EVALUATION OF THE IN-PLANE SEISMIC PERFORMANCE OF STONE MASONRY WALLS

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Summary

Although traditional historic masonry walls can be viewed as unsuitable structures to undergo seismic actions, they, in fact, exist and frequently represent the major structural elements of ancient buildings. Brick unreinforced masonry walls have been widely studied both from experimental and numerical point of view, but scarce experimental information is available for stone masonry walls. Therefore, the present work aims at increasing the insight about the behavior of typical ancient masonry walls under cyclic loading. Besides the strength and stiffness characterization, information about nonlinear deformation capacity is provided in terms of ductility factor and lateral drifts, which represent a step forward for the new concepts of performance based design.

Keywords: ductility, energy dissipation, lateral drift, performance based design, unreinforced masonry walls (URM).

1. Introduction

Unreinforced stone masonry walls were in the past the most relevant structural elements used in the construction of monumental and traditional buildings in the northern region of Portugal. These structural elements play a major role on the seismic response of the whole structure since they represent the basic resisting elements to horizontal seismic actions. In the absence of out-of-plane loading, the shear resistance of the walls can be prevailing on the stability conditions of masonry structures and, therefore, contributes to avoid their collapse. Although the northern region of Portugal has been classified as low to moderate seismicity zone, the assessment of the resisting and deformation conditions of the existing typical structural elements becomes of relevance due to the need of rehabilitation and retrofitting of ancient structures. On the other hand, the understanding of the seismic behavior of masonry stone walls represents an additional advantage in the perspective of new masonry structures design. This work aims at obtaining a better insight on the cyclic behavior of masonry stone walls that are frequently found in typical ancient structures. In order to attain this goal, twenty-four experimental cyclic tests were carried out at University of Minho using three distinct typologies regarding the textural arrangement of the stones in the walls. The experimental characterization focuses on the definition of the failure modes and force-lateral displacement hysteresis diagrams. Subsequently, the gathered data is analyzed in the scope of performance based design, where parameters like ductility and nonlinear deformation capacity represented by acceptable lateral drifts are key issues.

2. Experimental procedure

2.1 Geometry of the walls

As was aforementioned, the main goal of this work consists in the evaluation of the seismic performance of the masonry stone shear walls representative of ancient masonry structures. For this purpose, three distinct granitic textural typologies were adopted, as shown in Figure 1. The dimensions were selected so that suitable scaled specimens could be tested according to the available equipment existing at the laboratory of structures of University of Minho. The height to length ratio was 1.2 and the wall width was 200 mm. Walls WI and WR were constructed by introducing low strength mortar in head and bed joints, and built by the same experienced worker. Dry joints were considered in walls WS.

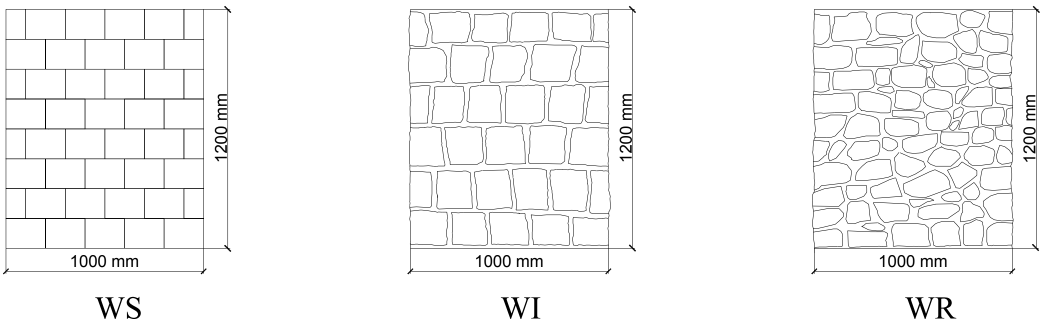


Figure 1: Geometry and texture of the walls, using S (sawn), I (irregular) and R (rubble) units

