Cyclic Behaviour of Steel Shear Walls with Beam-Only Connection Shear Panels

Hadi Monsef Ahmadi and Gianfranco De Matteis(✉)

Department of Architecture and Industrial Design, University of Campania "Luigi Vanvitelli", Aversa, CE, Italy
gianfranco.dematteis@unicampania.it

Abstract. Steel plate shear walls (SPSWs) with beam-only-connection provide a ductile behavior and reduce damage of columns when subjected to horizontal shear loads. In this study, the effectiveness SPSW with beam-only-connection on failure modes of three-story shear walls with two different types of beam-to-column connections is numerically investigated. The parametric studies are focused on the web plate slenderness and gap between columns. To achieve this aim, a finite element method for simulation and validation has been calibrated according to the recent experimental investigations. Therefore, two different gaps which were constant in the web plate length are selected. In addition, a solid shear panel is chosen as a reference study to compare the numerical results in terms of failure modes and structural behavior. The numerical results illustrated that the maximum shear strength, initial stiffness and energy-dissipating capacity of computational specimens are affected by changing the web plate slenderness and gap between the columns. Finally, the specimen which has improved the failure modes of shear walls is suggested.

Keywords: Steel shear wall · Numerical study · Beam-only-connection · Structural behavior · Failure mode

1 Introduction

Steel plate shear walls (SPSWs) have long been used as the principal lateral load-resisting solution of structures in high seismic zones [1]. SPSWs basically consist of shear panels, boundary elements, beam-to-column connections and plate-frame interactions [2]. As a basic seismic design of buildings, SPSWs can be classified in various categories according to the shear panel slenderness (as a function of the effective height of the infill panel divided by the web plate thickness) such as slender [3], semi-compact [4], and compact [5]. Normally, a shear panel characterized by a slenderness ratio more than 300 behaves as slender, meaning that it will undergo large buckling phenomena [6], which develops at a low ratio of drift under shear loading [7]. Therefore, a greater capacity of SPSWs is provided in the post-buckling behavior of the infill panel [8]. The direction of the main stress changes along the diagonal direction of the shear panel and the dissipative behavior derives from a tension field mechanism inside the infill panel after the elastic buckling of the shear panel. The boundary elements of slender SPSWs have to tolerate much more shear strength as well as bending moment through the

F. M. Mazzolani et al. (Eds.): STESSA 2022, LNCE 262, pp. 659–666, 2022.
https://doi.org/10.1007/978-3-031-03811-2_71

development of the tension field mechanism over the web plate [10]. The boundary elements must provide the required stiffness which causes the shear panel to be yielded before yielding the frame components [11]. Therefore, the envelope curves of SPSWs are affected by a pinching phenomenon due to an earlier buckling of the slender infill shear panels under shear load, particularly in the hinged structural frame [12]. Consequently, the energy-dissipation capacity of slender SPSWs is reduced, due to a significant influence of the pinching effect and the degradation stiffness on the hysteretic response [8].

Attempts to reduce column demands were recently solved by the employment of light-gauge, cold-formed steel plates, low-yield steel coupling beams to reduce overturning forces, perforations carried out in a defined regular pattern throughout the whole web plate, and web plates connected only to the beams. Limited experimental research on SPSW connected only to beam demonstrated that these systems have significant shear strength and may be utilized to isolate the shear walls from the main columns, therefore reducing column size. It was discovered that the frame has the potential to create a tension field in the wall plate, causing the wall plate to yield before the frame [10].

