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DESIGN OF STEEL FRAMES OF DISSIPATIVE SHEAR WALLS

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Abstract*. This paper presents the results of an experimental program carried out on two story frame models of dissipative shear walls. Rigid and semi-rigid beam-to column joint are considered. The experimental investigation is accompanied by a numerical simulation program in order to validate and extend the conclusions of the experimental study to real multi-story frames. Values of behavior factor q obtained from experimental tests are proposed.*

1 INTRODUCTION

Steel plate shear walls SPSW have been used as lateral force resisting systems since 80's, but the design specifications were rather incomplete or absent. Numerous research programs and also seismic experiences have confirmed their effectiveness. A major role on their development can be attributed to the introduction of design rules in the code provisions, e.g. AISC 2005 [1]. In Europe, the application of SPSW system is limited, partly due to the lack of design provisions in seismic code EN 1998-1 [2], particularly there are no recommendations for behavior and overstrength factors q and Ω, respectively. An additional problem refers to the prediction of the strength and stiffness capacity of the SPSW structures. Design practice requires simple models and conventional analysis software that are available and relatively simple to use. One of the models used to represent the behavior of SPSW is the strip model, developed by Thorburn et al. [3]. Thus, in order to model the steel shear walls, the steel plates are replaced by a series of truss members - strips, parallel to tension fields. A minimum of ten strips per panel are required to adequately represent the tension field action developed in the plate.

In the recent years, many researchers studied the behavior of SPSW and their viability as seismic force resisting systems. Choi & Park [4] and Habashi & Alinia [5] studied experimentally the interaction between the infill panels and the boundary elements. In another experimental program, Choi & Park [6] studied the ductility and energy dissipation capacity of various SPSW typologies, and compared them with other types of bracing. In their study, Ghomi & Gholhaki [7] investigated the influence of beam-to-column connections on the ductility of SPSW. Based on the results of the cyclic test, Li et al. [8] proposed a capacity design principle for boundary elements.

In order to address the issues presented above with regards to the performances of SPSW systems, a parametric study on different structural configurations was performed, on the aim to evaluate the demands in terms of strength and ductility [9]. Acceptance criteria and performance parameters (eg. behavior factor q) were taken according to [1]. Based on these results, an experimental program on steel plate shear wall systems has been developed at the Steel Structures Laboratory from the Politehnica University of Timisoara. The test results shown very good agreement with the parameters and acceptance criteria used in design. The paper summarizes the experimental results and gives some preliminary indications regarding the selection of behavior factor q.

2 EXPERIMENTAL PROGRAM

2.1 Analysis and design of steel plate shear wall structure

Previous studies of the authors [9] focused on the comparative analysis of the seismic performances of conventional centric and eccentric braced systems and new systems using buckling restrained braces and dissipative Steel Plate Shear Walls. Structures were investigated using pushover analysis. The parameters that were assessed in order to evaluate the seismic performance of the structural systems were story drift and plastic deformations in dissipative members. Particular attention was paid to the potential values to be used in design of overstrength factor Ω and to the global plastic mechanism at failure. Dual systems of SPSW have shown they are very effective both in terms of stiffness and ductility. Therefore, post-elastic behaviour of SPSW systems confirms the behavior factor q proposed in AISC 2005, which is similar to that used for high dissipative moment resisting frames or eccentrically braced frames.

Encouraged by the results of the numerical investigations, a test program was developed. In order to test realistic structural configurations, test specimens were extracted from a 6 story steel frame building (Figure 1.a). The 6 story structure was designed according to EN1993-1 [10], EN1998-1 and P100-1/2006 [11]. The steel materials were S235 grade for infill panels and S355 grade for frame members. A 4 kN/m^2 dead load on the typical floor and 3.5kN/m^2 for the roof were considered, while the live load amounts 2.0kN/m^2. Structure is located in a high seismic area, which is characterized by a design peak ground acceleration of 0.24g at a returning period of 100 years and soft soil conditions, with T_c=1.6sec. For serviceability check, acceleration amounts 0.12g and allowable inter-story drift is 0.008 of the storey height. For collapse prevention, acceleration amounts 0.36g. EN1998-1 does not provide any recommendations regarding the q factor for SPSW systems. For these structural systems, AISC 2005 provisions were taken as guidance. According to the later code, the behavior factor q for SPSW is similar to that of high dissipative moment resisting frames. Concluding, the design was based on a q factor equal to 6. Two-story specimens were extracted from the structure. Due to the limitations in testing capacity, the specimens were half-scale representative of the actual building (Figure 1.b).

