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SEISMIC VULNERABILITY ANALYSIS OF A STRATEGIC EXISTING RC BUILDING ACCORDING TO NEW SEISMIC CODE

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Abstract

The 2003 Boumerdes earthquake, magnitude 6.8, was the most devastating earthquake that occurred close to the capital Algiers, killing and injuring thousands of people and causing huge various degrees of damage to many elder RC existing buildings. Among them some were classified as strategic ones. The objective of this paper is to quantify the seismic vulnerability assessment of an old reinforced concrete strategic existing building designed prior to the new seismic code in force RPA99/version 2003 and erected in a moderate seismic zone, Constantine. The structural building system is a reinforced concrete resisting moment frames. The seismic input is considered according to the provisions of the Algerian seismic design code RPA99/version 2003. Story limit capacities are identified for first yield and ultimate states. The capacity curves were obtained using the concept of nonlinear analysis based on the capacity design. The damage parameters were obtained by performing the nonlinear time history analysis considering three different prescribed ground motion histories recorded during past earthquakes. The main damage parameters used in this study were story drift displacement. Hereafter, the main results of this analysis are discussed.

Keywords: seismic vulnerability; RC buildings; capacity design; nonlinear dynamic analysis; story drift displacement.

INTRODUCTION

Earthquakes can produce a strong negative impact on society because of human suffering and economic losses. During the recent Boumerdes earthquake 6.8 magnitude extensive and unexpected damage was observed in many structural elements as well as in nonstructural components and contents. Existing stocks of vulnerable structures arguably constitute the most critical hazard risk in seismic regions of all the northern part of Algeria.

A considerable portion of this stock date from the colonial period and consists of non-engineered buildings. In order to reduce this risk, the government decided firstly to protect the strategic existing buildings, from the adverse effects of future expected earthquakes. The selected five story building belongs the head office of the Wilaya of Constantine. The present study assesses the seismic performance of a an existing reinforced concrete moment resisting frames. It was designed with insufficient lateral stiffness to satisfy code drift limitations in zones with high seismic hazard. The concept of capacity design considering member rotations for local and inter story drifts for global deformations was used. Inelastic time history analysis was carried out to assess the structural performance under earthquake ground motions. Inter story drifts were considered as the potential damage parameter.

1. DEFINITION OF SEISMIC HAZARD

In general terms, the seismic hazard defines the expected seismic ground motion at a site, phenomenon which may result in destructions and losses. Seismic hazards are the intrinsic natural occurrence of earthquakes and the resulting ground motion and other effects. A relationship between hazards and their occurrence frequency can be derived through a process called seismic hazard analysis. the main purpose of seismic hazard analysis is to provide parameters for estimating seismic risk. The Wilaya of Constantine and the surrounding region has experienced moderate earthquakes at least a dozen times during the past 300 years. The level of the seismic hazard in this region has been performed on the basis of several seismogenic zones defined in the light of the most recent results obtained from seismotectonics analyses carried out in North Algeria. Results are presented as relationships between maximum values of expected peak ground acceleration (PGA) at bedrock and annual frequency of exceedance, and synthesized in maps of seismic hazard for return periods of 100 years and 500 years.

Amax = 0.15g, for 100 years return period, considered as a slight (moderate) earthquake and expected to occur many times during the lifetime of the building. The associated performance requirement is little or no damage, and without interruption of function. The response spectrum method is usually adequate.

Amax = 0.25g, for 500 years return period, considered as a strong (major) earthquake and expected to occur once during the lifetime of the building. 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 of building structures and characteristics of structural materials, as well as existing upgrading or/ and changes in the original systems of the buildings.

2.1. Linear analysis

For the defined vertical and horizontal loads, linear dynamic modal response spectrum analysis is performed using the program ETABS 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. 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 lateral performance of existing buildings can be carried out using the capacity design approach which is currently adopted by all the modern seismic codes and the best appropriate method for estimating capacity, deformability and decision making for structures safety. The analysis will consider the real bearing and deformability characteristics of the structures in the elastic and plastic ranges. This approach uses the theory of limit States (yield and ultimate) of reinforced concrete structures. 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 Ultimate Analysis of Rectangular Cross Sections (U.A.R.C.S), considering the following Eq. (1):

$$Qy = Qy_{\min} + \delta y_{\min}\left[\sum_{i=1}^{i=N-1}\frac{Qy_i}{\delta y_i}\right]$$

$$Qu = Qu_{\min} + \sum_{\delta u\,\min > \delta yi}\left[Qy_i + K_{2i}\left(\delta u_{\min} - \delta y_i\right)\right] + \sum_{\delta u\,\min \leq \delta yi} K_{1i}\,\delta u_{\min} \tag{1}$$

$$With: \quad K_{2i} = (Qu_i - Qy_i)/(\delta u_i - \delta y_i) \quad and \quad K_{1i} = Qy_i / \delta y_i$$

