COMPDYN 2013
4th ECCOMAS Thematic Conference on
Computational Methods in Structural Dynamics and Earthquake Engineering
M. Papadrakakis, V. Papadopoulos, V. Plevris (eds.)
Kos Island, Greece, 12–14 June 2013

DESIGN AND PERFORMANCE OF STEEL – CONCRETE HYBRID COUPLED SHEAR WALLS IN SEISMIC CONDITIONS

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Keywords: Hybrid coupled shear walls, dissipative links, innovative steel-concrete hybrid structures, steel frames, and seismic analysis.

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. Experimental tests have been designed to study the performance of the connection of a seismic link embedded in a concrete shear wall. 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 and concrete composite structures are common for buildings in seismic areas, conversely steel and concrete hybrid solutions are less diffused and many open problems and potentially interesting structural solutions require further investigation. In composite structures the deformation demands for the steel and concrete components are in the same range since concrete and steel are part of the same structural member. On the other hand, 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 possible 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.

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.

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 aims at developing hybrid systems characterised by innovative conceptions and connection systems between steel and concrete components. Two main typologies are considered: (i) hybrid coupled shear wall (HCSW) systems, and (ii) steel frame with reinforced concrete infill walls (SRCW). In this paper only the HCSW systems are considered and selected results obtained by the research units of the University of Camerino (UNICAM) and of the University of Liège (ULG) are illustrated.

2 COUPLED SHEAR WALLS 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 (e.g. 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 (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. 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 (ASCE 2009).

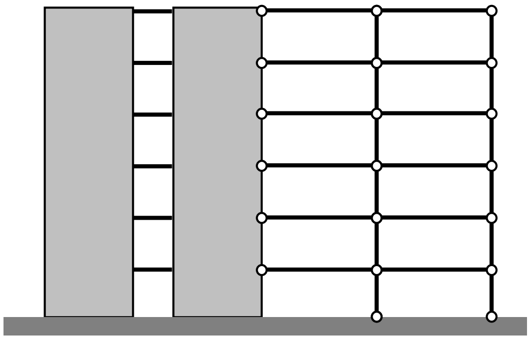

Figure 1. Example of conventional hybrid coupled shear wall system connected to a gravity-resisting steel frame with pinned beam-to-column joints.

2.2 Innovative hybrid systems

An example of innovative hybrid system is the reinforced concrete shear wall with steel links depicted in Figure 2. 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. Such a mechanism allows the reduction of the negative effects of the reinforced concrete wall on the foundations.

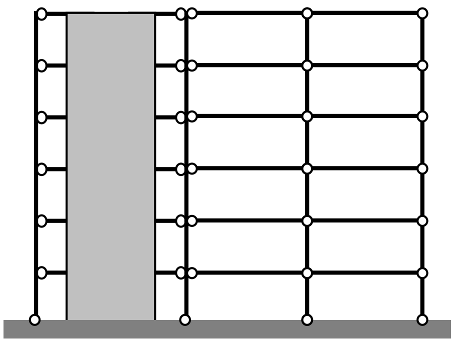

Figure 2. Example of innovative hybrid coupled shear wall system connected to a gravity-resisting steel frame with pinned beam-to-column joints.

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.

In this paper the problems encountered in a preliminary design of the above described innovative hybrid system based on the recommendations available in the Eurocodes are discussed using selected case studies. The relevant seismic performances of the designed structures are analysed in order to assess both potentiality and limitations encountered during the design for the proposed innovative HCSW systems.

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 kN/m^2 and Q_k = 3.0 kN/m^2, while the floor total seismic mass is 1200 tons. 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.

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

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

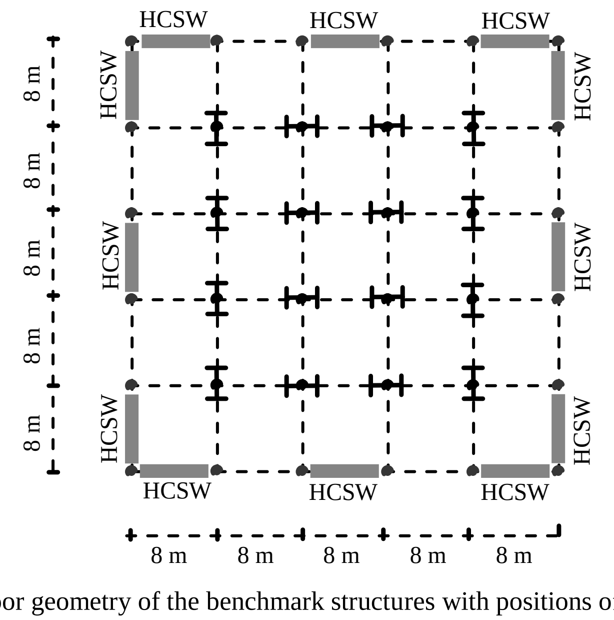

Figure 3. Floor geometry of the benchmark structures with positions of the HCSWs.

