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SEISMIC PERFORMANCE OF METALLIC BRACED FRAMES BY PUSHOVER ANALYSES

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Abstract. *A preliminary investigation is presented on a pushover analysis used for the seismic performance of metallic braced frames equipped with diagonal X-bracing and K-bracing systems.*

Three steel frames were analysed corresponding to 3, 6 and 10 floor regular buildings. The frames were modelled in the MIDAS/Civil finite element software and in the analyses non-linear static methods were used to obtain the pushover curve.

The principal objective of this article is to compare the evaluation of the structural performances of these buildings with respect to the proposed N2-method, and so also of the consequent convenience of using pushover methodology for the seismic analysis of structures.

1 INTRODUCTION

The recent role of performance based design led to the development and use of methods based on non-linear analysis, namely on the so-called pushover analysis. This analysis is based in a non-linear static analysis in which the magnitude of the structural load is increased according to a certain predefined pattern. With the process of increasing the magnitude of the loading, can be found week connections, sections or collapse modes of the structure. The loading is considered monotonic and the effects of the cyclic actions and strain reversals are calculated using a modified force-deformation approach with certain approximations for taking damping into account. This methodology is used in structural engineering applications to evaluate the capacity of the structure, and the actual practice indicates that it constitutes an useful and effective tool as a design methodology based upon performance based design principles.

This last fact, allied with the existence of significant scientific information on this type of analysis, somehow justifies the work developed by the authors in validating the use of pushover analysis in the design of braced frames and also of frames with base isolation devices [1]. For this study three steel frames were analyzed with 3, 6 and 10 floors for two different situations: frames without any bracing system; braced frames equipped with metallic diagonal braces. The frames were modelled using the finite element software MIDAS/Civil [2] and in the analyses were used the static non-linear pushover methods (based on the methodologies presented in this article). The main objective of this work consists on the comparative evaluation of the structural capacities and performances and therefore also in the convenient use of the pushover method in the seismic analysis of structures that, in an afterword phase will be equipped with base isolation devices with specific hysteretic behaviour [1] [3].

2 NON-LINEAR STATIC ANALYSIS

2.1 Introduction

In the design of structures under seismic actions several methodologies can be used, with distinct accuracy, to describe the structural seismic response. The non-linear dynamic analysis is the most realistic methodology and is based on the timely variation of the structural behaviour (of the materials and of the geometry) – therefore including material and geometric nonlinearities – under seismic actions. Although this methodology is the most accurate, its non-linear characteristics require knowledge of the structural behaviour and inherent theoretical developments and it also demand costly computational resources; such conditions are not often timely compatible with the design procedure besides the fact that most of the design do not justify the application of such elaborated models.

However, the design engineers need intuitive tools to determine the structural response under seismic actions, in particular for those that are strongly conditioned by dynamic actions. In this sense, several researchers try to develop simplified analysis and design methodologies based on non-linear analyses, for the determination of the structural response and that can be routinely used by the structural designers.

Concisely, the methodologies for analysis of buildings under seismic actions can divided in linear procedures and in non-linear procedures [4]. The linear procedures include the Linear Static Procedure (LSP) and the Linear Dynamic Procedure (LDP); the non-linear procedures include the Non-Linear Static Procedure (NLSP) and the Non-Linear Dynamic Procedure (NLDP).

The linear procedures use linear stress-strain relationships and introduce corrections associated to the building global deformation and to the behaviour of materials – indirectly incorporating the non-linear dynamic response – to obtain a credible seismic behaviour. These procedures are valid for regular structures (in plan and elevation) or for buildings in which the structural response is very close (or just deviates a few) to the elastic domain.

When the structures present strong irregularities or when the response occurs significantly in the non-linear domain, non-linear analyses (static or dynamic) should be used. Although the NLSP was initially applied to structures that did not present great sensibility for higher modes, some multimode load patterns have been developed and applied [5] [6]. Successful applications of this technique have been obtained for asymmetric structures with displacement dependent passive energy dissipation devices [7] [8].

These NLSP are non-linear static analyses of the structures with control of displacements and imposed loading (or pushover) that allow controlling the magnitude of the structural displacements and evaluating the seismic performance of the structure. The work developed herein is based in this pushover analysis [9]. Although this procedure is more correct than the procedures based on a linear analysis, its applicability should be thought carefully since it does not present sufficient sensibility to capture changes in the structural response as the stiffness degrades or when higher modes are also predominant in the response.

Finally, the NLDP are the ones that best represent the seismic behaviour and performance of the structures. Because of its realistic nature its applicability becomes complex, either for the calculation numerical processes and the computational resources involved, or for the necessary experience sensibility and advanced knowledge of the design engineer namely in the definition of the constitutive models.

2.2 Pushover Analysis

The pushover analysis is a simplified methodology to obtain the structural response to seismic actions through a non-linear static analysis. This analysis evaluates the performance of the structures through control of its displacements (at local and global levels), still giving information about the ductility and the resistant strength capacity.

FEMA 356	EC 8 (2003) – Method N2
• Modal* (fundamental mode) $$F_i = \frac{m_i h_i^k}{\sum_{j=1}^{n} m_j h_j^k} F_b$$ F_b – basal shear F_i – inertia forces at floor level i m_i – mass of floor level i h_i – coefficient associated with fundamental mode (height of floor level i) k=1.0 for T<0.5s k=0.75+T/2 for 0.5≤T<2.5s k=2.0 for T≥2.5 *Can be multimodal (association of 3 first modes, as proposed by Chopra and Goel) • Uniform (see detail under EC 8)	• Modal $$F_i = \frac{m_i \phi_i}{\sum_{j=1}^{n} m_j \phi_j} F_b$$ F_b – basal shear F_i – inertia forces at floor level i m_i – mass of floor level i ϕ_i – modal coefficient at floor level i • Uniform $$F_i = \frac{m_i}{\sum_{j=1}^{n} m_j} F_b$$

Table 1: Loading Pattern: FEMA 356 and EC 8.

These methods constitute a progress with respect to the methods associated to the linear behaviour (or with modified response spectra) because they are based on a more precise determination of the distribution of resistant capacity (ductility-yield) in a structure (instead of assuming a uniform ductility throughout).

