EARTHQUAKE RESISTING POTENTIAL OF AN INNOVATIVE HCW SYSTEM WITH LASER-CUT OPEN-TO-CHS CONNECTIONS

R. Das [(1)], B. Vandoren [(2)], H. Degee [(3)]

[(1)] *PhD researcher, Construction Engineering Research Group, Universiteit Hasselt, Belgium, rajarshi.das@uhasselt.be*
[(2)] *Associate Professor, Construction Engineering Research Group, Universiteit Hasselt, Belgium, bram.vandoren@uhasselt.be*
[(3)] *Professor, Construction Engineering Research Group, Universiteit Hasselt, Belgium, herve.degee@uhasselt.be*
…

Abstract

Although conventional reinforced concrete (RC) or hybrid coupled wall (HCW) structures have been used for a number of years as seismic resistant system thanks to their lateral strength, stiffness, and energy dissipation characteristics, some drawbacks such as expensive detailing, costly foundations, heavy superstructure, difficult restoration works etc. have limited their potential from a structural and an economic perspective. To minimize these drawbacks and make further advancement, an innovative HCW system is proposed in this study consisting of a single RC wall coupled with two steel circular hollow section (CHS) columns via steel coupling links. The RC 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 CHS columns rather than by the individual flexural action of the wall alone. The primary objective in designing this system is ensuring a “fuse”-like behaviour of the steel coupling links, i.e. concentrating the seismic damage in the coupling links while avoiding any damage in the RC wall as well as in the connections between the links and the primary vertical elements (RC wall and CHS column). Multiple case studies with varying coupling ratios are investigated through nonlinear pushover analyses to verify the above-mentioned design objective.

Different configurations are proposed to achieve an efficient link-to-RC wall composite connection, which can ensure the “fuse”-like behaviour of the coupling link. The steel coupling links are connected to a steel profile either partly embedded in the RC wall or passing through it. The connection zone is designed in such a way that the damage always occurs in the steel links (fuses) prior to any damage in the RC wall in general and in the connection zone in particular. Case studies have therefore been designed based on the force demands obtained from the global structure and further examined through detailed pushover analyses to highlight their applicability in the HCW systems.

A suitable link-to-CHS column connection is also necessary to ensure the “fuse”-like behaviour of the coupling links. However, the conventional open-to-CHS column connections, with beams (or links) directly welded on the tube, are often prone to severe local distortion of the CHS column surface, premature flange fractures and excessive welding quantity. To avoid such limitations, this paper introduces different types of innovative I-beam-to-CHS-column “passing-through” connections. These connections consist of coupling links connected to an I-beam stub (or vertical and horizontal plates) entering the CHS column through laser-cut slots. Standard design guidelines have been developed in accordance with Eurocode provisions for gravitational and seismic loading scenarios to calculate the ultimate joint resistances. Case studies have therefore been designed based on the global demands and further examined through detailed pushover analyses in order to validate their compatibility to the HCW systems.

Based on the design case studies, the primary design objective was achieved in both cases – the global and local perspective. Yielding was attained in the coupling links prior to any damage in the RC wall, CHS columns, and the connections. Encouraging validations are therefore presented regarding the earthquake resistant potential of the HCW system and the applicability of the innovative connections.

Keywords: Earthquake resistant design, hybrid coupled wall structure, embedded composite connection, open-to-CHS connection.

1. Introduction

In the modern construction industry, there is an increasing motivation towards developing better seismic resistant structures in order to avoid huge social and economic loses. Engineers and researches have therefore tried to develop innovative structures and corresponding design hypotheses to improve the structural resistance against the unpredictable challenges offered by the ground motions. Over the past few decades, conventional reinforced concrete (RC) or hybrid coupled wall (HCW) systems have proved to be among the most effective solutions in high to moderate seismic areas through an architecturally practical structural system, large lateral stiffness and strength provided by the coupling effect, and a competent energy dissipation mechanism provided by the coupling beams (or links) without significantly affecting the stability of the walls [1]. The coupling links act as "fuse" elements and attract the seismic damages prior to the primary load-carrying member i.e. the RC wall. Once the links are damaged, they can be easily replaced or repaired to restore the structural ability of the whole system. Certain drawbacks have however limited the potential of such coupled wall (CW) systems from a structural as well as economic perspective. For instance, two RC shear walls used in the conventional CW systems lead to a high overall weight of the superstructure and therefore demand an expensive substructure. Additionally, such huge amount of concrete reduces the overall ductility of the seismic resistant system, consequently limiting a crucial factor against earthquakes [2]. In order to minimize these drawbacks and further improve the combination between RC walls and steel coupling links, an innovative HCW system was proposed in the INNO-HYCO project [2] which consisted of a single RC wall coupled with two steel side "H" section columns via steel coupling links. These HCW systems are principally constructed as a part of a large frame structure (Fig. 1a). While the HCWs act as the earthquake resistant systems for the whole structure, the gravitational loads are resisted by the remaining parts of the frame (denoted as the "GR part" in Fig. 1a). These systems avoided the foretold drawbacks due to a reduced percentage of concrete and fulfilled the primary design objective i.e. yielding of the steel coupling links (either in flexure or shear or both) prior to any damage in the primary load carrying members (RC wall and steel columns) and therefore, provided an encouraging resistant system. The INNO-HYCO system has been further improved and refined in a doctoral research program [3]. Its major findings are summarized in the current article.

In order to ensure that the steel coupling links yield prior to the primary elements of the HCW (Fig. 1b), their connections with the RC wall as well as the steel columns should be designed in a suitable manner i.e. the connections should be able to provide sufficient strength and stay undamaged until yielding is obtained in the coupling links. This research study focuses on these connections from a detailed perspective.

