RE-CENTRING CAPACITY OF DUAL STEEL BUILDING FRAMES WITH REPLACEABLE THIN-WALLED SHEAR PANELS

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Abstract: Frames with replaceable thin-walled steel shear panels are efficient structural systems for resisting seismic loads owing to their high initial stiffness and reliable energy dissipation capacity. In order to reduce the economic losses from loss of building operation and cost of repairing, the shear panels may be designed to be easily removed and replaced if damaged by an earthquake. When replaceable steel shear panels and moment resisting frames are combined (dual system), allowing the MRF to remain in elastic reduces the residual drift after an earthquake and provides the re-centring capacity that is required for removing and replacing the damaged shear panels.
The paper presents a case study with application examples to buildings that have different configurations. Two different design cases are taken into account: moderate and high seismicity cases. The seismic performance is investigated using nonlinear static analyses.

Keywords: Re-centring capacity, dual steel frames, replaceable thin-walled shear panels.

1. Introduction

Conventional seismic design philosophy is based on dissipative response, which implicitly accepts damage of the structure under the design earthquake and leads to significant economic losses. Repair of the structure is often delayed by the residual displacement of the structure. In order to reduce the economic losses, the dissipative system may be designed to be easily removed and replaced if damaged by an earthquake. When the dissipative system and moment resisting frames are used together (dual system), allowing the moment resisting frame to remain in elastic reduces the residual displacement after an earthquake and provides re-centering capacity that is required for removing and replacing the damaged dissipative elements (Dubina D et al. 2011).

The paper presents application of this concept to a dual structure, obtained by combining moment resisting frames with replaceable thin-walled steel shear panels (SPSW).

Steel frames with steel shear panels have been previously studied and developed by Politehnica University Timisoara (UPT) (Neagu C 2011; Dubina D et al. 2014) in the frame of several research projects.

2. System description

Most of the structures designed to modern codes would experience inelastic deformations even under moderate seismic action, with residual displacements after an earthquake. Repair is difficult in such cases. One solution is the system that provides re-centering capability, through dual structural configuration (rigid-flexible) and removable dissipative members. One of this systems uses replaceable thin-walled steel shear panels. The proposed dual system is a specific system, whereby a link beam connects two shear panel bays between two parallel moment frames (Fig. 1). The panels are bordered by additional vertical boundary elements (stanchions, denoted as VBE) having pinned connections at their ends to the beams (denoted as HBE). The beam between the panels acts as a short, intermediate or long link, depending on the relative length of the shear panels and bay width.

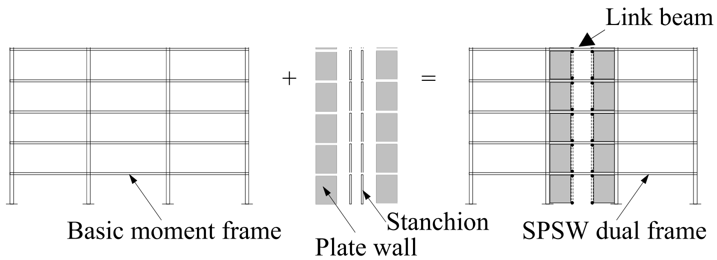


Fig. 1 - SPSW dual frame systems.

This systems has good seismic response, increased strength and stiffness, high dissipation capacity, and small residual drifts. The innovative application of such systems may be used for new constructions and also for upgrading the lateral resistance of existing ones.

A challenge for the SPSW system is also the construction efficiency and economic cost. Connection between shear panels and bordering elements can be done using fillet welds but when bolted connections are used, the construction time may be reduced. Recent developments in the field showed that steel panels, like other dissipative systems, may be designed and detailed to be replaced after an earthquake if the damage is limited in the panels but this requires special design and detailing conditions. If the more flexible MRFs are designed to remain elastic during earthquake, they provide the necessary restoring force to re-center the structure and to allow the replacement of the damaged shear panels (Dubina D et al. 2011).

