International Research Journal of Engineering and Technology (IRJET) e-ISSN: 2395-0056
Volume: 07 Issue: 05 | May 2020 www.irjet.net p-ISSN: 2395-0072

A Review on Nonlinear Seismic Response Analysis of an Innovative Steel-and- Concrete Hybrid Coupled Wall System

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Abstract – *This may be a review on the paper which studies the seismic response of an innovative hybrid coupled shear wall (HCW) system, obtained through the connection of a reinforced concrete (RC) wall to two steel side columns by means of steel links. This structural solution is conceived to take advantage of both the stiffness of the RC wall, required to limit building damage under low-intensity earthquakes, and therefore the ductility of the steel links, necessary to dissipate energy under medium-intensity and high-intensity earthquakes. The seismic performance of the proposed HCW system is evaluated through multirecord nonlinear dynamic analysis of a group of case studies designed on the idea of a selected ductile design procedure. The adopted finite-element models are illustrated and validated with experimental tests and more complex three-dimensional numerical models. a variety of results is presented and discussed to spotlight the potential of the proposed innovative HCW systems and therefore the possibility to develop a ductile mechanism during which plastic deformations are mainly attained within the steel links, with minor damages within the RC.*

Key Words: **Dissipative links; Finite-element analysis; Hybrid coupled walls; Seismic analysis; Steel-and-concrete hybrid structures.**

1. INTRODUCTION

The author talks about the good potential of Hybrid steel and concrete structures as seismic resistant solutions if properly designed to take advantage of the stiffness contribution of their ferroconcrete (RC) elements and therefore the dissipative capacity of their steel components. An example of such a synergy between steel and RC is hybrid coupled walls (HCWs), during which two RC walls are connected by steel or composite steel– concrete coupling beams. Steel links allow stable hysteretic behavior and potentially easy repair if the connection between the steel elements and therefore the RC walls is meant for link replacement. Possible alternatives are steel links including a replaceable fuse. This way, HCWs are ready to reduce the seismic risk under manifold points of view, i.e., attention to severe damage and fatalities at the last word limit state also concerns regarding economic loss for more frequent low and moderate seismic events.

The alternative HCW system proposed by Dall'Asta et al. (2015) within the european research project "Innovative hybrid and composite steel-concrete structural solutions for building in seismic area (INNO-HYCO)" to further exploit the potentialities of hybrid steel and concrete solutions was discussed. The proposed HCW system is formed of one RC wall coupled to two steel side columns by means of steel links (Fig. 1). Pinned connections are used between the links and therefore the side columns, ensuring the transmission of shear force only while the connections of the links to the RC wall transfer both shear and bending moment. As a consequence, the side columns are subject to an alternation of compression and traction, with small bending moments thanks to the eccentricity of the link connections and to the lateral displacements, whereas the RC wall is subjected to bending, with alittle and constant amount of axial compression force deriving from permanent loads. Experimental tests on wall-link-column subassemblies (Dall'Asta et al. 2015)[1] demonstrated the effectiveness of the adopted solution and its relevant connections. The varied HCW SYTEMS proposed by Zona et al. (2016)[2] was also discussed.

Accordingly, this paper investigates the seismic performance of the innovative HCW through multirecord nonlinear dynamic analysis. Such an analysis is formed employing a more advanced structural model than the one adopted by Zona et al. (2016)[2], and includes a bigger set of case studies. A frame finite-element model was developed into the open-source finite element software OpenSees (Mazzoni et al. 2007)[3] and validated by comparisons with experimental tests also as three-dimensional finite-element models adopted to explain the local behavior of the steel links. A group of buildings starting from 3 to 12 stories was designed assuming CR varying from 0.4 to 0.8. After preliminary vibrational analyses supported different modelling assumptions, the set of case studies was wont to gain insight into the seismic dynamic behavior of HCWs both at the element level, e.g., local stress and strain within the dissipative steel links and within the RC wall at the varied floors, and at the structural level, e.g., lateral displacements, yielding and damage sequence, and performance at assigned seismic intensities.

