SEISMIC RESPONSE ANALYSIS OF 3D STRUCTURES THROUGH SIMPLIFIED NON-LINEAR PROCEDURES

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ABSTRACT :

New tools for the analysis of the seismic behaviour of plan-asymmetric structures are herein presented and the concepts of "polar spectrum" and limit domains are discussed. Both polar spectrum and limit domains allow researchers to investigate the non-linear capacities of plan-asymmetric structures. The proposed procedure has been validated by comparing the results obtained with those of non-linear dynamic analyses for two benchmark structures.

KEYWORDS: Plan-asymmetric structures, non-linear analyses, polar spectrum

1. 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 require 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. 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).

However, non-linear static procedures can lead to inadequate results when applied to irregular structures because of the difficulties in taking into account dynamic latero-torsional effects. 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 ([Ayala and Tavera, 2002], [Chopra and Goel, 2004], [Fajfar and Kilar, 1997]). Some of the main problems concern the combination of forces in the two directions, and the most effective way to consider extra ductility demand for the elements near the soft edge.

Among the early proposals we mention one [Moghadam and Tso, 2000], 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 [Fajfar et al., 2002]. Further research has focused on the behaviour of framed structures with shear walls [De Stefano and Rutenberg, 1998] and on the accuracy of results on varying the loads plan distribution [Faella and Kilar, 1998]. Finally, Chopra has presented an extension of the MPA (Modal Pushover Analysis) procedure for asymmetric-plan structures [Chopra and Goel, 2004]. Available comparisons between these methods and the results of non-linear dynamic analysis (NDP) generally show limited success rates in the proposed procedures.

The in-plane elasto-plastic behaviour of plan-asymmetric structures has been recently investigated by means of resistance domains based on the results of pushover analyses [Petti et al. 2007]. The obtained results, checked by means of non-linear incremental dynamic analyses, have pointed out that non-linear static analyses, carried out for different in-plan directions of the incoming seismic action, have allowed us to accurately evaluate the least seismic resistant directions for complex structures. Such an approach allows us to take into account the torsional response in the inelastic range where, as analysed in recent studies, it seems less relevant than for elastic behaviour of the structure [Fajfar et al., 2008]. As is well-known, the collapse conditions for a three-dimensional system are also

governed by the spatial features of the seismic event.
The goal of this work is to investigate the seismic behaviour of plan-asymmetric structures by considering the least seismic-resistant directions and the spatial features of the seismic event. The capacity of the structure is described by using the limit domains based on the NSPs, while the seismic demand is analysed by introducing a new representation of the spectral response. This representation is based on the construction of a spectral surface obtained by the spectral seismic response for different in-plan directions. The in-plan projection of this surface is herein defined “Polar Spectrum”.

2. THE CONCEPT OF “POLAR SPECTRUM”

As we know, seismic codes are based on the application of equivalent static forces depending on the spectral response of an elastic single degree of freedom system. In order to take into account the spatial variability of the action, a combination of the effects in the two main direction can be considered.
However, analysis of the NS and EW components of real seismic events shows that the seismic shaking for a particular site presents **main directions** due to the event’s seismogenetic characteristics, the path from the source to the site and the local effects. This variability is herein assessed by means of spectra obtained for different α directions in plan. This study considers the following seismic events from the European Strong Motion Database (Table 1.1).

Table 1.1 Considered seismic events

Waveform Code	000199	000228	000535	006328	06334
Event Code	93	108	250	2142	
Event Name	Montenegro	Montenegro (aftershock)	Erzincan	South Iceland (aftershock)	
Country	Yugoslavia	Yugoslavia	Turkey	Iceland	
Date	15/04/79	24/05/79	13/03/92	21/06/00	
Station Code	67	67	205	2484	2488
P.G.A. (m/s²)	3,68	2,65	5,03	3,84	7,07

