

## DISSIPATIVE DIAPHRAGM CONNECTIONS FOR PRECAST STRUCTURES WITH CLADDING PANELS UNDER SEISMIC ACTION

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### ABSTRACT

Research activities carried out at European level emphasized the critical role of the cladding panels on the seismic performance of precast concrete structures. It has been shown that structural configurations of precast buildings involving an interaction between the outer frames and the cladding panels, owing to the panel-to-structure fastening systems, may draw high forces into the diaphragm or lead to strong distortions of the roof deck. The use of panel-to-panel dissipative devices, recently proposed to improve the seismic performance of precast structures with cladding panels, may also lead to this effect. This is particularly important for single-storey industrial precast structures that are not provided with a rigid diaphragm. The stiffness of the diaphragm usually depends on the mechanical connections of the roof deck only. Innovative design solutions based on the use of metallic roof-to-roof dissipative devices are hence proposed to mitigate these effects by improving the diaphragm action under controlled forces. The effectiveness of the proposed solutions is investigated by means of non-linear dynamic analyses.

*Keywords: Precast structures; Seismic performance; Diaphragm action; Cladding panels; Dissipative connections.*

### 1. INTRODUCTION

During the last two decades a series of European co-normative research projects has been performed to support the standardisation of seismic design criteria of precast structures. A preliminary research program with cyclic and pseudodynamic experimental tests on precast concrete columns (Figure 1a) was developed at the European Laboratory of Structural Assessment (ELSA) of the Joint Research Centre (JRC) of the European Commission in the years 1994-1998 during the drafting of the first version of Eurocode 8 (Toniolo and Saisi 1998). Probabilistic analyses and pseudodynamic tests on full scale prototypes of one-storey buildings (Figure 1b) were carried out within the ECOLEADER project (2002-2003) to show the equivalence of the seismic capacity of precast design solutions compared with corresponding cast-in situ solutions (Biondini and Toniolo 2009, 2010). The Precast Structures EC8 project (Growth Programme 2003-2006) addressed the seismic behaviour of precast structures for industrial buildings (Ferrara et al. 2006, Biondini et al. 2008), with pseudodynamic tests on full scale prototypes with different orientation of the roof members (Figure 1c). The SAFECAST project (2009-2012) investigated the role of the connections in precast structures (Toniolo 2012, Biondini et al. 2012, Bournas et al. 2013, Negro et al. 2013), with pseudodynamic tests of a full scale prototype of a three-storey building (Figure 1d). Based on the results of these projects, a set of principles and rules has been incorporated into Eurocode 8 to ensure the reliability of the seismic design of precast structures.

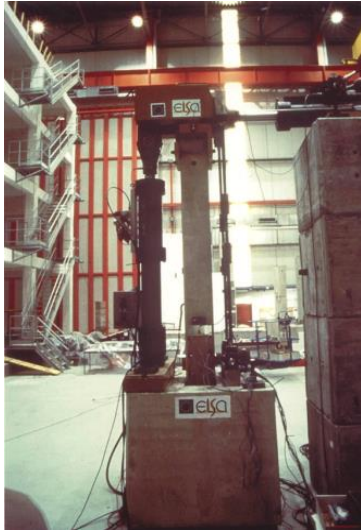
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(a)



(b)



(c)



(d)

Figure 1. Precast structure prototypes tested at ELSA Lab.: (a) Precast columns; (b) ECOLEADER project; (c) PRECAST STRUCTURES EC8 project; (d) SAFecast project

Recently, three strong earthquakes occurred in Europe in highly industrialised areas (L'Aquila, Italy, 2009; Lorca, Spain, 2011; Emilia, Italy, 2012). They represented a severe check for precast structures, as well as for any other type of structures. Mainly industrial single-storey buildings were involved. The experience of these earthquakes, further to confirm the validity of the code provisions for the design of the main precast frame structure, showed that there is still a pending problem to achieve a good seismic behaviour of the overall building system. This problem refers to the correct design of the connections of the wall panels to the structural frame. The collapse of this type of panels, with weight up to 10 tons, represents a mortal hazard for humans and may involve large direct and indirect economic losses for communities.

Figure 2a shows an emblematic picture of an industrial building after the 2009 L'Aquila earthquake: the main structure made of columns, beams, and roof elements is practically undamaged, while an entire façade of wall panels is collapsed down. A description of the effects of this earthquake on precast structures can be found in Menegotto (2009) and Toniolo and Colombo (2012). The 2011 Lorca earthquake also led to a relevant number of this type of failures (see Figure 2b). The 2012 Emilia earthquakes involved failures of several buildings for which the seismic design code was not in force at the construction time. Many falls of panels occurred also when the main structure did not collapse (see Figures 2c,d). A description of the effects of the 2012 Emilia earthquakes on precast structures can be found in Bournas et al. (2013) and Magliulo et al. (2014).