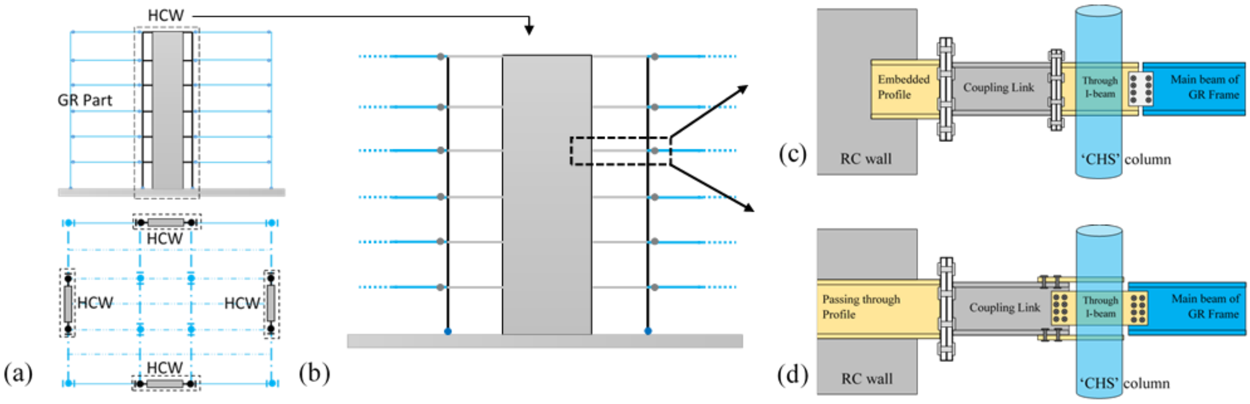

Fig. 1 – (a) Plan view and floor geometry of a 6-storey frame structure with positions of the HCW systems (b) schematic diagram of the HCW system, (c) an example of the "partly embedded" (denoted as WC-1) link-to-wall connection and the "through beam" (denoted as CC-1) link-to-CHS column connection, and (d) an example of the "passing-through" (denoted as WC-2) link-to-wall connection and the "through plates" (denoted as CC-2) link-to-CHS column connection.

Firstly, the link-to-wall “embedded” connections (i.e. coupling links are connected to the RC walls via splice/bolted connections with additional steel profiles embedded into the wall as shown in Fig. 1c) were investigated via optimizing design guidelines to ensure that the primary design objective of the HCW system is fulfilled i.e. yielding occurs in the coupling links prior to any damage in the RC wall or its connections. An alternative solution was also developed, namely a “passing-through” configuration (Fig. 1d), where the coupling link is connected to a steel profile passing through the RC wall. Detailed studies were conducted to analyze both types of configurations and optimized design guidelines were proposed with appropriate validations through numerical simulations [3]. Secondly, Circular Hollow Sections (CHS) were used for the steel side columns instead of the conventional “H” sections to improve the behaviour as well as the aesthetics of the HCW system. The CHS’ resistance against high axial forces and bending in all directions, lighter weight, lesser requirement for fire protection materials compared to its equivalent “H” section have already been emphasized in several research studies [3]. However, in today’s construction industry, the beam-to-column connections, involving a CHS column, are generally constructed by directly welding the beams to the CHS column wall or by using external ring diaphragms. These techniques lead to definite limitations such as local distortions on the CHS wall, substantial use of gusset plates and/or stiffeners, complicated fabrication works etc. [4], which have unfortunately limited the CHS’ preference from a designer's perspective. Several researchers have therefore searched for alternative solutions to improve the open-to-CHS column connections. Among such elaborate research work, a “passing-through” approach has proved to be a possible alternative to the traditional connections [5] where the primary members are connected to a secondary member passing through the CHS column. Nevertheless, a lack of knowledge regarding these “passing-through” open-to-CHS connections remained prevalent due to practical difficulties in the mechanical cutting process, fabrication as well as controlling the tolerance issues. Thanks to several advantages offered by Laser Cutting Technology (LCT) in construction engineering [6], these issues were positively managed through automated procedures and precise tolerance control. Therefore, several types of “passing-through” pinned (or shear resisting) and rigid (or moment resisting) open-to-CHS connections were investigated in the EU-RFCS research project LASTEICON [6]. Such “passing-through” mechanisms minimize the local distortions on the CHS column wall and the extensive use of gusset plates and/or stiffeners. They also result in a simpler fabrication process. Standard design guidelines were developed to design the LASTEICON connections under gravitational and seismic loading conditions. Two different “passing-through” LASTEICON moment resisting connections were therefore implemented in the HCW systems: (1) the “through beam” connection (see Fig. 1c), constituting of main beams (i.e. the coupling links of the HCW) connected to an I-beam stub passing through the CHS column via Laser Cut slots and (2) the “through plates” connection (see Fig. 1d), constituting of main beams (i.e. the coupling links of the HCW) connected via three plates (two transverse flange plates and a longitudinal web plate) passing through the CHS. The applicability of these LASTEICON “passing-through” connections in the HCW systems were validated through analytical and numerical simulations.

This paper aims at summarizing the previous research study [3], dealing with the above-mentioned advancements on the INNO-HYCO HCW system, through design case studies complying with the Eurocode 3 and 8 provisions. It therefore presents a newly modified HCW system consisting of one RC wall coupled to two steel side CHS columns via shear/intermediate/flexural coupling links. The link-to-wall connections (denoted as “WC”) are assumed to be rigid or moment resisting. The applicability of two different configurations, namely “partly embedded” (denoted as “WC-1” in Fig. 1c) and “passing-through” (denoted as “WC-2” in Fig. 1d) configuration, are used to design the link-to-wall connections. As the HCW system is a part of a large frame, each CHS column shares two different connections with the global structure: a rigid connection with the steel coupling links of the HCW system and a pinned connection with the primary beams of the GR part (see Fig. 1a). Only the coupling link-to-CHS column connection (denoted as “CC”) is detailed in this study to focus on the HCW system rather than the whole structure. Both types of configurations – “through beam” (denoted as “CC-1” in Fig. 1c) and “through plates” (denoted as “CC-2” in Fig. 1d) are used to demonstrate the applicability of the LASTEICON connections in the HCW systems.

2. Design Methodology

Detailed design guidelines regarding the HCW systems, the link-to-wall connection configurations (WC-1 and WC-2), and the LASTEICON moment resisting connection configurations (CC-1 and CC-2) were developed based on comprehensive parametric studies. Complete details are documented in the doctoral thesis [3] and are not discussed here. However, a general framework is provided in Fig. 2 to offer a clear perspective about the design procedure.

