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# FULL-SCALE PSD TESTING OF A TORSIONALLY UNBALANCED THREE-STOREY NON-SEISMIC RC FRAME

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## SUMMARY

In the framework of the research activity of the ELSA Laboratory of the Joint Research Centre, pseudo-dynamic testing of a real-size plan-wise irregular 3-storey frame structure is being carried out as the core of the research project SPEAR (Seismic PErformance Assessment and Rehabilitation of existing buildings).

The project, funded by the European Commission, sees the participation of many European and overseas Partners and is specifically aimed at throwing light onto the behaviour of existing old RC frame buildings lacking seismic provisions.

The main goal of the SPEAR project is contributing to the improvement of current design, assessment and retrofitting techniques and the development of new simplified approaches for the assessment and rehabilitation of existing building structures. This goal is pursued by means of a balanced combination of experimental and numerical activities.

In the paper the pre-test numerical work on the specimen is presented; the PsD test set-up is then described. Moreover, the results from the first PsD test are presented and discussed in detail in relation to the open issues of research in the field of torsionally unbalanced buildings.

## INTRODUCTION

One major source of hazard in southern European countries is represented by a number of existing RC and masonry structures, non-compliant with current codified requirements for earthquake resistance because designed following older codes and construction practice and not ensuring adequate provisions for earthquake-induced lateral loads. Among them, plan-wise asymmetric structures are quite common.

Given the economic costs of demolishing and re-building non-compliant structures, it is necessary to enforce a more rational approach for the seismic assessment and rehabilitation of existing structures, in order to reliably identify hazardous buildings and to conceive retrofitting interventions aimed at the most critical deficiencies only.

The research project SPEAR (Seismic PErformance Assessment and Rehabilitation), currently being carried out by a consortium of European partners, is specifically targeted at existing structures: evaluation

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of current assessment and retrofitting methods, development of new assessment and retrofitting techniques, contribution to the improvement of current codes are its main goals.

They will be achieved by a balanced combination of numerical work and experimental activities.

In the framework of SPEAR, a series of tests on small members and subassemblies has been carried out; however, the core of the experimental activity is the series of full-scale pseudo-dynamic tests, currently in progress, on a torsionally unbalanced three-storey RC frame structure, representing a common configuration of housing units in most earthquake-prone areas of Europe.

In the SPEAR structure, the issues brought about by plan-irregularity in older structures are further enhanced by generally poor local detailing, scarcity of rebars, insufficient confinement, weak joints and older construction practice, which will make the effects of torsional coupling more clearly stand out. Based on the test results in the as-built configuration, adequate retrofitting strategies will then be carried out before re-testing the specimen, thus collecting information on the effectiveness of the interventions.

The amount of data collected during the tests will be of much importance to improve the understanding of the behaviour of torsionally unbalanced buildings; moreover, once the calibration of computational models through experimental data is carried out, a whole range of different irregular structures can be studied with the certainty of reliable results.

The numerical activity in SPEAR (the SPEAR building is likely to become the most-analyzed structure in earthquake engineering, Elnashai [1]) consists of two parallel and complementary parts, both split into pre-test and post-test phases.

The first numerical activity was the application of current codified assessment procedures to the structure, in order to derive the predictions regarding global and local aspects of the expected response, to be compared to the experimental results as soon as they become available. In the pre-test phase, a comparison between the different (sometimes quite dispersed) results was carried out, in order to assess the level of code-dependency of the problem and to draw first conclusions on the available assessment tools at present; in the post-test phase, a direct comparison between the results and the predictions will also allow the relative quality of the assessments yielded by each approach to be judged, and their pros and cons to be highlighted.

The second parallel activity consisted in implementing a series of numerical models of the structure into research-oriented programs, ranging in refinement from element-models with lumped plasticity to fibre-models with distributed plasticity.

A range of modelling options was investigated; so far, the results derived from different approaches were compared in order to judge the level of model-dependency of the obtained results; in the near future, a comparison with the experimental data will possibly allow the best modelling techniques to be identified. Moreover, in the post-test phase, calibration and validation of these refined models by means of measured data will be carried out: in the end, well-tuned analytical tools will be available to run parametric analyses whose results can be used as a comparison to assess the effectiveness of current codified assessment approaches and to propose alternative or improved ones.

The experimental phase focused on the real-size specimen is articulated into three rounds of PsD tests, the first of which took place in late January 2004. The first tests were carried out on the structure in its original, “as built” configuration; as it was mentioned above, the results of this round of tests will be used to state the effectiveness of the predictions obtained both from codified assessment procedures and from research-oriented programs as to the earthquake resistance of the structure and its failure or damage patterns. Following these tests, a light (i.e., member-level) retrofitting intervention, most probably involving FRP wrapping of columns to improve ductility, is foreseen in the next few months. After the intervention is completed a new round of tests will be carried out in the retrofitted configuration, so that the effectiveness of currently available guidelines for the design of retrofitting interventions will be judged. Finally, the damage inflicted by the second round of tests will be repaired and the structure more heavily retrofitted, by means of interventions aimed at improving the global structural configuration.

In the following, the description of the set-up, execution and global results of the first round of bi-directional PsD tests is given. The results are discussed and the preliminary comparisons against the predictions are given, with particular focus on the issue of torsional effects.

## DESCRIPTION OF THE STRUCTURE

The main SPEAR structure is a simplification of an actual three-storey building representative of old constructions in southern European Countries, such as Greece, without specific provisions for earthquake resistance. It was designed for gravity loads alone, using the concrete design code applying in Greece between 1954 and 1995, with the construction practice and materials typical of the early 70s; the structural configuration shows the lack of consideration of the basic principles of earthquake resistant design.

The materials used for the structures are also those typical of older practice: for concrete a nominal strength $f_c$= 25 MPa was assumed in design; smooth rebar steel was used; given the scarcity of the current production, it was only possible to find bars with a characteristic yield strength larger than initially requested ($f_y \approx$ 450 MPa instead of $f_y$= 250 MPa); the end hooks for the steel bars were designed following the minimum requirements of old codes.

The structure is regular in elevation: it is a three-storey building with a storey height of 3 meters. The plan configuration is doubly non symmetric (Fig.1)), with 2-bay frames spanning from 3 to 6 meters; the presence of a balcony on one side and of a part of the structure 1 meter (in the weak direction) or 0.5 meter (in the strong one) longer than the rest increases the plan irregularity, shifting the centre of stiffness away from the centre of mass.

