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# PRELIMINARY EXPERIMENTAL TESTS ON STEEL-CONCRETE HYBRID COUPLED WALL SUBCOMPONENTS

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**Abstract**

*Single-pier steel and concrete hybrid coupled walls are a dissipative seismic-resistant system composed of a single reinforced concrete central wall connected to two side columns through horizontal steel dissipative beams. Previous studies proved that the system is a viable alternative to the more common solution with two reinforced concrete coupled walls, but also highlighted some issues to be solved to enhance the global performance of the system. The present paper shows possible enhancements of the original system in which, to limit the cracking of the wall due to the bending force acting at the base, the base cross section is reduced and two vertical steel profiles are added. In this way, the wall transfers the shear forces to the foundation, while the bending moment is resisted mainly by the vertical steel profiles. Experimental tests were conducted to preliminary investigate the global and local behavior of the system and to highlight the possible benefits gained from the proposed original solution. In this paper, the numerical study behind the design of the tested prototype and the main experimental outcomes are discussed.*

**Keywords:** Steel and concrete hybrid structures; dissipative links; seismic-resistant structures; experimental tests.

## 1 INTRODUCTION

Hybrid coupled walls (HCWs) represent a seismic resistant system [1]-[5], usually constituted by two reinforced concrete (RC) walls and a series of (steel or RC) coupling beams as schematically shown Figure 1a. HCWs offer stiffness and strength which are way larger than the sum of those provided by the single uncoupled walls. As highlighted in the figure, under horizontal actions, the two walls experience both bending and shear, as well as an alternation of tension/compression axial forces; the coupling beams are instead subjected to bending and shear. In order to eliminate the tension/compression axial forces on the RC wall, an alternative solution was presented [6] in which a single RC wall is coupled to two steel side columns

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through sets of horizontal dissipative steel links (Figure 1b). In this way the resulting RC wall is subjected to bending moment and the constant axial force stemming from permanent loads; the side steel columns experience an alternation of tension/compression (with some bending moments due to the eccentricity of the link connections). The links are pinned to the side columns and fixed to the RC wall, thus, transferring on it both the bending moment and the shear force. Another advantage of this solution is the possibility of replacing the damaged steel links [6][7].

The aforementioned HCW solution, hereafter called single pier hybrid coupled wall (SP-HCW), was extensively analysed [8]-[13] and proved to be effective as seismic protection strategy, although one critical aspect was identified, i.e., the possible damage at the base of the RC wall, a condition which would reduce the reparability of the system. For this reason, a new research project is currently in progress to find solutions able to limit or eliminate such vulnerability of the RC wall. In this context, innovative solutions which exploit the rocking motion of the wall and a pair of corner components (CC) [14] are being studied, according to two possible base connections: 1) perfectly hinged base RC wall; 2) tapered fixed base RC wall. While the former solution was preliminary analysed in [15] and the research is still ongoing, the latter solution (depicted in Figure 1c) is investigated, for the first time, in the present study, where an experimental test was designed and carried out to provide some preliminary indications on its feasibility and effectiveness.

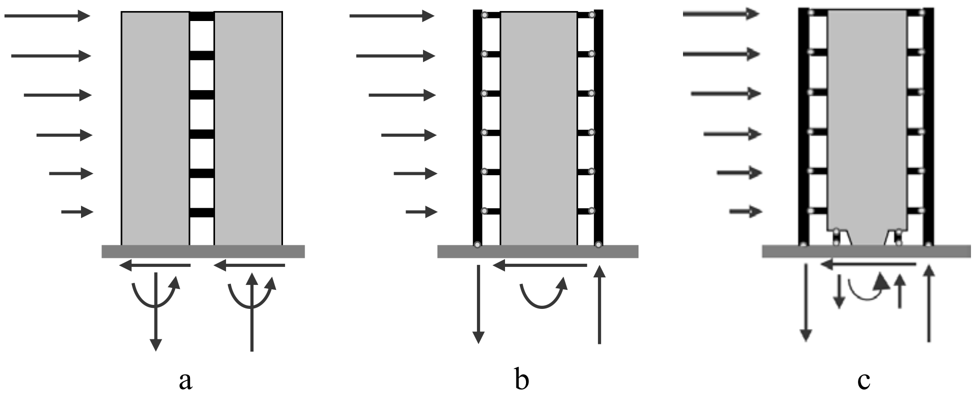


Figure 1: (a) Conventional HCW; (b) SP-HCW; (c) SP-HCW with CC and tapered fixed base.

