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An innovative solution for earthquake resistant hybrid steel-concrete systems with replaceable dissipative steel links

A. Zona, G. Leoni & A. Dall'Asta
Univesity of Camerino, Italy

T. Bogdan, C. Braham & H. Degée
University of Liège, Belgium

ABSTRACT: In this work innovative Hybrid Coupled Shear Walls (HCSW) are considered, their design is discussed, their efficiency and limitations evaluated by means of nonlinear static (pushover) analysis. Different numbers of storeys, wall geometries and design assumptions are studied in order to give an overview of situations of interest in European seismic prone areas. The design of an experimental test regarding the performance of the connection of a seismic link embedded in a concrete shear wall is presented. This study is part of a larger research project named INNO-HYCO (INNOvative HYbrid and COmposite steel-concrete structural solutions for building in seismic area) funded by the European Commission.

1 INTRODUCTION

Steel-concrete hybrid structures are of current limited application for construction in seismic areas, compared to classical steel-concrete composite structures. However, their use seems promising provided that suitable structural schemes and proper design methods are identified and developed. While the deformation demands for the steel and concrete components in traditional composite structures are in the same range (since concrete and steel are part of a same structural member), hybrid structures allow a more efficient design of both the reinforced concrete and the steel structural elements. In this context, the present paper proposes an innovative hybrid solution coupling concrete shear walls by steel link elements tailored to act as dissipative replaceable fuses. The concrete wall is designed to control the stiffness while the links are controlling the resistance and optimized for an efficient dissipation of the seismic energy. Preliminary design is discussed and efficiency and limitations of the system are evaluated by means of nonlinear static and dynamic analyses. Different numbers of storeys and different global geometries of the structural system are examined in order to give an overview of situations of interest in European seismic prone areas.

Objective of the research project INNO-HYCO (INNOvative HYbrid and COmposite steel-concrete structural solutions for building in seismic area), funded by the European Commission, is the study of steel-concrete hybrid systems obtained by coupling reinforced concrete elements (e.g., walls and shear panels) with steel elements (e.g., beams and columns). Such systems should permit to exploit both the stiffness of reinforced concrete elements, necessary to limit building damage under low-intensity earthquakes, and the ductility of steel elements, necessary to dissipate energy under medium- and high-intensity earthquakes. Such hybrid systems might represent a cost- and time-effective type of construction since: (i) simple beam-to-column connections could be used for the steel frame constituting the gravity-resisting part, (ii) traditional and well-known building techniques are required for the reinforced concrete and steel components.

2 COUPLED SHEAR WALL SYSTEMS

2.1 *Conventional systems*

Coupled shear wall systems obtained by connecting reinforced concrete shear walls by means of beams placed at the floor levels constitute efficient seismic resistant systems characterized by good lateral stiffness and dissipation capacity. Structural steel coupling beams (Figure 1) or steel-concrete composite coupling beams provide a viable alternative (e.g. El-Tawil et al. 2010), particularly for cases with restrictions on floor height. The coupling beam-wall connections depend on whether the wall boundary elements include structural steel columns or are exclusively made of reinforced concrete elements. In the former case, the connection is similar to beam-column connections in steel structures. In the past decade, various experimental programs (El-Tawil et al. 2010) were undertaken to address the lack of information on the interaction between steel coupling beams and reinforced concrete shear walls. Coupled shear wall systems suffer from being difficult to be repaired after strong earthquakes.

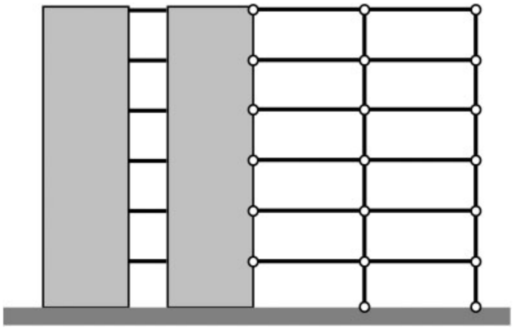

Figure 1. Example of conventional hybrid coupled shear wall system connected to a gravity-resisting steel frame with pinned beam-to-column joints.

