



**PRECAST VS. CAST-IN-SITU REINFORCED CONCRETE  
INDUSTRIAL BUILDINGS UNDER EARTHQUAKE LOADING:  
AN ASSESSMENT VIA PSEUDODYNAMIC TESTS**

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**SUMMARY**

The paper presents the results of pseudodynamic tests performed on a precast and cast-in-place reinforced concrete structures within the research project Seismic behaviour of reinforced concrete industrial buildings approved in July 2001 for an Ecoleader funding (European Consortium of Laboratories for Earthquake and Dynamic Experimental Research - contract n° HPRI-CT-1999-00059). Both prototypes consisted of two two-bay frames, connected by an interposed hollow-core slab. They were designed according to Eurocode 8 (draft May 2001), in order to withstand the same base shear force and hypothesising that the seismic behaviour of both structures allows to assume for them the same behaviour factor  $q$ , equal to 5. In order to provide a sound experimental evidence for this assumption, the reliability of which has been already widely checked by numerical analyses, pseudodynamic tests have been performed on two prototypes at the European Laboratory for Structural Assessment of the Joint Research Centre at Ispra. The results are of the utmost importance since, on one hand they provide a confirmation of the above said hypotheses, and, on the other, they stand as a dedicated reference for a proper calibration of the Eurocode 8 design rules.

**INTRODUCTION**

The attention paid all over the world by both research and design engineers to the behaviour of prefabricated concrete buildings under earthquake loads has been continuously growing over the past thirty years. A huge amount of work has been dedicated to the several different aspects of this broad topic and a consequent significant improvement of the knowledge has been gained. Despite these efforts, code writers and building officials have almost always looked with some scepticism at prefabricated concrete systems for earthquake resistance in buildings. As a matter of fact, many of the seismic design standards that are accepted for precast concrete buildings in several countries, introduce considerable conservatism often “to such an extreme cost penalty that the system no longer warrants consideration” (Englekirk, 1982). This scepticism was originally attributable to a lack of or limited knowledge about the seismic behaviour of prefabricated concrete systems and it was often recognised, over the past years that only with

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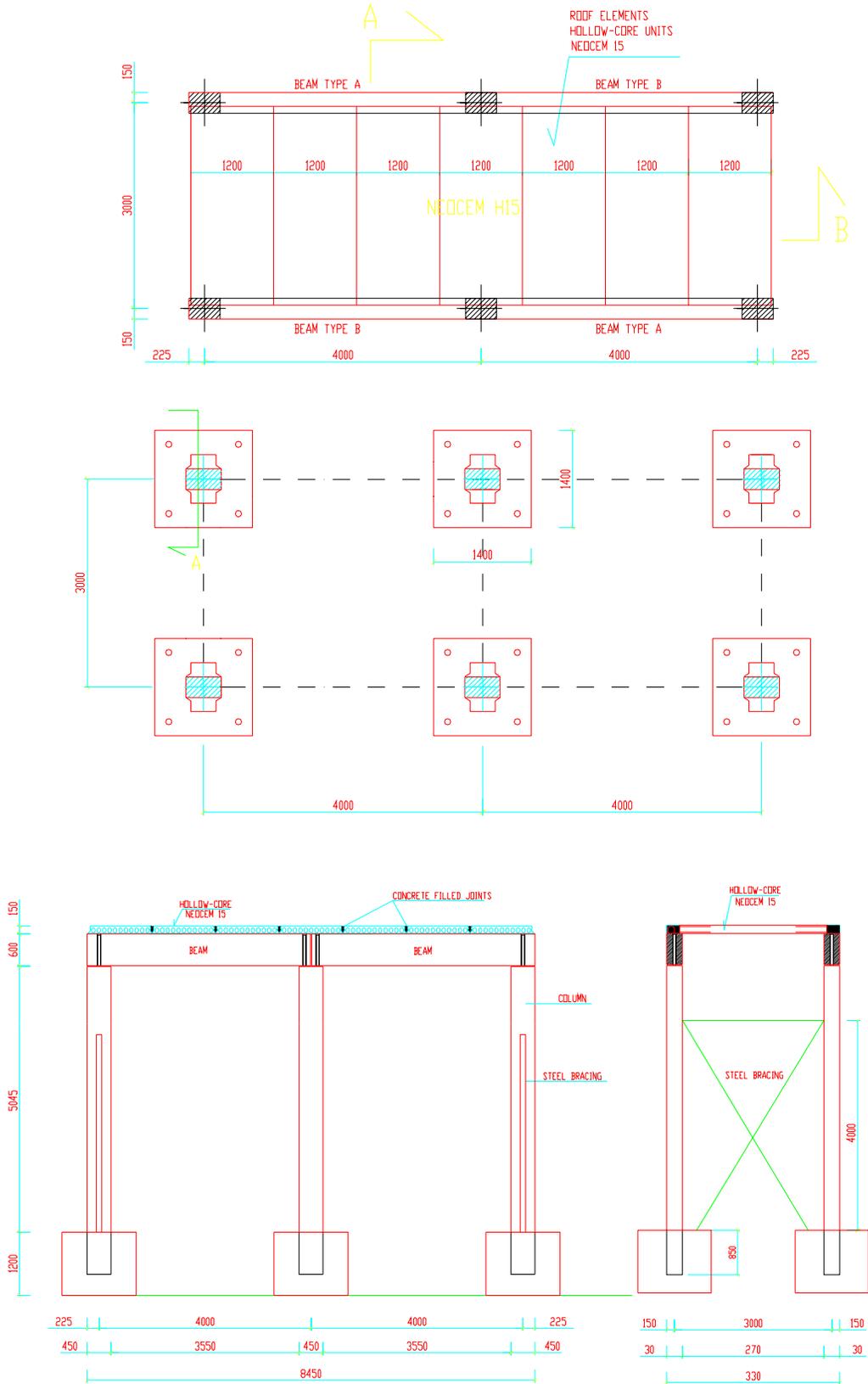
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the support of a good technical data base it was possible to achieve that confidence which would make prefabricated concrete buildings economically feasible even in regions of high seismic intensity. The very recent issue of the Fip Bulletin 27 “Seismic design of precast concrete building structures - State of art report” (2004) comes to fill this gap and the report itself stands as a clear witness of the huge amount of work which has been done over the past years in order to build up such a database. The basis for the whole work were obviously the lessons from past earthquakes. The analysis of the performance of existing prefabricated concrete buildings allowed to identify the main aspects of their seismic behaviour, from the proper definition of seismic input, to the design and detailing of structural members, joints and connections for earthquake resistance and ductility, to the prescriptions for an adequate development of diaphragm action by slabs and roof elements, as well as to specific provisions for cladding and other non structural elements. It was furthermore possible to draw from the above said analyses information about the seismic reliability of the adopted design methods and detailing procedures, also pointing out, within the above recalled fields, those problems needing further investigation before current knowledge may come together to suitable code prescriptions.