## 2 DESIGN OF THE TESTED SAMPLES

The designed prototype subjected to the experimental tests described in this article represents a reduced-scale reproduction of the SP-HCWs system with CCs at its base. Attention is focused on the study of the RC wall and the hinged CCs, omitting in this test the contribution of the horizontal links.

The first step of the project concerned the choice of the most suitable solution for the base section of the RC wall, considering the limitations in terms of dimensions and maximum applicable force of the available testing equipment. By fixing the overall height of the reinforced concrete wall, 3.00 m, and the length of the full section, 1.50 m, a series of preliminary numerical sensitivity analyses were carried out to evaluate the dimensional ratios of the RC base section, i.e., height, length, tapering, reinforcement geometric ratio, and of the CCs, i.e., profile dimensions, lever arm. The 2D model (Figure 2a) adopted for this purpose is linear elastic (despite more refined models accounting for system's nonlinearities could be used [15][16][17]), with the RC wall fixed at the base and the CCs rotationally released at their ends; horizontal rigid links are used to connect the RC wall to the CCs and rigid offsets are placed on top of the CCs to account for the connection required space.

The idea driving the test is to exceed the yield condition of the CCs and evaluate the capacity of the reduced section of the wall before the ultimate failure of the CCs. For this reason, the design of the CCs was carried out adopting stresses from static analysis performed applying a horizontal concentrated load of 300 kN at the top of the wall (Figure 2b); moreover, a partial safety factor equal to 1 was adopted for both the CCs and the bending design of the reduced section of the wall. Conversely, the remaining components of the system for which adequate overstrength is desired were designed by adopting the partial safety coefficients specified by the standards [18][19].

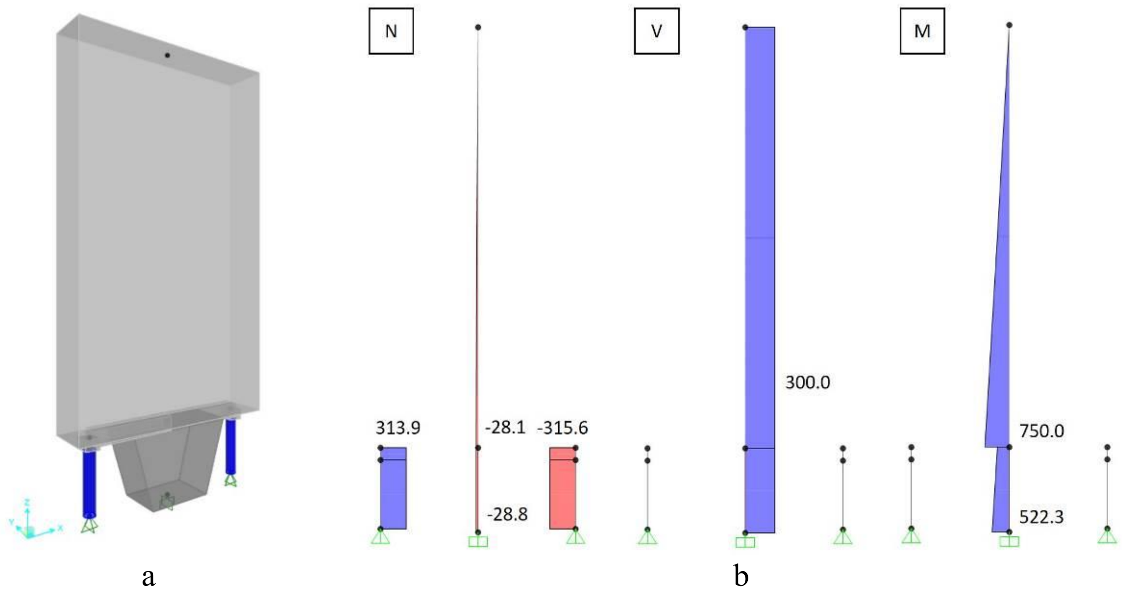


Figure 2: (a) Finite Element Model; (b) axial, shear, and bending moment diagrams from static analysis.