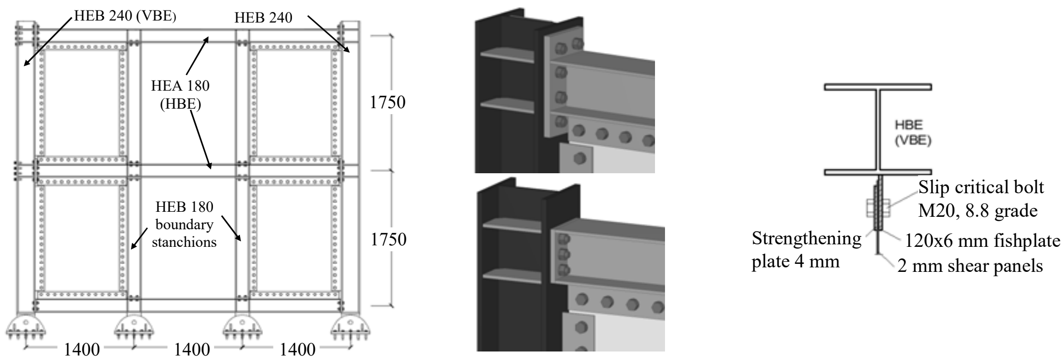
3. Experimental study

A large experimental program has been carried out at Politehnica University in Timisoara, Romania (Neagu C 2011; Dubina D et al. 2014), on steel frames with thin-walled steel shear panels, in order to determine the monotonic and cyclic performance.

3.1. Experimental setup

The specimens have been isolated from a six story frame structure (see Fig. 1). Two actuators were used for the experimental tests having 500 kN and 1000 kN capacity with 360mm stroke. Due to the stroke limitation, the specimens were half-scaled. The system had 2 shear panels per story of 2 mm thickness. The frames measured 4200 mm wide and 3500 mm high between member centrelines (Fig. 2a). The aspect ratio L/h of shear panels amounted 0.8 while the slenderness ratio L/t_w was 595.

In order to evaluate the contribution of the boundary frames to the strength and stiffness of the structure, two types of beam-to-column connections were used. According to EN 1993-1-8 (2005) classification, flush end plate connection was semi-rigid and weak partial strength ($M_{j,Rd}$=0.4$M_{b,Rd}$) (further refereed as semi-rigid) and extended end plate connection was rigid and strong partial strength ($M_{j,Rd}$ = 0.9$M_{b,Rd}$) (further refereed as rigid) see Fig. 2b. The shear panels were bolt connected to the boundary members using fish plates and M20 8.8 class slip-resistant bolts. Additional plates were used in order to strengthen the bolted areas to avoid failure by bearing.



(a) (b) (c)

Fig. 2 - SPSW: a) Half-scale tested frame; b) Semi-rigid and rigid beam-to-column connection; c) Connection between shear panels and boundary elements.

3.2. Loading protocol

Two hydraulic actuators were used, one at each story. Quasi-static cyclic testing was performed in accordance with ECCS (1985) Recommendations. The cyclic loading, involves generating four successive cycles of $\pm0.25D_y$, $\pm0.5D_y$, $\pm0.75D_y$, and $\pm1.0D_y$ amplitude ranges, followed up to failure by series of three cycles as presented in Fig. 3.

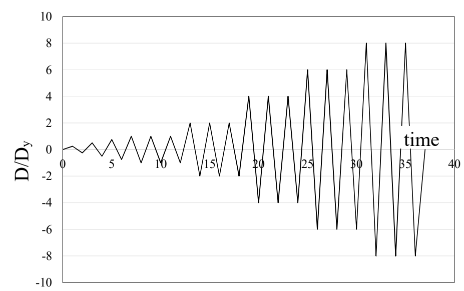
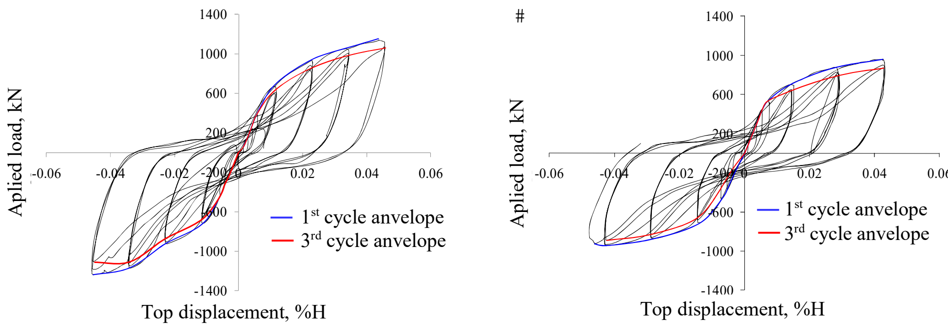


