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ORIGINAL RESEARCH PAPER

Role of wall panel connections on the seismic performance of precast structures

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Abstract Past seismic events, including the 2009 L'Aquila earthquake and the 2012 Emilia earthquake, clearly demonstrated the inadequacy of the current design approach for the connection system of the cladding wall panels of precast buildings. To clarify this problem the present paper investigates the seismic behaviour of a traditional precast structural frame for industrial buildings with a new type of connection system of cladding panels. This system consists of a statically determined pendulum arrangement of panels, each supported with two hinges to the structure, one at the top and one at the bottom, so to have under seismic action a pure frame behaviour where the wall panels are masses without stiffness. Adding mutual connections between the panels, the wall cladding panels become part of the resisting structure, leading to a dual frame/wall system or to a wall system depending on the stiffness of the connections. The seismic behaviour of this structural assembly is investigated for different degrees of interaction between frame and panels, as well as for an enhanced solution with dissipative connections. The results of nonlinear static (pushover) analyses and nonlinear dynamic analyses under recorded and artificial earthquakes highlight the role of the wall panel connections on the seismic behaviour of the structural assembly and show the effectiveness of the dual frame/wall system with dissipative connections between panels.

Keywords Seismic design · Concrete structures · Precast structures · Cladding wall panels · Connection devices

1 Introduction

In the tradition of precast concrete structures in Southern Europe, typical frame structures with cantilever columns and large floor spans have been widely used for industrial and commercial buildings since the sixties. For this type of buildings the use of large and heavy

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precast concrete cladding panels is quite common. They have been and still are considered as “non structural” members of the building, acting as seismic masses without stiffness and designed with local calculations, neglecting any contribution to the overall structural behaviour.

Past seismic events, including the 2009 L’Aquila earthquake and 2012 Emilia earthquake, clearly demonstrated the inadequacy of this approach to design the connections of the wall panels. Surveys on the effects of the 2009 L’Aquila earthquake are reported in Menegotto (2009) and Colombo and Toniolo (2012a). Many collapses of cladding panels due to underdesigned connections have been reported. While the typical frame is very flexible and high energy dissipation can be achieved at large drifts, the cladding-frame connections were statically undetermined but not designed for overall horizontal actions.

A comprehensive literature review on seismic performance of cladding systems in buildings can be found in NIST GCR 96-681 (1995). In particular, the problem of cladding-frame interaction has been studied in Palsson et al. (1984) and Henry and Roll (1986), showing that such interaction can dramatically change the seismic response. A proposal to exploit the cooperation of large cladding panels to reduce the seismic response through energy dissipation has been presented in Pinelli et al. (1995). The idea of exploiting the rocking motion of structural wall panels with dissipating couplers has been explored by Priestley et al. (1999) within the PRESSS program. Devices with dissipative capacities based on mechanisms of yielding or friction and designed to make mutual connections between wall panels have been investigated in Shultz et al. (1994) and Iqbal et al. (2007).

Colombo and Toniolo (2012b) presented a systematic framing of the problem, indicating three possible solutions to avoid panel collapses due to the failure of their connections. These solutions include:

(a) The adoption of a *statically determined support system* that makes the panels independent from the motion of the structure or allows a rigid motion of the panels;
(b) The adoption of an *integrated support system* adequately proportioned that makes the panels integral part of the resisting structure.

The present study starts from a particular arrangement of the statically determined solution (a), with a pendulum support system of vertical panels, consisting of two hinges applied one at the top and one at the bottom of each panel, and shows the effectiveness of mutual connections added between the panels so to integrate them into the resisting structural system as in the integrated solution (b). The seismic behaviour of this structural assembly is investigated for different degrees of interaction between frame and panels, as well as for an enhanced solution with dissipative connections (Biondini et al. 2012a). The results of nonlinear static (pushover) analyses and nonlinear dynamic analyses under recorded and artificial earthquakes highlight the role of the wall panel connections on the seismic behaviour of the structural assembly and show the effectiveness of the dual frame/wall system with dissipative connections between panels.

The case study stands within the scope of a wider problem that will engage for some years the researchers in the field of prefabrication to find adequate solutions. After more than 15 years of experimental and analytical research at European level, the achieved knowledge led to well defined criteria and reliable technical and normative documents for the design of precast frame structures (Biondini and Toniolo 2009, 2010). The problem of a proper design of the connections between the structural elements of the construction has been recently investigated within the European research project SAFECAST “Performance of Innovative Mechanical Connections in Precast Building Structures under Seismic Conditions” (FP7-SME-2007-2, Grant agreement No. 218417, 2009) with the involvement of 16 partners from the European countries more exposed to seismic risk (Toniolo 2012; Biondini et al. 2012b; Negro et al. 2012). The involvement of cladding panels in the resisting system of precast industrial buildings will be investigated within the scope of the recently started European research project SAFECLADDING "Improved fastening systems of cladding wall panels of precast buildings in seismic zones" (FP7-SME-2012, Grant agreement No. 314122, 2012).

2 The prototype

The investigated structural prototype is a one-storey precast frame building of 40.5 × 25.0 m of dimensions in plan, made of two lines of five columns 7.0 m high and spaced by 10.0 m. The roof is made of five transverse shed beams supporting ribbed elements. Under seismic condition the dead loads of the roof are related to:

- roof elements (including permanent finishings) = 2.8 kN/m^2;
- shed beams (average weight) = 17.5 kN/m;
- longitudinal beams (panel supporting) = 3.2 kN/m;

that lead to a vertical action of 600 kN on each of the 3 + 3 middle columns and 410 kN on each of the 2 + 2 end columns. All the columns have a square cross-section with side width of 60 cm and are reinforced with 12 ϕ 20 mm longitudinal bars corresponding to the minimum reinforcement ratio of 1 % given by Eurocode 8 (CEN-EN 1998-1 2004). The steel Class is B450C. The concrete Class is C45/55, with elastic modulus equal to 30 GPa.

A set of 16 + 16 vertical wall panels are placed along the two major sides of the building, with dimensions 2.5 × 8.0 m and an equivalent concrete thickness of 12 cm, leading to a weight of 3.0 kN/m^2. The panels are placed at the base on the foundation beam with a central hinged support that allows their free pendulum movement within the limits of the joint allowance left between the foundation beam and the panel. At the height of 7.08 m the panels are connected to the supporting beam with a central hinge. In this configuration the connection system 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 view of one cladding wall with the indication of the supports and panel connections. Figure 2 shows the equilibrium of the forces of one internal panel (Fig. 2a), with connections at both sides, and of an end panel (Fig. 2b), with connections at one side, respectively. The equilibrium conditions between the horizontal (H) and vertical (V) components of the forces in the supports and the forces F_1 and F_2 in the panel connections can be obtained from the geometrical dimensions b and h as follows for both the internal panel:

$$H = (F_1 + 2F_2)\, b/h$$

$$V = 0$$

and the external panel:

$$H = (F_1 + 2F_2)\, b/2h$$

$$V = (F_1 + 2F_2)\,/2 = Hh/b$$

These equations, applied to the total set of panels (with $F_1 \approx 0.8F_2$, as shown in Clause 3), provide the resistance of the connections required to sustain the horizontal action H associated to the limit state to be fulfilled.

