Extended abstract

Seismic Design of a reinforced concrete building by Capacity Design

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Abstract

In this work, a reinforced concrete building was modeled by linear analysis and designed by capacity design method. Subsequently a non-linear static analysis was performed and the results of the two methods were compared.

1. Introduction

The seismic stresses generated when designing structures to respond elastic, are too high and not economically justifiable, given the very low annual probability of occurrence. This implies that it is accepted that damages happen, but not collapse. Therefore, the need to exploit the energy dissipation capacity of structural elements is desirable, and to design structures that have sufficient ductility to withstand inelastic deformations without significant loss of strength.

The behavior of the structure is conditioned by the capacity for dissipating energy of its elements and the materials that constitute it. Steel and concrete exhibit different behavior about ductility, and there are ways of overcoming the less ductile behavior of concrete

Traditionally the seismic design of structures is based on forces reduced by a factor, which intends to consider the non-linear demands, and using response spectrum analysis (linear analysis). The limitation of this design process is in realizing what happens when the structural elements begin to yield.

Thus, methods based on nonlinear static analysis, which allow the estimation of the resistance and the deformation capacity. These methods considering the nonlinear characteristics of the materials that constitute the structural elements and the redistribution of the forces that occur when critical zones begin to yield.

2. Nonlinear static analysis

The beginning of the use of non-linear static analysis, or pushover analysis, dates back to the seventies, but only in the last twenty years has gained prominence among engineers and researchers, since it is a relatively simple method that allows to evaluate the complex problem associated to the behavior of structures under nonlinear regime in response to the seismic action.

The purpose of the pushover analysis is to evaluate the performance of the structural system, allowing to estimate the resistance and the capacity of deformation. The analysis takes into account the non-linear characteristics of the materials.

The procedure is to monotonically apply an incremental and invariant lateral force (or adaptive) shape to the structure, up to a predetermined displacement value or until it collapses the structure. This distribution of lateral forces must approximate the forces of inertia that develop during the earthquake, including in the analysis the presence of gravitational loads. In this way, each point of the pushover curve (Figure 2.1) represents the static equilibrium situation of the structure for a shear base V as a function of the top displacement of a control node, thereby giving information about the overall resistance and the capacity of deformation of the structure.

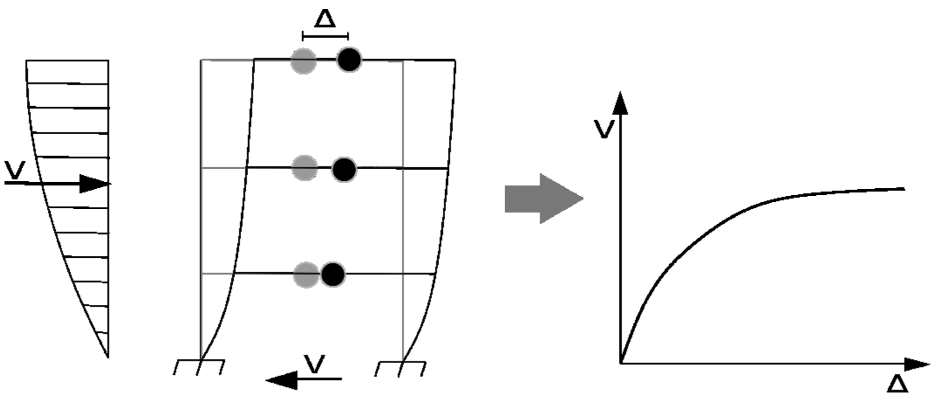

Fig. 2.1- Pushover curve

2.1 N2 method

The method N2 [Fajfar,2000], including in Eurocode 8-part 1-annex B [CEN, 2004], consists of the definition of the capacity of the structure, obtained by pushover analysis, adapted to an equivalent, idealized bilinear SDOF system, in which the seismic response is determined by a non-elastic response spectrum. The transformation of displacements and forces of the equivalent SDOF system to the model of the MDOF structure, is done by a transformation factor r.

$$\Gamma = \frac{\sum m_i \Phi_i}{\sum m_i \Phi_i^2} = \frac{m^*}{\sum m_i \Phi_i^2} \quad (2.1)$$

The transformation factor Γ is usually called the modal participation factor. Any reasonable shape of Φ can be assumed. Herein, the elastic first mode shape will be considered. The displacement shape is normalized with respect to the center of mass of the roof.

The elastic period of the idealized bilinear SDOF system T* is computed according Eq 2.2:

$$T^* = 2\pi \sqrt{\frac{m^* d_y^*}{F_y^*}} \quad (2.2)$$

The target displacement is determined using the rule of equal displacements, for the displacement corresponding to the period of the SDOF system and transformed to the MDOF system.

