



## SEISMIC BEHAVIOUR OF GROUTED SLEEVE CONNECTIONS BETWEEN FOUNDATIONS AND PRECAST COLUMNS

Nicola BURATTI<sup>1</sup>, Lorenzo BACCI<sup>2</sup> and Claudio MAZZOTTI<sup>3</sup>

### ABSTRACT

The paper presents the results of experimental tests on the cyclic behaviour of a column-foundation for precast concrete elements. The tested connection is built using steel tubes embedded in the footing, in which the column rebars are grouted. In particular the rebars are grouted with high strength mortar, in order to guarantee structural continuity between connected elements. The experimental results from the test carried out on the precast column footing joint were compared to the response of a reference cast in place specimen that was tested in the same experimental campaign. The behaviour of the tested specimens was analysed from different points of view: stiffness, strength, ductility and energy dissipation.

On the basis of experimental data, numerical models were developed to simulate the behaviour of elements tested.

*KEYWORDS: Precast Concrete Structures, Column Footing Joints, Test Results.*

### INTRODUCTION

Precast structures are widely used because they allow to minimize on-site construction time. Moreover precast and prestressed concrete building structures, assembled from high-quality, plant-produced products, provide superior flexibility for achieving the required degrees of energy efficiency, sustainability, and durability. In Italy precast concrete has been widely used to realize warehouses and industrial buildings. The structural configuration of these buildings is normally very simple, in fact it consists of cantilever columns connected by simply supported. The connection between columns and footings is often realized by inserting and grouting columns in precast concrete pocket foundations.

Recent collapses of precast industrial buildings due to Emilia Earthquake of May 2012 have highlighted how the effectiveness of these systems could be compromised. In effect the described structural typology has a low capacity of dissipating seismic energy through developing inelastic deformation, which are concentrated only at the base of columns. Moreover connections between main structural elements are simple bearings which work by gravity load friction, these types of connections are not suitable for the transmission of seismic actions. Therefore the need to guarantee a suitable speed of construction, but at the same time, to face seismic actions, have led to develop new precast structural systems, in which joints could guarantee continuity between main structural members, improving the static redundancy of the structure. This result can be effectively

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<sup>1</sup> Assistant Professor, Ph.D., DICAM - University of Bologna, Bologna, Italy, nicola.buratti@unibo.it

<sup>2</sup> Research Assistant, CIRI – University of Bologna, Bologna, Italy, lorenzo.bacci@unibo.it

<sup>3</sup> Associate Professor, Ph.D., DICAM - University of Bologna, Bologna, Italy, claudio.mazzotti@unibo.it

achieved with properly engineered connection details and with the use of field-cast concrete (Belleri and Riva, 2012).

The present work discusses the experimental behaviour of the column-foundation connection of the precast system developed by APE s.p.a.<sup>®</sup>, which is illustrated in Figure 1. The main feature of this system is that beam-column nodes and column-foundation connections cast in place, in order to realize moment resisting frames. The connection under investigation is realized introducing dowel bars, which come out from the bottom of the column, into steel boxes placed in the footing which are then filled with high strength mortar (Figure 2).

The paper describes the results cyclic-tests on two columns the first specimen was cast in place, while the second was precast and connected to the foundation as previously described. The experimental test showed the effectiveness of the base connection of the prefabricated column. Simple FEM models were also developed which allowed to simulate with good accuracy the experimental behaviour of the columns tested.



Figure 1. APE precast framing system



Figure 2. Column-foundation connection

## **EXPERIMENTAL PROGRAM AND TEST SETUP**

The present paper discusses the results of two different tests, aimed at analysing the effectiveness of the column-foundation connection previously described. The first test was carried out on a cast in place column, used as reference, and the second on a precast column connected to the foundation by grouting the rebars into corrugated tubes. Both columns had the same cross section of 400 mm × 400 mm and an height of 2330 mm. Both the columns tested

had also the same reinforcement: 8 longitudinal bars with 26 mm diameter and 8 mm diameter stirrups with a spacing of 50 mm. It is important to notice that at the bottom end of the precast

