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NON-LINEAR STATIC PROCEDURES IN PERFORMANCE BASED SEISMIC DESIGN

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SUMMARY

The performance of a structural system can be evaluated resorting to non-linear static analysis. This involves the estimation of the structural strength and deformation demands and the comparison with the available capacities at desired performance levels. This paper aims at evaluating and comparing the response of two reinforced concrete building systems by the use of different methodologies namely the ones described by the ATC-40 and the FEMA-273 and by the EC8 (Eurocode 8) design code using non-linear static procedures, with described acceptance criteria. Some results are also compared with the non-linear dynamic analysis. The methodologies are applied to a 4 and 8 storey frames system, both designed as per the Eurocodes in the context of Performance Based Seismic Design procedures.

INTRODUCTION

In recent years, the term Performance Based Design is being used as a popular buzzword in the field of earthquake engineering, with the structural engineer taking keen interest in its concepts due to its potential benefits in assessment, design and better understanding of structural behavior during strong ground motions. The basic idea of Performance Based Design is to conceive structures that perform desirably during various loading scenarios. Furthermore, this notion permits the owners and designers to select personalized performance goals for the design of different structures. However, there is a need to emphasis that some minimum level or minimum acceptable criteria are required to be fulfilled by all structures.

In the context of seismic design, it has been realized that the increase in strength may not enhance safety, nor reduce damage. The distribution of strength through the building rather than the absolute value of design base shear is now considered of importance, as endorsed by the capacity design principles. At the same time, the objective of most codes is to provide life safety performance during large and infrequent earthquakes. However, recent earthquakes have shown that structures may suffer irreparable or too costly to repair damages. Besides, inelastic behavior, indicating damage, is observed even during smaller earthquakes. It seems that Performance Based Design concepts, which consent multi-level design

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objectives, could provide a framework to improve the current codes; by obtaining structures that perform appropriately for all earthquakes.

Preferably, the performance assessment of structural systems subjected to the seismic action should be based on inelastic time history analysis, that assess with sufficient reliability damages and forces demands in all structural elements. This type of analysis requires a set of carefully selected ground motion records. Moreover, it needs to model adequately the cyclic behavior of all structural elements and to define carefully the viscous damping. It also requires additional computational effort, becoming him not very much suitable in current design.

Current research developments in seismic structural behavior indicate that the most suitable approach of achieving the performance objectives is by performing a damage-controlled design. The most important is to perform an evaluation process easy to be applied but that gets the main features that considerably influence the performance objective. Various recommendations are made in order to implement this ideology into the design procedure. The non-linear static procedures, which is the topic of this work, fulfils this purpose, regardless its limitations. Some results obtained by using different methodologies namely the ones described by the ATC-40 [1], the FEMA-273 [2] and the EC8 (Eurocode 8) [3] codes are presented and compared in this study.

NON-LINEAR STATIC ANALYSIS

As the name suggests this procedure is essentially a static analysis, in which the static loads are applied in an incremental fashion until the ultimate state of the structure is attained. The non-linear designation comes from the fact that the various components/elements are modeled using a non-linear mathematical model.

This section is dedicated to describe the main steps of this procedure, in a general manner. This is followed mainly because the concept of the non-linear static procedure is still relatively new and is the topic for discussion in this study. The various concepts and possible methodologies in its application are referred at various locations of this paper.

The employment of the non-linear static procedure involves four distinct phases as described below and illustrated in Figure 1:

1. Define the mathematical model with the non-linear force deformation relationships for the various components/elements;
2. Define a suitable lateral load pattern and use the same pattern to define the capacity of the structure;
3. Define the seismic demand in the form of an elastic response spectrum;
4. Evaluate the performance of the building.

The non-linear force deformation relationships for the components/elements should define the non-linear behavior, *i.e.* initial stiffness, yield point, post yielding stiffness, ultimate resistance and, if required, the behavior beyond the ultimate resistance of the section. These relationships are to be defined at all points where non-linear action can be expected and desired. Experimental results can be used to define the same, typically using a backbone curve obtained from a cyclic analysis (Figure 2(*i*)). Alternatively, numerical analysis may be performed to define the distinguishing points on the force-deformation relationship curve. Figure 2 (*ii*) represents a typical component with ductile behavior, characterized by an elastic range (point 0 to point 1 on the curve). Followed by a plastic range (points 1 to 3) that may include strain hardening or softening (points 1 to 2) and a strength-degraded range (points 2 to 3) in which the residual force that can be resisted is significantly less than the peak strength, but still substantial. Acceptance criteria for primary elements, that are required to have a ductile behavior, are typically within the elastic or plastic ranges between points 1 and 2, depending on the performance level.

