Repair and retrofitting of structural RC walls by means of post-tensioned tendons

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ABSTRACT: The introduction of un-bonded post-tensioned tendons or bars in the critical zone for the structural repair and retrofitting of structural R/C walls is investigated by means of non linear FE analyses. The numerical models are validated through comparison against experimental data on a 1:1 scaled traditional shear walls undergoing cyclic loadings. Comparative pushover and seismic analyses were performed.
The results showed that the introduction of unbonded post tensioned tendons concentrates the damage in a single large crack opening at the wall base section, whereas traditional walls develop an extended crack pattern over the critical zone. Despite similar behaviour is observed in terms of top storey maximum displacement, in case of post-tensioned structural walls the elastic behaviour of the post-tensioned bars, while reducing the energy dissipation capacity of the structure, avoids any residual displacement after a seismic event, thus limiting structural damage even under a design seismic event.

1 INTRODUCTION

Following strong earthquakes, traditional R.C. structural walls may show severe damage characterized by an extended crack pattern along the critical zone and concrete cover spalling close to the base section, followed by the onset of longitudinal reinforcement buckling and the failure of some rebars (Riva et al. 2003). R.C. walls might as well develop a large crack at the base section, causing the shear force to be transferred across the element by dowel effect of the longitudinal rebars only. In this case, sliding shear failure might occur (Riva et al. 2003).

When the residual displacement and the damage are not as severe as to inhibit any further use of the structure, shear wall structural rehabilitation might be considered. To this end it is of outmost importance to identify and test feasible and economic retrofitting techniques.

As for the retrofitting techniques, following the shear wall earthquake damage scenario, reinforcement replacement is a hardly viable solution. Partial replacement of the rebars obtained by means of overlap splicing might suffer problems related to joint efficiency and confinement. Similarly, if reinforcement is substituted by welding or by clamping part of a new bar, sufficient ductility might not be safely and confidently attained. The results of recent experimental tests on a repaired wall in which the longitudinal reinforcement was partially replaced and connected to the existing one by means of mechanical couplers, showed that these devices lead to an anticipated collapse due to excessive bearing stress close to their end section (Riva et al. 2004).

In this paper, the repair and retrofit of traditional R/C walls by partial or complete substitution of the damaged reinforcement with unbonded post-tensioned reinforcement is studied. Post tensioned reinforcement is introduced in the critical zone only, together with an appropriate shear key device.

This solution might be regarded as an application of the shear wall typology developed for new constructions by Kurama et al. (1999, 2000). Kurama et al. (1999, 2000) introduced prefabricated panels placed one on top of the other and linked by post-tensioned unbonded reinforcement along the entire wall height. These walls exhibit the same initial stiffness and strength of traditional RC walls but maintain a bilinear elastic behaviour throughout the seismic excitation. This way, large displacements can be adsorbed without significant damage, despite having a small energy dissipation capacity. Deformability localizes along the structural horizontal joints between adjacent panels. The structural efficiency of these joints is fundamental for the correct global behaviour of the structures. Transferring shear between the panels or the wall and the foundation has been widely studied (Soudki 1995a, b; Soudki 1996).

A similar approach was adopted for the seismic design of prefabricated frame structures, in which structural components are tightened by unbonded post-tensioned tendons. The small energy dissipation of these structures was proved to be easily overcome by introducing viscous dampers (Pampanin, 2005).

In this paper, the seismic behaviour of structural walls with post-tensioned unbonded high strength bars is investigated and compared to the behaviour of traditional RC walls by means of non linear Finite Element analyses. Post tensioned reinforcements are introduced in the critical zone only in partial or total substitution of the ordinary reinforcement.

In the following, three different solutions are analyzed and compared, namely: (i) shear walls with post-tensioned rebars entirely substituting traditional reinforcements in the critical zone, thus requiring the introduction of special shear resisting devices to allow shear transferring to the wall base, (ii) shear walls with post-tensioned rebars partially substituting traditional reinforcements in the critical zone, thus partially preserving the continuous reinforcement transferring the shear to the wall base (iii) shear walls with traditional reinforcement.

Shear walls were modelled by means of fiber beam elements, implemented within the FE Code MIDAS/Gen (2005). The numerical model was initially validated through comparison with experimental data on the performance of a 1:1 scaled reinforced concrete shear wall subjected to cyclic loadings.

Following analyses were performed on single walls of different typologies, namely with or without unbonded post-tensioned reinforcement, subjected to both pushover tests and to artificial seismic records compatible with Eurocode 8 (2004).

2 DESIGN OF SHEAR WALL RETROFIT

Shear wall retrofitting with unbonded post-tensioned reinforcement is obtained by means of Dywidag® bars placed in the critical zone of the cast-in-place, existing RC element. The unbonded rebars are fixed to the wall foundation and post-tensioned through anchors buried, beyond the critic zone, in the floor concrete slab. Outside the critical zone, the detailing of the traditional RC cast-in-place shear resisting walls is usually left untouched.

