13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 3304

EVALUATION OF CURRENT APPROACHES FOR THE ANALYSIS AND DESIGN OF MULTI-STOREY TORSIONALLY UNBALANCED FRAMES

Elena MOLA[1], Paolo NEGRO[2], Artur V. PINTO[2]

SUMMARY

Plan-wise irregular buildings are quite common in many earthquake-prone areas of Europe and worldwide, making up a remarkably important category of existing structures. Irregular structures exhibit a complex behaviour under uni- or bi-directional seismic excitation because of torsional coupling effects affecting the response; due to the inherent complexity of the problem only simplified models have been developed and studied so far and a number of open issues still exist on the subject. Experimental activity is therefore badly needed in order to validate analytical studies and to point out the way for their future developments. In the framework of the research activity of the ELSA Laboratory of the Joint Research Centre, bi-directional pseudo-dynamic testing of a real size plan-wise irregular 3-storey frame structure was carried out in January 2004, as the core of a 3-year research project named SPEAR (Seismic PErformance Assessment and Rehabilitation).

The SPEAR project, specifically targeted at existing buildings, pursues the aim of improving current codified approaches to the assessment of older non-seismically designed structures, by means of a balanced combination of numerical and experimental activity.

The data made available by the unique SPEAR experimental activity are very important in themselves, given the scarcity or absence of test data on the behaviour of irregular multi-storey structures; in the present paper, they have been compared to the predictions resulting from the application, to the same structure, of current codified assessment approaches, thus allowing some conclusions to be drawn on the effectiveness of the latter in dealing with torsionally unbalanced buildings.

INTRODUCTION

In the last decades, the progress of research has made it clear that regularity is a most desirable feature of seismically designed structures, because it limits the likelihood of local and global misbehaviours during the earthquake excitation.

Nevertheless, irregular buildings, both in plan and in elevation, do exist in earthquake-prone areas; the first category is the largest and the object of the following study.

[1] Research Fellow, ELSA Laboratory, Joint Research Centre, Ispra, Italy. E-mail elena.mola@jrc.it
[2] Research Officer, ELSA Laboratory, Joint Research Centre, Ispra, Italy. E-mail paolo.negro@jrc.it

Some of the existing plan-wise irregular buildings are newly designed ones, for which the choice of irregularity is accompanied by careful consideration of its effects on the seismic response, and design is based on somehow more refined approaches than those usually reserved to regular buildings.
The majority of existing plan-irregular buildings, though, are older ones, which were designed for gravity loads only, before the recent seismic provisions were enforced, particularly in southern European countries. These structures add to the drawbacks originating from the plan eccentricity the problems due to under-designed elements or joints, poor local detailing and older construction practice.
The study of the seismic response of plan-irregular buildings has challenged researchers and raised strong interest during the last decades.
The complex nature of the problem, though, required the adoption of simplified models, most of them single-degree-of-freedom (SDOF) ones, based on a number of simplifying assumptions. This approach has a range of drawbacks: strong model-dependency of the results, difficulty in finding general trends and rules applicable to a vast category of asymmetric systems, difficulty in extending to multi-storey buildings the conclusions drawn after many years' research on SDOF systems.
Furthermore, the experimental approach to asymmetric structures has been very limited so far, especially when bi-directional eccentricity and real-size structures are involved, even if the need of experimental data has been advocated by researchers for calibration of the analytical models and to improve the understanding of the influence of structural parameters on the response, Rutenberg [1].
At present, the available computational tools make it possible to turn to fully non-linear 3D dynamic analysis of asymmetric structures, even if some practical aspects, together with the slow evolution of codes into this direction, still play an important role against such practice; nonetheless, before such kind of black-box analysis tools can be used with confidence, the behaviour of asymmetric multi-storey buildings must be thoroughly understood from a conceptual point of view.
For this reason, the PsD testing activity currently being carried out at the ELSA Laboratory on a bi-eccentric, real size, RC frame structure appears very relevant as to the improvement of knowledge in the field, with the aim of contributing to future improvements in the design and assessment guidelines currently enforced in EC8, whose inadequacies have been repeatedly reported, e.g. Tso [2]. A more detailed description of the SPEAR research project is given in a companion paper, Negro [3]; in the following only the aspects more strictly connected to the features of the seismic response of torsionally unbalanced structures are discussed.

DESCRIPTION OF THE STRUCTURE

The SPEAR building model is a simplification of an actual three-storey building representative of older 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 enforced 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 final 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 non symmetric in both directions (Fig.1, Fig.2)), 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 (CM).

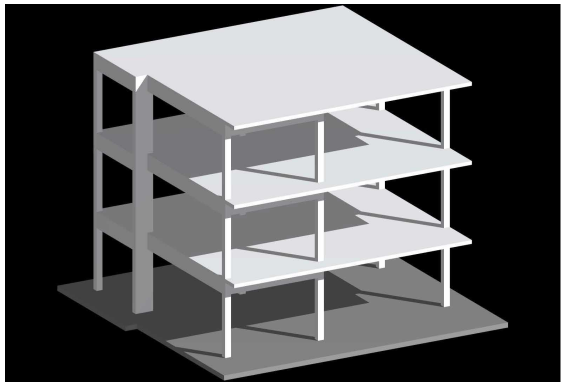

Figure 1 – The SPEAR structure

The concrete floor slabs are 150 mm thick, with bi-directional 8 mm smooth steel rebars, at 100, 200 or 400 mm spacing.
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 old practice; 8 mm smooth steel stirrups in beams 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. 2), has a cross-section of 250 by 750 mm, which makes it much stiffer and stronger than the others along the Y direction, which is the strong direction for the whole structure.
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 from the 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.

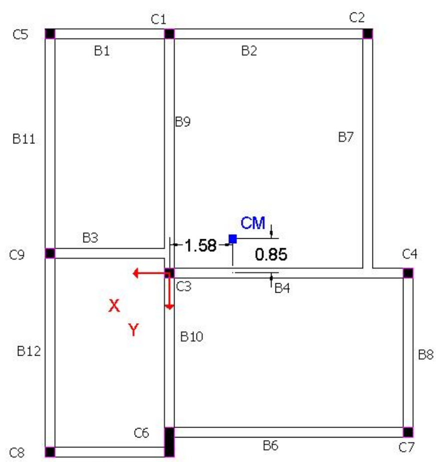

Figure 2 – Location of the CM of the structure

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.
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. 2). 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).

