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Finite element modelling and design of steel plate shear wall buckling-restrained by hat-section cold-formed steel members

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Abstract: Steel plate shear wall (SPSW) buckling-restrained by hat-section cold-formed steel is a new type of lateral bearing system that uses hat-section cold-formed steel members to restrain the buckling deformation of the SPSW. Integration of architectural and structural functions is achieved in this new shear wall system by externally bonding oriented strand board (OSB) and filling sound insulation material. A finite element (FE) model was developed and validated against seven experimental results in terms of failure modes, shear force-story drift angle curves and shear bearing capacities. Upon validation, an extensive parametric study was carried out to investigate the six key parameters including spacing of connecting bolts, geometries of hat-section steel, spacing of hat-section steel, steel material strength, width-height ratio of the SPSW and height-thickness ratio of the SPSW. Based on the 104 numerical results, new design rules are proposed to predict the stiffness, shear yielding strength and shear ultimate strength of this new type of buckling-restrained SPSW structure. By comparing the predicted results with experimental and numerical results, the proposed design method is found to be accurate and consistent. In addition, an equivalent cross-bar model is developed as a simplified substituted SPSW model to be used in theoretical modelling programmes in the future.

Keywords: Buckling-restrained; Cold-formed steel; Design rules; Numerical modelling; Steel plate shear wall

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34 1 Introduction

35 Steel plate shear wall (SPSW) is a new type of highly efficient lateral force resisting system
36 developed in the 1970s. Compared with the traditional reinforced concrete shear wall, the SPSW
37 enjoys the advantages of good ductility, superior seismic performance, light weight and high
38 construction efficiency, easy repairing, easy dismantling and recyclability [1-4]. In recent years, a
39 number of research have been carried out on the theory and test of the thin SPSW
40 (height-to-thickness ratio of steel plate (λ) >150). The research results show that there are some
41 deficiencies in the mechanical performance of thin SPSW, such as: the hysteresis loop “pinching”
42 phenomenon is caused by the out-of-plane buckling deformation of steel plate wall under cyclic
43 horizontal loading [5]; the noise and vibration of the deformation process affect the applicability
44 performance of the structure [6]; and the appearance of the tension field would bring additional
45 bending moment to the vertical edge component [7]. In order to change the insufficient seismic
46 behaviour caused by the excessive out-of-plane deformation of the steel plate, it is necessary to find
47 a reasonable technical measure to prevent the out-of-plane buckling of SPSW, for example the
48 welding stiffener constraint [8], the precast concrete cover constraint [5], the multi-ribbed grid
49 constraint [9], etc. The SPSW is mainly in the state of plane shear stress before shear buckling. With
50 the increase of the lateral shear deformation, the steel plate appears the external oblique buckling
51 deformation, and the lateral resistance of the structure is changed from the plane shear resistance to
52 the lateral resistance mode mainly borne by the oblique tension field. If the measures are taken to
53 reduce the surface deformation of the steel plate, the steel plate will have better shear resistance. The
54 reduction of out-of-plane deformation of the steel plate and the weakening of the tension field have
55 positive effects on the lateral stiffness, bearing capacity and energy dissipation capacity of the
56 structure. Therefore, it is of great significance to carry out the research on the buckling-restrained
57 SPWS structure.

58 In this study, from the perspective of buckling-restrained SPSW and integration of architectural
59 functional requirements of heat preservation, sound insulation, seepage and decoration of wall, a
60 new type of SPSW buckling-restrained by hat-section cold-formed steel is proposed, as shown in Fig.
61 1. The hat-section cold-formed steel members (Fig. 2) are installed vertically with bolt through
62 connection on both sides of the embedded SPSW. In order to prevent the hat-section steel members
63 participating in the force deformation with the SPSW and utilize their bending resistance effectively,
64 the size of bolt holes in the SPSW is slightly larger than the diameter of connecting bolts. The
65 hat-section cold-formed steel members have the advantages of light weight, easy manufacture,
66 convenient assembling and construction. Although there are many researches on the design method
67 of SPSW [10][13], the object of these researches is the pure SPSW, which may not be applicable to

the SPSW buckling-restrained by hat-section cold-formed steel proposed in this paper. Therefore, it is necessary to propose a design method for this new type of SPSW. Wang et al. [14] has carried out experimental research on seven specimens of SPSW buckling-restrained by hat-section cold-formed steel, however, the research only focuses on the preliminary qualitative exploration of the seismic performance of this new type of SPSW, which cannot be used as a reference for design. Therefore, in order to facilitate the practical application of this new type of SPSW and considering that the number of specimens considered for the test is too small, the parametric analysis is carried out based on the finite element (FE) model, and the design method of the SPSW buckling-restrained by hat-section cold-formed steel is proposed based on the FE analysis results.

In this paper, a FE model of this new type of buckling-restrained SPSW is presented. The model was validated against seven experimental results reported by Wang et al. [14]. Upon validation, a total of 104 FE results were generated in the parametric analysis. Based on the theoretical analysis and numerical results, a cross-bar model for calculating the stiffness and strength of the buckling-restrained SPSW is developed. Afterwards, a new design method is proposed to predict the stiffness, shear yielding strength and shear ultimate strength of buckling-restrained SPSW by hat-section cold-formed steel.