In this paper, the effectiveness of SPSWs beam-only-connection with various gaps and web plate slenderness on seismic behavior of 1:3 scale three-story one-bay slender shear walls with different beam-to-column joints (i.e., simple shear or resisting moment) has been numerically investigated. To this end, two experimental studies are primarily modelled and verified through Finite Element (FE) analysis. In order to evaluate the beam-to-column joints influence on the proposed, an experimental activity related to 1/3 scale three-story one-bay steel shear walls with either simple shear or moment-resisting beam-to-column connections is also verified by FE models. A comparison between the FE and experimental results showed a good agreement in terms of yielding shear strength capacity and failure modes. Accordingly, several 1:3 scale three-story shear walls, which are equipped SPSW beam-only-connection to reach the enabling objectives of this study, have been considered and analysed. Finally, the seismic behavior of the proposed arrangements of the SPSWs beam-only-connections are assessed in terms of maximum shear strength, ultimate shear strength, initial stiffness, energy-dissipating capacity.

2 Verification of the FE Model

Two 1:3 scale three-story one-bay unstiffened slender SPSWs with moment-resisting and simple shear beam-to-column connections were tested under cyclic shear loading. The welding connection types were used to introduce moment-resisting connections through welding of the beam's webs to the column's flanges directly. The simple beam-to-column connections had an angled cross-section and a 6 mm gap between the beam and column at the joints. The infill shear panel thickness was equal to 0.7 mm, with fishplates being utilized in the plate-frame interactions. The vertical load was not applied to the specimens, and, therefore, only a cyclic shear load was assigned to the shear walls according to the ATC-24 cyclic loading protocol. The material properties of the web plate and beams were steel types of st14 and st37, respectively. It is worth noting that the columns of the specimens were also composed of steel types of st52. More information about the experimental tests can be found in [10]. In order to verify the results of the tested specimens in terms of hysteretic performance and failure behavior in the global response of the SPSWs, two different numerical models according to the beam-to-column connections have been developed. S4R with reduced integration 4-point shell elements are implemented for all the components of the SPSWs. The elastic perfectly plastic material with steel properties with combined hardening is used in the FE models. A narrow mesh size approximately equal to 25 mm according to the mesh sensitivity analyses is adopted for both numerical specimens. The beam-to-column connections are introduced with TIE type connectors. The middle beams and top beam are constrained against out-of-plane deformations. All the degrees of freedom of the bottom beam are closed at the base of the FE models. The cyclic shear load, according to the reference experimental study condition, is applied to the top beam based on the displacement control method, by a reference point which was anchored to the loading beam with coupling type connectors. An imperfection is added to the shear panel according to the linear buckling analyses regarding the first, second and third buckling modes of the infill shear panel. The dynamic explicit solver is derived for both FE analyses. The results of the FE analyses in terms of hysteretic curves is verified by comparing the numerical results and experimental data as shown in Fig. 1.

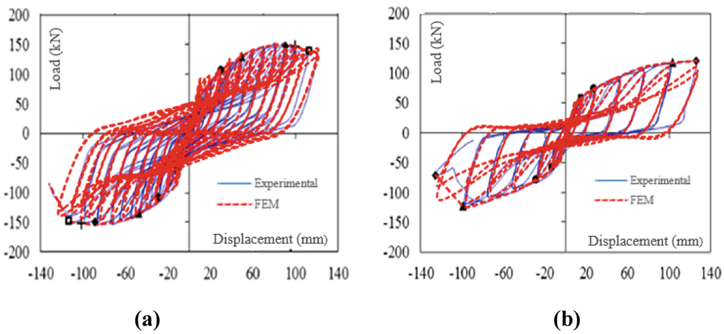

Fig. 1. Hysteretic curves of the specimen with a) simple shear and b) moment resisting connections.