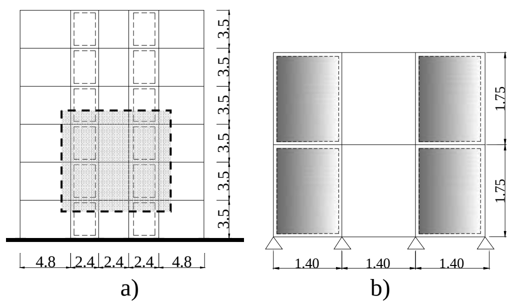

Figure 1. Six story model used for extracting the test specimens a) and two story half-scale specimen b).

2.2 Analysis and design of half-scale models

The two story specimens, extracted from the structure were half-scaled. In order to evaluate to contribution of the frame to the strength and stiffness of the structure, two types of beam-to-column connection were used. The first one is a flush end plate bolted connection and the second one is an extended end plate bolted connection. According to EC3 classification [12], flush end plate connection is semi-rigid and *weak* partial strength ($M_{j,\ Rd}$=0.4$M_{b,Rd}$) (further refereed as *semi-rigid*) and extended end plate connection is rigid and *strong* partial strength, with a capacity almost equal to that of the connected beam ($M_{j,Rd}$ = 0.9$M_{b,Rd}$) (further refereed as *rigid*). The infill panels were bolt connected to the boundary members using fish plates. To reduce the number of bolts, an edge plate was welded to the infill panels to increase the bearing resistance. Nonlinear pushover analyses were conducted on each model in order to evaluate the behavior and the characteristic parameters, like yield strength and yield displacements, ultimate capacity and dissipation capacity. The shear walls were modeled by 10 inclined pin-ended strips, oriented at angle α [13]. Beams and columns were modelled with plastic hinges located at both ends. Pushover analyses have shown the ultimate capacity of structure with rigid beam-to-column connections is 10% above that of the structure with semi-rigid connections, while the stiffness is not much improved. A total of 5 specimens were designed and fabricated (see Table 1). The frames measured 3500 mm high and 4200 mm wide between member centerlines (Figure 2). The slenderness ratio h/t_w of shear walls amounted 595 for 2 mm panels and 397 for 3 mm panels. The two actuators used for the tests have 360mm stroke and 1000 kN capacity and 360mm stroke and 500 kN capacity, respectively.

Table 1. Design of specimens.

Specimen	Infill panel	Column	Beam	Connection	Load type
SR-M-T2	2mm	HEB240 (HEB180)	HEA180	Semirigid	Monotonic
SR-C-T2				Semirigid	Cyclic
R-M-T2				Rigid	Monotonic
R-C-T2				Rigid	Cyclic
SR-C-T3	3mm			Semirigid	Cyclic