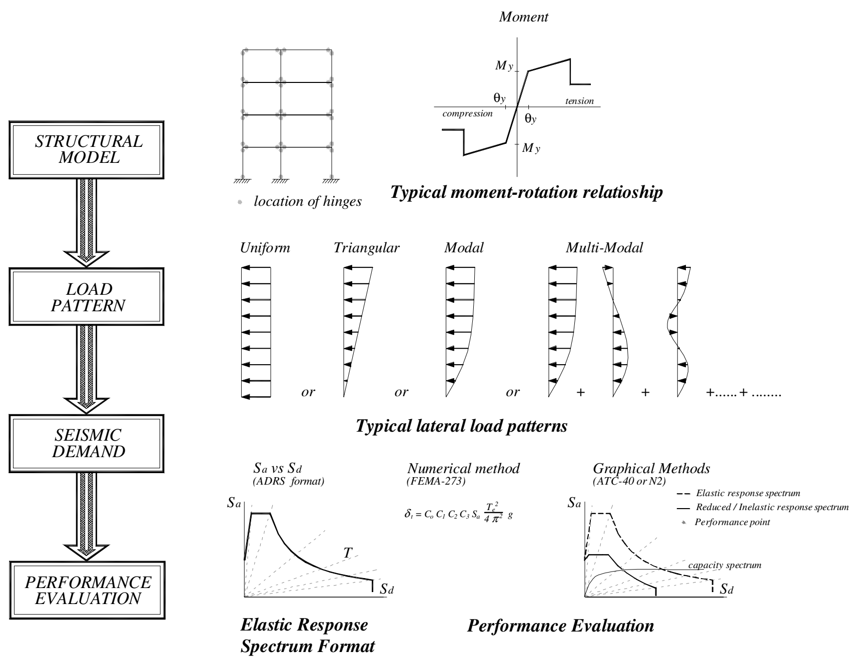

Figure 1. General flowchart for Non linear Static Procedure

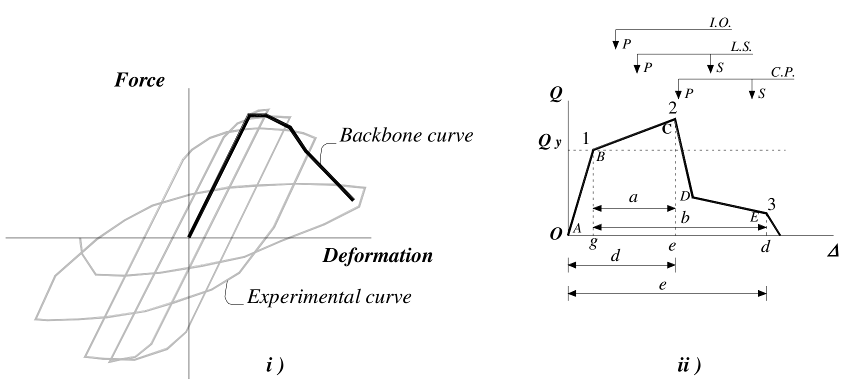

Figure 2. i) A typical backbone curve; ii) A typical force-deformation relationship used in the non-linear static procedure

The lateral loads are to be applied in profiles and should represent approximately the distribution of the inertial forces during the seismic event. It can be easily understood that, due to the changing stiffness and the different mode affects during the seismic event, the force distribution cannot be clearly distinguished. Various patterns have been proposed, right from the simple rectangular and inverted triangular patterns, to the more sophisticated ones, like modal and modal adaptive patterns. The patterns recommended by the various research committees are referred to along with their proposed format in performance evaluation.

In this procedure, the seismic demand is represented by the elastic response spectrum with a damping inherent of the structure under consideration and is used in different formats namely in the traditional

Spectral Acceleration *vs*. Period and in the Acceleration Displacement Response Spectrum format (ADRS format).

The Non-linear Static Analysis procedure generally introduced in this section is also referred to by the term *Pushover Analysis*, because of the nature of application of lateral loads while defining the capacity of the structure. In addition, it can be understood that this analysis technique offers means to establish/predict the inelastic forces, displacements, deformations etc., taking into account the non-linear behavior on the structural material during a seismic event. The concepts involved in the formulation of the pushover analysis procedure will be embarked upon in the following section. Various sophistications are possible in the application of the lateral load patterns and the performance evaluation formats.

Various methodologies have been developed for the performance evaluation using this procedure; the foremost of these are applied in this study:

1. The Displacement Coefficient Method – DCM (FEMA-273);
2. The Capacity Spectrum Method – CSM (ATC-40);
3. The N2 Method (adopted in EC8).

