Behaviour and design of innovative hybrid coupled shear walls for steel buildings in seismic areas

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SUMMARY:
In this work innovative hybrid coupled shear wall (HCSW) systems are considered, their preliminary design is discussed, their efficiency and limitations evaluated by means of nonlinear static (pushover) analysis as well as nonlinear dynamic (time-history) analysis. Different numbers of storeys and different geometries of the HCSW system are examined in order to give an overview of situations of interest in European seismic prone areas. The results presented in this paper show that promising solutions can be obtained and that some aspects require further research work. This study is part of a larger research project (INNO-HYCO - INNOvative HYbrid and COmposite steel-concrete structural solutions for building in seismic area) funded by the European Commission and currently under development.

Keywords: hybrid coupled shear walls, dissipative links, steel-concrete hybrid structures, steel frames.

1. INTRODUCTION

Steel and concrete hybrid structures have limited applications for building construction in seismic areas as for example compared to steel and concrete composite structures. However, steel and concrete hybrid structures might have very promising uses in seismic areas provided that suitable structural schemes and proper design method are identified and investigated. While the deformation demands for the steel and concrete components in composite structures are in the same range since concrete and steel are part of the same structural member, hybrid structures allow a more efficient design of the reinforced concrete structural elements as well as of the steel structural elements. In fact the deformation demands may be tailored to the capacity of the relevant materials. Thus, it becomes important to define innovative steel and reinforced concrete hybrid systems for the construction of feasible and easy repairable earthquake-proof buildings through the full exploitation of the properties of both steel and concrete constructions.

Examples of hybrid structural systems are: (i) shear walls with steel coupling beams; (ii) systems made of traditional reinforced concrete shear walls coupled with moment resisting steel frames; (iii) composite walls made of steel frames with infill reinforced concrete panels. Such systems are considered in Eurocode 8 but only limited information is given for their design, e.g., connection between steel components and reinforced concrete components. Hybrid systems may suffer from some drawbacks distinctive of reinforced concrete shear walls and of moment-resisting steel frames. Shear walls are low redundant structures, their post-yielding behaviour is characterised by deformations localized at the base and expensive detailing is required to avoid concrete crushing. Furthermore, the high overturning moments require expensive foundations. In addition, reinforced concrete shear walls are very difficult to restore because the damage could be extended for more than one inter-storey height. On the other hand, moment-resisting steel components have expensive connections and large yielded zones that make repair complicated.

The research project INNO-HYCO (INNOvative HYbrid and COmposite steel-concrete structural solutions for building in seismic area) funded by the European Commission and currently under development, focus its attention to hybrid structural systems characterised by innovative conceptions and connection systems between steel and concrete components. Attention is here limited to innovative hybrid coupled shear wall (HCSW) systems, where a reinforced concrete wall carries almost all horizontal shear while the overturning moment is mostly resisted by an axial compression - tension couple developed by two side steel columns; the reinforced concrete wall should remain in the elastic or should undergo limited damages and the steel links connected to the wall are the only dissipative elements. Such hybrid systems might represent a cost- and time-effective type of construction since simple beam-to-column connections could be used for the steel frame constituting the gravity-resisting part and traditional and well-known building techniques are required for the reinforced concrete and steel components.

In this paper the problems encountered in the preliminary design of innovative HCSW systems based on the recommendations available in the Eurocodes are discussed using selected case studies. The relevant seismic performances of the designed structures are analyzed in order to assess both potentiality and limitations encountered during the design for the proposed innovative HCSW systems. The presented results were obtained by the involved research units of the University of Camerino and of the University of Liege.