The assumed seismic 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) 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, using the behaviour factor q = 3.3 given for composite structural system type 3 according to the definition in 7.3.1.e; (ii) stiffness verification at DLS under the lateral forces assuming $\nu = 1.0$ in formula 4.31 in paragraph 4.4.3.2 of Eurocode 8.

A large number of HCSWs with different dimensions of the reinforced concrete shear walls and link lengths were evaluated in this preliminary design stage by the ULG unit, as detailed in Deliverable 2.1 Part 1 "Description of considered cases, comparison of results and conclusions about the effectiveness in withstanding seismic actions" produced by ULG and included as annex in the INNO-HYCO mid-term report (Dall'Asta et al. 2012). The seismic behaviour of the most promising designs was assessed through nonlinear static (pushover) analysis and multi-record incremental nonlinear dynamic analysis as detailed in Deliverable 2.1 Part 2 "Seismic behaviour assessment by means of incremental dynamic analyses" included in the mid-term report (Dall'Asta et al. 2012). These most promising designs and relevant static and dynamic seismic analyses were presented in Zona et al. (2011, 2012), providing the following indications. 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.

ID step1	Wall aspect ratio H/l_w	Wall length l_w (m)
4F10W25E	10.0	1.36
4F07W25E	7.5	1.81
4F12W25E	12.5	1.09
8F10W25E	10.0	2.72
8F07W25E	7.5	3.63
8F12W25E	12.5	2.18

Table 2. Designed cases: wall geometry.

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) as opposed to the four systems adopted in the preliminary design (Zona et al. 2011, 2012). In order to investigate the main geometric and design parameters, 18 cases were designed, comprising three height-to-length ratios of the wall (Table 2) taken with constant thickness (0.36 m) 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 (Tables 3 and 4 for the 4-storey and 8-storey cases, respectively, where d = section depth, b = flanges width, t_f = flange thickness, t_w = web thickness), 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). 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. Nevertheless, these design solutions were retained in the following seismic assessment phase in order to gain further insight into the influence of wall over-strength on the nonlinear behaviour up to failure.

ID step1	link #	d (mm)	b (mm)	t_f (mm)	t_w (mm)
4F07W25E	4	330	124	11.5	7.5
	3	400	170	13.5	8.6
	2	450	180	14.6	9.4
	1	500	160	16.0	10.2
4F10W25E	4	330	124	11.5	7.5
	3	400	170	13.5	8.6
	2	450	180	14.6	9.4
	1	500	174	16.0	10.2
4F12W25E	4	330	120	11.5	7.5
	3	400	165	13.5	8.6
	2	450	175	14.6	9.4
	1	500	180	16.0	10.2

Table 3. Designed cases (4-storey): wide-flange links.

Thus, there are a total of nine designs involving 4-storey frames (three wall geometries times three wall over-strengths) and a total of nine designs involving 8-storey frames (again three wall geometries times three wall over-strengths).

ID step1	link #	d (mm)	b (mm)	t_f (mm)	t_w (mm)
8F07W25E	8	400	124	13.5	8.6

	7	400	140	13.5	8.6
	6	550	170	17.2	11.1
	5	550	175	17.2	11.1
	4	600	180	19.0	12.0
	3	600	190	19.0	12.0
	2	600	190	19.0	12.0
	1	550	185	17.2	11.1
8F10W25E	8	400	130	13.5	8.6
	7	400	160	13.5	8.6
	6	550	180	17.2	11.1
	5	550	180	17.2	11.1
	4	600	185	19.0	12.0
	3	600	200	19.0	12.0
	2	600	210	19.0	12.0
	1	600	185	19.0	12.0
8F12W25E	8	400	140	13.5	8.6
	7	400	176	13.5	8.6
	6	550	186	17.2	11.1
	5	550	190	17.2	11.1
	4	600	190	19.0	12.0
	3	600	210	19.0	12.0
	2	600	220	19.0	12.0
	1	600	210	19.0	12.0

Table 4. Designed cases (8-storey): wide-flange links.

4 DESIGN OF THE DOWN SCALED CONNECTION

4.1 General assumptions

The tests aim at characterizing 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. Regarding 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 length of the links is defined as being intermediate according to Eurocode 8 chapter 6 classifications. The link stiffeners are 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 to guarantee the yielding to take place in the profile section or in the 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.