Fig. 3 - Cyclic loading protocol (ECCS 1985).

3.3. Cyclic test results

Plots of lateral load against top displacement of the three specimens tested under cyclic loading are shown in Fig. 4. All specimens exhibited stable force-displacement behaviour, with some pinching of hysteresis loops that are in line with the characteristics commonly observed in other tests on SPSW. The specimens, yielded at 0.65% and 0.7% top drift, respectively. This indicates that until the yielding, the stiffness of the beam-to-column joint has little effect on the behavior. Some local cracks were initiated at the panel corners at approximately 2% top drift. At the same drift level, local plastic deformations were observed at the beam flange under compression for rigid connections. For semi-rigid specimens the plastic deformations were initiated in the connections because of the beam end plate in bending, at approximately 2.5% top drift (Fig. 5). The ultimate displacement of the specimens was approximately 4.5% top drift, not owing to the specimen collapse but owing to the limitation of the actuator stroke. The contribution of the frame to overall response increased with lateral displacement. Thus, the difference between specimens with semi-rigid and rigid connections in terms of yield resistance and displacement was small, as mentioned before, but ultimate capacity decreased by 20%.



(a) (b)

Fig. 4 - Experimental frames hysteresis: a) specimen with rigid joints; b) specimen with semi-rigid joints.

Fig. 5 - Specimen after testing and connection details.

4. Case study

4.1. Design of case study building frames

4.1.1. Description of examined building frames

The case study presented hereafter was based on the isolation of an exterior plane frame from four and eight story buildings,

Fig. 6. The frames consist of a rigid moment resisting frames with three 8 m bays and two 3 m wide shear panels as lateral system located in the interior bay (see chapter 2). The height of all buildings was considered 4 m (see also Neagu C et al 2017).

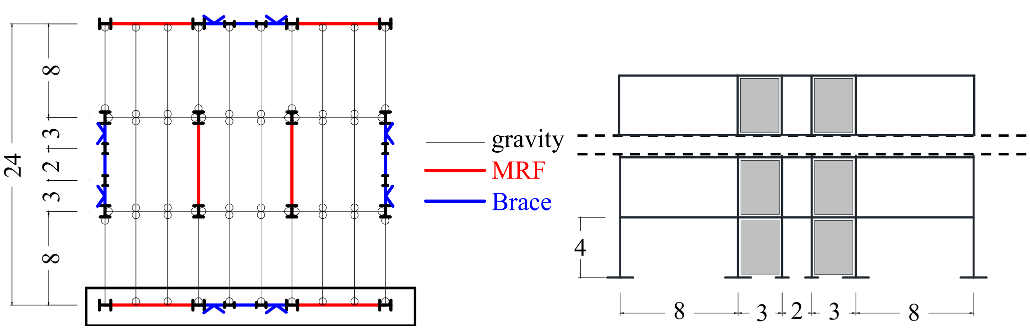


Fig. 6 - Building frames configuration, 3D upper view (left) and isolated exterior plane frame (right).

4.1.2. Preliminary assumptions

A number of analytical approaches are possible to achieve capacity design and determine the size of HBE, VBE and shear panel. One of these methods is the approximation of SPSW by a vertical truss with tension diagonals only (further denoted as equivalent frame) (Fig. 7), in line with AISC 341 (2010). The area of the equivalent braces is estimated in order to meet the structure's drift requirements.