As already referred several methodologies exist associated to this type of analysis, among which the following: (i) the method proposed by the Applied Technology Council (ATC) in the report ATC-40 [10] for the analysis of concrete structures, based in simplified pushover methods (Method of the Spectrum of Resistant Capacity); (ii) the method proposed by the Federal Emergency Management Agency (FEMA) in the regulations FEMA 356 [11] and FEMA 273/274 [12], that present the guidelines for the design and seismic rehabilitation of buildings through pushover analysis (Method of the Displacement Coefficient); (iii) the method N2 proposed in Eurocode 8 (EC 8) [13] and whose algorithm is comparable to the one proposed in FEMA 273/274 [12].

The resolution algorithm for this type of pushover analysis includes three stages: (i) defining the resistant capacity of the structure through the application of horizontal incremental loads (or displacements); (ii) determining the seismic action and the response of the structure based on a non-linear behaviour to establish the level of objective proposed performance that consists in the determination of the objective displacement (target displacement) or of the point of seismic performance (performance point); (iii) evaluating the performance of the structure for the predetermined loading level (corresponding to the target displacement or to the performance point).

One of the most critical phases of this process is the definition of the lateral loading to be applied to the structure, which can be either constant in height or associated with the vibration modes, the number of floors, etc. The loading pattern (Table 1) should be selected so that the final performance of the structure really translates its response.

A loading associated to the fundamental mode cannot satisfy this approach if the structure presents a behaviour governed by higher mode effects; or still, when a constant loading is applied along height that is non conforming with the stiffness distribution (and consequently of the yielding pattern) along height. Notice that the structural loading can be force-controlled (that is, applied previously until a predefined limit) or displacement-controlled (that is, the lateral load is applied until a certain lateral displacement is achieved). Usually, the gravity loads are force-controlled and the lateral loads are displacement-controlled.

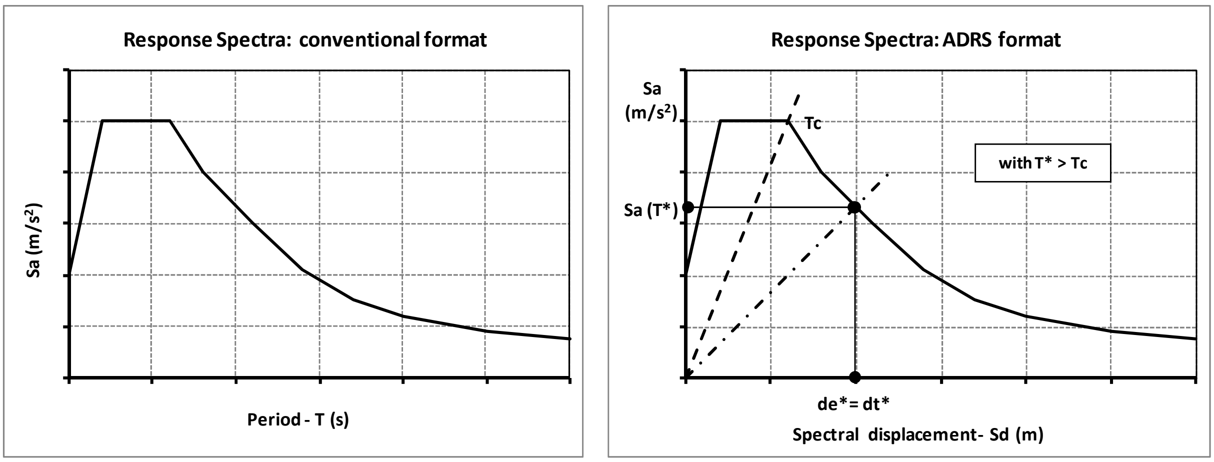


Figure 1: Response spectra: conventional format and ADRS.

The implementation of these types of analyses on the new regulations for structural design in Portugal imposes the need to study and to validate these methodologies.

In the case of EC 8, the proposed methodology is based on the method *N2* [13] [14] whose spectrum representation presents the spectral values of the acceleration in function of the spectral values of the displacement; that is to say, it is presented in the format Acceleration Displacement Response Spectrum (or ADRS).

Concisely an algorithm is presented for application of this methodology, which is composed by the following phases:

(1) The structure is modelled and the constitutive relations are selected to define the behaviour of the materials. The seismic action is defined according to the regulatory design criteria;

(2) The loading pattern should contemplate at least two distributions: modal and uniform. In the modal case, an acceleration distribution is assumed proportional to the fundamental mode and the inertia forces F_i at each floor level i are given by:

$$F_i = \frac{m_i \phi_i}{\sum_{j=1}^{n} m_j \phi_j} F_b \tag{1}$$

In this equation *Fb* is the basal shear, mi is the mass associated to the floor i and $Ø_i$ is the modal coefficient associated to the floor i. If the fundamental mode is considered linear, then the modal coefficient is proportional to the height of the floor (h_i) and the equation will be:

$$F_i = \frac{m_i h_i}{\sum_{j=1}^{n} m_j h_j} F_b \tag{2}$$

Finally, the curve of the resistant capacity is determined from the basal shear in function of the maximum displacement d_{max} (in the top of the building) through the progressive application of the lateral load pattern until the required performance is reached associated to the maximum displacement.

(3) The initial structure has to be treated as a SDOF system, since its response is obtained from the response spectra. Thus, the resistant capacity is determined for an equivalent SDOF. The transformation, to convert the capacity curves for an equivalent SDOF, is done through the following relationship:

$$F^* = \frac{F_b}{\Gamma} \tag{3}$$

$$d^* = \frac{d}{\Gamma} \tag{4}$$

whose transformation factor Γ is given by:

$$\Gamma = \frac{\sum_{j=1}^{n} m_j \phi_j}{\sum_{j=1}^{n} m_j \phi_j^2} \tag{5}$$

It is necessary to simplify the capacity curve for an elasto-perfectly plastic regime (Figure 2). In this graph F_y* represents the resistant strength capacity of the system with an equivalent SDOF and d_y* represents the idealized yield displacement of the equivalent SDOF system. With these values the elastic period T, of the equivalent SDOF system, is determined.

