SEISMIC BEHAVIOUR OF REINFORCED CONCRETE PRECAST TRADITIONAL ITALIAN FRAMES AND SUBASSEMBLIES

D. Bellotti[1] D. Bolognini[2] and R. Nascimbene[2]

[1] *Department of Structural Mechanics, University of Pavia, Via Ferrata 1, 27100 Pavia, Italy*
[2] *European Centre for Training and Research in Earthquake Engineering, Via Ferrata 1, 27100 Pavia, Italy*

ABSTRACT :

The results of a numerical and experimental research on the seismic behaviour of reinforced concrete (RC) precast frames and column-foundation connections are presented in this paper.
A traditional Italian multi-storey structure, characterized by a total height of 10.8m and span lengths of 5.5m, 7.2m and 14.4m, was designed in accordance with the recent seismic Code provisions for medium-high seismicity zones (OPCM 3431, 2005), then, one of the main two-bay three-storey frame was assembled in the laboratory in 3:4 scale and subjected to a quasi-static cyclic displacement history. A lateral load distribution calculated as a function of the mass and of the storey height was kept constant during the test.
The traditional beam-column connections can resist only shear actions; their design is based on the strength capacity without particular considerations about the local ductility.
Since it was expected that the behaviour of the connections strongly affects the in-plane global response, the main objective of the test was to quantify the initial stiffness, the maximum ductility and the energy dissipation of the frame.
In addition, two precast column-foundation connections in real scale were designed through a model based on local strut-and-tie mechanisms, then assembled in the laboratory and subjected to quasi-static loading – unloading cycles at increasing displacement at the top of the columns. The aim of these tests was to evaluate the reliability of the adopted design method and to compare the experimental results with the potential performances of alternative seismic solutions.

KEYWORDS: Reinforced concrete precast structures, Traditional Italian, Seismic behaviour

1. INTRODUCTION

The description of partial results obtained from the experimental tests planned within a three-year research project on the seismic behaviour of RC precast structures and subassemblies, developed in order to propose to Italian precasters rational solutions for traditional low-rise buildings, is presented in this paper.

Right from the start of its large employment, at the beginning of the 1950's, the great demand for reinforced concrete precast structures was targeted to the use of industrial and commercial buildings. Such situation, that immediately at the end of the war, seemed incidental and justified by the social and economical conditions, at present is still unchanged, so almost the entire National production is intended for industrial buildings.
The reasons why prefabrication of reinforced concrete and pre-stressed reinforced concrete elements seemed to feature greater potential for economy than cast-in-situ concrete lay in the speed of construction, higher performance of materials, flexibility combined with short delivery times, quality assurance and plant certification. At present, as opposed to other seismic prone countries, these reasons still represent the choice of reinforced concrete precast structures in Italy against the cast-in situ solution.

Generally Italian designer have a low confidence with the potential seismic performances of precast structures, in some measure justified by the lack of indications in the seismic Code, until the beginning of 2000. Such scant confidence in the pre-stressing and post-tensioning techniques in seismic zones and the poor practice in the construction of precast moment resisting frames still limit the use of precast solutions, so that buildings of high importance such as schools and hospitals are usually constructed using alternative not precast structural

typologies or not fully consistent with the seismic criteria.
The situation described above has favoured the development of one to three storey precast structures characterized by a relatively high degree of flexibility and with a structural system composed of monolithic columns fixed at the base and free at the top, with pinned beams on corbels or even, in few cases, barely supported beams on the top of the columns. The socket foundations on a reinforced concrete basement are the traditional solution, but the seismic design leads, in several cases, to a very large dimensions of the basement, so that a partial pre-cast solution or a cast-in-situ slab foundation may be more effective.

2. EXPERIMENTAL PROGRAM

2.1. Design

The three main basic structural typologies that can be currently considered representative of the National production of precast elements are depicted in the following figure, where the typical minimum and maximum dimensions usually utilized in the seismic design are also shown.

