Mechanical model of steel frames with reinforced concrete infill walls

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Abstract

This paper presents a mechanical model developed for the simulation of the monotonic behaviour of Steel frames with Reinforced Concrete infill Walls (SRCW). In particular, it deals with a specific typology of SRCW, obtained from the classical one through the interposition of dissipative elements in the columns and by stiffening and shaping the steel frame's corners in a way to prevent the brittle failure of the concrete in compression. This system has demonstrated in past researches to be able to overcome the typical problems of SRCWs and to assure, through a capacity design approach, a global ductile behavior. The selection of the main components to be included in the model is carried out on the base of the analysis of the available experimental tests and of the results of accurate 3D Finite Element model analyses. The behaviour of each component is represented through consolidated models present in the current state-of-the-art and, where necessary, calibrated using the results of the experimental and numerical analyses. The capacity of the proposed mechanical model in representing the global behaviour of the SRCWs is finally demonstrated comparing the results in terms of force–displacement curves with the ones obtained through the refined 3D Finite Element models.

Keywords Mechanical model · SRCW · Infill wall · Component-based model

1 Introduction

The effects of the interaction between steel or reinforced concrete (r.c.) frames with the infill panels, being the latter made by masonry or reinforced concrete, have been studied by several researchers in the last decades (Caprili et al. 2012; Ju et al. 2012; Faraji Najarkolaie et al. 2017; Milanese et al. 2018; Ricci et al. 2018; Elwardany et al. 2019) together with the possibility of improving this structural interaction (Preti et al. 2012; Morandi et al. 2018; Marinković and Butenweg 2019; Tsantilis and Triantafillou 2020).

Following the same principles, Steel frames with Reinforced Concrete infill Wall (SRCWs) are an hybrid structural system recently and specifically developed to exploit

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both the high lateral stiffness of the r.c. wall in the in-plane direction and the potential high dissipative capacity of the steel frames (Morino 1998; Hajjar 2002; Zona et al. 2008), aiming therefore at low or null damage for service level limit states and moderate damage for higher action levels (Braga et al. 2019). Moreover, they are characterized by an easy repair after moderate damage using epoxy resins on the cracked wall. The effective dissipative capacity of SRCWs can be severely reduced by a not optimized collaboration between the concrete wall and the steel frames that can lead to unfavorable resisting mechanisms and to severe early damage of the concrete wall (Dall'Asta et al. 2015, 2017; Cao et al. 2020). To overcome such limitations and aiming at fully exploiting the potentialities of SRCWs, the study of Dall'Asta et al. (2017) proposed a new dissipative version of the SRCW together with the associated capacity design method, characterized by the presence of steel dissipative elements placed within the steel columns, see Fig. 1, where most of the energy dissipation shall take place.

However, several past studies demonstrated, both experimentally and numerically, that the resisting mechanism of dissipative (Morelli et al. 2019) and traditional SRCWs (Liauw and Lee 1977; Liauw 1979; Te-Chang and Kwok-Hung 1984; Liauw and Kwan 1985a; Pallarés et al. 2020a, b) is strongly influenced in the in-plane loading direction by the effectiveness of shear studs distribution, proposing to divide them into three categories: the integral, non-integral and semi-integral infill frames. In the first case the shear studs are distributed all along the steel frame, in the second the influence of shear studs, if there are any, is completely negligible, and the last case represents an intermediate situation. No relevant studies have been performed in the out-of-plane direction or on the in-plane/out-of-plane interaction of such systems due the common approach of using the SRCWs as bracing systems for which the out-of-plane influence can be neglected. For all the three categories described, the advantages derive, indeed, from the composite actions developed between the infilled panels and the frames (Liauw and Kwan 1985b) resulting mainly in an increased stiffness and strength of the structures due to the in-plane bracing action of the panels and an increased ductility and energy dissipation due to the structural interface between the frame and the infill walls. The same authors showed that among the three different types of infilled frames the integral infilled frames provide higher stiffness, strength and ductility.

Morelli et al. (2019) studied the influence of the shear studs on the dissipative SRCW proposed by Dall'Asta et al. (2017) through detailed finite element models, showing that also in the case of dissipative SRCW, the integral, non-integral and semi-integral infill frames are characterized by deep differences, both in terms of global capacity and of failure mechanism. These studies showed that assumption of a resisting mechanism based simply on the formation of a diagonal strut, neglecting the influence of the shear studs distribution

Fig. 1 Dissipative SRCW proposed by Dall'Asta et al. (2017)

(Dall'Asta et al. 2017), leads to the underestimation of the resistance of integral and semi-integral infill frames. Moreover, the failure mechanisms for these two categories are different from the one associated to the formation of the diagonal strut.

It is so necessary to develop design tools able to foresee, in simple ways, the effective resisting mechanism and the associated resistance of the dissipative SRCW that cannot be treated as non-integral infill frames. Considering the scope of developing tools that aim at supporting the design phase of dissipative SRCW, it is straightforward that complex finite element model should be avoided.

The present paper illustrates and discusses the development of a component based mechanical model able to represent the monotonic behaviour of dissipative SRCWs. The related resisting mechanisms are so identified assessing the global resistance to horizontal actions by defining the mechanical behaviour of each single component. This model would be extremely useful for the design of the dissipative SRCW system through the conscious dimensioning of its main resisting mechanisms.

Indeed, while component based mechanical models are hardly used to represent the behavior of the whole building, they can be used in the pre-design phase to define the global mechanical characteristics of the dissipative SRCWs (e.g. initial stiffness and yielding force) to be included in an equivalent global model allowing the estimation of the forces acting on the structural elements. The component-based model can be then used to assess the local forces acting on the single components of the SRCWs, allowing their correct dimensioning and, eventually, the update of the global model.

To develop the component based mechanical model, firstly, numerical analyses are carried out adopting a 3D refined Finite Element (FE) model. These analyses, together with the experimental results described in (Dall'Asta et al. 2015 and Morelli et al. 2019), allow to identify the components needed for the development of the mechanical model. The behaviour of each component is then represented by force–displacement laws already present in the technical and scientific literature and, where necessary, calibrated on the base of the experimental and numerical results. The single components are then organized together in a global mechanical model; its validity is assessed comparing the global behavior, in terms of stiffness and resistance to the horizontal loads, with the results obtained experimentally and through the 3D refined finite element models. Figure 2 schematically resumes the methodology followed. For the development of the whole component based mechanical model a total of 11 different SRCWs configurations are considered, including the 2 experimentally tested in (Dall'Asta et al. 2015), the 6 numerically studied in (Morelli et al. 2019)

Fig. 2 Global methodology adopted for the development, calibration, and validation of the mechanical model

and further 3 new configurations, covering therefore all the possible main shear studs distributions for the SRCW systems.