Possible Adaptations in Pushover Analysis

Lateral Load patterns

For an adequate performance evaluation, the proper selection of the load pattern is imperative. These patterns should bound approximately the likely distribution of inertia forces in a design earthquake, thus requiring to incorporate, in some cases, higher mode effects into the selected lateral load pattern.

An invariant load pattern assumes that: i) the inertia forces will be almost constant throughout the earthquake and ii) the maximum deformations obtained with this constant load pattern will be close to that expected to occur during the design earthquake. These two assumptions are very close to the reality when the structural response is mainly influenced by the first mode and has only a single load yielding mechanism.

As no single load distribution can identify the variation of the local demands expected in a design earthquake, the use of at least two load patterns is recommended. For instance the FEMA-273 [2] and EC8 [3] propose two lateral load patterns in the non-linear static procedure:

- The uniform load pattern, leads to conservative values of demands in lower stories, compared to the upper values, and emphasizes the importance of story shear forces compared with overturning moments;
- A modal pattern, which can account for elastic higher mode effects, makes a good choice for the second load pattern.

The main concern in using the invariant load pattern for the pushover analysis is that, it is possible to detect only local mechanism that could occur in an earthquake while weaknesses due to dynamic characteristics change may not be identifiable. It is apparent that none of the invariant lateral load patterns referred can account correctly the inertia forces redistribution when some elements undergo non-linear behavior. A pattern is thus required, that could accompany the dynamic properties change because of non-linear behavior. Therefore, it is seems appropriate to use adaptive load patterns (for example, levels 4 and 5 of ATC-40 methodology [1]). More suggestions are proposed in Krawinkler [4]:

- Use of story loads proportional to the deflected shape of the structure at each load step;

- Based on mode shapes (Figure 3) derived from secant stiffness at each step and using SRSS method to combine their effects;
- Use of patterns such that the applied story loads are proportional to the story shears resistances of the previous step.

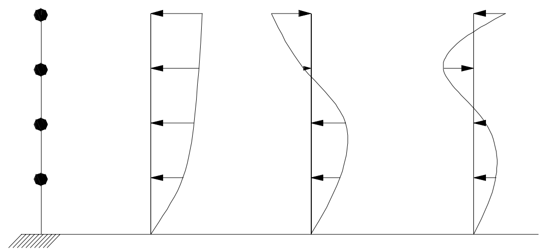

Figure 3. Load patterns due to higher modes

Recent studies, developed by some work groups (ATC-55 [5]), show that there will be potential improvements by recognize the effects of higher modes of vibration:

- Some studies suggest that the modification of the lateral load during the pushover analysis, to consider the non-linear behavior, can improve the Non-linear Static Analysis results compared with Linear ones;
- Secondly, it is suggested that combining the results of several pushover analysis, representative of different mode shapes for the same structure, can greatly improve the results.

Elnashai [6] proposed an adaptive pushover procedure, which had been further developed by Antoniou *et al.* [7]. This procedure is called 'adaptive' as the lateral load distribution is continually updated during the analysis, allowing the use of the correct force profiles defined by modal analysis at every step.

3D Structural Analysis

Initially, the non-linear static procedures have been applied to planar structures, thus not appropriate to asymmetric structures. More recently some studies have been performed to apply these methods to 3D building structures.

For instance, Fajfar [8] incorporated the torsional effect by means of pushover analyses of a 3D building structure, which applicability is reduced to torsionally stiff structures. The lateral load patterns are applied in mass centers in one direction only and the relations between base shear force and the correspondent lateral displacement of the control node (top displacement of the control node) are established. These base shear – top displacements curves are converted into an equivalent SDOF capacity curve, one for each horizontal direction. Following exactly the same procedures as in the case of planar structures, both target displacements are evaluated. Then the structure is pushed up to the target displacement defined at the mass center and the seismic demands evaluated. With the two separately planar analyses performed, the outcomes obtained are combined by means of SRSS rule.

The previous procedure could be improved if the loads are applied eccentrically in respect to mass centers. These eccentricities intend to consider some dynamic effects but no agreement has still been reached regarding the definition of the effective eccentricity values.

Ayala and Tavern [9] proposed an advanced approach. In this procedure the lateral loads include also the torsional moments. In fact, the structure is pushed with not only the lateral floor loads but also with torsional moments, considering the contribution of all modes of vibration. Thus, the curves base shear – top displacement and base moment – rotation are defined and used in the analysis.

PUSHOVER EXAMPLES

Non-linear static procedures are applied to a 4 and 8 storey frames system, both designed as per the Eurocodes in the context of Performance Based Seismic Design procedures.