ASSESSMENT EXERCISE

Introduction

The first part of the research activity of the SPEAR project consisted in a critical review of the current code-format assessment procedures for existing RC buildings. To do so, they were applied to a number of benchmark structures, among which the torsionally unbalanced SPEAR structure, whose results are presented and discussed in the following.

The assessment procedures considered in the study were:

- FEMA 356 Guidelines (and FEMA 310 Guidelines) [4]
- Japanese Assessment Guidelines [5]
- EC8 Part 3 [6]
- New Zealand Building Authority Guidelines (Draft 1996 and 2002) [7]

Each of them has its own procedure for dealing with plan (and elevation) irregularity to take into account the torsional coupling effects on the displacements and internal actions, resulting into different evaluation of the so-called target displacement.

The preliminary calculations were conducted by enforcing the basic prescriptions suggested by each procedure and on these grounds, when necessary, implementing a finite element model by means of a commercially available programme.

In parallel, a number of numerical simulations were carried out, implementing models of the structure into different research oriented 3D nonlinear dynamic analysis computer codes. The comparison between the results so derived and the assessment outcomes allowed an initial evaluation of the latter to be made, provided predictions for the test-set up, and led to draw some initial evaluation of the model-dependency of the results of the numerical approaches.

The experimental test, on the other side, revealed the behaviour of the structure under an artificially generated spectrum-compatible ground motion record in terms of maximum displacement, global interstorey drifts at the CM and single column interstorey drifts; these results have two-fold significance, being the term of comparison for the results of both branches of the research activity: when compared to the predictions of the numerical simulations, they will allow the choice of the best modelling options to be made, thus leading to better tuning and validation of the models. When compared to the assessment outcomes, they showed whether each procedure was able to predict the fundamental features of the behaviour of an existing building (which means providing adequate and possibly safe-side estimate of maximum displacement, approximating the failure mechanism and highlighting local concentrations of ductility demands). In the following, the latter aspect is discussed.

Application of the assessment procedures to the SPEAR structure

In the following the most significant outcomes of the assessment procedures are reported.

The outcomes of the FEMA and New Zealand guidelines are reported together because they are both displacement-based methods, requiring the implementation of pushover models with very similar features.

The EC8 and Japanese Guidelines methods are also discussed together because they are both force-based methods, whose results are given in terms of demand-to-capacity ratios.

FEMA and New Zealand outcomes

In Fig. 3), the pushover curves of the FEMA model, for the two lateral load patterns and the two principal directions (positive and negative) are reported. In this case the hinges rotational properties were modelled according to FEMA 356 prescriptions. From the pictures, it can be observed that the structure exhibits an unsymmetrical behaviour, which leads to different results for the positive and negative X and Y directions, depending on the relative importance of the torsional component of the response with respect to the flexural. The strong direction (±Y) has slightly larger stiffness, due to the presence of the rectangular strong column, and the behaviour in this direction exhibits smaller global displacement. These remarks are valid for the FEMA-NZ model too.

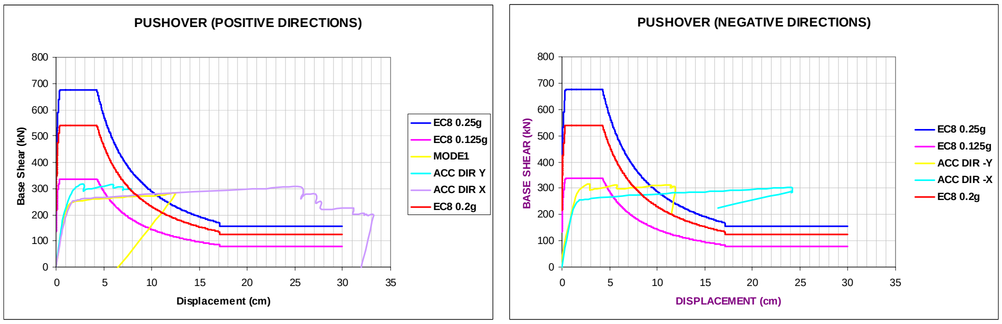

Figure 3 – FEMA model pushover

The predicted target displacements are above 50 mm for the 0.15g spectrum and about 85 mm for the 0.25g spectrum, both in the X and Y, positive and negative directions. It can be observed that the intersection with 0.25g spectrum is not reached by the capacity curve for the acceleration in the positive Y direction, which confirms the scarce ductility of the building in this direction.

The New Zealand force-based procedure allows for three different ways to perform the ductility checks for the structure, with an increasing level of difficulty.

The displacement-based approach uses the pushover curve too, but it requires some preliminary steps, such as the calculation of the storey sway potential index, to define the most probable failure mechanism. The sway potential index was above unity for the three storeys in both directions, thus indicating the large probability of column failure that also all the other procedures detected.

The New Zealand displacement-based procedure confirmed the different behaviour of the structure in the two directions; in Fig. 4 the capacity side of the application of the procedure is shown. The procedure yielded a response of incompliance for the 0.25 PGA excitation in the Y direction, with a demand to capacity ratio of about 1.28; for the same excitation in the X direction, on the contrary, a positive response came out, as the bare demand-to-capacity (DTC) ratio was of about 0.77. This result, which could be to a certain extent anticipated, since the displacement-based approach is less conservative than the force-based one, was obtained without taking into account the torsional effects that, as could be learnt from the other procedures, are more relevant in the weak direction.