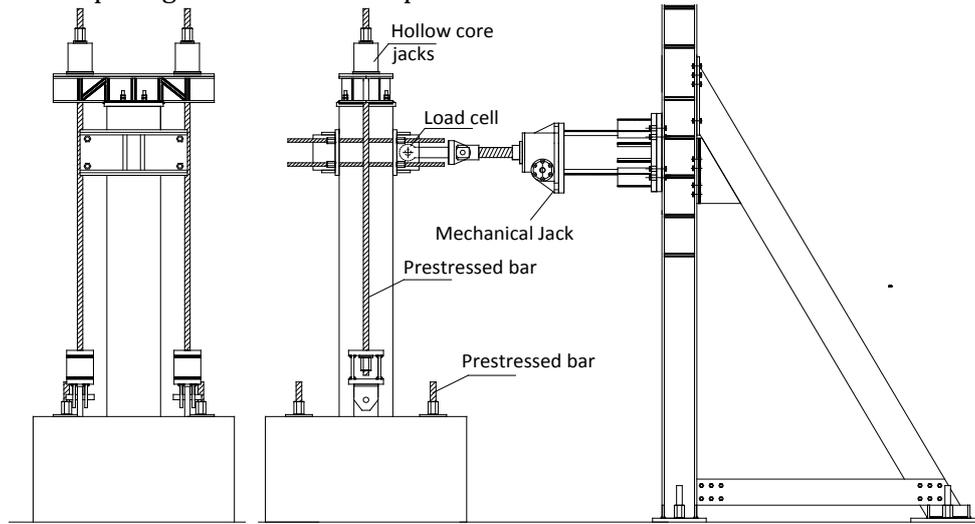


Figure 3. Experimental setup

column there is a steel tube (diameter 133 mm and thickness 16 mm) which is used in order to centre the column during the assembly phase (see Figure 2).

Figure 3 shows the experimental setup. The foundation of the specimen was connected to the strong-floor of the laboratory using 4 prestressed steel bars. Horizontal displacements were imposed at the top of the column using a electro-mechanical jack - with a maximum payload of 500 kN and a maximum stroke of 500 mm. The jack was supported by a steel reaction frame. During the tests an axial force of 500 kN , which represents gravity loads compatible with the structural framing system, was applied on the column using hollow-core hydraulic jacks and prestressed bars hinged at the base of the column.

During the tests sequentially increasing cyclic horizontal displacements were applied at the top of the column. For each displacement amplitude three complete cycles were repeated. Figure 4 and Table 1 report the lateral loading sequence and loading speed for each level of displacement. The horizontal loads and displacements were recorded, respectively, by a load cell and by two displacement transducers placed at the top of the column. The first horizontal displacement transducer was a potentiometer with a measuring of 125 mm, it was disposed to measure more accurately the lowest levels of displacement, while the second had a measuring range of 500 mm and was used to record largest displacements. Three pairs of displacement transducers were used to evaluate the column base rotation and average column curvatures (Figure 5). The base rotation was measured by two LVDTs at the base of column, while potentiometric transducers, which were disposed at two consecutive levels of the column base, were used to measure the curvatures over a length of 300 mm.

Horizontal displacement [mm]	Drift [%]	Loading rate [ $\mu\text{m/s}$ ]
5	0.3	200
10	0.5	250
18.5	1	1 <sup>st</sup> cycle 250; 2 <sup>nd</sup> cycle 300; 3 <sup>rd</sup> cycle 350
36.8	2	1 <sup>st</sup> cycle 400; 2 <sup>nd</sup> cycle and 3 <sup>rd</sup> cycle 450
55.2	3	500
92	5	600
129	7	600
200	11	600

Table 1. Loading protocol and loading rate.

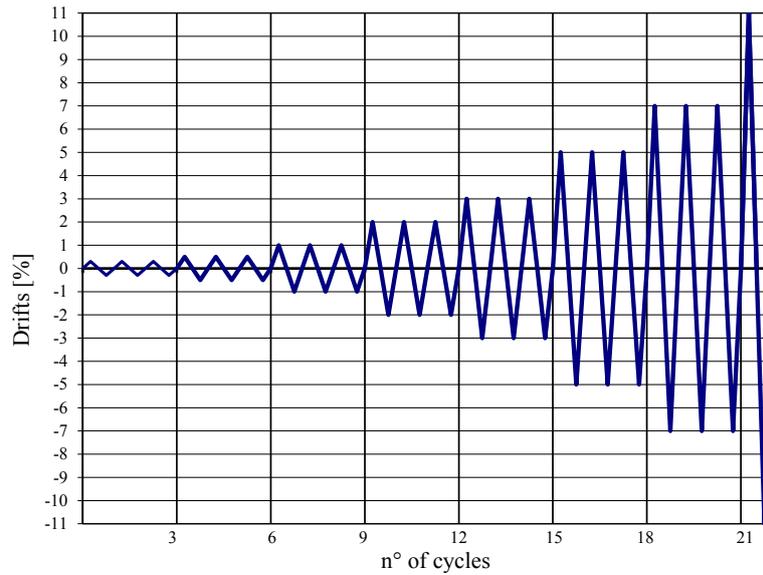


Figure 4. Lateral loading sequence



Figure 5. Transducers to measure base rotation and curvatures disposed on opposite sides of columns.

Further displacement transducers were used in order to monitor the possible displacements and rotations of the foundation block.

## **MATERIAL PROPERTIES**

Table 2 reports the material properties of the rebars which were measured by performing tensile tests. Table 3 lists the compression strength and the elastic modulus of the concretes used to cast the two columns testes.

Material properties of reinforcing bars				
	$f_u$ [MPa]	$f_y$ [MPa]	$A_{gt}$ [%]	E[MPa]
Cast in place	637.2	519.8	21.4	216857
Precast	628.8	520.1	19.2	205631

Table 2. Reinforcing bars mechanical properties

Material properties of concrete		
	$f_{cm}$ [MPa]	E [MPa]
Cast in place	43	34174
Precast	64	38453

Table 3. Concrete mechanical properties

### CYCLIC TESTS RESULTS

Figure 7 reports the experimental horizontal load versus drift curves for the cast in place column. The lateral load reported in Figure 7 was corrected by considering the effect of the horizontal component of axial force in the deformed configuration.

The system started to exhibit a strongly non-linear behaviour during the first cycle at 2% drift. In fact at this level of deformation one may notice the reduction of stiffness due to the yielding of the longitudinal bars. The corresponding load was about 200 kN while the maximum load attained during the test was equal to 230 kN. In the following cycles the effect of plastic deformation is clearly evident. Pinching may also be observed, most probably due to bar slip. At the end of the third cycle at 5% drift (displacement + 92 mm), the fracture of one of the longitudinal bars was observed. This rupture made all the subsequent cycles clearly unsymmetrical.

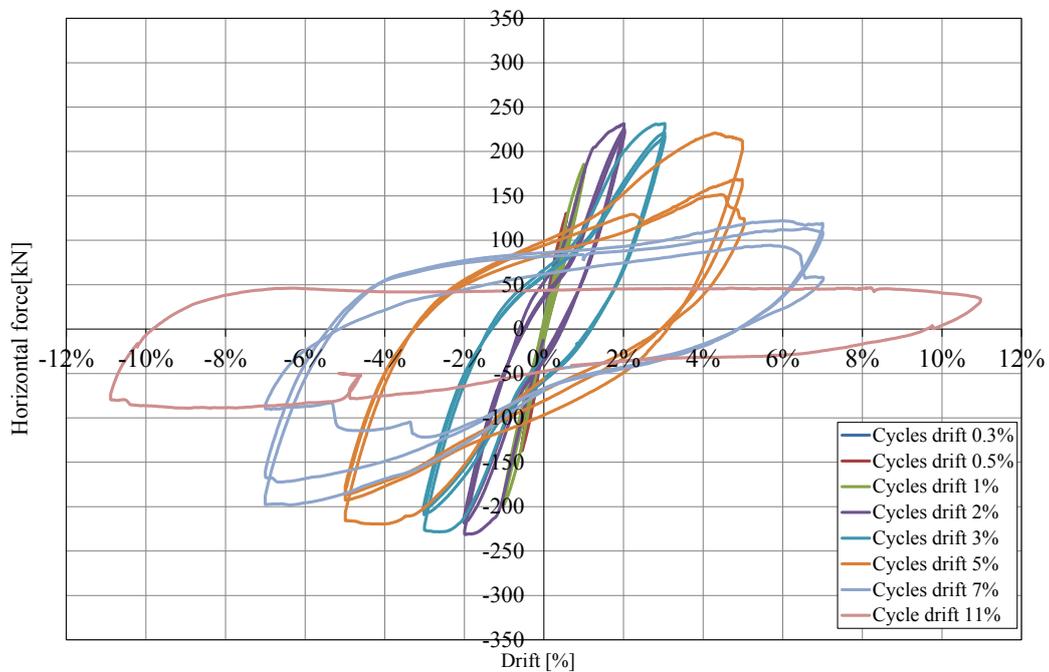


Figure 7. Experimental horizontal load versus drift curves for the cast in place specimen.