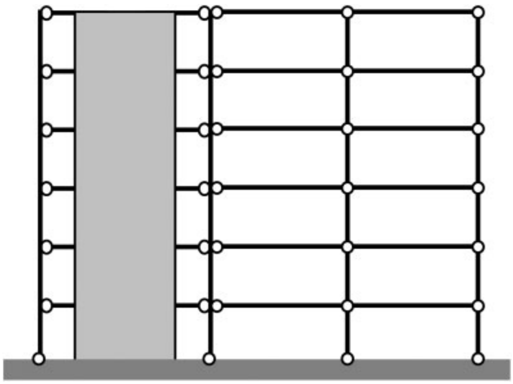

Figure 2. Example of innovative hybrid coupled shear wall system connected to a gravity-resisting steel frame with pinned beam-to-column joints.

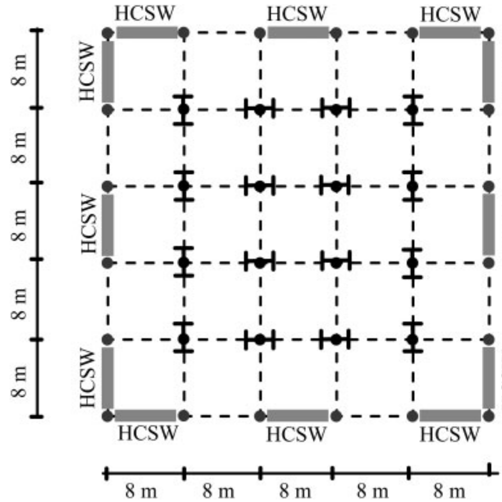

Figure 3. Floor geometry of the benchmark structures with positions of the HCSWs.

Table 1. Wide-flange cross sections of the columns of the vertical-resisting structure.

Storey #	4-storey case	8-storey case
8	–	HE 220 B
7	–	HE 220 B
6	–	HE 300 B
5	–	HE 300 B
4	HE 220 B	HE 450 B
3	HE 220 B	HE 450 B
2	HE 300 B	HE 450 M
1	HE 300 B	HE 450 M

2.2 *Innovative systems*

An example of innovative hybrid system is the reinforced concrete shear wall with steel links depicted in Figure 2. The connections between steel beams (links) and the side steel columns are simple: a pinned connection ensures the transmission of shear force only while the side columns are subject to compression/traction with reduced bending moments.

The structure is simple to repair if the damage is actually limited to the link steel elements. To this end, it would be important to develop a suitable connection between the steel links and the concrete wall that would ensure the easy replacement of the damaged links and, at same time, the preservation of the wall. Clearly, the proposed hybrid system is effective as seismic resistant component if the yielding of a large number of links is obtained.

3 DESIGN OF HCSW SYSTEMS

3.1 *Case study*

Two case studies are considered: 4-storey and 8-storey steel frames with the same floor geometry as shown in Figure 3. For each floor the vertical loads are $G_k = 6.5\,\mathrm{kN/m^2}$ and $Q_k = 3.0\,\mathrm{kN/m^2}$, while the floor total seismic mass is $1200\,\mathrm{kNs^2/m}$. Interstorey height is $h = 3.40$ m. The gravity-resisting frame has continuous columns (Table 1) pinned at the base. Beams (IPE500) are pinned at their ends. Steel S355 is used for columns, beams, and links. Selected materials for the reinforced concrete walls are concrete C25/30 and steel reinforcements S500.

The assumed seismic action is represented by the Eurocode 8 type 1 spectrum with $a_g = 0.25$ g and ground type C. Both verifications of the ultimate limit state (ULS) and of the damage limit state (DLS) are required.

3.2 *Preliminary design*

Preliminary designs based on the conventional force approach were made in order to identify possible optimal geometries. Two conditions were required in the design: (i) strength verification at ULS under the lateral forces derived from the design spectrum following the prescriptions in Eurocode 8.

A large number of HCSWs with different dimensions of the reinforced concrete shear walls and link lengths were evaluated and the seismic behaviour of the most promising designs was assessed through nonlinear static (pushover) analysis and multi-record incremental nonlinear dynamic analysis.

