Analysis of Seismic Behavior of A Few Reinforced Concrete Bridge's Columns of the East-West Highway in Algeria

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SUMMARY:
The objective of this paper is to analyze the seismic behaviour of a few columns of bridges realized on the east-west highway in Algeria, knowing that these bridges were designed before the application of the Algerian seismic code of bridges. First the method for nonlinear dynamic analysis of bridges was outlined. Next, we describe the evaluation method of resistant forces in reinforced concrete columns, based on numerical approaches. In the same context, the criteria of the strength and ductility of bridge piers in modern seismic codes were presented. In this study, the behaviour of columns of bridges, in terms of bending moment, shear and the ductility was analyzed under several seismic recordings in critical areas of columns of three bridges. The results obtained herein showed that the seismic behaviour of the considered columns was widely satisfactory under local seismic recordings even under strong earthquakes.

Keywords: behaviour, columns, bridges, nonlinear, dynamic

1. INTRODUCTION

During the last decade, Algeria realized the highway project for the cost of 11.4 billion dollars. This project covers over 1200 km joining the eastern to the western borders of Algeria in the northern part of the country. This region is characterized by a high level of seismic activity. Due the rugged topography this infrastructure contains more than 530 bridges (Smahi, 2011). Such a number of bridges is important and requires a detailed study of their seismic behaviour using nonlinear dynamics method. Real seismic recordings served as data for this investigation. Analysis of the performance of bridges constructed before the appearance of the new seismic code in terms of strength and capacity were necessary for the highlight of their endurance during a random strong earthquake. The objective of this research is to analyze the seismic behaviour of few columns of three bridges constructed on this highway, knowing that these bridges were designed before the implementation of the new Algerian seismic code recommendations for bridges RPOA2008 (RPOA-08, 2009).

2. ANALYTICAL PROCEDURE FOR NONLINEAR DYNAMIC RESPONSE OF BRIDGES

The behaviour of bridge columns is generally modeled using simple beam elements. Due to the large amplitude response, coupled inelastic deformations can occur in these elements. In fact, the modelling of the hysteresis behaviour of reinforced concrete columns, the stiffness degradation and strength loss are the main issues. Consequently, nonlinear beam elements that characterize really the inelastic hysteresis behaviour of the columns must be used (Chen et *al.* 2003). Figure 2.1 shows a bridge with a single degree of freedom.

Equation of motion for a degree of freedom for a bridge system expressing the dynamic equilibrium at time t is given by:

$$M\ddot{u}_t + C_t\,\dot{u}_t + K_t u_t = R(t) \tag{2.1}$$

Where:

M, C_t and K_t are the mass, damping and stiffness matrices, respectively, and $R(t)$ is the applied dynamic load vector.
$\ddot{u}_t$, $\dot{u}_t$ and u_t are the absolute acceleration, absolute velocity and absolute displacement Vectors, respectively.

When the structural system is nonlinear, the coupled equation of motion (2.1) is solved using step-by-step integration methods.
Considering the time interval Δt starting at time t and assuming that the stiffness and damping matrices at time t, K_t and C_t, can be applied over the full time interval, the equations of motion are obtained in the incremental form

$$M\Delta\ddot{u}(t) + C_t\,\Delta\dot{u}(t) + K_t\Delta u(t) = \Delta R(t) + MB\Delta\ddot{u}_g(t) \tag{2.2}$$

Where

$$\begin{cases} \Delta\ddot{u}(t) = \ddot{u}(t+\Delta t) - \ddot{u}(t) \\ \Delta\dot{u}(t) = \dot{u}(t+\Delta t) - \dot{u}(t) \\ \Delta u(t) = u(t+\Delta t) - u(t) \end{cases} \tag{2.3}$$

And

$$\begin{cases} \Delta R(t) = R(t+\Delta t) - R(t) \\ \Delta\ddot{u}_g(t) = \ddot{u}_g(t+\Delta t) - \ddot{u}_g(t) \end{cases} \tag{2.4}$$

To solve this equation, the generalized Newmark method (Wilson, 2002) is applied. For our analysis, the nonlinear method of software SAP2000/v14.1 (CSI, 2009) is used.