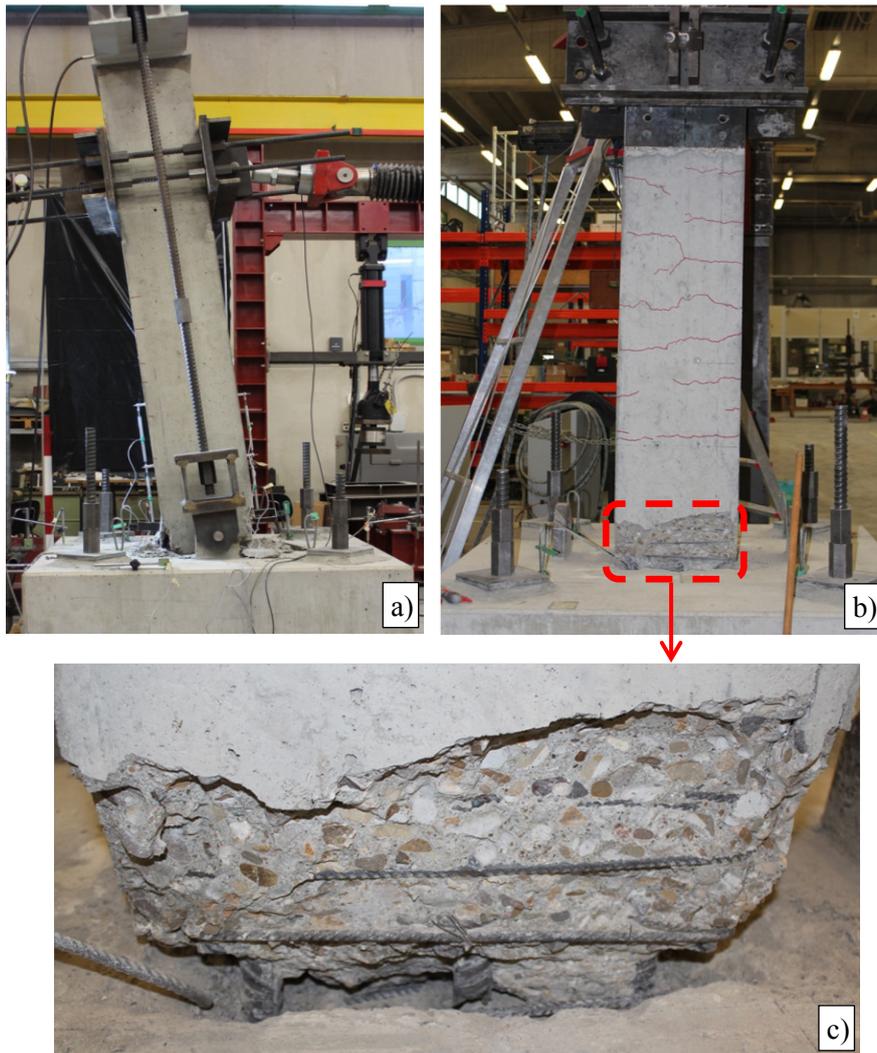


Figure 8. Damage observed on the cast in place column at the end of the test: a) maximum drift set during the test; b, c) damage at the base of the column.

Figure 8 reports the damage observed on the column at the end of the test. Inelastic deformations are concentrated at the base of column, in correspondence of the interface between column and foundation. The extent of the plastic region was limited. In fact high values of rotation have been registered at the column base, while deformations does not propagate over the height of column.

Figure 9 shows the force-drift curves of the prefabricated column. The precast system started to have a strongly non-linear behaviour during the first cycle at 2% of drift (+36 mm displacement), with a clear reduction of stiffness due to the yielding of the longitudinal bars, the applied load was about 250 kN. Comparing these curves with results obtained from cast in place specimen we can observe how globally the pinching effect is less evident. During cycles at 3% drift the system reached its maximum strength, with a horizontal force of approximately 300 kN. The Column, at least until the end of the first cycle to 7% of drift (+ 129 mm displacement), continued to present a stable and symmetrical behaviour, without sensitive reductions of resistance. The failure was reached at last cycle with a drift of 11% and a displacement level of 200 mm, with buckling of longitudinal bars. The analysis of the specimen at the end of the test (Figure 10) shows that the portion of column that experienced inelastic deformation was larger than the one in the cast in place specimen.