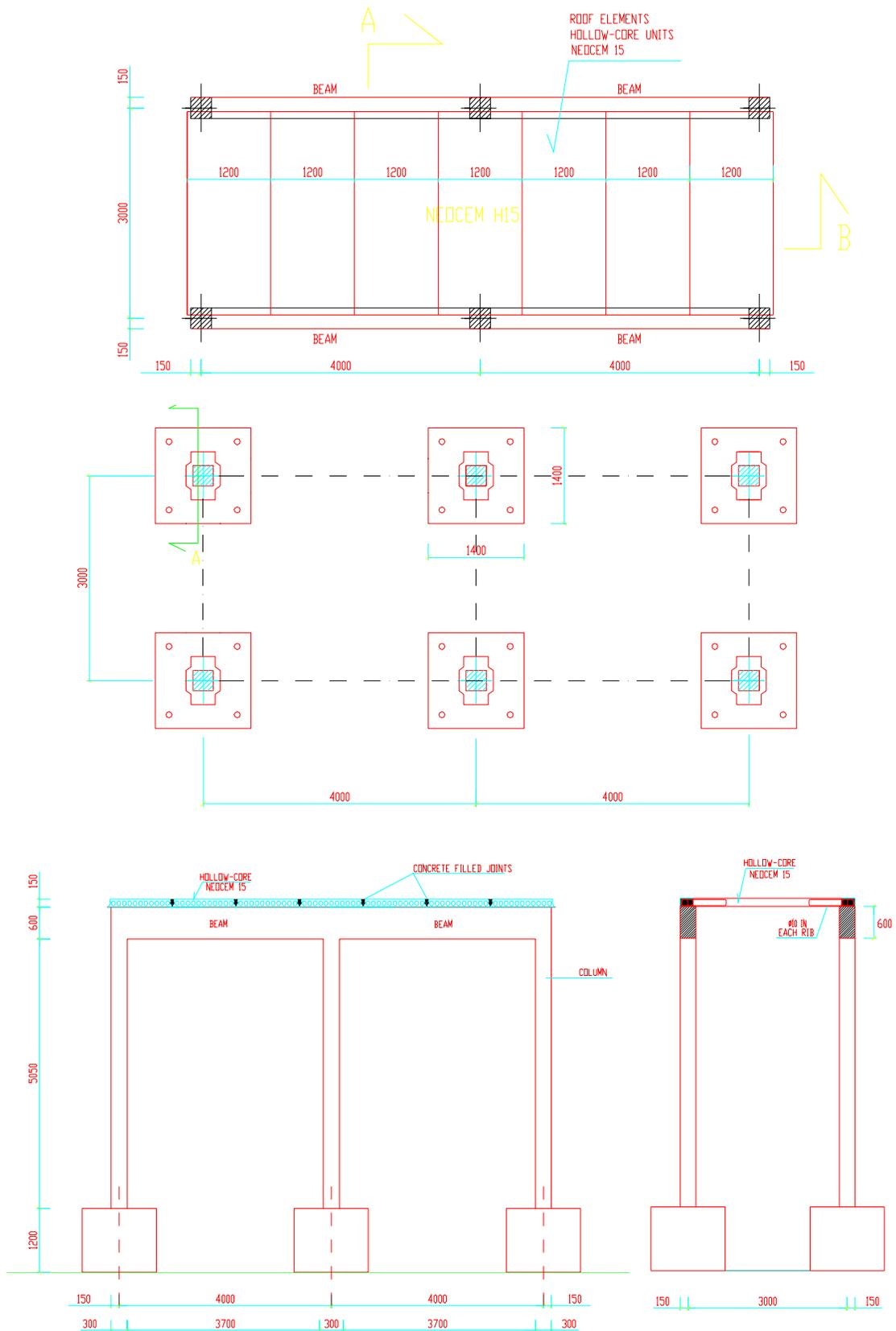
In the above said framework the one-storey precast concrete structures for industrial buildings, generally consisting of prestressed beams connected through dry hinged-joints to the top of columns, have been somewhat neglected, despite they are largely spread in many countries and, at least as far the Italian experience the authors are familiar to, they cover almost all the market of industrial building structures.

The European seismic code Eurocode 8 provides a section dedicated to precast concrete buildings (sect. 5.11): the evaluation of seismic behaviour and energy dissipation capacities of prefabricated concrete systems, which is in some way summarised within the behaviour factor  $q$ , is made by means of analogies with the behaviour of similar cast-in-place structures. Dedicated prescriptions for one-storey precast concrete structures, as far the evaluation of their behaviour factor, are given where it is explicitly specified (§ 5.1.2) that “*one storey frames with column tops connected along both main directions of the building and with the value of the column normalised axial load  $v_d$  nowhere exceeding 0,3 do not belong*” to the category of inverted pendulum systems. The above quoted statement, which exactly refers to the structure type here considered, clearly highlights the great importance of a proper design and detailing of connections as well as of provisions able to guarantee a reliable development of diaphragm action by precast slabs and roof elements, as far the seismic behaviour of prefabricated concrete structures and its analogy with the one of similar cast-in-place ones is dealt with. Lessons from past earthquakes have shown that well detailed and designed, in the sense above specified, precast concrete one-storey structures have a very good seismic behaviour. Their quite low translational stiffness, and the consequent quite long vibration period – ranging from 1 to 2 seconds – is instrumental at significantly lowering the seismic acceleration of the structure with respect to the ground one. Furthermore this type of structures is provided a large redundancy of resistance, since requirements imposed by the damage limitation state are generally more stringent than those by ultimate limit ones.

Besides these evidences, no practical application exists to assess the reliability of Eurocode 8 design rules. Furthermore an adequate calibration of the behaviour factor, based on dedicated experimental and theoretical/numerical investigations is needed. In this context, and as a natural prosecution of a previous experimental research focused on the behaviour of single precast concrete columns (Saisi and Toniolo, 1998), the research project “Seismic behaviour of reinforced concrete industrial buildings” can be framed. The project is aimed at providing dedicated experimental results as far the above referred topic is dealt with, and as a confirmation of the Eurocode 8 prescriptions, which substantially regarded, within the above specified limitations, one-storey precast concrete structures as equivalent to analogous cast-in-place ones, with a basic value of the behaviour factor  $q$  equal to 4.5. To this purpose two prototypes, a precast and a cast-in-place one, have been designed and built, as it will further explained in details, and submitted to a series of pseudodynamic tests to assess their seismic behaviour. This paper first provides a detailed report of the experimental results (see also Ferrara 2003 a-b) and a cross-examination of them in order to assess the equivalence between the seismic behaviour of the two investigated types of structures.



**Figure 1:** precast prototype: deck and foundation plans; side and front views



**Figure 2:** cast-in-situ prototype: deck and foundation plans; side and front views

## DESIGN OF THE PROTOTYPES AND ASSESSMENT OF PREDICTABLE RESPONSE

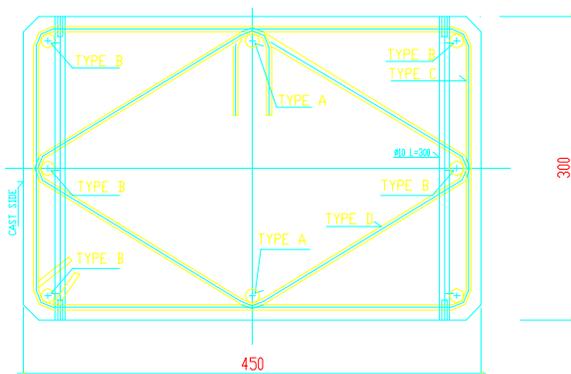
Both prototypes, the prefabricated and the cast in place one, consisted of two two-bay frames, each bay spanning 4 m, connected by an interposed hollow core slab, spanning 3 m. The clear height of columns measured 5,05 m from the edge of the footing socket. Figure 1 and 2 give a plan of deck and foundations and the side and front views of the precast and of the cast-in-place prototype respectively. Precast foundation sockets were used in both cases, tied by means of Diwidag bars to the floor of the laboratory.

The prototypes had to be representative of true structures with real dimensions, as common in the practice (for example beams spanning 12 m and slabs spanning 6m); for economy's sake the span of slabs and beams, which have not a direct influence on the seismic behaviour of the prototypes, have been scaled as above said. Missing weights, which are needed to induce the required values of axial loads in the columns, have been applied by means of vertical jacks through a suitable load distributing device.