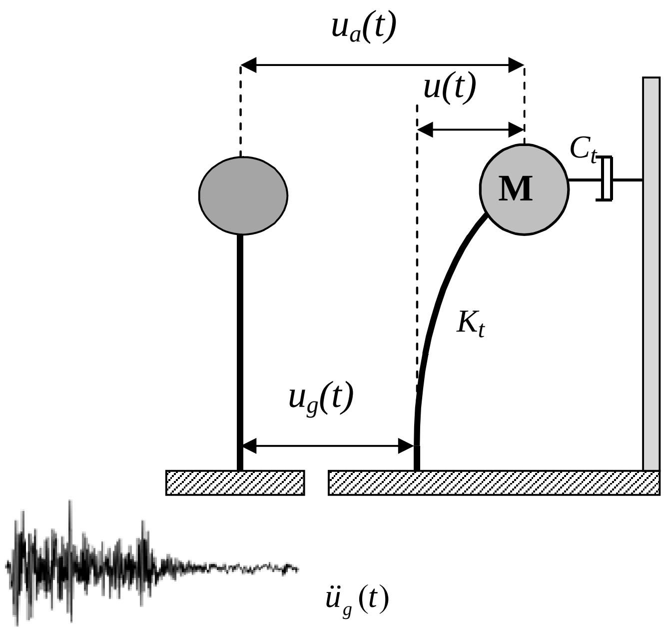


Figure 2.1. Bridge with a single degree of freedom**.**

3. RESISTING FORCES OF LATERALLY CONFINED COLUMNS AND DUCTILITY

The nonlinearity of the concrete material is integrated in the nonlinear analysis using the model of confined concrete behaviour of Mander et *al.* (Mander et *al.* 1988). Behavioural law of reinforcing steel, it using a conventional stress-strain diagram adopts by the Algerian concrete code (CBA-93 1994).

Based on the theoretical study of Mander`s model (Mander et *al.* 1988), for concrete subjected to uniaxial compressive loading and confined by transverse steel, the theoretical method proposed by Samra et *al.* (Samra et *al.*, 1996) is used to generate moment diagram curvature characteristics. The analysis method is summarized in the following steps:
1- Computing the cover`s strain in concrete and the maximum strain based on Mander`s model,
2- For compatibility, the deformations are taken equal to those of the extreme compressed fiber strain and the depth of compressed zone,
3- Using similar triangles theorem in strain diagram to find the strain in each longitudinal bar, and force for each bar. Dividing the compressed concrete into two parts, one part is confined the other is not. Dividing also the section into small rectangular layers, and calculating the concrete force for each layer, then summing up the forces to have the concrete force and the moment in the rebars.

In the analysis of the nonlinear behaviour of section elements, we use the curvature ductility, which is expressed by the available curvature ductility factor (Park, 1989):

$$\mu_c = \frac{\varphi_u}{\varphi_e} \tag{3.1}$$

Where φ_e is the curvature at yielding and φ_u is the ultimate curvature.

4. STRENGTH CRITERION AND CONDITIONS OF LOCAL DUCTILITY

According to the Algerian seismic code of bridges RPOA2008 (RPOA-08, 2009), the strength of sections in plastic hinge must satisfy the following relation:

$$\gamma_o S_d \leq \frac{1}{\gamma_R} R_d \tag{4.1}$$

with:

R_d Calculated strength section.

S_d Acting stress for the calculated combined action,

γ_o Coefficient of overcapacity, allow to take into account in empirical manner the design capacity of the structure

γ_R Coefficient providing additional security to brittle failure.

According to the Eurocode 8 (Eurocode 8, 2000), the ductile behaviour of the compression zone of concrete must be provided along the regions of potential plastic hinges. The confinement shall not be required in the columns with thin walled sections, under ultimate seismic loads, if the curvature ductility factor can be:

$$\mu_c = [13] \tag{4.2}$$

The maximum deformation due to the compression of the concrete does not exceed 0.35%.