Figure 10 also suggests that the steel tube (see Figure 2), used to centre the column, increased the stiffness and strength of the base section of the column, and shifted up, from the base of the column, the critical section, in which bars reached their ultimate strength for buckling.

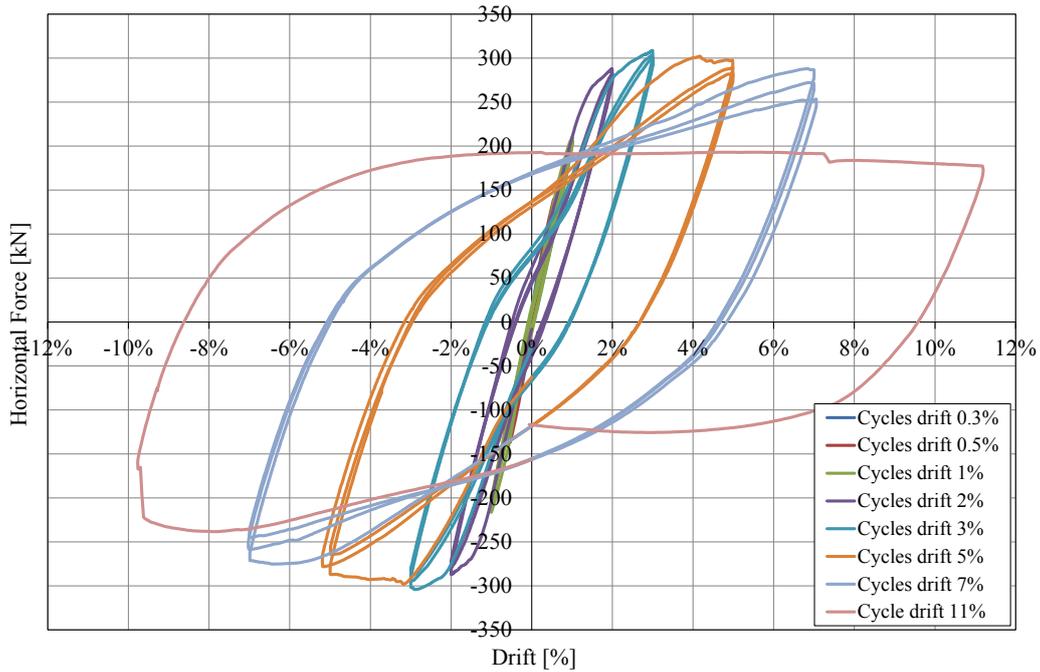


Figure 9. Experimental horizontal load versus drift curves for the precast specimen.

