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Seismic behaviour of an innovative hybrid coupled wall system investigated through cyclic tests

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Abstract. This article presents the experimental results obtained from the cyclic testing of an innovative hybrid coupled wall (HCW) system – a fixed-base reinforced concrete (RC) wall coupled with two steel side-columns via steel coupling links, where the wall carries almost all the horizontal shear force and the overturning moments are partially resisted by an axial tension-compression couple developed by the two steel columns rather than by the individual flexural action of the wall alone. The initial stiffness properties were primarily identified through the first cycle, estimating the yield force and displacement of the HCW system. Incremental cyclic tests were then conducted according to the ECCS 1986 provisions, targeting specific performance levels: (i) “reparability” of the HCW, i.e. the yield displacement, where the steel links yield with negligible damages in the wall and the self-centering capacity of the system is active, so that the actual replacement capacity of the elements can be validated; and (ii) a displacement level corresponding to a major earthquake with very low probability, which activates the wall as an additional dissipative element, eventually leading to a non-reparable damage state. Relevant results have been discussed through graphical and real-life illustrations. Finally, the constructional aspects are also discussed from a real-life application viewpoint.

Keywords: Earthquake resistant design, Hybrid coupled wall structure, Cyclic testing, Coupling links, Repairable systems.

1 Introduction

An innovative hybrid coupled wall (HCW) system, originally presented in the RFCS research project Inno-Hyco [1] offered encouraging results and attained the desired design goals, i.e. good lateral stiffness, ductile global behaviour, very limited damage in the RC elements, dissipative steel elements that are easily replaceable; proving to be an efficient alternative to the only coupled solution covered officially by EN 1998-1 [2] (see conventional solution in Fig. 1a). The innovative HCW is made of a RC shear wall coupled to steel side columns by means of replaceable steel links (see Fig. 1b). A number of advantages were obtained compared to the conventional one, such as: (i) the RC wall is subjected to shear and a fraction of the total bending moment without the alternation of tension and compression axial forces, making easier to control the wall damage; (ii) the dissipation of seismic energy is expected to be higher as the innovative system doubles the number of dissipative links; (iii) the innovative solution requires less horizontal space and has a smaller mass thanks to the reduced quantity of concrete with respect to steel, leading among other advantages to a lighter architecture. First developments were carried out in the frame of Inno-Hyco [1, 3], in which different innovative seismically resistant solutions were investigated. Those first developments have shown that the proposed system was theoretically working.

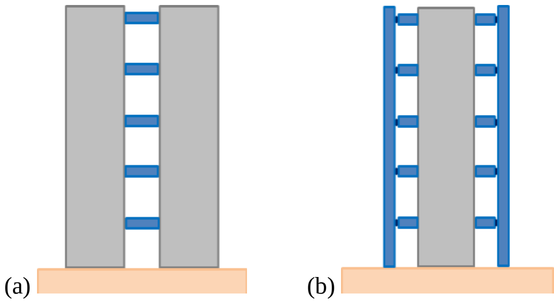

Fig. 1. (a) Conventional and (b) innovative HCW systems.

However, advanced strategies and real-life experimental evidences were deemed necessary to improve and subsequently characterize the overall seismic behaviour and the constructability/repairability of the proposed HCW system. To that purpose, 4 key components of the HCW have been identified and investigated in the frame of an EU-RFCS project HYCAD [4], namely, the wall, the links, the link-to-wall connections, the link-to-column connections and the foundation of the wall. The adopted strategies can be summarized as: (i) prefabricated DS walls to ease the construction process, (ii) shear critical links for better energy dissipation [5], (iii) post-tensioned link-to-wall connections using unbonded threaded bars and a steel box inserted in the DS wall to improve the replaceability of the damaged links [6], (iv) using rigid/semi-rigid link-to-side column connections [6] and (v) a fixed-base and rocking-base wall in order to characterize the influence of the support conditions at the base of the RC wall [7]. After further investigations through numerical and experimental analysis [5-8], combined with additional thoughts on the constructability and repairability of the HCW, two final solutions have been designed and tested. In both solutions, the RC wall is based on a prefabricated DS wall, with the links connected to the wall by unbonded threaded rods. The main difference lies at the base of the wall: (i) in the 1st configuration (C1) a classical rigid connection is implemented between the wall and its foundation, which means that the base of the wall itself can be activated as a dissipative zone in case of major earthquakes, once the plastic capacity of the links is exceeded; (ii) in the 2nd configuration (C2), the wall is connected to the foundation by a shear key, transferring the base shear and two vertical steel elements activated as plastic dissipators.