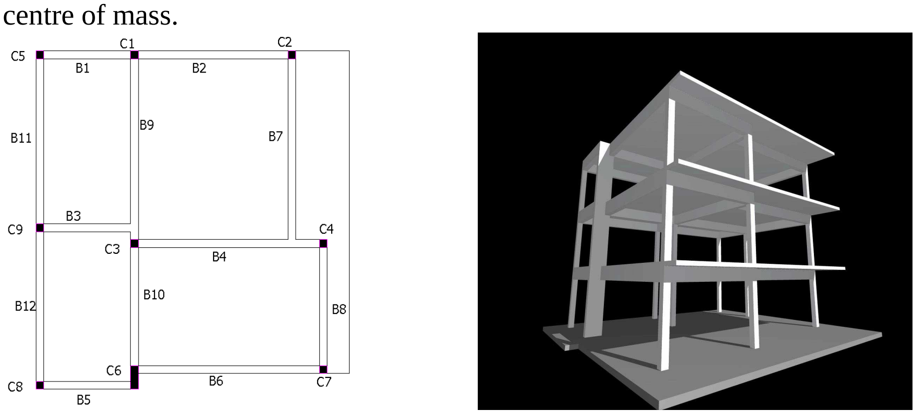


**Figure 1 – The SPEAR structure**

The concrete floor slabs are 150 mm thick, with bi-directional 8 mm smooth steel rebars, at 100, 200 or 400 mm spacing.

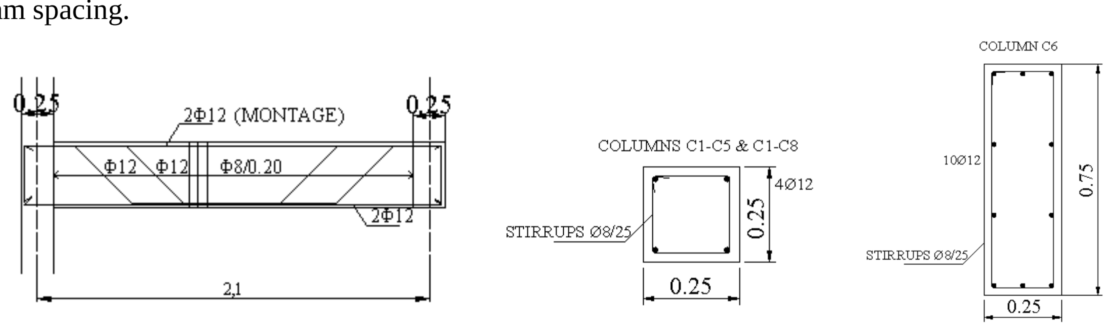


**Figure 2 – Details of the beam and column rebars**

Details of the rebar of one of the beams are shown in Fig. 2). Beam cross-sections are 250 mm wide and 500 mm deep. Beams are reinforced by means of 12 and 20 mm bars, both straight and bent at 45 degrees

angles, as typical in older practice; 8 mm smooth steel stirrups have 200 mm spacing. The confinement provided by this arrangement is thus very low.
Eight out of the nine columns have a square 250 by 250 mm cross-section; the ninth one, column C6 in Fig. 1), has a cross-section of 250 by 750 mm, which makes it much stiffer and stronger than the others along the Y direction, as defined in Fig. 3), which is the strong direction for the whole structure.
As can be seen in Fig. 2), all columns have longitudinal reinforcement provided by 12 mm bars (4 in the corners of the square columns, 10 along the perimeter of the rectangular one). Columns' longitudinal bars are lap-spliced over 400 mm at floor level. Column stirrups are 8 mm with a spacing of 250 mm, the same as the column width, meaning that the confinement effect is almost non-existent.
The joints of the structure are one of its weakest points: neither beam nor column stirrups continue into them, so that no confinement at all is provided. Moreover, some of the beams directly intersect other beams, so that beam-to-beam joints without the support of columns originate.
Design gravity loads are 0.5 kN/m$^2$ for additional dead load and 2 kN/m$^2$ for live load.
As described above, the structure is regular in elevation and has the same reinforcement in the beams and columns of each storey. The resisting elements in both directions are all of the same kind (frames), so that they present proportional stiffness matrices. All of these features mean that the structure belongs to a special class of multi-storey buildings, the so-called regularly asymmetric multi-story structures, in the sense that the centre of mass (CM), the centre of stiffness (CR) and the centre of strength (CP) of each storey are located along three vertical lines separated by the distances $e_r$ and $e_s$.
The centre of stiffness (based on column secant-to-yield stiffness) is eccentric with respect to the mass centre by 1.3 m in the X direction (~13% of plan dimension) and by 1.0 m in the Y direction (~9.5% of plan dimension).

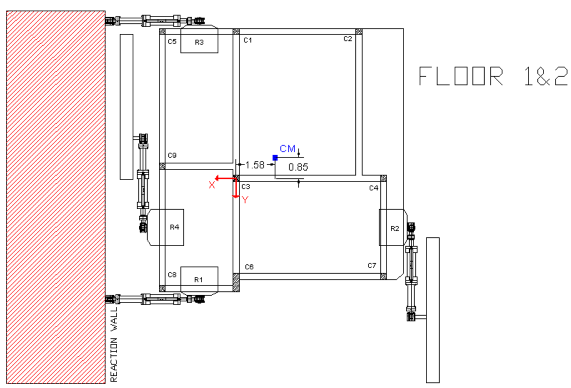


**Figure 3 – Location of the CM of the structure**

The reference system used in the PsD test and the location of the CM of the structure at the first and second floor are shown in Fig. 3). The origin of the reference system is in the centreline of column C3. The coordinates of the CM of the first two storeys with respect to this reference system are (-1.58m, -0.85m); at the third storey the coordinates of the CM vary slightly, becoming (-1.65m; -0.94m).

## TEST SET-UP

### Instrumentation

The layout of the instrumentation on the structure responded to different needs and considerations, both theoretical and experimental. The bi-directionality of the test made it difficult and too demanding to conceive an instrumentation layout to trace the local bi-directional behaviour of all the elements at all the storeys. Moreover, the significance of such a choice would have been debatable.

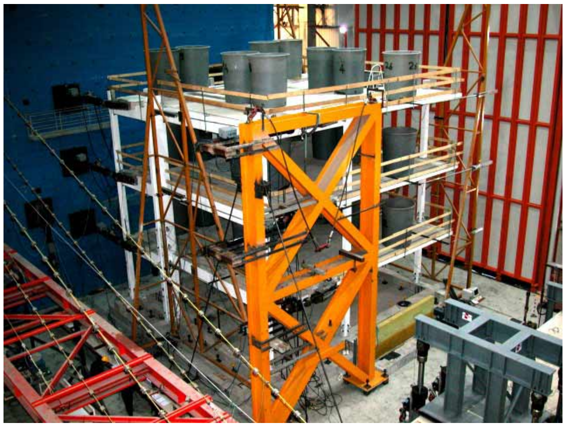

**Figure 4 – The SPEAR structure before the first round of PsD tests**

Based on the extensive preliminary numerical simulations, the expected damage pattern had been defined, and the elements likely to exhibit the most significant behaviour had been pointed out. The structure was expected to fail due to column failure, rather than developing significant damage in beams or joints; moreover, a soft-storey mechanism involving the first floor was expected in the weak direction and most of the damage was then expected on top and bottom of first storey columns, with the possibility of further damage taking place at the second floor. For this reason, the local instrumentation was mainly focused on the columns at the first and second floor, with inclinometers mounted at the member ends. To capture the effects of the hooks of the rebars, inclinometers were also placed above the splice level.