Figure 2. Failures of cladding panels through earthquakes: (a) L'Aquila 2009, (b) Lorca 2011, (c,d) Emilia 2012

Fastening systems of cladding wall panels of precast buildings have been widely investigated within the European research project SAFELCLADDING (EU Programme FP7-SME-2012, Grant Agreement n. 314122). The investigated fastening systems include innovative panel-to-panel dissipative devices (Biondini et al. 2013, Dal Lago et al. 2014, 2017a-c). The main results of this project are briefly recalled in this paper to show that the expected advantages of using panel-to-panel dissipative devices can be fully achieved provided that a stiff diaphragm is employed. The role of the diaphragm action is hence investigated by means of dynamic non-linear analyses performed on a typical structural arrangement of a precast industrial building equipped with mechanical connections. The effectiveness of an innovative solution concerning the use of roof-to-roof dissipative connections aimed at improving the diaphragm stiffness under controlled forces is finally discussed.

## 2. DISSIPATIVE PANEL CONNECTIONS: THE SAFELCLADDING PROJECT

Eleven partners, including universities, institutions and end users from different European countries, were involved in the SAFELCLADDING project (Colombo et al 2014). Different types of fastening devices and structural arrangements have been considered within this project and submitted to a campaign of experimental checks by means of a large number of tests performed at different levels. Local tests on single devices inserted between two supporting special frames, or between two parts representing the involved portions of the connected elements, have been performed for the definition of the intrinsic properties of the connector itself or of the whole connection. Tests on sub-assemblies made of groups of elements jointed by a number of connection devices representing the current module of the construction framework have been performed for the operational verification on a real structural assembly. Tests on full-scale prototypes of entire building structures have been performed, considering also the effects of construction tolerances and execution technologies, in order to investigate the seismic behaviour of real precast constructions.

Local and sub-assembly monotonic and cyclic tests have been performed at the laboratories for structural assessment of the University of Ljubljana (UL) (Zoubek et al. 2016a,b), Technical National University of Athens (NTUA) (Kaliviotis and Psycharis 2015), Politecnico di Milano (POLIMI) (Dal Lago et al. 2017a-d) and Istanbul Technical University (ITU) (Yuksel et al. 2018). Shaking table tests on structural sub-assemblies have been also performed at NTUA. Some pictures of this experimental activity are presented in Figure 3. A comprehensive set of design guidelines derived from the results of this experimental campaign can be found in EUR 27935 EN (2016).

