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Hybrid Coupled Walls with Replaceable Shear Links: Technical Solution and Seismic Performance of a 3D Multi-Story Building

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Abstract

As recent research findings evidenced a significant potential of the steel-concrete hybrid coupled wall (HCW) systems, further research activities are still needed for addressing particular issues and for developing advancements in the analysis, design, and detailing. These aspects are currently being addressed within the ongoing European research project “HYCAD”, which also represents the research framework of the current study. This paper investigates through a set of nonlinear static and dynamic analyses the behavior and the seismic performance of a newly introduced steel-concrete HCW system. In particular, a case-study was carried out on a 3D multi-story building structure with perimeter lateral load resisting frames, each composed of: (i) a HCW, i.e. a steel-concrete composite wall coupled to steel columns through a set of replaceable steel shear links; (ii) a moment resisting frame (MRF). Beside a brief state of the art, a description of the research framework and the case-study structural system, the current paper makes an overview of the following: (i) experimental program on components; (ii) proposed technical solution for connecting the replaceable dissipative shear links to the composite wall; (iii) experimental test set-up; (iv) response of the shear links based on pre-test FE investigations; (v) seismic demand in terms of rotation for the shear links.

Keywords

Hybrid Coupled Walls; Replaceable Dissipative Devices; Shear Links; Steel-Concrete Composite Structural Members; Seismic Performance; Experimental investigations; FEM / FEA;

1 Introduction

Two important aspects related to the design of building structures in seismic areas imply assuring: (i) an adequate seismic performance; (ii) the possibility of structural repair (i.e. replacement of yielded dissipative members). The focus of many researchers and engineers - in the last decades - has been on the development of innovative solutions fulfilling these conditions. One example is represented by the development of the “*coupled shear wall*” system. Later, a more advanced approach was made with the “*hybrid coupled wall*” system, and recently with the “*single-pier hybrid coupled wall*” system.

The “*coupled shear wall*” system, which is composed of reinforced concrete (RC) shear walls and RC coupling beams (i.e. placed at floor levels), was introduced by [1] [2] for a better exploitation of the stiffness, strength and dissipation capacity of the RC walls. Beside some advantages, the main disadvantage of the system is represented by the special reinforcement detailing required for the coupling beams and for the area in their vicinity. This resulted in construction difficulties, the need for highly skilled workers and elevated costs. Furthermore, the limited shear capacity of the coupling beams often implied the need of deep cross-sections [3].

Within the “*hybrid coupled wall*” system, the RC beams were substituted by steel beams [4], which were aimed as structural fuses, getting most of the damages and being later easily replaced. Steel beams are of great advantage in case of height restrictions, or when the required capacity and stiffness cannot be economically assured by RC coupling beams. According to [5], HCW systems are usually built in combination with steel framing systems, i.e.: (i) coupled core walls; (ii) coupled shear walls located on the building’s perimeter. Steel coupling beams dissipate energy similarly to the shear links in the eccentrically braced frames (EBFs), and can be short / intermediate / long links. The HCW systems offer an optimal combination of stiffness, strength and ductility - leading to adequate seismic performance.

