Seismic response of columns connected to the foundation through a fastening technique

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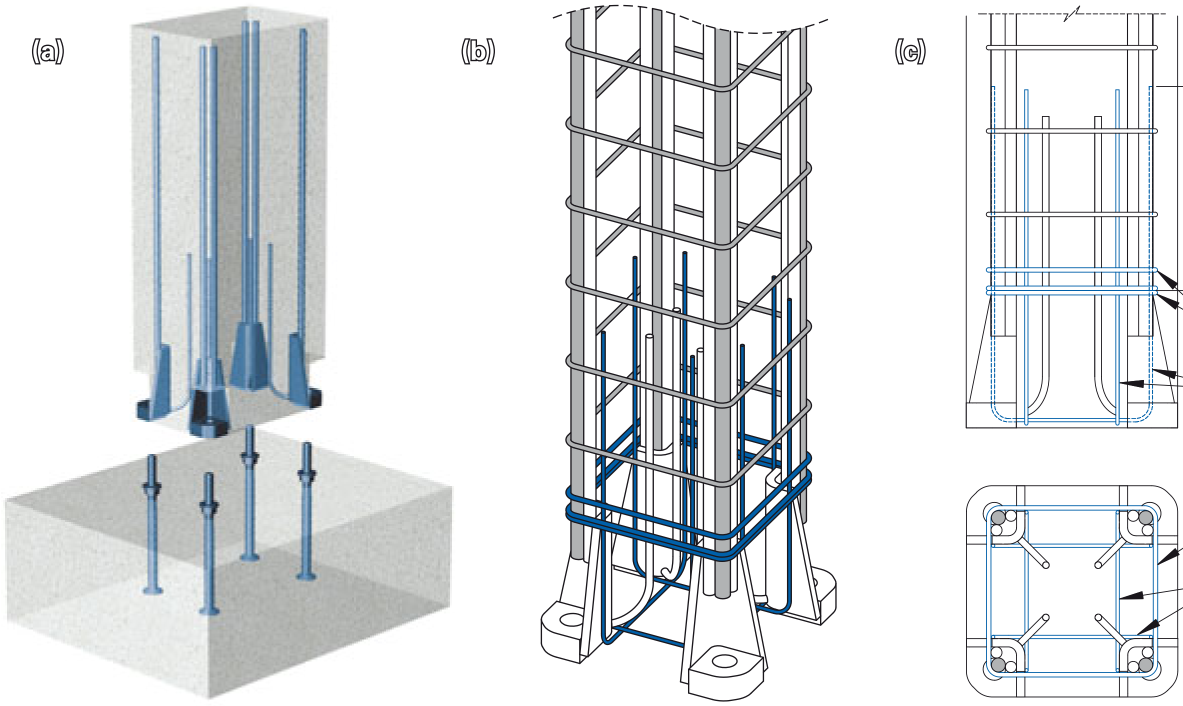


Fig. 1: *Representation of the studied column to foundation system: a) connection between the shoes and the anchor bolts; b) detail of the welded bars and of the lap splice along the height of the column; c) details of the additional reinforcement*

1. Foreword

The column to foundation connection system described in this research has been developed to be considered as an effective alternative of the use of traditional precast pocket foundations. This system is based on the mechanical connection between steel shoes embedded into the column base and protruding anchor bolts anchored into the foundation. The use of nuts and washers attached to the anchor bolts permits to control the vertical position, the height level of the column and the fixity of the connection. An additional injection of cement mortar into the void below the column (**Fig. 1a**) is required to complete the construction of the system.

The stress transfer from the anchor bolts – steel shoes system is based on a couple of longitudinal bars welded at the top of each shoe and on additional overlapped bars, which represent the steel reinforcement of the column (**Fig. 1b**).

Further details are to be considered in order to prevent the effects due to the eccentricity between the anchor bolt and the longitudinal bars, to improve the strength of the bottom part of the column and to tie effectively the longitudinal bars (**Fig. 1c**).

The seismic performances of the system have been evaluated by means of a numerical and experimental research, subdivided in two main topics: the investigation on the response of the single connectors and, subsequently, the study of the whole column – foundation system. The most relevant and deeply examined arguments are:

- The evaluation of the real response of the welded connection in steel column shoe, in order to demonstrate that the weldings remain in the elastic branch.
- The definition of the real global collapse mechanism and of the displacement ductility and dissipation capacity resources.
- The capacity of the base section of the column to resist shear actions.
- The initial stiffness of the column compared to the stiffness of other precast structural typologies.

2. Experimental tests on single connectors

The experimental response of the single steel shoes with welded bars – overlapped bars system to cyclic axial loads has been studied through a particular test configuration, opportunely studied in order to inhibit bending and torsional effects due to the eccentricity of the steel components of the specimen.

The specimens are composed of a steel system (whose components are the anchor bolt, the steel shoe (with welded bars) and the overlapping bar) embedded into a cast in place cylinder, characterized by a length equal to 1400 mm and with the diameter and the amount of reinforcing steel bars opportunely designed in order to impose a given lateral confinement level (equivalent to a 50 x 50 cm square column with two ø8mm-diameter stirrups @125mm).

C35/45 concrete and S500 steel bars were utilized, except for the anchor bolts; in fact they are made of 8.8 grade steel, in order to prevent it from the attainment of the failure condition and to study the remaining components of the specimen.

The typology of steel shoe that mark each specimen is summarized in the **Tab. 1**; in addition each group of 4 specimens that have the same column shoe, are divided in two subcategory accordingly to the adopted length of the lap splice: standard ("B") or reduced ("LP") length.