In the 4-storey case it is observed that reinforcements in the concrete wall yield together with yielding in bending of the steel links in the first three storeys. Afterwards, further plastic dissipation is fostered by the successive yielding in bending of the links at the last storey. Then the peak strength of the reinforced concrete wall is achieved leading to the maximum sustained base shear before leading the bracing system to failure due to failure of the reinforced concrete wall. A different seismic performance is observed in the 8-storey design, where the steel links at all storeys yield in bending before any damage in the reinforced concrete wall. Afterwards reinforcements yields and shortly afterwards all links yield in shear. The concrete peak strain is attained but the bracing system is still able to exhibit global hardening. Collapse is reached when the link at the fifth storey fails in combined banding and shear. Despite these differences, pushover verifications are satisfied (capacity displacement larger than target displacement) in both case studies. In addition, it is observed that the HCSW systems designed are not prone to soft storey formation, even when the yielding of the steel links is not simultaneous, thanks to the contribution of the reinforced concrete wall.

3.3 *Design based on rules inherited from other systems*

The results obtained in this first design stage directed the research towards the definition of a design approach that inherits recommendation for capacity design from other structural systems involving similar dissipative mechanisms in the links, i.e. eccentric braces in steel frames, as well as indication to reduce damages in the reinforced concrete wall. The design procedure attempted in this second stage of this research is subdivided in the following steps:

– Step 1: assign dimensions of the reinforced concrete wall by selecting height-to-length ratio and thickness;
– Step 2: design of the steel links based on bending and shear obtained from linear analysis (e.g. spectrum analysis) with assigned uniform over-strength; design of the steel side columns using the summation of the yield shear forces of the links (amplified with $1.1\gamma_{ov}$) as design axial force;
– Step 3: design of the wall longitudinal reinforcements to provide an assigned over-strength compared to the bending moment obtained from linear analysis; design of the transverse reinforcements to avoid shear collapse of the wall considering the maximum shear at the base derived from the limit condition of yielded steel links and yielded wall in bending; reinforcement detailing according to Eurocode 8 DCM rules.

The application of the above design rules and the necessity to limit the shear force at the base of the wall brought to the adoption of six HCSW systems for each direction (Figure 3). In order to investigate the main geometric and design parameters, 18 cases were designed, comprising three height-to-length ratios of the wall taken with constant thickness and relevant links (length 600 mm for the 4-storey case and 660 mm for the 8-storey case) designed with over-strength equal to one, longitudinal reinforcements in order to obtain three values of wall over-strength (1.00 in cases labelled as R10, 1.25 in cases labelled as R12, 1.50 in cases labelled as R15), as detailed in preliminary report “” produced by ULg. Reinforcement bars were concentrated in the lateral confined parts of the wall as for Eurocode 8 DCM rules. Significant quantities of longitudinal bars were required, especially for R15, in some cases exceeding Eurocode upper limits.

4 DESIGN OF DOWN SCALED CONNECTION

4.1 *General assumptions*

The tests aim to characterize the performance of the connection of a seismic link embedded in a concrete shear wall and the efficiency of the capacity design of such a system, with the objective of developing a plastic hinge in the replaceable part of the link, acting as a fuse, with all other components of the connection remain undamaged. For the materials, design values are used concrete C 25/30, steel grades (S275 for the link, S355 for the embedded part and S500 for reinforcements). The links are defined as being intermediate and the link stiffeners number is determined using a linear interpolation and the link rotation angle value θ_p. Face bearing plates are placed at the face of the concrete wall, in order to keep the integrity of the concrete part and for the yielding to take place in the profile section or rebars. The concrete wall dimensions are consistent with the experimental stand dimensions and the embedment length of the steel profile into the concrete part.

Two topologies are considered for what regards the embedment of the profile in the concrete shear wall and the connection of the replaceable part of the link to the embedded part.

– *Configuration No. 1 (Figure 4)* – The bending moment transferred by the link to the wall is resisted by shear studs, while the beam splice connection is placed at a distance of 100 mm from the top face of the concrete wall in order to allow an easy bolting of the removable part.

The design assumptions consists in forcing the creation of a plastic hinge in the replaceable part of the link and to capacity-design the part of the fixed part of the link embedded in the shear wall, the link-to-shear-wall connection and the bolted beam splice connection between the fixed and replaceable parts of the link.

