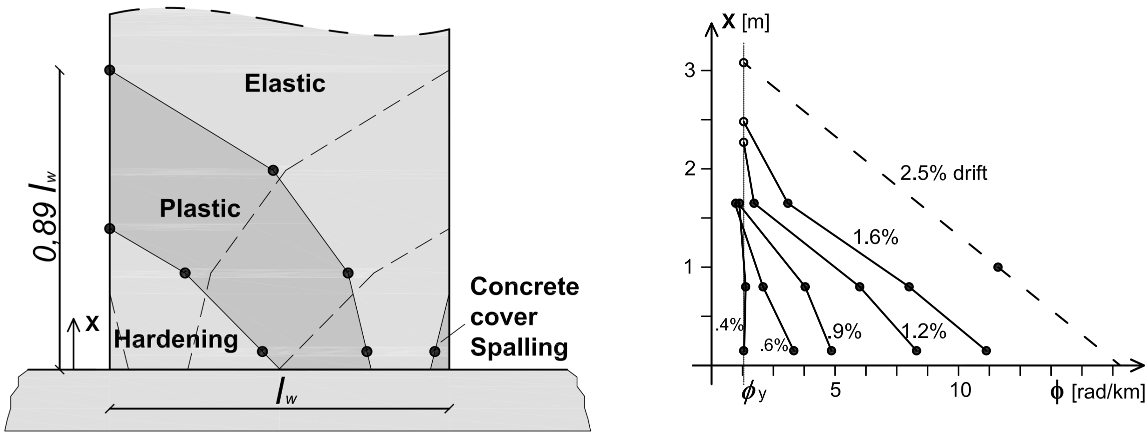



**Figure 9.** Extension of plastic and hardening areas in the critical region pictured at cycle XIII (1.6% drift) and average curvature measured along the wall height for increasing values of wall drift after yielding [Preti and Giuriani, 2011].

## 5. CONCLUSIONS

The results of the two test confirmed that dowel action is a reliable resource against shear sliding. In particular the dowel action is effective also at the open crack, offering sliding resistance during load reversals too, when friction may be deactivated.

The solution of unbounded dowels tested in the rocking wall showed to be effective. The dowels did not modify the deformation capacity of the wall, offered a source of energy dissipation and showed to be effective also for large crack opening, up to 38mm. Such solution can offer additional sliding resistance to both rocking and monolithic walls designed for a ductile behaviour, without introducing significant bending over-strength in the plastic hinge, which would recall additional seismic action in the dynamic response. In fact, the unbounded dowels allow uncoupling the sliding shear resistance from the bending one.

The full-scale experimental test allowed the measurement of local deformation in the plastic hinge, which are devoted to the calibration of the numerical modelling of structural walls.

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