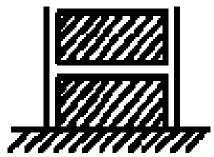


Fig. 7 – Approximation of SPSW with equivalent frame.

4.1.3. Code-based design

The equivalent building frames were then designed according to EN 1993-1 (2005), EN 1998 (2004) and AISC 341 (2010). Two different design cases were considered: moderate seismicity case with medium class ductile building frames (DCM) and high seismicity case with high class ductile building frames (DCH). Type 1-C spectrum was selected for design (EN 1998 2004) considering two peak ground accelerations 0.3 for high seismicity case and 0.15 for moderate seismicity case, respectively (Fig. 8). Based on previews research (Neagu C 2011; Dubina D et al. 2014) the reduction factor, *q*, was considered 5 for DCH and 3 for DCM.

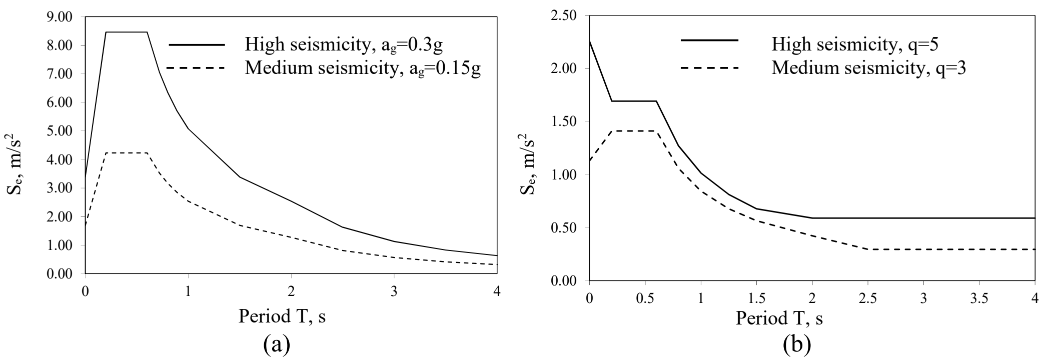


Fig. 8 – Ground type 1-C response spectra EN 1998 (2004): a) elastic; b) design.

Columns, beams and stanchions were designed as steel members according to EN 1998-1 (2005) and AISC 341(2010), with section varying depending on the floor and building. The shear panels had lower steel grade (S235) than the rest of the structural members (S355). The beams production was not considered to be fully controlled, so that the properties of the beam material had to comply with EN 1993-1-1 (2004) recommendations with γ_{ov} =1.25.

4.1. Re-centring capacity

In order to verify the plastic mechanism and re-centering capacity at ULS of the building frames, static non-linear analysis (push-over) was employed using SAP2000 (2000) software. The analysis has taken into account a "modal" shape lateral load. As the re-centering capacity is of interest, the target displacement corresponding to ULS was calculated. In order to model the shear panels, the so called "strip model" developed by Driver et al. (1998) was used. The shear panel is replaced by 10 inclined strips at angle α with respect to vertical, capable of transmitting tension forces only, and oriented in the same direction as the principal tensile stresses in the panel. Fig. 9a shows the strip model representation of a typical shear panel. The strips were modelled as double pinned beam elements having a trilinear plastic axial P type hinge at the middle (Fig. 9b). The presented behaviour was calibrated according experimental results (Neagu C 2011; Dubina D et al. 2014). Values for serviceability, ultimate and collapse prevention limit state acceptance criteria are proposed (Fig. 9b) in line with ASCE 41(2013).

