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ANALYSE DE LA PERFORMANCE SISMIQUE D'UN BÂTIMENT EXISTANT EN BÉTON ARMÉ A L'AIDE DE LA MÉTHODE N2

SEISMIC PERFORMANCE ANALYSIS OF AN EXISTING RC BUILDING USING N2 METHOD

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Résumé- L'Algérie est située dans la zone de collision des plaques lithosphériques africaines et eurasiennes. Les mouvements continus de ces plaques sont à l'origine de fréquents tremblements de terre de magnitudes et de conséquences variables. Presque toute la partie nord du pays a subi des tremblements de terre dévastateurs avec de nombreux dégâts importants et de décès. Le séisme de Boumerdès du 21 mai 2003 a été le tremblement de terre le plus violent jamais enregistré en Algérie, tuant près de 2300 personnes, plus de 11000 blessés et laissant plus de 120000 sans-abris. Environ 182000 logements ont été endommagés. Le séisme a généré un tsunami localisé qui a endommagé des bateaux au large des îles Baléares. La perte économique totale a été estimée à 5 milliards de dollars US. Pour évaluer les performances des réponses sismiques des structures, les codes de conception sismique classiques considèrent un facteur de comportement global R, pour prendre en compte la capacité à dissiper l'énergie par déformation plastique. Pour cela, des méthodes simplifiées telles que la méthode statique équivalente et la méthode dynamique modale spectrale utilisant le spectre de réponse sont proposées dans de nombreux codes. De nos jours, la conception sismique basée sur les performances (PBSD) considère les analyses statiques non linéaires avec différents niveaux de performance. La philosophie de conception est basée sur la détermination de la résistance structurelle optimale pour atteindre un état limite de performance donné, lié à un niveau de dommage défini, sous un niveau spécifié d'intensité sismique. Le premier mode est en général considéré dans ce type d'analyse. Cette étude évaluera les performances sismiques d'un bâtiment existant en béton armé à l'aide d'une analyse statique non linéaire selon l'Eurocode 8. Les résultats sont réalisés conformément aux Règles Parasismiques Algériennes en vigueur RPA99 / version 2003.

Mots - clés : Analyse pushover, Courbe de capacité, Spectre de réponse, Spectre de réponse en accélération-déplacement, Déplacement cible.

Abstract- Algeria is situated in the collision zone of African and Eurasian lithospheric plates. The continuous movements of these plates are the cause of frequent earthquakes of varying magnitudes and effects. Almost all the northern part of the country has experienced devastating earthquakes with many heavy damages and deaths of people. The Boumerdès earthquake of May 21, 2003, was the strongest ever-recorded earthquake in Algeria, killing nearly 2300, more than 11000 injured and left upwards of 120000 people homeless. About 182000 housing units were damaged. The quake generated a localized tsunami, which damaged boats off the coast of the Balearic Islands. Total economic loss was estimated at US$5 billion. To assess seismic response performance of structures, classical seismic design codes consider a global behavior factor R, to take into account the ability to dissipate energy through plastic deformation. For this purpose, simplified methods such as equivalent static method and dynamic modal spectral method using response spectrum are proposed in many codes. Nowadays, Performance Based Seismic Design (PBSD) considers nonlinear static analyses with different performance levels. The design philosophy is based on determination of the optimum structural strength to achieve a given performance limit state, related to a defined level of damage, under a specified level of seismic intensity. The first mode is in general considered in such kind of analysis. This study will assess the seismic performance of an existing RC building using nonlinear static analysis according to Euro code 8. The results are carried out in accordance with the Algerian Seismic Design Code in force RPA99/version 2003.

Keywords: Pushover analysis, Capacity curve, Response spectrum, Acceleration-displacement response spectrum, Target displacement.