Nevertheless, this paper focusses on the seismic behaviour of the 1st configuration, C1 (see Fig. 2a), along with its real-life design, construction and repairability aspects. The pre-design details are discussed in the following section. Engineering drawings are presented for relevant components. The test set-up, instrumentation, loading protocol considered for the cyclic tests are summarized while discussing the construction of the HCW system. The seismic behaviour of the HCW system is discussed through graphical and contour plots obtained from the tests. Real-life images are also provided for better clarity. The repairability aspects are highlighted. Relevant conclusions are stated in the final section.

2 Pre-design of the HCW test-specimen

The HCW system was designed according to a recently modified design approach documented in articles [9] as well as the design handbook of HYCAD [10]. The design outcomes are summarized in this section for Configuration 1 (C1), see Fig. 2a.

The specimen is designed as 4-storey HCW system, which can resist a total base bending moment of 900 kNm. A 50% coupling ratio (CR = 0.5) is considered. The storey height is 1.5 m. The total height of the structure is 6 m from base support to the central line of the topmost horizontal coupling link. A height-to-length ratio, H/l_w = 7.5 is considered to pre-dimension the RC wall cross-section. An overstrength factor of 1.5 is considered to design the flexural resistance of the RC wall section. Concrete grade is taken as class C30/37 (f_{ck} = 30 MPa) and reinforcements are taken as B450C (f_{yk} = 450 MPa). All steel sections are designed with a S355 steel grade (nominal yield stress, f_y = 355 MPa). The length and width of the RC wall is calculated to be 0.8 m and 0.4 m. 4 nos. Φ20 rebars are used at the corners and 16 nos. Φ16 rebars are used @ 56.25 mm spacing (see Fig. 2b and 2c). The design flexural resistance of the wall is calculated to be 675 kNm. Two UPN 400 profiles are attached to both sides of the walls in order to reduce concrete spalling at the faces as well as to connect the coupling links to the RC wall. They are however discontinued before the foundation, and therefore do not contribute to the flexural resistance of the wall. The steel links are designed to be shear critical built-up “I” sections with: height = 111.4 mm, width = 55 mm, flange thickness = 11.4 mm, web thickness = 4.1 mm and length = 190 mm. Based on the axial force demand, an HEB 160 profile is chosen for the steel columns. Φ10 stirrups are provided with a spacing of 100 mm throughout the height of the wall to provide suitable shear resistance to the RC wall. The link-to-wall connections are designed as full-strength moment resisting connections. The moment and shear force demand at the connection zone are calculated corresponding to an amplified shear resistance of the links. A steel end plate (200 mm x 245 mm x 15 mm) is welded at one end of each link to connect it to the UPN attached to the RC wall. 6 nos. M24 (grade 8.8) threaded bars are used for each connection (see Fig. 2d). One end of these threaded bars is essentially bolted with the link end plates, while the other end is bolted to the RHS box inserted at the center of the wall.