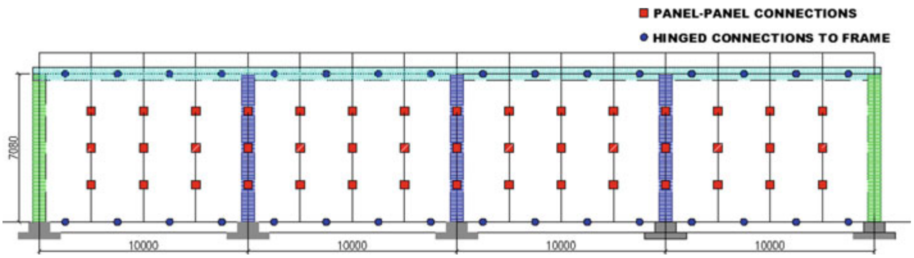


Fig. 1 View of the cladding wall system

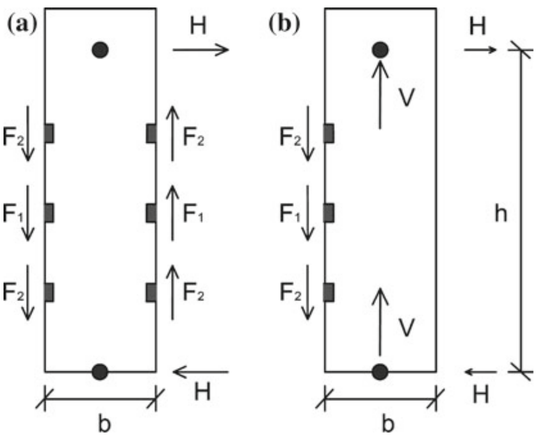


Fig. 2 Static scheme for in-plane equilibrium of **a** central and **b** side panels under horizontal loading

3 Linear elastic analysis

The analysis of the structural system is carried out by means of a finite element model with beam elements for columns and shell elements for panels (Fig. 3). A first series of linear elastic static analyses are performed on this model by applying at the top of the columns an horizontal force of 1,000 kN that corresponds to a ratio of about 1/3 of the competent roof and wall weights computed as follows:

$$2.8 \times 40.0 \times 25.0 = 2{,}800$$
$$5 \times 17.5 \times 25.0 = 2{,}187$$
$$3.2 \times 2 \times 40.0 = 256$$
$$3.0 \times 12.0 \times 8.0/2 = 1{,}440$$
$$W = 6{,}683 \text{ kN}/2{=}3{,}341 \text{ kN}$$

where the cladding walls are considered extended to the whole perimeter less two openings of 5.0 m each.

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, and perfectly rigid connections (stiffness $k = \infty$), with structural response given almost only by the wall panels. Intermediate cases are also studied with different values of the elastic stiffness k of the connections.

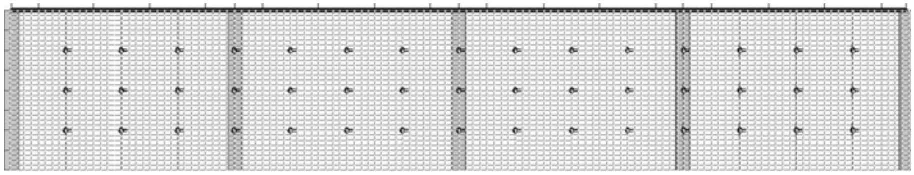


Fig. 3 Finite element model of the cladding wall system

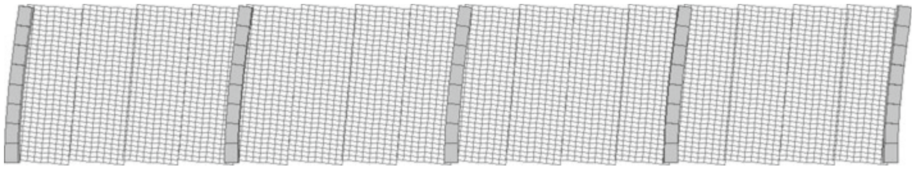


Fig. 4 Statically determined system: deformed shape of the structure ($d_{top} = 73.0$ mm)

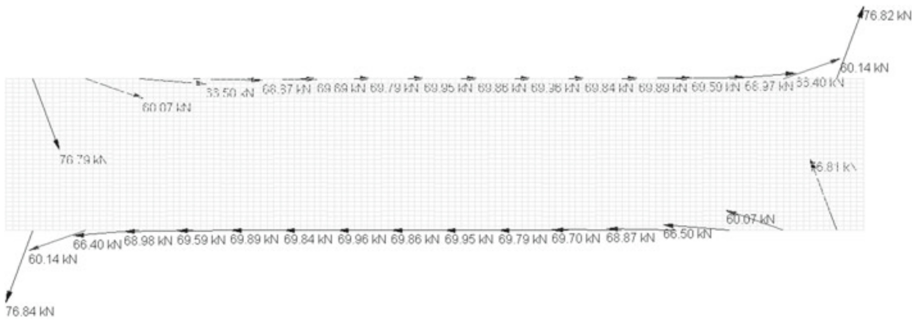


Fig. 5 Rigid connections: reaction forces on the fastenings (kN)

Figure 4 shows the deformed shape of the system without connections ($k = 0$). The top displacement is 73.0 mm and roughly corresponds to a roof drift of 1 %. The shear action on the columns is 200 kN and obviously corresponds to an equal repartition on the five columns of the global force applied at the top. The panel-to-beam fastenings do not receive any action from the roof: they can be proportioned with a local calculation based on the mass of any single panel.

For the case with perfectly rigid connections ($k = \infty$), the analysis leads to a top displacement reduced to 0.3 mm due to the much higher stiffness of the collaborating wall that takes almost all the applied action, leaving to the columns a very small portion of the base shear (0.5 kN). Figure 5 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 1,000 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 about 77 kN.

Figure 6 shows the distribution of the actions transmitted within 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

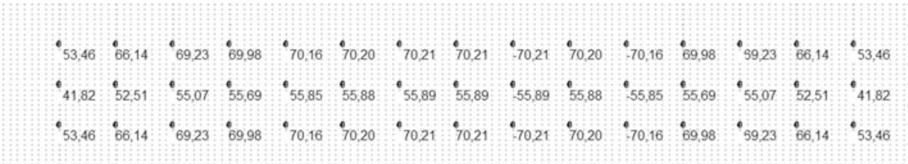


Fig. 6 Rigid connections: reaction forces on the panel-to-panel connections (kN)

panels the resultant of the forces of the three connections with the adjacent panel, missing the opposite panel, is equilibrated by the vertical resultant of the reactions of the two upper and lower fastenings to the structure. Figure 7 gives an indication of the concentration of the stresses on the panels over the fastenings and connections.

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. Following Eurocode 8 design rules (CEN-EN 1998-1 2004), for medium ductility class a behaviour factor $q = q_w q_o = 0.5 \times 3.0 = 1.5$ can be assumed for the squat wall system with rigid connections and, with reference to the maximum response amplification for a subsoil type B, a storey force $F_h = \alpha_g S_{max} W/q$ with $S_{max} = 1.2 \times 2.5 = 3.0$ can be computed. Therefore, with $F_h = 1{,}000$ kN, a seismic capacity $\alpha_g = 1.5 \times 1{,}000/(3.0 \times 3, 341) \approx 0.15$ 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.