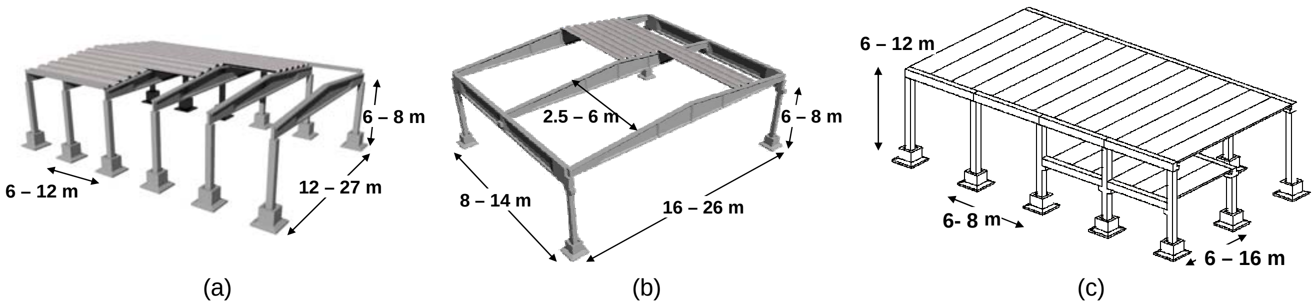


Figure 2.1 – Basic main structural typologies used in Italy: a) one-storey structure with portals; b) one-storey structure with portals and roof beams independent of the column grid; c) two- or three-storey structure.

A three-storey structure has been designed in a previous work (Calvi *et al.*, 2007) according to the indications of OPCM 3431 (2005) for medium-high seismicity (PGA equal to 0.25g) and for deep deposits of stiff clay (soil type C, in accordance with the classification of Eurocode 8, 2003), but with traditional not moment resisting connections.

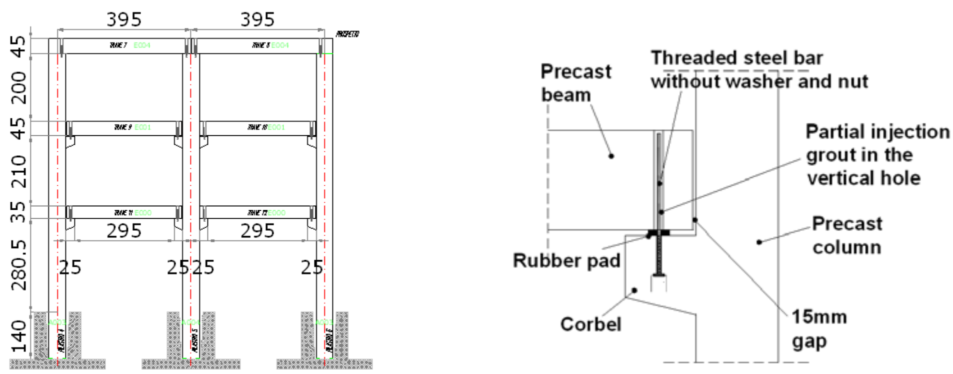


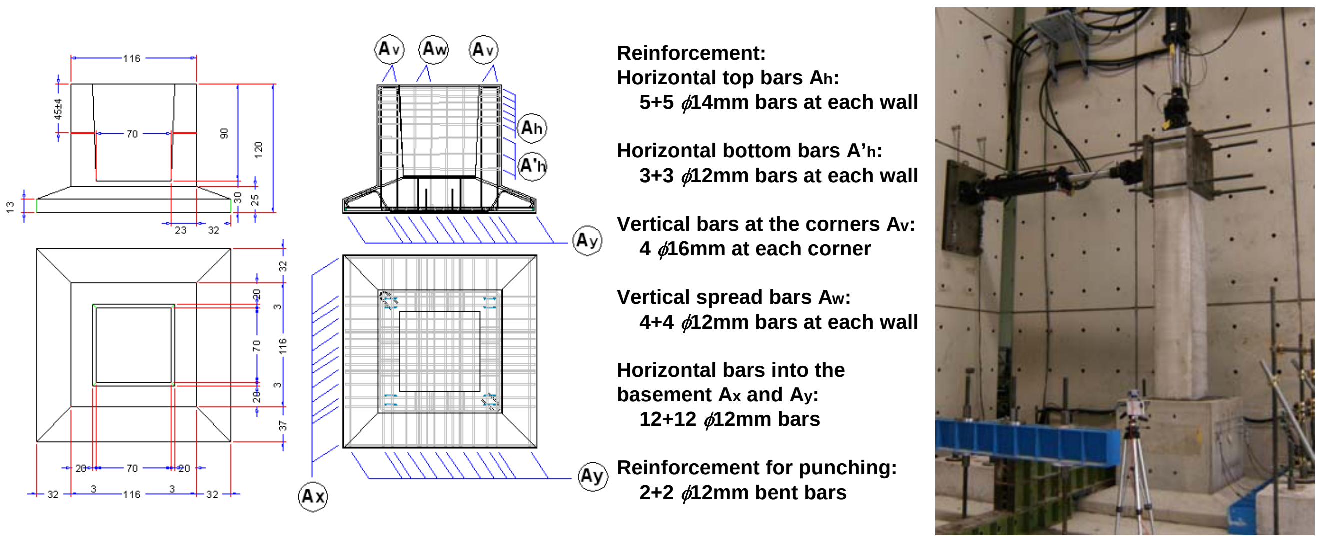
Figure 2.2 – Geometry of the specimen (left) and detail of the beam to corbel connection (right).

