IN-PLANE CYCLIC TESTING AND DYNAMIC MODELLING OF REINFORCED MASONRY WALLS

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ABSTRACT:

In the framework of the DISWall research project, funded by the European Commission, a new reinforced masonry system, made with horizontally perforated unit where the horizontal reinforcement is laid and vertically perforated units for the confining columns with vertical reinforcement, was purposely developed for typical low-rise residential buildings, to withstand in-plane actions. Squat and slender specimens, made with this construction technique, were tested under in-plane cyclic shear compression loads, in order to analyze the shear and flexural behaviour of the proposed reinforced masonry walls. According to the experimental results, a numerical research to evaluate reduction of elastic response of masonry walls due to their hysteretic behaviour was carried out. The response of SDOF systems with given ultimate ductility was determined through non-linear dynamic analyses, for a range of natural periods that characterize the elastic phase of unreinforced load bearing masonry buildings. In the present contribution, the results of the cyclic in-plane tests and the analytical model developed to reproduce the cyclic behaviour are presented, and the conclusions coming from the dynamic analyses carried out are introduced.

KEYWORDS: **reinforced masonry, in-plane cyclic tests, load reduction factor.**

1. INTRODUCTION

Reinforced and confined masonry have been developed in order to exploit the strength potential of masonry and solve its lack of tensile strength, improving significantly not only the resistance, but also the ductility and the energy dissipation capacity, that is the seismic behaviour, of masonry walls. In the last decades, a large variety of reinforced and confined masonry techniques have been developed. The different masonry systems depend on many parameters: geometric shape and material of the units, composition of the mortar and/or grout, quantity and layout of the reinforcement (Tomaževič 1999).
In the framework of the DISWall research project, funded by the European Commission, four innovative construction systems for load and non-load-bearing reinforced masonry walls were developed. One of these systems is based on the use of horizontally perforated clay units, with recesses for placing the horizontal reinforcement. The vertical reinforcement is concentrated at the wall's edges or intersections, in masonry columns made with vertically perforated clay units (see Figure 1). The mortar and the units have been expressly developed for this reinforced masonry system. The system and its features are described in Mosele et al. (2008).
The main advantages of the system are that all the problems related to cover of bars and mortar shrinkage are overcome, and it is possible to have the un-coupling of the reinforcement. This system also preserves a construction technique (masonry made with horizontal holes) which is very traditional for the countries facing the Mediterranean basin, as it allows reaching good thermal and acoustic insulation. However, the use of horizontally perforated units in seismic area has been generally avoided, due to lack of unit strength even to vertical loads and to possible unit brittleness.

Therefore, the system was tested under in-plane cyclic loads. On the basis of the obtained results, a new analytical model to represent the cyclic behaviour was developed. The model was used to carry out non-linear dynamic analyses on single degree of freedom systems with given ultimate ductility, using a group of 10 accelerograms numerically generated from a given response spectrum supplied by the new seismic Italian code (OPCM 3431, 2005). The analyses were aimed at estimating the load reduction factor $R\mu$ of this masonry system, and verifying its compliance with the seismic code requirements.

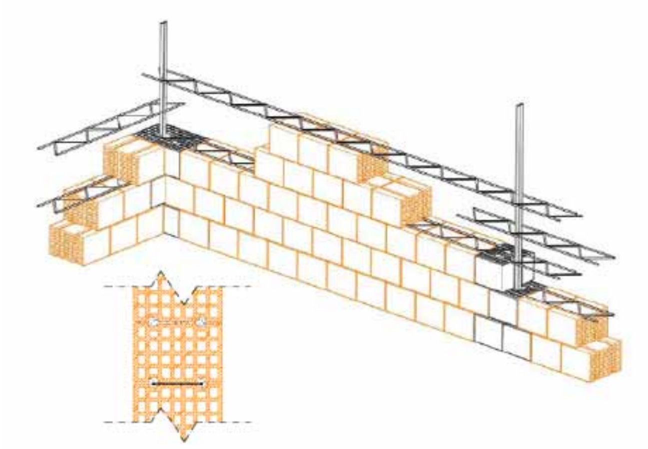

Figure 1 Construction system

2. CYCLIC SHEAR-COMPRESSION TESTS

2.1. Experimental program

For this reinforced masonry wall system, the main aim of the testing program was to assess the behaviour under in-plane cyclic actions. Tests were repeated on two series of specimens, with different horizontal reinforcement. One series was built with usual steel rebars (specimens named SR), the other with prefabricated truss reinforcement (specimens named TR). In all specimens, horizontal reinforcement was distributed on the specimen each other course.

