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Preliminary analyses of an innovative solution for reducing seismic damage in steel-concrete hybrid-coupled walls

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Abstract

Hybrid steel-concrete structures used as earthquake-resistant systems are an interesting solution for buildings in seismic prone areas, combining in effective ways the benefits of concrete and steel. In this context, 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 system is conceived to reduce the damage in the reinforced concrete wall while concentrating dissipation to the replaceable links. Although the numerical analyses for this innovative solution showed encouraging seismic performances and the desired ductile global behaviour, bottom zones of the concrete wall might experience undesired damages. Starting from the first proposed SP-HCW, in this study a new solution for its base is presented and preliminary investigated, i.e., the wall is designed as pinned at the base and equipped with additional vertical dissipative devices. In this way, this new configuration is expected to achieve lower 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 structural solution.

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

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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 by Dall'Asta et al. (2015): 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 Dall'Asta et al. (2015) and Morelli et al. (2016).

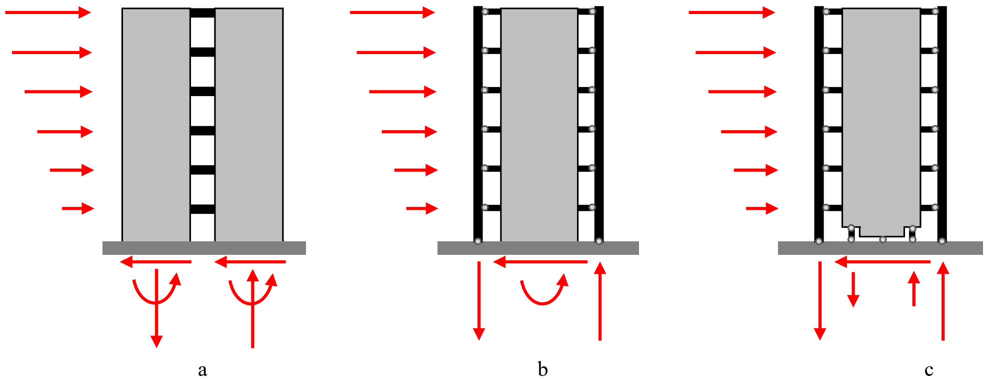

Fig. 1 (a) conventional HCW; (b) SP-HCW; (c) SP-HCW with RCC and hinged base.

SP-HCW were the object of various numerical studies for seismic behaviour simulation in Zona et al. (2016, 2018), Das et al. (2017, 2018), Salameh et al. (2020, 2021), that showed pros and cons of this structural solution. Among benefits there are the absence of alternate traction-compressions 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 reduce vulnerability of the RC wall and, hence, resilience, as in Caprili et al. (2021). Accordingly, the objective of this study is to explore the use of replaceable corner components (RCC), as those proposed and successfully tested by Liu and Jiang (2017) 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 testbed structures

The same 6-storey residential building adopted as testbed structure in Dall'Asta et al. (2015) as well as in Zona et al. (2016) 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 (2005) 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 Dall'Asta et al. (2015).

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 Zona et al. (2016) 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 (characteristic cylindrical compressive strength f_{ck} = 30 MPa) and reinforcements are B450C (characteristic yield stress f_{yk} = 450 MPa) in accordance with Eurocode 2 (2004). Reinforcements are designed following the DCM rules of Eurocode 8 (2004), i.e., using a confined area for the outer portions of the RC section as indicated in Zona et al. (2016). 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 Zona et al. (2016).

Table 1. Designed case studies.

Case	SP-HCW	SP-HCW with replaceable corner components
RC wall section	210 cm × 36 cm	210 cm × 36 cm
Steel rebars at the base: confined areas	10 + 10 d26	Hinged
Steel rebars at the base: non-confined area	8 d14	Hinged
Corner components	N/A	D 219,1 mm × t 10 mm
Steel link flange	100 mm × 9,8 mm	100 mm × 9,8 mm
Steel link web	220,4 mm × 6,2 mm	220,4 mm × 6,2 mm
Steel side columns	HE260B	HE260B

2.2. Nonlinear finite element model

A two-dimensional nonlinear model is implemented in the finite element software OpenSees (McKenna 2011), following the same approach detailed in Zona et al. (2018), as briefly described hereafter. The elastic axial and flexural behaviour of the steel link is model 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 Zona and Dall'Asta (2012) and Gu et al. (2014); 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, including modelling uncertainties (Badalassi et al. 2013, Franchin et al. 2018, Scozzese et al. 2017, 2018), and refined experimental-based description of the link behaviour (Caprili et al. 2018).