3 Numerical Study on SPSWs with Beam-Only-Connection

To alter the numerical specimens, two categories have been chosen. The first category includes SPSWs with full connections. The second category is for specimens with just beam connections. The gaps between the column and the web plate in the second category are 10 mm and 30 mm, respectively. Because the web plate thickness is 0.7 mm, 1.4 mm, and 2.1 mm for both categories, the web plate slenderness varies in each specimen. It is worth noting that, as previously indicated, beam-to-column connections are different. As a result, the efficiency of the beam-to-column connections will be evaluated for each category. Furthermore, as stated in the verification model, material properties, mesh sizes, constraints, analytical method, and applying load conditions are similar in both categories of specimens as defined in the verification model. SPSW-A-B-C-D indicates numerical specimens. Where A, B, C, and D indicate the category type, the gap between the column and the web plate, the thickness of the web plate, and the kind of beam-to-column connections, where P represents the pin type and M represents the resisting moment. It should be noticed that the B in the first category is typed “0”. The hysteresis curves will achieve for all specimens and the results in terms of maximum shear strength, ultimate shear strength, initial stiffness and seismic energy dissipation are deeply discussed in the next section.

4 FE Results

4.1 General

To evaluate the hysteretic behaviour of the FE specimens, the FE models have been cyclically analysed up to 4% drift. The FE results of the considered specimens have been compared in terms of maximum shear strength, ultimate shear strength, initial stiffness and cumulative energy dissipation capacity. Along the same line, the maximum shear strength is reported according to the maximum experienced shear strength in the hysteretic curves. Furthermore, the ultimate shear strength is calculated by subtracting the last experienced shear strength from the hysteretic curves. Likewise, the bilinear curve such as ECCS method [9] is used to evaluate the initial stiffness. The cumulative seismic dissipated energy is calculated by sums of the areas of the cyclic loops [6]. Table 1 is a summary of all numerical results.

4.2 Structural Behaviour

As demonstrated in Table 1, the structural behavior of SPSWs with resistant moment beam-to-column connections has significantly higher values in terms of maximum shear strength, ultimate shear strength, initial stiffness, and cumulative seismic energy dissipation capacity in both categories. As predicted, decreasing the slenderness of the web plate leads the structural behavior of the numerical specimens to increase. However, the stiffness of the boundary components must satisfy the specifications of the web plate thickness. Otherwise, the boundary elements, particularly the columns, would be prone to the buckling phenomena. Therefore, the yielding phenomenon will occur in the boundary elements rather than the web plate, which is in violation of the AISC code requirements. For better comparison, the von Mises stress of the three specimens of the first category are compared in Fig. 2.

Table 1. Summarize FE results.

Specimens	Maximum shear strength (kN)	Ultimate shear strength (kN)	Initial stiffness (kN/mm)	Energy dissipation (kN.mm)
	First category			
SPSW-1-0-0.7-M	112	95	6.2	130312
SPSW-1-0-0.7-P	100	80	4.6	70650
SPSW-1-0-1.4-M	199	169	11.3	202434
SPSW-1-0-1.4-P	162	114	8.5	168291
SPSW-1-0-2.1-M	255	202	14.4	286921
SPSW-1-0-2.1-P	199	129	9.8	253004
	Second category			
SPSW-2-10-0.7-M	73	66	3.6	94165
SPSW-2-10-0.7-P	70	63	3.2	29814
SPSW-2-10-1.4-M	108	96	5.2	124017
SPSW-2-10-1.4-P	95	93	4.7	62995
SPSW-2-10-2.1-M	139	122	6.2	130975
SPSW-2-10-2.1-P	109	94	5.4	97384
SPSW-2-30-0.7-M	72	65	3.3	91861
SPSW-2-30-0.7-P	70	61	3	29814
SPSW-2-30-1.4-M	108	85	4.8	116340
SPSW-2-30-1.4-P	91	81	4.6	64242
SPSW-2-30-2.1-M	129	119	5.9	124317
SPSW-2-30-2.1-P	105	90	5.7	101254

