Seismic vulnerability assessment of composite reinforced concrete-masonry building

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Abstract. During the last decades, many destructive earthquakes occurred in Algeria, particularly in the northern part of the country (Chlef (1980), Constantine (1985), Tipaza (1989), Mascara (1994), Ain-Benian (1996), Ain Temouchent (1999), Beni Ourtilane (2000), and recently Boumerdés (2003), causing enormous losses in human lives, buildings and equipments. In order to reduce this risk and avoid serious damages to the strategic existing buildings, the authorities of the country, aware of this risk and in order to have the necessary elements that let them to know and estimate the potential losses in advance, with an acceptable error, and to take the necessary countermeasures, decided to invest into seismic upgrade, strengthening and retrofitting of those buildings. To do so, seismic vulnerability study of this category of buildings has been considered. Structural analysis is performed based on the site investigation (inspection of the building, collecting data, materials characteristics, general conditions of the building, etc.), and existing drawings (architectural plans, structural design, etc.). The aim of these seismic vulnerability studies is to develop guidelines and a methodology for rehabilitation of existing buildings.

This paper presents the methodology, based on non linear and seismic analysis of existing buildings, followed in this study and summarizes the vulnerability assessment and strengthening of one of the strategic buildings according to the new Algerian code RPA 99/version 2003. As a direct application of this methodology, both, static equivalent method and non linear dynamic analysis, of composite concrete masonry existing building in the city of "CONSTANTINE", located in the east side of ALGERIA, are presented in this paper.

Keywords: vulnerability; masonry; strategic building; capacity design; non linear dynamic analysis; strengthening

1. Introduction

Algeria, a gateway between Europe and Africa, is located in Northern Africa at the margin

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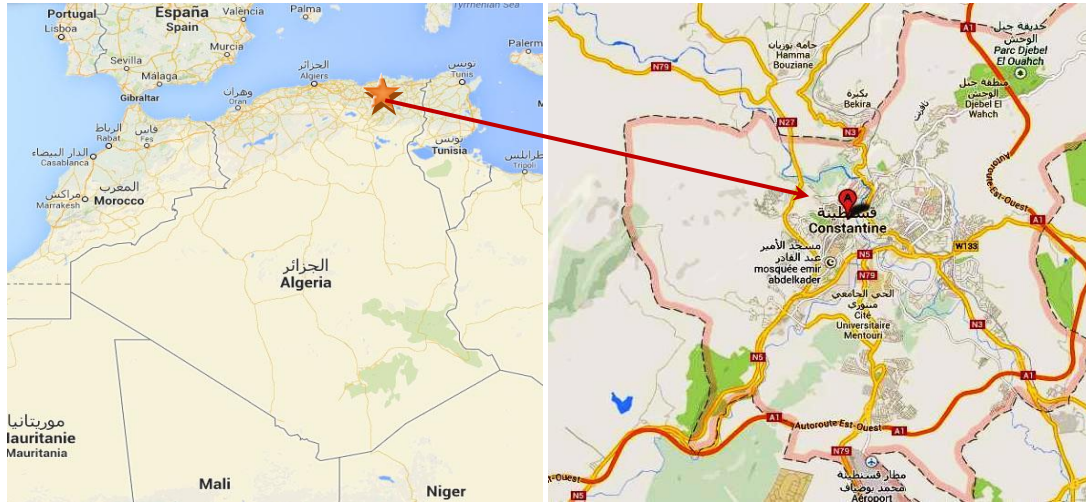[c]Associate Professor, Ph.D., E-mail: bourzam@gmail.com

Fig. 1 Constantine map (Source: Google Maps)

between the north moving African plate and the Eurasian plate, creating a zone of compression, which manifests itself by a series of thrust and normal faults that have been mapped in the area. This region has a rich history of seismicity and had experienced many destructive earthquakes in the past.

The northern part of Algeria has a high seismic activity, where a major part of this population, buildings and facilities are concentrated. Recently, many strong earthquakes occurred in this region, causing enormous losses in human lives, houses and infrastructure (Ousalem and Bechtoula 2005).

Within the framework of the policy of reduction of the seismic risk in Algeria and more particularly in the north part of the country, one of the privileged ways consists in installing tools for analysis and undertaking studies and actions for an effective prevention of the effects of this natural phenomenon which is the earthquake (Karantoni *et al*. 2014). In order to attenuate this phenomenon, the authorities of the country decided as a first step to protect the strategic existing buildings from the adverse effects of future expected earthquakes. Hence, seismic vulnerability study of this category of buildings has been considered (Belazougui *et al*. 2004).

The seismic vulnerability assessment of one of the strategic buildings in the city of CONSTANTINE (see Fig. 1), one of the most important strategic existing buildings, is presented in this paper, which has been built in the beginning of the twentieth century without any seismic regulations.

2. Methodology for seismic vulnerability assessment of composite reinforced concrete-masonry existing buildings

The seismic risk evaluation of a building or group of buildings located in a seismic hazard zone allows indicating the level of structural damage that could result by the action of an earthquake, depending on the vulnerability level of the structure. Analyzing the above mentioned, in general, it is worth noting that the seismic risk of buildings may be satisfactorily assessed by taking into
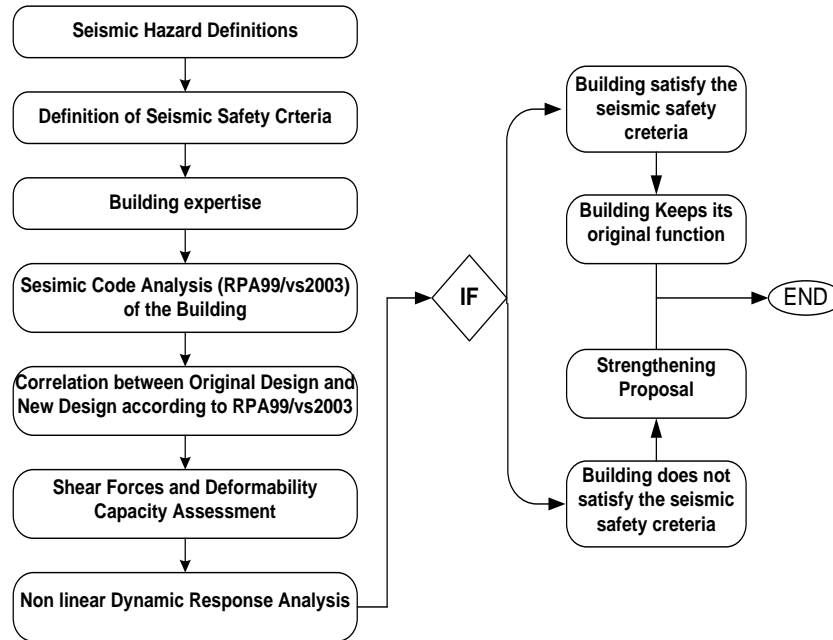
Fig. 2 Methodology flow chart

account the seismic hazard of the site and the vulnerability of the structure (Preciado *et al*. 2015)

The seismic vulnerability methodology for existing buildings used in this context was developed in the National Center of Applied Research in Earthquake Engineering CGS (Algiers-Algeria), with the cooperation of IIZIS University (Republic of Macedonia). The methodology takes into account (IIZIS/CGS, 1993), the definition of the seismic hazard, choice of the soil acceleration at the base, seismic safety criterion, structural building safety and damageability analysis of the building.

Fig. 2 shows basics steps of the developed methodology.

2.1 Definition of seismic risk and safety criterion

In this study, the definition of seismic hazard and attenuation laws are used to define the maximum expected bedrock acceleration as a function of a return period of 100 and 500 years are as follows (Figs. 3(a)-(b)):

Moderate earthquake: Amax=0.15 g for 100 years return period.

Major earthquake: Amax=0.25 g for 500 years return period.

The following sets of selected ground motion records are used in our methodology:

El Centro (California, USA) N-S May, 1940.

Ulcinj (Albatros, Montenegro) N-S, 1979.

Cherchel (Algeria) N-S, 1989.

In general, the safety criterion should be determined for two levels of the expected seismic action that are:

First level: corresponding to moderate earthquakes that are expected to happen many times
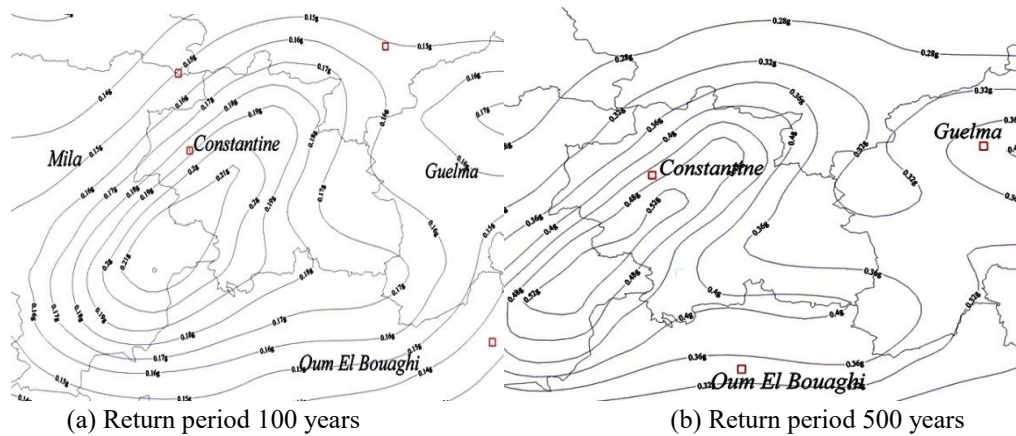
(a) Return period 100 years　　　　　　　(b) Return period 500 years

Fig. 3 Constantine iso-acceleration map (CGS-DAS-source, 2009)

Table 1 Maximum story drift displacement (first level)

	Story drift displacement	Ductility
RC Frame structure	$\delta = \min\left\{\delta_{cap-el}; \dfrac{H}{300-400}; \dfrac{H}{100}\right\}$	$\mu = 1-1.5$
RC Shear walls and frames	$\delta = \min\left\{\delta_{cap-el}; \dfrac{H}{350-400}; \dfrac{H}{100}\right\}$	$\mu = 1-1.5$
URM masonry walls	$\delta = \min\left\{\delta_{cap-el}; \dfrac{H}{500-700}; \dfrac{H}{100}\right\}$	$\mu = 0.8-1.0$

Table 2 Maximum story drift displacement (second level)

	Story drift displacement	Ductility
RC Frame structure	$\delta = \min\left\{\delta_{cap-el}; \dfrac{H}{125-150}; \dfrac{H}{100}\right\}$	$\mu = 2.5-3.0$
RC Shear walls and frames	$\delta = \min\left\{\delta_{cap-el}; \dfrac{H}{150-175}; \dfrac{H}{100}\right\}$	$\mu = 3.0$
URM masonry walls	$\delta = \min\left\{\delta_{cap-el}; \dfrac{H}{250-300}; \dfrac{H}{100}\right\}$	$\mu = 1.0-1.5$

during the life of the building, with a return period of 100 years. The behavior of the structures should remain in the elastic range, without any damage and the building can be used immediately.

The maximum allowable story drift displacement and ductility are as shown in Tables 1-2:

Second level: corresponding to major earthquakes that are expected to occur once during the life of the building, with a return period of 500 years. The structure may behave in the non linear range, with a controlled level of damage. No heavy damage or collapse is allowable, and the building must be reused after inspection with slight repairs.

To estimate the safety of the building, static and dynamic analysis for the moderate and major expected ground motions should be carried out and compared to the capacity of the structure.

2.2 Structural analysis

Structural analysis is mainly based on the structural systems of the building that must be able to resist the various combinations of gravity and horizontal loads (Fajfar and Kreslin 2010). The non-structural elements should be controlled based on the obtaining principal corresponding data (story deformation, flexibility, local instability, etc.). It must include the real data of the structure such as characteristics of structural materials, as well as different changes to the systems of the buildings.

2.3 Static and dynamic analysis

For the defined vertical and horizontal loads, linear static and dynamic analysis is performed with SAP2000v14 (Wilson and Habibullah 2009) to obtain the periods, the mode shapes, the story stiffness and relative displacements. Demands in terms of bending moments M, shear forces Q, and axial forces N, are checked for each element constituting the structure.

2.4 Seismic analysis according to the national seismic building code "RPA99/version 2003"

Structural elements of the building are checked according to the new Algerian code RPA 99/version 2003 (CGS, 2003) requirements. With the expected applied horizontal seismic forces, demands in terms of M, Q and N are computed and compared to the original (initial) design data if they are available. On the basis of this qualitative evaluation, a final decision will be made in the case of clear and satisfying safety criteria.

2.5 Analysis of strength and deformability capacities

The capacity approach considers the real bearing and deformability characteristics of the structures in the elastic and plastic state. This approach uses the theory of the Ultimate Limit State of RC structures. The capacity of the structure is determined using the Ultimate Analysis of Rectangular reinforced Concrete cross Sections of frames and walls systems, U.A.R.C.S (Bozinovski and Gavrilovic 1993a) and Static and Dynamic Ultimate Analysis of Masonry Buildings, S.D.U.A.M.B programs (Bozinovski and Gavrilovic 1993b) for each structural element and at each level of the structure, to obtain the periods, the mode shapes, the story stiffness and the relative displacements.

Demands in terms of bending moments, M, shear forces, Q, and axial forces, N, are checked for each element constituting the structure.

2.6 Dynamic response analysis

Dynamic response analysis of structures represents a numerical computation of structural systems with defined characteristics of masses, stiffness, damping, etc., and defined ranges of linear and nonlinear behavior expressed via displacements, forces and accelerations (A. Chopra 2001). To determine the nonlinear response of the structure, the Dynamic Response Analysis of Building Structures, D.R.A.B.S program (Bozinovski and Gavrilovic 1993c) program is used to assess the shear force-displacement (Q-δ) curve at each story of the structure.

2.7 Vulnerability assessment

The seismic vulnerability assessment will be done based on the previous steps that have been discussed and implemented; where a final decision and proposal should be submitted to the owner of the building, stating the fact that the building, either satisfies the stability criteria and in this case can be used in its existing state , or the building does not satisfy those criteria and in this case we will proceed to its strengthening if it does work (Calvi 2013), taking into account, the feasibility and the cost analysis of the proposed strengthening method.

3. Application to an existing composite reinforced concrete-masonry building

3.1 Description of the building

The building which will be a part of this study belongs to the fire fighters department. It was built in the beginning of the twentieth century, composed of five stories with a partial basement. The structural system is RC frame with stone masonry walls.

The masonry bearing walls have a variable thickness from 45 cm in the upper floors to 60 cm in the basement and the first floor. one floor to another; the floors currents the thickness of the walls of east shore of 45 cm; by against the DRC and in the basement the thickness of the walls is 60 cm The interior walls in masonry have a thickness ranging from 45 to 50 centimeters. The book presents a recess in elevation at the last level. The floors are in full slab of 20 cm thickness.

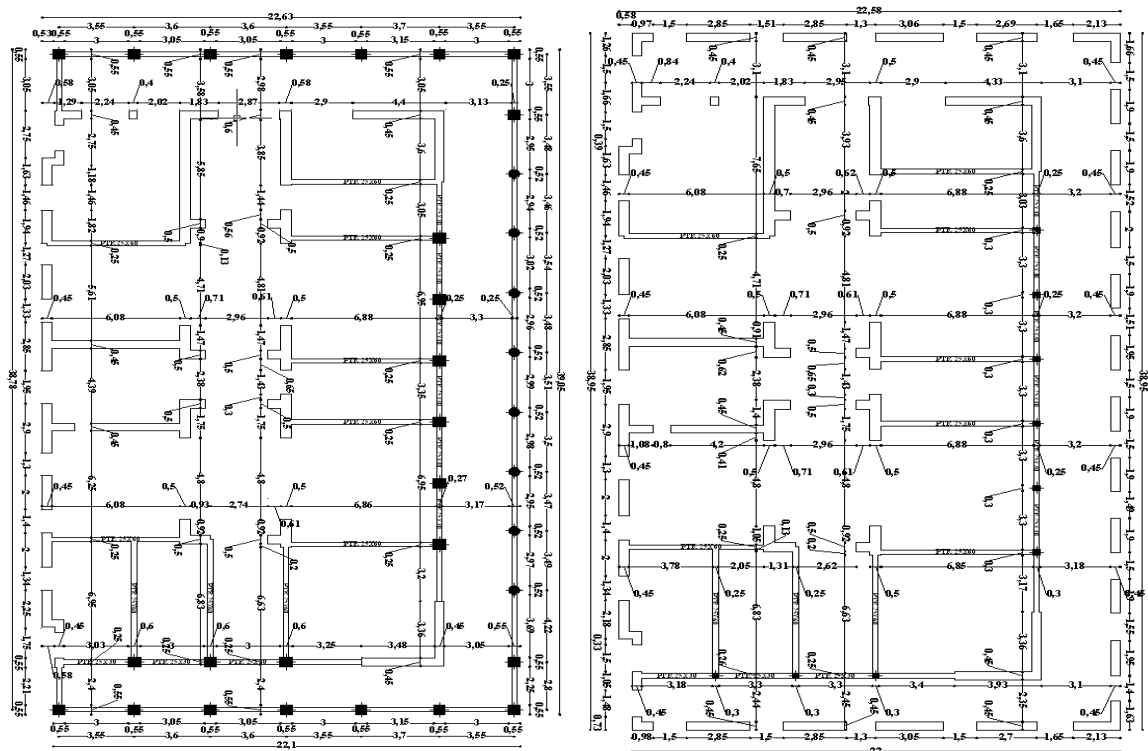The layout of different stories is shown in following figures:

Fig. 4 Plan view of 02 different stories

Table 3 mechanicals characteristics of the materials

Masonry	Concrete	Steel
Densité : d = 2.7 t/m^3 Elastic modulus: Em = 2.10^6 kpa Shear modulus : Gm = 5.10^5 kpa Comp. strength: σ_c = 1500 kpa Tensile strength : σ_t = 40 kpa	Comp. strength: f_{c28} = 20 MPa Tensile strength : f_{t28} = 1.8 MPa Yield strain: ε_e = 2 ‰ Ultimate strain: ε_u = 3.5 ‰	Yield strength: fe $\quad$ = 400 MPa Yield strain: : ε_e = 2‰ Ultimate strain: ε_u = 10 ‰

3.2 Mechanical characteristics of the materials

To determine the mechanical characteristics of building materials several samples coring have been carried out and taken to the laboratory for testing, which give us the following characteristics (see Table 3)

3.3 Structural analysis

The static and dynamic analysis has been performed by using the computer program SAP 2000v14 (Wilson and Habibullah 2009), in order to estimate the shear forces Q and the axial forces N. The building has been modeled in 3D, as shown in the following figure.

The periods of vibration for both directions have been calculated using the empirical formula of seismic code RPA99/2003 (CGS, 2003) which gave us
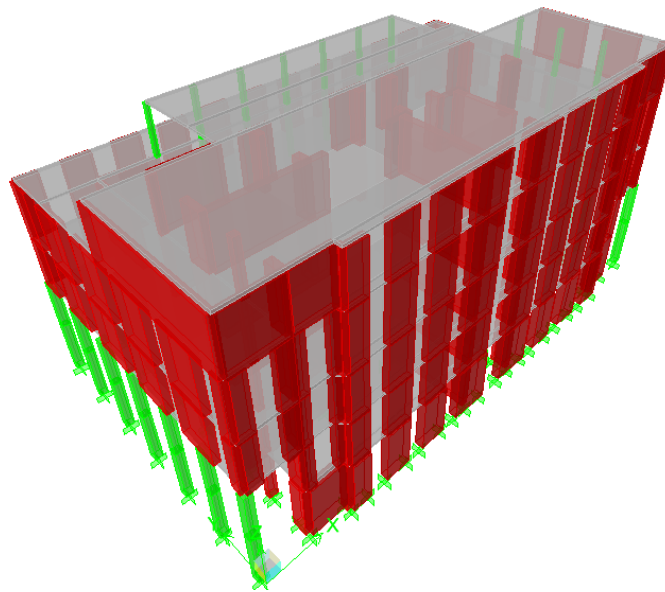
$$Tx = Ty = 0.590 \text{ s} \tag{1}$$

Fig. 5 3D view of the initial structure

3.4 Seismic analysis according to the ALGERIAN seismic code "RPA99/version 2003"

The base shear force is estimated using the static equivalent force procedure as

$$V = \frac{A.D.Q}{R}.W \tag{2}$$

Where:
V: Total design base shear force
A: Design base acceleration coefficient
D: Mean dynamic amplification factor, function of the fundamental natural period
Q: Quality factor, R: Behavior factor of the structure (composite structure masonry and RC frame)
T: Natural fundamental period of the structure
W: Total seismic weight.
The value of the shear force V must be distributed according to following formula

$$Fi = \frac{(V - F_t)W_i h_i}{\Sigma_{j=1}^{n} W_j h_j} \tag{3}$$

Where:
F_i: Seismic horizontal force at the i_{th} level.
F_t: Shall be assumed to be concentrated at the top of the structure in addition to F_n, and equal to 0.07
Hence, the distribution of the lateral seismic loads is shown in Fig. 6, for both transversal YY and longitudinal XX directions.

3.5 Deformability and strength capacity

The capacity of the structure is determined using the Ultimate Analysis of Rectangular reinforced Concrete cross Sections of frames and walls systems, U.A.R.C.S (Bozinovski and
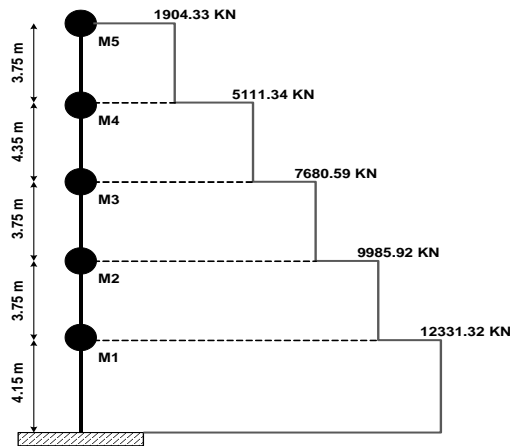

Fig. 6 Distribution of seismic forces over the stories

Table 4 shear forces and displacements

Level	Mass (KN)	δ_u^x (cm)	Q_y^x (KN)	Q_u^x (KN)	μ^xCap. Duct.	δ_u^y (cm)	Q_y^y (KN)	Q_u^y (KN)	μ^yCap. Duct.
5	10146	0.089	1209.31	1329.89	1.14	0.093	846.86	904.32	1.15
4	14730	0.167	1805.46	1975.36	1.69	0.137	1552.32	1650.18	1.14
3	13688	0.122	2376.33	2561.42	1.11	0.143	1939.00	2072.47	1.14
2	12282	0.189	3149.98	3241.75	1.09	0.200	1644.34	1979.90	1.36
1	11291	0.222	3539.84	3867.24	1.14	0.240	1985.51	2115.28	1.17

Table 5 Shear capacity; shear demand and safety factors of the structure in both directions

Level	V_i^x(KN)	Q_u^x(KN)	F_{sx}	V_i^y(KN)	Q_u^y(KN)	F_{sy}
5	2963.77	1329.89	0.45	2853.78	904.32	0.32
4	6811.45	1975.36	0.29	6612.67	1650.18	0.25
3	9753.57	2561.42	0.26	9511.35	2072.47	0.22
2	11709.21	3241.75	0.28	11532.94	1979.90	0.17
1	12411.16	3867.24	0.31	12321.36	2115.28	0.17

Gavrilovic 1993a) and Static and Dynamic Ultimate Analysis of Masonry Buildings, S.D.U.A.M.B programs (Bozinovski and Gavrilovic 1993b) for each structural element and at each level of the structure, to obtain the periods, the mode shapes, the story stiffness and the relative displacements.

Table 4 summarizes the results for both materials in terms of elastic and ultimate displacements, elastic and ultimate shear force and the ultimate ductility capacity for both directions XX and YY.

Q_y, Q_u, δ_u, and μ are respectively the elastic shear force, the ultimate shear force, the ultimate displacement and the ductility capacity in both directions XX and YY.

According to the Algerian seismic code regulation RPA 99/2003 (CGS, 2003), The structure is considered stable when the safety factor, **F$_s$,** is greater than 1.15 at each level and in both directions, where

$$F_s = \frac{Q_i}{V_i} \geq 1.15 \tag{4}$$

Q$_i$: Shear force capacity at level k.
V$_i$: Shear force demand at level k.

Table 5 summarizes the results of shear capacity, shear demand and the safety factors, Fs, for both transversal (YY) and longitudinal (XX) directions.

It is clearly shown from Table 5 that the capacities of the structure in term of shear forces do not satisfy the demand at all stories of the building.

3.6 Non linear dynamic response analysis

The Dynamic Response Analysis of Building Structures, D.R.A.B.S program (Bozinovski and Gavrilovic 1993c) program is used to assess the shear force-displacement (Q-δ) curve at each story of the structure. Three (03) selected earthquakes ground motion have been used, as shown in

Table 6.

The indicated duration of the selected accelerograms represents the total duration of the motion and not the duration of the strong motion only.

Fig. 7 shows the acceleration time-history of the three accelerograms used in this study.

Fig. 8 and Fig. 9 show a comparison between the capacity of the structure and the demand in terms of displacements for both, moderate (Amax=0.15g) and major earthquake (Amax=0.25g). It is clear that the capacity of the existing building expressed in terms of absolute displacement is exceeded at the level of all stories.

Table 6 Characteristics of selected motions

Earthquake	Country	Direction	Year	Duration (s)	Amax(m/s^2)
El Centro	USA	N-S	1940	40	3.42
Cherchell	Algeria	N-S	1989	24	2.26
Ulcinj	Serbia	N-S	1979	40	1.68
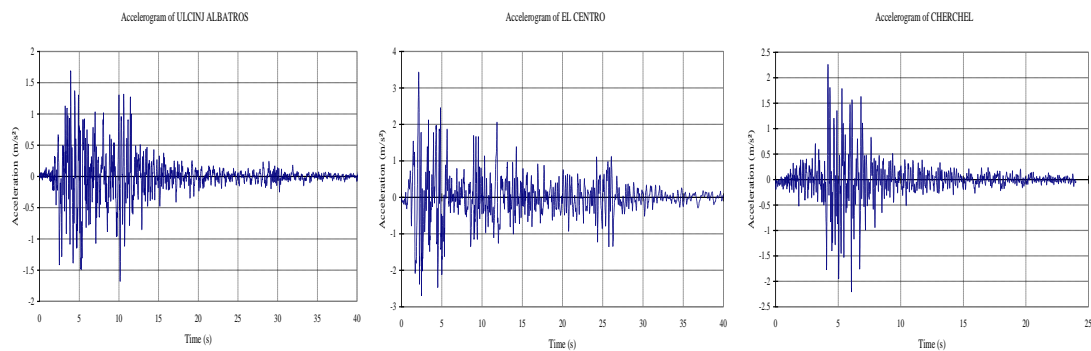
Fig. 7 Selected earthquakes accelerograms

(a) Longitudinal direction X-X   (b) Transversal direction Y-Y

Fig. 8 Displacement capacity and demand **Amax=0.15 g**

(a) Longitudinal direction X-X   (b) Transversal direction Y-Y

Fig. 9 Displacement capacity and demand **Amax=0.25 g**

(a) Longitudinal direction X-X   (b) Transversal direction Y-Y
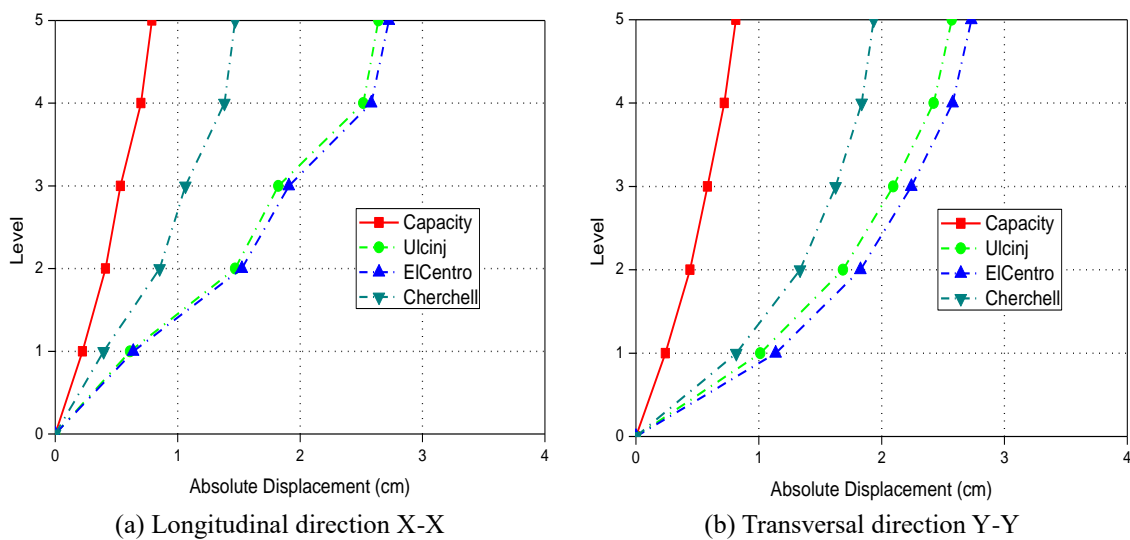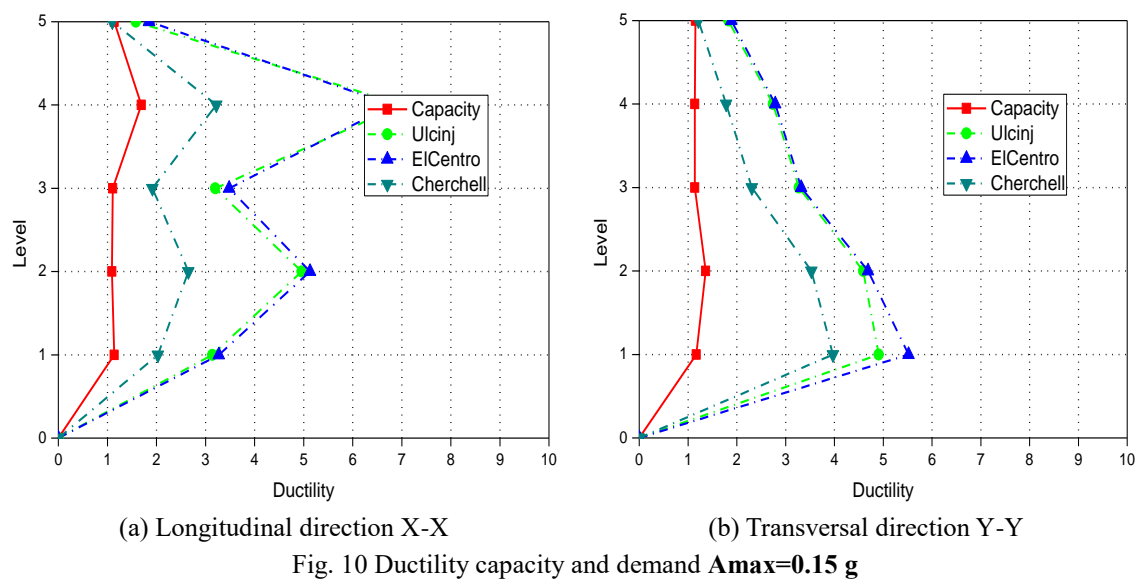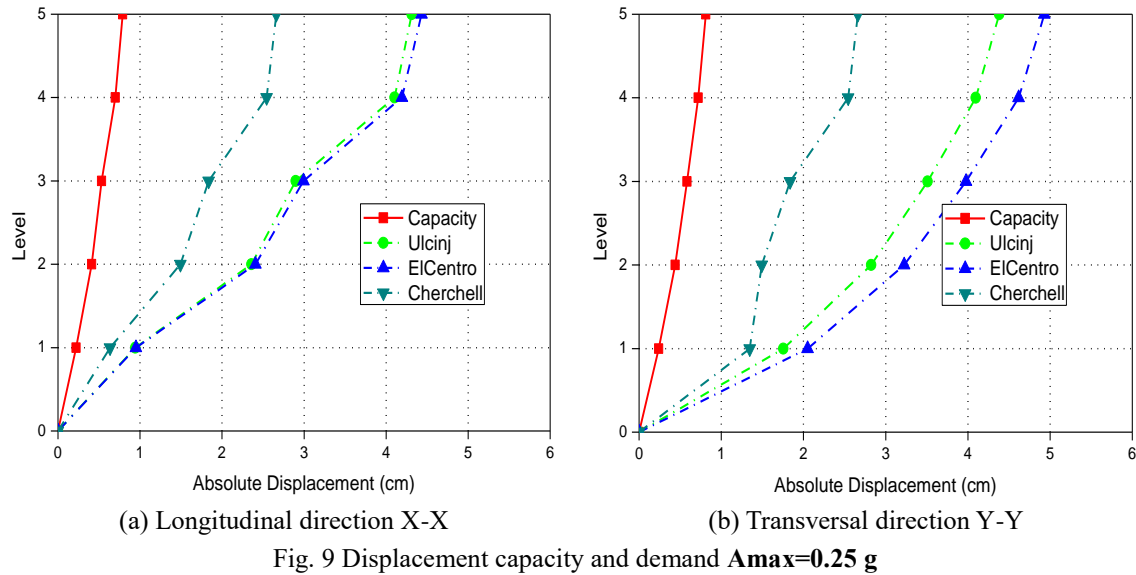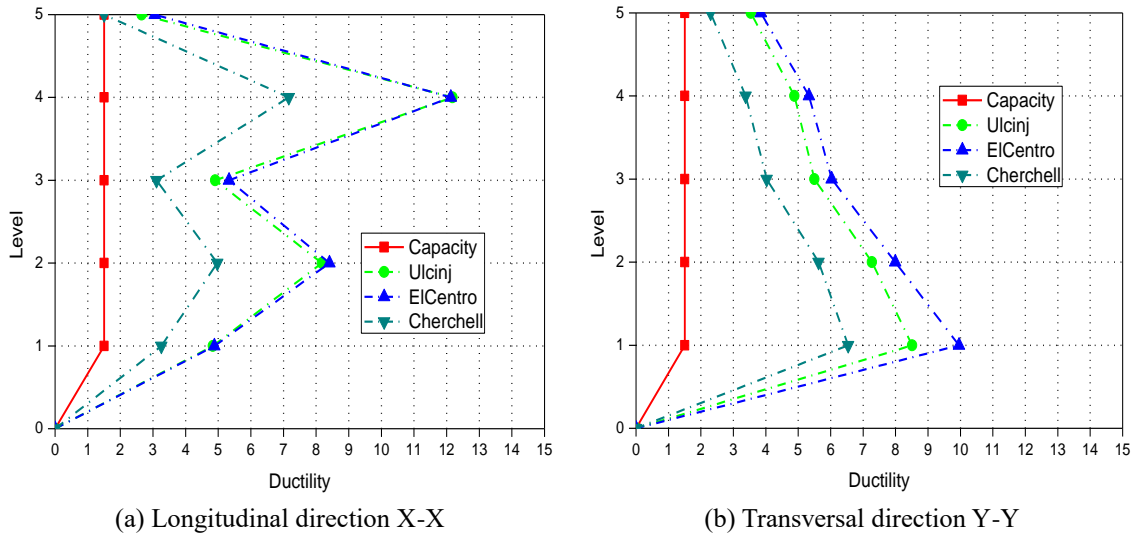
Fig. 10 Ductility capacity and demand **Amax=0.15 g**

Fig. 10 and Fig. 11 show a comparison between the capacity of the structure and the demand in terms of ductility for both, moderate (Amax=0.15 g) and major earthquake (Amax=0.25 g). It is clear that the capacity of the existing building expressed in terms of ductility is also exceeded at the level of all stories as advocated previously in the methodology.

From the above obtained results it is obvious that all computations led to the conclusion that the structure has huge lack of performance during a severe earthquake, due the fact that it has not been built based on seismic regulations, and hence it needs strengthening in order to increase the strength and to limit the absolute displacements under a major earthquake (Iskhakov *et al.* 2013). Many simulations have been tried in order to get the most economic and convenient solution.

(a) Longitudinal direction X-X   (b) Transversal direction Y-Y

Fig. 11 Ductility capacity and demand **Amax=0.25 g**

3.8 Strengthening proposal

The structural system of this deficient building should be adequately strengthened in order to attain the desired level of seismic resistance (FEMA 273, 1997). The term *strengthening* comprises technical interventions in the structural system of a building that improves its seismic resistance by increasing the strength, stiffness and/or ductility (Gorgulu *et al.* 2011), (Kaplan *et al.* 2011).

The types of intervention on masonry buildings consist of the improvement of the connections

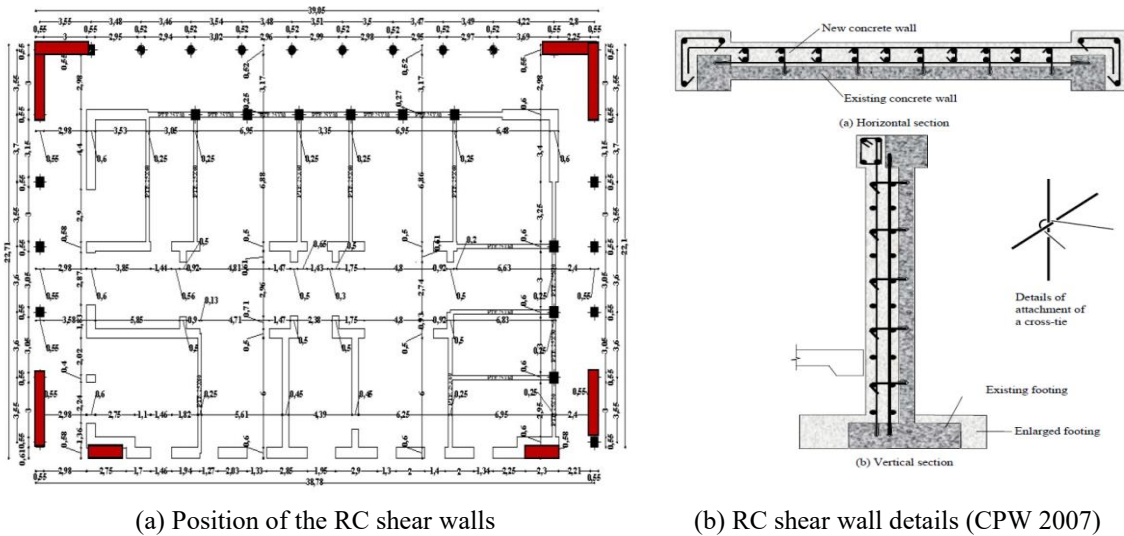(a) Position of the RC shear walls   (b) RC shear wall details (CPW 2007)

Fig. 12 View of the strengthened structure

between the resisting elements (e.g., tying of intersecting walls, tying of opposite parallel walls etc.), the reduction of the eccentricity between the mass and the stiffness centers, to avoid a large torsional effects, repair of cracked walls, application of vertical and horizontal confining elements to the walls, strengthening of and stiffening of horizontal diaphragms, strengthening of walls by means of RC jackets and last but not least strengthening of foundations (Gesualdo and Monaco 2011), (Sezen 2012).

Therefore, and based on our site investigation, it has been decided to insert RC shear walls (jackets) with a better behavior factor (thickness=15 cm) at the edges of the building in both longitudinal and transversal direction and also stiffening the horizontal diaphragms of the structure, as shown in Fig. 12(a)- (b)

The strengthened structure was reanalyzed using the same procedure (methodology). The main results of the strengthened structure are summarized in Tables 7-15, respectively, for the transversal and the longitudinal directions.

Fig. 13(a)-(b) and Fig. 14(a)-(b) show the new capacity and the demand in terms of absolute displacements and ductility for the proposed method of strengthening

Table 7 Distribution of the transversal and the longitudinal seismic forces for the strengthened structure

Level	W_i (KN)	H_i (m)	F_i^y (KN)	V_i^y (KN)	F_i^x (KN)	V_i^x (KN)
5	2577.35	3.75	1623.86	5233.58	1546.53	4984.36
4	4217.35	4.35	1280.05	6513.63	1219.09	6203.45
3	6707.35	3.75	1049.06	7562.69	999.10	7202.55
2	4117.35	3.75	804.38	8367.07	766.08	7968.63
1	5057.35	4.15	311.54	8678.61	296.70	8265.33

Table 8 Shear capacity; shear demand and safety factors in the transversal and the longitudinal directions for the strengthened structure

Level	Q_u^y (KN)	V_i^y (KN)	F_s^y	Q_u^x (KN)	V_i^x (KN)	F_s^x
5	14471.61	5233.58	2.76	14895.87	4984.36	2.98
4	20835.34	6513.63	3.19	10902.20	6203.45	1.75
3	22298.13	7562.69	2.94	13800.63	7202.55	1.91
2	23129.93	8367.07	2.76	15456.10	7968.63	1.93
1	21388.91	8678.61	2.46	15033.66	8265.33	1.81

Table 9 Capacity and demand of absolute displacements (cm) for Amax=0.25 g in the transversal direction for the strengthened structure

Level	δu cap	H/(150/175)	Ulcinj	El Centro	Cherchel
5	7.52	2.30	4.12	5.82	3.24
4	6.83	2.67	3.82	5.25	3.05
3	5.65	2.30	2.63	3.93	1.94
2	4.79	2.30	1.64	2.05	1.02
1	3.65	2.55	1.05	0.97	0.80

Table 10 Capacity and demand of absolute displacements (cm) for Amax=0.25 g in the longitudinal direction for the strengthened structure

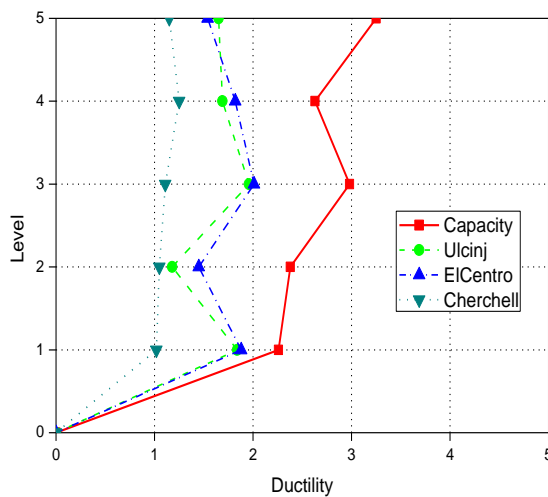| Level	δu cap	H/(150/175)	Ulcinj	El Centro	Cherchel
5	6.83	2.30	5.42	5.97	3.85
4	5.53	2.67	3.36	4.97	3.17
3	6.02	2.30	2.45	3.63	2.32
2	5.81	2.30	2.15	3.14	1.87
1	3.83	2.55	1.65	2.15	1.35
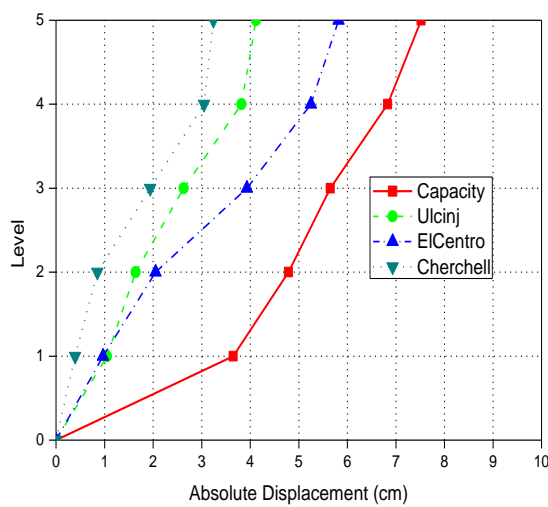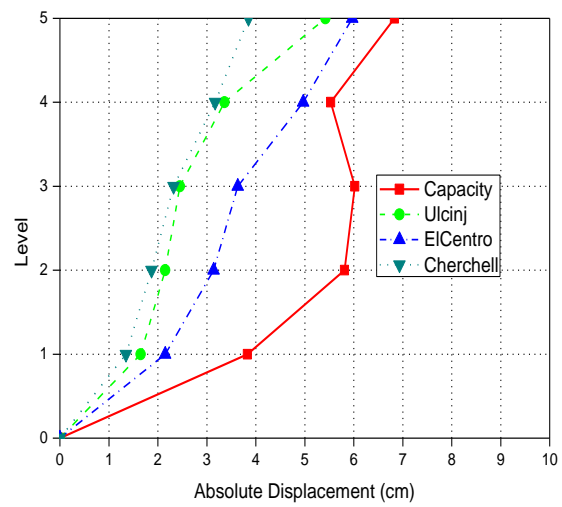
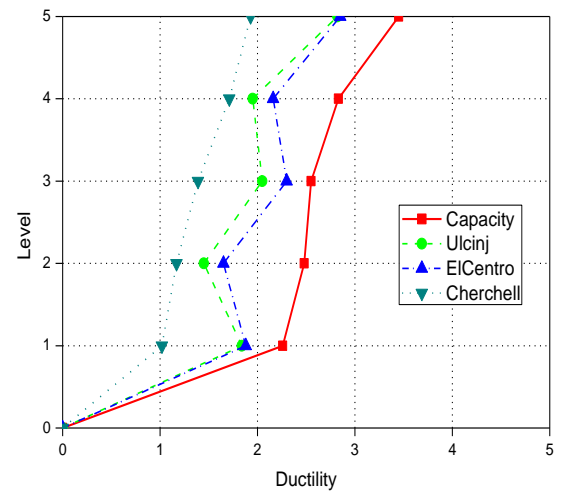(a) