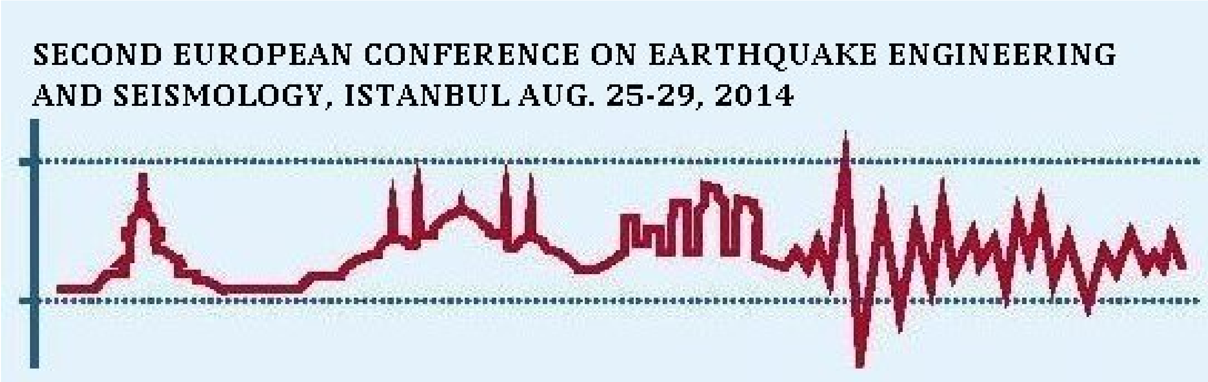


# SEISMIC PERFORMANCE OF EXISTING STRATEGIC BUILDINGSWITH CASE STUDY

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## ABSTRACT

The northern part of Algeria is prone to natural disasters. Some of the most intense earthquakes of the world have occurred in Algeria. Algeria has highly populous cities including the capital Algiers, located in zones of high seismic risk. Typically, the majority of the constructions in these cities are not earthquake resistant. Thus, any future major earthquake would turn into a major disaster. After the last May 21$^{st}$ 2003 Boumerdes Ousalem and Bechtoula (2003) major earthquakes a huge loss of lives due to collapse of buildings was observed. Most of buildings were designed and built during the colonial era, before the enforcement of the Algerian seismic code. The government realized that the seismic vulnerability assessment and retrofit of weak buildings became a pressing need against impending future major earthquake. Therefore, the Algerian authorities decided to invest firstly into the most important and densely populated towns such as Algiers the capital, Annaba, Constantine and Oran in seismic upgrade strengthening and retrofitting older strategic existing buildings designed basically for gravity loads which suffered from earthquakes causing enormous damage. In doing so, structural seismic vulnerability assessment of this category of buildings has been considered. Structural analysis is performed on the basis of site investigation (inspection of the building, collecting data, materials, general conditions of the building, etc.), and existing drawings (architectural plans, structural design, etc). The aim of these structural seismic vulnerability assessments is to develop guidelines and a methodology for rehabilitation of existing buildings.

This paper will provide insight to the structural seismic vulnerability assessment of the police department located in Constantine, according to the new Algerian seismic code RPA 99/version 2003. The analysis was carried out by means of, non linear static analysis using pushover procedure and non linear dynamic analysis using temporal analysis.

## INTRODUCTION

Seismic activity is high in almost all the northern part of Algeria. The majority of the population, the buildings and the facilities are concentrated in this region. Recently, many strong earthquakes occurred in this region, causing enormous losses in human lives, loss of houses and damage to infrastructure. In order to reduce this risk, the government decided firstly to protect the strategic existing buildings, from the adverse effects of future expected earthquakes IZIIS/CGS (1993). In

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doing so, seismic vulnerability study of this category of buildings has been considered. One of the most important strategic existing buildings is the Police department Center of Constantine. The seismic vulnerability assessment of this building is carried out in this paper.

## 1. DEFINITION OF SEISMIC HAZARD

Seismic hazard is a physical phenomenon associated with the occurrence of an earthquake and the resulting ground motion that may produce adverse effects on humans and affect the normal activities of people. The main purpose of seismic hazard analysis is to provide parameters for estimating seismic risk. In seismology, risk may be defined as the probability of earthquakes with a certain magnitude or greater striking at least once in a region during a specific period. The seismic hazard analysis in Constantine region has been performed on the basis of Seismic sources defined in the light of the most recent results obtained from seismotectonics analyses carried out in North Algeria. Results are presented as relationships between values of maximum expected peak ground acceleration (PGA) at bedrock and annual frequency of exceedance, and maps of hazard for return periods of 100 years and 500 years synthesis of the seismic hazard study of Algeria.

