The 14th World Conference on Earthquake Engineering
October 12-17, 2008, Beijing, China

NON-LINEAR METHODS FOR SEISMIC ASSESSMENT OF EXISTING STRUCTURES: A COMPARATIVE STUDY ON ITALIAN RC BUILDINGS

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ABSTRACT :
The present paper reports on an Italian collaborative research study on the applicability of nonlinear methods of analysis to regular and irregular existing RC buildings designed and constructed according to older building codes. The mentioned methods are applied to two case studies of practical interest and the results are compared in order to identify general trends and major unresolved issues with reference to the key properties of the considered structures. Non-linear time history analyses are carried out to obtain reference values for the expected response parameters, characterized in terms of median value and dispersion, to be compared with the results obtained through nonlinear pushover analyses.

KEYWORDS:
Nonlinear Methods of Analysis, Pushover Analysis, Nonlinear Dynamic Analyses, Existing Buildings, Seismic Vulnerability, Seismic Assessemnt

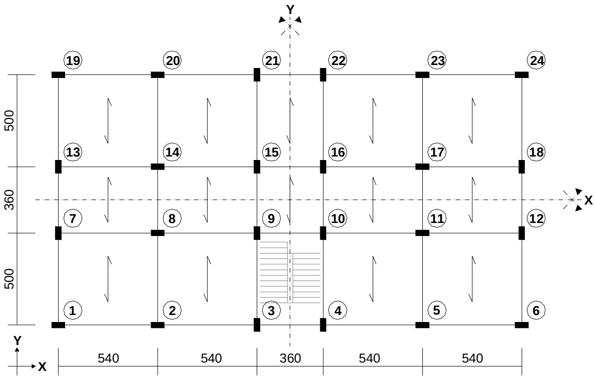
1. INTRODUCTION

Following the evolution of the earthquake engineering research in the past thirty years and the aging of the existing structures and infrastructures in many countries, a need has emerged to evaluate the seismic vulnerability of existing buildings. Many of the existing RC structures were built without accounting for seismic actions, thus much attention has been paid in recent years in developing reliable methods of analysis and assessment for these structures. Linear methods seem inappropriate in most cases; consequently, many current seismic codes and guidelines include provisions for nonlinear analyses (Eurocode 8, 2003a, EuroCode 8, 2003b, FEMA 356, 2000, ATC-40, 1996), which seem to be the natural choice for existing structures subjected to moderate and strong design earthquakes. Chapter 11 of the new Italian Seismic Design Guidelines (OPCM3431, 2005) is totally dedicated to existing structures. This is obviously a big issue in Italy, a seismically active country where many buildings of the '60s, '70s and '80s were often designed accounting for gravitational actions only. Following the publication of the new Italian Seismic Code, the Italian Department of Civil Protection (DPC) has launched a research program (ReLUIS) whose aim is to validate and improve the new code, to propose alternate procedures when deemed necessary, and to provide practical examples to practicing engineers. These activities are particularly important for testing code-specified nonlinear methods of analysis. Focus of these studies is not only the application of the nonlinear methods of analysis, but also the use of the results of the nonlinear analyses to assess the structures' seismic vulnerability. This paper presents the results of the ongoing research project on irregular RC buildings.

2. DESCRIPTION OF THE TWO CASE STUDIES

Two case studies, representative of wide classes of RC buildings, have been generated and considered for the application of different numerical models and analysis methods for quantifying their seismic response. The first one is a four-storey regular building whose plan view is represented in Figure 1. The plan shows its double symmetry with respect to two orthogonal axes. The floor is realized by means of a one-way ribbed slab

supported by deep beams parallel to the X-axis. All beams and columns have 30x60 cm^2 sections and further details about steel reinforcement are omitted herein for the sake of brevity.
A five-storey L-shaped building, whose plan view is shown in Figure 2, has been considered as a second case-study with the aim of investigating the various aspects of the seismic response of plan-irregular RC frames. Asymmetry in shape is only one of the aspects contributing to plan-irregularity of the structure. Indeed, various beam spans and column cross sections have been considered as usually occur in buildings designed for gravity loads only. In this case, the one-way ribbed slab of the horizontal floors runs in two different directions in the two wings of the L-shape. Deep beams, with 30x60 cm^2 section, are usually placed in the orthogonal direction with the aim of supporting the floor, while shallow beams (transverse section 25x50 cm^2) run parallel to the slabs. Thick lines in Figure 2 indicate deep beams.