The design of the prototypes has been performed in accordance with prescriptions of Eurocode 8 (draft May 2001) so that both structures are able to withstand the same base shear force equivalent to the earthquake. The following material properties have been assumed in design calculations:

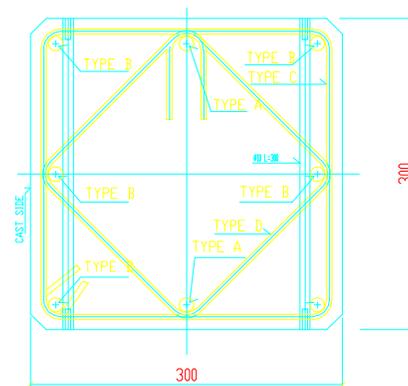
- concrete class C40/50  
characteristic compressive strength  $f_{ck} = 40 \text{ MPa}$   
design compressive strength  $f_{cd} = \frac{f_{ck}}{\gamma_c} = \frac{40 \text{ MPa}}{1.5} = 26,7 \text{ MPa}$
- steel B500 H  
characteristic yielding strength  $f_{yk} = 500 \text{ MPa}$   
design yielding strength  $f_{sd} = \frac{f_{yk}}{\gamma_s} = \frac{500 \text{ MPa}}{1.15} = 435 \text{ MPa}$

A classical design for non seismic conditions (snow and wind loads, bridge cranes etc.) yielded to the dimensions and reinforcement areas for the cross section of the columns shown in Figures 3-4:



$$A_s = 8 \phi 16 = 1608 \text{ mm}^2 (\rho_s = 1,19\%)$$

**Figure 3:** cross section of prefabricated columns



$$A_s = 8 \phi 14 = 1232 \text{ mm}^2 (\rho_s = 1,37 \%)$$

**Figure 4:** cross section of cast-in-place columns

Transverse reinforcement, seized as shown in Figures 3-4, consisted of stirrups  $\phi 6$  spaced 50 mm in critical zones of columns of both prototypes and  $\phi 6$  spaced 150 mm outside those zones. The length of critical zones was determined as 1 m from the bottom edge of the columns in the precast prototype and 1 m from both bottom and top edges in the cast-in-place ones.

Assuming a distributed vertical load equal to  $30 \text{ kN/m}^2$  (inclusive of the self weight of slabs and beams), which also accounts for the equivalence of the actual dimensions of the structures to the more realistic above referred ones, the following values of axial load in the columns have been obtained:

- *precast prototype* lateral columns  $N_{ad} = 90 \text{ kN}$  ( $v_{ad} = 0,025$ )  
central columns  $N_{ad} = 180 \text{ kN}$  ( $v_{ad} = 0,07$ )
- *cast-in-situ prototype* lateral columns  $N_{ad} = 67,5 \text{ kN}$  ( $v_{ad} = 0,025$ )  
central columns  $N_{ad} = 225 \text{ kN}$  ( $v_{ad} = 0,08$ )

For cast-in-situ prototype, only the effects of the beam redundancy and not the unbalancement of axial load in external columns due to seismic action has been considered.

The following design procedure, as classical in the practice of precast companies, was adopted. For the above referred values of axial loads in the columns, the design values of the resistant bending moment  $M_{rd}$  has been calculated for the critical column cross sections, through usual limit state calculations. The rotational stiffness of the cracked cross sections has been obtained as  $\kappa_{\phi} = M_y/\chi_y$ , where the yielding value of the bending moment,  $M_y$ , has been assumed equal to  $0,75 M_{rd}$  and the corresponding curvature  $\chi_y$  has been computed through an elastic cracked cross-section analysis. The translational stiffness of each column, reduced to account for second order effects of axial loads, has been hence computed as:

- *precast prototype*  $\kappa_{\delta} = \frac{3 \kappa_{\phi}}{h^3} - \frac{N_{ad}}{h}$
- *cast-in-situ prototype*  $\kappa_{\delta} = \frac{12 \kappa_{\phi}}{h^3} - \frac{N_{ad}}{h}$

Once computed the structure stiffness, by summation of those of each column, the vibration period of the structure has been evaluated as:

$$T_1 = 2\pi \sqrt{\frac{m}{\kappa_{\delta}}}$$

where  $m$  is the effective “vibrating” mass (72000 kg). Consequently, the design response spectrum  $S_d(T_1)$  has been calculated, hypothesising a ground type B and assuming behaviour factor equal to 4,95 for both prototypes. To the design response spectrum a design value of the horizontal base shear force  $E_{ad} = S_d W$  corresponds. By equating this value to the resistant one, simply computed as:

- *precast prototype*  $E_{rd} = \frac{\Sigma M_{rd}}{h}$
- *cast in situ prototype*  $E_{rd} = \frac{\Sigma M_{rd}}{h}$

the design value of the ground acceleration which would lead the structure to collapse has been finally computed. Table 1 summarises the results of the above explained calculations. In order to have information to be more reliably compared to experimental results as well as to evaluate how realistic is the assumption of fictitiously taking into account strength and stiffness reduction due to cyclic loading through partial safety factors for non seismic conditions, design calculations have been performed with reference to both design and characteristic values (i.e. with unit partial safety factors) of material properties. These reference values have been by the way checked by usual compliance tests, i.e. compression tests on companion cube specimens, 150 mm side, and tension tests on reinforcing bar segments, 600 mm long. At the age of testing the following results have been obtained:

- **concrete**
- *precast prototype*: mean cubic strength (2 cubes)  $R_c = 52.1 \text{ MPa}$   
mean compressive strength  $f_c = 0,83 R_c = 43,2 \text{ MPa}$
- *cast-in-situ prototype*: mean cubic strength (3 cubes)  $R_c = 51.5 \text{ MPa}$   
mean compressive strength  $f_c = 0,83 R_c = 42,7 \text{ MPa}$
- **steel**
- yielding strength (1 bar  $\phi$  16)  $f_y = 550 \text{ MPa}$
- tensile strength  $f_t = 657 \text{ MPa}$

Design calculations repeated with actual values of material properties and still assuming unit values for partial safety factors, yielded the results which are also summarised in Table 1 and which have been further confirmed by numerical predictions (Biondini and Toniolo, 2004).

The seismic action applied during pseudodynamic tests has been hence calibrated with reference to these last computed design value of ground acceleration, as it will be further explained.

<b>Design performed with <math>f_{c1} = 26,7</math> MPa and <math>f_{sd} = 435</math> MPa</b>			
<i>Precast prototype</i>		<i>Cast-in-situ prototype</i>	
<i>Central columns</i>	<i>Lateral columns</i>	<i>Central columns</i>	<i>Lateral columns</i>
$N_{ad} = 180$ kN	$N_{ad} = 90$ kN	$N_{ad} = 225$ kN	$N_{ad} = 67,5$ kN
$M_{rd} = 172$ kNm	$M_{rd} = 156$ kNm	$M_{rd} = 93,2$ kNm	$M_{rd} = 75,6$ kNm
$M_y = 129$ kNm	$M_y = 117$ kNm	$M_y = 69,9$ kNm	$M_y = 56,7$ kNm
$\chi_y = 5,944e-3$ m <sup>-1</sup>	$\chi_y = 5,927e-3$ m <sup>-1</sup>	$\chi_y = 9,405e-3$ m <sup>-1</sup>	$\chi_y = 9,375e-3$ m <sup>-1</sup>
$\kappa_\phi = 21702$ kNm <sup>2</sup>	$\kappa_\phi = 19742,7$ kNm <sup>2</sup>	$\kappa_\phi = 7432,171$ kNm <sup>2</sup>	$\kappa_\phi = 6047,929$ kNm <sup>2</sup>
$\kappa_\delta = 469,8939$ kN/m	$\kappa_\delta = 442,0446$ kN/m	$\kappa_\delta = 647,9504$ kN/m	$\kappa_\delta = 550,1595$ kN/m
<b><math>\kappa_\delta = 2707,9662</math> kNm</b>		<b><math>\kappa_\delta = 3496,5388</math> kNm</b>	
<b><i>Vibration period <math>T = 1,02</math> seconds</i></b>		<b><i>Vibration period <math>T = 0,90</math> seconds</i></b>	
<b><i>Design response spectrum <math>S_d = 0,296 \alpha_g</math></i></b>		<b><i>Design response spectrum <math>S_d = 0,336 \alpha_g</math></i></b>	
<b><math>E_{rd} = 192</math> kN - <math>\alpha_{grd} = 0,91</math> g</b>		<b><math>E_{rd} = 194</math> kN - <math>\alpha_{grd} = 0,80</math> g</b>	

