# Nonlinear Analysis of the Seismic Response of a Reinforced Concrete Structure Built in High Seismic Region in Algeria

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**SUMMARY:**
The objective of this paper is to analyze the seismic response of a reinforced concrete structure dimensioned according to Algerian seismic rules. First, we expose the approach of nonlinear time history method. Then, we describe a ten levels reinforced concrete building braced by shear walls, located in high seismicity region in Algeria. This structure is subjected with the regard to different seismic records. Finally, we analyze and interpret the seismic response in terms of the deformability, shear strength, flexural strength and the bearing capacity under the compressive stress as well as the local ductility. The results obtained have shown that the identified structure represents a case of satisfied nonlinear behavior under the medium seismic recording intensity contrary to that obtained under higher earthquake of El Centro, which remains unfavorable and requests a constructive improvement in the Algerian seismic rules.

*Keywords: seismic response, building, nonlinear dynamic analysis, seismic recording.*

## 1. INTRODUCTION

The structures located in northern Algeria are often subjected to the risk of damage induced by severe seismic actions. However, the intensity of seismic forces acting on a building is not conditioned only by the characteristics of seismic motion, but also by various structural properties and the type of bracing. In seismic areas, the types of bracing often used are the shear walls, which show their capacity of resistance and ductility. Many investigations have been conducted to examine the nonlinear seismic response of reinforced concrete structures braced by shear walls. Memari et *al.* (Memari et *al.* 2001) conducted a seismic study applied to 32-story reinforced concrete building. They used linear and nonlinear dynamic analysis using developed software programs. The variable parameters are the formation of plastic hinges and inter-storey drift. They concluded that the limits of inter-story drift in the seismic codes do not predict necessarily the degree of damage of this type of construction during an earthquake. In other study, Boivin et *al.* (Boivin et *al.* 2010) evaluated the seismic performance of 12-storey ductile concrete shear wall system designed according to 2005 National building code of Canada. A nonlinear analysis of this building has been conducted. They showed that the shear capacity is well underestimated compared with that predicted by the later code, which increases the risk of shear failure.

Although the northern Algeria is known for its high seismic activity, few studies have been conducted on seismic behavior of moderately high structures, designed by Algerian seismic rules (RPA99/v2003, 2004). The aim of this paper is to analyze the seismic response of a reinforced concrete building braced with shear walls which is widely used in northern Algeria. The analysis is done by using the nonlinear method under some real seismic records, to highlight the parameters that control the seismic response of the structure.

# 2. RESISTANT FORCES AND DUCTILITY IN SHEAR WALL

## 2.1. Behavior laws of materials

### *2.1.1. Behavior laws of concrete*

According to Eurocode 2 (Eurocode2, 2004), the concrete is defined in mechanical terms by its characteristic compressive strength noted $f_{ck}$. At the ultimate limit state, Eurocode 2 adopts a nonlinear diagram shown in Figure 2.1. This diagram is defined by the relationship of stress and strain as follows:

$$\sigma_c = \begin{cases} f_{cd}\left[1-\left(1-\frac{\varepsilon_c}{\varepsilon_{c2}}\right)^n\right] & pour \quad 0 \leq \varepsilon_c \leq \varepsilon_{c2} \\ f_{cd} & pour \quad \varepsilon_{c2} \leq \varepsilon_c \leq \varepsilon_{cu2} \end{cases} \tag{2.1}$$

Where $f_{cd}$ is the calculated value of the compressive strength of concrete, expressed by the following relationship :

$$f_{cd} = \frac{\alpha_{cc} \cdot f_{ck}}{\gamma_c} \tag{2.2}$$

Where $\varepsilon_c$ is the concrete compression deformation and $\varepsilon_{c2}$ is the concrete compression deformation at the maximum stress $f_{cd}$, expressed in Eurocode2 (Eurocode2, 2004).