Amax = 0.15g, for 100 years return period.

Amax = 0.25g, for 500 years return period.

In general, the safety design criteria should be determined for two performance levels.

1$^{st}$ performance level: Slight (moderate) earthquake ground motion, can be expected to occur within the service life of the building with a corresponding return period of 100 years. The associated performance requirement is little or no damage, and without interruption of function. The response spectrum method is usually adequate.

2$^{nd}$ performance level: Strong (major) earthquake ground motion, expected once during the life of the building with a corresponding return period of 500 years. It is considered as the maximum level of ground motion for which a structure is designed. The associated performance is that the building performs without catastrophic failure and should behave in the non linear range, with a controlled level of damage. No heavy damage or collapse is allowable, and the building can be used after inspection and some minor repairs.

## 2. STRUCTURAL ANALYSIS

Structural analysis should include the basic structural systems. The primary purpose is to support gravity loads. However, buildings may also be subjected to lateral forces due to wind or earthquake. It must be able to resist most efficiently the various combinations of gravity and horizontal loadings. The non-structural elements should be controlled on the basis of obtaining principal corresponding data (story deformation, flexibility, local instability, etc.).

Structural analysis shall include real data on building structures and characteristics of structural materials, as well as existing upgrading or/ and changes in the original systems of the buildings.

### 2.1. Static and dynamic analysis

For the defined vertical and horizontal loads, linear static and dynamic analysis is performed with ETABS 2013, Wilson and Habibullah (2013) for the purpose of obtaining the natural periods and mode shapes, story stiffness, inter story drift and absolute displacements. The resultant internal couples (bending moments and torques) and resultant internal forces (shear and normal forces) are checked for existing characteristic frames.

### 2.2. Seismic analysis according to the new code RPA 99/version 2003

In terms of demand, the resultant internal efforts M, N and V of the existing building are carried out according to the new Algerian seismic code in force RPA 99/version 2003, CGS (2003). In case of an existing original data, a comparison will be made with actual data for a qualitative evaluation.

## 3. CAPACITY ANALYSIS OF THE STRUCTUE

The capacity of the existing structure will be carried out using the nonlinear static analysis (pushover), the capacity design approach which is also the actual appropriate method for estimating capacity, deformability and decision making for structures safety, and the Algerian seismic code requirements Rocha et al. (2004). Each analysis will consider the real bearing and deformability characteristics of the structures in the elastic and plastic ranges. This approach uses the theory of ultimate Limit State of reinforced concrete structures.

### 3.1. Nonlinear static analysis

#### 3.1.1. Purpose of pushover analysis

The purpose of the pushover analysis is to assess the expected performance of a structural system by estimating its strength and deformation demands, and comparing these demands to available capacities at the performance levels of interests. It is expected to provide information many response characteristics that cannot obtained from an elastic static or dynamic analysis. The model incorporates the nonlinear force-deformation of the structural components by considering material non linearity.

#### 3.1.2. Background of pushover analysis

The pushover analysis is based on the assumption that the response of the structure can be related to the response of an equivalent single degree of freedom (SDOF) system. This implies that the response is controlled by a single mode where the shape of this mode remains constant throughout the time history response. Even if both assumptions are incorrect, results carried out by several pilot studies showed a good convergence of the maximum seismic response of multi degree of freedom (MDOF) structures with dominated by the fundamental mode shape.

#### 3.1.3. Formulation of pushover analysis

The pushover analysis is performed by imposing a constant gravity loads system and an assumed lateral loads distribution over the height of the structure increased monotonically from zero to the ultimate state corresponding to the collapse of the structure. The control node is usually located at the center of mass of the roof. The system of solved equations is given by Eq. (1)

$$[K_T]\{\Delta_U\} = \{\Delta F\} \tag{1}$$

Where $[K_T]$ is the matrix of global rigidity, $\{\Delta_U\}$ the vector of incremental displacement and $\{\Delta F\}$ the vector of incremental loads.

The pushover analysis provides useful results as the capacity of the structure in terms of a shear base force versus roof displacement relationship, maximum rotation and ductility of critical structural elements expressed in the form of local damage indices, distribution of plastic hinges, etc. The changes in slope of this curve give an indication of yielding of various structural elements. The main aim of the pushover analysis is to determine member forces and global and local deformation capacity of a structure. The information can be used to assess the integrity of the structure.

