

Behaviour of a support system for pre-cast concrete panels

G. Metelli

University of Brescia, Italy

P. Riva

University of Bergamo, Italy

ABSTRACT: Even though a large amount of precast concrete panels are produced every year in Europe, there is a lack of studies on the seismic behaviour of connection systems. In order to understand the interaction between the concrete panels and the structure, a numerical and experimental study on a connection system subjected to seismic action has been faced. Non-linear finite element analyses of a concrete panel portion connected to a concrete column by a steel system have been conducted with FE program DIANA. The non linear behaviour of the materials and of the contact surface between the panel and the support system are considered. Experimental tests have been carried out on prototype specimens representing a column portion linked by the connection system to a precast panel portion. The specimens have been subjected to cyclic horizontal displacement histories by imposed transverse displacement. The tests are an effective tool to validate the numerical results and to define an accurate force-displacement constitutive law of the connection.

1 INTRODUCTION

Precast concrete panels are often used for the façades of modern warehouses and commercial malls.

In Italy, the precast concrete structures are generally designed neglecting the interaction between the structure and the cladding panels. These elements are hung to the columns or the beams and are considered to be mass, contributing only to the dynamic properties of the structural skeleton of the building, having no influence on the lateral stiffness. The connection systems of the cladding panels should be designed in order to provide the displacement demand during serviceability earthquake without any damage occurring in the panel and in the structure. At the same time, they should be designed also to transmit forces to the structure due to ultimate limit state earthquakes (Eurocode 8, 2003). Furthermore, this assumption, which neglects the structural behaviour of the external cladding, is presumed to be conservative in the design of seismic resistant concrete frames. Linear static and linear dynamic analyses of reinforced concrete frames have shown a reduction of the lateral drift and a change in the natural frequency and member force distribution, by considering external precast concrete wall claddings as opposed to the bare frame (Henry and Roll, 1986). These numerical results indicate that neglecting the

structural role of the external concrete walls might not be conservative.

Although several works have been focused on the seismic behaviour of RC frames with masonry infills (Biondi et al., 2000; Mehrabi et al., 1996; Mehrabi and Shing, 1997) or moment resisting steel frames (De Matteis, 2005; Dogan et al., 2004; Pinelli et al., 1995), the dynamic and non linear behaviour of pre-cast structures under seismic actions, with pre-fabricated RC panels used as curtain walls, is still not well known due to the lack of studies on the interaction of the RC frame and the external pre-cast concrete panels. This aspect has been widely studied for moment resisting steel frames: a recent study (De Matteis, 2005) proves, by non-linear dynamic time-history analyses, that light sandwich panels bolted to edge members of the frame can improve the lateral stiffness of the structure, allowing a remarkable reduction in size of the MR steel bare frame. The hysteretic behaviour of the panels has been experimentally studied and has been taken into account in the numerical analyses. In (Pinelli et al., 1995) an advanced connection between architectural cladding panels and steel frames has been designed in order to dissipate energy in the engineered connection elements (Figure 1). The experimental behaviour of the connection is described in (Pinelli et al., 1996). The connection behaves like a passive dissipater, which provides lateral stiffness to the main steel structure because of the bracing effects of the cladding pan-

els. This way, the cladding walls are not only mass hung to the structure, but, at the same time, by providing a bracing action to the frame, they change the fundamental frequency of the building, in comparison with the bare resisting frame. The numerical results shown in (Pinelli et al., 1995) point out that the effectiveness of the cladding connection in the reduction of lateral displacements depends significantly on the ratio of the modified fundamental frequency of the structure and the critical frequency of the earthquake. The combination of the advanced cladding connections with a base isolator system should be more effective in high-rise steel frames. Some preliminary numerical studies show that this hybrid passive energy dissipation system provides a reduction of the base shear, earthquake input energy, and ductility demand in frame members (Dogan et al., 2004).

In precast concrete industrial buildings, the concrete cladding panels are often fixed to the structure by means of two support connections at the bottom, carrying the gravity loads, and two tie-back connections at the top of the panels, avoiding the out of plane movement of the panel (Figure 2). The four panel connections are designed to carry the same amount of the horizontal (parallel or normal to the panel) loads due to wind or earthquake. The cladding panels can be placed outside the structure, thus hiding it (case (a) in Figure 2), with the support system statically loaded in a plane perpendicular to the panel, or between the columns with the connection system, loaded in the plane of the façade, placed on the lateral edges of the panel (case (b) in Figure 2).

As previously mentioned, one of the problems related to precast RC panels, is that the behaviour of the supporting systems under earthquake actions is not well known, both with respect to their strength and ductility.

This paper aims at presenting finite element analyses (FEA) and the results of a wide experimental program of a particular steel support system connecting pre-cast concrete panels to concrete structures. This support system, called MT, has been designed and used since the '90s to carry only the concrete panel weight.

The numerical and experimental results presented in this paper provide a useful indications on the local behaviour of connection system loaded by a horizontal action and on diffusion phenomena in the concrete which cause the damage in the connected members. Furthermore the results can define an accurate force-displacement constitutive law of the connection, which should be an effective tool to study the seismic response of precast RC buildings.

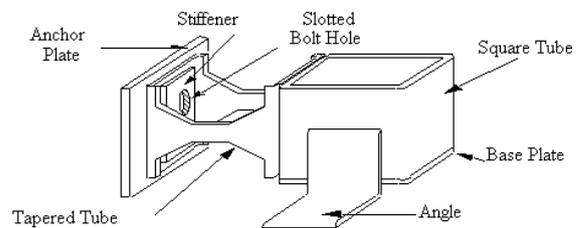
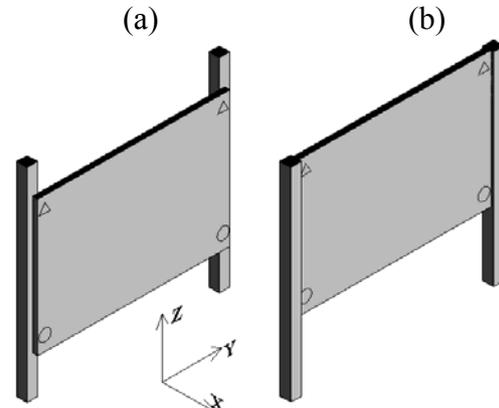


Figure 1. Ductile advanced cladding connection (Pinelli et al., 1995).