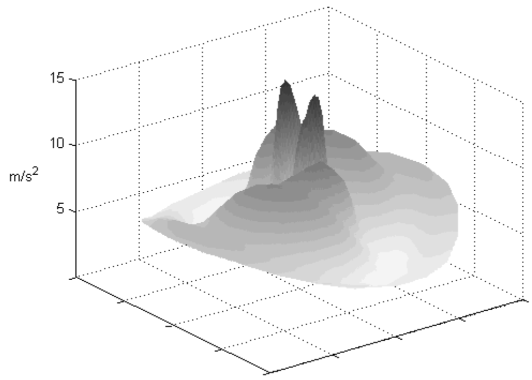

Figure 1 Spectral surface for the Erzincan seismic event (*000535*), ξ=0,05

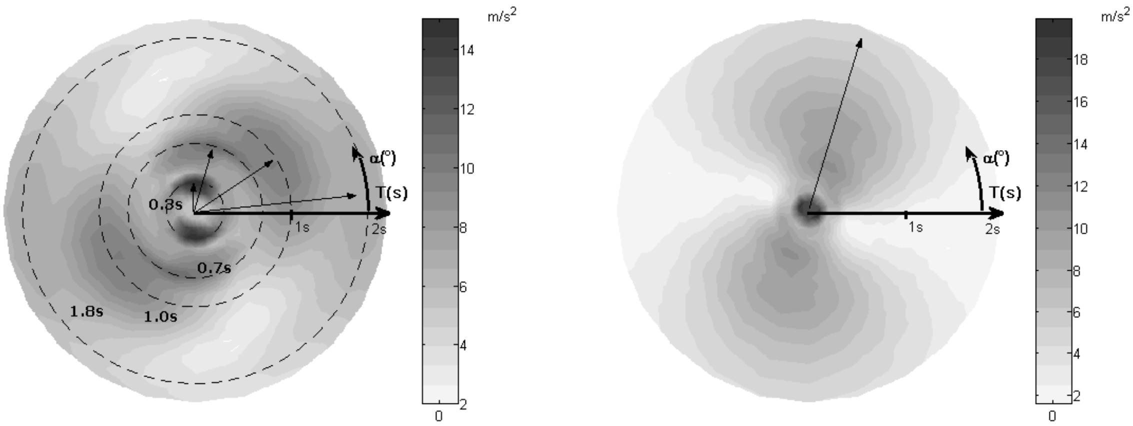

Figure 2 Polar Spectrum in terms of pseudo-acceleration evaluated for the seismic events: Erzincan (code *000535*, on the left), South Iceland aftershock (code *006334*, on the right)

Figure 1 shows, as an example, the spectral surface for the Erzincan event, evaluated with 15° increments of the direction in plan and 0.1s increments of the vibration period. To get a more useful visualization of the in-plan

variability of the seismic event it is possible to refer to the plane projection of the spectral surface, highlighting the values of the spectral response by means of a grayscale map. Such a representation is herein defined "polar spectrum".
Figure 2 shows, as an example, the polar spectra in terms of pseudo-acceleration for the seismic events Erzincan (code 000535) and South Iceland aftershock (code 006334) with a damping factor ξ=0,05. Analysis of results shows that for the Erzincan event (code 000535) the maximum response direction changes with the vibration period. In particular, with the increasing period, the maximum response is attained along directions varying from 90° (T=0.3s) to 5° (T=1.8s). For the South Iceland aftershock seismic event (code 006334) it can be observed, instead, that the peak response is attained along the same direction (75°) regardless of the vibration period value.

