Adjusting the Plastic Zone Development in Steel Plate Shear Walls — A Finite Element Study

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Abstract The current study presents a high-performance steel plate shear wall system, which is capable of dissipating large amounts of energy. A specific arrangement of perforations in the infill panel as well as the adjacent horizontal boundary elements is proposed. The aim is to increase the ductility of the system by inhibiting the pre-mature failure of the critical structural components. To this end, a sacrificial structural element, i.e., the steel panel, is implemented into the frame system to adopt the main role of energy dissipation. To allow an extensive development of the plastic field in the steel panel, perforated areas are designated at each of its corners. Furthermore, the concept of web-reduced beam sections is applied. Two other patterns and a benchmark case were also introduced for comparison. It was shown that the proposed system excels in energy dissipation for the price of slight initial stiffness loss. Furthermore, the synergistic effect of the suggested modifications delays the initiation of inelastic deformations in the surrounding structural members. Thus, these components and their connections remain protected under extreme loading regimes. One could take advantage of the proposed arrangement if harsh loading conditions are plausible.

Keywords: Computational mechanics, non-linear analysis, web-reduced beam section, perforated steel wall, plasticity, ductile failure

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1 Introduction

The steel plate shear walls (SPSWs) are used as effective lateral force-resisting components in a structure. Each SPSW consists of a steel infill panel (web plate) that is confined by horizontal and vertical structural elements of the building. The stiffness of the panel and its surrounding members dictate the lateral response of the system [26]. A properly-designed SPSW can be very ductile with relatively large energy dissipation capabilities. Moreover, it has a comparatively large initial stiffness that restricts the storey drift. The SPSW, compared to its reinforced-concrete counterpart, is lightweight, and thus it is seismically favoured. Reducing the overall dead weight of the structure renders the SPSWs highly efficient and economical against lateral loads [2].

There are two classes of SPSWs: stiffened (S-SPSW) and unstiffened (U-SPSW). The behaviour of the SPSW is controlled by adjusting the stiffness of the infill panel. For instance, thick or stiffened web plates (S-SPSW) undergo plastic deformations with no geometric instability whereas their thin counterparts exhibit crucial post-buckling behaviour [3]. Until 1980s, most SPSW-equipped buildings were designed with stiffened infill panels to inhibit the out-of-plane buckling. The lateral load in these S-SPSW systems is resisted by an in-plane shear field. Several studies have proved that the S-SPSWs perform well under cyclic loads and can greatly increase the dissipated energy [28]. However, achieving such a performance requires thick panels with several stiffeners that altogether increase the construction cost. On the other hand in 1931, Wagner [31] demonstrated that in U-SPSWs, buckling is not synonymous with failure and that the post-buckling strength of a thin shear panel can be significant. Namely, although the buckling resistance of the infill plate is negligible, a post-buckling diagonal tension field is created in the panel that contributes significantly to lateral strength and ductility [3].

The aforementioned post-buckling strength elegantly marries the principles of capacity design. Following the latter concept, the critical components of the structure must be protected by means of the ‘intentional’ weak links. The post-buckling tension field action of U-SPSWs extends the localised load-bearing capacity of the structure [5], and provides the potential for such a protection. Namely, with careful design considerations, the boundary elements of the U-SPSWs could remain elastic under extreme loading conditions, e.g., at high-risk seismic zones [13].

The use of either classes of the SPSWs, increases the design force of the adjacent columns [5]. Since the design of the columns is affected by the thickness of the panel, over-sized elements might result [5]. In order to reduce the moments and axial loads on the surrounding frame, various adjustments to the geometry and/or the material properties of the SPSWs have been proposed. In terms of materials, using cold-formed light-gauge steel [4, 30], or using low yield-strength steel [10, 30, 22, 23] are among these methods. In terms of geometry, perforating the infill panel could alleviate the severity of the load demand [21]. These openings could take various geometrical patterns, e.g., central circular hole [25, 6], multiple circular openings [7], or rectangular openings [16] are possible. Among the proposed methods of reducing

the column force, perforated panels are favoured since they provide passage for utilities or room for window openings.

Several experimental studies have been carried out on the U-SPSWs that formed the basis for analytical models, see [8, 12, 29, 20]. These analytical models were proposed to capture their post-buckling behaviour, see [14, 11, 15, 27]. Thorburn et al. [29] introduced the ‘strip model’, which represents the resistance provided by the tension field of the panel upon buckling. This theory was later verified by Timler et al. [19] through some experiments. They conducted cyclic tests on U-SPSWs and reported great ductility and energy dissipating capacity for the system. The strip model has been adopted by the Canadian Standard CSA-S16-01 [9] as a simple approach for the analysis of U-SPSWs.

