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SEISMIC PROTECTION OF STEEL BUILDINGS BY PURE ALUMINIUM SHEAR PANELS

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SUMMARY

The use of energy dissipation systems for the control of seismic response of structures is a valid alternative to conventional seismic design methods. To this purpose, several devices have been proposed and many of them are based on the use of metallic-yielding technology. Shear panels represent a convenient passive seismic protection system for framed buildings. Hence, this paper focuses on the application of pure aluminium shear panels, serving as dissipative and stiffening device in steel moment resisting frames. In particular, global numerical analyses are presented aiming at proving the effectiveness of the proposed system and providing useful information on the procedure that should be followed for economical design of the structure. Therefore, different frame-shear panels combined systems are considered, they being arranged in such a way to obtain dual structural configurations with equivalent performance at the serviceability limit state, which is the most stringent performance level for the bare frame configuration. Besides, different panel configurations are considered, namely full-bay and pillar type. Static and dynamic inelastic analyses are carried out. The comparison of obtained results allows useful information for the selection of optimal steel frame-shear panel combined system to be drawn.

INTRODUCTION

Shear panels represent an interesting solution to resist lateral forces and to control the dynamic response of framed buildings. Due to their considerable shear stiffness and strength, they can be favorably used as a seismic resistance system under both moderate and strong earthquake loading. In addition, when designed as dissipative elements, shear panels can be viably used for the seismic protection of the primary structure, due to the large energy dissipation capacity related to the large portion where plastic deformations take place. As far as the stiffening effect is concerned, it has been already recognized that even lightweight metal shear panels may considerably improve the structural performance of the structure at the serviceability limit state (Miller [1], De Matteis [2]). On the other hand, the use of shear panels as hysteretic devices represents a new trend that has been strongly promoted by the introduction of Low Yield Strength (LYS) steel, which allows the practical fabrication of compact dissipative shear panels, by using simple and acceptable stiffener configurations in order to avoid the occurrence of premature buckling phenomena (Nakagawa [3]). In such a case, tension field action is prevented and the shear panel is characterized by a stable inelastic cyclic behavior and a uniform yielding spread over the entire surface (Tanaka [4]). Generally, the use of low yield strength material allows not only saving fabrication costs by

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adopting larger width-to-thickness ratio, but also the protection of the primary structure for reduced interstorey drift angles, since incoming seismic energy is dissipated also for low intensity seismic motions (De Matteis [5]).

Owing to not easy availability of LYS steel on the world market, the use of pure aluminium as metal material to build shear panels has been proposed (De Matteis [6]). Such a material is characterized by a very large ductility and by a yield stress level even lower than LYS steel. For this reason it should be particularly adequate for the fabrication of dissipative devices based on material yielding. Therefore, a wide experimental research project has been recently undertaken at the Department of Structural Analysis and Design of the University of Naples Federico II. It aims at investigating by means of full scale monotonic and cyclic tests the seismic performance of shear panels made of aluminium with high degree of purity and thermically treated in order to enhance the material mechanical features.

In order to assess the seismic performance of dual systems made of steel frames combined with pure aluminium shear walls, a numerical study is presented in this paper. In particular, the results of static and dynamic inelastic analyses carried out on different frame-shear walls combined systems are shown. In addition to different configuration of the primary structures, two panel configurations are considered, namely full-bay and pillar type. In the whole, the obtained results emphasize the adequacy of the proposed solution and allow us to clarify many aspects related to the design methodology that should be adopted to optimize the seismic response of dual systems according to a performance based design approach.

SHEAR WALL SYSTEMS IN STEEL FRAMES

Structural configurations

Basically, shear panels may be introduced into a lateral forces resisting system according to two types of structural schemes:

- *standard scheme*, where beam-to-column joints are considered to be simply pinned and shear walls are assumed to be the only lateral-force-resisting system in the structure (Figure 1a).
- *dual scheme*, where steel shear walls and moment frames behave as a combined system to resist external lateral forces. The moment resisting frame is considered as the primary structure acting as back up system to shear walls, which however absorb a large part of the lateral forces at least in the early deformation stages (Figure 1b).

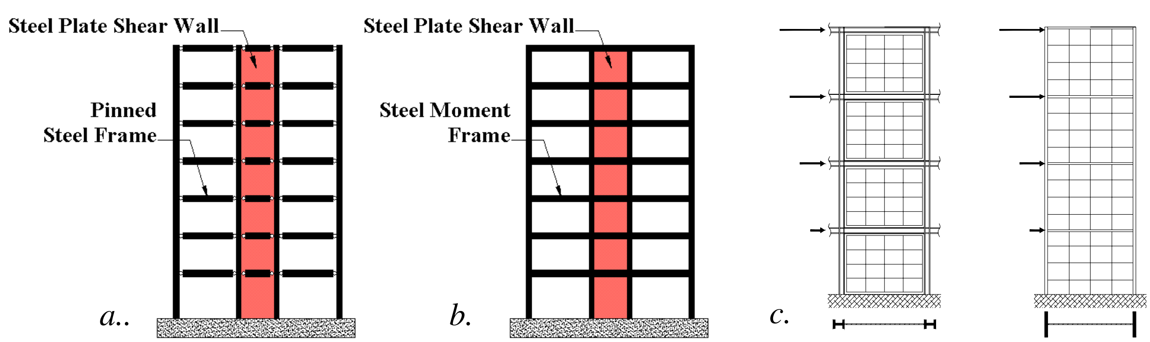
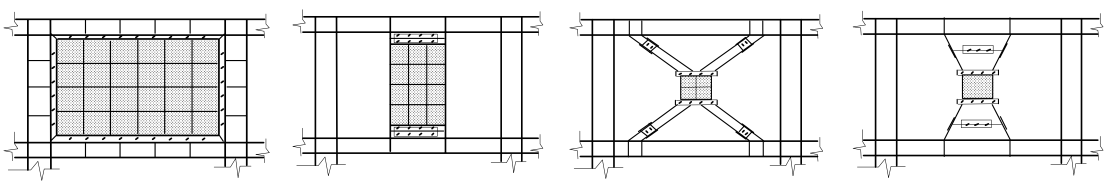


Figure 1. Typical systems with shear walls: a) Standard structural system, b) Dual structural system, c) structural analogy between shear wall and plate girder

In case single wall configuration (as shown in Figure 1) is adopted, the structural system can be schematised as series of plate elements, which are confined by boundary columns and horizontal floor beams. The steel plates, together with the lateral columns, resemble a vertical plate girder, where the columns act as girder flanges and the plate elements act as girder web (Figure 1c). On the other hand, horizontal floor beams act as the main transverse stiffeners placed in a plate girder, while the ribs of the single panel take the role of intermediate longitudinal and transversal stiffeners of the plate. The main advantages of this structural system are related to large lateral stiffness and strength as well as to the possibility to govern the collapse mechanism of the structure conferring large global ductility and high energy dissipation capacity.