3. DESCRIPTION OF THE MAIN FEATURES OF THE BENCHMARK STRUCTURES

For this study two benchmark structures are considered.
The first structure (structure *A*) is a five-storey L-shaped building with RC frames in two orthogonal directions, chosen from among the case-studies of the ReLUIS project [ref. ReLUIS]. A finite elements model has been assembled using the OpenSEES software [ref. OpenSEES]. In this model, beams and columns are spread plasticity non-linear elements with fiber sections applied; the floors are modelled by means of elastic shells with appropriate thickness [Petti et al., 2007]. The collapse condition for the generic section is defined in terms of section curvature, evaluated by imposing limit strain values for the concrete fibers (ε_c=0.006 in compression) and for the reinforcing steel fibers (ε_S =0.03 under tension).
The second structure (structure *B*) is the three-storey building considered in the Spear Project ([ref. SPEAR Project], [Fajfar et al. 2005]). In particular the finite element model is the "Post Test Model" [Dolsek and Fajfar, 2005] developed in OpenSEES. This model uses lumped plasticity with plastic hinges for beams and columns, while the floors are modelled by numeric constraints. The plastic hinges are defined through a tri-linear diagram and the collapse state is identified with the rotation corresponding to the 80% of the maximum bending moment on the softening branch of the diagram.
The dynamic properties for both models are summarized in table 3.1 (vibration period T, the mass participation ratios M%, the sum of the participation ratios up to the considered modal shape SUM).

Table 3.1 Modal properties for the benchmark structure *A* (on the left) and *B* (on the right)

Modo	T(s)	M%x	M%y	SumX	SumY
1	1,35	10,9%	52,1%	10,9%	52,1%
2	1,22	56,9%	19,6%	67,8%	71,8%
3	0,99	10,8%	9,2%	78,6%	80,9%
4	0,42	0,9%	8,5%	79,4%	89,4%
5	0,38	9,4%	1,9%	88,8%	91,2%
6	0,31	1,5%	1,1%	90,3%	92,4%
7	0,23	0,1%	4,1%	90,4%	96,5%
8	0,20	4,2%	0,5%	94,6%	97,0%
9	0,17	0,9%	0,5%	95,5%	97,5%

Modo	T(s)	M%x	M%y	SumX	SumY
1	1,03	67,4%	1,6%	67,4%	1,6%
2	0,84	15,1%	28,3%	82,5%	29,9%
3	0,67	1,8%	58,1%	84,4%	88,0%
4	0,31	7,6%	0,7%	92,0%	88,7%
5	0,25	3,6%	3,6%	95,6%	92,3%
6	0,21	2,3%	0,0%	98,0%	92,3%
7	0,21	0,5%	5,7%	98,4%	98,1%
8	0,17	1,6%	0,3%	100,0%	98,3%
9	0,12	0,0%	1,7%	100,0%	100,0%

4. IN-PLAN LIMIT DOMAINS

In order to assess the non-linear static response of plan-asymmetric structures it is possible to obtain limit domains, in terms of displacement or base shear. In particular such domains are obtained from the results of non-linear static procedures (pushover) carried out by varying the in-plan direction of the load distribution [Petti et al. 2007]. The domains depict the displacement of the control node (top floor centre of mass) or the base shear for the selected limit state. For the benchmark structures *A* and *B* the displacement and base shear collapse domains are shown in figures 3 and 4. The analysis of the figures indicates non-uniform in-plan behaviour of both the benchmark structures. In particular, the non-linear static procedures performed along the main axes overestimate the capacities of the structures.