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 characterised by good lateral stiffness and dissipation capacity. Coupling beams must be proportioned to avoid over coupling, i.e., a system that acts as a single pierced wall, and under coupling, i.e., a system that performs as a number of isolated walls. Extensive past research (Paulay and Priestley 1992) has led to well established seismic design guidelines for reinforced concrete coupling beams, typically deep beams with diagonal reinforcements, in order to satisfy the stiffness, strength, and energy dissipation demands. The diagonal reinforcement consists of relatively large diameter bars which must be adequately anchored and confined to avoid buckling at advanced limit states. Structural steel coupling beams (Fig. 1a) or steel-concrete composite coupling beams provide a viable alternative (El-Tawil et al. 2010), particularly for cases with restrictions on floor height. In contrast to conventionally reinforced concrete members, steel/composite coupling beams can be designed as flexural-yielding or shear-yielding members. 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 latter case the connection is achieved by embedding the coupling beam inside the wall piers and interfacing it with the wall boundary element. 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. However, coupled shear wall systems suffer from being difficult to be repaired after strong earthquakes. Design recommendations following the criteria of Performance or Displacement Based Design (PBD and DBD) and Force Based Design (FBD) are still missing or at their early stage of development (El-Tawil et al. 2010).

2.2. Innovative hybrid systems

An example of innovative hybrid system is the reinforced concrete shear wall with steel links depicted in Fig. 1b. The reinforced concrete wall carries almost all the horizontal shear force while the overturning moments are partially resisted by an axial compression-tension couple developed by the two side steel columns rather than by the individual flexural action of the wall alone. The reinforced concrete wall should remain in the elastic field (or should undergo limited damages) and the steel links connected to the wall should be the only (or main) dissipative elements. 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. Even if a capacity design is required, columns are expected to have a relatively small cross section. The negative effects of the reinforced concrete wall on the foundations would also be reduced.

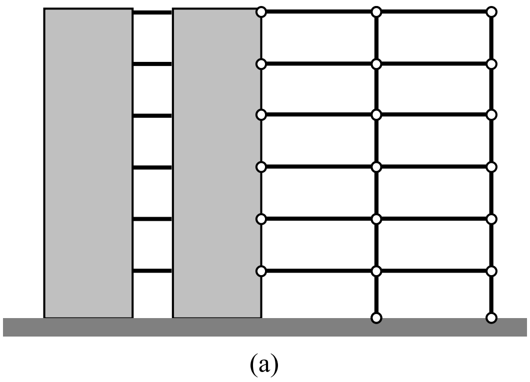


Figure 1. Examples of: (a) conventional hybrid coupled shear wall system, (b) innovative hybrid coupled shear wall system, both connected to a gravity-resisting steel frame with pinned beam-to-column joints.

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. As an alternative, links could include a replaceable fuse (El-Tawil et al. 2010) acting as a weak link where the inelastic deformations are concentrated while the remaining components of the system have to remain elastic. Clearly, the proposed hybrid system is effective as seismic resistant component if the yielding of a large number of links is obtained.

3. CASE STUDIES

3.1. Description and design data

Two case studies are considered: 4-storey and 8-storey steel frames with the same floor geometry as shown in Fig. 2. For each floor the vertical loads are G_k = 6.5 kN/m^2 and Q_k = 3.0 kN/m^2, while the floor total seismic mass is 1200 kNs^2/m. Interstorey height is h = 3.40 m.

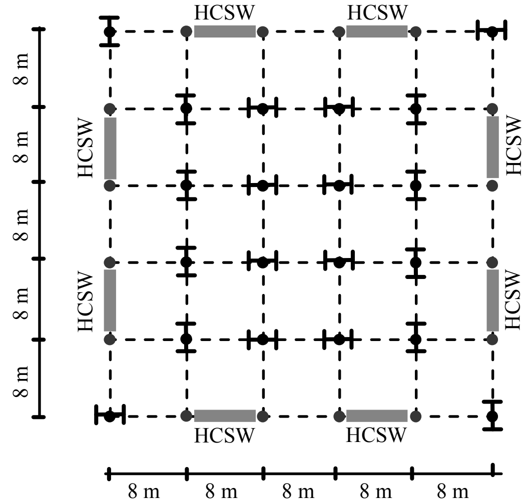


Figure 2. Floor geometry of the benchmark structures with positions of the HCSW systems.

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 and beams.