Main design unknowns for the retrofitting design are: (i) the initial deformation of the post-tensioned reinforcement (ε_{sp0}); and (ii) the unbonded reinforcement cross section (A_{sp}). The latter is obtained by enforcing the equality between the resisting and design bending moments at the ultimate limit state. The minimum level of post tension can be determined by imposing a maximum crack opening (i.e. 0.4 mm) at the damage limit state. Furthermore, by imposing the elastic behaviour of the unbonded reinforcement even in the case of a design earthquake excitation ($\varepsilon_{sp}<\varepsilon_{spy}$), no residual deformations are to be expected after the earthquake. Post tension must be adequately increased to account for post-tension losses induced by steel relaxation and creep of concrete. Details on the design of the shear walls can be found in (Oldrati et al., 2004).

The post-tensioned rebars, grease coated and positioned in polymeric extruded sheathings, are placed into confined concrete zones next to the wall edges. Steel spiral hoops are embedded underneath the anchorage plates to ensure adequate concrete confinement.

3 NUMERIC MODELLING AND VALIDATION

Shear walls with either traditional or unbonded post-tensioned reinforcement are modeled by means of force-based fiber elements, based on the Timoshenko beam theory, implemented within the Finite Element Code MIDAS/Gen (2005). Force-based elements are computationally more demanding than displacement-based elements, but they offer the main advantage of being “exact” within the beam theory framework used for the formulation (Spacone et al. 1996). This leads to the use of one element per structural member (beam or column) in a frame analysis, thus requiring a lower number of nodal degrees of freedom. This results in the faster assembly of the numerical mesh, which might in turn be a significant advantage for practitioner engineers.

The single multilayer shear wall (Fig. 1a) is therefore modeled by means of a number of elements equal to the number of floors in the building (Fig. 1b). Each element has 5 control sections. A simple linear shear force-shear deformation law was used at the section level.

Post-tensioned rebars are modeled by introducing two extra non linear truss elements in the critical zone (Fig. 1b). The trusses are linked to the wall by means of rigid beams. Post-tension is simulated by applying an initial distortion to the truss elements.

As for the boundary conditions, the base node is considered as fixed in case ordinary reinforcement connecting the wall to the foundation is present, i.e. in case of traditional walls and in case of post-tensioned rebars partially substituting the traditional reinforcement (Fig. 1c). On the other hand, in shear walls with post-tensioned rebars only, the rocking mechanism must be allowed, thus the base section must be enabled to rotate. The base node is therefore hinged to the ground (Fig. 1d). To account for resisting actions developed by the compressed concrete at the edges, two nonlinear springs where introduced at the wall base section and rigidly linked to the central base node.

For ordinary concrete, a classical Kent-Park law is adopted. Confined concrete is described by Kent-Park law, followed by a bilinear curve accounting for ductility. For the ordinary steel, Menegotto-Pinto constitutive law is used, whereas unbonded high strength steel is described by a bilinear tension-only law. The material parameters are summarized in Table 1.

A first set of analyses showed that the structural response is affected mainly by the shear capacity, whereas it is basically independent of the shear stiffness. The structural stiffness is governed by bending up to failure.

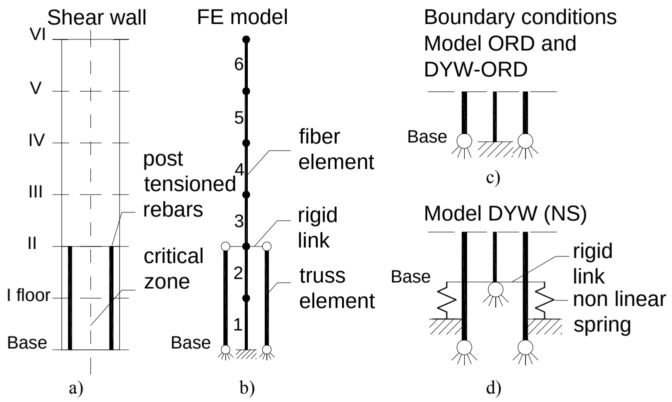


Fig 1. Model A – Force based fiber element mesh.

3.1 *Validation of the numerical model*

The numerical model (Model A, in the following) is initially validated through comparison against experimental data on the performance of a 1:1 scaled R.C. shear wall, shown in Figure 2, subjected to cyclic loading. The experimental study was carried out at the Laboratory of the University of Brescia. A detailed report of the test setup and results may be found in Riva et al. (2003). The experimental wall is shown in Figure 2a.

Model A results are also compared to the numerical shear wall response obtained by Oldrati et al. (2004), named Model B in the following.

In Model A, only three force-based fiber elements are used: one for the base segment, one for the critical zone, and one element extending to the wall top end (element 1-3, Fig. 2). In the analyses, shear capacity is assumed as equal to the force transmitted by the dowel action of the traditional reinforcement crossing the base section.

Model B refers to the analysis performed by Oldrati et al. (2004) with ABAQUS, using, for the r.c. wall, displacement based user defined fiber elements up to 11.5 m from the restrained end, and linear elastic beam elements beyond that section.

Mechanical properties for the materials, as well as the geometric characteristics of the elements used herein, are those reported in (Riva e al. 2003, Table 1). The same properties were used by Oldrati et al. (2004).

Experimental and numerical responses are shown in Figure 3. The comparison shows a good agreement of numerical and experimental responses in terms of cyclic behavior and resistance.

Table 1. Material properties.