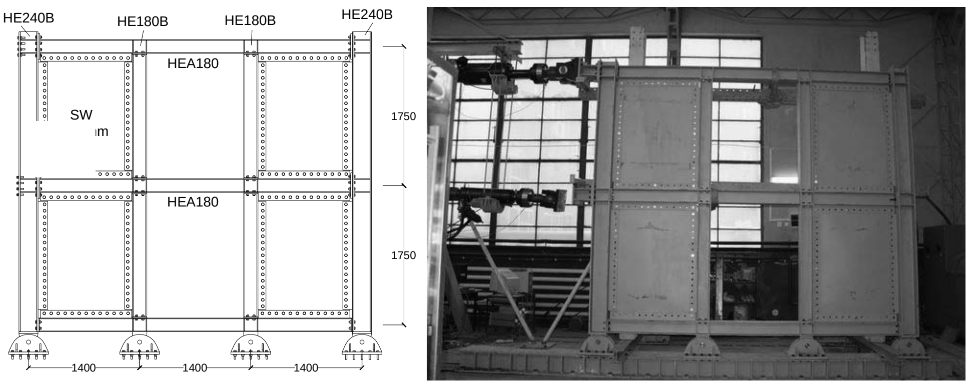

Figure 2. Typical half-scaled test specimen: designed (left) and testing set-up (right).

Table 2 and Table 3 show the measured average values of yield stress f_y, tensile strength f_u and elongation at rupture A_u. It can be observed that there are important differences between nominal and actual material characteristics. The material overstrength is larger for hot rolled sections than for plates. Thus, for hot rolled sections the maximum yield strength scattering amounts 105 N/mm^2, while for plates amounts 80 N/mm^2. The connection of the steel panels to the beams and columns used bolts. The connection was designed to develop expected shear strength of the steel panels. Extended end plate bolted connections were used for rigid beam-to-column connections while for semi-rigid ones flush end plate bolted connections.

Table 2. Mechanical properties of rolled profiles.

Profiles	Nominal steel, ordered (EN 10025-2/2004)	Element	Tests			Actual steel grade (supplied)
			f_y, N/mm^2	f_u, N/mm^2	A_u, %	
HEB240	S335	Flange	457	609	40	S460
		Web	458	609	31	
HEB180	S355	Flange	360	515	44	S355
		Web	408	540	40	
HEA180	S355	Flange	419	558	32	S420

Table 3. Mechanical properties of flat steel (panels).

Element	Thickness, mm	Nominal steel, ordered	f_y, N/mm^2	f_u, N/mm^2	A_u, %	Actual steel grade
Plate	2	S235	305	429	24	S275
Plate	3	S235	313	413	25	S275

All specimens were tested monotonically and cyclically using ECCS procedure [14]. According to this procedure, first a monotonic test is done in order to evaluate the force-displacement curve that is used to evaluate the yield displacement. Yield displacement D_y and yield force F_y are obtained by intersecting the initial stiffness α_y and a tangent at the curve *F - D* with a slope of 10% of the initial stiffness. Yield displacement D_y is then used to calibrate the cyclic load history. This contains four elastic cycles ($\pm0.25D_y$, $\pm0.5D_y$, $\pm0.75D_y$ and $\pm1.0Dy$), followed by groups of 3 cycles of amplitudes multiple of $2D_y$ ($3\times2D_y$, $3\times4Dd_y$, $3\times6D_y$, …). Based on experimental results, the evaluation of yield displacement was adjusted to take into account the specific behavior of SPSW. Thus, the slope of initial stiffness was corrected and amounted 20% of the initial stiffness.

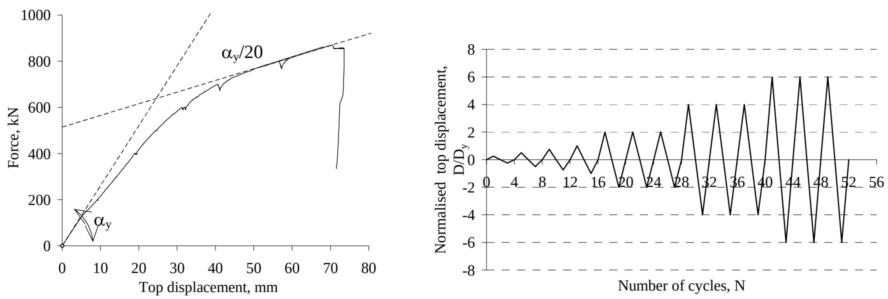

Figure 3. ECCS cyclic loading protocol.