Moreover, on the two large faces of column C6, displacement transducers were located, to measure the shear deformation of the column, without including the effects of bar slippage at the bottom.

Finally, two locations of interest were chosen to be more carefully investigated on the soffit of the first and second floor: the beam-on-beam intersections were in fact particularly interesting because of the possible local torsional effects that might develop during the response. For this reason, they were both instrumented with two inclinometers (one in each direction) and two crossed displacement transducers.

To increase the significance of the test, and to exploit the rare opportunity of a bi-directional excitation and response, the use of optical measurements was also decided. The optical equipment consisted of four cameras, two for each location, placed at two different angles, so that they gave a stereovision of the target area. Comparisons between optical measurements and traditional mechanical measurements will be available in a later phase, thus allowing the effectiveness of both methods and their relative errors to be judged.

**PsD technique and algorithm**

*Introduction*

In the following a short description of the PsD technique used in the tests is given. A more detailed description of the method and of the mathematical approach is given in Molina [2], [3].

The bi-directionality of the PsD test, consisting in the simultaneous application of the longitudinal and the transverse component of the earthquake to the structure, introduces a higher degree of complexity, both from the analytical and from the technical point of view, with respect to usual unidirectional PsD testing.

In fact, three degrees of freedom (DoFs) per floor need to be taken into account: two translations and one rotation along the vertical axis, as opposed to the single degree of freedom that is usually taken into account in conventional unidirectional PsD testing.

Four actuators per storey were connected to the structure, three of which were strictly necessary. The control of a redundant number of actuators thus required a more complex control strategy.

*Analytical background*

The PsD integration of the horizontal response of the structure was performed in terms of three generalized DoFs at each floor, consisting of the in-plane displacements $d_X$ and $d_Y$ and of the rotation along the vertical axis $d_\theta$ at the centre of mass (CM) of the structure. They were collected in the vector of generalized floor displacements.

The in-plan restoring forces $R_X$, $R_Y$ and the torque $R_\theta$ were collected in the vector of conjugated generalized restoring forces.

Assuming for each floor the hypothesis of rigid-body behaviour, its horizontal motion is completely described by the generalized displacements and its equations of motion were derived from the application of D'Alembert's Principle, when the whole structural mass was assumed to be concentrated at the floor level.

Thus a 3N system of equations of motion governed the structural response, where N was the number of storeys and the variables were the generalized displacements of the CM.

However, the control system used for the test was based on a set of linear actuators and displacement transducers attached at prescribed locations at each floor. For this reason, the necessary transformations between the two systems of co-ordinates were developed.

The measurement of floor displacements for control purposes was achieved using high-resolution linear displacement transducers attached to each floor. During the test, the computed generalized displacement of the floor was imposed by means of the actuators with feedback from these displacement transducers; thus, in order to determine the target displacement at the transducer level, a geometric transformation was first performed.

At each step, each displacement transducer was associated to an actuator acting along the same direction; once the prescribed displacements of each transducer at each step were reached, the acting axial force in each actuator was measured by its load cell. It was then necessary to express such forces as resultant generalized forces at the CM of each floor, by means of a static transformation.

When more than three actuators act on a rigid floor, as in this case, the use of individual displacement transducers on the structure as feedback signals for the actuators can lead to control instability. For this reason, only a number of feedback displacements equal to that of the DoFs was used, whereas the redundant actuators were controlled by other means with the aim of guaranteeing an acceptable distribution of loads among all the actuators themselves. A dedicated algorithm computed the optimal distribution of piston loads compatible with the known set of generalized floor forces.

Two different approaches are usually employed to step-by-step solve the system of equations of motion: the explicit Newmark method or the $\alpha$ operator splitting method, which are both particular cases of the $\alpha$-generalized method, an extension of the Newmark scheme. In this case, the explicit Newmark method was used because the time step was small enough in comparison to the natural frequencies of the specimen.

*Hardware and software set-up*

The servo-control units used for the test were MOOG actuators with ±0.5m stroke and a load capacity of 0.5 MN. The control displacement transducers were Heidenhain sensors with a stroke of ±0.5m and a resolution of 2μm. Each actuator was equipped with a strain-gauge load cell and a Temposonics internal displacement transducer.

*Mass distribution*

When modelling the structure and implementing the time integration algorithm, the structural mass considered is the one that takes into account the presence of finishings and of the quota of the live loads which is assumed to act at the time of the earthquake, therefore the mass properties were those resulting from the preliminary numerical simulations. The coordinates of the CM of each floor were calculated with reference to the system of coordinates originating in C3 and shown in Fig. 3); the mass values, the coordinates of the CM and the moment of inertia with respect to the CM are given in Table 1.

**Table 1 – Coordinates of the CM and mass moment of inertia I at each floor**

| Storey | Mass (kg) | $x_{CM}$ (mm) | $y_{CM}$ (mm) | I (kg $m^2$) |
|---|---|---|---|---|
| 1 | 67264 | -1583 | -849 | 1500663 |
| 2 | 67264 | -1583 | -849 | 1500663 |
| 3 | 62804 | -1646 | -937 | 1363409 |
| total | 197332 | | | |

The laboratory specimen, though, did not have finishings and live loads on it. For this reason, to reproduce the corresponding stress on the structural elements, a distribution of water tanks on each floor was studied, to account for finishings and 30% of the live loads at each floor; the tanks were distributed so that the gravity loads on columns would be the closest to the values used in design.

## Choice of the input signals

The input signals for the test were chosen after preliminary analyses and discussions among the partners. A suite of 7 pairs of semi-artificial records modulated after historic events was chosen for the preliminary analyses. They were consistent with the Soil C, type I spectrum of Eurocode 8 [4]. Due to the plan-irregular configuration of the structure (which makes the direction of the excitation an important parameter affecting the response) and the possibility of interchanging the longitudinal and transverse component of each record, the initial number of analyses foreseen for each PGA level was very high. In fact, when all possible combinations of signs and direction were considered, the final number of 7x2x2x2=56 analyses (for each PGA level) came out. After the preliminary phase, the Montenegro Herceg-Novi record was chosen (Fig.5), based on the results of the preliminary numerical simulations, an excerpt of which is shown in Fig. 6).