2 Review of the mechanical behaviour of dissipative SRCWs and identification of the resisting mechanisms

2.1 2.1 Review of the studies on of the dissipative SRCWs

The mechanical behaviour of dissipative SRCWs is strongly influenced by the disposition of the shear studs (Dall'Asta et al. 2017; Morelli et al. 2019). These studies analyzed numerically and experimentally two configurations of dissipative SRCW system, one representative of an integral infill frame and one of a non-integral infill frame, and 6 configurations representative of semi-integral frames only through numerical models. Figure 3 shows these 8 dissipative SRCW configurations.

The configurations referred in the cited studies as “configuration 1” and “configuration 2” and hereafter respectively named SRCW-1 and SRCW-2, can be respectively seen as an example of non-integral and integral infill frames and can be assumed as the two boundary situations within which all the other configurations are included. Indeed, as Fig. 4a shows, the force–displacement curves related to cases referred as SRCW-a to SRCW-f in Fig. 3 are included in the fuse delimited by the two force-displacements curves of SRCW-1 and SRCW-2. Analogously, the failure mode of those two configurations can be threatened, even if with some approximations, as the two boundary possibilities: the rigid rotation of the infill wall within the steel frame up to the failure in compression of the concrete as experienced by SRCW-1; the initial reaction mainly associated to the shear stiffness of the infill wall up to the failure of the wall itself due to combined tensile-shear forces, as experienced by SRCW-2. The approximation is mainly related to the presence of few shear studs in SRCW-1 introducing a supplemental source of resistance due to the pullout effect that should not be present into an actual non-integral infill frame (Liauw and Kwan 1985a). However, considering that a limited number of shear studs are necessary to avoid the rigid

Fig. 3 Dissipative SRCW's configurations studied in the research of (Dall'Asta et al. 2015; Morelli et al. 2019)

(a)

(b)

Fig. 4 **a** Force–displacement curves and **b** scheme of the failure modes for the configurations studied by (Morelli et al. 2019)

out-of-plane falling of the infill wall, the SRCW-1 can be reasonably treated as the lower boundary case.

2.2 Definition of a detailed finite element model

Aiming at individuating the resisting mechanisms that contribute to the global resistance, detailed 3D FE models of the boundary cases SRCW-1 and SRCW-2 are prepared. To this end, the indications contained in the research of Morelli et al. (2019), that already analyzed the possible approaches for the accurate Finite Element modelling of dissipative SRCWs, are used with limited different modelling choices to improve the precision of the model. The detailed 3D FE models are developed in Abaqus (ver.6.14) (Hibbitt Karlson and Sorensen 2012) using shell elements for the steel profiles (external frame and dissipative elements), solid elements

for the concrete of the infill wall and beam elements for the steel reinforcing bars and studs embedded within the infill wall. The degrees of freedom of the embedded elements, re-bars and studs, are constrained to the interpolated values of the degrees of freedom of the host region (concrete wall). The use of embedded elements provides the highest quality of approximation but it is hindered with high computational cost (Markou and Papadrakakis 2012) and neglects the slip phenomena between the bar and the hosting concrete. The contact between the shell elements representing the steel frame profiles and the solid element of the wall is defined as a “surface to surface contact” with a “penalty tangential behaviour”, see Abaqus (ver.6.14) user manual (Hibbitt Karlson and Sorensen 2012). The value of the friction coefficient between the two materials is strongly influenced by the corrosion level of the steel surface and the cracking conditions of the concrete (Cairns et al. 2007) and, within the present research, it was assumed a value of 0.45, as suggested by Baltay and Gjelsvik (1990). A tetrahedral linear mesh with reduced integration (C3D4R Abaqus elements) with maximum dimension of 30 mm, Fig. 5a, is used for the solid model of the concrete of the infill wall instead of 50 mm sized hexahedral one suggested in (Morelli et al. 2019) to increase the reliability of the model for the detailed representation of the concrete cracks propagation.

The stress–strain relationship of the concrete is modeled following the indications of the Model Code 2010 (Fib 2013), expressed in equation for the concrete in compression and in Eq. (2) for the concrete in tension. The meaning of the symbols is graphically explained in Fig. 5b and c, while Table 1 reports the numerical values of the adopted parameters.

$$\sigma(\varepsilon) = \frac{\left[\frac{E_{ci}}{E_{cl}} \cdot \frac{\varepsilon}{\varepsilon_{cl}} - \left(\frac{\varepsilon}{\varepsilon_{cl}} \right)^2 \right]}{1 + \left(\frac{E_{ci}}{E_{cl}} - 2 \right) \cdot \frac{\varepsilon}{\varepsilon_{cl}}} \cdot f_c \quad (1)$$

$$\sigma(\varepsilon) = \begin{cases} E_{ct} \cdot \varepsilon & \varepsilon < \varepsilon_{ct} \\ f_{ct} \cdot e^{-\beta \cdot \left(\varepsilon - \frac{f_{ct}}{E_{ct}} \right)} & \varepsilon > \varepsilon_{ct} \end{cases} \quad (2)$$

Fig. 5 Concrete infill wall: **a** Tetrahedral meshes adopted for the finite elements and stress–strain relationships for the behaviour in **b** compression and in **c** tension

Table 1 Values of the concrete parameters adopted for the modelling

Parameter	Numerical value adopted	Source	Parameter	Numerical value adopted	Source
f_c	33.2 N/mm ²	(Morelli et al. 2019) (NTC 2018)	E_{c0}	21,500 N/mm ²	(Fib 2013)
$f_{ct} = 0.3f_{cm}^{2/3}$	3.5 N/mm ²		f_{c0}	10 N/mm ²	
$f_{cm} = f_c + \Delta f$	41.2 N/mm ²		$E_{cl} = \varepsilon_{cl} \cdot f_c$	15,091 N/mm ²	
Δf	8 N/mm ²	(Fib 2013)	ε_{cl}	0.0022	
$E_{ci} = E_{c0} (f_c + \Delta f)^{1/3} / f_{c0}$	34,467 N/mm ²				

The value of the factor of degradation in tension, β , is calibrated on the base of the studies described in (Lee and Fenves 1998) in which a similar concrete grade was subjected to experimental tensile tests. The concrete material is then defined using the Concrete Damage Plasticity Model (CDPM) (Lubliner et al. 1989), already used in several other studies (Nana et al. 2017; Behnam et al. 2018) and assuming a dilatation angle of 36° , an eccentricity of 0.1, a value of $f_{b0}/f_{c0} = 1.16$ and a value of $2/3$ for the parameter K (Morelli et al. 2019). This CDPM allows the definition of a viscosity parameter that fosters the numerical convergence of the material. As observed in the studies of (Michał and Andrzej 2015) this parameter has a significative influence on the stress distribution within the concrete. Therefore, preliminary parametric analyses, carried out varying the value of this parameter, are executed to evaluate the optimal values in terms of reliability of the results and convergence capacity. The push-over, static, analyses are executed applying an increasing horizontal displacement to the top beam of the steel frame, restraining the out-of-plane movements in order to correctly represent the experimental test setup.

Figure 6 reports, for the SRCW-1 configuration, the experimental force–displacement curves and the numerical ones obtained varying the viscosity coefficient and the analysis's increment size.