1-Introduction

During the past earthquake disasters in the world, significant damage occurred to buildings despite, they were designed according to the conventional earthquake resistant design codes based on the philosophy of force-based approach. The current Force based design method is predominantly based on the minimum strength required for the structure. The fundamental shortcomings in this method are mainly the assumed stiffness of structural elements, the inappropriate global behavior factor, the determination of ductility and other considerations. One of the major developments in seismic design over these two last decades was to facilitate the implementation of Performance Based design methodology in many seismic design codes, since it is now considered as the seismic design methodology of the future. The application of the Performance based Seismic Design in engineering projects has become possible due to the advances in knowledge of earthquake engineering as well as in computing technology. It allows the design team to work together to determine the appropriate level of ground motion and performance objectives to evaluate how the building is likely to perform under a design earthquake, in order to meet the owner's expectations. A building may be designed based on one or multiple performance objectives. The selected performance objectives will depend on the intended use of the structure. The target building performance levels range from fully operational, in which structural and nonstructural components are expected to sustain almost no damage in response to the design earthquake, to collapse prevention, in which the structure should remain standing, but is extensively damaged.

The capacity spectrum method, the direct displacement based design and the N2 method are considered as the most suitable and techniques approaches used to assess the seismic performance of existing structures, or design of new structures.

The capacity spectrum method compares the capacity of a structure to resist lateral loads, to the demands of earthquake response spectra. The capacity is represented by a lateral load force-displacement diagram that takes into account the sequential yielding of structural elements as the structure is laterally displaced beyond its linear elastic limits.

2- Performance Based Seismic Design

Modern codes, such as the Euro code 8 have introduced important innovative seismic design practices for building structures, such as performance based design with respect to real nonlinear behavior of structural elements using non linear static analysis (pushover) procedure with several performance levels. The basic theory considers capacity design approach which is a design process where it is beforehand decided which structural elements will yield (ductile components) and which structural elements will remain elastic (brittle components) according to a certain performance level Réf. [1].

2.1- Nonlinear static analysis

A nonlinear static analysis allows a more accurate estimation of the inelastic structural response than linear methods using behavior factors, since the formation of plastic effects and the redistribution of forces are considered. It estimates the overall building load carrying capacity by means of a nonlinear force-displacement capacity curve. A nonlinear static analysis can play an important role in the design of new and existing buildings. It involves more effort to be performed and should be approaches with specific objectives in mind. Nonlinear analysis methods are best applied when either geometric or material nonlinearity is considered Réf. [2].

a- Geometric nonlinearity: This is a type of nonlinearity where the structure is still elastic, but the effects of large deflections cause the geometry of the structure to change, so that linear elastic theory breaks down. Typical problems that lie in this category are the elastic instability of structures, such as in the Euler bulking of struts and the large deflection analysis of a beam-column member. In general, it can be said that for geometrical non-linearity, an axially applied compressive force in a member decreases its bending stiffness, but an axially applied tensile force increases its bending stiffness. In addition, P-Delta effect is also included in this concept.

b- Material nonlinearity: In this type of nonlinearity, material undergoes plastic deformation. Material nonlinearity can be modeled as discrete hinges at a number of locations along the length of a frame (beam or column) element and a discrete hinge for a brace element as discrete material fibers distributed over the cross-section of the element, or as a series of material points throughout the element.

c- Static pushover analysis: It is a nonlinear static procedure; the structural model is subjected to a constant gravity loading and an incremental lateral loads whose distribution represents the inertia forces expected during a ground motion shaking. The lateral loads are applied until a target displacement is reached which represents the displacement demand that the earthquake ground motion would impose on the structure. Once loaded to the target displacement, the demand parameters for the structural components are compared with respective acceptance criteria for the desired performance state. The capacity curve defines in general four structural performance levels:

- Fully operational: No significant damages have occurred to structural and nonstructural components. Building is suitable for normal occupancy and use.

- **Operational:** no significant damage has occurred to structure, which retains nearly all of its pre-earthquake strength and stiffness. Non-structural components are secure and most would function.

- **Life Safety:** significant damage to structural elements, with substantial reduction in stiffness, however, margin remains against collapse. Nonstructural elements are secured but may not function. Occupancy may be prevented until repair can be instituted.

- Near Collapse: substantial structural and nonstructural damage. Structural strength and stiffness substantially degraded. A little margin against collapse. Some falling debris hazards may have occurred.

