Assessment of Seismic Response of Plan-Asymmetric Structures through simplified Static Non-linear Analyses

Petti, Prof. L.; Marino, Ph.D. I.; Cuoco, Dr. L.; Palazzo, Prof. B.

ABSTRACT

In the last few years, the need to evaluate the seismic performances of buildings while sustaining strong motion has encouraged the development of simplified non-linear static procedures. Several procedures are available today to assess the behaviour of plane-frame systems or plan-regular framed buildings suitably for engineering purposes. Less accurate procedures are instead available for non-regular plan structures. This study introduces the concept of resistance and displacement domains in plan and shows how these can be useful in evaluating the direction of minor seismic resistance in the case of asymmetrical plan building. Results have been preliminary tested by non-linear time history analyses.

KEYWORDS

Non-linear Analysis, Asymmetric Plan Building, Latero-Torsional behaviour

INTRODUCTION

Traditional seismic design considers Linear Dynamic Procedure (LDP) with assigned response spectrum (Response Spectrum Analysis) as the standard analysis. More complex procedures, such as non-linear time history analysis (NDP), are seldom used because they need the definition of an accurate hysteretic model to describe the behaviour of the materials under cyclic actions and the choice of a set of accelerograms that describes the real site conditions. Moreover, as is well- known, NDP analyses present convergence difficulties and require major computational effort.

The need to evaluate the seismic performances of buildings while sustaining strong motion has fostered the development of simplified non-linear static procedures (NSP). These analyses, which provide an evaluation of deformation capacities in the post-elastic range, allow us to relate hazard levels to those performance targets described in modern seismic codes (Performance Based Design).

Non-linear static analysis is also becoming the key method to evaluate the seismic response of existing structures. This method, which is able to evaluate the collapse mechanism of structures, is suitable for the analysis of structures not explicitly designed for seismic action.

However, non-linear static procedures can lead to unsuitable results when applied to irregular structures because of the difficulties in taking into account dynamic latero-torsional effects. As is already known, an asymmetric distribution of masses and stiffness, or of strength in plan, leads to high ductility demand for the elements near the soft or weak edge (14, 15). In the last few years several proposals have been put forward to extend traditional pushover analysis, calibrated on plane systems, to the assessment of three-dimensional models (1, 2, 3, 9, 13, 22, and 23). At present no simplified procedure, which can adequately account for the torsional behaviour of asymmetric structures is available,. Some of the main problems concern the combination of the forces in the two directions, and the best way in which to consider extra ductility demand for the elements near the soft edge.

Among the early proposals we mention one (17), which is based on the study of the non-linear static behaviour of the critical frames only, identified by means of LDP analyses performed on three-dimensional models. Afterwards, Fajfar extended the N2 method to three-dimensional structures (9, 10). Further research has focused on the behaviour of framed structures with shear walls (4) and on the accuracy of results on varying the loads plan distribution (5). Finally, Chopra has presented an extension of the MPA (Modal Pushover Analysis) procedure to asymmetric-plan structures. A wider, performance-based analysis of the behaviour of asymmetric structures can be found in (30), with particular attention being paid to the torsional influence on the permanent drift investigated by uncoupling the translational and the torsional effect. Available comparisons between these methods and the results of non-linear dynamic analysis (NDP) generally show limited success of the proposed procedures.

In this framework, this study will present and discuss the initial results of a simplified force-based static non-linear analysis to evaluate the latero-torsional response of plan-asymmetric structures. In particular, the latero-torsional response of a benchmark structure has been investigated by using a 3D model through pushover procedures and non-linear time history analyses.

In particular, this study presents limit domain in terms of strength and displacement capacities evaluated by means of pushover analyses by applying force distributions, triangular and uniform, at different angles in the plane to search for the least seismic-resistant direction. The results have been compared to preliminary non-linear dynamic responses of the benchmark structure to recorded earthquake accelerograms.

DESCRIPTION OF THE STRUCTURE

The benchmark structure considered in this study is one of the case studies proposed in the ReLUIS project (Rete dei Laboratori Universitari di Ingegneria Sismica – Earthquake Engineering Test Labs Network) (25).

The structure, designed without seismic actions, is a five-.storey, L-shaped building with RC **moment** frames in two orthogonal directions (figure 1). The floors are realized by means of one-way ribbed concrete slabs supported by deep beams (400x500 and 400x600 mm). Moreover, shallow beams (500x250 mm) run parallel to the slab and complete the floor structure. Three RC sections are considered for the columns: 400x700, 400x400, 550x400 mm. The transverse reinforcement is made up by 8 mm stirrups spaced at 100 mm for columns and 50 mm for beams. Material properties are characterized by the mean values: 25 MPa is the compressive strength for concrete and 400 MPa the yielding stress for both transverse and longitudinal reinforcement. Elastic modulus are 210000 MPa for steel and 28960 MPa for unconfined concrete.