△tie-back connection ○support connection
Figure 2. Typical arrangement of connections for precast concrete panels.

2 THE CONNECTION SYSTEM

As shown in Figure 3, the MT support system is composed of three main components: the anchor steel plate with welded inserts embedded in the column (1); the anchor steel distributing plate (2), built in the precast panel; the steel bracket supporting the panel (3); a leveling bolt to adjust the vertical position of the panel during installation (4); two bolts transferring the dead load of the panel to the column (5). Figure 4 shows a typical application of the support connection system. Figure 5 shows the tie-back system, which is composed of an anchor channel built in the column (6) and a steel bracket embedded in the panel (7). Furthermore, the head of the levelling bolt and the distributing plate, built in the precast panel, are characterised by a saw-toothed surface in order to improve the friction between the bolt and the distributing plate.

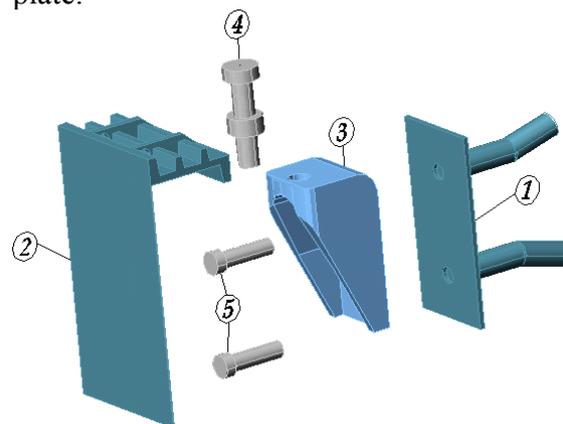


Figure 3. Components of the support system: exploded view.



Figure 4. Typical application of the support system.

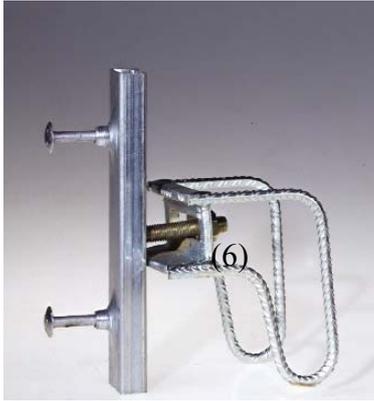


Figure 5. The tie-back connection.

3 NUMERICAL ANALYSIS

3.1 FE model

A three dimensional Finite Element (FE) model of the MT6 support system has been carried out by adopting 8-node tetrahedral elements. Over 12,000 elements have been used, with a dimension varying from 5mm to 50mm away from the diffusion zone (Figure 6). The restraints and the geometry of the model are shown in Figure 7: the panel restraints avoid out of plane movements, while the column is assumed to be fixed. The column is loaded by a vertical pressure equal to 8 MPa, in order to simulate the load of the roof of a typical one storey industrial precast building. A weight of 120 kN has been assigned to the panel, which is carried by two support systems. A horizontal displacement parallel (Y axis in Fig. 2) or normal (X axis in Fig. 2) to the building façade has been applied to the precast panel in order to evaluate the behaviour of the system loaded by seismic actions. All the details of the FE model can be find in the work of Metelli and Riva (2006).

The non linear analyses have been carried out with the FE program DIANA V.9.1. The non linear behaviour of the materials and of the contact surface between the panel and the support system have been considered. A rotating smeared crack approach has been used to model the concrete

cracking zone around the anchors of the support system.

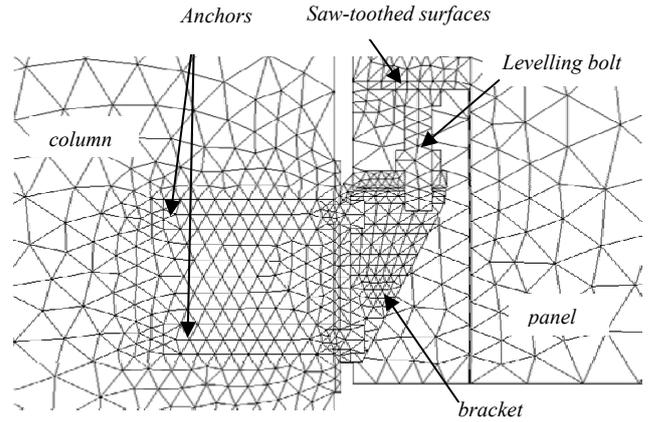


Figure 6. FE model of the assembled components of the support system.

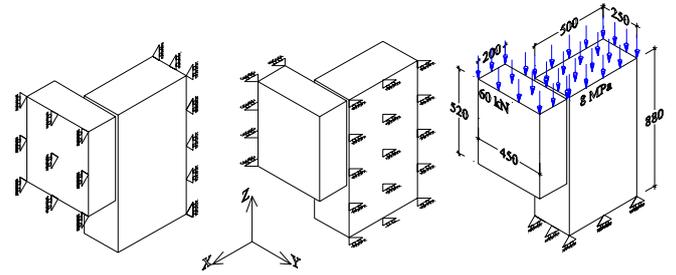


Figure 7. FE model: restraints and dimensions.