Tests were made for basic materials characterization and to characterize the behaviour of masonry under uniaxial compression. Results are reported in Mosele et al. (2008). Specimens built with entire reinforced masonry system and masonry panels without confining columns ("H") were tested under in-plane cyclic shear-compression loading. Shear compression tests on entire reinforced masonry system were carried out on specimens characterized by two slenderness ratios, in order to force shear behaviour (slenderness ratio "a" equal to 1.09) and flexural behaviour (slenderness ratio "b" equal to 1.64). For these specimens, vertical reinforcement was constituted by two rebars with diameter equal to 16 mm at each masonry edge (squat specimen "a") and by one rebar with diameter equal to 16 mm at each masonry edge (slender specimens "b"). Table 1 lists the shear-compression tests carried out.

Table 1 Test matrix for in-plane cyclic shear-compression tests

Series	**n°**	**Dimensions (mm)**	**Vertical Reinf.**	**Vertical Renf. ratio**	**Horizontal Reinf.**	**Horizontal Renf. ratio**
HSa	2	1550x300x1690	-	-	-	-
SRHSa	2	1550x300x1690	-	-	2Φ6/20cm	0.045%
TRHSa	2	1550x300x1690	-	-	1murfor/20cm	0.040%
SRSa	2	1550x300x1690	4Φ16	4x0.043%	2Φ6/20cm	0.045%
TRSa	2	1550x300x1690	4Φ16	4x0.043%	1murfor/20cm	0.040%
SRSb	2	1030x300x1690	2Φ16	2x0.065%	2Φ6/20cm	0.045%
TRSb	2	1030x300x1690	2Φ16	2x0.065%	1murfor/20cm	0.040%

The specimens were tested with cantilever type boundary condition, with fixed base and top end free to rotate, by applying a centred and constant vertical load equal to 11% and 16% of the measured maximum compressive strength of the reinforced masonry system walls. The corresponding compressive stress levels (0.4 N/mm^2 for the lower pre-compression rate, 0.6 N/mm^2 for the higher one) are adequate to represent the typical vertical loads for two up to four-storey height buildings. Each series reported in Table 1 is constituted by two specimens, one for each pre-compression level.

2.2. Results of the in-plane cyclic tests

During in-plane cyclic tests the attainment of four main limit states, which can be used to idealize the masonry wall behaviour, can be observed. These states are related to the occurrence of flexural cracking on the horizontal joints (flexural cracking limit, H_f, δ_f), to appearance of the first significant diagonally oriented shear crack (crack limit, H_{cr}, δ_{cr}), to attainment of the maximum resistance H_{max} at a corresponding displacement level δ_{Hmax}, and, finally, to attainment of maximum displacement δ_{max}, to which a consequent value of residual lateral resistance H_{dmax} corresponds.

All specimens presented the first non-linearity related to opening of flexural cracks at the base of the wall for displacements of about 1÷2 mm, independently by applied pre-load and specimen type. For specimens designed to fail in shear (SRSa and TRSa; λ=1.09), the first diagonal oriented shear crack was visible at displacement amplitudes of about 5 mm. The attainment of maximum lateral load corresponded to displacements of about 12 mm (walls preloaded with 0.6 N/mm^2) or more (walls preloaded with 0.4 N/mm^2). Collapse was reached at imposed displacement amplitudes immediately following δ_{Hmax} (Table 2), and was characterized by well defined diagonal crack pattern, with cracks passing through the joints and the units, falling out of the unit shells and buckling of the vertical rebars.