From this first phase, it was decided to choose CHS 76.1x3.2 profiles of S275 steel grade ($f_y$=275 N/mm$^2$, $f_t$=430 N/mm$^2$) for the CCs. The reduced section of the RC wall develops from a 45cm x 30cm section at the base with 12$\phi$26 reinforcement bars, to a 75 cm x 30 cm section at the conjunction with the full section with 12$\phi$26 reinforcement bars. 4-arm $\phi$10 ties (shear reinforcements) every 8 cm were also arranged. The full section is reinforced with 8$\phi$26 rebars in the two confined regions and by 2-arm $\phi$10 horizontal bars every 20 cm for the entire height development; in the confined regions additional ties was provided, with a spacing of 5 cm where the anchor bolts are inserted, then increased to 10 cm for the remaining height. The steel adopted for rebars is B450C grade ($f_y$=450 N/mm$^2$), while the concrete is class C30/37 ($f_c$=30 N/mm$^2$). The main dimensions of the prototype are shown in Figure 3.

The second step involved the design of the connections of the CCs to the ground and to the wall: the two elements should operate as two struts, being rotationally free at the ends. It was decided to use pin connections, designed in accordance with Eurocode 3 [18], to which the steel profiles are welded, and which are connected to the ground and the wall by means of a system of anchor bolts capable of transmitting the tensile forces. The transmission of concentrated compressive loads in the plates and concrete was designed according to the T-stub scheme; in addition, supplementary confining reinforcement was provided in the wall. The entire connection system of the CCs, i.e., pins and anchor bolts, was designed to ensure a degree of overstrength and to prevent any potential connection-side damage.

From this second step, pinned connections were designed consisting of three plus two 15 mm thick plates in S450 steel grade ($f_y$=440 N/mm$^2$, $f_t$=550 N/mm$^2$) and $\phi$28 mm diameter pin in M8.8 steel grade ($f_y$=640 N/mm$^2$, $f_t$=800 N/mm$^2$); four $\phi$14 mm diameter anchor bolts in M8.8 steel grade extend approximately 700 mm into the wall. Details about connections are shown in Figure 3. The executive drawings of the wall and of the steel elements were made considering the possible assembly difficulties and adopting the adequate tolerances to ensure the smoothest possible realization of the system.

The CHS profiles, designed to be made of S275 steel grade, were then changed into S235 steel grade, keeping the same dimensions, due to availability issues of grade S275. This permitted to explore the impact of a lower capacity of the CCs and, hence, an earlier activation of their dissipative mechanism.