In Configuration No. 1 (Figure 4), the bending moment transferred by the link to the wall is balanced by a couple of horizontal forces and 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 consist 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 values from the mechanical model presented in Figure 4 are:

M_1 – characteristic maximum bending moment value at the replaceable part of the link;

M_0 – bending moment value at the interface link/concrete;
M_x – the assumed value of the maximum bending moment in the embedded part of the profile.

According to AISC (2010) recommendations, the embedment length values are increased by the concrete cover thickness due to the risk of spalling of the concrete near the wall face. It is assumed that steel link does not behave as having a fixed boundary condition at the face of the wall, and the effective fixed point is taken at one third of the embedment length from the face of the wall.

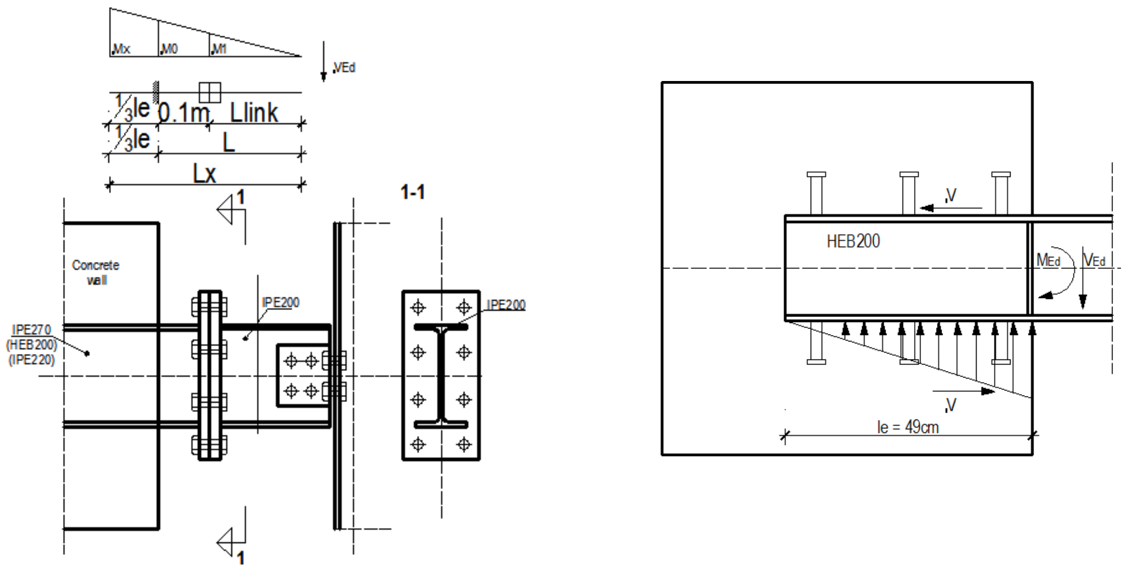

Figure 4. Configuration No. 1- mechanical model.

According to the chosen static scheme and to capacity-design principle, the design is based on the amplified value of the bending moment, with reference to the value at the face.

$$M_{Ed} = 1.1 \cdot \gamma_{ov} \cdot \Omega \cdot M_o \quad (1)$$

$$V_{Ed} = \frac{M_{Ed}}{L_x} \quad (2)$$

The required number of shear studs is governed by the value of the force V, as shown in Figure 4. The embedment length is determined using equilibrium equations of the mechanical model (3) and (4) and shall not be less than 1.5 times the height of the steel profile according to Eurocode 8 recommendations.

$$V_{Ed} = \frac{1}{2} \cdot f_{cd} \cdot l_e \cdot b \quad (3)$$

$$M_{Ed} = V \cdot h - \frac{1}{2} \cdot f_{cd} \cdot l_e \cdot b \cdot \frac{1}{3} \cdot l_e \quad (4)$$

where:
b = 200 mm (HE200B);
h = 200 mm (HE200B);
l_e = 0.49 m – the embedment length of the profile inside the concrete wall.

The number of shear studs is given by (5) and their geometry fulfills the Eurocode 4 Part 1 and 2 specifications.

$$n = \frac{V}{P_{Rd.L}} \quad (5)$$

where:
$P_{Rd.L}$ - the design resistance of a headed stud according to Eurocode 4 Part 2 Annex C;
n – number of shear studs.

For 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 located right at the face of the wall using threaded bushings.

The design objective is similar to the first configuration but the main differences between the two configurations are:

- The positioning of the beam-splice connection of the fuse with respect to 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 can be resisted by a couple of vertical loads, activating the compression resistance of the contact profile-concrete.

