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Procedia Engineering 199 (2017) 3516–3521

Procedia Engineering

www.elsevier.com/locate/procedia

X International Conference on Structural Dynamics, EURODYN 2017

Performance-Based Seismic Design Of An Innovative HCW System With Shear Links based on IDA

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Abstract

Structural members yielding in shear are used in earthquake resistant systems, such as eccentrically braced steel frames and systems with passive energy dissipation devices, in a conscious effort to concentrate the energy dissipation capacity of the structure in components that can be repaired or replaced after a major earthquake. This paper presents an improved approach to a newly suggested design procedure of an innovative Hybrid Coupled Shear Wall (HCW) system consisting of a RC shear wall coupled with steel side columns via dissipative steel shear links. The primary design objective is to reduce or possibly avoid the damage in the RC wall while concentrating the seismic damage to the replaceable steel links which are shear critical, i.e. intended to fail in shear rather than flexure. To this purpose, a performance based approach is followed considering several limit states such as IO (Immediate Occupancy based on a limitation of the interstorey drifts), LS (Life Safety with yielding of the link, restoration possibilities of the links and limitation of the damages in the RC wall) and CP (near collapse situation with possible significant damages in the RC wall). The proposed design procedure is applied to several case studies which are analysed through nonlinear static and incremental dynamic analyses using finite element models in order to optimize the respective contribution of the wall, side columns as well as the links in terms of strength, stiffness and energy dissipation capacities.

Peer-review under responsibility of the organizing committee of EURODYN 2017.

Keywords: *Steel and concrete hybrid structures; Shear links; Energy dissipation; Seismic-resistant structures; Performance based design*

1. Introduction

In recent years, coupled walls have proved to be a very interesting solution for the design of seismic-resistant structures, starting from reinforced concrete (RC) walls coupled by RC beams [1-4] to more recent steel and concrete

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Peer-review under responsibility of the organizing committee of EURODYN 2017.
10.1016/j.proeng.2017.09.500

hybrid coupled walls (HCWs) where coupling between RC walls is achieved through steel or steel-concrete composite beams [5], which are often organized as structural fuses that can be easily substituted when damaged. However, due to some drawbacks, the combination of the RC wall and steel elements was thus improved into a new innovative HCW system in the INNO-HYCO research [6]. This system consists of a RC shear wall with dissipative steel links and steel side columns. 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 side steel columns. Although the ZDLD (Zona-Degee-Leoni-Dall'Asta) design approach [7] has provided promising results, the links were designed to yield both in shear as well as flexure. According to the previous researches, shear links provide better energy dissipation characteristics than flexural links and function primarily as a metallic yielding device (fuse), limiting the maximum lateral force that can be transmitted to primary structural members thus providing significant energy dissipation potential. Therefore, in this paper, the emphasis is upon developing a simple procedure to design the steel links as shear critical elements (SCSL) so that; the RC wall remains in the elastic range while the steel links connected to the wall be the primary dissipative elements and undergo ultimate shear deformations prior to any damage in the RC wall. As Incremental Dynamic Analysis (IDA) provides a more in-depth indication of seismic structural response as actual ground motion records are utilized, a performance based approach based on IDA was further implemented to verify the suitability of the structure towards the desired design objectives.

2. Design Methodology

The design procedure proposed [7] by the authors in the INNO-HYCO project is followed and modified to design the coupling links as shear critical. Step 1, 2, 3, 5 and 6 are all adopted without any variations from the ZDLD design procedure [7], whereas, the newly derived modifications are introduced as step 4.

***Step 1**:* Dimensions of the RC wall are chosen by selecting suitable height-to-length ratio H_w/l_w and thickness b_w as suggested in the INNO-HYCO project [6]. The longitudinal reinforcements are chosen based on EC 8 [8] DCM rules.

***Step 2**:* A Coupling Ration (CR) is chosen initially to study the response of the structure through a parametric analysis. This CR is then varied to acquire a more detailed information about the structural behaviour of the coupling system.