2 Summary of tests

A total of seven SPSW buckling-restrained by hat-section cold-formed steel members have been tested under low-cycle reversed shear load [14]. As illustrated in Fig. 1, the specimens were comprised of one embedded SPSW and hat-section cold-formed steel members on both sides, connected by M12 high-strength steel bolts. The dimensions of the embedded steel plates were all $1080 \times 1080 \times 2.66$ mm (height $\times$ width $\times$ thickness); the plates were connected to four sides of the loading device (see Fig. 3) by twelve M24 high-strength steel bolts of grade 10.9. The dimensions of oriented strand board (OSB) were $1000 \times 1000 \times 15$ mm (height $\times$ width $\times$ thickness). Note that the specimen NRSP was a pure SPSW as a reference comparison specimen. The detailed parameters of all specimens are reported in Ref. [14] and the corresponding material properties are shown in Table 1. The test was carried out by load-displacement hybrid control [14, 15].

3 Finite element modelling

The non-linear finite element (FE) analysis package ABAQUS 6.14 [16] was employed in the numerical investigation herein. For the FE model, geometric nonlinearity and material nonlinearity are considered, and Newton-Raphson method is used to solve the balance equation in ABAQUS/Standard analysis module. An automatic stabilization method is adopted to improve the convergence of the calculation. The loading of the experimental programme was reasonably

101 simplified in the modelling process to balance the efficiency and accuracy of the program calculation.
 102 A new FE model of the SPSW buckling-restrained by hat-section cold-formed steel members was
 103 developed, as illustrated in Fig. 4.

104 3.1 Material models

105 An ideal elastic-plastic constitutive model was adopted for the hat-section cold-formed steel
 106 members with the yield stress $\sigma_y = 345$ MPa. A two-stage cyclic skeleton constitutive was adopted
 107 for the embedded SPSW [17] based on the results of tensile coupon, as given by Eq. (1):

$$\tilde{\sigma} = \begin{cases} \tilde{\varepsilon} & , \tilde{\varepsilon} \leq 1 \\ \frac{\tilde{\varepsilon} + 5.796}{2.486 + 0.608 \cdot (\tilde{\varepsilon} + 5.796) - 0.00046 \cdot (\tilde{\varepsilon} + 5.796)^2} & , \tilde{\varepsilon} > 1 \end{cases} \quad (1)$$

108 where, $\tilde{\varepsilon} (\varepsilon = \varepsilon / \varepsilon_y)$ and $\tilde{\sigma} (\tilde{\sigma} = \sigma / \sigma_y)$ are the regularized strain and stress respectively; ε_y is the
 109 yield strain, σ_y is the yield stress.

110 The stress and strain curves of steel materials were determined from tensile coupon tests. For
 111 FE model, the true stress σ_{true} and the plastic strain ε_{pl} are calculated based on nominal (engineering)
 112 strain ε_{nom} and nominal (engineering) stress σ_{nom} , as shown in Eqs. (2) and (3), where E is the
 113 Young's modulus.

$$\sigma_{true} = \sigma_{nom} (1 + \varepsilon_{nom}) \quad (2)$$

$$\varepsilon_{pl} = \ln(1 + \varepsilon_{nom}) - \frac{\sigma_{true}}{E} \quad (3)$$

114 3.2 Finite element type and mesh

115 The two columns of the loading device were considered as rigid body, so beam element (B31) was
 116 employed for the simulation. Shell element (S4R) was used for both embedded SPSW and
 117 hat-section cold-formed steel members as widely employed for metallic structures [18, 19]. After the
 118 mesh sensitivity analysis with consideration of the balance of mesh density and time cost, the final
 119 mesh size was selected as 10×10 mm.

120 3.3 Loading and boundary conditions

121 A uniaxial simplified constitutive model of steel under cyclic load was adopted and the hysteretic
 122 cyclic loading can be simulated by monotone loading [17]. A reference point RP1 (see Fig. 4) was
 123 established, and the translational degrees of freedom of U1, U2 and U3 at the top of the column on
 124 both sides of the loading device and the shear wall on the steel plate were coupled to reference point
 125 RP1. Horizontal lateral displacement (U1) was applied to the reference point RP1 and the vertical

translational degree of freedom U2 was released. Meanwhile, the translational degree of freedom U3 and three rotational degrees of freedom UR1, UR2 and UR3 were fixed (see Fig. 4).

At the bottom of columns, the one-way hinge constraint which only releases the rotational degree of freedom in the vertical plane (UR3) was adopted to achieve the four-side hinged connection of the loading device. The columns were connected to both sides of the embedded SPSW by "Tie" constraints. The bottom edge of the steel plate was constrained by fixing the translational degrees of freedom. The translational freedom U3 within the range of 60 mm on the four edges of the embedded steel plate (i.e. the outer edge range of the nut of the high-strength bolt in the test) was restricted to consider the constrain effect of fishtail plate on the embedded steel plate in the loading process.

3.4 Contact interaction and interfaces

The "surface-to-surface" contact was used to simulate the interaction between the hat-section cold-formed steel members and the SPSW. The master surface was defined as the SPSW surface and the slave surface was chosen as the surface of hat-section cold-formed steel member [20]. In the tangential direction, the contract surface was considered as frictionless.

The sliding connection bolts were simulated by "Connector" in ABAQUS. The connectors were adopted at the bolt geometric centre of hat-section cold-formed steel member and the SPSW. The hat-section steel members and SPSW were not allowed to separate in the normal direction; in the tangential direction the relative sliding range was allowed to simulate the partially sliding connection bolts.

3.5 Initial geometric imperfection

The thickness of the steel plate in the SPWS structure was small, and corresponding out-of-plane stiffness was also small. Therefore, in the process of manufacture, transportation and installation, initial geometric imperfections on steel plates and hat-section steel members are unavoidable. This study considers the initial geometric imperfection of the specimens by conducting eigenvalue buckling analysis subjected to the monotonic loading. The first buckling mode obtained from FE modelling of each specimen is shown in Fig. 5. The magnitude of the imperfection was taken as 1/1000 of the height of the SPSW.