Specimens designed to fail in flexure (SRSb and TRSb; λ=1.64) were characterized by crack pattern with damage concentrated at the bottom of the wall and on the compressed toes. Maximum load was attained at displacements of about 25 and 20 mm, according to the level of applied pre-stress (0.4 or 0.6 N/mm^2). TRSb specimens, with horizontal prefabricated reinforcement, were characterized by higher maximum displacement (Table 2). Collapse occurred with crushing of the toe at the higher pre-compression level, while it occurred with yielding and failure of vertical reinforcing bars, at the lower pre-compression level. Specimens of the HS series (without vertical reinforcement) developed rocking, as commonly observed for unreinforced masonry, with damage concentrated at the bottom of the specimens. Table 2 lists the values of maximum lateral load and maximum displacement, and load and displacement ratios at the relevant limit states for the tested reinforced masonry system.

Table 2 Maximum lateral load and displacement; load and displacement ratios at the limit states.

Specimen	H_{max} [kN]	d_{max} [mm]	H_{cr}/H_{max}	H_{dmax}/H_{max}	H_{dmax}/H_{cr}	d_{cr}/d_{Hmax}	d_{max}/d_{Hmax}	d_{max}/d_{cr}
TRSa 0.6	211	12,51	0,81	0,89	1,10	0,42	1,03	2,43
SRSa 0.6	218	15,01	0,73	0,89	1,21	0,42	1,22	2,93
TRSb 0.6	93	45,55	0,92	0,81	0,87	0,54	2,18	4,05
SRSb 0.6	90	30,00	0,89	0,47	0,53	0,42	1,42	3,42
TRSa 0.4	201	19,99	0,71	0,83	1,15	0,37	1,50	3,93
SRSa 0.4	201	19,99	0,68	0,85	1,25	0,29	1,16	3,99
TRSb 0.4	81	79,99	0,91	0,36	0,39	0,49	3,27	6,64
SRSb 0.4	79	64,99	0,85	0,29	0,34	0,33	2,43	7,37

3. MODELLING OF THE IN-PLANE CYCLIC BEHAVIOUR

3.1. Available models

According to observed behaviour, various models have been developed to reproduce the hysteretic behaviour of reinforced masonry walls. Modena (1992) proposed a model based on five non-dimensional parameters: three shape parameters, obtained as the ratios between horizontal loads and displacements at the relevant limit states, one able to describe stiffness degradation, and the fifth defining available ductility. He later proposed a procedure based on the use of dissipated energy to obtain the envelope curves of dynamic tests from static tests or the envelope curves of hysteresis loops from monotonic tests. Tomaževič and Lutman (1996) also defined the envelope curve of the hysteretic behaviour of reinforced masonry walls, starting from the experimental results of monotonic tests, by introducing fictitious input energy. To model hysteretic behaviour, these authors used three parameters, two depending on experimental stiffness degradation, and the third on the amount of dissipated hysteretic energy during one loading cycle, corresponding to strength degradation. Bernardini et al. (1997) further developed this model, taking into account the amount of adsorbed instead of dissipated energy to evaluate strength degradation. Magenes and Baietta (1998) again proposed hysteretic non-linear law for reinforced masonry shear walls. The model was based on five empirical relationships, which evaluate strength and stiffness degradation and other energy and displacement parameters. A different approach was followed for reinforced masonry by Wakabayashi and Nakamura (1984) and Tassios et al. (1984), who developed semi-empirical criteria based on the description of, respectively, global and local resisting mechanisms.

3.2. Applied and developed models

To reproduce the experimental cyclic behaviour, the model proposed by Tomaževič and Lutman (1996) was applied. The idealized envelope curves on which construction of hysteresis loops is based were taken as quadri-linear curves defined by the four limit states, and given in Mosele et al. (2008). The cyclic degrading model is obtained introducing parameters C_k and C_f. The first gives the slope of the unloading branch of cycles, and is based on the ratio of elastic stiffness to stiffness at the maximum displacement. The second influences the shape of cycle and is evaluated on the basis of energy equivalence. However, due to the particular shape of experimental cycles, in this case the equivalence was calculated in two different ways on squat specimens (with shear behaviour) and slender specimens (with flexural behaviour), and respectively equivalence was made on the amount of input (squat walls) and dissipated (slender walls) energy. Finally, β gives strength degradation and allows modeling repeated cycles. As can be seen in Figure 2, the model was able to reproduce fairly well the experimental behaviour. From the comparison between experimental and modelled values of the ratio between dissipated and input energy (Figure 5) it can be seen that energy balance of the model was not as in the tests, particularly in the case of flexural behaviour, after attainment of the maximum load.