The relatively complicated test protocol (**Tab. 2, Tab. 3**) has been defined in order to utilized the first specimen of each subcategory to experimentally calibrate the reference value of the fundamental parameters of the twin specimen and to evaluate the experimental response for a number of cycles compatible with a seismic event.

The instrumentation consists of strain-gauges attached to the reinforcing steel bars and on the steel shoes, and of linear potentiometers to the measurement of relative axial displacements. Additional potentiometers have been utilized to evaluate the lateral displacement profile along the height of the specimen in the plane of the eccentricity between the anchor bolt and the longitudinal reinforcing bars. In particular, a strain-gauge has been provided to each shoe in the region characterized by the maximum deformability, evaluated through appropriate finite element analysis (**Fig. 2**).

The experimental tests highlighted that the weldings do not govern the response of the system. In fact, independently of the examined steel shoe type, other components of the system exceeded the yielding condition so that the welded connections remained into the elastic branch without suffer damage. This result guarantees an appropriate energy dissipation capacity and prevents from undesired brittle failure mechanisms.

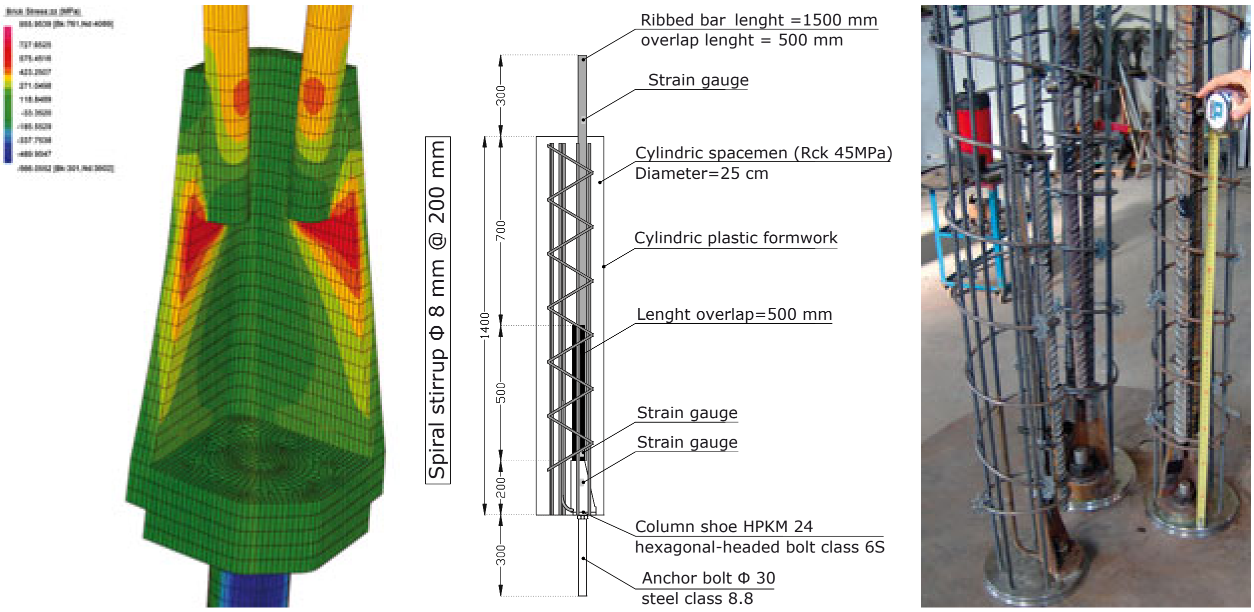
If the shoe with welded bars – overlapped bar system composed by elements with different stiffness and subjected to the same axial load is considered, the global response depends on the behaviour of the longitudinal bars and not on the weldings. In particular the shoes and the overlapping region are more deformable than the longitudinal bars, even though the plastic branch is reached only by the longitudinal bars or in the region where the bars are overlapped, depending on the type of the examined specimen ("B" or "LP").

The results obtained from the first series of tests are summarized in **Tab. 4**. It has to be noticed that it makes

Tab. 1: Details of the steel reinforcement of the specimens

Type of shoe	Type of specimen	Amount of specimens	Type of anchor bolts	Diameter of the rebar [mm]	Diameter of the overlapped bar HAK-SE [mm]	Lap splice length [mm]
HPKM 24	B	2	M24	16	25	840
	LP	2				500
HPKM 30	B	2	M30	20	32	1000
	LP	2				650
HPKM 39	B	2	M39	25	32	1150
	LP	2				800

Fig. 2: *Finite element model of the column shoe (left); geometry and reinforcement of the specimen with HPKM24 shoe (in the middle); assembly of the reinforcement in the laboratory (right)*