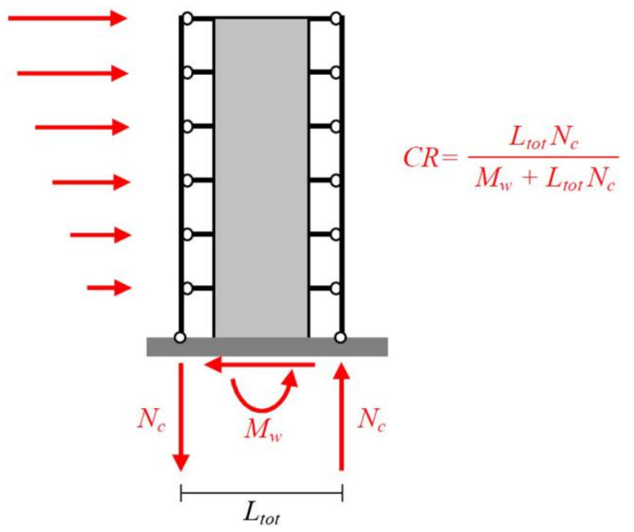

Fig. 1. Innovative hybrid coupled wall.(Zona et.al(2018)]

2. Finite-Element Model of Proposed HCWs

2.1 Modeling of Steel Links

The definition of a model for the steel links in the proposed HCW system benefits from the research studies made for the steel links in eccentrically braced frames (EBFs), given the similarities in their hysteretic behavior, as corroborated by the experimental results of Dall'Asta et al. (2015)[1] compared with those of Hjelmstad and Popov (1983)[4], Malley and Popov (1984)[5], Okazaki and Engelhardt (2007)[6], Dusicka et al. (2009)[7], and Ji et al. (2015)[8].

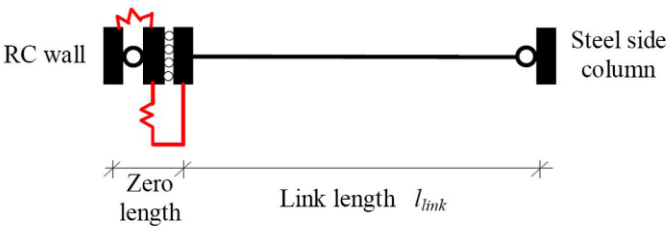

Fig. 2. Steel link model(Zona et.al(2018)]

The modelling approach of Bosco et al. (2015)[9], based on the elastoplastic model introduced by Zona and Dall'Asta (2012)[10] and implemented in OpenSees in its time-discrete implicit formulation (Gu et al. 2014)[11], was shown to produce accurate results for a wide range of link geometries with a proper description of the isotropic hardening that characterize the link hysteretic behavior. For this reason, the Bosco et al. model was used as a starting point for the definition of a specific link model (Fig. 2 and Table 1) in this study.

Table-1: Components of hysteretic steel link model

Behavior	Bending	Shear
Elastic range	Euler-Bernoulli linear elastic beam	Zero-length element with elastoplastic law
Plastic range	Zero-length element with rigid-plastic law	

A linear elastic Euler-Bernoulli beam element with length llink describes the elastic axial and flexural behavior of the link. The plastic flexural response and also the elastoplastic shear response are lumped at the link end connected to the RC wall through the utilization of rigid-plastic zero-length elements available in OpenSees (Mazzoni et al. 2007)[3]. Both the flexural and shear hysteretic responses are described through the elastoplastic model (Zona and Dall'Asta 2012)[10] that needs the subsequent constitutive parameters: initial stiffness, yielding force, maximum force for asymptotically fully developed hardening, postelastic stiffness, elastic-to-plastic transition shape parameter α, hardening rate
parameter δr.

The rigid-plastic flexural model uses a sufficiently high value of the initial stiffness so as to possess negligible elastic rotations without causing numerical instabilities, whereas the yield and maximum bending moments are often computed on the idea of the link cross section properties. The elastoplastic shear model uses the linear elastic shear stiffness of the link, whereas the yield and maximum shear forces are often determined from the link properties. In both the flexural and shear models the opposite constitutive parameters, i.e., postelastic stiffness, elastic-to-plastic transition shape, and hardening rate, are often easily calibrated supported experimental results also as more complex three-dimensional finite-element simulations ready to reproduce the experimental response results (Bosco et al. 2015)[9].An example is additionally illustrated within the paper.