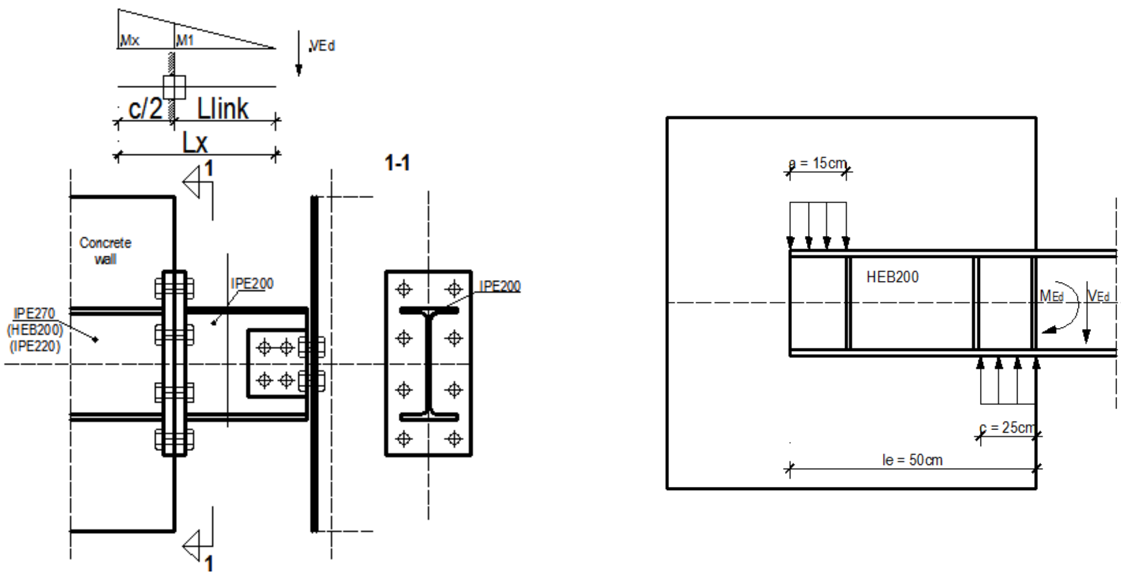

Figure 5. Configuration No.2- mechanical model.

According to the transfer mechanism, the embedded part of the profile is designed assuming conservatively that the bending moment increases linearly until the location of the first reaction force applied by the concrete on the profile. This value is then increased by the overstrength factors similarly to the procedure described in (1) and (2) for Configuration 1. These forces can be largely resisted by an HEB200 profile.

In order to determine the embedded length the following equilibrium equations are used:

$$f_{cd} \cdot a \cdot b + V_{Ed} = f_{cd} \cdot b \cdot c \tag{6}$$

$$M_{Ed} - f_{cd} \cdot a \cdot b \cdot \left(l_e - \frac{a}{2} \right) + f_{cd} \cdot c \cdot b \cdot \frac{c}{2} = 0 \tag{7}$$

where:
b = 200 mm (HEB200);

h = 200 mm (HEB200);
l_e = 0.50 m– the embedment length of the profile inside the concrete wall;
a = 0.15 m;
c = 0.25 m.

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 the failure of the steel bolts, before yielding of the beam flange of the profile.

4.2 Numerical model

In order to assess the actual behaviour prior to the physical testing to come, the numerical model was developed with the computer program SAFIR used for the analysis of structures under ambient or elevated temperature conditions. The program based on Finite Element Method, was developed at University of Liege and can be used to study the behaviour of one, two or three dimensional structures. The numerical model detailed here for Configuration No.1 presented in Figure 6, along with the test setup of the connection, Figure 7.

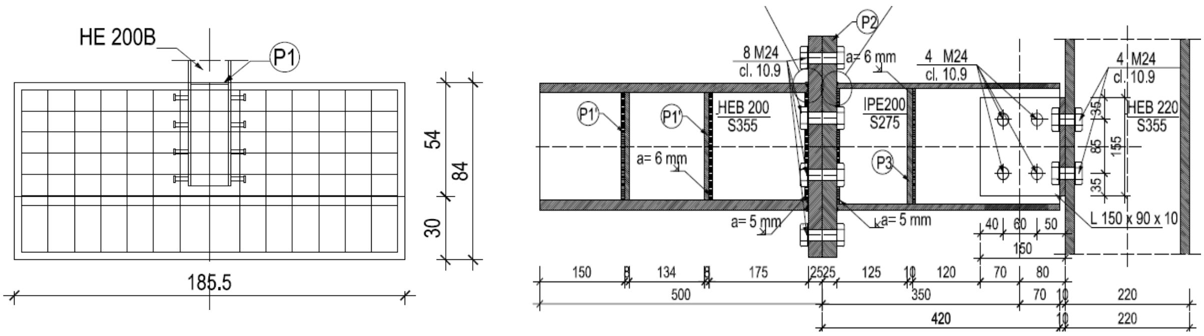

Figure 6. Configuration No.1- design detail drawings.