Description of the structures

The 4-storey frame was designed and studied as part of a *Cooperative Research on the Seismic Response of Reinforced Concrete Structures*, carried out in collaboration between members of the *European Association of Structural Mechanics Laboratories*, under the coordination of the LNEC [10]. The 8-storey frame system was designed and detailed at the University of Patras [11] acording to EC8. The geometric layout of both structures, along with the direction of analysis, as it is studied in only one direction, are represented in Figure 4. This figure presents a plan view, a cross section and the dimension of the various structural elements for each structure. In this work, the structures are studied without the consideration of infill material.

Essentially, the four-storey framed structure is orthogonal in both directions, with plan dimensions 10x10m and interstory heights of 3m, except for the ground story 3.5m. The eight-storey building is also orthogonal in both directions, with plan dimensions 20x15m and all interstory heights of 3m.

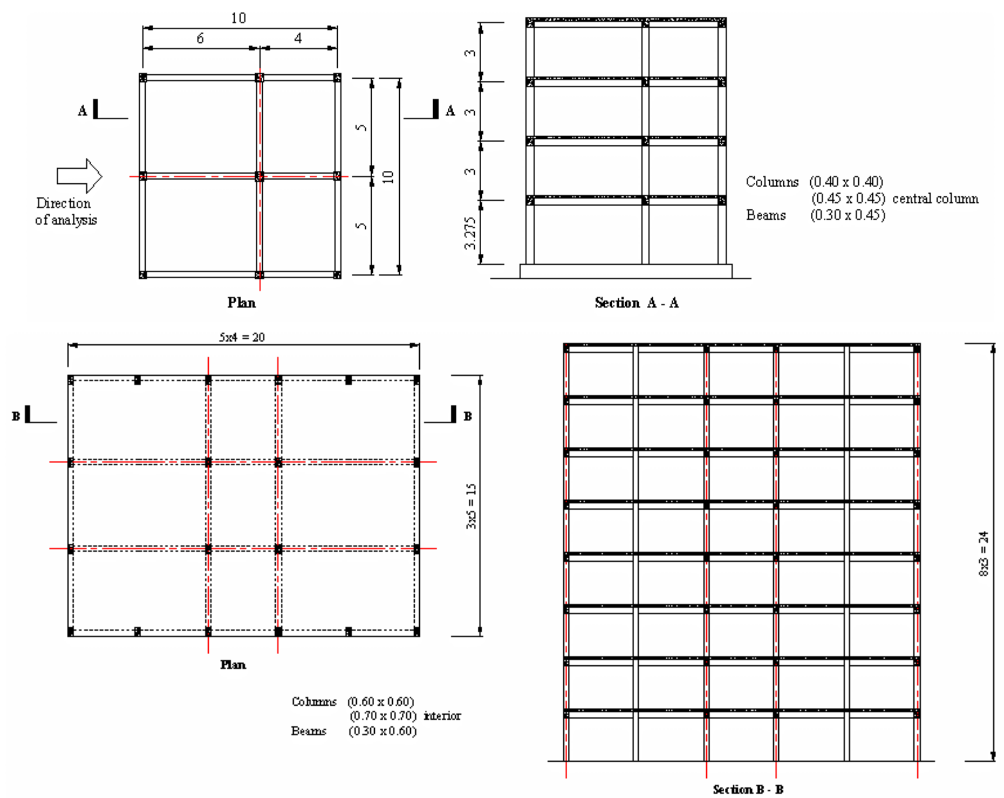

Figure 4. Geometric characteristics for the 4 and 8 frame systems

For the four-storey reinforced concrete frame building most of the analyses were performed using the program IDARC version 5 [12], in particular the non-linear static analyses based on ATC40 and FEMA-273 as well as the non-linear dynamic analysis. Beams and columns elements are modeled similarly for flexural and shear deformations, and axial deformation is considered in columns, however neglected in beams. The discretization is such that single bar elements are used for columns and beams, in plane frames. For the N2 method it was used the SAP2000 version 8 [13], especially all the studies developed with the eight-storey building.

Seismic Action

The four-storey structure was tested pseudo-dynamically at the JRC-Ispra [14] for an accelerogram based on the 1976 Friuli earthquake and consistent with the Eurocode response spectrum for intermediate soil (type B), damping equal to 5% and for a peak ground acceleration (PGA) equal to 0.3g – Figure 5.

As far as the non-linear static methodologies are concerned, all analysis are performed using the smooth Eurocode response spectrum for intermediate soil (profile B) and damping equal to 5% (Figure 5) and a ground motion level of 0.3g.