The elastic stiffness of the mutual panel connections, with intermediate values $0 < k < \infty$, obviously influences the structural response. Figure 8 shows how the ratio between the base shear in the panels and the total base shear (columns plus wall) increases with the connection stiffness. For very low stiffness the structural response is close to a pure frame behaviour. For intermediate values of the connection stiffness a combined behaviour typical of the dual frame-wall systems is achieved. For high stiffness values the structural response stabilises on a wall system behaviour with almost total release of the column forces. Furthermore, Fig. 8 shows the complementary stiffening effect of the connections in terms of top displacement. Finally, Fig. 9 shows the repartition ratio of the shear force exchanged between the adjacent panels on the connections, the middle one F_1 and the upper (or lower) one F_2. For high stiffness values the middle connections have a force 0.8 time lower than the others, while for low stiffness values the forces get uniform ($F_1/F_2 \to 1$). It is worth noting that the transition interval for the repartition ratio in the connections (Fig. 9) is moved towards higher stiffness values with respect to both the repartition ratio of the base shear and the top displacement (Fig. 8). The difference of the forces F_1 and F_2 is negligible for values of stiffness lower than 10 kN/mm.

4 Non-linear static analysis

The seismic response of the structure is affected by the non-linear behaviour of its members. In particular, the effectiveness of the mutual panel connections depends primarily on their

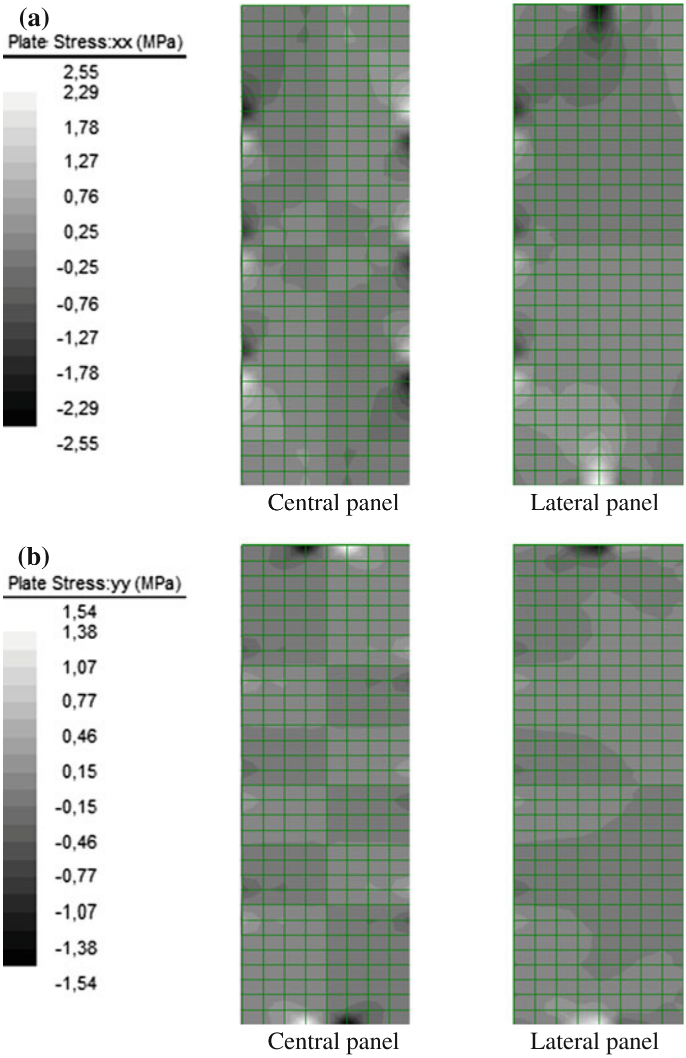


Fig. 7 Rigid connections: contoured maps of (**a**) horizontal normal stresses σ_{xx} and (**b**) vertical normal stresses σ_{yy} (MPa)

ductility capacities. To investigate this aspect a series of non-linear static analyses with monotonic loading (pushover) is performed.

Figure 10 shows the moment-curvature relationship of the column cross-sections for the two levels of axial forces. These diagrams are computed with a parabola-rectangle stress-strain model for concrete, neglecting its tensile strength, and with a bi-linear hardening stress-strain model for the steel reinforcement. An elastic behaviour is assumed for the wall panels relying on their over-strength with respect to the capacity of the mutual connections. For these connections an elastic-plastic behaviour is assumed with different values of the initial elastic stiffness and of the ultimate resisting force. The ultimate deformation of the connecting devices is not specified and the plastic branch is left indefinite.

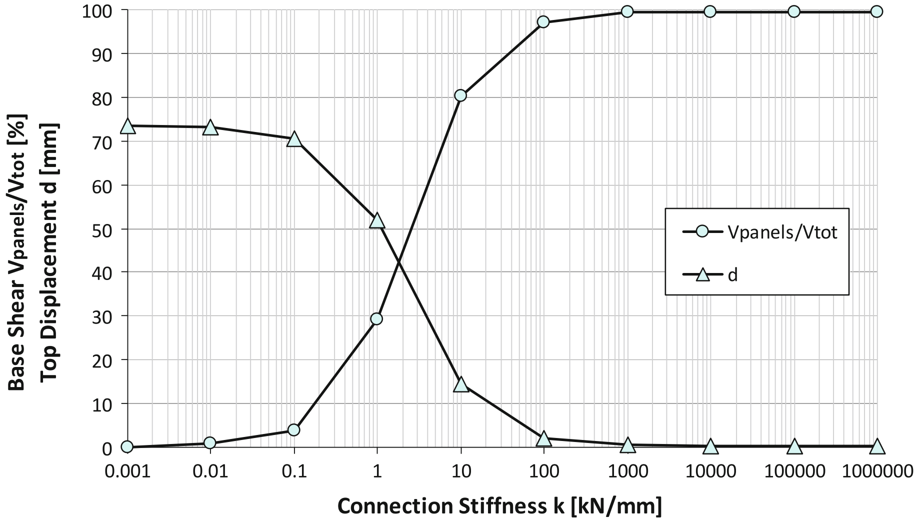


Fig. 8 Base shear in the panels and top displacement versus the connection stiffness

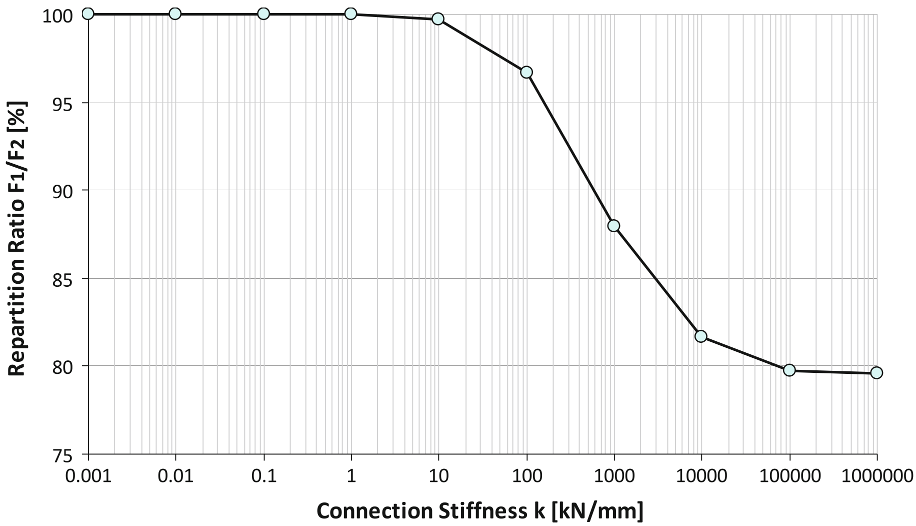


Fig. 9 Repartition ratio F_1/F_2 versus the connection stiffness

Figure 11 shows a family of pushover curves of the structural system for different values of the initial elastic stiffness of the connections and for a very high ultimate resisting force (R = 200 kN). From the initial slope of the curves the elastic stiffness of the structure is deduced and the natural vibration period T is evaluated. This period depends on the collaboration degree between columns and panels, provided by the stiffness of the connections, and it varies from 1.2 to 0.05 s.