2.3 Experimental results

2.3.1 Monotonic tests

The first test was done on the semi-rigid structure with 2mm panels SR-M-T2. The loading scheme twisted the top beam and also the left hand side column. Due to safety reasons, the test was stopped before reaching the failure of the specimen. An attempt was made to correct the problem but the frame structure was affected and therefore the test was ceased. For the R-M-T2, the out-of-plane restraint was improved and there were no impediments to conclude the test. The specimen exhibited desirable and stable behavior, as shown in Figure 4.

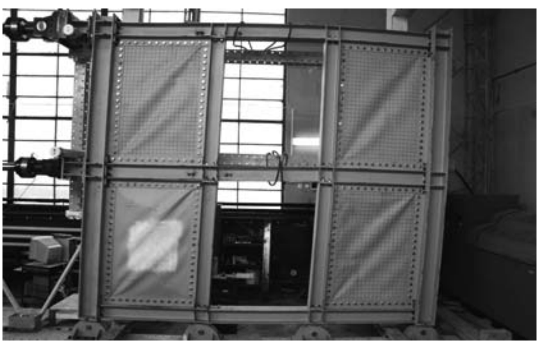
Figure 4. Damage to the specimen R-M-T2.

Some local fractures were initiated at the corners of the panels at interstory drifts of approximately 2%. There were no damages of the beam-to-column connections before plastic deformations took place in panels. Local plastic deformations started to initiate at the beam flange in compression for drifts larger than 2%. The test was stopped at a top displacement of 240mm as force started to drop. The plastic rotation for the maximum capacity amounted 0.03rad. Results of the monotonic test were compared with the results of the pushover test. The strip model used in the pushover test did not describe accurately the behavior of the specimens. When the strip area was reduced by 10%, the accuracy of the model was very much improved (Figure 5). This reduction of the panel area conservatively reduces the strength and stiffness of the structure. The model permits the evaluation of the initial stiffness and ultimate capacity of the SPSW structure.

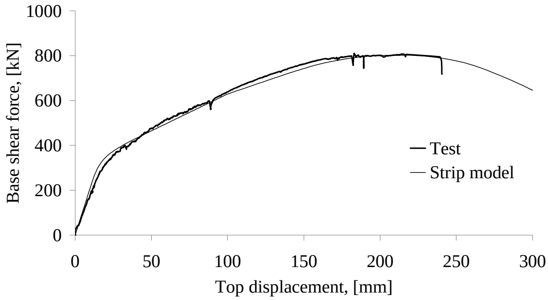

Figure 5. Comparison of experimental results for rigid specimen with 2mm thick panel and numerical results using the strip model.

2.3.2 Cyclic tests

All specimens exhibited stable behavior up to cycles of 4% interstory drift. Very little pinching were recorded in the hysteresis loops for large drifts and semi-rigid specimens, only. Panels yield at approximately 0.006 of the story height for semi-rigid specimens and 0.008 of the story height for rigid specimen. Some local fractures were initiated at the corners of the panels at drifts of approximately 2%. Local plastic deformations were observed at the beam flange in compression for rigid connections. Bolted connections between infill panels and the fish plates shown small slippages but no plastic deformations either in plates or bolts. This enabled a very easy dismantling of the steel panels after the test. Associated with a small residual drift (for rigid structure), this can assure an easy intervention after a moderate earthquake to replace the damaged panels.

Figure 6 plots the hysteresis of rigid and semi-rigid specimens and Figure 7 plots the envelopes of the hysteresis (results from each third cycle were considered). Contribution of rigid connections on the ultimate capacity of the specimens is larger than in the monotonic tests. As the initial stiffness is mainly attributed to the panels, differences between rigid and semi-rigid specimens in terms of stiffness are not as important as differences in terms of strength.