For the non-linear dynamic analyses, five different artificial accelerograms are chosen from all the artificial accelerograms defined. All these artificial accelerograms generated (Figure 6) are consistent to the Eurocode response spectrum for intermediate soil (profile B) and viscous damping equal to 5% (Figure 5).

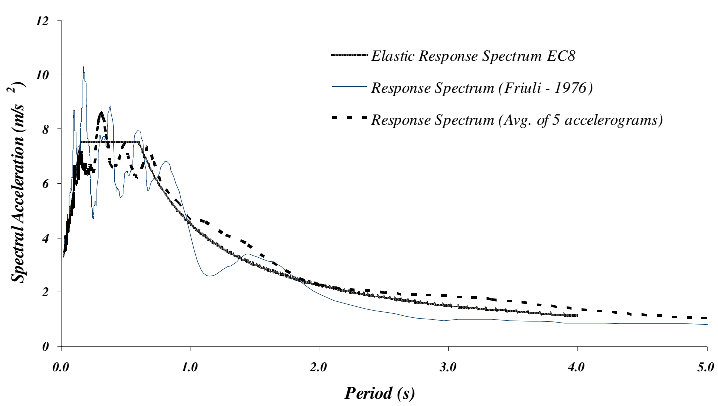

Figure 5. Response Spectra adopted in this work

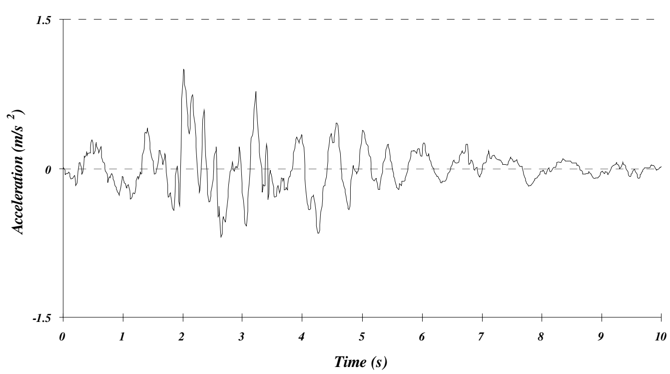

Figure 6. Reference accelerogram (normalised)

Results

This section presents a summary of the results obtained for the both structures studied. In particular, for the four-storey building a comparison of the outcomes reached for all the non-linear static analyses performed and for the non-linear dynamic analyses are presented.

4 storey frame system

Figure 7 represents the capacity curve, base shear *vs*. control node displacement (top displacement), using three different lateral load patterns. This curve gives important properties of the structures, such as the initial stiffness, the maximum strength and yield global displacement. From Figure 7 it can be seen that the uniform load produced larger shear forces the same displacement values.

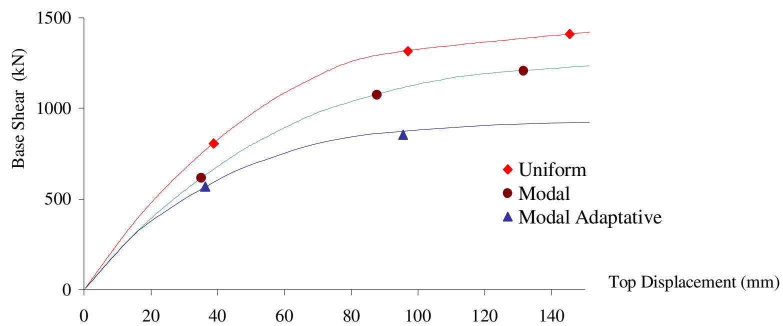

Figure 7. Capacity curves for Uniform, Modal and Modal Adaptative lateral load pattern

Table 1 shows the top displacements, drift (the maximum value occurs always in the second floor) and the base shear for all the methodologies performed. Based on the results presented in Table 1, it seems that the methodologies lead to similar maximum displacements for a 0.3g seismic action; however, the base shear differs significantly. In general, the results are similar and the N2 results are the closest to the non-linear dynamic analysis (NDA). The NDA, as expected, gives similar values when compared to the pseudo dynamic (PsD) test results [15].

Table 1. Summary of the various analysis methodologies – 4 storey building

	NDA	ATC40	FEMA-273		N2
			Uniform	Modal	
Top displacement [mm]	119.43	95.58	97.00	87.74	100.69
Drift	1.29	1.05	1.11	0.96	1.11
Base Shear [kN]	1190.2	854.8	1316.3	1075.1	1167.9

The detailed distribution of story displacements, story shear and the interstory drifts are presented in Figure 8. The CSM, the DCM and the N2 evaluation techniques lead to similar results, as far as the displacements are concerned.