5. DESCRIPTION OF STUDIED BRIDGE COLUMNS

The majority of bridges of East-West Highway in Algeria are similar, with a slight difference in the soil foundations, which did not affect significantly the columns reinforcement of these bridges. For this reason, three bridges located in the province of Chlef were chosen. Chlef is a zone of high seismicity in Algeria, and the studied bridges were seen as most representative in the field of engineering. The first bridge in question is 180 m long and 5 spans of 36 m each span. The deck is composed of a series of seven post-tensioned prestressed concrete beams. The two central columns are mentioned by P11FC and P12FC. The second bridge is 216 m long, with six spans of 36 m each span. The two central columns are selected and noted P21FC and P22FC. While the third bridge is 144 m long, 4 span of 36 m each span, the two columns are studied and noted P31FC and P32FC. Table 5.1 summarizes the necessary data concerning the geometry, the longitudinal and transverse reinforcement for the columns of three considered bridges.

According, to the basic data of the three bridges shown in Table 5.1, the constructive provisions concerning the spacing and the percentage of transverse reinforcement as well as the longitudinal reinforcement, verify widely those percentages recommended by Eurocode 8 (Eurocode8, 2000) and RPOA2008 (RPOA-08, 2009). This observation allows avoiding in advance all premature failures due to the minimum required constructive provisions.

Table 5.1. Databases Of Considered Columns Of Three Bridges

bridge	1		2		3	
column	P11FC	P12FC	P21FC	P22FC	P31FC	P32FC
Height (mm)	10545	12045	6655	9549	10515	5509
diameter (mm)	1600	1600	1600	1600	1600	1600
coating (mm)	70	70	70	70	70	70
Spacing at the base (mm)	100	100	100	100	100	100
Spacing at mid-heigh (mm)	200	200	200	200	200	200
transverse reinforcement A_t (mm^2)	401.92	401.92	401.92	401.92	401.92	401.92
longitudinal reinforcement (mm^2)	41799.	41799.	40192	41799.7	41799.	28938.
Percentage of longitudinal reinforcement (%)	2.08	2.08	2	2.08	2.08	1.44

6. ANALYSIS OF SEISMIC BEHAVIOR OF A FEW BRIDGES COLUMNS

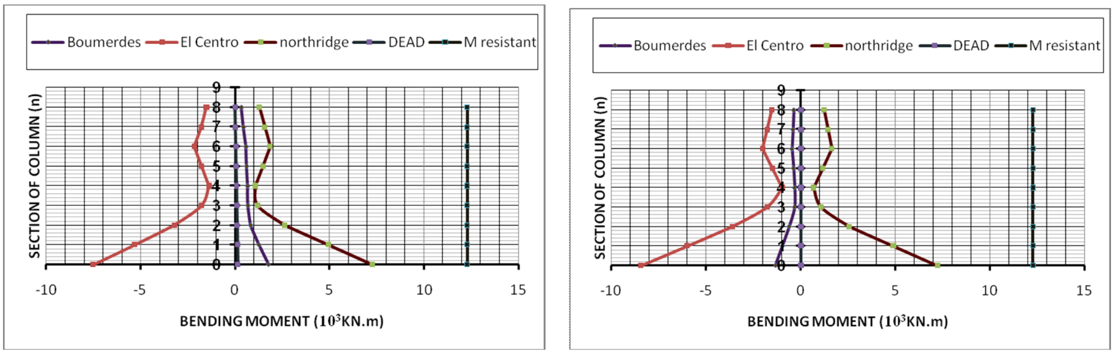
In this section, detailed highlights on the parameters that control the seismic behaviour of bridges columns, in terms of flexural, shear, bearing capacity and ductility were emphasized.

6.1. Analysis of the bending strength

The bending of columns is usually represented by the bending moment in critical sections. In this context, the bending behaviour of columns identified in paragraph 3 is shown in Figures (6.1, 6.2 and 6.3) under various seismic recordings of Boumerdes (Algeria) (2003), El Centro (1940) and Northridge (1994).