3.6 Validation of FE model

Fig. 6 shows the failure modes of SPSW obtained by test and FE model. The cross fold and tear of SPSW observed in the test also occurred at the corresponding place of the FE results. Fig. 7 shows the comparison between the shear force-story drift angle curves obtained by the FE simulation

monotone loading and the hysteresis loading test. It can be seen that experimental and FE curves of all specimens agree well. Table 2 summarises the shear yielding strength and shear ultimate strength by test [14] and FE modelling. According to Ref. [15], the drift ratio at yielding is determined by the yield inflection point of skeleton curve and the ultimate drift angle is the displacement angle corresponding to 85% of the ultimate load. To conclude, good agreements between the experimental and FE results have been achieved in terms of failure modes, shear-story drift angle curves and shear strength. The accuracy of the new FE model simulating the loading performance of buckling-restrained SPSW is demonstrated.

For the test results, according to Ref. [15], the stiffness of the specimen is the initial stiffness before the yield of the specimen. According to the skeleton curve of the specimen obtained from the test results, the yield bearing capacity of the specimen is the load corresponding to obvious yield on the skeleton curve. The ultimate bearing capacity of the specimen is the peak load on the skeleton curve. For the FE results, the stiffness of the specimen is the elastic stiffness of the load-displacement curve obtained by FE analysis. The yield bearing capacity is the load corresponding to the obvious yield in the load-displacement curve of the specimen. Because it is difficult to simulate the damage and fracture of steel plate, the load-displacement curve of the specimen obtained by FE analysis does not show a descending stage. Therefore, according to Ref. [21], defined as the load at the displacement angle of $\theta_u=0.02$. According to the load-displacement curve of the experimental results, the bearing capacity of the specimen the displacement angle of $\theta_u=0.02$ is less than its ultimate bearing capacity. Therefore, the method to determine the ultimate bearing capacity of the specimen calculated by FE method is conservative and feasible.

4 Parametric study

4.1 General

The FE model validated against test results was employed in the parametric study to extensively investigate key parameters including spacing of connection bolts, section geometries and spacing of the hat-section cold-formed steel members, strengths of steel materials, width-height ratio and height-thickness ratio of the embedded SPSW. In order to better reflect the structural characteristics of the structure, a simplified two-stage skeleton of $F-\theta$ curve model is proposed (see Fig. 8). Effects of the aforementioned parameters on the lateral stiffness (K_e), shear yielding strength (F_{ym}) and shear ultimate strength (F_{um} , defined as the load at the displacement angle of $\theta_u=0.02$ [21],) of the SPSW buckling-restrained by hat-section cold-formed steel members are discussed in this section.

189 4.2 Spacing of connection bolts

190 In parametric study, the benchmark construction FE model of SPSW with dimensions of $1080 \times$
191 1080×2.66 mm (Height $H \times$ width $L \times$ thickness t) was employed, and the spacing of connection
192 bolts s_m is considered from 120 to 840 mm. As shown in Fig. 9, with the increase of the spacing of
193 connection bolts s_m : the stiffness K_e and the shear yielding strength F_{ym} are basically unchanged;
194 whereas, the shear ultimate strength F_{um} is rather stable when the spacing of connection bolts s_m is
195 smaller than 168 mm (i.e. $s_m/t_m = 112$, where, t_m is the thickness of hat-section cold-formed steel
196 member), and the shear ultimate strength F_{um} is gradually decreased when the spacing of connection
197 bolts s_m is greater than 168 mm. The spacing of connection bolts affected the restraint effect of the
198 hat-section steel members to the embedded SPSW. According to the analysis results, it seems that
199 when the s_m/t_m became larger than 112, F_{um} would decrease gradually. Therefore, it is recommended
200 to limit the spacing of connection bolts s_m in the structure of SPSW buckling-restrained by
201 hat-section cold-formed steel members as shown in Eq. (4):

$$s_m / t_m \leq 112 \quad (4)$$

202

203 4.3 Geometries of the hat section

204 In parametric study, the benchmark construction FE model of SPSW with dimensions of $1080 \times$
205 1080×2.66 mm (Height $H \times$ width $L \times$ thickness t) was employed.

206 4.3.1 Length of the crimping

207 In parametric study, the length of the crimping l_m of the hat-section (as shown in Fig. 2) is
208 considered from 0 to 40 mm. Results summarized in Fig. 10 shows that the length of the crimping l_m
209 has limited effect on the stiffness K_e and shear yielding strength F_{ym} of the shear wall structure. The
210 shear ultimate strength of F_{um} of structure improves slightly with increasing of l_m , and F_{um} remains
211 basically unchanged when l_m is larger than 20 mm. According to the height-thickness ratio limit of
212 T-section flexural members [22], the length of the crimping of hat-section cold-formed steel member
213 is suggested to be limited by Eq. (5). If the l_m is larger than this limit, the crimping might buckle
214 locally first, which would greatly reduce the out-of-plane restraint effect on the steel plate.

$$l_m / t_m \leq 15\sqrt{235 / f_y} \quad (5)$$

215 4.3.2 Section geometries

216 The geometries of the hat-sections are expected to have significantly influences on the
217 buckling-restrained SPSW. Therefore, the section height h_m (30-90 mm) and the thickness t_m (0.5-4

mm) are considered in the parametric study. As shown in Fig. 11(a), when the section height $h_m=90$ mm, as the thickness t_m of the hat-section increases: the stiffness K_e and the shear yielding strength F_{ym} are basically unchanged; the shear ultimate strength F_{um} increase slightly when the thickness t_m is smaller than 2.5 mm and keeps the same when the thickness t_m is greater than 2.5 mm,. As can be seen from Fig. 11(b), when the thickness is kept constant as $t_m=1.5$ mm , the section height h_m has no effect on the curve stiffness K_e and shear yielding strength F_{ym} , but positive effects on the shear ultimate strength F_{um} . Fig. 11(c) presents the shear ultimate strength F_{um} with varying section height h_m and cold-formed steel thickness t_m .