The required number of shear studs is influenced by the value of the fore V, as shown in Figure 4. The

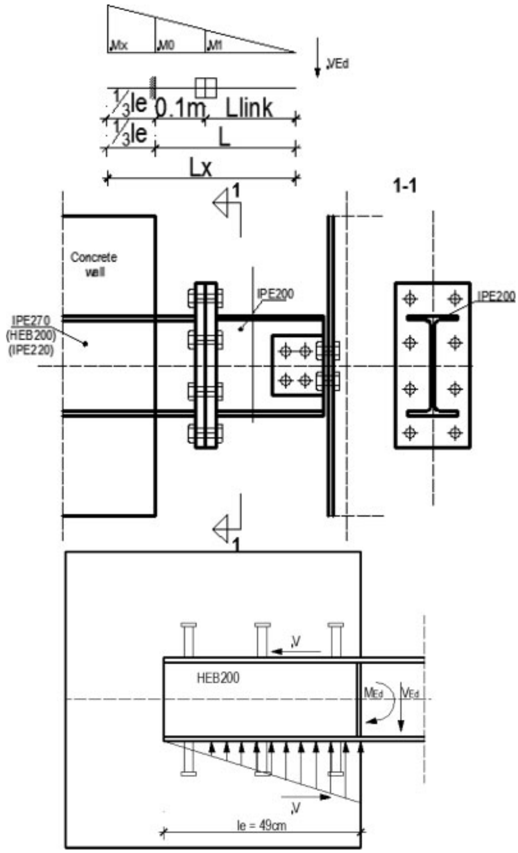

Figure 4. Configuration No. 1 – mechanical model.

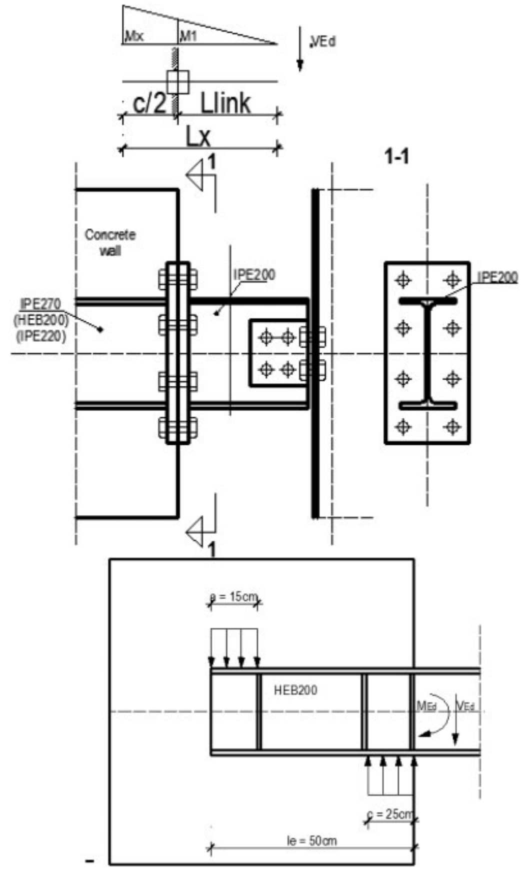

Figure 5. Configuration No.2 – mechanical model.

embedment length is determined using equilibrium equations of the mechanical model.

– *Configuration No. 2 (Figure 5)* – The bending moment transferred by the link to the wall is balanced by a couple of vertical forces. The connection link – embedded steel profile is placed at the top of the wall using threaded bushings.

The main differences with configuration 1 are:

– The positioning of the beam-splice connection of the fuse with the embedded part of the link. The end-plate is positioned right at the face of the shear wall, acting thus also as face bearing plate. Moreover, the connection is assumed to be realized in practice with threaded bushes instead of regular bolts;
– The assumed load transfer mechanism between the embedded part of the link and the concrete shear wall. In this case, no shear studs are used and a longer embedded part is considered in order to develop a static scheme where the applied bending moment is resisted by a couple of vertical load, activating the compression resistance of the contact profile-concrete.

According to the transfer mechanism, the embedded part of the profile is designed assuming conservatively that the bending moment increased linearly until the location of the first reaction force applied by the concrete on the profile.

The design of the beam splice connection has been realized with CoP, software developed by Universities of Aachen and Liege, based on the component method. The aimed and governing failure of the connection is failure of the steel bolts, before yielding of the beam flange of the profile.

5 SEISMIC BEHAVIOUR OF HCSW SYSTEMS

5.1 *Modelling assumptions and selected results*

The seismic behaviour of the designed HCSW systems was assessed through displacement-controlled nonlinear static analysis under applied lateral loads (pushover analysis), using the software FinelG. For the sake of simplicity, the evaluation of the seismic performances is based on a plane model of a single HCSW connected to two continuous columns equivalent to the relevant parts of the gravity-resisting steel frame, as shown In Figure 6 (4-storey case).