### 3.2. Capacity design approach

The Capacity Design approach is currently adopted by all the modern seismic. The basic idea is to force the member to fail in a ductile manner by making the capacity of the member in other possible failure modes greater. It involves the simple application of plastic analysis on an element-wise basis. Capacity design is based on the fundamental concept that the element should not exhibit brittle failure modes and is designed to be stronger than the maximum expected stresses they possibly get from the adjacent ductile members. Hence, in order to ensure an overall dissipative and ductile behavior, brittle failure or the premature formation of unstable mechanisms shall be avoided. The envelope of yield and ultimate capacity curve is obtained using the computer program U.A.R.C.S Bozinovski and Gavrilovic (1993) and considering the following Eq. (2).

$$
\begin{aligned}
&Qy = Qy_{\min} + \delta y_{\min}\left[\sum_{i=1}^{i=N-1}\frac{Qy_i}{\delta y_i}\right] \\
&Qu = Qu_{\min} + \sum_{\delta u\min > \delta yi}\left[Qy_i + K_{2i}\left(\delta u_{\min} - \delta y_i\right)\right] + \sum_{\delta u\min \le \delta yi} K_{1i}\delta u_{\min} \\
&With: \quad K_{2i} = (Qu_i - Qy_i)/(\delta u_i - \delta y_i) \quad and \quad K_{1i} = Qy_i/\delta y_i
\end{aligned}
\tag{2}
$$

### 3.3. Capacity according to the seismic code prpa99/version 2003

According to the Algerian seismic code in force RPA99/version 2003, the damage limitation required is considered satisfied when the inter story drift displacements are less than one percent (1%Hi). So, one of the possibilities to consider the no collapse requirement is to use the limit of the inter story drift displacement as the capacity curve.

## 4. NONLINEAR DYNAMIC ANALYSIS

The nonlinear dynamic analysis is used to compute deformations, stresses and section forces more accurately by considering the time dependent nature of the dynamic response to earthquake ground motion. It is also conducted to avoid many limitations of simplified response methods. The overall objective is to develop a set of time histories that are representative of site ground motions that may be expected for the design earthquake and that are appropriate for the types of analyses planned for specific structures. According to the new concept in the Algerian seismic code, during major earthquakes, structures are allowed to undergo deformations beyond the elastic limit state to absorb deformation energy. A nonlinear dynamic time history analysis using step by step integration method is a very useful tool to determine the most appropriate realistic response of elements, and hence the performance of the whole structure. Dynamic response analysis of structures represents a numerical computation of structural systems with defined characteristics of masses, stiffness, damping, etc, and defined ranges of elastic (linear) and plastic (non linear) behavior expressed via displacements, velocities, accelerations and forces Chopra (2001). The most general approach for solving the nonlinear dynamic response of structural system is the direct numerical integration of the dynamic equilibrium equations. This involves the attempt to satisfy dynamic equilibrium at discrete equal time intervals after the solution has been defined at time zero. The solution of the nonlinear dynamic equilibrium equations is carried out in incremental form using the following Eq. (3):

$$[M]\{\Delta\ddot{U}\} + [C]\{\Delta\dot{U}\} + [K]\{\Delta U\} = -[M]\{I\}\ddot{U}_g \tag{3}$$

Where To determine the non-linear response of the structure, the D.R.A.B.S Bozinovski and Gavrilovic (1993) program is used and the bilinear model is adopted. The figure 1 represents the relationship force-displacement (F-δ).

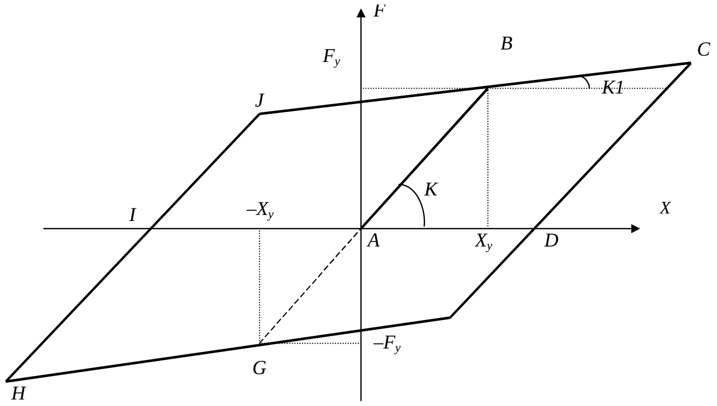


Figure 1. Bilinear model

Where: K1=(Fu-Fy)/(Xu-Xy) Lp = K2/K=αK/K.