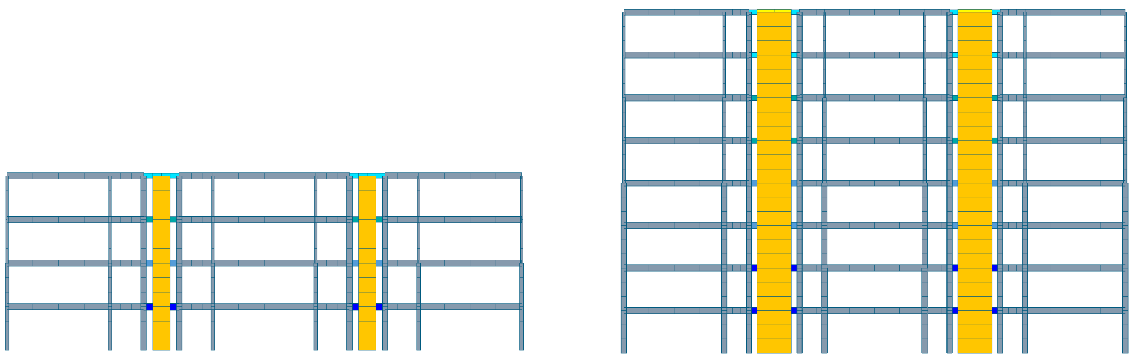
Table 1. Wide-flange cross sections of the columns of the vertical-resisting structure.

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

The assumed seismic design action is represented by the Eurocode 8 type 1 spectrum with a_g = 0.25g and ground type C. Both verifications of the ultimate limit state (ULS) and of the damage limit state (DLS), assuming 0.005h as allowed interstorey drift, 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, (ii) stiffness verification at DLS under the lateral forces derived from the elastic spectrum with ordinates divided by 2.5, i.e., equivalent to assuming ν = 0.4 in formula 4.31 in paragraph 4.4.3.2 of Eurocode 8. Horizontal static design loads were computed according to Eurocode 8, using the behaviour factor q = 3.3 corresponding to composite structural system type 3 according to the definition given in 7.3.1.e of Eurocode 8. Various HCSWs with different dimensions of the reinforced concrete shear walls and link lengths were evaluated. Concrete C25/30 with steel reinforcements S500 was used for the shear walls. Plane views of the most promising geometries obtained are shown in Fig. 3 and hereafter described.



(a) (b)

Figure 3. Innovative HCSW system with additional columns: (a) 4-storey case, (b) 8-storey case.

The HCSW in the 4-storey case has 1.36 m × 0.30 m reinforced concrete shear walls (length 1/10th of the total height of the building) with 13 layers of 2ϕ28, additional steel side columns (HE450B) to reduce the link length to 0.70 m (being the bay span assigned and equal to 8.00 m), links pinned to the side column and clamped to shear wall with cross sections as given in Table 2.

The HCSW in the 8-storey case has 2.72 m × 0.30 m reinforced concrete shear walls (again length 1/10th of the total height of the building) with reinforcements in 6 layers of 2ϕ36 for each of the two free-edge wall ends and 10 layers of 2ϕ10 in the central region (in accordance with Eurocode 8), additional steel side columns (wide flange double-T cross section with depth 450 mm, flanges 450 mm

$\times$ 40 mm, web thickness 21 mm) to reduce the link length to 0.66 m, links pinned to the side column and clamped to shear wall with cross sections as given in Table 2.

Table 2. Links in the optimal preliminary force-based designs.

Storey #	4-storey case	8-storey case
8	-	IPE 400
7	-	IPE 400
6	-	IPE 450
5	-	IPE 450
4	IPE 400	IPE 550
3	IPE 450	IPE 550
2	IPE 500	IPE 550
1	IPE 550	IPE 550

In both cases links are made of steel S355 and are intermediate links according to the Eurocode 8 definition in 6.8.2(9). The additional side columns were designed in order to meet the capacity design requirements of the columns in eccentrically braced frames, i.e., their amplified design axial force is given by formula 6.30 in paragraph 6.8.3 of Eurocode 8.