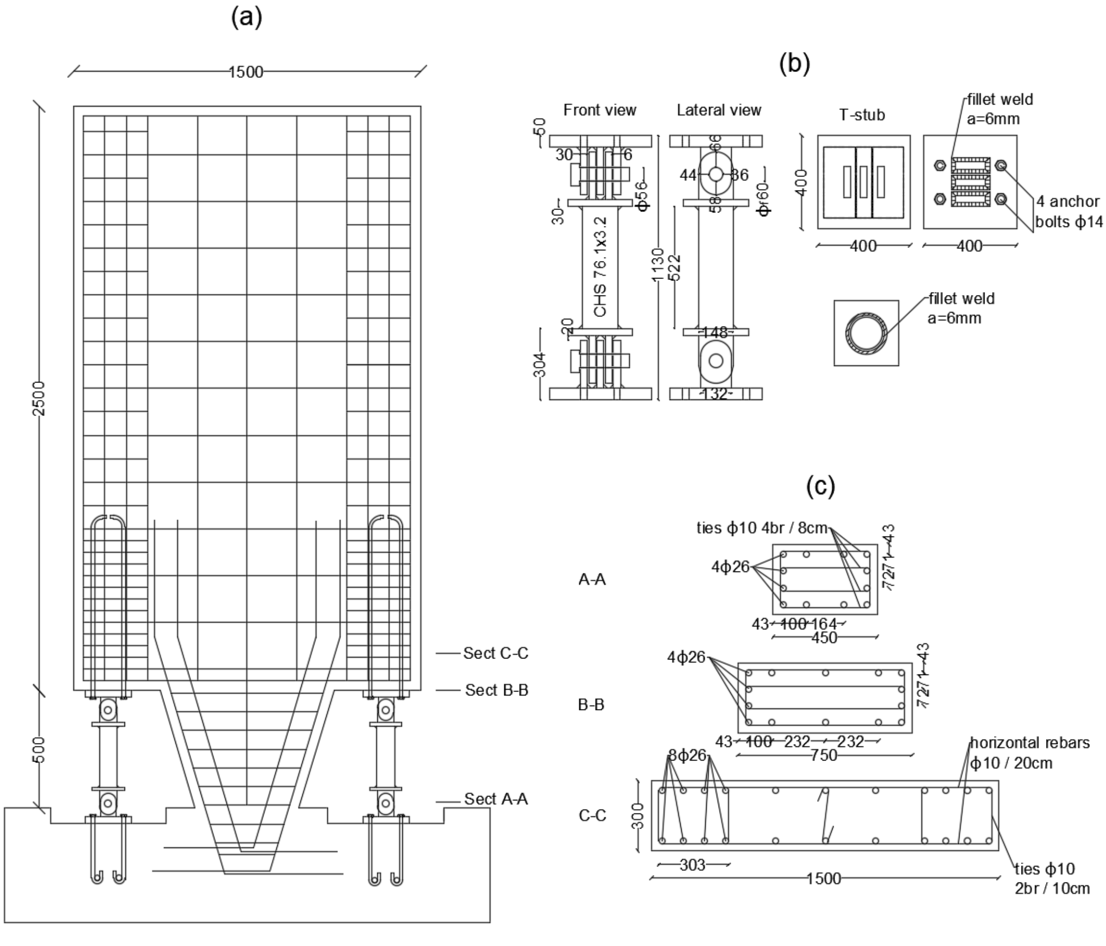


Figure 3: General dimensions and main sections of the tested component.

## 3 EXPERIMENTAL ANALYSIS

The designed system with the S235 CHS was tested under increasing monotonic load. Figure 4 shows the direction of the applied load, the adopted test set up and measuring instruments. The wall is fixed to a steel base through 4 anchoring bolts, placed at the four vertexes of the concrete base, aligned to the tubes' anchorages. Horizontal load is applied by two hydraulic jacks completed by loading cells to measure it. The load is transferred to the wall through a horizontal frame. Vertical load is indirectly applied through two other hydraulic jacks acting at the top of the wall and pushing on a contrast beam which activate two DYWIDAG bars. The vertical load is constant and applied at the very beginning of the test, and equal to 70 kN, which corresponds to the weight of the two other upper-floors walls, being the system conceived not to carry gravity vertical loads except its self-weight. Two loading cells allow to control that this load stands still during the whole test. Finally, the out-of-plane stability of the wall is provided by an adequate surrounding frame. This is necessary only in case of strong damage during the test, that may trigger this kind of phenomenon. About the measuring instruments (Figure 4), the type 1 displacement sensors monitor the horizontal displacement of the wall at different heights; the type 2 ones control the vertical displacement of the reduced part; the type 3 ones measure the vertical displacement of the lateral parts in correspondence of the tubes. The tubes are also equipped with 3 strain gauges placed at the mid-section of each element, to measure deformations. The type 4 displacements sensors are placed to measure the eventual opening of horizontal cracks in the connecting RC zone with the steel pieces; the type 5 ones control that the base does not move in the horizontal direction, and monitor the eventual uplift of the base. Finally, the type 6 displacement sensors control that the steel base to which the whole system is anchored remains still.

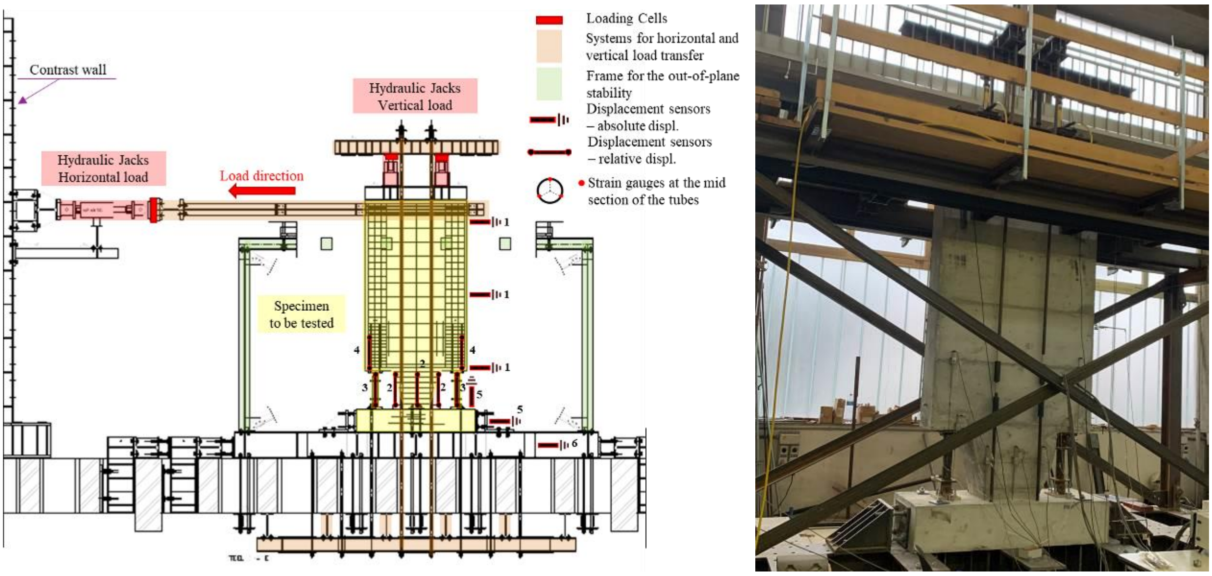


Figure 4: Test set up and measuring instruments.