A bending stiffness ratio η , defined as the ratio of the out-of-plane bending flexural rigidity of the hat-section members and that of the embedded SPSW on the unit width (given by Eq. (6)), is introduced herein to quantify the restraining effect of the hat-section cold-formed steel member.

$$\eta = \frac{E_m I_m}{Dl} \quad (6)$$

$$D = \frac{Et^3}{12(1-\nu^2)} \quad (7)$$

where, D is the out-of-plane bending flexural rigidity per unit width of embedded SPSW (given by Eq. (7), where E , t and ν are the elastic modulus, thickness and Poisson's ratio of embedded steel plates, respectively); E_m is the elastic modulus of the hat-section cold-formed steel member; l is the width of embedded steel plate restrained by a pair of hat-section members; I_m is the sum of the moment of inertia of the hat-section members.

The relationship between the shear ultimate strength F_{um} and the bending stiffness ratio η is shown in Fig. 11(d). When the thickness of the cold-formed steel is small, as the bending stiffness ratio η of the section increases, the shear ultimate strength F_{um} of the structure improved slightly. For the same bending stiffness ratio η , the larger of the cold-formed steel thickness t_m lead to a higher corresponding structural shear ultimate strength F_{um} . Fig. 11(d) shows that the shear ultimate strength F_{um} reaches its upper bound when the bending stiffness ratio η is larger than 211.4 (corresponding to specimen 90-2.5). Based on the above analysis results, it is recommended to limit the bending stiffness ratio as follows:

$$\eta = \frac{E_m I_m}{Dl} \geq 210 \quad (8)$$

Besides, in order to prevent local buckling of the hat-section members, some limits on the cross-sectional dimensions are suggested as follows, with reference to the width-thickness limit reported by Cui and Long [22]:

$$b_{lm} / t_m \leq 40\sqrt{235 / f_y} \quad (9)$$

$$b_{2m} / t_m \leq 30\sqrt{235 / f_y} \quad (10)$$

$$h_m / t_m \leq 60\sqrt{235 / f_y} \quad (11)$$

where, b_{1m} , b_{2m} and h_m are the flange width, the lower flange width and the web height of the hat section, respectively, as shown in Fig. 2.

4.4 Spacing of constraints

A FE model of SPSW with dimensions of $3000 \times 3000 \times 6$ mm (Height $H \times$ width $L \times$ thickness t) was employed to investigate the effects of the spacing of hat-section members (S), ranging from 375, 430, 500, 600, 750, and 1000 mm. As illustrated in Fig. 12, some conclusions can be drawn: (1) before the structure yields, the decreasing spacing leads to slight increase of the shear yielding strength F_{ym} , but no change on the stiffness K_e ; (2) after yielding, the structure enters an elastic-plastic stage, and an out-of-plane buckling tension field occurs in the steel plate between two hat-section cold-formed steel members; (3) the spacing S has negligible effect on the shear ultimate strength F_{um} when the spacing S was less than 450 mm (accordingly, $S/t=72$). According to the analysis results, considering the influence of steel strength, it is recommended that the spacing of constraints is limited by Eq. (12):

$$S / t \leq 72\sqrt{235 / f_y} \quad (12)$$

4.5 Yield strength of steel material

The yield strengths of the cold-formed steel materials of the hat-section members (f_{ym}) were considered as 235, 345, 390 and 420 MPa, and the benchmark construction FE model of SPSW with dimensions of $3000 \times 3000 \times 6$ mm (Height $H \times$ width $L \times$ thickness t) was employed. The FE results are shown in Fig. 13. It is found to have limited influence on the lateral performance of the structure.

Similarly, the yield strengths of the steel material of embedded SPSW (f_y) were considered as 235, 345, 390 and 420 MPa. It can be seen, as shown in Fig. 14, that the yield strength f_y of the embedded steel wall steel has no effect on the structural stiffness K_e . With the increase of the yield strength of steel material, the shear yielding strength F_{ym} increases consistently. That was probably because when spacing S is within the reasonable range (see Eq. (12)), the in-plane buckling deformation did not occur before the yielding of the embedded steel wall.

4.6 Geometries of the SPSW

A FE model of SPSW with dimensions of $3000 \times 3000 \times 6$ mm (Height $H \times$ width $L \times$ thickness t)

was employed to set as the benchmark construction model.

4.6.1 Width-height ratio

The width L of the embedded SPSW varied from 2250 mm to 5250 mm with an interval of 750 mm, and the corresponding width-height ratios α (equal to L/H) are 0.75, 1, 1.25, 1.5 and 1.75. As can be seen from Fig. 15(a)-(c), the structural stiffness K_e , the shear yielding strength F_{ym} and shear ultimate strength F_{um} all increase linearly with of the width-height ratio. In order to evaluate the utilization efficiency of steel materials of SPSW, an average shear stress of the cross section of the SPSW ($\bar{\tau}_{um}$) is defined as:

$$\bar{\tau}_{um} = \frac{F_{um}}{tL} \quad (13)$$

Fig. 15(d) shows the relationship between the width-height ratio α and the average shear stress $\bar{\tau}_{um}$, which generally follows a parabolic trend.