The friction between the levelling bolt head and the distributing plate has been modelled by linking the symmetrical nodes of the two elements with two springs parallel to the contact surface (x and y directions in Figure 8), thus allowing the support system to carry the horizontal loads. A no tension gap element allowed the vertical load to be transferred from the panel to the column. As shown in Figure 8, the stiffness k of the longitudinal spring depends on the geometrical characteristic of the toothed surfaces and has been calculated considering the force F_f causing the panel to slide along the toothed surface. Considering a slip metal to metal friction coefficient μ equal to 0.3 and the equilibrium along the tooth face, it is possible to define an equivalent interlocking coefficient ξ between the distributing plate and the levelling bolt (Fig. 8):

$$F_f \cos \alpha = \mu N \cos \alpha + N \sin \alpha \quad (1)$$

$$F_f = (\mu + \tan \alpha) N = \xi N \quad (2)$$

This way, by assuming a tooth angle α equal to 63° , the horizontal force F_f causing the panel slip is 2.3 times the vertical load N carried by the support system. Hence, the stiffness k of the longitudinal springs is given by equation (3):

$$k = \frac{N \xi}{n} \frac{1}{\delta_{0.1}} \quad (3)$$

where n =number of the linked nodes on the head of the bolt; $\delta_{0,1}$ =conventional slip of 0.1 mm corresponding to a longitudinal force equal to $2.3N$ (slip onset).

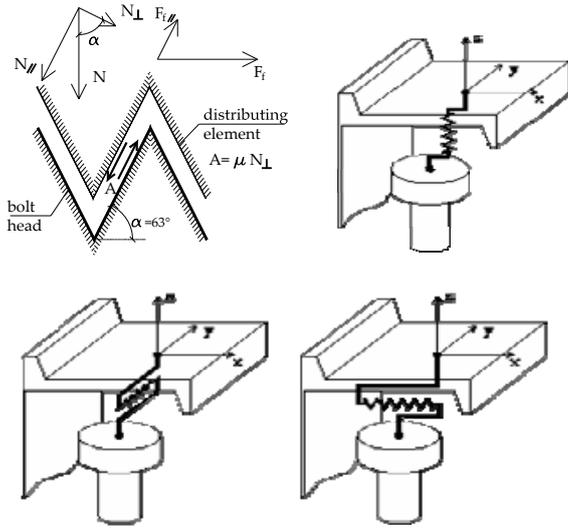


Figure 8. FE modeling of the friction between the distributing plate and the head of the leveling bolt.

A C35/45 concrete has been assumed with the compressive stress–strain relationship provided by EC2 (Eurocode 2, 2004). In tension, a linear cohesive crack model has been assumed with a tensile strength f_t equal to 3.2 MPa and a fracture energy G_f equal to 0.15 N/mm. The behaviour of the concrete far from the connection system has been assumed as linear elastic. For the steel components of the system, an elasto-plastic with linear hardening constitutive model and a Von Mises yield criterion has been assumed. The material properties are summarised in Table 1.

Table 1. Mechanical properties of the materials.

Concrete	f_c [MPa]	E_c [GPa]	ν	G_f [N/mm]	f_t [MPa]	ϵ_{c1} [%]	ϵ_{cu} [%]
	35	35	0.2	0.15	3.2	0.22	0.34
Steel		E_s [GPa]	ν	f_y [MPa]	f_u [MPa]	ϵ_{su} [%]	
Bolts		210	0.3	640	800	11	
Brackets and plates		210	0.3	350	520	29	
Anchors		210	0.3	500	500	12	

E : modulus of elasticity; ν : Poisson's ratio; f_c : compressive strength; f_t : tensile strength; ϵ_{c1} : strain at the peak compressive stress; G_f : fracture energy; ϵ_{cu} : ultimate strain; ϵ_{su} : ultimate plastic strain; f_y : yield strength; f_u : ultimate strength.

3.2 Numerical Results

As previously mentioned, the analyses have been conducted considering the case of the cladding panel external to the structure (case (a) in Fig. 2), applying a horizontal displacement to the panel along the Y axis (δ_y , parallel to the panel) or X axis (δ_x , normal to the panel). For each displacement direction, three analyses have been carried out by assuming a vertical load N in the panel equal to

100%, 66% and 33% of the panel weight in order to simulate a vertical component of the seismic action. The numerical results with normal displacement δ_x are representative also of the case with the cladding panel placed between the two columns (case (b) in Figure 2), being the imposed displacement parallel to the bracket and to the anchors embedded in the column.

The main results of the numerical analyses are summarized in Table 2 and Table 3, where the failure load $F_{y,u}$ or $F_{x,u}$, the lateral displacement $\delta_{y,u}$ or $\delta_{x,u}$ at failure, and the tangent stiffness k_y or k_x are shown for six loading configurations. The load-displacement curve is illustrated in Figure 9 for a vertical load equal to 60 kN and a longitudinal displacement δ_y . In order to evaluate the contribution of each component to the lateral deformation of the support system, the lateral absolute displacement of four points has been plotted: the displacement imposed to the panel (δ_1), the bolt head (δ_2) and the bracket (δ_3 and δ_4). The results of all other studied cases are discussed and reported in details by the work of Metelli and Riva (2006). Based on the results, the following main observations may be made:

- the value of the vertical load N governs the failure and the resistance of the support system due to the friction provided by the interlocking of the bolt head and the distributing plate: for longitudinal displacement with a vertical load reduced to the 33% of the panel weight (Table 2 – case ay3) or for normal displacement with a vertical load reduced to the 66% of the panel weight (Table 2 – case ax2 and ax3) the failure of the system is due to the panel slip. In all other cases, the failure is due to the bolt yield;
- the support system loaded in the direction normal to the panel shows a better performance than the one loaded in the longitudinal direction, both in term of resistance and stiffness. The resistance of the system varies from 46.0 kN for case ay3 to 95.1 kN for case ax1, while the stiffness varies from 27.5 kN/mm to 51.2 kN/mm;
- by representing the experimental results of each analysis by means of a bilinear curve, it is possible to define a ductility factor q as the ratio between the ultimate lateral displacement $\delta_{x,u}$ or $\delta_{y,u}$ and the yield displacement $\delta_{x,y}$ or $\delta_{y,y}$,

$$q = \frac{\delta_{x,u}}{\delta_{x,y}} \quad \text{or} \quad q = \frac{\delta_{y,u}}{\delta_{y,y}} \quad (4)$$

The ductility factor q varies from 1.45 in case ax2 to 13.5 in case ay1. The support system loaded in the longitudinal direction shows a good ductility due to the low torsional resistance of the bracket. The main lateral deformation δ of the system in the

case of the support system loaded in the longitudinal direction is caused by the bracket torsion, which accounts for approximately 50% of the total imposed displacement δ_y , while in the case of the support system loaded along the normal direction it is mostly due to the bolt deformation, accounting for 63.8% to 80.3% of the total imposed displacement δ_x .