The same modelling approach is used to build the refined 3D FE models for the following 5 configurations:

- the SRCW-1 and -2, already described in Fig. 3. These configurations are used to calibrate and validate the 3D refined FE models and to individuate the main mechanisms contributing to the global resistance of the SRCW system, as described in this paragraph.
- The SRCW-0, characterized by the absence of shear studs. This configuration is used to study and better understand the influence of the pure contact between the steel frame and the reinforce concrete (r.c.) infill wall and to calibrate some of the components associated to this mechanism, as described in the calibration paragraph.

Fig. 6 Calibration of the viscosity parameter: comparison between the experimental and numerical force–displacement curve for the SRCW-1 configuration

- The SRCW-g and -h configurations, representing two further possible distributions of shear studs and completing therefore the set proposed by (Morelli et al. 2019). They are used to validate the results of the mechanical model, as described in the paragraph dedicated to the validation of the model.

The results already obtained by (Morelli et al. 2019) on the configurations SRCW a-to-f, see Fig. 3, are used to calibrate some of the components defining the mechanical model.

Figure 7 shows the global view of the 3D refined FE model and summarizes the configurations modelled in the present research (SRCW 0-to-2 and of SRCW-g and -h configurations).

The SRCW-1 and SRCW-2 3D refined FE models are subjected to an imposed horizontal displacement applied to the top beam of the frame while all the displacement in the out-of-plane direction are constrained, reproducing the experimental test configuration described in (Dall'Asta et al. 2015). The numerical and experimental behaviors are compared in terms of global force–displacement curves, see Fig. 8, and of cracking pattern, see Fig. 9. These figures testify the capacity of the developed 3D refined FE models of representing the experimental results in both aspects, even if some difference can be noticed in the force–displacement curves for the higher displacements in configuration SRCW-1. The same modelling approach is used to model the SRCW-0 and SRCW h-g configurations for the calibration and validation of the component based mechanical model described in the following paragraphs.

2.3 Individuation of the single mechanisms contributing to the global resistance

To better understand the mechanical behaviour of the two SRCW boundary cases, SRCW-1 and SRCW-2, necessary to figure out the mechanisms present in all the configurations considered within this research, it is useful to deeply analyze the distribution of internal forces/stresses within the steel frame and the r.c. infill wall and the field of internal displacements when the external horizontal force (or the imposed displacement) increases. As already

Fig. 7 Global view of the SRCW 3D refined FE model (left) and view of the steel elements composing the modelled configurations (right)

Fig. 8 Comparison between the numerical and experimental force–displacement curves for the SRCW-1 and SRCW-2 configurations

Fig. 9 Comparison between the cracking pattern recorded during the experimental tests and the ones obtained through the 3D refined FE models for the **a** SRCW-1 and **b** SRCW-2 configurations

noted in (Morelli et al. 2019), the global force–displacement curves associated to both the boundary configurations SRCW-1 and SRCW-2 are well approximated by a trilinear curve, and to each branch of these trilinear curves it is associated a different behaviour of the SRCW. The Fig. 10 shows the trilinear approximation for both configurations and individuates some significative points on the curves used in the following as reference points to describe the distribution of internal forces/stresses and the field of internal displacements. The definition of these points, together with a general description of the status of the steel frame and of the infill wall are reported in Table 2 and in Table 3 respectively for the configurations SRCW-1 and SRCW-2.

Analyzing first the configuration SRCW-1, for a low level of the imposed displacement Δ (before reaching the point “1” of Fig. 10a and of Table 2), the forces are transferred from the top beam to the base of the systems also through a shear mechanisms. After reaching the imposed displacement associated to point “1”, the contribution in shear becomes practically negligible and the wall movement consists almost of a pure rotation. This can be seen analyzing the field of internal horizontal and vertical displacement and individuating the center of rotation of the infill wall (as the point characterized by null displacements).

Fig. 10 Trilinear approximation of the force–displacement curves and individuation of significant points for the **a** SRCW-1 and **b** SRCW-2 configurations

Table 2 Individuation of the significant points on the force–displacement curve for the configuration SRCW-1

Point	Δ [mm]	V_b [kN]	Infill Wall Steel Frame				
				Cracking	Shear	Strut	Tie
0	0.00	0	Elastic	Not present	Active	Active	None
1	0.50	149		Crack initiation			
2	1.90	321	Dissipative element yielding initiation	Cracking propagation	Negligible		Progressive yielding propagation
3	3.10	396	Dissipative element complete plasticization				
END	12.00	490					

Figure 11 shows this field of displacement for an imposed displacement of 0.1 mm (considered sufficiently small) and of 0.5 mm (associated to point “1”). Initially the horizontal displacements are one order of magnitude higher than the vertical ones. When the imposed horizontal displacement Δ increases, reaching values close to 0.5 mm, the order of magnitude of vertical and horizontal displacements becomes almost the same and the center of rotation shifts horizontally in the direction of the applied force, modifying the deformation field of the infill wall from a shear one to the one associated to a rotation around the bottom corner in compression.

When the external imposed displacement reaches the point “1”, the infill wall starts to crack, see Fig. 12a, the shear mechanism completely loses its efficacy and the main resistance is based on a strut & tie mechanism. Indeed, it is possible to individuate a diagonal portion of the infill wall that works in tension thanks to the presence of the shear studs placed in the four corners of the steel frame and the opposite diagonal of the infill wall

Table 3 Individuation of the significative points on the force–displacement curve for the configuration SRCW-2

Point	Δ [mm]	V_b [kN]	Steel Frame	Infill Wall	Shear	Strut	Tie
0	0.00	0	Elastic	Cracking	Active	Active	None
1	0.50	247		Horizontal cracking initiation			
2	0.70	292		Significative horizontal cracking	Negligible		Several vertical rebar plasticization
3	1.20	382		Diagonal cracking initiation			
4	2.00	321	Dissipative element yielding initiation				
5	2.80	384	Complete plasticization				
6	4.00	609		Significative diagonal cracking			Several diagonal rebar plasticization
END	12.00	611

Fig. 11 SRCW-1 horizontal and vertical displacements at: **a** $\Delta = 0,1 \text{ mm}$ and **b** $\Delta = 0,5 \text{ mm}$ top displacement

working in compression thanks to the direct contact between the steel frame and the infill wall, see Fig. 12b. The rotation of the infill wall determines a wider contact zone in bottom corner respect to the opposite top corner of the steel frame, inducing the development of a further resisting mechanism not directly related to the strut & tie one. Indeed, the portion of the column close to the frame corner where the center of rotation lies (the bottom right

Fig. 12 **a** isostatic curves (red – compression, blue – tension) with crack initiation front for $\Delta = 0.5 \text{ mm}$; **b** contact and openings zones; minimum principal stresses (kN/mm²) around contact zones for **c** $\Delta = 6 \text{ mm}$ and **d** $\Delta = 12 \text{ mm}$

corner in Fig. 12b) tends to oppose the rotation of the infill wall through its bending stiffness and the pressure applied by the concrete to the steel column has approximately a triangular distribution, as shown in Fig. 12c-d. This last observation is needed in the later part of the research when defining the equivalent stiffness of the described resisting mechanism.