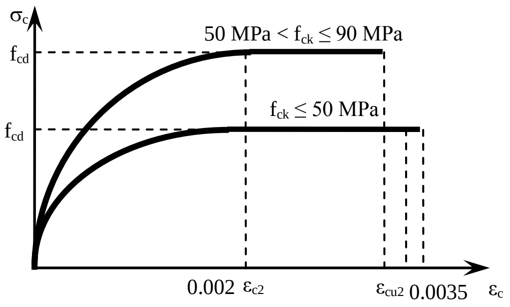



**Figure 2.1.** Stress - Strain diagram of unconfined concrete for (Eurocode2, 2004)

### *2.1.2. Behavior laws of steel*

According to Eurocode 2, the steel is characterized by its yield stress of the longitudinal reinforcement $f_{yk}$, and its stress reinforcement $f_{yd}$,expressed by the relation:

$$f_{yd} = \frac{f_{yk}}{\gamma_s} \tag{2.3}$$

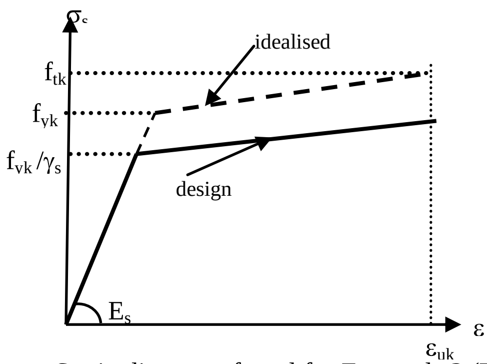



**Figure 2.2.** Stress - Strain diagram of steel for Eurocode 2 (Eurocode2, 2004)

The design of reinforced concrete sections, according to Eurocode 2, at ultimate state used a conventional stress-strain diagram defined in Figure 2.2. This diagram is characterized by an elastic strain of the reinforcement at maximum load, $\varepsilon_{sy,d} = f_{yd}/E_s$, , and the ultimate strain of reinforcement at maximum load $\varepsilon_{uk}$.

## 2.2. Axial force and bending moment resistance

Figure 2.3 represents the cross section of a shear wall with typical distributions of the stresses for various positions of the neutral axis. This section is subjected to a bending moment M and an axial compressive force N. The position of the neutral axis c corresponds to the ultimate curvature after crushing of the concrete.

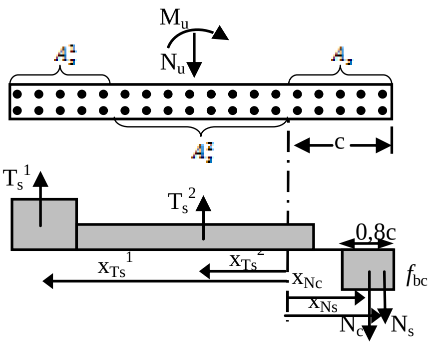



**Figure 2.3.** Behavior of a shear wall section under the different stresses

Consider the equilibrium equations of forces acting on the section shown in Figure 2.3. The axial force ($N_u$) and the bending moment resistance ($M_u$) are expressed as follows:

$$N_u = A_s^1.f_{yk} + A_s^2.f_{yk} - 0.8.c.f_{cd} - A_S.f_{yk} \tag{2.4}$$

$$M_u = A_s^1.f_{yk}.x_{T_s^1} + A_s^2.f_{yk}.x_{T_s^2} + 0.8.c.f_{cd}.x_{N_c} + A_S.f_{yk}.x_{N_s} \tag{2.5}$$

With $A_s^1, A_s^2$, et $A_S$ are the reinforcement sections for edge area, medium area and edge compressed area respectively. $x_i$ are the distances between the application point of force and the neutral axis.

## 2.3. Shear resistance

To establish the equation for the shear resistance, we have used the analogy of Mörsch (Boeraeve, 2008).To ensure a ductile failure, Eurocode 2 requires that the shear resistance ($V_{Rd}$) will be expressed as follows:

$$V_{Rd} = Z.cot\theta.\left[l_w.\nu_1.f_{cd}.sin^2\theta + \frac{A_{sw}.f_{ywd}}{s}\right] \tag{2.6}$$

Where:

- $\theta$ : Angle of inclination of diagonal compressive stress,
- $A_{sw}$ : area of one leg of the transverse reinforcement
- $l_w$ : length of cross-section of wall,
- $f_{ywd}$ : design value of the yield strength of the transverse reinforcement,
- $s$ : spacing of transverse reinforcement.
- $Z = 0.9\ l_w$ : Reduction coefficient, and $\nu_1 = 0.6\left(1 - \frac{f_{ck}}{250}\right)$.