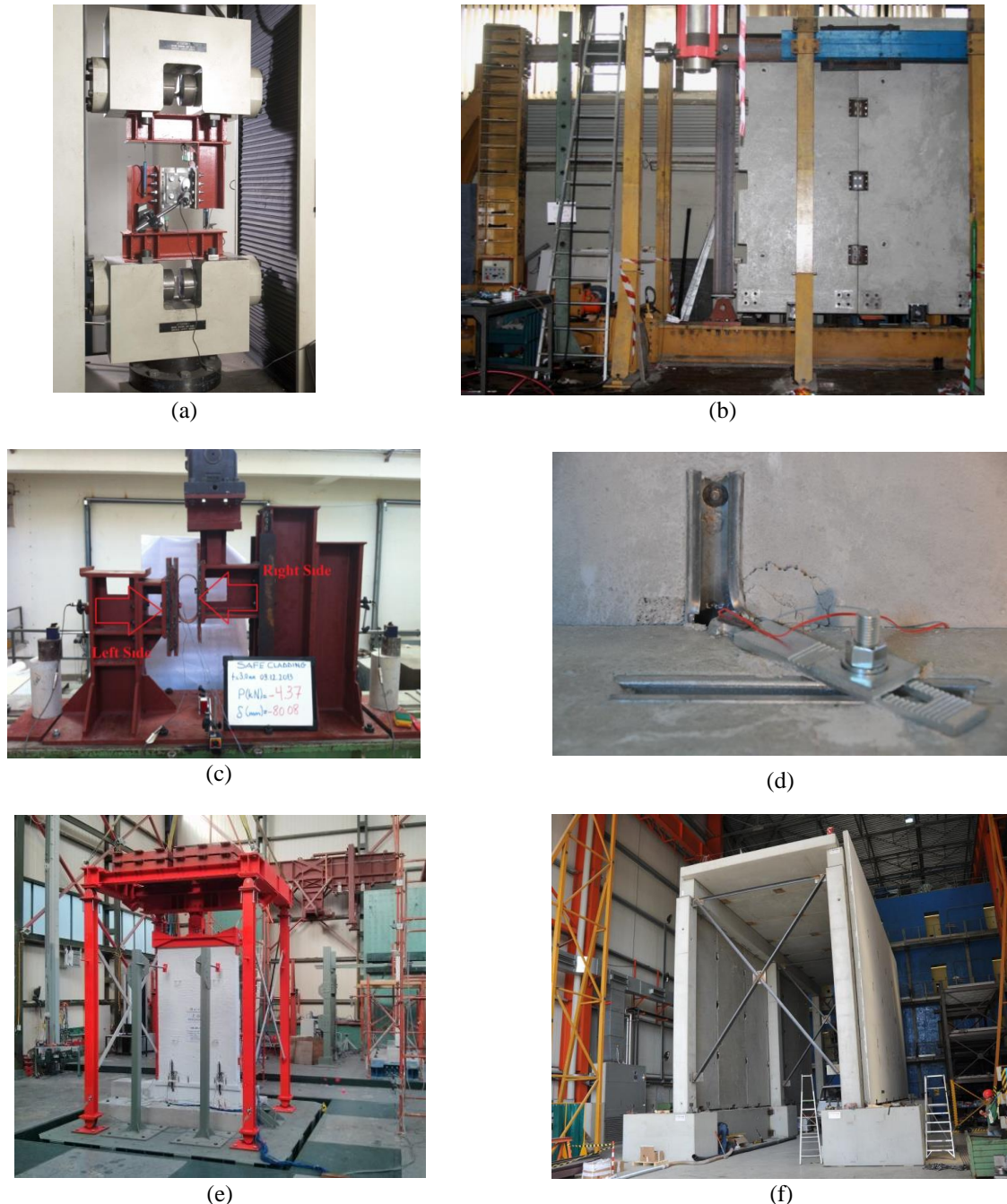


Figure 3. Experimental activity carried out within the SAFECLADDING project: (a) POLIMI Lab.: set-up for local cyclic tests on friction dissipative devices, (b) POLIMI Lab.: set-up for sub-assembly cyclic tests on panel-to-panel friction dissipative connection, (c) ITU Lab.: set-up for local cyclic tests on plastic dissipative devices (steel cushions), (d) UL Lab.: detail of a local cyclic test on a fastener of current production, (e) NTUA Lab.: set-up for shaking table tests on the base connections of cantilever panels, (f) ELSA Lab.: full-scale prototype of precast structure - configuration with vertical panels

An extensive series of cyclic and pseudodynamic tests has been performed on a full scale prototype of precast structure at the ELSA laboratory (Figure 3f). Three possible solutions of the problem have been proposed, consisting of isostatic, integrated or dissipative systems, to overcome the inadequacy of the present design criteria of panel-to-structure connections (Biondini et al. 2013). For each solution considered, the effectiveness of the connection devices under earthquake conditions has been tested, both for vertical and horizontal panels, at different levels of seismic intensity and with different arrangements of the fastening system (Dal Lago et al. 2017a-d, Negro and Lamperti Tornaghi 2017, Toniolo and Dal Lago 2017). A comprehensive set of design guidelines derived from this experimental campaign can be found in EUR 27934 EN (2016).

Based on the outcomes of the SAFECLADDING project, complemented with the research results obtained from other European projects developed over the last twenty years, new innovative solutions using dissipative wall connections are now available to preserve the integrity and ensure good seismic performance of precast structures. However, the positive features ensured by a dissipative system of connections of the cladding panels can be fully exploited provided that a stiff diaphragm is employed. The problem of the diaphragm behaviour of precast roofing systems has been addressed in literature, among others, by Fleischman et al. (2001) and Ferrara and Toniolo (2008) with reference to specific dry-assembled structural arrangements. In general, the horizontal actions stress the diaphragm based on the distribution of mass and stiffness of the lateral force resisting systems acting in parallel. When the diaphragm action cannot take place, for instance in case of single-ribbed floor members connected with typical hinged connections or double-ribbed elements similarly connected only at one rib, each frame of the overall structural assembly behaves substantially independently from the adjacent, with dynamic behaviour depending only on its own stiffness characteristics and pertinent mass.

A structural system with homogeneous distribution of stiffness and mass tends, therefore, to behave in a more uniform way, with moderate stresses into the diaphragm. Relevant differences of mass and/or stiffness modify the seismic behaviour of the building, largely stressing the diaphragm, if stiff, or creating possible out-of-phase responses with large roof distortions, if flexible. Such a behaviour is magnified if dissipative cladding panel connection arrangements are introduced in the structure, since in this case the peripheral frames are characterised by a much larger stiffness in comparison to the internal frames.

This issue is investigated in the following by means of dynamic non-linear analyses performed on a typical structural arrangement of a precast industrial building equipped with mechanical dissipative connections.