	σ_{tu} ε_{tu}	σ_{tmax} ε_{tmax}	σ_{tc1} ε_{tc1}	σ_{cy} ε_{cy}	σ_{c1} ε_{c1}	σ_{cu} ε_{cu}
Confined Concrete	-0.145 -0.1%	1.45 -0.027%	23.5 0.107%	47.0 0.7%	47.0 2.46 %	4.0 2.71%
Ordinary concrete	-0.145 -0.1%	1.45 -0.27%	15.0 0.047%	30.0 0.2%	30.0 0.35 %	4.0 0.7%

	E_s	f_{sy} ε_{sy}	f_{sm} ε_{sm}
Steel	200000	500 0.25%	580 8%

	E_{sDYW}	σ_{syDYW}	σ_{suDYW} ε_{suDYW}
Post tensioned rebars	205000	1080.0	1230.0 8%

Note: σ [MPa], f [MPa], E [MPa]

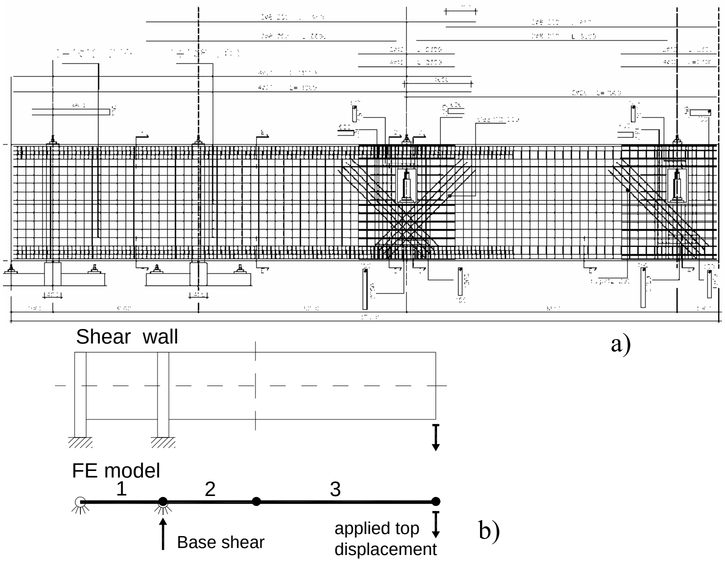


Figure 2. (a) Experimental 1:1 scale R.C. wall (Riva e al. 2000) and (b) numeric model.

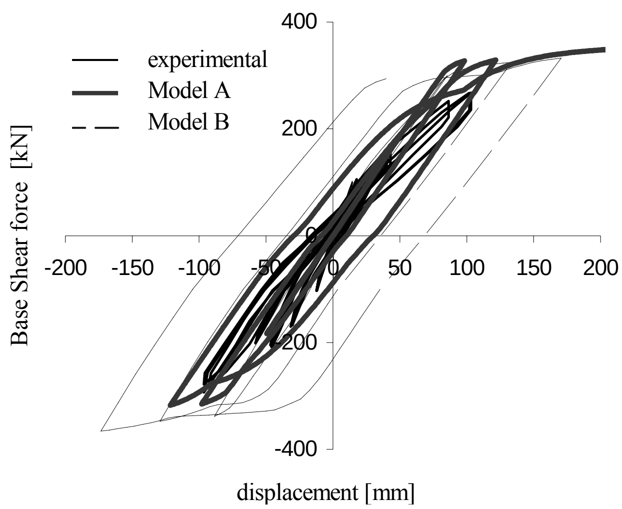


Figure 3. Numerical and experimental response.

4 ANALYSIS OF POST-TENSIONED SHEAR WALL

The behaviour of shear walls with post-tensioned, unbonded reinforcement stretching across the critical zone is investigated by means of finite element numerical analyses. The study refers to a single shear wall of a six storey building. The wall has a 0.3x3.5 m section, and has a height of 19.2 m. With reference to Figure 1, different models are used to study the three different critical zone reinforcement typology and layout; namely: (i) Model ORD: critical zone with traditional reinforcement (Fig. 4a); (ii) Model DYW-ORD: shear wall critical zone with unbonded rebars integrating the ordinary reinforcement at the base section (Fig. 4b). Ordinary reinforcement is proportioned to resist the shear force according to Eurocode 8 (2004) requirements; (iii) Model DYW: shear wall critical zone with unbonded rebars entirely substituting the longitudinal reinforcement. In this case, additional devices must be designed to allow shear force transfer across the base section. Beyond the critical zone, an ordinary reinforced concrete section is assumed.

Material properties are those described in the previous chapter. Six Dywidag® bars, having a diameter of 32 mm, were introduced at the wall edges in model DYW and four rebars in model DYW-ORD. Each unbonded rebar was post-tensioned to 120 kN. The choice of the post tension level is briefly discussed in section 4.2 and more extensively in Oldrati et al. (2004). The tendons were stretched from the base section to the second floor level (node 3 in Figure 1). Beside this default setting, the case with four Dywidag bars was also analyzed.

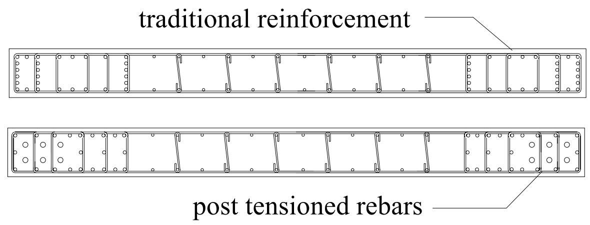
4.1 *Non linear "push-over" analysis*

A pushover analysis is performed by applying a monotonically increasing lateral displacement to the top storey. This pushover analysis follows a finite element investigation previously performed by Oldrati et al. (2004).