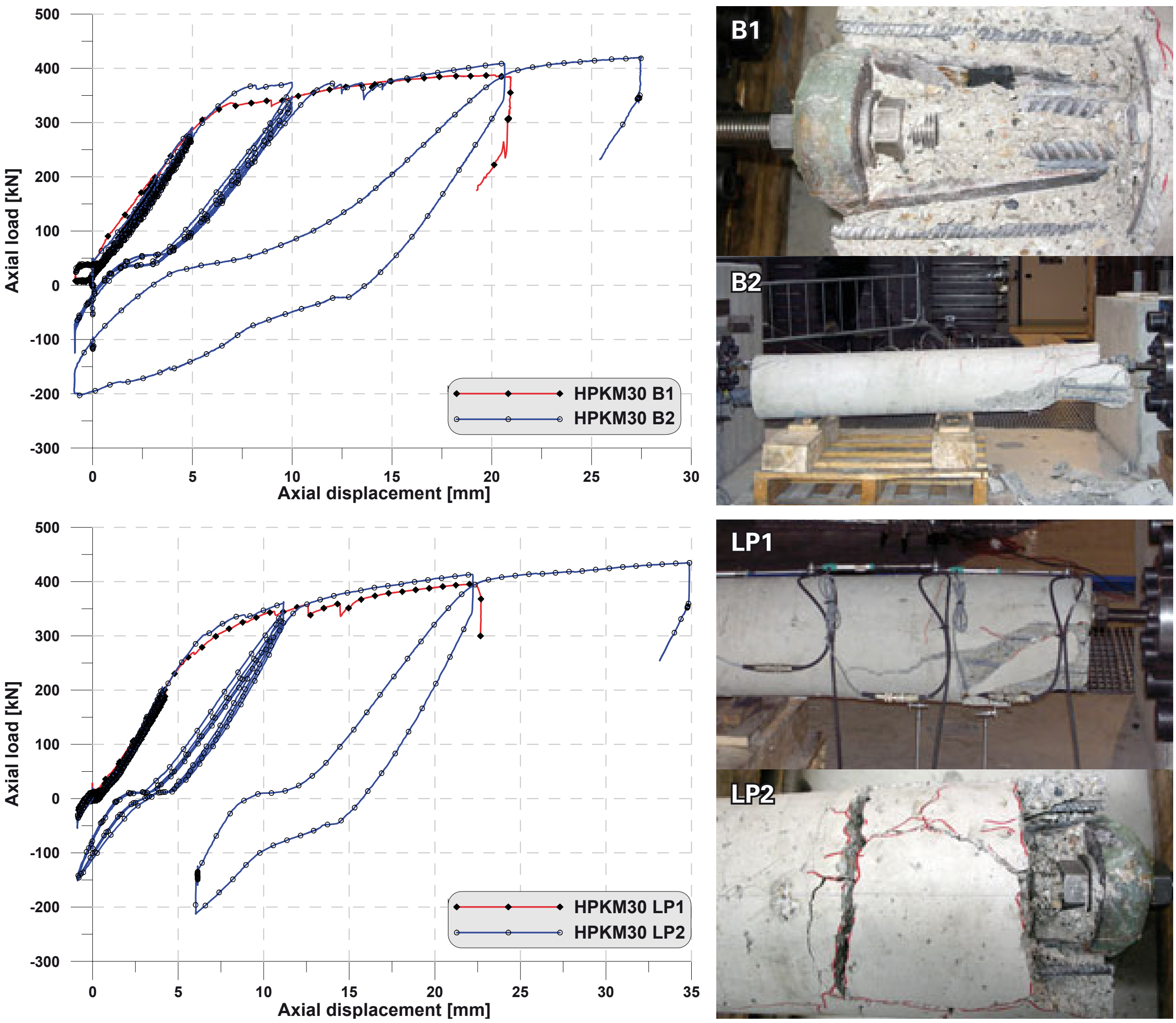
Tab. 2: *Planning of the tests (3 groups of 4 specimens for each type of column shoe, HPKM 24, 30, 39)*

n.	ID	Collapse mechanism	Protocol	Objective
1	B1	Longitudinal bars	A	Monotonic response and evaluation of the incipient yielding
2	B2		B	Hysteretic behaviour and effectiveness of the lap splice length
3	LP1	Longitudinal bars and slippage	A	Monotonic response and evaluation of the incipient yielding
4	LP2		B	Hysteretic behaviour and effectiveness of the lap splice length

Tab. 3: *Test protocols*

		Run 1	Run 2	Run 3	Run 4
Prot. A	N° cycles	20	1	-	-
	Tensile load	66% nominal strength	Monotonic up to failure	-	-
	Compressive load	20% nominal strength	-	-	-
Prot. B	N° cycles	20	4	< 4	1
	Tensile load	95% experimental strength	2 x experimental deformation at yielding	4 x experimental deformation at yielding	Monotonic up to failure
	Compressive load	20% experimental strength from protocol A	20% experimental strength from protocol A	20% experimental strength from protocol A	-

Fig. 3: *Axial load – axial displacement curves of the 4 specimens with HPKM 30 shoe type and damage pattern at the end of the test*



no sense to speak about ductility in this stage of the work, because of the fact that the response of the anchor bolts and the bending – shear behaviour have not been studied yet. It seems worth, instead, to highlight that the maximum strength of the system is conditioned by the steel components for specimens characterized by medium – large shoes (HPKM 30 and 39), and by the concrete component for specimens characterized by smaller shoes (HPKM 24). This hierarchy has to be considered only if the column without the anchor bolts is considered.

The effects due to the eccentricity between the longitudinal bars and the anchor bolt (maximum measured value = 25 mm, depending on the arrangements of the steel reinforcement and on the geometrical dimensions of the cross section of the column) are effectively resisted by the bars welded at the back of the column shoes.

Summarizing, the type "B" specimens were characterized by a damage restricted to the end where the shoe was embedded into the concrete cylinder; in addition, the shoes was subjected to high deformations and a relevant rotation between the bottom plate and the thin wall. It was expected that the "LP" type specimens reached the failure condition by a mechanism characterized by the slippage between the longitudinal bars, due to the short lap splice length imposed during the construction of the specimen in order to study and quantify its effectiveness.