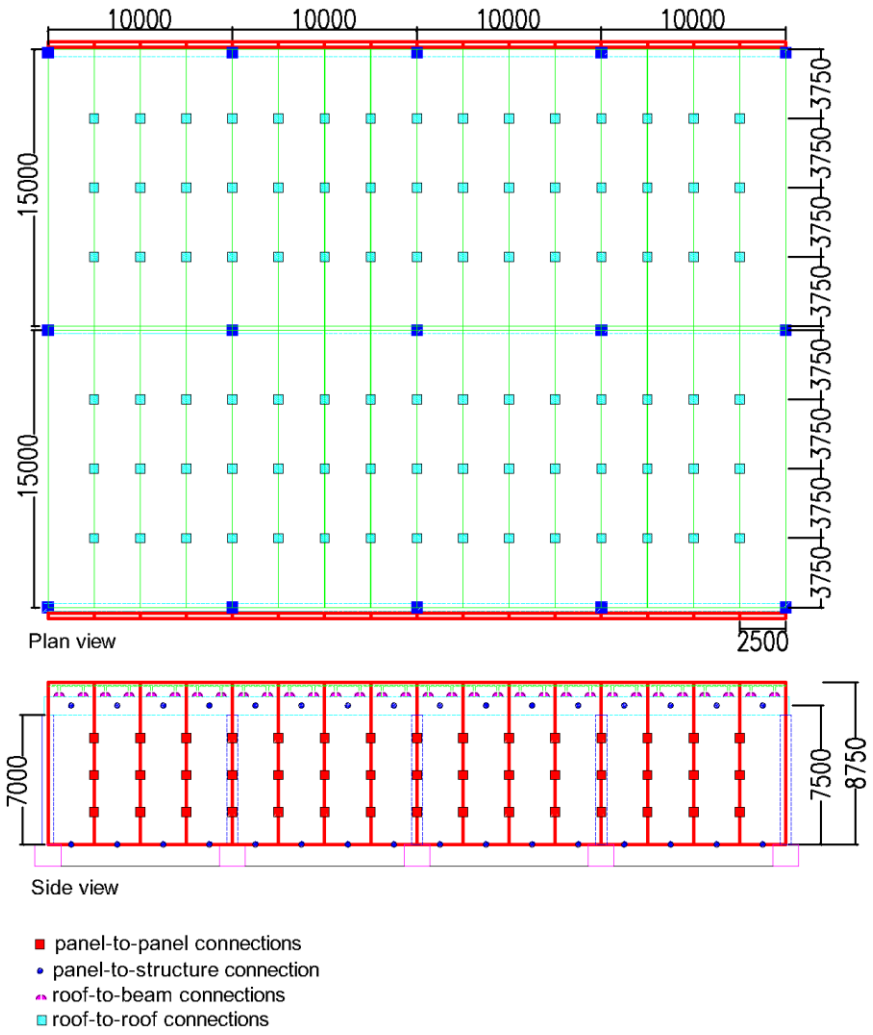
### 3. CASE STUDY

A single-storey precast structure with dimensions typical of industrial buildings in Southern Europe is considered as a case study. The precast frame has two bays and three frames with columns spaced by 15 m. The columns have height of 7 m and square cross-section 0,6×0,6 m. The beams have span of 10 m and rectangular cross-section 0,6×0,8 m. The roof elements have span of 15 m and TT cross-section with 2,5 m of width. A distributed mass of 320 kg/m<sup>2</sup> is considered for the structural and non-structural dead load. The lateral vertical cladding panels have overall dimensions 2,50×8,75 m with equivalent constant thickness of 0,12 m, leading to a spread mass of 300 kg/m<sup>2</sup>. The cladding panels are placed along the two sides of the building in the direction of the beams. All structural members are made with concrete C45/55 and reinforced with steel B450C.

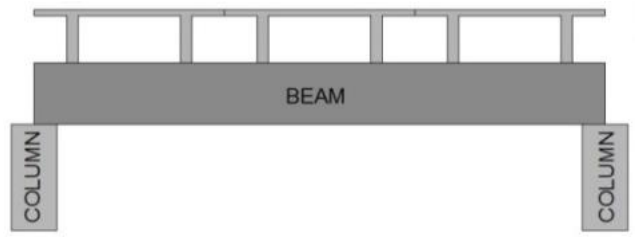
Figure 4a shows the geometry of the building and the scheme of the connections. Roof-to-roof connections are located at the ¼, ½ and ¾ of the elements. Panel-to-panel connections are placed at ¼, ½ and ¾ of the distance between bottom and top hinges of each panel. Figure 4b shows a schematic view of the deck cross-section.

Columns are reinforced considering a geometric reinforcement ratio  $\rho = 1\%$ , with 12  $\Phi 20$  steel bars placed as shown in Figure 5, with a concrete cover of 30 mm. The steel bars and the concrete core are considered effectively confined by closed stirrups.

The Sargin model is assumed for unconfined concrete (Sargin and Handa 1969). This model is modified with a constant peak stress branch to consider the effect of transversal reinforcement for the confined core of the columns. A linear-parabolic stress-strain curve is assumed for reinforcing steel.



(a)



(b)

Figure 4. Case study building: (a) plan and frontal views (measures in mm); (b) schematic roof section

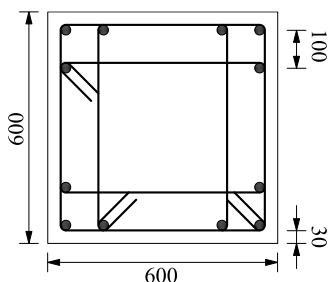


Figure 5. Column cross-section (measures in mm)

Columns are fixed at the foundation base. The beam-to column connection is realized with two dowels spaced in the direction orthogonal to the beam element. This is idealised with a hinge in the main bending plane of the beam and with a fixed joint in the other planes. The physical clearance between the top of the column and the centroid of the beam is taken into account by using coupling links.

Columns and beams are modelled by using beam elements. Roof elements and cladding panels are modelled with shell elements. The non-linear behaviour of columns and selected connections is considered into the structural model.

The non-linear behaviour of columns is based on a smeared plasticity model associated with the cross-sectional bending moment-curvature relationships under axial load for the seismic load combination. The hysteretic behaviour is based on the Takeda model (Takeda et al. 1970). Roof-to-beam connections are realized with dowels and angles and modelled with spring elements. Large stiffness is assumed in both vertical direction and horizontal direction normal to the beam axis. Non-linear behaviour is considered in the direction parallel to the beam, with load-displacement model calibrated on the basis of test results (Psycharis and Mouzakis 2012, Dal Lago et al. 2017e). Similarly, panel-to-panel and roof-to-roof connections are modelled by axial springs with non-linear behaviour calibrated on the basis of test results (Dal Lago et al. 2017b-c). An elastic-plastic model is used for Friction Based Devices (FBDs, Figure 6; Dal Lago et al. 2017c) and dowels. An elastic-hardening model is used for Multiple Slit Devices (MSDs, Figure 7; Dal Lago et al. 2017b) and angles. A kinematic hardening is considered for the hysteretic behaviour of FBDs and MSDs. A Takeda hysteretic behaviour is assumed for dowels and angles (Takeda et al. 1970). Figure 8 shows the monotonic load-displacement models of the connections.

