

NONLINEAR ANALYSIS METHODS FOR REINFORCED CONCRETE STRUCTURAL ELEMENTS SUBJECTED TO STATIC AND DYNAMIC LOADS

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ABSTRACT:

Civil engineering structures are generally designed to be sufficiently ductile and capable of dissipating energy when subjected to movements caused by seismic action. The aim is to study and assess the behavior in terms of global ductility of reinforced concrete beam-column structural elements. Seismic behavior is studied using the push-over method. Initially, we are interested in modeling the nonlinear bending behavior, up to failure, of a reinforced concrete frame under increasing monotonic static loading. The modeling is carried out using the finite element method within the framework of the Navier-Bernoulli hypothesis. The load-displacement curves of the elements analyzed are plotted, enabling the global ductility factor to be deduced. The second part of the work consists of applying the push-over method to a nonlinear static analysis of the frame studied, up to failure, in order to assess its seismic response and deduce its performance point. It was used to obtain the load-displacement curves and to deduce the global ductility factor. After analyzing the nonlinear behavior of the frame using the two proposed methods, approximate global behavior was observed. It was found that in both approaches, the frame exhibits sufficiently ductile behavior, indicating a less advanced state of plasticization. The performance point coincides with the elastic displacement of the frame, so the behavior of the frame has not reached the plastic phase, and the frame can withstand the load applied to it.

1. INTRODUCTION

The analysis of structures designed in reinforced concrete was one of the major areas of research in the last century, which saw great progress in the 1960 and early 1970, thanks to the development of computer tools. Several methods have been developed, the most widely used of which is the finite element method (FEM). The effects of material and geometric nonlinearities have been taken into account by several researchers, including: (Frinklin, H. A., 1970, Grelat. Alan., 1978, Filippou, F. C. and Kwak, G., 1990, Alioua, N. et al., 2014, Benyahi, K et al., 2018, Benyahi, K. et al., 2021a, Benyahi, K et al., 2021b, M. Belhocine et al., 2023).

We are interested in studying and assessing the global behavior of reinforced concrete building elements of the column-beam type, whose behavior is governed by flexion. Firstly, we present the process for formulating the numerical analysis method used to model the nonlinear behavior of reinforced concrete column-

beam structures (frame built at CEBTP-CEA, France (A.A. Taï., 1981)). The calculation steps are described in the form of an algorithm, leading to the computer program "Thin Beam 2." This tool will be used to calculate resistance and displacements and, subsequently, to assess the ductility factor of the behavior of the structures studied with respect to monotonic static stresses. The modeling is carried out using the finite element method (Filippou, F. C. and Kwak, G., 1990) under the Navier-Bernoulli hypothesis. This is achieved using the theory of nonlinear elasticity, which takes account of the real stress-strain curves of concrete and steel. This discretization method seeks an approximate solution and provides reasonably accurate results for various engineering problems (H. Belmioub et al., 2022, K. Ghouilem et al., 2023). It is widely used to assess the behavior of all types of structures. The structure is discretized into 2-node "beam" finite elements with six degrees of freedom (6 ddl).

Next, we applied the "push-over" method to the same frame (CEBTP-CEA), subjected to a variable horizontal load. It is the

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preferred method for assessing seismic performance because of its simplicity. It is also a static analysis that is directly defined by nonlinear material characteristics (Poursha et al., 2014). It was originally developed by Freeman, S.A., 1978. Various improvements and applications of the push-over method (Tehrani-zadeh, M. and Moshref, A., 2011, Fajfar, P. and Gaspersic, P., 1996, Poursha et al., 2014) are still in use today.

A major challenge in the performance-based seismic design of structures is the simple and effective development of methods to analyze and evaluate structural demands. Analysis procedures to predict structural demands for forces and deformations under earthquakes are required in current building codes. Thus, simplified non-linear analysis procedures, including the capacity spectrum method, have been incorporated into the (ATC40.,1996) documents to evaluate the displacement demand imposed on a building that is usually expected to deform inelastically. The main idea is to determine the capacity and demand diagram of an inelastic single-degree-of-freedom system using pushover analysis and response spectrum analysis, respectively, and obtain the target displacement of the performance point corresponding to the intersection of the capacity and demand graphs. Then, the displacement of the single-degree-of-freedom system is converted into that of a corresponding multi-degree-of-freedom system. Finally, by comparing the displacement obtained with the permitted displacement, the structure is evaluated to see whether it can meet the requirements of the seismic design or not (ATC 40., 1996). It is recommended above all for existing buildings, where a more favorable assessment of earthquake safety can avoid costly reinforcement measures.

We therefore apply the capacity spectrum method, applied in ATC 40 to the CEB TP gantry to test its performance under an increasing lateral load until failure, or we introduce the 5% damped spectrum of (RPA., 2003). The capacity curve of the frame is obtained by performing a "push-over" analysis; the shear force at the base is given as a function of the lateral displacement at the head of the frame. The capacity spectrum is then transformed into spectral acceleration and spectral displacement (S_a - S_d) values using the "ETABS "(Extended 3D Analyses of Building Systems) software. The performance point is obtained by applying the (ATC 40. 1996) method, which allows direct assessment of the structure's performance at each limit state, in an Excel "Log Seismic" file, where the 5% damped elastic spectrum of the Algerian seismic regulations (RPA., 2003) has been introduced, to assess the maximum displacement that the frame will undergo and its degree of penetration into the plastic domain. This enabled the behavior and global ductility of the frame to be assessed.