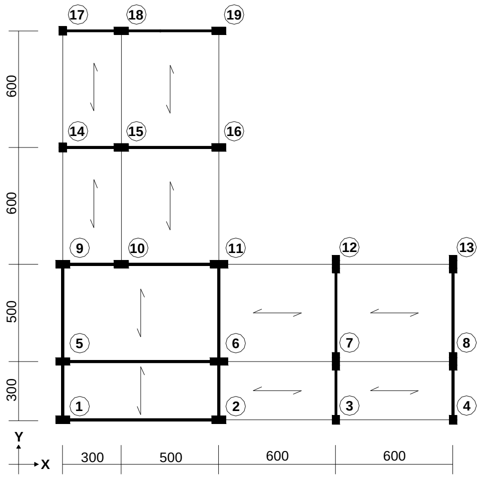


Figure 1: Case-Study1 –Rectangular building Plan Figure 2: Case-Study2 – L-Shaped Building: Plan

As for the loads, Dead Load G_k = 6 kN/m^2, Live Load Q_k= 2 kN/m^2. The additional self-weight of the 30 cm-thick masonry infill exterior walls was assumed to be 8.00 kN/m^3. The main material characteristics are the compressive strength R_{ck} = 27 MPa for the concrete and the yield stress f_{sk} = 440 MPa for the reinforcing steel.

3. RESULTS

Non-linear static analyses were carried out according to the N2-method (Fajfar, 2000, and more recent updates) using the Elastic Demand Spectra proposed by the European Code 0, while non-linear time-history analyses were performed using input ground motions selected from the European strong motion database in order to obtain spectrum-compatible inputs, as specified by OPCM3431 (2005) for Zone 1 and Class B soil. SAP2000 (CSI, 2005) was the basic package used for the analyses. SAP2000 uses a lumped plasticity model which possibly account for N-M interaction in columns under uniaxial and biaxial bending. Rotational capacity of the plastic-hinges is defined in terms of plastic rotations which can also be defined for different values of the axial stress N with the aim of accounting for the possible interaction between ultimate (and yielding) rotation and axial force N. Shear plastic-hinges can also be defined by specifying a threshold value for the shear force in the members. More advanced software packages, such as OpenSEES (Fenves et Al., 2004) , CANNY99 (Li, 1996) and MIDAS (Midas, 2005), were also used in the project.

3.1. Case study #1

First, a direct comparison in terms of capacity curves obtained through pushover analyses by means of two different numerical codes, both based on lumped-plasticity models, is represented in Figure 3. The same differences can be observed with reference to both X- and Y-direction; more specifically, the ultimate values of the base shear obtained from SAP2000 and MIDAS are quite close. The slight difference basically derives from the stability of the convergence algorithms which can severely affect the value of the displacement capacity. In all cases, Young Modulus has been suitably reduced to represent cracked conditions (50% reduction has been assumed for both beam and column stiffness, as indicated in OPCM3431, 2005).