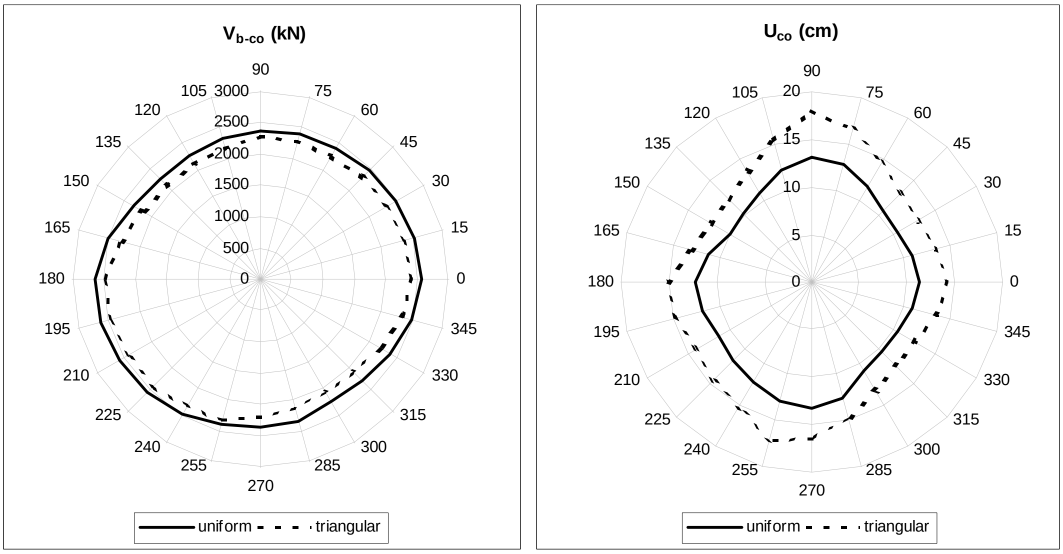

Figure 3 Limit domains for the benchmark structure *A*: in terms of base shear (on the left) and control node displacement (on the right)

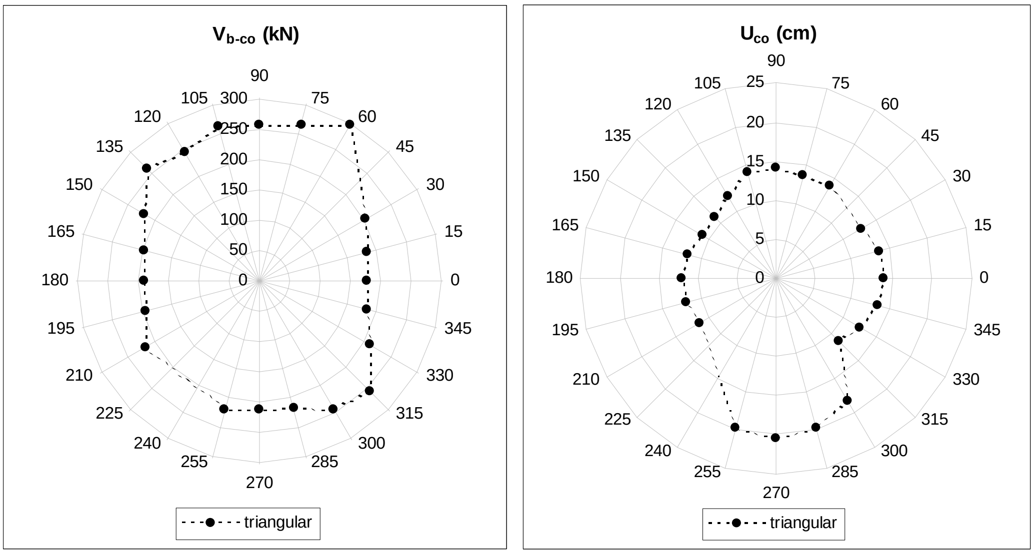

Figure 4 Limit domains for the benchmark structure *B*: in terms of base shear (on the left) and control node displacement (on the right)

5. NON-LINEAR DYNAMIC ANALYSES

In order to evaluate the applicability of the limit domains to accurately describe the collapse state of the structure, incremental dynamic analyses have been performed. For both the considered in-plan dispositions of the structures, 0° and 90°, the analyses have been carried out by increasing the amplification factor of the seismic event up to the collapse.

Therefore, for same sample events applied to each structure, the following results are shown:

- the polar spectrum with the indication of the main translational vibration periods of the structure
- the comparison between the results of the non-linear dynamic procedures and the collapse domain in terms of top displacement
- the comparison between the results of the non-linear dynamic procedures (NDP) and the non-linear static procedures (NSP) in terms of storey displacements along the collapse direction

Figures 5-7 depict the results for the Montenegro seismic event (code *000199*) applied to *structure A*.

