Seismic behaviour of precast buildings with cladding panels

F. Biondini, B. Dal Lago & G. Toniolo
Department of Structural Engineering, Politecnico di Milano, Milan, Italy

SUMMARY:
Past seismic events, including L'Aquila earthquake in 2009, have clearly demonstrated that the current connection techniques of wall cladding panels to the typical frames of precast buildings are far from excluding them to interfere with the overall structural seismic behaviour, while the current approach is to design these connections basing just on local calculations. Possible solutions to avoid panel collapses due to the failure of their connections include: (a) The adoption of a statically determined support system that makes the panels independent from the motion of the structure; (b) The adoption of an integrated support system adequately proportioned that makes the panels integral part of the resisting structure. This study starts from a particular arrangement of the statically determined solution (a), with a pendulum supporting system of the panels, consisting of two hinges applied one at the base and one at the top, and shows the effectiveness of mutual dissipative connections added between the panels so to integrate them into the resisting structural system as in the integrated solution (b). The results of non-linear static and dynamic analyses under recorded and artificial accelerograms show the effectiveness of the dual frame/wall system with dissipative connections between panels. Two case studies illustrate the features of the configurations with vertical and horizontal panels.

Keywords: Precast concrete structures, seismic performance, mechanical connections, cladding panels

1. INTRODUCTION

The present seismic design approach for the connections of large concrete cladding panels in precast buildings regards their out-of-plane behaviour, neglecting their possible interaction with the whole structure within their in-plane behaviour. Many earthquakes provided the evidence that, if those panels are not provided with a connection set that makes them totally independent from the motion of the frame, dramatic collapses of those heavy panels can occur. Surveys on the effects of the L'Aquila earthquake are reported in Menegotto (2010) and Colombo and Toniolo (2010). The typical precast frames in Southern Europe consist of very flexible cantilever type columns with hinged beams, which energy dissipation capacity can be large, but achievable at relatively high drifts (Biondini and Toniolo 2009). Thus, more reliable design criteria have to be introduced to ensure that the cladding system is statically determined even at high drifts or to correctly design the inserts, considering the real in-plane forces arisen in the event of an earthquake (Colombo and Toniolo 2010).
This study starts from a particular arrangement of the statically determined solution, with a pendulum supporting system of the panels, consisting of two hinges applied one at the base and one at the top, and shows the effectiveness of mutual connections added between the panels so to integrate them into the resisting structural system. It is shown how the structural behaviour is modified moving from the statically determined to the integrated solution and how the design criteria have to be adapted (Biondini *et al.* 2010 and 2011). The use of dissipative connections is also considered to provide an effective and economical solution for the frame-panel integrated structural system. Among the different typologies of dissipative connections (Symans *et al.* 2008), a friction system is used in the case studies, which considerations can be extended to displacement-dependent connections in general.

2. CASE STUDY 1: VERTICAL PANELS

The investigated structural prototype is a one-storey precast frame building of 40,5x25,0 m of dimensions in plan, made of two lines of five columns, 7,0 m high, spaced by 10,0 m. The roof is made of five transversal shed beams supporting ribbed elements. Under seismic condition the dead loads of the roof are:

- roof elements with finishings	2,8 kN/m^2
- shed beams (average weight)	17,5 kN/m
- longitudinal beams	3,2 kN/m

that lead to a vertical action of 600 kN on the 3+3 middle columns and 410 kN on the 2+2 end columns. All the columns have a square cross-section with side width of 60 cm and reinforced with 12ϕ20mm longitudinal bars corresponding to the minimum reinforcement ratio of 1%. The steel Class is B450C. The Concrete Class is C45-55, with an elastic modulus assumed equal to 30 GPa for elastic analysis.
Along the two major sides of the building 16+16 vertical wall panels are placed, with dimensions 2,5x8,0 m and an equivalent concrete thickness of 12 cm, leading to a weight of 3,0 kN/m^2. They are placed at the base on the foundation beam with a central hinged support that allows their free rocking within the limits of the joint allowance left between the beam and the panel. At the height of 7,08 m the panels are connected to the supporting beam with another central hinge. In this configuration the structural assembly represents a statically determined solution. In the integrated solution the panels are jointed between them by means of three connections placed in the adjacent sides at 1/4, 2/4 and 3/4 of the support height. Figure 1 shows the frame system with the cladding wall panels with the indication of the supports.

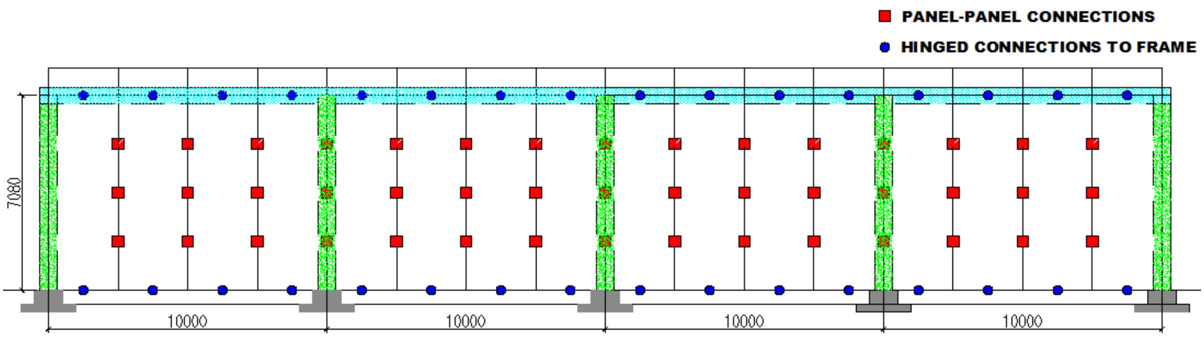


Figure 1. View of the cladding wall system with vertical panels.