2.2 Modeling of RC Wall

The paper discusses about the behavior of RC walls subjected to lateral loads is notably influenced by the ratio, i.e., the ratio between the general height Hw and therefore the base length lw, as for instance discussed by Kolozvari et al. (2015)[12]. For slender walls like those within the proposed HCW system, i.e., designed to avoid shear collapse and with sufficient lateral deformability to permit link yielding (Zona et al. 2016)[2] leading to a suggested ratio adequate to 10, the structural behavior is dominated by flexure. Variety of models for flexure-controlled RC walls are found within the literature.

Among them, force-based frame elements with the so-called fiber cross section integration are often presented because the best modeling strategy (e.g., Pugh et al. 2015)[13]. For this reason, this study used the force-based distributed-plasticity fiber frame element available in OpenSees to explain the flexural behaviour of the RC wall of the considered innovative HCW system. The shear behavior of the RC wall elements was modeled as linear elastic by aggregating to the flexural stiffness of the section an elastic initial stiffness adequate to Gc, Av, where Gc is that the elastic tangential modulus of the concrete, and Av is that the shear area, evaluated as 5/6 times the world of the oblong cross section.

An example of calibration and validation of the adopted RC wall model was shown using as benchmark the experimental tests for the downscaled 4-story wall specimen RW2 reported by Orakcal et al. (2006)[14]. The preliminary calibration of the fabric parameters was conducted on the idea of the experimental stress strain relations measured during the specimen construction. The Chang and Mander model as implemented in OpenSees (Mazzoni et al. 2007)[3] was used for the confined and unconfined portions of the concrete cross section, whereas the steel reinforcements were modeled with the Zona and Dall'Asta (2012)[10] model with k1=3,700 MPa, δr= 5.0, and α=0.485. A mesh convergence study was performed with the target of evaluating the optimal number of elements and integration points within the force-based frame element. Uniform meshes of 4, 8, and 16 elements were wont to describe the wall specimen and 5, 7, and 9 integration points were utilized in each element.

2.3 Modeling of Hybrid System

The global model was obtained by assembling the only components models previously described. Elements were placed in their actual positions and connected by means of rigid elements to elucidate the eccentricity of the connection systems. Beam elements with null flexural stiffness connected the HCW system to an equivalent elastic column that simulated the presence of the gravity-resistant frame. Mass related to the self-weight of structural elements was lumped at the highest nodes of each element, and mass related to the superimposed permanent loads and live loads was applied to the equivalent columns according to the tributary area. Damping apart from that determined by the hysteretic behaviour of materials was modeled using the Rayleigh model with the damping matrix proportional to the mass matrix and initial stiffness matrix so on possess a 5% damping factor for the first two vibration modes.

3. Seismic Response Analysis

3.1 Parametric Study

This paper presents a parametric study so as to research the seismic behavior of the proposed innovative HCW system through nonlinear dynamic analysis using as seismic input a group of natural accelerograms. Two parameters were varied, i.e., the building height and therefore the design CR. Four buildings heights, i.e., 3, 6, 9, and 12 stories with interstory height adequate to 3.50 m, were considered so as to spotlight differences within the seismic behavior of the proposed hybrid system for low-rise to medium-rise buildings, covering the potential range of application of HCWs. the planning used three values of the CR, i.e., 0.40, 0.60, and 0.80, so as to explore how this design parameter influences the seismic dynamic behavior and possibly to point its optimal value.