Bending moment versus base section rotation is plotted in Figure 5a for the analyzed shear wall typologies (Curve 1, 2 and 3, for the described reinforcement layout). Ordinary longitudinal reinforcement significantly affects the flexural behaviour of the shear wall. The adoption of unbonded post-tensioned rebars, partially substituting the traditional reinforcement (DYW-ORD), results in a significant increase in the ultimate bending moment when six bars are introduced (curve 6) and it is appreciatively the same when only 4 bars are introduced. The ultimate bending moment of the traditional wall (curve 1) is comparable to the flexural capacity of the wall without ordinary reinforcement (curve 2). Failure of the shear wall with unbonded reinforcement occurs when concrete reinforcement occurs when concrete overcomes its compressive strength, whereas traditional shear walls failure follows reinforcement yielding.

Considering this rebars layout, the yield moment (M_y) is maximum when no shear reinforcement is used (17290 kNm, curve 2), and decreases in case of shear wall with shear reinforcement (16790 kNm, curve 3). The minimum yield moment is reached for traditional shear walls (15240 kNm, curve 1).

Upon yielding of the unbonded bars, an abrupt change in the curve slope is observed. In case of shear walls having both traditional and post-tensioned rebars (DYW-ORD), the change in slope is less pronounced, as the yielding of unbonded reinforcement occurs after that of web reinforcement. On the other hand, traditional walls (ORD) show a smoother transition towards the plastic stage. This is the result of the progressive yielding of the reinforcement occurring along the cross section.



a)
b
)

Figure 4. Typical critic zone cross section for: a) traditional walls; b) reinforced shear wall with post-tensioned rebars.

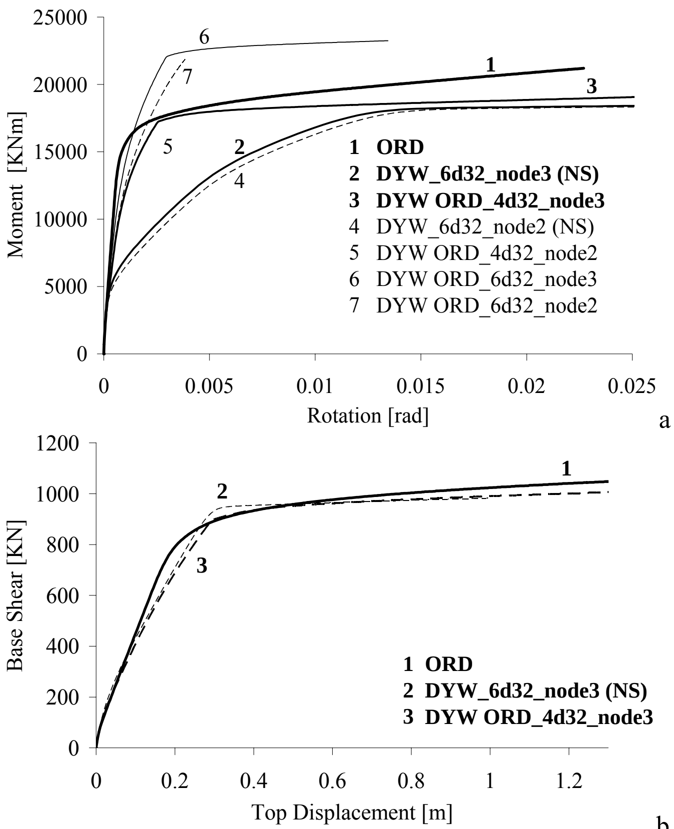


Figure 5. Pushover test: a) bending moment versus base section rotation; b) Base shear versus top displacement.

The upper floor maximum displacement is the result of both a rigid translation induced by the base section rotation (rotation times the wall height), and a displacement produced by the flexural deformation of the structure. Pushover analyses were performed by applying an increasing top displacement until concrete reached failure (Fig. 5b). This resulted in unreasonably high values of the total displacement, and the analyses fail to have a physical meaning. Considering a reference value of top displacement of 0.5 m, equivalent to a 5% drift, the base section rotation and the following displacement are listed in Table 2. It is worth noting that when lacking the shear reinforcement (DYW), the damage is localized at the wall base section. In this case, 76% of the total top displacement is induced by the base section rotation. The adoption of both traditional and unbonded rebars (DYW-ORD) limits this effect, despite damage localization is still evident.