3. LINEAR ELASTIC ANALYSIS

The analysis of the structural system shown in Figure 1 is carried out based on a finite element model with beam elements for columns and shell elements for panels.
A first series of linear elastic static analyses are performed by applying at the top an horizontal force of 1000 kN that corresponds to a ratio of about 0,30 of the competent roof and wall weights computed as follows:

$$
\begin{aligned}
&2{,}8\text{x}40{,}0\text{x}25{,}0 &&= 2800\\
&5\text{x}17{,}5\text{x}25{,}0 &&= 2187\\
&3{,}2\text{x}2\text{x}40{,}0 &&= 256\\
&3{,}0\text{x}120\text{x}8{,}0/2 &&= 1440\\
&\qquad W &&= 6683 \text{ kN} \ / \ 2 = 3341 \text{ kN}
\end{aligned}
$$

with the cladding walls extended to all the perimeter less two openings of 5,0 m each.
Figure 2 shows the displacement profile of the structure with statically determined walls.

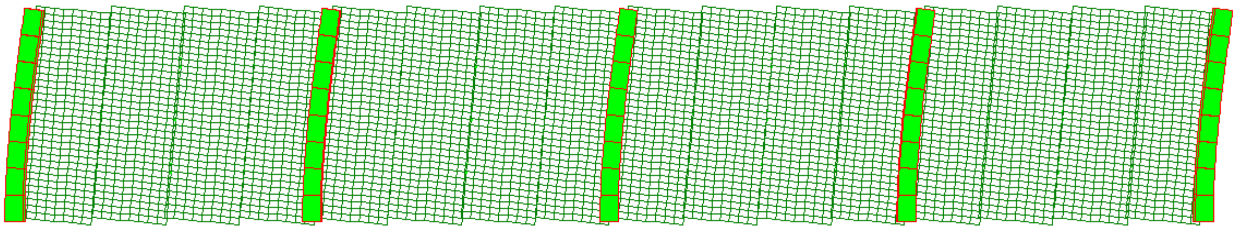


Figure 2. Disconnected vertical panels: deformation of the structure (d_{top}=73,0 mm).

With reference to the degree of reciprocal connection between the panels, the following two limit cases are studied: absence of connections (stiffness k=0), with structural response given only by the columns; perfectly rigid connections (stiffness k=∞), with structural response given almost only by the panel walls.

For the case with perfectly rigid connections, the analysis leads to a top displacement reduced to 0,3 mm due to the much higher stiffness of the collaborating wall. They take almost all the applied action, leaving to the columns a very small shear of 0,5 kN. Figure 3 shows the distribution of the forces on the fastenings of the panels to the structure. An almost uniform distribution of the horizontal forces on the 16 fastenings, with values up to around 70 kN, displays for the global equilibrium of the force of 1000 kN applied at the top. At the ends of the wall there are strong vertical components on the last fastenings for the global equilibrium of the overturning moment of the top force with respect to the base. In these fastening the forces rise up to around 77 kN.

Figure 4 shows the distribution of the actions transmitted among the panels through their mutual connections. They are vertical reaction forces against the reciprocal sliding of the panels. The values are almost constant over all the length of the wall: 70 kN on the upper and lower connections and 55 kN on the middle connections. It can be noted that for the end panels the resultant of the forces of the three connections with the adjacent panel, since the opposite panel is missing, is equilibrated by the vertical resultant of the reactions of the two upper and lower fastenings to the structure.

The obtained results provide an indication of the level of the forces in the connections for a seismic action applied to a very rigid structural arrangement of squat walls at the ultimate limit state of no-collapse.

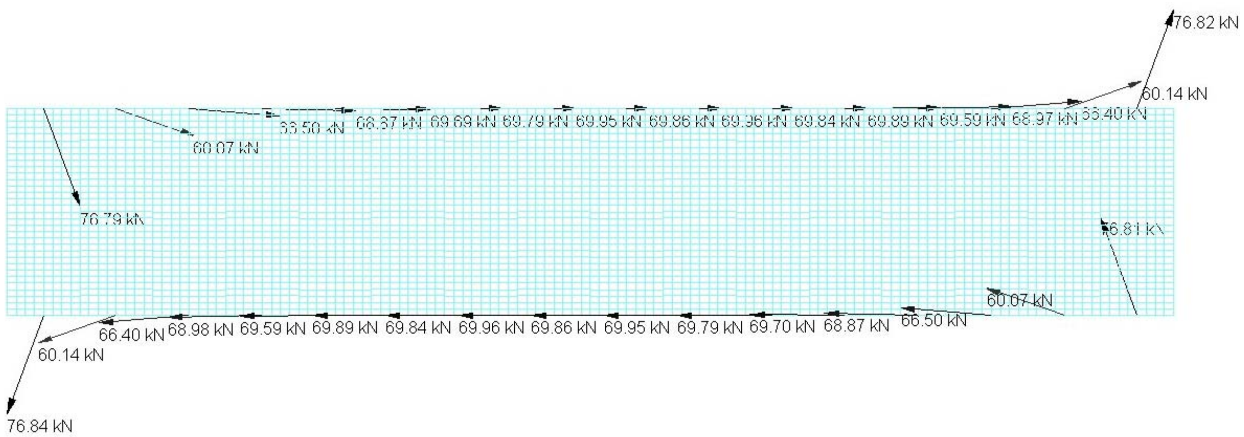


Figure 3. Rigid connections among vertical panels: reaction forces on the fastenings (kN).

