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NONLINEAR ANALYSIS AND EXPERIMENTAL BEHAVIOUR OF AN INNOVATIVE STEEL FRAME WITH REINFORCED CONCRETE INFILL WALLS

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Abstract. *Steel frames with reinforced concrete infill wall (SRCW) are characterized by an high stiffness that minimize the inter-storey drift and the associated damage to non-structural elements. They are however characterized by critical drawbacks, that limit their use, such as the low dissipative capacity of the concrete wall that causes high seismic force and high overturning moment at the wall base foundation. Within the present study, an innovative steel frame with reinforced concrete infill shear wall system is proposed. It consists of a reinforced concrete infill walls surrounded by a steel frame which contains energy dissipation replaceable elements within the columns. In this way the force transmitted to the other structural elements, and in particular to the foundations, are limited and an important portion of the seismic energy dissipated through plastic deformation. Within this paper the behaviour of the proposed system is investigated through nonlinear analyses based on advanced models supported by the experimental results on real scale specimens, highlighting advantages and drawbacks of the system.*

1 INTRODUCTION

Steel frames with Reinforced Concrete infill Walls (SRCWs) are structural systems that offer many advantages such as their high initial stiffness, able to reduce the building damage under low-intensity earthquakes, and potentially easy repairs after moderate damage through the use of epoxy resins on the cracked wall [1]. In this way, SRCW systems provide a promising structural solution potentially able to reduce the seismic risk in all its aspects, i.e., economic loss and fatalities, as for example contemplated in recent studies [2][3] following the 2012 Emilia earthquakes in Italy and highlighting the necessity of an adequate structural behaviour also for low-to-moderate seismic events [4].

In SRCWs three different resisting mechanisms to horizontal actions can be identified as depicted in Figure 1: 1) flexural behaviour of the steel frame; 2) direct interactions between the steel frame and the compression strut in the reinforced concrete (RC) infill walls; 3) interactions between steel frames and the RC infill walls through friction and shear connectors. The ratio of horizontal forces resisted by each of the load transferring paths depends on the mechanical properties and geometrical configuration of the considered system.

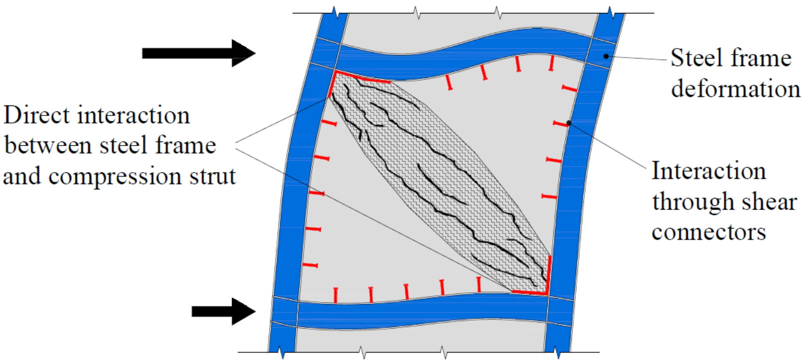


Figure 1: SRCWs resisting mechanisms to horizontal actions.

Despite all the available research studies carried out in the last decades, it is not easy to design a SRCW system in seismic areas due to the lack of specific capacity design rules that allow controlling the formation of a proper dissipating mechanism. Refined numerical analyses previously carried out on SRCW systems [5] designed according to the Eurocodes, in fact, pointed out an unsatisfactory fragile behaviour due to the severe damage occurring to concrete long before yielding of the ductile elements. The failure mechanism is generally characterised by yielding of the steel frame concentrated mainly in the elements near the bottom of the wall (more specifically at the connections of the horizontal to the vertical parts). The plastic deformation on the concrete infill walls concentrates in a diagonal path clearly highlighted by the distribution of cracking. In addition, localized plastic deformations are also present near the corners of the infill walls due to the local action of the first studs of the horizontal and vertical elements (Figure 2a). In order to overcome the observed critical aspects possibly affecting the considered structural system, an innovative approach for ductile design of SRCWs, recently studied during a European research project [5] that also involved innovative steel and concrete hybrid coupled walls [6], is presented in this work leading to the solution depicted in Figure 2b. The RC infill walls are not connected to the vertical elements where the energy dissipation is expected. The system is conceived to control the formation of diagonal struts in the infill walls and behaves as a latticed brace instead of a shear wall. The energy dissipation is expected to take place only in the vertical elements of the steel frame subjected mainly to axial forces without involving the reinforcements of the infill walls.