The strut & ties mechanism and the one opposing the rotation of the infill wall through the bending resistance of the steel column continue to contribute to the global behaviour even for higher values of the imposed horizontal displacement Δ , even when the dissipative element placed within the column starts to yield (point “2” of Fig. 10a and of Table 2) or reaches the complete plasticization (point “3”) and the system develops its maximum resistance. Together with the spreading of the dissipative element plasticization, also the yielding of the reinforcing bars starts in correspondence of the wall corners (bottom left and top right corners of Fig. 12a-b) and spreads into wider zones involving a higher number of rebars section. The failure of the system is then reached either for the pull-out of the bottom corner of the concrete in tension or due to excessive compression of the other bottom corner as already observed experimentally and reported in Fig. 9a.

The configuration SRCW-2 has a similar behavior, even if the presence of a higher number and more distributed shear studs determines a higher contribution in shear of the infill wall and the development of further resisting mechanisms. For the lower level of the external imposed displacement Δ , before reaching the point “1” of Fig. 10b and of .

Table 3, the resistance of the system is mainly guaranteed by the shear resisting mechanism: the horizontal displacements are far higher than the vertical ones for small imposed horizontal displacement ($\Delta=0,1$ mm) and the center of rotation lays in the middle of the infill wall, see Fig. 13a. The imposed displacement Δ for which the shear mechanism loses its efficacy, determining a displacement field more similar to a rotation and the shifting of the center of rotation more close to the bottom-right corner, is practically the same of the one obtained for the SRCW-1 configuration, but the presence of a higher number and more distributed shear studs allows the shear mechanism to offer an higher contribution in terms of stiffness and resistance to the global behavior, as resulting from the comparison of the resistances shown in Fig. 10a and b.

Once the imposed displacement Δ exceed the value associated to point “1”, the infill wall starts to crack and due to the presence of diffused shear studs, the main crack generates in correspondence of the bottom part of the dissipative element in tension and its pattern is firstly horizontal. Once the crack crosses the concrete zone characterized by a higher number of vertical rebars (point “2”), the pattern deviates, the crack propagates diagonally (point “3”) and the vertical rebars close to the dissipative element in tension start to yield, see Fig. 14b and d. Increasing the imposed displacement, the dissipative element in tension also starts to yield (point “4”) and reaches the complete plasticization (point “5”) Fig. 14e. Finally, the crack continues to spread again with a horizontal pattern, the portion of the infill wall positioned over the crack continues to rotate determining the plasticization of further rebars, ideally located on the diagonal in tension formed within the infill wall, see Fig. 14g.

After the first phase associated to the cracking of the infill wall, also in the SRCW-2 configuration the resistance of the system can be associated to a strut and tie mechanism, where the strut in compression forms practically in the same way seen for the SRCW-1 configuration, see Fig. 15a. The infill wall part in tension, on the contrary, is wider and has a more complex behaviour due to the presence of more numerous and more spread shear studs and it is possible to individuate two different ties in tension. The first one to form is a vertical tie, close and working in parallel to the dissipative element in tension, see Fig. 15b. Its resistance is mainly associated to the global resistance of the vertical rebars positioned

Fig. 13 SRCW-2 horizontal and vertical displacements at: **a** $\Delta=0,1$ mm and **b** $\Delta=0,5$ mm top displacement

in the infill wall vertical edges. The second r.c. tie is a diagonal one, similar to the one individuated in the SRCW-1 configuration, but wider and involving a higher number of rebars present within the wall, Fig. 15c.

Finally, also in the SRCW-2 configuration, the lower part of the steel column reacts to the rotation of the infill wall with a mechanisms similar to the one represented in Fig. 12b for the configuration SRCW-1. However, considering that the portion of infill wall effectively rotating in the configuration SRCW-2 is smaller (only the part over of the main crack), this contribution can be considered negligible, as testified by the comparison between the global force–displacement curves of Fig. 16, obtained considering or neglecting the contact forces in the bottom part of the column.

3 Analytical definition and calibration of the components

On the base of the observation made on the distribution of stresses and deformations through the numerical analysis on the two boundary configurations SRCW-1 and SRCW-2, the single components contributing to the global resisting mechanism can be represented in the mechanical model shown in Fig. 17a. Figure 17b reports the simplified mechanical model in the case of the configuration SRCW-0, obtained from the one associated to SRCW-1 and SRCW-2 and removing the components directly related to the presence of the shear studs. This last model is used to calibrate the components related to the diagonal strut.

Fig. 14 Isostatic curves (red – compression, blue – tension) with crack initiation front for **a** $\Delta = 0.50$ mm, **b** $\Delta = 0.70$ mm and **c** $\Delta = 4.0$ mm; yielding propagation within the dissipative element for **d** $\Delta = 2.0$ mm and **e** $\Delta = 2.8$ mm; stress distribution within steel components: in red plasticization propagation within rebars for **f** $\Delta = 0.70$ mm and **g** $\Delta = 4.0$ mm

Fig. 15 **a** Diagonal strut mechanism within SRCW-2 (Phase III) identified from compressive isostatic curves, **b** vertical tie identified from plasticization path of rebars ($\Delta\delta = 0.70$ mm), **c** diagonal & vertical tie identified from plasticization path of rebars ($\Delta\delta = 4.00$ mm)

Configurations SRCW-1 and SRCW-2 can, on the contrary, be represented by the same mechanical model assuming that the single components have different behaviour depending on the number and disposition of shear studs.

Fig. 16 Numerical force-displacements curves obtained for the configuration SRCW-2 considering and neglecting the influence of the contact between the infill wall and the bottom part of the steel column and confrontation with the experimental force-displacement curve

With reference to the numbering of the elements shown in Fig. 17a–b, Table 4 reports a summary description of the single components, together with the type of element adopted for the building of the mechanical model in OpenSEES (Mazzoni et al. 2007) and the source for the definition of the generalized force–displacement relationship. Some of these relationships were already defined in the scientific and technical literature, while others needed a specific calibration through the numerical 3D models.