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. 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 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.

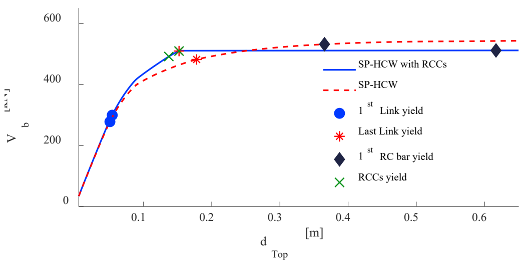

Fig. 2. Capacity curves comparison and limit states.

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).

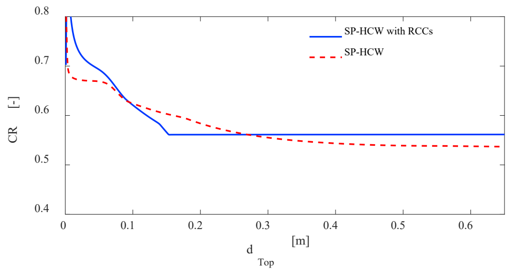

Fig. 3. Evolution of the coupling ratios obtained from nonlinear analysis.

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. 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.

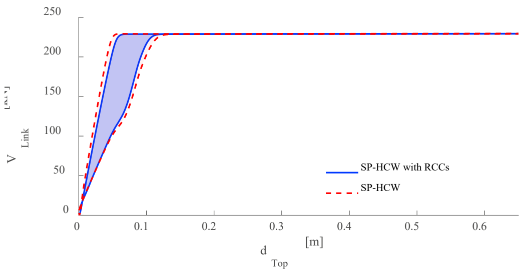

Fig. 4. Shear response of the horizontal links.

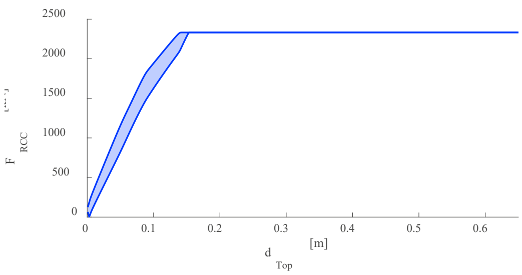

Fig. 5. Axial response on the RCCs.

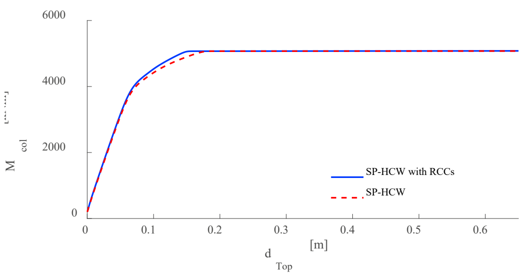

Fig. 6. Base moment contributed by the steel columns.

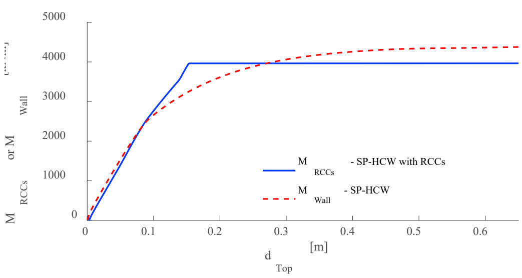

Fig. 7. Base moment contributed by the RC wall or the RCCs (depending on the system type).

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 with added replaceable corner components (RCCs) and hinged at the foundation interface. Results obtained from nonlinear static (pushover) analysis show that the proposed innovative solution (SP-HCW with added RCCs) is a viable alternative to reduce damage at the base of the RC wall while preserving the desired seismic performance. Further studies encompassing a larger number of case studies and more refined modelling are indeed necessary to gain more insight and validate this preliminary results.

Acknowledgements

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