Figure 12 shows how the natural vibration period decreases with the higher values of the elastic stiffness of the connections. The same diagram also indicates the corresponding variation of the structural response in terms of ratio a/a_g between the maximum acceleration

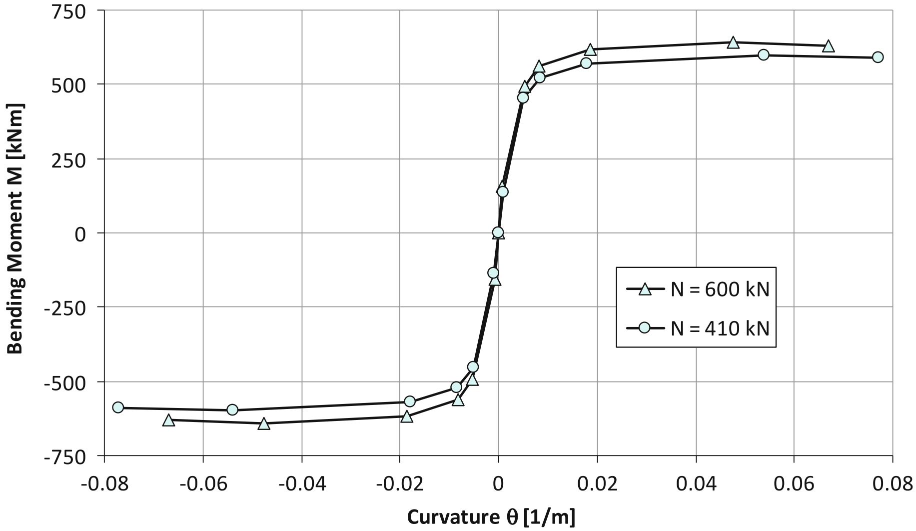


Fig. 10 Bending moment-curvature relationship of the column cross-sections

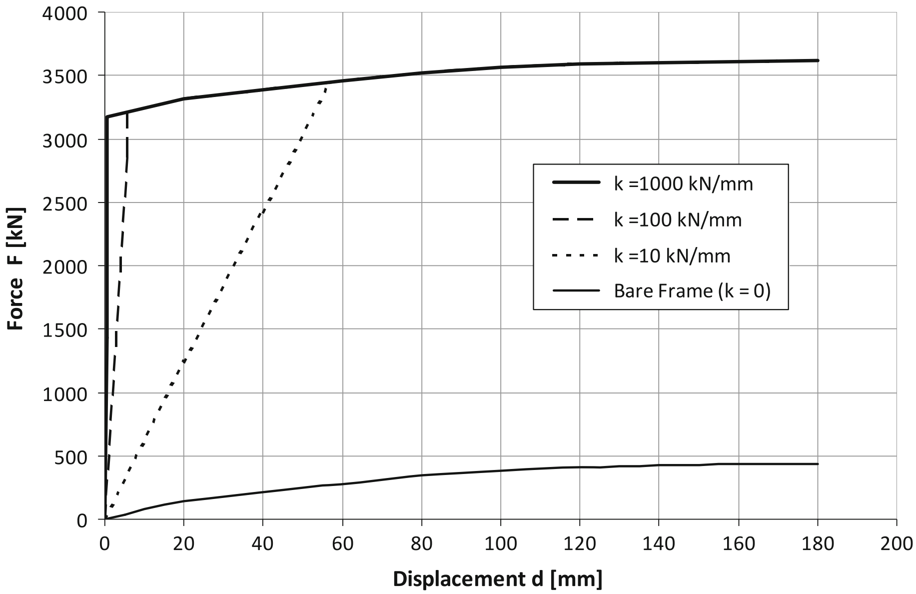


Fig. 11 Pushover curves for R = 200 kN

a and the peak ground acceleration a_g, computed from the elastic response spectrum given by Eurocode 8 for subsoil type B. A large increase of the seismic force can be noted in the solution with wall panels integrated in the structural assembly, with values of the ratio a/a_g that initially increases with the connection stiffness from 1.3 to 3.0 and then decrease to 1.8 for the case of rigid connections. This response factor does not consider the energy dissipation possibly offered by the structural system.

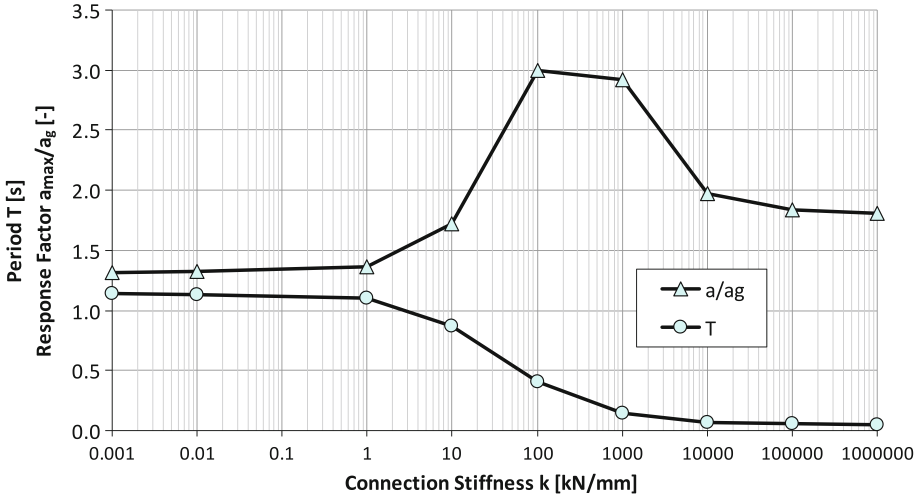


Fig. 12 Parameters of the dynamic response

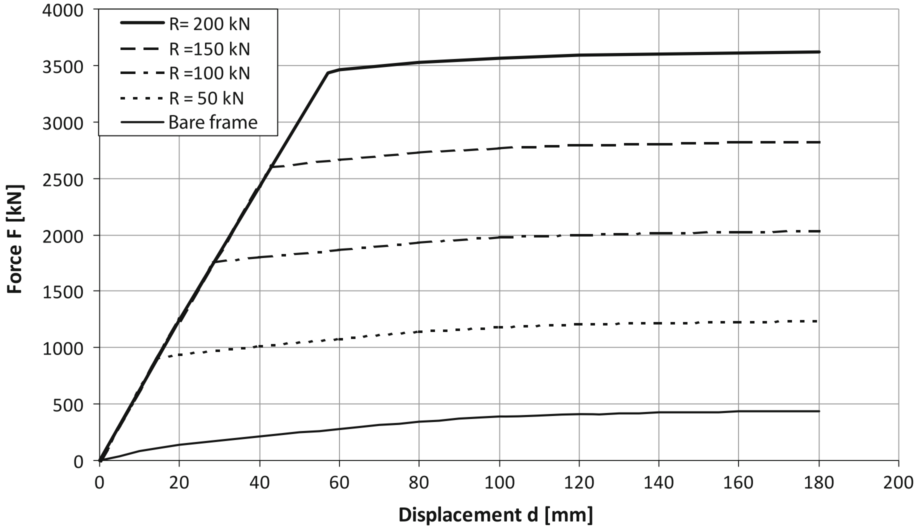


Fig. 13 Push-over curves for k = 10 kN/mm

Figure 13 shows that the pushover curve of the structural system is limited to the level corresponding to the different possible strengths of the connections. It is worth noting that the shear ratio taken by the wall panels in the elastic stage (see Fig. 8) decreases progressively after the yielding of the connections with the development of their plastic deformation until the yielding of the columns, as shown in Fig. 14. From these results it is clear that, if the connections have sufficient plastic deformation capacity, the choice of their strength level can influence the energy dissipation capacity and properly regulate the structural response with reference to the intensity of the seismic action expected on site. An example of this