3.1 Analytical definition of the single components

3.1.1 Components 1 and 2

The components 1 and 2 in Table 4 represent respectively the steel frame (beams and columns) and the steel dissipative elements, realized by tubular elements, included within the columns. The numerical and experimental analyses have demonstrated that the former remains in the elastic field besides a small portion close to the center of

Fig. 17 Single components organized in a mechanical model for the configurations **a** SRCW-1 and -2, and **b** SRCW-0

Table 4 Single components constituting the mechanical model for the dissipative SRCWs

Comp.ID	Element/mechanism	Type definition	Geometrical and mechanical characteristics		
			SRCW0	SRCW1	SRCW2
1	Steel beams/columns	ElasticBeamColumn	Effective geometry and material	Effective geometry and material	Effective geometry and material
2	Steel dissipative element	NonLinearBeamColumn (Fiber section)	Effective geometry and material	Effective geometry and material	Effective geometry and material
3	Wall-column contact element	Zerolength (Elasto-Plastic)	Analytical definition based on the numerical analysis	Analytical definition based on numerical analysis	Negligible
4	Diagonal strut	Two-NodeLink (Elastic)	Calibration of the effective width from the numerical model	Calibrated on the SRCW-0 configuration	Calibrated on the SRCW-0 configuration
5	Connection system/steel rebars	(Multiple) Two-NodeLink (Elasto-Plastic)	Not present	Analytical definition based on numerical analysis and Standard Code provisions	Analytical definition based on numerical analysis and Standard Code provisions
6	Shear element	Two-NodeLink (Elasto-Plastic)	Not present	Calibration and definition based on numerical analysis	Calibration and definition based on numerical analysis

rotation of the infill wall that can experience plastic deformation. The contribute of this small portion is taken under consideration in the mechanical model through the component 3 of Table 4 representing the contact between the infill wall and the frame. Therefore, the component 1 is represented by linear elastic beam elements whose stiffnesses are derived directly from the geometry and mechanical characteristics of the steel profiles adopted. The steel fuses, on the contrary, go in the plastic field due to the combined axial-shear-bending actions associated to the rotation of the infill wall. Their contribution is taken into account in the OpenSEES model through a nonlinear beam element (nonlinear beamcolumn (Mazzoni et al. 2007)) with a fiber section and an elastic-perfectly plastic stress–strain law, with a yielding strength $f_{ym} = 243 \text{ N/mm}^2$ as obtained experimentally in (Morelli et al. 2019).

3.1.2 Component 3

The mechanism deriving from the contact between the infill wall and the exterior frame, named as component 3 in Table 4, depends on the flexural stiffness of the portion of the column that opposes the rigid rotation of the infill wall. This component is present, or significant, only when the collapse mechanism foresees the rigid rotation of the whole wall and its action is assimilable to an elasto-plastic spring opposing the rotation of the wall. The initial stiffness, k_3 , of the equivalent spring representing the component 3 depends on the distribution of the contact stresses and on the position of the spring itself in the mechanical model. To be consistent with the organization of the mechanical model shown in Fig. 17, the component is represented by a linear spring located horizontally at the level of the imposed displacement and the resultant stiffness k_3 is evaluated to assure an equivalent stabilizing moment M_3 as the one offered by the wall-frame contact mechanism. Analyzing the contact pressures acting on the steel column of the configuration SRCW-1, Fig. 18, it can be seen that the main pressures are applied to the bottom of the column itself and, to simplify all the calculation without losing in accuracy, they can be approximated with a linear distribution characterized by the same overturning moment.

Adopting this linear distribution and following the scheme reported in Fig. 19, the equivalent spring stiffness can be therefore evaluated as:

$$K_3 = \frac{10 \cdot EJ}{h.H^2} f_y \quad (3)$$

3.1.3 Component 4

The component 4 derives from the contact between the infill wall and the steel frame and represents the formation of a concrete diagonal strut between two opposite angles of the frame itself. The initial idea about the global resistance of the SRCW system was originally based on the development of this mechanism and it is therefore present in all its possible configurations. Some hypothesis about the possible shape and the design of the concrete diagonal strut were provided in (Dall'Asta et al. 2017) but without supplying detailed information about the transverse section of the diagonal strut (only supposing that a fan-shaped stress fields are expected to form at the bearing plates). Considering that the components 1 to 4 are the only ones needed to complete the proposed mechanical model of the

Fig. 18 **a** Contact pressure derived from the numeral model acting on the part of the steel column placed under the dissipative device for the configuration SRCW-1 and **b** equivalent linearization of the pressure over the height

Fig. 19 Equivalent spring for the component associated to the contact between the infill wall and the steel column: **a** derivation of the initial elastic stiffness and **b** force–displacement relationship

configuration SRCW-0, that model is used to calibrate the value of α_m used to obtain the mean value of the width of the diagonal strut from the bearing plate width L_b , see Fig. 20. The optimal value of α_m deriving from such calibration is equal to 1.13.

3.1.4 Component 5

The component 5 represents the effects of diagonal and vertical ties forming within the infill wall thanks to the connection offered by the presence of the shear studs and it is not therefore, for example, present in the mechanical model of the configuration SRCW-0. The effectiveness of this component depends on the position of the shear studs, being more effective when the shear studs are placed closer to the corners of the steel frames, and it is strictly related to the formation of the cracking pattern and to the resisting mechanism. As depicted in the previous section, the distribution of the tensile stresses within the r.c. wall is very different in the case of the two boundary configurations SRCW-1 and SRCW-2. In the former, they are mainly present in the diagonal of the wall while in the latter there is a significant presence of tensile stresses also in the of the r.c. wall portion placed close to the edge of the steel frame in tension.

The influence of the different zones in which the shear studs can be placed is not completely independent, and it is rather difficult to assign them a precise and unique contribution on the determination of the cracking formation and propagation and, therefore, of the equivalent tie. It is however possible to make the following deductions, based also on the results obtained in (Morelli et al. 2019):

- Configurations SRCW-b and -d develop a cracking pattern similar to the one of the boundary case SRCW-2, with an initial horizontal crack that goes down approximately at 45° after crossing the concrete zone, close to the steel column, reinforced with vertical rebars.
- Configurations SRCW-a, -c and -f have a cracking pattern similar to the one of the boundary case SRCW-1, with a cracking pattern that involves mainly the bottom corner in tension.
- Configuration SRCW-e has a cracking pattern that does not resemble none of the boundary cases SRCW-1 or SRCW-2 and therefore the simplified model would be very different from the other cases. The analysis of this specific case is not in the objectives of the present research and will be therefore not analyzed in the following.

Fig. 20 Concrete diagonal strut (component 4) **a** schematic representation and **b** calibration of the parameter through the numerical model

Considering the groups of shear studs already proposed in (Morelli et al. 2019) and represented in Fig. 21, the C1, C2 and C3 shear studs groups are the ones that influence the most the behaviour of the SRCWs since the group C4 is closer to the center of rotation and therefore its influence is almost negligible in the cracking formation and propagation. It becomes relevant only in the case of the SRCW-e configuration where, due to the absence of the C1 and C2 groups, C3 together with C4 have a significant influence on the global behavior. In general, it can be noted that the contemporary presence of at least the C1 and C3 shear studs groups is required to develop a cracking pattern similar to the boundary case SRCW-2.

Depending on the shear studs disposition, therefore, component 5 can be characterized by the presence of different r.c. ties. In the case of absence of shear studs (SRCW-0 configuration), the component 5 is not present considering that no direct tensile forces can be generated by the steel frame–r.c. wall interaction, see Fig. 22a. In the general case (configurations SRCW 1 and 2 and configurations SRCW a-d and SRCW-f), the component 5 can be modelled with a vertical tie and a diagonal tie, which represent the tensile resistance of the rebars within the wall, and a corner-link, which represent the local behaviour of the shear studs, see Fig. 22b.