4.6.2 Height-thickness ratio

In order to investigate effects of the height-thickness ratio of SPSW (λ , defined as H/t), the thickness of the embedded SPSW (t) are varied from 5 mm to 12 mm, resulting in the corresponding height-thickness ratio from 250 to 600. As illustrated in Fig. 16, effects of height-thickness ratio are similar to the width-height ratio of SPSW - for larger height-thickness ratios, the structural stiffness K_e , the shear yielding strength F_{ym} and shear ultimate strength F_{um} all became greater, but the average shear stress $\bar{\tau}_{um}$ becomes smaller.

5 Development of design rules

Based on parametric analysis results, for SPSW buckling-restrained by hat-section cold-formed steel members, it is found that the governing parameters of the structural stiffness and shear strengths are the yield strength, width-height ratio and height-thickness ratio of the embedded SPSW. It was assumed that the hat-section cold-formed steel members, that satisfies the requirements of Eqs. (4)-(12), can fully exert the buckling constraint effect on the embedded SPSW. In this study, 104 models of SPSW buckling-restrained by hat-section cold-formed steel members satisfying Eqs. (4)-(12) were established. Within the range of steel strength grade from Q235 to Q420, width-height ratio from 0.6 to 2.5, and height-thickness ratio from 250 to 600, based on the theoretical with numerical analysis, a design method of the buckling-restrained SPSW structure is proposed in this section. The two-stage skeleton curve of the lateral force resistance of the reaction structure shown in Fig. 8 which can be determined by calculating the stiffness K_e , the shear yielding strength F_{ym} and the shear ultimate strength F_{um} of the buckling-restrained SPSW structure.

302 5.1 Stiffness

303 As shown in Fig. 8, at the elastic stage, the hat-section members and the SPSW slide in the plane,
 304 and the lateral stiffness of the structure is provided by the shear deformation of the SPSW. According
 305 to the Hooke's law [23], the relationship between the shear stress τ and the shear strain γ of steel
 306 plate are described by Eq. (14):

$$\tau = G \cdot \gamma \quad (14)$$

307 where, G is the shear modulus of steel ($7.9 \times 10^4 \text{ N/mm}^2$).

Before yielding
$$\tau = \frac{F}{L \cdot t} = G \cdot \gamma = G \cdot \theta \quad (15)$$

Structural stiffness
$$K_e = \frac{F}{\theta} = G \cdot L \cdot t \quad (16)$$

308 5.2 Shear yielding strength

309 For buckling restrained SPSW, the yield strength might be enhanced by the external members (i.e.
 310 the hat-section cold-formed steel members in this study). However, considering the effect of cyclic
 311 hardening of buckling-restrained SPSW structure in practice is not as good as that observed in test,
 312 therefore, it is suggested to ignore this enhancement in practical design for safety reasons. That is to
 313 say, the shear yielding strength of the SPSW buckling-restrained by hat-section cold-formed steel
 314 members structure (F_{ym}) is calculated by Eq. (17). It should be noted that this conclusion is restricted
 315 to buckling-restrained SPSW structures which satisfied the requirements of Eqs. (4)-(12).

$$F_{ym} = F_y = \frac{f_y}{\sqrt{3}} \cdot l_w \cdot t_w \quad (17)$$

316 5.3 Shear ultimate strength

317 Similarly, a ratio (ξ_m) of the structural shear ultimate strength F_{um} to the nominal shear yielding
 318 strength F_y is introduced (see Eq. (18)) to reflect the improvement of the shear ultimate strength of
 319 the structure owing to the contribution of hat-section steel members.

$$\xi_m = \frac{F_{um}}{F_y} \quad (18)$$

320 Considering the effects of the governing parameters such as the yield strength f_y , the
 321 width-height ratio α and the height-thickness ratio λ of SPSW, new design rule for the variable ξ_m is
 322 proposed based on the 104 FE parametric analysis results, (see Eq. (19)). Therefore, the shear
 323 ultimate strength F_{um} of the SPSW buckling-restrained by hat-section steel members is suggested to
 324 be calculated by Eq. (20):

$$\xi_m = 0.101 \left(3.595 - \sqrt{\frac{f_y}{235}} \right) (5.026 + 0.166^\alpha) \left(0.216 + 0.97^{\frac{\lambda}{300}} \right) \quad (19)$$

$$F_{um} = \xi_m \cdot \frac{f_y}{\sqrt{3}} \cdot L \cdot t \quad (20)$$

5.4 An equivalent cross-bar model

In the global analysis of high-rise buildings, a cross-bar model is normally used to simplify the SPSW structure. In this study, a cross-bar model (see Fig. 17) of the SPSW buckling-restrained by hat-section cold-formed steel members is proposed to provide equivalent structural stiffness and strength, which could be used in the computational analysis programmes such as SAP2000 and ETABS, etc. The member AB is identical to the member CD, oblique to each other and hinged to the upper and lower frames at both ends.

Fig. 18 shows the deformation of cross-bar model under a lateral force F , i.e. the two bar ends C and B generate the same horizontal lateral displacement Δ , and two bars shift to AB' and C'D respectively after deformation. When the cross-bar produces a horizontal side shift Δ , a certain axial deformation δ and a reaction force of the bars AB and CD are generated. According to the small deformation geometric relationship and the material mechanics, the relationship between force F and horizontal deformation Δ is described by Eqs. (21)-(27).