Shear panel modeling

Shear panels may be inserted into the single mesh of the frame as a large panel rigidly and continuously connected to columns and beams, serving also as cladding panel (Figure 2a). As an alternative, smaller element installed at the nearly middle depth of the storey could be used. In such a case shear panels have to be connected to the beams by rigid support members in order to transfer the incoming shear forces, according to different schemes, namely partial bay type (Figure 2b), bracing type (Figure 2c) and pillar type (Figure 2d).



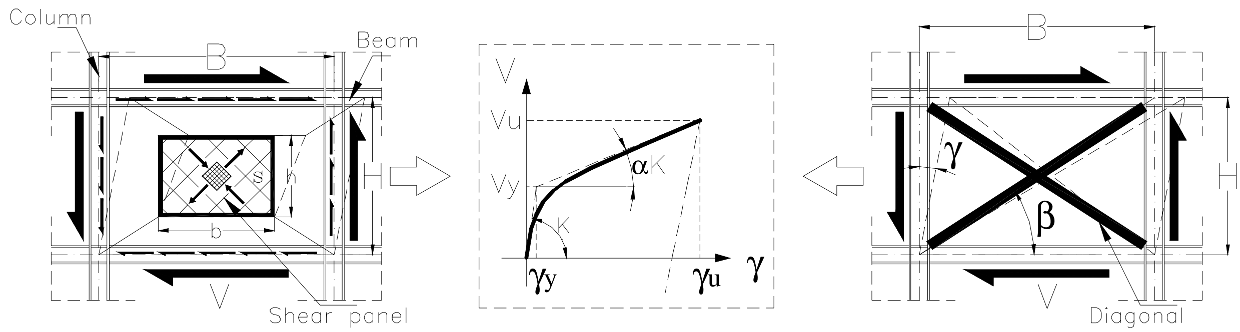
a. Full bay type *b. Partial bay type* *c. Bracing type* *d. Pillar type*

Figure 2. Typical arrangements for shear panels.

Processing time is an important aspect when performing numerical analysis of framed building provided with dissipative devices. For this reason, it is necessary to set up simplified models reproducing the effect of shear panels in terms of strength, stiffness and dissipative behavior. With reference to slender full bay type shear panels, several studies have been already carried out demonstrating the effectiveness of the so called strip model to predict the results of experimental tests (Driver [7]). The strip model can be easily extended and applied for compact full bay type shear panels as well, by taking into account compression principal stresses also. Therefore, full bay compact shear panels may be modeled as a double series of strips oriented in both tension and compression directions. When the effect of flexural interaction between shear panels and the boundary members is negligible, a further simplification can be based on the adoption of two trusses only, which are placed through diagonal directions, connecting the opposite corners of the frame mesh according to the X-bracing model. Mechanical features of diagonal trusses, namely section area A_d and material yield strength f^*_y, may be easily determined equating the shear behavior of the panel under consideration with that provided by equivalent diagonal members in terms of stiffness, elastic strength and post elastic behaviour. The same simplified model can be also assumed in case of different panel configurations - namely partial bay type, pillar type or bracing type – by relating the above relationships to the ratio between panel depth (h) and storey height (H) (see Figure 3).

Design criteria

Introduction of shear panels into steel framed structures allows the improvement of structural performance levels under lateral loads due to increasing of stiffness, strength and ductility. In addition, compact shear panels are also able to enhance the energy dissipation capacity of the whole structure, acting as sacrificial devices, absorbing a large amount of seismic input energy and protecting the primary framed structure from relevant structural damages. Therefore, compact shear panels can act as hysteretic dampers, whose dissipative function is activated by interstorey drifts occurring during the loading process of the structure. On the other hand it has to be taken into account that stiffening effect provided by shear panels produces an important increase of lateral stiffness of the whole structure and therefore the shifting of the structural period into the range of higher spectral acceleration. Such an effect should be considered in the design process.



Mechanical and geometrical properties of shear panel

E= Young's modulus
ν= Poisson's modulus
f_y= yield stress
ε_u= ultimate strain
s= thickness
V_y=conventional elastic shear strength
V_u= ultimate shear strength

Equivalence relationship

$$A_d = \frac{B \cdot s}{4(1+\nu)\cdot \mathrm{sen}\beta \cdot \cos^2\beta} \cdot \frac{H}{h}$$

$$f_y^* = \frac{2(1+\nu)\cdot \mathrm{sen}\beta \cdot \cos\beta}{\sqrt{3}} \cdot \frac{h}{H} \cdot f_y$$

$$\alpha \approx \frac{2(1+\nu)\cdot \mathrm{sen}\beta \cdot \cos\beta}{\sqrt{3}} \cdot \frac{(V_u - V_y)}{V_y} \cdot \frac{h}{H} \cdot \frac{f_y}{E \cdot \varepsilon_u}$$

Mechanical and geometrical properties of equivalent diagonal

E= Young's modulus
ν= Poisson's modulus
f^*_y= yield stress
α= strain hardening factor
ε_u= ultimate strain
A_d= diagonal cross-section area

Figure 3. Equivalence between shear compact shear panel and X-bracing model

Generally, owing to their large lateral flexibility, bare frames designed according to strength and therefore with reference to ultimate limit state only are not able to meet also serviceability limit state requirements prescribed by current structural codes. Hence, shear panels may be profitable used also as upgrading system, which provides the complementary rigidity to the frame to fulfil minimum stiffness requirements. In this way, the whole structure has to be intended as a composite (dual) system, where the primary structure exhibits elastic deformations only under moderate earthquakes, while it becomes a useful supplementary energy dissipation system for medium and high intensity earthquakes, developing plastic hinges in beams and columns. On the other hand, shear panels have to be intended as the main energy dissipative system, supplying also additional lateral stiffness and strength to the whole structure (see Figure 4). Design criteria for a dual system have to be applied aiming at optimising the structural performance of the whole structure to allow the achievements of predefined performance targets keeping the minimum fabrication costs. The main variables are stiffness and strength ratios between the primary structure and the complementary one, which should be determined following a sort of trial and error procedure.