<b>Design performed with <math>f_{ck} = 40</math> MPa and <math>f_{yk} = 500</math> MPa</b>			
<i>Precast prototype</i>		<i>Cast-in-situ prototype</i>	
<i>Central columns</i>	<i>Lateral columns</i>	<i>Central columns</i>	<i>Lateral columns</i>
$N_{ad} = 180$ kN	$N_{ad} = 90$ kN	$N_{ad} = 225$ kN	$N_{ad} = 67,5$ kN
$M_{rd} = 195,3$ kNm	$M_{rd} = 178,8$ kNm	$M_{rd} = 105,4$ kNm	$M_{rd} = 87,3$ kNm
$M_y = 146,475$ kNm	$M_y = 134,1$ kNm	$M_y = 79,05$ kNm	$M_y = 65,5$ kNm
$\chi_y = 6,927e-3$ m <sup>-1</sup>	$\chi_y = 6,889e-3$ m <sup>-1</sup>	$\chi_y = 11,077e-3$ m <sup>-1</sup>	$\chi_y = 10,987$ m <sup>-1</sup>
$\kappa_\phi = 21146,23$ kNm <sup>2</sup>	$\kappa_\phi = 19461,12$ kNm <sup>2</sup>	$\kappa_\phi = 7136,538$ kNm <sup>2</sup>	$\kappa_\phi = 5959,473$ kNm <sup>2</sup>
$\kappa_\delta = 456,9402$ kN/m	$\kappa_\delta = 435,5942$ kN/m	$\kappa_\delta = 620,4041$ kN/m	$\kappa_\delta = 541,9174$ kN/m
<b><math>\kappa_\delta = 2656,2572</math> kNm</b>		<b><math>\kappa_\delta = 3408,4778</math> kNm</b>	
<b><i>Vibration period <math>T = 1,04</math> seconds</i></b>		<b><i>Vibration period <math>T = 0,92</math> seconds</i></b>	
<b><i>Design response spectrum <math>S_d = 0,293 \alpha_g</math></i></b>		<b><i>Design response spectrum <math>S_d = 0,332 \alpha_g</math></i></b>	
<b><math>E_{rd} = 219</math> kN - <math>\alpha_{grd} = 1,04</math> g</b>		<b><math>E_{rd} = 222</math> kN - <math>\alpha_{grd} = 0,92</math> g</b>	

<b>Design performed with <math>f_c = 43,2</math> (42,7) MPa and <math>f_y = 550</math> MPa</b>			
<i>Precast prototype</i>		<i>Cast-in-situ prototype</i>	
<i>Central columns</i>	<i>Lateral columns</i>	<i>Central columns</i>	<i>Lateral columns</i>
$N_{ad} = 180$ kN	$N_{ad} = 90$ kN	$N_{ad} = 225$ kN	$N_{ad} = 67,5$ kN
$M_{rd} = 211,2$ kNm	$M_{rd} = 194,7$ kNm	$M_{rd} = 113$ kNm	$M_{rd} = 95,1$ kNm
$M_y = 158,4$ kNm	$M_y = 146,025$ kNm	$M_y = 84,75$ kNm	$M_y = 71,325$ kNm
$\chi_y = 7,559e-3$ m <sup>-1</sup>	$\chi_y = 7,527e-3$ m <sup>-1</sup>	$\chi_y = 12,054e-3$ m <sup>-1</sup>	$\chi_y = 12,01e-3$ m <sup>-1</sup>
$\kappa_\phi = 20594,77$ kNm <sup>2</sup>	$\kappa_\phi = 19400,26$ kNm <sup>2</sup>	$\kappa_\phi = 7031,103$ kNm <sup>2</sup>	$\kappa_\phi = 5938,803$ kNm <sup>2</sup>
$\kappa_\delta = 452,4802$ kN/m	$\kappa_\delta = 434,091$ kN/m	$\kappa_\delta = 610,58$ kN/m	$\kappa_\delta = 540$ kN/m
<b><math>\kappa_\delta = 2641,3244</math> kNm</b>		<b><math>\kappa_\delta = 3381,126</math> kNm</b>	
<b><i>Vibration period <math>T = 1,04</math> seconds</i></b>		<b><i>Vibration period <math>T = 0,829</math> seconds</i></b>	
<b><i>Design response spectrum <math>S_d = 0,292 \alpha_g</math></i></b>		<b><i>Design response spectrum <math>S_d = 0,33 \alpha_g</math></i></b>	
<b><math>E_{rd} = 238</math> kN - <math>\alpha_{grd} = 1,13</math> g</b>		<b><math>E_{rd} = 240</math> kN - <math>\alpha_{grd} = 1,01</math> g</b>	

**Table 1:** summary of design calculations – no-collapse limit state

As far the check against damage limitation state, according to Italian seismic code, the elastic design spectrum scaled by a factor equal to 2.5 has been assumed for the calculation of displacements  $d_r$  at the deck level. A “rough” computation has been performed, for each of the four levels of seismic intensity prescribed by the Italian code, with reference to global translational stiffness, as calculated before considering actual values of material properties and unit partial safety factors. As remarked in the introduction, design calculations show that the two structures at issue have a large redundancy of seismic resistance against ultimate limit states (no-collapse), the requirements imposed by the damage limitation state being far more stringent than the previous ones.

<i>Seismic intensity</i>	<i>Precast - <math>S_d(T_1) = 0,578 \alpha_g</math></i>	<i>Cast-in-situ - <math>S_d(T_1) = 0,653 \alpha_g</math></i>
$\alpha_g = 0,05$ g (cat. IV)	$E_{ad} = S_d W = 20,81$ kN $d_r \cong 8$ mm < 0,01 h = 50,5 mm	$E_{ad} = S_d W = 23,51$ kN $d_r = E_{ad} / \kappa_\delta \cong 7$ mm < 0,01 h = 50,5 mm
$\alpha_g = 0,15$ g (cat. III)	$E_{ad} = S_d W = 62,42$ kN $d_r = E_{ad} / \kappa_\delta = 23,6$ mm < 0,01 h	$E_{ad} = S_d W = 70,52$ kN $d_r = E_{ad} / \kappa_\delta = 20,9$ mm < 0,01 h
$\alpha_g = 0,25$ g (cat. II)	$E_{ad} = S_d W = 104,04$ kN $d_r = E_{ad} / \kappa_\delta = 39,4$ mm < 0,01 h	$E_{ad} = S_d W = 117,54$ kN $d_r = E_{ad} / \kappa_\delta = 34,8$ mm < 0,01 h
$\alpha_g = 0,35$ g (cat. I)	$E_{ad} = S_d W = 145,66$ kN $d_r = E_{ad} / \kappa_\delta \cong 54$ mm $\cong$ 0,01 h	$E_{ad} = S_d W = 164,56$ kN $d_r = E_{ad} / \kappa_\delta = 48,6$ mm < 0,01 h