Considering the limitations of both analytical models in dealing with geometrical complexities and the cost and limitations of the experimental setups, computational methods seem to offer more flexibility [24]. Among various cost-effective numerical approaches, the robustness of the finite element (FE) method [17, 18] satisfies the demands from both industry and academia. More importantly, the FE method is used to validate the experimental results and provide a more comprehensive range of results. In the current context, Elgaali et al. [15] used some FE models to replicate the results obtained from a previous experiment [8]. It was shown that the panels with thicker plates are not necessarily stronger when the failure is controlled by the yielding of columns.

In the current study, it is hypothesised that implementing proper perforation patterns in the infill plate could protect other critical structural components. More specifically, it will be shown that the development of the plastic zone within the infill wall could be controlled. In the following section, the benchmark model is validated by an experimental result from the literature. Then, the rationale behind the proposed arrangement is clarified. The details of the FE analysis is the last part of Sec. 2. In the following section, the performance of various arrangements are discussed and compared against each other. Finally, the study concludes the key findings and provides some suggestions for future studies.

2 Methodology

2.1 Finite Element Prototype

In order to validate the FE modelling procedure, the numerical results were compared to those from a previously-conducted experiment in [12]. The setup consisted of a four-storey single-span U-SPSW frame with no perforations, see Fig. 1. All storeys have a height of 1829 mm and a span of 3050 mm except the first one that is 1927 mm high. The panel thickness was 4.8 mm and 3.4 mm for the lower two and upper two storeys, respectively. The dead load of the structure was simulated by exerting an axial force of 720 kN per column of the last floor. The FE prototype replicates this setup.

Fig. 1: Validating the model: (a) the schematic presentation of the experimental setup used in [12] (F is the applied lateral load), and (b) the corresponding FE model

The load-displacement graphs of the experimental model versus the FE prototype are illustrated in Fig. 2. The current FE results show an acceptable agreement with the experiment. The small discrepancy between the two responses can be attributed to the failure of the welded connections during the experiment [12]. Therefore, in the current study, the connections were deemed to be rigid. Moreover, since initiating plastic joints are inhibited at the connections, any specific connection details were neglected in the FE models.

2.2 Case Studies

Following the adopted approach for the prototype in the previous section, a series of single-storey single-span FE models were created using the ANSYS commercial FE package [1]. More specifically, four case studies were considered:

1. the benchmark case with no perforation (SPSW-A¹),
2. top boundary beam mid-span perforation only (SPSW-B),

¹ Note that all cases are U-SPSWs but for brevity, they are named only with the 'SPSW' prefix.

Fig. 2 Verification of the FE model with experiment

3. top and bottom boundary beam mid-span perforations (SPSW-C), and
4. the suggested pattern with both (mid-span) beam and panel openings (SPSW-D).

In Fig. 3, the meshed models are shown along with the incorporated perforation patterns. A symmetrical arrangement is adopted to elicit the same behaviour if the direction of the lateral loads inverts. The SPSW-A is the benchmarking model, which is a reduced version of the experimental model from the previous section.

Fig. 3: Finite element models of the case studies: (a) without perforations (SPSW-A), (b) with only top beam opening (SPSW-B), (c) with top/bottom beam openings (SPSW-C), and (d) the suggested pattern with panel and top/bottom beam openings (SPSW-D)

In order to meet the capacity design requirements, the structural boundary members should be designed in a way that the tension field could develop plastically in the infill plate. This consideration gives the frame ductility and toughness. On the other hand, it is assumed that the beams are rigidly welded to the columns, and thus any plastic deformation in the connection zone can cause fracture that may lead to a holistic failure. Namely, in the U-SPSW systems, the panel could overload the boundary elements causing the connections to fail. In summary, maximising the performance is achieved via addressing the following issues:

1. limiting the plastic strains to small values at joints,

Fig. 4 Schematic geometry of the WRBS and panel openings

2. allowing the development of the plastic strains in the infill plate and the horizontal frame members—away from the connection zone, and
3. avoiding the local buckling or occurrence of a plastic hinge in the columns.

In order to prevent brittle failure in moment-resisting joints, two remedies are feasible:

- strengthening the connections, and
- using the reduced beam sections (RBSs).

Herein, a special case of the RBSs is adopted to reduce the stiffness of the web at the mid-span—the so-called web-reduced beam section (WRBS). In order to re-route the stress path, it is suggested that some corner openings to be incorporated into the infill panel. The proposed perforations of the panel and the WRBS are schematically illustrated in Fig. 4; the corresponding parameters are listed in Table 1.