2.2- Capacity curve-pushover

The capacity curve is determined by first loading statically the structure with realistic gravity loads combined with a set of lateral forces to assess the roof displacement and base shear. The yielding elements are then relaxed to form plastic hinges and incremental lateral loading is applied until a nonlinear static capacity curve is created. The capacity curve can then be combined with a demand curve, typically in the form of an Acceleration Displacement Response Spectrum (ADRS). This combination essentially reduces the problem to an equivalent single degree of freedom system. Static pushover analysis is most suitable for systems in which the fundamental mode dominates behavior. Results provide insight into the ductile capacity of the structural system, and indicate the mechanism, load level, and deflection at which failure occurs.

2.3- Pushover analysis according to Eurocode 8

Among the different approaches proposed in the literature, the Euro code 8 describes and recommends the N2 method developed by Fajfar Réf. [3] for evaluating the seismic response of newly designed and existing buildings. Pushover analysis is performed under two lateral load patterns. A load distribution corresponding to the fundamental mode shape (Modal Distribution) and a uniform distribution proportional to masses (Uniform Distribution).

Classical steps of the Euro code 8 to implement N2 method are then:

- Develop a cantilever model of the structure with concentrated masses and elastic behavior (uncracked cross sections).

- Determination of the intrinsic dynamic characteristics of the structure (fundamental period of vibration and first mode shape).
- Normalization of the fundamental mode shape such as $\phi_n = 1$.
- Determination of the participating modal mass of the first mode.
- Transformation of the multi degree of freedom system to an equivalent single degree of freedom with an equivalent mass m* determined as:

$$m^* = \sum m_j \phi_{j,1} = \sum \overline{F}_j \qquad (1)$$

- Determination of the modal participation factor of the first mode:

$$\Gamma = \frac{m^*}{\sum m_j \phi_{j,1}^2} = \frac{\sum \overline{F}_j}{\sum \left(\frac{\overline{F}_j^2}{m_j} \right)} \qquad (2)$$

The force F* and displacement d*

of the equivalent single degree of freedom are computed as:

$$F^* = \frac{F_b}{\Gamma} \quad \text{et} \quad d^* = \frac{d_n}{\Gamma} \qquad (3)$$

Where:

F_b and d_n are the base shear force and the control node displacement of the multi degree of freedom system respectively.

- Determination of the idealized elasto perfectly plastic force displacement relationship.

The yield force F_y^*, represents the ultimate strength of the equivalent single degree of freedom system, and is equal to the base shear force at the formation of the plastic mechanism. The initial stiffness of the equivalent single degree of freedom system is determined by equalizing the areas under the actual and the equivalent single degree of freedom system force displacement curves.

Based on this assumption, the yield displacement of the equivalent single degree of freedom system d_y^* is given by:

$$d_y^* = 2\left(d_m^* - \frac{E_m^*}{F_y^*} \right) \qquad (4)$$

Where:

E_m^* is the actual deformation energy up to the formation of the plastic mechanism. Figure 1 shows the principle of energies idealization.

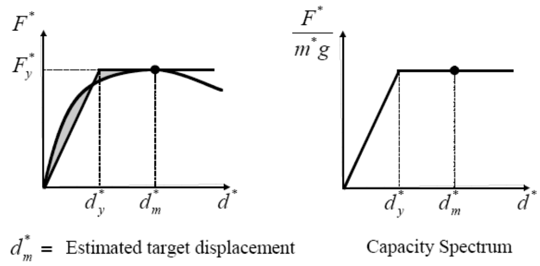

Figure 1: Linéarisation de la courbe de capacité

Figure 1: Linearization of capacity curve

- Determination of the period of the idealized equivalent single degree of freedom system T* as:

$$T^* = 2\pi \sqrt{\frac{m^* d_y^*}{F_y^*}} \qquad (5)$$

- Determination of the target displacement for the equivalent single degree of freedom system as:

$$d_{et}^* = S_e(T^*) \left(\frac{T^*}{2\pi} \right)^2 \qquad (6)$$

Where:

$S_e(T^*)$ is the elastic acceleration response spectrum at the period T*. For the determination of the target displacement d_t^* for structures in the short period range and for structures in the medium and long period ranges, different expressions should be used.

a) $T^* < T_C$ (short period range)

- If $F_y^* / m^* \geq S_e(T^*)$, the response is elastic and thus:

$$d_t^* = d_{et}^* \qquad (7)$$

- If $F_y^* / m^* < S_e(T^*)$, the response is nonlinear and:

$$d_t^* = \frac{d_{et}^*}{q_u}\left(1+(q_u-1)\frac{T_C}{T^*}\right) \geq d_{et}^* \quad \text{with} \quad q_u = \frac{S_e(T^*)m^*}{F_y^*} \qquad (8)$$

Where:
q_u is the ratio between the acceleration in the structure with unlimited elastic behavior $S_e(T^*)$ and in the structure with limited strength F_y^* / m^*.

b) $T^* \geq T_C$ (medium and long period range), the response is nonlinear and:

$$d_t^* = d_{et}^* \quad \text{with} \quad d_t^* \leq 3d_{et}^* \qquad (9)$$

- The target displacement of the multi degree of freedom system is then:

$$d_t = \Gamma d_t^* \qquad (10)$$

3- Case study

The building is six (06) stories with regularity in plan and elevation. The structure is reinforced concrete resisting moment frames with hollow clay bricks infill walls. A configuration of three (03) longitudinal bays, nine (09) transversal bays is considered in this study. The building is located in a high seismic zone (Zone III) with housing usage Réf. [4]. The site inspection has permitted to identify the main characteristics of the structure. Some details are shown in figure 2.The details of structural elements beams and columns are shown in table1.

The grade of concrete is f_{c28} =20 MPA and was obtained by non-destructive tests (ultra sonic tests). The grade of steel is f_e = 400 MPa. The roof dead load is G_T=6.54 KN/m². The roof live load is Q_T=1KN/m². The current level dead load is G_E=6.19 KN/m².
The current level live load is Q_E=1.5 KN/m².

Vue générale du bâtiment

General view of the building

Ferraillage d'une dalle typique en BA

Reinforcing of a typical RC slab

Ferraillage type d'un poteau

Reinforcing of a typical column

Ferraillage type d'une poutre

Reinforcing of a typical beam

Figure 2: Détails de la structure après inspection sur site

Figure 2: Details of the structure after site inspection

Tableau 1: Dimensions et ferraillage des éléments structurels

Table 1: Dimensions and reinforcing of structural elements

Story	Transversal beams (cm²)		Longitudinal beams (cm²)		Columns (cm²)	
6	35x20	2HA12	50x20	2HA12	40x35	10HA12
5	35x20	2HA12	50x20	2HA12	40x35	10HA12
4	35x20	2HA12	50x20	2HA12	40x35	10HA12
3	35x20	2HA12	50x20	2HA12	40x35	10HA12
2	35x20	2HA12	50x20	2HA12	40x35	10HA12
1	35x20	2HA12	50x20	2HA12	40x35	10HA12

All transverse reinforcement is mild steel with diameter of 8 mm.

Figure 3 shows 3D model of the structure using ETABS program Réf. [5].

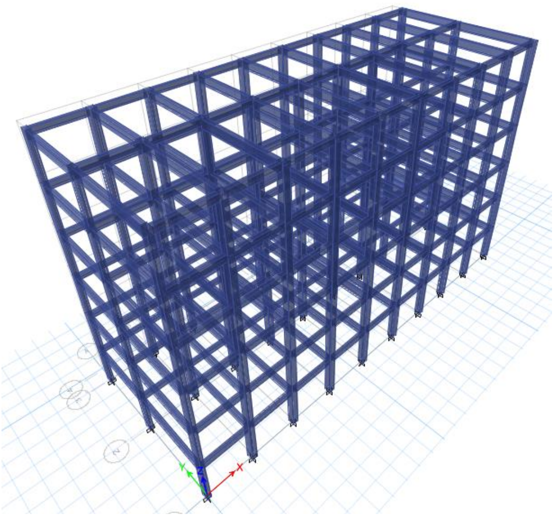

Figure 3 : Modèle structurale tridimensionnel

Figure 3 : Tridimensional structural model

4- Results

4.1- Linear static analysis

The main intrinsic dynamic characteristics of the structure in terms of natural periods and mode shapes are summarized in tables 2 and 3.