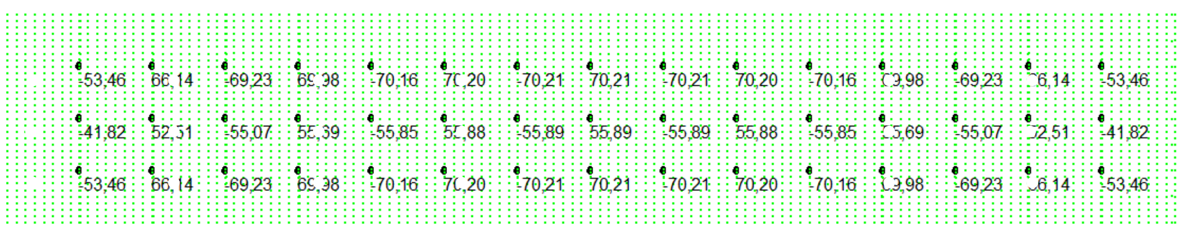


Figure 4. Rigid connections among vertical panels: reaction forces on the connections (kN).

Following Eurocode 8 rules (EN 1998-1:2004), for medium ductility class a behaviour factor $q=q_w q_o=0,5x3,0=1,5$ can be assumed for the squat wall system with rigid connections and, with reference to the maximum response amplification for a subsoil of category B, a storey force $F_h=\alpha_g S_{max} W/q$ with $S_{max}=1,2x2,5=3,0$ can be computed. Therefore, with $F_h=1000$ kN, a seismic capacity α_g = 1,5x1000/(3,0x3341) ≈ 0,15 is obtained, corresponding to a medium-low seismicity zone in the Italian territory.
The forces obtained under this moderate seismic intensity show very large values that put difficult design problems for the connections, problems that become even more critical for zones with higher seismic intensity. This emphasizes the importance of solutions able to attenuate the seismic response of the integrated wall-frame arrangement of the structure, possibly with dissipative effects of the connections.

4. NON LINEAR ANALYSIS

Several non-linear dynamic analyses of the prototype have been performed, in particular with reference to a specific type of connections.
Shultz et al. (1994) reported the characteristics of a set of devices designed in USA to make mutual connections between wall panels, devices with dissipative capacities based on different mechanisms of yielding or friction. The present study refers to a dissipative connection device tested at the Laboratory for material testing of Politecnico di Milano for SPAV Company, Martignacco, Italy (Ferrara et al. 2010).
The “SPAV” connection is made of two steel T shaped parts, obtained from the cut of an IPE profile, that are fixed to the adjacent panels in special cavities and jointed with two lateral bolted steel plates, as shown in Figure 5.
The length of the slotted holes gives the limit to the reciprocal slide between the parts. The tightening torque given to the bolts, controlled by dynamometric wrench, activates the friction between the plates determining the slip threshold.
Two brass sheets are interposed between the steel plates and the profiles to ensure the stability of the repeated slide cycles.

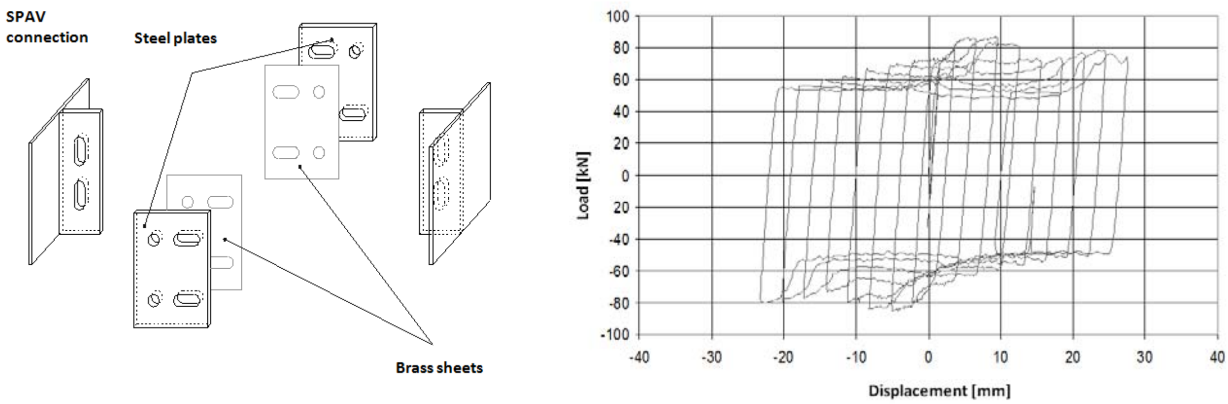


Figure 5. Components of SPAV connection and experimental load-displacement cycles.

In the design model the constitutive law is assumed as perfectly elastic-plastic with parameters directly evaluated from the experimental curves. The ultimate “failure” deformation of the connection has been assumed as that corresponding to the end stroke of the slotted holes, that is ±40 mm for a perfect initial centring of the bolts.
The corresponding top displacement of the structure is d=±40x7,08/2,5=±113 mm.
The columns have been modelled with non linear moment-curvature relationships.
Figure 6 shows a pushover curve of the structure with SPAV connections computed with the elastic-plastic model based on an initial stiffness of 60 kN/mm, a slip threshold of 60 kN and a “functional” stroke limit of ±25 mm.