3. Experimental tests on column – foundation systems

The combined bending – shear behaviour has been evaluated by experimental tests on three column – foundation systems, subjected to quasi – static cyclic horizontal top displacement.

A reference structure was considered in order to design the specimens. It is a RC precast multi-storey rectangular building with span lengths equal 14m and 8m, respectively along the two principal directions. The design was developed under the hypotheses of high ductility class, peak ground acceleration value of 0.25g and soil composed of medium dense sands. Both the geometrical dimensions and the steel reinforcement of the specimens; the cross section is 40 x 40 cm and HPKM 30 steel shoes were used together with standard HPM 30 P anchor bolts.

The experimental loading history consists of a series of horizontal displacement cycles at the top of the column with increasing target drift (**Fig. 6**). The vertical distance between the axis of the actuator and the base of the column is 2150 mm; it is expected that the shear component of the displacement profile of the specimen will be negligible, due to the value (5.37) of the ratio between the height and the depth of the cross section of the column. A constant vertical load will be imposed at the top of the column during the tests: the planned values are 200 kN, 400 kN and 600 kN, respectively and equal to 5%, 10% and 15% of the non-dimensional axial load (obtained by the ratio $N/(A_c f_{cd})$, where N is the axial load, f_{cd} the design compressive strength of the concrete and A_c the area of the cross section).

The instrumentation (**Fig. 7**) has been arranged in order to measure relative displacements, deformations and the curvatures of the cross section at different height levels.

Tab. 5: *Horizontal displacement history at the top of the column*

DRIFT	DISPLACEMENT [mm]
± 0.40	± 8.60
± 0.80	± 17.20
± 1.20	± 25.80
± 2.40	± 51.60
± 3.60	± 77.40
± 4.80	± 103.20

4. Main results obtained from the tests

The results obtained from the tests highlighted that the collapse mechanism is governed by the behaviour of the anchor bolts without any significant damage of the specimen, independently of axial load level imposed on the top of the column. The non-linear branch of the base shear – top displacement curve (**Figures from 8 to 10**) is due to the yielding of the anchor bolts, which are the only components of the system to reach the plastic branch,

Tab. 4: *Summary of the main results of the tests on single connectors*

ID Specimen	Strength [kN]	Ultimate dis-placement [mm]	Ratio between strength of the specimen and of the welded bars		
			Numerical prediction	Experimental result	Diff. [%]
HPKM 24-B1	158	9.5	0.57	0.70	18.57
HPKM 24-B2	275	20.4	0.99	1.22	18.85
HPKM 24-LP1	300	27.0	1.08	1.33	18.80
HPKM 24-LP2	248	26.0	0.89	1.10	19.09
HPKM 30-B1	387	21.8	0.89	0.93	4.30
HPKM 30-B2	420	27.9	0.97	1.01	3.96
HPKM 30-LP1	395	23.1	0.91	0.95	4.21
HPKM 30-LP2	453	28.2	1.04	1.09	4.59
HPKM 39-B1	417	22.9	0.62	0.66	6.06
HPKM 39-B2	501	60.5	0.74	0.79	6.33
HPKM 39-LP1	507	29.8	0.75	0.80	6.25
HPKM 39-LP2	461	24.5	0.68	0.73	6.85

Fig. 4: *Geometrical dimensions and steel reinforcement of the column – foundation system*

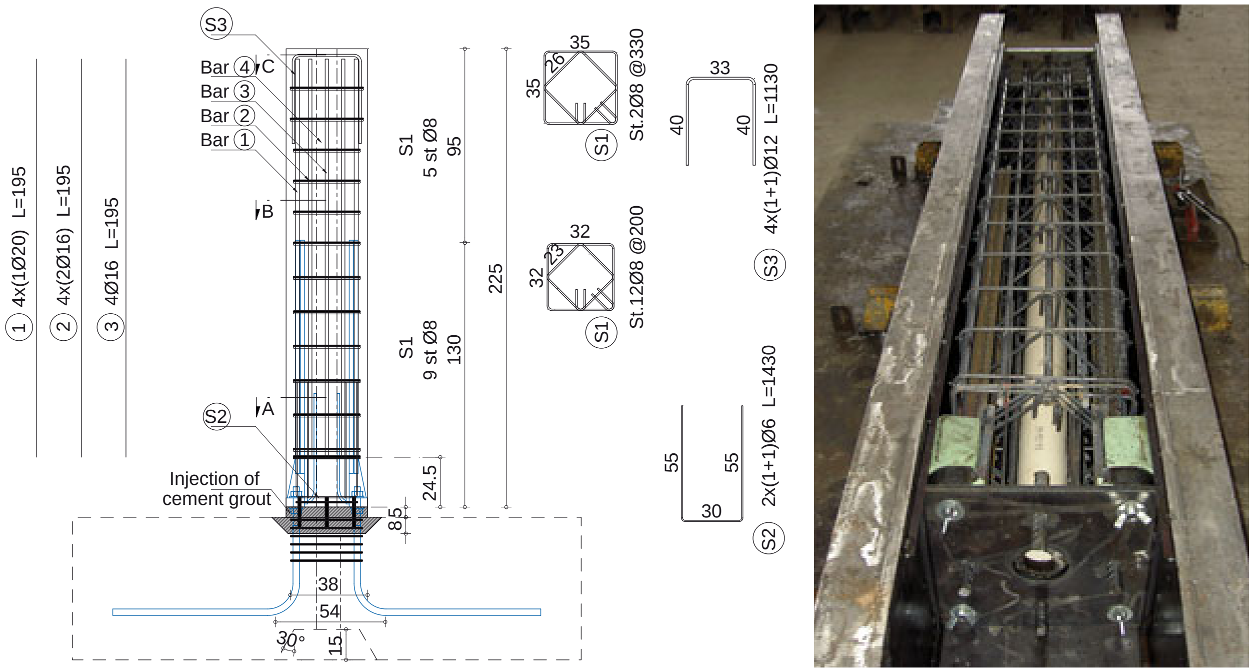


Fig. 5: *Details of the reinforcement of the connection between column and foundation (4 ø24mm steel pins added in order to resist shear actions)*