MODEL PROPERTIES

The model of the benchmark structure has been implemented with OpenSees (v 1.7.3), a finite element software developed at University of California, Berkley (28).
In particular, a finite element model with spread plasticity has been examined to perform non-linear static and dynamic analyses. Due to prevalent observed collapse mechanisms for the case under investigation, non-linear shear behaviour is not considered. The behaviour of sections has been modelled through the use of fiber integrations by strain and stress which has been updated, step by step.
Concrete has been described as non-linear behaviour using a no-tension four-parameter Kent-Park-Scott model (*Concrete01* – figure 2). The concrete strength mentioned above is for non-confined material, and is used only for the bound fibers. For the inner core, the behaviour of the concrete has been modified by means of the Richart model. A bilinear elasto-plastic model with hardening has been used to describe the behaviour of the reinforcement (*Steel01)*. The materials' numerical models have no deformation limit..
Floors are modelled by means of 50 mm (27) elastic shells whose modulus is set to the value of 30000 MPa.
Modelled elements such as these do not present the ramps and the well-defined state-change points like the plastic hinges proposed, for example, in FEMA 356, but have a smooth reduction in stiffness as deformation increases. Seismic codes usually propose hinge rotation limits, because lumped plasticity is the most common way to perform non-linear analyses. In order to define a limit state for this spread plasticity approach the following limit values for fiber strain have been considered:

- 0,006 (in compression) for concrete fibers
- 0,03 (in tension) for steel fibers

Positive and negative curvatures have then been evaluated for beams and columns by considering positive and negative flexural actions.
In the case of the columns, the linear interpolation in the plane of the section has been used to evaluate limit curvature (rhomboidal domain) as shown in figure 3. Axial forces have also been considered and a linear interpolation is used between the domains referring to two axial force sample values.
In addition to this approach (*Fiber*), the limit hinge rotations proposed in the Italian Seismic code (26) have been considered too (*OPCM*); the code provides several formulas for collapse hinge rotation and plastic hinge length:

[1] $$\varphi_u = \frac{1}{\gamma_{el}} 0{,}016 \cdot (0{,}3)^{\nu} \left[\frac{\max(0{,}01;\omega')}{\max(0{,}01;\omega)} f_c \right]^{0{,}225} \left(\frac{L_v}{h} \right)^{0{,}35} 25^{\left(\alpha \cdot \rho_{sx} \frac{f_{yw}}{f_c}\right)} (1{,}25^{100\rho_d})$$

[2] $$L_{pl} = 0{,}1 L_v + 0{,}17 h + 0{,}24 \frac{d_{bL} \cdot f_y}{\sqrt{f_c}}$$

[3] $$\varphi_u = \frac{1}{\gamma_{el}} \left(\varphi_y + (\theta_u - \theta_y) L_{pl} \left(1 - \frac{0{,}5 L_{pl}}{L_v} \right) \right)$$

from these two values it is possible to go back to the section maximum curvature by considering a plasticity distribution similar to that adopted in [3]. Even for this approach, a linear interpolation in the plane of the section is used (rhomboidal domain); no interpolation is needed for axial force because the code's formula explicitly considers the axial action influence (ν in [1]).

ANALYSIS PROCEDURES

The analyses performed are modified force-based non-linear pushover. In particular, a force multiplier is changed (increased or decreased) to reach a target displacement, so that the analysis can also describe softening behaviour in the structure.
Two force distributions are used along the height of the structure. The first one is a triangular distribution, taking into consideration the inertial mass and the height above the ground. The second one considers only the floor masses. Forces are applied at beam-column joints, proportionally to the joint mass.
For each force distribution 24 pushover curves have been obtained by rotating the direction of action in the x-y plane, 15° at each step. The control node is assumed to be the centre of mass on the top floor in all pushover analyses.
Using the described limit values for section curvature, the state at which the first primary element goes beyond its limit is assumed as global collapse (U_{co}-Vb_{co}). Each pushover curve is bilinearized according to the FEMA 356.
Collapse states for each analysis in the plane led us to define limit domains in terms of collapse displacement and base shear. Each domain is plotted both for 'fiber' and 'OPCM' limit value criteria.
The structure's capacity described above is compared to the non-linear dynamic response to recorded earthquake activity, selected from among those proposed by ReLUIS. In particular, the selected accelerogram is taken from the European Strong Motion Database (25), and reports an event recorded in Iceland on 21/6/2000. The record, considering both NS and EW components, is scaled to study the structural response as ground acceleration varies (IDA).

DESCRIPTION OF RESULTS

Figures 4-7 show the results of the described pushover analyses. In particular, figures 4 and 6 refer to the collapse displacement of the control node on varying the direction of action in the plane in the case of triangular (TR) and uniform (UN) force distribution. Both cases show non-regular behaviour of the structure in terms of limit displacements. In particular, analyses showed that the benchmark structure presents lower displacement capacities in the 150° and 315° directions (highlighted by means of arrows in the figures). Otherwise, structural behaviour seems to be more regular regarding strength (figures 5 and 7). Finally, differences between 'fiber' and 'OPCM' collapse criteria result as unimportant; in particular, the 'fiber' approach seems to be a little more conservative than the 'OPCM' approach.
Figures 8-9 show a comparison between the pushover results for the two force distributions examined. Domains, similar in shape for both forces and displacements, show the same critical directions. However, the behaviour according to the uniform distribution presents minor collapse displacements and major strengths. Figure 10 shows the comparison in terms of control node rotation at collapse, identifying the directions of maximum and minimum torsional response of the building.