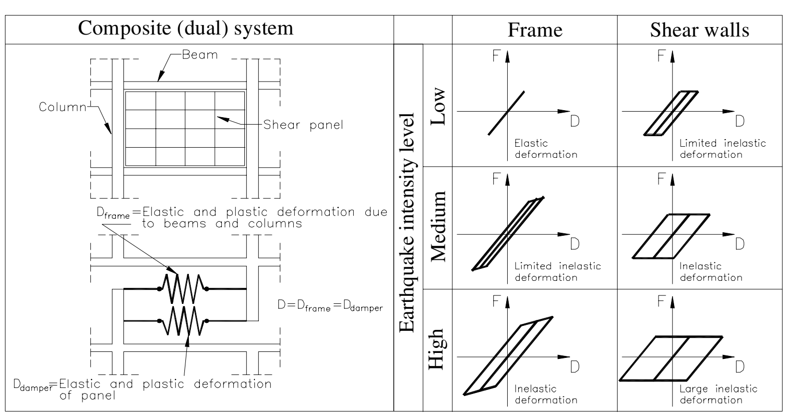


Figure 4. Schematized modeling for frame-shear wall combined systems

THE NUMERICAL STUDY

General

In the following, the results of a numerical investigation carried out on moment resisting steel frames equipped with compact pure aluminium shear panels are presented. Initially, full bay type shear panel configuration is considered. Then, for the lake of example, pillar type shear panel configuration is taken into account. Aiming at determining optimal design solutions, several frame-shear panel combinations are considered, taking on as an equivalence criterion the same global lateral stiffness of the whole structure. Therefore, the frame component is varied, starting from a standard system given by a simple pin-jointed steel frame designed according to gravity load, where shear wall is the only lateral force resisting system, passing through different dual system configurations where lateral forces are shared between steel frame and shear walls, and arriving to the opposite solution given by a rigid steel frame designed according to the serviceability limit state, where shear panels are not needed to fulfil the design requirements. The performance of these systems is evaluated by checking the fulfilment of difference performance levels and also the relevant failure mode exhibited by the structure. Obviously, favourable collapse mechanisms are those concerned with the progressive yielding of shear panels and plastic hinges development in the end sections of the beams of the primary structure. In particular, for the optimisation of passive protection exercised by shear panels, it would be valuable that developing dissipative mechanisms follow a hierarchical in such a way the development of plastic hinges in the frame sections takes place only after shear panels have undergone significant plastic deformations.

Analyzed structural system configurations

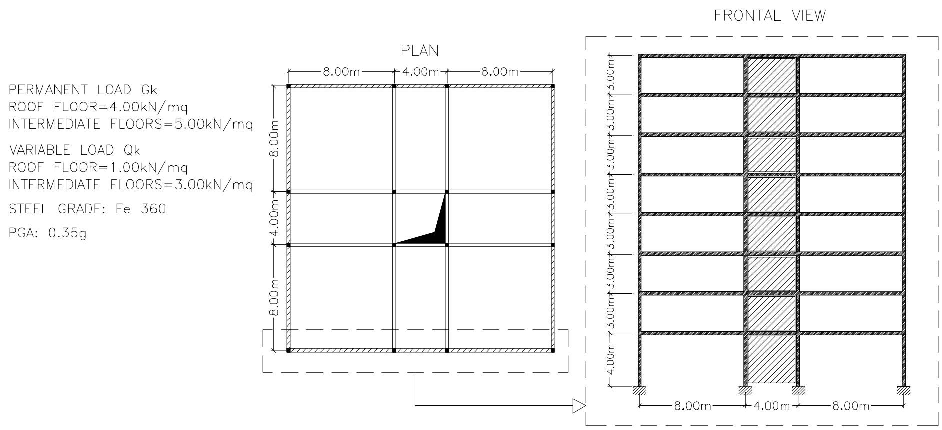
The steel building under investigation is an eight-story building with a 20.00 m side square plant. The total height of the building is 25.00 m, the interstorey height being 4.00 m and 3.00 m for the first floor and all the other floors, respectively. The building resistant system is given by moment resisting frames, which are placed though the perimeter of the building, and by internal pin-jointed frames. It is assumed that structural members are made of Fe360 steel grade. As far as the design seismic action is concerned, the envelope of the linear elastic design response spectra provided by EC8 [8] (PGA= 0.35g) has been taken into account. Main characteristics of analyzed building, in terms of both geometry and design loads, are given in Figure 5a.

According to the scope of the study, different configurations of lateral force resisting frames have been considered (Figure 5b). Therefore, perimeter frames have been designed according to:

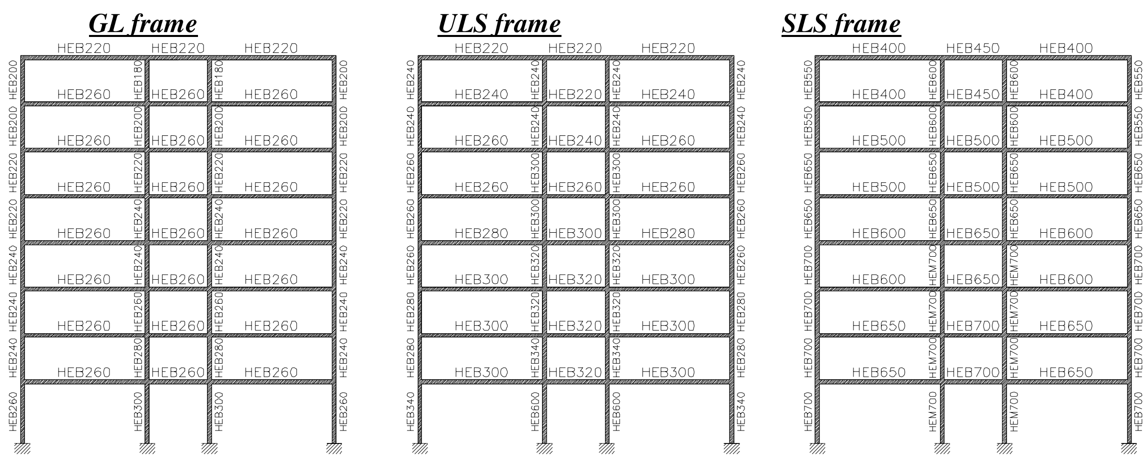
a. gravity loads only (GL configuration);
b. lateral strength, considering only the fulfillment of the ultimate limit state provided by EC8 assuming a design q-factor (q_d) equal to 6 (ULS configuration);
c. lateral stiffness, considering also the fulfillment of the serviceability limit state related to an elastic interstorey drift limit equal to 0.6% related to earthquake loading equivalent to the half of the elastic base shear force (SLS frame configuration).

In a second design phase, in order to allow the fulfillment of the serviceability limit state as well, GL and ULS frame configurations have been upgraded by means of compact shear panels placed into the central bay of the frame, according to the single shear wall configuration and full bay shear panel arrangement. For the accomplishment of such a requirement, different solutions have been examined, varying the strength of the shear wall component, which has been proportioned in such a way to absorb 50%, 75% or 100% of the whole design base shear force related to SLS frame configuration, since the latter should have a fundamental period of vibration similar to the one of structures upgraded by shear wall. Besides, for the sake of comparison, an additional system has been considered, using the same basic configuration of GL frame, but with simple pinned beam-to-column joints (P configuration). In such a case the whole seismic action is carried by shear panels, which therefore have been assumed as the ones designed according to the 100% of the design base shear force related to SLS frame configuration.