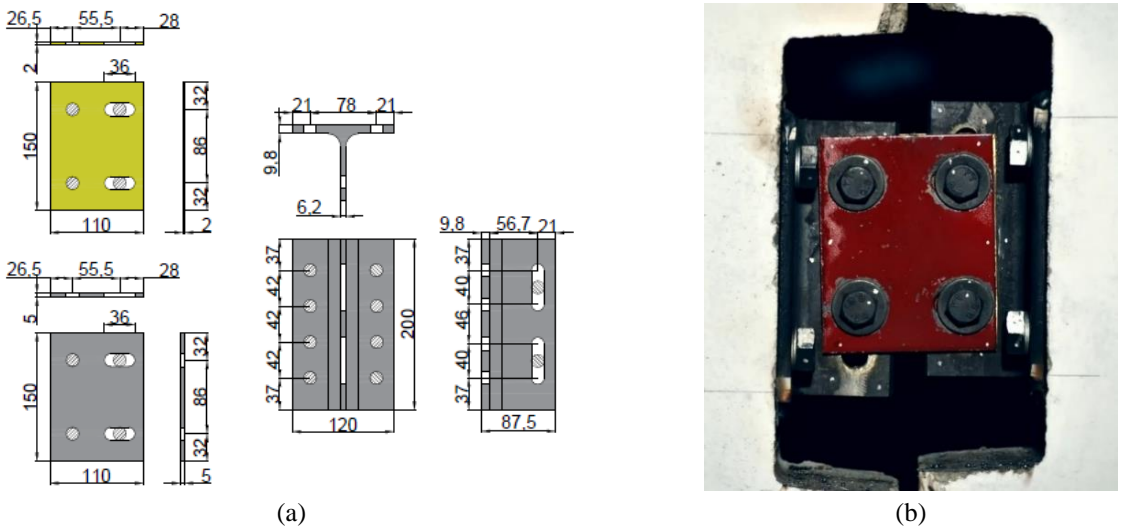


Figure 6. Friction Based Device (FBD): (a) components; (b) device in operation

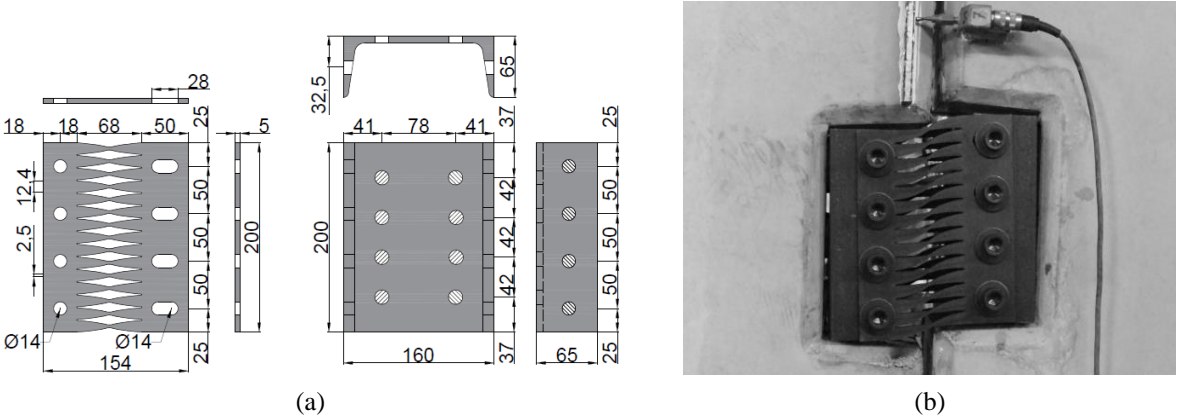


Figure 7. Multiple Slit Device (MSD): (a) components; (b) device in operation

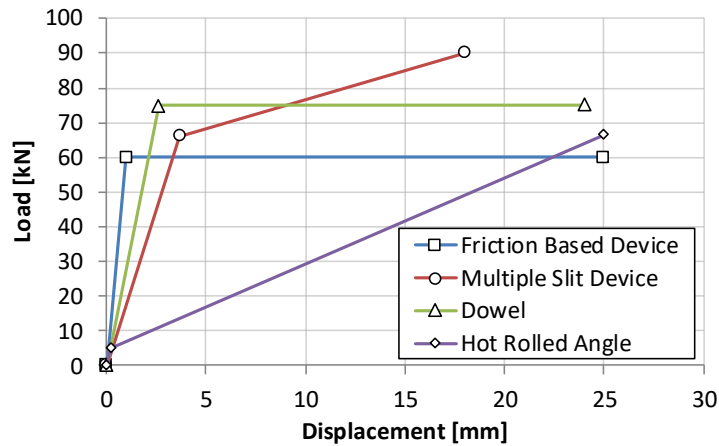


Figure 8. Non-linear load-displacement diagrams of connections

The distance between the centroid of the beam and the corner node of the rib plate element is covered by a translational coupling link. The cladding panels are connected to the foundation and to the peripheral beams at the top with central hinged connections. Thus, the central nodes at the base of the plate elements of the panels are restrained, while the top connection is modelled with a connection element with high stiffness which rigidly link all the relative displacements between the beam and the panel node. The mass is spread through the density associated to all structural elements ( $2500 \text{ kg/m}^3$ ). The additional mass associated to waterproofing and finishes on top of the roof elements is negligible.