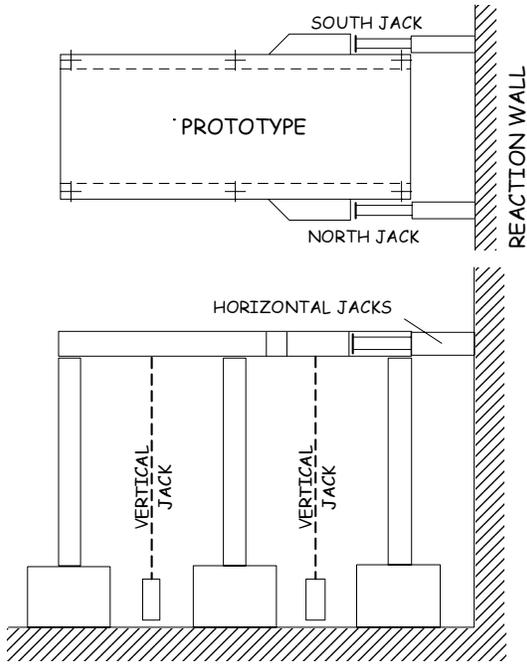
**Table 2.** check of damage limitation state for precast and cast-in-situ prototypes

## TEST SET-UP AND TESTING PROCEDURE

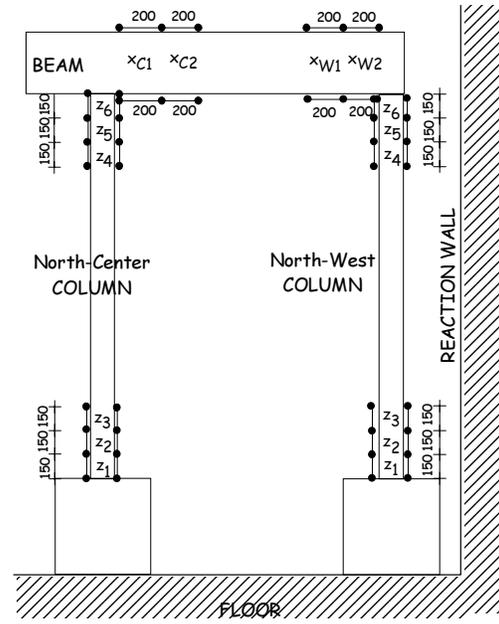
Figure 5 shows a scheme of the set-up adopted for pseudodynamic tests of both prototypes. Horizontal actuators were commanded according to a master-slave scheme in order to imposed a uniform translation to the deck, thus minimizing the effects of accidental torsion, which was absolutely neglected in the design. The actuators were connected through spherical hinges to a suitable shaped cast-in-place enlargement of the peripheral curb of the deck.

Imposed deck displacement were measured at the opposite side by means of transducers placed on a fixed reference frame, so to minimize errors due to local deformations of the structure close to loading devices and of the devices themselves. Besides displacements and reaction forces, a dedicated set of instruments was placed at the bottom edge of prefabricated columns, and at the bottom and top edges of cast-in-place ones, as shown in Figures 6-7, to measure the distribution of curvatures in critical zones of columns themselves. Actually four columns of the precast prototype were instrumented on both faces (all the three ones of a frame and just the central one of the other) while only two columns (one central and one lateral) were provided with curvature measuring instruments at the bottom and top edges; this for economy's sake in the data acquisition system and also taking into account the data recorded for the precast prototype, which showed a perfectly symmetric behaviour of the two frames the prototype consisted of.. The system controlling the whole test, and actually performing the numerical part of it, also provided values of velocities and accelerations of the deck and of the different energy-like quantities featuring the evolution of the structure behavior under the simulated earthquakes.

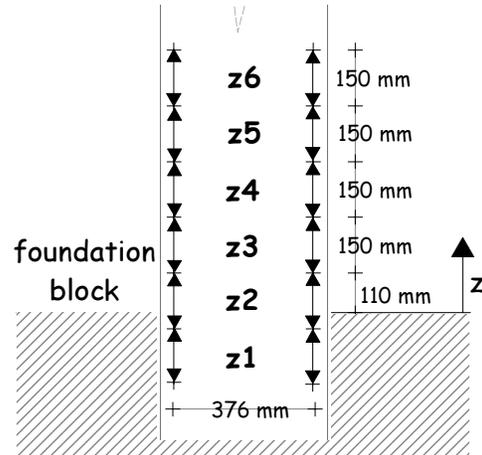
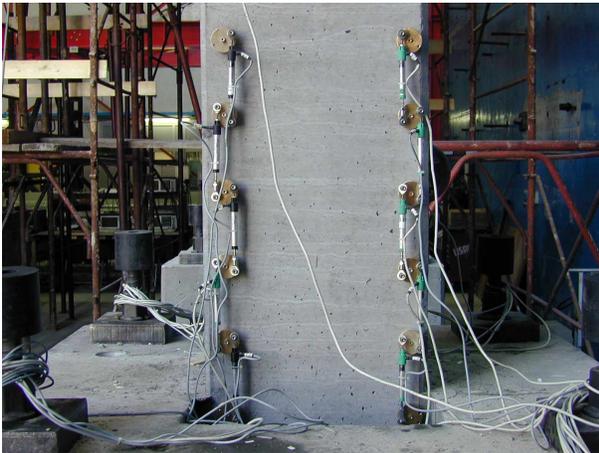
As far this last, seismic ground motion has been assigned through an artificial accelerogram, the spectrum of which was consistent with the one given by Eurocode 8 for ground type B (Figure 8). The seismic intensity was actually calibrated on the previously computed seismic resistant capacity of the structures. Three tests have been performed for each type of structure, fixing the value of the peak ground acceleration (pga) respectively to 1/3, 2/3 and 3/3 of the theoretical maximum one (respectively  $\alpha_g \cong 1,10$  g for the precast prototype and  $\alpha_g \cong 1,00$  g for the cast-in-place one). Since for precast prototype it was not possible to complete the test due to the attainment of the maximum stroke capacity of horizontal jacks, the pga at the third test level for cast-in-place structure was kept equal to 80% of the maximum one.