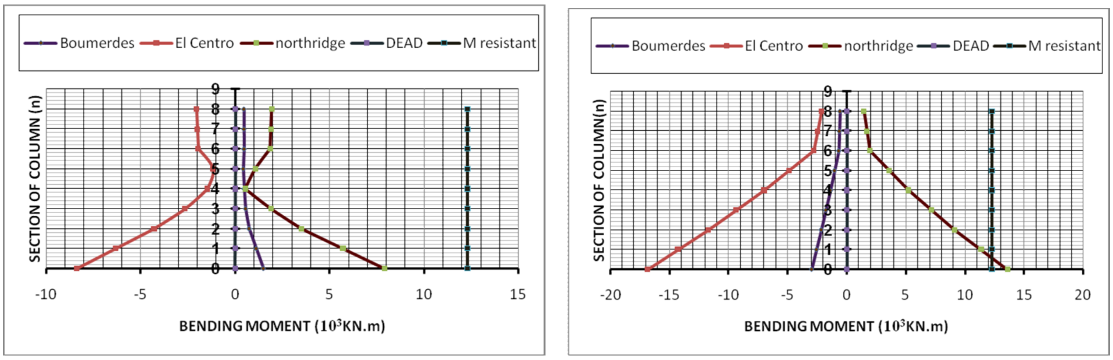
Figure 6.1a shows the flexural behaviour of the central columns (P11FC) of Bridge (1). From this Figure, we see that the bending moments in the various seismic recordings remain strictly less in absolute values than the resisting moment even under strong seismic recordings of El Centro. From this figure, we see that the intensity of the resisting moment of this column is equal to 12.2×10^3 KNm, greatly exceeding the maximum moment in the seismic recording of El Centro estimated at 7×10^3 KNm, which is more than 4 times that of Boumerdes quantified to 2×10^3 KNm, while in the midst of this column the various seismic recordings produced almost the same amount of the moment in absolute values, because this location is the inflection point of the column. The same observation is

noticed in Figures 6.2 and 6.3 on the columns of the bridges 2 and 3, where the flexural behaviour of these columns is largely verified. These figures also show that the flexural behaviour of these columns remains in a very favourable state, because the sections of longitudinal and transverse reinforcement required in these columns are more sufficient as shown in Table 5.1.
Therefore, from the bending strength point of view, the columns of the three bridges in question have evidently a good bending behaviour under different seismic recordings, even under strong recordings similar to those of El Centro.



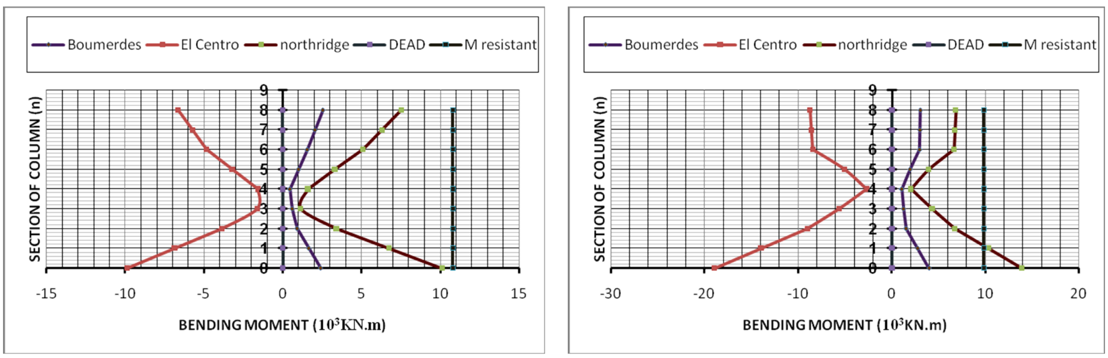
a- P11FC b- P12FC

Figure 6.1. Variation of bending moment under different seismic recordings along the height of the central columns of bridge 1



a- P21FC b- P22FC

Figure 6.2. Variation of bending moment under different seismic recordings along the height of the central columns of bridge 2



