STUDIO PRELIMINARE PER LA RIDUZIONE DEL DANNO SISMICO IN PARETI IBRIDE ACCIAIO-CALCESTRUZZO

PRELIMINARY STUDY FOR REDUCING SEISMIC DAMAGE IN STEEL-CONCRETE HYBRID-COUPLED WALLS

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ABSTRACT

An innovative single-pier hybrid coupled wall (SP-HCW), made of a single reinforced concrete wall coupled to two steel side columns by means of steel link, was recently proposed. The numerical analyses for this innovative solution showed the achievement of the designed seismic performances and the desired ductile global behaviour. However, the bottom zones of the concrete wall might experience undesired damages in case of strong seismic inputs. Hence, a new solution is presented and preliminary investigated, i.e., the wall is pinned at the base and equipped with additional vertical dissipative devices. This new configuration is expected to achieve no damage of the wall without reducing its dissipative capacity. In this article the results of preliminary pushover analyses are discussed to evaluate the expected performances of the proposed solution.

SOMMARIO

Un sistema strutturale innovativo composto da una singola parete in calcestruzzo armato accoppiata a due colonne in acciaio attraverso link in acciaio, è stato di recente proposto dagli autori. Le analisi numeriche di questa soluzione hanno mostrato prestazioni sismiche in linea con gli obiettivi progettuali e il comportamento globale duttile desiderato. Tuttavia, la zona inferiore della parete in calcestruzzo armato potrebbe subire un danneggiamento indesiderato in caso di elevato impegno sismico. Per questo motivo è stata studiata una nuova soluzione che prevede la parete incernierata alla base ed equipaggiata con dei dispositivi verticali dissipativi. In questo modo si prevede di evitare il danneggiamento della parete senza ridurre la sua capacità dissipativa. In questo articolo sono

discussi i risultati delle analisi statiche non lineari preliminari al fine di valutare le prestazioni della soluzione proposta.

1 INTRODUCTION

Hybrid coupled walls (HCW) are commonly made by two reinforced concrete (RC) walls connected by means of steel coupling beams or steel-concrete composite coupling beams, as depicted in Fig.1a. The walls are subjected to bending, shear, and an alternation of tension and compression axial forces while the coupling beams are subjected to bending and shear; the resulting stiffness and strength are greater than the summation of the contributions of the individual uncoupled walls. A different configuration for HCWs, called single pier hybrid coupled wall (SP-HCW) was developed in [1]: a single RC wall is coupled to two steel side columns through steel links (Fig.1b). In this case the RC wall is subjected to bending and constant axial force from permanent loads while the side steel columns are subject to an alternation of compression and traction plus bending moments due to the eccentricity of the link connections. Pinned connections are used between the links and the side columns while the connections of the links to the RC wall transfer both shear and bending moment. The damaged steel links can be replaced if detailed as proposed and tested in [1] and [2].

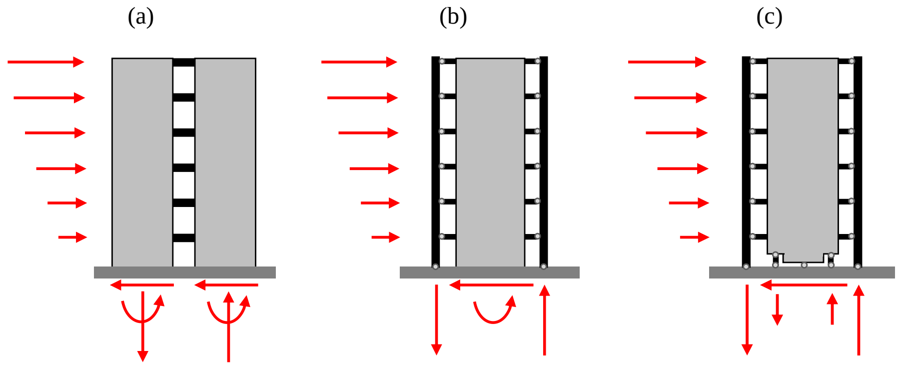

Fig. 1. (a) conventional HCW; (b) SP-HCW; (c) SP-HCW with RCC and hinged base.