For each shear wall component, the bare frame configuration has been modified accordingly, starting from those given in Figure 5b and by increasing the column sections of the central bay in order to allow them to resist the normal forces transmitted by the confined shear walls, according to the plate girder analogy, as well as to allow the combined frame-shear wall system fulfilling serviceability limit state requirements. For this reason, it has been necessary to use hollow square sections, since standard I-shape sections were not able to satisfy strength and stiffness requirements (see Figure 9).



a. Plan and front view of examined building



b. configurations of bare frames designed according to: Gravity Load (GL), Strength (ULS), Stiffness (SLS)

Figure 5. The analyzed structure

Adopted shear panels

For seismic upgrading of steel frames, compact shear panels characterized by a pure shear dissipative mechanism have been considered. Compact shear panels are generally made of low yield strength metals, which thanks to the high E/f_y ratio allow larger plate width-to-thickness ratio to be adopted. As an alternative to Low Yield Strength (LYS) steel, which is usually characterized by a nominal yield stress of about 90-120 MPa and the same Young's modulus of ordinary steel, the material proposed in this paper is the 'pure' aluminium, which is characterized by a very low percentage (less than 0.5%) of alloying elements. Such a material, which is easily available on the world market, is able to provide a yield stress even lower than LYS steel. On the basis of some preliminary material tests carried out at the University of

Naples Federico II, after adequate heat treatment, the E/f_y ratio can be assumed equal to 2.800, therefore greater than the one related to LYS, see De Matteis [6]. The possibility to provide shear panels made with the above material with adequate stiffeners to delay the occurrence of buckling phenomena to shear deformation higher than the required one has been successfully analyzed and drawn in De Matteis [9], where adequate design charts are given.

In current numerical study, according to the above material test results, the following values of the basic mechanical properties have been considered: conventional yield strength f_y=25 MPa, hardening ratio V_u/V_y = 3, Poisson's modulus ν=0.33, ultimate elongation ε_u=50%. For each configuration, the thickness of shear panels has been determined in relation to the relevant design shear force, by assuming uniform shear stress distribution on panel shear area.

Analysis program and result evaluation

Seismic performance of steel frame equipped with pure aluminium shear panels has been evaluated by means of static and dynamic inelastic analyses, where shear panels have been modeled through equivalent inelastic truss members characterized by a stable inelastic cyclic behavior.

For static analyses, the structures is laterally pushed towards increased displacements according to a typical pushover procedure. The output of analysis is given in terms of normalized base shear force ($V^*= V/W$, V and W being the base shear force and the global seismic weight, respectively) and maximum interstorey drift ratio of the structure (γ). The main information gained by such analyses are: normalized base shear force (V^*_{DP}) and associated maximum interstorey drift ratio (γ_{DP}) corresponding to the first yielding of shear panels; normalized base shear force (V^*_{DF}) and associated maximum interstorey drift ratio (γ_{DF}) corresponding to the first plastic hinge in frame member sections; collapse mechanism of the structure; sequence and spreading of plastic deformations throughout the whole structure (shear panels and frame member sections).

Dynamic analyses are carried out using the so-called Incremental Dynamic Analysis (IDA) procedure, see FEMA 350 [10]. Such a procedure consists in scaling up the peak ground acceleration (PGA) of a given ground motion record relating the normalized elastic spectral acceleration ($S_{a,e}/g$) to the maximum interstorey drift of the structure (γ). The obtained diagram allows the design performance objectives of the structure to be easily checked. In particular, four different performance levels are considered in this study: (1) elastic limit state of shear panels (DP-Damage of Panels), related to the first significant yielding of the most stressed panel; (2) elastic limit state of the frame (DF-Damage of Frame), related to the first significant yielding of frame sections; (3) Serviceability Limit State (SLS), related to the achievement of interstorey drift ratio γ_{SLS}=0.006 rad; (4) Ultimate Limit State (ULS), conventionally assumed for a maximum interstorey drift ratio γ_{ULS}=0.03 rad.

Four different natural records have been selected for carrying out dynamic inelastic analyses. The main characteristics of considered records are given in the Table 1. It can be observed that the selected accelerograms can be considered to be rather homogeneous, since they have similar values of the Trifunac duration (Trifunac [11]).

Table 1. Main characteristics of considered natural records

Earthquake	*Ground Acceleration Record*			
	Name of the station	Direction of the component	PGA (g)	*Trifunac Duration (s)*
Campano-Lucano (Italy, 1980)	Sturno	EW	0.32	38.53
	Sturno	NS	0.22	40.02
	Bagnoli-Irpino	EW	0.18	31.81
	Bagnoli-Irpino	NS	0.14	41.08

In order to evaluate the seismic performance of analysed structures, on the basis of obtained pushover curves and IDA curves, the following performance indexes have been defined (see Figure 6):

- safety factors related to SLS (sf_{SLS}) and ULS (sf_{ULS}), defined as the ratio between elastic spectral acceleration when these limit states are achieved ($S_{a,e,SLS}$ and $S_{a,e,ULS}$) and the corresponding minimum values prescribed by EC8 ($S_{a,e,SLS}{}^{EC8}$ and $S_{a,e,ULS}{}^{EC8}$);
- seismic protection index (SPI), defined as the ratio between the elastic spectral acceleration corresponding to first plastic hinge in the frame ($S_{a,e,DF}$) and the one related to first plastic deformation in the panels ($S_{a,e,DP}$), such a ratio being representative of the dissipative capacity of the structure before the occurrence of damages in primary structure;
- actual q-factor (q), defined as $S_{a,e,ULS}/S_{a,e,DP}$, such a ratio being representative of the global dissipative capacity of the system.
- structural weight saving, evaluated with respect to the SLS frame configuration. Owing to higher fabrication costs of pure aluminium shear panels than ordinary steelworks, the weight of shear panels has been conventionally considered five times higher than the actual one.