Table 2. Influence of the base section rotation on the top displacement (δ = 0.50 m)

Model	base rotation ϑ_{nase} [rad]	Rigid displacement induced by base rotation . δ_{ϑ} [m]	δ_{θ}/ δ [%]
DYW-ORD	0.88E-02	0.17	34
DYW (NS)	2.00E-02	0.38	76
ORD	0.40E-02	0.07	14

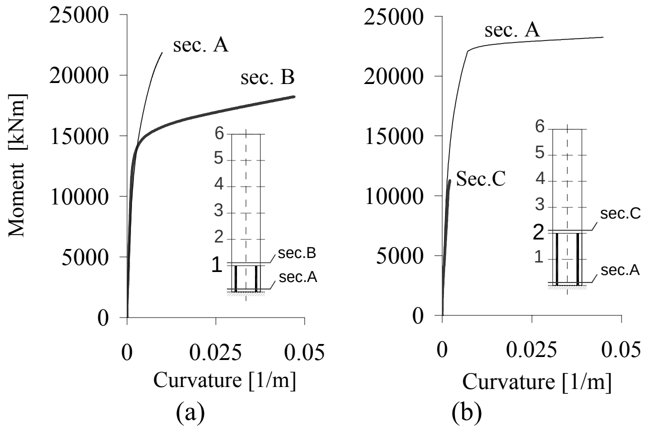


Figure 6. Moment vs curvature for DYW-ORD_6d32 models as a function of the post compressed zone length.

Pushover analyses highlighted that, when unbonded rebars integrate the traditional reinforcement, the critical zone might extend well beyond the first floor. The structural response is proved to significantly depend on the length of the wall post-compressed zone. To emphasise this aspect, DYW-ORD models having 6 post-tensioned rebars extending through the first (Fig. 5a, curve 7) or the second floor (Fig. 5a, curve 6) were analyzed. Figure 6 plots the moment versus sectional curvature for the two models (base section A moment values include the unbonded rebars contribution).

When the post compressed zone is limited to the first interstorey height, the base section behaves elastically throughout the analysis because the yielding moment is first reached outside the post compressed zone, where all damage is accumulated (Fig. 6a). On the other hand, when unbonded rebars are stretched up to the second floor, the element outside the post tensioned zone remain basically elastic and the damage accumulates around the base section (Fig. 6b).

4.2 *Dynamic analyses*

A EC8 (1997) compatible artificial acceleration time-histories having a peak ground acceleration (PGA) of 0.6g (5.88 m/s2) was used to perform dynamic analyses of the shear walls (Fig. 7). The large value of PGA is the result of selecting 0.35g PGA, an Importance Factor of 1.4 and a Type C Soil (Eurocode 8, 2004). Reduced PGA was used to verify the structure under damage limit state seismic excitation.

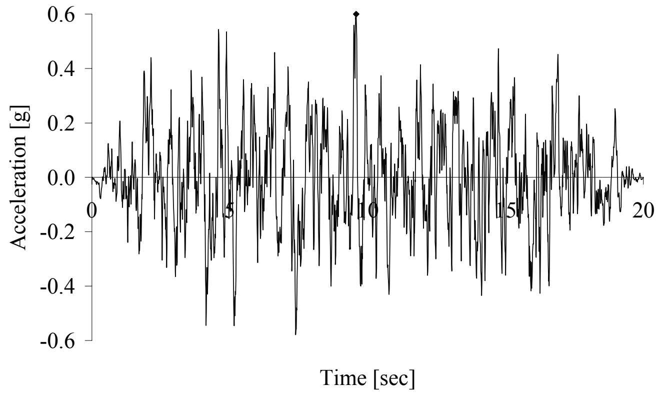


Fig.7. Adopted EC8 compatible earthquake.

A pilot parametric study clarified the effect of varying post-tension level (either by varying the tendon post-tensioning, or the tendon number) on the wall structural seismic response. The post-tension level affects the onset of the cracking process. For increasing post tension forces the structure cut the vibratory phenomena more easily, and it exhibits a larger energy dissipation capacity. This is due to the increase in the concrete stress level and in the possible Dywidag® bars plastic deformation. For increasing post tension, a delay in the development of the maximum displacements is also observed. Maximum displacements are approximately constant for post tension larger than 120 kN (Oldrati et al, 2004). It is worth noting that an excessive post tension might result in excessive concrete stresses and might jeopardize the capacity of the unbonded bars to allow the elastic recovery of the undeformed shape in ultimate conditions. Based on the parametric study, the optimal structure response was found when adopting 6 and 4 tendons for DYW and DYW-ORD models, respectively. Each Dywidag® rebar post tension level was set to 120 kN, equal to 15% of the yielding stress.

Base bending moment versus base section curvature is shown in Figure 8 for all structural typologies. The curves are characterized by a few large cycles where all energy dissipation is concentrated.

It is interesting, however, to note that pinching of the curves is very pronounced in case of unbonded post-tensioned rebars, exhibiting a smaller capacity to dissipate energy but, at the same time, a pronounced self centring behaviour.

Figure 9 shows the top displacement versus time, for the analysed structural typologies. The traditional shear wall (Fig. 9, ORD) shows a greater dissipation capacity with a prompt reduction of the oscillatory phenomena. As a drawback, a larger 12.4 mm residual displacement, caused by the yielding of the reinforcement, is observed.

In case of walls having post-tensioned rebars partially substituting the traditional reinforcement (Fig. 9, DYW-ORD) and tightened to the second floor level, the maximum top displacement is equal to 0.24m, similar to that of the ORD model, but residual displacements are almost null 10 seconds past the seismic event. Following the seismic event the structure recovers its undeformed shape.

It can be shown that, by limiting the post-compressed zone to the first floor, a residual displacement of 7 mm is recorded after the earthquake. The base section behaves almost elastically throughout the seismic event, whereas large plastic deformation accumulates right beyond the post-compressed zone, proving that the post-tensioned zone must be conveniently extended beyond the first interstorey height.