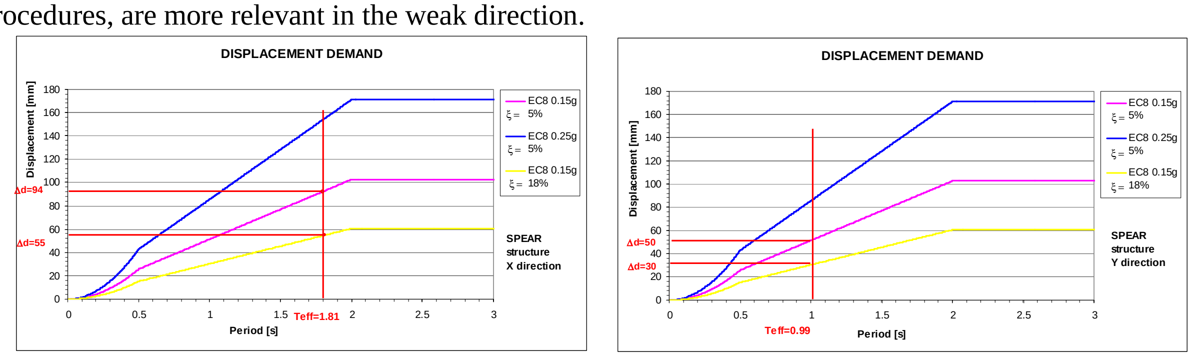

Figure 4 – New Zealand displacement-based procedure; demand in X and Y directions

A drawback of the procedure is the lack of a clearly specified means to account for torsional effects and leads to considering the structure incompliant for the 0.25 PGA excitation because a DTC of about 0.8; when increased to account for torsional effects is too likely to become greater than unity to consider the structure safe. The X direction yielded a result of compliance for the 0.15g excitation, with a DTC of about 0.5 and, for this excitation, also in the Y direction the evaluation was positive, but with a DTC of about 0.7.

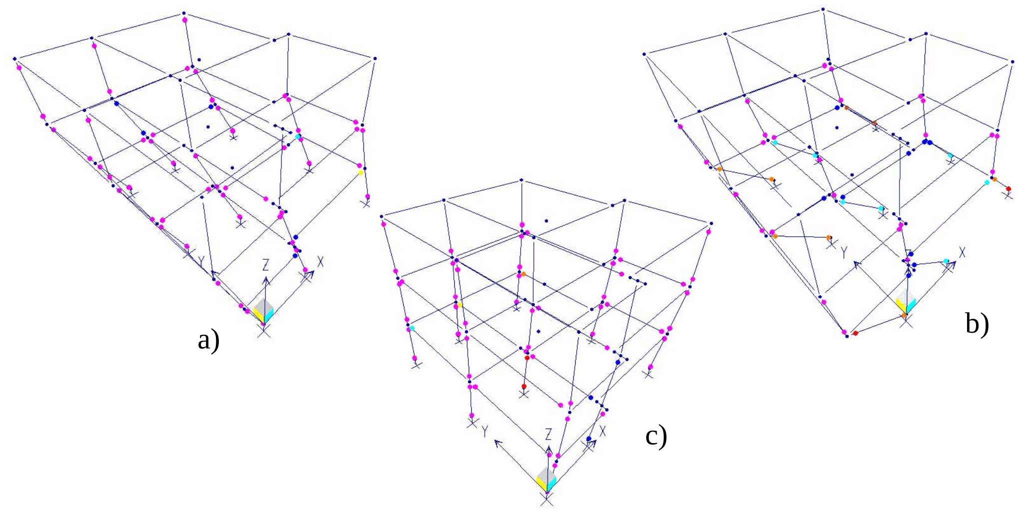

Figure 5 – Pushover mechanisms: a) Mode 1 load pattern, b) Acceleration +X load pattern, c) Acceleration +Y load pattern

As it can be observed from Fig. 5)-6), the Y direction pushover shows that the central column, (C3), reaches its ultimate curvature first, due to larger axial load, whereas the other columns show a quite uniform and more limited rotation pattern. This is an effect of the presence of the strong column, which spreads the developing hinges all over the three floors.

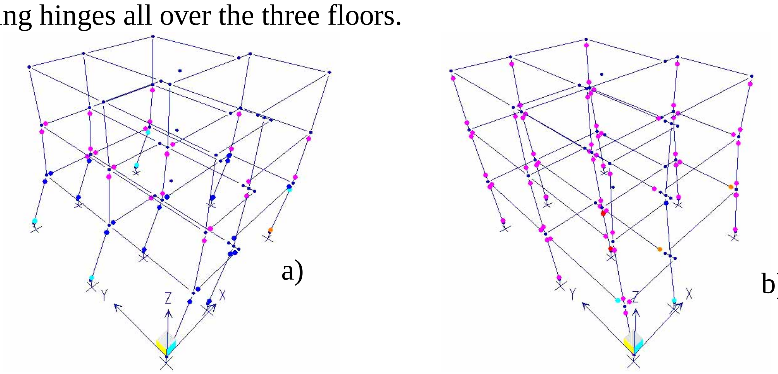

Figure 6 – Pushover mechanisms: a) Acceleration –X and b) Acceleration –Y

On the contrary, the X direction corresponds to a more dangerous rotation pattern, developing a mechanism involving above all the first floor columns, thus originating a “soft-storey” mechanism leading to large displacements. The plastic hinges form in the columns, resulting into a weak column-strong beam failure mechanism, which does not comply with the basic seismic capacity design rule enforcing the strong column-weak beam as the fundamental prerequisite for seismically safe structures, Priestley [8]. The failure mechanisms derived from the pushover analyses for the SPEAR structure for the Y direction show a smaller ductility and a quite marked descent in the final part, which is produced by the presence of the strong column. The influence of this strong column is clearly proven by the formation of plastic hinges in it only at its base, and only after reducing the development of plastic hinges in the weaker columns during the Y direction pushover. This shows that the final effect of this element might be beneficial.

On the contrary, the predictions for the weak direction, in which the difference in capacity and stiffness of the strong column with respect to the others is less relevant, led to predict larger rotations, particularly concentrated in the first floor columns, thus leading to larger displacements.

Japanese Guidelines and EC8 procedures outcomes

The outcomes of the Japanese procedure for the SPEAR structure, being based on force and sectional capacity considerations, led to predict the formation of column hinges in all columns, except for the strong one, in both directions. The outcome of the Japanese procedure for the main SPEAR structure was negative at all levels for a PGA value of the spectrum of 0.25g, as it could easily be anticipated from the results obtained by the pushover analyses performed for the other procedures; on the other hand, for a PGA value of 0.125g the response was negative both at level 1 and at level 2, and only at level 3 could a positive evaluation come out. In particular, even if the procedure is exclusively a force based one, therefore lacking the ability to properly highlight local ductility problems, it was able to draw the attention on the formation of plastic hinges almost exclusively in columns.