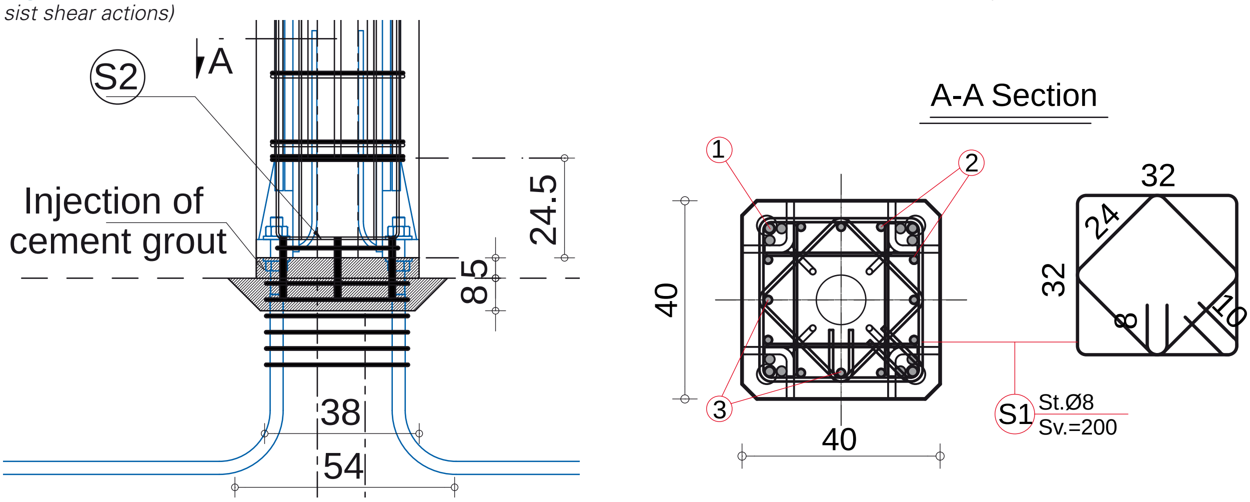


Fig. 6: *Test set – up (left) and experimental maximum displacement of the test with target drift equal to 4.8% (right)*

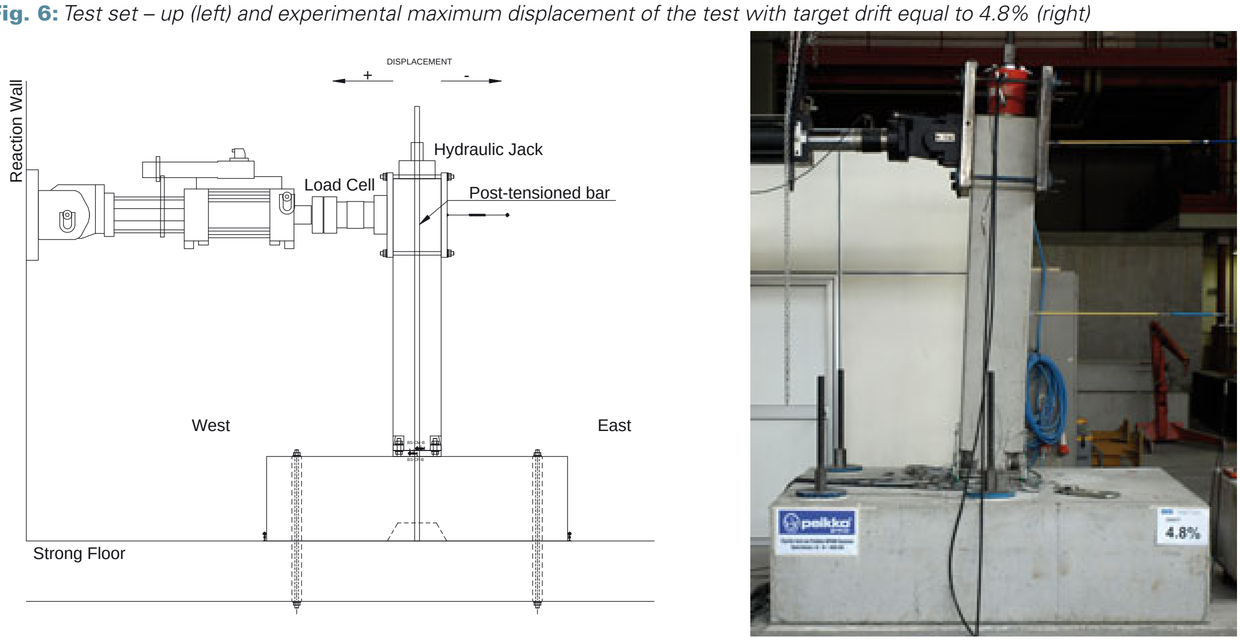
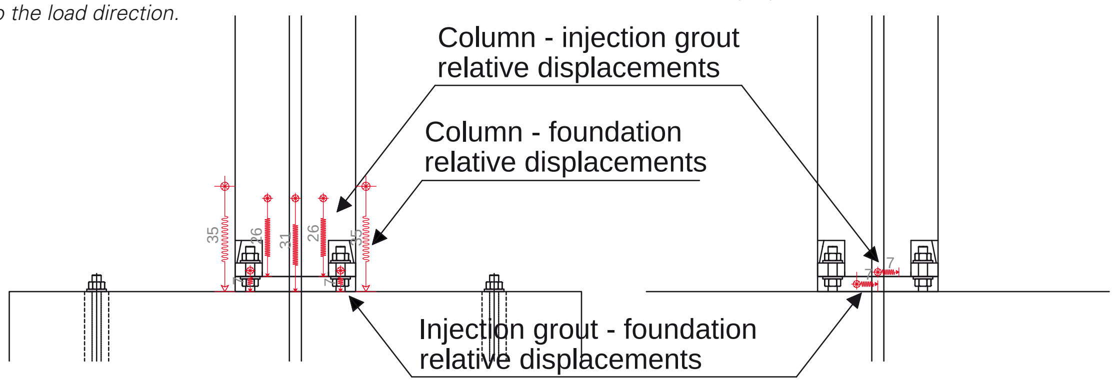