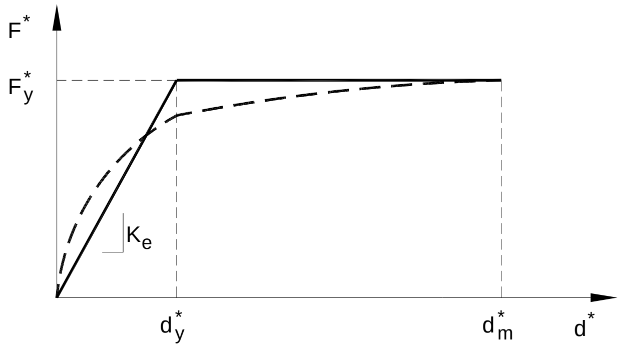


Figure 2: Idealization of the capacity curve (EC 8).

(4) The idealized target displacement dt* is determined, depending on the dynamic characteristics of the equivalent SDOF system, enabling to quantify the seismic response of the idealized equivalent SDOF system and to obtain the seismic performance of the equivalent SDOF system.
For medium and long periods, such that $T \geq T_c$

$$d_t^* = S_a \frac{T^2}{4\pi^2} \tag{6}$$

For low periods, such that $T < T_c$, and inelastic behavior

$$d_t^* = S_a \frac{T^2}{4\pi^2} \frac{1}{q} \left[1 + (q-1) \frac{T_c}{T} \right] \tag{7}$$

with

$$q = \frac{S_a}{\left(F_y^* / m^* \right)} \tag{8}$$

and

$$m = \sum_{j=1}^{n} m_j \phi_j \tag{9}$$

In these equations: T is the period of the equivalent SDOF system, S_a is the spectral acceleration for the period T, and T_c is the period that defines the transition between constant velocity and constant acceleration (defined in the ADRS format response spectra of Figure 1, as the intersection between the straight line and the descending branch of the graph S_a vs S_d).

(5) After determining the performance point of the equivalent SDOF system it is necessary to determine the seismic performance of the structure. The loading pattern is applied until the maximum displacement d_{max} is reached, whose value is obtained multiplying the target displacement by the transformation factor Γ.

(6) Finally, the resistant capacity and the floors drifts are verified, for a target displacement $d_t \leq d_{max}/1,5$.

Another very important aspect in this pushover analysis is the definition of the material model that is used to simulate the ductility of the structural members of the complete structure. Figure 3 presents the simplified force-deformation relationship used to model the beam elements or columns, and the deformation criteria (for actions controlled by deformation) for the several materials used [4] [15].

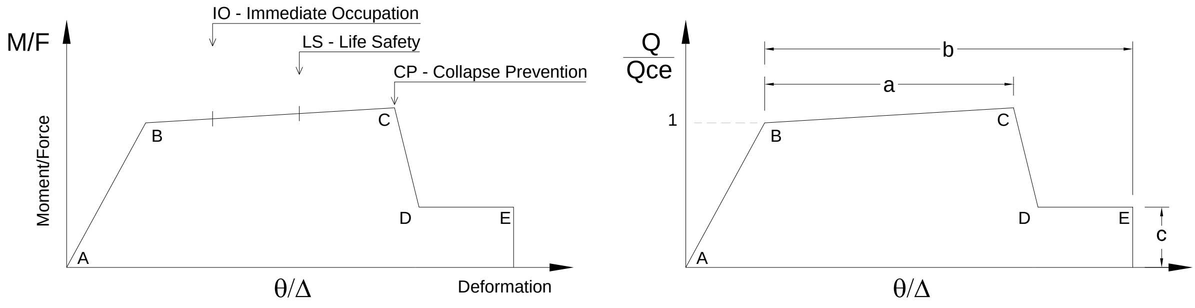


Figure 3: Constitutive relationship for pushover analyses (FEMA 356).

In the first line AB a linear response is shown with a yield point at B. The inclination of the second line BC is usually low (0 to 10% of the value of the inclination of the elastic regime AB) and it represents some hardening. The third line CD represents the degradation of the resistant capacity while the line DE corresponds to the plastification of the structural element.

The criteria of acceptable deformation is also included by appropriate deformation ratios for primary elements (P) and secondary elements (S), which are also presented qualitatively in Figure 3 for three safety levels: Collapse Prevention (CP), Life Safety (LS) for the human life and Immediate Occupation (IO) for usefulness or serviceability of the structure.

The values attributed to each point of the curve vary in function of the type of structural element, and they still depend on other parameters as specified in the ATC-40 and in the FEMA-356. In simple framed structures the non-linear behaviour occurs in sections or nodes that can be previously identified and introduced in the calculation model through hinges with non-linear behaviour defined with material characteristics as represented in Figure 3.

3 APPLICATION OF THE NON-LINEAR STATIC ANALYSIS (PUSHOVER)

3.1 Pushover analysis through the software MIDAS

The pushover analysis presented in this section is based in the algorithm included in the software MIDAS/Civil. The application of this software in the determination of the seismic performance of structures is validated by several researchers whose works served as base for the definition of the models presented in this study [1].

In agreement with the criteria of seismic design, the forces induced in the structure during a high intensity earthquake surpass the yield limits causing great inelastic deformations. These deformations, caused by a combination of gravity loads and lateral loads, are located in the zones that possess larger internal forces and that constitute the so-called critical zones in which occurs energy dissipation through plastification mechanisms.

The plastification mechanisms should represent conveniently the capacities of the resistant elements, especially with respect to the capacity associated to non-linear deformations namely the rotation capacity. Such elements should not present a significant loss of resistance for larger deformations. The designer should conceptualize and define the seismic-resistant members and select construction dispositions that guarantee the correct formation of the plastic hinges in the places chosen previously.

The analysis process begins with the elaboration of the 2D or 3D structural model and later on defining the location of the plastic hinges and the criteria associated to their behaviour. The software includes a data base with several predefined behaviours (according to FEMA 273/274) and it still allows the introduction of bilinear and tri-linear relationships defined by the user. Although these predefined characteristics can be useful in a preliminary analysis, the designer should validate its applicability for final analyses (preferentially through experimental validation).