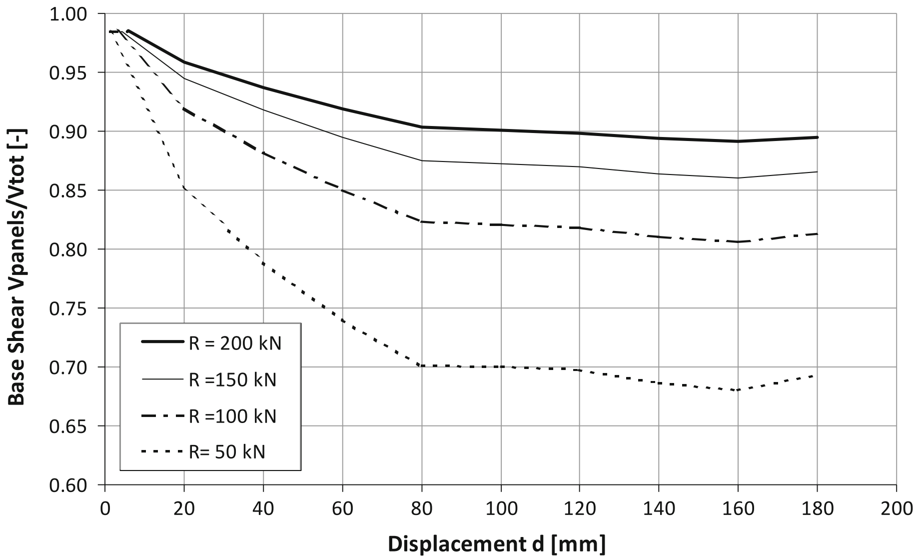
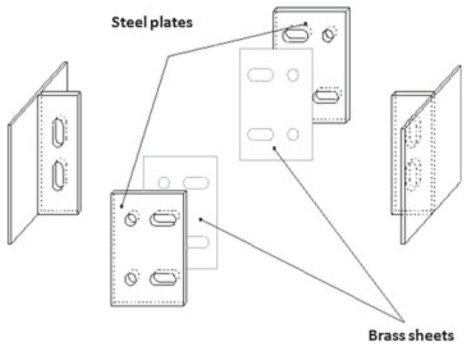


Fig. 14 Plastic repartition of base shear for k = 100 kN/mm

Fig. 15 Components of a dissipative connection device (SPAV)



regulation will be presented in the next section devoted to nonlinear dynamic analysis of the prototype.

5 Non-linear dynamic analysis

Non-linear dynamic analyses of the prototype are carried out considering a dissipative connection device tested at the Laboratory for material testing of Politecnico di Milano for SPAV Company, Martignacco, Italy (Ferrara et al. 2011). 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 Fig. 15. The length of the bolt 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 resistance. Two brass sheets are interposed between

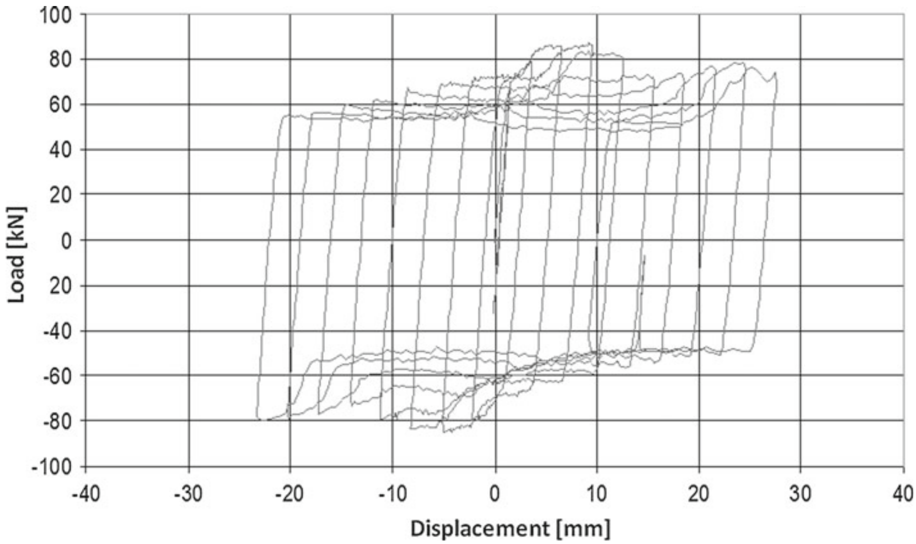


Fig. 16 Force-displacement cycles of the SPAV connection

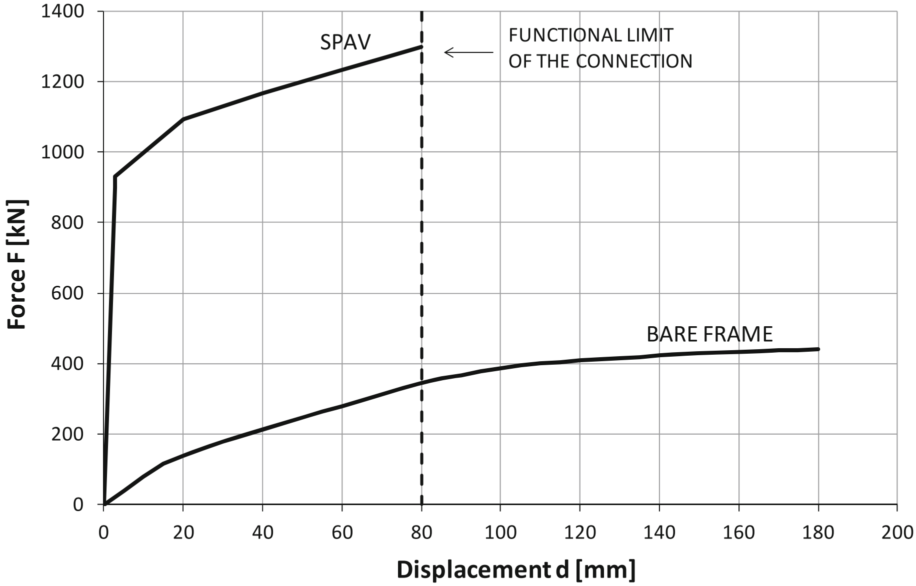


Fig. 17 Pushover curve with SPAV connections

the steel plates and the profiles to ensure the stability of the repeated slide cycles, as shown in Fig. 16.

The use of slip resistant connections with pre-loaded bolts is common in steel construction and regulated by Eurocode 3 (CEN-EN 1993-1-8 2005). For the control of the pre-loading related to the required tightening force, reference can be made to EN1090-2 (CEN-EN 1090-2 2008). Protection and durability of the connections can be obtained like in steel construction, with the possibility of inspections and re-adjusting of the tightening force at due times.