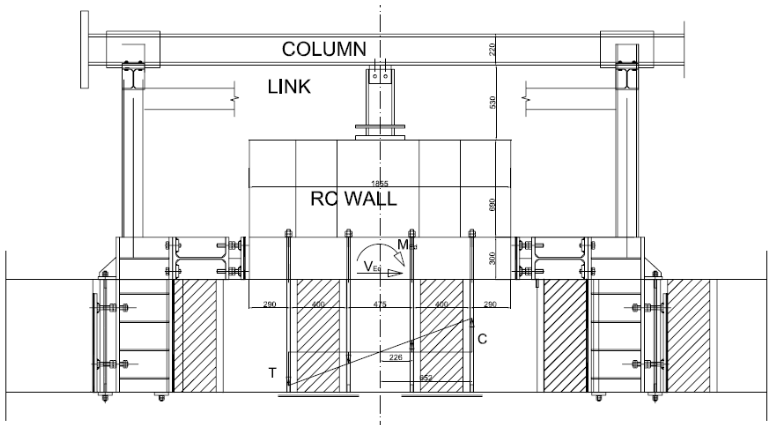

Figure 7. Configuration No.1- test setup.

Concrete wall is defined as a shell element with a constant width all over the wall, reinforcements are defined as truss elements and the steel profile is modelled by means of Bernoulli-type 3D beam elements. Materials properties are the design values defined in Section 4.1. For concrete a parabolic–linear law in compression was used, while the steel part was modeled with a bilinear- elastic-perfectly–plastic law.

Figure 8b) confirms the assumption that the link beam cannot be considered as fixed at the face of the concrete wall and due to that the maximum value of the bending moment occurs inside the concrete wall.