An important objective of the experimental program was the evaluation of the behavior factor q. The q factor can be expressed as a product of the ductility factor q_μ, that accounts for the ductility of the structure and the overstrength factor q_s, that accounts for the reserve in strength of the structure (due to structural redundancy, material overstrength, member oversize due to design, other non seismic load combinations and serviceability requirements). The overstrength may vary significantly and is affected by the contribution of gravity loads, material overstrength, etc. Therefore, in order to calibrate the behavior factor q, it is more important to focus on the ductility component, which can be taken equal to the displacement ductility factor μ [15]. The displacement ductility factor q_μ is therefore defined as the ratio of the ultimate displacement D_u and the yield displacement D_y. The parameter D_u corresponds to a reduction of the load carrying capacity of 10% compared to the maximum one. The yield displacement D_y can be considered the one corresponding to the modification of the elastic stiffness. Based on the observation of the hysteresis curves, the ultimate displacements D_u for SR-C-T2 and SR-C-T3 were corrected to take into account the slippage during the load reversal. Table 4 presents the q_μ factor values for the specimens. Comparing these values, it may be seen the specimens have similar ductility factors q_μ, with values ranging between 4.9 and 5.2. These values show SPSW structures have a good ductility and can provide q factors similar to those corresponding to other dissipative structure, like for example high dissipative moment resisting frames, which have ductility factors q_μ equals to 5 (see [2]).

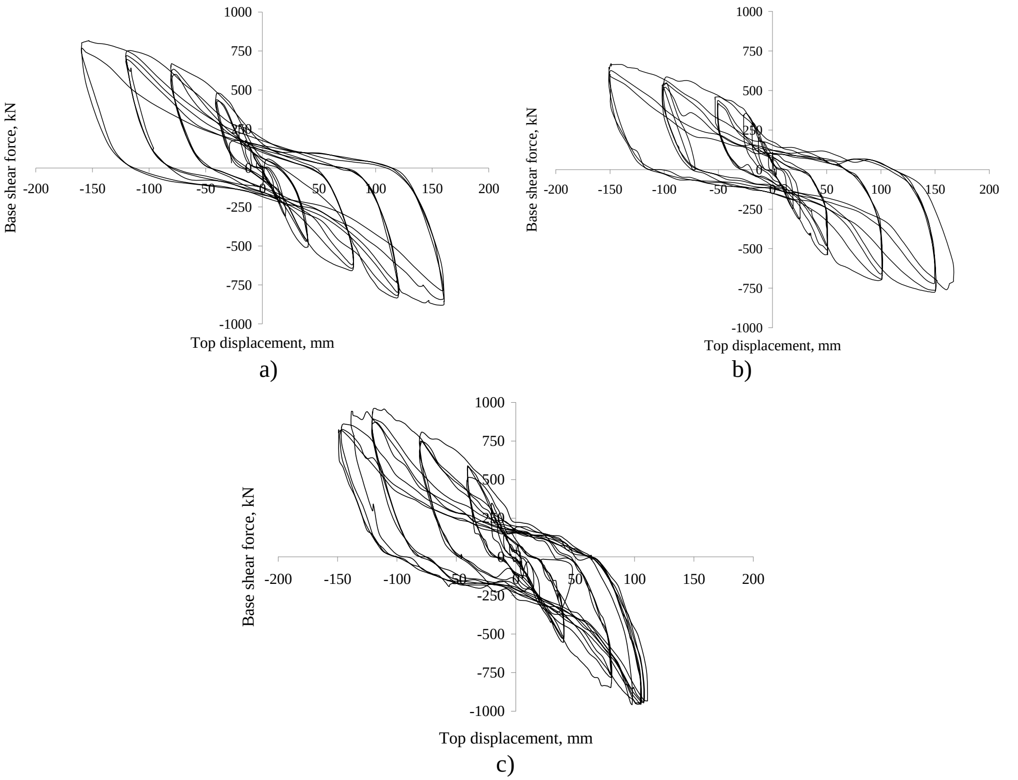

Figure 6. Hysteresis curves: a) R-C-T2 specimen; b) SR-C-T2 specimen; c) SR-C-T3 specimen

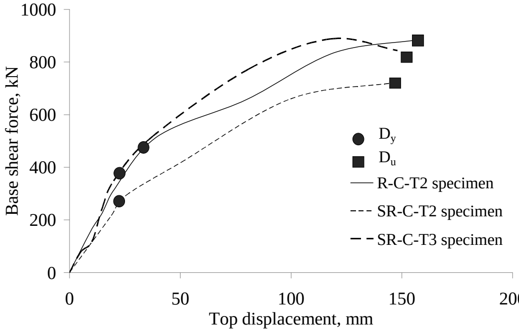

Figure 7. Envelope curves of the hysteresis.