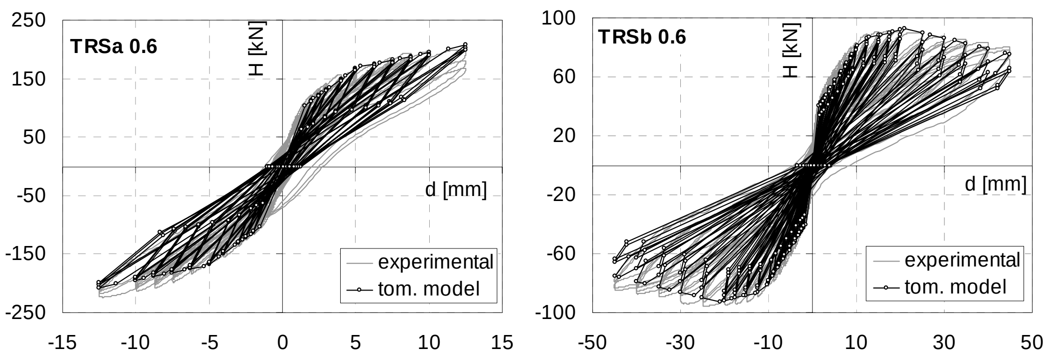

Figure 2 Modelled and experimental hysteresis loops

Given the above mentioned issues of the applied model, a new one was developed. The new model is still based on the quadri-linear envelope curves defined by the four limit states. The construction of the cycles is based on four main observations: 1) the intersection between the loading branch of the hysteresis loops and the envelope curve is constant for the first cycles, than starts lowering for cycles of greater amplitude. This particularly happens in the slender walls. 2) The unloading branch is characterized by two phases. The first phase has steeper slope, and can be related to the load attained at the corresponding displacement level. For larger displacement amplitudes, the first phase becomes larger, as it regulates the energy dissipation during the cycle. 3) The second phase of the unloading branch is almost parallel to the loading branch, then the slope increases again, closer to the point where the loops have intersected the envelope curve during the loading branch. These two last features give to the loops the particular ‘S’ shape observed during the experimental tests. 4) Finally, in the repeated cycles, the strength of the loading branch is reduced, but the unloading branch is similar to that of the first cycle, that makes the dissipated energy lower. Starting from these considerations, the cycles were modelled on the basis of four main points (A; B; C; D) and their symmetrical. These points were found on the basis of the parameters C_1 and C_2, which depend on the amount of the absorbed and dissipated energy during the cycle, and Z, which is a ductility parameter. Figure 3 shows the geometrical scheme for the loops’ construction.

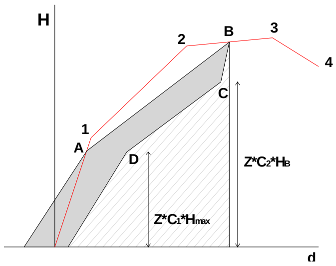

Figure 3 Scheme for the loops’ construction

The slopes of the various loading and unloading phases are given by stiffness parameters, as in Tomaževič and Lutman’s model. Other two parameters, R_1 and R_2, are used to model the repeated cycles on the basis of the ratio between input and dissipated energy in the first and, respectively, the second and the third cycle. Overall, the model uses four independent parameters, and the others are all based on those. A more detailed description of the model features is given in Nicolini (2008). Figure 4 shows the experimental hysteresis loops and those generated by the model. It can be seen that, as in Tomaževič and Lutman’s model, the agreement is satisfactory. Figure 5 shows that the new developed model reproduces fairly well also the energy balance of the test (ratio between dissipated and input energy at each cycle), also in the case of flexural behaviour.