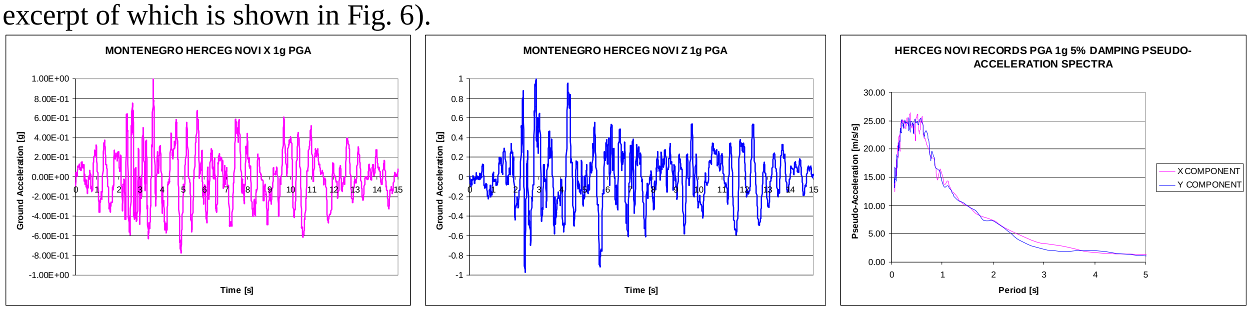


**Figure 5 – Herceg-Novi records PGA=1g; a) longitudinal component b) transverse component c) acceleration response spectra of the X and Y components**

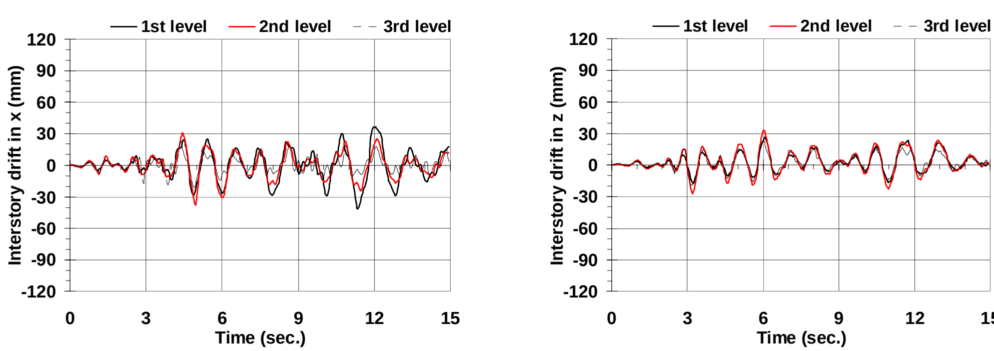


**Figure 6 – Numerical prediction of interstorey drifts for PGA=0.2g**

The response of the structure under the Herceg-Novi pair of records (H-N longitudinal and H-N transverse) scaled to a PGA of 0.2g, shown in terms of interstorey drift time histories, presented no

pronounced peaks and an increase in amplitudes in the latter part of the response: these were considered desirable features for the test.
Once the accelerogram was chosen, the most appropriate direction of application was to be determined.
Given the bi-directional plan irregularity of the structure, 8 combinations of the directions of application and orientations were possible for the chosen signal. The aim was to maximize the effects of torsion on the response when determining the final combination for the test. To quantify the effects of torsion on the response, the standard deviation of the displacement demand imposed on the columns was examined: the larger this parameter, the larger the influence of torsional effects. Based on this criterion, it was decided to adopt the combination that consisted of the application of the X component in the –X direction of the reference system of Fig.3), and of the Y component in the –Y direction of the same reference system. This choice was the one that, according to the pre-test predictions, should highlight the influence of plan eccentricity on the response in terms of displacements.
Finally, the level of peak ground acceleration (PGA) had to be defined. This was not an easy task, considering that such level was the critical parameter in determining the damage pattern of the specimen. The aim of the test, in view of the subsequent phases of the project, was to investigate the behaviour of the structure with a significant damage, but not so severe as to be beyond repair. In fact, the following repair and retrofitting phase was intended to consist into a light intervention, meaning that the level of damage inflicted in the first round of test should have been carefully and cautiously limited.
To choose the acceleration level for scaling, damage levels of the structure under the Herceg Novi record scaled to different PGA values were investigated. The degree of damage was represented by the interstorey drift demand-to-capacity ratio of the columns. Due to the torsional irregularity, a number of the columns were the critical ones: C3 because it had the highest axial load, C1 and C2 because they were the edge columns farthest from the CR. Based on the preliminary analyses carried out by the MAE Center, the University of Rome and the University of Ljubljiana, the intensity level proposed for the test ranged between 0.14g and 0.20g. It was thus decided to run the test with a PGA level of 0.15g, because, due to the inherent brittleness of the structure, even a slight overestimation of the PGA level could have inflicted too heavy damage, thus compromising the future phases of the project.

## EXPERIMENTAL RESULTS

When starting a round of PsD tests, especially in the case of full-size structures, initial low-intensity tests are carried out before the actual test, at the chosen level, can take place.
The primary aim of the low-intensity tests is to check the functioning of both the hardware and software equipment. The control and integration algorithms are put to the test, together with the acquisition system. The significance of the results obtained is checked to ensure the perfect execution of the test at the larger intensity level. Moreover, from the low-intensity level test, very useful information about the elastic properties of the structure can be obtained, as discussed in the following. For this reasons, the round of tests presented in the following began with the test at the value of PGA of 0.02g.

### 0.02g PGA input

#### *Frequencies and modal shapes*

One of the main purposes of the initial low-intensity test at 0.02g PGA was to obtain the initial stiffness matrix of the structure, in the elastic range of excitation.
It was thus possible to obtain the first mode shapes of the structure, along with their frequencies (f) and modal damping values (z). In Fig. 7), the nine 3D mode shapes are represented.
It can be observed that a degree of coupling between the flexural and torsional behaviours is evident starting from the very first modes.