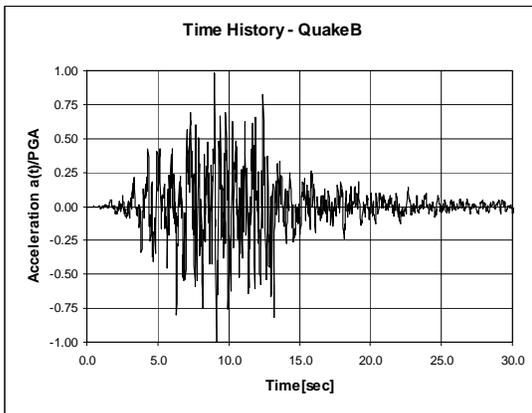
**Figure 5:** scheme of the test set-up



**Figure 6:** scheme of instruments measuring the distribution of curvatures – cast-in-place prototype



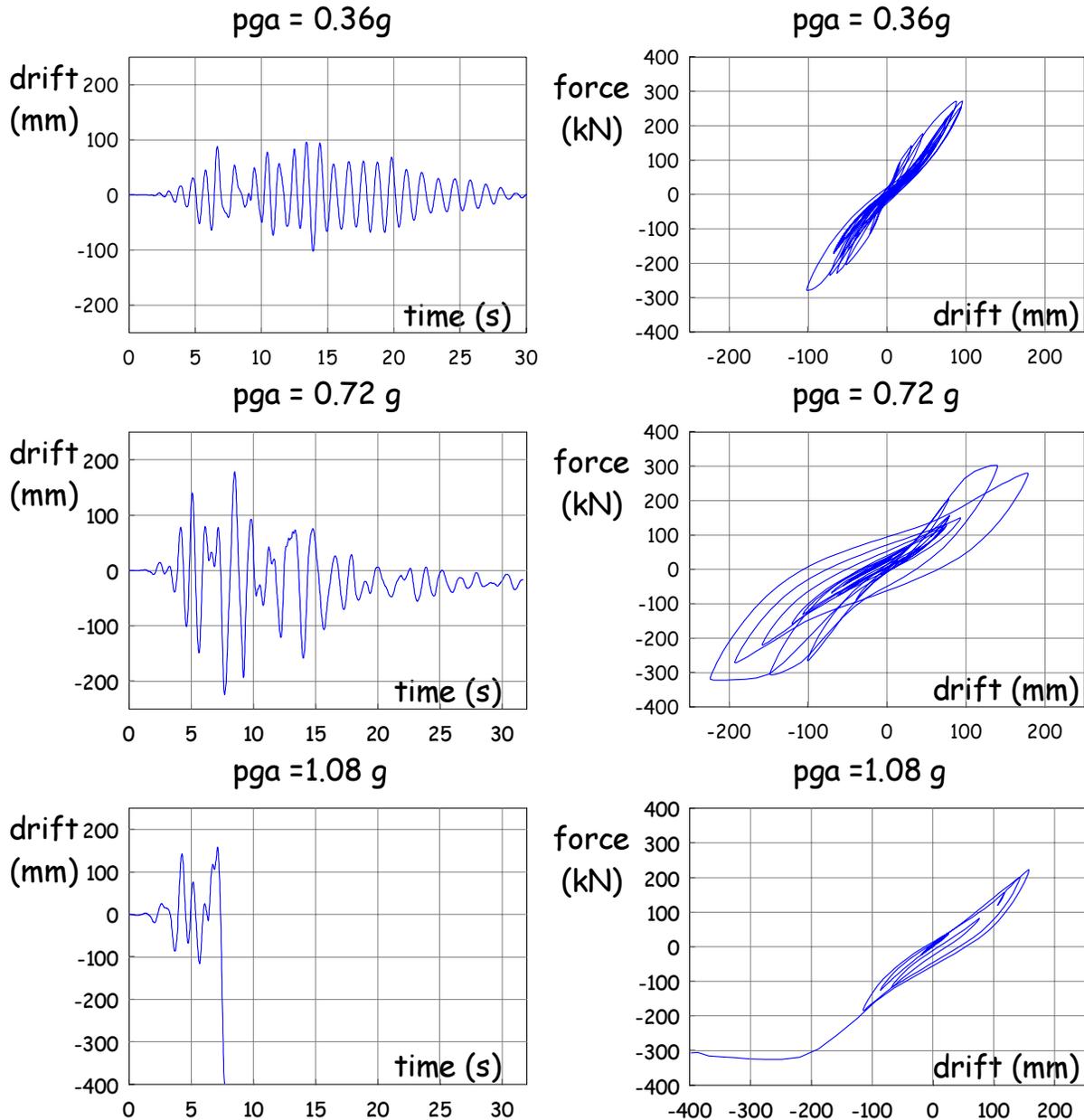
**Figure 7:** instrument to measure the distribution of curvatures at the base of prefabricated columns



**Figure 8:** Artificial employed accelerogram and corresponding spectrum

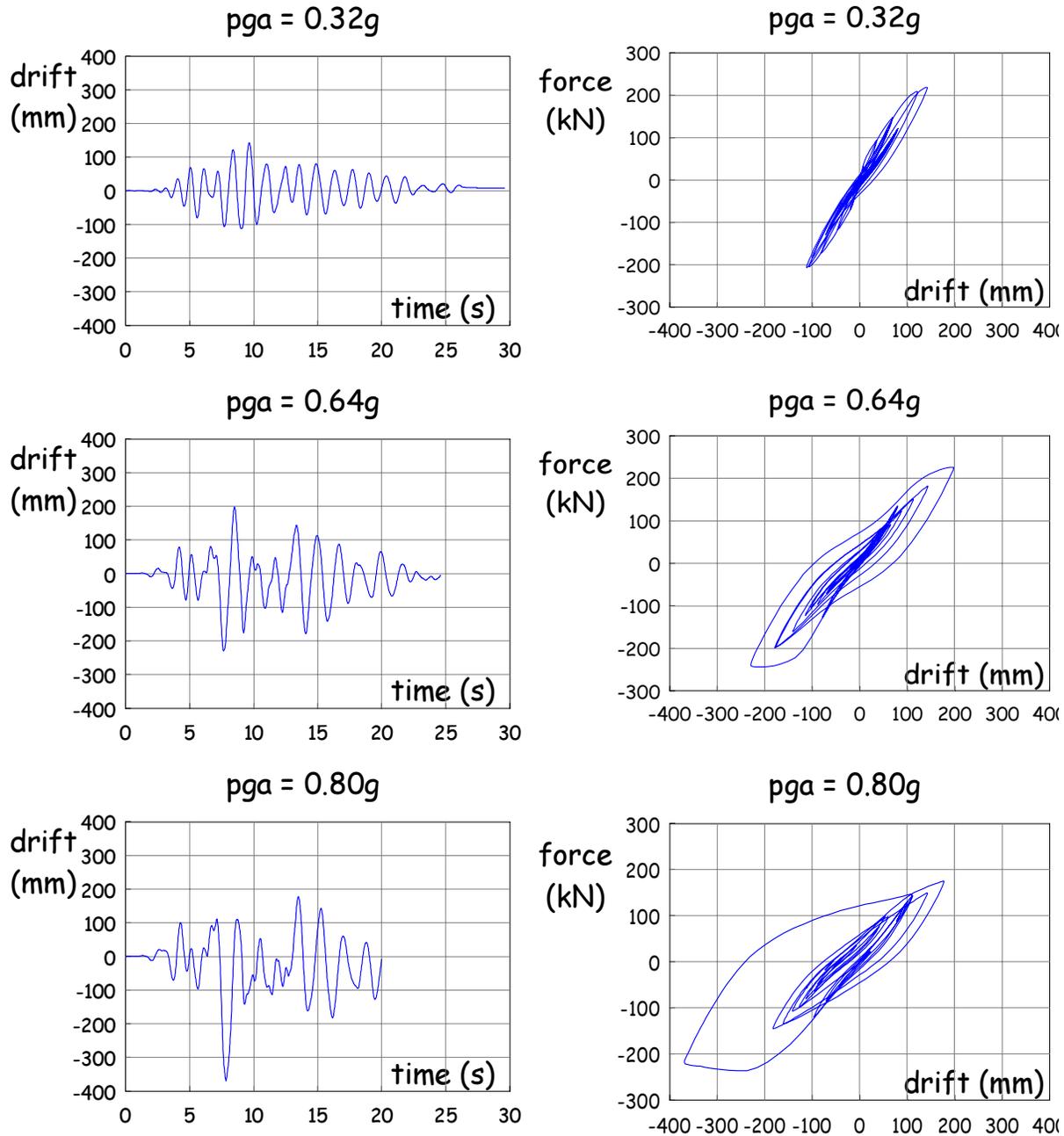
## EXPERIMENTAL RESULTS

A first synoptic view of the experimental behaviour of the two prototypes – for all the three investigated levels of seismic intensity – is here given through the time history of displacements, measured at the deck level as previously specified, and the corresponding force-displacement evolutions (Figures 9 and 10, respectively for the precast and the cast-in-situ prototype).