3.2 Design of Case Studies

The design of the considered case studies (12 buildings) started from the plant configuration of a residential building selected as a benchmark project in Dall'Asta et al. (2015)[1]. The gravity-resistant steel frame (floors, beams, and columns) had pinned beam-to-columns joints and base restraints. Beams and columns of the gravity-resistant frame were designed consistent with Eurocode 3 using steel grade S275 (nominal yield stress fy=275 MPa) and a limitation to the vertical deflection at service limit state adequate to L=250, where L is that the beam span length. the ground was assumed made from corrugated sheets and a concrete slab of maximum thickness 130 mm and minimum thickness of 65 mm, hence the ground are often considered as a rigid diaphragm. The characteristic values of the dead loads Gk (structural and non structural elements) were 4.30 kN=m2 for the ground type, 3.30 kN=m2 for the roof floor, and 4.75 kN=m2 for the steps . The superload Qk was 2.0 kN=m2 for every floor and 4.0 kN=m2 for the steps . Dall'Asta et al. (2015)[1] gave details of the planning of the gravity-resisting frame. The gravity-resistant frame was connected to the HCW systems (two systems for every direction) that were the sole components providing lateral resistance against horizontal actions. Links and side columns were made from S355 steel (nominal yield stress fy = 355 MPa). C30 concrete (characteristic compressive strength fck =30 MPa) and B450C reinforcements (characteristic yield stress fyk =450MPa) were used for the ferroconcrete wall.

Ground A of Eurocode 8 with design peak ground acceleration (PGA) ag = 0.20g was considered because the seismic input. The seismic mass of every floor was 316 kNs2=m which of the roof was 264 kNs2=m, both computed consistent with Eurocode 8. The ductile design procedure specifically developed for the innovative HCWs by Zona et al. (2016)[2] was used for dimensioning the structural elements involved within the seismic-resistant mechanism.

The dissipative steel links were designed with a consistent distribution with the cross sections selected among the commercial European IPE hot-rolled profiles, all of which were compact cross sections in bending (Class 1 consistent with Eurocode 3). Eurocode 8 link classification for eccentrically braced frames was considered and therefore the design was performed by enforcing the selection of short or intermediate links thanks to their more stable hysteretic behavior compared with long links.

Commercial HE profiles with compact cross sections in compression, i.e., Classes 1 and a couple of consistent with Eurocode 3, were adopted for the side steel columns. However, within the case studies with a better number of stories and a better CR, thanks to the many increase of the planning axial force (capacity design enforces over strength within the side columns), more-efficient hollow circular sections were used. The link-to-wall connections and link-to-side column connections were designed to avoid damage within the connection by using the capacity design consistent with Eurocode 8. On the idea of the solutions studied by Dall'Asta et al. (2015)[1], a splice reference to threaded bushings was adopted within the wall side, whereas a double angle bolted connection was used between the link and therefore the steel column

3.3 Definition of Model Material Properties

The materials within the finite-element model were defined using the mean values of their constitutive parameters consistent with the Eurocode safety format for nonlinear analysis of Eurocode 2 and Eurocode 3.

3.4 Vibration Analysis

Prior to the nonlinear dynamic analyses, a vibration analysis was performed so as to guage the dynamic properties within the linear behavior range of the designed case studies. The eigen analysis was supported the structure inertial properties derived from the planning loads, i.e., self-weight and permanent and live loads. The linear elastic stiffness properties of the steel elements were those derived from the steel coefficient of elasticity under three hypotheses for the RC wall: (1) non cracked concrete with coefficient of elasticity adequate to the mean coefficient of elasticity computed consistent with Eurocode 2, i.e., Ec = 29,064 MPa; (2) cracked concrete with a conventionally reduced coefficient of elasticity adequate to 0.50Ec; and (3) cracked concrete with bending stiffness of the RC wall determined at the bottom of every story from the initial slope of the moment-curvature relation with axial force determined by the vertical loads at the seismic load combination. within the latter case, the determined reduced coefficient of elasticity was about 0.42Ec for the 3-story cases, 0.32Ec for the 6-story cases, 0.27Ec for the 9-story cases, and 0.25Ec for the 12-story cases.

Differences between the varied levels for an assigned building height were limited. The natural period of vibration and modal participating mass ratio for the horizontal direction were computed for rock bottom three vibration modes. The lowest three vibration modes were typical lateral modes of cantilever-type structures, and their modal participating masses within the horizontal direction cumulatively account for 100% within the 3-story buildings, about 95% within the 6-story buildings, about 93% within the 9-story buildings, and about 92% within the 12-story buildings.