Tableau 2 : Périodes naturelles et taux de participation de la masse modale

Table 2: Natural periods and modal participating mass ratios

Direction	T (s)	α_1 (%)	Rz (%)
Long (XX)	1.120	82.530	0.080
Trans (YY)	0.998	77.140	5.180

It can be noticed that in the transversal direction, the fundamental vibration mode is coupled with a little amount of torsion (5.18%) because of the eccentric position of the stairs.

Tableau 3 : Déformées modales des deux premiers modes

Table 3: Mode shapes for the two first modes

Story	First mode shapes	
	Long (XX)	Trans (YY)
6	1.000	1.000
5	0.928	0.923
4	0.799	0.791
3	0.617	0.608
2	0.395	0.387
1	0.159	0.154

4.2- Non linear static analysis

Figures 4 and 5 show the capacity curves for modal distribution (MD) and uniform distribution (UD) obtained from the non-linear static analysis using pushover approach, for the main longitudinal (XX) and transversal (YY) directions.

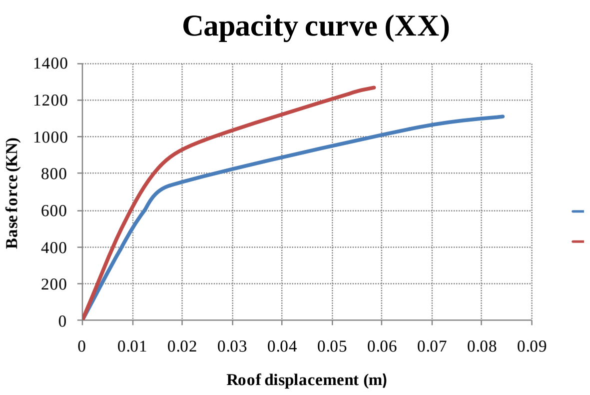

Figure 4 : Courbe de capacité pour la direction longitudinale (xx)

Figure 4: Capacity curve for longitudinal direction (xx)

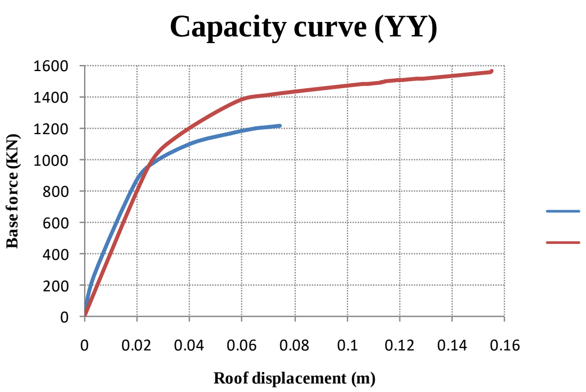

Figure 5 : Courbe de capacité pour la direction transversale (yy)

Figure 5: Capacity curve for transverse direction (yy)

The two vertical distributions of lateral loads are not the same, because one is considered as a uniform pattern based on lateral forces that are proportional to mass regardless of elevation, and the second is considered as modal pattern proportional to lateral forces consistent with the lateral force determined in elastic analysis.

The results of the main parameters of the equivalent single degree of freedom Réf. [6] for both distributions and main directions are shown in table 4.