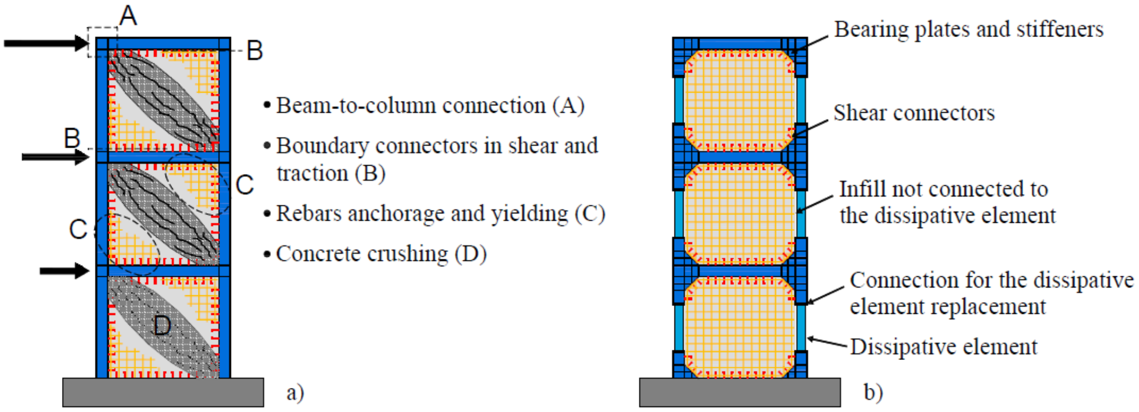


Figure 2: a) critical aspects in the behaviour of conventional SRCW systems; b) innovative SRCW system.

Detailing of the connection of the dissipating elements should allow their replacement and the presence of the infill wall limits the axial force in the compressed dissipative element avoiding its buckling. The formation of the diagonal strut is ensured by joint stiffeners and bearing plates. The joint may be welded in shops allowing speeding up the erection phases. The stud connectors are not required to transfer shear forces but they are mainly used to connect the infill and the frame together during the seismic shakings. A capacity design procedure, able to assure the desiderate energy dissipation mechanism, is proposed consistently with the Eurocode 8 [7] framework and explained in details. Sophisticated and simplified numerical models are then adopted to better understand the global behaviour of the system and to check the validity of the design procedure proposed. Finally, the results of an experimental campaign on two different 2/3 downscaled one-storey specimens are illustrated to validate the design procedure and to provide insight into the influence of the shear studs distribution on the behaviour of the proposed SRCW system.

2 DUCTILE DESIGN OF SRCW SYSTEMS

The proposed innovative SRCW is composed by elements with specific tasks according to the proposed capacity design. The design procedure consists of 9 steps, it is force-based and is applied by considering the simple statically determined scheme representing the limit behaviour of the SRCW depicted in Figure 3. The procedure is described hereafter.

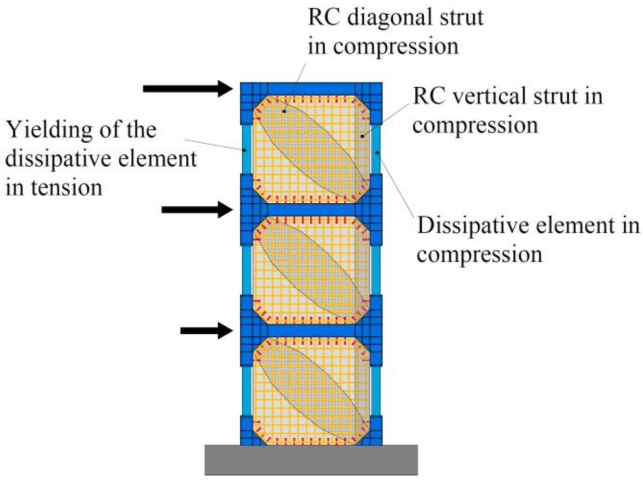


Figure 3: Limit behaviour under seismic action of the innovative SRCW system.

Step 1: definition of the static equivalent lateral loads and calculation of the truss actions.

A design spectrum reduced by a suitable behaviour factor is considered; the definition of the behaviour factor is rather delicate because the limit structural scheme adopted may not represent the behaviour of the system especially in the linear range for moderate earthquakes.

Step 2: design of the cross sections of the ductile boundary elements in traction.

These elements are subjected also to compression under the reversed loadings but they are not expected to yield in compression. Even if in principle no specific provisions are necessary, it is better to assure the element cross section being compact, e.g. at least class 2 according to Eurocode 3 [8].

Step 3: capacity design of the connection of the ductile elements and of the adjacent elements.

The design of the connection of the ductile elements and of the adjacent elements is performed with the formula

$$R_d \geq 1.1\gamma_{ov} R_{fy} \tag{1}$$

where γ_{ov} is the over-strength coefficient of the element with plastic resistance R_{fy} of the connected dissipative member based on the design yield stress of the material. The connection should be designed to resist the force in the linear range in order to reduce damage in the non-ductile elements and to permit the possibility to replace the ductile element after seismic damage. End-plates connections should be preferred to other types with the ductile element connected to the split plate by means of full penetration welding. The adjacent vertical elements to which the ductile elements are connected should be over-strengthened; this can be assured by using a different steel grade or by suitably enlarging the resisting cross section that should have width equal or greater than the infill wall thickness.

Step 4: calculation of geometric over-strength factors.