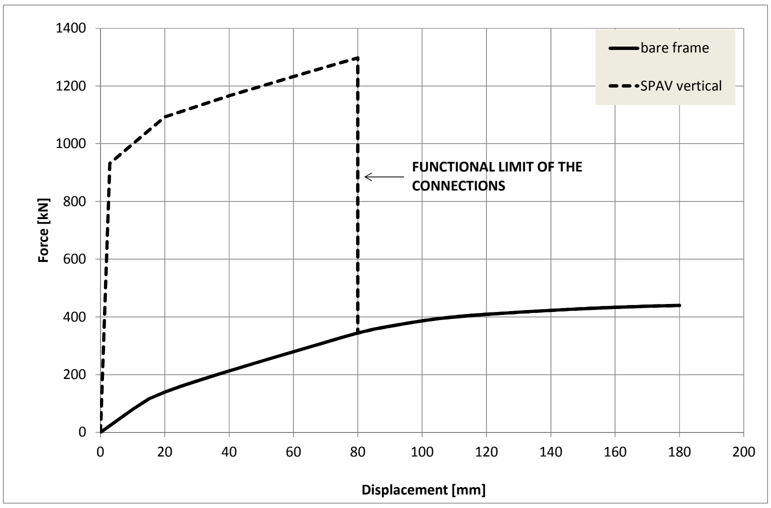
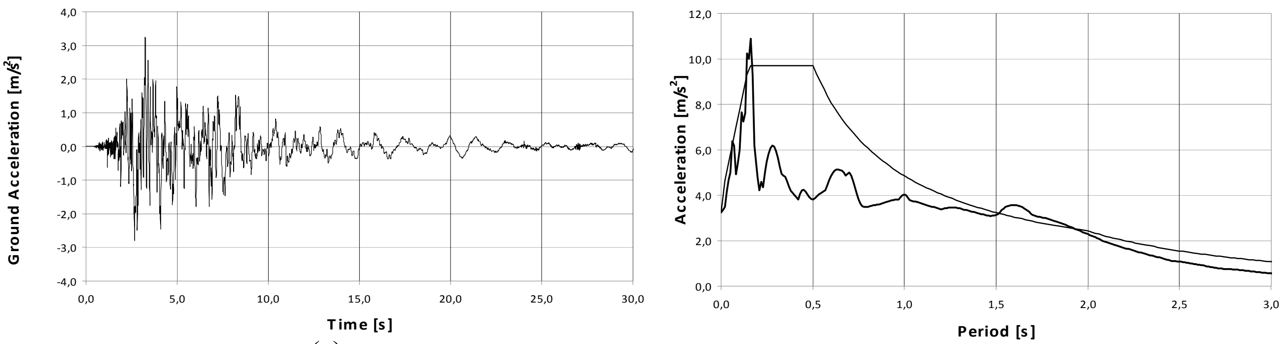


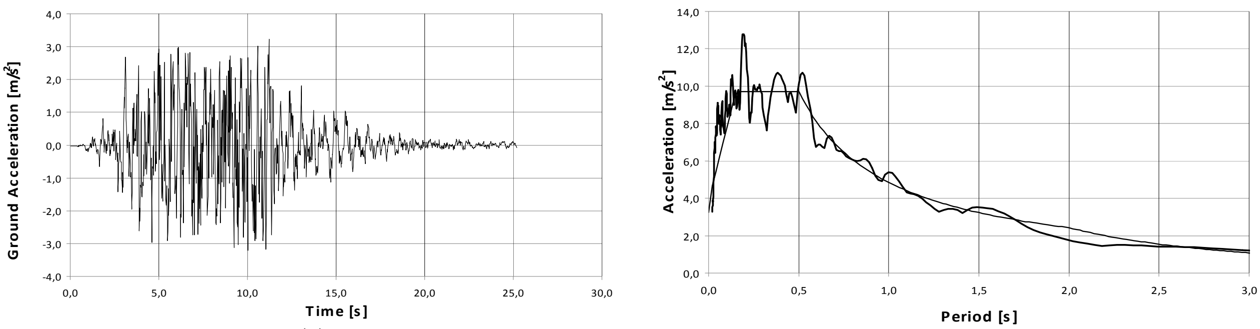
Figure 6. Pushover curve with SPAV connections for the structural system with vertical panels.

The dynamic non linear analysis is performed with a Takeda model (Takeda et al. 1970) for the columns, the elastic-plastic model described above for SPAV connections, and a linear elastic model for the panels. The analyses are repeated with the recorded L'Aquila 2009 accelerogram (AQK-WE), and one artificial accelerogram (SC) compatible with the response spectrum given by Eurocode 8 for subsoil B and scaled to the peak ground acceleration of the AQK-WE earthquake (PGA≈0,32g). Figures 7 and 8 show these accelerograms together with the corresponding elastic response spectra compared to Eurocode 8 spectrum. It is worth noting that the scanty compatibility of L'Aquila response spectrum with the Eurocode 8 standard spectrum, due to its poor content of frequencies, could lead to a weaker impact on the actual structural response.



(a) (b)

Figure 7. L'Aquila earthquake (AQK-WE): (a) accelerogram, and (b) response spectrum compared with the model of EC 8.