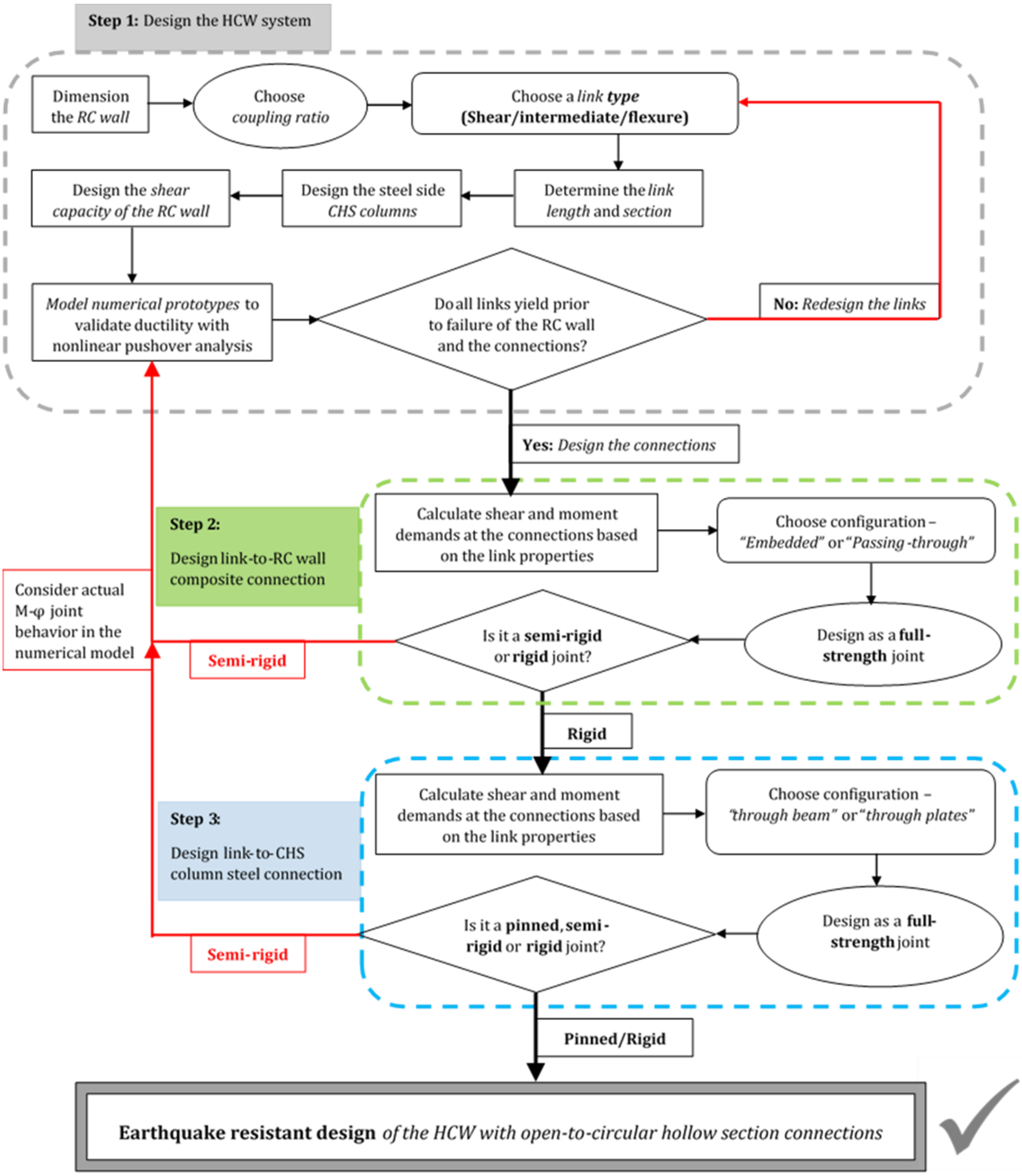

Fig. 2 – General framework for earthquake resistant design of the HCW system with laser-cut open-to-CHS connections

3. Design Case-studies

Two different sets of 6-storey HCW systems were designed using different coupling ratios (CR) according to the aforementioned design guidelines to check the compatibility of the LASTEICON moment resisting/rigid connections for both link types:

1. LRS-HCW – a HCW system consisting of LASTEICON Rigid connections and Shear links
2. LRF-HCW – a HCW system consisting of LASTEICON Rigid connections and Flexural links.

The seismic design loads for both systems were calculated in accordance with Eurocode 8. The interstorey height = 3.50 m, permanent floor loads = 4.30 kN/m^2, variable floor loads = 2.00 kN/m^2, permanent roof loads = 3.30 kN/m^2, variable roof loads = 1.97 kN/m^2 were considered. Concrete for the RC wall was taken as class C30 (f_{ck} = 30 MPa) and reinforcements were taken as B450C (f_{yk} = 450 MPa) as per EN 1992-1-1 [7]. The reinforcements were designed according to the DCM rules stated in EN 1998-1 for ductile walls [8]. To maintain an economic efficiency, adequate Built-up I Profiles (BuIP) were developed to design the shear critical coupling links for the LRS-HCW system instead of the standard IPE profiles. However, the flexural links for the LRF-HCW systems were designed with standard IPE profiles. The link lengths were calculated from the corresponding design procedures of the LRS-HCW and LRF-HCW systems [3]. The steel side CHS columns were designed for a combination of axial and bending forces and were chosen from the European standard CHS profiles classified as Class 1 or 2 sections in compression (i.e. compact sections) according to EN 1993-1-1 [9]. The column section, chosen for a specific CR, was used for all storeys to avoid variations in link lengths as well as any complications in practical construction. Steel grade S355 (nominal yield stress, f_y = 355 Mpa) was adopted for both the coupling links and the CHS columns. Finally, the shear design of the RC wall was carried out. In relation to the total height of the building (H) = 21 m and a previously suggested ratio, H/l_w = 10 [2], the length and width of the RC wall was calculated to be 2.1 m (l_w) and 0.36 m respectively as shown in Fig. 3a. The same cross section of the RC wall was used for both the LRS-HCW and LRF-HCW systems. Table 1 shows the outcomes of link, column design and estimated base shears.

Once the primary design objectives were achieved for the global structure, the link-to-wall connections were designed. The moment and shear demands on the link-to-wall connections were determined based on the link properties used in the global structure. In order to present alternative approaches, both configurations – "partly embedded" ("WC-1" as shown in Fig. 3b) and "passing-through" ("WC-2" as shown in Fig. 3c) - were used to design each link-to-wall connection located in the different storey levels of both the LRS-HCW and LRF-HCW systems. The offset length, l_0, was considered as 100mm for all cases. Two plates with dimensions 300mm x 400mm x 25mm were used for the splice connection. The embedment lengths, embedded/passing-through profiles were determined from the design guidelines mentioned in Section 2. Relevant outcomes of the design calculations such as the embedment lengths and sections for WC-1, passing through sections for WC-2, are listed in Table 1. Stiffeners were then provided at a distance of 1/6th of the embedment length (l_e in Fig. 3) from the face of the RC wall and from the end of the embedded profile respectively, to prevent local deformation of the flanges in the steel-concrete bearing zones. Steel grade S355 (nominal yield stress, f_y = 355 MPa) was adopted for the embedded profiles, the stiffeners and the beam splice connection plates.