The analysis of the obtained results (figures 6-7) points out that the structure attains the collapse along the same direction regardless of its in-plan orientation. It can be seen, by analysing figure 5, that this direction is the one along which the maximum spectral response is attained for the 1° and 2° vibration period. In both cases the collapse is attained in accordance with the limit domain obtained with the triangular force distribution. Moreover, 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.

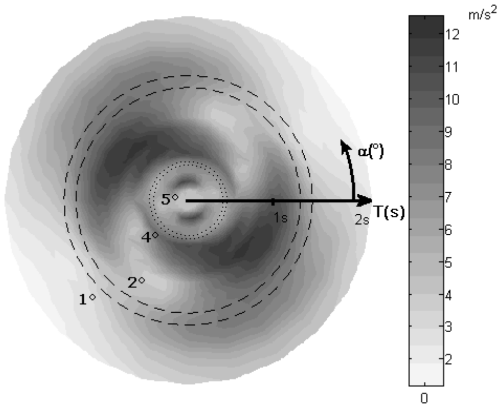

Figure 5 Polar Spectrum – Montenegro Event (code *000199*) – Main vibration periods for structure *A*

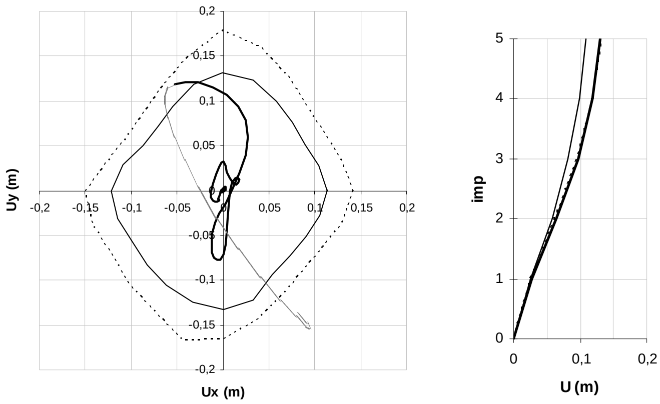

Figure 6 *Structure A* - comparison between NDP and NSP in terms of control node displacement and deformed shape for the Montenegro event *000199* and in-plan orientation *0°* (NDP collapse amplification factor 0.35) -thin line: NSP uniform distribution; dashed line: NSP triangular distribution; bold line: NDP-

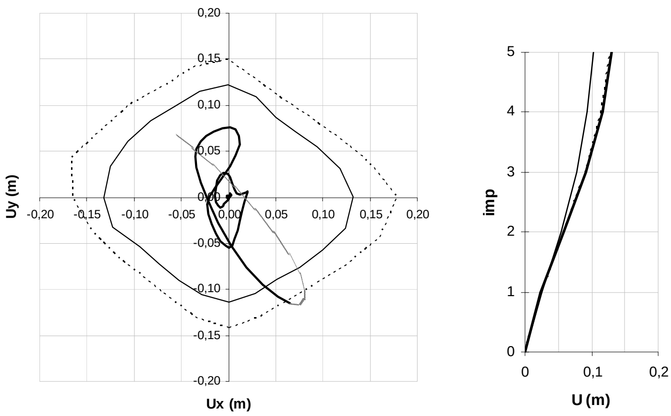

Figure 7 *Structure A* - comparison between NDP and NSP in terms of control node displacement and deformed shape for the Montenegro event *000199* and in-plan orientation *90°* (NDP collapse amplification factor 0.30) -thin line: NSP uniform distribution; dashed line: NSP triangular distribution; bold line: NDP-

Figures 8-10 depict the results related to the Erzincan event (code *000535*) applied to *structure B*.
The collapse direction is the same for the two different orientations of the structure. Such a direction results as being the one along which the maximum spectral response is attained for the main vibration periods (figure 8).
Figures 9-10 show that the collapse state is fully consistent with the statically obtained domain for the analysis with orientation 0°, but not so much for the analysis with orientation 90°. Focusing on figure 10 it can be seen that the static deformation for orientation 90° is not consistent with the triangular force distribution, whose aim should be to simulate the first modal shape. This circumstance, which is due to the activation of plastic hinges at intermediate floors, suggests the application of adaptive procedures in performing pushover analysis ([Aydinoglu, 2003], [Bento and Bhatt, 2008]).