We then compare the results obtained with the two static non-linear analysis methods, in terms of curve shape and global ductility, to estimate the behavior of the frame studied.

2. THEORICAL MODEL

2.1 Situation

We are interested in the modeling and numerical simulation of the nonlinear bending behavior of a reinforced concrete frame subjected to fixed vertical loads and increasing horizontal loading until failure. The nonlinear analysis based on the finite element method is obtained by using the elaborated program "Thin Beam 2". We checked the reliability of the "Thin beam 2" program by

comparing our results with those of the studies carried out by Low and Tai. The seismic behavior is obtained by applying the "push-over" analysis method.

2.2 Nonlinear finite element analysis method for reinforced concrete elements

2.2.1 Approach used: We consider the "Thin beam 2" element in, based on the Navier Bernoulli hypothesis (Belhocine et al., 2023), operating in bending. The beam is presented by the segment gA gB in Figure 1:

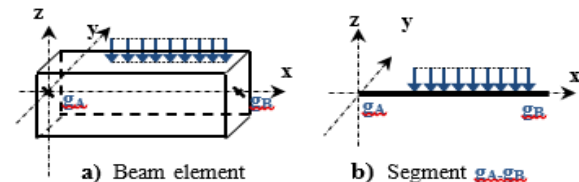


Figure 1. Bernoulli's beam element

2.2.2 Formulation: The relationship between deformations $\varepsilon_x(x)$ and displacements $u_0(x)$ of a point in the cross-section of the beam is linear (Eq.1):

$$\varepsilon_x(x) = \varepsilon_0(x) + z \phi(x) \quad (1)$$

With

$$\varepsilon_0(x) = \frac{du_0(x)}{dx} \quad (2)$$

$$\phi(x) = -\frac{d^2w(x)}{dx^2} \quad (3)$$

Were

$w(x, z)$: Transverse displacement along z
 $u_0(x)$: axial displacement at the beam's reference axis
 $\phi(x)$: Curvature of the cross-section

The relationship between forces and deformations in the cross-section is given by (Eq.4):

$$\begin{Bmatrix} N \\ M \end{Bmatrix} = \begin{bmatrix} \overline{EA} & \overline{ES} \\ \overline{ES} & \overline{EI} \end{bmatrix} \begin{Bmatrix} \varepsilon_g \\ \phi \end{Bmatrix} = [K] \begin{Bmatrix} \varepsilon_g \\ \phi \end{Bmatrix} \quad (4)$$

Were

M: Bending moment
N: Normal force
 $\overline{EA}$: Normal force stiffness
 $\overline{ES}$: Stiffness due to bending normal force coupling
 $\overline{EI}$: Flexural rigidity
[K]: The secant stiffness matrix of the section
 ε_g : Longitudinal deformation at the geometric center g of a section
 ϕ : Curvature of the cross-section

2.2.3 Finite element discretization: The discretization is performed using a beam finite element with 2 nodes having three degrees of freedom per node (or 6 degrees of freedom per element), as shown in Figure 2:

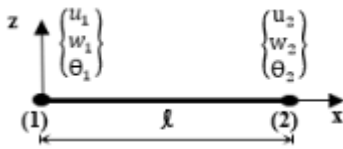


Figure 2. Two node elements

The displacements are represented by a vector of dimension 6×1 in the following way (Eq.5):

$$\{U\} = \{u_1 \quad w_1 \quad \theta_1 \quad u_2 \quad w_2 \quad \theta_2\} \quad (5)$$

The longitudinal displacement $u_0(x)$ is approximated by Lagrange-type interpolation functions of degree 1 (Eq.6):

$$u_0(x) = N_1 \cdot u_1 + N_4 \cdot u_2 \quad (6)$$

With

u_1 and u_2 : the longitudinal displacement values at nodes 1 and 2 respectively

The transverse displacement is approached by Hermite interpolation functions of degree 3. (Eq.7):

$$w(x) = N_2 \cdot w_1 + N_3 \cdot \theta_1 + N_5 \cdot w_2 + N_6 \cdot \theta_2 \quad (7)$$

With

w_1 , and w_2 : Transversal displacement values at nodes 1 and 2 respectively
 θ_1 and θ_2 : The values of the rotation at nodes 1 and 2 respectively
 N_1, N_2, N_3, N_4, N_5 , and N_6 : Interpolation functions

The relationship between cross-sectional deformations at the element's x-axis and nodal displacements is as follows (Eq. 8):

$$\begin{Bmatrix} \epsilon_{0x} \\ \phi \end{Bmatrix} = [B] \{u_n\} \quad (8)$$

With

$[B]$: Deformation matrix

$\{u_n\}$: Nodal displacement vector

Or

$$\{u_n\} = \langle u_1 \quad w_1 \quad \theta_1 \quad u_2 \quad w_2 \quad \theta_2 \rangle \quad (9)$$

$$[B] = \begin{bmatrix} \frac{-1}{l} & 0 & 0 & \frac{1}{l} & 0 & 0 \\ 0 & \left(\frac{6}{l^2} - \frac{12x}{l^3}\right) & \left(\frac{4}{l^2} - \frac{6x}{l^3}\right) & 0 & \left(\frac{6}{l^2} - \frac{12x}{l^3}\right) & \left(\frac{2}{l^2} - \frac{6x}{l^3}\right) \end{bmatrix} \quad (10)$$