The hinges are defined through constitutive relationships as presented in Figure 3 by the diagram moment-curvature, which translates the no-linear behaviour expected at each plastification section. The hinge-type chosen for column locations (usually in the column extremities) has to consider the interaction between the axial force and the bending moments (P-M2-M3), but the interaction surface is user-defined. In the case of MIDAS/Civil it is possible to define an interaction surface or allow that the software calculates the envelope through the characteristics of the structural members. The beam members are simulated just considering the contribution of the bending moment (M3), locating the bending hinges at appropriate sections in the extremities of the members.

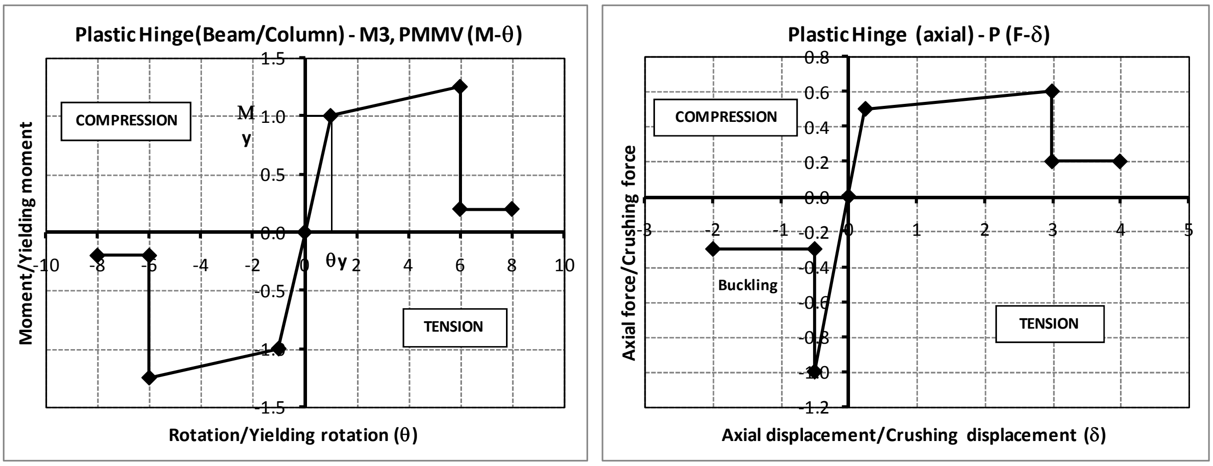


Figure 4: Plastic hinges: (a) bending hinge (M/My vs θ/θy); (b) hinge also under axial force.

When stiffening walls exist, which introduce significant rigidity in the structural global assemblage, they should be incorporated in the calculation model to obtain a model leading to a realistic capacity curve. The contour walls (even if they do not present any resistant function) can be simulated through a model of equivalent connecting rods with a behaviour defined by a shear plastic hinge located in the center of the wall panel. In the case of an experimentally verified yielding of another type, as the crushing failure of the compression connecting rod, a model should be used that represents such behaviour including the instability effect by axial compression (corresponding to a model of equivalent connecting rods with a flexure hinge, also with axial force, located in the diagonals) [16].

3.2 Description and characterization of the application models

In this section are presented the models used to study the seismic behaviour of several structural systems. The purpose of this work is to study three framed structures (regular in plan and elevation), constituted by metallic steel members (making 3, 6 and 10 floors structural frameworks) through a pushover analysis.

The structures were modelled using the commercial FEM package MIDAS/Civil, and for each structure three structural solutions (one solution without bracing and two bracing solutions) were considered: (i) the structure is built without any bracing element; (ii) the building presents glass facades and a bracing system composed by diagonals in X-braces; (iii) the building presents glass facades and the bracing system, by architectural or by strength reasons, is constituted by K-braces.

The first structural case intends to simulate the occurrence of an earthquake during the construction phase, considering that the framed structure is totally built but it does not possess other bracing or stiffening members. The second and the third structural cases intend to simulate construction options enforced by architectural or by strength requirements.

Therefore the bracing system is constituted by metallic steel elements (diagonal X-braces or K-braces) placed at the corners of the peripheral 2D-frames of the 3D-building. The pushover analyses were done to evaluate the capacity of these 2D-frames, since global structural capacity depends on the resistant capacity of these substructures especially for the structural model adopted in which the slabs are represented as infinitely rigid diaphragms (simulated through the functionality "Rigid Link-Plane" in MIDAS/Civil).

The results of the two structural cases (ii) and (iii) were compared with the results obtained in the case (i) when the resistant structure is only composed by a skeleton of beams and columns. In any case of the structural layouts (Figure 5) the analyzed structures present regularity both in plan and in elevation.