2.2 Testing setup

The static cyclic tests were carried out for three distinct pre-compression levels (σ_0=0.5N/mm^2, σ_0=0.875N/mm^2 and σ_0=1.25N/mm^2) by using the typical testing setup depicted in Figure 2. The cantilever wall was fixed to the reaction slab through a couple of steel rods. The pre-compression loading was applied by means of a vertical actuator with reaction in the slab given by the cables. A stiff beam was used for the distribution of the vertical loading and a set of steel rollers were added to allow relative displacement of the wall with respect to the vertical actuator. The seismic action was simulated by imposing increasing static lateral displacements according to the history indicated in Figure 2, by means of a hinged horizontal actuator appropriately connected to the reaction wall. The deformation of the wall was measured by means of the LVDTs indicated also in Figure 2.

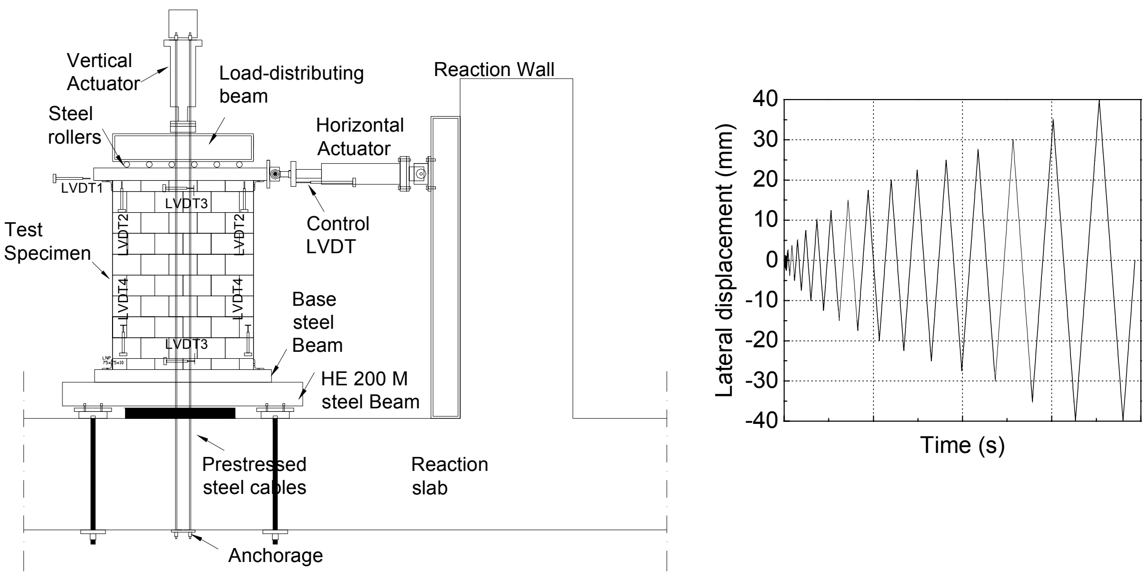


Figure 2: Testing setup, transducers and lateral displacement history

3. Results

The results of the cyclic static tests are analyzed firstly in terms of failure modes and qualitative analysis of the force-displacement hysteresis diagrams, and then based on the values of shear strength, lateral stiffness and deformation characteristics, namely ductility factor and lateral drifts corresponding to distinct performance levels.