(a) (b)

Figure 8. Artificial earthquake (SC): (a) accelerogram, and (b) response spectrum compared with the model of EC 8.

In order to study the different response of the 3 different structural configurations (disconnected, integrated and dissipative), L'Aquila AQK-WE and the artificial accelerograms have been applied to each of them. The computed vibratory curves are shown in Figure 9. The diagram at the top gives the displacement time history response for a zero connection stiffness (pure frame structure), with large storey drifts; the diagram at the bottom shows the structural response with rigid connections (wall structure), with small storey drifts and high connection forces; the diagram in the middle shows the structural response with dissipative SPAV connections (dissipative structure), with intermediate storey drifts and limited connection forces.
The strong effectiveness of the connections can obviously be noticed in lowering the maximum storey drifts from 80 mm to 4 mm and 0,6 mm under L'Aquila earthquake, and from 80 mm to 6 mm and 0,8 mm under the artificial accelerogram, for SPAV connections and rigid connections, respectively. The large increment of the vibration frequencies of the structural responses of the integrated systems can also be noted. In particular, the response with rigid connections corresponds to a pure elastic behaviour of a stiff wall system. The residual displacement of the integrated structure with dissipative connections indicate that an inelastic slide occurred in the connections and, therefore, the friction mechanism worked in dissipating energy. The energy dissipation is mainly provided by the friction behaviour of the devices, while the columns remain in almost elastic range.
It is finally noted that, with reference to the maximum values of the elastic responses with rigid connections a force reduction factor by 2,28 is deduced for the friction connections herein studied in the case of AQK-WE accelerogram.

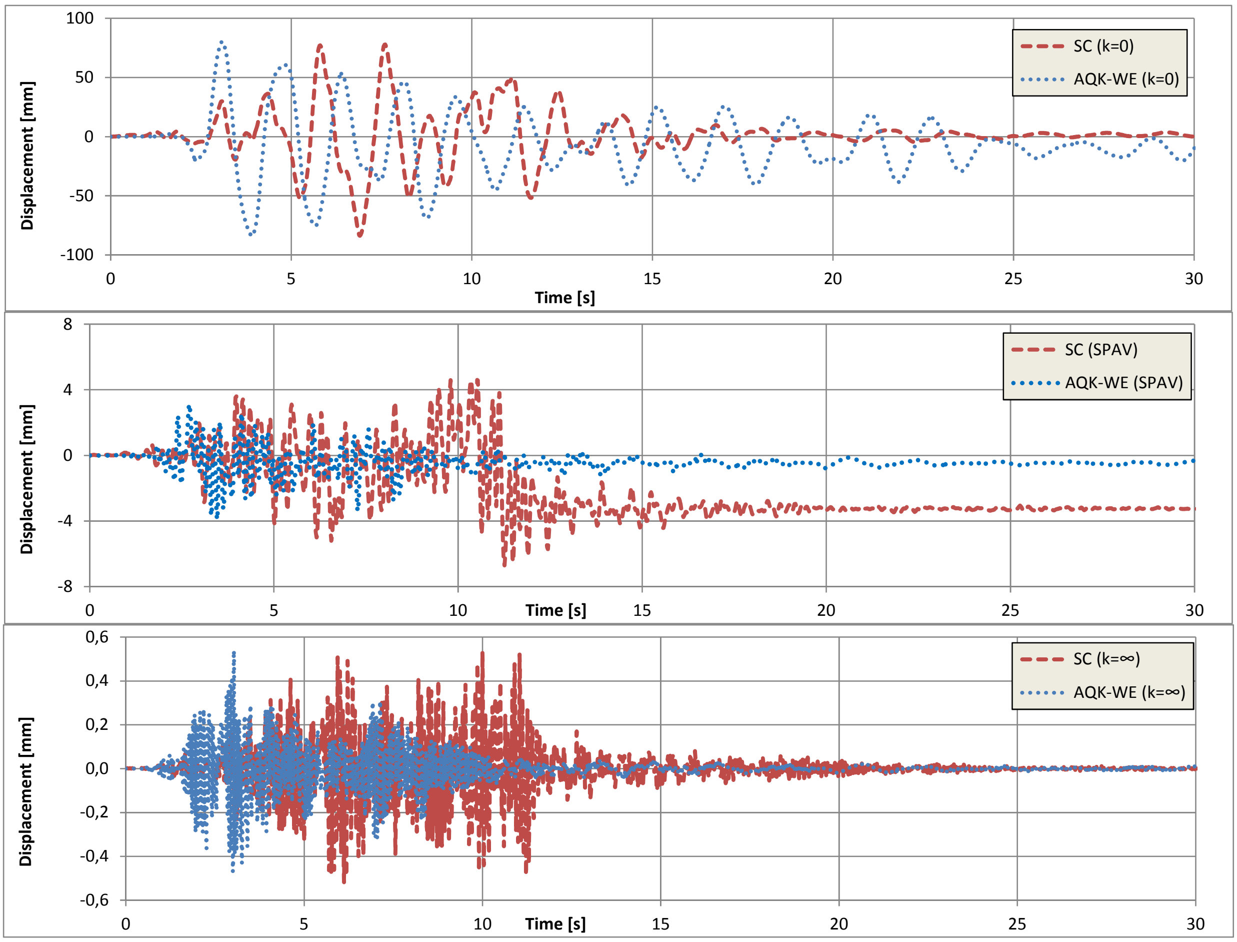


Figure 9. Displacement time histories under L'Aquila earthquake (AQK-WE) and the artificial accelerogram (SC).