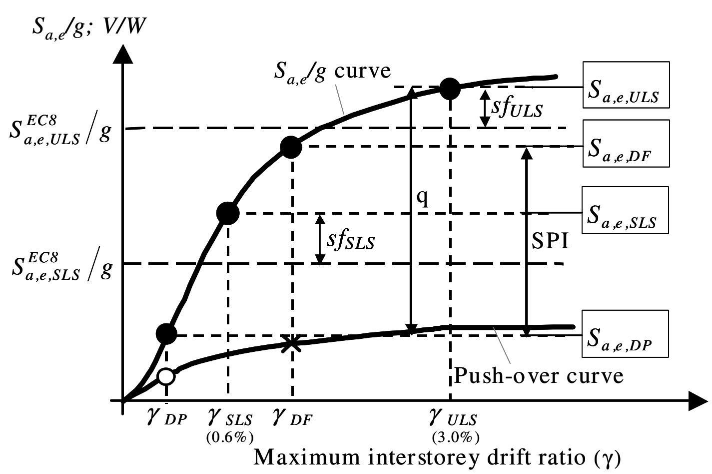


Figure 6. Definition of performance indexes

THE OBTAINED RESULTS

Pushover analyses

Pushover curves of the examined frame-full bay shear panels combined systems are given Figure 7. In particular, Figure 7a is related to GL dual system configuration, where the frame designed according to gravity loads only is upgraded by means of shear walls with different strength ratios, going from a shear wall component ($V_{d,P}$) able to resist the 100% of the global design shear force (V_d) - conventionally assumed as the one related to the SLS configuration - to a shear wall component able to resist 50% V_d. For the sake of comparison, pushover curves related to the bare GL frame configuration and to the SLS frame configuration are given. Analogously, Figure 7b is related to the ULS dual system configuration, where the frame designed according to ultimate limit state only is upgraded by means of shear walls having a strength ratio ($V_{d,P}/V_d$) ranging from 0.5 to 1.0. In both cases, the response curve for the pin-jointed frame configuration P upgraded with a shear wall component absorbing the seismic base shear force V_d is depicted.

The above figures emphasise clearly the upgrading effect due to aluminium shear wall. Obviously the performance of considered systems improves gradually as far as the shear wall component increases. Anyway, it should be observed that the SLS frame configuration gives rise to seismic performance much higher than dual system, but this is due to the extremely large overstrength ratio concerned with this frame configuration, which is caused by the difficulty in fulfilling the serviceability limit state checks for the analysed steel framed building. Such an overstrength ratio, which can be defined as $V^*_{DF}/(S_{a,e,ULS}/q_d)$, where q_d is the design q-factor assumed equal to 6 in the case being, is equal to about 2.5. This means that the SLS frame is very far from representing an efficient solution in relation to prescribed performance levels and structural economy. Also, it is worth noticing that SLS frame exhibits structural damage in members for a maximum interstorey drift ratio equal to about 0.4%, which is smaller than the one associated to serviceability limit state check (γ_{SLS} =0.6%).

On the other hand, the obtained results highlight that all frame-shear wall combined systems actually have an initial lateral stiffness similar to SLS frame, emphasising the strong stiffening effect provided by adopted shear walls. Also, for all the examined cases it can be observed that the first significant damage of steel frame is now related to an interstorey drift ratio γ_{DF} = 0.9%, therefore greater than 0.6%. Such a value seems to be independent of the strength ratio of shear wall ($V_{d,P}/V_d$) as well as of the primary structure (GL or ULS frame configuration). In the whole, this result proves the seismic protection effect provided by aluminium shear panels, which produce a large shifting of the interstorey drift ratio γ_{DF}, keeping the primary structure acting as an elastic back up system up to large deformation levels.

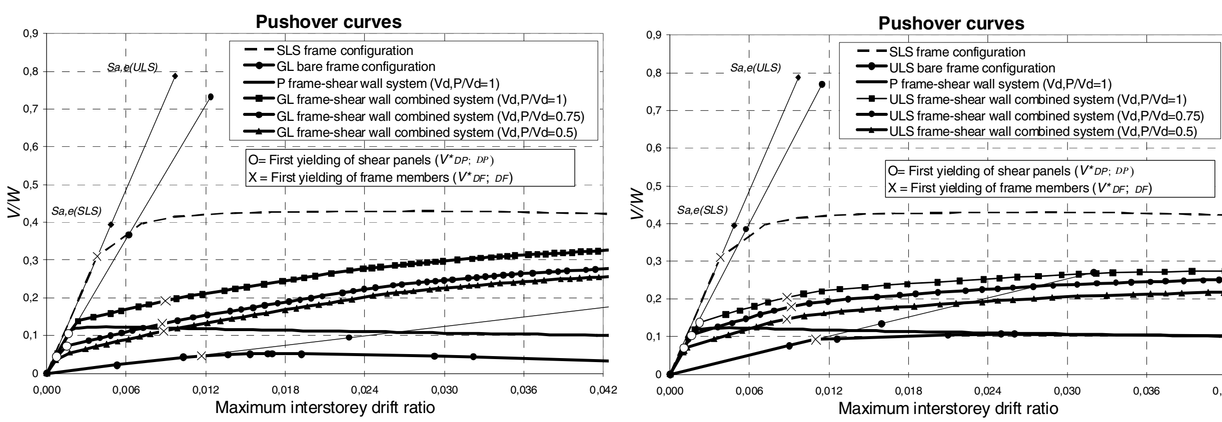


Figure 7. Pushover curves for GL (a) and ULS (b) system configurations

Also, it is worthy noticing that design strength ratio of shear wall ($V_{d,P}/V_d$) rules the level of seismic force producing the first damage in the structure, both for shear panels and for frame members. In particular, it is interesting to compare the value of the normalized base shear force corresponding to the first yielding of steel frame (V^*_{DF}) and the design seismic base shear force prescribed by the relevant seismic provision (in the case being, $S_{a,e,ULS}/q_d$). As a matter of the fact, for GL dual system configurations, the ratio V^*_{DF} / ($S_{a,e,ULS}/q_d$) is equal to 0.9, 1.1 and 1.6, for shear wall strength ratio ($V_{d,P}/V_d$) equal to 0.5, 0.75 and 1, respectively. Similarly, in the case of ULS dual system configurations, the ratio V^*_{DF} is equal to 1.2 $S_{a,e,ULS}/q_d$, 1.5 $S_{a,e,ULS}/q_d$ and 1.7 $S_{a,e,ULS}/q_d$, for shear wall strength ratio ($V_{d,P}/V_d$) equal to 0.5, 0.75 and 1, respectively. For this reason, it can be concluded that the optimal configurations for GL and ULS dual system among those analysed could be defined the ones having a shear panel strength ratio ($V_{d,P}/V_d$) equal to 0.75 and 0.50, respectively. In fact, in these cases the normalized base shear force producing the first yielding of the primary structure (V^*_{DF}) is very similar to the design spectral acceleration provided by the code for the whole structure ($S_{a,e,ULS}/q_d$=0.72/6=0.12g).

Figure 7 highlights as dual frame-shear wall solutions are more effective than standard shear wall configuration associated to a pinned frame. In fact, even if the response of the latter configuration appears to provide the same initial stiffness and a higher base shear force corresponding to the first yielding of the system (V^*_{DP}), the better performance of dual systems for large deformation levels is noticeable. This is due to the cooperation of ductile shear walls with the primary structure, the latter, through the development of frame plastic mechanism, providing a significant contribution to the global energy dissipation capability of the compound system, which therefore can benefit of a considerable plastic overstrength.