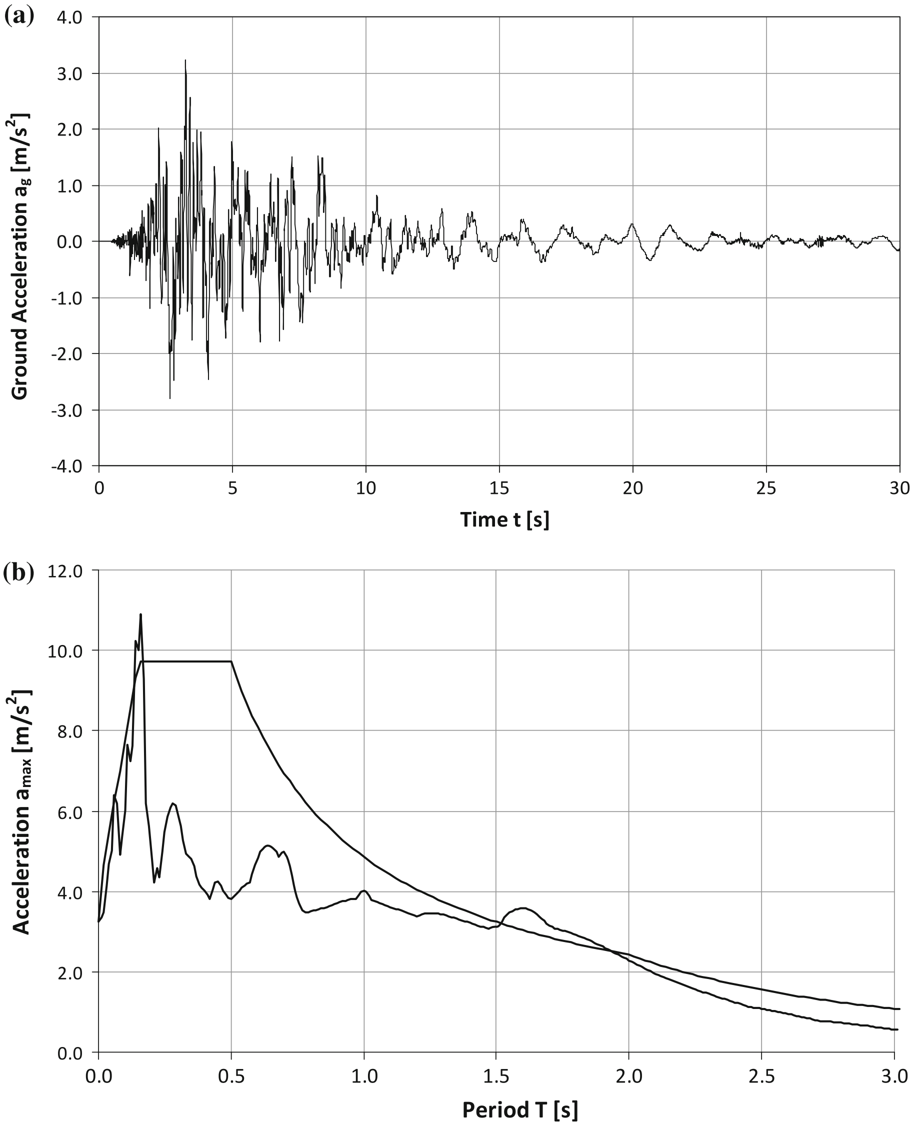


Fig. 18 L'Aquila earthquake (AQK-WE): **a** accelerogram and **b** response spectrum compared with the spectrum of Eurocode 8

Assuming the slip threshold as corresponding to the yielding limit, the friction phase as corresponding to the plastic deformation, and the slip resistance as corresponding to the ultimate strength of the connection, the constitutive law of the connection device is modelled 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 bolts within the slots, that is ±40 mm for a perfect initial centering of the bolts. The corresponding top displacement of the structure is $d = \pm 40 \times 7.08/2.5 = \pm 113$ mm. Figure 17 shows a pushover curve of the structure with

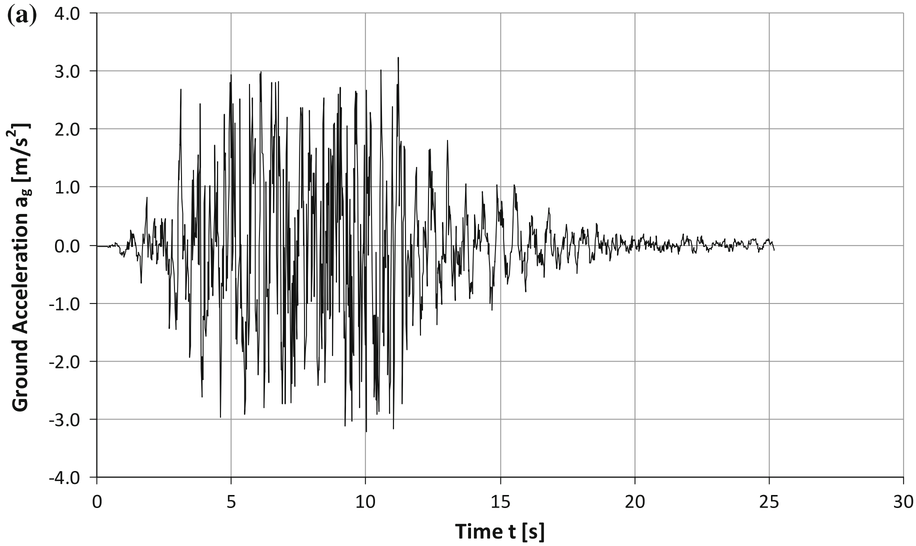


Fig. 19 Set of artificial earthquakes: **a** one accelerogram from the set (SC), and **b** response spectra of the set of accelerograms compared with the model of Eurocode 8

SPAV connections computed with the elastic-plastic model based on an initial stiffness of 600 kN/mm, a slip resistance of 60 kN and a "functional" stroke limit of ±25 mm.

The dynamic non linear analyses are 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 performed under a recorded accelerogram of the 2009 L'Aquila earthquake (AQK-WE) with PGA = 0.32 g, and ten artificial synthetic accelerograms (SC) compatible with the response spectrum given by Eurocode 8 for subsoil type B and scaled to