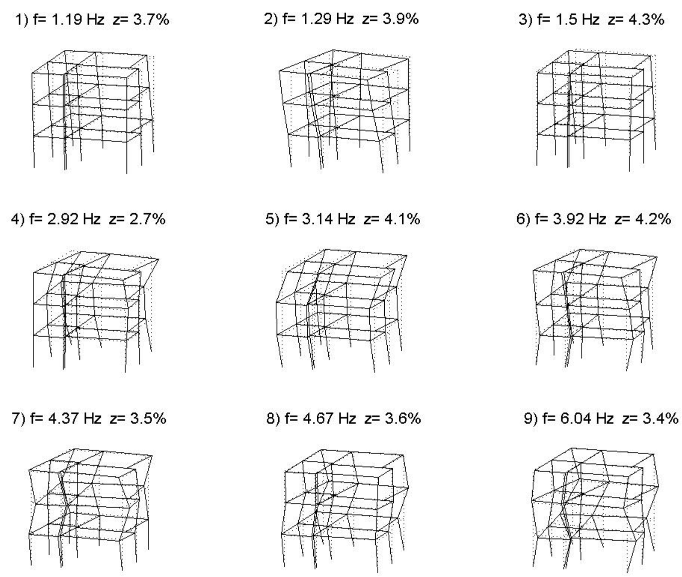


**Figure 7 – Frequencies and mode shapes obtained from the 0.02g PGA PsD test**

**0.15g input**

As mentioned above, this test was expected to cause significant damage, without going beyond the reparability stage. In particular, the damage pattern expected from the numerical predictions was to be limited to columns. It was expected to consist of spalling and flexural cracking at the bottom and top of the square columns. At the bottom, cracking was expected in the area immediately above the lap splicing of first and second storey columns. At the top, damage was expected right below the beams at the first two storeys. In any case, cracks large enough to be epoxy-injected afterwards were to originate.
As can be seen in Fig.8), the peak values of displacements and drift reached during the tests were compatible with this damage pattern, taking into account the poor structural detailing.
Nonetheless, the inspection of the structure during and soon after the test revealed that only minor damage had occurred for the 0.15g PGA: only minor cracking, mainly concentrated at the top of columns and in the beams intersecting into column C6, could be spotted and no damage at all at the bottom of columns originated.

*Displacements time-histories*

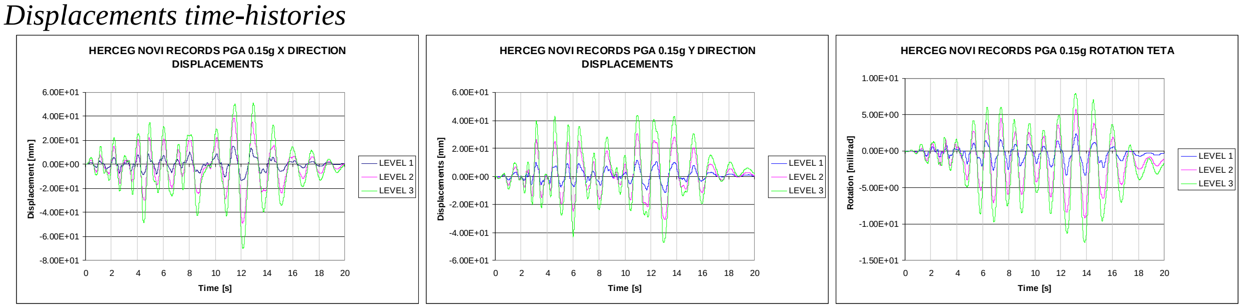


**Figure 8 – Displacements time-histories: a) X direction, b) Y direction, c) rotation θ**

In Fig. 8) the displacement time-histories in the X, Y and θ directions are represented at the three levels of the specimen. The displacements and the rotation are those of the CM, whose coordinates were given in Table 1. The maximum displacements were about 70 mm in the X direction and 50 mm in the Y direction; the maximum rotation was about 12 mrad at the third storey.

The displacements where significantly different from those predicted by means of the numerical simulations. In particular, the amplitude of the torsional response was much larger.

*Floor hysteresis loops*

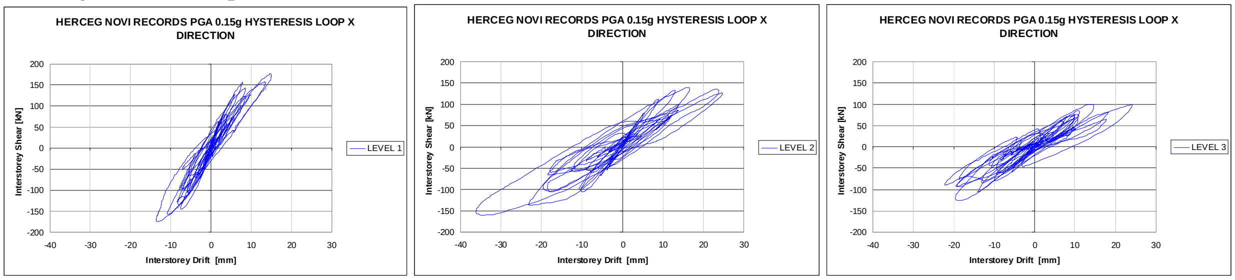


**Figure 9 – Hysteresis loops in the X direction**

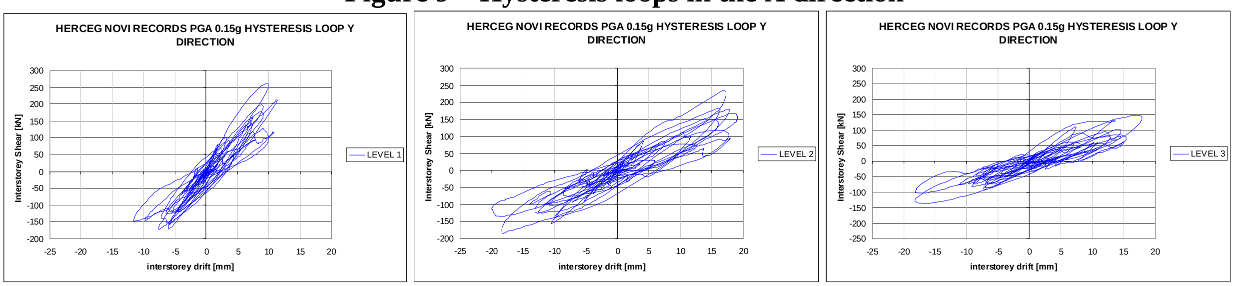


**Figure 10 – Hysteresis loops in the Y direction**

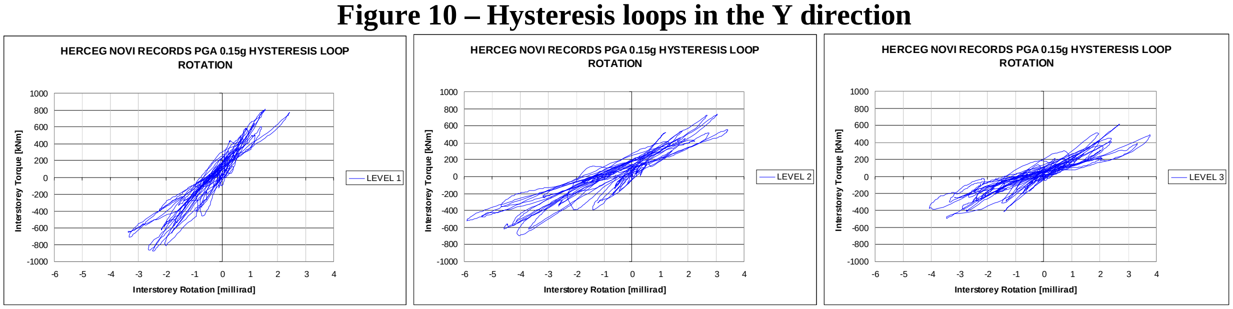


**Figure 11 – Hysteresis loops in rotation θ**

In Fig. 9)-11) the storey-level hysteresis loops for the X, Y and θ directions are reported. The interstorey shears (or torques) are plotted against the interstorey drifts (or rotations). It can be observed that the second level absorbs a larger energy with respect to the other storeys, followed by the third storey and then by the first one.