At the level of the element's x cross-section, the relationship between internal forces and nodal displacements is evaluated using (Eq. 4) and (Eq. 8):

$$\begin{Bmatrix} N \\ M \end{Bmatrix} = [D][B]\{u_n\}; [D] = \begin{bmatrix} EA & ES \\ ES & EI \end{bmatrix} \quad (11)$$

Or

$[D]$: Matrix of the elastic properties of the element

2.2.4 Resolution algorithm of the program "Thin beam2":

The nonlinear analysis in the program "Thin Beam 2" is carried out using an iterative procedure. The nonlinear resolution algorithm is as follows:

1. For stage $j-1$, corresponding to external loading $\{F\}^{j-1}$, the state of the structure being studied is assumed to be known (displacement $\{U\}^{j-1}$, residual forces $\{\Psi\}^{j-1}$), the initial step ($j=1$) corresponds to zero loading ($\{U\}^0 = \{0\}$ and $\{\Psi\}^0 = \{0\}$).

2. External load increment:

$$\{F\}^j = \{F\}^{j-1} + \{\Delta F\}^j \quad (12)$$

2.2 Start of iterative procedure ($i = 1$):

2.3 Incrementing the residual forces to be balanced.

$$\{\Psi\}^i = \{\Delta F\}^j + \{\Psi\}^{j-1} \quad (13)$$

3. Evaluation of elementary stiffness matrices $[K]^i$, then assemble the global stiffness matrix $[K]$ as a function of the solution $\{U\}^{i-1}$, from the previous iteration ($i-1$):

$$[K]^i = [K(U)^{i-1}] \quad (14)$$

*If this is the first iteration ($i = 1$), then: $\{U\}^{i-1} = \{U\}^{j-1}$ from the previous convergence step $j - 1$.

4. Solving the system of equations:

$$\{K\}^i \{\Delta U\}^i = \{\Psi\}^i \quad (15)$$

5. Cumulative vector of nodal displacements:

$$\{U\}^i = \{U\}^{i-1} + \{\Delta U\}^i \quad (16)$$

6. Calculation of the elementary nodal force vectors resulting from the internal forces, at iteration i : $\{F_R\}_e^i$ and assembly in the global vector $\{F_R\}^i$ of the resistant forces in the structure.
7. Evaluation of the vector of unbalanced residual forces at iteration i :

$$\{\psi\}^i = \{F\}^j - \{F_R\}^i \quad (17)$$

8. Convergence test $\{\psi\}^i = \{0\}$:

8.1. If yes ($\{\psi\}^i = \{0\}$), convergence criteria are met $\rightarrow$ go to step (2) for the next loading increment ($j+1$) and return to step 2

8.2. If no: divergence test: $i \leq i_{max}$: the maximum number of iterations is not excited $\rightarrow$ move on to the next iteration ($i+1$) $\rightarrow$ return to step (3).

8.3. If the maximum number of iterations is exceeded ($i > i_{max}$): the process diverges $\rightarrow$ stop calculation.

With

i_{max} : maximum iteration value set by the user.

The calculation chart is presented in Figure 3.

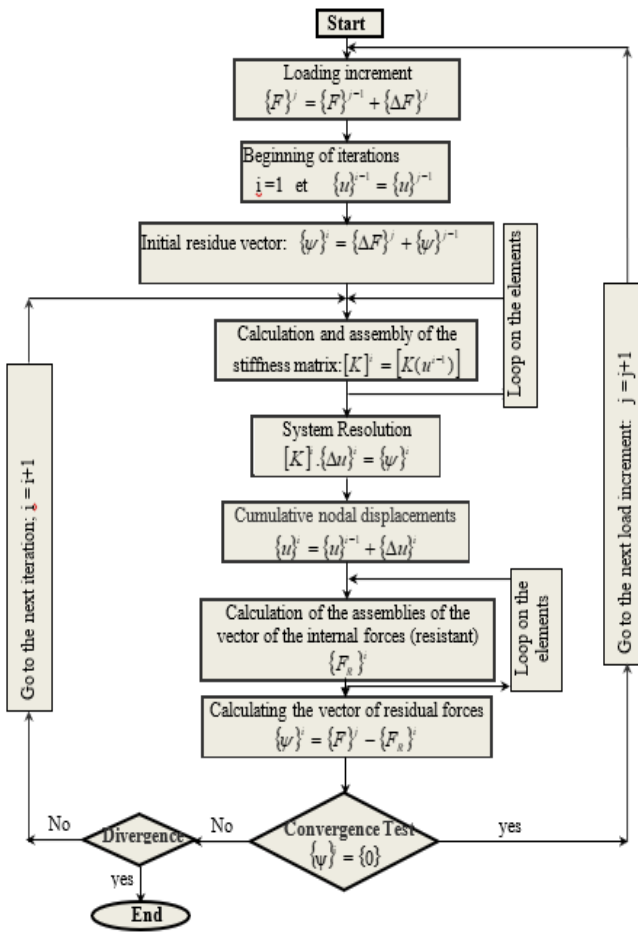


Figure 3. Calculation flowchart for the program "Thin beam 2"

2.2.5 Modelling materials: Several approaches are used in the literature to describe the behavior of concrete in compression. In the present study, we use the model of (Sargin, 1971), which includes the parabolic law and the parabola-rectangle law. For the behavior of concrete in tension, we use the model of (Grelat, 1978), which takes into account the contribution of tensioned concrete after cracking (tension stiffening effect). To model the behavior of steels, we use a perfect elastoplastic law (BAEL, 1999).