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. (2):

$$[M]\{\Delta\ddot{U}\}+[C]\{\Delta\dot{U}\}+[K]\{\Delta U\}=-[M]\{I\}\ddot{U}_g \qquad (2)$$

Where:

$[M]$: mass matrix.

$[C]$: damping matrix.

$[K]$: stiffness matrix.

$\{\Delta\ddot{U}\}$: incremental acceleration vector.

$\{\Delta\dot{U}\}$: incremental velocity vector.

$\{\Delta U\}$: incremental displacement vector.

$\ddot{U}_g$: ground acceleration.

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).

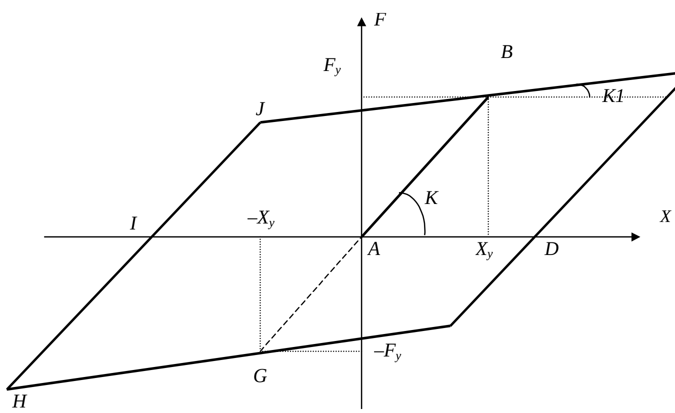

Fig. 1 - Bilinear model

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

The plots in figure 2 show 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.