3.1 Failure modes

Among the different types of tested masonry walls, distinct crack patterns and failure modes were found. In dry masonry walls WS, similar deformation mechanisms based on the occurrence of stepped flexural cracks govern the response under cyclic loadings. Behind the opening of the stepped diagonal crack, rocking mechanism develops and depending on the pre-compression, this could result in sudden collapse by toe crushing. In the cases where only rocking mechanism occurs, the final crack patterns are characterized by head joint cracks associated to inelastic sliding of the stones along the bed joints. However, no damage of the stones was recorded even for the higher levels of initial stress level. These crack patterns correlate reasonably well with the results obtained by [1] through limit analysis. In the same manner, the failure patterns attained in the walls WI and WR depend on the pre-compression level, similarly to what was reported previously by other authors [2-4]. Although with higher level of scattering, flexural response associated to the opening of horizontal cracks in the bed joints at the base of the wall and the associated rocking mechanism are typical of wall specimens WI and WR subjected to low pre-compression levels. The combination of the rocking mechanism with the opening of diagonal stepped cracks is also characteristic of the walls WI submitted to intermediate levels of pre-compression (σ_0 = 0.875N/mm^2). For the higher levels of pre-compression, shear cracks associate with rocking and toe crushing occurs. Finally, the failure patterns of walls WR submitted to pre-compression levels of 0.875 and 1.25N/mm^2 are characterized by shear cracks and significant development of secondary stone-mortar interface cracks in the middle of the walls and progressive damage in the stones.

To sum up, three basic failure modes were recorded to occur in masonry stone walls under cyclic loading. Thus, for low to moderate pre-compression levels, the behavior under cyclic loading is governed by flexural response characterized by cracks in the bed joints at the base of the wall or spreading along the height of the wall. In general, this failure mode does not lead to collapse of the structure subjected to in-plane loading, even if a combination of in-plane and out-of-plane loading would probably lead to premature collapse. Collapse limit sates can be attained by shear cracking and rocking mechanism.

3.2 Load-displacement diagrams

Some examples of the load-displacement hysteresis diagrams are exhibited in Figure 3. Here, the number 100, 175 and 250 indicate the vertical load in kN, producing a stress of 0.5, 0.875 and 1.25 N/mm^2, respectively. In general, the shape of these diagrams reflects to great extent the failure modes briefly discussed in the previous section. The typical load-displacement diagram of the walls WS is represented by the one obtained for wall WS1.100. Apart from the distinct maximum values of the lateral resistance and lateral displacement, all walls exhibit significant nonlinear response to the cyclic loading. The inelastic displacements are attributed to the bed joint sliding of the stones adjacent to the flexural and diagonal stair stepped cracks that developed during the reversal loading. These walls present high level of lateral displacement and simultaneously lower capacity to dissipate energy, with almost no degradation of its stiffness. Despite the fact that the rocking mechanism governs the response of irregular walls submitted to low and intermediate pre-compression levels, the hysteresis loops reveal that large amount of dissipated energy is present in this type of walls, see W2.175. Analogous tendency is observed by the irregular walls under the higher level of vertical stress, which additionally, exhibit significant stiffness degradation in the post-peak regime. The hysteresis response of the walls WR indicates dissipative features after peak load is reached, namely post-peak stiffness degradation and further deformational capacity after cracking loading (f_{cr}). It should be mentioned that walls WR2.250 and WR3.250 present somewhat asymmetric behavior regarding the maximum lateral resistance. This fact can be attributed to the intrinsic heterogeneity of the textural arrangement due to variation in workmanship.