A two-bay three-storey frame, extrapolated from such structure, was constructed in 3:4 scale (the geometric dimensions are indicated in the Figure 2.2(left)). The beams are supported on the corbels or on the top of the columns through a 15mm-thick rubber pad. Two rectangular hollow steel profiles (dimensions 80 x 50 mm, thickness 3 mm) are embedded at the ends of the beams; two ϕ16mm vertical steel threaded bars placed inside the steel profiles and anchored into the corbel or directly into the top of the column and an injection grout connect the beams to the columns. A height equal to 200 mm (i.e. about the mid-height of the beams) has been imposed for the injection grout, so that the bars are forced to failure through a shear mechanism. A 15mm-gap is leaved between the ends of the beams and the internal side of the columns. The steel used for the profiles and the threaded bars is 8.8 grade (equivalent yielding axial strength of 560 N/mm^2).

The frame is anchored to the strong floor of the laboratory through post-tensioned socket foundations, opportunely overdesigned in order to inhibit possible cracking and to capture, during the experimental tests, only the response of the structure fixed at the base, mainly affected by the behaviour of the connections.
The column have 500 x 500 mm square sections and a maximum percentage of reinforcing bars equal to 2.70%. The rectangular sections of the pre-tensioned beams have width of 500 mm and depth of 350 or 450 mm. Concrete type C45/55 and C40/50 were used for the beams and for the columns, respectively. The reinforcement consists of mild steel (yield strength f_{yk} = 430 N/mm^2) and strands that were initially stretched up to 70% of the nominal yielding load (f_{pk} = 19860 N/mm^2).

Two additional socket foundations has been tested separately from the frame, in order to validate the proposed design method (Calvi *et al.*, 2007), characterized by some changes compared to the provisions of CNR 10025 (1998), and to quantify the expected large capacity to demand ratio. Mori & Pereswiet-Soltan (1981), in fact, have already demonstrated that a typical socket foundation, designed through the allowable tension simplified method and a different arrangement of the reinforcing bars in order to resist a bending moment of 60 kNm, was characterised by an ultimate bending moment capacity equal to 6 time the corresponding design value. Some year before, Leonhardt (1977) indicated that the same arrangement of reinforcing steel bars might be effective, but he stated that the local strut-and-tie mechanism that resist the bending moment cannot develop simply with a single strut, due to the presence of the vertical bars along the wall of the foundation, which cause a more complex mechanism with several struts that are more inclined to the vertical direction than the single global strut. The first version of the CNR 10025 (1994) suggested a modified heavier arrangements of the reinforcing bars with a particular concentration of the reinforcement into the corners of the walls and, in addition, partially considered the effect of the friction between the column and the socket foundation in the design method. The new version (CNR 10025, 1998), instead, inhibited the friction effects for safety reasons. Both in such methods and in the proposed one, the capacity domain is defined as the minimum envelope of the capacities of the simplified local strut-and-tie mechanisms and of the global ones, such as rigid overturning, sliding and punching.

The 3D model of the studied three-storey structure was characterized by bending moment design values at the base notably higher than the values considered in the work developed by Mori & Pereswiet-Soltan. A new socket foundation was designed according to the new proposed method for a bending moment equal to 300 kN and an axial load of 500 kN. In the following Figure 2.3(a) and (b) the geometry and the reinforcing bars of the two specimens are shown. The Figure 2.3(c) shows the test setup, described in the following.



(a) (b) (c)

Figure 2.3 – (a) Geometry of the socket foundation; (b) details of the reinforcement; (c) test setup

In high seismicity zones, the design of the socket foundations is generally governed by the rigid overturning, the bending of the basement and, partially, by mechanism that involves the vertical bars into the corners of the walls and the diagonal struts of the lateral walls. These conditions result in a very large demand of the dimensions of the basement, even up to 2 times the maximum dimensions coming from a static design (see Figure 2.4(b)), so that in the case of multi-storey buildings, it seems worth to consider the monolithic slab foundation as a possible alternative and more effective solution, combined with a partially precast socket foundation.