***Step 3**:* If the CR is chosen, the axial force N_c is therefore derived as described in the ZDLD design [7]:

$$N_c = \frac{M_c}{L_{tot}} = \frac{CR}{1-CR}\frac{M_w}{L_{tot}} = \frac{CR}{1-CR}\frac{M_{w,Rd}}{L_{tot}\gamma_w} \quad ..\{1\}$$

where M_c is the moment resisted by the two side columns, L_{tot} is the length of the RC wall plus twice the link length, γ_w is a safety coefficient used to limit or possibly avoid damage in the RC wall by reducing its bending moment capacity [7] and is equal to 1.5. Thus, the shear force demand on the links can be calculated as:

$$V_{link,i} = \psi_i \frac{N_c}{n_{links}} = \psi_i \frac{1}{n_{links}} \frac{CR}{1-CR} \frac{M_{w,Rd}}{L_{tot}\gamma_w} \quad ..\{2\}$$

Where n_{links} is the no. of links on each side of the RC wall and ψ_i is a distribution coefficient. Two different values of ψ_i are used and compared, namely, uniform distribution (similar sections are used throughout all the height level of the structure):

$$\psi_i = 1$$

and non-uniform distribution (different sections are used for different height levels of the structure):

$$\psi_i = \begin{cases} 1+\frac{2}{3}\ for\ z_i \le \frac{N}{3}; \qquad 1\ for\ \frac{N}{3} < z_i \le \frac{2N}{3}; \qquad 1-\frac{2}{3}\ for\ \frac{2N}{3} < z_i \le N \end{cases} \quad ..\{3\}$$

Geometric nonlinear effects are controlled with the amplification of seismic loads according to EC 8 Part 1, paragraph 4.4.2.2, equation 4.28. Now, the link design resistances in bending and shear are computed according to Eurocode 8.

$$M_{p,link} = f_y b t_f (d - t_f);\ V_{p,link} = \frac{f_y}{\sqrt{3}} t_w (d - t_f) \quad ..\{4\}$$

***Step 4**:* Links can be designed as shear critical in three ways: using larger sections; decreasing link length; or increasing the flange thickness of the built-up I profile. As the first option proves to be rather uneconomical, a new relationship using both the later options was developed based upon W. S. Park and H. D. Yun's research work [9] to ensure greater energy dissipation. Therefore, built-up I profiles (BuIP) were adopted for the steel links. The appropriate slenderness ratios were also taken into account to safely design the links against local flange or web buckling. While a link section is chosen based upon the moment and shear demands, the required section modulus, Z_{req}, is determined and checked to ensure the steel coupling beam is shear critical. Therefore, the section should be chosen to develop a moment resistance, M_s, greater than or equal to the moment corresponding to the development of strain hardening in shear.

$$Z_{req} \geq \frac{M_s}{\theta_b f_y}; \quad M_s = 1.35 V_{p,link} l_{link} \quad ..\{5\}$$

Where l_{link} is the link length (from the face of column to the face of RC wall therefore considering eccentricities), θ_b is the material reduction factor typically taken as 0.9 and the factor 1.35 accounts for the development of strain hardening in the web of the steel coupling link [9]. Thus, to make the link shear critical, $M_{p,link}$ should be equal to M_s and following this concept a relationship can be derived as stated in Eq. 6.

$$\frac{bt_f}{t_w} = \frac{1.35 l_{link}}{\sqrt{3}}; \qquad e \leq L_{link} = 0.74 \frac{M_{p,link}}{V_{p,link}} \quad ..\{6\}$$

Link section and link length can be finalized as per this relationship. Also, following these derivations, a new factor of 0.74 was found (Eq. 6) and thus used to define a short link, which was 7.5% lesser than the factor of 0.8 recommended by Eurocode 8 as mentioned earlier. The following checks are required to ensure sufficient resistance of both the flange and web of the built up steel coupling link sections against local buckling.

$$\frac{b}{t_f} \leq 0.31 \sqrt{\frac{E_s}{f_y}} \ ; \quad \frac{H}{t_w} \leq 3.05 \sqrt{\frac{E_s}{f_y}} \quad ..\{7\}$$

Where b and t_f are the width and thickness of the steel coupling link flange, and H and t_w are the total height and thickness of the steel coupling link web, respectively.