The evaluation techniques exhibit quite different results while comparing the story shear distribution. This can be attributed to the nature of lateral load pattern used. Even so, the N2 results show similar distribution pattern to the NDA results except for the higher story where they differ significantly. It can be understood from the Figure 7 that the modal adaptive load pattern results in the least base shear value (only the first mode contribution is considered in the modal adaptive load pattern). Similarly, the interstory drifts also differ in magnitude, however maintaining similar profiles in all analysis cases.

Based on the results in Figure 8, it is observed that the response of the structure is sensitive to the shape of the lateral load distribution. It is also observed that the story shear values using the uniform load pattern overestimates the maximum base shear in the building (compared to non-linear dynamic methodologies), apparently providing a conservative prediction of base shear seismic demands.

From Figure 8, it can be observed that the maximum inter-story drift ratio is always attained at the second story in all methodologies. This could be attributed to the sudden decrease section capacity in that region (there is a variation of column longitudinal reinforcement between the first and the second story).

The nonlinear dynamic analysis (NDA) achieves the largest value of inter-story drift ratio: 1.29% for the PGA of 0.30g considered. The maximum drift value for the design earthquake (1.29%) is in the range for the maximum total drift defined in ATC-40. It is also observed that the maximum inter-story drift ratio occurs at the second story level for all levels of seismic action, indicating a probable soft story mechanism at this level.

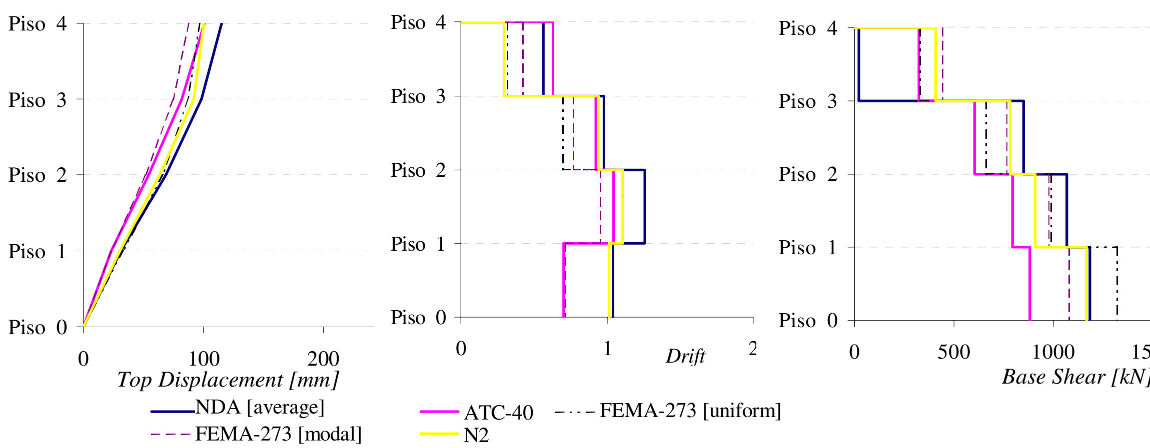

Figure 8. Story displacement, inter-story drift and story shear for all the methods

8 storey frame system

Figure 9 presents the curves base shear *vs.* top displacement for the eight-storey structure, using uniform, triangular and modal lateral load pattern. As noticed with four-storey structure, the response is sensitive to the load pattern adopted and the uniform load produced larger shear forces for smaller displacements. The differences between the triangular and the modal capacity curves are minimum as the first mode is very close to a triangular distribution. In Figure 10 the three first modes are represented.