SP-HCW were the object of various numerical studies for seismic behaviour simulation in [3][4][5][6][7][8], that showed advantages and disadvantages of this structural solution. Among benefits there are the absence of alternate traction-compression forces in the RC wall as well as smaller dimensions thanks to the smaller size of the steel side columns with respect to the two RC walls. Among critical issues the main one was identified to be the possible damage at the base of the RC wall that would reduce the actual reparability of the system. Hence, to improve the seismic performance of SP-HCWs, it is important to study solutions able to reduce vulnerability of the RC wall. Accordingly, the objective of this study is to explore the use of replaceable corner components (RCC), as those proposed and successfully tested in [9] in RC walls, arranged in the configuration depicted in Fig.1c where a hinged connection is inserted between the RC wall and the foundation. To this end, a case study is designed, a nonlinear finite element model adopted, and preliminary results obtained from nonlinear static (pushover) analysis illustrated and discussed.

2 CASE STUDIES

2.1 Design of the tested structures

The same 6-storey residential building adopted as testbed structure in [1] as well as in [3] is considered. Floors have an extension of 25.00 m × 14.15 m and inter-storey height is 3.50 m, floor loads are permanent G_k = 4.30 kN/m^2 and variable Q_k = 2.00 kN/m^2, roof loads are permanent G_k = 3.30 kN/m^2 and variable (snow) Q_k=1.97 kN/m^2. The considered case study is designed as having a gravity-resistant steel frame structure (floors, beams, columns) where beam to column joints and restraints at the base of the columns can be considered as pinned connections. Beams and columns of the gravity-resistant frame are designed according to Eurocode 3 [10] prescriptions, having assumed steel grade S275 (nominal yield stress f_y = 275 MPa) and a limitation to the vertical deflection at service limit state equal to L/250, L being the beam span length. Details in the design of the gravity-resisting frame can be found in [1].

Table 1. Designed case studies.

Case	SP-HCW	SP-HCW with replaceable corner components
RC wall section	210 cm x 36 cm	210 cm x 36 cm
Steel rebars at the base: confined areas	10 + 10 ϕ26	Hinged
Steel rebars at the base: non-confined areas	8 ϕ14	Hinged
Corner components	N/A	D 219,1 x t 10 mm
Steel link flange	100 mm x 9,8 mm	100 mm x 9,8 mm
Steel link web	220,4 mm x 6,2 mm	220,4 mm x 6,2 mm
Steel side columns	HE260B	HE260B

The gravity-resistant frame is connected to two SP-HCW for each direction that are the only components providing the lateral resistance against horizontal actions. The seismic resistant SP-HCWs were designed according to the methodology proposed in [3] for the site of Camerino, Italy, following the indication for the seismic input provided by the Italian seismic building code. The design was made assuming a coupling ratio equal to 60% for both the SP-HCW and the SP-HCW with added corner vertical components. The results of the design are reported in Table 1. Concrete is taken as class C30/37 (characteristic cylindrical compressive strength f_{ck} = 30 MPa) and reinforcements are B450C (characteristic yield stress f_{yk} = 450 MPa) in accordance with Eurocode 2 [11]. Reinforcements are designed following the DCM rules of Eurocode 8 [12], i.e., using a confined area for the outer portions of the RC section as indicated in [3]. Steel grade S355 (nominal yield stress f_y = 355 MPa) is adopted for links, side columns, and corner components. Links and side columns were sought among double-T profiles while corner components among circular hollow profiles. Links were designed using the uniform distribution assumption as described in [3].

2.2 Nonlinear finite element model

A two-dimensional nonlinear model is implemented in the finite element software OpenSees [13], following the same approach detailed in [4]. The elastic axial and flexural behaviour of the steel link is modelled with a Euler-Bernoulli beam element with finite length while the plastic flexural and the elasto-plastic shear response are lumped at the link end connected to the RC wall, using rigid-plastic zero-length elements. A force-based distributed-plasticity fibre frame element is used to describe the flexural behaviour of the RC wall, with different constitutive descriptions used for the confined and unconfined portions of the concrete cross section. The shear behaviour of the RC wall elements is modelled as linear elastic by aggregating to the flexural stiffness of the section an elastic initial stiffness equal to G_cA_v, where G_c is the elastic tangential modulus of the concrete, and

A_v is the shear area, evaluated as 5/6 times the area of the rectangular cross section. A couple of truss elements, transmitting axial force only, are used to model the RCCs; their nonlinear behaviour is described using the OpenSees SteelBRB model presented in [14] and [15]; the material parameters assigned are the value of the yield strength, f_y = 355 MPa (steel S355), and the initial elastic modulus, E_s = 210 GPa. For the sake of simplicity, the kinematic hardening of rebars, steel links and RCC is set to negligible values. This assumption reduces post-yielding redistributions, hence, providing a clearer representation of the plastic behaviour, for the benefit of the presented preliminary investigation. More refined nonlinear models for steel will be adopted in future studies programmed for this structural solution.