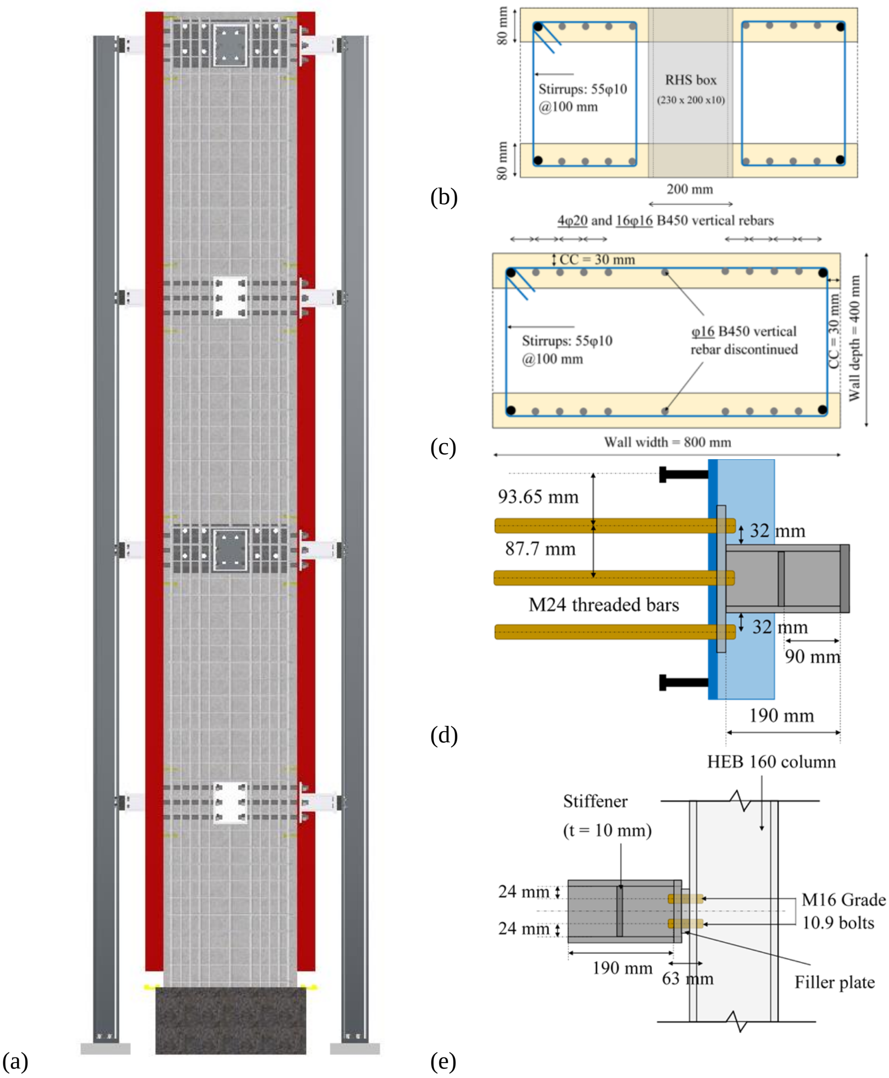

Fig. 2. (a) Overview of the designed specimen, (b)-(c) sectional view of the wall cross-section with and without the RHS box, (d) link-to-wall connection and (e) link-to-column connection.

4Φ13 shear studs are welded to the UPNs corresponding to each connection zone (2 on top and 2 at the bottom of the connection), to provide suitable shear resistance. The threaded bars as well as the shear studs are placed at the hollow part of the pre-cast DS walls, so that each part could be assembled on-site before the final concrete casting. The link-to-column connections are designed as shear resisting. A steel end plate (132 mm x 112 mm x 15mm) is welded to the other end of the coupling links, which is then connected to the flanges of the steel columns. 4 nos. M16 bolts are used at each link-to-column connection zone to provide the required shear resistance (Fig.

2e). A 30 mm thick filler plate is also used between the link end plate and the column flanges to have enough space for the link to be removed after damage. The column base connections and the wall foundation have been designed according to the standard Eurocode design guidelines [11, 12].

3 Experimental campaign

3.1 Test set-up and support conditions

Fig. 3 shows the real-life view of the full test set-up with the lateral supports. A pair of hydraulic cylinders are attached to the reaction wall corresponding to the intermediate level (3 m above ground) and top level (6 m above ground) of the HCW system. A load cell (capacity = 250 kN) is connected to one of the cylinders at each level. The cylinders are then attached to solid steel boxes, which are then attached to the load plates directly connected to the specimen. The foundation and the column base plates are connected to the strong floor using dywidag bars (each pretensioned with a 30 kN force) to achieve fixity.