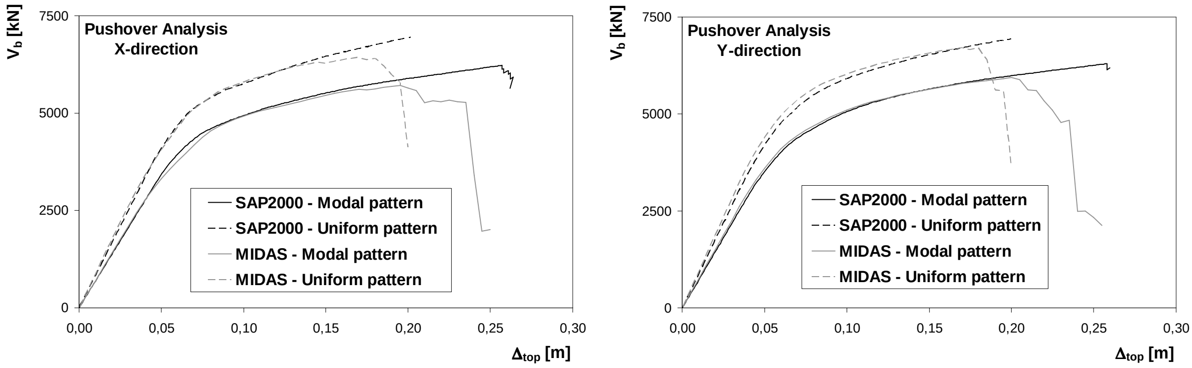
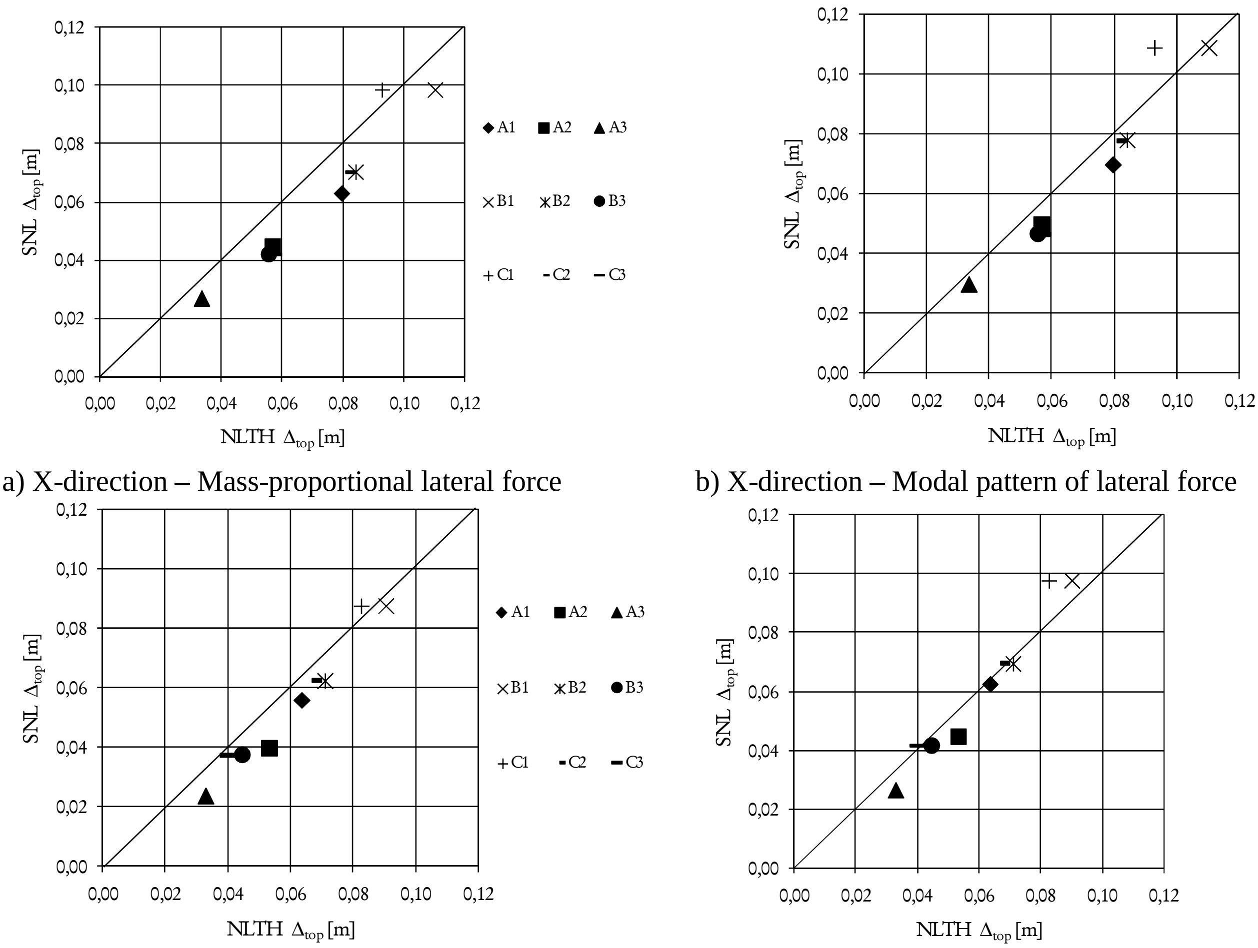


Figure 3: Case-Study #1 – Comparison between two Numerical Codes (MIDAS vs SAP2000).