### 2.4. Curvature ductility factor

The curvature ductility factor is defined by the ratio of maximum curvature ($\varphi_u$) to the curvature at yield ($\varphi_y$) (Park, 1989). For local ductility in section of wall, Eurocode 8 suggests the following formulas:

$$\mu_\varphi = \frac{l_w(0.0035 + 0.1\alpha_1 \omega_w)}{2\varepsilon_{sy.d} l_{w0}(\nu + \omega_v)}\left[\left(1 - \frac{\varepsilon_{c2.c}}{\varepsilon_{cu2.c}}\right)\left(\frac{f_{cc}}{f_{ck}}\right)\left(\frac{b_{w0}h_{w0}}{b_w h_w}\right) + 2\omega_v\right] \tag{2.7}$$

Where :

$l_w$, $b_w$, $h_w$ :are the dimensions of the wall,
$l_{w0}$, $b_{w0}$, $h_{w0}$ :are the dimensions in boundary elements of the wall,
$f_{cc}$ : characteristic resistance of confined concrete,
$\alpha_1$ : is a shape coefficient of the wall,
$\varepsilon_{cu2,c}$ : ultimate strain of confined concrete
$\varepsilon_{sy,d}$ : design value of tension steel strain at yield;
$\omega_v$ : Mechanical ratio of vertical web reinforcement,
$\nu$ : is the reduced axial force,
$\omega_w$ : Mechanical volumetric ratio of confining reinforcement,

## 3. NONLINEARE ANALYSIS METHOD

The method of nonlinear time history analysis depends on direct integration of motion equations, where the algorithms containing elastic-plastic deformations of the structure are used. This analysis is conducted using real seismic recordings. In this method, the structure is simulated by a vertical console as shown in Figure 3.1. The hysteresis system used for nonlinear behavior law is the modified Takeda 1970 (Wilson, 2002).

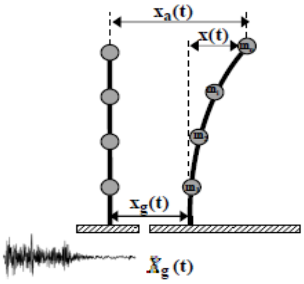



**Figure 3.1.** Schematic representation of the oscillator under seismic excitation

The nonlinear motion equation is expressed by:

$$M.\ddot{X} + C.\dot{X} + F_s(X) = -M.\ddot{X}_g(t) \tag{3.1}$$

Where :

M : matrix masses $m_i$,
C : matrix damping of system,
$F_s(X)$ : the variable rigidity of the structure.
$X_a$ : absolute displacement of the structure.
$\ddot{X}_g$ : ground acceleration
$\ddot{X}, \dot{X}, X$ are the acceleration, velocity and relative displacement of the structure.

The solution of equation (3.1) is obtained by the numerical integration method, using SAP 2000/v14.1 software (CSI, 2009).

## 4. DESCRIPTION OF THE BUILDING AND LOADING

### 4.1. Description of building

The selected structure is a building including ten storeys. It is located in a high seismic region in Algeria. Its total height is H= 36,5m, height of ground floor is $h_{RDC}$= 5m and the other floors are $h_{ec}$= 3,5m. The geometric data are illustrated in Figure 4.1 and Table 4.1. The initial characteristics of the materials are 30MPa for the compressive strength of concrete $f_{ck}$, and 500 MPa for yield strength of the reinforcements $f_{yk}$.

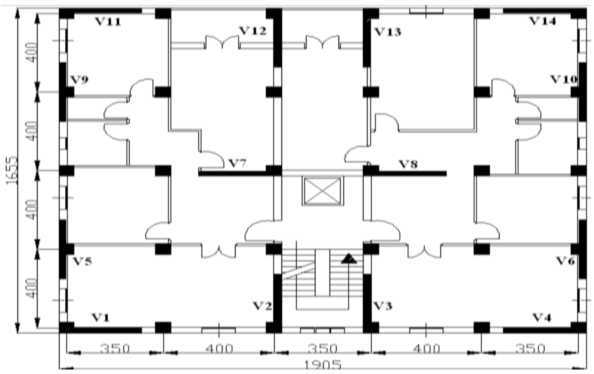



**Figure 4.1.** Plan of the structure

**Tableau 4.1.** Geometric Data of the Structure Elements

| | | level | Dimension | reinforcement |
|---|---|---|---|---|
| columns | | 1, 2, 3<br>4, 5, 6<br>7, 8, 9, 10 | 55x55 cm²<br>45x45 cm²<br>35x35 cm² | 4T25+12T16<br>4T20+8T16<br>8T16 |
| beams | | All levels | 30x40 cm² | 10T12 |
| Shear walls | Columns edge | 1, 2, 3<br>4, 5, 6<br>7, 8, 9, 10 | 40x40 cm² | 4T20+4T16<br>8T16<br>8T14 |
| | wall | 1, 2, 3<br>4, 5, 6<br>7, 8, 9, 10 | e= 25 cm | T12 (s=10cm)<br>T12 (s=15cm)<br>T12 (s=20cm) |

### 4.2. Seismic loading

A set of seismic of accelerograms was established to perform this analysis. It includes four seismic records of both directions (N-S) and (W-E), where their characteristics are shown in Table 4.2.