(a)

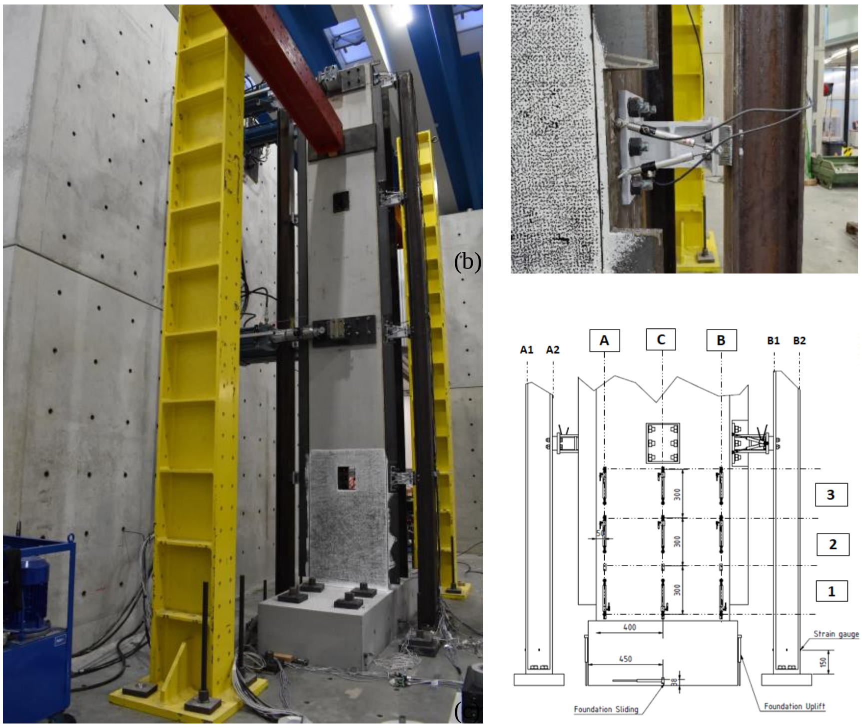

Fig. 3. (a) Real-life test set-up with instrumentation on (b) links and (c) wall

3.2 Instrumentation

Several sensors are used for specific observations: (i) LVDTs (one on each hydraulic cylinder) and wire sensors (one at each level of load introduction) – to measure the global displacements of the HCW system; (ii) one load cell at each level of load introduction; (iii) inclined LVDTs on each link to measure the local deformations (see Fig. 3b); (iv) strain gauges on the column flanges and rosettes on their web to estimate the axial forces taken by the side columns, while being aware of any torsional influence; (v) LVDTs on the back face of the wall to measure the crack widths at the bottom part of the wall (see Fig. 3c); and (vi) LVDTs to measure the foundation uplift and any occurrence of sliding.

3.3 Loading protocol

The ECCS 1986 [13] cyclic testing procedure is followed. A load proportion = 0.4 is maintained while applying the load at two levels of the HCW, i.e. loading at intermediate level = 40% of the loading at top level (chosen with regards to the fundamental mode of vibration of the HCW – obtained and verified through numerical models). The yield force (total base shear), F_y and the yield displacement (at top), e_y, could be estimated from the first cycle. An yield displacement, e_y = 28 mm is considered to set up the history of the cyclic test as shown in Table 1.

Table 1. Cycles amplitude and history followed for specimen C1

N° of cycles [progressive N° of cycles]	Amplitude	
1 [1]	$\pm e_y/4$	± 7 mm
1 [2]	$\pm e_y/2$	± 14 mm
1 [3]	$\pm 3e_y/4$	± 21 mm
1 [4]	$\pm e_y$	± 28 mm
3 [7]	$\pm 2e_y$	± 56 mm
3 [10]	$\pm 3e_y$	± 84 mm
1 [11]*	$\pm 4e_y$	± 112 mm

*testing was stopped after 1 cycle for safety reasons

3.4 Constructing the HCW system

The step-by step construction of the HCW system is summarized as follows:

Stage 1 – *Production of steel parts*: The individual steel parts were produced, cut and fabricated together for the relevant assemblies, i.e. making bolt holes; welding end plates (with bolt holes) to the horizontal links, columns; welding studs to UPNs, etc.

Stage 2 – *Prefabricating double (DS) walls*: The RHS boxes were sent to the concrete production unit to cast them inside the DS RC walls (see Fig. 4).