2.3 Results of pushover analysis

The behaviour of the designed SP-HCWs is assessed through nonlinear static (pushover) analysis, considering a triangular forces distribution. The global response of the two case studies is described by the capacity curves reported in Fig.2. The steps related to the activation of the first and last horizontal links, the yielding of the first reinforcement bar, and yielding of the RCCs are highlighted by coloured markers.

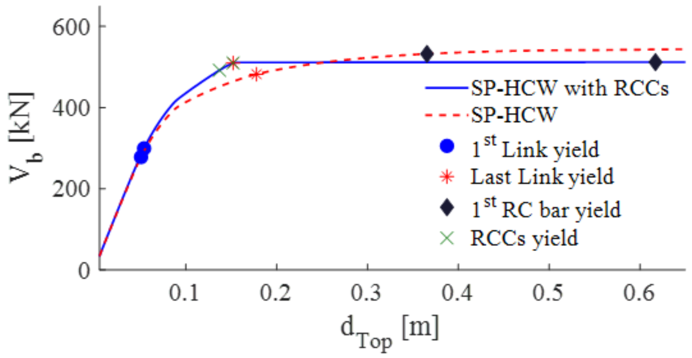

Fig. 2. Capacity curves comparison and limit states.

The elastic phase is identical for both systems, while differences between the capacity curves are observed with the first link activation. In the SP-HCW, the progressive yielding of the horizontal links leads to a gradual reduction in stiffness, which, once all the links yielded, is only supported by the reinforced concrete wall as hardening of materials is neglected in the adopted model. The capacity curve continues to increase slightly until the concrete and the reinforcing bars of the RC wall reach crushing and yielding strength, respectively. The SP-HCW with RCCs has a lower reduction in the stiffness after the horizontal links yielded. However, when the RCCs yield, no further hardening is possible, due to the simplified constitutive modelling assumption in the post-elastic behaviour of steel. Both the designs are effective in protecting the RC wall, anticipating its damage by activating the horizontal links and RCCs. The yielding of the wall reinforcement bars occurs in both cases for large displacements and the initiation is localised, for the SP-HCW and SP-HCW with RCCs systems respectively, at the base of the wall and in the area above the vertical links. The SP-HCW with RCCs offers a better contribution in resistance for medium displacements, while the SP-HCW has a slightly higher resistance for large displacements by virtue of the contribution offered by post-elastic response of the RC wall.
The effective CR and its evolution for rising lateral loads is compared in Fig.3. The CR values fluctuates strongly until the system attains the plastic conditions and then become stable close to

the design value 0.6. The difference between the stable CR value and the design value is slightly more pronounced for the systems-HCW (0.54) than for the SP-HCW with RCCs (0.56).

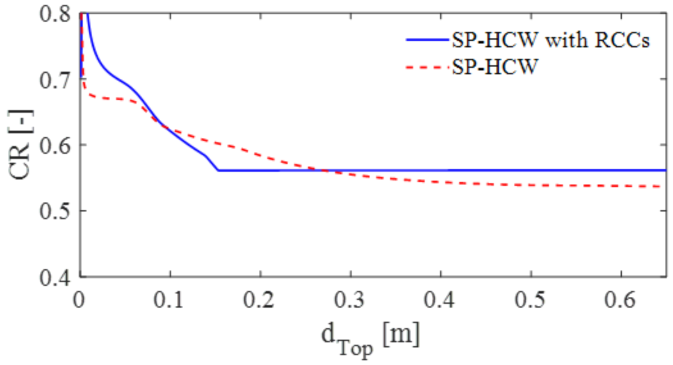

Fig. 3. Evolution of the coupling ratios obtained from nonlinear analysis.

The shear response of the links in the first and last floor is shown in Fig.4. A slight delay is observed between the links at the lower and higher storeys. The SP-HCW with RCCs shows slightly less differences in this regard.

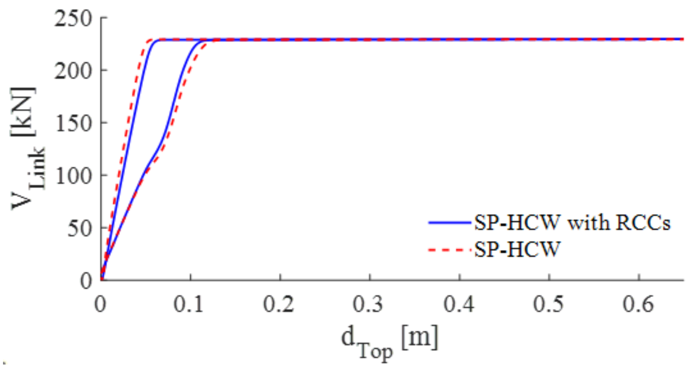

Fig. 4. Shear response of the links at the first and last floor.