The reinforced concrete shear wall is represented by frame elements using a fibre description for the behaviour of the concrete in the longitudinal direction, allowing an accurate estimation of the behaviour

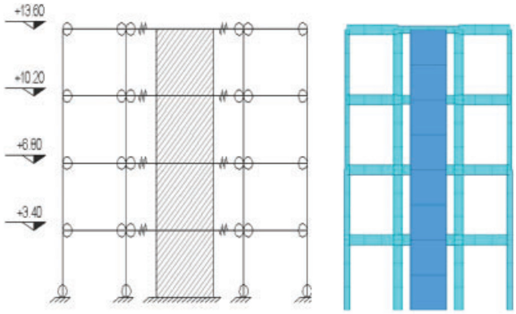

Figure 6. Numerical model.

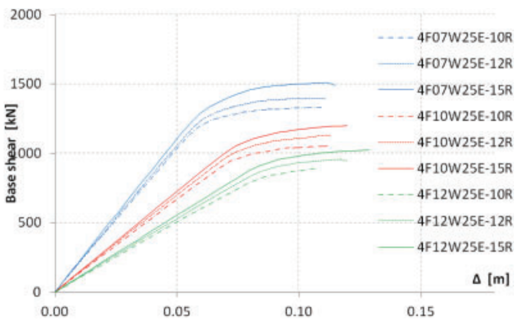

Figure 7. Pushover curve – global comparison.

in bending and thus of the possible plastic hinge at the bottom of the wall. Reinforcement of the shear wall is assumed to be sufficient to avoid a shear failure of the wall. The steel shear links are modelled using non linear frame elements for the bending contribution as well as non linear shear springs introduced at mid span of each steel link to account for the shear deformability of the link and for the possible yielding in shear of the links. This additional shear spring is required for a correct modelling of the system since most links are classified as intermediate according to EC8 definition and are thus more or less equally prone to shear or bending failure. Interaction between flexural and shear plastic deformations is considered in the post-processing of the results.

For all materials, the design values of the resistance are used ($f_{cd} = 20$ MPa, $f_{y,s} = 435$ MPa, $f_{y,p} = 355$ MPa). Steel behaviour is modelled by a bilinear elastic-perfectly-plastic law without strain hardening. Concrete is by a parabola-rectangle in compression and a linear behaviour in tension with tension stiffening. Particular values of the strain are $\varepsilon_{c2} = 0.002$ (end of the parabolic behaviour) and $\varepsilon_{cu2} = 0.0035$ (end of the rectangular behaviour). No confinement is considered for the concrete in compression.

Pushover curves (total base shear versus top displacement) for the 4-storey cases are shown in Figure 7, while for the 8-storey case is shown in Figure 8.

It is observed that increasing the wall over-strength has positive effects as the yielding of reinforcements and failure of the wall (attainment of the ultimate strain in the concrete) are delayed, resulting in an increment of the lateral load capacity as well as in an increment of the global ductility of the HCSW systems. However, these benefits are obtained at the expenses of an often excessive congestion of reinforcements. It is also observed that a reduction of the wall aspect ratio allows a higher lateral load capacity as well as a higher ductility. Given that a reduction of the aspect ratio means smaller steel links and longer walls that allow more space for reinforcements, the design should be based in practice on a limitation of the wall aspect ratio, possibly limited to suggested values $H/l_w \leq 10$.

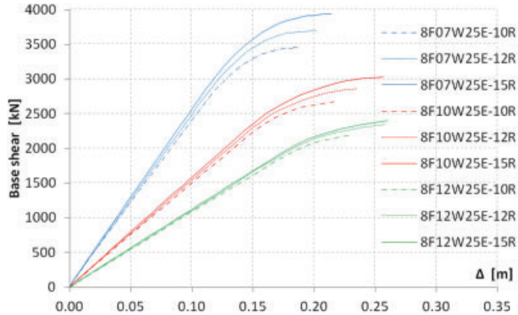

Figure 8. Pushover curves – global comparison.