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The development of the "*single-pier hybrid coupled wall*" system was part of the European research project INNO-HYCO [6], which involved the investigation of innovative and improved hybrid steel and concrete systems consisting of a RC shear wall, coupled to steel side columns by means of steel shear links. The aim of the RC wall is to resist the horizontal shear force, while the overturning moments are partially resisted by the axial compression-tension couple developed by the two external steel columns rather than by the flexural action of the wall alone. The strength and ductility of the shear links – directly influences the performance of the system (e.g. allowing the structure to deform inelastically without a significant loss of strength), limits the maximum lateral forces transmitted to the non-dissipative structural members and provides hysteric energy dissipation when designed correctly [6]. The shear links are designed and modelled as fixed at the connection to the wall and pinned at the connection with the steel column. Consequently, only the shear force is transferred to the external steel column, while both shear and bending moment are transferred by the connection to the RC wall. The wall is therefore subjected to bending and shear, while the steel columns are subjected mainly to axial forces (i.e. alternating tension / compression in dependence to the seismic action). In contrast to the solution with two RC walls coupled by steel beams, the single-pier HCW system presents several advantages, i.e.: ▪ a better control of the damage within the RC wall; ▪ smaller horizontal dimension of the HCW for the same coupling action; ▪ reduced total mass of the structure – due to the use of steel columns instead of an additional RC wall. Following a seismic event, it is considered that the yielded shear links can be easily replaced, provided that a suitable connection detailing is used. Consequently, as part of the INNO-HYCO [6] project, two types of connections have been proposed and tested (see also [7]), with the following main components: (i) an embedded element; (ii) the dissipative link; (iii) a beam splice able to transfer the bending moment and shear force from link to the embedded element; (iv) an angle joint between the shear link and the external column.

Recent research findings, e.g. [8] [9] [10], showed that single-pier hybrid coupled wall (HCW) systems can achieve: (i) controlled post-elastic ductile behaviour under medium- and high-intensity earthquakes; (ii) seismic energy dissipation effectively concentrated in steel dissipative components that can be easily replaced after seismic events; (iii) very limited damage in the RC wall. Nevertheless, further research activities are still needed for addressing particular issues and for developing advancements in the analysis, design, and detailing. Consequently, the ongoing European research project HYCAD [11] (*"Innovative steel-concrete HYbrid Coupled walls for buildings in seismic areas: Advancements and Design guidelines"*) aims at: (i) making the HCW system an effective and competitive alternative to other seismic-resistant structural solutions; (ii) facilitating the design and foster the application of HCWs as seismic-resistant solutions in the European construction market.

The current paper makes an overview of the following: (i) experimental program on components; (ii) proposed technical solution for connecting the replaceable dissipative shear links to the composite wall; (iii) experimental test set-up; (iv) response of the shear links based on pre-test FE investigations. In addition, the paper summarises the outcomes of a preliminary numerical investigation (i.e. nonlinear static and dynamic analyses), which aimed at assessing the behaviour and the seismic performance of the newly introduced HCW system. In particular, a case-study was carried out on a 3D multi-story building structure with perimeter lateral load resisting frames, each composed of: (i) a single-pier HCW, i.e. a steel-concrete composite wall coupled to steel columns through a set of replaceable steel shear links; (ii) a moment resisting frame (MRF). Furthermore, the current study also aimed to assess the seismic demand for the replaceable links in terms of shear deformation – important for the upcoming experimental investigations.

2 Experimental program: HCW component tests

Tests on components are currently being carried out within the HYCAD research framework [11], with the aim to identify the configurations with the best performance – and to use these for the pseudo-dynamic testing of two large scale structures. In particular, the shear link, as well as the link-to-wall connection are investigated corresponding to the following HCW configurations:

- HCW configuration 1 characterised by: ▪ cast in-situ reinforced concrete wall; ▪ post-tensioned link-to-wall connection; ▪ steel plate with shear studs embedded in the RC wall; ▪ replaceable shear link;
- HCW configuration 2 characterised by: ▪ composite wall with encased steel profiles (including welded shear studs); ▪ double-slab pre-cast concrete wall – with poured-in-situ concrete infill; ▪ rectangular hollow section (RHS) embedded in the RC wall; ▪ threaded rods used for the connection between shear link and RHS; ▪ replaceable shear link.

An overview of the experimental program corresponding to the link-to-HCW component tests at RWTH is presented in Table 1. For each configuration a number of three tests will be performed considering the following loading types: (i) monotonic loading; (ii) cyclic loading using the ECCS-1986 [12] and the EN-15129 [13] loading protocols. The main tests will be preceded by standard tests for the material characterization (i.e. concrete and main steel components).