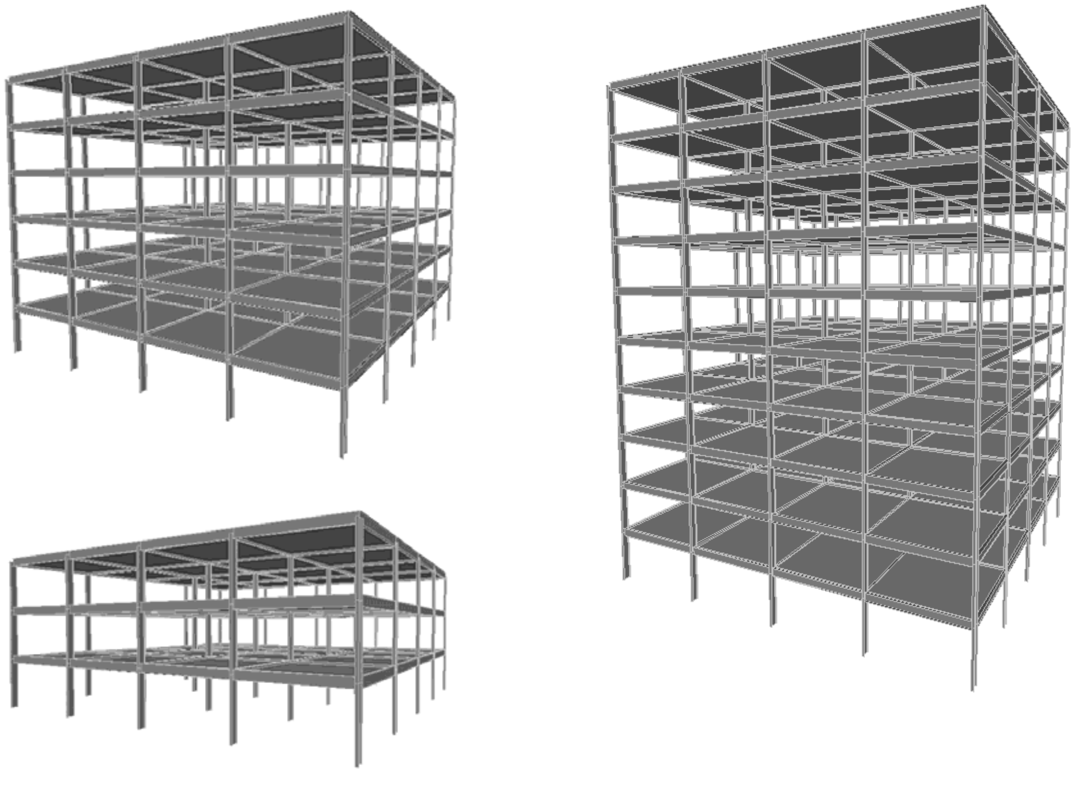
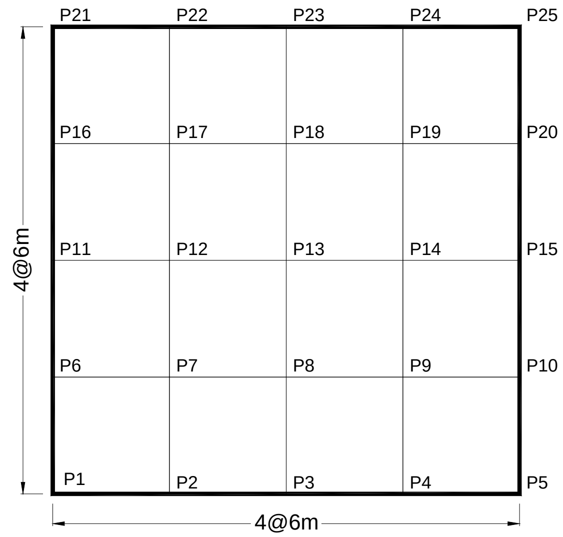


Figure 5: Structural outlines of the buildings analyzed in this study (3, 6 and 10 floors).

The structural elements were predesigned in agreement with Portuguese design code RSAEEP and its characteristics are synthesized in Figure 6. The seismic design was elaborated in agreement with the EC8 criteria for soil type B and a damping ratio of 5%. For the quantification of this action two elastic response spectra were used, associated with seismic actions of type 1 (moderate earthquake at short focal distance) and type 2 (strong earthquake at higher focal distance) in seismic zone 1 defined in the proposal of the EC8 National Annex of 2006 for Portugal.



Characteristics of the resistant elements

Beams: HE 320A

Columns:

- Frame with 3 floors: 3 floors with HE 300B
- Frame with 6 floors: 3 floors with HE 500B +
  3 floors with HE 300B
- Frame with 10 floors: 2 floors with HE 900B +
  2 floors with HE 800B + 3 floors with HE 500B +
  3 floors with HE 300B

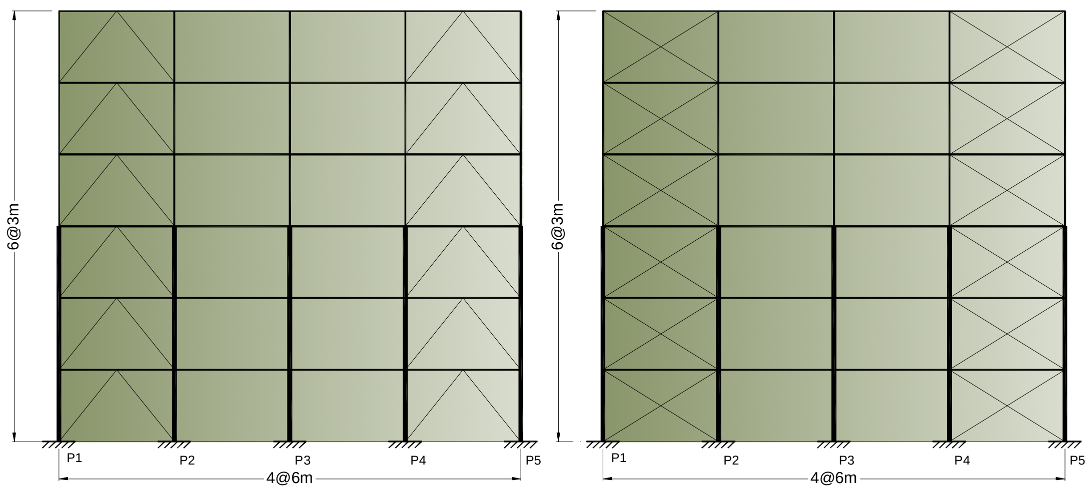
Floor mass: 480 ton
Roof mass: 370 ton

Figure 6: Plan view and characteristics of the buildings resistant elements.

Relatively to the distribution of the lateral loads, two patterns of lateral loading were considered: uniform distribution, defined as a constant acceleration; proportional distribution to the first vibration mode associated with the floor masses.

The gravity loads (G) include the own weight of the beams, columns and slabs and still the walls (external and interior partitions) floor coverings and wall coatings and revetments. The load of the wall partitions was 2 kN/m2 and still was considered a life-use overload (Q) of 3 kN/m2. The mass was calculated through the combination of actions: G + $\psi_2 \times$ Q, with ψ_2=0.4 (accounting for reduction in the live load mass, when earthquake occurs).

The metallic steel members are of the class S 275 and they present an elastic longitudinal modulus E=210 GPa. The steel braces used in the structural solutions (ii) and (iii) are commercial profiles HE 100A and bracing schemes are shown in Figure 7.



a) b)

Figure 7: Elevation of the periphery frames: (a) K-bracing; (b) diagonal X-bracing.

In this study, the procedure to model the critical zones of the resistant elements was the following: (1) plastic hinges of simple bending (M), in the extremities of the beams; (2) plastic hinges of deviated composed biaxial bending (P-M2-M3) in the extremities of the columns; (3) plastic hinges of axial force (P) in the diagonals of metallic steel bracings.