The Y direction, the one of the strong column, is globally stronger, so the levels of interstorey shears reached along this direction are larger, as larger is the global value of the absorbed energy, as expected; on the contrary, much larger displacements were reached in the X direction at the second floor.

These findings were confirmed by the time histories of the absorbed energy at each floor.

## 0.20g input

After performing the 0.15g PGA test, it was clear that the test failed in reproducing a significant level of damage, therefore a higher intensity test was necessary.

The upper boundary of the initial confidence range for the definition of the intensity, 0.2g, appeared to be the best choice.

New sets of numerical simulations were thus carried out, taking into account the effects of the test that had just been performed. They confirmed the suitability of the choice of having one more test at that intensity. It was thus decided that this test at the intensity level of 0.2g would be carried out as the final one of the first round.

*Displacements time-histories*

In Fig. 12), the displacement time-histories in the X, Y and θ directions are represented at the three levels of the specimen. The displacements and the rotation are those of the CM, whose coordinates were given in Table 1. As for the 0.15g case, the resulting displacements were significantly different from the predicted, in particular the torsional response turned out to be much larger.

The maximum displacements were above 100 mm both in the X direction and in the Y direction; the maximum rotation was about 20 mrad at the top storey.

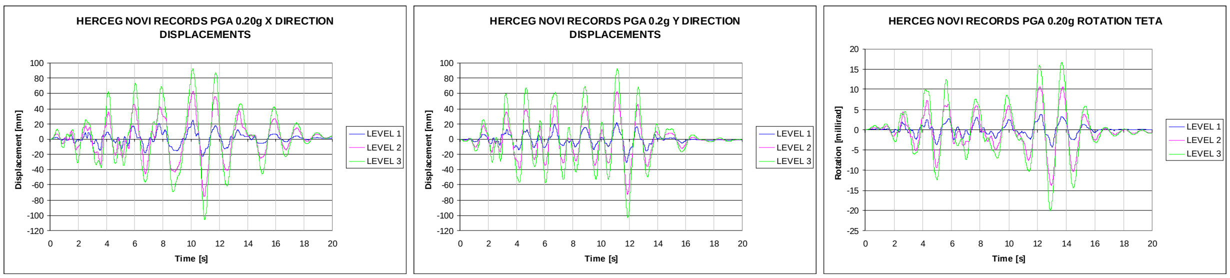


**Figure 12 – Displacements time-histories: a) X direction, b) Y direction, c) rotation θ**

*Floor hysteresis loops*

In Fig. 13)-15) the storey hysteresis loops for the X, Y and θ directions are reported. The interstorey shears (or torques) are plotted against the interstorey drifts (or rotations). The same comments as for the 0.15g test can be reported for the 0.2g test.

The most affected level is once again level 2 and the direction where the absorbed energy is larger is the Y direction.

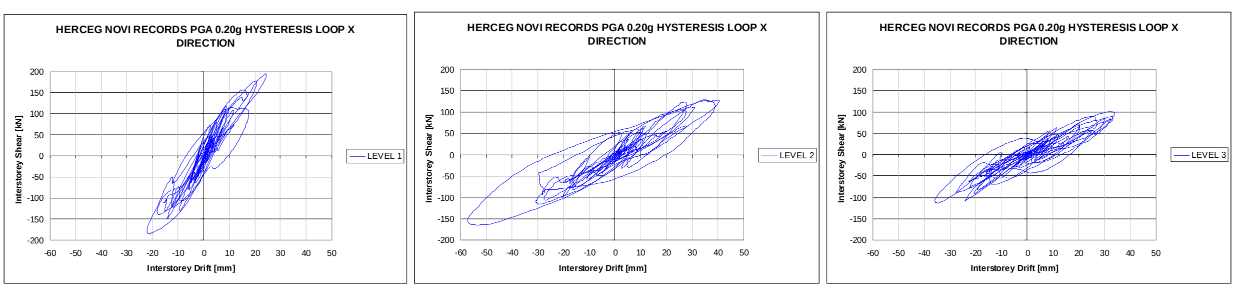


**Figure 13 – Hysteresis loops in the X direction**

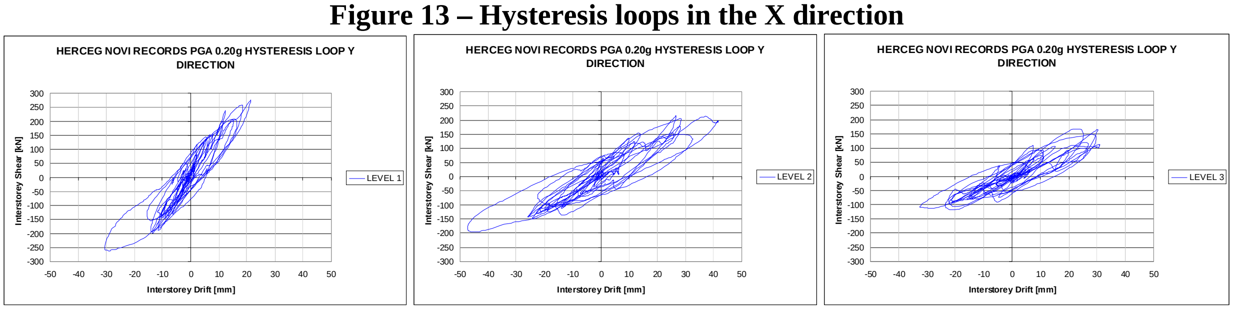


**Figure 14 – Hysteresis loops in the Y direction**

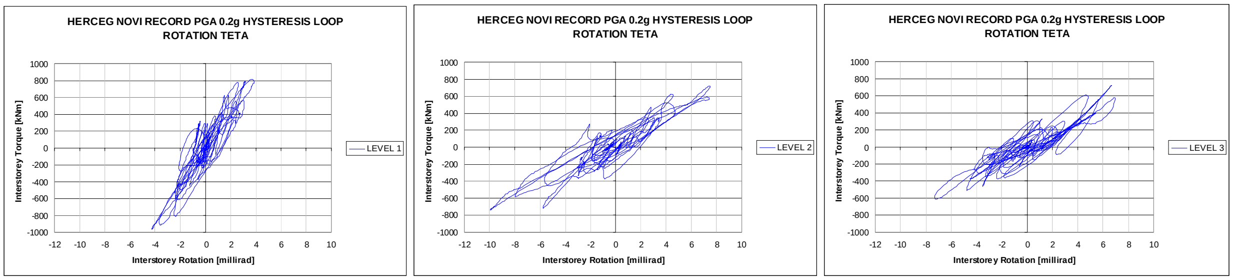


**Figure 15 – Hysteresis loops in rotation θ**

*Column interstorey drifts time-histories*

The interstorey drifts at each storey are different for each of the nine columns of the specimen, due to torsional effects.