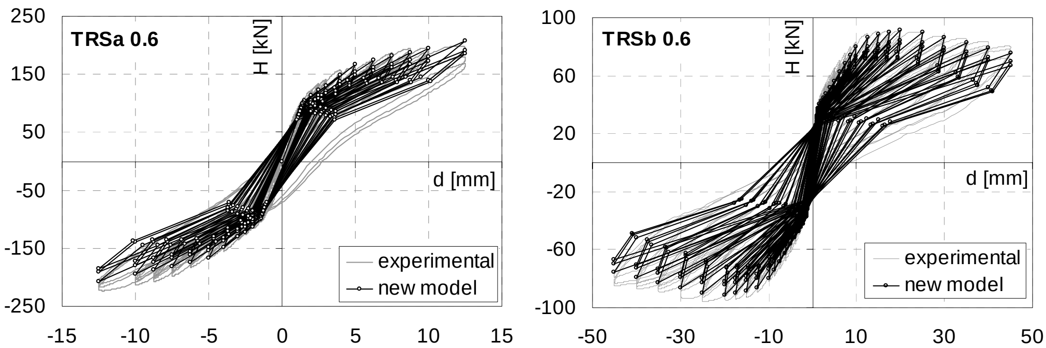

Figure 4 Comparison between hysteresis loops generated by the new model and experimental ones

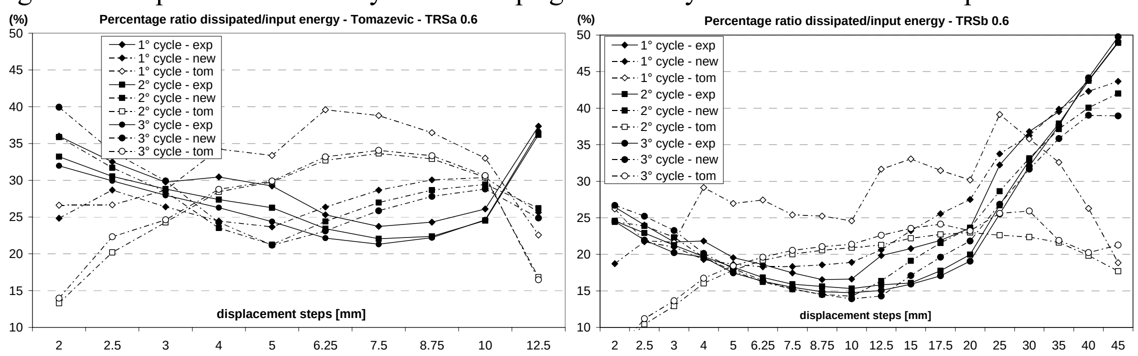

Figure 5 Ratio of dissipated-input energy: comparison between experimental and modelled values for squat (left) and slender (right) specimen.

4. DYNAMIC ANALYSES

4.1. Seismic input used

Dynamic analyses were carried out on 10 synthetic time-histories composed of 2048 points taken at a sampling frequency of 100 Hz. The time-histories were created in MATLAB™, and are compatible with the spectra of OPCM 3431 (2005) with less than 10% deviation between generated and code-prescribed spectra in the period range from 0.10 to 2 s. Their peak ground accelerations (PGA) are normalized to 1g. The value of the PGA is arbitrary, because the analyses are not affected by it. Definition of the response spectra varies according to the different types of soils and to the range of natural period. The main five soil categories are: A, rock or other rock-like geological formation; B, very dense sand, gravel, or very stiff clay; C, medium-dense sand, gravel or medium stiff clay; D, loose-to-medium cohesionless soil or predominantly soft-to-firm cohesive soil; E, soil profile consisting of a surface alluvium layer. It should be noted that soils B, C and E have the same spectral parameters.