**Tableau 4.2.** Summary of Characterizing Chosen Earthquakes

| Seisme | Date | Magnitude | Location of registration | pic ground acceleration (PGA) | Direction |
|---|---|---|---|---|---|
| El Asnam – Algeria (NRC, 1984) | 08/11/1980 | 5,6 | EL ASNAM (Algeria) | 0,211 g | N-S |
| Boumerdes - Algeria (Laouami et al. 2003) | 23/05/2003 | 6,8 | KADDARA (Algeria) | 0,340 g | W-E |
| Loma Prieta - USA (Ayrault et al.2001) | 17/10/1989 | 7,1 | SAN ANDREAS (USA) | 0,346 g | N-S |
| El Centro - (Choi et al.2005) | 19/05/1940 | 7,2 | EL CENTRO (USA) | 0,349 g | N-S |

## 5. SEISMIC RESPONSE ANALYSIS OF THE BUILDING

### 5.1. Deformability Analysis

The deformability is studied in terms of displacements inter-storey and in the top of structure. According to seismic rules, the inter-storey displacement for the structure should not exceed 0.01h and 0.0075h for the Algerian seismic Rules (RPA99/v2003) and Eurocode8 (Eurocode8, 2005) respectively.

Figure 5.1 shows a comparison between the admissible inter-storey displacement recommended by the Algerian seismic Rules (RPA99/v2003) and Eurocode8 (Eurocode8, 2005), and the inter-storey displacement obtained under different seismic recording identified previously. The structure analysis shows that inter-storey displacement for different seismic records are less than the deformations limited by the admissible displacement 0.01h for RPA99/v2003, and 0.0075h for Eurocode 8, except for the displacements of seven upper storey displacement obtained under the seismic record of El Centro, which exceed that of Eurocode 8, and those obtained for the upper floors of Loma Prieta earthquake, which reached the allowable limit of Eurocode 8. Is to be noted that the deformation field is largely high for the results obtained under the local seismic records in Algeria, however for high seismic intensity the deformation is always unfavorable. In this context, it is important to note that the Algerian seismic Rules (RPA99/v2003) allow a wide deformability equals 1% of storey height comparing with that of Eurocode 8, which allows 0.75% of the storey height.

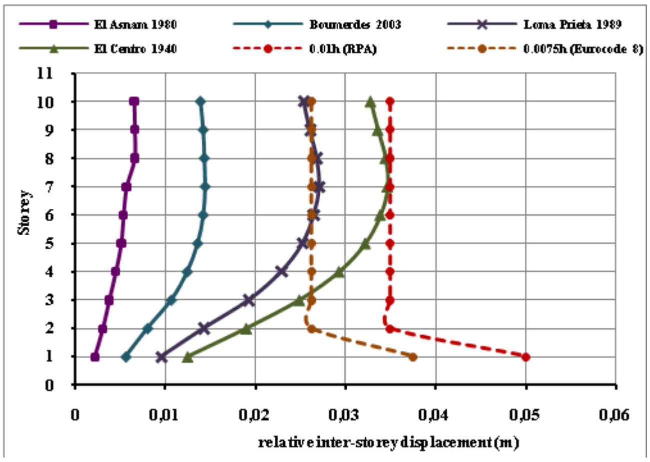
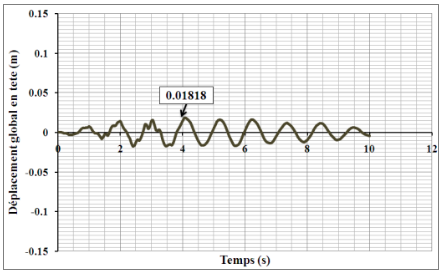