Considering the case of complete connection between the steel frame and the r.c. infill wall (configuration SRCW-2, see Fig. 22b), the component 5 can be represented by the system of springs shown in Fig. 23a that takes into account the main possible failure mechanisms of the connection between the r.c. wall and the steel frame provided by the shear studs and of the r.c. ties developed within the wall. The force–displacement behaviour of each of these springs can be derived from the provisions of the Eurocode 2–part 4 (EN 1992–4:2018) and the InFaSo Design Manual (Kuhlmann et al. 2014), see Fig. 23b. All the calculation regarding the force–displacement curves for the springs of the component 5 are resumed in the Appendix 1.

The standard identifies three types of possible failures in tension (for C2, C3, C4 groups), each of them represented by a single spring in Fig. 23a: failure of the Stud (S), Pull-out of the stud (P), failure for the formation of the Concrete Cone (CC). The last of them does not necessary lead to the failure of the subsystem if there are steel rebars crossing the cracks. The stiffness and resistance of the mechanism associated to the presence of

Fig. 21 Individuation of the relevant shear studs' groups

Fig. 22 Representation of the component 5 in the mechanical model for the configurations **a** SRCW-0 and **b** SRCW-1, SRCW-2, SRCW-a-d, SRCW-f

Fig. 23 **a** Springs composing the component 5 of the mechanical model and **b** their force–displacement relationships

these steel rebars depends on their quantity and their anchoring type and their contribution is represented in Fig. 23a by the springs named RS/RB that consider the resistance of the Rebars (RS) themselves and of their bonding (RB).

For the subsystem working in shear (as C1 group), Eurocode 2–part 4 (EN 1992–4:2018) indicates that the global resistance of the mechanism associated shall be evaluated as the lower between the resistance associated to the failure of the Stud (S) and the one associated to the pry-out of the concrete (CP); the edge failure mechanism suggested by the Code is not applicable to the particular geometry of the SRCWs. The RB components can be considered as rigid, considering that each resisting bar belongs to an electro-welded steel mesh or is anchored in a concrete zone confined by closed stirrups.

The activation of the different springs represented in Fig. 23a or the activation of only a part of them depends on the cracking pattern associated to the global resisting mechanism

that is, in turn, influenced by the shear studs distribution, as already discussed at the beginning of the present paragraph. On the base of the observation made about the formation of the cracking pattern, the following indications can be given:

- Configurations SRCW-b and -d develop a cracking pattern similar to the one of the boundary case SRCW-2, therefore both the C1 and C2 subsystems activate and the component 5 assumes the aspect of Fig. 24a.
- Configurations SRCW-a, -c and -f have a cracking pattern more similar to the one of the boundary case SRCW-1 and, therefore, for these configurations it can be assumed that only the C1 subsystem activate and the component 5 assumes the aspect of Fig. 24b.

3.1.5 Component 6

The component 6 represents the shear behaviour of the r.c. infill wall and, similarly to the component 5, its effectiveness depends strongly by the number and disposition of the shear studs, because they influence the portion of r.c. effectively connected to the steel frame. The phenomena inducing the interaction in shear of the r.c. wall with the steel frame is very complex and an exact theoretical approach goes beyond the objectives of the present research. It is however possible to consider the contribution in shear of the shear wall through a simplified approach and focusing it into a single spring, see spring “6” in Fig. 17, characterized by an elasto-plastic behavior. The contribution in shear of the wall can be, indeed, considered as linear elastic for low levels of forces. After the first cracking takes place, it can be reasonably assumed that the wall loses its monolithic behaviour and the contribution in stiffness of the wall in shear drops to a negligible value and the strut and tie mechanisms give a predominant contribution.

Considering the rather complex interaction among the different zones in which the shear studs can be positioned, a full-comprehensive definition of this component is not feasible in reliable terms with the few results available on the dissipative SRCW system. However, it is possible to calibrate the stiffness of the spring “6” depicted in Fig. 17 on

Fig. 24 Activation of the **a** C1 and C2 subsystems for the SRCW-2, -b and -d configurations and **b** of only the C1 subsystem for the SRCW-1, -a, -c and -f configurations

Fig. 25 Force–displacement curve for the spring associated to component 6

Table 5 Values of k_t and of $\delta_{6,y}$ for the different configurations considered

ID	Type of Cracking	k_t	$\delta_{6,y}$ (mm)
SRCW-1	At corner	0.05	0.5
SRCW-2	Along surface	0.15	0.5
SRCW-a	At corner	0.1	0.4
SRCW-b	Along surface	0.075	0.6
SRCW-c	At corner	0.075	0.6
SRCW-d	Along surface	0.1	0.45
SRCW-f	At corner	0.05	0.5

the base of the force–displacement curves obtained through the 3D numerical models for all the configurations considered in Fig. 3 besides the SRCW-e.

Therefore, the initial stiffness K_6 and the maximum force F_6 of the spring representing the component 6 can be calculated as expressed in Eqs. (4) and (5), as represented in Fig. 25:

$$K_6 = k_{st} \cdot \frac{6}{5} \cdot \frac{b_w \cdot t_w}{h_w} \cdot G \quad (4)$$

$$F_6 = K_6 \cdot \delta_{6,y} \quad (5)$$

where b_w , t_w , h_w are respectively the length, thickness and height of the r.c. infill wall, G is the transversal elastic modulus of the concrete, $\delta_{6,y}$ is the displacement of the steel frame top beam for which the cracking starts and k_{st} is a coefficient that depends on the distribution of the shear studs.

Table 5 reports the values of $\delta_{6,y}$ obtained by the 3D refined numerical models as the values associated to the first cracking of the infill wall and of k_t as the value that allows the mechanical model to best approximates the refined numerical force–displacement curves, see Figs. 26 and 27.

Table 5 shows that, for the cases considered, a mean value of $\delta_{6,y}=0.5$ mm can be considered as constant for all the configuration.

To predict the contribution of the component 6 in configurations with a shear studs distribution different from the one assumed in the previous paragraph, it is convenient to search the contributions $k_{t,Ci}$ that each specific group of shear studs supplies to the global stiffness k_t . This searching is done minimizing the mean error on the values of

Fig. 26 Force displacement curves obtained through the experimental tests, the 3D refined numerical model and the mechanical model for the **a** SRCW-1 and **b** SRCW2 configurations

k_t reported in Table 5 and obtaining therefore the following values of $k_{t,C1}$ reported in Table 6.

4 Validation of the proposed mechanical model

The proposed mechanical model is validated comparing the global force–displacement curves with the ones obtained through the refined 3D numerical models. The SRCWs configurations adopted for the validation are shown in Table 7. The concrete class and its mechanical characteristics are assumed equal to the ones adopted in the SRCW-1 and -2 configurations.