Step 5: The steel side columns at the *i*-th floor are designed using the summation of the yield shear forces of the links:

$$N_{c,Ed} = 1.1 \gamma_{ov} \sum_{i=1}^{n_{links}} V_{link,i} \quad ..\{8\}$$

with $\gamma_{ov} = 1.25$. The effect of the eccentricity between the column axis and the shear connection between the link and the column should also be considered in the design.

Step 6*:* The design of the wall transverse reinforcements as suggested by the previous authors [7] is made to provide a shear resistance $V_{Rd,w}$ in the wall that exceeds the maximum shear $V_{pl,w}$ (estimated base shear):

$$V_{Rd,w} > V_{pl,w} = \frac{1}{H_1} \left[M_{w,Rd} + 1.1 \gamma_{ov} (l_w + 2 l_{link}) \sum_{i=1}^{n_{links}} V_{link,i} \right] \quad ..\{9\}$$

where H_1 is the resultant height of the fundamental mode inertial force distribution or a fundamental-mode based equivalent lateral force distribution.

3. Description of the Case Study

A 6 storied structural system was investigated. Seismic design loads were calculated in accordance with EC 8. The system was assumed to be a part of a residential building for the seismic design considerations. The interstorey height is taken to be 3.50 m and the HCWs are designed for 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. A previously recommended behaviour factor, q = 3 [7] is considered in the present designs. Concrete for the RC wall is taken as class C30 and reinforcements are taken to be B450C following the Eurocode 2 [10] guidelines. Reinforcements are designed according to the DCM rules stated in Eurocode 8, Clause 5.4.3.4 for ductile walls. Steel links were designed as Built-up I Profiles (BuIP). Steel grade S355 is adopted for both; coupling links and steel side columns. The links lengths are chosen through a path of trial and error, depending upon their mechanical properties and their compatibility towards being short links. Columns are chosen from European HE profiles (Class 1 or 2 sections in compression according to Eurocode 3 [11]). Lastly, the shear design of the RC wall is carried out for the base shear estimated from Eq. 9. Therefore, in relation to the total height of the building (H) = 21 m and the suggested ratio, H/l_w = 10, the length of the RC wall section is calculated to be 2.1 m (l_w). The width is primarily assumed to be 0.36 m as shown in Fig. 1.

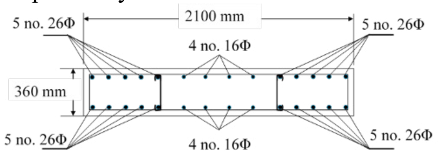

Fig. 1. Detailing of the reinforcements designed for the RC wall of the 6 storied building

The HCW systems are modelled and analysed in SAP 2000 [12]. The RC wall, steel shear links and the steel side columns are modelled using linear elastic frame elements. Plastic flexural hinges are introduced at both ends of each element (between two subsequent floors) of the RC wall. Plastic shear hinges are introduced at the midpoint of the links. As maximum bending moment occurs at the link end fixed to the shear wall, plastic flexural hinges are introduced at those connection points only to verify that the sections yield in shear prior to flexure. Eccentricities of the connections are incorporated through links provided by SAP 2000. The steel coupling links are assumed to be pinned to the face of the steel side columns and are fixed to the face of the RC wall. The base of the steel side columns are pinned to the ground, whereas the base of the RC wall is fixed. Nonlinear direct integration time histories with isotropic hysteresis behaviour of the plastic hinges were adopted for the nonlinear dynamic analyses.