Fig. 7: *Instrumentation of the connection between column and foundation: (a) side perpendicular to the load direction; (b) side parallel to the load direction.*



whilst the other components do not exceed the yielding condition and are not significantly damaged.

The hysteretic behaviour has been evaluated up to a drift level equal to 4.8%, since higher displacement values have not been considered significant. It is characterized by negligible strength degradation and by a global ductility level always greater than 5. The total damping, given by the sum of the viscous (considered equal to 2%) and the hysteretic (calculated as a function of the ratio between the dissipated energy and the elastic energy of each cycle) damping is about 8.5% in the elastic branch (up to a drift level equal to 1.2%), then it increases and is included in the range 16% - 20% for the first cycles of the following drift levels. Such damping value, although it is partially limited by the particular shape of the unloading branch dominated by the response of the anchor bolts, is characterized by satisfying and effective values. In addition, the displacement measured at the zero loading condition may be smaller than the case of a cast-in-place equivalent system.

The good seismic response of the examined specimens can be also noticed through the damping – displacement ductility relationship (**Fig. 12**), where the first experimental cycles obtained from the tests are characterized by a results very close to the typical behaviour of cast-in-place frames (red dashed line) for medium-low axial load values and close to the behaviour of RC bridges for higher axial load values.

In seismic design, high values of the shear demand are expected, so particular care may be required. In particular, 4 ø24mm (**Fig. 5**) steel pins embedded into the column of the specimens were used in order to resist the shear action and permit to the anchor bolts to be subjected only to axial loads. The adoption of the steel pins was essentially due to particular requirements of the test set-up. In the practice, the design may develop in accordance with the following steps: 1) to compare the shear demand with the capacity of the concrete only (depending on the axial load level) and check for a high safety factor; 2) if the previous condition is not satisfied, it is necessary to consider a different solution. The adoption of a steel profile embedded into the column or a cast-in-place basement characterized by an appropriate steel reinforcement are effective solution.

The incipient yielding condition of the examined columns is attained at a drift level included in the range 1% - 1.3%; in addition, since the column does not exceed the elastic branch and it is not subjected to significant damage, a more rational design than the case of traditional RC precast structures characterized by monolithic columns and pinned beams

Fig. 11: *Damping calculated for each cycle of the specimens subjected to the experimental tests*

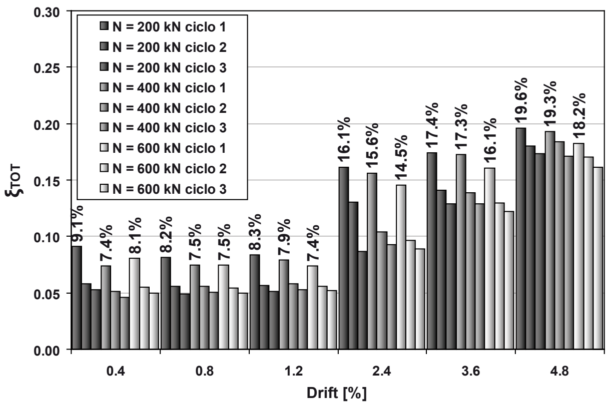
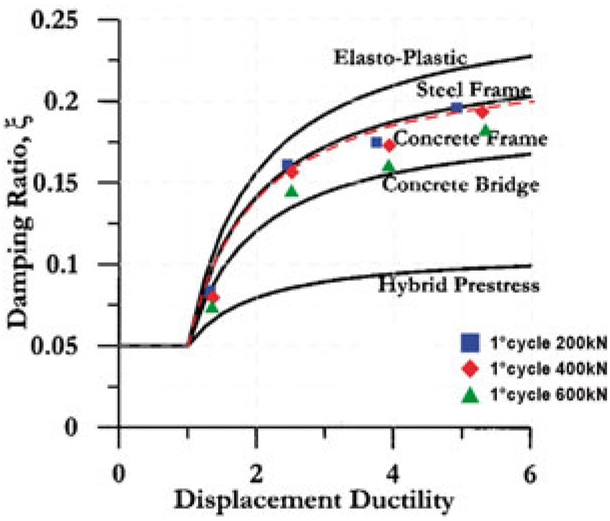


Fig. 12: *Damping as a function of the displacement ductility, comparison between the examined system and characteristic curves of other structural typologies (after Priestley et Al, 2007)*



(yielding condition at 2% drift level), can be obtained.