Fig. 4. Prefabricated DS walls and steel parts

Stage 3 – *Transportation and installation at site*: All steel and concrete parts arrived at the testing laboratory. The prefabricated DS wall panels were temporarily stored and then placed at the testing positions – inside the foundation formwork.

Stage 4 – *Placing foundation rebars for first stage of casting*: Necessary rebars were placed in the foundation (see Fig. 5a). Dywidag bars were used to fix the foundation in to the strong floor of the laboratory. The first stage of casting is conducted.

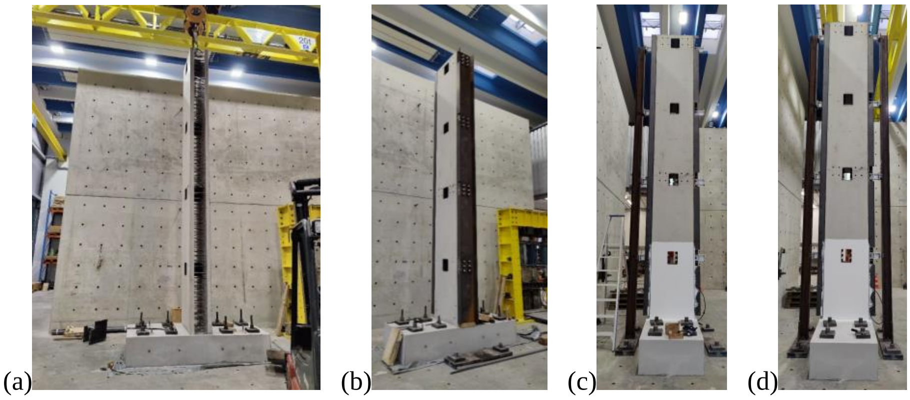

Fig. 5. (a) First stage and (b) second stage of casting, (c) & (d) attaching links and columns

Stage 5 – *Installing threaded bars, UPNs for second stage of casting*: Threaded bars are then placed at each storey level, on both sides of the DS walls, through plastic pipes to maintain the unbonded nature of the connections. The UPN profiles (with welded shear studs) are then attached to both sides of the wall using the threaded bars (see Fig. 5b). The central part of the prefabricated DS walls is then casted in two stages (half of the wall height) to avoid high punching stresses on the precast shells.

Stage 6 – *Installing links and columns*: After concrete setting, the foundation is pretensioned through the dywidag bars to minimize any possibility of sliding at the intersection between the foundation of the system and the strong floor. The threaded bars are loosened to attach the horizontal links on both sides of the wall (see Fig. 5c). The columns are subsequently placed with the base plates and connected to the horizontal coupling links (see Fig. 5d). Filler plates are used at each link-to-column connection to allow for an easy removal of the link. All bolts at the link-to-wall and link-to-column connections are tightened.

4 Experimental results

The force-displacement behaviour obtained at the top of the HCW is plotted in Fig. 6 to illustrate its cyclic behaviour. The total base shear at the yield displacement, e_y = 28 mm, is obtained to be approximately 199 kN, corresponding to 142 kN load at the top level and 57 kN at the intermediate level of the HCW. At this point, the structure is observed to enter in the plastic zone. Results obtained from the local components are subsequently plotted with respect to the load applied at top of the HCW for specific observations.