The following sets of real ground motion records are used in the nonlinear dynamic analysis taking into account the soil conditions, frequency content and the aspect of near field and far field.

- Ulcinj (Albatros, Montenegro) N-S 1979.
- El Centro (California, USA) N-S May 8th, 1940.
-Cherchell (Algeria) N-S October 29th, 1989.

The figure 2 shows the selected recorded earthquakes.

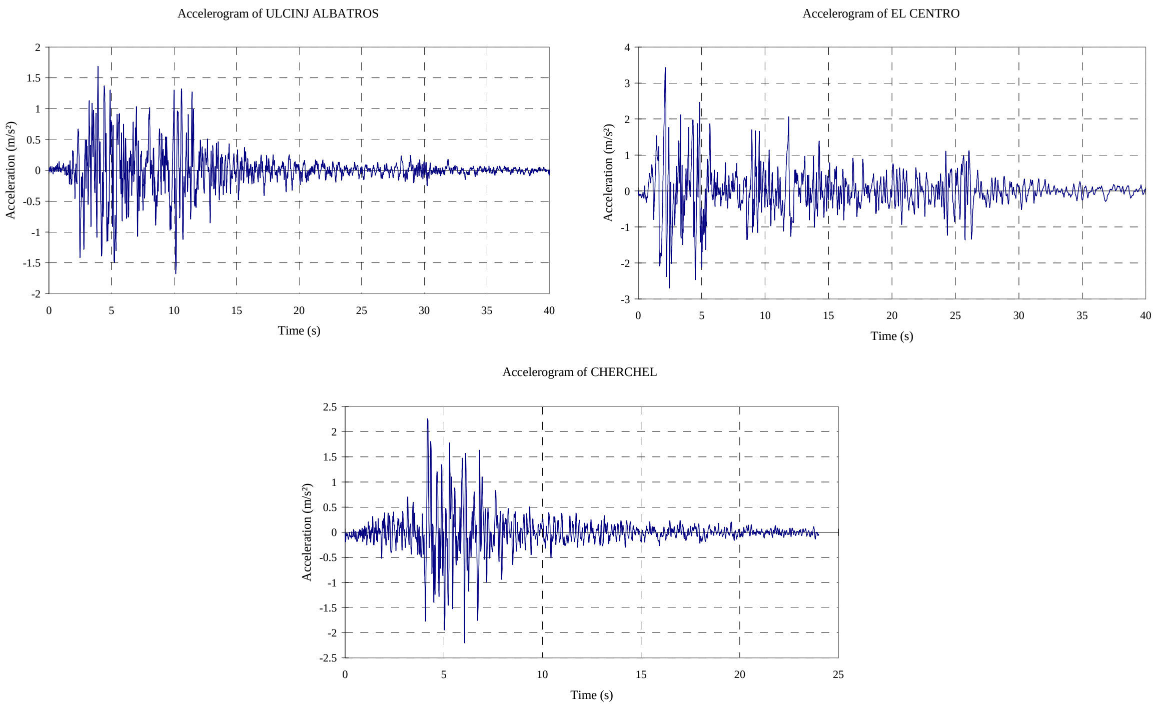


Figure 2. Selected earthquake accelerograms

## 5. LIMIT STATE

There are numerous limit states that can be considered in seismic vulnerability studies. Based on previous research, inter story drift has been well correlated with structural damage and is mostly used as the key to predict the behavior of the structure Dimova and Negro (2005). It is considered as a potential damage parameter in terms of structural response. The top displacement response is also used in order to evaluate the seismic joint in case of adjacent constructions.

## 6. VULNERABILITY ASSESSMENT

Based on the results of the demand and capacity analyses, a final decision and proposal should be submitted to the building owner.

1- If the stability criteria in accordance with the building function are satisfied, the building is safe and can still be used with no retrofitting.

2- If the stability criteria in accordance with the building function are not satisfied, strengthening is needed; otherwise the building is downgraded to a lower group.

3- If the elementary stability criteria in accordance with the building function are not satisfied, structure does not satisfy the elementary criteria, the building must be retrofitted and downgraded to a lower group or in the case demolished.

The final decision should be made after an economic cost analysis.