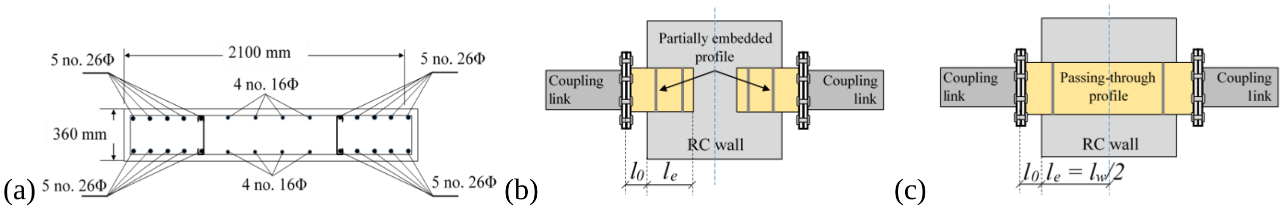

Fig. 3 – (a) Reinforcement detailing for RC wall of the 6-storey HCW system [3] and schematic diagram of (b) WC-1 configuration (c) WC-2 configuration

Table 1 - Design outcomes for the coupling links, CHS columns, link-to-wall connections and link-to-column connections

Step 1: Design of the HCW system	**LRS-HCW system**					**LRF-HCW system**				
Coupling Ratio (CR)	**0.4**	**0.6**	**0.8 Non-uniform**			**0.4**	**0.6**	**0.8 Non-uniform**		
Link Section	BuIP_17	BuIP_18	BuIP_18	BuIP_19	BuIP_20	IPE200	IPE220	IPE240	IPE400	IPE450
Link Length (mm)	400.00	500.00	500.00	500.00	500.00	700.00	700.00	1200.00	1200.00	1200.00
Section Depth, H (mm)	200.00	220.00	220.00	400.00	500.00	200.00	220.00	240.00	400.00	450.00
Flange Width, b (mm)	75.00	85.00	85.00	105.00	115.00	100.00	110.00	120.00	180.00	190.00
Flange Thickness, t_f (mm)	11.60	13.50	13.50	16.00	17.20	8.50	9.20	9.80	13.50	14.60
Web Thickness, t_w (mm)	5.60	5.90	5.90	8.60	10.20	5.6	5.90	6.20	8.60	9.40
Storey level	All	All	6,5	4,3	2,1	All	All	6,5	4,3	2,1
Shear Demand (kN)	103.45	217.74	193.55	580.65	967.74	85.71	192.86	133.33	400.00	666.67
Moment Demand (kNm)	20.69	54.44	48.39	145.16	241.94	30.00	67.50	80.00	240.00	400.00
Yield Shear, V_n (kN)	216.20	249.69	249.69	676.95	1009.19	219.80	254.92	292.53	681.28	838.87
Yield Moment, M_n (kNm)	58.37	84.27	84.27	228.47	340.60	57.79	75.73	96.10	333.41	428.77
Axial Demand on column, N_{Ed} (kN)	1783.66	2059.93		5323.52		1813.39	2103.09		4620.91	
Flexural Demand on column, M_{Ed} (kNm)	90.80	134.20		701.89		90.76	126.72		729.08	
CHS diameter, d_c (mm)	244.50	323.90		610.00		244.50	323.90		610.00	
CHS thickness, t_c (mm)	20.00	20.00		20.00		20.00	20.00		20.00	
Estimated Base Shear for H_1 = 2/3 H (kN)	658.70	745.40		1468.10		742.63	815.06		1774.58	
Step 2: Design of the link-to-wall connection										
WC-1: "Embedded" Configuration	WC-1.1	WC-1.2	WC-1.2	WC-1.3	WC-1.4	WC-1.5	WC-1.6	WC-1.7	WC-1.8	WC-1.9
Embedded Length, l_e (mm)	385.00	440.00	440.00	780.00	1000.00	285.00	325.00	340.00	650.00	740.00
Embedded Profile	HEB240	HEB280	HEB280	HEB500	HEM500	HEA220	HEA240	HEA260	HEA450	HEA500
WC-2: "Passing-through" Configuration	WC-2.1	WC-2.2	WC-2.2	WC-2.3	WC-2.4	WC-2.5	WC-2.6	WC-2.7	WC-2.8	WC-2.9
Passing through Profile	HEM220	HEM240	HEM240	HEM360	HEM500	HEB220	HEB240	HEB240	HEB400	HEB500
Step 3: Design of the link-to-CHS column connection										
CC-1: "through beam" Configuration	CC-1.1	CC-1.2	CC-1.2	CC-1.3	CC-1.4	CC-1.5	CC-1.6	CC-1.7	CC-1.8	CC-1.9
Through I section	IPE300	IPE330	IPE330	IPE400	IPE500	IPE270	IPE300	IPE300	IPE400	IPE450
CC-2: "through plates" Configuration	CC-2.1	CC-2.2	CC-2.2	CC-2.3	CC-2.4	CC-2.5	CC-2.6	CC-2.7	CC-2.8	CC-2.9
Through web plate height, h_w (mm)	150.00	160.00	160.00	160.00	160.00	150.00	160.00	160.00	160.00	160.00
Through flange plate width, b_f (mm)	75.00	85.00	85.00	105.00	115.00	100.00	110.00	120.00	180.00	190.00
Through flange plate thickness, t_f (mm)	12.00	12.00	12.00	12.00	12.00	12.00	12.00	12.00	12.00	12.00
Through web plate thickness, t_w (mm)	16.00	16.00	16.00	16.00	16.00	16.00	16.00	16.00	16.00	16.00

Similarly, after determining the moment and shear demands from the global structure, the link-to-CHS column connections at each storey level were designed using two different types of LASTEICON configurations – "through beam" or "CC-1" and "through plates" or "CC-2" – for both types of HCW system. A S355 steel grade was considered for all elements. Fig. 4a and Fig. 4b illustrates the schematic diagrams for configuration CC-1 and CC-2 respectively with the common geometrical dimensions used for all the design case studies. All the connections were investigated through nonlinear numerical simulations in order to validate the different types of link failure – a shear failure of the coupling link in the LRS-HCW system and a flexural failure of the coupling link in the LRF-HCW system.