These drifts were calculated, based on the hypothesis of infinite rigidity of the floor slabs, by means of simple geometric considerations.

The X and Y direction drifts of each column are in fact obtained adding to the drifts of the CM in the same direction the rotation at the CM multiplied by the distance between the CM and the considered column, measured in the direction perpendicular to the one in which the drift is being calculated.

It can thus be concluded that the maximum drifts in the X direction are those of the Y edge columns (columns C1, C2, C5 and C8); in the Y direction the maximum interstorey drifts are those of columns (C8, C9, C5 and C4, C7).

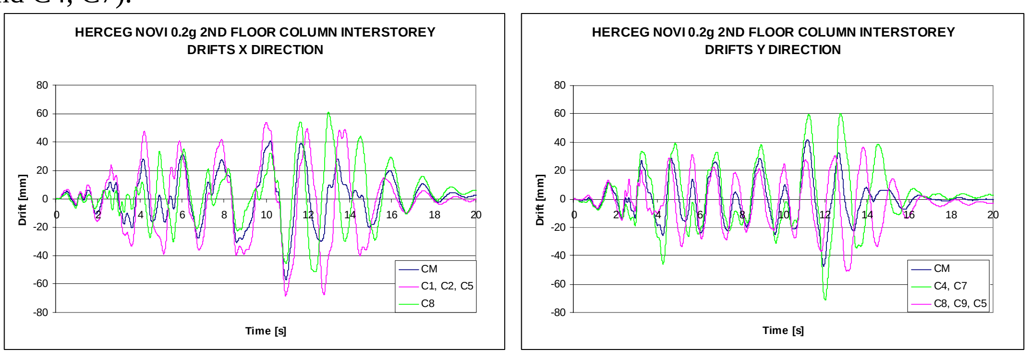


**Figure 16 – Second storey largest column drifts compared to those of the CM a) in the X direction, b) in the Y direction**

Fig.16) allows a comparison between the drift measured at the CM and the drifts of the edge columns, the most displaced ones, to be made. As it could be expected for an irregular structure, a difference exists between the two, so that it is not on the safe side to estimate the displacements and to assess the structure based on the displacements of the CM only.

The extent of such difference is the most important outcome, taking into account the torsional response which was much larger than predicted: the test showed that, despite an eccentricity that could be defined not too large (in the order of 10% of the plan dimension), the effects of torsion on the drifts of the edge columns are significant in both directions. In the X direction, where the structure is less rigid and the drift at the CM is already quite large, the maximum drift reached at the CM is 55 mm, whereas the maximum drift reached at the edge columns C1, C2 and C5 was about 70 mm, a difference which is not negligible. In the Y direction the maximum drift reached at the CM was 45 mm, whereas the maximum drift of the edge columns C4 and C7 was above 70 mm, i.e., more than 50% larger.

Fig. 17)-19) show a different representation of interstorey drifts of the second storey (the largest ones) for each of the nine columns; the drifts in the Y direction are plotted against those in the X direction.

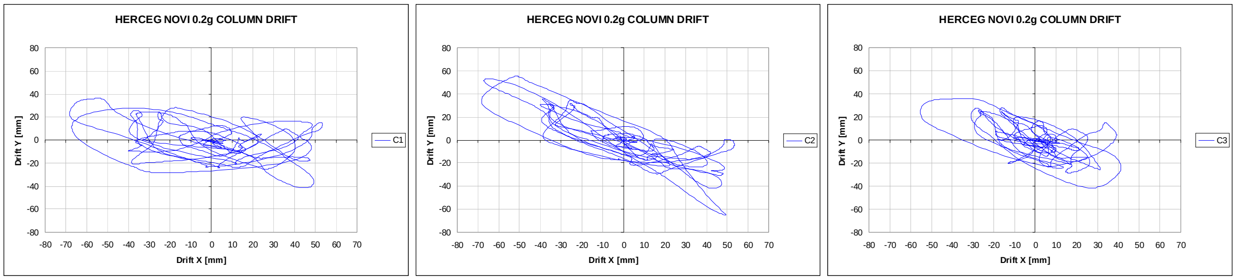


**Figure 17 – Second storey X and Y direction drifts of columns C1, C2, C3**

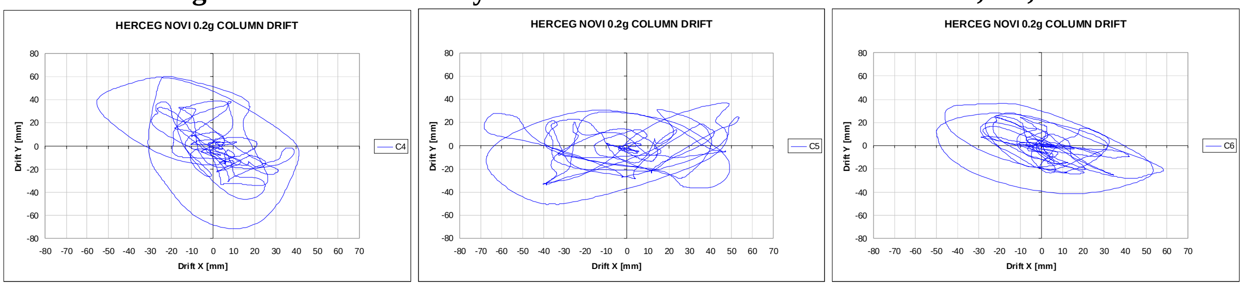


**Figure 18 – Second storey X and Y direction drifts of columns C4, C5, C6**

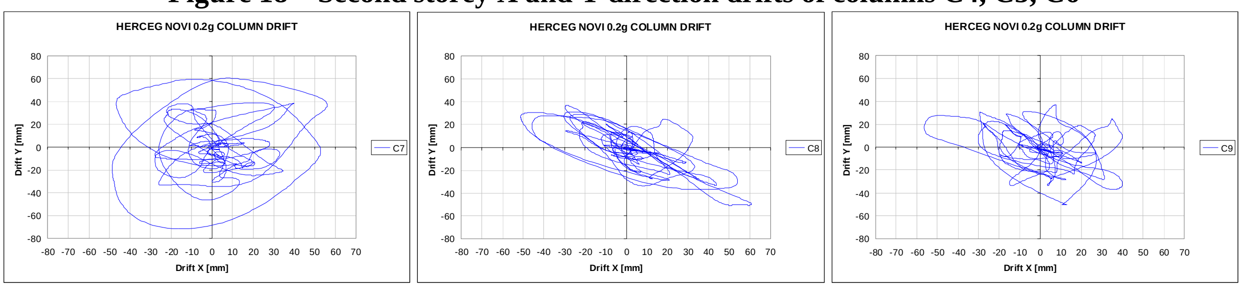


**Figure 19 – Second storey X and Y direction drifts of columns C7, C8, C9**

## Damage Pattern

To better understand the evolution of damage in the structure during the two tests, in Fig. 20), the evolution of the natural frequencies of the first modes, computed by applying a characterization algorithm to the acquired test results, Molina [5], are reported for both levels of excitation.
It can be observed that the first test corresponded to a 40% drop in the fundamental frequency, which is consistent with the relatively low level of damage. The second test resulted into a cumulative drop of the fundamental stiffness of more than 50%.