Table 1 Experimental program: Link-to-HCW component tests

Configuration	Wall type	Loading	Nr.
Configuration 1 [**HCW_C-1**]	Cast in-situ reinforced concrete wall	Monotonic	1
		Cyclic	2
Configuration 2 [**HCW_C-2**]	Composite wall with encased steel profiles	Monotonic	1
		Cyclic	2

The proposed technical solution for connecting the replaceable shear link to the composite HCW, respectively to the external steel column – is illustrated in Figure 1 (with a focus on the connection details and components) and in Figure 2 (view of the specimen and test set-up).

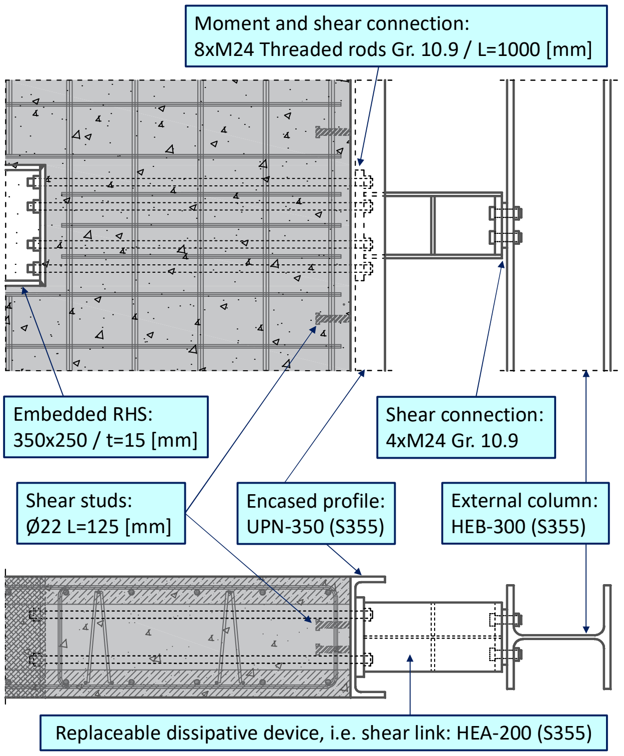


Figure 1 Technical solution for the HCW Configuration 2 "*HCW_C-2*", and detail of the connection zone between shear link and: ▪ composite wall; ▪ external steel column.

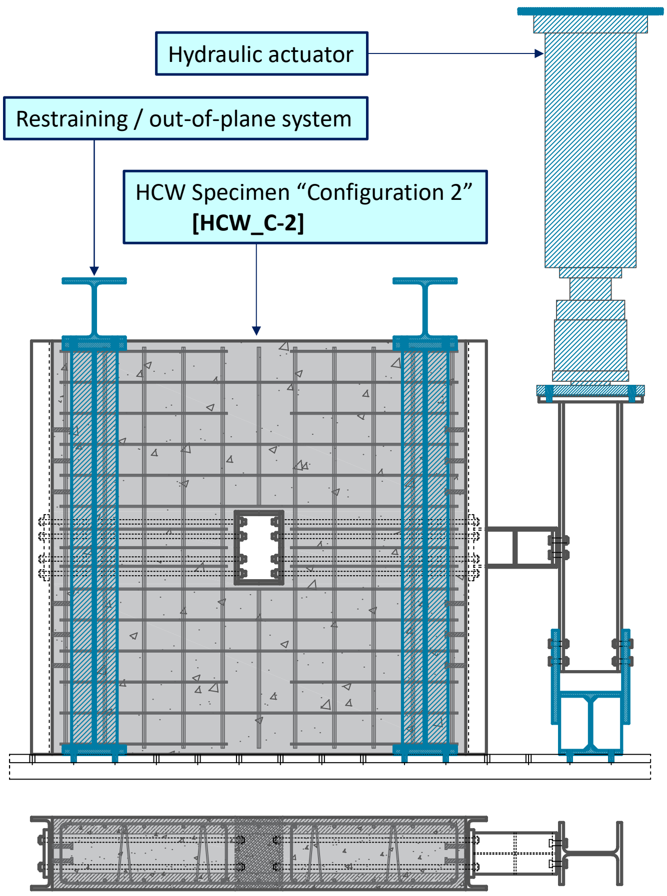


Figure 2 Experimental test set-up and *"HCW_C-2"* specimen

The lateral and top view from Figure 2 illustrate the *"HCW_C-2"* specimen configuration, as well as the test set-up with the following main components: ▪ the restraining / out-of-plane system (located around the wall and at the lower part of the external steel column; ▪ a hydraulic actuator connected to the external column. Regarding dimensions, the RC wall has 350 mm width, and respectively 2000 mm length / height. The length of the shear link is equal to 400 mm. Further details of the technical solution (see Figure 1) can be summarized as follows: ▪ link-to-wall connection realised with "*8xM24 Gr. 10.9 / L=1000 mm*" threaded rods; ▪ link-to-column connection realised with "*4xM24 Gr. 10.9*" bolts and a "*15 mm / S355*" filler plate; ▪ partially encased "*UPN-350 / S355*" profiles, each with "*12xΦ22 / L=125 mm*" shear studs; ▪ "*HEB-300 / S355*" external steel column; ▪ "*HEA-200 / S355*" shear links with - "*t=10 mm / S355*" web stiffeners, "*t=25 mm / S355*" extended end-plate, and "*t=20 mm / S355*" flush end-plate; ▪ an embedded "*350x250 mm / t=15 mm / S355*" rectangular hollow section (RHS); ▪ "*Φ10÷16 mm / B450*" reinforcement; ▪ double slab precast concrete wall with "*C35/45*" in-situ concrete infill. The "*link - to - composite wall*" connection acts as a moment and shear connection due to the following: ▪ extended end-plate welded to the shear link; ▪ four rows of pre-stressed threaded rods – aimed to transfer the shear force to the encased UPN profile, and the bending moment to the composite wall. In contrast, the "*link - to – external column*" connection acts as a shear connection as it contains: ▪ two rows of pre-stressed bolts placed close to the longitudinal axis of the link; ▪ a filler plate, aimed to facilitate the replacement of the dissipative links, as well as to prevent the contact between the flanges of the link and the external column and to reduce the tension / compression couple within the connection. The final design of the specimens, including the "*HCW_C-2*", was investigated numerically with the finite element (FE) modelling software Abaqus [14]. Consequently, the main design objective was confirmed (i.e. development of large plastic deformations in the shear link and the elastic response of the wall and external column – see Figure 3) and the corresponding force-deformation curve (see Figure 6) was determined.

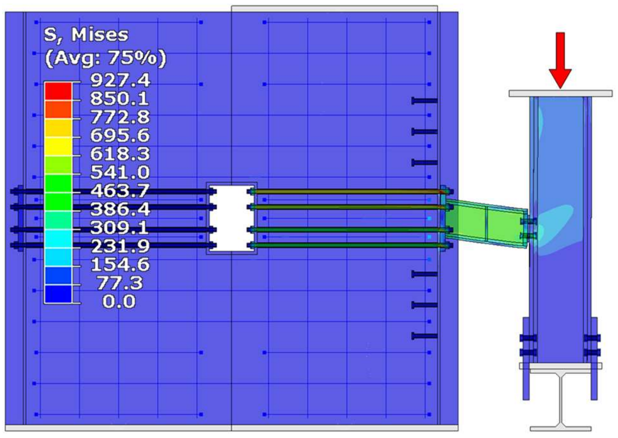


Figure 3 FE numerical investigation of the "*HCW_C-2*"

Further details regarding the specimen design and the FE investigations are available within: [15] [16].

3 Seismic performance evaluation

The aim of the current study, presented in detail within [17], was not only to investigate the seismic performance through a set of nonlinear static and dynamic analyses, but also to evaluate the deformation demand for the shear links in the context of the ongoing experimental investigations. Consequently, a case-study was carried out on a 3D multi-story building structure (see Figure 4) with perimeter lateral load resisting structures (see Figure 5), each composed of: (i) a single-pier HCW, i.e. a composite wall coupled to steel columns through a set of replaceable steel shear links; (ii) a moment resisting frame (MRF). It is to be noted that the structural system is identical on the two horizontal directions. The case-study building has *14 m* height with 4 floors of *3.5 m* height. As it can be observed in Figure 5**,** each perimeter frame has three spans, i.e.: ▪ *6.6 m* – which contains secondary beams with "pinned" connections; ▪ *3.2 m* – which contains the HCW subsystem; ▪ *8.2 m* – which contains the MRF subsystem. The design was performed considering a permanent load of *5 kN/m²* (slab surface load) and *3.5 kN/m* (façade line load), as well as a live load of *3.3 kN/m²*. The seismic load was defined by seismic spectrum according to EN 1998-1 [18] (*spectrum type 1, ground type C,* $S = 1.15$, $\gamma_I = 1.0$, $a_g = 0.3g$, $\psi_{2,i} = 0.3$, $\varphi = 0.8$, $\psi_{E,i} = 0.24$, *behaviour factor* $q = 4$).

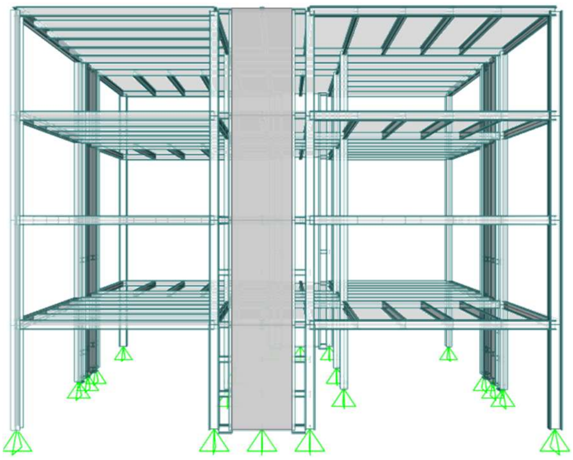


Figure 4 Structural model of the 3D multi-story building with perimeter hybrid coupled walls (HCWs) and replaceable shear links (*Sap2000 [19] model - view with extruded elements*)

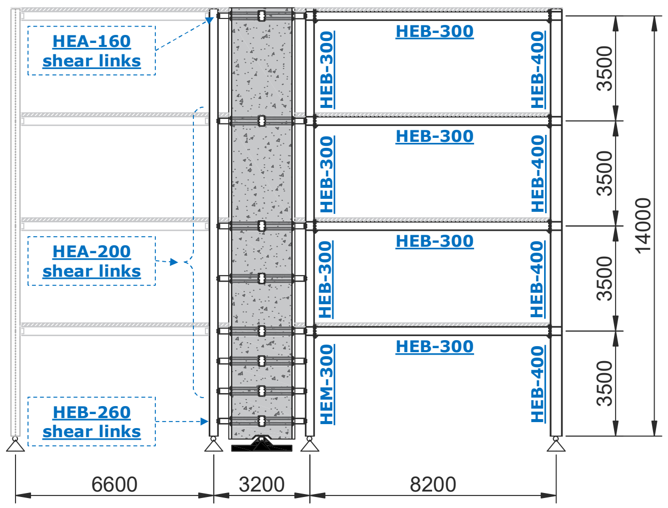


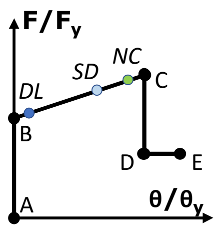
Figure 5 Perimeter lateral load resisting structure: [HCW+MRF]

The design and analysis of the case-study building structure was performed with Sap2000 [19]. The main design and modelling considerations are: ▪ all column bases were pinned; ▪ the base of the composite walls was defined as pinned – based on [20]; ▪ the MRF subsystem was aimed to respond in the elastic range (also for near collapse seismic intensity) with the purpose of providing structural re-centring – as proposed within [21]; ▪ the "*link-to-wall*" and "*link-to-column*" connections were modelled as fixed and respectively pinned based on the technical solution in Figure 1; ▪ the material properties correspond to those described in Section 2, and the beams and columns from the MRF subsystem were made of S355 steel grade; ▪ a diaphragm effect was assigned at each floor – considering the presence and the constraining effect of the concrete slab; ▪ the composite wall, aimed to respond in the elastic range, was modelled using beam elements – to which the cross-section from Figure 2 was assigned (i.e. concrete / reinforcement / encased UPN profiles); ▪ the secondary composite beams and the internal framing system were aimed to transfer the gravity loads only; ▪ rigid end-length offsets were defined for the beam elements of the shear links located in the composite wall and in the external steel column; ▪ the same cross-section was used for all shear links (i.e. HEA-200 / S355), with the exception of those located at the top floor and close to the foundation. The outcome of the design phase was represented by the configuration of the perimeter lateral load resisting structure – as illustrated in Figure 5. As it can be observed, due to the pinned column / wall bases, a larger number of "*identical*" shear links was needed at the 1st and 2nd floor. From the modal analysis, the fundamental period of vibration was obtained in amount of $T_1 = 0.759\ s$. The nonlinear response of the structural members was defined by means of concentrated plasticity, i.e.:

– For *beams / columns* – a plastic hinge type for members subjected to "flexure" / "*axial load and flexure*" was defined, with the acceptance criteria from FEMA-356 [22]: $1\cdot\theta_y$ at Damage Limitation (DL), $6\cdot\theta_y$ at Significant Damage (SD), $8\cdot\theta_y$ at Near Collapse (NC), where θ_y is the yield rotation;
– For *shear links*: a plastic hinge type for members subjected to shear was defined according to FEMA-356 [22], with the modelling parameters and the acceptance criteria as summarised in Table 2.

Table 2 Modelling parameters | acceptance criteria - shear links [22]

Point	A	B	C	D	E
F/F_y	0	1	1.5	0.8	0.8
θ/θ_y	0	0	0.075	0.075	0.1
Acceptance criteria			**DL**: $0.0025\cdot\theta_y$ **SD**: $0.055\cdot\theta_y$ **NC**: $0.07\cdot\theta_y$		



The parameters from Table 2 were used within the simplified model in Figure 6a. From a nonlinear static analysis, the capacity curve corresponding to the shear link was obtained (see Figure 6b). The comparison with the force-deformation curve from Abaqus (see Figure 6b) confirmed that the simplified shear link model was able to reproduce the response in terms of stiffness and capacity. The nonlinear static analysis of the 3D multi-story building with HCWs – performed only on the X-direction, and using a modal distribution of forces – allowed assessing the: (i) overall capacity curve on the X-direction, i.e. [HCW+MRF]; (ii) target displacements corresponding to the DL | SD | NC limit states; (iii) contribution of the [HCW] and [MRF] subsystems. As it can be observed in Figure 7, using the N2 Method [23] [18], the following target displacements were computed: ▪ $D_{t,DL}=0.067\ m$; ▪ $D_{t,SD}=0.135\ m$; ▪ $D_{t,NC}=0.202\ m$. The state of the structure corresponding to each of the three seismic intensities – was characterised by an elastic response of the MRFs, respectively by a nonlinear response of the HCWs (i.e. plastic hinges developed in all shear links). Related to the contribution of the two subsystems, Figure 7 evidences the following: ▪ a linear elastic response of the MRFs even beyond NC intensity; ▪ the HCWs have the main contribution to the stiffness and capacity.

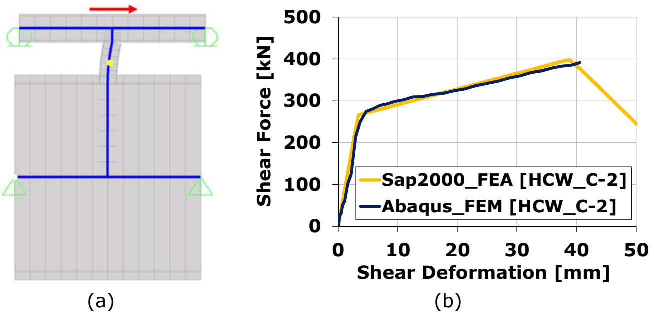


Figure 6 FEA simplified model of the shear link: (a) detail of the Sap2000 model; (b) comparison (in terms of force-deformation curve) between the simplified (Sap2000) and the advanced (Abaqus) model.

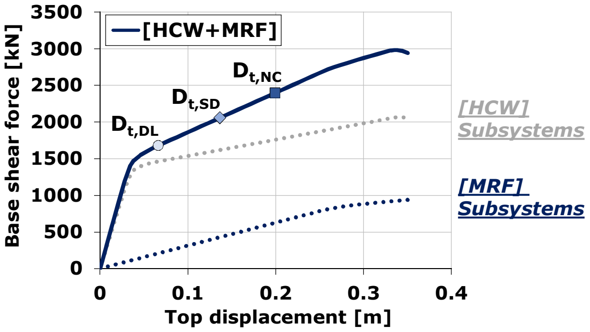
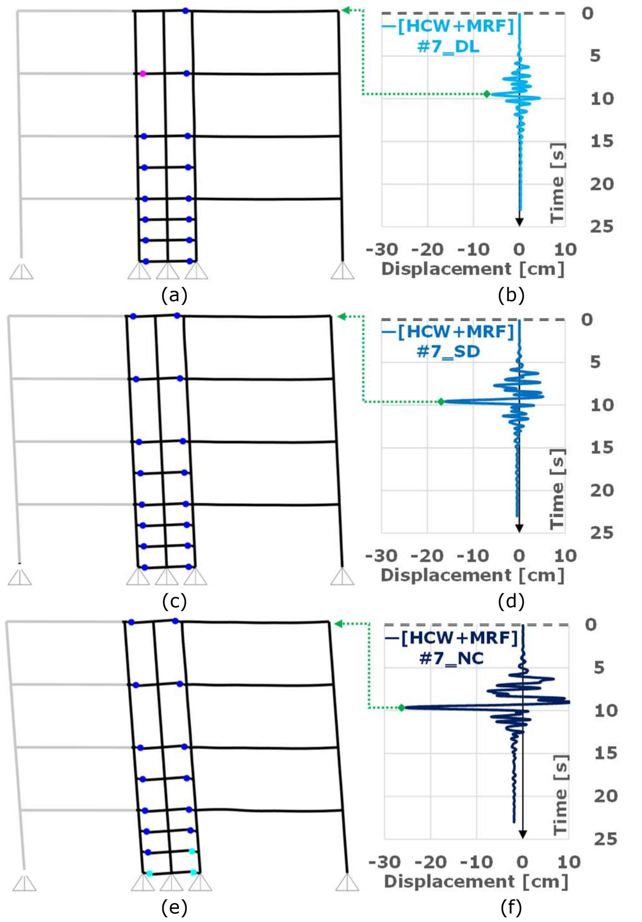


Figure 7 Outcomes of the nonlinear static analysis: capacity curve | target displacements | contribution of [HCW] and [MRF] subsystems