4. Performance Assessment Based On Pushover Analysis and IDA

The pushover curves corresponding to the lateral loads proportional to the first modal deformation reporting the base shear versus roof horizontal displacement for the 6 storied structure are shown in Fig. 2. In each curve four events are marked. The all links yielded case varies significantly with the CRs. For CRs ≤ 60%, all the coupling links yield with reinforcements of the RC wall still staying in the elastic range, whereas, for CRs > 60%, the reinforcements yield prior to the yielding of all links. So, the higher CRs are characterized by damage in RC walls before all links start to dissipate seismic energy through plastic deformation. This situation contradicts with the design objectives. Thus a nonuniform distribution of steel links were adopted as per previous recommendations [7] for coupling ratios greater than 60%. This provided an effective solution as shown in the pushover curves in Fig. 2b. The design objectives can be fulfilled for a CR equal or below a specific value with the uniform link distribution. This can be called as the critical limit of coupling ratio. Thus, the critical limit of Coupling Ratio (CR) for uniform link distribution of a 6 storied building is obtained as 60%. Pushover curves for the uniform link distribution of the 6 storied structure are shown in Fig. 2a for CRs 40%, 60%, and 80% and curves illustrating the comparison between the uniform and nonuniform link distributions for CR 80% is shown in Fig. 2b. The all links yield displacement (ALY displacement) was obtained from the pushover curves, to be 0.112m, 0.09836m and 0.06736m for the 40%, 60% (uniform) and 80% (non-uniform) structure respectively and is considered to be the performance displacement.

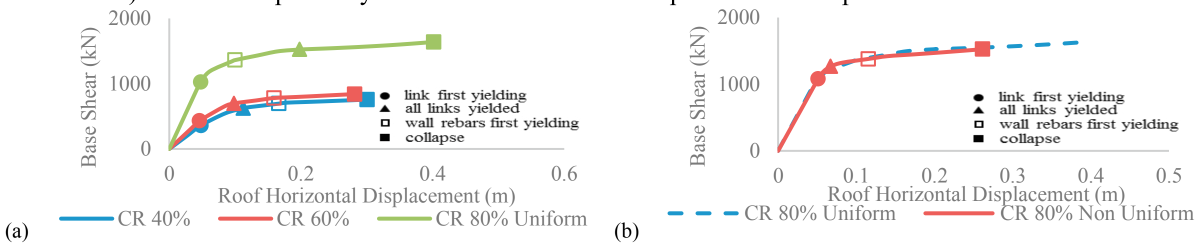

Fig. 2. Pushover curves for (a) CR 40%, 60%, 80% using Uniform link distribution; (b) 80% uniform and non-uniform link distribution

Two artificial time histories LT1 (PGA = 2.64 m/s2) and LT2 (PGA = 2.45 m/s2) and eleven natural time histories (TH1 to TH11) were used as seismic input, preliminarily scaled to match the EC8 spectras for their corresponding soil type and amplified with scale factors (SF) 0.33, 0.67, 1, 1.33, 1.67, 2, 2.33, 2.67, 3, 3.33, 6.67, and 10 for the IDA. The natural ground motion records were taken from Rexel [14]. The ground motions were chosen based on magnitude, distance from the nearby fault, and site conditions. Selected ground motions were recorded at 5-27 km from the closest point of the fault rupture. Moment magnitudes (M_w) of these earthquakes vary from 6.4 to 7.1. The average IDA curves for eleven natural and two artificial accelerograms were highlighted in Fig. 3a, Fig. 3b, and Fig. 4 for CR 40%, 60% and 80% (with nonuniform distribution of links) respectively to study their comparison with the relevant pushover curves. The vertical dotted line in the subsequent graphs represents the corresponding performance displacement at which all links yield (ALY displacement) in that particular structure. Since a successful application of the designed shear links largely depend on the ability to function most effectively as an energy dissipator, energy dissipation capacity is the primary means of measuring the performance of these links. Fig 5 illustrates the comparison of hysteretic behaviour between the structures designed as per ZDLD approach [7] and the shear critical approach. The area enclosed by the curve for the latter case is always greater than that of the previous one. The hysteretic behaviour is also observed to be more stable in the latter case.