5.2 *ULS and DLS verifications*

Outcomes in terms of pushover curves presented in section 5.1 can be analyzed with respect to different ULS and DLS criteria. This allows in particular the calculation of specific performance points associated the activation with different limit states and hence, based on the N2 method proposed in Annex B of Eurocode 8, the calculation of the acceleration level corresponding to these performance points. In the present contribution, only preliminary assessments are given. The outcomes will be more deeply investigated in the next publications on the INNO-HYCO project. Three different limit states are taken into consideration:

- *ULS1: Maximum compressive strain reached in the concrete shear wall*. No confinement is conservatively taken into account. Accounting for the confining effect by imposing rules similar to those prescribed by EC8 for reinforced concrete walls could certainly improve the global behaviour of the system.
- *ULS2: Maximum rotation of the steel links*. In this preliminary assessment, indicative conservative values of the maximum possible rotation capacity are estimated according to the criteria proposed by FEMA 356 for seismic links in eccentrically braced structures. The values must be adjusted in a next stage on the base of experimental test results to be carried out in the context of the INNO-HYCO project.
- *DLS: Maximum allowable inter-storey drift*. Associated values of the acceleration should be referred to acceleration level for DLS, smaller than the reference design value of 0.25 g.

Table 2. Maximum sustainable accelerations from pushover curves and ULS/DLS criteria [in g].

ID step1	ULS1	ULS2	DLS
4F07W25E – 10R	0.22	0.19	0.11
4F07W25E – 12R	0.23	0.18	0.11
4F07W25E – 15R	0.26	0.14	0.07
4F10W25E – 10R	0.18	0.18	0.11
4F10W25E – 12R	0.18	0.18	0.09
4F10W25E – 15R	0.15	0.14	0.06
4F12W25E – 10R	0.14	>0.14	0.08
4F12W25E – 12R	0.14	>0.14	0.08
4F12W25E – 15R	0.13	>0.13	0.06
8F07W25E – 10R	0.27	0.24	0.14
8F07W25E – 12R	0.3	0.25	0.15
8F07W25E – 15R	0.35	0.29	0.18
8F10W25E – 10R	0.24	0.24	0.11
8F10W25E – 12R	0.26	0.25	0.12
8F10W25E – 15R	0.29	0.26	0.12
8F12W25E – 10R	0.21	>0.24	0.10
8F12W25E – 12R	0.24	0.24	0.10
8F12W25E – 15R	0.25	0.25	0.10

Table 3. Behaviour factors at $a_g = 0.25$ g.

ID step1	10R	12R	15R
4F07W25E	2.77	2.59	2.30
4F10W25E	2.81	2.94	2.55
4F12W25E	1.86	2.15	1.97
8F07W25E	1.30	1.28	1.41
8F10W25E	1.22	1.23	1.24
8F12W25E	1.17	1.21	1.19

Results are provided in Table 2 in terms of maximum acceleration for the 4-storey and 8-storey cases. Additionally, Table 3 gives the values of the behaviour factor estimated from an equivalent bilinear curve on the base of the ductility.

6 CONCLUDING REMARKS

A selection of results involving an innovative steel-concrete hybrid coupled shear wall systems developed under the European research project INNO-HYCO was briefly illustrated. The analysis of the case studies designed according to the adoption of existing rules in the Eurocodes has highlighted the potentialities of the proposed innovative HCSW systems, namely: it is actually possible to develop a ductile behaviour where plastic deformation are attained in the steel links and limited damage occurs in the reinforced concrete wall; the interstorey drifts up to collapse are quite regular regardless of the non-simultaneous activation of the plastic hinges in the steel links and/or in the reinforced concrete wall; the adopted design approach based on well-known concepts and procedures already available in the Eurocodes give a promising starting design solution, although it appears that using the behaviour factor proposed by Eurocode 8 for composite walls overestimates the deformation capacity of the HCSW, in particular for short walls. On the other hand, the following issues have been encountered: the designed solutions require additional studies to clarify the relationships between wall and links in order to provide additional design recommendations as integration of the Eurocodes; the slenderness of the HCSW systems needs to be better controlled in order to limit the negative effects of geometric nonlinear effects and improve the behaviour at the damage limit states. However, the sensitivity of the response to rotation capacity of the links and to confinement of the concrete needs also to be deeper studied, including in the context on time-history analysis. The upcoming developments of this research work involve the definition of a design procedure compatible with the current Eurocode 8 recommendations and in-depth experimental studies on the connections between reinforced concrete wall and replaceable dissipative steel links.

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