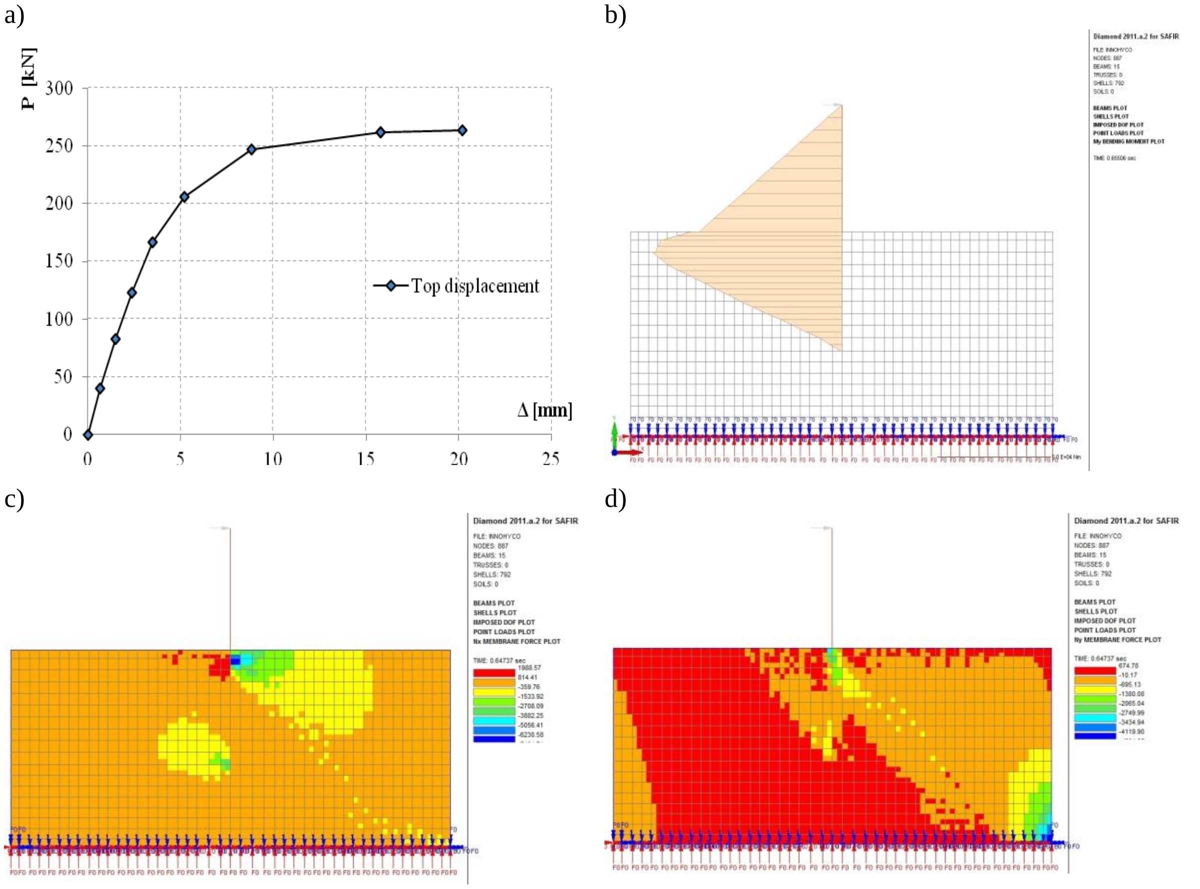

Figure 8. SAFIR-Numerical results: a)Force - top displacement curve; b) M_y – beam link; c) Nx – membrane force plot; d) N_y – membrane force plot.

L_{link}	0.35 m
$M_1 = M_{pl.RdIPE200}$	44.76 kNm
$V_{Ed}=M_1/L_{link}$	127.89 kN
V_{Sdmax} - jack limit	400kN/1.5=266.67 kN
$1.1*\Omega*\gamma_{ov}*V_{Ed}$ - max value	263.76 kN
V_{SAFIR}	258.95kN
$1.1*\Omega*\gamma_{ov}*V_{Ed}/V_{SAFIR}$	98%
$M_x = M_1 \cdot \frac{L_x}{L_{link}}$	60.75 kNm
$1.1*\Omega*\gamma_{ov}*M_x$ -design value	125.30 kNm
M_{SAFIR}	124.6 kNm
$1.1*\Omega*\gamma_{ov}*M_{Ed}/M_{SAFIR}$	100%

Table 5. Comparison between FEM model and design results.

Maximum design values obtained using the simple static scheme shown in Figure 4, are compared with the outcomes of the numerical model and numerical model. It can be observed that the values obtained using SAFIR numerical model are similar to those obtained by hand design. The model will be further calibrated with respect to the last results when available.

5 SEISMIC BEHAVIOUR OF HCSW SYSTEMS

5.1 Modeling assumptions

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. This software is being developed at the University of Liège for more than 40 years and is used for both academic activities and regular design purposes. It accounts for geometric and material non linear effects. 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. An illustration of the model is given at Figure 9 for the 4-storey case. Loads and masses are those of the HCSW system as well as loads and masses from the relevant part of the gravity-resisting steel frame

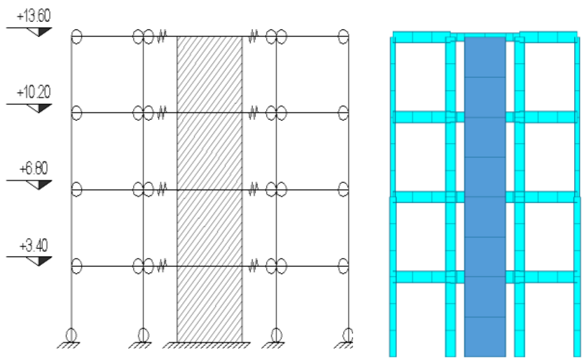

Figure 9. Numerical model.

The reinforced concrete shear wall is represented by beam elements using a fibre description for the behaviour of the concrete in the longitudinal direction, allowing an accurate estimation of the behaviour 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 ε_{c2} = 0.002 (end of the parabolic behaviour) and ε_{cu2} = 0.0035 (end of the rectangular behaviour). No confinement is considered for the concrete in compression.

5.2 Selected results for the 4-storey cases

Pushover curves (total base shear versus top displacement) for the 4-storey (15R) cases are shown in Figures 10 to 16. The end of the curves corresponds to reaching the maximum allowed compressive strain in the concrete ($\varepsilon_{cu,2}$= 0.0035).

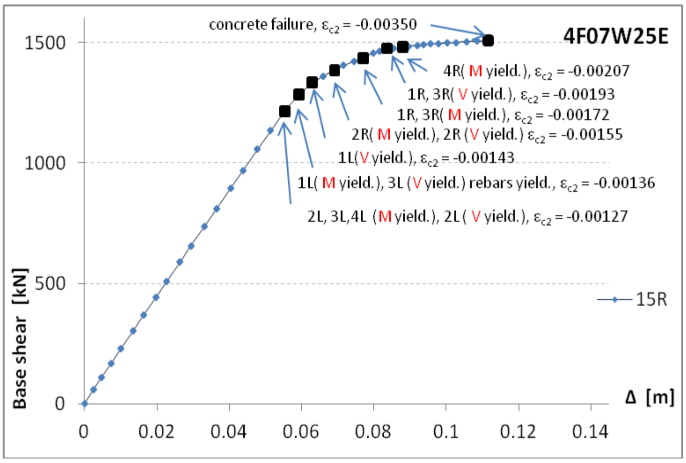

Figure 10. Pushover curve for case 4F07W25E15R and relevant yielding sequence.