4.2. Numerical analyses

Dynamic analyses were aimed at defining the response of an SDOF system for the nine different values of natural periods between 0.10 and 0.5 s (each 0.05 s), which characterise the elastic phase of unreinforced load-bearing masonry buildings from one to more than five storeys high. They were repeated for each soil group classified by the Italian code (OPCM 3431, 2005). Analyses were carried out on the basis of the given value of ultimate ductility factor μ (last column of Table 2), related to the definition of the idealized elastic limit as the actual shear crack limit obtained during tests. The aim was to estimate load reduction factor R_μ due to energy dissipation and non-linear behaviour of the reinforced masonry system, taking into account shear and flexural failure modes. For a given ground motion, computation of the inelastic strength demand, which allows the load reduction factor to be calculated, involved iteration, for each period and target ductility μ on lateral strength, until the computed ductility demand was the same as target ductility factor μ within a tolerance of 2%. The procedure is described in Grendene (2006). At that point, the analysis gives the value of R_μ for the given time-history at the given value of period. Analyses were repeated for nine chosen values of natural period and for 10 generated accelerograms, i.e., 90 analyses for each tested specimen, repeated for the three groups of soils (A: rock soil; B, C and E: medium stiffness soils; D: soft soil) given in the Italian code (total 2160 analyses).

5. RESULTS OF THE ANALYSES

Figure 6 shows the results of the analyses in terms of load reduction factor R_μ at the various natural periods, evaluated on the different soil types and for the different specimens (squat walls with shear failure mode and slender walls with flexural failure mode). The dots into the diagrams represent the results of all the analyses carried out (ten accelerograms, various tested specimens, two different pre-compression levels). The continuous lines represent the mean value and the two dashed lines in each diagram represent the mean value plus and minus the standard deviation. Figure 7 gives the average values (and the average plus and minus the standard deviation) on all soil types for squat and slender specimens, tested under pre-compression level of 0.4 and 0.6 N/mm^2.
A similar trend, characterized by lower natural period values on which R_μ increases and higher periods on which R_μ value is stationary, is obtained on all soil types. In general, values for squat walls are lower than for slender walls. The differences are generally more marked for rock and medium stiffness soils than for soft soils. For all the various specimens, the lowest values of R_μ are obtained on the soft soils, the highest on the rock soils. On average on the different soil types, the lowest value of R_μ for squat walls is 1,76 (at T_n=0,10 s), for slender walls it is 2,15. When the stationary phase is reached, at a natural period value of about 0,25-0,30 s (generally lower for slender walls, and higher for squat walls), the values of R_μ raise to about 3,0 for squat walls, and 3,6 for slender walls. Finally, it can be observed that values of R_μ obtained for lower pre-compression level are always higher than those obtained at higher pre-compression level. At the lower natural periods, this difference is generally around 20%.

It has to be noted that squat walls, dominated by shear failure mode, exceed the R_μ value of 2,5 for a natural period between 0,15 and 0,20s. In the same natural period range, slender walls exceed the R_μ value of 3,0. These values are significant as the Italian code prescribes to use load reduction factors of 2,5 in case of reinforced masonry with shear failure, and 3 in case that capacity design is pursued and thus flexural behaviour dominates (OPCM 3431, 2005). These values have to be multiplied by over-strength factor in order to obtain the final value of behaviour factor q. On the contrary, in the European seismic code, values of behaviour factor q between 2,5 and 3,0, independently from the failure mode, are given (EN 1998-1, 1998). In this case, those are the final values of behaviour factor to be adopted, as contributions in terms of load reduction factor of SDOF system and over-strength of MDOF system are not distinguished.