If the possible foundation system is compared with a traditional precast pocket foundation and the same design conditions of the reference case study are assumed, it results in a less volume of concrete (about 20%) and less weight of steel (about 30%). In seismic design, both the solutions must be completed with additional RC tie-beams along two orthogonal directions in order to inhibit the effects of the relative displacements of the soil on the structure. The solution described in this research avoid the difficult related to the connections between the tie-beam and the precast pocket foundation and provide a potentially more effective connection.

Fig. 8: *Base shear vs. horizontal top displacement of the specimen subjected to a vertical load equal to N = 200 kN and damage pattern at the end of the test*

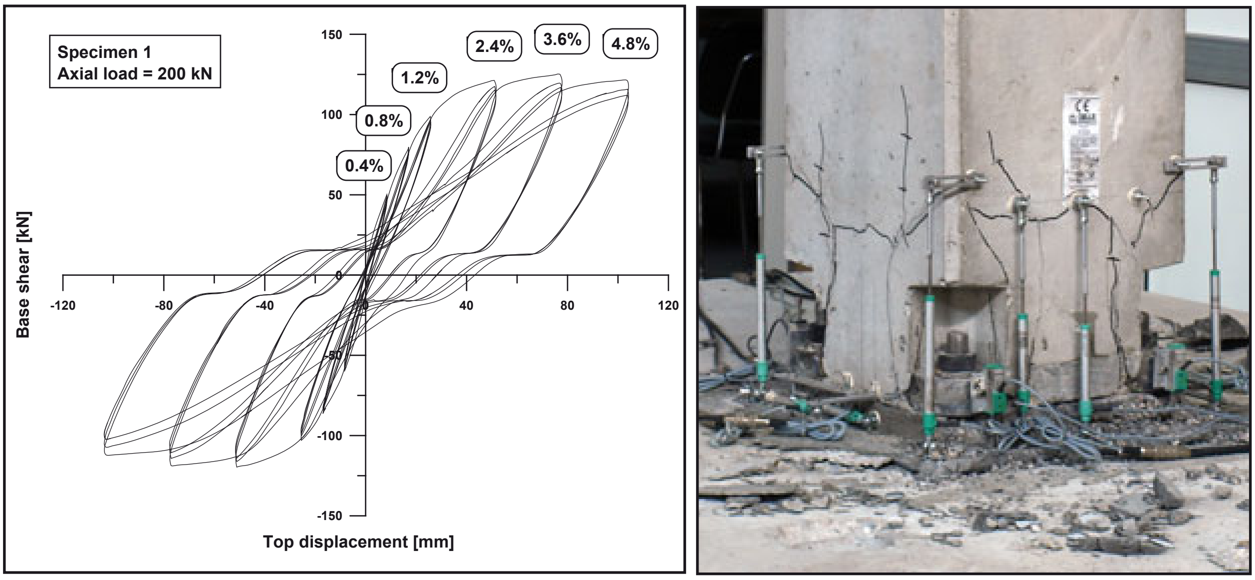


Fig. 9: *Base shear vs. horizontal top displacement of the specimen subjected to a vertical load equal to N = 400 kN and damage pattern at the end of the test*

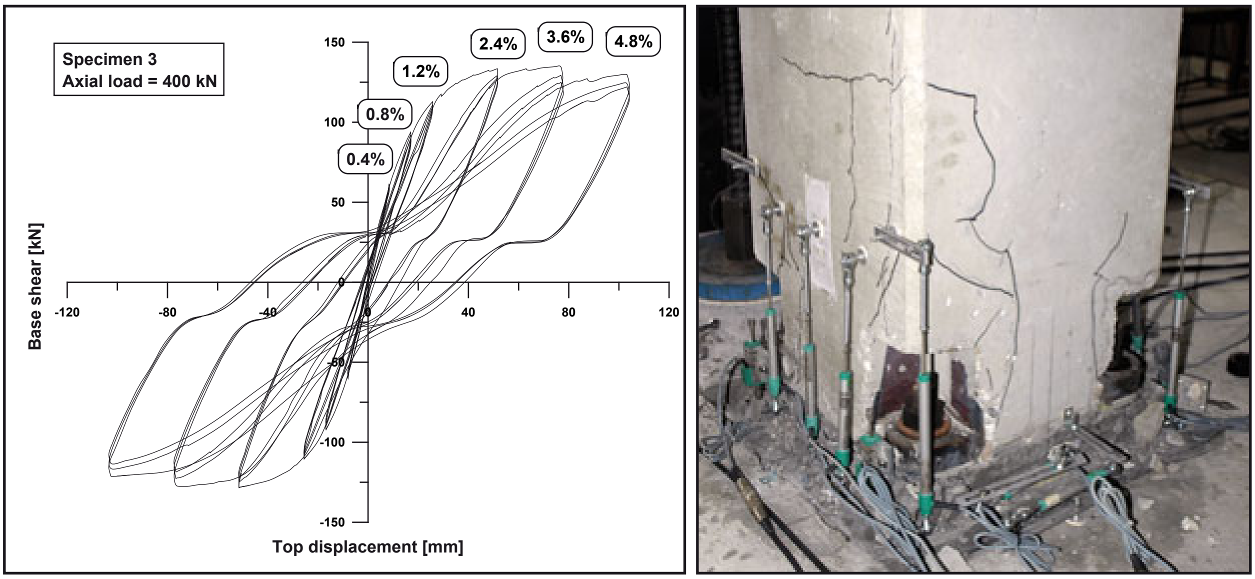
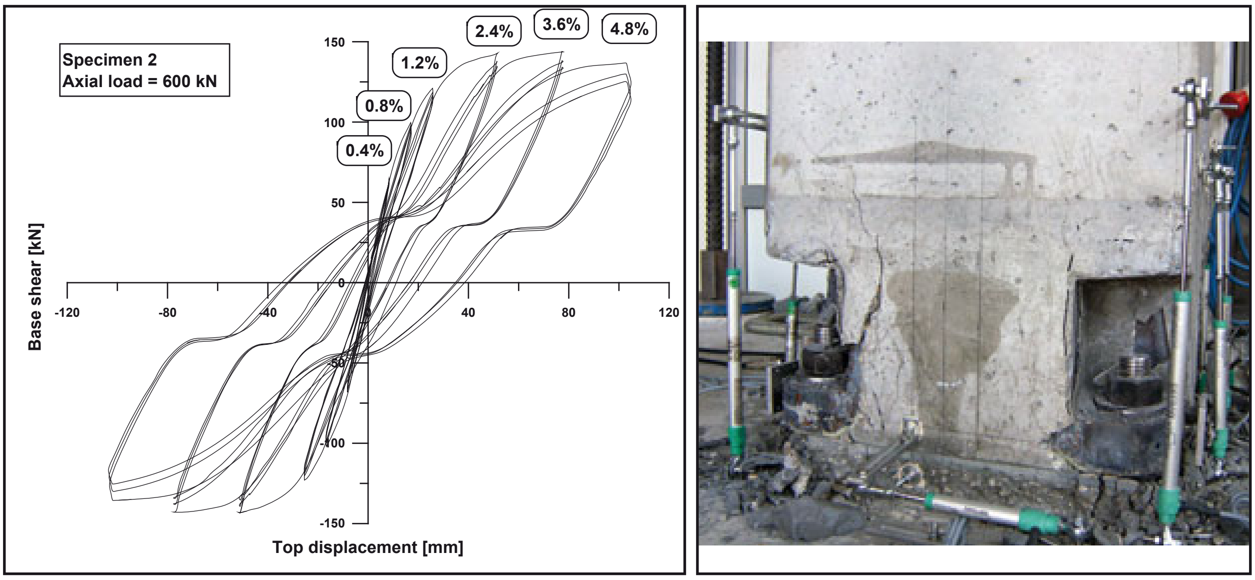