Tableau 4 : Principaux paramètres des équivalents SDOF et MDOF

Table 4: Main parameters of equivalent SDOF and MDOF

Direction	Modal Distribution		Uniform Distribution	
	Long (xx)	Trans (yy)	Long (xx)	Trans (yy)
Γ_1	1.279	1.285	1.279	1.285
d_m^* (m)	0.065	0.057	0.045	0.120
d_y^* (m)	0.033	0.026	0.024	0.046
T_C (s)	0.5		0.5	
T^* (s)	1.544	1.303	1.249	1.536
$S_e(T^*)(m/s^2)$	3.835	4.286	4.24	3.854
μ = Ru	6.945	7.043	7.030	4.924
$d_{et}^*(m)$	0.231	0.184	0.175	0.230
$d_t^*(m)$	0.231	0.184	0.175	0.230
$d_t(m)$	0.296	0.237	0.223	0.295
$1.5 * d_t(m)$	0.444	0.356	0.335	0.443

The main results in terms of absolute displacements for both distributions and main directions obtained from the nonlinear static analysis are shown in figures 6 and 7. These displacements demand obtained with the N2 method are compared to the limit state of RPA99/2003.

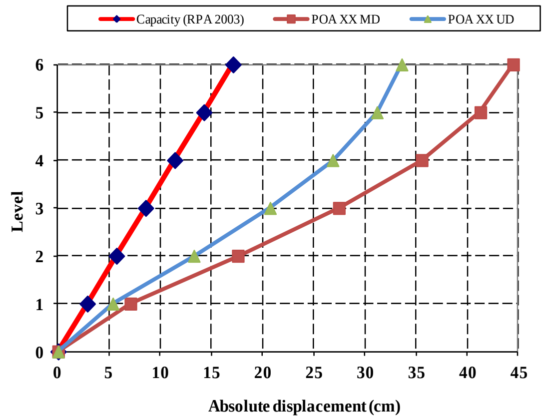

Figure 6 : Déplacements absolus pour la direction longitudinale (xx)

Figure 6: Absolute displacements for longitudinal direction (xx)

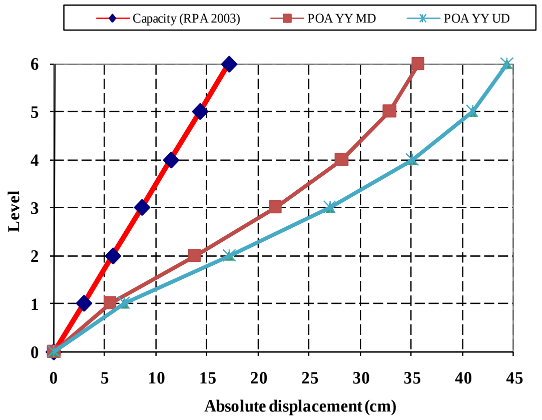

Figure 7 : Déplacements absolus pour la direction longitudinale (yy)

Figure 7: Absolute displacements for longitudinal direction (yy)

5- Discussion

- The analysis of the main results in terms of absolute displacements for both modal distribution and uniform distribution Réf. [7] and for both main longitudinal (XX) and transversal (YY) directions showed that:
- The demand in terms of absolute displacements is larger than the capacity for both distributions and for both main directions Réf. [8].
- The demand in terms of absolute displacements of the modal distribution is the worst in the longitudinal (XX) direction Réf. [9].
- The demand in terms of absolute displacements of the uniform distribution is the worst in the transversal (YY) direction Réf. [10].
- For both directions, the obtained results are not in conformity with the limits of the RPA99/3003 Réf. [11] i.e., larger than the limits 1% H_i.
- Even for inter story displacements; the obtained results are not in conformity with the limits of the RPA99/2003 (larger than 1% H_i).
- Since the gap is less than five (05) centimeters between adjacent buildings the effect of hammering is inevitable Réf. [12].
- On the basis of different obtained results, the absolute displacements demand are larger than the limits of the RPA99/2003 (see Fig. 6 and 7), so we can conclude that the structure needs strengthening Réf. [13].

6- Conclusion

The analysis of the building with the nonlinear static analysis using N2 method according to the Euro code 8 has been performed. All the results showed a large demand in terms of absolute displacements for both distributions and for both main directions.
One can say that in case of a major earthquake, the building is unsafe and heavy structural damage or collapse can occurs. The building is in a need of a strengthening by the incorporation of reinforced concrete shear walls in order to diminish.

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