Comparing the results of static non-linear (SNL) and non-linear time-history analyses (NLTH)is a key step for assessing the reliability of the static analysis. Nonlinear dynamic analyses of the building of case study 1 were carried out on a fiber-based numerical model developed in OpenSEES (Fenves et Al., 2004). Nine sets of seven couples of natural accelerograms proposed by Iervolino et al. (2006) were considered. Each set of seven couples is deemed to be equivalent to a Seismic Zone (ranging from 1 to 3 as seismic hazard decreases) and a Soil Type (ranging from A to C, from the stiffer to the softer one) according to Eurocode 8. Figure 4 shows the results of these analyses in terms of the average values of the top displacement Δ_{top} evaluated by NLTH analyses considering for each Seismic Zone and Soil Type the seven seismic records mentioned above, while the corresponding design spectra have been considered for determining the same quantities through SNL.



a) X-direction – Mass-proportional lateral force
b) X-direction – Modal pattern of lateral force
c) Y-direction – Mass-proportional lateral force
d) Y-direction – Modal pattern of lateral force

Figure 4: Case-Study #1 – Comparison between Non-Linear Static and Dynamic Analyses.

Figure 4 points out that in both directions the results obtained by non-linear static analyses are reasonably close to those of the non-linear time history analyses. In both directions, static pushover analyses carried out considering the so-called modal pattern of horizontal forces results in closer agreement with the dynamic analyses because the building is regular and basically controlled by the first mode.

Following Eurocode 8 – part 3 (2005) the ratios ρ between the demand and the capacity in terms of plastic rotations (for ductile failure mechanisms) and in terms of shear (for brittle failure mechanisms) was also evaluated (Berto et al., 2008). Figure 5a shows the capacity curve for the uniform pattern distribution and the bilinear approximation, according to the pushover procedure described in the European Standard. As often observed, the governing failure mechanism is shear collapse, as evidenced by the shear ratios ρ (Demand/Capacity) in Figure 5b (ro_SH). Assuming that such a failure can be prevented, the analysis is continued until the occurrence of ductile mechanism in correspondence of the sharp decrease of the capacity curve, evidenced also by the ro_CR curve which exceeds the value 1 (Figure 5b).

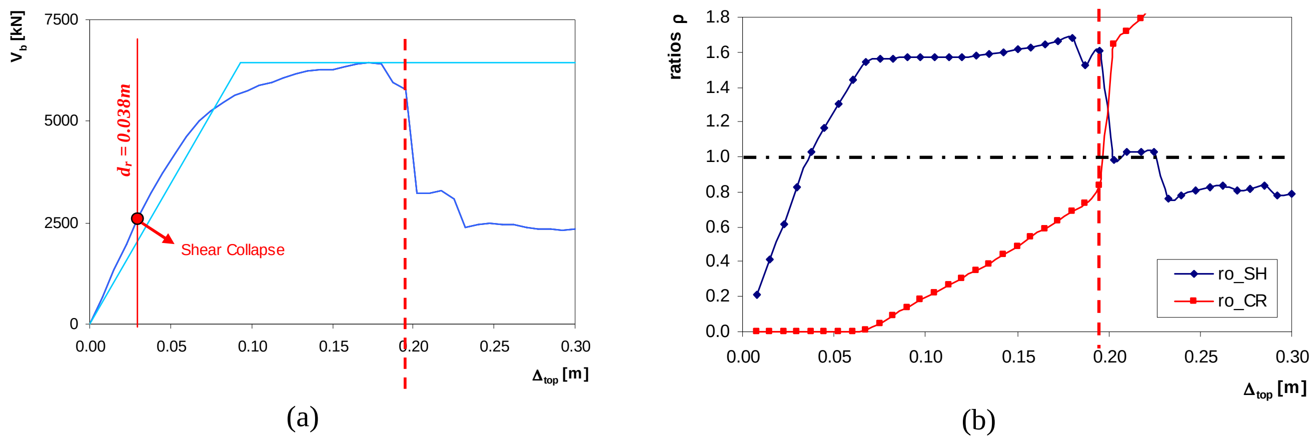


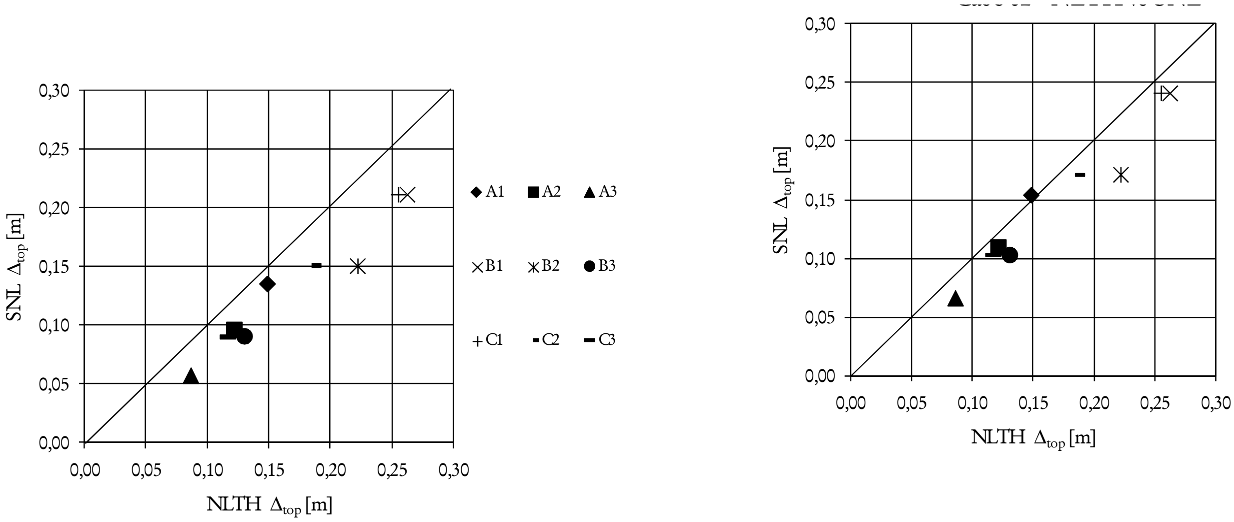
Figure 5: Case-Study #1 –Capacity curve for the uniform pattern distribution and ratios ρ vs displacement.

3.2. Case study #2

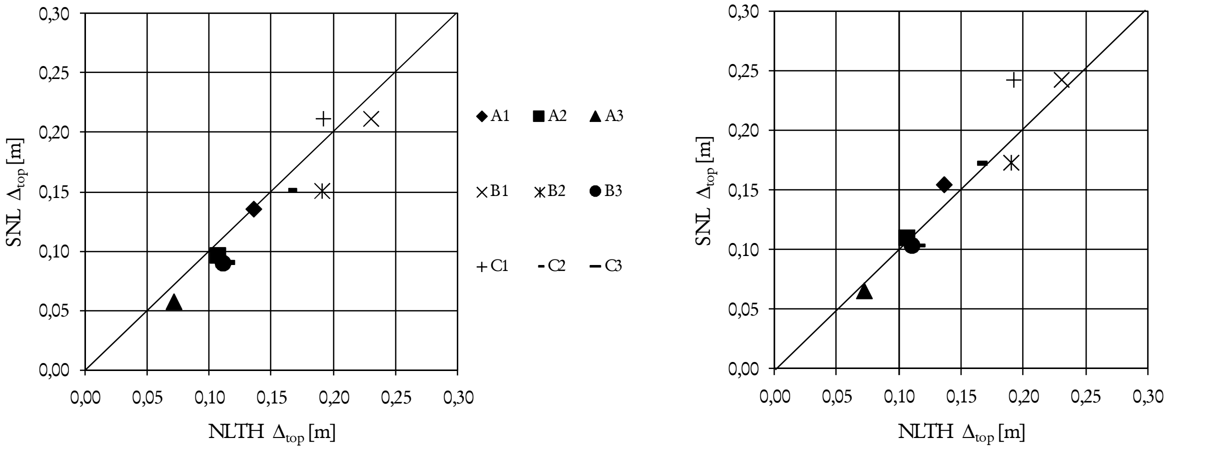
The comparison between NLS and NLTH of Figure 4 were repeated for the irregular building of case # 2, using again OpenSEES and a fiber-based distributed plasticity model. Figure 6 shows the results. Although a good correlation can be observed in both directions, static analyses lead to more accurate results in the Y-direction as the points in Figure 6c and Figure 6d lie on the equivalence line. On the contrary, the static analyses performed in X-direction often result in an un-conservative estimation of the average value of the top displacement. A possible reason for this (though small) discrepancy can be found by considering that mass and stiffness centroids are basically aligned in the Y-direction, while a larger eccentricity exists in the X-direction. Consequently, higher modes participation in the X-direction is more relevant than in the Y-direction, thus the modal pushover analyses yield better predictions for NLS analyses in the Y-direction.

In order to improve the results obtained with the SNL pushover based on the N2-,method, Multimodal Pushover Analyses, as proposed by Chopra ad Goel (2004), were performed for the L-shaped building. Figure 7 shows that for this specific application there is no major improvement with respect to Figure 6.

For plan-asymmetric structures it is insightful to obtain limit domains, in terms of displacement or base shear. These domains are obtained from the results of NLS pushover analyses carried out by varying the in-plan direction of the load distribution [Petti et al. 2007]. Figure 8 and Figure 9 show the results of NLS analyses using different load shapes. The results are shown in terms of elastic limit and collapse displacements. These results were obtained using program SAP2000. Figure 10 and Figure 11 show the results in terms of demand ductility and demand rotation. The results show a general high complexity in the plan behaviour. Overall, the directions in the plan near to 90° and 270° lead to lower force and deformation demands while the 45° and 210° directions to the higher torsional demands. These results confirm previous observations indicating that applying the loads along the building reference axes may lead to unconservative demands.



a) X-direction – Mass-proportional lateral force b) X-direction – Modal pattern of lateral force



c) Y-direction – Mass-proportional lateral force d) Y-direction – Modal pattern of lateral force

Figure 6: Case-Study #2 – Comparison between Non-Linear Static and Dynamic Analyses.

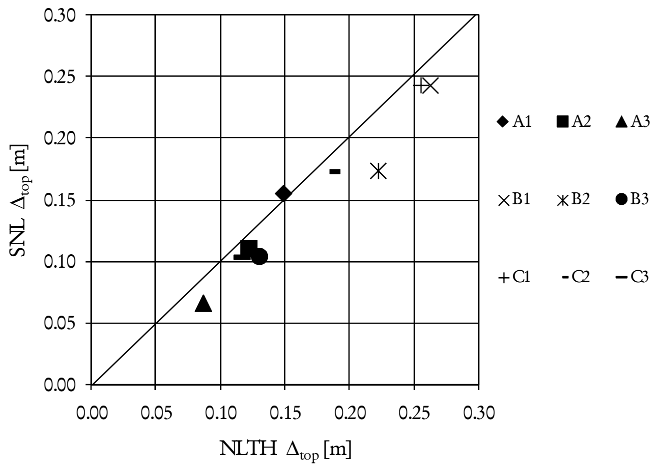


Figure 7: Case-Study #2 – Multi-Modal Pushover Analysis (MPA) in X-direction.

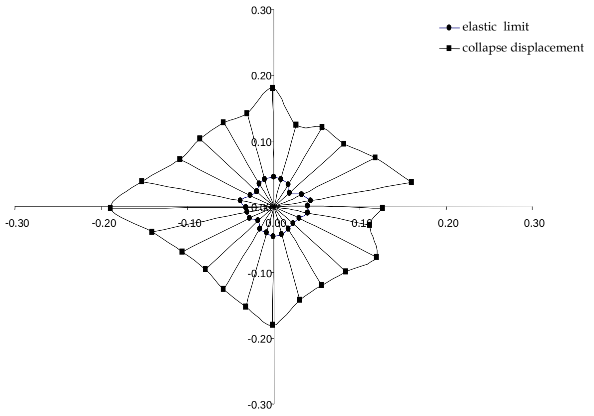


Figure 8: Case-Study #2 – Limit domains in terms of collapse and elastic displacement [m] – Uniform load distribution

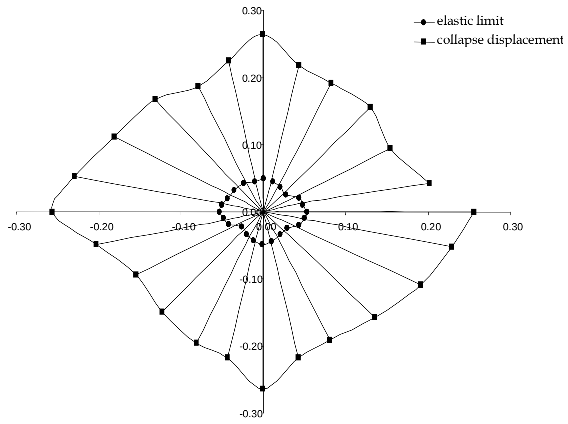


Figure 9: Case-Study #2 – Limit domains in terms of collapse and elastic displacement [m] – Triangular load distribution

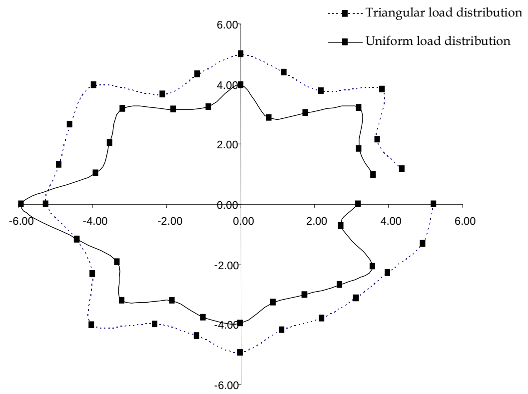


Figure 10: Case-Study #2
Collapse domains in terms of demanded ductility

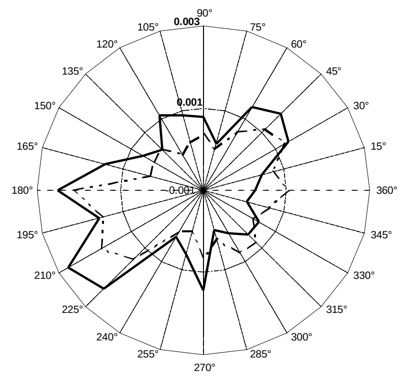


Figure 11: Case-Study #2 – Rotation of centre of mass [rad] in the case of uniform [dashed line] and triangular [continues line] load distributions

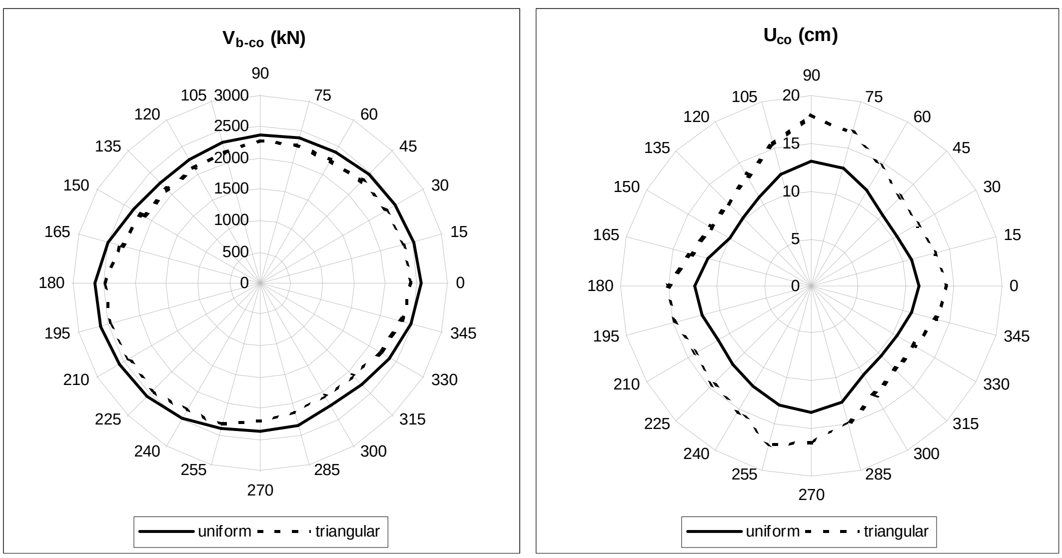


Figure 12 Case-Study #2: Limit domains in terms of base shear (left) and control node displacement (right)

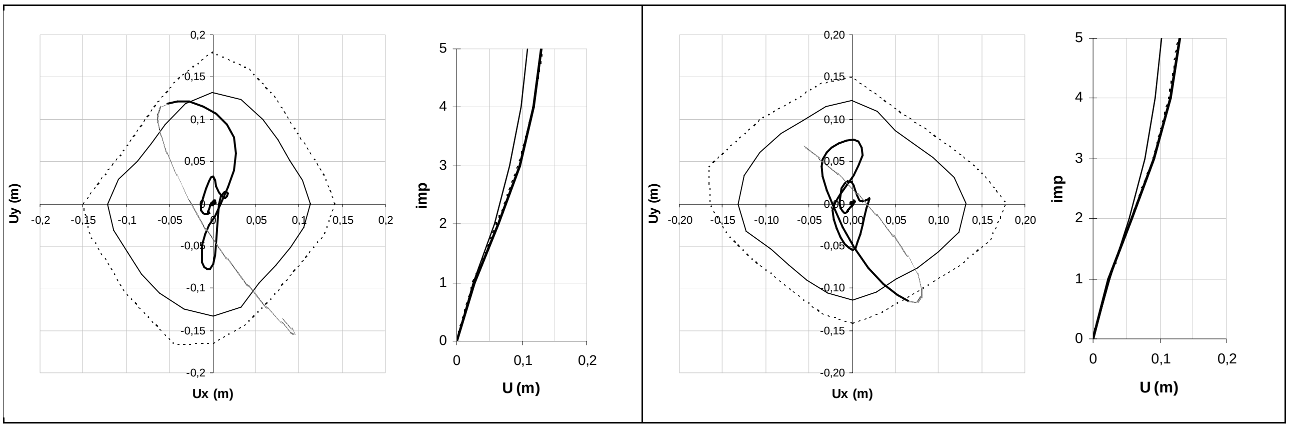


Figure 13 Case-Study #2*:* Comparison between NDP and NSP in terms of control node displacement and deformed shape for the Montenegro event (1979) in the case of: (Left) in-plan orientation *0°* - NDP collapse amplification factor 0.35; (right) in-plan orientation *90°* - NDP collapse amplification factor 0.30.

Similar conclusions were reached using a different program, as shown in Figure 12, which shows collapse domains in terms of base shear and displacement of the control node (top floor centre of mass). It is interesting to note that different analysts using different programs obtain different results (Figure 8, Figure 9 and Figure 12-right). In order to evaluate the applicability of the limit domains to accurately describe the collapse state of the structure, incremental dynamic analyses up to collapse were also performed. Figure 13 shows the results for the Montenegro seismic event (1979). In both cases the collapse is attained in accordance with the limit domain obtained with the triangular force distribution. A good match can be observed between the dynamic storey displacements and the deformed shape associated with the static procedure for the triangular force distribution.

4. OPEN SISSUES

The above discussion refers to an ongoing project. The following unresolved issues have emerged so far:

1) According to EC8, the design verifications are based on the chord-rotation. However, this check requires a heavy and lengthy post-processing. Furthermore, chord rotation capacity formulas are obtained from simple tests that do not necessarily reproduce the complex load history on the structure. The plastic hinge rotation demand would seem an easier check, as done in FEMA 356 (2000);
2) The definition of a given limit state is not clear. If only a single section in a very complex building has reached a given limit state (let's say the ultimate limit state) it would be difficult to argue that the entire structure has reached such a state. On the other hand, if a single base column has reached shear failure, the building has definitely reached its ULS. In other words, how is the design check of the different sections related to the overall check of the entire structure?
3) Modeling the stiffness of the floor diaphragms is an open issue. Older structures have fewer beams, thus the assumption of floor diaphragms rigid in their plane may be too crude an assumption. However, the computational cost of modeling the floor slabs needs to be further investigated.
4) Time-history analyses on building n.2 show that a critical angle of seismic incidence, i.e. the angle of application of the records providing the maximum structural response, exists and the increment of response parameters is not negligible; consequently, such critical angle should be taken into account in the seismic structural analyses.

5. CONCLUSIONS

A quick review of the results of an ongoing research is reported in the present paper with the key purpose of pointing out both comparisons among well-established methods of analysis for the seismic response prediction of structures and open issues about their modeling.
Comparisons have dealt primarily with different ways of modeling structures taking into account the mechanical and geometric non-linearities in different ways. The result of such comparisons are usually encouraging when

different lumped plasticity models where compared, while further work is needed for reducing the differences in results obtained with models that use lumped and distributed plasticity. Static versus dynamic analyses have been also compared. Although significant differences have been found in terms of the demand-to-capacity ratios derived by means of the two aforementioned methods, the parametric studies carried out point out that in most cases that less accurate methods of analyses lead to more conservative demands on the structures.
The analyses carried out so far also indicate that the influence of member cracking on the assessment result is quite low, while another parameter, the footing flexibility, often neglected in the practitioners' analyses, seems to play a relevant role in predicting the buildings' seismic performances. The study also indicates that the nonlinear analyses should be carried out for different angles of applied forces (both for nonlinear static and dynamic analyses).
Finally, the study this paper stems from opens more questions than answers; this means that a lot of work has to be still done in computational mechanics, especially when seismic performance of existing structures is of concerns, before nonlinear methods of analyses can be safely used by the engineering community at large.

ACKNOWLEDGEMENTS

The authors wish to acknowledge the Italian Department of Civil Protection (DPC) and the ReLUIS Consortium that granted the present research as part of the DPC/ReLUIS Research Project 2005-2008–Line 2–Task 3 (seismic behavior of RC irregular buildings).

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