5. CASE STUDY 2: HORIZONTAL PANELS

The seismic response of the structural system previously described is now investigated by considering the cladding walls made by horizontal panels. It is assumed that the panels are linked to the frame with statically determined (double pin) connections at mid height. Two lines of panel-to-panel connections are inserted at 2,35 and 4,70 m of height, in between adjacent panels, with a number of three connections for each panel contact line. By neglecting friction effects in between the contact lines, each panel is considered statically determined. The front view of the building is shown in Figure 10.

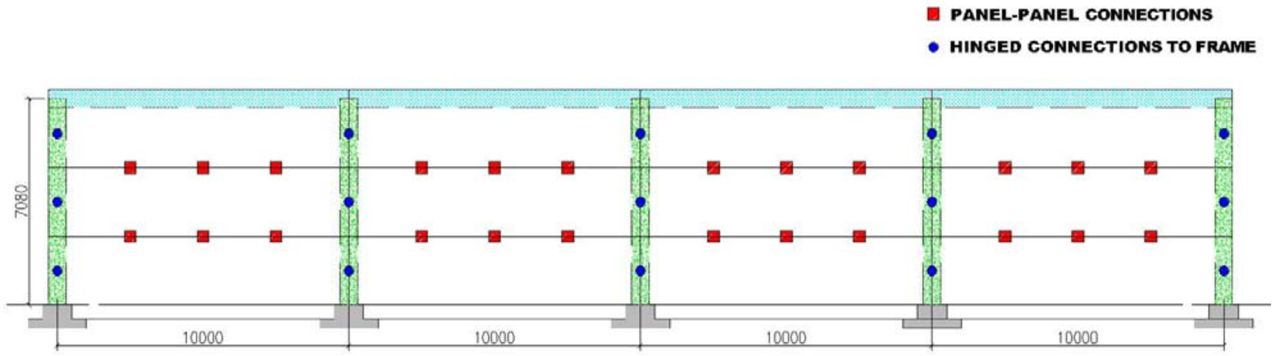


Figure 10. View of the cladding wall system with horizontal panels.

The structural response of the system is depending on the stiffness of the connections. For negligible stiffness, when a static horizontal load of 1000 kN applied at the top of the building, the deformed configuration is shown in Figure 11, where the panels are free to slide one over the other. The entity of the relative slide increases on the height of their horizontal contact surface, since they follow the deformation of the cantilever columns. For rigid connections, the panels act as a single shear wall, thus significantly reducing the top displacement (equal to that of the vertical panels case) but developing very high forces, up to 157 kN (Figure 12). The forces acting on the connections are different depending on the position of the devices even on the same line for such high stiffness. It can also be noticed how the upper line is subjected to higher forces. This is due to the different relative slide of the two lines. The panels act as horizontal flexural bracers, and their equilibrium brings to strong axial loads on the columns through the pinned support, which balance leads to the counter-acting bending moment. The columns are also strongly subjected to shear in the lower part, since all the base shear is carried by them.

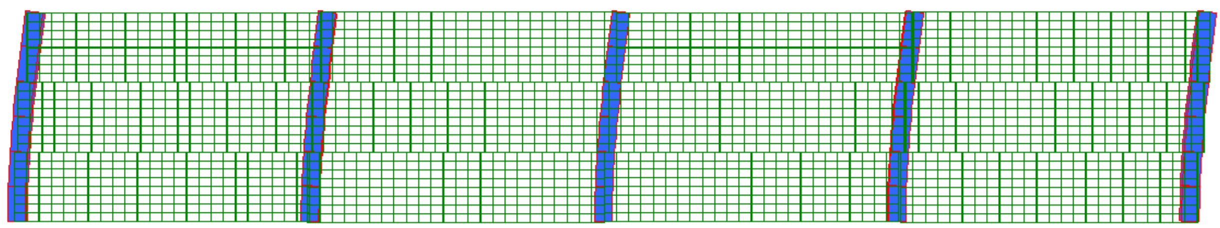


Figure 11. Disconnected horizontal panels: deformation of the structure (d_{top}=73,0 mm).

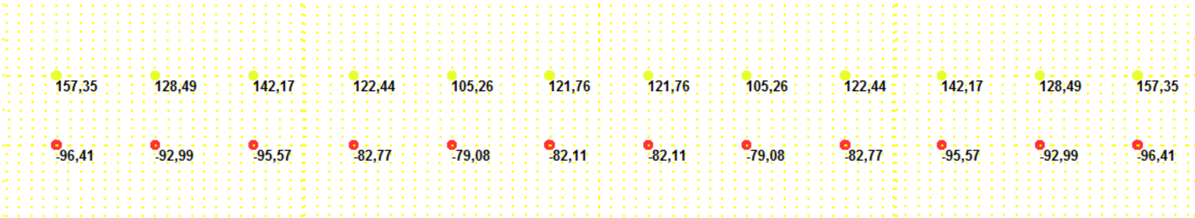


Figure 12. Rigid connections among horizontal panels: reaction forces on the connections (kN)

If using the previously described SPAV connection and performing a non linear static analysis, we obtain the curve described in Figure 13, where it can be noticed that the connections reach their functional limit at different drift levels for the two rows.