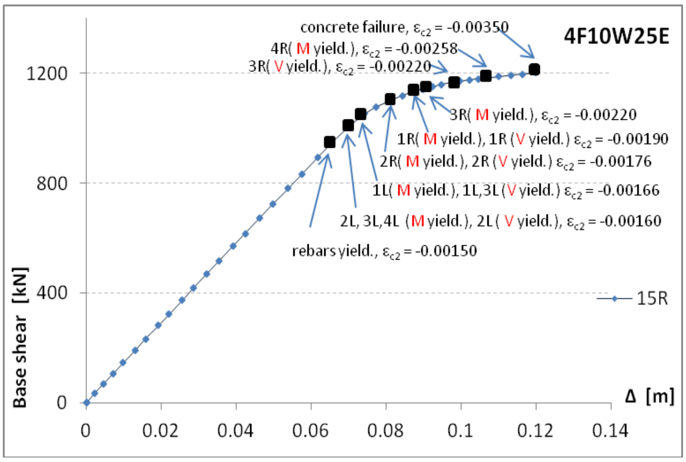

Figure 11. Pushover curve for case 4F10W25E15R and relevant yielding sequence.

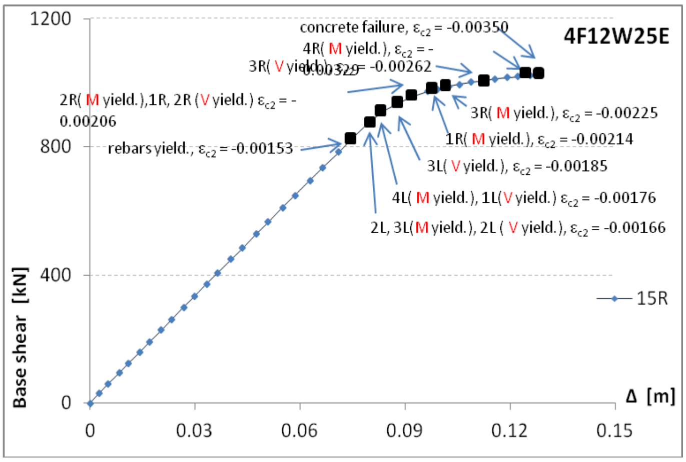

Figure 12. Pushover curve for case 4F12W25E15R and relevant yielding sequence.

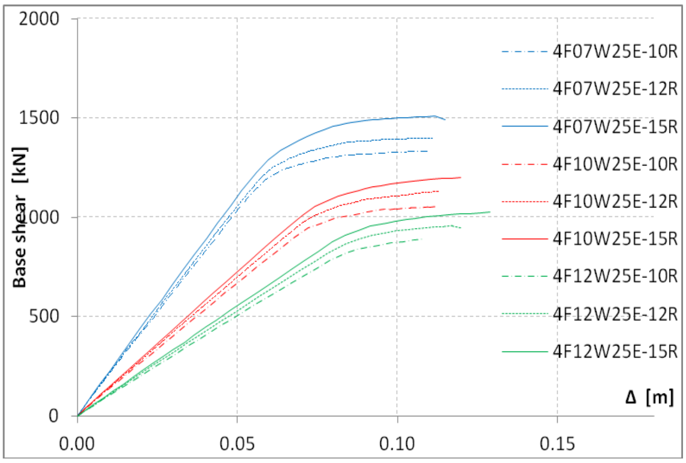

Figure 13. Pushover curve – global comparison

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 $H/l_w \leq 10$.

5.3 Selected results for the 8-storey cases

Pushover curves for the 8-storey (15R) cases are shown in Figures 18 to 29. The observation made for the 4-storey cases can be repeated for these taller structures, where problems for the higher values of the wall aspect ratio H/l_w are even more evident.

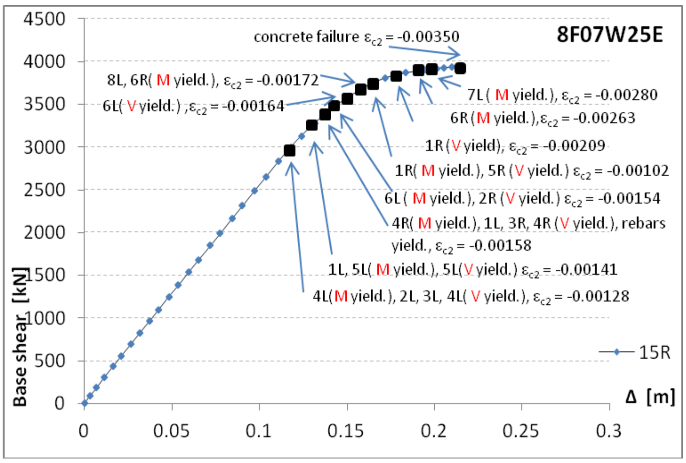

Figure 14. Pushover curve for case 8F07W25E15R and relevant yielding sequence.