On the other hand, it is apparent that the global strength of the above frame-shear panel combined systems is limited in comparison to the one related to SLS bare frame configuration. In addition to the high structural overstrength of SLS frame configuration, this is due to the adopted system based on full bay type, where the limited shear deformations of the panels do not allow the full exploitation of the hardening resources. For this reason, shear panels with pillar type configurations are considered. In fact, the adoption of shear panels having depth (*h*) lower than frame interstorey height (*H*) gives rise to higher ductility demand for shear panels and therefore to higher contribution in dissipating input seismic energy. In the following results, pillar type shear panels are characterised by a height ratio (*h/H*=1/3) and a width equal to 225 cm. For the sake of example, pillar type shear panels have been determined aiming at upgrading the above ULS bare frame, by assuming a design base shear $V_{d,P}$ equal to 50%, so to allow a large plastic involvement of shear panels. Relevant results are given in Figure 8, where pushover curves related to frame-pillar type combined systems are compared with the ones related to selected frame-full bay type combined systems.

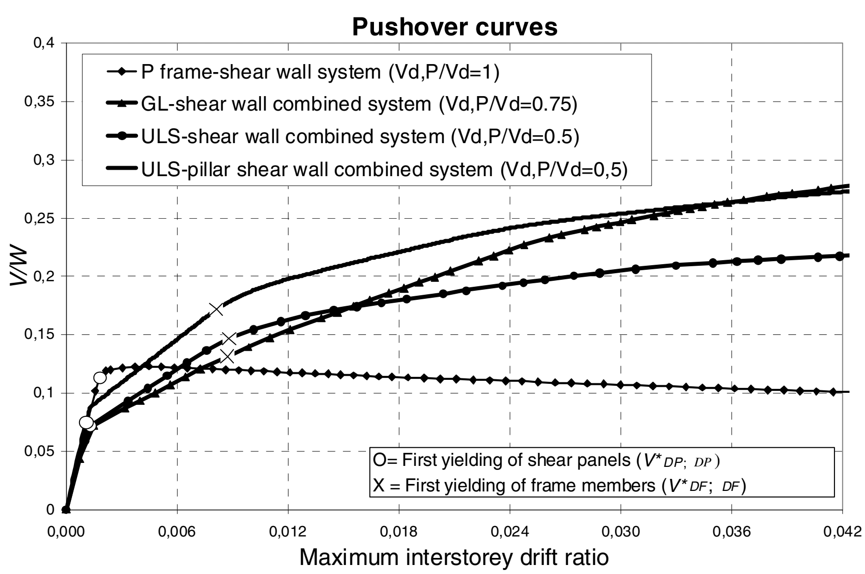
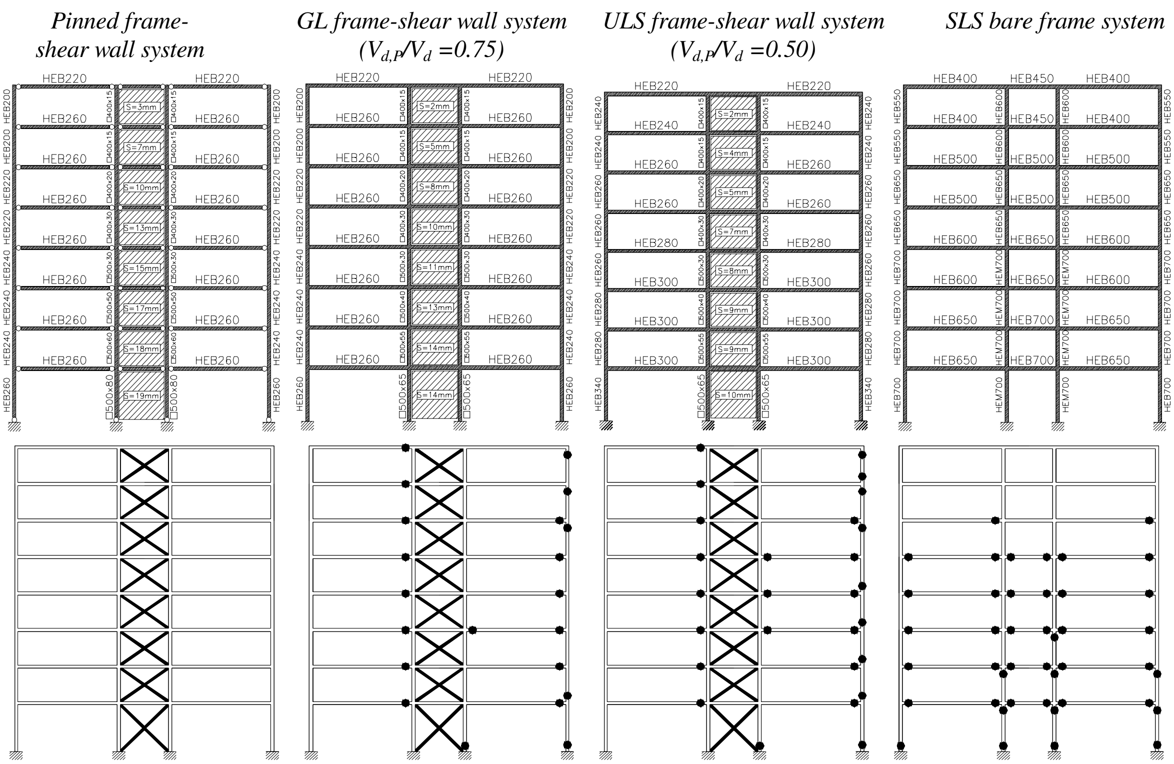
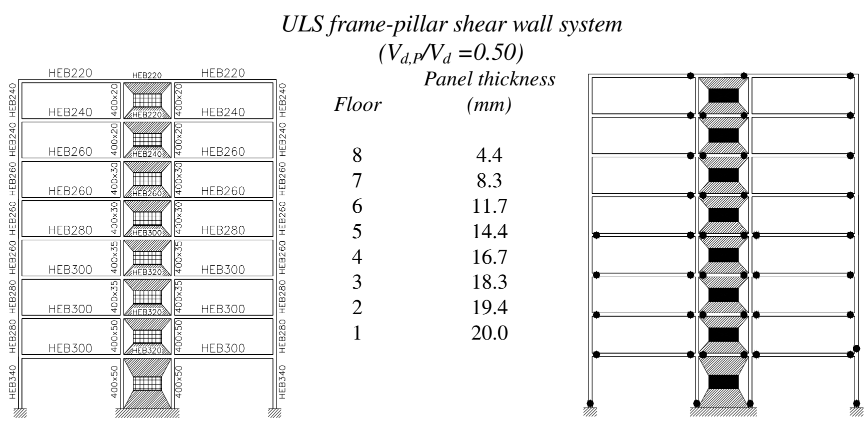


Figure 8. Pushover curves for selected system configurations

It is apparent that global structural performance of ULS frame with pillar type panels is better than the one of ULS frame with full bay type panels, especially for reduced deformation levels, proving the higher efficiency of this shear panel configuration. Only for larger interstorey drift demand (> 3%), seismic performance of ULS frame with pillar type panels proportioned with 50% of design base shear $V_{d,P}$ is similar to that of GL frame with full bay shear panels proportioned with 75% of design base shear $V_{d,P}$. Besides, it can be observed that the first significant damage of steel frame is now related to interstorey drift ratio $\gamma_{DF} = 0.8\%$, slowly lower than to one related to full bay shear panels systems.