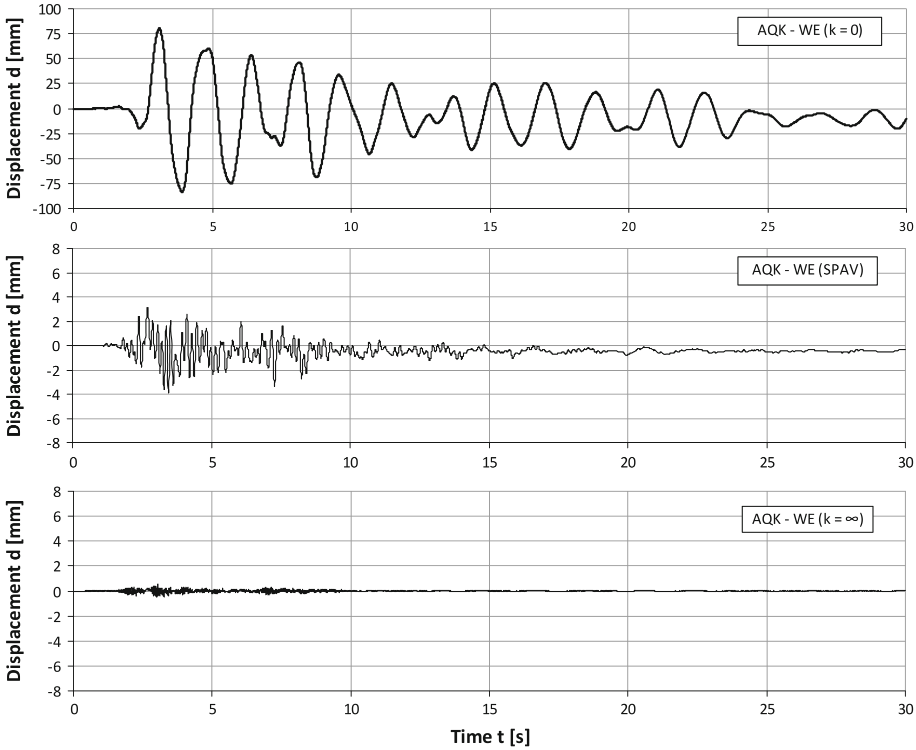


Fig. 20 Displacement time-histories under L'Aquila earthquake (AQK-WE)

PGA = 0.32 g. Figures 18 and 19 show these accelerograms and their elastic response spectra compared to the Eurocode 8 spectrum. It is worth noting that the scanty compatibility of the response spectrum of L'Aquila earthquake with the Eurocode 8 standard spectrum, due to its poor content of frequencies, could lead to a weaker impact on the actual structural response.

The different response of the three structural configurations (statically determined, integrated and dissipative) under L'Aquila earthquake (AQK-WE) is shown in terms of vibratory curves in Fig. 20. 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 same analyses are performed for the set of ten artificial accelerograms. Figure 21 shows the vibratory curves of the system under one artificial accelerogram from the set (SC, Fig. 19a).

The strong effectiveness of the connections can be noticed in lowering the maximum storey drift of the statically determined system from 80 to 4 mm and 0.6 mm under L'Aquila earthquake, and from 80 to 6 mm and 0.8 mm under the artificial accelerogram, for SPAV connections and rigid connections, respectively. The large increase of vibration frequencies of the structural responses of the integrated systems can also be noted.

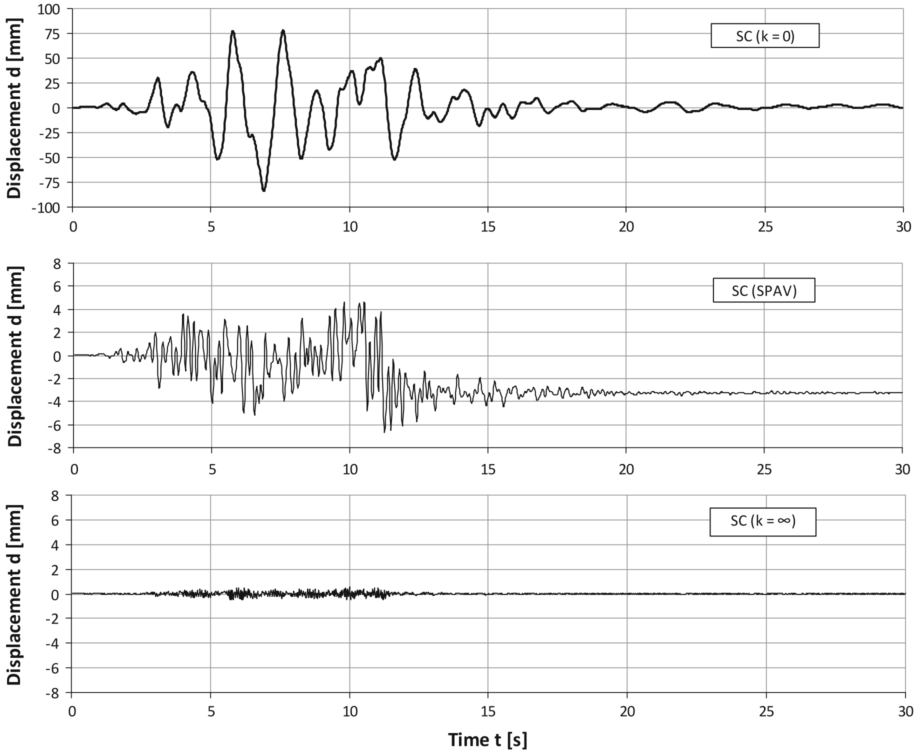


Fig. 21 Displacement time-histories under an artificial earthquake (SC, Fig. 19a)

In particular, the response with rigid connections corresponds to a pure elastic behaviour of a stiff wall system.

The residual displacements 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. This is more evident for the artificial accelerograms that have, with their full content of frequencies, a stronger impact on the structure. The measure of the energy dissipation can be deduced from the force-displacement diagrams shown in Fig. 22, where the area within the cycles is much wider for the artificial accelerogram. The vibratory curves corresponding to the whole set of artificial accelerograms are compared in Fig. 23 for the configuration with dissipative devices. Figure 24 shows the absolute values of the maximum and residual displacements associated to the set of artificial earthquakes. The values of representative parameters of the structural response for the set of artificial earthquakes and L'Aquila earthquake are listed in Table 1.

It is worth noting that the maximum displacements reached with the artificial accelerograms are much higher than with the recorded one, and are characterized by a relatively low dispersion. A mean residual displacement of 2.71 mm is expected under the artificial earthquakes, with a large scatter due to the variability of time history of each accelerogram. The energy dissipation is mainly provided by the nonlinear behaviour of the devices, while the columns remain almost linear elastic. In all the studied cases the maximum total horizontal

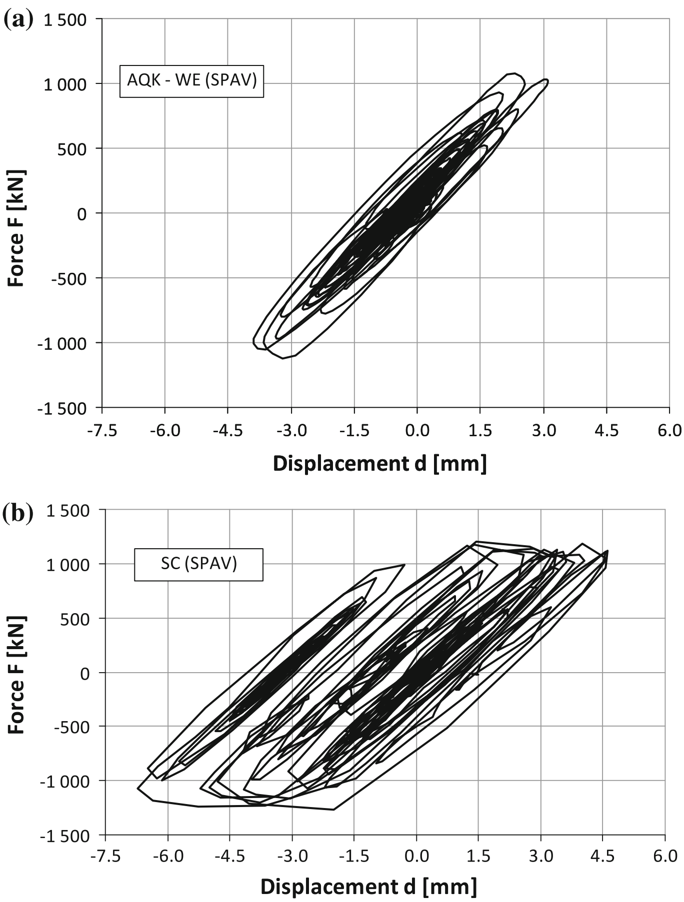


Fig. 22 Force-displacement cycles of the system with dissipative connections (SPAV) under **a** L'Aquila earthquake (AQK-WE) and **b** one artificial earthquake (SC, Fig. 19a)