2.2 Capacity Spectrum Method (CSM)

The method has gained acceptance and popularity among researchers and engineers of structures, and is included in the ATC-40 [ATC-40.1996]

The method consists in comparing the capacity of the structure in the capacity curve format of the pushover analysis with the reduced response spectrum to estimate the maximum displacement. In order to take into account the nonlinear behavior of the structure, reduction factors are applied to the response spectrum as a function of the values of the effective viscous damping coefficient.

In order to estimate the maximum displacement in the structure that occurs during the earthquake, it is necessary to perform an iterative process, where we try to find the point of intersection between the capacity spectrum and the reduced response spectrum,

The iterative process is briefly presented in the following essential steps: (i) construct a bilinear representation of the capacity spectrum; (ii) calculate the spectral reduction factors and plot the response spectrum; (iii) determining the coordinates of the point corresponds to the intersection of the response spectrum with the capacity spectrum; (iv) if the value of the displacement determined is in the tolerance range of 5% with respect to the starting point, then this is the estimated value of the maximum displacement, otherwise it is necessary to continue with the iterative process.

2.3 Capacity Spectrum Method - (CSM)-FEMA 440

FEMA 440 [ATC-2005] introduces some changes to the CSM method of the ATC-40, based on a large statistical study performed using SDOF oscillators with a variety of different hysteretic behaviors.

3 Numerical modeling

3.1 Confined concrete

The confinement of the concrete causes a change in the stress-strain relationship, both the strength and the ultimate extensions are higher, which can significantly increase its ductility, contributing to an increase in the deformation capacity of the structural element. In the Mander analytical model [Mander et al., 1984] for confined concrete elements subject to cyclic uniaxial loading, the stress-strain relationship after reaching the maximum stress value, presents a downward line representative of the degradation of strength and stiffness that characterizes the envelope of a cyclic loading, Fig 3.1

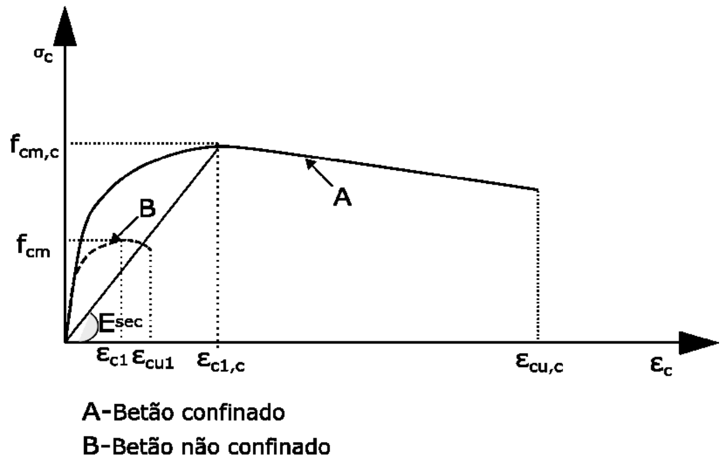

Fig 3.1- Stress-strain curve [Mander *et al*,1984]

3.2- Steel

The steels present two important characteristics for the seismic behavior of the structural elements:

- The extension of the steel to the ultimate strength, ε_{su}, which may influence the maximum value of the ultimate curvature of the sections, and therefore influence the local ductility.
- The relationship between tensile strength and yield stress, f_t / f_y, which translates the hardening of the steel, and which has an influence on the length of the plastic hinge.

In the first load cycle the behavior is the same as the steel subjected to monotonic loading, which can be represented by three regions:

$$0 \le \varepsilon_s \le \varepsilon_y \qquad f_s = E_s \cdot \varepsilon_s \le f_y \tag{3.1}$$

$$\varepsilon_y \le \varepsilon_s \le \varepsilon_{sh} \qquad f_s = f_y \tag{3.2}$$

$$\varepsilon_{sh} \le \varepsilon_s \le \varepsilon_{su} \qquad f_s = f_y + (f_t - f_y) \cdot \left(\frac{\varepsilon - \varepsilon_{sh}}{\varepsilon_{su} - \varepsilon_{sh}}\right)^{1/2} \tag{3.3}$$

3.3- Plastic hinge length

In the models of concentrated plasticity, the deformation of the structural elements and the consequent displacements in the structure are dependent on the length of the plastic hinge. The greatest difficulty of these models is to be able to establish the ideal value of said length. In this work was adopted the solution proposed by EC8-2, Annex E for beams and columns, Eq. 3.1, and for reinforced wall the solution proposed by Priestley [Priestley *et al.*,2007], Eq3.2

$$L_P = 0{,}10 \cdot l + 0.015 \cdot f_{sy} \cdot d_{bl} \tag{3.4}$$

$$L_P = k \cdot l + 0.1 \cdot l_w + 0.022 f_{sy} d_{bl} \tag{3.5}$$

Where:

$$k = 0.2\left(\frac{f_u}{f_{sy}} - 1\right) \le 0.08 \tag{3.6}$$

3.4 Plastic hinge model

The CALTRANS model [Caltrans, 2009], introduced automatically by SAP2000 [CSI, 2009], is based on the idealization of the perfect elastoplastic moment-curvature (M-φ). The elastic branch of the idealized relation must pass at the point corresponding to the first armature to be yielded, and the plastic moment is obtained by balance of areas above the point of yield between the idealized relation and the real curve.