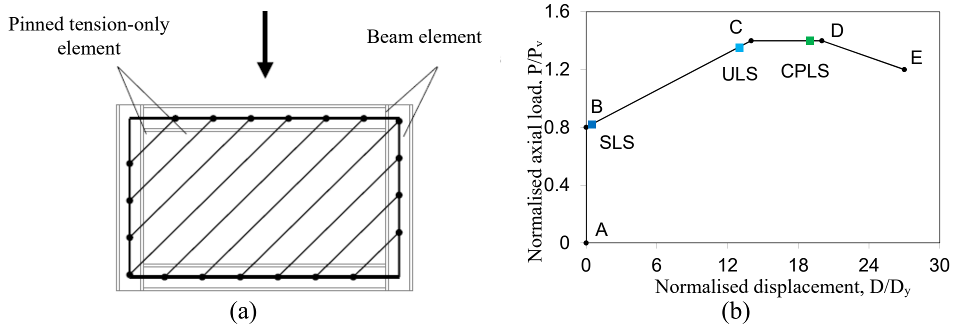


Fig. 9 - Modelling of shear panels: a) Strip model; b) Tension only plastic hinge.

After running the push-over analyses, yielding was observed in MRF elements before the attainment of ultimate limit state, for all 4 building frames. Therefor some of the sections were replaced in order to keep the MRF elements in elastic up to ultimate limit state. The objective of having no yielding in the MRFs before the attainment of ULS, represents the basic design requirement for dual frames with removable dissipative members. Fig. 10 and Fig. 11 present the final sections for all 4 buildings.

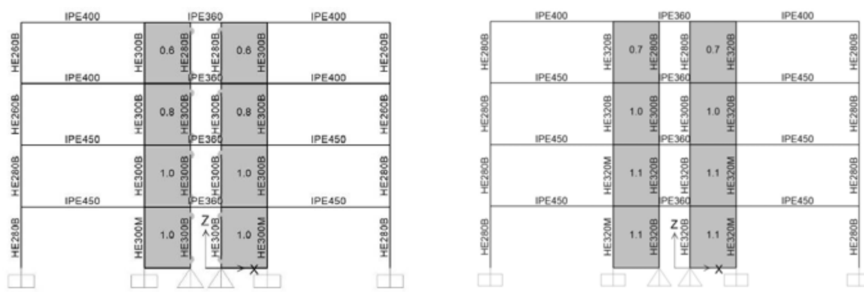
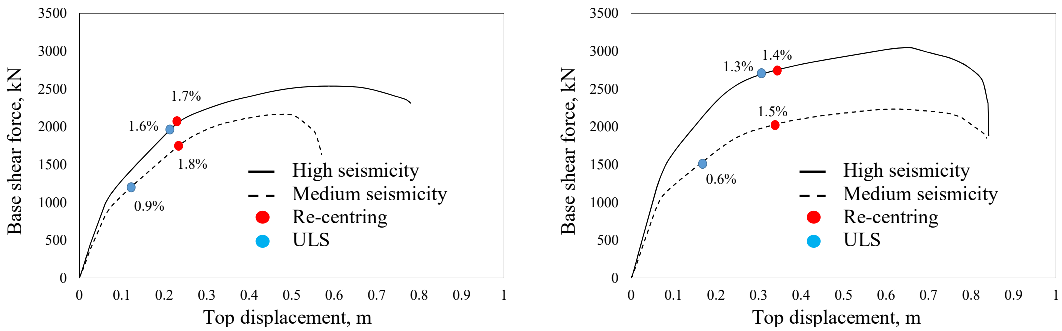


Fig. 10 - 4 story building frames for moderate seismicity case (left) and high seismicity case (right).

Fig. 11 - 8 story building frames for moderate seismicity case (left) and high seismicity case (right).