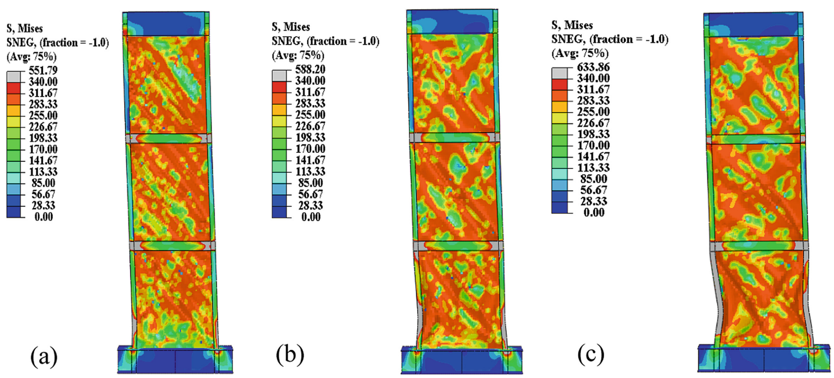

Fig. 2. Von Mises stress distribution of the a) SPSW-1-0-0.7-M, b) SPSW-1-0-1.4 and c) SPSW-1-0-2.1

Buckling can occur when the thickness of the web plate is increased, as seen in Fig. 2. This outcome may also be obtained by comparing the cumulative seismic energy dissipation value in Table 1.

In terms of maximum shear strength, the gap in the specimens of the second category is not efficient. However, changing the gap has a far greater impact on the cumulative seismic energy capacity. Additionally, buckling of the column in specimens with large gaps can be prevented. For better comparison, two specimens with gaps 10 mm and 30 mm are shown in Fig. 3.

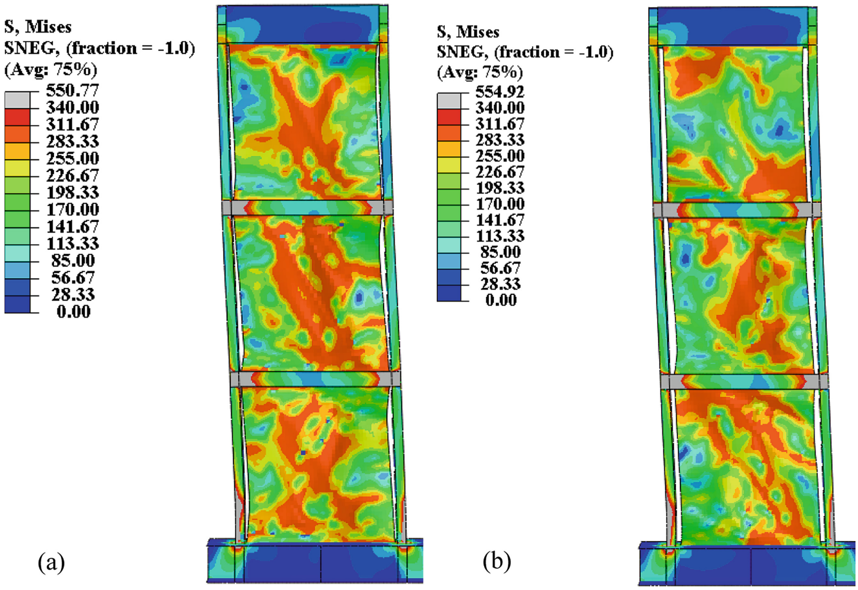

Fig. 3. Column buckling of a) SPSW-2-10-2.1-M and b) SPSW-2-30-2.1-M.

4.3 Fracture Tendency

The maximum equivalent strain value, PEEQ, can be used to predict the fracture tendency of steel plate shear walls. Table 2 summarizes these results.

As demonstrated in Table 2, when the web plate slenderness was reduced, the PEEQ values increased. The greatest PEEQ value is associated with SPSWs with pin beam-to-column connections. Because, as mentioned in the reference study, rotation of the pin connections based on the angles might occur. As a result, the connections are more prone to failure. However, when only a beam connection is utilized with the web plate, the PEEQ values are considerably reduced. As a result, employing just beam connections can benefit in reducing the fracture tendency in the SPSW with moment resistant beam-to-column connections.

Table 2. Summarize PEEQ results.