Table 1: Parameters of the panel and beam openings (h_w is the height of the beam web, and the numerical values are in millimetres)

Component	h_f	l_f	r_f	h_p	l_p	r_p
Beam web	700	$0.8h_w$	70	n/a	n/a	n/a
Panel	n/a	n/a	n/a	100	300	h_p

2.3 Finite Element Analysis

This research investigates the behaviour of four different U-SPSW systems for the purpose of increasing the ductility of the system under monotonically-applied lateral loading. An intermediate frame is selected to represent the typical repeating structure

in multi-storey buildings. In Table 2, the geometrical dimensions of the structural components are summarised.

Table 2: Geometry of the structural components (all numerical values are in millimetres)

Component	Flange		Web	
	Width	Thickness	Height	Thickness
Column	350	20	350	15
Beam	200	15	350	12
Panel	$2650 \times 1650 \times 3$			

The nonlinear push-over analyses were carried out on the introduced cases while the effects of large deflections and large strains were considered. The geometry was discretised by means of several finite strain iso-parametric shell elements (SHELL 181). Every element of this type has four nodes, each with six degrees of freedom (DoFs), and could properly represent thin to moderately thick shell-like entities. Moreover, it is well-suited to capture large rotations and/or finite strains.

It was assumed that all the connections are welded, hence the beam-column connection was considered rigid and the infill plate was directly connected to the structural boundary elements. In consequence, the beam-to-column connections experience large moments, which may cause local buckling. To circumvent this condition, the columns were equipped with stiffener plates.

The pushover analysis is carried out by applying a displacement-controlled monotonically increasing lateral load along the centre-line of the top beam. To simulate the restrictions imposed by the rigid floor slabs, the out-of-plane nodal translations of the beam at the roof and support levels were constrained. In addition, all the DoFs of the columns at the support level were constrained to simulate a fully fixed connection.

In order to enforce the formation of the diagonal tension field in the panel and to consider its fabrication imperfections, a small out-of-plane perturbation should be initially applied. The perturbation follows the eigenshape of the first buckling mode of the structure, which is obtained from the eigenvalue buckling analysis. Thus, the geometry of each model is perturbed according to its first eigenmode to alleviate the artificial over-stiffness of the ideally-flat plate and induce the diagonal tension field.

The von Mises yield criterion was used as the measure of plasticity to quantify the accumulation of plastic strains. Since this criterion is independent of the first stress invariant, it is a reasonable choice for incompressible ductile deformations. A multi-linear elasto-plastic material model with isotropic hardening is adopted for the structural components while the maximum plastic strain is conservatively limited to 14 percent, see Table 3. In addition to material nonlinearity, the geometrical nonlinearity sources were also included. The full Newton-Raphson method was used to solve the simultaneous system of nonlinear equations at each increment.

Table 3: Mechanical properties of the components

Component	Yield strength (MPa)	Failure strength (MPa)	Elastic modulus (GPa)
Column	314	481.7	202.991
Beam	333.7	478	203.947
Panel	342	456	208.835

3 Results and Discussions

The pushover analysis results of the four cases are shown in Fig. 5. From the force-displacement curves, the initial stiffness, ultimate strength, final drift (as the ductility measure), and energy dissipation are the chosen criteria to quantify the behaviour of different types of U-SPSWs.

Since the SPSW is a dual lateral load carrying system, both the infill plate and the boundary elements contribute to the overall stiffness. There is a reduction in the initial lateral stiffness of the U-SPSWs as the number of openings increases. Following this trend, the initial stiffness of the panels of type B, C and D were respectively 82.1, 74.6 and 72 percent of that of the benchmark type A. This compliant behaviour can be attributed to the loss of stiffness in the panel and boundary elements, which alters the wall-frame interactions.

Fig. 5: Pushover analysis results of the case studies

In spite of about 28 percent loss in the initial stiffness, the SPSW-D model was able to provide 94 percent of the benchmarking shear strength (that of SPSW-A). As discussed earlier, in an ideally-designed U-SPSW system, the infill wall should yield in tension while the columns and beam-column connections remain elastic under lateral loads.