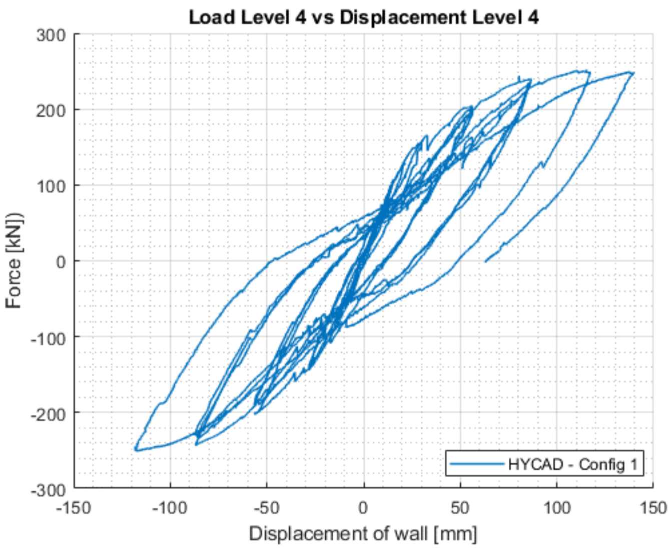

Fig. 6. Global force-displacement curve obtained from the top of the HCW

Fig. 7 shows the local displacements obtained from the diagonal sensors installed on the links at each storey level with respect to the shear load applied at the top of the HCW. At a load = 142 kN (4th cycle, e_y = 28 mm), yielding is achieved in all links. Furthermore, a permanent deformation (≈ 1.5 – 2 mm) can be noticed in the links from the diagonal sensors (see Fig. 7) after the final cycle – which corresponds to a vertical shear deformation of 6 – 8 mm. The damage in the webs also validates the fact that the links behaved to be shear critical in nature, fulfilling one of the design objectives. The evolution of crack widths at the bottom part of the wall are illustrated in Fig. 8. At a load = 142 kN (4th cycle, e_y = 28 mm), all crack widths are noticed to be negligible i.e. < 0.3 mm. However, bigger cracks formed later during the higher cycles ($2e_y$, $3e_y$…) as expected. The columns also stayed in the elastic zone until the end of the test. Finally, real-life images are provided to show the damages in the wall and one of the links (Fig. 9). Further details can be found in the relevant deliverables of project HYCAD [14, 15].

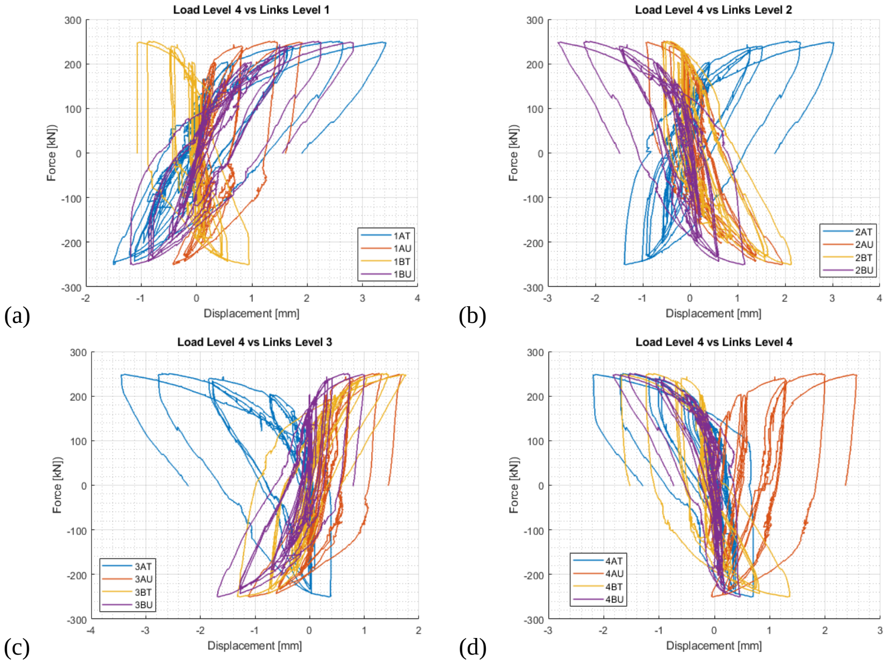

Fig. 7. Deformation in the links corresponding to the load applied at the top of the HCW, (a) 1st storey, (b) 2nd storey, (c) 3rd storey and (d) 4th storey