## 7. APPLICATION FOR AN EXISTING STRATEGIC R/C BUILDING

### 7.1. Description of the building

The building is for a security purpose, and its principal functions are the judicial investigations. It was built in 1958, with no seismic design. The building is composed by six n stories and a basement. The structural system is a reinforced concrete resisting moment frames. The thickness of the floor is 35 cm with hollow concrete blocks. The building is set on a medium soil quality. The figure 3 shows the location of the building by Google map.

Figure 3. Mathematical model of the structure

### 7.2. Mechanical characteristics of the materials

Material characteristics were defined using a range of in-situ and laboratory testing and inspection techniques CEB (1996), to obtain the necessary information.

***Concrete:***

- Characteristic compressive cylinder strength at 28 days: $f_{c28}$ = 20 Mpa.
- Design tensile strength: $\sigma_t$= 1.8 Mpa.
- Yield strain: $\varepsilon_e$= 0.002.

-Ultimate strain: $\varepsilon_u$= 0.0035.

***Steel:***

- Characteristic tensile yield strength of reinforcement: $f_e$= 400 Mpa.
- Characteristic tensile strength of shear reinforcement: $f_t$= 235 Mpa.
- Yield strain of reinforcement: $\varepsilon_y$= 0.002.
- Yield strain of shear reinforcement: $\varepsilon_e$= 0.0018.
- Ultimate strain: $\varepsilon_u$= 0.010.

### 7.3. Structural analysis

#### 7.3.1. Mathematical model

Considerable advances in computer technology and availability of increased computational resources brought more detailed approach for modeling reinforced concrete structures using finite elements. For this purpose, the structure was modeled in 3D space frames with rigid diaphragms and a fixed base, using the nonlinear computer program ETABS V.13. The figure 4 shows a three dimensional view of the existing structure.

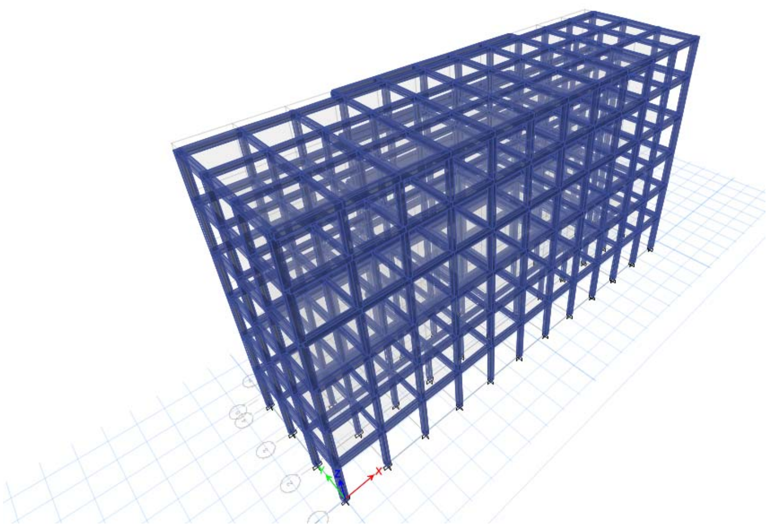

Figure 4. Three-dimensional view of the existing structure

#### 7.3.2. Dynamic characteristics of the structure

The regularity of the structure in plan and elevation gives dominant fundamental modes in the two main directions (longitudinal XX and transversal YY). Table 1 shows the two first fundamental modes.

Table1. Periods and mass factors participation

| Mode | Period (Sec) | UX (%) | UY (%) | RZ (%) |
|---|---|---|---|---|
| **1** | 1.660 | 0.007 | **82.569** | 0.873 |
| **2** | 1.516 | **85.622** | 0.017 | 0.198 |

### 7.3.3. Seismic assessment by the code RPA 99/version 2003

The total design seismic base shear force is estimated using the static equivalent force procedure, and determined from Eq. (4) given by:

$$V = \frac{ADQ}{R} W \tag{4}$$

Where:

V: Total design base shear force.
A: Design base acceleration coefficient.
T: Fundamental natural period of the structure.
W: Total seismic weight.
Q: Quality factor.
R: Behavior factor of the structure.
D: f (T), Mean dynamic amplification factor, function of the fundamental natural period.

The total force is distributed over the height of the structure in conformance with the following equation:

$$F_K = \frac{(V - F_t) W_K h_K}{\sum_{i=1}^{N} W_i h_i} \tag{5}$$

Where:

Fk: Seismic horizontal force at the Kith level.
Ft : Shall be assumed to be concentrated at the top of the structure in addition to Fn, and equal to 0.07 TV, except that Ft need not exceed 0.25 V and may be considered as zero when T does not exceed 0.7 sec.
Wk : Seismic weight at level k.
hk : Height of level k from the base.