These factors are calculated as usual for steel structures by the ratio of the real plastic resistance of the ductile element and the relevant design force

$$\Omega = min\left\{ \Omega_i = \frac{N_{pl,Rd,i}}{N_{Ed,i}} \right\} \tag{2}$$

To guarantee yielding of the edge steel elements at the different levels, and to avoid formation of soft storeys, the maximum over-strength Ω_i should not differ from the minimum value by more than 25%. At the higher levels, this condition may difficultly be satisfied. In such a case, the yielding takes place only at a limited number of storeys and the ductile elements should be designed to guarantee the global ductility, e.g. [9][10][11].

Step 5: calculation of axial forces in non-ductile elements by combining the effects of gravity loads with those of the seismic action suitably magnified.

These forces are calculated by suitably magnifying the seismic design component accounting for the material and geometric over-strength of the ductile elements with the usual formula

$$N_{Ed} = N_{Ed,G} + 1.1\gamma_{ov}\Omega N_{Ed,E} \tag{3}$$

Step 6: capacity design of the reinforced concrete infill against concrete crushing (definition of wall thickness t_w and width of the bearing plate l_b).

This step is crucial as it assures the good performance of the system that should not be affected by the wall failure (concrete crushing). As previously described, bearing plates are placed at the beam-to-column nodes to control the formation of the diagonal strut within the wall (Figure 4a).

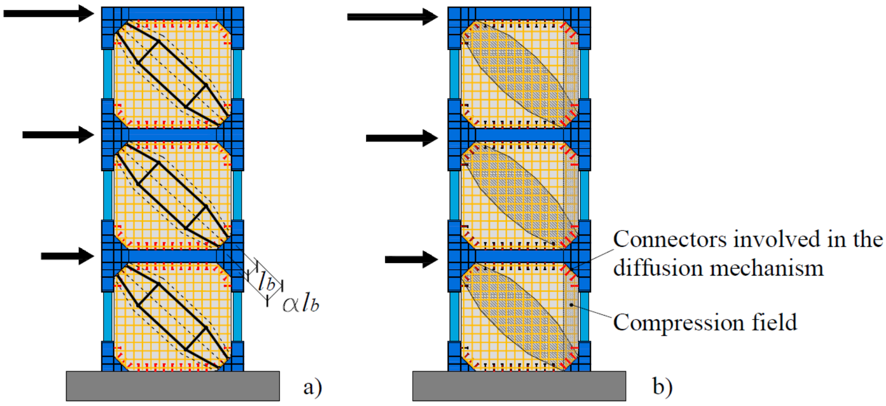


Figure 4: (a) diagonal struts within the infill walls; (b) compression fields involved to resist the axial force in the case lateral elements failed due to instability.

A fan-shaped stress field is expected at the bearing plate; the effective width of the wall should be equal to the bearing plate width l_b at the diagonal ends whereas the effective width is imposed by a coefficient $\alpha > 1$ at mid diagonal. The design formula

$$\left\{0.85\frac{f_{ck}}{\gamma_c}t_w l_b;\, 0.85\frac{f_{ck}}{\gamma_c}\nu\left(1-\frac{f_{ck}}{250}\right)(\alpha t_w l_b)\right\} \geq N_{Ed,G} + 1.1\gamma_{ov}\Omega N_{Ed,E} \tag{4}$$

is derived from Eurocode 2 (paragraph 6.5) [12]; the second value of the strut strength takes into consideration the transverse tension ($\upsilon = 0.6$ may be assumed) whereas the first value considers a simple compression field. The two design parameters l_b and coefficient α can be determined with a trial procedure or by imposing a tentative value for α. The bearing plate should be then proportioned and stiffened to avoid stress localization in the concrete.

The wall reinforcements should be checked to guarantee the diffusive mechanism that depends on the choice of the parameter α; for this purpose rules for partial discontinuity regions suggested in Eurocode 2 (paragraph 6.5.2) [12] are considered. In this case, tractions to be resisted by reinforcements is evaluated by means of the formula

$$T = \frac{1}{4}\left(1-\frac{1}{\alpha}\right)N_R \tag{5}$$

Two different reinforcement layouts may be adopted (Figure 5), the former is constituted by two sets of orthogonal reinforcements whereas the latter is constituted by a set of specific transverse (with respect to the strut direction) reinforcements. In the first case, vertical and horizontal reinforcements should fulfil the conditions

$$\frac{T^2}{f_{yd}^2} = A_{sl}^{v\,2} + A_{sl}^{h\,2} \qquad\qquad \frac{A_{sl}^h}{A_{sl}^v} = \frac{L}{h} \tag{6}$$

It is worth noting that the first reinforcement layout is simpler but possibly less stiff than the second, which instead requires a third order of reinforcements that can be placed only in the case of sufficiently thick walls.