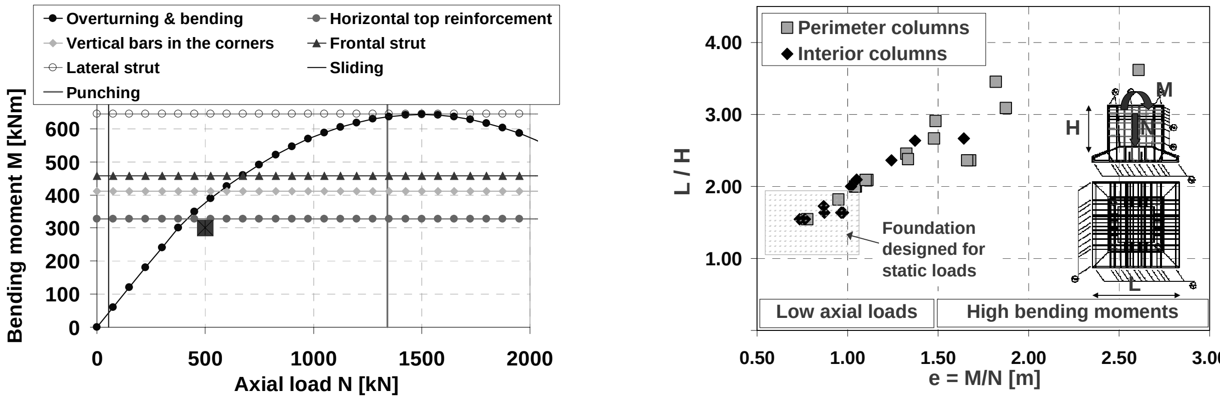


Figure 2.4 – Envelope of the design capacity domains of the specimens (left) and L/H ratio of socket foundation of three storey buildings as a function of the eccentricity of the seismic loads (PGA = 0.25g, Soil type C) (right).

2.2. Test setup and loading history

The frame was subjected to quasi-static cyclic displacement history with a constant profile of lateral forces, calculated as a function of the mass m_i distributed on the beams and of the storey height h_i: $F_i / F_{TOP} = (h_i \cdot m_i)/(h_{TOP} \cdot m_{TOP}) = \{1;0.98;0.71\}$ (see Figure 2.5).

The displacement is imposed by the actuator at the third floor, that controls the other two actuators by imposing the given force ratio. The loading history consisted in a series of three cycles at increasing drift levels (0.5, 1.0, 1.5, 2.0, 3.0%), with an additional final monotonic loading up to failure.

The instrumentation consisted of 64 linear potentiometers LVDT for the evaluation of: i) three interstorey absolute displacements; ii) three values of curvature at different heights at the base of each side of the three columns; iii) relative translations and rotations between the beams and the columns (see Figure 2.5).

The high quality of the basic materials resulted in a very low standard deviations of the mechanical properties.

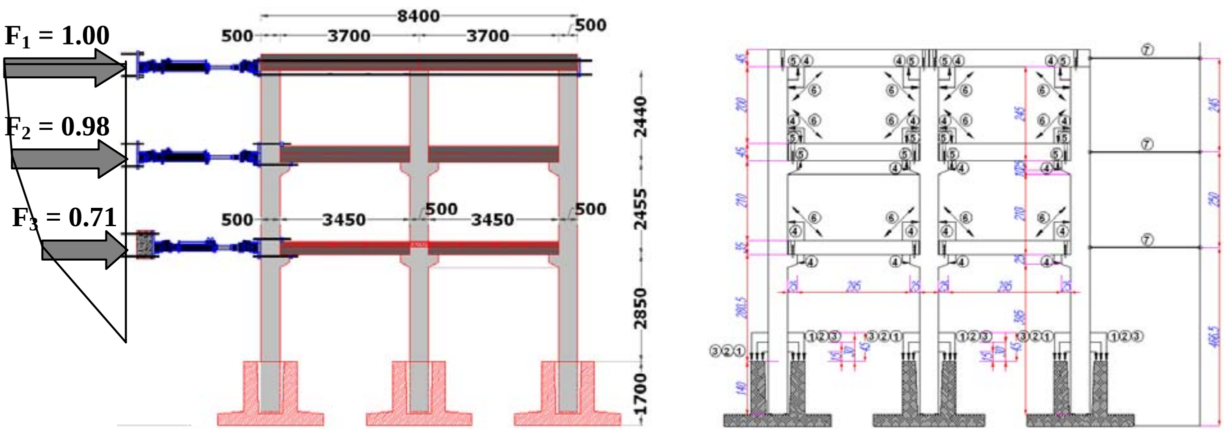


Figure 2.5 – Lateral force distribution (left) and instrumentation on the frame (right).