#### 4. NON-LINEAR DYNAMIC ANALYSIS

The analyses have been performed under the accelerogram shown in Figure 9. The original signal, recorded in Tolmezzo, Italy, during the 1976 earthquake, is artificially modified to make it compatible with the elastic response spectrum of Eurocode 8 for soil type B (EN 1998-1:2004) and scaled to a peak ground acceleration  $\text{PGA} = 0,32\text{g}$ .

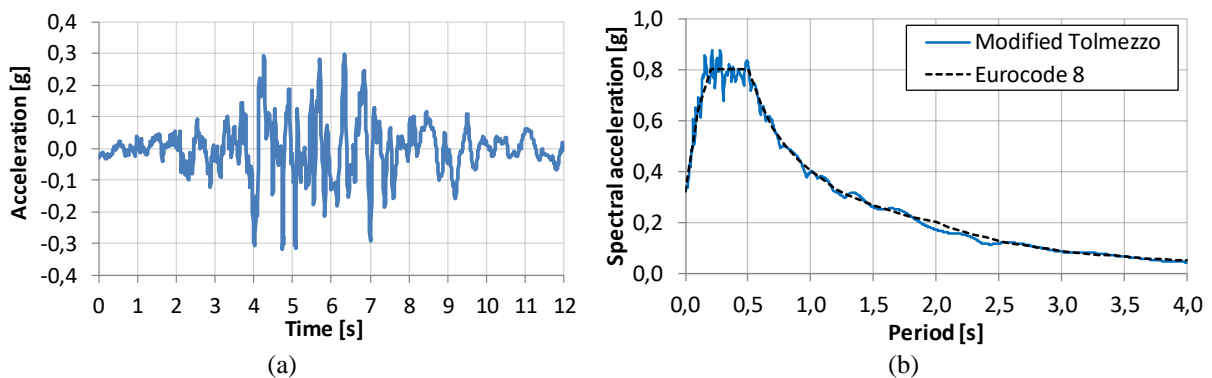


Figure 9. Modified Tolmezzo accelerogram: (a) Accelerogram ( $\text{PGA} = 0,32\text{g}$ ); (b) Elastic response spectrum compared with the spectrum of Eurocode 8 for soil type B

The results for the structure without mutual connections in between panels and floor elements are shown in Figure 10 in terms of top displacement time-history. Since the distribution of mass and stiffness among inner and outer frames is quite uniform, the response of these frames is similar even if the roof elements are connected to the beams with soft angle connections, with small differences not involving significant out-of-phase vibrations (Figure 10a). On the contrary, the use of dowels to connect roof and beam elements leads to a coupled response typical of structures with rigid diaphragm (Figure 10b). In both cases, the response is characterised by remarkable flexibility, with maximum displacements attained of about 200 mm and yielding of all columns at the base.

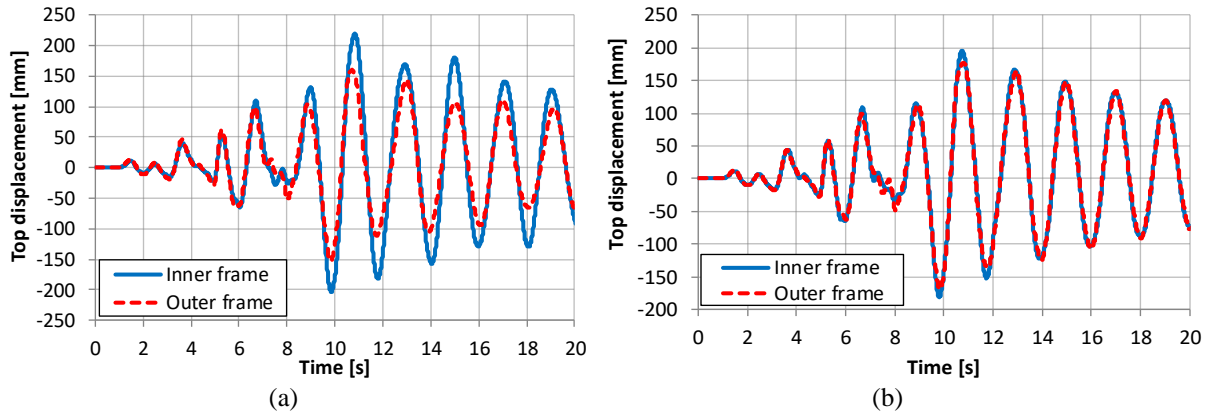


Figure 10. Top displacement time-history of the building with no panel-to-panel and no roof-to-roof connections: building with (a) angle roof-to-beam connections, and (b) dowel roof-to-beam connections

The seismic response of the structure with panel-to-panel FBD connections dramatically changes for the outer frames. The consequent response modification of the inner frames relies on the diaphragm effectiveness. Figure 11 shows the results of the analyses considering angles and dowels as roof-to-beam connections. With angles (Figure 11a), the inner frame tends to vibrate independently from the outer frames, displacing up to about 200 mm and bringing its columns to yield at the base. With dowels (Figure 11b), the collaboration between the frames improves and the response of the inner frame is remarkably modified with respect to the case without FBDs. However, this effect is not sufficient to prevent the columns of the inner frames from yielding.