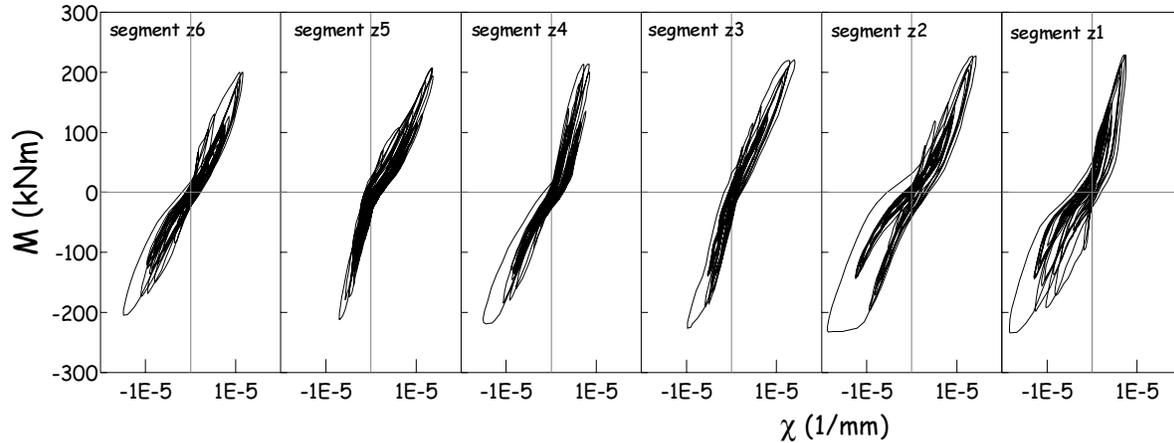
**Figure 9:** time histories of recorded deck displacement and force-displacement evolutions pseudodynamic tests on precast prototype

As far the first level ( $\alpha_g = 0.36$  for precast prototype and  $\alpha_g = 0.32$  for cast-in-situ one) an elastic behaviour has been substantially detected in both cases, with quite “closed” force-displacement cycles, furthermore showing that, under maximum intensity pulses, the onset of steel yielding is attained.

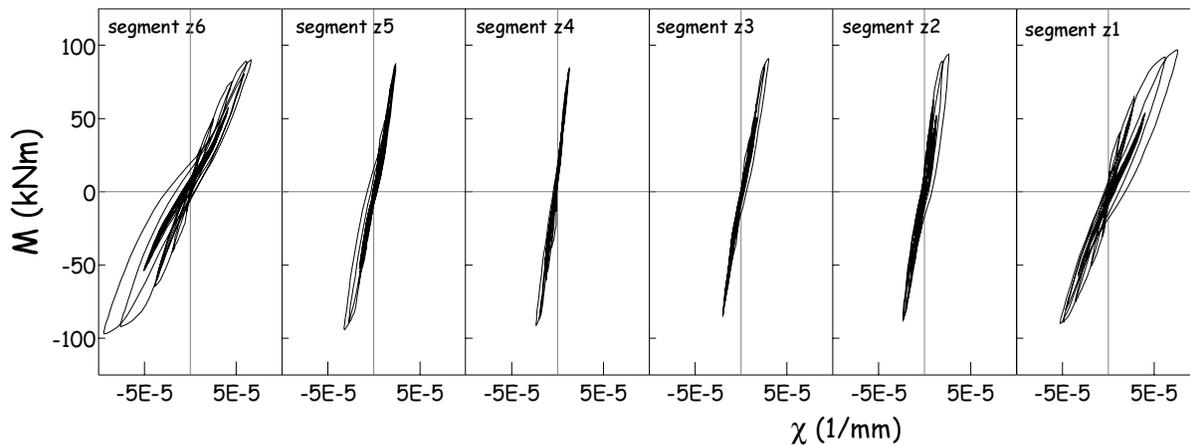


**Figure 10:** time histories of recorded deck displacement and force-displacement evolutions pseudodynamic tests on cast-in-situ prototype

This is confirmed also by moment-curvature diagrams plotted, for one central column of both prototypes in Figures 11 and 12. It has to be remarked that the bending moment for column segments at different heights has been computed referring to the mid-height point of each segment, hypothesising the measured total reaction force to be equally shared among the six columns of the prototypes and applied at the column top-edge. The maximum values of displacements stand, to the one computed for the check of damage limitation state in a 1<sup>st</sup> category seismic zone (actually pgas of the first test level are roughly equivalent to the one prescribed for this zone) in a ratio which is quite close to the behaviour factor 2.5 assumed, as from Italian seismic code, in design calculations, the reliability of which is here confirmed.



**Figure 11:** moment-curvature relationships along the critical zone of prefabricated columns ( $\alpha_g = 0.36$ )



**Figure 12:** moment-curvature relationships along the critical zones of cast-in-place columns ( $\alpha_g = 0.32$ )

No significant residual displacement of the structure nor curvature in critical zones of columns was detected after the complete load removal, despite on one hand the onset of steel yielding and on the other some cracks at column bottom edges open up to a few hundred microns were observed under the attained maximum displacements.

Force-displacement (Figures 10-11) and moment-curvature (Figures 13-14) diagrams of the second test ( $\alpha_g = 0,72$  and  $\alpha_g = 0,64$  for the precast and the cast-in-place structures respectively) show in both cases several cycles of significant hysteresis denoting the full yielding of steel and an appreciable capacity of dissipating energy by the structures, either precast or cast-in-situ, taking profit of the material resources beyond the elastic limits. Some residual displacements were observed in both cases after load removal, as well as some fairly visible cracks in critical zones of columns, as a witness of the irreversible effects of the yielding of steel, cracking of concrete and non-linear behaviour of compressed concrete, as also confirmed by local measurements. The maximum attained value of the shear force was consisted with theoretical predictions for the cast-in-place prototype, while a significantly higher (+20%) value was recorded for the precast one. The differences in casting of columns (horizontal for the latter, obviously vertical for the former) as well as the higher degree of quality control which features the production of prefabricated structural elements, mainly in the detailing of reinforcement, may be probably called as a partial explanation for this (Dimova and Negro, 2004). This topic surely deserves further investigation.

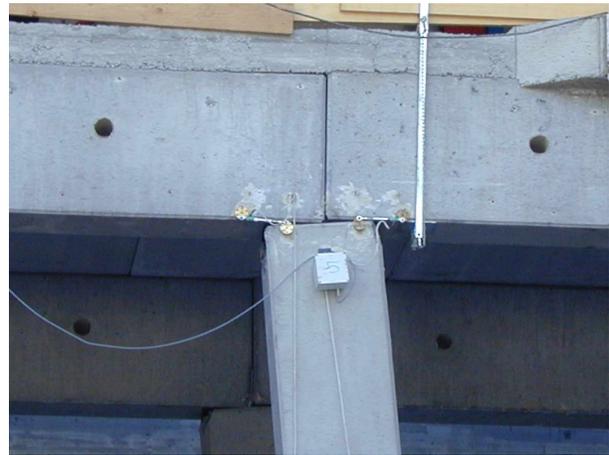
The same statements hold for the structure behaviour under the highest value of seismic intensity. Force-displacement diagrams for the second structure (for the first one the test is somewhat meaningless) show a

very large hysteresis cycle under maximum intensity pulse, with a slight reduction of load capacity after the maximum value was attained, as probably due to second order effects of gravity loads for the quite high attained values of displacement. Some cover rupture occurred at bottom edges of columns after high intensity pulses, but no buckling of reinforcement was observed (Figure 13). Something similar would have featured the behaviour of the precast prototype, if the test could have been completed (Biondini and Toniolo, 2004). The only observed damage was in this case some local cover rupture at column top edges, caused by large relative rotations between beam ends and top edges of columns (Figures 14). All what above said is also confirmed by moment-curvature diagrams in critical zones of columns (Figure 15-16-17). From such graphs it can be estimated the extension of the regions where energy dissipation takes place through material deformation beyond the elastic limit. A rough estimation showed that the largest part of energy dissipation occurred in a zone which hardly proceeds at a distance further than the cross section height from both top and bottom edges of cast-in place columns and one and a half the section height from the bottom edge of the prefabricated ones. This would mean that in the prefabricated structure the energy dissipation through which seismic resistance is developed, takes place in a volume of material which, despite concentrated into a lower number of critical zones is the same as in analogous cast-in-place structures, where it is spread over a large number of plastic zones.