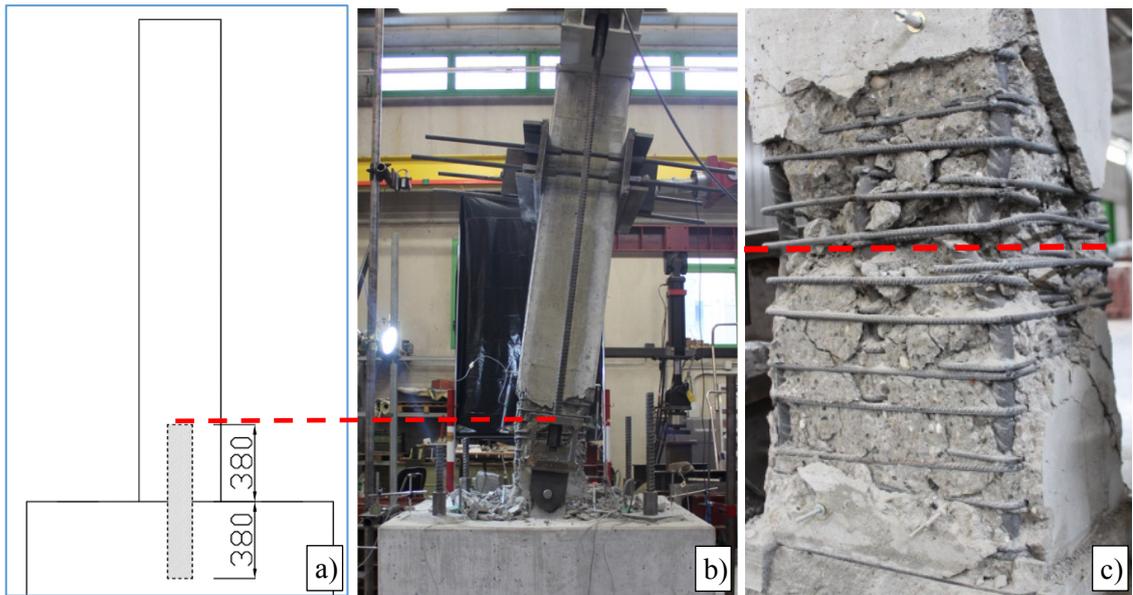


Figure 10. Damage observed on the prefabricated column at the end of the test: a) length of the steel tube at the base; b) effect of the steel tube on the stiffness; and c) damage at the base of the column. Quotes in mm.

At the end of the tests the available ductility of the systems was evaluated by defining an index of ductility in terms of displacement ( $\mu_D$ ). This index was calculated as the ratio of the ultimate displacement ( $\Delta_u$ ) and yield strength ( $\Delta_y$ ).

The ultimate displacement was evaluated according to two different criteria (Park, 1989): in the first one  $\Delta_u$  is calculated as the displacement at which the first reinforcement failure occurred; in the second one the ultimate displacement was defined as the point on envelope curve which corresponded to a reduction of the maximum strength equal to 20%.

The yield displacement  $\Delta_y$  was evaluated considering an equivalent elastic-plastic behaviour with a reduced stiffness, corresponding to a value of force equal to 75% of the maximum. As we can

see from Table 4, the obtained ductility values are comparable and span for 4.2 to 5.6. A column is generally considered ductile if the index of available ductility in terms of displacement varies between 4 and 6 (Hwang et al., 2005).

It is important to underline that tests carried out are "quasi-static", so they provide a conservative estimation of the actual resistance of structures. In effect if we compared results obtained by a pseudo static test, with those obtained by a dynamic test, we would notice an increase of material resistance, due to higher rate of deformation.

Figure 11, shows is a direct comparison of the two envelope curves, through which it is possible to understand the differences between the two specimens. Even if the initial stiffness is comparable, the precast column shows a greater resistance capacity, and a symmetric behaviour also for high drift values. The cast in place specimen shows a faster decay of resistance and an asymmetric behaviour under the effect of the cyclic loading for higher displacement levels.

Ductility Index				
	Cast in place		Precast	
	Displ. -	Displ. +	Displ. -	Displ. +
$\Delta_y$	28	32	21	22
$\Delta_{u1}$	130	130	92	92
$\Delta_{u2}$	156	144	136	102
$\mu_{D1}$	4.6	4.1	4.4	4.2
$\mu_{D2}$	5.6	4.5	6.5	4.6

Table 4. Yield displacements,  $\Delta_y$ , ultimate displacements,  $\Delta_u$ , and ductility indexes  $\mu_D$ .

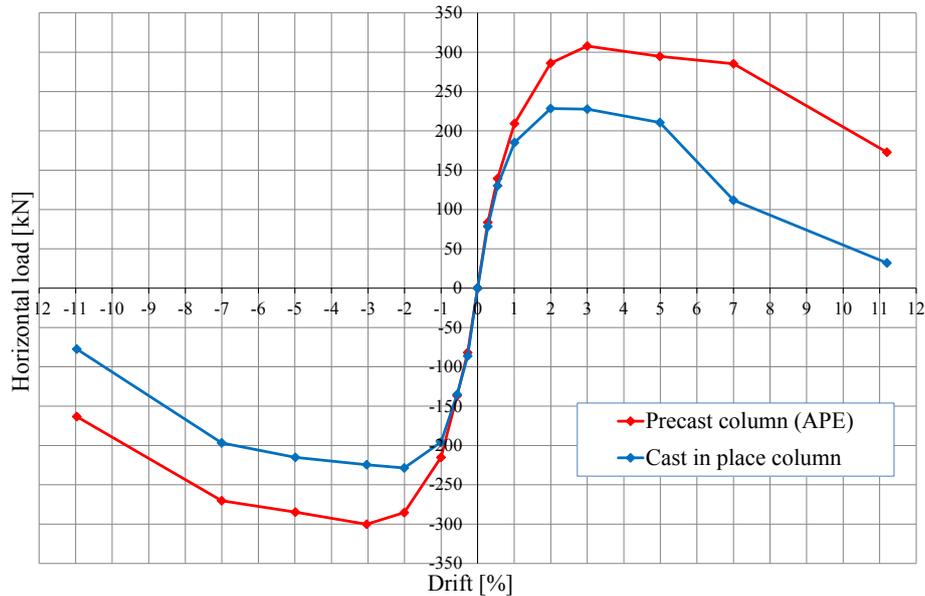


Figure 11. Comparison between the envelopes of the experimental force versus drift curves.