2.3 Nonlinear push-over analysis method

2.3.1 Principles and purpose of the push-over method: The principle of the "push-over" method is to superimpose a curve representing the resistance capacity of a structure, resulting from a nonlinear analysis under incremental static loading, with a curve representing the stress caused by the earthquake (the response spectrum) (Manasri et al., 2010). The intersection of these two curves represents a performance point for estimating the maximum displacement undergone by the structure under study and its degree of penetration into the nonlinear domain (Sermin, Oguz., 2005). It therefore amounts to superimposing seismic capacity and demand.

2.3.2 Capacity curve

2.3.2.1 Diagram of the capacity curve: It reflects the capacity of structures to withstand earthquakes, representing the shear force at the base V_b acting on the structure as a function of the

displacement at the top. This curve is formed by a linear elastic phase followed by a nonlinear phase, which corresponds to the formation of plastic hinges (Figure 4).

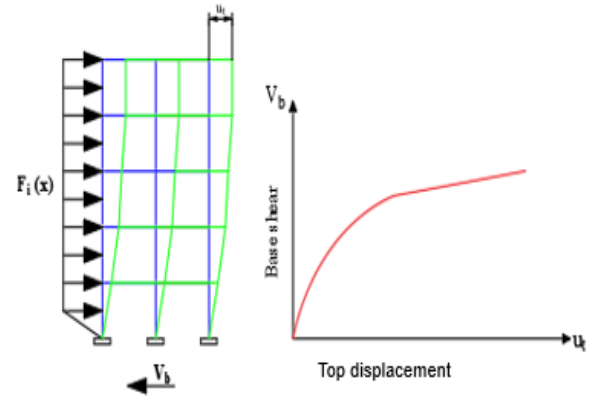


Figure 4. Damage levels described by a capacity curve

2.3.2.2 Capacity spectrum transformation: During the push-over analysis, the curve obtained represents the variation of the shear force at the base as a function of the displacement at the top, resulting in a curve in the $(V_b - u_t)$ format. This must be transformed into another format $(S_a - S_d)$ (spectral acceleration, spectral displacement) to give a capacity spectrum (Figure 5).

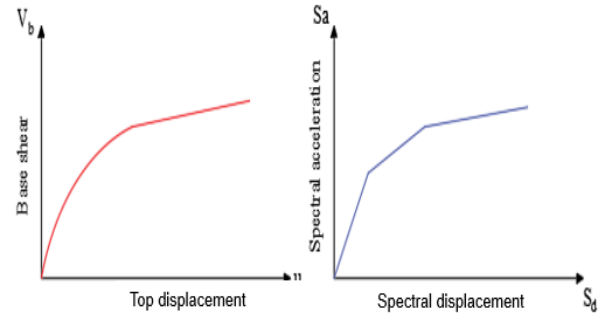


Figure 5. Transforming the capacity curve

2.3.3 Determination of the damped spectrum at 5%: We use the spectrum from (RPA., 2003), which is a 5% damped spectrum plotted with an acceleration response curve (S_a/g) for a system with one degree of freedom. Subjected to a given excitation for successive values of natural periods T , to obtain a 5% damped elastic spectrum, simply assign a value equal to unity for the damping factor η (Eq. 12).

$$\frac{S_a}{g} = \begin{cases} 1,25 A \left[1 + \frac{T}{T_1} \left(2,5\eta \frac{Q}{R} - 1 \right) \right] & 0 \leq T \leq T_1 \\ 2,5\eta (1,25 A) \left(\frac{Q}{R} \right) & T_1 \leq T \leq T_2 \\ 2,5\eta (1,25 A) \left(\frac{Q}{R} \right) \left(\frac{T_2}{T} \right)^{2/3} & T_2 \leq T \leq 3 s \\ 2,5\eta (1,25 A) \left(\frac{Q}{R} \right) \left(\frac{T_2}{3} \right)^{2/3} \left(\frac{3}{T} \right)^{5/3} & T \geq 3.0 s \end{cases} \quad (18)$$

The spectrum obtained using (RPA., 2003) is in $(S_a - T)$ format. To convert it to $(S_a - S_d)$ format, simply use the ATC 40., 1996 method, where (Eq. 13):

$$S_{de} = \frac{T_n^2}{4\pi^2} S_{ae} \quad (19)$$

Were

A: Zone acceleration coefficient
 η : Damping correction factor
 R: Behavior coefficient of the structure
 T_1, T_2 : Characteristic periods associated with the site category
 Q: Quality factor

2.3.4 Determining the performance point: We use procedure B of (ATC 40., 1996) for the determination of the performance point, which is based on the assumption that the slope of the elastic segment, the point (a_y, d_y) , and the slope of the elastic post segment are constant (ATC., 1996). This simplifying assumption implies that the effective damping depends only on the term (d_{pi}) . It is described as follows:

1. Development of the elastic spectrum damped at 5%.
2. Representation of the 5% damped spectrum and a family of reduced $\beta_{eff} > 5\%$ spectra.
3. Transformation of the capacitance curve in $(V_b - u_t)$ format into a capacity spectrum in $(S_a - S_d)$ format and representation of the capacitance spectrum with the seismic demand spectra.
4. Development of the bilinear idealization curve of the capacitance spectrum. The initial slope of the capacitance spectrum represents the initial stiffness of the structure. The point of intersection between the slope of the capacity spectrum and the 5% damped elastic spectrum is projected onto the capacity spectrum to give the point (a^*, d^*) (Figure 6). The post-elastic segment must be determined by rotating the post-elastic segment around the point (a^*, d^*) , respecting the equality of sections A_1, A_2 (Figure 7):

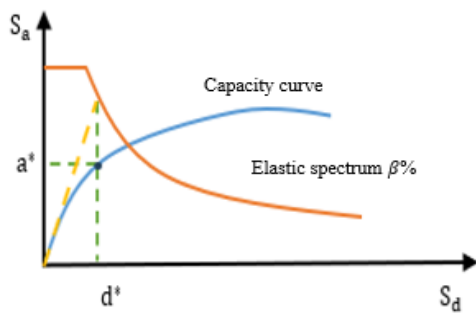


Figure 6. Search the point (a^*, d^*)

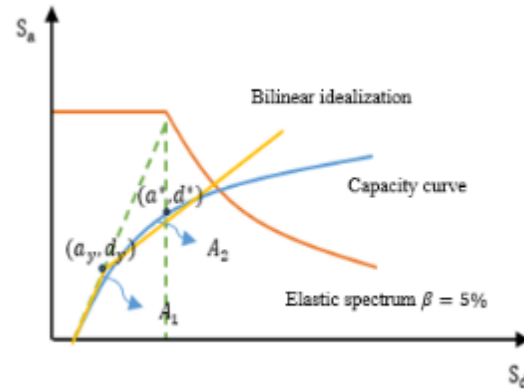


Figure 7. Determine the point (a_y, d_y)

5. Calculation of the effective damping for various displacements chosen close to the point (a^*, d^*) . The slope of the post-elastic segment of the idealized curve is given by the equation (Eq. 20):

$$\text{Post-elastic slope} = \frac{a^* - a_y}{d^* - d_y} \quad (20)$$

For any point a_{pi}, d_{pi} of the post-elastic segment of the idealized curve, the slope is given by (Eq. 21):

$$\text{Post-elastic slope} = \frac{a_{pi} - a_y}{d_{pi} - d_y} \quad (21)$$

Since the slope of the post-elastic segment is constant, we can equalize the equations Eq. 20 and Eq. 21:

$$\frac{a^* - a_y}{d^* - d_y} = \frac{a_{pi} - a_y}{d_{pi} - d_y} \quad (22)$$

By solving the equation, we can find the a_{pi} values as a function of d_{pi} (Eq. 23):

$$a'_{pi} = \frac{(a^* - a_y)(d_{pi} - d_y)}{d^* - d_y} + a_y \quad (23)$$

The value of a_{pi} can be introduced into the equation to calculate β_{eff} (Eq. 24):

$$\beta_{eff} = \frac{63,7 \text{ k} (a_y d_{pi} - d_y a'_{pi})}{a_{pi} d_{pi}} + 5 \quad (24)$$

Solving equation 8.15 for a series of d_{pi} values. Replace the expression for β_{eff} by (Eq. 25):

$$\beta_{eff} = \frac{63,7 \text{ k} (a_y d_{pi} - d_y a_{pi})}{a_{pi} d_{pi}} \quad (25)$$

6. For any point d_{pi} considered in step 5, plot the points (d_{pi}, β_{eff}) in the same graph as that of the reduced spectra and the capacitance spectrum.
7. Connect the various points created in the previous step to form a line. The intersection of this line with the capacitance spectrum defines the performance point, (Figure 8):

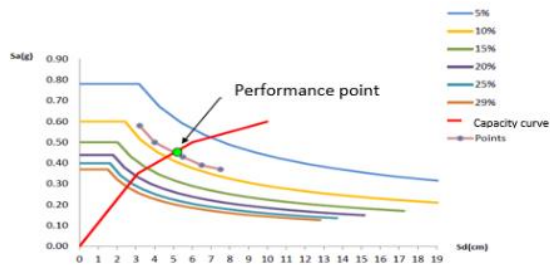


Figure 8. Determining the performance point

3. EXAMPLE STUDIED

3.1 Low Moehle example

This is a 514.4 mm high column, embedded at its base (Limkatanyu, S. and Samakrattakit, A., 2003), subjected to a fixed vertical load N (44.5 kN) and a variable horizontal load F variable up to failure. The geometric characteristics of the column are shown in Figure 9. The characteristics of the materials used are presented in Table 1:

Characteristics of concrete	
Elasticity module E	MPa 33.10 ³
Compressive strength f_{cj}	42,13
Compressive strength out concrete	44
Tensile strength f_{tj}	3.05
Poisson coefficient ν	0.2

Table 1.a Characteristics of concrete

Characteristics of steel	1 ^{er} et 3 ^{eme} bed	2 ^{ieme} bed
Elasticity module E	MPa 200.10 ³	MPa 200.10 ³
Elasticity limit	465.1	444
Resistance à la rupture	500	500
Poisson coefficient ν	0.3	0.3