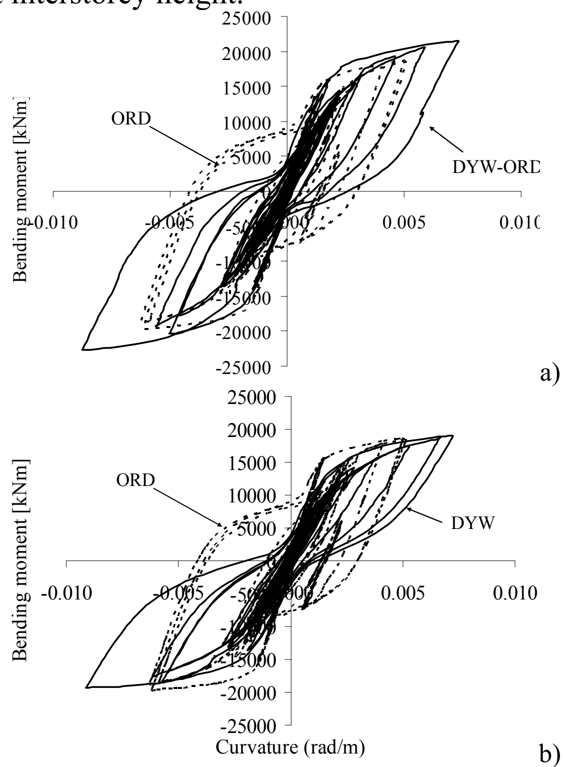


Figure 8. Moment versus base rotation curve for: (a) ORD and DYW-ORD; (b) ORD and DYW models.

Walls having unbonded rebars entirely substituting the traditional reinforcement (Fig. 9, DYW), show the largest top displacement (0.3 m), mainly induced by the large rotation at the wall base triggered by the rocking mechanism. The oscillatory phenomena reduce more slowly, showing a smaller energy dissipation capacity, but again no appreciable residual displacements are observed (residual displacement is equal to 20% of the traditional wall residual displacement).

Figures 10 and 11 show base shear force and bending moment versus time histories, respectively, for the analysed structural typologies. Both shear and bending moment are larger for DYW model.

Figure 12 illustrates concrete and reinforcement strain distribution at three different wall heights (namely: 0.50m, 1.6m e 2.7m from the base section). In case of shear walls with post-tensioned rebars, strain concentration is observed at the base section, compatibly with the large rotation induced by the rocking motion; whereas the traditional wall spreads the strain along the critical zone depth, compatibly with a cantilever resisting mechanism.

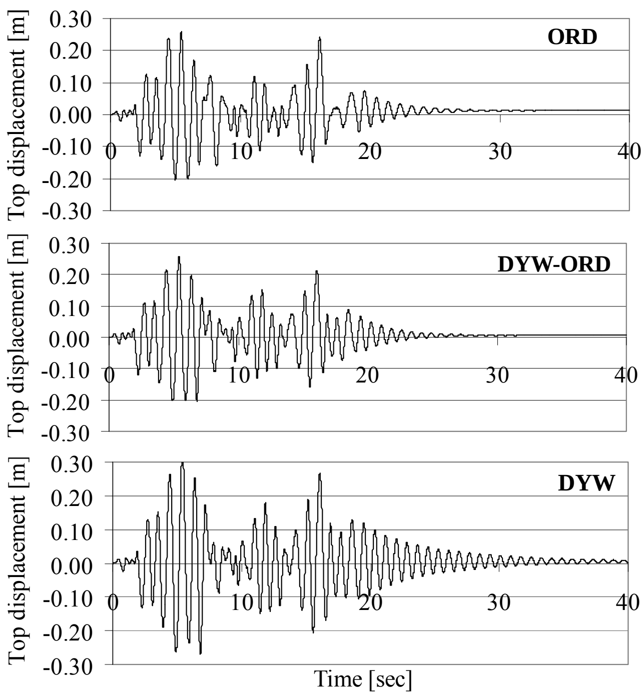


Figure 9. Top displacement versus time for varying critical zone typologies.

In DYW model, damage, in terms of concrete and reinforcement stress level, is confined near the base section because of the prevalence of the base rotation over the flexural deformed shape. Slightly above the base section and beyond, reinforcement behaves elastically and the maximum concrete stresses are never larger than 23 MPa. In DYW-ORD model, damage extends up to 2.5 m above the base section, with traditional reinforcement overcoming the yield stress and maximum concrete stresses equal to 40 MPa. Traditional walls (ORD) exhibit severe damage over the investigated height due to the yielding of the reinforcement, which results in a large residual displacement. In ORD model concrete stress is limited.