- The damage of the concrete is limited to the zone around the anchors as shown by the crack pattern of the column at failure: the average crack width around the anchors is approximately equal to 0.30 mm.

Table 2. Numerical results with an applied displacement in the plane of the panel (Y axis).

Case	N	$F_{y,u}$	$\delta_{y,u}$	k_y	Panel δ_1 - δ_2	Bolt δ_2 - δ_3	Bracket δ_3 - δ_4	Column δ_4	Failure
MT6	[kN]	[kN]	[mm]	[kN/mm]	[mm]	[mm]	[mm]	[mm]	
ay1	60	50.7	26.0	27.5	0.10 0.4%	5.20 20%	12.94 49.8%	7.75 29.8%	Bolt yield
ay2	40	50.7	26.0	27.5	0.10 0.4%	5.20 20%	12.94 49.8%	7.75 29.8%	Bolt yield
ay3	20	46.0	7.00	25.5	0.81 11.6%	1.19 17.0%	3.24 46.4%	1.75 25.0%	Panel slip

Table 3. Numerical results with an applied displacement in the plane of the panel (Y axis).

Case	N	$F_{x,u}$	$\delta_{x,u}$	k_x	Panel δ_1 - δ_2	Bolt δ_2 - δ_3	Bracket δ_3 - δ_4	Column δ_4	Failure
MT6	[kN]	[kN]	[mm]	[kN/mm]	[mm]	[mm]	[mm]	[mm]	
ax1	60	95.1	3.80	49.6	0.21 5.5%	3.05 80.3%	0.44 11.6%	0.10 2.6%	Bolt yield
ax2	40	92.0	2.90	51.2	0.24 8.3%	2.20 75.9%	0.37 12.7%	0.09 3.1%	Panel slip
ax3	20	46.0	0.90	51.0	0.13 14.6%	0.57 63.8%	0.15 16.9%	0.04 4.7%	Panel slip

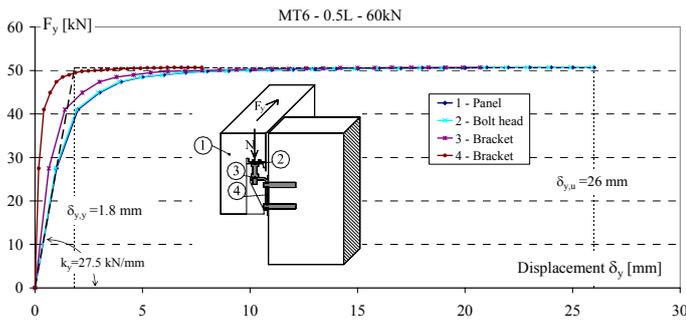


Figure 9. Longitudinal Load F_y – Lateral Displacement δ_y , with a dead load equal to 60 kN.

4 EXPERIMENTS

4.1 Test Set-up

The aims of the experimental program are to evaluate the stiffness, the ductility and the energy dissipation characteristic of each type of MT support system as well as to investigate the role of the vertical load and of the extension length of the levelling bolt on the lateral behaviour of the support system. The experimental tests concern 11 proto-

type specimens representing a column portion linked by the support system to a precast panel portion. Three batches of tests have been carried out at the P.Pisa Laboratory of the University of Brescia in order to investigate the behaviour of different support system, called MT4, MT6 and MT9, where the number refers to the nominal vertical load transferred by the support system to the column (half weight of the concrete panel) (see Table 4). The MT4 and the MT9 series consist of 3 specimens, each one with a different vertical load, while the MT6 series consists of five specimens with a varying vertical load and different extension length of the levelling bolt. All the specimens tested had a 250x250 mm column cross section and a column length equal to 1250 mm. The panel portions were 200 mm depth, 300 mm wide and 480÷580 mm high. The geometry of the tested specimens and the mechanical characteristic of concrete and reinforcing steel are shown in Figure 10, while Figure 11 and Figure 12 show the experimental setup adopted. The bench consists of a 1800x2150 mm steel ring frame (a) designed in order to allow the panel to move in its plane and to avoid any movement of the column portion. For all the tests, the vertical load N , varying from 13 kN to 90kN, was applied and kept constant during the test by means of two tendon rods (b). The horizontal displacement was applied at the top of the panel by means of threaded steel bars (c) (16 mm in diameter), which are instrumented with strain gauges to measure the applied load F . As in the numerical analysis, the column is loaded by a vertical pressure equal to 8 MPa, by means of four 24 mm diameter steel bars (d).

The specimens were subjected to eight cyclic horizontal displacement histories by imposed transverse displacement δ of increasing amplitude (Figure 13) with a step increment of about 0.1 mm/s. In order to evaluate the contribution of each component to the lateral deformation of the support system, the lateral absolute displacement of four points were measured: the displacement imposed to the panel (δ_1), the bolt head and bottom displacements (δ_2 , δ_3), and the bracket displacement (δ_4) (Figure 13). In order to verify that the in plane panel rotation was correctly controlled by the two vertical tendons, the horizontal displacement δ_5 of the panel bottom (b) was also measured.