Assuming that the horizontal displacement is much less than the length of the bar, then,

$$\delta = \Delta \cdot \cos \theta = \Delta \cdot \frac{L}{\sqrt{L^2 + H^2}} \quad (21)$$

Then, the strain ε of the cross-bar is:

$$\varepsilon = \frac{\delta}{\sqrt{L^2 + H^2}} = \frac{L}{L^2 + H^2} \cdot \Delta \quad (22)$$

The axial force of the bar AB' is:

$$F_{AB'} = E_s \varepsilon A = \frac{L}{L^2 + H^2} \cdot E_s A \cdot \Delta \quad (23)$$

Hence,

$$F_{AB'} \cdot \cos \theta = \frac{L^2}{(L^2 + H^2)^{\frac{3}{2}}} \cdot E_s A \cdot \Delta \quad (24)$$

Similarly,

$$F_{DC'} = E_s \varepsilon A = \frac{L}{L^2 + H^2} \cdot E_s A \cdot \Delta \quad (25)$$

$$F_{DC'} \cdot \cos \theta = \frac{L^2}{(L^2 + H^2)^{\frac{3}{2}}} \cdot E_s A \cdot \Delta \quad (26)$$

To sum up,

$$F = F_{AB} \cdot \cos \theta + F_{DC} \cdot \cos \theta = \frac{2L^2}{(L^2 + H^2)^{\frac{3}{2}}} E_s A_s \cdot \Delta \quad (27)$$

where, A_s and E_s are the section area and elastic modulus of the bars, respectively.

Eqs. (21)-(27) defines the elastic characteristic of the cross-bar model. In order to define a cross-bar model for determining the characteristics of buckling-restrained SPSW structure after yielding, the following key parameters should be provided: the structural stiffness K_e (see Eq. (16)), the yield shear strength F_{ym} (see Eq. (17)), the ultimate shear strength F_{um} (see Eq. (20)) and the ultimate displacement angle $\theta_u=0.02$ of buckling-restrained SPSW structure; the width L , the height H , the material elastic modulus E_s , the reinforced modulus E_{s1} , the yield stress f_{ys} , the ultimate strength f_{us} , and the cross-sectional area A_s of the bars.

According to the Fig. 8, Eqs. (16) and (27), the structure stiffness K_e is expressed as:

$$K_e = \frac{F}{\theta} = \frac{F}{\Delta/H} \quad (28)$$

$$K_e = \frac{F}{\Delta/H} = \frac{2L^2 \cdot H}{(L^2 + H^2)^{\frac{3}{2}}} E_s A_s \quad (29)$$

Thus, the cross-sectional area of the bar can be determined:

$$A_s = \frac{(L^2 + H^2)^{\frac{3}{2}}}{2L^2 \cdot H \cdot E_s} \cdot K_e \quad (30)$$

When the buckling-restrained SPSW structure yields, the yield strain ε_y of the cross-bar is:

$$\varepsilon_y = \frac{L}{L^2 + H^2} \cdot \Delta_y = \frac{L}{L^2 + H^2} \cdot \theta_y \cdot H = \frac{L \cdot H}{L^2 + H^2} \cdot \frac{F_{ym}}{K_e} \quad (31)$$

Afterwards, the material yield strength f_{ys} of the cross-bar can be determined:

$$f_{ys} = E_s \varepsilon_y = \frac{E_s \cdot L \cdot H}{L^2 + H^2} \cdot \frac{F_{ym}}{K_e} \quad (32)$$

The two-stage stress-strain relationship was employed for cross-bar material and the reinforced modulus E_{s1} of the cross-bar material is:

$$E_{s1} = E_s \cdot \frac{K_p}{K_e} = \frac{E_s}{K_e} \cdot \frac{F_{um} - F_{ym}}{\theta_u - F_{ym}/K_e} \quad (33)$$

The ultimate strength f_{us} of the cross-bar material is:

$$f_{us} = \frac{F_{um}}{F_{ym}} \cdot f_{ys} \quad (34)$$

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360 6 Result comparisons

361 New design equations for the prediction of the stiffness, shear yielding strength and shear ultimate
362 strength of this new type of SPSW structure have been developed in Section 5. Fig. 19 shows the

comparison the structural stiffness K_e between numerical and predicted results. Good agreement is achieved with the mean value of K_{e1}/K_{e2} is found to be 1.02.

As shown in Fig. 20, for the specimens NRSP1, NRSP2, NRSP3 and NRSP6 which are satisfying Eqs. (4)-(12), the predicted results of F_{um} are in good agreement with numerical (and experimental) results of F_{um} with good accuracy and consistency - the mean value of the predicted results to numerical (and experimental) results is 1.001 and the corresponding coefficient of variation (COV) = 0.0069, variance = 0.009.

7 Conclusions

A finite element model of the steel plate shear wall (SPSW) buckling-restrained by hat-section cold-formed steel members was developed and validated against the existing experimental results in terms of failure modes, shear force-story drift angle curves and shear strength. In the parametric study, a total of 104 numerical results were generated to consider six key parameters of this new buckling-restrained SPSW structure, including spacing of connecting bolts, geometries of hat-section steel, spacing of hat-section steel, steel material strength, width-height ratio of the SPSW and height-thickness ratio of the SPSW. Numerical results show that: (1) effects of all the aforementioned parameters on the structural stiffness of the buckling-restrained SPSW structure are limited; (2) with the increase of the width-height, the shear yielding strength and shear ultimate strength of SPSW increase linearly; (3) the shear yielding strength and shear ultimate strength increase as the height-thickness ratio decreases; (4) with the decrease of the spacing of hat-section cold-formed steel members, the shear yielding strength improves slightly, but its effect on the shear ultimate strength is negligible when it is less than 450 mm. Based on numerical study and theoretical analysis, requirements on the geometries of this new buckling-restrained SPSW structure are suggested to ensure effective buckling-restraint on the SPSW. Within the range of steel strength grade from Q235 to Q420, width-height ratio from 0.6 to 2.5, and height-thickness ratio from 250 to 600, new design equations for the prediction of structural stiffness, shear yielding strength and shear ultimate strength of this new buckling-restrained SPSW structure are proposed. In addition, a cross-bar model as a simplified equivalent model for the SPSW buckling-restrained by hat-section cold-formed steel members is proposed, which could be used in the computational analysis programmes such as SAP2000 and ETABS, etc. By comparing the predicted results with experimental and numerical results, the proposed design method is found to be accurate and consistent.