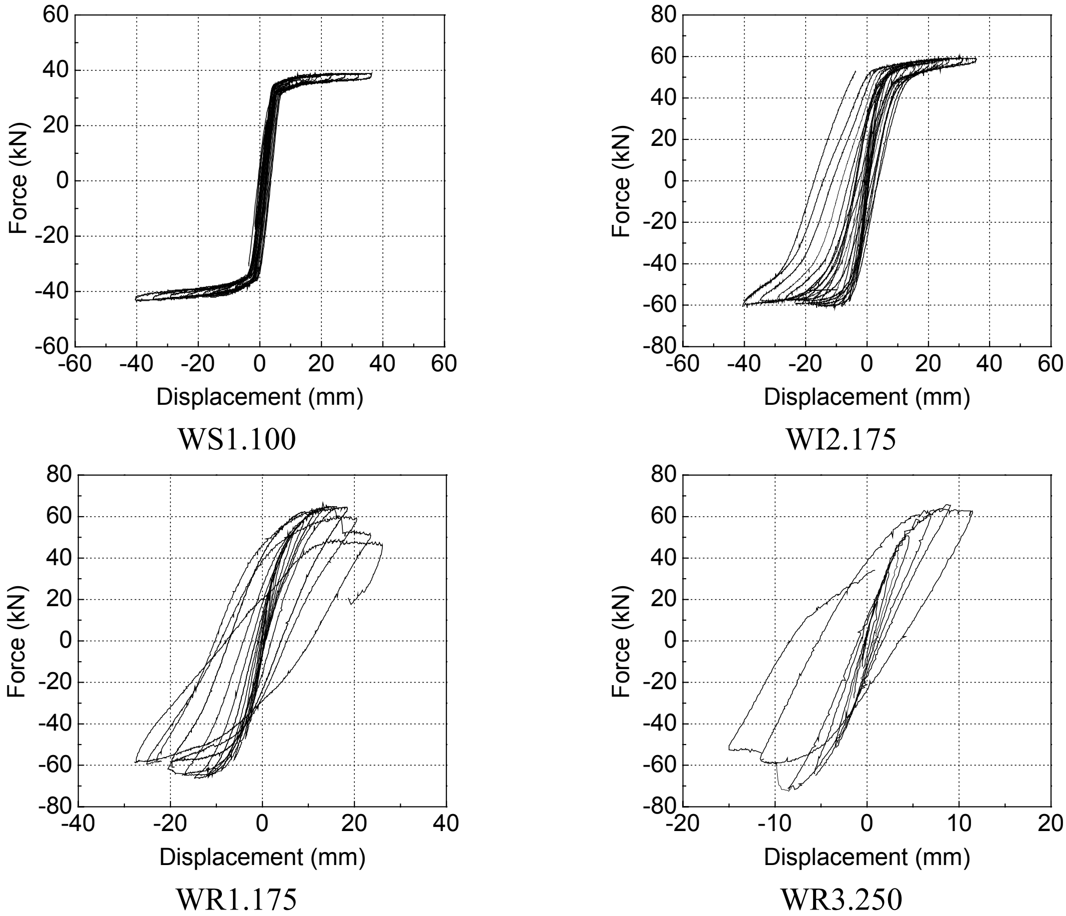


Figure 3: Load-displacement hysteresis diagrams

3.3 Performance analysis

The combination of the experimental data provided by the analysis of the failure patterns and the hysteresis loops allows the definition of sets of values (force, displacement) that are linked to specific damage levels, identified during the application of reversal cyclic loading simulating the seismic action. Each damage level defines a certain limit state or, as recently defined, performance level. In the scope of performance based design concepts, three performance levels are usually taken into consideration: Immediate Occupancy (IO) associated to minor extents of cracking, Life Safety (LS) characterized by substantial damage but with considerable amount of its original stiffness and Collapse Prevention (CP) corresponding to extreme damage leading to collapse if further deformation beyond this point is applied. Besides, the first cracking state is also taken into account. These performance levels are herein identified, respectively, with the force and displacement associated to the formation of the first crack (f_{cr}, δ_{cr}), shear cracking (f_s, δ_s), the maximum lateral force (f_{max}, δ_{Fmax}) and the maximum displacement ($f_{\delta max}$, δ_{max}), see Figure 4. In general, the performance levels are expressed in terms of lateral drift [5], which means that besides the strength and stiffness, the characteristics of deformation have to de determined. In conjunction with the lateral drift (δ/h), the ductility factor, μ, is also a parameter that provides insight about the seismic performance of the masonry stone walls under study and further assessment of its suitability to seismic zones in the scope of the performance based new concepts. The quantitative assessment of the performance of the walls is carried out by taking into account a bilinear idealization of the cyclic envelop, which has been a common approach in terms of design and performance assessment [6]. It is defined by means of the secant stiffness, k_e, at the formation of cracks calculated as f_{cr}/δ_{cr}, and the ultimate resistance load, f_u. Once the effective stiffness is available, the ultimate load is obtained taking into consideration that in terms of energy dissipation, the bilinear diagram should be equivalent to the experimental cyclic envelop. This is accomplished by ensuring equal areas under the bilinear and experimental envelops [3]. As can be seen in Figure 4, the ultimate idealized displacement, δ_u is defined as the displacement where the idealized bilinear diagram intersects the descending branch of the experimental envelop. The ultimate ductility factor is defined as the ratio between the ultimate idealized displacement and the idealized elastic limit, $\mu=\delta_u/\delta_e$.