Fig. 6: Distribution of the plastic strain upon failure in each case study: (a) without perforations, (b) with only top beam opening, (c) with top/bottom beam openings, and (d) the suggested pattern with panel and beam openings

The SPSW types A and B demonstrated the ultimate drift values of 1.2 and 1.4 percent, respectively. It is clearly illustrated in Figs. 6a and 6b that the beam-column connections controlled the failure, i.e. the panel could not work at its maximum capacity. In the SPSW-A, with no panel perforations and no WRBS, an almost diagonal tension field was formed. Consequently, the plastic field was mainly developed around the corners of the panel and the adjacent face of the column. Such a concentrated overstress resulted in a pre-mature failure of the beam-column connection, which is confirmed by the just over one percent final drift value. The SPSW-B took a similar position except that the plastic region was slightly developed towards the implemented WRBS. Therefore, a slight improvement was observed for SPSW-B in terms of ductility.

The SPSW-C is the intermediate case that reached a final drift of 3 percent. Although the ductility seems to be further improved, the failure mechanism was dominated by the connection between the panel and the surrounding structural members. A vertical plastic band was formed between the two beam openings and developed the maximum plasticity at the wall-beam interface, see Fig. 6c. This behaviour is readily justified by the weakened beam at the mid-span, where it was welded to the panel. Next to this area, the fracture was initiated by the large inelastic deformations, which finally led to failure.

As indicated in Fig. 6d, a larger portion of the panel in SPSW-D has undergone plastic deformations. In type D, a higher maximum plastic strain was induced compared to the benchmark value, i.e., 12.87% versus 4.97%. Moreover, the boundary elements of the suggested case did not attract a great magnitude of plasticity. Namely, in the SPSW-D model, the wall acted as a well-tuned weak link that sacrificed itself to protect the columns and the frame connections. The von Mises plastic strain reached its ultimate value within the panel and away from the frame. In another word, most of the load carrying capacity of the panel has been utilised. The plastic hinge occurred in the roof beam, and thus enabled the panel to undergo large drifts before failure. The development of plasticity within the panel allowed a ductile behaviour that is confirmed by the final drift of 4.7 percent.

In terms of shear strength, the performance of all models were almost of the same order ranging from 2309 to 2525 kN. The SPSW-A slightly outperformed the other three models, see Fig. 5. The SPSW-D model ranked second with an only 6.22 percent decrease in the shear strength compared to SPSW-A. The shear strength of types C and B were 91.4 and 91 percent of that of the benchmark value, respectively.

Table 4 presents the total dissipated energy by each SPSW system under the applied load (inclusive of the elastic portion). The SPSW-B arrangement did not contribute significantly to the energy dissipation (only an increase by 3.9 percent). In contrast, the C and D types improved the dissipated energy by 146.5 and 303.3 percent, respectively. This result is in accordance with the large developed plastic regions in the D type and highlights the successful engagement of the panel in energy absorption.

Table 4: Dissipated energy in the models

Case	Dissipated energy (kJ)
SPSW-A	47.708
SPSW-B	49.636
SPSW-C	117.644
SPSW-D	192.394

According to the chosen criteria, it is observed that combining the suggested perforation patterns along with the WRBS approach has a synergistic effect. Namely in SPSW-D, the plastic field was developed into the infill panel instead of being retained at the structural boundary elements. Simultaneously, the WRBS modification allowed for larger drifts of the frame. Along with an excellent performance in shear, this model could reach the final drift of 4.7 percent, which is significantly greater than other models. Furthermore, the dissipated energy was about three times more than that of the benchmark value.

4 Conclusion

In the current study, by combining the WRBS concept and perforations in the infill panel, a new arrangement was proposed for the U-SPSWs. The hypothesis stated that this specific pattern (SPSW-D) performs better than the conventional SPSW (benchmark model) and two other arrangements, i.e., one beam with WRBS (SPSW-B) and two beams with WRBS (SPSW-C). The aim of the design suggestions was to transfer the plastic deformations from the column and beam-to-column connections into the panel. By conducting a series of pushover analyses using the FE method, it was found that the proposed model can successfully deflect the plastic deformations from the column and beam-column connection into the infill panel. Among the analysed cases, the SPSW-D takes advantage of the proposed pattern and outperforms other models in ductility and energy dissipation. While sacrificing the initial lateral stiffness by 28 percent, the dissipated energy is increased by 300 percent and the ultimate drift is increased around the same order.

The hypothesis of the study is approved and it was shown that the incorporating openings in both the infill panel and the beams could alter the stress path in favour of ductility and draw away the inelastic deformation from the critical structural elements. Namely, the infill plate could act as a fuse and dissipate the input energy under extreme conditions such seismic activities. In future studies, a parametric analysis of the proposed arrangement could reveal insightful information on its behaviour, e.g., the optimum location of the perforations.

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