## NUMERICAL MODEL

A numerical model was implemented using the finite element code Opensees, to reproduce the results obtained by tests (McKenna et al. 2000).

The FEM model was built using the *beamWithHinges* beam-column elements (Scott and Fences, 2006), which require the definition of the length of the plastic hinge  $L_{pl}$  in order to avoid localization issues (Calabrese et al., 2010). In the present work this parameter was defined after Priestley, et al. (2007). The nonlinear behaviour of cross sections was described using fibre models. In particular, the

Mander's model was used for defining concrete properties (Mander et al. 1989) while the model proposed by Kunnath et al. (2009) was adopted for the rebars.

The fixed-end rotation of the column base, due to bar slip, was identified as an important factor to reproduce the effective behaviour of tests. To describe this phenomenon in the numerical model two approaches were considered. In the first approach the model proposed by Zhao and Sritharan (2007) was used. In the second approach a rotational spring was inserted at the column base. The stiffness of the spring was evaluated after the empirical model proposed by (Elwood and Eberhard, 2006):

$$K_{slip} = \frac{8 \cdot u \cdot E \cdot J_{eff}}{\varphi_{long} \cdot f_y}, \quad (1)$$

where  $\varphi_{long}$  is the diameter of longitudinal reinforcement,  $f_y$  the yield strength of longitudinal reinforcement,  $u$  value of the stress that is created at the interface between reinforcement and concrete (variable between  $6\sqrt{f_c}$  and  $12\sqrt{f_c}$ ), and  $EJ_{eff}$  effective stiffness of the element. As proposed by (Elwood and Eberhard, 2006)  $EJ_{eff}$  was defined as:

$$E \cdot J_{eff} = 0.2 \cdot E \cdot J_g, \quad (2)$$

where  $J_g$  is the inertia of concrete gross section. The best agreement between experimental data and numerical data was obtained by adopting the model with rotational spring for precast system with  $u$  set to  $12\sqrt{f_c}$ . Figure 12 shows a comparison between experimental and numerical results for the precast column. In general the numerical model provided good results both in terms of stiffness, strength and hysteretic behaviour up to drift values as large as 5%.