Fig. 10: *Base shear vs. horizontal top displacement of the specimen subjected to a vertical load equal to N = 600 kN and damage pattern at the end of the test*



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Authors

Lorenzo Bianco, Civil Engineer, got his Laurea in the University of Genoa in December 1992. He cooperated with different engineering companies for design of steel and concrete structures; he worked with software houses that developed and sold software for engineers and architects. Cooperated as engineer on site for the building of the enlargement of the paper mill Verzuolo 9 of Cartiere Burgo S.p.a. (total investment of 500 million Euro).

Nowadays he is Managing Director of Peikko Italia S.r.l. as a part of Peikko Group. Peikko is European leader in production and sales of steel inserts used in the building branch.

Silvia Santagati received her Degree in Building Engineering from the Politecnico of Milan, Italy in 2007. For her degree's dissertation, she completed a study on a precast connection between column and foundation, with experimental tests at the GIMED Laboratory (Department of Structural Mechanics of Politecnico of Milan).

In the last year, she has been working at the European Centre for Training and Research in Earthquake Engineering (Eucentre) of Pavia, where she was involved in the research projects of the Precast Structures Area. Since September 2008 she is studying at the ROSE School to attend the MSc in Earthquake Engineering.

Davide Bolognini took his Laurea degree in Civil Engineering at the University of Pavia in October 1997. Up to September 2003 he worked at the Department of Structural Mechanics (DMS) of the same University and in a construction firm.

The experience developed in the past concerns: numerical and experimental evaluation of the seismic vulnerability of R.C. structures and structural subassemblies, reinforced and unreinforced masonry buildings, seismic design and retrofit of R.C. structures and bridges with fibre-reinforced polymers (FRP), seismic design of long span R.C. precast structures, development of software for static and dynamic non-linear analyses of masonry and R.C. structures.

Currently he is the head of the Precast Structures Section at the European Centre for Training and Research in Earthquake Engineering (Eucentre) of Pavia. Among the projects developed in latest year: numerical and experimental evaluation of the seismic response of R.C: precast structures and structural subassemblies, seismic design of traditional Italian precast structures, socket foundations and connections with RC tie-beams, innovative column-foundation connections, RC bearing walls, use of steel fibre reinforced concrete (SFRC) in seismic areas, evaluation of design methods of moment resisting frames with concentrically steel braces.

Roberto Nascimbene is a Researcher at the Precast Section of the European Centre for Training and Research in Earthquake Engineering (EUCENTRE). He graduated in Civil Engineering from University of Pavia, Department of Structural Mechanics. Then he completed the PhD in Structural Engineering in 2001 at the Department of Structural Mechanics in Pavia. Actually he is Adjunct Professor of "Design of shell structures" in the Department of Structural Mechanics at the University of Pavia. He has authored more than 50 publications (journal and conference papers) in the field of computational mechanics and earthquake engineering. His main research interests are (i) development of numerical - static/dynamic and linear/nonlinear – models of earthquake resistant structures (reinforced concrete and precast), (ii) SFRC (steel fibre reinforced concrete) for reinforced concrete earthquake resistant structures, (iii) development of advanced models for analysis of precast structures and precast connections subjected to seismic action, (iv) existing buildings.