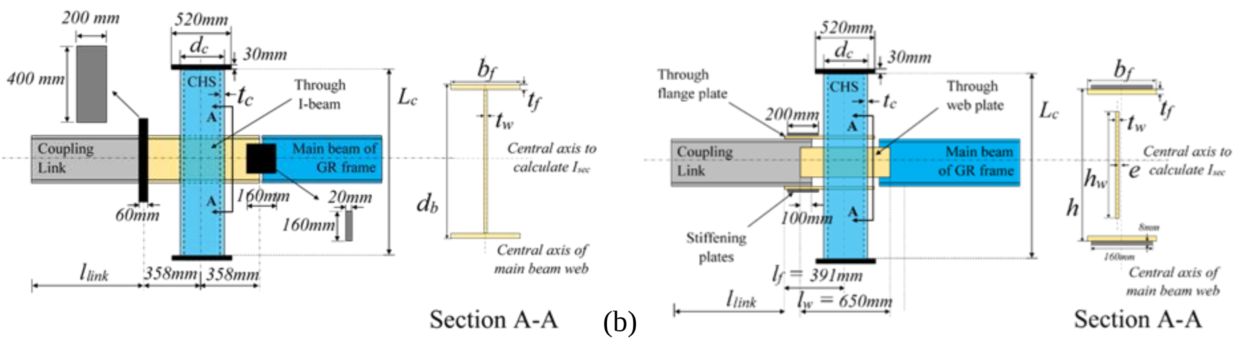

Fig. 4 – Schematic diagram of LASTEICON (a) CC-1 configuration and (b) CC-2 configuration

4. Modelling approach

The global HCW systems were modelled and analyzed by means of a plane model. The RC wall, links and columns were modelled using linear elastic frame elements (axial, flexural and shear deformability) while the nonlinear behavior was included using shear/flexural hinges. Fixed boundary conditions were assumed for both, the link-to-wall and link-to-CHS column connections. Eccentricities of the connections were incorporated through rigid links. The base of the steel side columns were pinned to the ground, whereas the base of the RC wall was fixed. In the next step, the detailed link-to-wall connection configurations (WC-1 and WC-2) were modeled and analyzed through nonlinear static analyses by means of a full 3D model using solid elements implemented in DIANA 10.1 [10]. All members were modelled using eight-node hexahedral solid elements. A perfect bond was assumed at the steel-concrete interface. Nonlinear plasticity was introduced in the RC wall according to EN 1992-1-1, Clause 3.1.7(2) [7]. Elastic-perfectly plastic material properties were chosen for the B450C steel reinforcements. Nominal material properties, with strain hardening, were adopted for steel grade S355 according to Table 3.1 (EN 10025-2), EN 1993-1-1 [9]. Finally, the different link-to-CHS column configurations (CC-1 and CC-2) were similarly modelled with 3D solid elements and analyzed in DIANA FEA. To avoid any secondary connection failure and focus on the "passing-through" zone, the laser cut slots were made with a zero tolerance, thus assuming a perfectly welded connection. The connection between the "through" member(s) and the coupling links was also modeled as a welded connection in order to avoid complicated numerical models and other failure mechanisms. Similar material properties were considered for S355 as stated above. The detailed models for the connections were calibrated against experimental results [3]. Additional details regarding the modelling approach of the different systems can also be found in the same study [3].

5. Results and Discussions

5.1 Behaviour of the HCW systems – LRS-HCW and LRF-HCW

The HCW systems were analyzed through a nonlinear pushover analysis considering a lateral load distribution proportional to the first modal deformation. The pushover curves illustrating the base shear versus the horizontal displacement of the top most point of the structure are shown in Fig. 5 and Fig. 6 respectively for the LRS-HCW and LRF-HCW systems. Both types of systems were investigated for three different coupling ratios i.e. CR = 40%, 60% ("U" denotes a uniform link distribution i.e. the links at all storey levels are designed with the same section) and 80% ("NU" denotes a non-uniform link distribution i.e. the links at different storey levels are designed with different sections). Each pushover curve highlights four important events: (a) first yielding in the steel links; (b) all links yielded; (c) first yielding in the steel reinforcements in the RC wall (attained at the base of the RC wall); and (d) collapse, i.e. ultimate deformation in the RC wall. The pushover curves in Fig. 5 and Fig. 6 show that all the steel coupling links yielded prior to yielding of the RC wall for both the LRS-HCW and LRF-HCW systems. Therefore, the primary design objective was fulfilled. Furthermore, the design hypothesis was verified as all the links in the LRS-HCW system yielded in shear and all the links in the LRF-HCW system yielded in flexure.