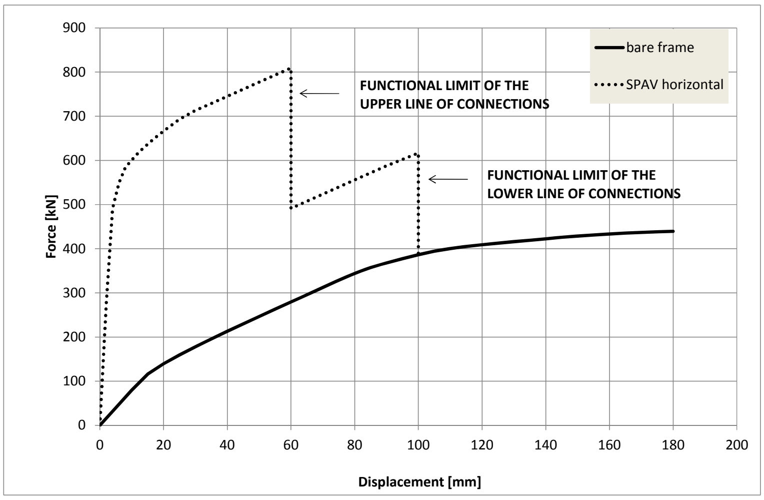


Figure 13. Pushover curve with SPAV connections for the structural system with horizontal panels.

The non linear dynamic analyses performed under L'Aquila AQK-WE accelerogram and an artificially generated earthquake show that, in comparison with the response of the bare frame (equal to that of the previous case), the additional energy dissipation provided by the devices is very effective in reducing the top displacement, which decreases from 80 mm to 26 mm for L'Aquila earthquake and 13 mm for the artificial accelerogram. In both schemes a residual displacement of about 5 mm is found at the end of the earthquake. The contribution of the columns to the overall response is significant for these cases. The vibratory curves are shown in Figure 14. It is finally noted that, with reference to the maximum values of the elastic responses with rigid connections, equal to 2510 kN for the AQK-WE accelerogram, a force reduction factor by 3,59 is deduced for the friction connections herein studied in the case of AQK-WE earthquake.

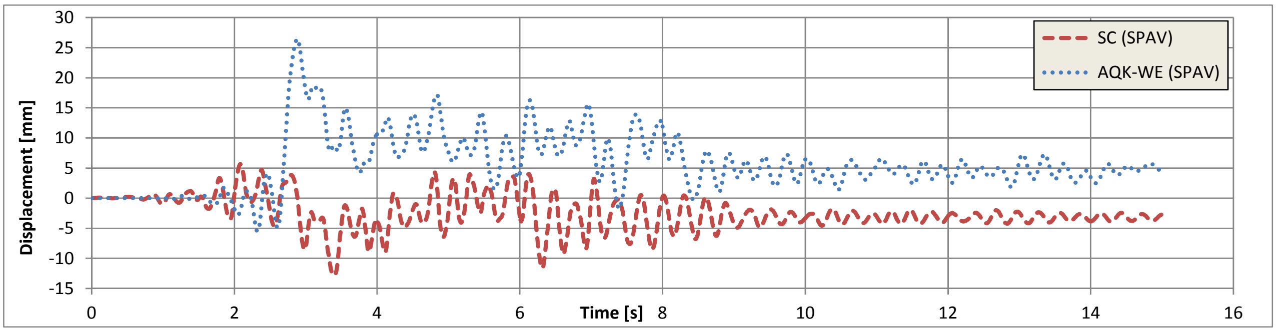
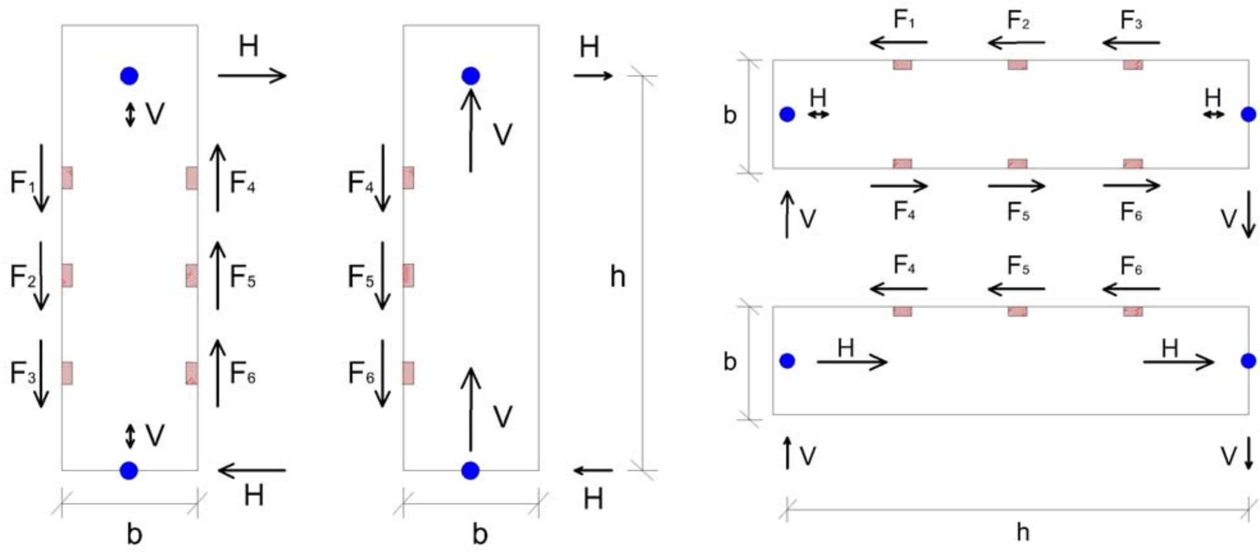


Figure 14. Vibratory curves for the structure with SPAV connections between horizontal panels.