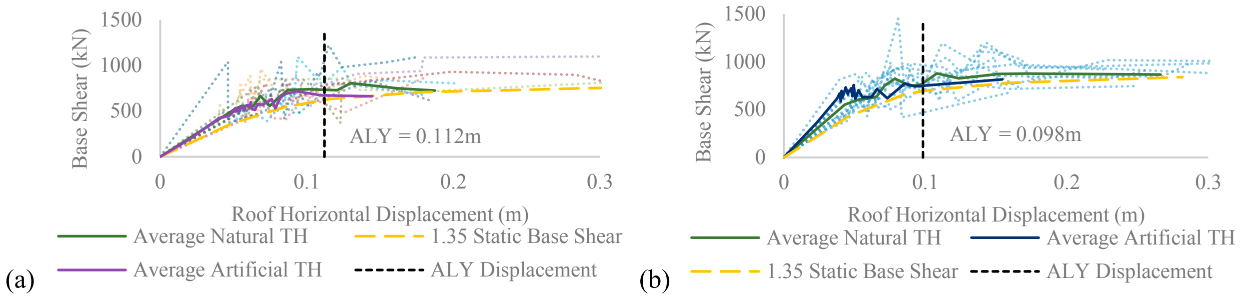

Fig. 3. Artificial and natural IDA curves comparison with the static pushover curve for (a) CR 40% and (b) CR 60% using Uniform link distribution

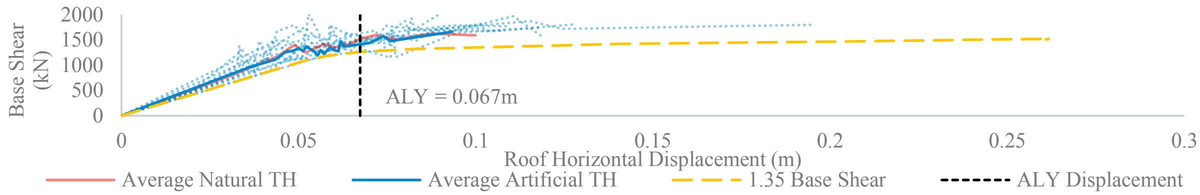

Fig. 4. Artificial and natural IDA curves comparison with the static pushover curve for CR 80% using Non-Uniform link distribution.

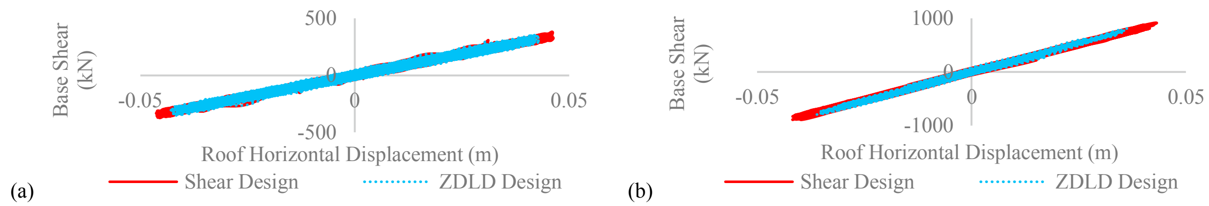

Fig. 5. Comparison between hysteretic loops (a) for CR 40% (uniform) and TH7; (b) for CR 80% (non-uniform) and TH9 Time history (SF=1).

Fig. 6a and 6b demonstrate the maximum link rotations (average from all amplified THs) in all the shear critical steel links for structures with 40% (uniform) and 80% (non-uniform) link distributions w.r.t their corresponding max roof horizontal displacements. The rotation values till the ALY displacement were compared with the FEMA 356 [13], Table 5-6 guidelines to verify their suitability for the proposed design procedure and it was observed that the rotations in each links were well below the LS limit (Life Safety). For example, the maximum rotation till the ALY displacement is obtained as 0.0078 rad and 0.0084 rad for 40% and 80% CR respectively to acceptance limits of 0.005 rad (IO), 0.11 rad (LS) and 0.14 rad (CP). Also, when compared with AISC 341-16 [14], Clause F.3.4a., it is observed that the maximum rotation in each links stays well below the recommended acceptance limit of 0.08 rad.