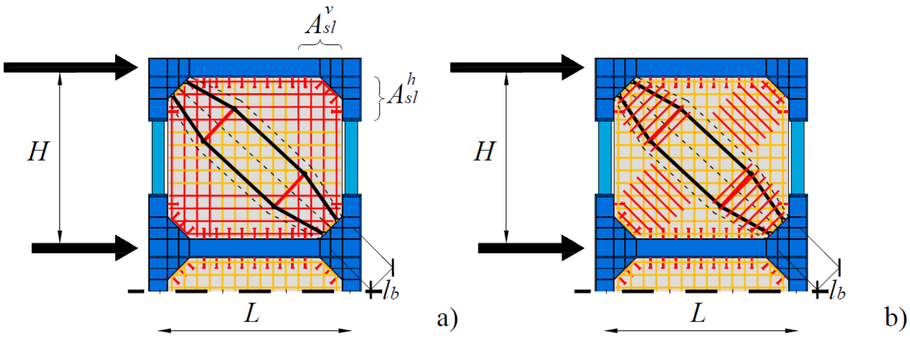


Figure 5: (a) reinforcements constituted by orthogonal rebar layout; (b) reinforcements constituted by additional stirrups.

Step 7: design of the beams in traction.

These elements are designed to resist magnified axial forces. To improve the system feasibility, it is better that their width is compliant with the wall thickness.

Step 8: check and possible re-design of the compressed edge elements.

The ductile elements are checked for instability by using the formula

$$\frac{Af_y\chi}{\gamma_{M1}} \geq N_{Ed,G} + 1.1\gamma_{ov}\Omega N_{Ed,E} \tag{7}$$

The effective length of the element can be selected to be equal to the distance between beam-to-column intersection nodes taking into account the real dimension of the enlarged zone necessary for realizing the bearing plate. In the case the verification is not satisfied, it is expected that the adjacent strip of the concrete wall collaborates to bear the compression force (Figure 4b); in such a case, the following points (8-1 and 8-2) have to be carried out.

Step 8-1: design of the shear connection between the wall and the frame.

The shear connection is designed to transmit to the adjacent RC wall the force in excess with respect to the bearing capacity of the ductile element given by

$$\upsilon_{Rd} \geq N_{Ed,G} + 1.1\gamma_{ov}\Omega N_{Ed,E} - \frac{Af_y\chi}{\gamma_{M1}} \tag{8}$$

In this case, shear connection has to be placed at the vertical elements. It has to be designed by taking into account that possible splitting failure mechanisms of the wall may occur instead of the usual failures due to concrete crushing and yielding. For this purposes, rules suggested by Eurocode 4 Part 2 (paragraph 6.6.4 and Annex C) [13] may be considered.

Step 8-2: check of the vertical strut developing in the wall.

This element withstands the same force calculated with equation (8) and has to be suitably reinforced with confinement stirrups. The same detailing rules suggested in Eurocode 8 (paragraph 5.4.3.4.2) [7] for RC walls may be adopted.

Step 9: calculation of the length of the dissipative element, in order to ensure the compliance between local and global ductility.

For this purpose, formulas derived by considering simplified mechanisms (Figure 6) can be adopted. To a first approximation, the following formula may be used:

$$H' = \frac{(\mu_s - 1)}{(\mu_m - 1)} \frac{\delta_{el} L}{\varepsilon_y \sum_{i=1}^{N} (z_N - z_{i-1})} \tag{9}$$

where μ_s and μ_m are the ductility of the structure and of the material of the dissipative element respectively; the summation is extended only to the N elements that fall in the $\Omega \div 1.25\ \Omega$ range, δ_{el} is the structure elastic displacement evaluated for the static equivalent loading inducing the first yielding; δ_{pl} is the structure plastic displacement evaluated considering only the elements expected to yield.

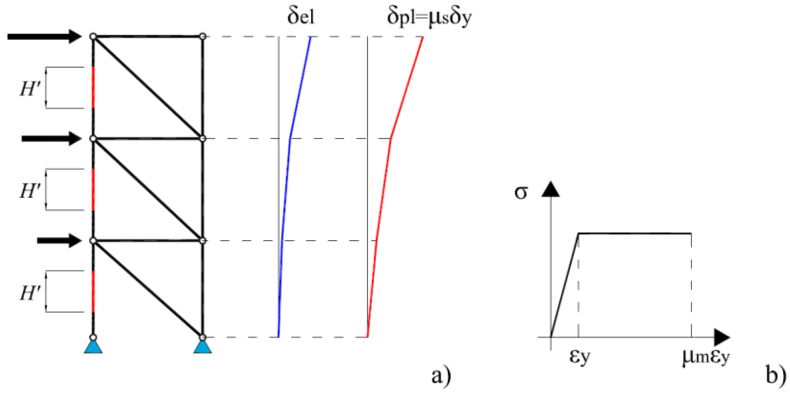


Figure 6: a) Elastic and plastic deformations of the SRCW; b) material stress-strain relationship.