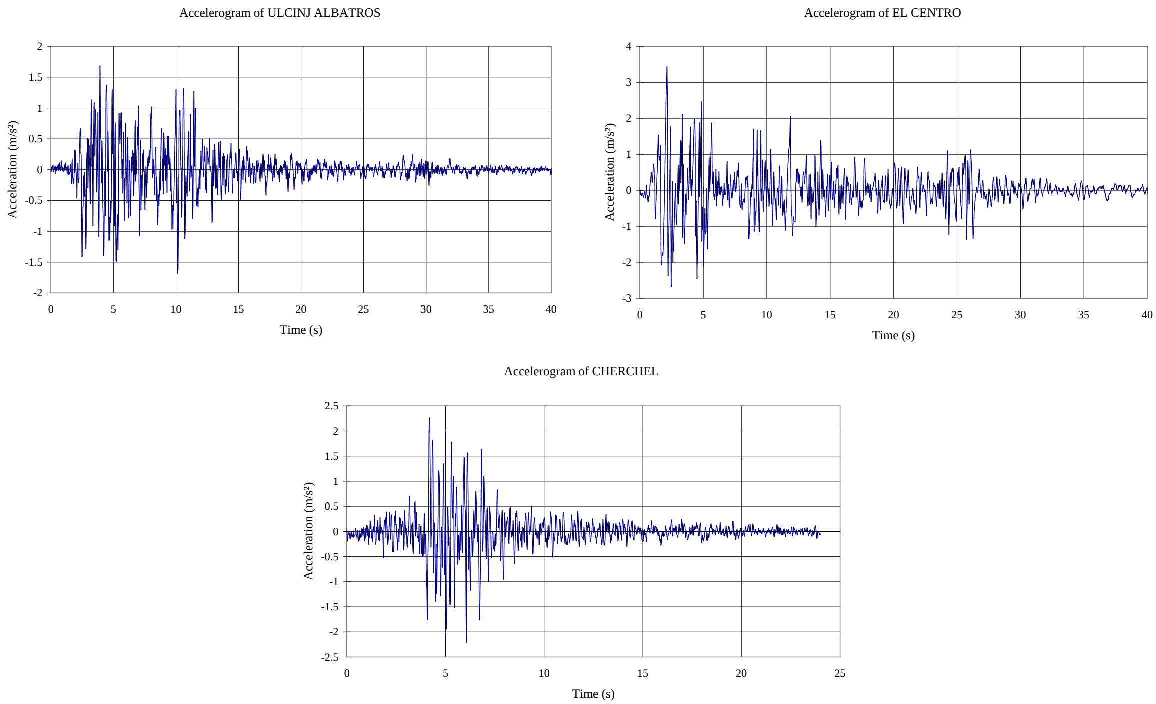

Fig. 2 - Selected earthquake accelerograms

5. LIMIT STATE

There are numerous limit states that can be considered in seismic vulnerability studies. In the traditional approaches, two limit states are considered. The elastic and the ultimate limit states. The first is defined in terms of strength and calculated using the building material properties, whereas the second is estimated in terms of displacements using a given ductility factor, eventually converted to forces using a reduction factor. More recent approaches consider multi-linear behavior relationships for the elements and define different damage states as break points in the behavior curves either in displacement or rotation (drift).

The structural performance criteria levels are generally set depending on the construction material and structural type, the importance of the building, the economic relevance of business interruption, the role of the structure in a post earthquake emergency and socio economic criteria. These levels can be specified as limits on any response parameter such as stresses, strains, displacements, velocity and accelerations. Obviously, different limit states have to be cross-correlated to the level of the seismic action, in other words to the earthquake design level.

Two structural performance levels, operational and life safety are considered for the system assessment carried out in the present study.

Limit inter story displacement for moderate earthquake: $\Delta_m = \left[\frac{H}{400} \,\&\, \frac{H}{300} \right]$

Limit inter story displacement for major earthquake: $\Delta_M = \left[\frac{H}{150} \,\&\, \frac{H}{125} \right]$

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 analyzed building is part of a state complex, located in Constantine, which was designed according to requirements of the 1999 Algerian seismic design code and constructed in the 2008s. All buildings are separated by 50 mm seismic gaps. Our focus will be on the building Bloc C2 for any potential impact with the adjacent buildings. The building is composed by five stories and a basement. The partition and exterior enveloping walls are made of hollow clay bricks. The structural system is a reinforced concrete resisting moment frames. which consists of reinforced concrete columns, beams and slabs. the building has a regular shape in plan and an irregular shape in elevation. The building is set on a medium soil quality. The figure 3 shows the location of the building by Google map.

Fig. 3 - Building location by Google map

7.2. Mechanical characteristics of the materials

Mechanical material characteristics were defined using a range of in-situ and laboratory testing and inspection techniques to obtain the necessary information.

Concrete:

- Characteristic compressive cylinder strength at 28 days: f_{c28} = 20 Mpa.
- Design tensile strength: σ_t= 1.8 Mpa.
- Yield strain: ε_e= 0.002.

-Ultimate strain: ε_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: ε_y= 0.002.
- Yield strain of shear reinforcement: ε_e= 0.0018.
- Ultimate strain: ε_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 and based on existing drawings and the site inspection, the structure was modeled in 3D space frames with rigid diaphragms and a fixed base, using the nonlinear computer program ETABS V.13. The figures 4 and 5 show the structural system in plan and three dimensional view of the existing structure.