The nonlinear response history analysis (NRHA) of the 3D multi-story building with HCWs – performed on the X-direction only, and using a set of seven accelerograms matching the target spectrum (see [17]) – allowed assessing for each accelerogram (i.e. *#1÷7*) and each limit state (i.e. *DL | SD | NC*) the following: (i) displacement at the top floor over time; (ii) state of the [HCW+MRF] structure at maximum lateral displacement; (iii) min./max. inter-story drifts; (iv) residual displacements at the top floor. An illustration of the response, i.e. under to the most demanding accelerogram (#7), is shown in Figure 8 and Figure 9 in terms of: ▪ damage state of the structure; ▪ displacement at the top floor vs. time; ▪ inter-story drifts. Consequently, the HCW subsystems developed plastic hinges in all shear links and for each limit state – including for DL (see Figure 8a). The deformations within shear links corresponded mainly to DL criteria, and only at NC intensity – three plastic hinges reached LS criteria. The MRF subsystems responded in the elastic range. Regarding the displacement at the top floor, it was observed that the NRHA average values ($D_{Av.,DL}=0.061\ m$, $D_{Av.,SD}=0.130\ m$, $D_{Av.,NC}=0.207\ m$) were very close to the target displacement values computed with the N2 method [23]. As it can be observed in Figure 9, the inter-story drift limits (*7.5 mrad* for DL, and *15 mrad* for SD) were not reached by the investigated [HCW+MRF] system. Furthermore, a relative uniform distribution of the inter-story drifts was observed over the height of the building, with the lowest values corresponding to the 1st floor. The average values of the residual top displacements were: ▪ *11 mm* at DL; ▪ *31 mm* at SD; *43 mm* at NC. The deformation demand for the shear links (i.e. average #1÷7) was: ▪ *7.6 mm* at DL; ▪ *15.6 mm* at SD; ▪ *23.9 mm* at NC.



Plastic hinge legend: pre-DL; DL; SD; NC;

Figure 8 State of the structure at max. lateral deformation generated by Acc.#7, and displacement at the top floor vs. time - corresponding to the three seismic intensities: (a)-(b) DL; (b)-(c) SD; (d)-(e) NC.

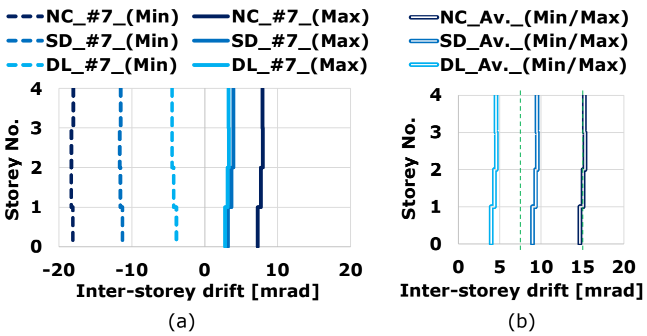


Figure 9 Inter-story drifts corresponding to the DL | SD | NC seismic intensity: (a) min./max. drifts from Acc.#7; (b) average drifts (#1÷7)

4 Conclusions

The aim of the current study was to investigate the behaviour and the seismic performance of a newly introduced steel-concrete HCW system. In particular, a case-study was carried out on a 3D multi-story building structure with *[HCW+MRF]* perimeter lateral load resisting frames. In addition, an overview was shown regarding the: (i) technical solution for connecting the shear links to the composite wall – *"HCW_C-2"*; (ii) test set-up and experimental program; (iii) response of the shear links based on pre-test FEM; (iv) modelling | design | analysis – of the 3D structure and the corresponding seismic performance; (v) deformation demand for the shear links. Advanced FE investigations confirmed the performance (i.e. intended load transfer mechanism | failure mode) of the *"HCW_C-2"* proposed technical solution. The nonlinear static and dynamic analyses, evidenced an adequate seismic performance of the *[HCW+MRF]* system, i.e.: ▪ the *[HCW]* subsystems had the main contribution to the stiffness and capacity, and were capable of developing a global plastic mechanism (i.e. plastic hinges formed in all shear links, however also at DL intensity); ▪ the *[MRF]* subsystems were characterised by an elastic response even beyond NC intensity; ▪ the average displacements at top floor (i.e. from the NRHA) were close to the target displacements computed with the N2 method; ▪ a uniform distribution of inter-story drifts was observed, and the allowed limits (i.e. DL: *7.5 mrad*; SD: *15 mrad*) were not reached; ▪ regarding repairability, low residual top displacements were evidenced (*11 mm* at DL; *31 mm* at SD; *43 mm* at NC); ▪ the deformation demand for shear links was: *7.6 mm* at DL; *15.6 mm* at SD; *23.9 mm* at NC.

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