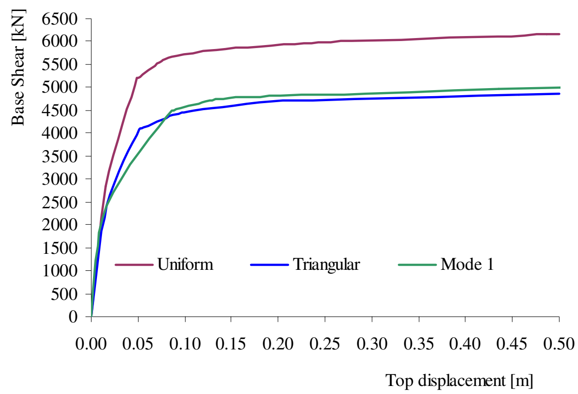

Figure 9. Capacity curves for Uniform, Triangular and Modal lateral load pattern

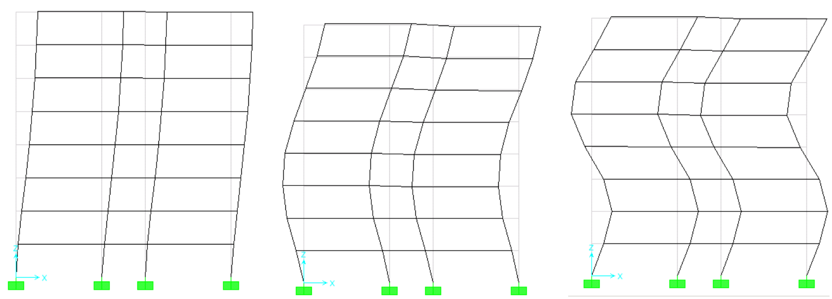

Figure 10. Modes of vibrations: 1st, 2nd and 3rd, respectively

Intending to overcome the shortcomings of the non-linear static analysis regarding the longer period structures, it is decided to perform, for the eight-storey building, a Multi-Mode Pushover procedure (MMP) using the N2 method. The MMP follows the procedures of a typical pushover analysis except the load patterns are based not only in the first mode but also in higher modes. In this work the first three modes are considered. The load patterns are based on the three elastic mode shapes (Figure 10) and are defined by multiplying the mass at each level by the mode shape. Having defined the load patterns for each mode, a pushover analysis is performed for each load pattern and the correspondent capacity curves defined (Figure 11). Then the MMP procedure used in this work pursues the traditional N2 method for each load pattern. Finally the total response (demand) is evaluated by combining the peak 'modal' response using the SRSS rule.

The MMP procedure adopted in this study was based in the work of Chopra and Goel [16] but this procedure still need to be evaluated for a wide range of buildings and the different results compared with non-linear dynamic analyses to verify the precision of the method. Nevertheless, some limitations could be already addressed to this procedure, in particular the fact of considering the 'addition' of different non-linear responses of the structure.

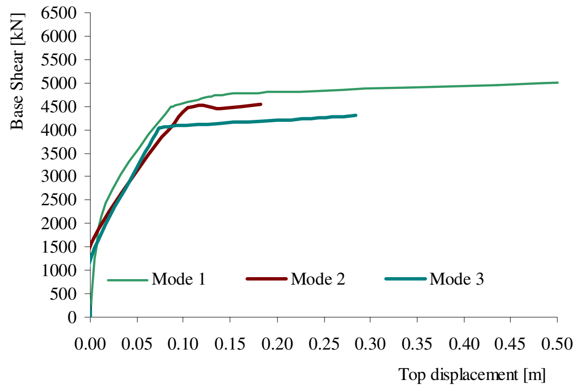

Figure 11. Pushover curves for the 1st, 2nd and 3rd modes

Figure 12 shows the distribution of the story displacement, the inter-story drift and story shear for the 0.3g design ground motion considered. This figure shows the differences between the results of the different pushover analyses for the N2 method used. It is clear that the response of the structure is sensitive to the shape of the lateral load distribution. Regarding the shear story values it is observed that the uniform load overestimates the maximum base shear in the building, providing a conservative prediction of base shear seismic demands. The differences between the triangular and the modal (only with mode 1) are minimum, as expected.

As shown in Figure 12, the maximum top displacement is 80 mm for the modal load pattern (mode 1) and the maximum drift (1.57%) is reached in the third floor for the same lateral load distribution. For this structure small changes are noticed between the modal lateral load distribution where only the first mode contribution is considered with the MMP procedure which includes the effect of the first three modes.

It is also observed that the story shear values using the uniform load pattern leads to the maximum base shear value in the building (5700 kN), apparently providing a conservative prediction of base shear seismic demands.

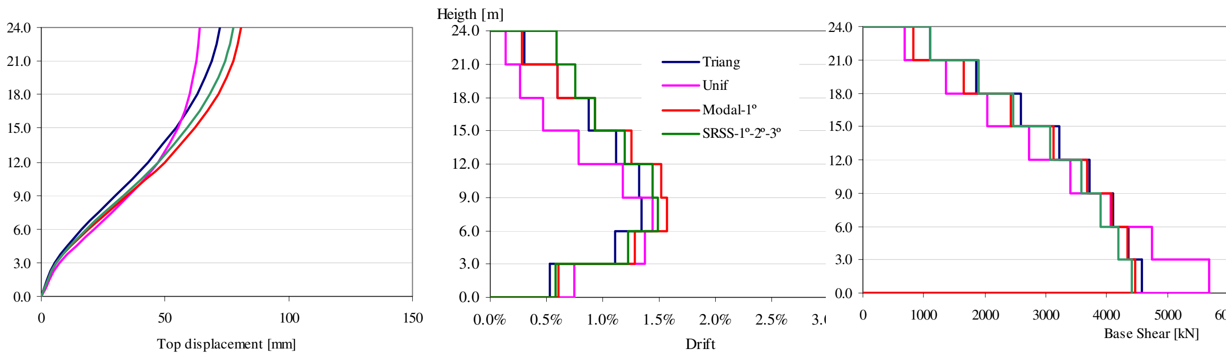

Figure 12 Story displacement, inter-story drift and story shear using the uniform, triangular, modal (mode 1 and mode 1+2+3)