3 NONLINEAR ANALYSIS OF THE INNOVATIVE SRCW SYSTEM

3.1 Finite element model

High fidelity finite element models are developed with the computer program ABAQUS for a preliminary analysis of the behaviour of the presented design approach for SRCW systems. A 4-storey system is considered and designed according to the procedure previously described. Details on the design parameters (i.e. geometry, vertical and horizontal loads, material properties) are reported in [5]. The geometry of the system is closely reproduced by using shell finite elements both for the steel frame and for the concrete infill walls (Figure 7); these are assumed to be connected to the frame only at the inclined bearing plates where stud connectors are placed. In particular, 4-node linear shell elements with 5 degrees of freedom per node (reduced integration, small membrane strains) are used. Wall reinforcements are considered by introducing two layers of reinforcements according to the construction drawings. A coarse mesh (mean size of 0.5 m) is adopted for the concrete walls to avoid numerical convergence problems, whereas a more refined mesh (mean size of 0.1 m) is adopted for steel members.

The behaviour of the concrete is described adopting a smeared cracking model with full shear retention. The concrete behaviour is assumed to follow Mander's law in compression and a linear elastic law in tension up to cracking, followed by a softening tract as later illustrated. Cracking is assumed to occur when the stress reaches a failure surface defined by a linear relationship between the equivalent pressure stress and the Mises equivalent deviatoric stress. When a crack is detected, its orientation is stored for subsequent calculations. Subsequent cracking at the same point is restricted to being orthogonal to this direction because stress components associated with an open crack are not included in the definition of the failure surface used for detecting the additional cracks. The behaviour of cracked concrete is modelled with a strain-softening branch that allows to simulate the interaction between reinforcements and concrete (tension stiffening); in particular, it is assumed a linear strain softening with zero stress at a total strain of about 10 times the strain at tensile failure. Elastoplastic-hardening models are considered for the reinforcements and the steel frame. The yielding point, the material plastic hardening and the ultimate stress for each steel grade considered are defined according to the mechanical characteristics of materials adopted in the design.

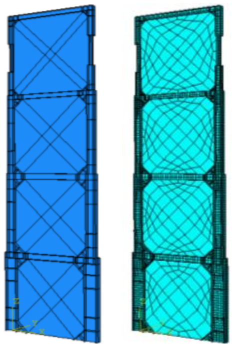


Figure 7: SRCW finite element model.

3.2 Results

Figure 8a; shows the yielding pattern of the steel frame for three displacement levels measured at the top level. It is observed that plastic strains are attained only at the ductile elements consistently with the dissipating mechanism assumed in the proposed design procedure. The resisting mechanism adopted in the design is also corroborated by the stress field in the wall (Figure 8b) that clearly depicts the formation of the diagonal struts in the concrete panels. The values of the principal stresses do not increase with the overall displacement demonstrating that the wall is protected against crushing due to yielding of the side fuses.

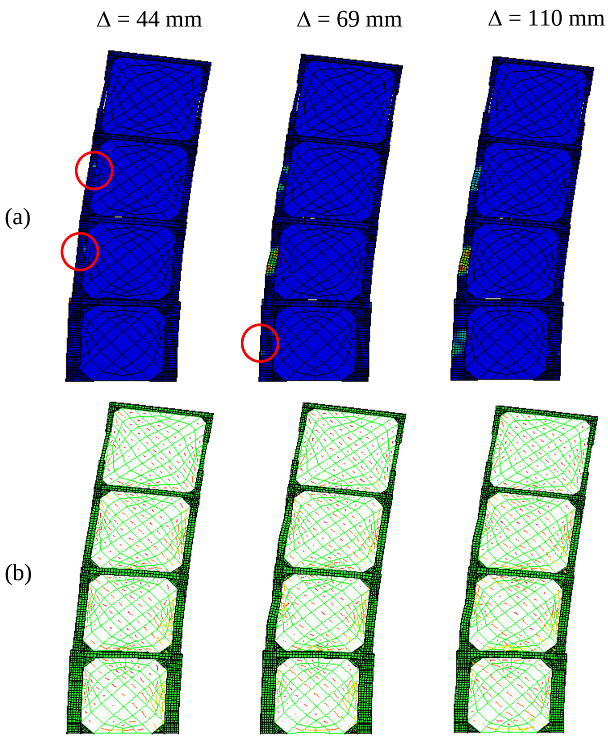


Figure 8: Results of finite element analysis: a) yielding pattern in the steel frame; b) stress field in the wall.

4 EXPERIMENTAL BEHAVIOUR OF THE INNOVATIVE SRCW SYSTEM

4.1 Introduction

An experimental campaign was carried out on two two-third downscaled specimens in order to validate the predicted behaviour of the proposed SRCW, evaluate possible problems related to the realization of such system, and highlight the influence of different shear stud distributions along the steel frame perimeter. The specimens, respectively named S1 and S2 are shown in Figure 9. S1 is characterized by the presence of shear studs only in the steel frame corner zones while S2 has shear studs distributed all along the steel frame perimeter (excluded the dissipative zones). In both cases the RC wall is 12 cm thick.

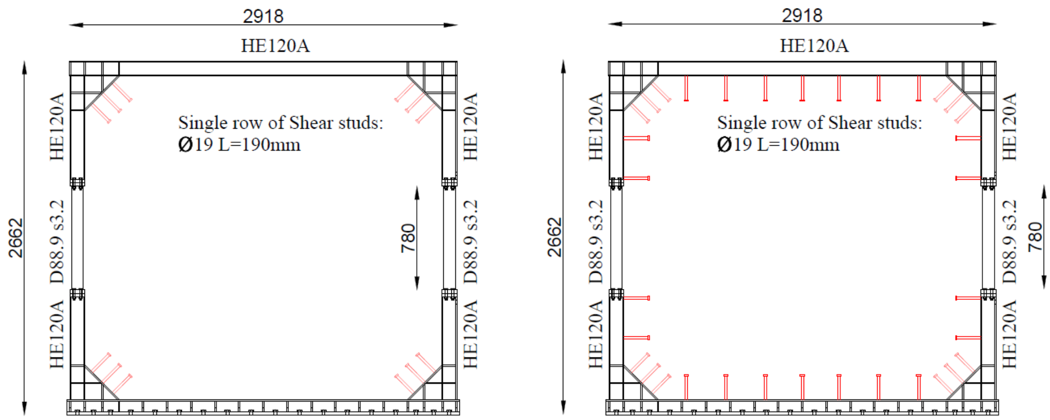


Figure 9: SRCW Steel frame specimens: a) Specimen S1 with shear studs only in the corner; b) Specimen S2 with shear studs all along the perimeter (excluded the dissipative zones).

4.2 Test setup

The SRCW specimen is bolted to a steel base firmly connected to the strong floor by means of an anchoring system and horizontal reaction system (Figure 10), while a lateral stabilizing system avoids transversal displacements of the wall.

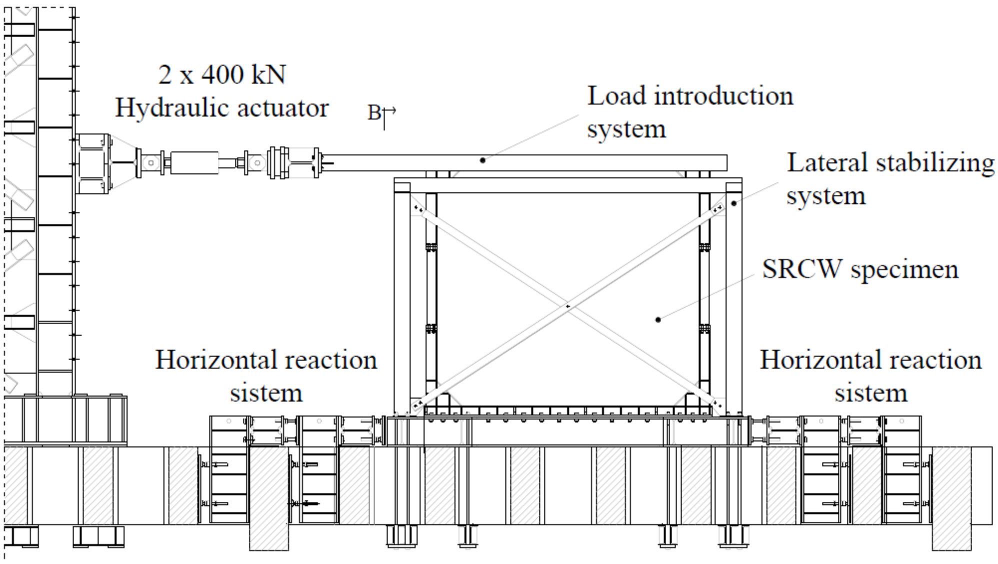


Figure 10: Global test setup.

The system is connected to the wall specimen by ten friction connections and it is independent from the lateral supporting system, allowing so the free tensile deformation of the dissipative elements. The displacements of the wall, the force applied, the deformation of dissipative elements and of the load introduction system are recorded by several sensors placed as shown in Figure 11.