Table 1. b Characteristics of steel

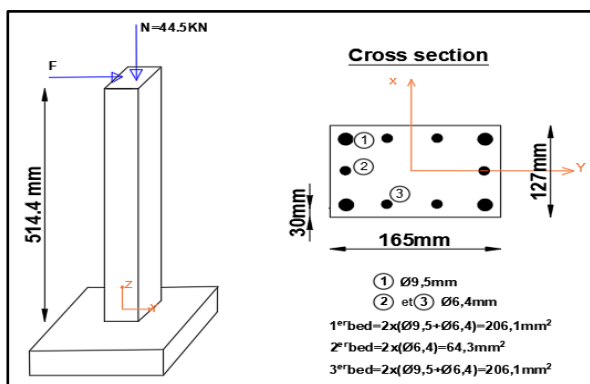


Figure 9. Geometric data of the Low Mohle column

3.2 A. Tai example

This is a bi-embedded gantry, defined by the CEA and built and tested by the CEBTP research department (A.A. Tai., 1981). The geometric characteristics of the frame materials tested are given in Figure 10. The frame is subjected to monotonic lateral load H and permanent axial loadings applied to the beam.

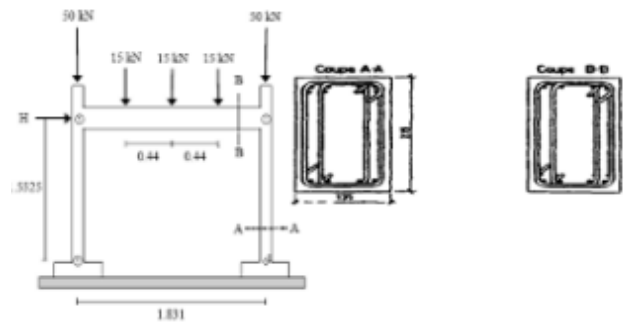


Figure 10. Geometric characteristics of the frame materials tested

The cross sections of the frame are defined by:

- Section A-A: 0.135 x 0.135 cm, (8T8 + 2T6)
- Section B-B: 0.235 x 0.135 cm, (8T8 + 4T6)

The characteristic of materials is giving in Table 2:

Characteristics of concrete	
Elasticity module E	MPa 32,5.10 ³
Compressive strength f_{cj}	33
Tensile strength f_{tj}	3.3

Table 2.a Characteristics of concrete

Characteristics of Steel	
Elasticity module E	MPa 213.10 ³
Compressive strength σ_s	475

Table 2. b Characteristics of steel

The non-linear analysis based on the finite element method is obtained using the program "Thin Beam 2" (Belhocine et al., 2023). To validate it, the results obtained with "Thin beam 2" are compared with the experimental studies carried out by Low and Tai.

The "push-over" analysis is carried out by applying a distribution of incremental lateral forces to the CEBTP-CEA frame until the displacement at the top reaches a defined target displacement. The global resolution method is of the step-by-step type, taking into account any nonlinear behavior, both material and geometric. The capacity curve of the structure ($V_{base} - U_t$) is determined using the ETABS software (Hamou., 2007). The properties of concrete and steel materials have been introduced. The nonlinear behavior laws of concrete and steel materials used are the models of (Mander et al., 1988) for concrete and (Park and Paulay, T., 1975) for steel to establish the moment-curvature laws associated with the flexural plasticization of plastic hinges.

Beams and columns are modeled by elements with linear elastic properties. Their nonlinear behavior is reflected by the introduction of plastic hinges at the sections likely to plasticize (Mohamed, Nour El-Din Abd-Alla., 2007). The formation of plastic hinges is limited to the ends of columns and beams (ATC 40., 1996, FEMA 273., 1997).

4. RESULTS AND DISCUSSION

4.1 Numerical Result with the program “Thin beam 2”

4.1.1 Low column results: Figure 11 and Table 3 illustrate the difference between the numerical result obtained with the program "Thin beam 2", the numerical results of Low, and the experimental curve. The comparison between our numerical results and the experimental results from the studies carried out by Low is very satisfactory because the curves obtained are very similar, which shows that the program "Thin Beam 2" works well.

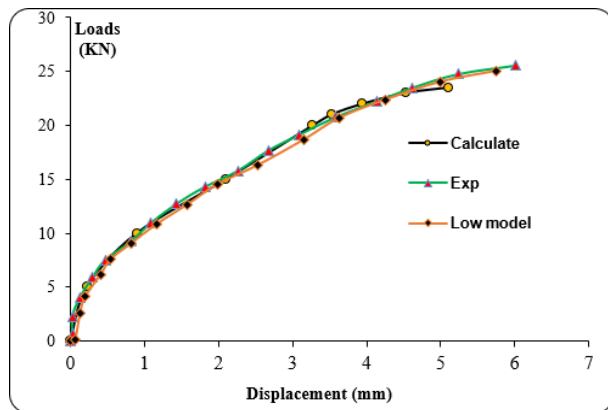


Figure 11. Loads-deflections curves of the Low Moehle column

	Exp	Calculate	Low model
F_{max}	KN 25.53	KN 23,5	KN 25,52
D_{max}	mm 6,001	mm 5,11	mm 6,23

Table 3.a The maximal loads and displacements of Low column

Difference between (%)	
Experimental and calculate	Calculate and Tai model
5,7	4,87
20,7	19

Table 3.b Comparison between experimental results, low model, and calculation with “Thin beam 2”

4.1.2 Tai frame results: Figure 12 and Table 4 illustrate the difference between the numerical result obtained with the program "Thin beam 2", the numerical results of Tai, and the experimental curve. It can be seen that the behavior of the frame is well approximated by the calculation. The simulation shows a good estimate of the maximum loads: 5.7% difference between the experimental study and ours, 4.87% between our study and that of Tai. The difference between the maximum displacement

obtained with our study and that of the experimental study is 20.7%. This is due to the fact that the transverse reinforcements are not taken into account in our study (stirrups). This is why there is a slight difference between the calculated curve and the experimental curve. The behavior is slightly more rigid than the experimental one.