As for the EC8 procedure, it can be observed that it highlighted a large number of members (columns) whose DTCs are larger than unity, even for a PGA value of 0.125g; in particular, the procedure could find out that, in an equivalent elastic analysis, in the stiffer direction (Y) the structure behaves in a less satisfactory way (ductility-wise) than in the soft direction (X). Moreover, the procedure predicted very large flexural actions in the strong column, especially at the first storey; the same results were not derived by the Japanese procedure, which is based on a global storey capacity evaluation. It can be concluded that the approximate analysis by EC8 is less adequate, because the structural behaviour is far from being linear and because of the uncertainties on the behaviour factor to be assumed, but it was able to predict some important features of the structural response.

EXPERIMENTAL RESULTS

Introduction

In the following a short preliminary description of the experimental activity carried out in the frame of the SPEAR project is given; for a more detailed description see Negro, [3], Molina [9].

The first round of tests consisted of three PsD tests at different values of PGA: a small test at a PGA of 0.02g, a test at the PGA value of 0.15g and a higher level test at 0.2g PGA.

The bi-directionality of the PsD test, consisting in the simultaneous application of the longitudinal and transverse component of the earthquake to the structure, introduced 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 needed to be taken into account: two translations and one rotation along the vertical axis, as opposed to the single degree of freedom that is considered in conventional unidirectional PsD testing.

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

The PsD integration of the horizontal response of the structure was performed in terms of three generalized DoFs at each floor, consisting in the in-plane displacements d_X and d_Y and of the rotation along the vertical axis d_θ at the center of mass (CM) of the structure.

The input signal chosen for the test was the Herceg-Novi pair of records (H-N longitudinal and H-N transverse), due to the stability of the response of the structure under those records in terms of interstorey drift time histories, which were obtained from pre-test numerical simulations.

Once the accelerogram was chosen, the most appropriate direction of application was determined: based on the criterion of maximizing the effects of torsion on the response, 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.2), and of the Y component in the –Y direction of the same reference system.

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 and its intensity.

The aim of the test, in view of the subsequent phases of the project, was to investigate the behaviour of the structure with significant damage, but not so severe as to be beyond repair.

The outcomes of the assessment procedures were all negative when a PGA of 0.25g was considered, whereas they had given positive responses as for the 0.125g PGA and, in some cases, also for the 0.20g PGA. Based on the results of preliminary nonlinear numerical simulations, the intensity level advised 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. Before the main test, a preliminary, low-level, control test was performed, with a PGA of 0.02g.

0.02g input results

The primary aim of the control test at low PGA level, apart from that of a technical check of the computational, control and acquisition equipments, was to derive information about the initial properties of the structure, mainly represented by its mode shapes and frequencies.
Nine frequencies were derived, (nine DoFs were assumed for the system); the three lower frequency modes were: 1.19 Hz for the fundamental mode, with shape mainly flexural in the X direction, 1.29 Hz for the second mode, mainly flexural in the Y direction, 1.5 Hz for the third, mainly torsional, mode.

0.15g input results

Displacements time-histories
In Fig. 7) 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 Fig.2). The maximum displacements were about 70 mm in the X direction and 50 mm in the Y direction; the maximum rotation was around 12 mrad at the third storey.

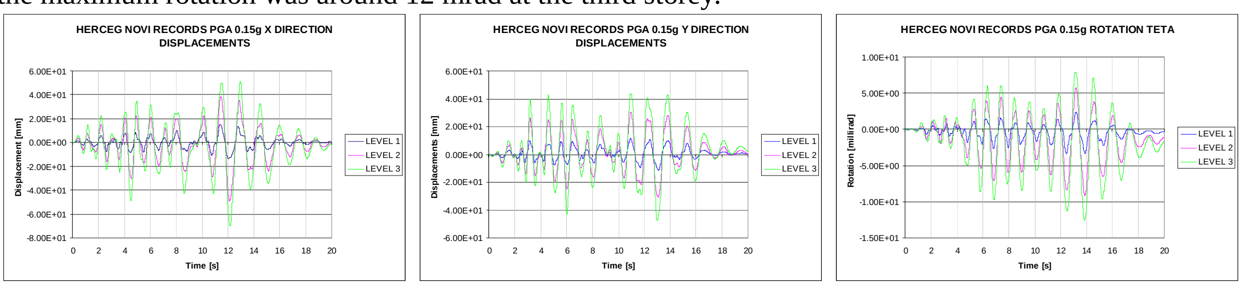

Figure 7 – Displacements time-histories: a) X direction, b) Y direction, c) rotation θ

After performing the 0.15g PGA test, it was decided to run a further test to increase the level of non-linearity in the behaviour of the specimen.

0.20g input outcomes

Displacements time-histories
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. The maximum displacements were slightly above 100 mm both in the X direction and in the Y direction; the maximum rotation was around 20 mrad at the third storey.

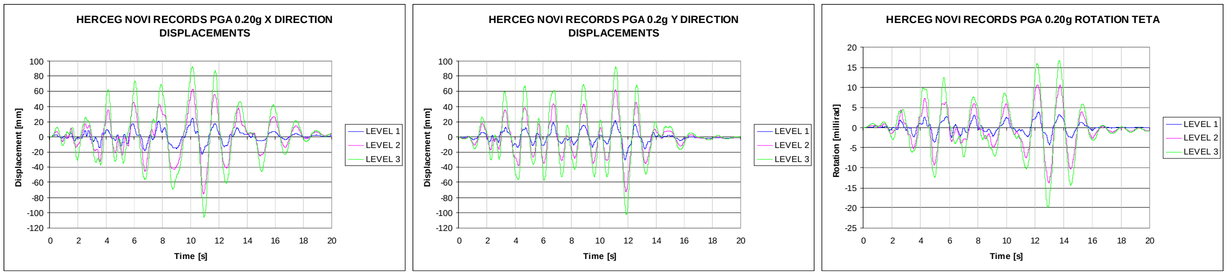

Figure 8 – Displacements time-histories: a) X direction, b) Y direction, c) rotation θ

Floor hysteresis loops

In Fig. 9)-11) the storey hysteresis loops for the X, Y and θ directions are reported. The most affected level is level 2 and the direction where the absorbed energy is larger is the Y direction.

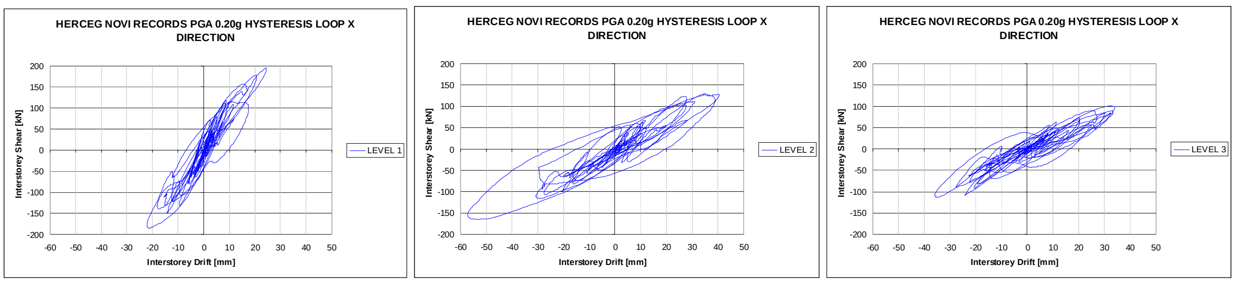

Figure 9 – Hysteresis loops in the X direction

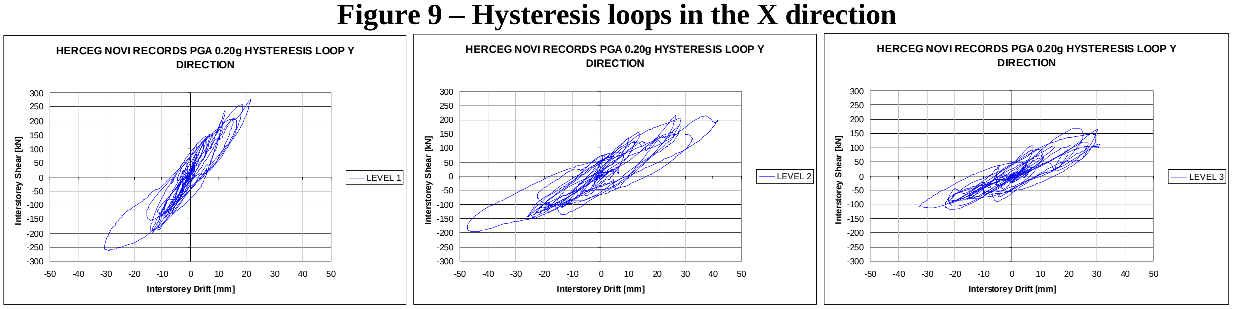

Figure 10 – Hysteresis loops in the Y direction

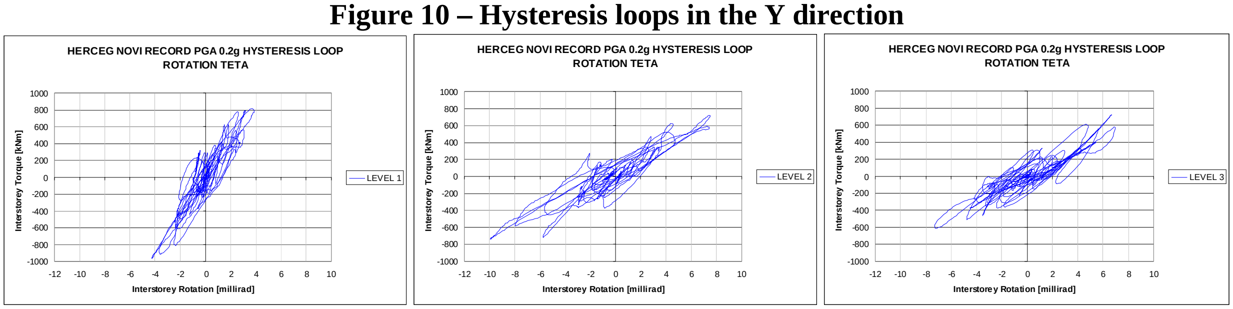

Figure 11 – Hysteresis loops in rotation θ

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, which was also assumed in the test, by means of simple geometric considerations.

The X and Y direction drifts of each column were 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 was being calculated.

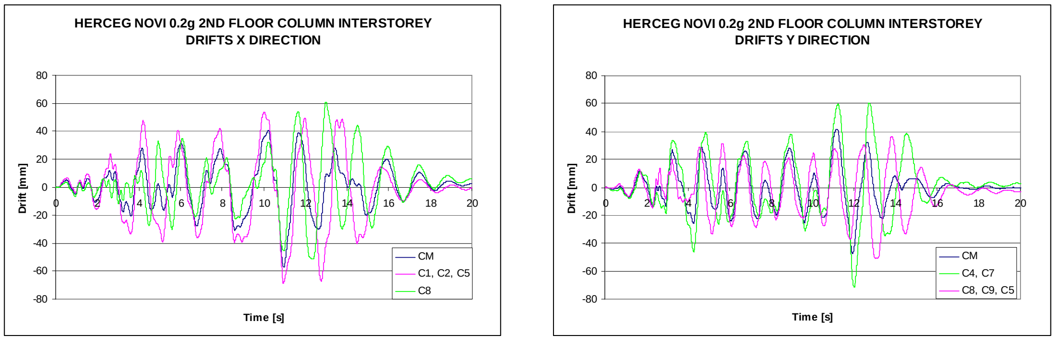

Figure 12 – Second storey largest column drifts compared to those of the CM a) in the X direction, b) in the Y direction

It is 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, C4, C7).

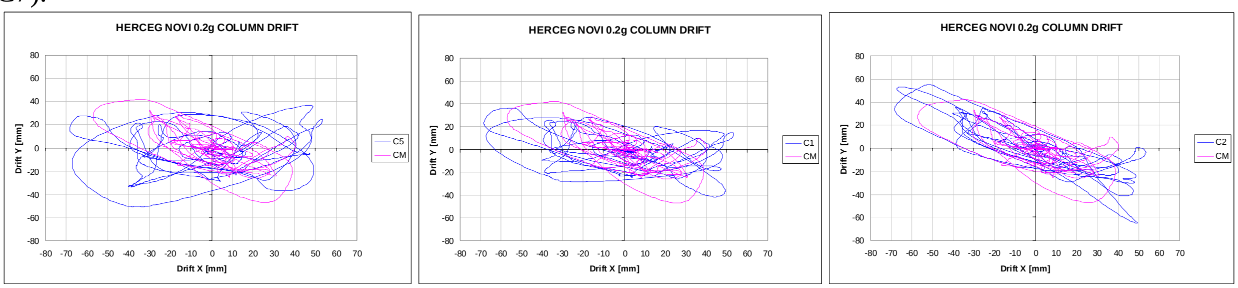

Figure 13 – Second storey column drifts compared to that of the CM: columns C5, C1, C2

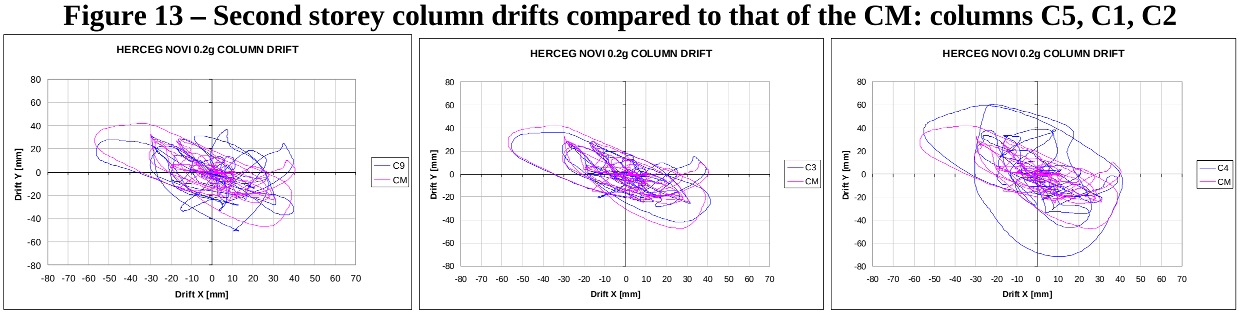

Figure 14 – Second storey column drifts compared to that of the CM: columns C9, C3, C4

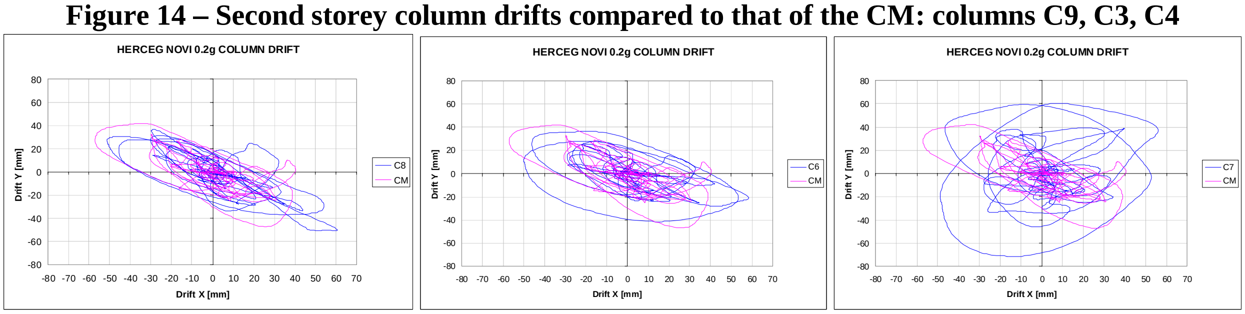

Figure 15 – Second storey column drifts compared to that of the CM: columns C8, C6, C7

Fig. 12) allows a comparison between the drift measured at the CM and the drifts of the edge columns, the most deformed ones, to be made. The effects of torsion on the drifts of the edge columns are remarkable in both directions. In the X direction, where the structure is more flexible and the drift at the CM is already quite large, the maximum drift reached at the CM was 55 mm, whereas the maximum drift reached at the edge columns C1, C2 and C5 was about 70 mm, a difference that 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. 13)-15) show a different representation of the interstorey drifts of the second storey for each of the nine columns; the drifts in the Y direction are plotted against those in the X direction. The plots refer to the second floor because it was the one exhibiting the largest drifts. The different levels of bi-directional drift demands of each column can thus be observed, together with the correlation between the X and Y components.

COMPARISON BETWEEN THE EXPERIMENTAL AND THE ASSESSMENT RESULTS

Maximum displacements

In Fig. 16), the CM drifts are plotted (Y direction drift against X direction one) together with the boundaries of target displacements that were derived from the FEMA and FEMA-NZ model pushover in terms of interstorey drifts.
It can be observed that the two procedures and the experimental results did not agree in determining the storey with the largest demands in terms of displacements: as mentioned above, the failure mechanism highlighted by the assessment procedures was of the soft-storey kind in the X direction at the first floor; due to the larger axial load that the first storey columns had to sustain, they resulted the least ductile and those who would suffer the most, up to the formation of a storey mechanism leading to large displacements.
The test, though, showed that the floor with the largest drifts was the second one; the second level also absorbed more energy than the other two.

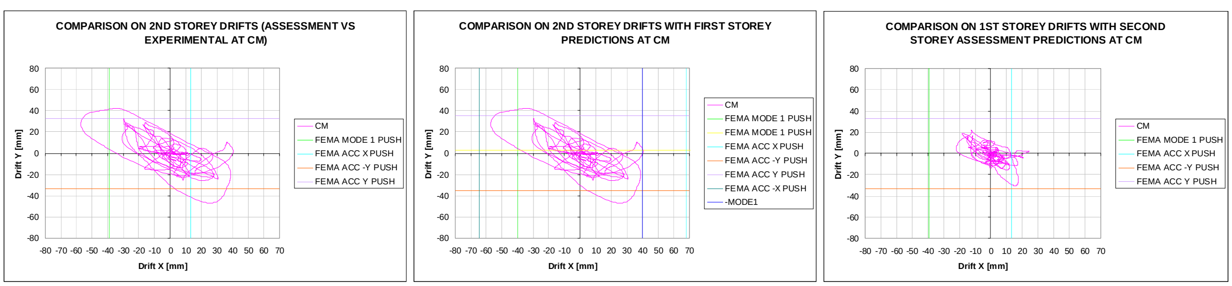

Figure 16 – First and second storey drifts at CM compared to the target displacements from the FEMA procedure

This can clearly be observed from Fig. 16): Fig. 16 a) in fact shows that in the X direction the FEMA procedure considerably underestimated the displacements at the CM of the second storey; the upper bound in the X positive direction, given by the light blue line, is less than half the value reached during the test; in the negative X direction the agreement was better, but still the displacements reached during the test were about 60 mm against 40 mm of the prediction.
The main difference remains with the fact that the storey mechanism took place at the second storey, not at the first one.
To better understand the discrepancy, in Fig. 16 b) the experimental second storey CM drifts are plotted together with the FEMA boundaries for the first storey drift in the X direction; from this comparison it can be understood whether the order of magnitude of the estimates was correct but assigned to the wrong storey. From Fig. 16 b), though, it can be observed that in this case there was a remarkable overestimation of the maximum displacements mainly in the positive X direction when the light green line is considered; in the negative X direction once again there is a better match. Finally, in Fig. 16 c), the first storey experimental drifts at the CM are compared to the limits obtained by the procedure for the second storey; also in this case from the comparison it can be understood whether the order of magnitude of the

predictions was correct. In this case, in the negative X direction a strong overestimation of the displacement is shown, whereas in the positive X direction the underestimation of the displacements is evident: the second storey estimation is still smaller than the first storey experimental drift.
The conclusion that can be drawn is that in the test the second storey took the role that was predicted for the first; the predicted failure mechanism was then wrong and led to an underestimation of the maximum displacements at the second storey and an overestimation of those at the first storey. The different load patterns of the pushover analyses yielded different target displacements: it can be concluded that the load pattern following the first modal shape performed better than the uniform acceleration pattern in the weak direction; it must also be observed that the positive drifts estimates were worse than the negative ones. This latter observation highlights the difficulties in obtaining equally reliable predictions in both fundamental directions, arising from the asymmetry of the structure.
Moreover, it must be taken into account that the drifts at the CM, that were considered in this paragraph, are smaller than the drifts of the individual columns, for which the effects of the rotation at the CM are to be taken into account, as it will be discussed in the following. This means that a further gap between the predictions and the edge column interstorey drifts (those that are to be considered when assessing an irregular structure) came out.

Column drifts

Interesting comments can be made about Fig. 13)-15), when observing the plots of the single column interstorey drifts together with the plot of the drifts of the CM: the remarkable effects of torsion on the response are evident.
In Fig. 13)-15) the plots of the drifts are arranged along the plan-wise configuration of the structure, as can be seen from Fig.2): this is helpful in understanding how the relative positions of the CM, the centre of stiffness (CR) and of each column, all lead to develop different drift histories and considerably different demands on the individual elements.

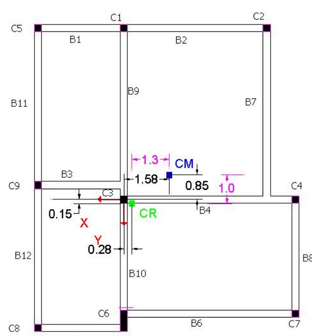

Figure 17 – Relative location of the CM and the CR of the structure in plan

First of all, to better understand the motion of the structure, it is necessary to locate the CR of the system. It is known that different definition of this locus can be adopted: in this case the location CR of the structure was computed following the prescription of EC8 [10], taking into account only the contribution of the moments of inertia of the columns, and is represented in Fig. 17 a). At floors 1 and 2 the CR is eccentric with respect to the CM by 1.3 m in the X direction and 1.0 m in the Y direction, respectively about 13% and 9.5% of the total plan dimensions. At floor 3 a very slight difference in the eccentricity occurs; it becomes 1.34 m and 1.04 m in the X and Y directions respectively. The CM and the CR are almost perfectly located along the diagonal connecting column C8 and C2; on the other hand, the CM is located along the diagonal connecting C4 and C5, whereas the CR is displaced of about 1.6 m along the opposite diagonal, as can be observed in Fig.17 b).
The input signal consisted of two components applied along the – X direction and the –Y direction.

The structure mainly displaced along a 45° direction (the C4-C5 diagonal), as can be seen from the observation of the CM drifts in Fig. 16).
Moreover, it can be observed that column C3, which is located very close to the CR, and close to the CM, exhibited the least significant drift increments in both directions, but especially in the X direction, where the eccentricity is smaller. This is in agreement with the reconstruction of the motion of the structure that was drawn from the experimental data.
On the contrary, all the other columns were affected by torsion: the edge columns in the Y direction were affected by torsional effects in their drifts in the X direction, whereas the reverse happened for the X direction edge columns, which were affected in their maximum drifts in the Y directions.
When comparing the relative importance of torsion-effected drift increments for the X and Y edge columns, it can be seen that columns C7 and C5 are the most affected: this means that the Y direction edge columns suffered the most. This is in good agreement with the relative position of CM and CR above analysed: along the line C1-C4 there is a remarkable distance between CR and CM, the Y eccentricity is larger and the two columns are the farthest from the CR.
Finally, it must be observed that the significant extent of the difference between the drifts at the CM and the edge column drifts is one of the most important outcomes of the test: the torsional effects on the response turned out to be much larger than predicted.
In fact, 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 remarkable 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.
This result confirmed one of the basic remarks that had been drawn at the end of the assessment exercise with regard to the use of pushover analysis in assessing the displacement capacity of irregular multistory buildings: conventional pushover tends to underestimate the displacements because the pushover curve refers to the CM of the model. The prescription of the procedure to take into account the increases of displacements on the edge elements should thus be more precise and compelling, because, as was demonstrated in this case, to reach a safe-side estimation of the flexible edge elements drifts, increases as high as 50% of the displacements at the CM can be necessary even for moderate eccentricity.
These large increases are larger than could be expected when considering the values of plan eccentricity of the structure in themselves: an eccentricity of around 10% is defined in all procedures as not likely to have major effects; for example, the structure falls in the EC8 category where the separate analysis in the two main direction is allowed and in the Japanese Guidelines it is in the category with the lowest capacity-reductive factor.
Nevertheless, it was proven by the test that in this case the interaction of the eccentricity in both directions had a role that cannot be neglected in enhancing the torsional effect on the response even if the two eccentricity values were not large.
From Fig. 8) it can also be observed that the rotation time-history exhibited its maximum effects in the final range of the response (from 10s to 15s); it was in fact observed that the structure moved mainly in its translational modes in the initial part of the excitation, following the first modal shape, which was mainly flexural in the weak (X) direction. In the meanwhile the values of the total energy absorbed by the rotational mode were smaller in comparison to those absorbed by the translational DoFs, as can be seen in Fig.18); moreover, in the last part of the response in Fig. 12) it is shown that when the rotation strongly increased and the torsional component became very important, it caused a shifting of the column drifts with respect to that of the CM and the strongest increase in the maximum values of the edge column drifts. This can also be seen in Fig. 18): in the same time range, the energy absorbed by the rotation reaches its maximum (negative, since there is no input energy associated to torsion, therefore the absorbed energy is the opposite of kinetic energy) values.

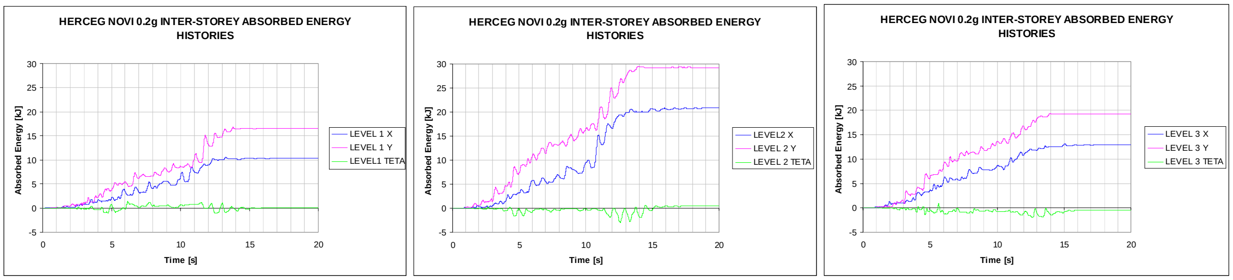

Figure 18 – Herceg-Novi 0.2g input: interstorey total absorbed energy histories

This behaviour highlights one of the widely agreed conclusions drawn from research on the torsional response of asymmetric structures: the effects of higher modes and the dynamic amplification of eccentricity are very important factors in affecting the behaviour of multi-storey plan-irregular buildings, Rutenberg [1]. For this reason the drawbacks of the static equivalent approach even corrected with the use of design eccentricity, must not be forgotten and the caution in extending to multi-storey buildings the conclusions drawn from single storey models was justified by the results of the test hereby described.

CONCLUSIONS

From the comparison between the outcomes of the assessment procedures and the experimental results important drawbacks of the current codified approaches for the assessment of torsionally unbalanced multi-storey buildings were highlighted.
Besides establishing relative comparisons among the effectiveness of the different procedures that were considered in the study, it can be concluded that all approaches failed to give correct and safe-side estimations of the important features of the structural response.
The procedures were correct in assessing the role and behaviour of the strong column though: even if at first sight this column might appear critical because of its higher stiffness and because it is the largest source of irregularity, its effect turned out to be beneficial in improving the behaviour of the structure in the Y direction, without suffering any damage. This was correctly predicted by all assessment procedures, both the force- and the displacement-based ones.
On the other hand, they failed in predicting the global failure mechanism of the structure; in fact, a first-floor soft-storey mechanism was predicted by all the pushover analyses, whereas in the test the second storey was the most affected, with larger drifts and absorbed energy. This confirms that much care should be paid in applying simplified SDoF procedures to multi-storey irregular buildings.
Neither the absolute values of such drifts, though, were adequate: the maximum displacements at the CM were underestimated; moreover, it must be noted that the effects of torsion on the individual column drifts were very large even if the eccentricity had been dismissed by many procedures as “minor”. This led to a further underestimation of edge displacements and to neglecting the possible interaction of the two plan eccentricities, that during the test corresponded to unpredicted effects.
In addition to these preliminary conclusions, the results of the experimental activity represent an unprecedented wealth of data from which much more information about the seismic behaviour of irregular structures will be derived.

ACKNOWLEDGEMENTS

The Project SPEAR is being funded by the European Commission under the “Competitive and Sustainable Growth” Programme, Contract N. G6RD-2001-00525. Access to the experimental facility took place by means of the EC contract ECOLEADER N. HPRI-1999-00059.
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.

REFERENCES

1. Rutenberg, A. "EAEE Task Group (TG) 8: Behaviour of irregular and complex structures, asymmetric structures – Progress since 1998", *Proc. of 12th European Conference on Earthquake Engineering*, Paper N. 832, Elsevier Science Ltd, London, 2002
2. Tso, W.K., Wong, C.M. "Eurocode 8 seismic torsional provisions evaluation", J. *of European Earthquake Engineering*, 1, 23-33, 1995
3. Negro, P., Mola, E., Molina, F.J., Magonette, G., "Full-Scale PsD Testing Of A Torsionally Unbalanced Three-Storey Non-Seismic RC Frame", *Proc. of 13th World Conference on Earthquake Engineering*, Paper N. 968, Vancouver, 2004
4. ASCE-FEMA Prestandard and commentary for the seismic rehabilitation of buildings, FEMA 356, November 2000
5. Japan Building Disaster Prevention Association, "Standards for seismic capacity evaluation of existing reinforced concrete buildings", 1977 revised 1990 (in Japanese)
6. Eurocode 8: "Design of structures for earthquake resistance, Part 3: Strengthening and repair of buildings", Doc CEN/TC250/SC8/N293, Draft No 1, 2001
7. New Zealand National Society for Earthquake Engineering (2002) "The Assessment and improvement of the structural performance of earthquake risk buildings", Draft 06 May 2002
8. Priestley, M.J.N., (1997) "Displacement-based seismic assessment of reinforced concrete buildings", *J. of Earth. Eng*., 1(1)
9. Molina, F. J., Buchet, Ph., Magonette, G.E., Hubert, O., Negro, P., "Bidirectional pseudodynamic technique for testing a three-storey reinforced concrete building", *Proc. of 13th World Conference on Earthquake Engineering*, Paper N. 75, Vancouver, 2004
10. Eurocode 8: "Design of structures for earthquake resistance, Part 1: General rules, seismic actions and rules for buildings", Doc CEN/TC250/SC8/N269, Draft No 1, 2000