3.3. Finite element model

The seismic behaviour of the designed HCSW systems was assessed through multi-record nonlinear incremental dynamic analysis (IDA) using the software SAP2000 Advanced 14.2.4. Preliminary nonlinear static analysis under applied lateral loads (pushover analysis) were also run. A plane model of a single HCSW connected to a continuous steel column equivalent to the pertinent part of the gravity-resisting steel frame was used in the analyses, including loads and masses of the HCSW system as well as loads and masses from the pertinent part of the gravity-resisting steel frame. Material nonlinearity was included in the model using nonlinear hinges. The reinforced concrete shear walls were modelled using linear elastic frame elements (axial, flexural and shear deformability) with flexural elastic stiffness as obtained from the initial slope of the nonlinear moment curvature relation of their cross section with concrete confinement included according to Eurocode 2. The nonlinear moment curvature relations were also used to define the relevant idealized moment rotation curves of the nonlinear hinge located at the base of the wall. Mean values of the material properties were used for the concrete and reinforcements. The steel shear links were modelled using linear elastic frame elements (axial, flexural and shear deformability) with nonlinear flexural hinges introduced in each link at the end clamped to the shear wall (point of maximum bending moment) as well as shear hinges introduced at mid span of each steel link (shear force constant along the link). In the adopted software there is no available option of plastic hinges with interaction between bending moment and shear. Thus, the analyses were run with the flexural hinges and shear hinges independent from each other. Afterward, results were post-processed in order to evaluate if the cumulate plastic rotation given by the flexural hinge and by the shear hinge in each link exceeds its ultimate plastic rotation. A preliminary evaluation of the influence of geometric nonlinearity was made. It is found that geometric nonlinear analysis results in non negligible reductions of the load bearing capacity. This observation is in agreement with the fact that the interstorey drift sensitivity coefficient θ (Eurocode 8 Part 1, paragraph 4.4.2.2, equation 4.28) assumes always values larger than 0.1 while still acceptable being lower than 0.3. Consequently, geometric nonlinearity was included in the static and dynamic analyses.

3.4. Results of nonlinear static analysis

The obtained pushover curves (shear base versus top displacement) are shown in Fig. 4 while the evolution of the plastic rotations in the nonlinear hinges are shown in Fig. 5. 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.

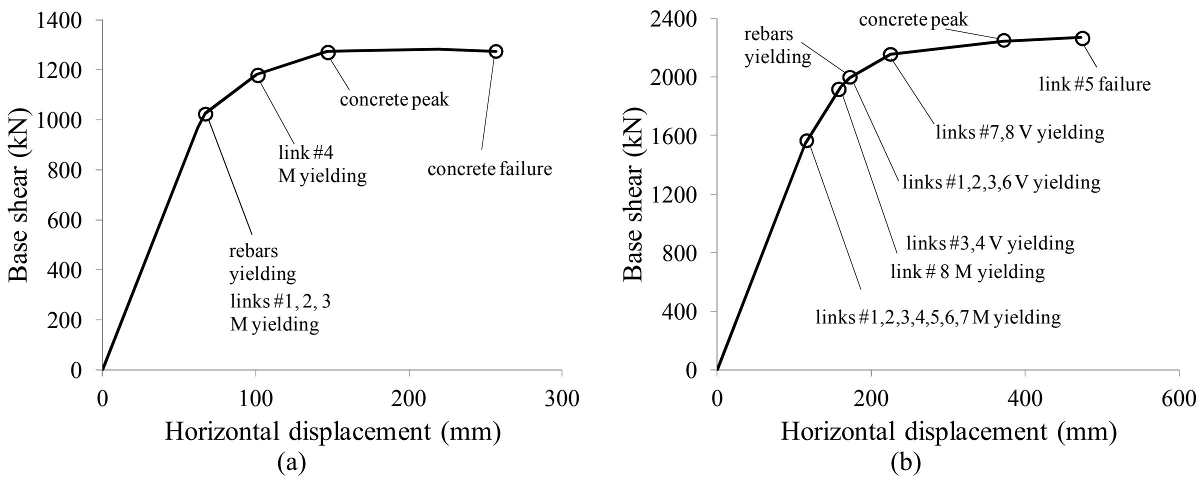


Figure 4. Pushover curves for: (a) 4-storey case, (b) 8-storey case.