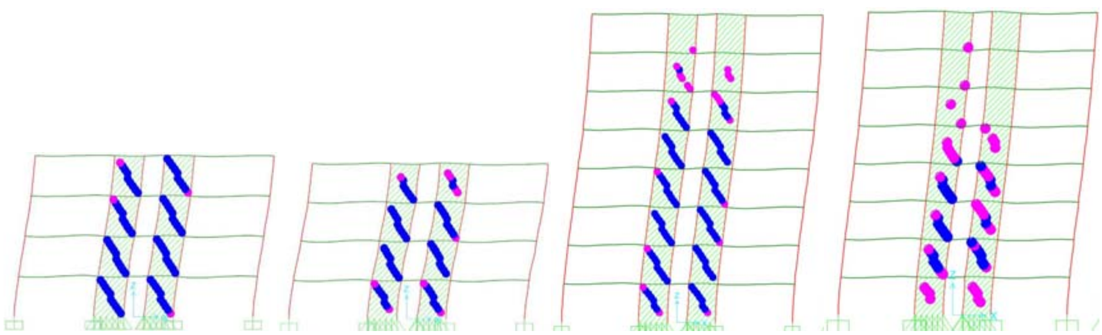
In Fig. 12 is presented the nonlinear static behaviour for all build frames in terms of base shear force and top displacement. The target displacements corresponding to ULS (blue dot in Fig. 12) and the maxim displacement of re-centering capacity, $d_{re\text{-}centering}$ (red dot in Fig. 12) with corresponding maximum inter-story drift ratios are also presented. The frames designed at high seismicity case, have a larger capacity and ductility than the ones designed at moderate seismicity case. The 8-story frames are more ductile than the 4-story frames and were designed to resist similar seismic forces (within the same ductility class).



(a) (b)

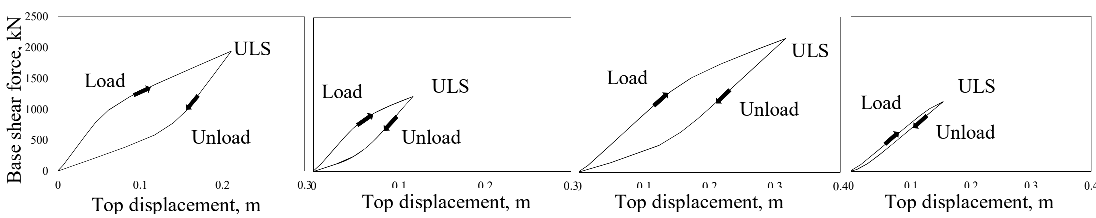
Fig. 12 - Capacity curves: a) 4 story building frame; b) 8 story building frame.

The plastic mechanisms are presented in Fig. 13 at target displacement corresponding to ULS. It can be seen that the plastic hinges are developed only in the shear panels with no damage in the MRF. Thus, the MRF has the necessary restoring force to re-center the building frames and then replace the damaged shear panels. Also, a re-centring analysis was performed, loading the frames up to ULS and then unloading to 0 force. The analysis showed that after unloading, no residual drift are present (Fig. 14) and confirmed the re-centring capacity of the frames.



(a) (b)

Fig. 13 - Plastic mechanism at ULS: a) 4 story building frame at high (left) and low (right) seismicity case; b) 8 story building frame at high (left) and low (right) seismicity case.



(a) (b)

Fig. 14 – Re-centring capacity at ULS: a) 4 storey building frames at high (left) and low (right) seismicity case; b) 8 storey building frames at high (left) and low (right) seismicity case.

5. Conclusions

Steel frames with thin-walled shear panels have been studied experimentally and the results showed that the system can be very efficient increasing both strength and stiffness of structural system, and having high ductility, with a stable cyclic behaviour (e.g. high dissipation capacity).

An extended design and analysis was conducted on dual building frames with replaceable thin-walled steel shear panels considering two building heights, i.e. 4 and 8 story, respectively. Two different design cases were considered: high seismicity case with high ductility class structures and moderate seismicity case with medium ductility class structures. Nonlinear static analyses was used in order to verify the collapse mechanism and re-centering capacity of building frames.

The study has shown that this innovative application of replaceable SPSW system can seismically protect multi-storey dual frames working as a “structural fuse” solution. When the SPSW frames are unloaded, they pass close to “0” displacement, with practically not residual displacement. Since the MRF are elastic up to ULS, the dual SPSW frames behave as a self-centering system.

The proposed acceptance criteria for the three limit states have to be validated through an extensive parametric study with buildings having different configurations designed at different seismicity cases.

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