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403

404 Nomenclature

λ	height-to-thickness ratio of steel plate
σ_y	the yield stress of steel materials
$\tilde{\varepsilon}$	the regularized strain of steel materials
$\tilde{\sigma}$	the regularized stress of steel materials
ε_y	the yield strain of steel materials
σ_{true}	the true stress of steel materials
ε_{pl}	the plastic strain of steel materials
ε_{nom}	the nominal (engineering) strain of steel materials
σ_{nom}	the nominal (engineering) stress of steel materials
E	the Young's modulus of steel materials
K_e	the lateral stiffness of steel plate shear wall
F_{ym}	the shear yielding strength of steel plate shear wall
F_{um}	the shear ultimate strength of steel plate shear wall
θ_u	the displacement angle of buckling-restrained SPSW structure
s_m	the spacing of connection bolts
t_m	the thickness of hat-section cold-formed steel member
l_m	the length of the crimping of the hat-section
h_m	the section height of the hat-section
η	the ratio of the out-of-plane bending flexural rigidity of the hat-section members and that of the embedded SPSW on the unit width
D	the out-of-plane bending flexural rigidity per unit width of embedded steel plate shear wall
t	the thickness of embedded steel plates
ν	the Poisson's ratio of embedded steel plates

E_m	the elastic modulus of the hat-section cold-formed steel member
l	the width of embedded steel plate restrained by a pair of hat-section members
I_m	the sum of the moment of inertia of the hat-section members
b_{1m}	the flange width of the hat section
b_{2m}	the lower flange width of the hat section
h_m	the web height of the hat section
H	the height of steel plate shear wall
L	the width of steel plate shear wall
S	the spacing of steel plate shear wall
f_y	the yield strength of steel plate shear wall
$\bar{\tau}_{um}$	the average shear stress of the cross section of steel plate shear wall
τ	the shear stress of steel plate
γ	the shear strain of steel plate
G	the shear modulus of steel
ζ_m	the ratio of the structural shear ultimate strength F_{um} to the nominal shear yielding strength F_y
α	the width-height ratio of steel plate shear wall
F	the lateral force cross-bar model
Δ	the horizontal lateral displacement of steel plate shear wall
δ	the axial deformation of the bars
A_s	the section area of the bars
E_s	the elastic modulus of the bars
E_{s1}	the reinforced modulus
f_{ys}	the yield stress of the bars
f_{us}	the ultimate strength of the bars

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406 **References**

- 407 [1] Dowden DM, Bruneau M. Quasi-static cyclic testing and analytical investigation of steel plate
408 shear walls with different post-tensioned beam-to-column rocking connections. Engineering
409 Structures, 2019, 187:43-56.
- 410 [2] Liu WY, Li GQ, Jiang J. Mechanical behavior of buckling restrained steel plate shear walls with
411 two-side connections. Engineering Structures, 2017, 138:283-292.

- 412 [3] Bypour M, Gholhaki M, Kioumarsi M , et al. Nonlinear analysis to investigate effect of
413 connection type on behavior of steel plate shear wall in RC frame. *Engineering Structures*, 2019,
414 179:611-624.
- 415 [4] Ozcelik Y, clayton PM. Seismic design and performance of SPSWs with beam-connected web
416 plates. *Journal of Constructional Steel Research*, 2018, 142: 55-67.
- 417 [5] Guo YL, Zhu JS QL, Zhou P, Zhu BL. A new shuttle-shaped buckling-restrained brace.
418 Theoretical study on buckling behavior and load resistance. *Engineering Structures*, 2017,
419 147:233-241.
- 420 [6] Driver RG, Kulak GL, Kennedy DJL, et al. Cyclic Test of Four-story Steel Plate Shear Wall.
421 *Journal of Structural Engineering*, ASCE, 1998, 124(2): 112-120.
- 422 [7] Lubell AS, Helmut GL, el al. Unstiffened Steel Plate Shear Wall Performance Under Cyclic
423 Loading. *Journal of Structural Engineering*, 2000,126(4):453-460.
- 424 [8] Takanashi Y, Takemoto T, Tagaki M. Experimental Study on Thin Steel Shear Walls and
425 Particular Bracing Under Alternative Horizontal Load//Resistance and Ultimate
426 Deformability of Structures Acted on by Well-Defined Repeated Loads. Portugal:1973.
- 427 [9] Ge ML, Hao JP, Yu JG, Yan PZ, Xu SC. Shaking table test of buckling-restrained steel plate
428 shear walls. *Journal of Constructional Steel Research*, 2017, 137:254-261.
- 429 [10] Thorburn LJ, Kulak GL, Montgomery CJ. Analysis of steel plate shear walls. Department of
430 civil engineering, the university of Alberta, Edmonton, Alberta, 1983.
- 431 [11] Yu JG, Yu HS, Feng XT, Dang C, Hou TF, Shen J. Behaviour of steel plate shear walls with
432 different types of partially-encased H-section columns. *Journal of Constructional Steel*
433 *Research*, 2020, 170:106123.
- 434 [12] Moghimi H , Driver R G . Performance-Based Capacity Design of Steel Plate Shear Walls. I:
435 Development Principles. *Journal of Structural Engineering*, 2014, 140(12):04014097.
- 436 [13] Moghimi H , Driver R G . Performance-Based Capacity Design of Steel Plate Shear Walls. II:
437 Design Provisions. *Journal of Structural Engineering*, 2014, 140(12):04014098.
- 438 [14] Wang YH, Gu CW, Tang Q, Shi Y, Xu L, Tan JK, Luo W. Experimental Study on Cyclic Pure
439 Shear Behaviour of Hat-Section Cold-formed steel members Buckling-Restrained Steel Plate
440 Shear Walls without Effect of Frame. *Engineering Structures*, 2019, 201:109799.
- 441 [15] JGJ 101-2015 Specification of testing methods for earthquake resistant building. Beijing: China
442 Architecture & Building Press,2015
- 443 [16] ABAQUS, ABAQUS/Standard Version 6.14 User's Manual vols. I-III, Dassault Systèmes
444 Simulia Corp, Rhode Island, USA, 2014.
- 445 [17] Shi YJ, Wang M, Wang YQ. Study on constitutive model of structural steel under cyclic loading.
446 *Engineering Mechanics*,2012,29(09):92-98+105.