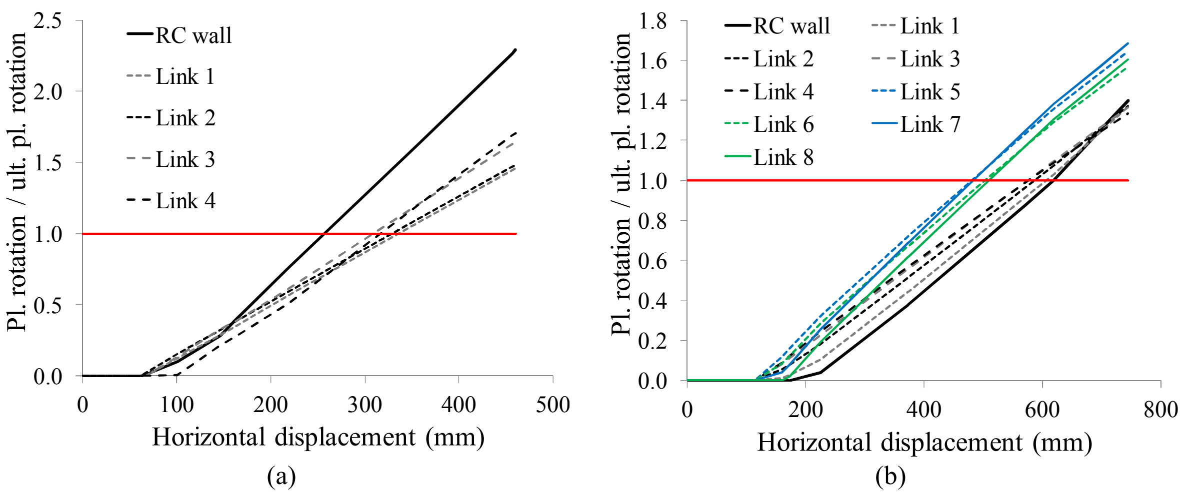


Figure 4. Plastic rotations for: (a) 4-storey case, (b) 8-storey case.

3.5. Results of nonlinear dynamic analysis

Seven artificial accelerograms were used as seismic input, scaled from the scale factor (SF) 0.3 up to 1.0 with steps of 0.1 amplitude, then using steps of 0.2 amplitude. Results were averaged over the seven accelerograms using the running mean with zero-length window (Vamvatsikos and Cornell 2002), i.e., calculating values of the engineering demand parameters (EDPs) at each level of the intensity measure (IM) and then finding the average and standard deviation of EDPs given the IM level. In Table 3 the number of failed models for each IM is given for SF up to 1.6. It is observed that the 4-storey HCSW system experiences failures for a number of accelerograms before the design seismic level at the ultimate limit state is attained, i.e., one failure for SF = 0.6, two failure for SF = 0.8, four failures for SF = 0.9, and four failures for SF = 1.0. The 8-storey HCSW system shows a better overall behaviour with failures reached for the first time at SF = 1.2.

Table 3. Overview of the observed failures for the assigned scale factors.

	Observed failures	
SF	4-storey	8-storey
0.3	0	0
0.4 (DLS)	**0**	**0**
0.5	0	0
0.6	1	0
0.7	0	0
0.8	2	0
0.9	4	0
1.0 (ULS)	**4**	**0**
1.2	3	3
1.4	3	7
1.6	5	7

Table 4. Interstorey drifts in the 4-storey HCSW system from IDA (average values).