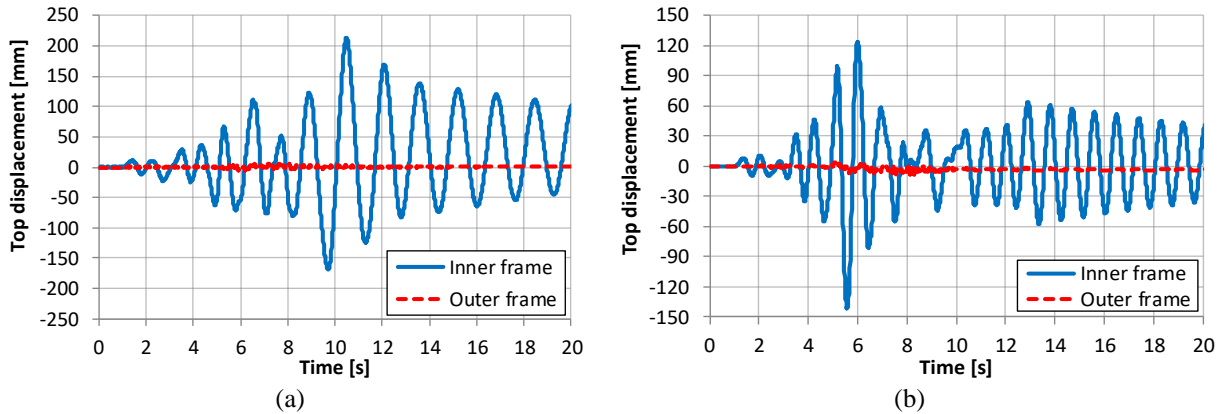


Figure 11. Top displacement time-history for the building with FBD panel-to-panel and no roof-to-roof connections: building with (a) angle roof-to-beam connections, and (b) dowel roof-to-beam connections

Finally, Figures 12-14 show the results of the analyses where both panel and roof mutual connections are activated. The top displacement time histories shown in Figure 12, corresponding to the use of angle (Figure 12a) and dowel (Figure 12b) beam-to-floor connections, indicate that the collaboration among frames is stronger with respect to the case without floor mutual connections. The maximum displacement of the inner frame is reduced to about 80 mm and 40 mm, respectively. Furthermore, the outer frames are subjected to a more relevant motion compared with the noise elastic vibration achieved for the case without floor mutual connections.

Figures 13 and 14 show that the seismic response with angle or dowel beam-to-roof connections is very different. When angles are used, the diaphragm effect is weaker, leading to a moderate slippage of the FBDs (Figure 13a) and a strong deformation of the MSDs (Figure 14a) with relevant energy dissipation from the latter. When dowels are used, FBDs slip more (Figure 13b), dissipating a large amount of the seismic energy, while MSDs undergoes moderate plasticisation (Figure 14b). This type of behaviour is preferable, since the displacement capacity of FBDs can be remarkably larger with respect to MSDs, leading to a more effective seismic response. In addition, FBDs do not damage

under operation (Dal Lago et al. 2017c), while strongly deformed MSDs would need to be replaced after earthquakes, leading to a cost. It is worth noting that in both cases permanent residual deformations occur due to the non-re-centring capacity of the investigated connections. However, these residual deformations can be considered fully acceptable since the vertical residual displacement can be zeroed by subsequently untightening and re-tightening the FBDs, thanks to the elastic recovery ensured by the frame structure. The use of combined panel and roof dissipative connections allowed a remarkable reduction of structural drift and damage under forces controlled by both the friction threshold of FBDs and yield threshold of MSDs. The alternative use of over-resisting connections would lead to significantly higher stresses without the beneficial activation of hysteretic damping.

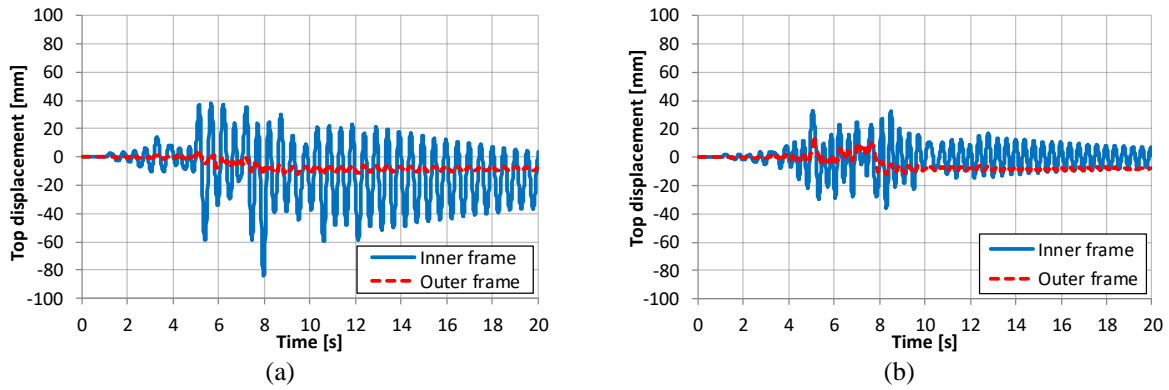


Figure 12. Top displacement time-history for the building with FBD panel-to-panel and MSD roof-to-roof connections: building with (a) angle roof-to-beam connections, and (b) dowel roof-to-beam connections