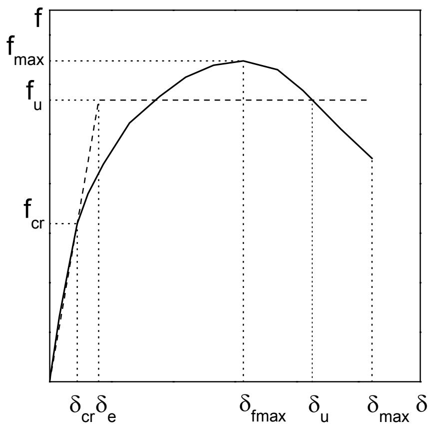


Figure 4: Cyclic envelope and the bilinear idealization

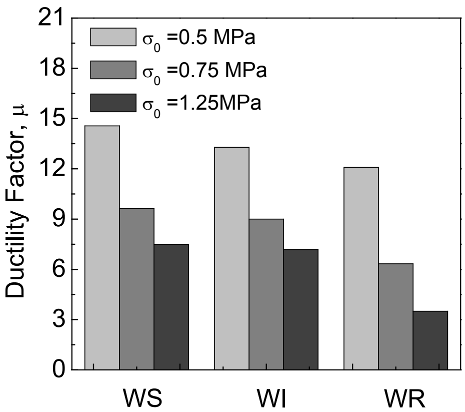
The characteristic values of the experimental envelop, which was considered for the positive part of the hysteresis loops, and the parameters that define the corresponding bilinear idealization are summarized in Table 1. The number of tested specimens for each type of wall is indicated values inside brackets. For walls WR, the lateral force corresponding to the opening of diagonal shear crack is referring to the negative hysteresis loops. These results allow interesting conclusions about the shear behavior of the masonry stone walls. No significant differences were found for the maximum lateral resistance, f_{max}, exhibited by the different types of wall for low and intermediate pre-compression levels. Nevertheless, for the higher level of pre-compression, the walls WR undergo a reduction in its lateral resistance. The reduction is particularly significant in Wall WR1.250 that presents smeared vertical cracking since early stages of loading, meaning that the compression stresses can govern its shear behavior. The statistical fitting to the experimental data composed by the values of the maximum lateral resistance obtained for the walls WS leads to a linear correlation $\tau=0.39\sigma$ and $\tau=0.36\sigma$, where τ is the normalized shear force and σ is the vertical pre-compression stress level. These relations are rather close the expression found by Oliveira [7] for stone masonry walls under monotonic loading. Apart from walls WR.250, in the remaining walls, the lateral resistance increases as the level of pre-compression increases. Lateral stiffness depends clearly on the level of pre-compression for all types of wall. Higher levels of pre-compression are associated to larger values of stiffness. This finding is in agreement with the results reported in [3]. Similarly to the lateral resistance, no significant differences in stiffness were recorded among the distinct types of walls. The discussion about the values of ultimate ductility factor is performed in next section.