Figure 5 shows the horizontal load vs. top displacement curve obtained from the test and highlights the most significative steps (see Table 1), also reported in Figure 6.

The most significative steps of the test were the following: point 1 corresponds to the appearance of the first slight cracks in the reduced part of the wall (Figure 6a); point 2 corresponds to the starting of the crack at the interface of the reduced part with the base of the wall (Figure 6b); at point 3, the test paused to track the evolution of the cracks in the reduced part and to report the base starting to lift in the tense side; at point 4, tube under tensile load yielded (deformation at yielding evaluated according to nominal values of yielding stress of the material); at point 5 the tube under compression started to buckle (Figure 6c); at point 6, an unloading-loading cycle was performed. Re-loading was up to 250 kN (value of the load at the ended of the first loading phase, point 8). Figure 6d and Figure 6e show the status of the system, and highlight the crack pattern of the reduced part and of the base of the specimen. In the reduced part, the resisting mechanism is well depicted (compressed struts), but it should be highlighted that the cracks are slight and quite superficial. A higher damage can be noticed instead in the base of the specimen, due to the uplift and of the local pressure of the DYWIDAG rebars on the concrete holes: indeed, due to the relevant horizontal displacement reached, the rebars lost verticality and came into contact with the base, inducing local damage in the base itself. Although the cracks are wider than those in the reinforced part, they are superficial, and this damage did not influence the global behaviour of the specimen, as resulting from the test. At point 9, the tube under compression is completely buckled (Figure 6f), and at point 10, the test needed to be ended due to reaching the maximum stroke of the hydraulic jack (Figure 6g). Although the post maximum load branch was not reached, the test can be considered concluded, due to reaching the complete failure of the tube in compression and a displacement equal to 4.3% of the height of the component, which is comparable to the Near Collapse Limit State according to FEMA 350 [20].The final configuration of the specimen is similar as the one at the end of the first loading phase, with the only exception of the tube in compression, which is completely damaged. The reduced part of the wall is cracked, but cracks are superficial and do not go through the wall. The most damaged part is the base, but in any case, this didn't influence the global behaviour of the system. Anyway, the damage level on the concrete base could be further reduced by adopting a stronger and more diffuse anchorage system effective in preventing the tube uplift.

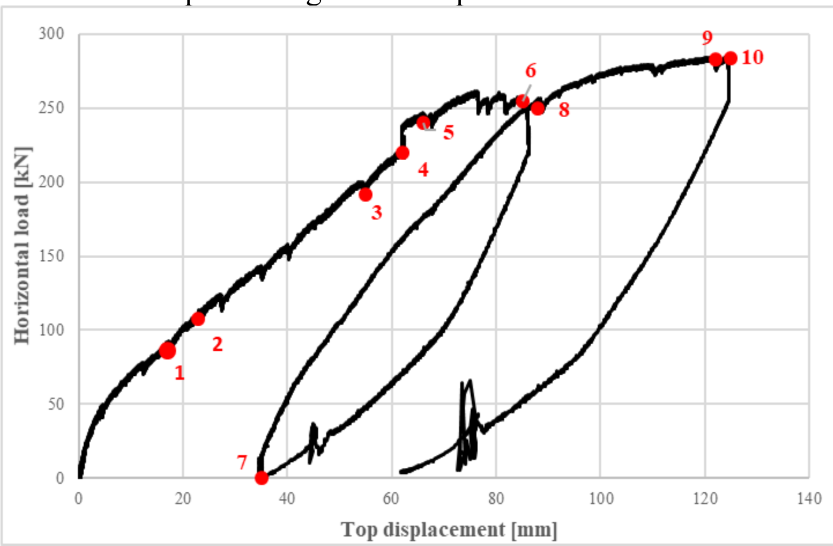


Figure 5: Horizontal load vs top displacement of the system and significative points of the curve.

| Point | Load [kN] | Displacement [mm] |
|---|---|---|
| 1 | 87 | 17 |
| 2 | 108 | 23 |
| 3 | 192 | 55 |
| 4 | 220 | 62 |
| 5 | 240 | 66 |
| 6 | 255 | 85 |
| 7 | 0 | 35 |
| 8 | 250 | 88 |
| 9 | 283 | 122 |
| 10 | 284 | 125 |

Table 1: Values of load and displacement of all the relevant points shown in the experimental curve (Figure 5).