The tests on the two column to socket foundation connections have been done similarly to the tests described in Mori & Pereswiet-Soltan (1981). The changes regarded: i) the dimensions and the reinforcement of the foundation (Figure 2.3(b)), whose arrangement was in accordance with the CNR 10025 (1998); ii) the capacity, 5 times greater than the previous experimental tests (Figure 2.4(a)); iii) the axial load, kept constantly equal to

500 kN during the test; iv) the loading history programme consisted in a first series of three loading – unloading cycles up to 400 kNm with step of 100 kNm. Then, target drift values were imposed (2%, 4%, 5%, 6% of the column height).
The bending moment is controlled by imposing a force at the top of the column (Figure 2.3(c)), opportunely overdesigned in order to study the behaviour of the foundation only. One side of the basement of the foundation was fixed through a system of steel plates and beams post-tensioned to the strong floor of the laboratory in order to prevent the rigid overturning.
A C40/50 concrete and a FeB44k steel were used for the basic elements of the specimens. The injection grout between the foundation and the column has a nominal compressive strength on cube of 23 N/mm^2 and it has to be considered equivalent to an optimal solution compared to the current in-site solutions.
The instrumentation consisted in 22 linear potentiometers LVDT and 18 internal and external strain-gauges used to measure the main displacements related to the strut-and-tie local mechanisms and the axial deformation of the steel bars (Figure 2.6).

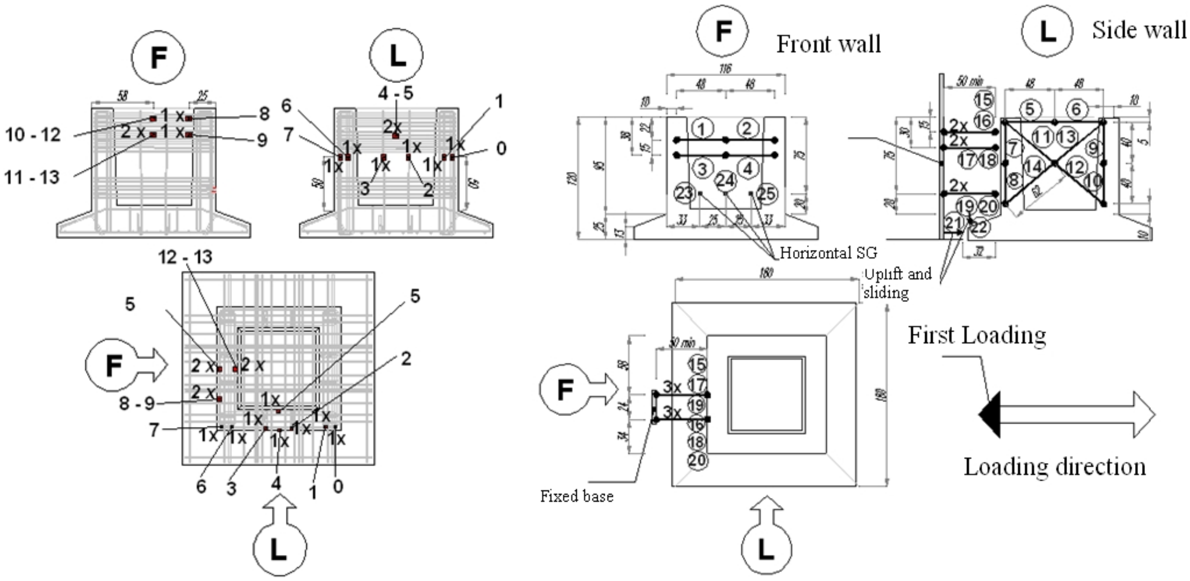


Figure 2.6 – Instrumentation of the column to foundation connections: strain-gauges (left); LVDT potentiometers (right)

3. EXPERIMENTAL AND NUMERICAL RESULTS

The experimental response of the frame is characterised by an expected high flexibility (incipient yielding between 120and 150mm of top displacement, i.e. between 1.54% and 1.95% of total drift) and a lateral deformed shape similar to a vertical cantilever (Figure 3.2), in accordance with the fact that the frame is a sub-assemblies of a structure composed by monolithic columns fixed at the base and pinned beams not able to provide lateral stability, whose fundamental period of vibration is 1.2s. The low residual displacements (about 6% of the maximum top displacement, see Figure 3.1) obtained up to the drift 2%, is therefore just due to the pseudo-elastic response of the system, rather than a good seismic behavior. The dissipation, in fact, is almost negligible, so that the associated equivalent viscous damping, calculated in accordance with the equation: $\xi_{eq} = E_d / (4\pi \cdot E_{el})$, where E_d is the hysteretic damping and E_{el} is the elastic energy of each cycle, is characterised by values up to 8.9%, which are decreasing with the target drift, due to the narrowed shape of the cycles (see Figure 3.1).