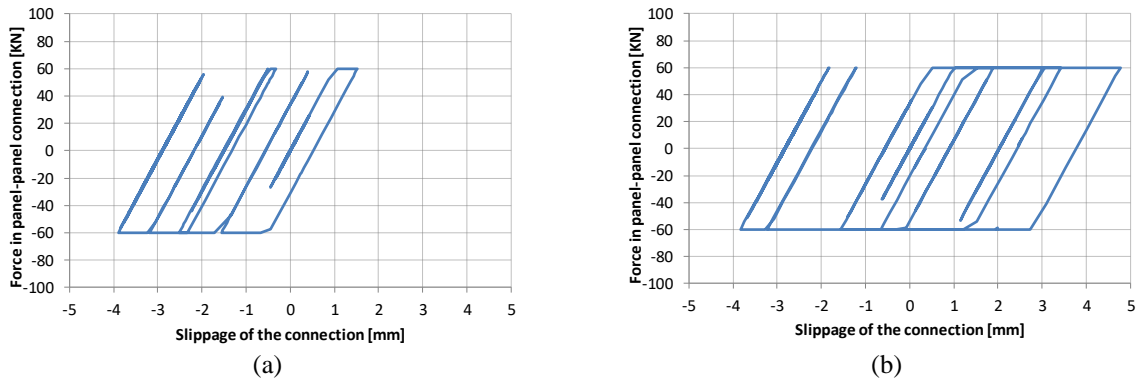


Figure 13. Panel-to-panel connection hysteretic diagrams of the building with FBD panel-to-panel and MSD roof-to-roof connections: building with (a) angle roof-to-beam connections, and (b) dowel roof-to-beam connections

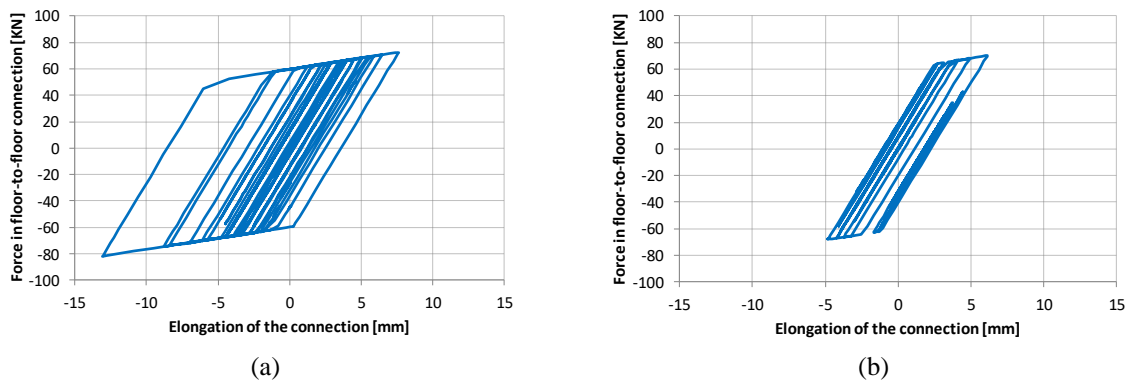


Figure 14. Floor-to-floor connection hysteretic diagrams of the building with FBD panel-to-panel and MSD roof-to-roof connections: building with (a) angle roof-to-beam connections, and (b) dowel roof-to-beam connections

## 5. CONCLUSIONS

The results of the dynamic non-linear dynamic analyses carried out on a typical precast concrete structural arrangement pointed out that the use of dissipative cladding connections aimed at reducing the overall structural drift and consequent damage can be very effective, given a relevant diaphragm stiffness is ensured. For roof decks made of slab elements not mutually connected, the use of a relatively soft roof-to-beam connection at the ends of both ribs of TT elements, like angles, provides a negligible stiffening of the diaphragm. The use of a stiffer roof-to-beam connection, like dowels, relevantly modifies the seismic behaviour of the structure, although the reduction of displacement of the inner frame is limited to about 30%, which is a low percentage compared to the potential beneficial effects of the cladding dissipative system with FBDs. For roof decks with adjacent roof elements the introduction of dissipative roof-to-roof MSD connections provides, on the contrary, a more relevant reduction of the seismic drift, around 60% when angles are employed at the roof member ends, and 80% when dowels are employed. If softer floor-to-beam connections are used, the MSDs tend to be more stressed and the FBDs are subjected to a lower slippage and, therefore, are less effectively exploited. The displacements attained in angles or dowels are by far below the ultimate limits. In particular, dowels remained in the elastic range. The remarkable drift reduction provided by the combination of dissipative cladding and roof connections is also accompanied by a total protection of the structure from damage. Therefore, the structure can be designed to be fully operational even after the occurrence of earthquakes with intensity up to the design seismic capacity at collapse limit state.

## ACKNOWLEDGMENTS

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