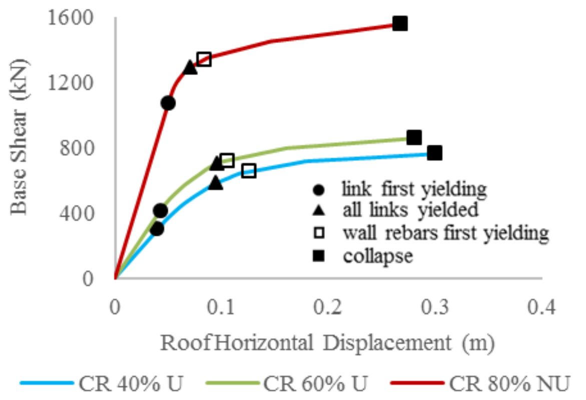

Fig. 5 - Pushover curves for a 6-storey LRS-HCW system

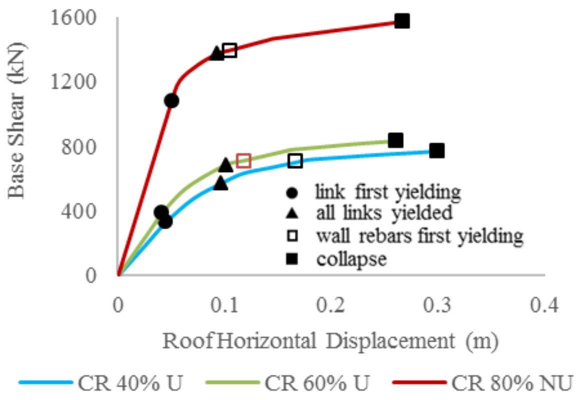

Fig. 6 - Pushover curves for a 6-storey LRF-HCW system

The steel coupling links in both systems successfully redistributed the shear forces in an approximately uniform manner at the point of all links yielding as shown in Fig. 7 and Fig. 8. Due to qualitative similarity, results are only shown for CR = 60% (with uniform link distribution) to avoid overcrowding of data. Fig. 9 and Fig. 10 illustrate the bending moments developed in the links at different storey levels of the LRS-HCW and LRF-HCW system with CR = 60%. "WC" and "CC" refer to the two different connections of a link respectively: the link-to-Wall Connection end and link to-Column Connection. At the point of all links yielding in the LRS-HCW, unequal bending moments were noticed at both ends of a particular link (below the plastic flexural resistance, $M_{p,link}$, of the link section). As shown in Fig. 9, the bending moment at "WC" was always observed to be higher than the bending moment obtained at "CC". This occurred due to the different stiffnesses of the RC wall and the steel side CHS columns. At further stages of the analysis (after the yielding of all steel links and the RC wall), multiple links were found where the moment demand reached $M_{p,link}$ at the link-to-wall connection end, whereas, it stayed well below the link resistance at the link-to-column connection end. Nevertheless, determining the unequal bending moments at each end of the steel shear links at the condition of all links yielding was found to be rather complicated. As a consequence, the maximum possible bending moment which can develop at both ends of the links was assumed equal to $M_{p,link}$. The steel side CHS columns were therefore designed based on this assumption and a certain overdesign of the columns could not be avoided. On the other hand, the LRF-HCW systems ably avoided such an overdesign of the CHS columns. As the links were designed to be intermediate/flexure critical, they yielded in flexure (or simultaneously in shear and flexure), hence producing equal bending moments (plastic flexural resistance of the link, $M_{p,link}$) at both ends of the link: "WC" and "CC". Therefore, the CHS columns as well as the connections could be designed with the appropriate demands. The interstorey drifts for both the LRS-HCW and the LRF-HCW system were also checked. At the point of all links yielding in the LRS-HCW system, the maximum drifts were measured to be 0.56% for CR = 40% U, 0.58% for CR = 60% U and 0.42% for CR = 80% NU. These values stayed compatible with the 1% limit recommended by the EN 1998-1 guidelines [8] for buildings having non-structural elements of brittle materials attached to the structure. Similar agreements were obtained from the LRF-HCW systems as the maximum drifts at the point of all links yielding i.e. 0.58% for CR = 40% U, 0.61% for CR = 60% U and 0.46% for CR = 80% NU, were noted to be less than the Eurocode limit.

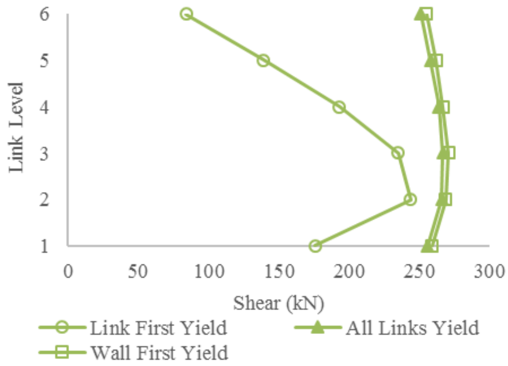

Fig. 7 - Shear in the links for the LRS-HCW with CR 60% and uniform link distribution

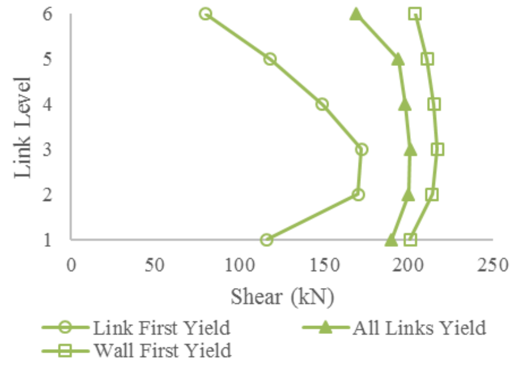

Fig. 8 - Shear in the links for the LRF-HCW with CR 60% and uniform link distribution

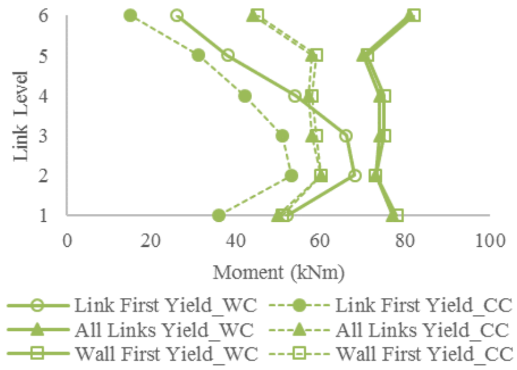

Fig. 9 - Moment in the links for the LRS-HCW with CR 60% and uniform link distribution

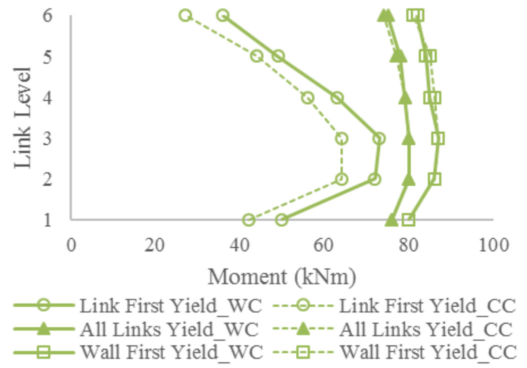

Fig. 10 - Moment in the links for the LRF-HCW with CR 60% and uniform link distribution

4.2 Behaviour of the link-to-RC wall connection.

Both types of configurations, WC-1 and WC-2, were used to design each link-to-wall connection in order to show the alternative approaches (see Table 1). They were evaluated through a nonlinear static analysis (vertical load acting downward at the opposite end of the coupling link incremented until yielding of the link) to validate the global design objective, i.e. yielding of the steel links prior to any or minimal damage in the RC wall and the link-to-wall connection. Detailed results are only shown for the LRS-HCW and LRF-HCW system with CR = 60% and uniform link distribution due to qualitative similarity. The primary objective was achieved, for both WC-1 and WC-2, as the web of the steel links yielded in shear for the LRS-HCW systems (as shown by the von Mises stresses in Fig. 11a and 11b) and in flexure for the LRF-HCW systems (as shown by the von Mises stresses in Fig. 11c and 11d) prior to any yielding in the embedded/passing through profile. The von Mises stresses in the embedded profiles for all case studies maintained an elastic behaviour at the point of link yielding. However, few vertically cracked elements were noticed on the face of the RC wall right above the top flange-to-concrete interface. These cracks originated prior to link yielding and could not be avoided with any reasonable capacity design procedure, due to obvious spalling of the concrete at the aforementioned location. They were thus considered as inevitable minimal damage. These damages could however be reduced by placing a face bearing plate, according to the requirements of DCH rules of Eurocode 8. Few longitudinally (along the RC wall length) cracked elements were observed at the point of link yielding due to gradual "pulling out" of the profile. Table 2 presents the maximum crack widths obtained in the longitudinal and vertical directions at the yielding point of the coupling link for both configurations. The maximum crack widths in all cases remained well below the recommended serviceability limit of 0.3 mm according to EN 1992-1-1, which indicated that no significant damage occurred in the RC wall prior to the link yielding. Hence, the primary design objective was fulfilled.