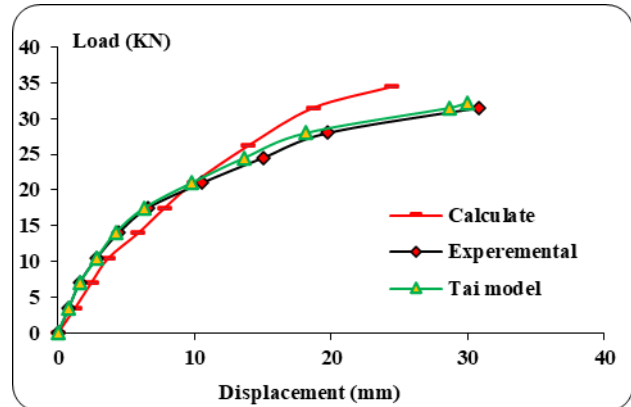


Figure 12. Load-displacement curve of the frame (CEBTP)

	Experiment	Calculate	Tai model
F_{max}	KN 32.5	KN 34,47	KN 32,79
D_{max}	mm 30,84	mm 24,44	mm 30,19

Table 4.a The maximal loads and displacements of the Tai frame

Difference between (%)	
Experimental and calculate	Calculate and Low model
7,75	7,91
14,84	17,72

Table 4.b Comparison between experimental results, Tai model, and calculation with “Thin beam 2”

With:

F_{max} (KN): Maximal loads

D_{max} (mm): Maximal displacement

4.1.2.1 Frame ductility using “Thin beam 2”: Ductility is a very important criterion that describes the behavior of structures and their degree of energy dissipation in the plastic domain (Thomas, Paulay., 1999).

The global ductility of the frame using the curve load-displacement given by the program “Thin beam 2” is calculated by the factor μ , which represents the maximum displacement achieved over the elastic displacement of the frame. The elastic limit state of the frame corresponds to: $D_e = 6.1\text{mm}$. The maximum displacement achieved by the frame is: $D_{max} = 24.441\text{cm}$. Or a global ductility, which can be estimated by the ratio:

$$\mu_D = D_{max} / D_e = 4.01$$

4.2 Numerical results using “push-over” analysis

The capacity curve for the frame studied is shown first (Figure 13), giving the shear force at the base of the frame as a function of the lateral displacement at the head. Next, we present the capacity spectrum in the format ($S_a - S_d$) (Figure 14), obtained by “pus-hover” analysis using ETABS software.

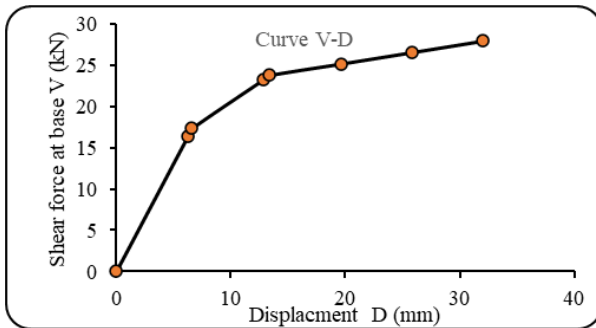


Figure 13. Capacity curve of the frame ($V_{base} - u_t$)

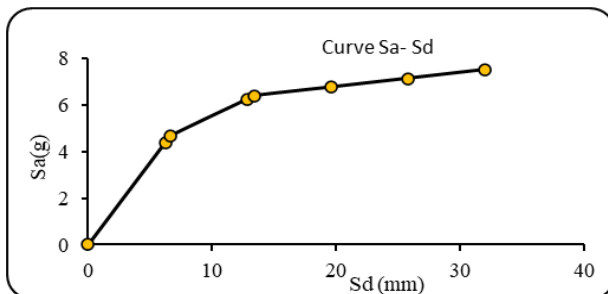


Figure 14. Capacity curve for the equivalent system of the single-DDL of the frame ($S_a - S_d$)

Figure 15 and Table 5 show the difference between the numerical result obtained with the program “Thin Beam 2”, the numerical results of A.A. Tai, ETABS, and the experimental curve. It can be seen that the behavior of the frame is well approximated by the calculation. The simulation shows the difference in maximum loads: 18.98% between ours and that obtained with the ETABS software. There was also a difference of 23.67% between the maximum displacement obtained with our study and the ETABS study.