Differences within the first vibration period between the 2 cracked models generally range from about 3 to six, with two exceptions being the 9-story and 12-story cases with CR 0.60, during which the discrepancies reach about 11%.On the opposite hand, the differences within the predicted playing period between the cracked and non cracked models generally range from about 14 to twenty and increase to about 27% for the 9-story and 12-story cases with CR 0.60. Regarding the influence of the CR within the vibration properties, a non negligible effect with lower periods for increasing CR was observed, needless to say, being the resulting design characterized by higher lateral stiffness.

In order to match the results obtained within the design of the innovative HCW to those achievable using conventional RC walls, the same RC cantilever wall decided for every case study. The equivalence between the 2 walls was defined as having an equivalent elastic lateral stiffness measured by the amount of the primary vibration mode. The wall thickness bw was kept equal between the hybrid solution and therefore the RC wall while only the length lw was varied.

Except for the 3-story case with lower CR, the rise within the wall length was always significant when a RC-only wall was considered rather than the coupled system. the equivalence in terms of the primary modal period doesn't imply equivalence in term of overturning moment. so as to qualify this aspect, the resisting moment MRCW of the stiffness equivalent RC cantilever wall, with reinforcements designed and detailed following an equivalent DCM rules of Eurocode 8 as used for the wall of the HCW system, was compared with the resisting moment MRCW of the HCWs.

Apart for the 3-story case, for the lower CR the RC wall and HCW system had similar resisting moments. On the opposite hand, difference were quite important for the upper CRs during which the resisting moment within the RC wall was significantly less than the resisting moment of the HCW system, despite the substantial increase of lw; RCW. Nevertheless, the 2 structural systems had very different mechanisms in terms of seismic-energy dissipation and relevant developed damages.

3.5 Multirecord Dynamic Analysis at Design Seismic Level

Nonlinear dynamic analyses were performed using as seismic input a group of 30 natural ground motion records from the ecu Strong Motion Database scaled to match, on the average, the Eurocode 8 Type I soil with A pseudo-acceleration response spectrum with PGA =0.20g using REXEL software Response results were computed for every accelerogram and their extreme values were afterward identified and averaged over the 30 accelerograms.

The results shows that The designed HCWs had good lateral stiffness at the planning seismic input, with roof displacements always below H=200 and interstory drift always below 1%.The normalized lateral displacements and interstory drifts decrease with the amount of stories because displacements increase but proportionally with the building height. Interstory drifts within the upper two thirds of the building height tend to be uniform for the lower values of CR. All links yielded in bending within the HCW systems designed with CRs 0.40 and 0.60 apart from the last floor of the 3-story CR 0.60 case. No yielding in shear was activated. The shear force and bending moment for CRs 0.40 and 0.60 were basically uniform over the peak, according to the planning assumption of uniform link distribution and with the results obtained from pushover analysis.

A significantly different state of stress was attained within the steel links within the cases designed with CR adequate to 0.80. In these cases, the shear forces. Acceleration response spectra of spectrum-compatible natural records.. Lateral displacements averaged over 30 accelerograms. and bending moments decrease with the peak and only the links of a couple of lower stories yielded within the 9-story and 12-story buildings. Accordingly, the uniform link distribution over the building height assumed within the design doesn't appear to be the foremost effective option, and a non uniform distribution should be preferred. Additionally, during this case the results of the nonlinear dynamic analysis accept as true with the pushover analysis results. The uniform distribution of the bending moment within the links of the HCWs designed with CR adequate to 0.40 and 0.60 was reached thanks to post elastic stress redistribution. Such redistribution requires a plastic rotation within the steel links that increases significantly when a lower CR is adopted and when the peak of the building is increased.θp is normalized with reference to the last word value θpR determined ccording to Eurocode 8 point 6.8.2(10). As a consequence, the planning of upper buildings with lower CRs could produce excessive rotations within the steel links, as was the case of the 9-story and 12-story designs with CR 0.40.

It was confirmed that the planning objective requiring no damage within the RC wall was satisfied, whereas just some yielding within the reinforcements at the wall base was attained within the 3-story case with CR= 0.80.The other design objective aiming at avoiding any damage within the side steel columns was satisfied also , i.e., side columns designed to possess over strength with reference to the condition of all links yielded in shear, a condition that wasn't attained at the considered design seismic level.