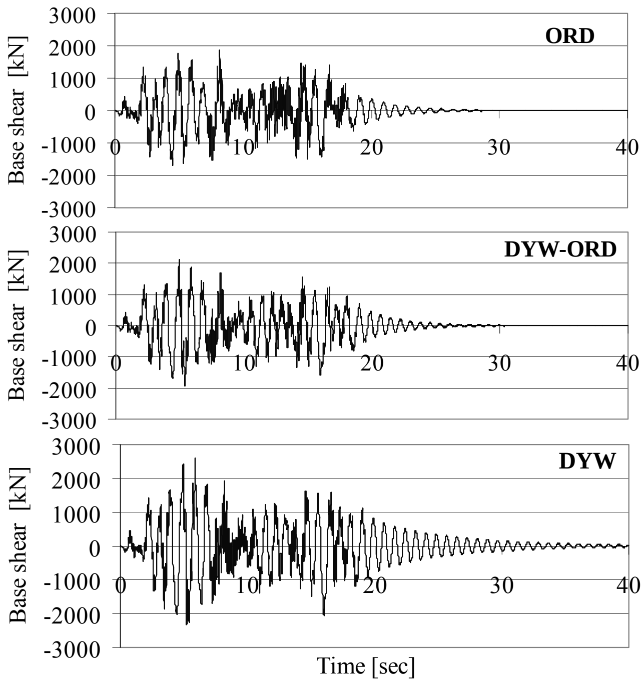


Figure 10. Base shear force versus time for varying critical zone typologies.

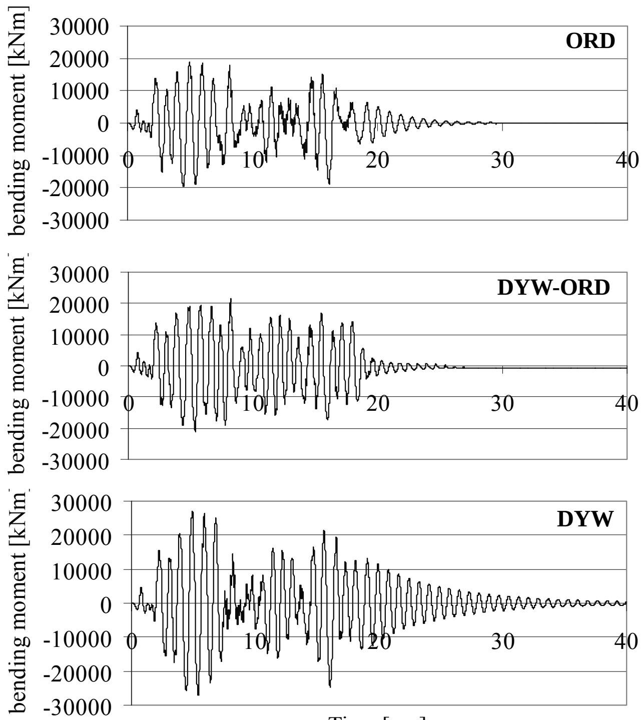


Figure 11. Bending moment versus time for varying critical zone typologies.

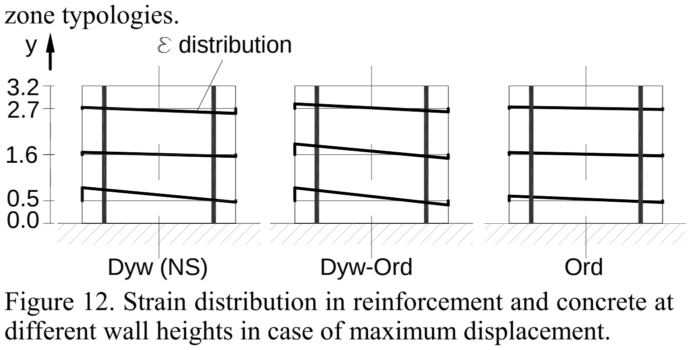


Figure 12. Strain distribution in reinforcement and concrete at different wall heights in case of maximum displacement.

Concrete confinement is therefore required anytime unbonded post-tensioned rebars are adopted.

Further analyses allowed verifying that in case of moderate earthquakes (PGA equal to 0.15g) all models behave elastically, with limited cracking extending over the critical zone. For increasing PGA values (0.25g), plastic behaviour of the shear reinforcement and larger crack openings can be observed in traditional walls. Walls lacking traditional reinforcement show larger top drifts, mainly induced by the base rotation.

It is worth noting that, in order to reduce the top wall drift, and to cut the oscillatory phenomena more rapidly, additional dissipating devices might be introduced (Kurama 1999).

The behaviour of the three wall typologies at the damage limit state was also investigated. Damage was evaluated by analysing the concrete and steel stress level, as well as the crack opening. In case of shear walls having post-tensioned rebars entirely substituting the traditional reinforcement, the onset of a large crack at the base can be observed as a result of the large rotation required by the rocking mechanism. On the other hand, in case of both post-tensioned rebars partially substituting the traditional reinforcement and in case of ordinary shear walls, diffuse cracking is observed. Crack opening can be calculated by assuming a crack spacing equal to the spacing of the stirrups along the confined zone. The crack opening values obtained by the analyses and the estimate provided by EC2 (2004) for traditional walls are listed in Table 4. It is worth noting that the 0.4 mm crack opening limit value assumed in the proportioning of the shear wall at the damage limit state, is verified by traditional shear wall only. However, the slightly larger crack opening obtained in case of walls having post-tensioned rebars in partial substitution of the reinforcement is not impairing the structural functionality at the damage limit state.

Table 3. Crack opening at the damage limit state.