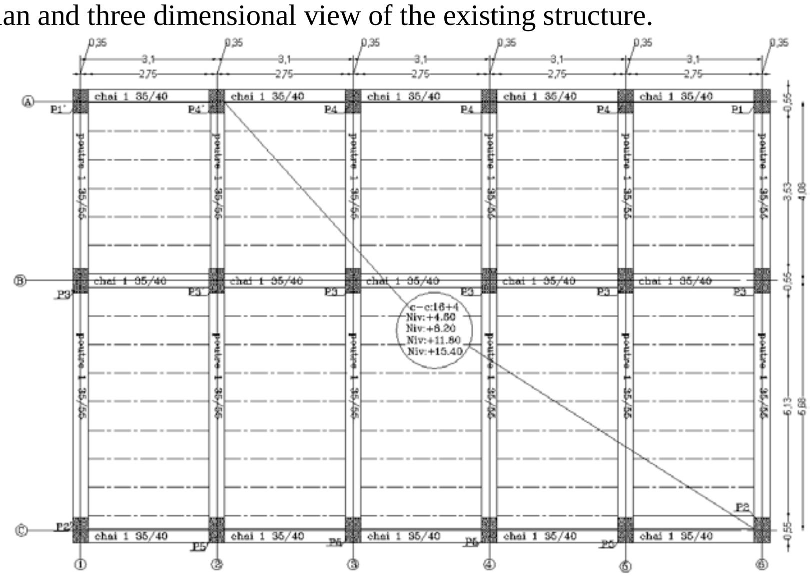

Fig. 4 - Structural system in plan

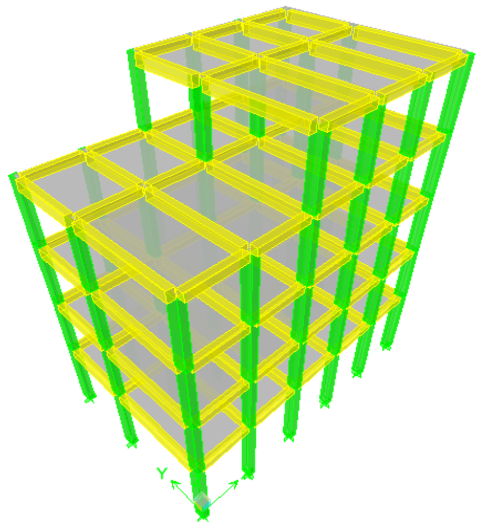

Fig. 5 - Three-dimensional view of the existing structure

7.3.2. Dynamic characteristics of the structure

The analysis of the structure gives translational coupled with rotational modes in the transversal direction. The main results for the first fives mode shapes are summarized in Table 1 for the two main directions (longitudinal XX and transversal YY).

Table1 - Periods and mass factors participation

Mode	**Period (Sec)**	**UX (%)**	**UY (%)**	**RZ (%)**
1	0.806	**90.406**	0.004	0.136
2	0.619	0.071	**59.007**	**29.458**
3	0.558	0.067	**29.498**	**59.920**
4	0.265	**7.253**	0.000	0.012
5	0.198	0.006	**5.474**	3.213

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. (3) given by:

$$V = \frac{ADQ}{R} W \tag{3}$$

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 base shear force is distributed to each story in accordance to the distribution of the story mass with its height from the base given by Eq. (4):

$$F_K = \frac{(V - F_t) W_K \, h_K}{\sum_{i=1}^{N} W_i \, h_i} \tag{4}$$

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)**
5	220.05	220.05	247.57	247.57
4	320.44	540.49	355.83	603.40
3	269.82	810.31	299.77	903.17
2	209.01	1019.32	225.17	1128.34
1	143.03	1162.35	142.30	1270.64

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

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 shear forces using the capacity design approach. The safety factor "S" permits to compare the shear capacity to the demand in accordance with the Algerian seismic code RPA99/version 2003 for both main directions. It should be greater than the value of 1.15. Table 3 and figures 6 and 7 show the main results.

Table 3 - Safety factor S for main directions

	Longitudinal XX			Transversal YY		
Level	**Quxi (KN)**	**Vxi (KN)**	**Sxi**	**Quyi (KN)**	**Vyi (KN)**	**Syi**
5	1146.95	220.05	**5.21**	1998.54	247.57	**8.07**
4	1660.38	540.49	**3.07**	2899.46	603.40	**4.81**
3	1638.68	810.31	**2.02**	2867.60	903.17	**3.18**
2	1628.42	1019.32	**1.60**	2899.54	1128.34	**2.57**
1	1372.32	1162.35	**1.18**	2453.81	1270.64	**1.93**

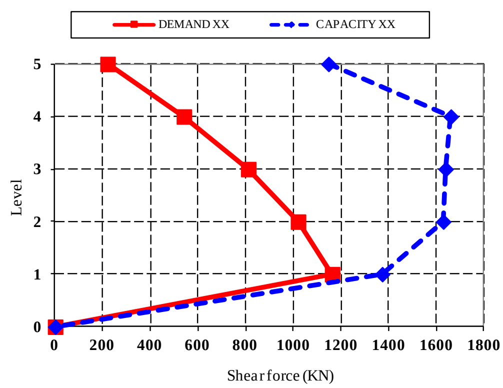

Fig. 6 - Capacity and demand in terms of shear forces for longitudinal (XX) direction.