6. CONSIDERATIONS ON THE ROLE OF PANEL-TO-PANEL CONNECTIONS

For the considered accelerograms, the maximum horizontal force applied to the was limited by their friction threshold strength. By isolating single panels with connections, their equilibrium displays as shown in Figure 15.



(a) (b)

Figure 15. Equilibrium of (a) vertical and (b) horizontal cladding panels with connections.

The following maximum contribution of the devices to the overall response can be derived, based on the assumption that all connections enter the friction region ($F_i = R_c$):

For the configuration with vertical panels

$F_{max} \approx n_c\,(n_p - 1)\,R_c\,b\,/\,h = 953$ kN with:

$n_c = 3$	number of connections for each joint;
$n_p = 16$	number of the panels;
$R_c = 60$ kN	strength of one connection;
$b = 2{,}5$ m	width of a panel;
$h = 7{,}08$ m	height of the upper support.

For the configuration with horizontal panels

$F_{max} \approx n_c\,n_s\,(n_r - 1)\,R_c\,b\,/\,H = 478$ kN with:

$n_c = 3$	number of connections for each joint;
$n_r = 3$	number of panel rows;
$n_s = 4$	number of spans;
$R_c = 60$ kN	strength of one connection;
$b = 2{,}35$ m	height of a panel;
$H = 7{,}08$ m	height of the load application.

This is the maximum value for the panels, to be added to the contribution of the columns in the overall response, which action might not be negligible, depending on the entity of the additional devices.

7. CONCLUSIONS

The results of this investigation provides important information related to the magnitude distribution of the forces that the mutual connections between the panels have to transmit. The forces are obtained for a medium-low seismicity zone in the Italian territory with α_g=0,15 and would become much larger for higher risk areas. These are very high forces that put difficult problems for the design of connectors. The problems would become even more difficult for a seismic force acting in the transversal direction of the building along which its effects would be subdivided by a much lower number of panel connections, involving also the connections of the roof in its diaphragm behaviour. A possible solution obviously consists of leaving the cladding panel reciprocally disconnected and connected to the structure only by means of a statically determined support system. Therefore, this latter could be designed as a common frame system following the traditional approach, with much lower forces and with the attention addressed more to the storey drift rather than to the column strength. The other solution, suggested by the present study for both vertical and horizontal panels, employs on the contrary statically undetermined connections with limited strength, calibrated on the slip friction threshold of special connectors up to the requested level, in expectation of a reduced response due to the dissipative capacities of connections.

ACKNOWLEDGEMENT
The present study is supported by the European programme FP7-SME-2007-2, research project SAFECAST, Grant agreement No. 218417, 2009.

REFERENCES

Biondini, F., Toniolo, G. (2009). Probabilistic calibration and experimental validation of the seismic design criteria for one-storey concrete frames, *Journal of Earthquake Engineering*, 13(4), 426-462.

Biondini F., Dal Lago B., Toniolo G. (2010). Influenza delle connessioni tra pannelli sulla risposta sismica di costruzioni prefabbricate; *18th CTE Congress*, Brescia, Italy, November 11-13, 2010.

Biondini F., Dal Lago B., Toniolo G. (2011). Seismic behaviour of precast structures with dissipative connections of cladding wall panels; *14th ANIDIS Congress*, Bari, Italy, September 18-22, 2011.

CEN-EN 1998-1:2004 2004. *Eurocode 8: Design of structures for earthquake resistance – Part 1 General rules, seismic actions and rules for buildings*, European Committee for Standardization, Brussels.

Colombo, A., Negro, P. (2009). Tayloring experimental strategy and set-up – the long story of the seismic behaviour of precast structures, *3rd International Conference on Advances in Experimental Structural Engineering*, San Francisco, CA, USA, October 15-16, 2009.

Colombo A. & Toniolo G. (2012). Precast constructions: the lesson learned from L'Aquila earthquake, *Structural concrete,* Journal of fib.

Ferrara, L., Felicetti, R., Toniolo, G., Zenti, C. (2010). Dispositivi dissipatori ad attrito per pannelli di tamponamento in edifici prefabbricati – una indagine sperimentale, *18th CTE Congress*, Brescia, Italy, November 11-13, 2010.

Menegotto, M. (2010). Experiences from L'Aquila 2009 earthquake. *3rd International fib Congress*, Washington DC, USA, May 29-June 2, 2010.

Shultz, A. E., Magana, R. A., Trados, M. K., Huo, X. (1994). Experimental study of joint connections in precast concrete walls, *5th U.S. National Conference on Earthquake Engineering*, Chicago, IL, USA, July 10-14, 1994.

Symans, M. D., Charney, F. A., Whittaker, A. S., Constantinou M. C., Kircher, C. A., Johnson, M. V. and McNamara, R. J. (2008). Energy Dissipation Systems for Seismic Applications: Current Practice and Recent Developments**,** *Journal of Structural Engineering,* 134(3), 3-21.

Takeda T., Sozen M.A. and Nielsen N.N. (1970). Reinforced Concrete Response to Simulated Earthquakes, *ASCE Journal of the Structural Division,* 96(12), 2557-2573.