Dynamic non-linear behaviour has been evaluated by considering the structure oriented with its x-axis aligned both with the recorded EW (*analysis a*) and NS (*analysis b*) components. Figures 11-13 show the dynamic response for *analysis a*. The figures show that the collapse is attained in the direction of about 154° according to the minor displacement capacity direction found by means of the pushover analyses carried out. Moreover, the collapse displacement coincides with that of the pushover domain for triangular distribution, and the collapsed element, column 15 at ground level, is the same for both dynamic and static analysis. Figure 13 reports the evolution of torsional response showing that the control node rotation is roughly internal to the domain before collapse (highlighted with a +), but became unstable after it.
As a further comparison between static and dynamic responses, figure 14 shows storey displacements at collapse for both analyses in the collapse direction (150°) and in the main plan directions (0° and 90°). Analysis of results shows that the lesser seismic resistant directions found by means of pushover analysis well describe its true dynamic behaviour.
Moreover, results show that the collapse mechanism is a global one, because it is well described by the triangular force distribution. In particular, in figure 11, it can be seen that the collapse displacement is almost equal to the triangular pushover displacement (outer domain) and figure 12 shows that the collapse shear is that associated with triangular distribution too (inner domain). In figure 14 the deformation through the height of the structure is internal to the range defined by the two statically obtained deformations.
Figure 15 shows the comparison between the behaviour evaluated by means of pushover analyses and that obtained by incremental dynamic analysis. For this purpose, several proposals can be found in literature regarding the representation of the time history data (29). In the study several criteria have been considered: maximum displacement vs. maximum base shear (*max*), maximum displacement vs. corresponding base shear (*inst*) and maximum displacement vs. mean base shear within the interval +/- 0,25 s (*mean +/- 0,25s*). Moreover, a new criterion has been proposed: the maximum displacement vs. maximum base shear within the interval +/- ¼ T (*max +/-(1/4)T*) with T the dominant natural period of vibration in the direction under examination.
Figures 16-20 show the same data for *analysis b*. For this case the collapse has been attained in the direction of about 286° and generally similar conclusions to those of *analysis a* can be reached.
Preliminary results show that results obtained by static non-linear analysis carried out in the least seismic resistant direction will effectively describe the dynamic non-linear response (figures 14 and 19). Moreover, the proposed criteria for comparing dynamic and static analysis seems to supply us with the best results for all investigated displacements (figures 15 and 20).

CONCLUSIONS

In the field of seismic engineering, an open issue, which is currently of great importance, is the need to design a feasible procedure to evaluate the seismic *performance* of three-dimensional structures, because at present code regulations only refer to plane structures, and pushover analyses have shown that main directions in the plane might not be the key ones.
In this framework, the study introduces a new concept of limit plan domain to assess the non-linear behaviour of a latero-torsional plan building. In particular, such a limit domain, evaluated by means of pushover analyses by varying the force direction in plan, can lead to a search for less seismic resistant directions.

The preliminary results obtained by investigating the static and dynamic non-linear response of a benchmark structure show that the dynamic seismic behaviour can be effectively described by classical pushover analyses carried out in the lesser seismic resistant directions.
Moreover, the study proposes a new criterion to compare dynamic and static analysis which seems to supply the best results for all investigated cases.
The results obtained have encouraged us to extend the proposed analysis methodology to other cases and recorded excitations.

ACKNOWLEDGMENTS

The presented results are part of research carried out in the ReLUIS project by University of Salerno.

REFERENCES

1. Ayala A. G. and Tavera, E. A., "A new approach for the evaluation of the seismic performance of asymmetric buildings", *Proc. of the 7th National Conference on Earthquake Engineering, Boston*, (2002)
2. Aydinoglu M. N. [2003], "n incremental response spectrum analysis procedure based on inelastic spectral displacements for multi-mode seismic performance evaluation", *Bulletin of Earthquake Engineering 1(1)*, (2003), pp. 3-36.
3. Chopra A. K. and Goel, R. K., "A modal pushover analysis procedure to estimate seismic demands for unsymmetric-plan buildings", *Earthquake Engineering and Structural Dynamics 33(8)*, (2004), pp. 903-927.
4. De Stefano M. and Rutenberg A., "Prediciting the dynamic response of asymmetric multistory wall-frame structures by pushover analysis: two case studies", *Proc.,11th Eur. Conf. Earth. Engrg., A.A. Balkema, Rotterdam*, (1998).
5. Faella G. and Kilar V., "Asymmetric multistory R/C frame structures: push-over versus nonlinear dynamic analysis", *Proc., 11th Eur. Conf. Earth. Engrg., A.A. Balkema, Rotterdam*, (1998).
6. Fajfar P., "A nonlinear analysis method for performance-based seismic design", *Earthquake Spectra 16(3)*, (2000), pp. 573-592.
7. Fajfar, P., "Structural analysis in earthquake engineering - a breakthrough of simplified nonlinear methods", *Proc. of the 12th European Conference on Earthquake Engineering, London, UK, Keynote lecture*, (2002).
8. Fajfar P. and Gaspersic P. [1996], "The N2 method for the seismic damage analysis of RC buildings", *Earthquake Engineering and Structural Dynamics 25*, pp. 23-67.
9. Fajfar P. and Kilar V. [1997], "Simple push-over analysis of asymmetric buildings", *Earthquake Engineering and Structural Dynamics*, Vol. 26, pp. 233-249.
10. Fajfar P., Kilar V., Marusic D., Perus I. and Magliulo G. [2002], "The extension of the N2 method to asymmetric buildings", *Proc. of the fourth forum on Implications of recent earthquakes on seismic risk, Technical report TIT/EERG, 02/1*, Tokyo Institute of Technology, Tokyo, pp. 291-308.
11. Fajfar P., Marusic D. and Perus I., "Influence of ground motion intensity on the inelastic torsional response of asymmetric buildings", *Proc. of the 13th World Conference on Earthquake Engineering, Vancouver, Canada, Paper No. 3496*, (2004).