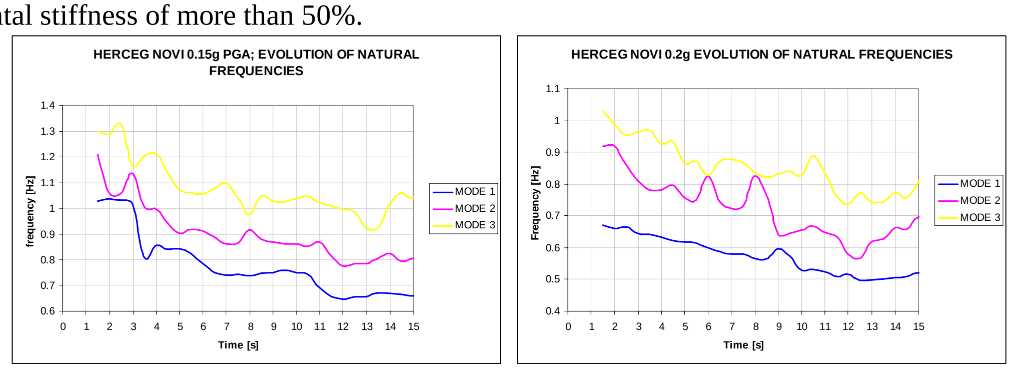


**Figure 20 – Evolution of the natural frequencies during the: a) 0.15g PGA test b) 0.20g PGA test**

As mentioned above, the damage pattern expected from the numerical prediction was to be limited to the columns. It was expected to consist of light spalling and flexural cracking at the bottom and top of the square columns, in particular at the first floor. Cracking was expected to be evident in the area immediately above the lap splicing at the bottom of first and second storey columns (at about 400-500 mm from the floor slab). At the top of the columns damage was expected below the roof beams at the first two storeys. No major damage was expected at the joints or in the beams.

**Figure 21 – Damage after the 0.2g PGA test: a) extensive spalling at the top of columns (C3); b) light spallling and flexural cracking at the top of corner columns (C1); c) base of column C3**

This level of damage was initially expected to be reached with the PGA value of 0.15g, based on the outcomes of the numerical simulations, which referred to a range of PGAs starting from 0.14g up to 0.2g; given the intrinsic fragility of the structure, and the risk of inducing too extensive damage with the choice of an even slightly too high PGA value, it was decided to stay on the safe side with the first test, thus choosing the 0.15g PGA value. The structural response under the 0.15 PGA earthquake though, did not induce any particular damage, except for light cracking at the top of the first and especially the second storey column. It was thus clear that a significant damage pattern would be reached subjecting the structure to an additional test, with a larger PGA value, which was chosen to be 0.2g.
During the second test the same damage trends were followed, only partly following the predictions. The damage was concentrated in columns, as expected. It was particularly evident at the top rather than at the base of the columns, and at the second floor rather than at the first; in Fig. 21 a) it can be seen that columns C3, the one with the highest axial load, suffered the most for extended spalling at the top and that also the flexible edge columns exhibited a significant level of spalling and cracking at the top.
Unexpectedly, though, the bottom parts of the columns did not show either cracking at the level of the lap splices, or spalling, except for some minor damage particularly evident in column C3, as it can be seen in Fig.21 c). Finally, some damage was detected in the beams and floor slabs connected to the strong column C6: due to its large stiffness, it did not suffer any damage in itself, but caused cracking in the surrounding elements.

## CONCLUSIONS

In the framework of the activity of the SPEAR research project, a round of bi-directional PsD tests was carried out on a full-scale three-storey plan-wise irregular RC frame structure.
The description of the experimental activity and the discussion of the results were the object of this paper. The test-set up, the instrumentation, the control and integration algorithm were described.
The data collected from the tests, in terms of mode shapes, storey displacement time histories, hysteresis loops and column drifts time histories were presented and commented. The damage pattern originated from the excitation was also briefly discussed.
The tests highlighted the strong effects of torsional irregularity on the column drifts, even for a limited level of plan eccentricity and relatively low levels of excitation. The comparison with the predictions derived in the pre-test numerical phase allowed some preliminary conclusions on the degree of confidence that should be given to conventional assessment procedures and modelling techniques in the case of plan-

wise irregular structures to be drawn. These issues are addressed in more details in a companion paper, Mola [6].

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A group of European partners take part in the SPEAR Project together with the ELSA Laboratory: the University of Patras (Greece), the Imperial College of London, the University of Ljubljana (Slovenia), the Universities of Rome and Pavia (Italy), the National Laboratory of Earthquake Engineering (LNEC) of Lisbon, the Higher Technical Institute of Nicosia and EQE International (London). The project is being co-ordinated by the ELSA Laboratory, whereas the University of Patras holds the administrative co-ordination. Overseas partnerships were also created with the Mid-America Earthquake (MAE) Centre of Urbana-Champaign, Illinois, the Building Research Institute (BRI) of Tsukuba (Japan), the Technical University of Tokyo and the Middle-East Technical University of Ankara.

Professor Michael Fardis, from the University of Patras, provided the original design of the structure and the suite of artificial accelerograms. The preliminary numerical analyses were a common effort of the whole SPEAR consortium. Mr. Seong-Hoon Jeong (MAE Center), Mr. Antonis Kosmopoulos and Prof. Michael Fardis (University of Patras) participated in the tests by providing the numerical simulations to decide the level of peak ground acceleration for the second test, based on the results of the first.

A bi-directional pseudodynamic test on a full-scale complete building is a highly innovative and challenging experiment, and whenever doing experimental work, many difficulties can be encountered. In the test described in this paper, difficulties were much more severe than expected: the test, which was a common effort of the whole ELSA staff, was made possible by the expertise, dedication, as well as forbearance and friendship of all staff. The contributions of Messrs. Philippe Buchet and Olivier Hubert are particularly acknowledged.

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