The distribution of the lateral seismic loads and shear forces for the two main directions is presented in Table 2.

Table 2. Seismic loads and shear forces in the longitudinal (XX) and the transversal (YY) directions

| Level | $F_{xi}$ (KN) | $V_{xi}$ (KN) | $F_{yi}$ (KN) | $V_{yi}$ (KN) |
|---|---|---|---|---|
| **6** | 1316.33 | 1316.33 | 1316.33 | 1316.33 |
| **5** | 1297.84 | 2614.17 | 1297.84 | 2614.17 |
| **4** | 1063.11 | 3677.29 | 1063.11 | 3677.29 |
| **3** | 804.15 | 4481.44 | 804.15 | 4481.44 |
| **2** | 545.18 | 5026.63 | 545.18 | 5026.63 |
| **1** | 291.87 | 5318.50 | 291.87 | 5318.50 |

$$V_i^y = F_t + \sum_{k=i}^{N} F_k^y \qquad V_i^x = F_t + \sum_{k=i}^{N} F_k^x$$

$V_{yi}$: Transversal shear force at level i.
$F_{yk}$: Transversal horizontal force at level i.
$V_{xi}$: Longitudinal shear force at level i.
$F_{xk}$: Longitudinal horizontal force at level i.

## 7.4. Deformability and strength capacity

The capacity of the structure was assessed for the yield and ultimate states in terms of absolute displacement using the nonlinear static analysis (pushover), the capacity design approach and the Algerian seismic code restrictions. The lowest capacity has been considered in the following study. Tables 3 and 4 show the different capacity curves in terms of absolute displacements for the yield and ultimate states. The figures 5 to 8 represent the different capacity curves.

Table 3. Capacity curves for the yield states

| Direction | | Longitudinal XX | Transversal YY | |
|---|---|---|---|---|
| **Level** | **NLSA** | **Cap des** | **NLSA** | **Cap Des** |
| **6** | 3.08 | 16.40 | 3.39 | 14.19 |
| **5** | 2.95 | 14.64 | 3.15 | 12.70 |
| **4** | 2.60 | 12.09 | 2.70 | 10.53 |
| **3** | 2.08 | 9.33 | 2.08 | 8.26 |
| **2** | 1.43 | 6.75 | 1.32 | 5.88 |
| **1** | 0.63 | 3.15 | 0.55 | 3.33 |
| **0** | 0.00 | 0.00 | 0.00 | 0.00 |

Table4 . Capacity curve for the ultimate states

| Direction | Longitudinal XX | | Transversal YY | | XX and YY |
|---|---|---|---|---|---|
| **Level** | **NLSA (cm)** | **Cap Des (cm)** | **NLSA (cm)** | **Cap Des (cm)** | **Code** |
| **6** | 13.34 | 33.65 | 16.76 | 32.40 | 22.70 |
| **5** | 13.02 | 24.95 | 16.00 | 23.75 | 19.40 |
| **4** | 12.22 | 18.73 | 14.35 | 17.53 | 15.60 |
| **3** | 10.91 | 13.18 | 11.63 | 13.07 | 11.80 |
| **2** | 8.91 | 8.77 | 7.88 | 9.02 | 8.00 |
| **1** | 6.22 | 4.97 | 3.85 | 4.95 | 4.20 |
| **0** | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |

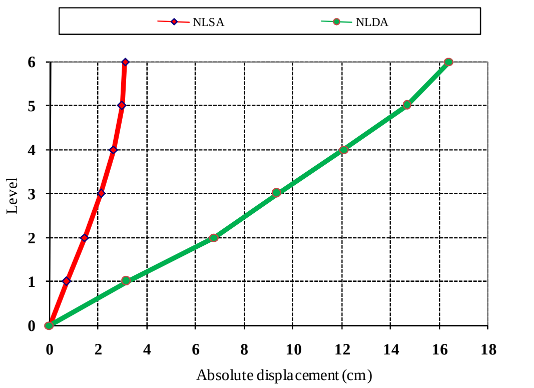


Figure 5. Longitudinal (XX) direction
Yield capacity

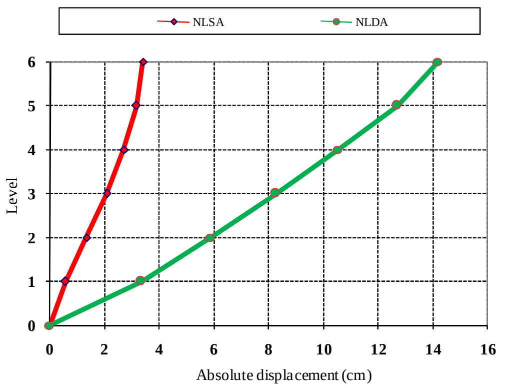


Figure 6. Transversal (YY) direction
Yield capacity

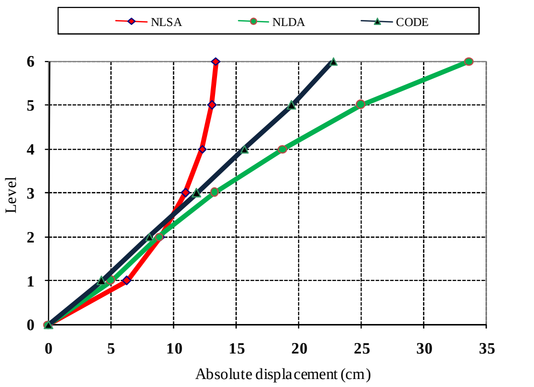


Figure 7. Longitudinal (XX) direction
Ultimate capacity

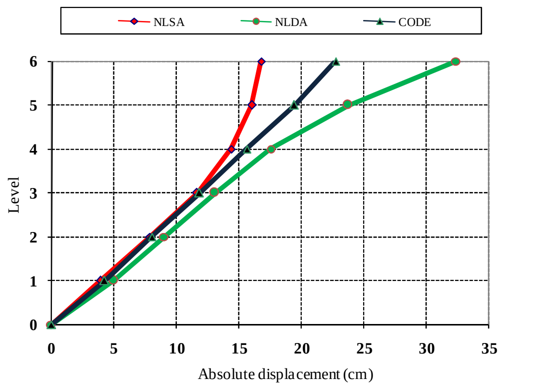


Figure 8. Transversal (YY) direction
Ultimate capacity

### 7.5. Nonlinear dynamic response analysis

The nonlinear dynamic response analysis of the structure is carried out using the D.R.A.B.S Bozinovski and Gavrilovic (1993) program and the selected ground motion records. The absolute displacements are compared the capacity for the moderate and strong earthquakes. Tables 5 to 8 resume the results of the nonlinear dynamic analysis in terms of absolute displacements. Figs. 9 to 12 show the obtained results for the moderate and strong earthquakes in the two main directions.

Table 5. Capacity and demand in terms of absolute displacements (cm) for Amax=0.15 g, in the longitudinal (XX) direction

| Level | Δy cap | Ulcinj | El Centro | Cherchell |
|---|---|---|---|---|
| 6 | 3.08 | 14.17 | 14.26 | 3.00 |
| 5 | 2.95 | 13.64 | 13.54 | 2.81 |
| 4 | 2.60 | 12.17 | 11.61 | 2.36 |
| 3 | 2.08 | 10.12 | 9.20 | 1.87 |
| 2 | 1.43 | 7.56 | 6.51 | 1.39 |
| 1 | 0.69 | 4.42 | 3.57 | 0.82 |
| 0 | 0.00 | 0.00 | 0.00 | 0.00 |

Table 6. Capacity and demand in terms of absolute displacements (cm) for Amax=0.15 g, in the transversal (YY) direction

| Level | Δy cap | Ulcinj | El Centro | Cherchell |
|---|---|---|---|---|
| 6 | 3.39 | 11.73 | 11.11 | 3.34 |
| 5 | 3.15 | 11.21 | 10.58 | 3.15 |
| 4 | 2.7 | 9.86 | 9.15 | 2.67 |
| 3 | 2.08 | 8.16 | 7.41 | 2.10 |
| 2 | 1.32 | 6.07 | 5.37 | 1.51 |
| 1 | 0.55 | 3.58 | 3.10 | 0.90 |
| 0 | 0.00 | 0.00 | 0.00 | 0.00 |

Table 7. Capacity and demand in terms of absolute displacements (cm) for Amax=0.25 g, in the longitudinal (XX) direction

| Level | Δu cap | Ulcinj | El Centro | Cherchell |
|---|---|---|---|---|
| 6 | 13.34 | 23.43 | 20.20 | 5.00 |
| 5 | 13.02 | 22.59 | 19.29 | 4.69 |
| 4 | 12.22 | 20.36 | 16.92 | 3.94 |
| 3 | 10.91 | 17.38 | 13.79 | 3.13 |
| 2 | 8.00 | 12.43 | 9.66 | 2.31 |
| 1 | 4.20 | 7.09 | 5.45 | 1.37 |
| 0 | 0.00 | 0.00 | 0.00 | 0.00 |