Nevertheless, the tension/compression axial force within the columns are often very large when high values of CR are adopted, and this impacts both the planning of the columns and therefore the design of the foundations. the difficulty of the planning of the steel columns for giant axial forces are often handled through the adoption of proper steel sections having an efficient behavior when compressed, e.g., hollow sections rather than I-sections, as was the case of the upper CRs within the 9- and 12-story buildings of this study. More critical is that the issue of the planning of the foundations when high values of CR are used, because the substructure has got to handle uplift, which is sort of perilous from a seismic geotechnical point of view as long as net tension results in loss of shear capacity of the soil. Thus, high CRs have significant repercussions on the substructures, requiring large and stiff foundations to be verified against uplift also as compression.

3.6 Incremental Multirecord Dynamic Analysis

In order to supply more insight into the nonlinear seismic behaviour of the proposed HCWs, an equivalent case studies were analyzed for an equivalent set of 30 accelerograms multiplied by an element varying from 0.1 (PGA 0.02g) to 3.5 (PGA0.70g). during this way it had been possible to get information about the performance of the HCWs for occurrence probabilities of the seismic events aside from the one assumed for the planning , i.e., return period TR= 475 years for PGA0.20g, and to realize information about the sequence of injury activation within the steel and RC structural components. As previously mentioned, no deformation limits were introduced for the materials. However, the incremental analyses were terminated if the utmost concrete strain averaged over the 30 accelerograms reached the last word concrete strain before PGA0.70g was attained. The adopted analysis protocol assumed that the averaged response results were computed conditional to survivals.

4. Conclusions

This paper studied the seismic performance of an innovative hybrid coupled wall system made by one RC wall coupled to 2 side steel columns by means of dissipative steel links. to the present end, a group of case studies was designed by employing a recently proposed design procedure that aims at activating hysteretic dissipation altogether the steel links while the RC wall remains undamaged. Two parameters were varied within the design of the case studies, i.e., the building height and therefore the coupling ratio, i.e., the ratio between the instant resisted by the 2 side columns and therefore the total resisted moment.

Four buildings heights were considered, i.e., 3, 6, 9, and 12 stories, and for every building height three values of the CR were utilized in the planning , i.e., 0.40, 0.60, and 0.80, exploring the potential range of application of HCWs in low-rise to medium-rise buildings with low, medium, and high CRs. A computationally efficient finite element model capable of simulating the nonlinear cyclic behaviour of its steel and RC elements was developed. Nonlinear dynamic analyses with a group of natural ground motions as seismic input were wont to monitor variety of response quantities describing the worldwide behavior of the proposed HCW system and therefore the local behaviour of its components.

The main results of the paper are as follows:

• within the designs made with CRs 0.40 and 0.60, all the steel links yielded while the RC wall was still in its elastic range;

• within the designs made with CRs 0.40 and 0.60, the resulting stress distribution within the steel links was basically uniform along the building height as a consequence of a big redistribution of forces that, however, required rotations that within the taller buildings (9- and 12-story cases) and lower CR (0.40) exceeded the link rotation capacity;

• When the designs were made with CR 0.80, the obtained HCWs were stiffer and unable to yield all steel links before yielding the RC wall; thus, the planning objectives weren't completely met albeit the system was ready to withstand the planning seismic input also as higher accelerations;

• Regardless the worth of CR adopted within the analysis, failure of the RC wall either in bending or in shear was reached for seismic intensities significantly above those considered within the design; thus, the planning objective aiming at a dissipative behaviour that excludes collapse within the RC wall was met

• The side steel columns remained in their elastic range and much from buckling no matter the adopted CR and building height; thus, the planning objective of side columns undamaged was satisfied.

The results indicate that the proposed HCW has promising potential as a seismic resisting system ready to effectively exploit the lateral stiffness of the RC wall and therefore the dissipation capacity of the steel links. The utilization of CR adequate to 0.60 appears to be the foremost suitable option to provide good ductile behavior without excessive deformation demand within the dissipative elements (steel links) and without large over strength demand within the non dissipative components (RC wall and side steel columns).

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