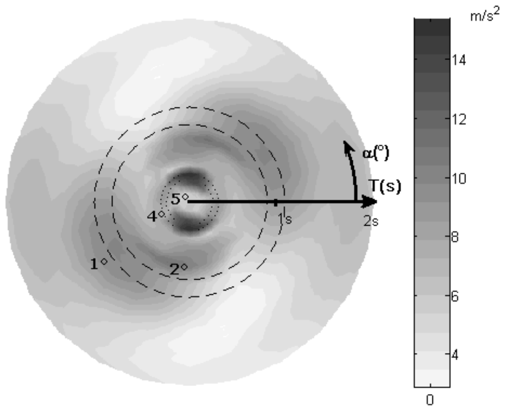

Figure 8 Polar Spectrum –Erzincan Event (code *000535*) – Main vibration periods for *structure B*

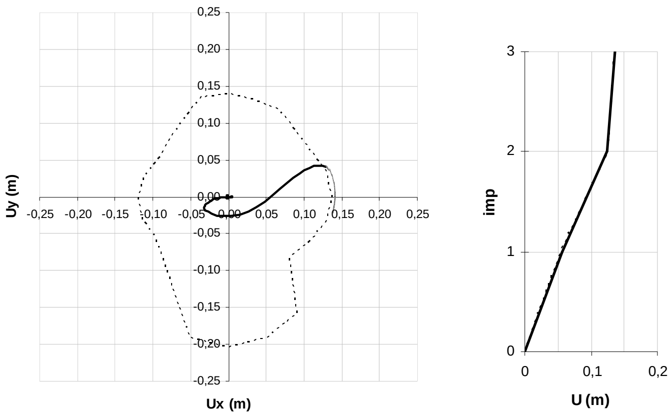

Figure 9 *Structure B* - comparison between NDP and NSP in terms of control node displacement and deformed shape for the Erzincan event *000535* and in-plan orientation *0°* (NDP collapse amplification factor 0.32) -thin line: NSP uniform distribution; dashed line: NSP triangular distribution; bold line: NDP-

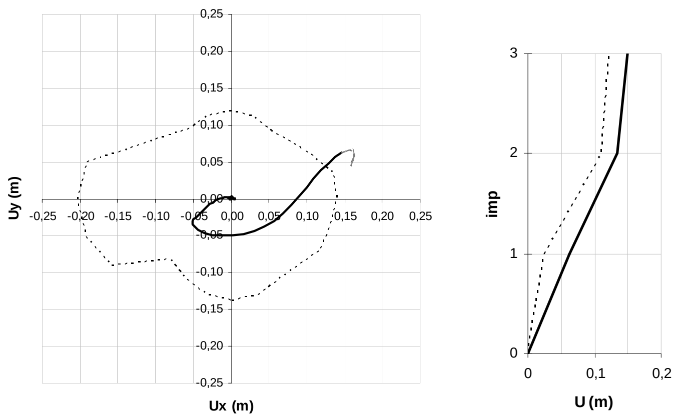

Figure 10 *Structure B* - comparison between NDP and NSP in terms of control node displacement and deformed shape for the Erzincan event *000535* and in-plan orientation *90°* (NDP collapse amplification factor 0.44) -thin line: NSP uniform distribution; dashed line: NSP triangular distribution; bold line: NDP-