Figures 8, 9 and 10 represent the curves of resistant capacity of the frames (3, 6 and 10 floors) characterizing the performance of the structures for two schemes of distribution of the lateral loads (uniform and proportional) acting on the three structural configurations (without bracing; with X-braces; with K-braces).

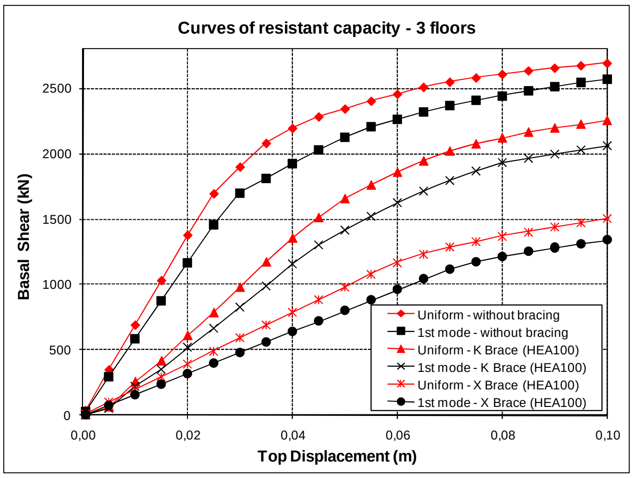


Figure 8: Curves of resistant capacity: Basal shear vs Top displacement (3 floors).

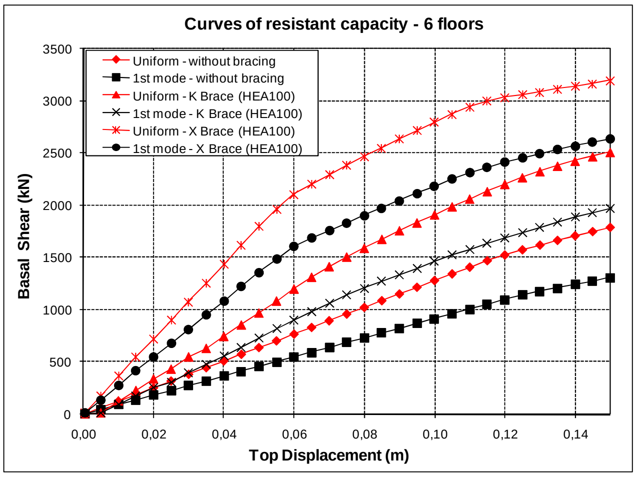


Figure 9: Curves of resistant capacity: Basal shear vs Top displacement (6 floors).

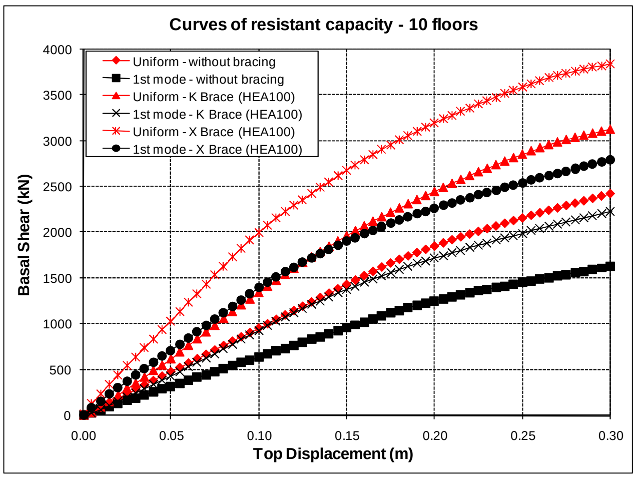


Figure 10: Curves of resistant capacity: Basal shear vs Top displacement (10 floors).

In these curves it is possible to identify several important parameters in the seismic response of the analyzed structures, namely the yielding displacement and the stiffness variation with the increase of the load. This representation still supplies information about the non-linear behaviour of the structure.

In a first observation it is verified that the resistant capacity depends on the loading pattern. For the analyzed structures, a uniform distribution of lateral loads led to higher resistant capacity. This effect is more significant with increasing number of floors. Notice that, in a simplified manner, the distribution associated to the first mode can be substituted by a triangular distribution since such configurations are very similar.

As it would be expected the worst performance corresponds to the un-braced structures; for the same top displacement of each of the three un-braced frames, corresponds a lower basal shear comparatively to the braced structural configurations.

When bracing members are added, the corresponding structural configurations have higher resistant capacity. The K-bracing system and the X-bracing system are modelled as pin-articulated struts and yield by axial force. The K-bracing configuration although stiffer (than configuration without braces) did not provide as high resistance as with the X-bracing configuration. The percentual increase of resistant capacity, when including a bracing system, decreases with increasing number of floors. This unexpected behaviour is associated to the reduced section of the strut bracing members (higher sections are generally used) and to the yielding model used for the hinges (yielding by axial force).

In fact structural solutions using K-braces were analyzed for several yielding modes of these bracing members (hinges P, hinges M and hinges P-M2-M3) to determine an inferior limit in the collapse mode of these elements; that was obtained with hinges of axial effort, the ones that conditioned these results in the present study. The worst performance of the K-bracing system, as compared with the diagonal X-bracing system, is associated with a premature yielding by axial force at the connection with the slab.

The resistant capacity of the structure depends on the structural configuration model (used and idealized by the structural designer) and the yielding of the structure – whose collapse mode is linked to the choice of the critical zones and to the yielding relationship – cannot correspond to the real collapse mode if the adopted non-linear behaviour does not correspond to the effective behaviour.

Because of that parametric studies are needed to identify the critical zones associated to bracing system configurations and the corresponding yielding modes, as form of validation of the structural models used in the calculation programs of various analysis and design software. This information is of paramount importance for the seismic design of structures. According to some researchers, this type of analysis is more adequate for low-rise structures and those that present higher frequencies.

Figures 11, 12 and 13 represent the spectra of resistant capacity of the frames (3, 6 and 10 floors) and the response spectra in the format ADRS, for each structural configuration (as regards to stiffening bracing system). In these graphs the response spectra were evaluated according to EC 8, for soils A-B-C-D and E, damping ratio of 5% and peak ground acceleration of 0,25g.