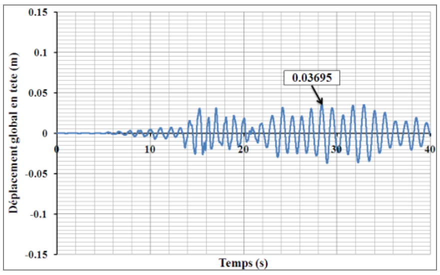

**Figure 5.1.** Inter-story displacement of structure

Figures 5.2 illustrate the results of global displacements obtained at the top of the identified structure by the nonlinear analysis under different seismic records mentioned above. From this figure, we can observe that the maximum displacement under the seismic record of El Asnam is reached at 0,01818m for t=4,10s, Boumerdes (Algeria) at 0,03695m for t= 28,94s, Loma Prieta at 0,08273m for t=5,39s, but under the earthquake of El Centro the displacement is reached at 0,1388m for t=12,36s. It can be said here that the total displacement at the top of the structure under El Centro earthquake exceeds four times that of Boumerdes with twice less duration, and more than seven times that of El Asnam for a duration with four times more. Despite the difference in time reaching the maximum displacement, we can consider that the local earthquakes have non-explosive behavior.

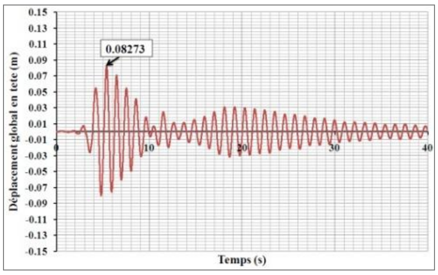
We can notice that the structure under local earthquakes has a large possible deformability, because the displacement peak under the seismic record of El Asnam (Algeria) and Boumerdes (Algreria) does not exceed 0.05% and 0.1% of the total height of the structure respectively.



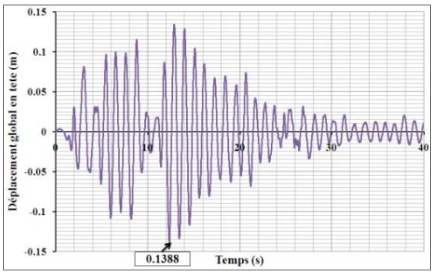

(a) El Asnam earthquake 1980.




(b) Boumerdes earthquake 2003.




(c) Lome Prieta earthquake 1989.

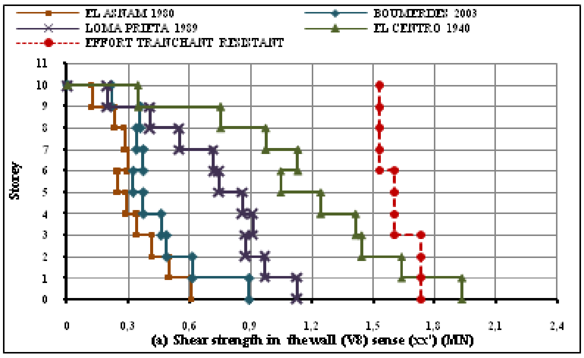



(d) El Centro earthquake 1940.

**Figure 5.2.** Global displacement at the top of the structure in function of time

## 5.2. Shear analysis

The results of nonlinear dynamic analysis of shear force obtained for shear walls (V8-direction xx') and (V2-direction yy') of the structure are illustrated in Figure 5.3, which presents a comparison between the shear resistance determined by the method described in paragraph (2.3), and those assessed under different seismic loads considered. From this figure, it is clear that the shear forces under different seismic records used are widely less than to the shear resistance, except the shear at the base under El Centro earthquake which is slightly higher. We note that the shear forces obtained under the seismic records with medium intensity at the base do not exceed 50% of the value of the resistance shear contrary to that of El Centro which exceeds 10% of shear resistance value. The same observation is shown for wall (V2) of the direction (yy ') (see Figure 5.3, b). Also, the structure has a good performance of the shear resistance the local earthquakes in Algeria, comparatively under that of El Centro earthquake.