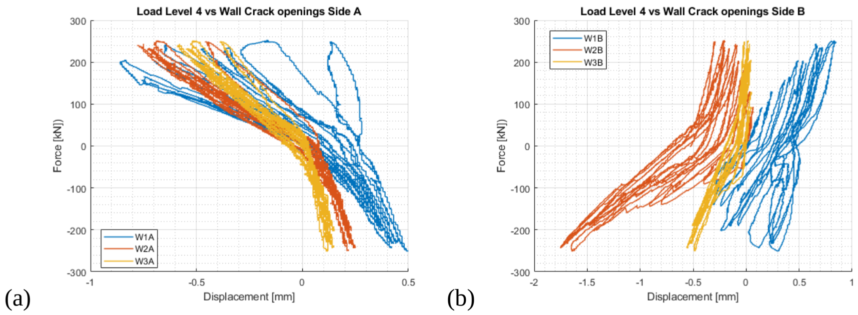

Fig. 8 Evolution of cracks at the ground storey of the wall with respect to the load applied at the top of the HCW, (a) Side A and (b) Side B

Fig. 9. Images showing the final state of (a) the bottom part of the RC wall and (b)-(c) a link

5 Repairability of the HCW system

As per the design hypothesis of the HYCAD HCW systems, the steel columns should stay in the elastic zone with minimal damage to the RC wall, while all damages concentrating on the dissipative steel links. Therefore, in this case, the coupling links can be replaced with new ones following the steps described below (see Fig. 10):

i. the link-to-column connection are unbolted and the filler plate is removed.
ii. then the threaded bars of each link-to-wall connection can be loosened (and pulled inside the RHS box) to take out each link.
iii. the new link can be integrated using the same approach in a reverse manner.

However, some critical observations could be made during the experimental campaign regarding the replaceability of the HYCAD HCW system. It is absolutely necessary to maintain the unbonded nature of the link-to-wall connections, i.e. the threaded bars. Therefore, the plastic tubes should be cut in a precise manner and rubber heads can be used at both ends of the threaded bars to avoid any concrete getting inside the plastic tubes during the casting stages. Furthermore, casting one storey at a time can prove to be beneficial as in that case the concrete pressure will be reduced, hence decreasing chances of concrete getting inside the plastic tubes.

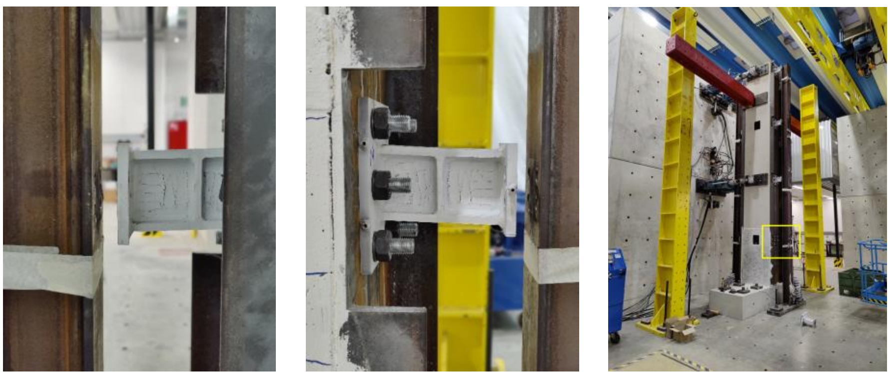

Fig. 10. Removing a damaged link at the ground storey level of the HCW

6 Conclusion

The seismic behaviour of an innovative hybrid coupled wall (HCW) system made of a fixed-base reinforced concrete (RC) wall connected to two steel side columns via steel horizontal coupling links, is investigated in this study through a full-scale cyclic test as per the ECCS 1986 guidelines. A 4-storey specimen is pre-designed and constructed for the experimental campaign. The steel coupling links are designed as shear critical "fuses", i.e. concentrating all damages in the links prior to any or limited yielding of the RC wall and the steel side columns. The pre-design details are provided with relevant engineering drawings. Measurements obtained from the load cells, displacement transducers and strain gauges validate the fact that, the design objectives are achieved i.e. all links yielded in shear at an estimated yield displacement of the global HCW system (e_y = 28 mm), prior to minimal damage in the RC wall (crack width < 0.3 mm), while the columns stayed in the elastic zone throughout the experiment. Real-life images are shown for the ultimate state of the HCW system and its different components. Finally, the constructability and replaceability aspects of the proposed HCW system is discussed in details to offer a clear view of the HCW system from a practitioner's point of view.

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