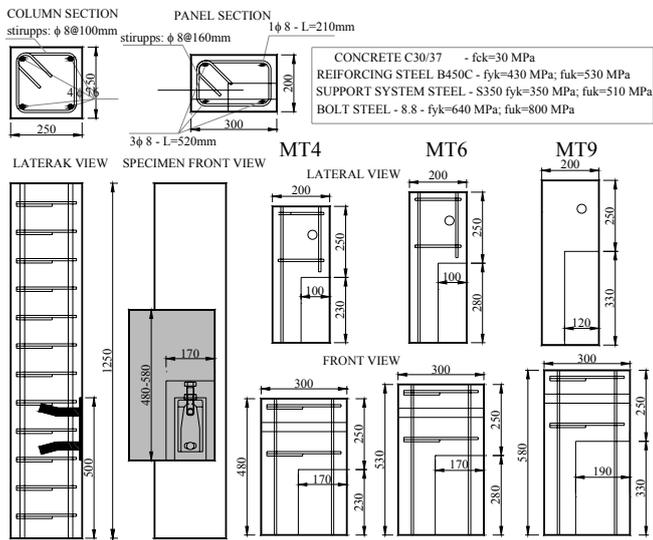


Figure 10. Dimensions of the specimens and mechanical characteristic of the materials.

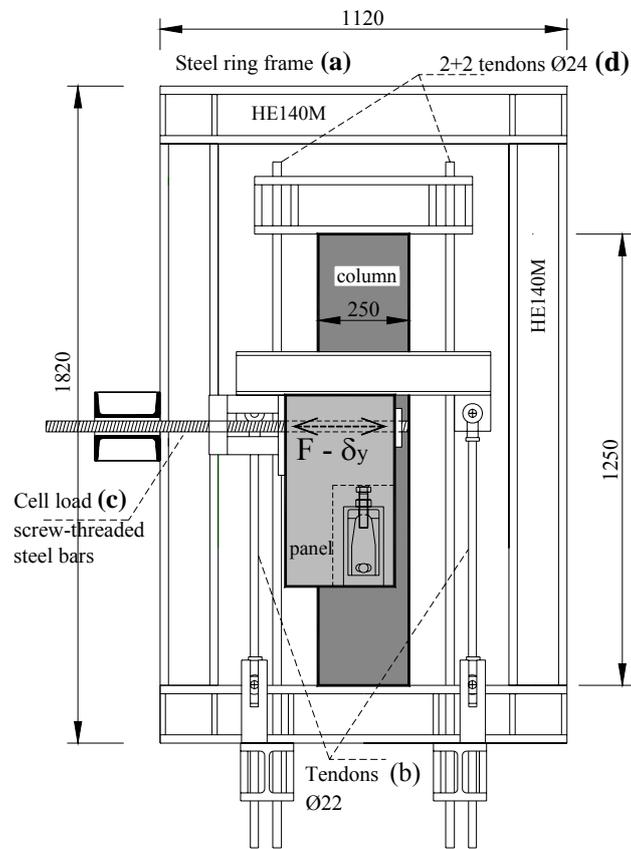


Figure 11. Experimental set-up: steel frame and specimen.

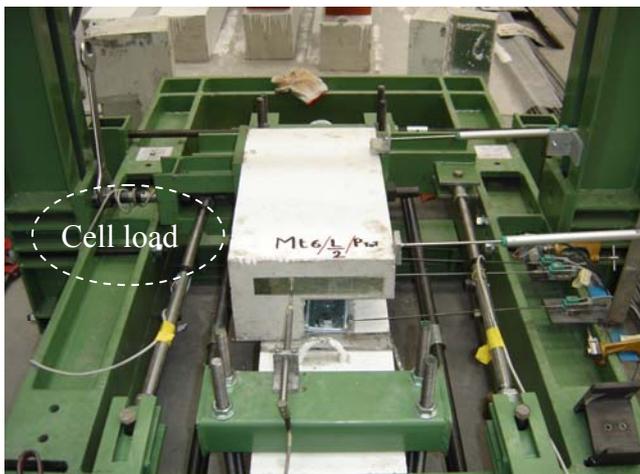


Figure 12. Picture of the bench with MT6 support system.

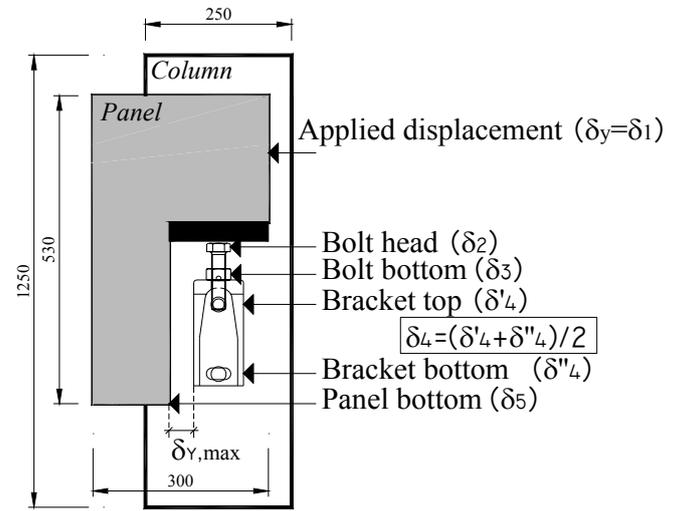


Figure 13. Measured points.

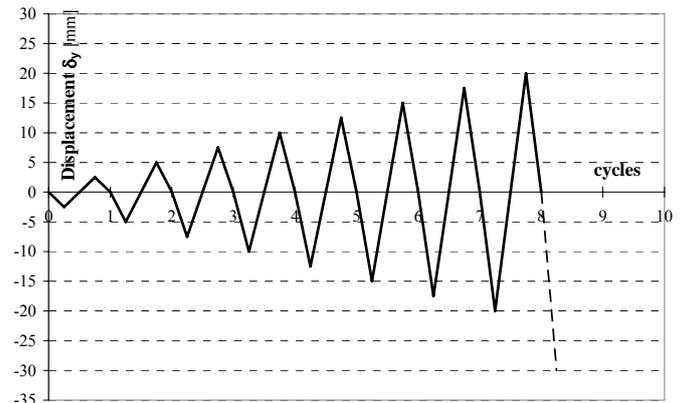


Figure 14. Loading history.