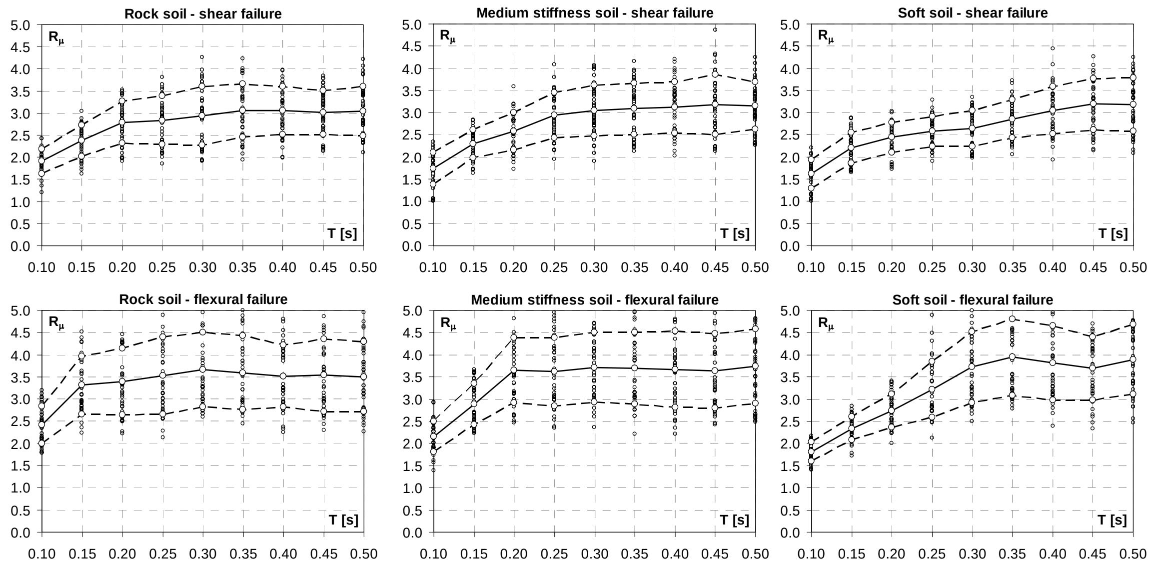

Figure 6 Load reduction factors for different types of soils and of failure modes

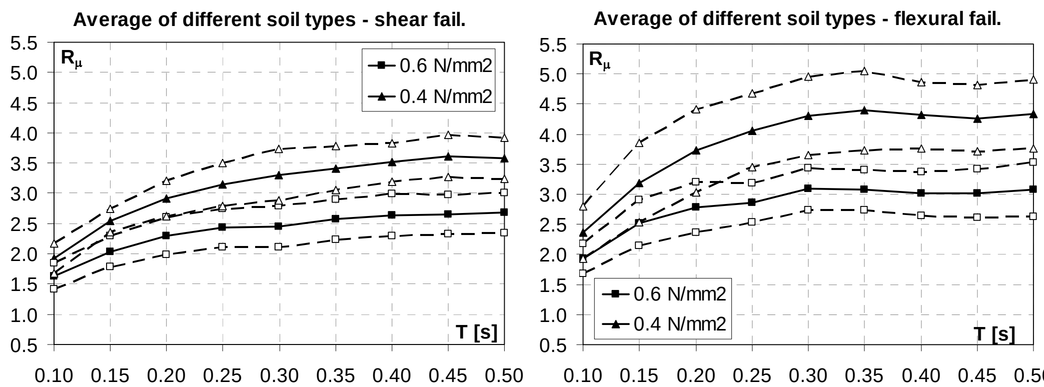

Figure 7 Load reduction factors for different types of failure modes and pre-compression level

6. CONCLUSION

A new reinforced masonry system based on the use of horizontally perforated clay units has been developed and tested under in-plane cyclic shear compression tests. The specimens were designed in order to obtain flexural failure (slender walls) and shear failure (squat walls). The tests results showed the main differences in terms of failure modes. The slender walls were characterized by high displacement capacity and good energy dissipation, the squat walls were more brittle, with collapse occurring few cycles after the attainment of the maximum load. In any case, the behaviour of the new system was satisfactory, and the horizontally perforated units were able to transfer the loads from the masonry panel to the confining columns with the vertical reinforcement. A model available in literature was applied to reproduce the experimental cyclic behaviour of the walls. It gave fair results, but some limitations were found in the application to the current case. Therefore, a new model, still based on energy considerations and stiffness degradation rules, was developed and implemented into a numerical procedure, which allows carrying out non-linear dynamic analyses for the evaluation of load reduction factors of SDOF systems. 2160 analyses were carried out and, overall, R_{μ} average value is 2,74 for squat walls and 3,35 for slender walls. These results are good as they demonstrate that the proposed system is characterized by load reduction factors more optimistic of those given by the Italian code, on almost all the range of analyzed periods. The use of code prescribed values can be thus conservatively proposed for the design of this new construction system, and the application of capacity design principles to that construction system reflects the results of both experimental tests and numerical analyses.

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