Table 1: Characteristic parameters of the cyclic behavior

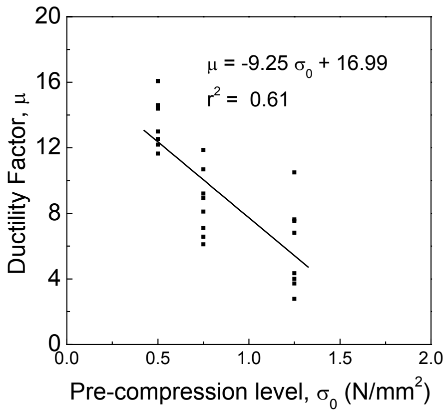
Wall	f_{cr} (kN)	δ_{cr} (mm)	k_e (kN/mm)	f_s	δ_s	f_{max} (kN)	δ_{Fmax} (mm)	δ_{max} (mm)	f_u (kN)	δ_e (mm)	μ
WS.100 (4)	17.99	1.48	12.54	-	-	36.94	12.89	32.69	34.93	2.85	14.56
WS.175 (3)	25.97	1.45	17.88	-	-	62.88	22.66	32.31	60.02	3.36	9.64
WS.250 (3)	33.56	1.76	19.07	-	-	86.31	24.30	32.31	81.18	4.29	7.49
WI.100 (2)	20.75	1.6	13.07	-	-	37.62	13.82	37.40	36.83	2.83	13.29
WI.175 (3)	28.76	1.83	15.74	-	-	55.72	19.75	29.20	51.75	3.29	9.00
WI.250 (2)	39.41	1.92	20.63	65.97	6.60	82.99	18.09	25.66	73.75	3.59	7.18
WR.100 (2)	17.14	1.49	11.52	-	-	37.02	14.22	34.81	34.36	2.89	12.09
WR.175 (2)	27.33	1.79	15.28	55.98	8.89	63.14	12.94	24.16	57.12	3.81	6.33
WR.250 (3)	33.13	1.75	18.91	68.76	5.45	65.97	6.99	10.29	58.87	2.94	3.50

3.3.1 Ultimate ductility factor

The capacity of shear walls to deform nonlinearly beyond the peak strength without pronounced strength loss or stiffness degradation is evaluated by means of the ductility factor. Therefore, the ductility factor is a useful measure that makes possible the evaluation of reduction of elastic seismic actions in the seismic design by means of a behavior factor, since it gives indications about the ability of the structure to dissipated energy [6]. In terms of performance based concepts, it is usually taken as the force reduction factor that reduces expected elastic levels of the base shear strength to acceptable design levels [5,8]. The evaluation of the ductility of the masonry stone walls is performed based on the values indicated in Table 1. It is stressed that the conclusions provided must be adopted with the necessary precaution due to the reduced number of tested specimens. Figure 5a summarizes the mean values of the ductility factor for the different types of walls and for distinct levels of pre-compression.



(a)



(b)

Figure 5: Analysis of the ultimate ductility factor; (a) for distinct type of walls; (b) for different levels of pre-compression

Additionally, it is possible to verify that for all levels of pre-compression there is a tendency for the ductility decreases from the walls WS to walls WI and WR. The reduction is quite moderate in case of low to medium pre-compression levels, but rather significant for the walls WR submitted to the higher levels of pre-compression (σ= 1.25N/mm^2). This is the result of similar failure patterns governed by the flexural response (rocking mechanism) for low levels to moderate levels of pre-compression verified for the distinct types of walls, whereas shear compression failure mechanisms rule the behavior under higher levels of compression, leading to a more brittle response. It can also be observed that ductility factor depends clearly on the level of pre-compression. For all walls, the ductility factor decreases significantly with the increase on the pre-compression. This dependency can also be confirmed through the rough linear fitting to the experimental data depicted in Figure 5b. In spite of the scatter (related to the data concerning the three types of walls), a clear tendency for the ultimate ductility decreases as the pre-compression level increases was found. Similar results were pointed out by [3]. The dependence of ductility capacity on the axial load ratio was also referred by [8]. If on one hand, the level of pre-compression leads to higher values of the lateral resistance under cyclic loads, on the other hand, it yields more brittle responses and, thus, to lower capacity to deform nonlinearly.