Models	Maximum strain	Concentrated crack [mm]	Distributed crack [mm]
DYW-ORD	0.00296	1.5	0.45
DYW	0.01741	8.7	-
ORD	0.00178	0.9	0.27 (0.23 - EC2)

5 DISCUSSION AND CONCLUDING REMARKS

Following strong earthquakes, traditional R.C. structural walls may show large residual displacements and significant damage extended along the critical zone, with concrete cover spalling, as well as buckling and failure of some rebars. However, when residual displacements and damage are not as severe as to prevent any further usage of the structure, shear wall structural rehabilitation might be considered.

In this paper, the repair and retrofitting of traditional R.C. walls by means of unbonded post-tensioned reinforcements was investigated. Post tensioned rebars were introduced in partial or total substitution of the traditional reinforcement in the critical zone only.

The level of post-tension was evaluated by enforcing crack opening to be smaller than 0.4 mm at the damage limit state, whereas the post-tensioned rebar area was proportioned to ensure the necessary resisting bending moment at the ultimate limit state. Concrete confinement followed the EC8 detailing recommendations.

When subjected to earthquake loadings, unlike traditional R.C. walls, the unbonded, post-tensioned structure are characterized by: i) the elastic behaviour of the unbonded tendons, which results in negligible residual displacement, thus in the recovery of the undeformed shape; ii) the confinement of the damaged zone. The structural response of shear walls with unbonded post tensioned rebars entirely substituting traditional reinforcement is characterized by a concentrated rotation, resulting in a discrete large crack opening at the base section, induced by the rocking motion. In case of partial substitution of the reinforcement, the crack pattern shows both a large crack at the base and a diffuse crack pattern which extends to a limited zone above the base section.

Given the localization of the damage and the recovery of the undeformed shape after a seismic event, small confined restoration works aimed at restoring the damaged concrete, re-tensioning the unbonded rebars and retrofitting the base shear resistance might be sufficient.

As a drawback, the adoption of this retrofitting structural solution results in a smaller energy dissipation capacity and in larger compression stresses in the concrete. For this solution to be the adopted, special devices ensuring the confinement of the concrete at the base edge, as well as dampers improving the energy dissipating capacity should be introduced.

It is worth noting that the significant upgrade of the base section resistance might cause the critical section to "migrate" to the second floor level. Accordingly, tendons should be stretched beyond the new critical zone in order to avoid anticipated failure of the post tensioned structural wall.

Post-tensioned walls might exhibit a slight increase of the inter-storey drift. This phenomenon, which might adversely affect non structural elements in the building, should be further studied for this solution to be applied.

Future development of the research will focus on the experimental behaviour of retrofitted shear walls, as well as on the design and testing of all structural detailing necessary for the correct structural behaviour.

REFERENCES

Eurocode 2 (2004). Design of concrete structures - Part 1-1: General Rules, and Rules for Buildings, EN 1992-1-1:2004, European Committee for Standardization, December 2004.

Eurocode 8. 2004. Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings. En 1998-1:2004, *European Committee for Standardization*.

Kurama, Y., Sause, R., Pessiki, S. and Lu, L. W. 1999. Lateral Load Behavior and Seismic Design of Unbonded Post-Tensined Precast Concrete Walls, *ACI Structural Journal* 96 (4): 622-632.

Kurama, Y., Sause, R., Pessiki, S. and Lu, L. W. 2000. Seismic Design of Unbonded Post-Tensined Precast Concrete Walls with Supplemental Viscous Damping, *ACI Structural Journal* 97(4): 648-658.

MIDAS/Gen. 2005. *Analysis Manual for MIDAS/Gen,* Midas information technology Co., Seohyenon-dong.

Oldrati M., Riva P. and Marini A. 2004. Analisi sismica di un edificio con pareti in c.a. con armature post-tese non aderenti. *XI Congresso Nazionale "ANIDIS L'ingegneria Sismica in Italia"*, Genova 25-29 gennaio 2004.

Pampanin, S. 2005. Solutions for high seismic performances of precast/prestressed concrete buildings, *Journal of advanced concrete technology* 3(2): 207-223.

Riva, P., Meda A., and Giuriani, E. 2003. Cyclic behaviour of a full scale R.C. structural wall, *Engineering and Structures* 25: 835-845.

Riva, Meda, A., Giuriani, E. 2004. Experimental test on a full scale repaired r.c. structural wall, *13th World Conference on Earthquake Engineering*, Vancouver, August 1 - 6.

Soudki K. A., West, J. S., Rizkalla, S. H., Blackett, B. 1996 Horizontal connections for Precast Concrete shear wall panels under cyclic shear loading, *PCI Journal*, September-October: 65-76.

Soudki K. A., Rizkalla, S. H., LeBlanc, B. 1995 a. Horizontal connections for Precast Concrete shear walls subjected to cyclic deformations. Part1: Mild steel connections, *PCI Journal* 40 (3): 78-96.

Soudki K. A., Rizkalla, S.H., Daikiw, R.W. 1995 b. Horizontal connections for Precast Concrete shear walls subjected to cyclic deformations. Part2: Prestressed connections, *PCI Journal,* September-October: 87-96.

Spacone, E., Filippou, F.C., and Taucer, F.F. 1996. Fibre beam-column model for nonlinear analysis of R/C frames. Part I: formulation, *Earthquake Engineering and Structural Dynamics* 25: 711-725.