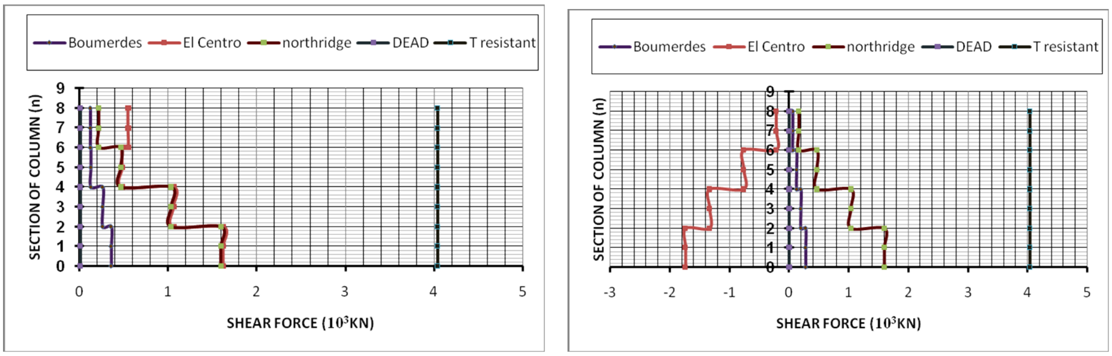
a- P31FC b- P32FC

Figure 6.3. Variation of bending moment under different seismic recordings along the height of the central columns of bridge 3

6.2. Analysis of shear bridges columns

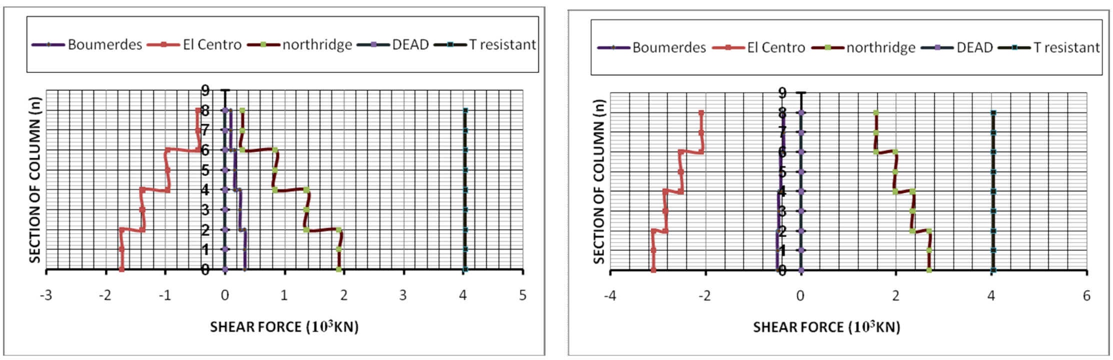
The behaviour of bridge columns under shear is shown in Figures (6.4, 6.5 and 6.6), under the different seismic recordings described above. These Figures show a comparison between the resisting shear and the variation of shear recorded during the seismic activity of: Boumerdes (2003), Northridge (1994) and El Centro (1940).

Figure 6.4 illustrates the shear behaviour of the central column (P11FC) of the bridge (1). According to the Figure, we see that the shear forces under different seismic recordings remain strictly less than the shear resistance. This indicates the importance of shear resistance that possesses this column. The same figure schematizes clearly the variation of maximum shear forces at the base of the column (P11FC). From this figure, we see that the value of the shear resistance of this column equal to $4x10^3$ KN, far exceeds the values of shear forces in the various recordings, even the maximum effort in recording seismic of El Centro estimated at $1.64x10^3$ KN, which exceeds more than 4 times that obtained in the recording of Boumerdes quantified at $3.8\ x10^3$ KN. On the central column (P12FC) of the bridge, the same observation is recorded in Figure 8, where the shear behaviour of this column is widely verified. Concerning the columns (P21FC) and (P22FC) of the bridge (2), Figure 6.5 (a and b) illustrates that their behaviour to shear strength remain similar to bridge (1) with a difference, about 400KN to 800KN. In addition to the Bridge (3), Figure 6.6 (a and b) illustrates the same observations as previous columns.
Consequently, from the point of view of the shear behaviour at the base, the columns of the three bridges in question are performing well in base shear under different average and high seismic recordings.