	Storey			
SF	#1	#2	#3	#4
0.3	0.28%	0.41%	0.44%	0.45%
0.4	0.37%	0.52%	0.57%	0.56%
0.5	0.46%	0.63%	0.66%	0.65%
0.6	0.87%	0.91%	1.02%	0.99%
0.7	0.59%	0.82%	0.88%	0.81%
0.8	1.72%	1.43%	1.61%	1.73%
0.9	2.35%	1.20%	1.13%	1.26%
1.0	2.36%	1.62%	1.54%	1.50%
1.2	2.36%	1.11%	1.24%	1.28%
1.4	2.11%	1.54%	1.93%	1.94%
1.6	2.52%	1.58%	1.91%	2.25%

Table 5. Interstorey drifts in the 8-storey HCSW system from IDA (average values).

	Storey							
SF	#1	#2	#3	#4	#5	#6	#7	#8
0.3	0.13%	0.24%	0.28%	0.30%	0.32%	0.34%	0.37%	0.39%
0.4	0.17%	0.31%	0.38%	0.39%	0.40%	0.46%	0.49%	0.51%
0.5	0.20%	0.37%	0.46%	0.47%	0.48%	0.52%	0.55%	0.59%
0.6	0.21%	0.42%	0.52%	0.55%	0.59%	0.61%	0.63%	0.67%
0.7	0.22%	0.44%	0.56%	0.60%	0.66%	0.67%	0.68%	0.71%
0.8	0.24%	0.47%	0.59%	0.65%	0.71%	0.72%	0.72%	0.76%
0.9	0.27%	0.50%	0.64%	0.69%	0.75%	0.76%	0.76%	0.83%
1.0	0.30%	0.55%	0.69%	0.75%	0.78%	0.81%	0.82%	0.89%
1.2	0.41%	0.65%	0.77%	0.86%	0.89%	0.89%	0.92%	0.97%
1.4	1.46%	0.92%	1.06%	1.12%	1.11%	1.06%	1.03%	1.10%
1.6	1.53%	1.06%	1.19%	1.29%	1.35%	1.38%	1.38%	1.42%

The interstorey drifts as derived from the IDA are given in Table 4 and in Table 5 for the 4-storey and 8-storey cases, respectively. The table rows in grey are relevant to the averaged values unconditional to survivals, i.e., the mean values are computed considering also the interstorey drifts obtained for the accelerograms that cause plastic rotations in the hinges that actually exceed their capacity. It is observed that the targeted limitation at the damage limit state (DLS SF = 0.4, interstorey limit 0.50%) is not satisfied (by a small over quota) in the upper three floors of the 4-storey case and in the last floor of the 8-storey case. It is also observed that the distribution of the interstorey drift along the height is quite regular but for the first one (4-storey case) and first two (8-storey case) storeys influenced by the base restraint of the reinforced concrete wall.

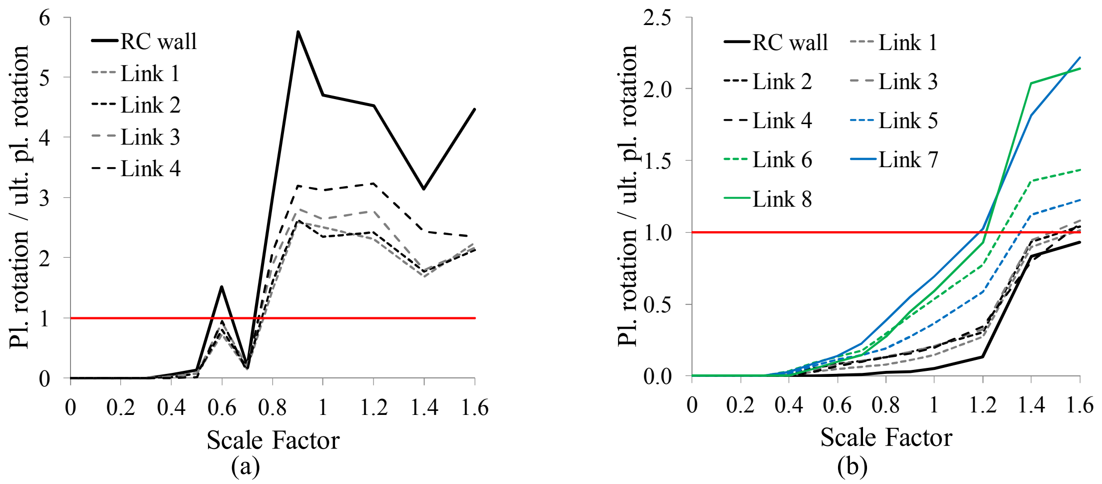


Figure 4. Evolution of plastic rotations (IDA average values) for: (a) the 4-storey case, (b) 8-storey case.