Specimens	PEEQ
SPSW-1-0-0.7-M	1.45
SPSW-1-0-0.7-P	2.89
SPSW-1-0-1.4-M	2.48
SPSW-1-0-1.4-P	3.35
SPSW-1-0-2.1-M	3.37
SPSW-1-0-2.1-P	3.57
SPSW-2-10-0.7-M	1.53
SPSW-2-10-0.7-P	2.41
SPSW-2-10-1.4-M	1.69
SPSW-2-10-1.4-P	5.06
SPSW-2-10-2.1-M	1.93
SPSW-2-10-2.1-P	10.3
SPSW-2-30-0.7-M	1.14
SPSW-2-30-0.7-P	2.41
SPSW-2-30-1.4-M	1.17
SPSW-2-30-1.4-P	4.61
SPSW-2-30-2.1-M	1.31
SPSW-2-30-2.1-P	9.89

5 Conclusion

The cyclic behavior of perforated steel shear walls 1:3 scale three-level one-bay with beam only connection was examined in this paper. The numerical specimens differed in terms of web plate slenderness, gap between column and web plate and beam-to-column connections. The obtained numerical results indicate that the specimens with moment resisting beam-to-column connections have a higher shear capacity, initial stiffness, and energy dissipation than the specimens with simple shear beam-to-column connections. Likewise, the finite element results illustrate that the structural behaviour of the considered specimens increases when the web plate slenderness decreases. On the other hand, the column buckling of the SPSWs with beam only connection can be prevented in comparison with specimens with full connections. Finally, it can be proposed that column buckling, and fracture tendency reduced with high gap and thick web plate in the steel plate shear walls.

References

1. Takahashi Y, Takemoto Y, Takeda T, Takagi M (1973) Experimental study on thin steel shear walls and particular bracings under alternative horizontal load. In: Preliminary report, IABSE, symposium on resistance and ultimate deformability of Tsructures acted on by well-defined repeated loads, Lisbon, Portugal
2. De Matteis G, Mazzolani FM, Panico S (2008) Experimental tests on pure aluminium shear panels with welded stiffeners. Eng Struct 30:1734–1744
3. De Matteis G, Formisano A, Mazzolani FM, Panico S (2005) Design of low-yield metal shear panels for energy dissipation. In: Improvement of buildings' structural quality by new technologies. CRC Press, pp 677–688 (2005)
4. ANSI/AISC 341-16 (2016) Seismic provisions for structural steel buildings. American Institute of Steel Construction (AISC), Chicago
5. De Matteis G, Sarracco G, Brando G (2016) Experimental tests and optimization rules for steel perforated shear panels. J Constr Steel Res 123:41–52
6. De Matteis G, Ahmadi HM (2020) Hysteretic behavior of steel shear panels with internal rectangular-shaped links. J Constr Steel Res 177:106451
7. Monsef Ahmad H, Sheidaii MR, Tariverdilo S, Formisano A, De Matteis G (2020) Experimental and numerical study of perforated steel plate shear panels. Int J Eng 33:520–529
8. Formisano A, Lombardi L, Mazzolani FM (2016) Perforated metal shear panels as bracing devices of seismic-resistant structures. J Constr Steel Res 126:37–49
9. Ahmadi HM, De Matteis G (2020) Seismic performance of steel shear panels with butterfly-shaped links. Ingegneria Sismica 37:85–101
10. Azandariani MG, Gholhaki M, Kafi MA (2020) Experimental and numerical investigation of low-yield-strength (LYS) steel plate shear walls under cyclic loading. Eng Struct 203:109866
11. Wang M, Shi Y, Xu J, Yang W, Li Y (2015) Experimental and numerical study of unstiffened steel plate shear wall structures. J Constr Steel Res 112:373–386
12. Ahmadi HM, Sheidaii MR, Boudaghi H, De Matteis G (2020) Experimental and numerical study on largely perforated steel shear plates with rectangular tube–shaped links. Adv Struct Eng 23(15):3307–3322. https://doi.org/10.1177/1369433220937147