The axial response of left and right RCCs is shown in Fig.5. The two vertical links exhibit the same behaviour (symmetrical material in tension and compression) with a slight divergence due to the vertical loads acting differently on the two elements. Fig.6 and Fig.7 directly show the terms involved in the calculation of the CR represented in Fig.3. The behaviour of the steel columns shown in Fig.6 is very similar for the two systems. On the other hand, in Fig.7 a softer transition between the elastic and plastic phases is observed in the case of the SP-HCW, due to the gradual crushing of the concrete and yielding of the bars in the cross-section and along their height. The transition in the SP-HCW with RCCs is more abrupt following the yielding of the vertical links.

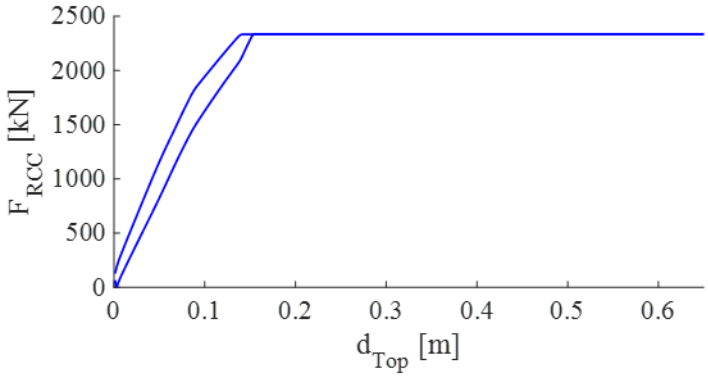

Fig. 5. Axial response of the RCCs.

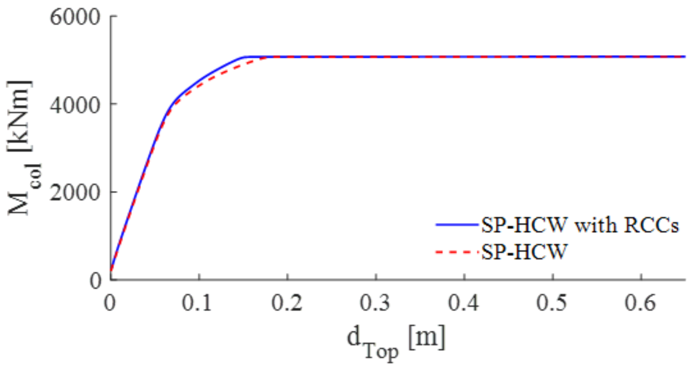

Fig. 6. Base moment contributed by the side steel columns.

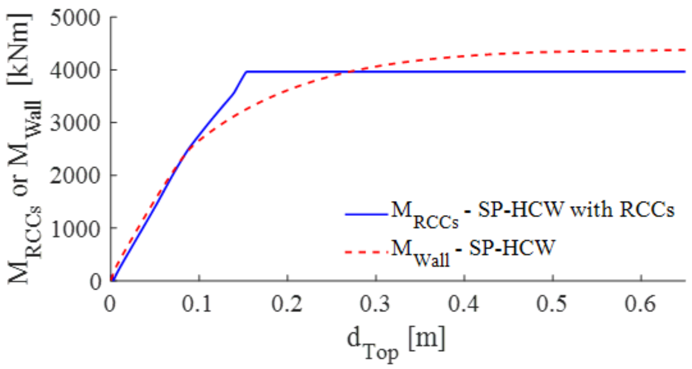

Fig. 7. Base moment contributed by the RC wall or the RCCs (depending on the system type).

CONCLUSIONS

This preliminary study, part of a larger research project on single-pier hybrid coupled wall (SP-HCWs), focuses on the comparison between two different base design for the reinforced concrete (RC) wall, i.e., RC wall continuous with the foundation and RC wall hinged at the foundation with added replaceable corner components (RCCs). Results obtained from nonlinear static (pushover) analysis show that the proposed innovative solution (SP-HCW with added RCCs) is a potentially appealing alternative to reduce damage at the base of the RC wall while preserving, or even increasing, the desired seismic performance. Further studies involving a larger number of case studies and more refined modelling are underway to gain more insight with respect to this preliminary results.

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KEYWORDS

Steel and concrete hybrid structures; steel structures; dissipative links; seismic design; seismic-resistant structures.