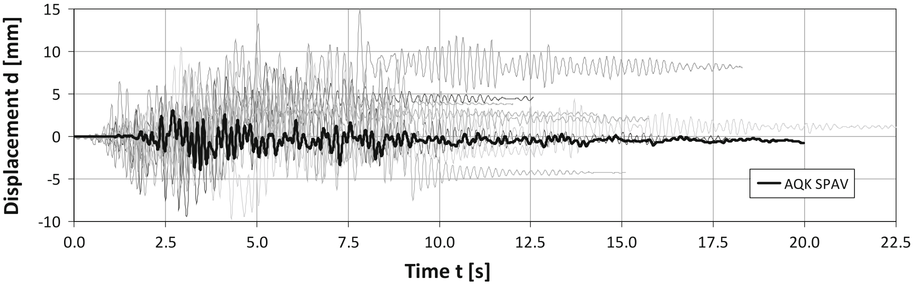


Fig. 23 Displacement time history for a set of artificial earthquakes and L'Aquila earthquake (AQK-WE)

force F_{max} applied to the structure was limited by the slip resistance (strength) of the connections:

$$F_{max} \approx n_c \left(n_p - 1\right) R_c b/h = 953 \text{ kN}$$

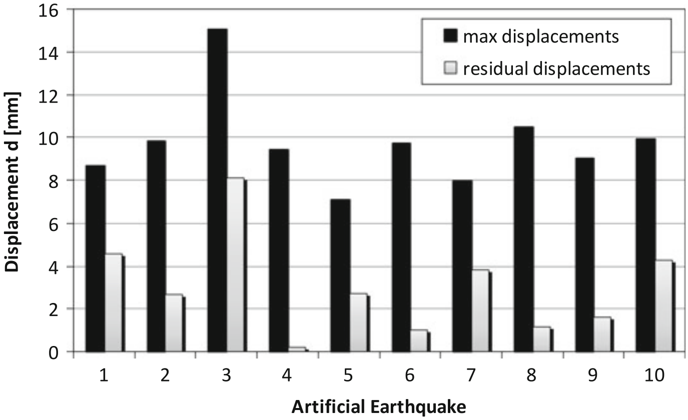


Fig. 24 Maximum and residual displacements (absolute values) for the set of artificial earthquakes

Table 1 Parameters of the structural response under artificial earthquakes and L'Aquila earthquake

	Artificial accelerograms (SC)				L'Aquila (AQK-WE)
	Max	Min	Mean	St. Dev.	
Maximum displacement (mm)	15.0	7.1	9.7	2.1	3.9
Residual displacement (mm)	8.1	0.2	2.7	2.3	0.5
Maximum total base shear (kN)	1,577	1,212	1,405	113	1,120
Maximum shear with fixed devices (kN)	8,170	2,920	5,790	1,394	2,510
Force reducing factor (behaviour factor)	6.09	2.92	4.59	0.91	2.24
Maximum force in devices (kN)	60	60	60	0	60
Maximum device slippage (mm)	5.4	2.5	3.5	0.8	1.4
Energy dissipated by devices (kNm)	80.70	26.67	53.13	17.29	17.74
Energy dissipated by columns (kNm)	0.03	0.00	0.00	0.01	0.00

where:

$n_c = 3$	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.

This is the maximum value of the base shear for the panels, to be added to the contribution of the columns in the overall response, which is much smaller but not negligible.

It is finally noted that, with reference to the maximum force values of the elastic response with rigid connections shown in Table 1, force reduction factors varying from 2.24 to 6.09 are deduced for the dissipative connections herein studied. This range provides an indication of the order of magnitude of the behaviour factor to be used in a typical elastic analysis of this type of systems. A more reliable value should be deduced, as a function of the slip resistance of the devices, from a much wider parametric investigation based on large sets of accelerograms.

6 Conclusions

The involvement of cladding panels in the resisting system of precast industrial buildings leads to a number of design problems related to the effects on the overall structural assembly. The role of the roof elements in their diaphragmatic behaviour shall be defined with reference to the layout in plan of the building, where the stiffening contribution of the wall panels can be differently distributed. The high level of forces, carried on the connections by the high stiffness of the walls, requires new types of high strength devices. Also the design of panels shall be revised to account for their actual role into the resisting system.

These problems will be widely investigated within the scope of the European research project SAFECLADDING devoted to "Improved fastening systems of cladding wall panels of precast buildings in seismic zones" (FP7-SME-2012, Grant agreement No. 314122, 2012). The scope of the present paper is limited to a proposal for the use of dissipative connections between the panels in order to calibrate the repartition of forces on the structural parts (frame and walls) and keep in this way forces and displacements within allowable levels. It is worth noting that, even though the working principles of the proposed system are clearly stated, several questions remain pending for its actual application.

For what concerns the current elastic analysis used in the design of precast structures, the dissipative connections require a preliminary parametric investigation comparing linear elastic and non-linear dynamic analyses so to evaluate a force-reducing factor (behaviour factor) to be applied to the elastic seismic response of the integrated stiff structure. An example of such evaluation is given in this paper. With respect to the integrated stiff structure, for which the maximum elastic amplification of the ground motion is expected, the lowering of the resistance of the dissipative connections leads to an attenuation of the nonlinear response down to what expected for the bare frame with no contribution of the panels. The detailed study of this behaviour will be the topic of a future paper.

In general, in the current practice the detailing of the cladding panels does not fulfil the code requirements for structural shear walls, especially for their minimum thicknesses. Following the results of the quoted SAFECLADDING research project, special rules could be added in Eurocode 8 for this type of elements, allowing their structural use in a systematic way.

When a codified approach for the strength verification of panels cannot be applied, a double analysis shall be performed: one analysis for the verification of columns referred to the bare frame where the panels are masses without stiffness, and one analysis for the verification of panel connections referred to the dual wall/frame system where the panels are taken into account with their actual stiffness.

The results of the investigation presented in this paper provide important information related to the entity of the forces that the mutual connections between the panels have to transmit. The forces of 55 and 70 kN obtained for a medium-low seismicity zone in the Italian territory with $\alpha_g = 0.15$, would become of 92 and 117 kN for a medium-high seismicity zone with $\alpha_g = 0.25$ and would become of 128 and 163 kN for a high seismicity zone with $\alpha_g = 0.35$. 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 supported only by a statically determined system of connections to the structure. With this solution the structure could be designed as a common frame system following the traditional approach, with much lower forces and with the attention focused more on the storey drift rather than on the column strength. Special connections would be needed to allow the very large displacements expected under earthquake conditions. The solution suggested by the present study employs on the contrary a statically undetermined system of connections with limited strength, calibrated on the slip resistance of special connectors up to the requested level, in expectation of a reduced response due to the dissipative capacities of the connections. It seems a promising way, based on very simple low cost devices, which addition could be largely compensated by the reduction of the column size.

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