The possibility of evaluating the maximum structural response moving from observation of the polar spectrum suggests performing dynamic analyses with the structure oriented so that its least resistant direction (assessed by means of the statically obtained domains) is aligned with the max solicitation axes (evaluated by means of the polar spectrum).
Figures 11 presents two example (one for each benchmark structure) of analyses performed with such a "critical" alignment.
For the South Iceland aftershock event (code *006334*) *structure A* attains the maximum spectral response approximately along the axis 75°-255°; considering that for *structure A* the least resistant direction corresponds to α=135° and the alignment is obtained by rotating the structure 120° counter-clockwise. With this orientation the NDP collapse amplification factor is equal to 0.30, while for the standard orientation it is 0.40 (*0°*) and 0.35 (*90°*).
In the case of structure *B* the least resistant direction corresponds to α=150° and the critical alignment is obtained by rotating the structure about 100° counter-clockwise. The collapse is attained with a NDP amplification factor equal to 0.24, while for the two standard orientation it is equal to 0.31 and 0.29.

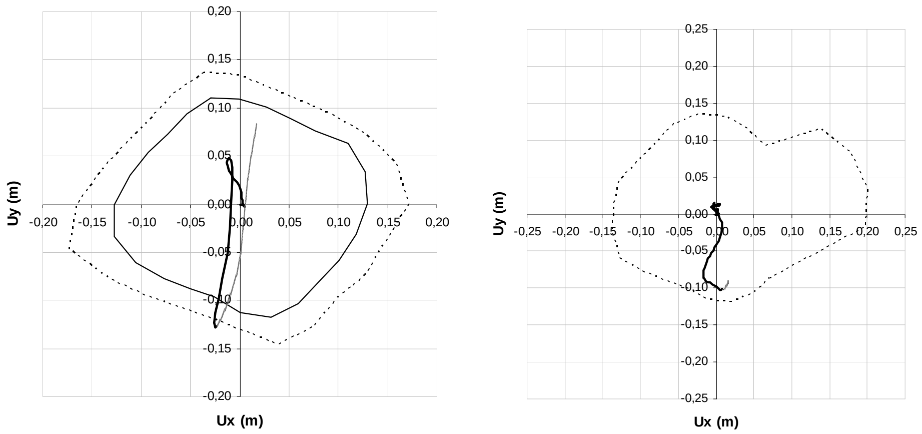

Figure 11 Top displacement for the analysis with "critical" alignment for *structure A* on the left (NDP collapse amplification factor 0.30) and *structure B* on the right (NDP collapse amplification factor 0.24)

The obtained results generally confirm a good degree of agreement between the static proposed procedure and non-linear dynamic analyses**.** Some differences could be observed in the case of seismic events able to excite higher modes or for structures exhibiting a non-standard collapse mechanism. For these cases, the limit domains should be based on adaptive or/and multimodal pushover procedures ([Chopra A. K. 2008], [Goel R.K. 2008]).

6. CONCLUDING REMARKS

The seismic behaviour of plan-asymmetric structures has been investigated here by means of limit domains, based on the results of non-linear static procedures, and by considering the spatial characteristics of the seismic events. In particular, to completely describe the in-plan variability of the seismic shaking a new representation of the dynamic demand has been proposed: the "polar spectrum".
The analysis of the results obtained for two benchmark structures indicates the following:

- The collapse condition, assessed by means of non-linear dynamic procedures, turns out to be coherent with the limit domains based on the results of non-linear static procedures;
- The non-linear dynamic analyses have shown that the directional components of the seismic event govern the collapse direction;
- The worst condition for the structure is obtained by aligning its least resistant direction, evaluated on the basis of the limit domains, with the maximum response direction gathered from the analysis of the polar spectrum.

Therefore, for the examined case studies, the combined use of the limit domains and of the polar spectrum allows us to identify a lower bound safety factor for the structure. Such a "critical alignment" also results as being meaningful when the post-elastic behaviour is not fully consistent with the force distribution assumed to perform the non-linear static analyses.
In the case of seismic events capable of exciting higher modes or for structures exhibiting a non-standard collapse mechanism, the limit domains should be based on adaptive or/and multimodal pushover procedures.

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