Table 8. Capacity and demand in terms of absolute displacements (cm) for Amax=0.25 g, in the transversal (YY) direction

| Level | Δu cap | Ulcinj | El Centro | Cherchell |
|---|---|---|---|---|
| 6 | 16.76 | 20.17 | 17.66 | 5.58 |
| 5 | 16.00 | 19.46 | 16.95 | 5.27 |
| 4 | 14.35 | 17.58 | 15.11 | 4.47 |
| 3 | 11.63 | 15.14 | 12.79 | 3.52 |
| 2 | 7.88 | 11.50 | 9.57 | 2.52 |
| 1 | 3.85 | 6.85 | 5.65 | 1.50 |
| 0 | 0.00 | 0.00 | 0.00 | 0.00 |

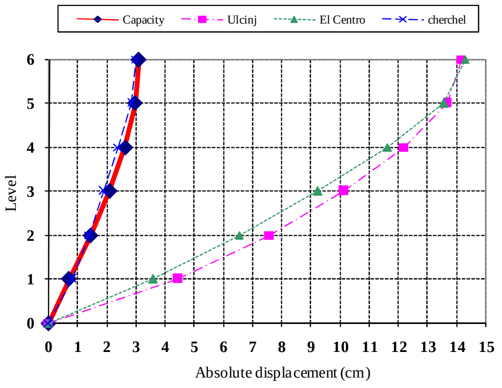


Figure 9. Longitudinal (XX) direction
Absolute displacements (0.15 g)

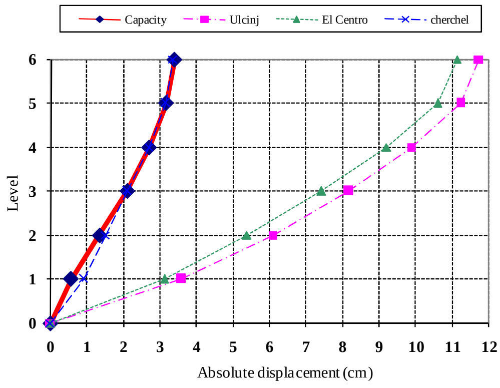


Figure 10. Transversal (YY) direction
Absolute displacements (0.15 g)

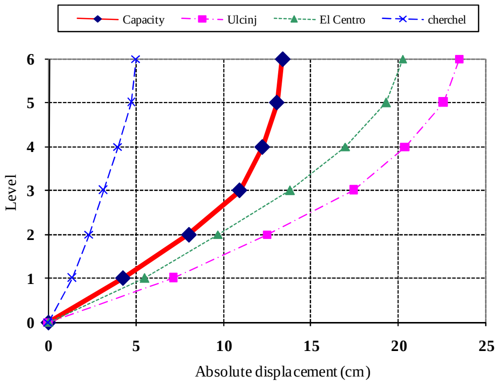


Figure 11. Longitudinal (XX) direction
Absolute displacements (0.25 g)

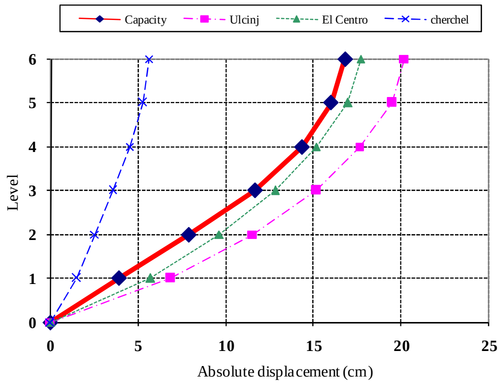


Figure 12. Transversal (YY) direction
Absolute displacements (0.25 g)

## 8. CONCLUSION

On the basis of the different obtained results, the deficiency of the structure is its excessive flexibility. Absolute displacements under lateral forces exceeded considerably the capacity values. The seismic joint dimension is important. In addition, dimensions of many columns are under estimated, since they were designed to bear vertical loads only. All calculations led to the conclusion that the structure needs strengthening CGS (1994). Two reinforced concrete shear walls in each main must be added, in order to diminish the top displacement, and the retrofitting of the weak columns is necessary.

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