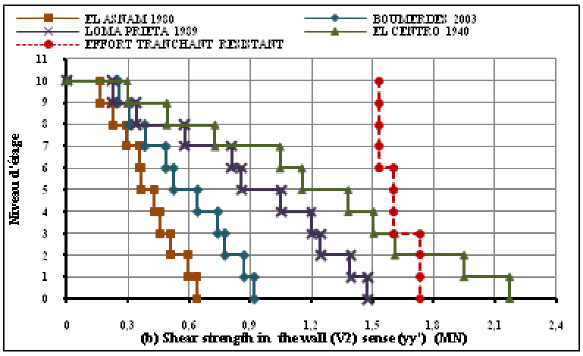




**Figure 5.3.** Shear strength for each floor of the structure

## 5.3. Flexural strength analysis

The variations of curves of bending moments under different seismic records on the walls (V8-direction xx') and (V2-sense yy') are shown in Figure 5.4, which illustrates a comparison between the bending moment determined by the method described in paragraph (2.2), and those measured under different considered seismic loads. It is shown in Figure 5.4 that the bending moments under the different seismic records considered are largely less than the curve of the resistance moment, except for El centro records at the base. This shows that the shear walls dimensioned according to RPA99/v2003 ensures a very adequate bending resistance under the local seismic records in Algeria; however, this behavior under strong earthquakes such as El Centro requires a large improvement for integration of other recommendations and constructive arrangements.

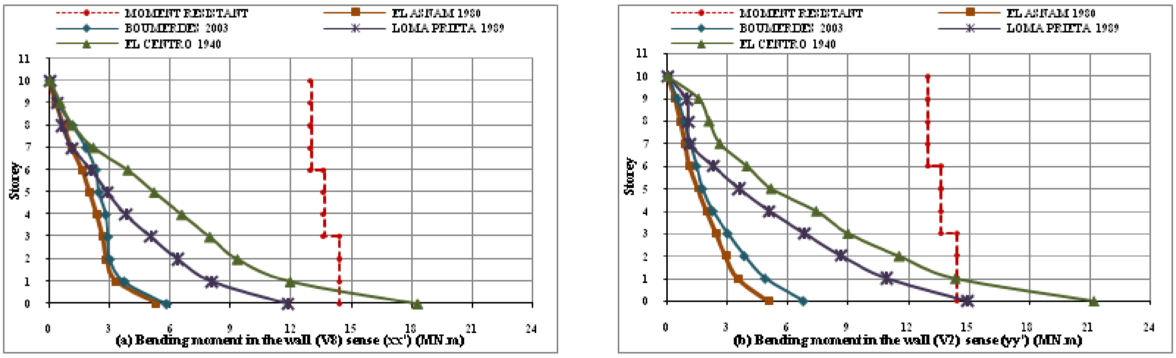



**Figure 5.4.** Bending moment for each floor of the structure

## 5.4. Bearing capacity analysis

The variation of compression axial forces of the walls (V8-direction xx ') and (V2-sense yy') of the structure is illustrated in Figure 5.5, which shows a confrontation between the axial force resistance determined by the method described above in paragraph (2.2) formula (2.4), and the axial forces of compression determined by the nonlinear analysis method under different given seismic records in this study.

From this figure, we can see that the compressive forces in the wall (V8-direction xx ') under the different local seismic records of El Asnam and Boumerdes (Algeria) remain largely less than axial force resistance. Moreover, the bearing capacity at the base of this shear wall, under the seismic record of El Centro is still insufficient where the structure shows a suffering in the resistance at the base. Regarding (V2-sense yy ') wall, the same seismic response is found over the whole height of the shear wall and under the different seismic records.

As a result, the bearing capacity of the studied structure under compressive stress remains unfavorable under the major earthquake intensities, which needs a considerable improvement in the seismic design, contrary to that obtained in the local earthquakes.