On the base of what assumed and obtained in the previous paragraphs, and in particular of the observations highlighted for the component 5 regarding the influence of the shear studs distribution on the cracking pattern, the two SRCW configurations SRCW-g and -h adopted for the assessment of the mechanical model will be characterized by cracking patterns respectively similar to the boundary case SRCW-2 and SRCW-1. Indeed, the configuration SRCW-g has both the C1 and C3 shear studs groups, while the SRCW-h lacks the C1 group. Consequently, for both cases the mechanical model will be similar, see Fig. 28, with the only difference laying in the component 5 and therefore in the absence of the sub-system C1 in the SRCW-h configuration.

Even if the two SRCW configurations, -g and -h, do not include all the possible combinations of shear studs' disposition for the component 5, their models, and therefore their validation, involves all the components calibrated in the present research.

The analyses on the refined 3D numerical models confirm the assumed crack patterns, as depicted in Fig. 29, and the effectiveness of the mechanical model in assessing the global force–displacement curves is testified by Fig. 30.

The comparison of the force–displacement curves obtained through the mechanical model and the refined one shows the capacity of the former in representing the global behavior of the dissipative SRCWs. Some differences can be seen for the SRCW-g configuration, in particular to the -g2 case in terms of post-elastic stiffness. This deviation can be

Fig. 27 Force displacement curves obtained through the 3D refined numerical model and the mechanical model for the **a** SRCW-a, **b** SRCW-b, **c** SRCW-c, **d** SRCW-d, and **e** SRCW-f configurations

Table 6 Contribution of each shear studs group to the global shear stiffness

Conf. ID	$k_{t,C1}$	$k_{t,C2}$	$k_{t,C3}$	$k_{t,C4}$	$\Sigma k_{t,i}$	k_t (from calibration)
SRCW-1		5.20%			5.20%	5,00%
SRCW-2	2.05%	5.20%	3.70%	2.50%	13.45%	15,00%
SRCW-a		5.20%	3.70%	2.50%	11.40%	12.50%
SRCW-b	2.05%		3.70%	2.50%	8.25%	7.50%
SRCW-c	2.05%	5.20%		2.50%	9.75%	10,00%
SRCW-d	2.05%	5.20%	3.70%		10.95%	10,00%
SRCW-f	2.05%	5.20%			7.25%	7.50%

Table 7 SRCW configuration adopted for the validation of the mechanical model

Configuration ID	Shear studs groups	Dissipative element steel grade
SRCW-g1	C1,C3	S235
SRCW-g2		S275
SRCW-h1	C2,C3	S235
SRCW-h2		S275

Fig. 28 Schematic representation of the mechanical model for the **a** SRCW-g and **b** -h configurations

Fig. 29 Cracks formation and propagation for the **a** SRCW-g1 and **b** -h1 configurations obtained through the 3D refined numerical models

Fig. 30 Comparison between the force–displacement curves obtained from the 3D refined numerical model and the mechanical model for the **a** SRCW-g1, **b** -g2, **c** -h3 and **d** -h4 configurations

attributed to the more widely extended diffusion of cracks that involves a greater number of rebars in the later stage of the global resisting mechanism.

5 Conclusions

The paper presents a proposal for a component-based mechanical model of dissipative Steel frames with Reinforced Concrete infill wall (SRCWs). The mechanical model is developed on the base of both experimental and numerical results obtained in past researches and considers the possible resisting mechanisms contributing to the monotonic global behaviour of the system in terms of stiffness and strength.

To have a reliable mean for calibrating and assessing the proposed mechanical model, first of all a refined 3D finite element model is developed on the base of the indications of past researches and calibrated on the base of the available experimental results.

The component-based mechanical model considers 6 types of resisting mechanisms and associate to each of them a specific component, whose monotonic behaviour is derived from the current technical standard provisions or technical literature and, where necessary due to the lack of specific indications, calibrated on the base of the experimental tests and of the results of the 3D refined finite element models.

The typology and disposition of the single components of the mechanical model are strongly influenced by the cracking pattern of the infill wall and, consequently, by the type of failure of the system. The present paper proposes, on the base of the analysis of the experimental and numerical results already present in the scientific literature, a way to foresee the cracking pattern of the infill wall and, consequently, to define the component-based model. The results obtained show a good agreement with the ones derived from the 3D refined finite element models both in terms of initial stiffness and resistance and the mechanical model can be regarded as a useful tool for the design and verification of dissipative SRCWs.

There are, however, still some issues to be solved in the proposed model, mainly inherited from the limitations of the experimental tests. First, the model can represent only the monotonic behavior, while the seismic action induces cyclic forces. Further experimental and numerical studies would be, however, required to calibrate the cyclic behavior of each component of the mechanical model and the development of refined 3D FE models is very challenging due to high non-linearity sources, included the stiffness and resistance degradations effects. The model still does not consider the presence and variation of vertical loads acting on the wall. Moreover, the influence of the infill wall concrete strength on the component 6, the one considering the contribution of the infill wall in shear, should be further deepened. Finally, the model is currently unable to supply, as most of the component-based models, information about the ductility capacity of the single component and of the global system.

Appendix 1–evaluation of the component 5

The force–displacement relationships of Fig. 22b can be analytically evaluated following the provisions of the Eurocode 2 – part 4 (EN 1992–4:2018) and of the InFaSo Design Manual (Kuhlmann et al. 2014).

Studs Failure (S-component)

The behaviour is assumed to be elastic-perfectly plastic and the yielding force and elastic stiffness are evaluated as follows:

$$F_{y,S} = n_h \cdot \frac{\pi \cdot \varnothing_h^2}{4} \cdot f_{y,s}$$

$$K_S = n_h \cdot \frac{\pi \cdot \varnothing_h^2}{4} \cdot \frac{E_s}{h_{ep}}$$

where

- $\varnothing_h = 19$ mm is the diameter of studs
- n_h is the number of shear studs
- $E_s = 210,000$ MPa is the elastic modulus for studs
- $h_{ep} = 180$ mm is the effective studs' height
- $f_{y,s} = 500$ MPa is the yield strength for studs.

The yielding displacement can be derived as:

$$\delta_{y,S} = \frac{F_{y,S}}{K_S}$$

Concrete Cone Failure (CC-component)

The concrete cone failure mechanism has a rigid—linear softening behavior, which depends on the number of the studs (n_h), the strength of each concrete cone (F_{CC}°) individually considered and on various geometric factors which are considered by means of the φ -coefficients defined according to Eurocode 2—part 4 (EN 1992–4:2018):

$$F_{max,CC} = n_h \cdot F_{CC}^\circ \cdot \varphi_{A,N} \cdot \varphi_{S,N} \cdot \varphi_{EC,N} \cdot \varphi_{RE,N} \cdot \varphi_{M,N}$$

where

- $\varphi_{A,N}$ considers of the mutual overlap of the cones and the proximity to the edges
- $\varphi_{S,N}$ considers of the disturbance of the stresses distribution due to the proximity of edges
- $\varphi_{RE,N}$ considers the effect of dense reinforcement between which the studs are located
- $\varphi_{M,N}$ considers of the effect of compressive forces between fixture and concrete in cases of bending moments
- $\varphi_{EC,N}$ considers of eccentricity of loads application.