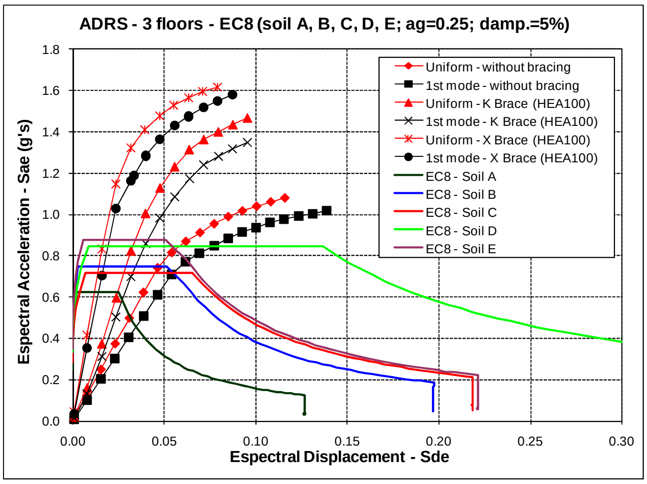


Figure 11: Capacity spectra and response spectra ADRS (frame with 3 floors).

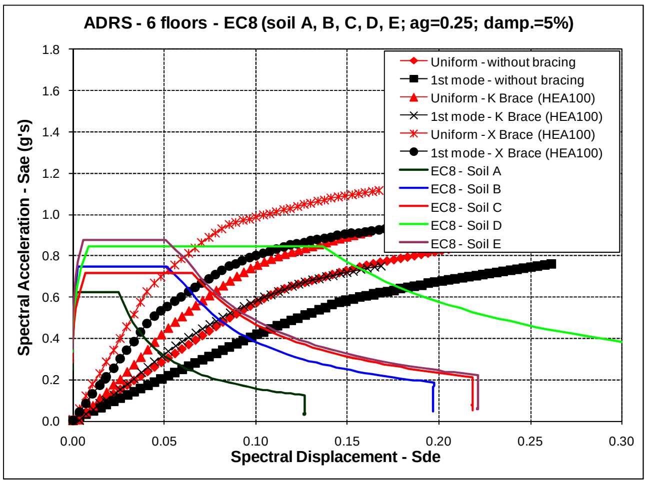


Figure 12: Capacity spectra and response spectra ADRS (frame with 6 floors).

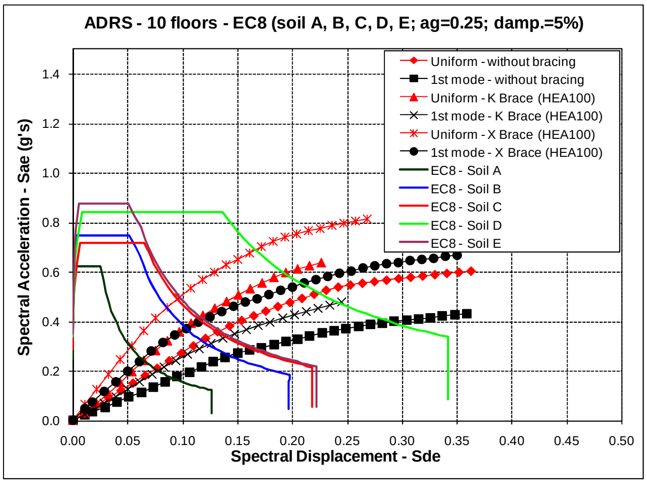


Figure 13: Capacity spectra and response spectra ADRS (frame with 10 floors).

Although the capacity curves shown in these figures, that represent the MDOF systems instead of the equivalent SDOF system, do not allow computing the performance point related with the *N2* method, some relevant conclusions can be obtained after comparing the relative performance of each structural system.

The transformation of the MDOF system into an equivalent SDOF system is based on the application of a reduction factor, which even without being constant for every case causes an expected overall reduction in capacity. In this context, the 3-floor frames present building seismic responses associated to low periods (T<Tc) and the 10-floor frames present seismic responses associated to medium or long periods (T>Tc) in agreement with criteria of method *N2*. This behaviour can be expected in the equivalent SDOF system since there is a general reduction in all capacity curves as mentioned before. Also, the increase of number of floors implied an increase in the spectral displacement and a decrease in the spectral acceleration.

The next step is the transformation of the MDOF capacity curve to an equivalent SDOF in order to compute the performance point in the response spectra. This new curve is obtained applying the transformation factor and the desired point can be determined through the intersection of this new resistant capacity curve with the response spectrum. Such methodologies for the determination of the seismic performance point are addressed in the context of ATC-40 and the method *N2* (in EC-8) and, as explained earlier, are based on the definition of the point of seismic performance of an equivalent SDOF system (1-EDF system).

Although the study that was carried out involved the analysis of all structural systems, only the system of 3 floors frame without bracing will be addressed herein to exemplify the purpose of this procedure (Figure 14); its application is similar for all other cases, according with the methodology for low and long periods as described in method *N2* (in EC-8).

To perform the MDOF system reduction to an equivalent SDOF system it is necessary to determine the transformation factor Γ, expressed by equation (5). This factor is a function of the structural mass and the modal configuration associated with the 1^{st} vibration mode. After calculating this transformation factor, the reduced capacity curve of the equivalent SDOF (1-EDF) system can be obtained dividing the basal shear and top displacement, in the MDOF capacity curve, by this factor (Figure 14).

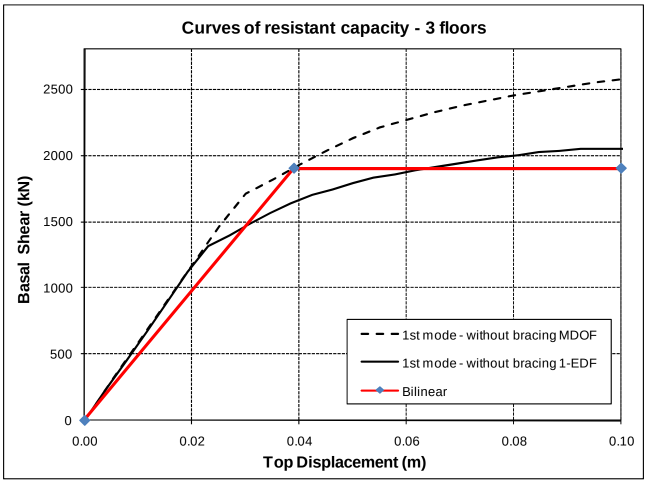


Figure 14: Equivalent Bilinear curve for 1^{st} mode without bracing (frame with 3 floors).