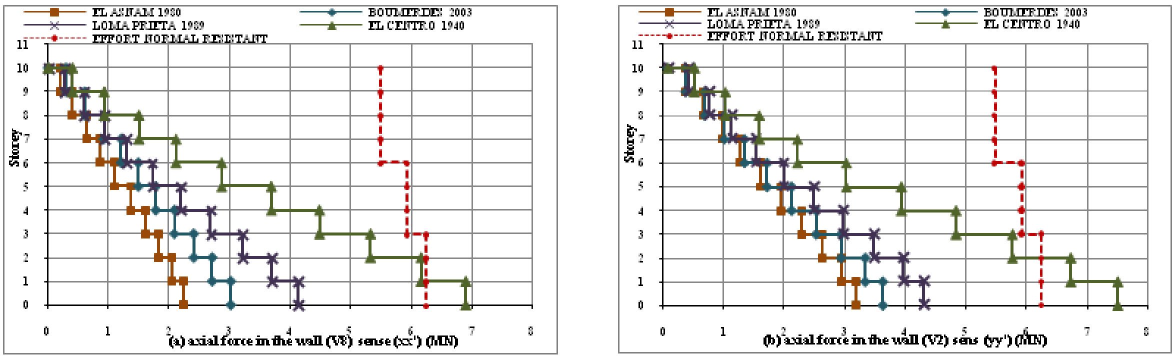
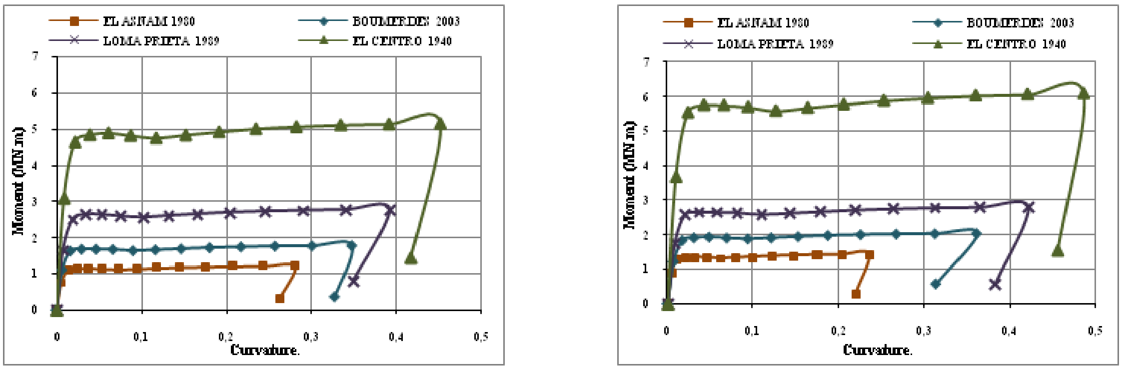



**Figure 5.5.** Axial force for each floor of the structure

### 5.5. Analysis of the local ductility at the base of the structure

From Figure 5.6, the curvature ductility factor of V8 wall and V2 wall varies from 16,25 to 13,98 and from 17 to 14,02 for various seismic records respectively (see Table 5.1). From these values, it is noted that the ductility is largely sufficient in these walls; however, the ductility decreases significantly with the increasing of the earthquake magnitude.




(a) Moment-Curvature in the wall (V8) sens (xx')

(b) Curvature in the wall(V2) sens (yy')

**Figure 5.6.** Moment-curvature diagram

**Tableau 5.1.** Summary of the Curvature Ductility Factors

| | | El Asnam | Boumerdes | Loma Prieta | El Centro |
|---|---|---|---|---|---|
| (V8) shear wall of direction (XX') | $\varphi_u$ | 0.24 | 0.30 | 0.34 | 0.39 |
| | $\varphi_e$ | 0.02 | 0.02 | 0.02 | 0.03 |
| | $\mu_\varphi$ | **16.25** | **15.02** | **14.17** | **13.98** |
| (V2) shear wall of direction (YY') | $\varphi_u$ | 0.20 | 0.31 | 0.37 | 0.42 |
| | $\varphi_e$ | 0.01 | 0.02 | 0.03 | 0.03 |
| | $\mu_\varphi$ | **17.00** | **15.66** | **14.62** | **14.02** |

## 6. CONCLUSION

From the analysis of the seismic response of a ten level building dimensioned according to Algerian rules, in terms of the inter-storey displacement, shear strength, flexural strength, bearing capacity and the local ductility; we can draw the following conclusions:

- The identified structure shows a large deformation under the local seismic records, however under major earthquakes such as El Centro states the performance of the deformability requires a very special structural design.
- The structure identified states a very appropriate shear strength under local earthquakes of medium intensity (Boumerdes- Algeria), however, under high-intensity earthquakes (El Centro) the structure showed a very poor seismic performance.
- The shear walls dimensioned according RPA99/v2003; provide a very adequate flexural strength under the local seismic records; however this behavior under strong earthquakes such as of El Centro needs a large improvement by integrating other constructive recommendations and provisions.
- The bearing capacity of the studied structure under the compression force remains unfavorable under the major earthquake intensities, which requires a considerable improvement in the seismic design.
- The structure studied has presented a large sufficient local ductility.

In general, the results obtained showed that the identified structure has an acceptable performance under the local seismic records of the construction dimensioned by Algerian seismic Rules

(RPA99/v2003). Moreover, under high intensity seismic recording like that of El-Centro, this performance requires exceptionally a constructive improvement in the recommendations of seismic code.

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