SYMMETRIC 3D R/C BUILDINGS SUBJECTED TO BI-DIRECTIONAL INPUT GROUND MOTION

Giuseppe FAELLA[1], Vojko KILAR[2] And Gennaro MAGLIULO[3]

SUMMARY

The influence of the orthogonal horizontal component of the input ground motion on the seismic response of a reinforced concrete building is analysed by carrying out a comparison with the response under unidirectional excitation. To gain reliable results, a preliminary phase of the paper is devoted to the numerical modelling of a four storey full scale r/c bare frame building tested in the ELSA laboratory at Ispra. The beams are idealised by uniaxial bending models; the columns either by multi-spring models either by two coupled uniaxial bending models in order to identify the modelling which better matches the experimental results.

The numerical analysis of the seismic response of the building under bi-directional input ground motion is performed for using an ensemble of five real earthquakes. The results are analysed in terms of global and local response parameters. It is shown that the increment in damage produced by the action of the orthogonal horizontal component is particularly evident when the seismic behaviour is analysed at the local level of the member sections.

INTRODUCTION

The seismic design of R/C structures should be performed by considering the variability of the ground motion incidence angle, or alternatively, by taking into account both the horizontal components, in addition to the vertical component when it is relevant. As regards the horizontal seismic action, Eurocode 8 provisions [CEN 1994a, CEN 1994b], as other seismic codes, require to consider two orthogonal components taken as independent and represented by the same response spectrum. The maximum value of each action effect on the structure due to the two components may be estimated by the square root of the sum of the squared responses to each of them. As an alternative, for each direction the action effects may be computed adding to those due to the application of the seismic action along that direction the 30% of the effects due to the application of the seismic action along the orthogonal direction. The sign of each component in this combination is to be taken as the most unfavourable for the effect under consideration.

When time history analysis is used and a spatial model of the structure is adopted, simultaneously acting accelerograms are to be considered for the horizontal components. As far as special provisions for columns [CEN 1994c] of High Ductility Class, the biaxial bending has to be considered in the design resistance evaluation and verification; for columns of Medium and Low Ductility Class biaxial bending may be considered in a simplified way by carrying out the verification separately in each direction, with the bending resistance reduced by 30%. These provisions should take into account the larger input energy due to the presence of both earthquake horizontal components, the triaxial interaction effects (variation of the section strength domain due to the interaction of the two bending moments and of the axial force), the variability of the ground motion incidence angle and the capacity of the resisting elements to withstand earthquake forces acting in any horizontal direction. However, their adequacy for reinforced concrete three-dimensional frame buildings is steel to be verified.

[1] *Università degli Studi di Perugia, via Duranti, 06125 Perugia, Italy. E-mail: faella@unipg.it*
[2] *University of Ljubljana, Jamova 2, 1000 Ljubljana, Slovenia. E-mail: vkilar@ikpir.fagg.uni-lj.si*
[3] *Università degli Studi di Napoli Federico II, p. Tecchio 80, 80125, Napoli, Italy.gmagliul@unina.it*

In some papers the biaxial response of single mass two-degree-of-freedom structures has been studied [De Stefano & Faella, 1996], but the adopted models are too simple to represent the actual seismic behaviour of a reinforced concrete building. In [Kunnath & Reinhorn 1990, Zeris & Mahin 1991] spread plasticity models have been developed in order to analyse the response of reinforced concrete members subjected to cyclic biaxial flexure with constant or varying axial force; such models, however, are used to analyse the behaviour of members which are just parts of a three-dimensional structure. Nowadays reliable computer programs for 3D R/C buildings nonlinear dynamic analysis which take into account the triaxial interaction effects, as DRAIN-3DX [Prakash et Al. 1993, Powell & Campbell 1994] and CANNY-E [Li 1996a, b] are available. Nevertheless, to obtain reliable results a faithful modelling is to be set up: because of the great number of parameters which condition the seismic response of 3D reinforced concrete structures and the difficulty to determine them only according to theoretical evaluations, a comparison with experimental tests is necessary.

In this paper the seismic response of a suitably modelled reinforced concrete building under both the two horizontal components of several real earthquakes is compared to its response under the principal component of the same earthquakes, to evaluate how the second component modifies response and damage of the structure. A faithful numerical model is set up by fitting the results obtained from pseudodynamic tests performed on the building in the ELSA laboratory at Ispra [Negro et Al. 1994].

Figure 1. Reference test building [Negro et Al. 1994].

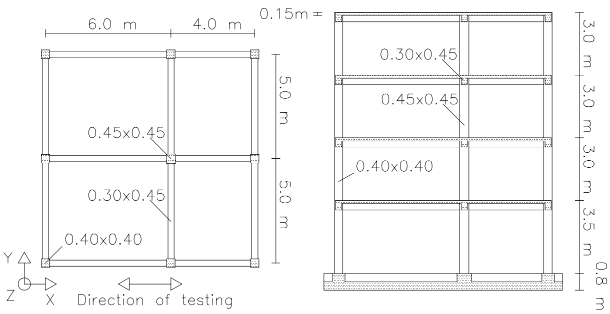
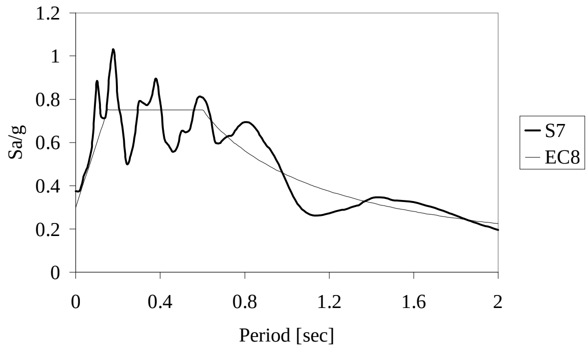


Figure 2. Eurocode 8 and S7 elastic spectra.



REFERENCE TEST BUILDING AND RESULTS

The reference test structure [Negro et Al. 1994] is a four-story full-scale r/c frame building [Fig. 1], designed in accordance with the prescriptions of Eurocode 2 and 8 for High Ductility Class structures. Dimensions in plan are 10 m x 10 m; interstorey heights are 3.0 m, except for the ground storey which is 3.5 m high. The structure is symmetric in the direction of testing (X direction), with two equal spans of 5.0 m, whilst in the other direction it is slightly irregular due to the different span lengths (6.0 and 4.0 m). All columns have square cross section with 400 mm side, except for the interior column which is 450 mm x 450 mm; all beams have rectangular cross section, with total height of 450 mm and width of 300 mm. A solid slab, with thickness of 150 mm, was adopted for all storeys. The materials used are normal-weight concrete C25/30 and B500 Tempcore rebars. In the preliminary design of the building additional dead load to represent floor finishing and partitions equal to 2.0 kN/m^2, live load equal to 2.0 kN/m^2, peak ground acceleration of 0.3g, soil type B, importance factor equal to 1 and behaviour factor q equal to 5 were assumed.

The seismic input is an artificial accelerogram, called S7, generated by using the waveforms derived from real signals recorded during the 1976 Friuli earthquake: its response spectrum fits the one given by EC8 for soil profile B at 5% damping [Fig. 2]. High and low level pseudodynamic tests were performed in the European Laboratory for Structural Assessment (ELSA) of the Joint Research Centre of the European Commission at Ispra (VA), using such accelerogram S7 scaled by 1.5 and 0.4 respectively. Instrumentation included both active and passive measurements in order to obtain the four floor slabs displacements, restoring forces, the column ends rotations, the columns axial deformations and the energy dissipation [Negro et Al. 1994]. The first three periods of the structure amount to: 0.56 sec (X), 0.54 sec (Y), 0.39 sec (R).

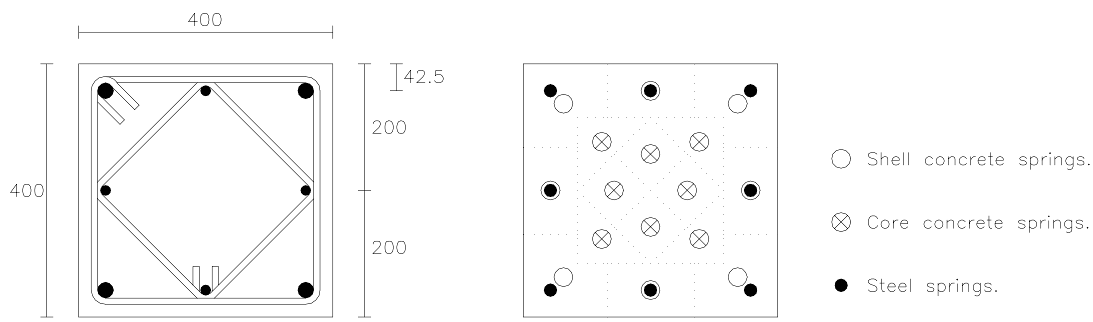


Figure 3. Geometry and modelling of a corner column section

NUMERICAL MODELLING

The model was set up using all the available theoretical and experimental data and comparing the numerical results to the tests in terms of top displacement and base shear time histories in order to achieve appropriate values for relevant parameters; CANNY-E computer program was used to perform the dynamic numerical analyses and reference to the high level pseudodynamic test was done.

The geometry of the modelled building and the masses, concentrated in the centre of mass of each floor, are taken from [Negro et Al. 1994]. The damping matrix is assumed to be proportional to the instantaneous stiffness matrix, assigning a damping ratio equal to 2%. This value comes from dynamic snap-back tests [Negro et Al. 1994] performed in ELSA laboratory on the reference building before the pseudodynamic ones.

The beams are idealised by a non-linear uniaxial bending model [Li, 1996a], with elastic shear deformation; their axial and torsional deformations are not taken into account. The inelastic flexural deformation is lumped at the element ends and is given by the rotation of two nonlinear springs, which are connected to the joint by a rigid zone. The moment-rotation relationships in the two rotational springs are computed based on moment-curvature relationships, assuming an asymmetrical moment distribution along the length of the element. A tri-linear skeleton curve is assigned to represent the cross section behaviour before and after cracking and yielding. The characteristic moments and curvatures are computed according to either experimental results, which are obtained from tests conducted on the materials constituting the structure [Negro et Al. 1994], either theoretical evaluations: the core concrete cracking and yielding curvatures are computed increasing the maximum and the ultimate strength and the deformation by the Mander & Priestley formulae [Mander et Al. 1988, Paulay & Priestley 1992]. For the after yielding stiffness, when the upper part of the section is in tension, an increased value is assumed with respect to the one theoretically computed: in particular, it is 5% of the initial stiffness for the beams of the external frames and 10% for the beams of the internal ones. In this way the progressive spreading of the region in which slab reinforcement yields, when the beam rotation increases, is modelled, even though the section is assumed rectangular. The hysteretic behaviour follows the Takeda rules, but the pinching effect is also considered. Best correlation with the experimental results are obtained assuming small values for the unloading stiffness (in each cycle it is reduced of 50% with respect to the previous one).

The columns are idealised by two different models (non-linear uniaxial bending models and multi-spring models [Li, 1996a]) in order to compare the effects of modelling on the seismic response of the structure. As regards the first modelling, each column has been idealised by two coupled uniaxial bending elements to provide resistance capacity in two orthogonal directions. Furthermore, two different sets of values are selected for the hysteretic parameters of such models while their skeleton curves are equal and assigned as for the beams. In the first set the values which define the simplest possible hysteresis behaviour are settled (simple uniaxial): the unloading stiffness from the primary curve has the biggest possible value, the unloading stiffness in the interior loops is always constant and the pinching effect is not considered. In the second one (best uniaxial), the hysteretic parameters are selected to fit better the experimental results: the settled values are the same as for the beams, but the internal loop unloading stiffness is assumed equal to that from the primary curve. This is due to the lower damage suffered by columns with respect to the beams, according to the provisions of the Eurocode 8 seismic design. As a consequence, the variation of the parameters which govern the nonlinear behaviour of the beams influences more the seismic response than the variation of parameters that govern the column nonlinear behaviour.

The multi-spring model has a central linear elastic element and two multi-spring elements at the base and the top of the column, connected to the joint by a rigid zone. Each multi-spring element consists of an assigned number of uniaxial springs, which carry axial force and receive axial displacement; the spring displacement is based on a plane section assumption and determined from the inelastic flexural rotation and the axial deformation of the whole multi-spring element. It includes the interaction among the bi-directional bending moments and the axial load. The element has an elastic shear and torsional behaviour. The assumed element is characterised by 24 springs [Fig. 3]: steel force-displacement relationships are assigned to 8 of them, shell and core concrete force-displacement relationships to the other 16 springs. These relationships are assigned according to either experimental tests conducted on the materials [Negro et Al 1994], either theoretical evaluations as for the beams. In the modelling, the experimental values of the deformations correspondent to the concrete maximum strength and the steel yielding strength are increased: this is assumed to try to model the stiffness degradation caused by cracking which may develop over a zone longer than the inelastic end segment prefixed length [Li, 1996a] and the rotation caused by bond slip of tension reinforcing steel along its embedded length in the joint. The steel unloading stiffness is taken equal to the initial one, while a concrete unloading stiffness degradation is assumed as advised in [Li, 1996a].

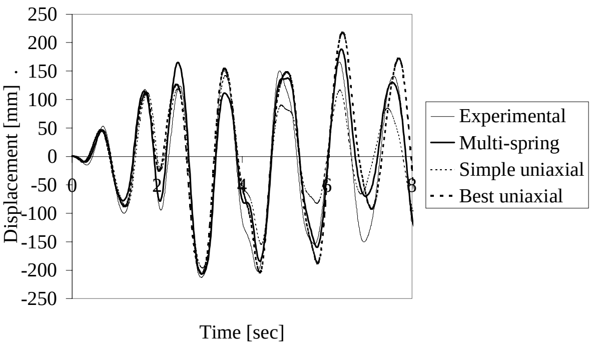


Figure 4. Top displacements for different models.

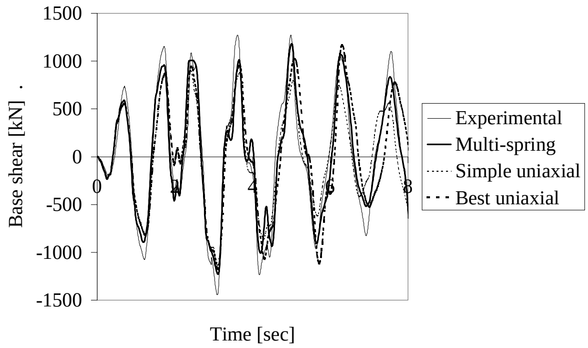


Figure 5. Base shear for different models.

In Figure 4 the experimental response in terms of top displacement time history is compared with the responses obtained by the three used models. The figure shows that the “best uniaxial” and the “multi-spring” models provide similar performance even though the latter one lead to the best correlation with the experimental results, especially after several cycles in the inelastic range of behaviour. Such results are confirmed in terms of base shear, as it is shown in Figure 5.

SEISMIC RESPONSE UNDER BI-DIRECTIONAL INPUT GROUND MOTION

To study the response of a reinforced concrete frame building under bi-directional ground motion, the above modelled structure is lightly modified. A doubly symmetric building is obtained either shifting the central frame in Y direction on the axis of symmetry and by changing the reinforcement in some sections of the beams. In this way the effects of the action of the secondary horizontal component on the seismic behaviour are isolated from the effects of plan asymmetry.

Table 1. Earthquake records used as input ground motion.

Earthquake	Date	Station	Duration [sec]	Primary component PGA [g]	Secondary component PGA [g]
Imperial Valley	18.05.40	El Centro	53.40	0.348	0.214
Kern County	21.07.52	Taft	54.40	0.179	0.156
Montenegro	15.04.79	Petrovac	19.60	0.438	0.305
Valparaiso	03.03.85	El Almendral	72.02	0.284	0.159
Northridge	17.01.94	Newhall	59.98	0.590	0.583

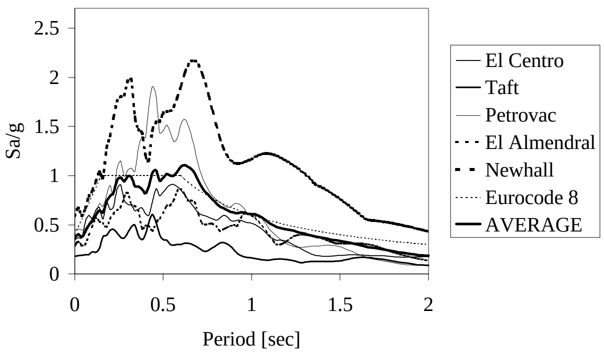


Figure 6. Elastic spectra of primary components.

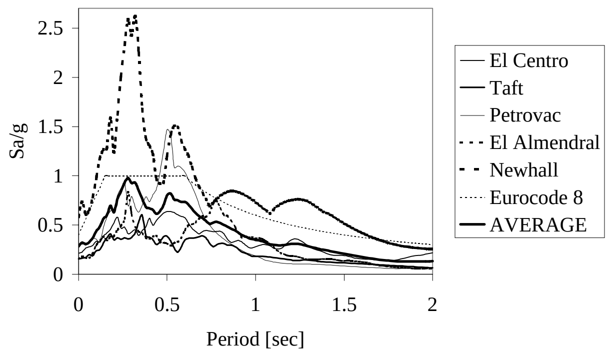


Figure 7. Elastic spectra of secondary components.

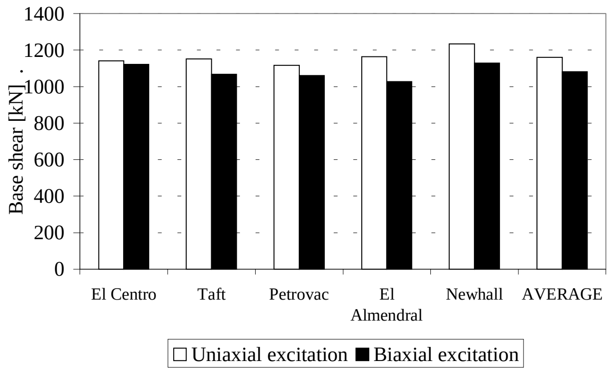


Figure 8. Maximum base shear.

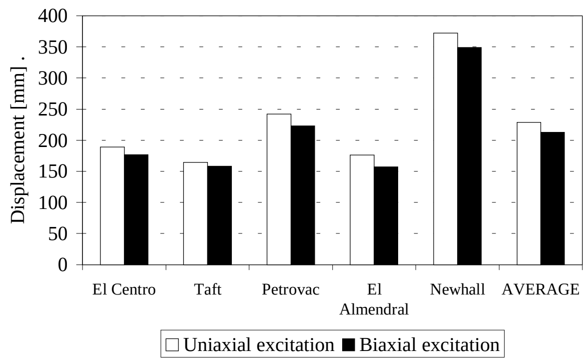


Figure 9. Maximum top displacement.

The main data of the real earthquakes used as input ground motion are reported in Table 1, where the component having the greatest peak ground acceleration is identified as “primary component”. The selected records represent an ensemble since the mean of the primary component elastic spectra, at 5% damping, matches well the Eurocode 8 elastic spectrum, computed assuming a PGA equal to 0.4 g and soil type B [Fig. 6]. Reference to a peak ground acceleration grater than 0.3 g (used in the design of the tested building) was done either because it is closer to the value used in the modelling phase (0.45 g = 0.3 g * 1.5) either to analyse a significant nonlinear behaviour of the building. Moreover, the usage of five records gives also the information on the influence of different earthquakes on the problem under examination. In Figure 7 the elastic spectra of the secondary components of the chosen five earthquakes and their average, compared to the above described Eurocode 8 elastic spectrum, are shown.

To evaluate the influence of the secondary horizontal component, the seismic response of the symmetric building under both the components of each earthquake is compared to its response under the primary component of the same earthquake acting in the X direction; this comparison is performed in terms of global and local seismic response parameters.

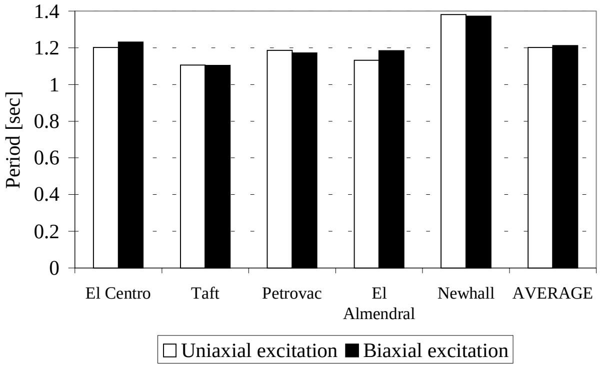


Figure 10. Mean of the instantaneous periods.

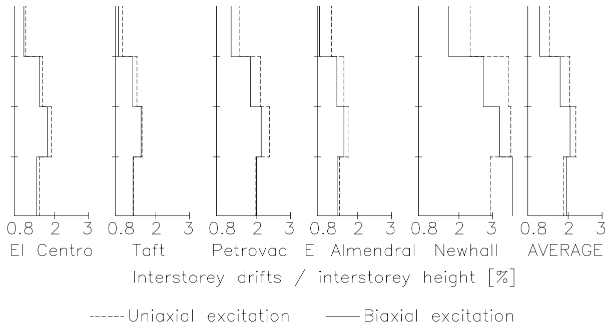


Figure 11. Maximum interstorey drifts.

For a global point of view the maximum top floor displacement of the mass centre, the interstory drifts, the maximum base shear and the average of the instantaneous periods of the symmetric building are compared in X direction. The maximum base shear under bi-directional excitation is lower than the one under uni-directional excitation for every earthquake, as it is shown in Figure 8. Lower internal forces correspond to lower floor displacements as it is confirmed in Figure 9 for the maximum top displacements of the mass centre. As predictable, base shear and floor restoring forces are not always the right parameters to catch the damage of a structure under seismic action. A well known global damage index is based on the variation of the instantaneous period of the structure: it globally describes the decreasing of the stiffness of the resisting elements. In Figure 10, the arithmetic means of the instantaneous first period (in X direction), computed over the total duration of the earthquakes, are compared. The last two histograms in Figure 10 represent the average over the five earthquakes and show a slight increment (just of 1%) for bi-directional excitation. These apparently contradictory results are confirmed by the maximum interstory drifts plotted in Figure 11. The mean value over the five earthquakes for bi-directional excitation is lower in the 2nd, 3rd and 4th level, but greater in the 1st level; this is due to the local damage of the elements, which is so much stronger in the 1st level that can more than compensate the observed difference in terms of floor restoring forces. These results prove that, even though the base shear and the top displacement are lower, the action of the secondary component can increase the damage. However, it has to be noted that the above results are relevant to the response of the structure in the X direction.

To confirm this conclusion local investigations must be carried out. In Figure 12 the mean of the maximum rotations of all the multi-spring elements at the ends of all the columns of an external and of the internal frame in the X direction are plotted. For each earthquake, the difference between the nonlinear maximum rotations under bi-directional and unidirectional excitation is related both to the intensity of the secondary component and to the ratio between the two components: as an example, the horizontal components of the Newhall earthquake, which have large and very close pick ground accelerations, lead to nonlinear rotations 20% grater than the ones under uni-directional excitation. The importance of the element local damage in the issue under examination is clearly marked in Figure 13, where the means (over the five earthquakes) of the maximum rotations of all the multi-spring elements at the columns ends are plotted for the four storeys. In the case of bi-directional excitation the rotations at the 1st floor, where the damage of the columns is maximum, are about 13% grater than those obtained under unidirectional excitation; this increment becomes 4% and 2% at the 2nd and the 3rd storey respectively, while a decrement of about 13% is observed at the 4th storey, where the damage is very low and the floor internal force effect prevails. Globally under bidirectional excitation an increment of about 6% in terms of nonlinear maximum rotations has been evaluated. The increase in damage produced by the action of the secondary component is clearly evident if the damage is assessed at the fibre level of the column section (Figs. 14 and 15). In particular, the mean of the ratios between the maximum elongation and the yielding elongation of the most damaged reinforced bar of the sections at column ends of an external and the internal frame in the X direction is shown in Figure 14 for all storeys of the building. The increment in damage due to the presence of both the components varies from the 53% at the 1st storey to the 11% at the 4th storey, with an average value of 43%. In Figure 15 it is reported the number of cracked, yielded and crushed sections among the 240 examined total sections (48 sections by 5 earthquakes); as above, the analysed sections are at column ends of an external frame and of the internal frame in X direction. Obviously almost all sections are cracked for both the kinds of considered seismic actions. The yielding for bi-directional excitation occurs in 117 sections, with an increment equal to 56% compared to the response under unidirectional excitation; moreover, in the former case the concrete achieves the maximum compressive strength in a number of sections almost 5 times greater than in the latter case.

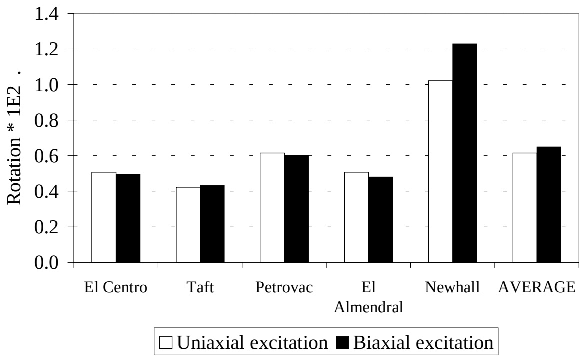


Figure 12. Maximum rotations at columns ends.

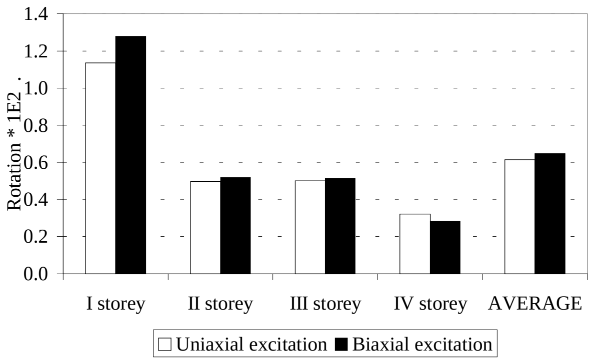


Figure 13. Maximum rotations at columns ends.

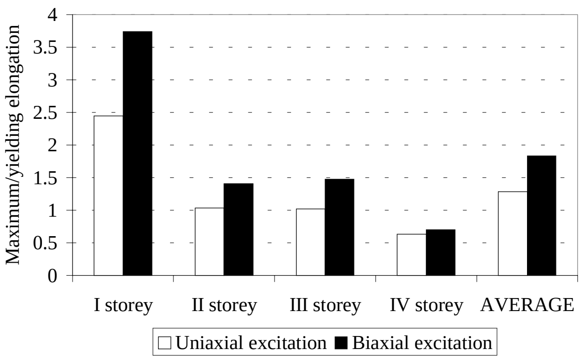


Figure 14. Maximum/yielding reinforcement elongation ratio.

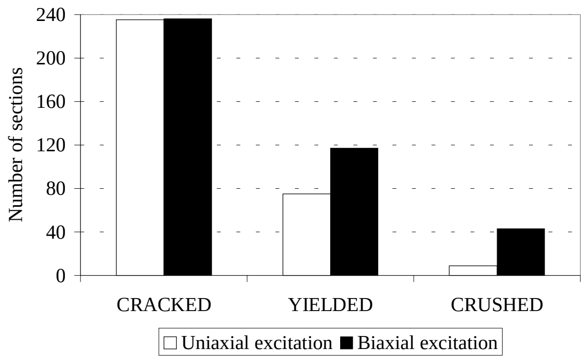


Figure 15. Damage of the columns sections.

CONCLUSIONS

The results obtained in the first part of the paper show that a suitable modelling of a bare frame reinforced concrete building to analyse the nonlinear seismic response can be achieved idealising the beams by uniaxial bending models and the columns by multi-spring models. With reference to the analysed building, indications are provided about the corrections to apply to the experimental stress-strain relations of the materials when computing the skeleton curves of the moment-rotation relationships assigned at the beam ends and those of the force-displacement relationships assigned at the springs of the multi-spring elements; indications are also provided on how the inelastic parameters of the hysteresis loops can be settled. Following such indications a very satisfactory match between numerical and experimental values in terms of displacements and base shear time histories are obtained. Such modelling is used to analyse the effect of the secondary horizontal component of the input ground motion on the inelastic behaviour of the building.

The comparison between the response under bi-directional and unidirectional ground motion shows no significant difference in terms of global response parameters (base shear, top displacement and instantaneous periods). However, the damage suffered by the structure under the bi-directional seismic action is grater than damage produced by the uni-directional one. The performed analyses clearly evidence that such increment in damage cannot be assessed through the variation of global response parameters. Neither the top displacement, nor the base shear are the right parameters; not too much can be also gained by the variation of the instantaneous period of the structure; since this damage parameter strongly takes into account the behaviour of the beams, which are "unidirectional resisting elements". More information can be obtained from the interstorey drifts, which clearly evidence the significant and larger damage produced by the simultaneous action of both the horizontal components at the lower storeys of buildings. Moreover, the values computed for the analysed building seem to evidence that the biaxial excitation is more damaging than the uniaxial one as the earthquake is more violent. In the examined case, the results at the member cross section show that, under bi-directional excitation, the nonlinear rotations in the direction of the primary component action increase of about 10% compared to the unidirectional excitation. The increment in terms of yielded and crushed sections is about 60% and 400% respectively. Based on the results of the paper it seems that the provisions of seismic codes which require to adopt two orthogonal components acting simultaneously are adequate to cover the increment in damage produced by the secondary component. However, the necessity to consider both the horizontal components in the seismic design of r/c buildings and then the adequacy of the seismic codes require analyses relative to different buildings and above all the development of a reliable biaxial damage index.

ACKNOWLEDGEMENTS

The authors would like to express their acknowledgement to dr. Paolo Negro for providing the experimental results. This research has been partially supported by the "Programma di scambi internazionali tra l'Università degli Studi di Napoli Federico II ed Università ed Istituti di ricerca stranieri per la mobilità di breve durata di docenti e ricercatori" of University of Naples Federico II.

REFERENCES

CEN 1994a. Eurocode 8. Design provisions for earthquake resistance of structures. Seismic Actions and General Requirements of Structures.

CEN 1994b. Eurocode 8. Design provisions for earthquake resistance of structures. General rules for buildings.

CEN 1994c. Eurocode 8. Design provisions for earthquake resistance of structures. Specific Rules for Various Materials and Elements.

De Stefano, M. & Faella, G. 1996. "An evaluation of the inelastic response of systems under biaxial seismic excitations". *Engineering Structures*. Vol. 18, No. 9, pp. 724-731.

Kunnath, S.K. & Reinhorn, A.M. 1990. "Model for inelastic biaxial bending interaction of reinforced concrete beam-columns". *ACI Structural Journal*. Vol. 87, No. 3, pp. 284-291.

Li, K.N. 1996a. CANNY-E. Three-dimensional nonlinear dynamic structural analysis computer program package. Canny Consultants Pte Ltd. Technical manual.

Li, K.N. 1996b. CANNY-E. Three-dimensional nonlinear dynamic structural analysis computer program package. Canny Consultants Pte Ltd. Users' manual.

Mander, J.B., Priestley, J.N. & Park, R. 1988. "Theoretical Stress-Strain Model for Confined Concrete". *Journal of Structural Engineering*. Vol. 114, No. 8, pp. 1804-1825.

Negro, P., Verzelletti, G., Magonette, G.E. & Pinto, A.V. 1994. Tests on a Four-Storey Full-Scale R/C Frame Designed According to Eurocodes 8 and 2: Preliminary Report.

Paulay, T. & Priestley, M.J.N. 1992. *Seismic design of reinforced concrete and masonry buildings*. A Wiley Interscience Publication, John Wiley & Sons Inc.

Powell, G.H. & Campbell, S. 1994. DRAIN-3DX element description and user guide for element type 01, type 04, type 05, type 08, type 09, type 15, and type 17. Version 1.10. Department of Civil Engineering, University of California, Berkeley, California.

Prakash, V., Powell, G.H., Campbell, S. 1993. DRAIN-3DX base program user guide. Version 1.10.

Zeris, C. A. 1990. "An investigation of the nonlinear biaxial response of simple structural systems", Fourth U. S. National Conf. on Earthquake Engng, Palm Spring, CA, 20-24 May 1990, Vol. 2, pp. 985-994.

Zeris, C.A. & Mahin, S.A. 1991. "Behavior of reinforced concrete structures subjected to biaxial excitation". *Journal of Structural Engineering*. Vol. 117, No. 9, pp. 2657-2673.