- 447 [18] Chen MT, Young B. Behavior of cold-formed steel elliptical hollow sections subjected to
448 bending. *Journal of Constructional Steel Research*, 2019, 158: 317-330.
- 449 [19] Chen MT, Young B. Structural behavior of cold-formed steel semi-oval hollow section beams.
450 *Engineering Structures*, 2019, 185: 400-411.
- 451 [20] Su MN, Young B, Gardner L. Deformation-based design of aluminium alloy beams.
452 *Engineering Structures*, 2014, 80:339-349.
- 453 [21] JGJ99-2015 Technical specification for steel structure of tall building. Beijing: China building
454 industry press, 2015.
- 455 [22] Cui J, Long LP. Basic principles of steel structure. China building industry press, 2008.
- 456 [23] Liu DH, Huang C. material mechanics I. Chongqing university press, 2010.
- 457 [24] Computers and Structures, Inc. CSI analysis manual, Berkeley, California, USA, 2018.
- 458

Fig. 1 Steel plate shear wall buckling-restrained by hat-section cold-formed steel

Fig. 2 Dimensions of hat-section cold-formed steel members

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Fig. 3 Schematic drawing of the loading device

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Fig. 4 FE model of SPSW buckling-restrained by hat-section steel members

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(a) NRSP

(b) BRSP1

(c) BRSP3

(d) BRSP4

(e) BRSP5

Fig. 5 Buckling modes of specimens obtained from FE models

(a) Specimen NRSP

(b) Specimen BRSP1

(c) Specimen BRSP3

(d) Specimen BRSP4

(e) Specimen BRSP5

Fig. 6 Comparison of failure modes between test and FE modelling

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(a) Specimen NRSP

(b) Specimen BRSP1

(c) Specimen BRSP3

(d) Specimen BRSP4

(e) Specimen BRSP5

Fig. 7 Comparison of shear force-story drift angle curves between test and FE modelling

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Fig. 8 Two-stage skeleton of F - θ curve

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(a) F - θ curves

(b) F_{um} - s_m curve

Fig. 9 Effects of the spacing of connection bolts on the structure performance of SPSW

Fig. 10 Effects of the length of crimping on the structure performance of SPSW

(a) Thickness of the section t_m

(b) Height of the section h_m

(c) Spatial distribution of shear ultimate strength F_{um} of the structure

(d) bending stiffness ratio η

Fig. 11 Effects of the section design of hat-section steel members on the structure performance of SPSW

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(a) The effects of number of constraints pairs on the F - θ curves

(b) The F_{um} - s curve

Fig. 12 Effects of the spacing of hat-section steel members on the structure performance of SPSW

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Fig. 13 Effects of yield strength of hat-section cold-formed steel on the $F-\theta$ curves of SPSW

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Fig. 14 Effects of yield strength of embedded steel plates on the F - θ curves of SPSW

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(a) The F - θ curves

(b) The K_e - α curve

(c) The F - α curves

(d) The τ_{um} - α curve

Fig. 15 Effects of aspect ratio of embedded steel plates on the structure performance of SPSW

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(a) The F - θ curves

(b) The K_e - t curve

(c) The F - t curves

(d) The τ_{um} - λ curve

Fig. 16 Effects of height-thickness ratio of embedded steel plates on the structure performance of SPSW

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Fig. 17 Equivalent cross-bar model

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Fig. 18 Deformation of the cross-bar model

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Fig. 19 Comparison of the structural stiffness between predicted results with test and numerical results

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Fig. 20 Comparison between predicted results ζ_{Pred} with numerical (and experimental) results ζ_{FE}

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Table 1 Properties of steel materials (unit: MPa)

Components		Yield strength f_y	Ultimate strength f_u
SPSWs	NRSP		
	BRSP1	321.67	463.67
	BRSP2		
	BRSP3		
	BRSP4		
	BRSP5	303.67	439.33
	BRSP6		
Hat-section cold-formed steel members	BRSP1	427.33	551.00
	BRSP2	420.00	539.67
	BRSP3	425.67	550.00
	BRSP4		
	BRSP5	404.00	541.33
	BRSP6	430.67	556.00

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Table 2 Comparison of shear bearing capacities obtained from test [14] and FE modelling

	Specimens	NRSP	BRSP1	BRSP3	BRSP4	BRSP5	Mean	COV
Shear yielding strength	Test (kN)	637.7	654.2	650.2	622.2	601.9		
	FE (kN)	629.5	666.5	669.3	632.6	613.4		
	Test/FE	1.01	0.98	0.97	0.98	0.98	0.98	0.001
Shear ultimate strength	Test (kN)	686.7	792.4	800.0	732.5	729.3		
	FE (kN)	727.3	809.4	823.2	739.8	743.8		
	Test/FE	0.94	0.98	0.97	0.99	0.98	0.97	0.004