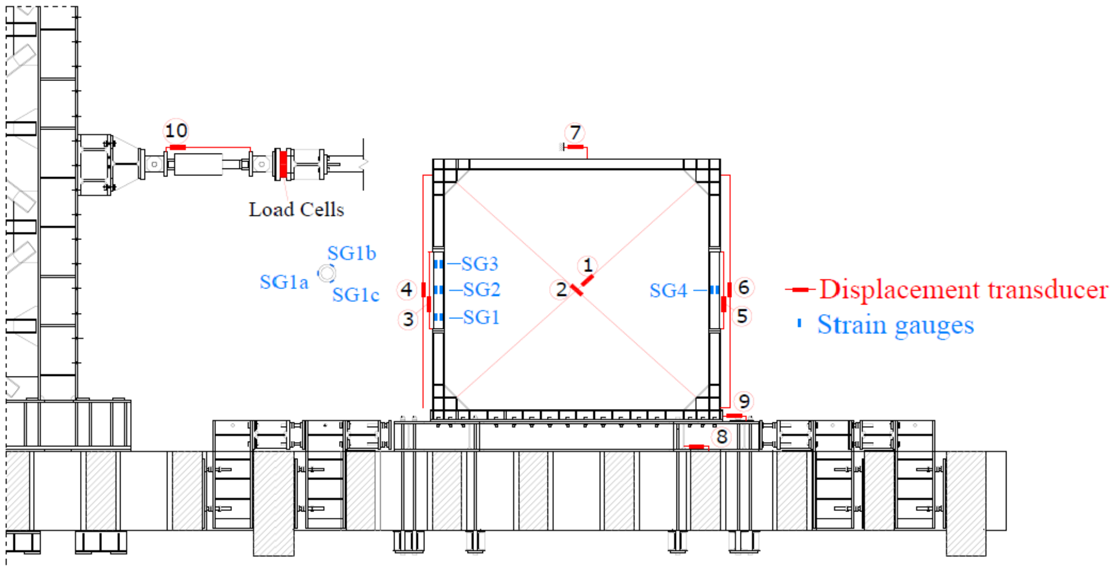


Figure 11: Sensors position.

The tests are carried out in displacement control and the displacement history imposed to the jacks end is reported, for both tests, in Figure 12. An initial maximum displacement equal to 20 mm is imposed cyclically in order to assess a displacement ductility equal at least to 3 (during the test a yield displacement equal to about 6 mm is observed). The imposed maximum displacement is then raised to about 30 mm.

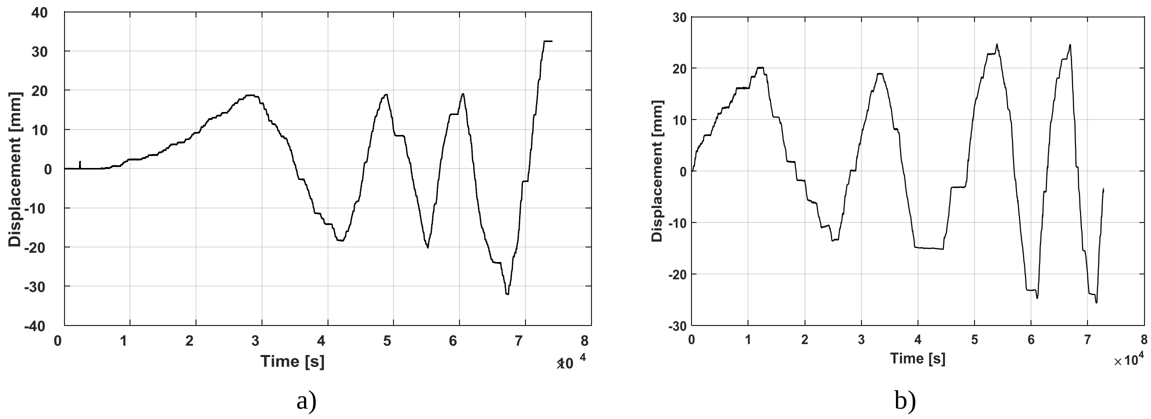


Figure 12: Loading history of tests on a) specimen S1 and b) specimen S2.

4.3 Experimental results

In Figure 13 the experimental cyclic behaviour of specimen S1 is shown. The first loading cycle highlights a relatively "fat" hysteretic behaviour, while pinching phenomena, with the maximum resistance that remains practically constant, is exhibited during the subsequent cycles. The first semi-cycle shows that the system is characterized by a behaviour very close to an ideal elastoplastic one with a displacement ductility equal, at least, to 3.