4.2 Experimental Results

The experimental horizontal load F – displacement δ_l curves of five tested specimens are shown in Figure 16 while Table 4 show in details the main results of each test. For the MT6 support system with 60kN vertical load and leveling bolt at the middle extension (0.5L), the comparison between the numerical and experimental results is also shown. On the basis of the results, the following main observations can be drawn:

- the shear resistance $F_{y,u}$ of the support system and failure are strongly affected by the leveling bolt extension length. For the MT6 support system the failure load decreases from 51.0 kN in the case of completely screwed leveling bolt (MT6_0.0L_60kN) to 18.8 kN (MT6_1.0L_60kN) in case of completely unscrewed leveling bolt. At the same time, longer extension length of the leveling bolt allows the system to develop larger lateral deformation $\delta_{y,u}$ due to the bolt plastic deformation (see pictures in Figure 16);
- in case of short leveling bolt extension or small vertical load, the failure is due to the panel slip with a severe damage of the saw-toothed surfaces of bolt and plate. In the case of completely screwed bolt (MT6_0.0L_60kN) the lateral deforma-

tions of the bolt and the panel sliding correspond respectively to the 17% and 68% of the total lateral deformation. On the contrary the completely unscrewed bolt allows a lower panel slip (37%) and a larger bolt deformation (53%);

- the vertical load N affects the behaviour of the support system both in terms of components deformation and ultimate load. A decrease of 67% of the vertical load N causes a reduction of shear resistance F_u equal to 9% for MT6 support system, 28% for MT9 and 36% for MT4. As lower the vertical load is, as lower the friction between the distributing plate and the head of the leveling bolt becomes, causing the damage of the saw-toothed surfaces and a failure by panel slip. The teeth damage and the slip panel are pointed out by the horizontal path in the F - δ curves;

- the support systems showed good energy dissipation properties with very high lateral displacements and without loosing the capability to sustain the vertical load. The specimen Mt6_0.0L_60kN shows a larger energy dissipation capacity than the other specimens. Furthermore most of the energy is dissipated by friction, so that it increases with the vertical load. Figure 15 shows the comparison of the specific dissipated energy, normalized with respect to the vertical load N , among all of tested specimens;

- in Figure 14 the comparison between numerical monotonic and hysteretic experimental curves is presented for the MT6_0.5L_60kN specimen. It is worth pointing out that the numerical analyses catch the same failure mode than the tests with a shear ultimate load equal 1.20 time the experimental value. Nevertheless, the numerical initial stiffness k of the system is double than the experimental one because a conventional infinitely high stiffness (given by eq. (3)) of the longitudinal springs which connect the bolt head and the distributing plate of the panel is assumed in the FE model;

- the tests pointed out that the damage in the supporting system is localized either in the saw-toothed surfaces or in the plastic deformation of

the leveling bolt. Very little damage pattern was observed in concrete elements: in the MT6_0.0L_60kN, MT6_0.5L_60kN, MT9_0.5L_90kN specimens a crack may be observed, departing from the right top of the bracket toward the side column with an angle approximately equal to 50°.

Table 4. Summary of the experimental results.

	N	k_y	$F_{y,u}$	$\delta_{y,u}$	Panel			E	Failure
					sliding $\delta_1 - \delta_2$	Bolt $\delta_2 - \delta_3$	Bracket $\delta_3 - \delta_4$		
	[kN]	[kN/mm]	[kN]	[mm]	[mm]	[mm]	[mm]	[kJ]	
MT6_1.0L	60	10	18.8	34	12.5 (37%)	18.0 (53%)	2.9 (8%)	1.43	Bolt yield
MT6_0.0L	60	23	51.0	26	17.5 (68%)	4.4 (17%)	4.8 (18%)	2.79	Panel slip ^{a)}
MT6_0.5L	60	11	41.0	26	11.6 (45%)	5.8 (22%)	8.6 (33%)	2.53	Bolt yield
MT6_0.5L	40	15	43.3	29	16.2 (56%)	5.7 (20%)	7.1 (24%)	2.21	Bolt yield
MT6_0.5L	20	15	37.5	29	21.4 (74%)	2.2 (8%)	5.4 (18%)	1.83	Panel slip ^{a)}
MT9_0.5L	90	30	43.8	35	11.2 (32%)	15.5 (44%)	8.0 (23%)	2.71	Bolt yield
MT9_0.5L	60	25	46.8	35	8.7 (25%)	14.1 (40%)	15.1 (43%)	2.19	Bolt yield
MT9_0.5L	30	20	31.4	30	13.7 (45%)	8.0 (26%)	14.1 (47%)	1.67	Panel slip ^{a)}
MT4_0.5L	40	12	30.5	12	3.1 (26%)	4.3 (36%)	4.7 (40%)	0.71	Panel slip ^{a)}
MT4_0.5L	27	13	20.3	12	6.2 (50%)	3.8 (30%)	1.6 (13%)	0.68	Panel slip ^{a)}
MT4_0.5L	13	11	19.3	16	9.4 (59%)	3.0 (19%)	2.9 (18%)	0.58	Panel slip ^{a)}

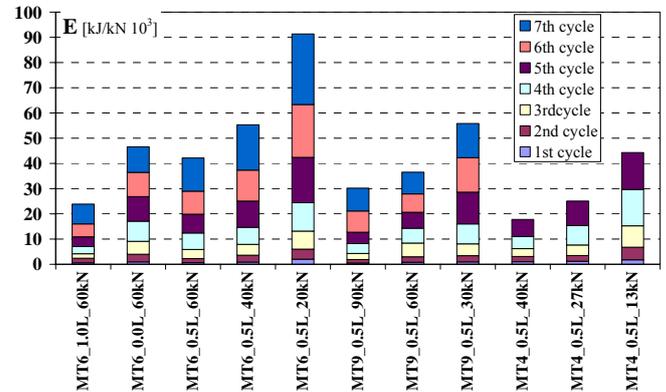


Figure 15. Dissipated energy.