The analysis of the evolution of the plastic rotations (Fig. 5) reveals that the wall in the 4-storey case undergoes significant plastic rotations and the rotation at peak in the moment rotation curve is soon attained, leading to the premature collapse of the bracing systems, before the design level (SF = 1) is reached. On the other hand, the wall in the 8-storey case has a much better behaviour with limited damage while the plastic deformation are basically limited in the steel links of the last storeys. In this way collapse is attained for SF larger than the design level.

4. COMMENTS ON PROSPECTIVE DEVELOPMENTS

4.1. Proposal of a design procedure for the innovative HCSW system

The assessment through nonlinear static and dynamic analyses performed after the preliminary design of the proposed innovative HCSW system shown that the adoption of existing rules in the Eurocodes is a promising starting point. Based on the assessment results, the following design procedure was defined within a force-based approach as contemplated in the Eurocodes and is currently under assessment.

1. Design of the dimensions of the reinforced concrete wall assuming an aspect ratio (height-to-length ratio) with a possible suggested value of about 10.
2. Preliminary linear elastic analysis (static approach based on equivalent horizontal forces or dynamic approach based on design spectrum analysis) of an assumed first trial linear model. A suitable structural factor at this stage is kept q = 3.3 as adopted in the preliminary design. The influence of geometric nonlinear effects is controlled with the suggested amplification of seismic loads as a function of the interstorey drift sensitivity coefficient (Eurocode 8 Part 1, paragraph 4.4.2.2, equation 4.28).
3. Design the dimensions of the links by means of a trial and error design based on the bending moment and shear distributions obtained from preliminary linear elastic analysis. Steel links are dimensioned as intermediate links.
4. The over-strength regularity is verified by the condition $\Omega_{max} / \Omega_{min} < 1.25$, with the over-strength coefficient Ω defined in Eurocode 8 paragraph 6.8.3 for the high ductility design of eccentrically braced steel frames.
5. The design of steel columns of the HCSW system is based on capacity design, i.e., using the axial force amplification based on the coefficient Ω as in column design of high ductility braced frames:

$$N_{Ed} = N_{Ed,G} + 1.1\gamma_{ov}\Omega N_{Ed,E} \quad (1)$$

where γ_{ov} is the over-strength coefficient defined in Eurocode 8.
6. Finally the reinforced concrete wall is detailed according to the prescriptions for high ductility walls (Eurocode 8 paragraph 5.5.3.4) and an over-strength coefficient could be introduced to limit wall damage.

The advantage of the proposed design approach is its relatively ease of use by any structural engineer familiar with the concepts and procedures of the Eurocodes. For the same reason, this design approach could be straightforwardly implemented as integration in Eurocode 8, once it adequately validated.

4.2. Link connection typologies and proposal for their design

Connections between the reinforced concrete wall and the replaceable dissipative steel links are a critical aspect that requires specific attentions in their design. Two typologies were selected for the embedment of the steel link in the reinforced concrete wall: in one case (typology 1) the bending moment transferred by the link to the wall is resisted by shear studs; in the other case (typology 2) the moment transferred by the link to the wall is balanced by a couple of vertical forces, as depicted in Fig. 5. Two solutions are considered for the link splice: the splice connection is placed at a distance from the concrete wall that is sufficient to allow an easy bolting of the replaceable part (solution associated to typology 1); the splice connection is placed at the face of the wall and threaded bushings to allow replacement of the dissipative tract of the steel link (solution associated to typology 2), as depicted in Fig. 6.