The collapse of the frame was reached at the second cycle of drift level 3%, through the failure of the vertical steel bars of the connection at the second floor for an interstorey shear of 144 kN. The design strength of the two steel bars and the shear strength related to the incipient damage of the concrete were 129 kN and 33 kN, respectively (corresponding to a base shear of 347 kN and 89 kN). It has to highlight that the structure resist,

during the first cycle at 3% drift, an interstorey shear of 168 kN. Before the collapse, a relevant damage of the concrete of the corbels was observed since the drift 0.5%; the cracks gradually developed until partial expulsions of the corners of the corbels or of the top of the columns were reached (see Figure 3.3).
The ends of the beams are subjected to rotations that generated a global deformed shape of the beams closer to a double bending system, rather than simply supported beams. After an horizontal relative displacement of 15mm is reached (the dimension of the gap), however, the rotation is restrained by the column, resulting in a concentration of stresses on the column or additional demand on the connections, depending on the direction of the loads. This situation is emphasized, in real cases, by the additional cast-in-place topping on the beams and on the slabs.

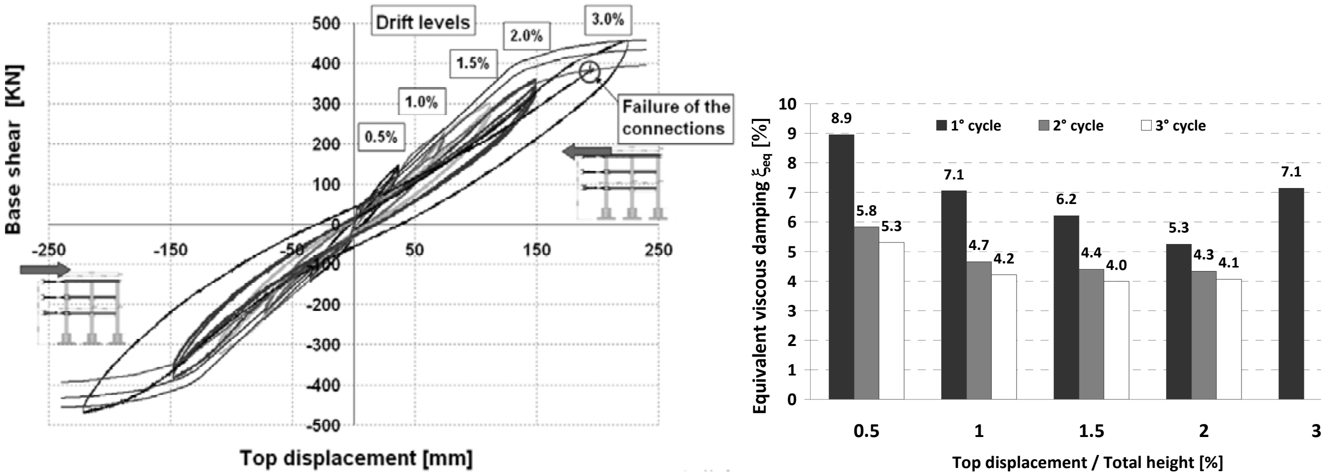


Figure 3.1 – Base shear – top displacement experimental curves (left); equivalent viscous damping (right)

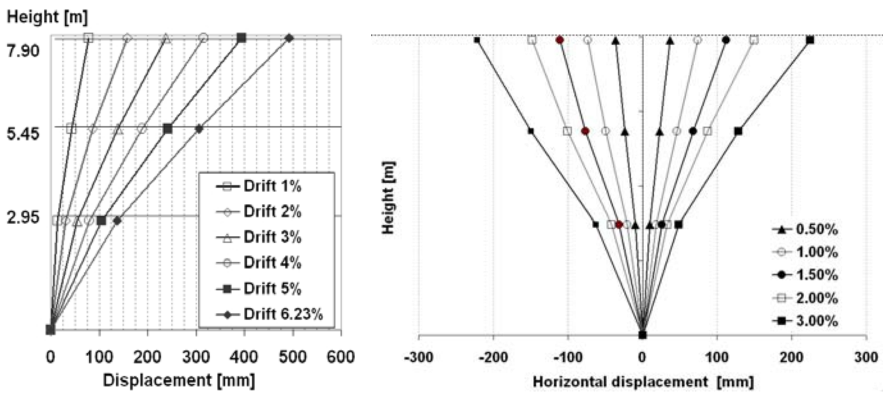


Figure 3.2 – Lateral deformed shape: numerical prediction (left); experimental result (right)