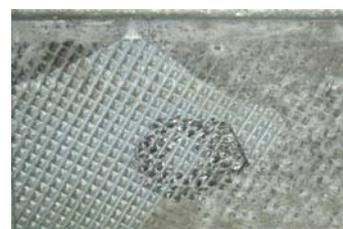
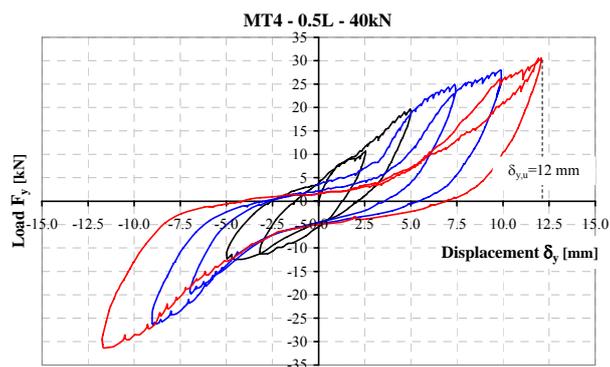
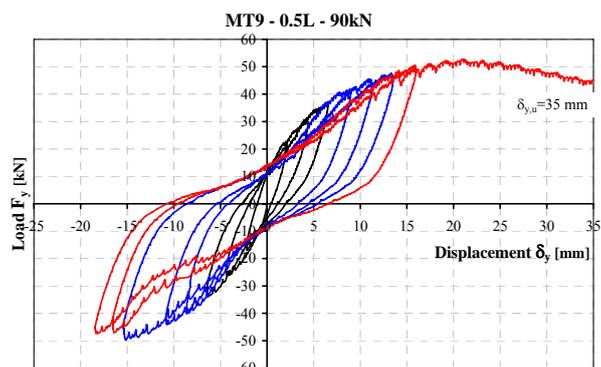
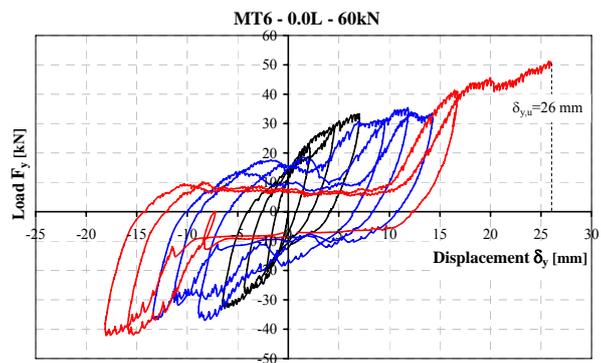
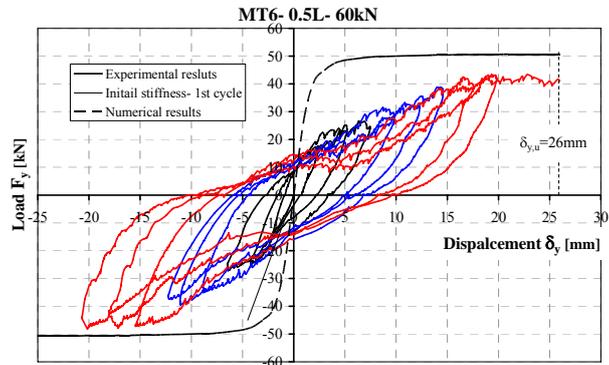
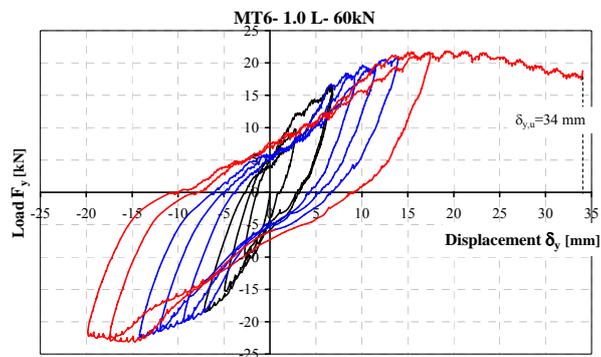


Figure 16. Force F_y –displacements δ_y curves and pictures at failure.

5 DESIGN CONSIDERATIONS

On the basis of the experimental results, some design considerations may be proposed concerning the MT support system behaviour. According to Eurocode 8 (§4.3.5.2), the non structural elements, as well as their connections and attachments or anchorages, shall be verified for the seismic design situation by applying a horizontal force F_a , acting at the centre of mass of the non-structural element, which is defined as follows:

$$F_a = (S_a \cdot W_a \cdot \gamma_a) / q_a \quad (5)$$

where W_a is the weight of the element, S_a is the seismic coefficient applicable to non-structural elements, (defined by the eq. (6)), γ_a is the importance factor of the element assumed equal to 1, q_a is the behaviour factor of the element assumed equal to 2. The seismic coefficient S_a may be calculated using the following expression:

$$S_a = \alpha \cdot S \cdot \left[3(1+z/H) / (1+(1-T_a/T_1)^2) - 0.5 \right] \quad (6)$$

where α is the ratio of the design ground acceleration (PGA) on type A ground, a_g , to the acceleration of gravity g ; S is the soil factor; T_a is the fundamental vibration period of the non-structural element; T_1 is the fundamental vibration period of the building in the relevant direction; z is the height of the non-structural element above ground; and H is the building height. Assuming the ratio T_a/T_1 equal to 0 and z/H equal to 1, a PGA equal to 0.35g and $S=1.4$, equal to the most severe PGA and soil conditions for European sites, the seismic coefficient S_a and the seismic action F_a are given by the following expressions:

$$S_a = 0.35 \cdot 1.4 \cdot \left[3(1+1) / (1+(1-0)^2) - 0.5 \right] = 1.225 \quad (7)$$

$$F_a = (S_a \cdot W_a \cdot \gamma_a) / q_a = \frac{1.23 \cdot W_a \cdot 1}{2} \cong 0.61 W_a \quad (8)$$

As previously mentioned, the label of each type of the MT brackets, is followed by a number which represents the half weight of the supported panel. Assuming that the seismic action of the panel F_a is equally distributed among the two brackets and two tie-back systems, the seismic action on each connection system F_{as} is equal to $F_a/4$. So that it is possible to calculate the safety factor ψ for each tested bracket, as shown in Table 5. In all cases, the seismic action is smaller than the bracket strength, and the safety factor $\psi > 1.5$, with the exception of the MT6-1.0L-60 (levelling bolt completely unscrewed and a vertical load equal to 60kN) and MT9-0.5L-30 (levelling bolt partially screwed and a vertical load equal to 30kN) specimens for which the safety factor is smaller than 1.5, but larger than 1.