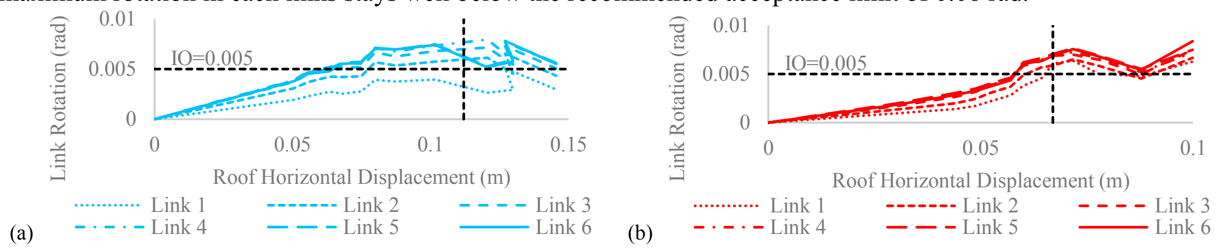

Fig. 6. Maximum Link Rotations for (a) CR 40% with uniform and (b) CR 80% with non-uniform link distribution

Fig. 7a and 7b demonstrate the interstorey drifts at different levels for structures (averaged from all THs) with 40% and 60% uniform link distributions w.r.t their corresponding roof displacements. The drifts till the ALY displacement were compared with the FEMA 356 [13], Table 6-19 limits to verify their suitability for the proposed design and it was observed that the drifts were safely below the LS limit. However, when the maximum drifts far after the ALY point were compared with FEMA 356, certain values surpassed the codal restrictions, thus leading towards complete collapse. The interstorey drift values before the ALY displacement, also agreed with the Eurocode 8 for buildings having non-structural elements of brittle materials attached to the structure, and stayed well below the limit of 1%.

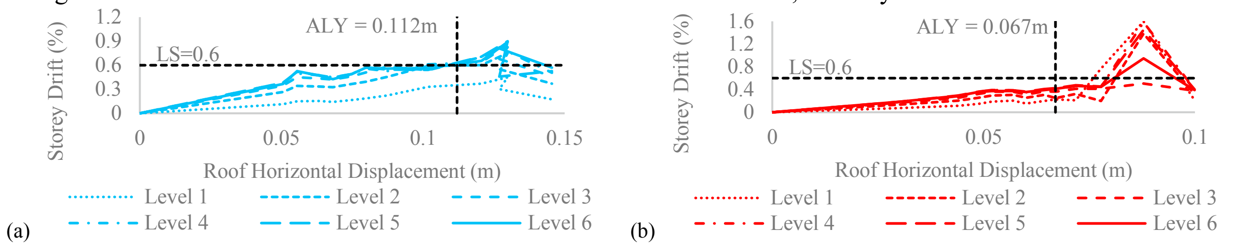

Fig. 7. Maximum Interstorey drifts for (a) CR 40% with uniform and (b) CR 80% with non-uniform link distribution

5. Conclusions

In this present piece of study, an innovative hybrid coupled shear wall system consisting of a RC wall coupled via steel shear critical links with steel side columns on each side is investigated. Some important conclusions were drawn from the nonlinear pushover and IDA results obtained as discussed in the previous sections. Design objectives can be fulfilled using shear critical steel links in a pattern similar to the ZDLD design approach. Hysteretic behaviors obtained from the nonlinear time history analyses characterizes higher energy dissipation potential of the shear critical steel links than the regular steel links. Comparing the maximum link rotation values with the FEMA 356 and AISC 341-16, evident link resistance was also noticed as the maximum rotation in each links remained well below the acceptance limits for a time history scale factor rising up to 10; which in term creates a PGA of 15g. The interstorey drift values also stayed within the FEMA 356 and Eurocode 8 limitations and thus agreed with a suitable ductile structural performance. These results prove the effectiveness of the shear critical steel links and their efficiency towards innovative hybrid coupled wall systems as energy dissipating devices. However, further assessments based on experimental investigations should provide more realistic responses.

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