Finally, in Figure 9, for the above examined structures, relevant dissipative mechanisms for a conventional collapse corresponding to a maximum interstory drift ratio of 3% are depicted. It can be observed that in all the cases the hierarchical order of plastic deformations panels-beams-columns is respected. All panels are in the plastic field and plastic hinges taken place in almost all the beams. This is true in case of both full bay shear panel configurations and pillar shear panels configurations, where global type collapse mechanisms are of concern. On the contrary, in SLS steel frame configuration, plastic deformations are mainly concentrated at lower floors and a partial type collapse mechanism takes place.





Floor	*Panel thickness (mm)*
8	4.4
7	8.3
6	11.7
5	14.4
4	16.7
3	18.3
2	19.4
1	20.0

Figure 9. Plastic deformation distribution at 3% maximum interstorey drift ratio

Inelastic dynamic analyses

The results of inelastic dynamic analyses related to full bay type dual systems selected on the basis of pushover analyses together with the ones related to SLS frame configuration and standard shear wall (pinned frame) configuration are given in Figure 10. IDA curves are provided for each analysed record as well as in terms of average among the four analysed records. From the examination of these results it is apparent that the main design objective, which is related to the fulfilment of the serviceability limit state, is practically achieved for all the cases. In fact, maximum interstorey drift displacements are in the order of 0.6% for a normalized elastic spectral acceleration corresponding to a frequent earthquake defined by EC8 ($S_{a,e,SLS}^{EC8}$).

In Figure 11a, results related to pillar type dual system are provided, while in Figure 11b the comparison among seismic performance of different analyzed structural configurations is given with reference to the average curves of the considered earthquake records. It is clearly shown that the difference between SLS frame configuration selected steel frame-shear wall combined systems are much more reduced as respect to what evidenced by pushover analyses. This can be explained by considering the significant contribution provided by low-resistant shear panels in terms of dissipation of input seismic energy. In particular, it is apparent that better seismic performance is the one related to pillar type shear wall, due to the major plastic involvement of shear panels.

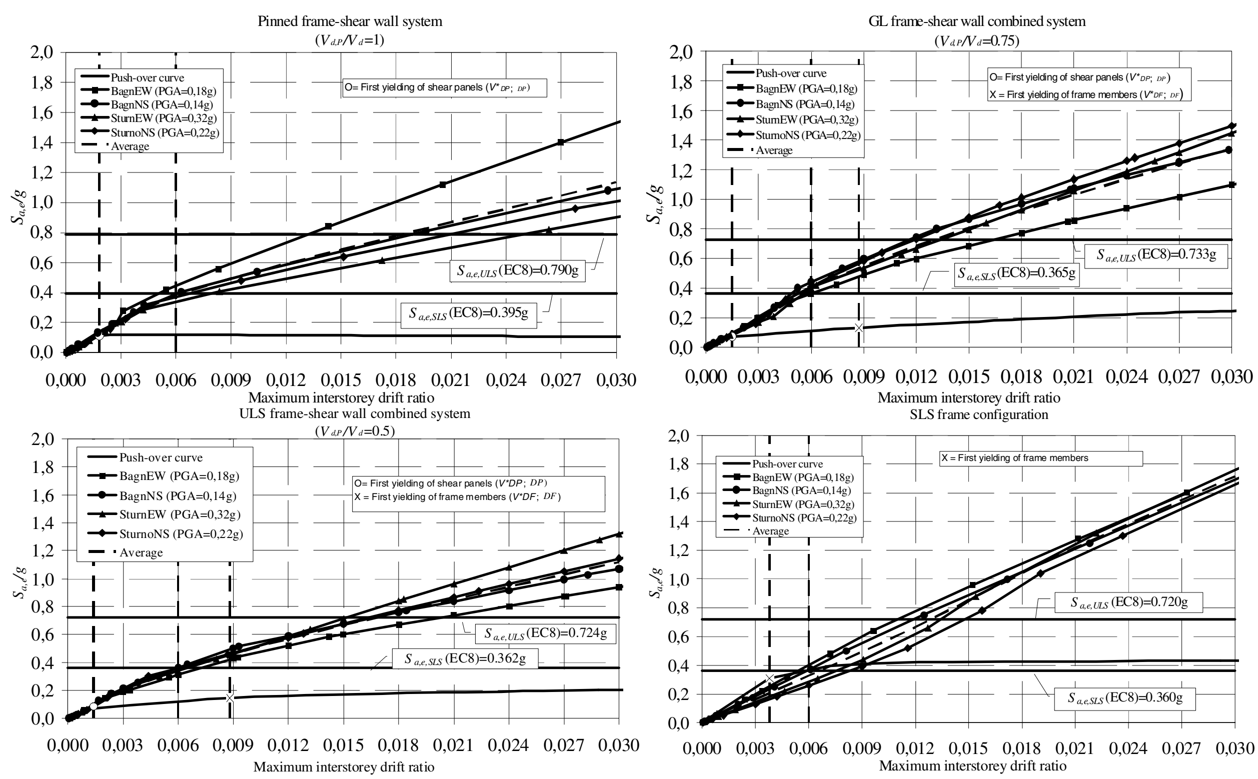


Figure 10. Incremental dynamic analyses curves for full-bay type shear panel structural configurations

In order to have a numerical evidence about the comparative seismic performance of analyzed structural configurations, values assumed by previously defined performance indexes are given in Table 2. It can be observed that the serviceability limit state check is strictly satisfied (sf_{SLS} factor around 1), proving that it is the most conditioning for the structural design. On the other hand, the safety factor related to ultimate limit state check of frame members is rather high for the SLS frame configuration (sf_{ULS}=2.4), proving the significant and useless overstrength of this structural solution. The same factor, for the other configurations, assumes a more favorable value. It is also interesting to note the value assumed by the Seismic Protection Index (SPI), which provides a measure on how the primary structure damage is delayed with respect to the activation of energy dissipation in shear panels. The obtained values emphasize the effectiveness of the proposed seismic protection strategy. Finally, the q-factor value gives a measure of the global dissipative capacity of the system. It is apparent that higher values are referred to dual frame-shear wall systems, and in particular to pillar type shear wall, the latter having also a safety factor for ultimate limit state check even larger than SLS frame configuration. This stresses how the combination of shear wall and rigid steel frame exalts the dissipative capacity of the system, enlarging the portion of the behavioral curve of the structure where energy dissipation takes place. The above outcomes, are corroborated by examining structural weight saving index, where it appears that ULS frame-panel combined system provide optimal solution, especially when pillar type shear panels are employed, this structural configuration being associated to the minimum fabrication cost and also to better seismic performance.