The strength of each concrete cone F_{CC}° can be derived from the following relation:

$$F_{CC}^\circ = k_1 \cdot \sqrt{f_{ck,cube}} \cdot h_{ep}^{1.5}$$

where

- $f_{ck,cube}$ is the concrete cube strength
- k_1 depends to the cracked condition of concrete: $k_1 = 10.3$ was assumed as average value between the cracked ($k_1 = 8.9$) and uncracked ($k_1 = 12.7$) values.

The negative stiffness of the backbone phase can be evaluated as:

$$K_{CC} = -537 \cdot \sqrt{f_{ck,cube} \cdot h_{ep}} \cdot \varphi_{A,N} \cdot \varphi_{S,N} \cdot \varphi_{RE,N}$$

Finally, the ultimate displacement, which corresponds to zero residual strength condition is equal to:

$$\delta_{u,CC} = \frac{F_{max,CC}}{K_{CC}}$$

Pull-out failure (P-component)

Following the InFaSo Design Manual (Kuhlmann et al. 2014) the pull-out mechanism can be defined as rectangular-parabolic curve. To simplify the mechanical behaviour it was defined as a trilinear curve, in which the plastic branch is perfectly coincident with the rectangular-parabolic curve. The first significant point is defined by the following force–displacement

$$\delta_{1P} = k_p \cdot \left(\frac{F_{max,CC}}{A_h \cdot f_{ck,cube} \cdot n_h} \right)^2$$

$$F_{1P} = \frac{A_h \cdot f_{ck,cube} \cdot n_h}{\sqrt{k_p}} \cdot \frac{\sqrt{\delta_{1P}}}{0.5}$$

where $A_h = 511 \text{ mm}^2$ is the studs head area minus the stud's area and $k_p = 0.0131$. The second significant point is defined from:

$$\delta_{2P} = 2 \cdot k_p \cdot \left(\frac{\min(F_P, F_{y,RS})}{A_h \cdot f_{ck,cube} \cdot n_h} \right)^2 - \delta_{1,P}$$

$$F_{2P} \cong \frac{A_h \cdot f_{ck,cube} \cdot n_h}{\sqrt{2 \cdot k_p}} \cdot \frac{\sqrt{\delta_P}}{0.5} - F_{1P}$$

where $F_P = n_h \cdot p_{uk} \cdot A_h$ and $p_{uk} = 12 \cdot f_{ck,cube}$.

Pry-out failure (CP-component)

The pry-out failure mechanism is analogous to the concrete cone failure, the maximum strength (rigid) can be derived from the following equation:

$$F_{CP} = 0.75 \cdot k_8 \cdot F_{\max,CC}$$

where $k_8 = 2$ ($h_{cp} > 60$ mm).

Reinforcements failure (RS₁, RS₂, RS₃-component)

Following the suggestions of InFaSo Design Manual (Kuhlmann et al. 2014) the reinforcement behaviour could be defined by a rectangular-parabolic curve. In this research it was simplified and modelled as elastic-perfectly plastic, where the yielding point is individuated by the following equations:

$$F_{y,RS} = n_s \cdot \frac{\pi \cdot \varnothing_s^2}{4} \cdot f_{y,r}$$

$$\delta_{y,RS} = \frac{2 \cdot F_{y,RS}^2}{12100 \cdot f_{ck,cube} \cdot \varnothing_s^4 \cdot n_s^2}$$

where n_s is the number, $\varnothing_s$ the diameter and $f_{y,r}$ ($=391.5$ MPa) the yielding strength of rebars. For the RS₂-component, which has a local influence, the number of rebars can be obtained considering the bars that intersect the 45°-inclined cones from the head of the studs. In case of RS₁ and RS₃-component, whose have a wider influence, the number of rebars was obtained through the observation of numerical model results obtained from the configurations SRCW-1 and SRCW-2.

The following tables resume the numerical values obtained for the component 5 in the validation phase for the SRCW-g and SRCW-h configurations.

SRCW2-like macro-models

Vertical tie	Vertical reinforcement rebars	n_s	2	$\varnothing_s$	8 mm
			6		6 mm
	Rebars Failure	$F_{y,RS1}$	= 105.705 kN		
		$\delta_{y,RS1}$	= 0.35 mm		
		(assumed from the numerical results obtained from the SRCW-2 FE model, see Fig. 15)			
	Concrete Cone Failure	$F_{y,CC}$	= 143.323 kN		
		n_h	= 2		
		$\varphi_{A,N}$	= 0.143		
		$\varphi_{S,N}$	= 0.9		
		$\varphi_{EC,N}$	= 1		
		$\varphi_{RE,N}$	= 1		
		$\varphi_{m,N}$	= 1		
		$F_{max,CC}$	= 32.792 kN		
	Concrete Pry-Out Failure	F_{CP}	= 49.056 kN		
	Total Yielding Force	F_{RS1+CP}	= 154.761 kN		

SRCW2-like macro-models

Diagonal tie	Horizontal reinforcement rebars	n_s	7	$\varnothing_s$	6 mm
	Vertical reinforcement rebars	n_s	7	$\varnothing_s$	6 mm
	Total Rebars Area (diagonal)	224 mm ²			
	Rebars Failure	$F_{RS3} = 137.167$ kN $\delta_{y,RS3} = 2$ mm (assumed from the numerical results obtained from the SRCW-2 FE model, see Fig. 15)			

SRCW1-like macro-models

Corner link	Concrete Cone Failure	$F_{y,CC}^o = 143.323$ kN $n_h = 3$ $\varphi_{A,N} = 0.075$ $\varphi_{S,N} = 0.8$ $\varphi_{EC,N} = 1$ $\varphi_{RE,N} = 1$ $\varphi_{m,N} = 1$ $F_{max,CC} = 25.798$ kN			
		Studs Failure			
		$n_h = 3$ $F_{y,S} = 425.293$ kN $K_S = 992.351$ kN/mm $\delta_{y,S} = 0.43$ mm			
		Pull-Out Failure			
		$n_h = 3$ $\delta_{1P} = 0.01$ mm $\delta_{2P} = 0.08$ mm $F_P = 610.747$ kN $F_{1P} = 98.377$ kN $F_{2P} \cong 184.046$			
		Horizontal reinforcement rebars	n_s	8	$\varnothing_s$ 6 mm
		Vertical reinforcement rebars	n_s	2	$\varnothing_s$ 8 mm
				2	6 mm
	Total Rebars Area (diagonal)	271 mm ²			
	Rebars	$F_{RS2} = 106.103$ kN			
		$\delta_{y,RS} = 0.10$ mm			

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Data availability All data generated or analysed during this study are included in this published article.

Code availability Available upon request.

Compliance with ethical standards

Conflicts of interest The authors declare they have no financial interests.

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