4. Case study

The building under study consists of 6 floors in reinforced concrete structure, with a height of 16.8m above ground. It develops in an area of implantation of approximate dimensions of 28.4m X 11.4m, constant in all the floors. It is located in Lisbon. (a_{gr}= 1,5 m/s^2)   Table 4.1 show the materials adopted.

Table 4.1- Materials

Concrete C25/30	f_{cd}	16,7 MPa
	f_{ck}	25,0 MPa
	f_{ctm}	2,6 MPa
	$E_{c,28}$	31,0 GPa
	v	0,2
Steel A400 NR	f_{syk}	400,0 MPa
	f_{syd}	348,0 MPa
	E_s	200,0 GPa

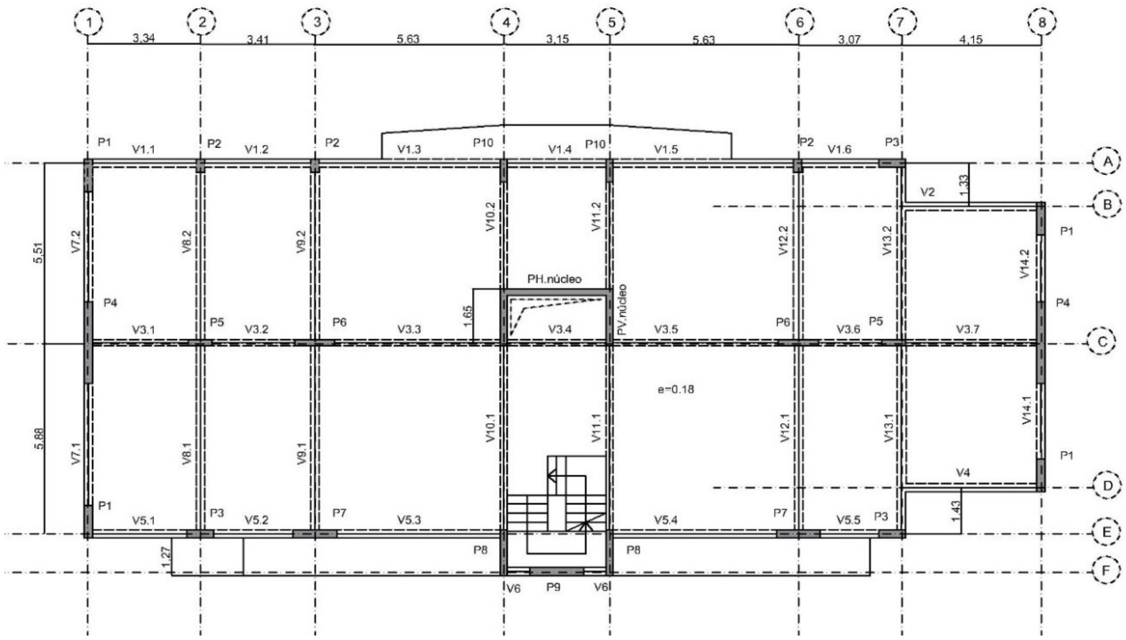

Fig.4.1- Plant

4.1 Modal response spectrum analysis

Table 4.2 show the results for the dynamic analysis.

Table 4.2 – Effective masses and periods

Modo	T (s)	U_x (%)	U_y (%)
1	0.68	0.0	72,1
2	0.65	76,7	0,2
3	0.53	0.0	3.6
4	0.20	0.1	13.8
5	0.19	14.6	0.2
6	0.15	0.0	0.4
	∑	91.4	90.3

4.2 Structural type of the building and behavior factor

According to EC8-1, the investigated building represents an uncoupled wall system in both horizontal directions. The structural system is considered as a wall system, when more than 65% of shear force resistance at building base is taken by walls.

The behavior fator in both direction is equal to 3.0.

4.3 Nonlinear analysis

In the modeling of nonlinear analysis, the post-yield behavior of the structural elements should be included, through models that simulate plasticity in zones where inelastic deformations are predicted to occur. In the case of the study building, a nonlinear spatial analysis was performed using the SAP2000 automatic calculation program. This software allows modeling nonlinear behavior through concentrated plasticity models, providing automatic force-displacement relationships.