Table 4. q factor values for rigid and semi-rigid specimens.

Structure	D_y [mm]	D_u [mm]	q_μ	q_μ^*
R-C-T2	32	157	4.9	4.9
SR-C-T2	23	148 (114)	6.4	4.9
SR-C-T3	23	150 (120)	6.5	5.2
Average values			5.9	5.0
* *corrections due to slippage at the load reversal*				

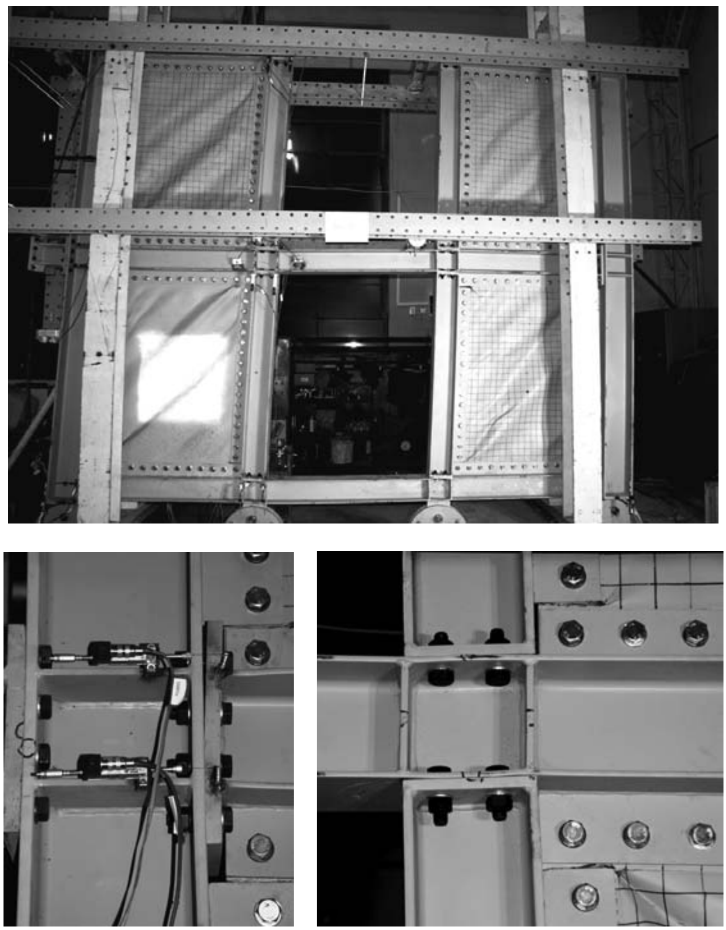

Figure 8. General view and details of the R-C-T2 specimen

3 CONCLUSIONS

The paper investigated the performances of the dual SPSW structures. Due to the limitations of the testing capacity, the models were half-scaled. A total of 5 specimens were designed and fabricated, which included specimens with semi-rigid and rigid connections. Specimens were tested monotonically and cyclically. The results of the tests have confirmed the conclusions of the previous numerical studies developed by the authors, on the performances of dual SPSW structures. The strip model developed by Thorburn et al. [3] was used for the preliminary analysis of the models. Based on the experimental results, this model was modified by correcting the area of the strips by a factor of 0.9. Rigid connections increased the yield resistance and the ultimate capacity of the structures. The initial stiffness is also improved when rigid beam-to-column connections are adopted. Behavior factor q amounts in average 5, considering the contribution of the ductility, only. These values indicate SPSW structures exhibit a dissipative behavior, similar to other dissipative structures, like for example moment resisting frames. However, further investigations are needed to calibrate the behavior factors q for SPSW structures.

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