An additional check was done following the joint classification guidelines proposed in EN 1993-1-8, Clause 5.2.2 [11] to validate if such joints can be assumed as rigid joints in the global structure. Zone 1, Zone 2 and Zone 3 stands respectively for rigid, semi-rigid and nominally pinned. The rotational stiffness ($S_{i,ini}$) of all four abovementioned joints were determined from the numerical models and were checked with respect to their corresponding coupling link properties (k_bEI_b/L_b) where k_b = 8 as the RC wall acts as a bracing system, E is the young's modulus, I_b is the second moment of area of the link and L_b is the span of the coupling link (= l_{link}). The joint showed a rigid behaviour in all the four design cases as illustrated in Fig. 12. This validated the aforementioned global assumption, since the evaluated deformability remains limited enough for the joint to be considered as rigid according to the EN 1993-1-8 definition.

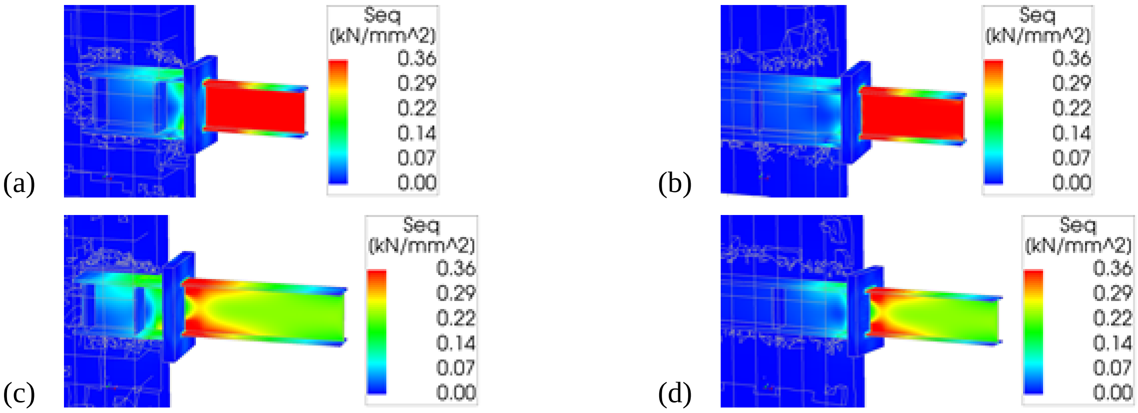

Fig. 11 - von Mises stresses at link yielding for (a) WC_1.2, (b) WC_2.2, (c) WC_1.6 and (d) WC_2.6

Table 2. Maximum crack widths at the load level corresponding to link yielding

	WC-1.2	WC-1.6	WC-2.2	WC-2.6
Max. Longitudinal crack width (mm)	0.093	0.150	0.069	0.069
Max. Vertical crack width (mm)	0.018	0.230	0.210	0.160

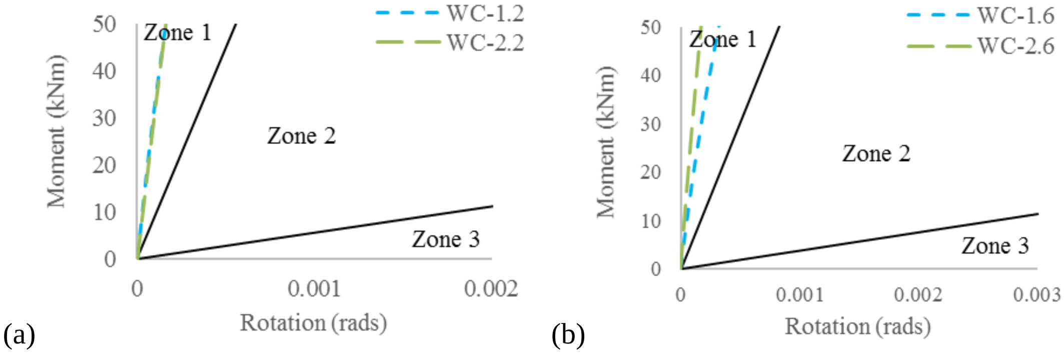

Fig. 12 - Joint classification for (a) WC_1.2 and WC_2.2, (b) WC_1.6 and WC_2.6

4.3 Behaviour of the link-to-CHS column connection.

The design configurations for the link-to-CHS column connections were similarly evaluated through nonlinear static analyses where a vertical load was applied at the left end of the steel coupling link and was incremented until its yielding. As mentioned in the previous sections, detailed results are only shown for the connections designed for the LRS-HCW and LRF-HCW system with CR = 60% and uniform link distribution due to qualitative similarity. As presented in Table 1, CC-1.2 and CC-2.2 were designed for the shear critical links in the LRS-HCW system whereas CC-1.6 and CC-2.6 were designed for the flexural links in the LRF-HCW system. The von Mises stresses for CC-1.2 (Fig. 13a) and CC-2.2 (Fig. 13b) highlight the fact that the design objective is achieved as the web of the link yields in shear prior to any damage in the connections. The front face of the CHS column is deliberately cut to present the stresses in the through parts of the connection. Similarly, the design objective for configurations CC-1.6 and CC-2.6 was also achieved successfully as the flanges of the coupling links yielded in flexure prior to any damage in the LASTEICON connections as shown in Fig. 13c and Fig. 13d respectively. von Mises stresses in the "through" members and the CHS column were observed to be well below the yield limit. Qualitatively similar results were obtained for all the other connection case studies as the primary design objective was achieved in each case.