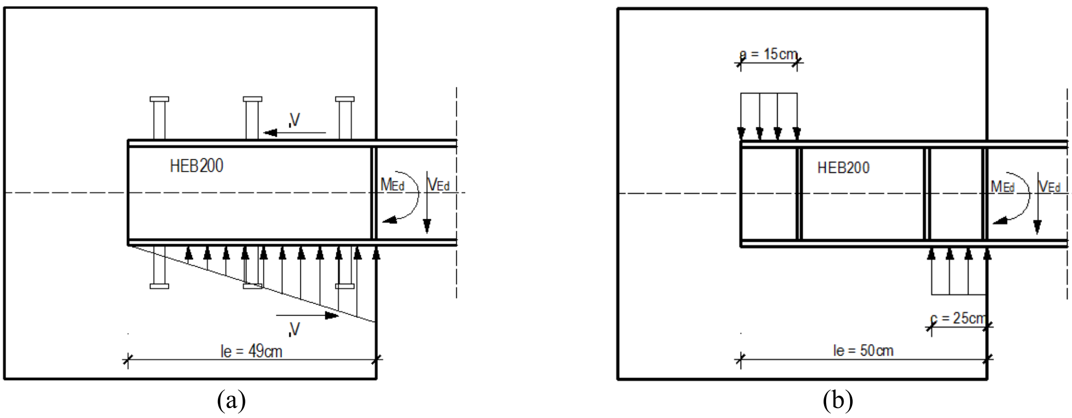
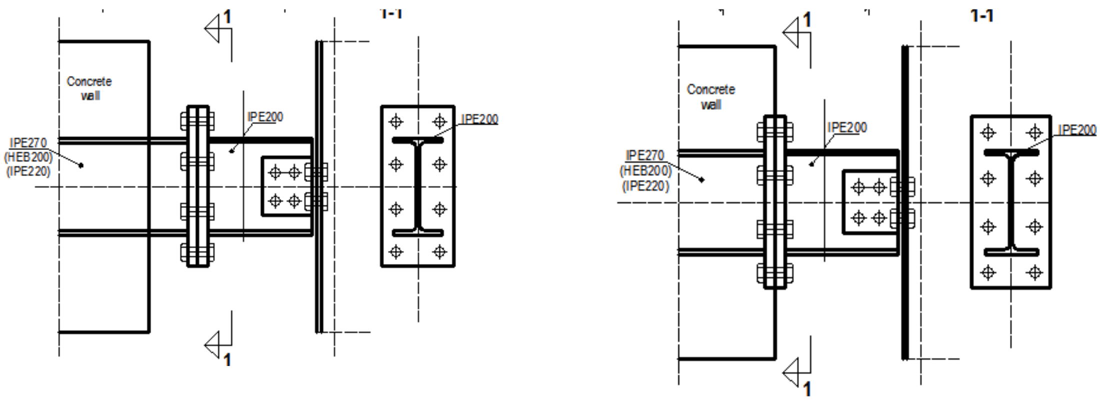


Figure 5. Connection typology 1 (a) and typology 2 (b).

The design assumptions of typology 1 force the creation of a plastic hinge in the replaceable part of the link and give adequate over-strength (capacity design) to 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 objective of the adopted design is to get the replaceable part of the link acting as a fuse, while the embedded part of the link and the concrete are designed to remain undamaged after the formation of the plastic hinge in the fuse. The design assumptions of typology 2 are similar to those in typology 1, i.e., formation of the plastic hinge in the replaceable part of the link while all other components remain undamaged. 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. This value is then increased by the over-strength factors. The presented design approach for the connection systems will be integrated in the design procedure proposed for the HCSW systems as described in the previous paragraph and its assessment based on the results of the programmed experimental tests.



(a) (b)

Figure 6. Link splice typology 1 (a) and typology 2 (b).

5. CONCLUSIONS

In this paper the research work on 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. On the other hand, the following issues have been encountered: the designed solutions, although a promising starting point, require additional studies to clarify the relationships between wall over-strength 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. 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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