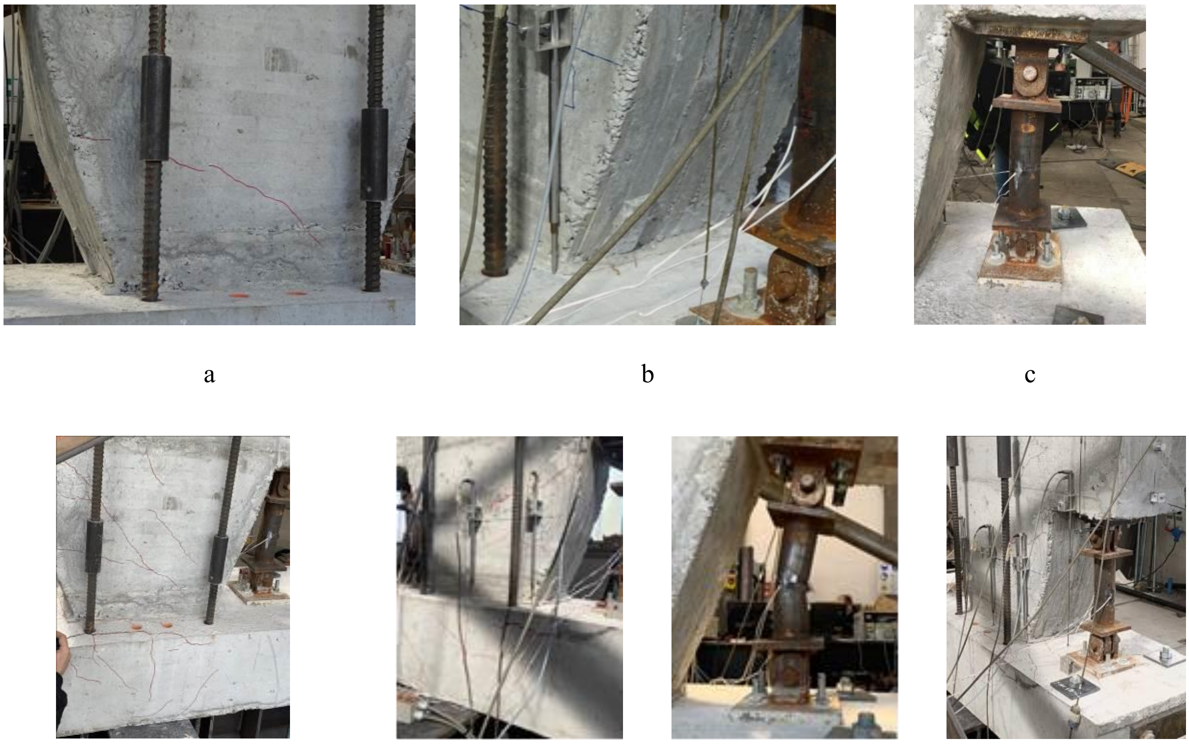

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Figure 6: Images of the significative points of the curve (Figure 5).

## 4 CONCLUSIONS AND FUTURE ACTIVITIES

This paper presented a selection of the results of preliminary experimental tests performed on an upgraded base solution for innovative seismic-resistant single-pier steel and concrete hybrid coupled walls. The tested configuration was developed to avoid major damage of the concrete wall at its base. To this end, the cross section of the wall was reduced at its base to limit its bending stiffness, and two lateral vertical steel profiles were added to resist the bending force acting on the wall. A prototype of the whole system was designed, and then a monotonic experimental test was performed on the reduced-section wall. The test outcomes highlighted the benefits of this introduced upgrade, since few superficial cracks appeared on the reduced concrete part, with higher damage in the steel tube under compression, that is a replaceable component. Besides, the system reached 125 mm of top displacement, which is almost 5% of the height of the specimen, proving a high rotational capacity with limited damage in the hardly reparable part of the system. In the next phases, the system will be tested also under cyclic loading, together with the use of stronger steel tube, to assess the actual resistance and deformation capabilities of the RC wall.

## ACKNOWLEDGEMENTS

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