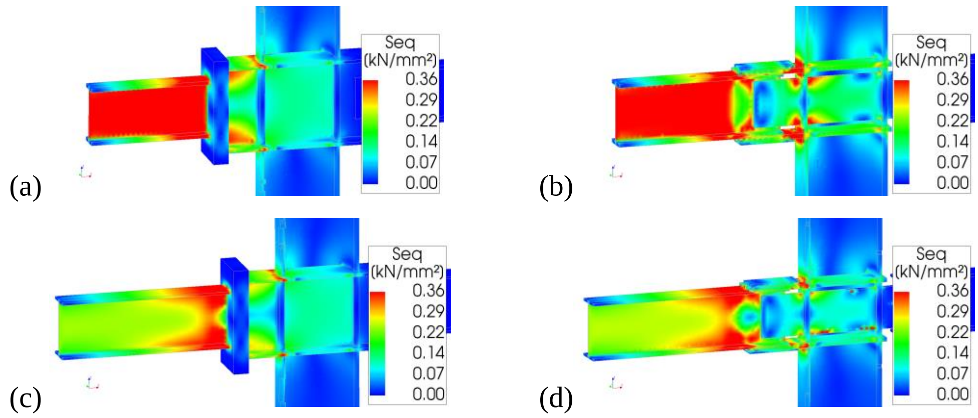

Fig. 13 - von Mises equivalent stresses obtained at link yielding for (a) CC-1.2, (b) CC-2.2, (c) CC-1.6 and (d) CC-2.6 connection

Similarly like the link-to-wall connections, an additional check was done as per the joint classification guidelines proposed in EN 1993-1-8, Clause 5.2.2 [11] to validate if the LASTEICON joints can be truly assumed as rigid joints in the global structure. The rotational stiffness $S_{i,ini}$ obtained from all four design cases are plotted in Fig. 14, where the zones were defined based on $k_b EI_b/L_b$ of the corresponding coupling links. Zone 1, Zone 2 and Zone 3 respectively stands for rigid, semi-rigid and nominally pinned. C1.2, C1.6 and C2.6 were noticed to fulfill the criteria of rigid joints and therefore justified the assumption made in the global HCW system. However, as CC-2.2 displayed a semi-rigid behaviour, the actual moment-rotation properties of this connection was determined and implemented in the global model to check its influence on the structural behaviour. The differences were obtained to be negligible from both a qualitative as well as a quantitative standpoint.

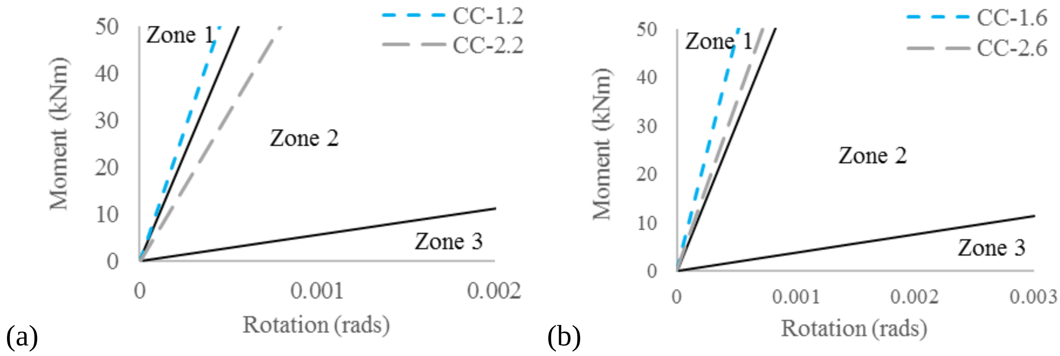

Fig. 14 - Joint classification for (a) CC-1.1 and CC-2.1, (b) CC-1.2 and CC-2.2

Although full-strength joints could be achieved in all cases using either configuration for both the shear and flexure critical links, the “through plates” configuration showed a rather semi-rigid behaviour of the joint when they were applied to shear links. However, such an issue can be easily solved by considering thicker plates, or the “through beam” configuration can be prioritized for the shear critical links.

5. Conclusions

This paper investigates the behavior of a newly introduced steel and concrete HCW system made by a single RC wall coupled to two steel CHS columns by means of steel links through nonlinear static analyses. Encouraging results are obtained, which therefore validate the primary design hypothesis of the coupled wall structures, i.e. the considered HCW can be conceived as a seismic resistant system where the RC wall, steel CHS columns and their corresponding connections with the links remain undamaged, while the seismic energy is dissipated by yielding concentrated in the steel links only. Suitable link-to-wall composite connections and innovative open-to-CHS “passing-through” link-to-column connections are proposed which can allow an effective concentration of seismic damage in the replaceable steel links prior to any or minimal damage in the RC wall as well as the respective connections. Two different types of configurations, namely “partly embedded” and “passing-through”, are discussed for the link-to-wall connections. Relevant design case studies are presented in order to highlight their compatibility in the HCW systems. Full-strength rigid joints could be achieved using either configuration for both the shear and flexure critical links. Similarly, two different types of configurations, namely “through beam” and “through plates”, are discussed for the link-to-CHS column connections. Suitable design case studies are performed.

6. References

[1] El Tawil et al., (2010): Seismic design of hybrid coupled wall systems: state of the art. *ASCE Journal of Structural Engineering*.

[2] Zona A, Degee H, Leoni G, Dall’Asta A, (2016): Ductile design of innovative steel and concrete hybrid coupled walls. *Journal of Constructional Steel Research* 117, 204–213.

[3] Das R, (2019): Earthquake resisting potential of innovative hybrid coupled shear walls with laser-cut open-to-circular hollow section connections, Doctoral Thesis, Hasselt University, Belgium.

[4] Kanyilmaz A, (2019): The problematic nature of steel hollow section joint fabrication, and a remedy using laser cutting technology: A review of research, applications, opportunities. *Engineering Structures*, 183, 1027-1048.

[5] Voth AP, (2010): Branch Plate-to-Circular Hollow Structural Section Connections. Doctoral Thesis, University of Toronto, Canada.

[6] Kanyilmaz A, Castiglioni CA, (2018): Fabrication of laser cut I-beam-to-CHS-column steel joints with minimized welding. *Journal of Constructional Steel Research*, 146, 16–32.

[7] European Committee for Standardization, Eurocode 2: Design of Concrete Structures — Part 1–1: General Rules and Rules for Buildings. EN 1992-1-1, December 2004.

[8] European Committee for Standardization, Eurocode 8: Design of Structures for Earthquake Resistance — Part 1: General Rules, Seismic Actions and Rules for Buildings. EN 1998-1, December 2004.

[9] European Committee for Standardization, Eurocode 3: Design of Steel Structures - Part 1–1: General Rules and Rules for Buildings. EN 1993-1-1, 2005.

[10] DIANA User’s Manual, DIANA Release 10.1, March 2017.

[11] European Committee for Standardization, Eurocode 3: Design of steel structures - Part 1-8: Design of joints. EN 1993-1-8, 2005.