The equivalent SDOF (1-EDF) capacity curve is then simplified to an elastic-perfectly plastic equivalent regime. To obtain this simplified curve it is necessary to determine both the resistant strength capacity (F_y*) and the idealized yield displacement (d_y*) of the equivalent SDOF (1-EDF) system. Since both curves must have the same dissipation energy, the area below the simplified curve must be equal to the integrated area of the curve for the equivalent SDOF (1-EDF) system. The strength capacity has the same value as the one obtained for the maximum basal shear associated with the collapse mechanism in the equivalent SDOF curve and the yielding displacement is obtained applying the equal energy principle (Figure 14).

This is a fundamental step in the seismic performance characterization of the structure since a slight change in the bilinear curve can significantly influence the structural response and the corresponding seismic performance. Notice that the bilinear curve shown in Figure 14 has a reduced initial stiffness as compared with the equivalent SDOF curve; that leads to a less stiff structure and consequently to a higher fundamental period system.

The initial stiffness must be chosen according with the regime in which the structure is expected to operate. If the seismic structural performance is expected to occur at a quasi-linear regime with slight inelastic hinge behaviour then the initial stiffness assumes a very important role in the bilinear curve definition.

If a deep nonlinear behaviour is expected in the medium inelastic deformation range, then the chosen yielding values become more significant and the value assumed by the initial stiffness can be somehow neglected or disregarded.

With the bilinear equivalent curve defined it is possible to determine the equivalent period T* according to the following equation:

$$T^* = 2\pi \sqrt{\frac{m^* \cdot d_y^*}{F_y^*}} \tag{10}$$

After determining the bilinear equivalent curve translating the elastic-perfectly plastic behaviour of the equivalent SDOF, it is necessary to perform a unit transformation to an ADRS format in order to characterize the performance point. This point is determined extending the initial stiffness bilinear branch to intersect the elastic spectral response. In this case (3 floors frame without bracing) the performance displacement is equal to $d_{max}=d_t^*=S_d=0.053$ m , as shown in Figure 15. Also a ductility factor can be determined dividing the performance displacement by the yielding displacement.

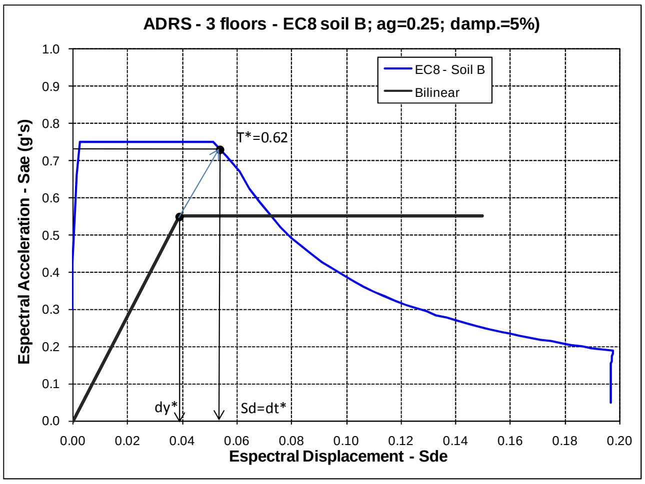


Figure 15: Performance point for 1st mode without bracing (frame with 3 floors).

At this point the maximum (top) displacement of the MDOF system is obtained from the performance point amplified by the transformation factor:

$$d_{\max} = \Delta_{top} = \Gamma \cdot d_t^* = \Gamma \cdot S_d \tag{11}$$

The seismic structural performance is then computed increasing the loading pattern and assuming this top displacement as the control displacement. With this process it is possible to access structural damage, hinge evolution, internal forces and inter-stories displacements.

These results can be compared with the resistant capacity values associated with the desired seismic performance. Obviously, this information is very important for seismic analysis and design purposes. Similar conclusions can be obtained by applying this methodology to the other structural systems. The equivalent SDOF capacity curve is determine by a MDOF reduced curve for all cases and the behaviour that was observed in the original capacity curve is also visible in the reduced curves. Generally the bracing system increase the overall inelastic performance compared with the frames without these bracing systems. As expected lower top displacement are obtained with the corresponding decrease in the inelastic range.

The presented methodology is of paramount importance for the evaluation of the seismic performance of buildings and structures, because it is easily applicable and permits to obtain realistic non-linear response of structures (conditioning their seismic design). It still presents other advantages [17], namely the identification of the structural critical zones (where localized losses of resistant capacity occur) and the visualization of the collapse sequence. Although some drawbacks have been pointed in the past [17] (some of them already overcome, like the inclusion of effects of higher modes) and still exist, like the insensibility for variations of the geometry and of the dynamic characteristics of the structure, if it is used appropriately this method constitutes a very good means for estimating the non-linear structural capacity.

4 CONCLUSIONS

The main objective of this article consists on the presentation of a simplified methodology (pushover analyses) that allows obtaining the response of a structure under seismic actions considering its non-linear behaviour. To reach this objective three structures (3, 6 and 10 floors) were analyzed under seismic loading (considered applied in two alternative loading patterns) with three variations of their resistant structural system (with and without bracing)..

In these analyses it was verified that this pushover methodology allows evaluating the performance of structures through control of their displacements (at local and global levels), still giving additional information about the ductility and the resistant capacity. The introduction of bracing members in the structural model influences significantly the obtained results, altering the resistant capacity and the associated collapse mode.

This contribution allows improving the knowledge about the seismic response of the analyzed structures, above all for the redistribution of the damaged zones during the occurrence of a high intensity earthquake.

The response of the structures is sensitive to the loading pattern justifying the adoption of an envelope of the resistant capacity associated to the possible loading patterns. In the studied structural cases, it was verified that the largest resistant capacity was obtained with a uniform distribution of the lateral loads. This pushover analysis started to be implemented in the seismic regulations because it is an advantageous methodology for the evaluation of the seismic performance of structures, therefore justifying this present study and the need for continuing this research to better evaluate and characterize the applications of these analyses.

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