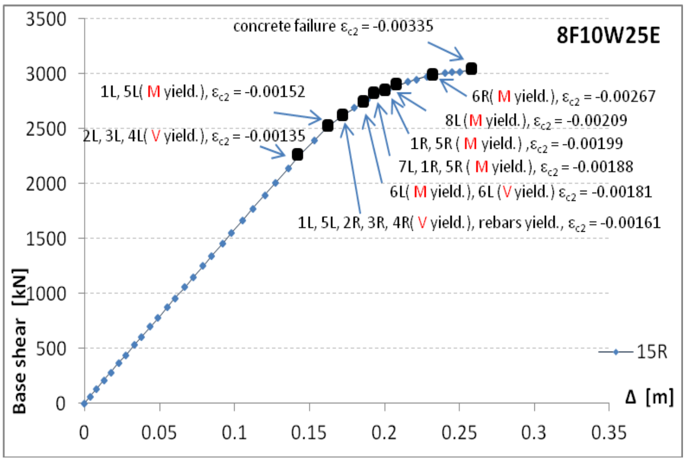

Figure 15 Pushover curve for case 8F10W25E15R and relevant yielding sequence.

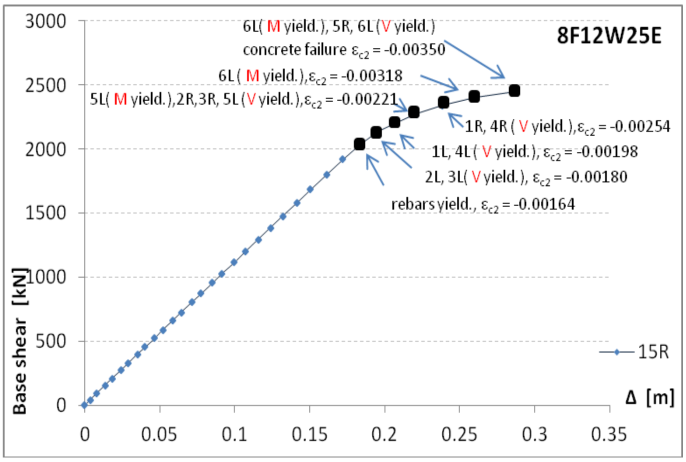

Figure 16. Pushover curve for case 8F12W25E15R and relevant yielding sequence.

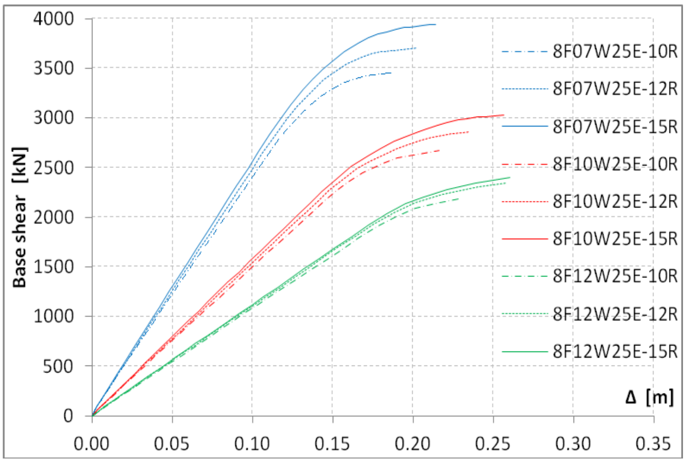

Figure 17. Pushover curves – global comparison.

5.4 ULS and DLS verifications

Outcomes in terms of pushover curves presented in section 4.3 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 0.25g.

Results are provided in table 6 in terms of maximum acceleration for the 4-storey and 8-storey cases. Additionally, table 7 gives the values of the behaviour factor estimated from an equivalent bilinear curve on the base of the ductility.

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 6. Maximum sustainable accelerations from pushover curves and ULS/DLS criteria [in 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
4F12W25E	1.17	1.21	1.19

Table 7. Behaviour factors at ag = 0.25g.

6 CONCLUSIONS

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 gives 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. Capacity design of connections will be confirmed by experimental results. 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.

ACKNOWLEDGEMENT

Support for this research from the European Commission, Research Fund for Coal and Steel, Steel Technical Group TGS 8 "Steel products and applications for building, construction and industry", is gratefully acknowledged

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