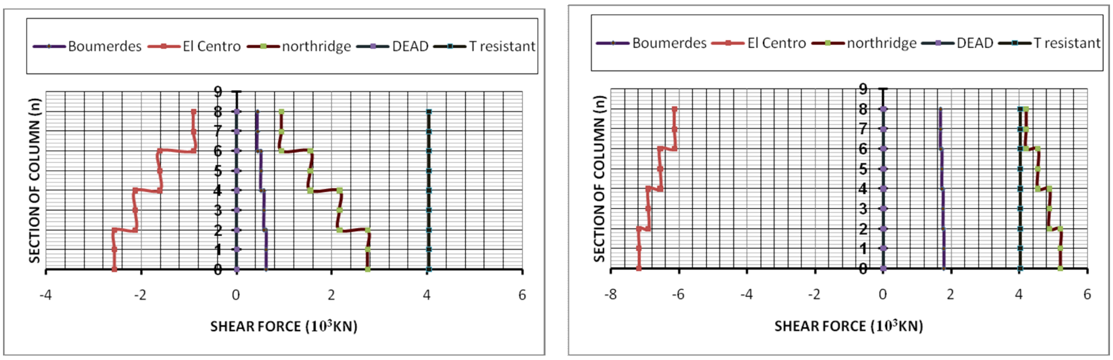
a- P11FC b- P12FC

Figure 6.4. Variation of the shear force along the height under different seismic recordings of the central columns of bridge 1



a- P21FC b- P22FC

Figure 6.6. Variation of the shear force along the height under different seismic recordings of the central columns of bridge 2



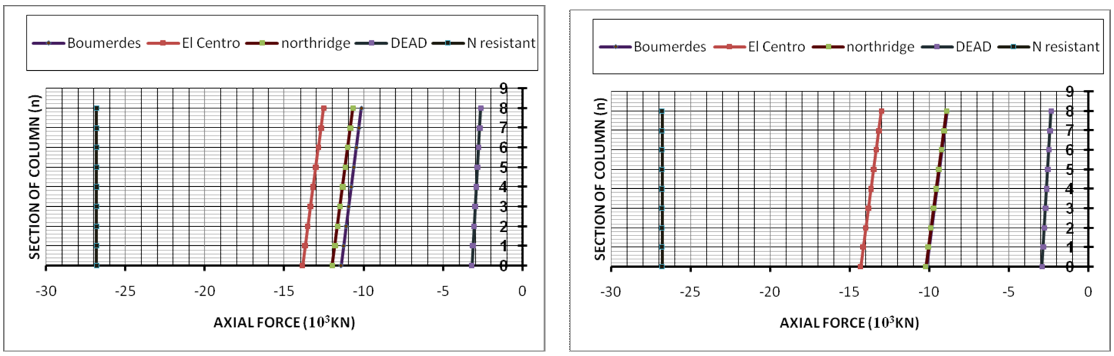
a- P31FC b- P32FC

Figure 6.6. Variation of the shear force along the height under different seismic recordings of the central columns of bridge 3

6.3. Analysis of bearing capacity of bridge columns

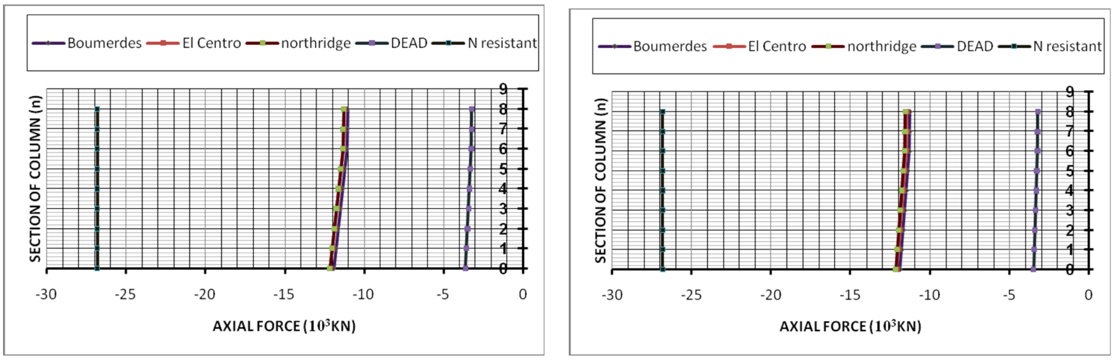
The variation of bearing capacity represented by the axial force at the base of each column, under different seismic recordings described above is illustrated in Figures 6.7, 6.8 and 6.9. In the following, we try to analyze this capacity in each column considered.
Figure 6.7 (a) shows the comparison of axial forces under different seismic recordings, of the column (P11FC) of the bridge (1). According to this Figure, we see that the axial forces under different seismic recordings remain strictly less than the axial resistance; although under strong seismic recordings as El Centro. In the same figure, we observe that the intensity of the axial resistance of the column exceeds 26.8x10^3KN, against the axial forces in the various recordings range from 11x10^3KN t to 14 x10^3KN, indicating that the column possesses a good bearing resistant seismic records treated. Figure 11b shows that the bearing capacity of the center column (P12FC) is similar to the center column (P11FC). Figure 6.7b shows that the bearing capacity of the center column (P12FC) is similar to the center column (P11FC). Concerning the bearing capacity of the bridge columns (2), Figure 6.8 (a and b) shows similar behaviour as in bridge 1, with a difference in the intensity of axial forces. For column P31FC and P32FC, the same observation is noticed in Figure 6.9 (a and b) as described in earlier columns, despite the significant increase in the intensity of axial forces in the various recordings.