Figures 4.2 and 4.3 show pushover curves obtained for the X and Y directions, respectively. Pushover curves produced by uniform loading have, in both directions, shear force values at the upper base of the modal load.

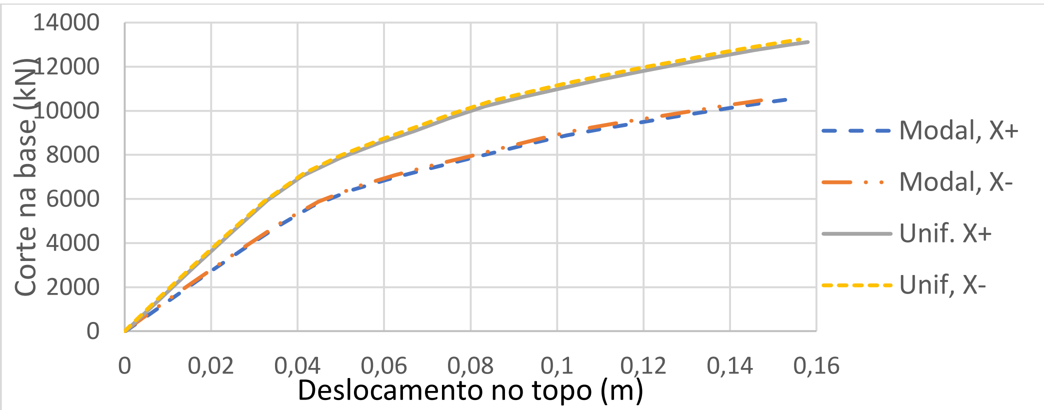

Fig.4.2- Pushover curve, X dir.

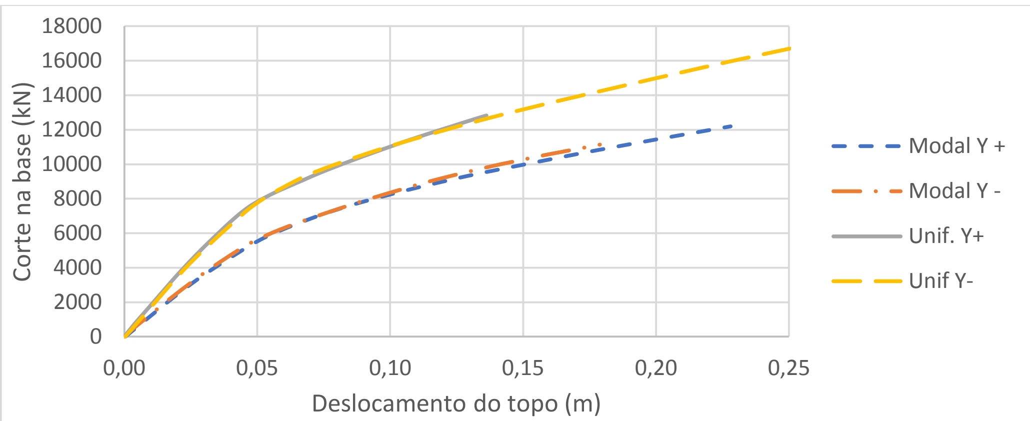

Fig.4.3- Pushover curve, X dir.

4.3.1 N2 method

Table 4.3 show the results for N2 analysis.

Table 4.3 – Results from método N2, X and Y dir.

	Modal Dir. Y	Unif. Dir. Y	Modal Dir. X	Unif. Dir. X
m*	1338.5		1513,9	
F* (kN)	8710	12152	7686	9655
d*y (m)	0.115	0.125	0.081	0.080
d*u (m)	0.162	0.185	0.112	0.113
d*u/d*y	1.41	1.48	1.38	1.41
T* (s)	0.83	0.73	0.78	0.71
Sa (m/s^2)	6.5	9.0	5.1	6.4
Sae (m/s^2)	4.0	4.6	4.4	4.8
d*t (m)	0.07	0.06	0.07	0.06
Δ (m)	**0.098**	**0.084**	**0.095**	**0.081**

From the results, the following can be observed:

• The value of the period T * for modal loading is greater than the uniform loading period.

• The period T * is greater than the fundamental period of each direction determined in the elastic analysis.

• The target displacement and top displacement caused by modal loading is greater than that caused by uniform loading.

• The response of the equivalent SDOF structure is elastic for the seismic action considered, d * t <d * y, Sae <Sa

• The ductility of the SDOF structure, d * u / d * y, has values close to 1.4, revealing the low ductility available in the structural system.

5. Conclusions

A structural wall system building was analyzed according to EC8-1 definitions. Structural wall systems are characterized by high stiffness, deformations are relatively small and there are generally no problems of limiting storey drifts.

Of the results is concluded that the ductility values are lower than the one adopted in the linear analysis.

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