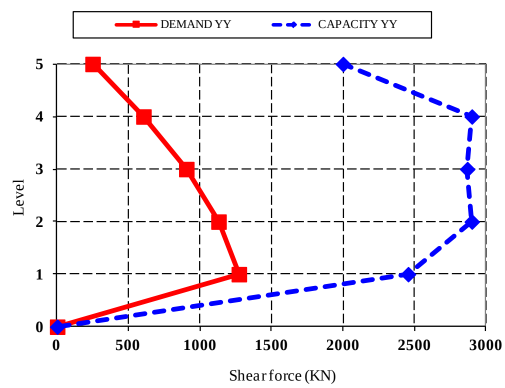

Fig. 7 - Capacity and demand in terms of shear forces for transversal (YY) direction.

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. Variations of maximum inter story drifts in the assessed structure are summarized in Table 5 and plotted in figures 8 and 9. Furthermore, to investigate the effect of hammering between adjacent buildings, the peak roof displacement is evaluated for each record corresponding to the maximum demand at that story throughout the duration of the event. Table and figures resume the results of the nonlinear dynamic analysis in terms of inter story displacements in case of a major earthquake in the two main directions.

Table 5 - Capacity and demand in terms of inter story displacements (cm) for Amax=0.25 g, in the longitudinal (XX) and transversal (YY) directions

Level	Earthquake	Δx (cm)	Δy (cm)	1% hi (cm)	Δi (cm)
5	**Ulcinj**	0.8	0.3	3.6	**2.6**
	El Centro	0.7	0.5		
	Cherchell	0.5	0.3		
4	**Ulcinj**	1.2	0.5	3.6	**2.6**
	El Centro	1.0	0.8		
	Cherchell	0.8	0.4		
3	**Ulcinj**	1.5	0.8	3.6	**2.6**
	El Centro	1.4	1.1		
	Cherchell	1.1	0.6		
2	**Ulcinj**	1.8	0.9	3.6	**2.6**
	El Centro	1.6	1.2		
	Cherchell	1.3	0.7		
1	**Ulcinj**	**5.3**	2.2	4.6	**3.3**
	El Centro	**3.7**	3.1		
	Cherchell	2.9	1.6		

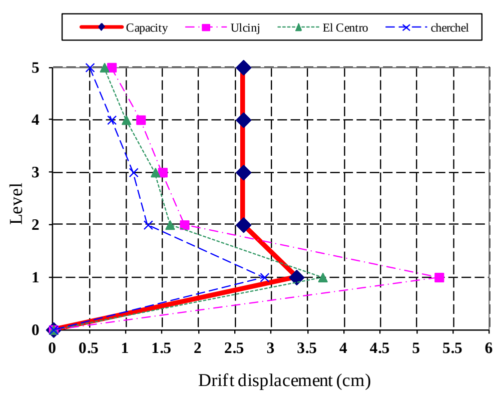

Fig. 8 - Capacity and demand in terms of inter story displacements for longitudinal (XX) direction

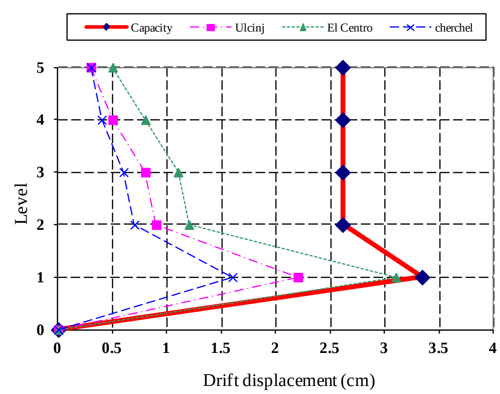

Fig. 9 - Capacity and demand in terms of inter story displacements for transversal (YY) direction

8. CONCLUSION

The present analytical work shows that:
- Dynamic analysis of the structure shows coupled modes (translation with torsion) in the transversal direction YY for the first mode.
- Inter story displacements demand exceed capacity for the first level in the longitudinal direction (XX) in case of a strong earthquake.
- Absolute displacements under lateral forces exceed considerably the expansion gap of 5 mm in the two main directions in case of a strong motion.

The performed comparative analyses confirm that the structure is very flexible and needs strengthening CGS (1994). Adding two reinforced concrete shear walls in each main decreases displacements demands significantly. Thus, existing deficiencies in frame elements are less pronounced and poor construction quality in buildings is somehow compensated.

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