12. Fajfar P., Marusic D. and Perus I., “Torsional effects in the pushover-based seismic analysis of buildings”, *Journal of Earthquake Engineering*, Vol. 9, No. 6, (2005), pp. 831-854.
13. Fujii K., Nakano Y. and Sanada Y. [2004], “Simplified nonlinear analysis procedure for asymmetric buildings”, *Proc. of the 13th World Conference on Earthquake Engineering, Vancouver, Canada, Paper No. 149*, (2004).
14. Hejal R. and Chopra A.. “Earthquake response of torsionally-coupled buildings”, *Report UCB/EERC-87/20*, (1987).
15. Kan C. L. and Chopra A. K. (1979). “Linear and nonlinear earthquake responses of simple torsionally coupled systems”, *Report UCB/EERC-79/03*, (1979).
16. Kilar V. and Fajfar P., “On the applicability of pushover analysis to the seismic performance evaluation of asymmetric buildings”, *European Earthquake Engineering 15*, (2001), pp. 20-31.
17. Moghadam A. S. and Tso K., “Pushover analysis for asymmetric and set-back multistory buildings”, *Proc. of the 12th World Conference on Earthquake Engineering, Auckland, New Zealand, Paper No. 1093*, (2000).
18. Rutenberg A., EAEE Task Group (TG) 8: “Behaviour of irregular and complex structures asymmetric structures - progress since 1998”, *Proc. of the 12th European Conference on Earthquake Engineering, London, UK, Paper No. 832*, (2002).
19. Rutenberg A., Levy R. and Magliulo G., “Seismic response of asymmetric perimeter frame steel buildings”, *Proc. of the 12th European Conference on Earthquake Engineering, London, UK, Paper No. 107,* (2002).
20. Tso W. K. and Moghadam A. S. [1997], “Seismic response of asymmetrical buildings using pushover analysis”, *Proc. of the International Workshop on Seismic Design Methodologies for the Next Generation of Codes, Bled, Slovenia*, eds. P. Fajfar and H. Krawinkler (A.A. Balkema, Roterdam, Brookfield), (1997), pp. 311-321.
21. Tso W. K. and Myslimaj B., “Effect of strength distribution on the inelastic torsional response of asymmetric structural systems”, *Proc. of the 12th European Conference on Earthquake Engineering, London, UK, Paper No. 081*, (2002).
22. Yu Q.S.K., Pugliesi R., Allen M. and Bischoff C., “Assessment of modal pushover analysis procedure and its application to seismic evaluation of existing building”, *Proc. of the 13th World Conference on Earthquake Engineering, Vancouver, Canada, Paper No. 1104*, (2004).
23. Zarate A. and Ayala A. G., “Validation of single storey models for the evaluation of seismic performance of multi-storey asymmetric buildings”, *Proc. of the 13th World Conference on Earthquake Engineering, Vancouver, Canada, Paper No. 2213*, (2004).
24. ReLUIS project - Rete dei Laboratori Universitari di Ingegneria Sismica – Earthquake Engineering Test Labs Network, www.reluis.unina.it.
25. www.isesd.cv.ic.ac.uk, web site of the European Strong Motion Database.
26. OPCM 3274/05, Ulteriori modifiche ed integrazioni all’ordinanza del Presidente del Consiglio dei Ministri n.3274 del 20 marzo 2003 recante “Primi elementi in materia di criteri generali per la classificazione sismica del territorio nazionale e di normative tecniche per le costruzioni in zona sismica”, Annex 2, ch. 11.
27. Petti L., Marino I., De Iuliis M., Giannattasio G., “Problematiche di modellazione delle strutture esistenti in c.a. per la verifica sismica mediante analisi statica non lineare”, *L’ingegneria sismica in Italia, XII convegno, Pisa*, (2007), paper 442.
28. Open System for Earthquake Engineering Simulation (OpenSEES), Pacific Earthquake Engineering Research Center, University of California, Berkley (USA); http://opensees.berkley.edu.

29. Ferracuti B., Pinho R., Savoia M., Francia R., “Pushover versus Incremental Dynamic Analyses”, *L’ingegneria sismica in Italia, XII convegno, Pisa*, (2007), paper 234.

30. Pettinga J. D., Priestley M. J. N., Pampanin S., Christopulos C., “The Role of Inelastic Torsion in the Determination of Residual Deformations”, *Journal of Earthquake Engineering, vo.l 11 2007, supplement 1, Proceedings of the Sixth ROSE Seminar, (2007)*

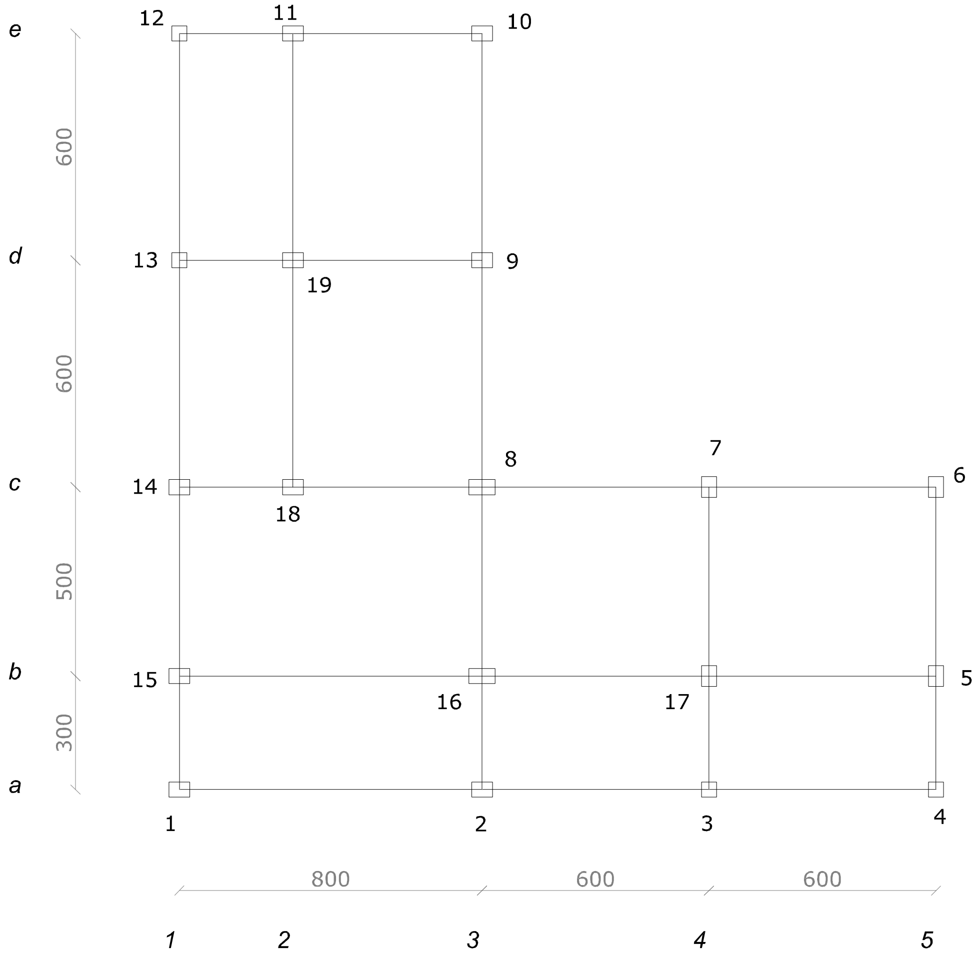

Fig. 1 – Benchmark Structure Plan

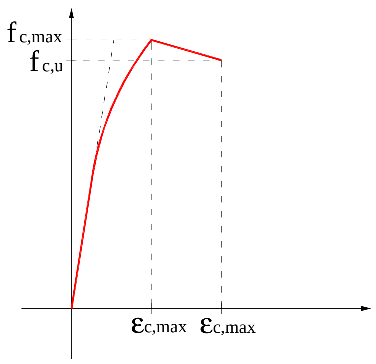

Fig. 2 - Concrete01: Stress-Strain Relationship

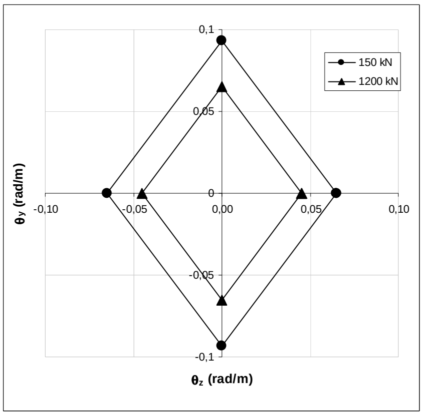

Fig. 3 – Example of Collapse Curvature Domain for Columns

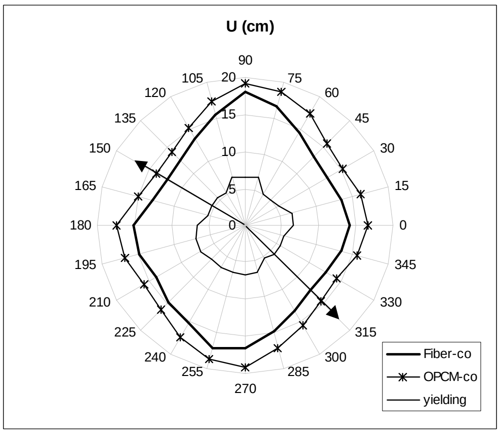

Fig. 4 - Collapse and Yielding Displacement-domain for Triangular Distribution

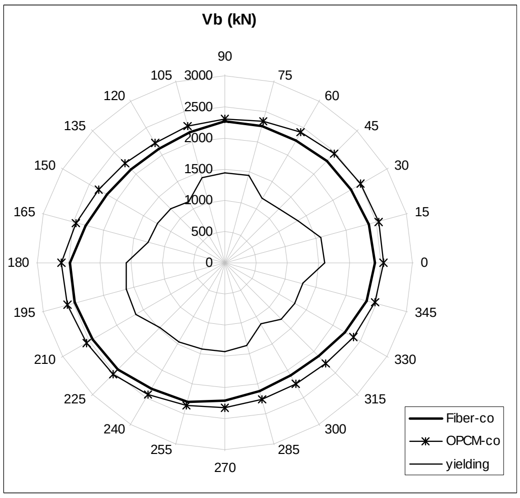

Fig. 5 - Collapse and Yielding Force-domain for Triangular Distribution

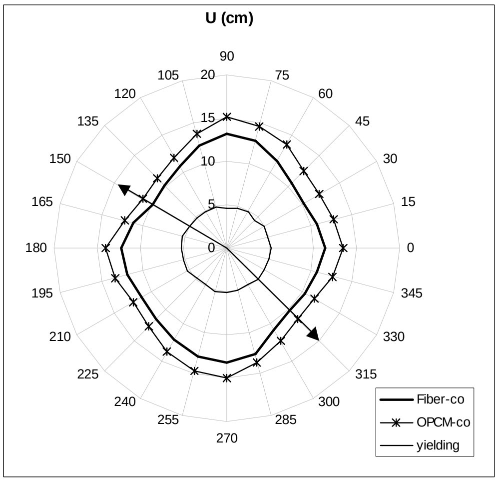

Fig. 6 - Collapse and Yielding Displacement-domain for Uniform Distribution

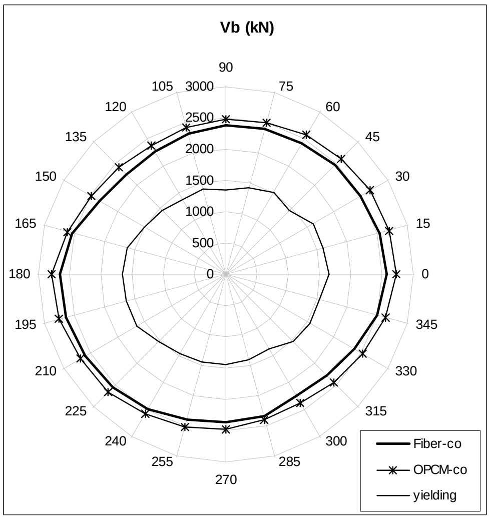

Fig. 7 - Collapse and Yielding Force-domain for Uniform Distribution

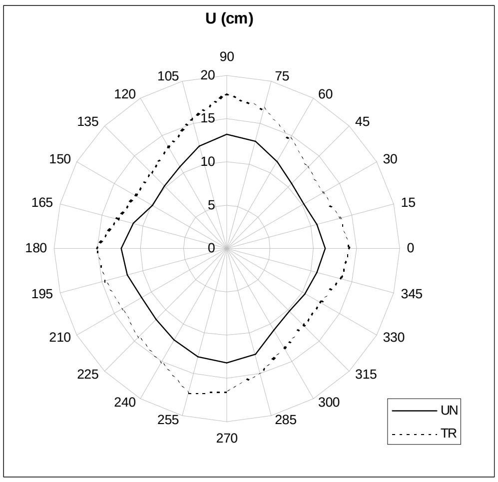

Fig. 8 - Collapse Displacement-Domain for the Force Distributions under examination.

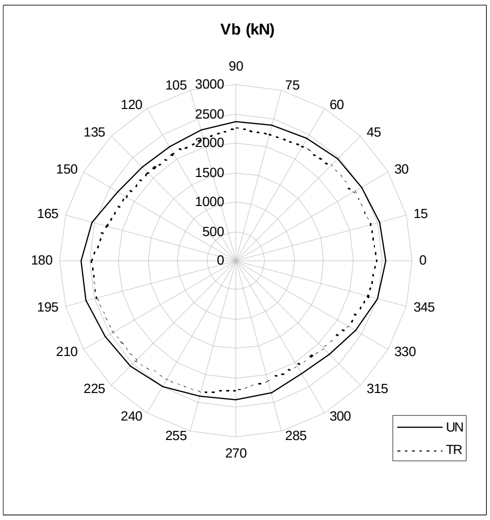

Fig. 9 - Collapse Force-Domain for the Force Distributions under examination

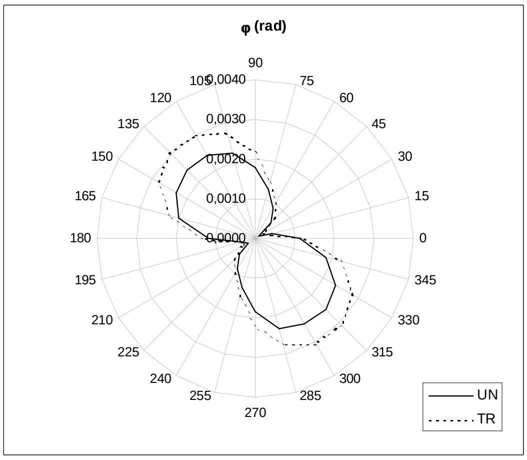

Fig. 10 - Collapse Rotation-Domain for the Force Distributions under examination

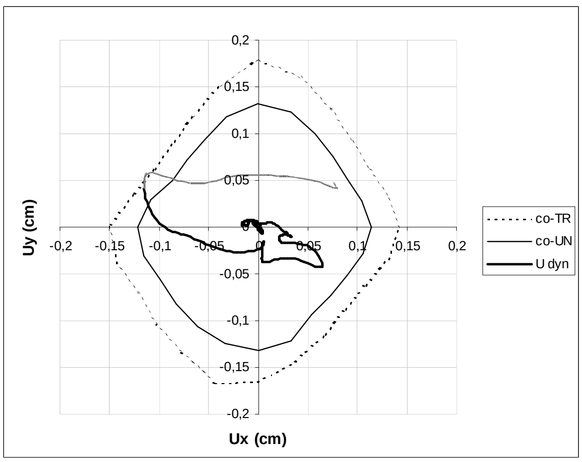

Fig. 11 – Time History Response (*analysis a*) – Iceland 21/6/2000 earthquake ReLUIS, European strong motion database

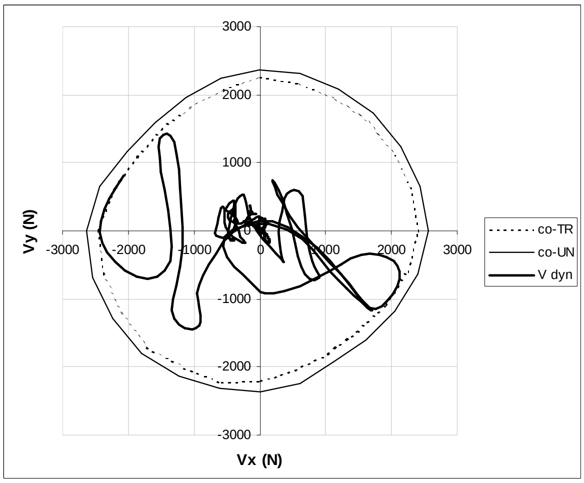

Fig. 12 - Time History Response (*analysis a*) – Iceland 21/6/2000 earthquake ReLUIS, European strong motion database

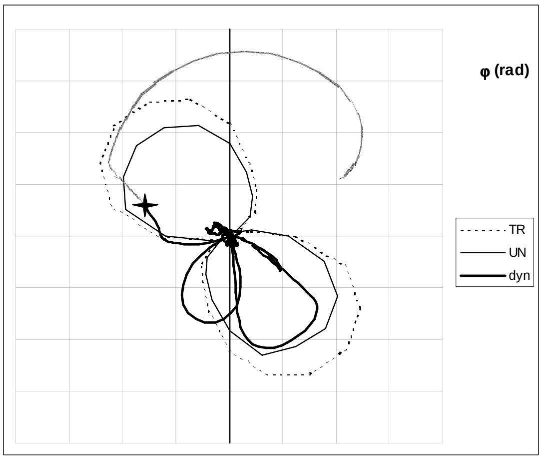

Fig. 13 - Time History Response (*analysis a*) – Iceland 21/6/2000 earthquake ReLUIS, European strong motion database

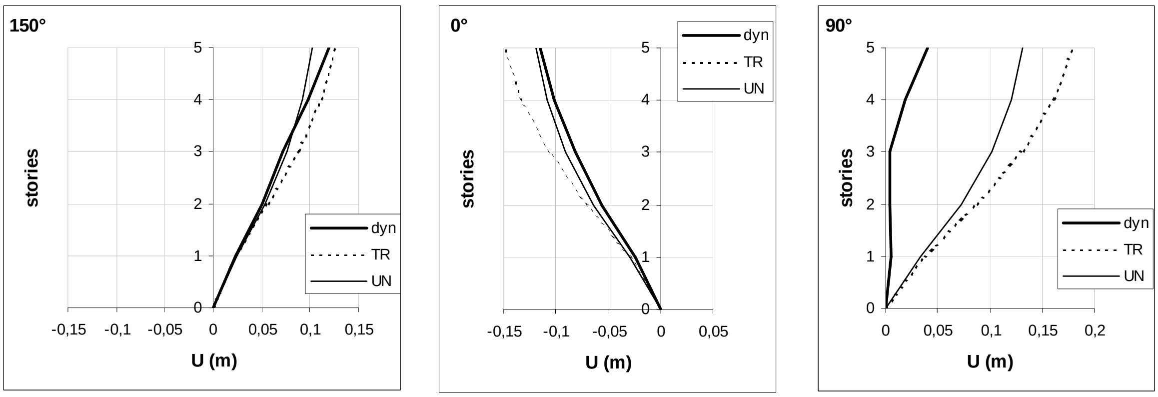

Fig. 14 - Storey Displacement Comparison at Collapse

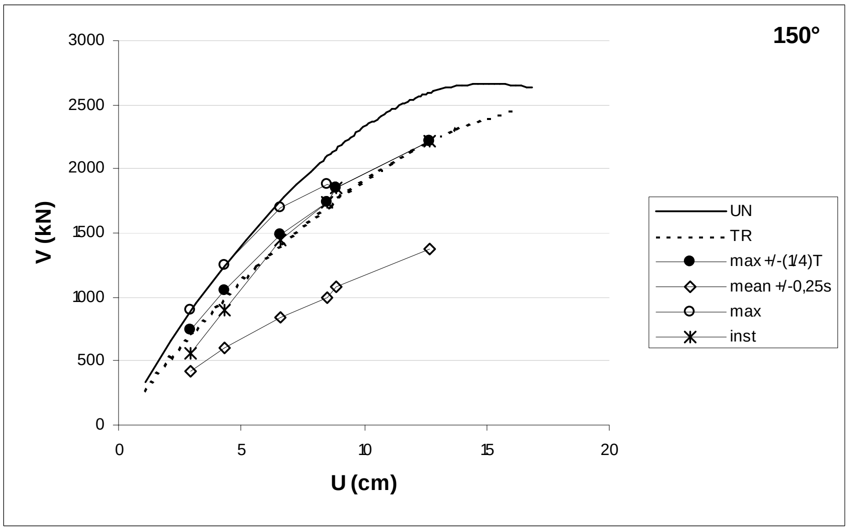

Fig. 15 – Comparison between IDA (*analysis a*) and pushover analyses

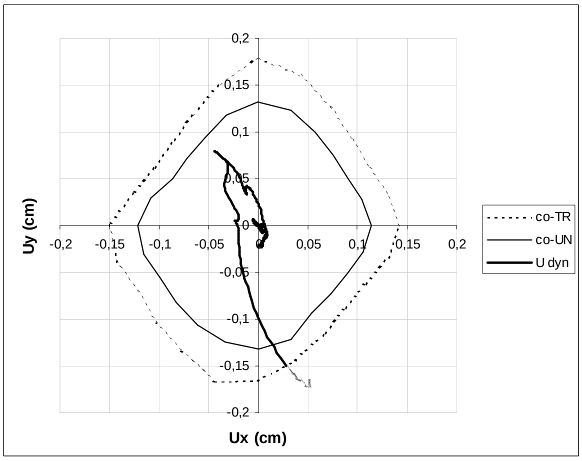

Fig. 16 – Time History Response (*analysis b*) – Iceland 21/6/2000 earthquake ReLUIS, European strong motion database

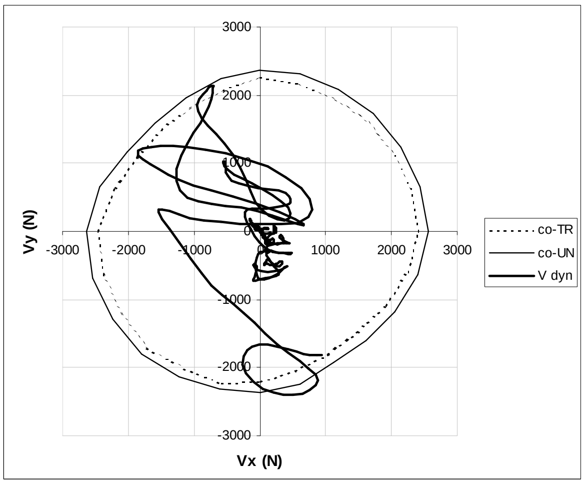

Fig. 17 – Time History Response (*analysis b*) – Iceland 21/6/2000 earthquake ReLUIS, European strong motion database

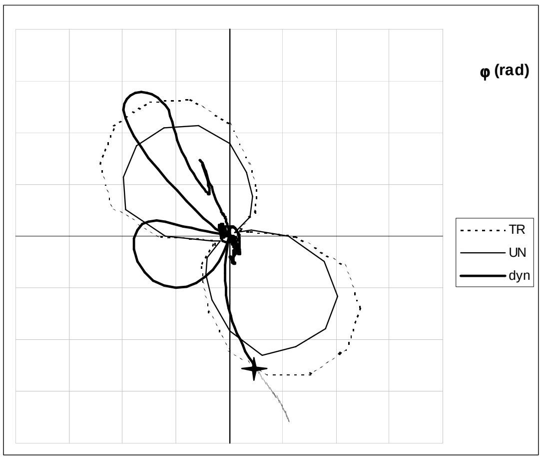

Fig. 18 – Time History Response (*analysis b*) – Iceland 21/6/2000 earthquake ReLUIS, European strong motion database

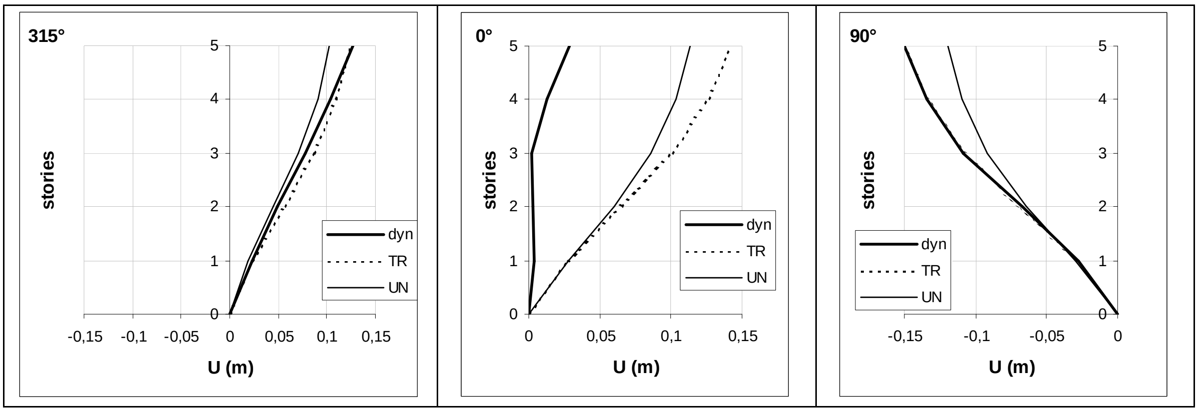

Fig. 19 - Storey Displacement Comparison at Collapse

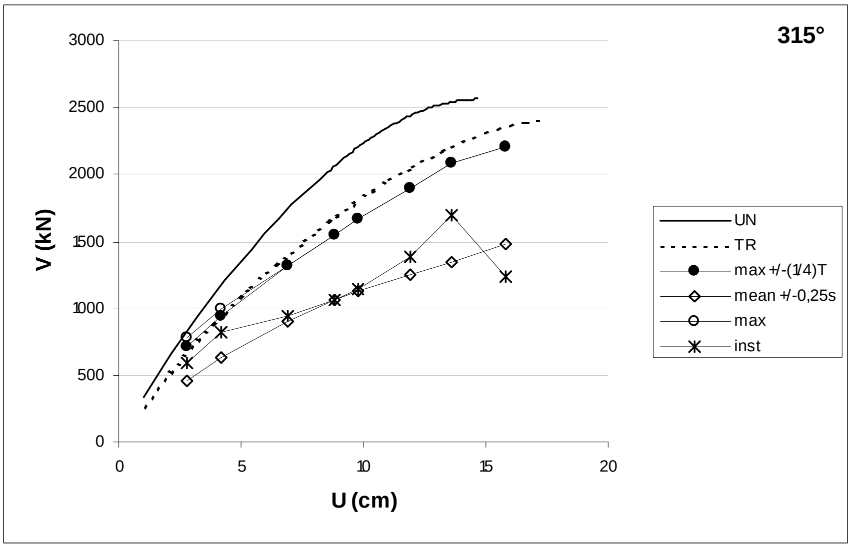

Fig. 20 – Comparison between IDA (*analysis b*) and pushover analyses