DISCUSSION: ADVANTAGES AND LIMITATIONS

The pushover analysis is an effective tool for the performance evaluation of a structural system, by estimating its strength and deformation demands induced during a seismic event, by means of a static non-linear analysis. The demands are then compared to available capacities at the performance levels of interest. The evaluation is based on assessment of important performance parameters such as global drift, interstory drift and inelastic element deformations (either absolute or normalized with respect to a yield values). The pushover analysis can be viewed as a methodology for predicting seismic force and deformation demands, which can account for, in an approximate manner, the redistribution of internal forces occurring when the structure is subjected to inertia forces that no longer can be resisted in the elastic range. The main advantages of pushover analysis over the linear methods (Linear Static and Linear Dynamic analysis) are:

- The design is achieved by controlling the deformations in the structure;
- Consideration of the non-linear behavior, which avoids the use of behavior coefficients (reduction factors), that can not be rigorously assessed;
- Allows tracing the sequence of yielding and failure on the member and the structure levels, as well as the progress of the overall capacity curve of the structure;
- Its applicability to performance-based seismic design approaches as it permits different design levels to verify the performance targets.

Nevertheless, some unresolved issues that need to be addressed through more research and development:

- The techniques in this methodology incorporate the use of an effective nonlinear model to represent the global structure. In some cases, initial stiffness is used, as is the case, for instance, of the DCM method, while the secant stiffness is used by the CSM, to represent the initial condition of the structure. Care should be taken on the selection of this stiffness as some parameters, further in the procedures, are based on the initial stiffness of the structure;
- Selection of appropriate load distribution is crucial to predict accurately higher mode effects in the post-elastic range, mainly if they play an important role in the structural response. The modal adaptive pattern is thought to provide better results as they account for the inelastic response by suitably adjusting the load pattern based on the mode shape in the previous step. Nevertheless, other load patterns proposed can also be considered with caution;
- Incorporation of torsional effects, due to mass, stiffness and strength irregularities, is difficult to account for, without a 3-dimensional analysis. However, this has a great inconvenience in the definition of a lateral load pattern in both the direction. An alternative could be in performing independent analysis in both directions;
- Some researchers prefer to use site-specific spectra.

CONCLUSIONS

Three non-linear static procedures are used for the seismic assessment of a four-storey reinforced concrete structure and the N2 method chosen as the non-linear static procedure for the seismic assessment of the eight-storey building. Some conclusions could be addressed regarding the structures and the results obtained:

- The modal lateral loads (including only the mode 1 or the first three modes) show similar results, and for the structures studied the modal pattern (defined for the first mode) is very close to a triangular pattern;
- The uniform load pattern seems to indicate conservative results regarding the base shear evaluation but they may be misleading in some cases.

Concluding, and regarding the non-linear static analysis, one can say that:

- More appropriate for low rise and high frequency structures, i.e. for structures that vibrate primarily in the fundamental mode;
- It may expose design weaknesses that may remain hidden in an elastic analysis, such as: weaknesses due to story mechanisms, excessive deformation demands and strength irregularities;

- It is not able to represent accurately dynamic phenomena, without the use of more sophisticated lateral load patterns;
- It is not possible to account for phenomena like stiffness and strength degradation, P-Δ effects and the duration of the seismic action;
- It may not detect some important deformation modes that may occur in a structure subjected to severe earthquakes, and it may exaggerate others;
- Inelastic dynamic response may differ significantly from predictions based on invariant or adaptive static load patterns, particularly if higher mode effects become important.

Because of some of the pushover analysis's limitations referred previously, sometimes it is necessary to use the non-linear dynamic analysis (time-history analysis) as a verification tool at this developmental stage. Nevertheless, there are still some reservations to adopt this method, which are mainly related to its complexity and suitability for practical design applications. Another limitation of the non-linear dynamic analysis is its sensitivity to the characteristics of the input motions and thus selection of representative acceleration time-histories is fundamental. Besides, the hysteretic behavior of all the critical sections should be carefully defined. Finally, the efforts in computation and assimilation of results, contribute to the methods aloofness from practical design utilization.

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