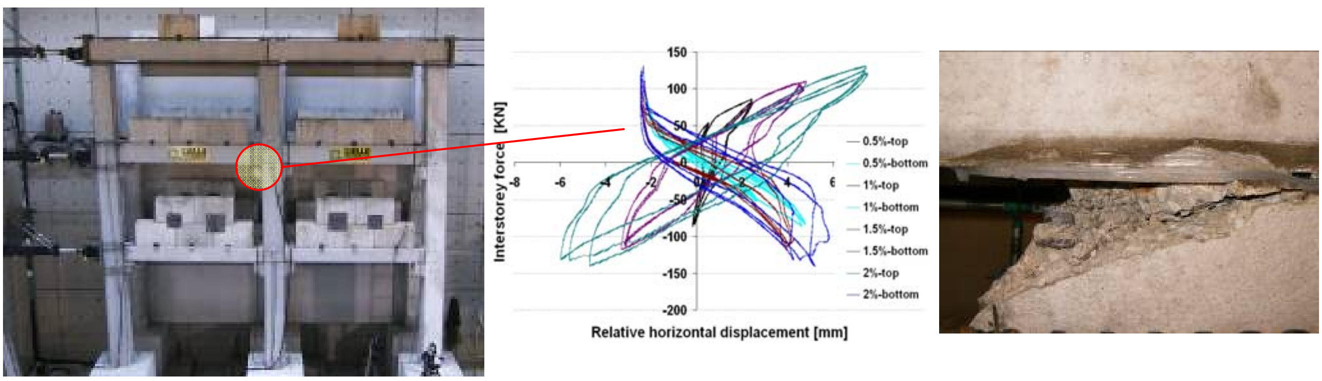


Figure 3.3 – Total lateral deformed shape during the test at drift 2% (left); expulsion of the concrete at a support (centre); interstorey shear – relative top and bottom horizontal displacement curves of a connection (right).

The R_μ reduction factor of the frame, determined in accordance with the procedure proposed by Miranda (2000) and under the hypothesis of infinite strength of the connections, is equal to 3.2 and 3.1 for stiff and dense soil, respectively, and for a displacement ductility $\mu_\Delta = 3$. If the collapse mechanisms due to the connections is included, R_μ decreases to values less than 2.
The peak ground acceleration (PGA) necessary to attain a given top displacement has been evaluated both through pushover analysis and incremental dynamic (time-history) analysis (IDA method) with 7 accelerations compatible with the spectrum of the Eurocode 8 defined for dense soil. Although the structure can resist PGA values greater than 0.5g, the failure of the connections is predicted from values equal to 0.30 ÷ 0.35g. The IDA method is strongly influenced by the main frequencies of the accelerograms and the characteristics of the sytructure.

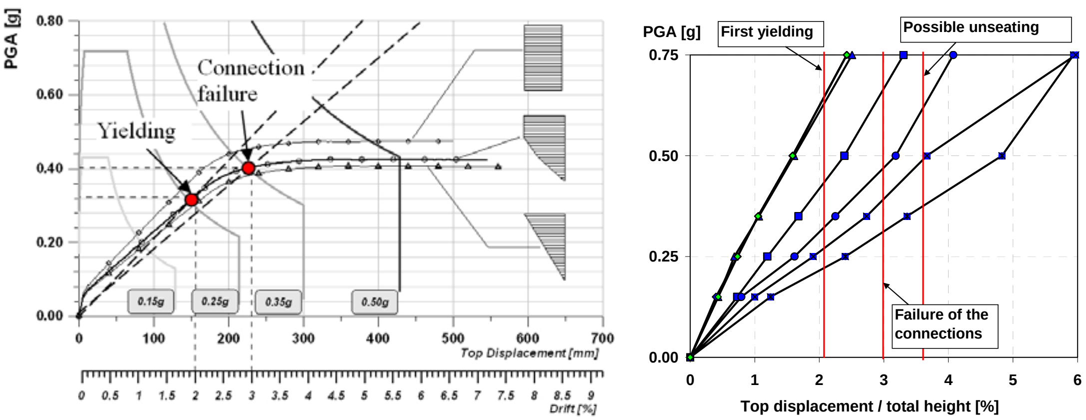


Figure 3.4 – Pushover curves for different lateral force profiles in the plane acceleration – displacement (left); acceleration – displacement curves obtained from the incremental dynamic analysis method with 7 accelerograms compatible with the spectrum defined in the Eurocode 8 for dense soil (right)

The socket foundations experimentally confirmed that the design method is characterized by a high capacity to demand ratio (> 2). The failure, in fact, was generated by a large horizontal crack between the basement and the back wall, but the walls were substantially undamaged (Figure 3.5). This means that the flexural domain of the basement depicted in the Figure 2.4 is reliable, but the horizontal lines (local domains) should be shifted upward.

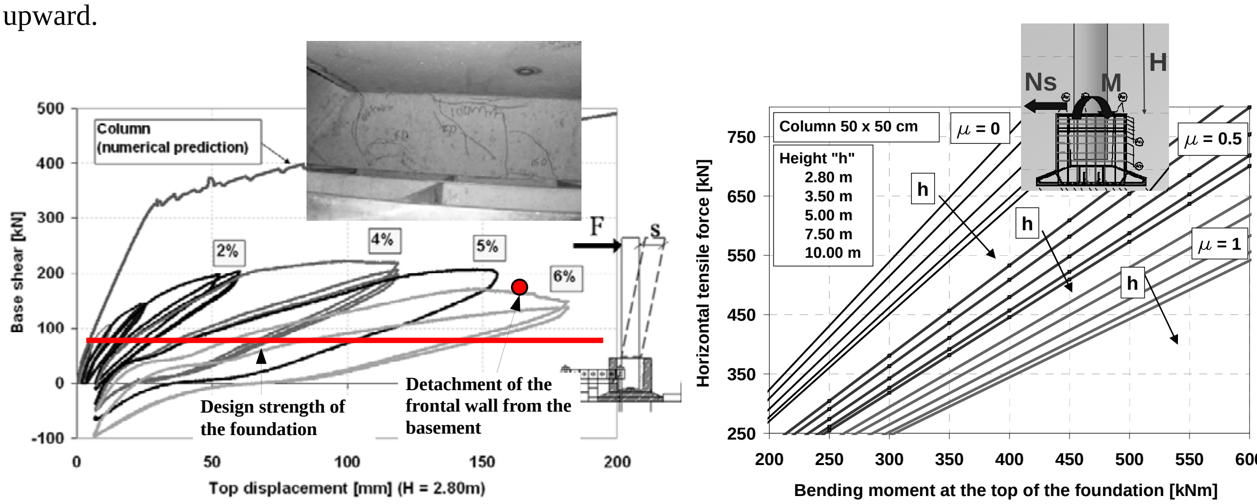


Figure 3.5 – Base shear – top displacement curve of the column of the socket foundation (left); cracking of the frontal wall of the foundation (top left); tensile force – bending moment curves as a function of the friction coefficient and of the height of the column (right)

The column – walls – injection grout system tends to be monolithic, with the exception of the injection grout, which permit to the column to rotate inside the socket foundation, so that, for a bending moment equal to the design value, such rotation generates a top displacement equal to 2% of the height of the column. Nonetheless,

the injection grout is representative of a good quality in-situ solution (Rck 250).
Another reason that justifies the high level capacity of the foundation is the effect due to the friction. If the method proposed by Osanai et al. (1996) is considered, the design capacity increases of 29% or 45% for a friction coefficient μ = 0.5 or 1, respectively. In conclusion, socket foundations designed with the proposed method in medium – high seismicity zones, will be characterized by collapse mechanisms due to rigid overturning or sliding.

4. CONCLUSIONS

The partial results of the research work presented here highlighted the following apects:
i) The displacement ductility μ_Δ of the examined multistorey precast structure is equal to 3 without considering the failure of the connections. On the contrary, it decreases to values less than 2. Low values of the R_μ factor, not greater than 3.5, is therefore associated.
ii) Since the second order effects and the displacement limitations related to the serviceability limit states governs independently of the ductility class, combined with the fact that the lateral stability is provided only by the fixed supports at the base of the columns, the design of this structural typology resulted in an almost elastic behaviour up to a PGA value of 0.3g. In particular, the failure of the connections was reached before the plastic hinges at the base of the columns could develop. The costs of columns with great cross-sections is generally less than innovative seismic solutions, which are considered only for more important or higher structures.
iii) the representation of the beam – column connections through a hinge is very simplifying, because the rotation capacity is limited. This aspect should be considered in the design, in order to avoid the damage at the ends of the beams, particularly if great thicknesses of the cast-in-place topping are used.
iv) The high capacity to demand ratio obtained from the recent design method of the socket foundations has been highlighted, as well as the importance of a good quality for the injection grout, which may lead to additional rotations of the columns equal to the 0.2% of his height.

ACKNOWLEDGEMENTS

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