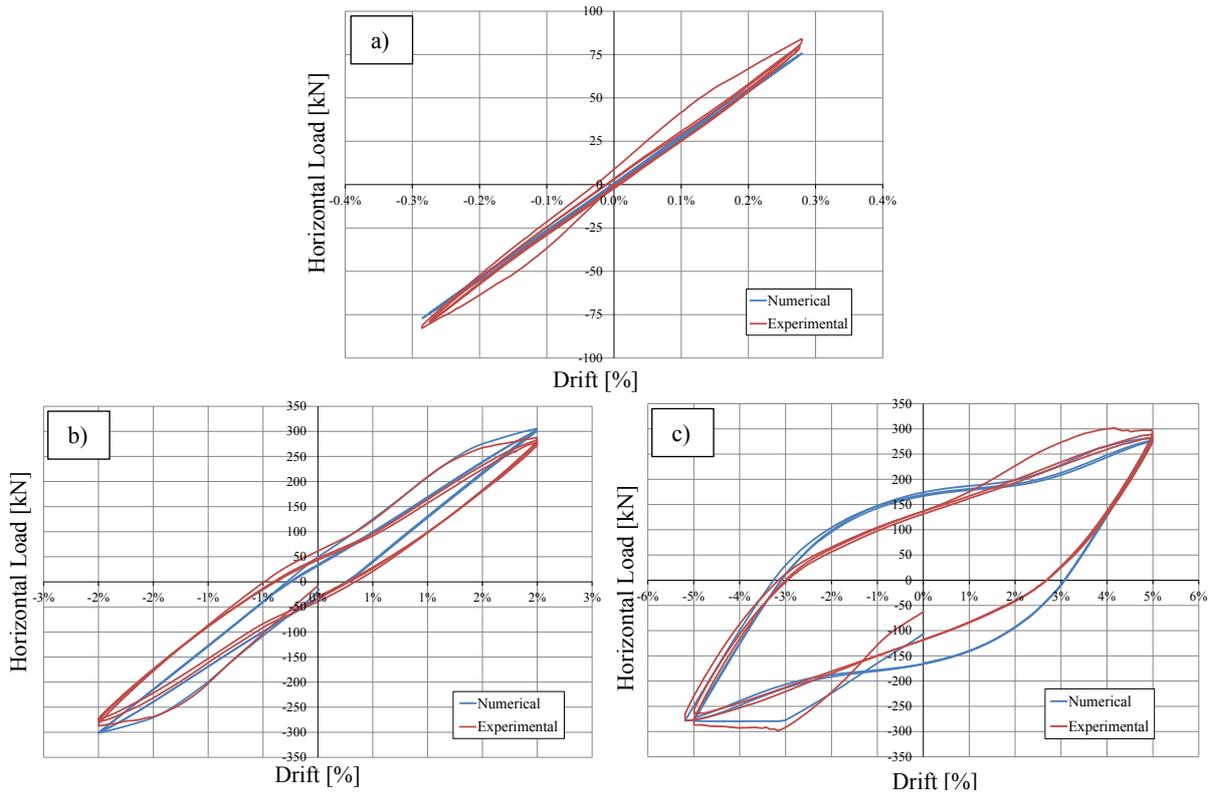


Figure 12. Comparison between the numerical and experimental load-drift curves for the precast column: a) low drift level - b) yielding drift level - c) high drift level

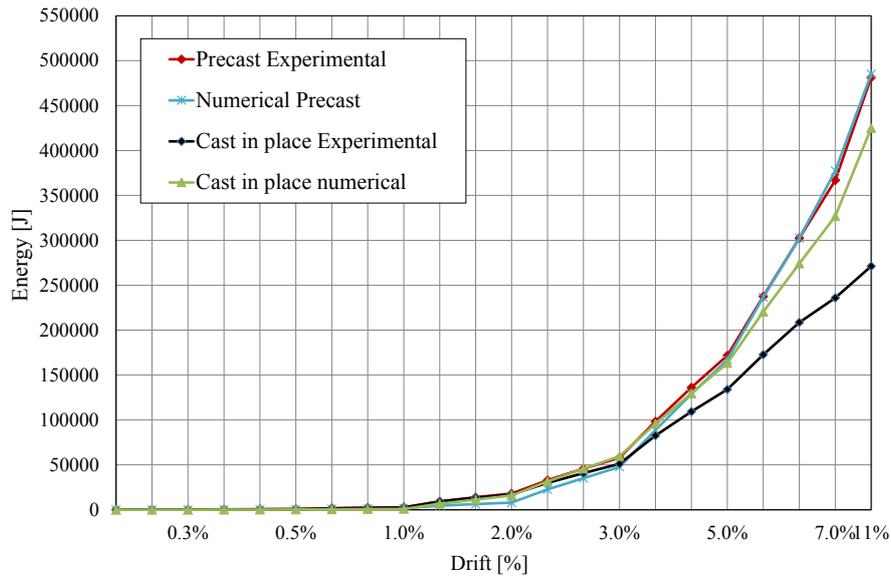


Figure 13. Comparison between the energy dissipated by hysteresis in the two tested columns and in the numerical models.

The results of the numerical model were compared to the experimental results also in terms of amount of energy dissipated by hysteresis. Figure 13, reports dissipated energy values on the vertical axis and the drift ratio on the horizontal axis. The precast column shows a better dissipative behaviour, if compared to the cast in place column, in effect the energy dissipated at the end of the test is almost double.

## CONCLUSIONS

The present work presented the results of experimental tests that allowed to investigate the performance of a column-foundation connection based on grouted corrugated tubes. In particular the behaviour of the prefabricated element was compared to the one of a cast-in-place column. The experimental results on a full-scale precast specimen showed a good performance of connection, in fact a stable hysteretic behaviour was observed up to 5 % drift ratios.

The tests have clearly shown that the constraint at the base of the cast in place column does not behave as a perfectly fixed end, on the other hand, the connection detail of the precast column realized an higher structural continuity between column and footing.

Simple numerical models were used to simulate with good accuracy the experimental behaviour of the two columns tested. The comparison between numerical and experimental results has highlighted the need to consider the bar slip effect at the base of the column to properly reproduce the experimental results.

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