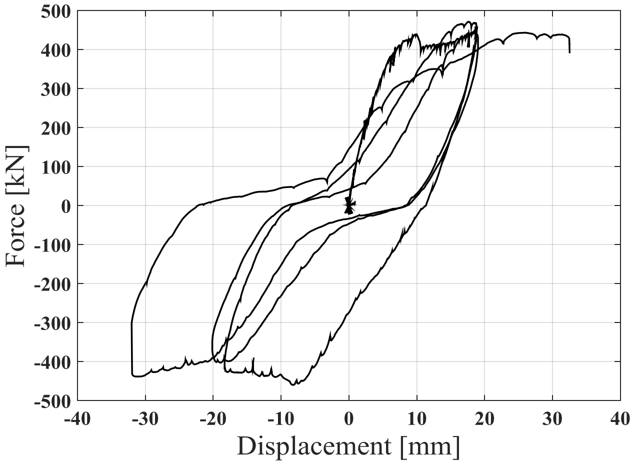


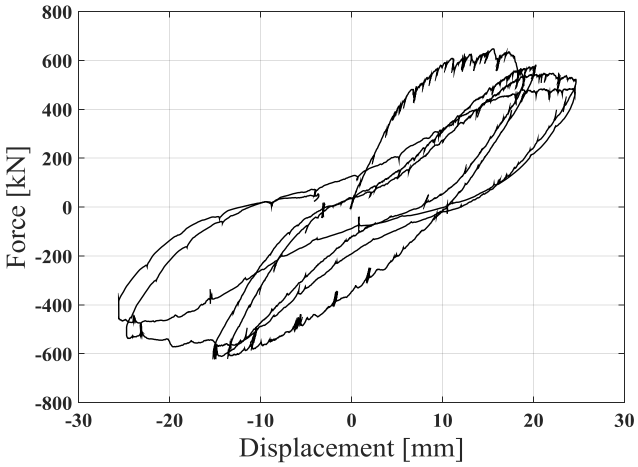
Figure 13: Specimen S1: cyclic force-displacement curve.

At the end of the first unloading phase, the concrete wall exhibits practically no damage, exception made for a little detachment from the lateral steel boundary elements. It can be inferred that, mainly due to the low number of shear studs connecting the RC wall to the steel boundary elements, the wall behaves as a rigid body within the steel frame, avoiding any damage, except for the corner zones. Due to the continuous accumulation, cycle after cycle, of the vertical displacements in the lower edge, the force application point of the compressed concrete diagonal strut changes and causes the failure of the specimen due to an excessive shear deformation of the non-dissipative vertical steel element (Figure 14a). At the same time, the spalling of the concrete on the opposite lower corner of the infill wall and the complete detachment of the infill wall from the steel frame occurs (Figure 14b). Practically no other damages are visible within the RC wall.

a) b)

Figure 14: Lower corners of S1 after failure: a) shear failure of the non-dissipative zone and b) spalling of the concrete and complete detachment by the steel frame.

Specimen S2 shows a behaviour similar to the one of specimen S1 as can be seen in Figure 15, with evident pinching phenomena but with an higher resistance. The diffused presence of the shear studs all along the perimeter of the steel frame allows the transmission of horizontal forces also through shear mechanism, as testified by the diffused diagonal cracking observed within the wall. The more efficient connection between the wall and the boundary steel elements causes the propagation of a main crack from the base of the dissipative element in tension, as illustrated in Figure 16, avoiding any detachment phenomena between the RC wall and the steel frame. Due to the displacement accumulation, the vertical and horizontal reinforcing bars crossing the main crack break (Figure 16), causing the loss of some horizontal forces carrying capacity.



a) b)

Figure 15. Specimen 2: a) cyclic force-displacement curve; b) first loading and unloading phase

a) b)

Figure 16. Specimen S2 at the end of the test: a) global view; b) failure of the steel reinforcements in tension crossing the main crack.

5 CONCLUSIONS

A new steel frame with reinforced concrete infill wall (SRCW) is proposed and its actual behaviour assessed through experimental tests. The system is characterized by the presence of a dissipative element within the boundary steel columns that allows the activation of a proper dissipative mechanism. Consequently, the force transmitted to the other structural elements, and in particular to the foundations, are limited and an important portion of the seismic energy dissipated through plastic deformation. To allow the activation of the dissipative mechanism and in order to optimize the design of non-dissipative elements, a force-based capacity design method, applied by considering the simple statically determined scheme representing the limit behaviour of the SRCW, is proposed. Rules and indications are supplied for the design of each element composing the system, selecting and adapting the design philosophy and the indications of the Eurocodes. The real behaviour of the proposed system is then studied through an experimental campaign on two 2/3 downscaled specimens, characterized by a different shear stud distribution. Both specimens results highlighted a good monotonic displacement ductility and cyclic pinching phenomena, a practically constant maximum resistance and a tendency to accumulate tensile plastic deformation on the dissipative elements. The specimen with shear studs only on the corners failed due to excessive shear damage to the non-dissipative zone of the column and the spalling of the concrete on the opposite lower corner of the infill wall. The failure was probably caused primary by the uplifting of the concrete infill wall and the consequent uplifting of the contact point between the concrete strut and the steel frame. The other specimen, with shear studs distributed all along the non-dissipative steel boundary elements, failed due to the tensile failure of concrete wall reinforcements. It showed the same tendency to accumulate vertical displacement, but the accumulation took place on the main crack formed on the RC wall, while no detachment was observed between the wall and the steel frame. While the first specimen exhibited a latticed structural behaviour, the second specimen highlighted also a shear resistance mechanism of the shear wall, with a consequent initial lateral resistance higher than the one of first specimen. Further researches are however needed for better understanding of the shear studs distribution influence and to study the effects on the failure mechanism of the vertical forces transmitted by the dead and live load to the SRCW systems. The presence of the vertical loads could, in fact, modify the effective failure mechanism, avoiding the accumulation of the vertical displacements. In this way, the re-centring capability, today accepted as one of the most important features for limiting the structural damage due to the earthquake actions [14][15], would be demonstrated also for SRCW systems.

6 ACKNOWLEDGMENTS

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