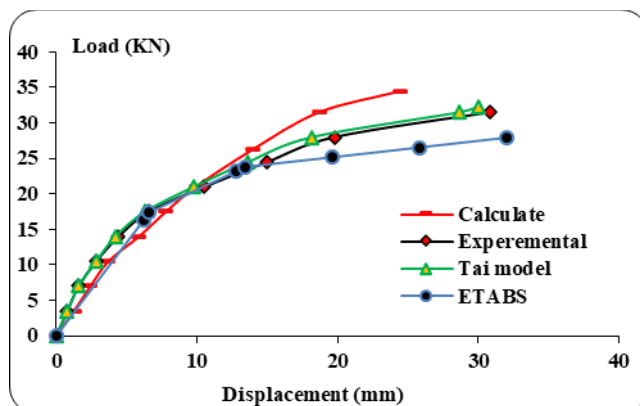


Figure 15. Different curve loads-displacement of the frame

Difference (%)	
Experimental and calculate	Calculate and ETABS
5,7	18,98
20,7	23,67

Table 5. Comparison between the different results

4.2.1 Frame ductility using “Push-over”: The global ductility of the frame using the curve load- displacement giving by ETABS is calculated by the factor μ , which represents the maximum displacement achieved over the elastic displacement of the frame. The elastic limit state of the frame corresponds to: $D_e = 6.21\text{mm}$. The maximum displacement achieved by the portal frame is: $D_{max} = 32.021\text{cm}$

Or a global ductility, which can be estimated by the ratio μ :

$$\mu = D_{max} / D_e$$

$$\mu_D = D_{max} / D_e = 5.15$$

4.2.1 Determining the performance point of the frame: In an Excel file, “Log seismic” (Figure 16), we introduce the capacity curve in the ($S_a - S_d$) format of the CEBTP frame (capacity spectrum), obtained with ETABS and its idealized curve. We also introduce the 5% damped spectrum calculated using equation (18) of (RPA.2003) and the family of reduced $\beta_{eff} > 5\%$ spectra, reduced using formula (19) of the (ATC 40., 1996) method ($\beta_{eff} = 9.68\%$ to 14.93%). The intersection of the line formed by the reduced spectra with the capacitance spectrum defines the performance point. The performance point of the CEBTP CEA frame is obtained from Figure 15:

S_a : 4.66g
 S_d : 6.598cm
 S_a : Spectral acceleration 1DDL system (g)
 S_d : Spectral displacement system 1 DDL (m)

The performance point ($S_d = 6.57\text{ cm}$) coincides with the elastic displacement of the frame ($D_e = 6.21\text{ cm}$), so the behavior of the frame has not reached the plastic phase and the frame can withstand the load applied to it.

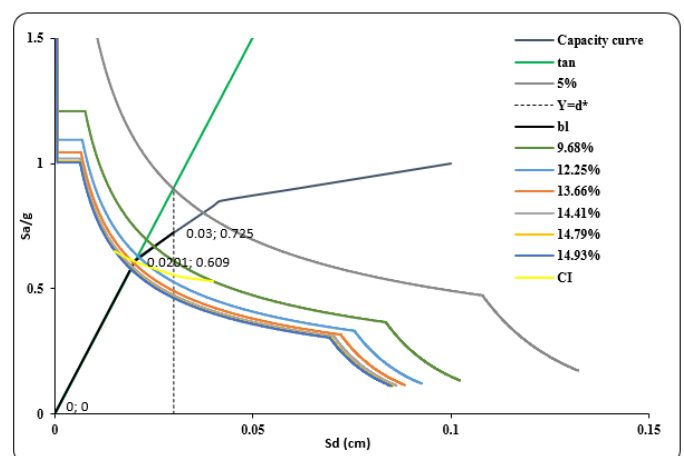


Figure 16. Performance point for the frame CEBTP CEA equivalent system

5. CONCLUSION

Two nonlinear analysis methods are presented. The first method allows the numerical simulation of the nonlinear behavior of structural elements of the reinforced concrete column-beam type under monotonic static loading, increasing until failure, as summarized in the program "Thin Beam 2." It is applied to the CEBTP CEA frame (France) and the low Moehlle column for validate the program. The results obtained with the program, by comparison between the numerical curves obtained and the experimental ones, are very satisfactory.

The second is the nonlinear "push-over" method, applied to the same frame (CEBTP) to assess its seismic response and performance. We proposed the spectral capacity method, recommended by the American code ATC 40, where we introduced the capacity spectrum of (RPA, 2003). The equivalent system capacity curve (S_a-S_d) of the frame (calculated using ETABS software) and the reduced response spectra were presented. Then, by entering them into an elaborate Excel file called "Log seismic," we were able to observe the performance point of the frame and calculate its global ductility.

After analyzing the behavior of the frame with the two methods, it was found that the two load-displacement curves using the two methods are close and that the frame exhibits moderately ductile behavior ($\mu_D = 4.01$, $\mu_D = 5.15$), which is indicative of a less advanced state of plasticization. The performance point ($S_d = 6.57$ cm) coincides with the elastic displacement of the frame ($D_e = 6.21$ cm), so the behavior of the frame has not reached the plastic phase, and the frame can withstand the load applied to it.

Therefore, the evaluation of seismic demands is of paramount importance in the para-seismic design of structures and in the evaluation of the vulnerability of existing structures, which requires the use of calculation methods that take into account post-elastic effects for the accurate prevention of seismic demands.

In terms of perspectives, we can envisage taking into account:

- Displacements due to shear forces
- Apply the pushover method to a structure and assess its performance under seismic stress.

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