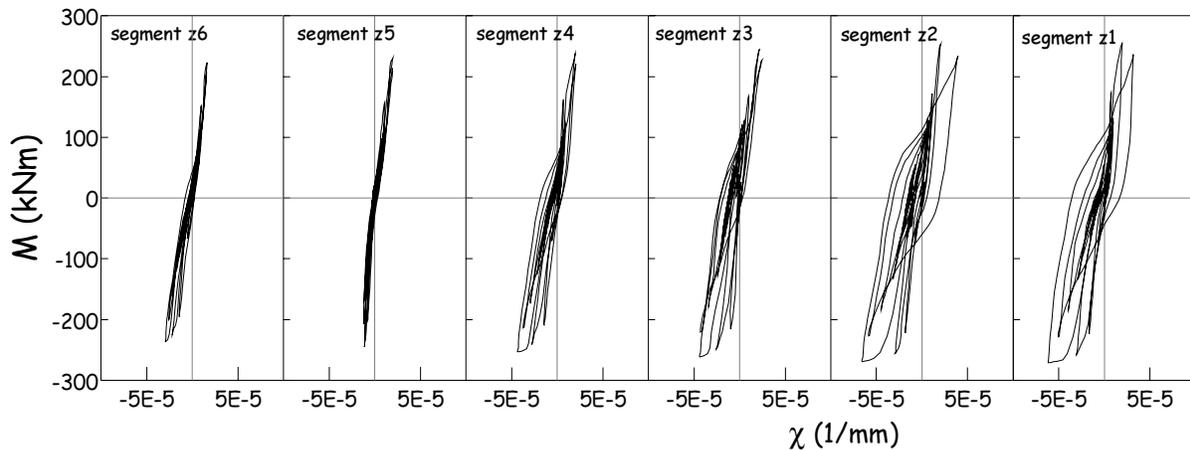
Figures 18 and 19 finally show the two prototypes under maximum displacements attained during the third tests.



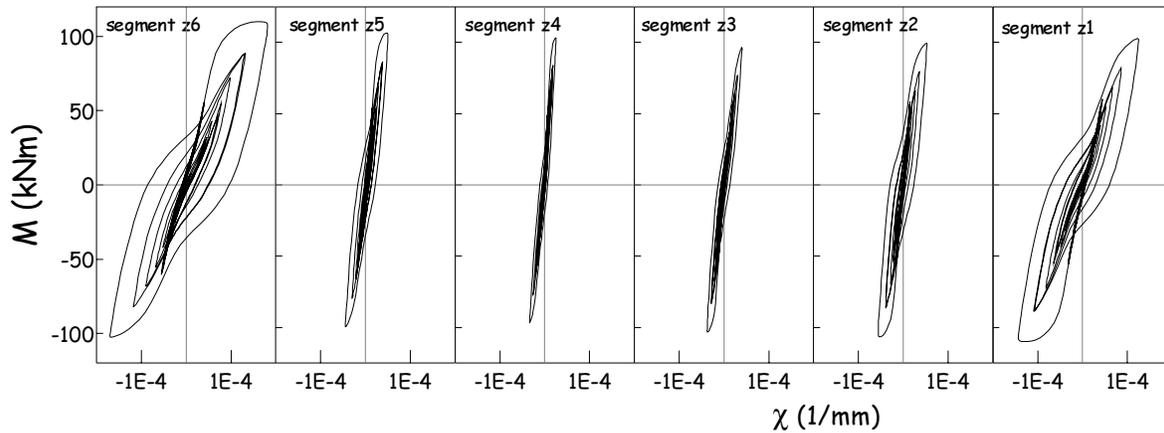
**Figure 13:** cover spalling at bottom edge of cast-in-place columns at the end of the third test



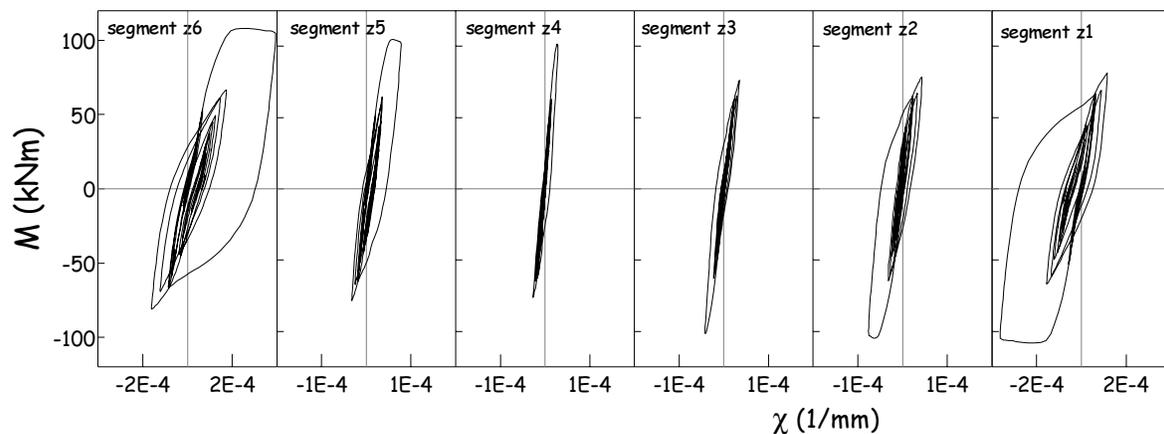
**Figure 14:** rotation at the beam-column hinge at the end of the third test (prefabricated prototype)



**Figure 15:** moment-curvature relationships along the critical zones of prefabricated columns ( $\alpha_g = 0.72$ )



**Figure 16:** moment-curvature relationships along the critical zones of cast-in-place columns ( $\alpha_g = 0.64$ )



**Figure 17:** moment-curvature relationships along the critical zones of cast-in-place columns ( $\alpha_g = 0.80$ )

## CONCLUSIONS

A prefabricated and a cast-in-place prototype of one storey reinforced concrete structures for industrial buildings have been submitted to pseudodynamic tests in order to assess their response under earthquake loading. Both structures were designed according to Eurocode 8 (draft May 2001) in order to withstand the same base shear force and assuming the same behaviour factor equal to  $q = 4.95$ . Results showed that precast concrete structures are able to resist to earthquake loading as reliably as analogous (in the sense above specified) cast-in place ones. Due to the cantilever behaviour of precast columns, their seismic resistance may only rely upon the flexural resistance of the bottom edge sections. By the way the energy dissipation in prefabricated columns occurs within a volume of material which is almost equal to that involved at top and bottom edge sections of cast-in-place columns designed to withstand the same base shear force. A further confirmation to these hypotheses, mainly to the true equivalence of the behaviour factor  $q$ , will come from currently undergoing analyses.

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**Figure 18:** precast prototype at the end of the third test



**Figure 19:** cast-in-place prototype under maximum displacement attained during the third test

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