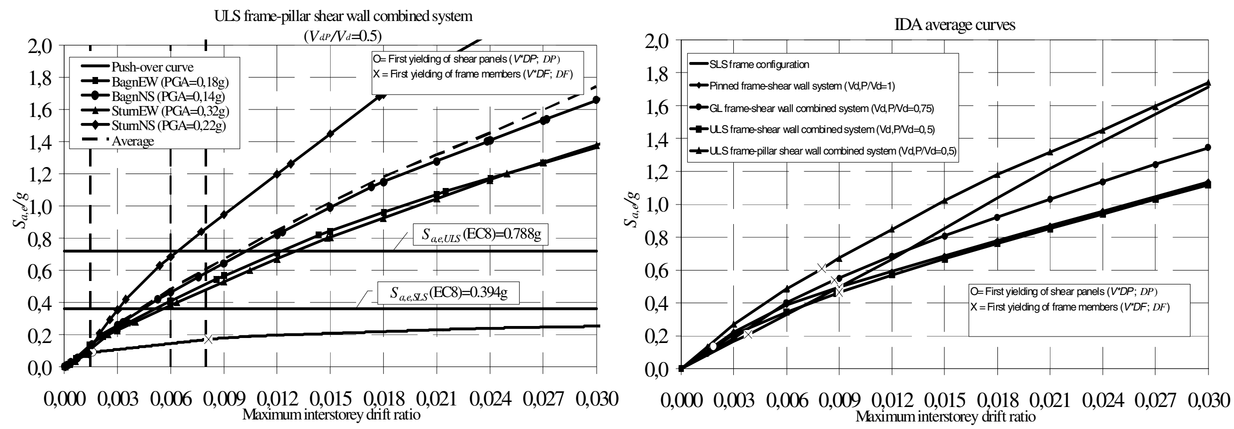


Figure 11. IDA curves for pillar type shear panels (a) and comparison among analyzed structural configurations (b)

Table 2. Performance indexes

System	*Performance indexes*				
	sf_{SLS}	sf_{ULS}	SPI	q-factor	Saving weight
Pinned frame-shear wall system ($V_{d,P}/V_d$=1)	1.0	1.4	-	10	+10.4%
GL frame-shear wall combined system ($V_{d,P}/V_d$=0.75)	1.1	1.8	8.1	16	-1.9%
ULS frame-shear wall combined system ($V_{d,P}/V_d$=0.5)	1.0	1.5	6.3	14	-5.9%
ULS frame-pillar shear wall combined system ($V_{d,P}/V_d$=0.5)	1.2	2.2	4.5	12.5	-12.5%
SLS frame configuration	1.0	2.4	-	6.8	Ref. value

For each considered limit state, the mean return period T_r (in years) of the corresponding earthquake can be approximately calculated scaling the code spectral value through the following expression [12]:

$$S_{a,e}(T_r)=\left(\frac{T_r}{475}\right)^n \cdot S_{a,e,ULS} \tag{1}$$

where $S_{a,e}(T_r)$ is the elastic spectral acceleration obtained at the considered limit state. According to Eurocode 8, $S_{a,e,ULS}$ is related to return period of 475 years and n factor is assumed equal to 0.43, so to have T_r = 95 years for a seismic event corresponding to the serviceability limit state checking with reduction factor ν=2. The obtained results are given in Table 3.

Table 3. Mean return period T_r (years) and corresponding probability of exceedance

Structure	*Limit state*			
	DP	*SLS*	*DF*	*ULS*
Pinned frame-shear wall system	8 (71%/10 y.)	95 (10%/10 y.)	-	1105 (4.4%/50 y.)
GL frame-shear wall combined system	3 (96%/10 y.)	117 (8.2%/10 y.)	230 (4.3%/10 y.)	1949 (2.5%/50 y.)
ULS frame- shear wall combined system	3 (96%/10 y.)	95 (10%/10 y.)	162 (6%/10 y.)	1304 (3.8%/50 y.)
ULS frame-pillar shear wall combined system	3 (96%/10 y.)	156(6.2%/10 y.)	261(3.8%/10 y.)	2997 (1.7%/50 y.)
SLS frame configuration	-	76 (12%/10 y.)	24 (34%/10 y.)	3579 (1.4%/50 y.)

It is shown that the dual systems are able to increase the mean return period T_r of the earthquake producing the occurrence of first plastic hinge in the frame up to ten times. On the other hand, it should be noted that the plastic deformation of shear panels is associated to very low intensity earthquake. But, it is also worthy noticing that first damage of a shear panel does not require its replacement, since significant degradation effects due to plastic deformation are usually related to much higher interstorey drift limits [9].

CONCLUSIONS

In this paper the seismic performance of steel frame-pure aluminium shear wall combined systems has been investigated. With reference to a typical perimeter frame steel building, static and dynamic inelastic analyses have been carried out. Several dual structural systems have been taken into consideration, where the basic steel frames have been designed according to different resistance levels. Two different types of shear panels have been considered, namely full-bay panels and pillar type panels. The obtained numerical results show the effectiveness of shear wall-frame combined systems for the seismic protection of steel framed buildings in comparison with structural solutions based on bare steel frame configurations. In fact, the protection provided by aluminium shear panels delays the first significant damage of frame members at interstorey drift ratios higher than the one corresponding to serviceability limit state. Also, it has been observed that the ultimate limit behaviour of steel frame-shear panel dual structural systems is characterized by favourable collapse mechanisms, following a hierarchical order characterized by the progressive yielding of shear panels and successive plastic hinge development in the end sections of all the beams of the primary structure. In order to have a clear evidence of the beneficial effects provided by shear panels, the seismic behaviour of analysed structures has been evaluated by different performance indexes related to some significant limit states. The above results clearly show the upgrading effect of aluminium shear wall, which provides a useful stiffening effect at serviceability limit state, a significant energy dissipation contribution for low intensity earthquakes as well as a remarkable contribution for the global behaviour of the whole system at ultimate limit state. This allows the optimisation of the seismic behaviour of dual systems in relation to different performance levels. In particular, it has been observed that better seismic performance is related to pillar type shear wall, due to the major plastic involvement of shear panels and consequent significant contribution provided in terms of dissipation of input seismic energy. The same findings are emphasized by the assessment of structural weight related to examined configurations, where it appears that steel frame-aluminium shear panel dual systems allow a significant economical saving, giving rise to structural configuration more convenient than standard systems (bare steel frames and pinned frame-shear panel systems), especially when pillar type shear panels are employed.

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