3.3.2 Deformation capacity – maximum drifts

As was aforementioned, in terms of performance based concepts, the evaluation of the suitability of masonry structures to resist lateral actions is based on the assessment of performance levels based on the lateral deformation capacities. Besides the ultimate ductility factor, the capacity of nonlinear deformation is also evaluated from the lateral drifts connected to each performance levels previously considered. As can be seen from Table 2, where the average values of the lateral drifts are displayed, high lateral deformation capacity was found for all types of walls and for distinct levels of pre-compression. As expected, the larger values are associated to rocking failure patterns, which occurs in a pronounced manner in walls WS and for low levels of pre-compression.

Table 2: Lateral drifts for the performance levels

Wall	FC	IO	LS	CP
WS.100	0.12%	-	1.18%	3.17%
WS.175	0.12%	-	1.89%	2.69%
WS.250	0.15%	-	2.02%	2.69%
WI.100	0.13%	-	0.91%	3.12%
WI.175	0.15%	-	1.65%	2.43%
WI.250	0.16%	0.55%	1.51%	2.14%
WR.100	0.12%	-	1.19%	2.90%
WR.175	0.15%	0.74%	1.19%	2.31%
WR.250	0.15%	0.45%	0.58%	0.86%

Lateral drifts of approximately 2.5% were reported by [5] in unreinforced masonry piers, whose response under lateral cyclic loads was governed by rocking. The lateral displacements obtained in dry masonry walls are also well correlated with the results reported by Oliveira [7] with respect to in-plane monotonic behavior of dry masonry walls. Maximum lateral drifts ranging between 2.25 and 3.0% were recorded. Similar levels of maximum lateral drift were found for walls WS and WR subjected to low level of pre-compression of $\sigma = 0.5\text{N/mm}^2$. Decreasing, but even considerable high values of the lateral drift were obtained for high level of pre-compression. These values confirm that rocking mechanism play the major role in the failure patterns under reversal cyclic loading. The large ultimate lateral drifts are also associated to sliding and interlocking mechanism developed in the opening cracks. Walls WR subjected to high level of pre-compression of $\sigma = 1.25\text{N/mm}^2$, exhibit lateral drifts close to the ones indicated by Magenes and Calvi [9] for masonry brick walls failing in shear. In the scope of performance based design for masonry structures it is even advisable that behavior of walls and piers is governed by rocking or bed joint sliding. Despite rocking behavior is poor in energy dissipation, due to its highly nonlinear elastic, it is, in general, associated to large nonlinear deformation capacity, without relevant damage [5]. If this is true for dry masonry shear walls, in the case of irregular masonry stone walls subjected to intermediate pre-compression levels, rocking is frequently associated to diagonal cracks whose opening thickness increases with lateral imposed displacements leading to hysteretic behavior with high stiffness degradation and inelastic deformation. On the other hand, shear failure patterns and toe crushing are force controlled processes, occurring when a certain stress is attained and causing sudden and large strength deterioration, which happens more frequently in walls WR.

4. Conclusions

Although no significant differences were found in terms of strength and lateral stiffness among the distinct types of walls, low mortared strength masonry walls, WI and WR, exhibit markedly higher level of energy dissipation when compared with dry stacked masonry. However, this is accomplished with higher level of stiffness degradation corresponding to large damage. The analysis of the experimental data referring to the reversal cyclic tests reveals also that, for the adopted geometry, the behavior of stone masonry shear walls submitted to low and moderate pre-compression is governed by rocking mechanism. This implies that large amount of nonlinear deformation can be withstand by stone masonry walls, which is confirmed by the large values of the lateral drift. Nevertheless, care must be taken regarding the irregular and especially rubble textural walls concerning the quality of workmanship, since it can significantly influence the shear behavior.

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