Regarding the serviceability limit state, it is not possible to verify each analyzed support system because it depends on the concrete panel dimension. Assuming a panel height h equal to 2.50 m, the MT6 and MT9 support system can develop deformations consistent with an interstorey drift of $0.01h$.

Table 5. Verification of the MT support system at ultimate limit state.

Support system	F_u	F_{as}	ψ
MT6-0.5L-60 kN	41.0	18.3	2.24
MT6-0.5L-40 kN	43.3	18.3	2.36
MT6-0.5L-20 kN	37.5	18.3	2.05
MT6-1.0L-60 kN	18.8	18.3	1.03
MT6-0.0L-60 kN	51.0	18.3	2.79
MT9-0.5L-90 kN	43.8	27.5	1.59
MT9-0.5L-60 kN	46.8	27.5	1.70
MT9-0.5L-30 kN	31.4	27.5	1.14
MT4-0.5L-40 kN	30.5	12.2	2.50
MT4-0.5L-27 kN	20.3	12.2	1.66
MT9-0.5L-13 kN	19.3	12.2	1.58

6 CONCLUSIONS

The behaviour of a typical support system for pre-cast panels has been analysed both numerically and experimentally to assess its capacity to resist seismic (i.e. transverse) forces.

Although the support system was not originally designed to carry lateral loads, the results show a good performance of the support bracket, both in terms of resistance, energy dissipation and ductility with a very limited damage in the concrete elements.

The transverse force transmission of the supporting system is ensured by the enhanced friction due to the saw toothed surface of the levelling bolt head and of the corresponding plate.

Further numerical analyses involving the FE modelling of the saw-toothed surfaces should be carried out in order to fine-tune the constitutive model of the levelling bolt-to-plate interface.

Finally, based on the FE analysis and experimental results, a simplified constitutive model of the support system has to be developed in order to allow the study of the seismic response of industrial buildings taking into account the interaction between the concrete structure and the cladding panels.

ACKNOWLEDGMENTS

The authors gratefully acknowledge the support of Edilmatic srl (Pegognaga (MN), Italy) for financing this research project on concrete panel support systems. The cooperation of Gino Antonelli and Luca Scartozzoni in carrying out the numerical analysis and Andrea Facchinetti in carrying out the tests are gratefully acknowledged.

REFERENCES

- Biondi, S., Colangelo, F., Nuti, C. 2000 La Risposta Sismica dei Telai con Tamponature Murarie, *CNR- Gruppo Nazionale pe la Difesa dei Territori*.
- De Matteis, G. 2005. Effect of Lightweight Cladding Panels on the Seismic Performance Of Moment Resisting Steel Frames, *Engineering Structures*, 27, n° 11, 1662-1676.
- Cohen, J. M., Powell, G. H. 1993. A Design Study of an Energy-Dissipating Cladding System, *Earthquake Engineering and Structural Dynamics*, 22, 617-632.
- Diana v. 8.1.2. 2003. User's manual. TNO DIANA BV, Delft.
- Dogan, T., Goodno, B. J., Craig, J. I. 2004. Hybrid Passive Control In Steel Moment Frame Buildings, *Proceedings of the 13th World Conference on Earthquake Engineering, Paper 2387*.
- Eurocode 2: Design of concrete structures - Part 1-1: General Rules, and Rules for Buildings, EN 1992-1-1:2004, *European Committee for Standardization*, December 2004.
- Eurocode 8: Design of structures for earthquake resistance - Part 1: General Rules, Seismic Actions and Rules for Buildings, PrEn 1998-1, *European Committee for Standardization*, December 2003.
- Henry, R. M., Roll, F. 1986. Cladding-Frame Interaction, *Journal of Structural Engineering*, 112, n° 4, 815-834.
- Goodno, B. J., Craig, J. I., Dogan, T., and Towashiraporn, P. 1998. Ductile Cladding Connection Systems for Seismic Design, *Building and Fire Research Laboratory, NIST, Report GCR 98-758*.
- Pinelli, J. P., Craig, J. I., Goodno, B. J. 1995. Energy-Based Seismic Design of Ductile Cladding Systems, *Journal of Structural Engineering*, 121, n° 3, 567-578.
- Pinelli, J. P., Moor, C., Craig, J. I., Goodno, B. J. 1996. Testing of Energy Dissipating Cladding Connections, *Earthquake Engineering and Structural Dynamics*, 25, 129-147.
- Mehrabi, A. B., Shing, B. P., Schuller, P., Noland, J. 1996. Experimental Evaluation of Masonry-Infilled RC Frames, *Journal of Structural Engineering*, 122, n° 3, 228-237.
- Mehrabi, A. B., Shing, B. P. 1997. Finite Element Modelling of Masonry-Infilled RC Frames, *Journal of Structural Engineering*, 123, n° 5, 604-613.
- Metelli, G. and Riva, P. 2006. Numerical analyses of a support system for pre-cast concrete panels. *Proceedings of the First European Conference on Earthquake Engineering and Seismology*, Geneve, Switzerland, 3-8 September 2006, paper n° 612.)
- Paulay, T. and Priestley, M.J.N. 1992. Seismic Design of Reinforced Concrete and Masonry Buildings, *John Wiley & Sons*, New York.