Consequently, the columns of the considered bridges maintain a good reserve of bearing capacity after different seismic records.



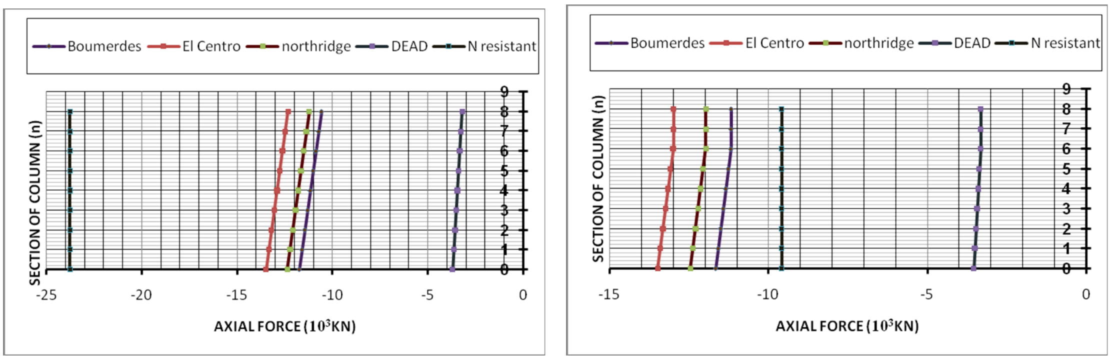
a- P11FC b- P12FC

Figure 6.7. Variation of the axial force along the height of central Columns under different seismic recording of Bridge 1



a- P21FC b- P22FC

Figure 6.8. Variation of the axial force along the height of central Columns under different seismic recording of Bridge 2



a- P31FC b- P32FC

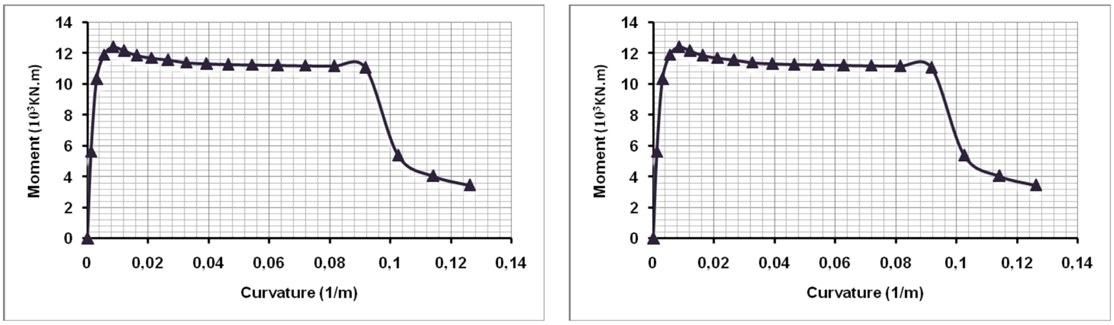
Figure 6.9. Variation of the axial force along the height of central Columns under different seismic recording of Bridge 3

6.4. Analysis of ductility of bridges columns

Figures 6.10, 6.11 and 6.12 show the variation of Moment-Curvature Relationship in critical sections of the columns of bridges considered in this study. Table 6.1 summarizes the significant values for the curvatures at yield and ultimate states, and the corresponding curvature ductility factor available. From this table, we note that the factors of the ductility curvature available for the six columns are considered strictly larger than 13, the minimum ductility curvature factor recommended by Eurocode 8, as clarified in paragraph 4. This result clearly highlite the potential of local ductility possessed by these columns. Consequently, the columns of the considered bridges columns are widely ductile.

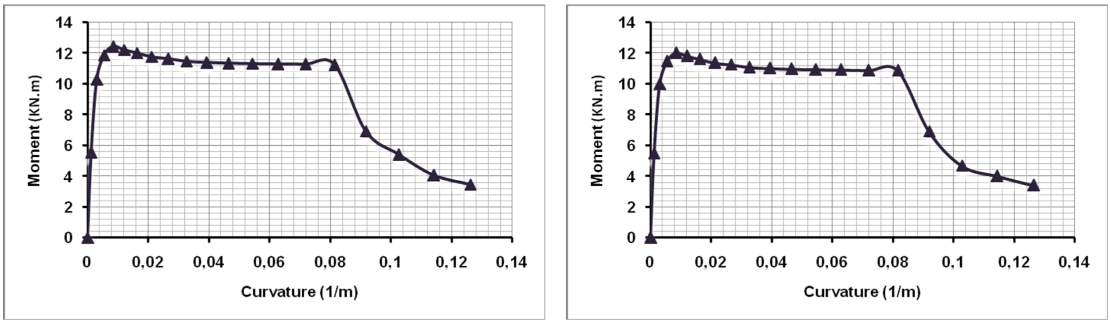
Table 6.1. Different Values For Significant Curvature At Yield And Ultimate And Their Availabe Curvature Ductilty Factor Corresponding

columns	Curvature at yield	Curvature at ultimate	Avalaible curvature ductilty factor	ductility factor of EC8
P11FC	0.003016	0.088	29.17	13
P12FC	0.003016	0.088	29.17	13
P21FC	0.003016	0.079	26.19	13
P22FC	0.003022	0.077	26.48	13
P31FC	0.003016	0.088	29.17	13
P32FC	0.003016	0.0785	26.03	13



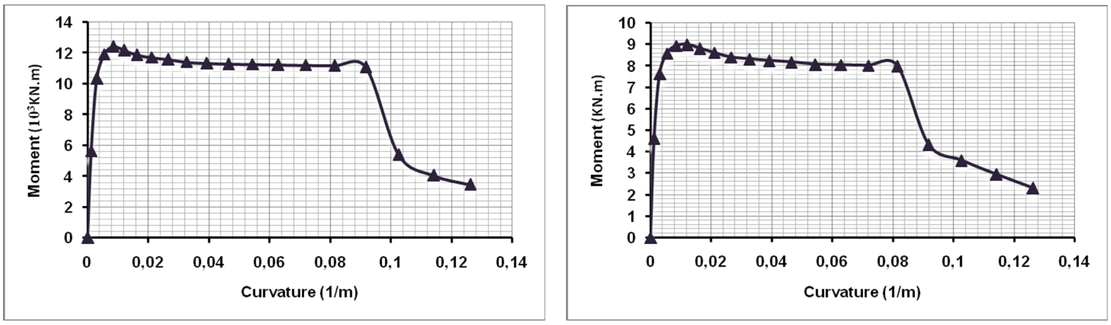
a- P11FC b- P12FC

Figure 6.10. Moment-Curvature Relationship for the columns of Bridge 1



a- P21FC b- P22FC

Figure 6.11. Moment-Curvature Relationship for the columns of Bridge 2



a- P31FC b- P32FC

Figure 6.12. Moment-Curvature Relationship for the columns of Bridge 3

7. CONCLUSION

From the analysis of seismic behaviour of columns of the three bridges of the East-West Highway in Algeria, enables us to identify the following remarkable conclusions:

1. From the point of view of flexural strength, it's evident that the columns of the three selected bridges have good bending behaviour under different earthquake records.
2. Concerning the shear, the columns of the three treated bridges show a good behaviour at the base under different seismic records (medium and high).
3. From the point of view of bearing capacity under different earthquake records, the studied columns are located within a highly reliable interval of security.
4. The considered bridge columns are widely ductile.

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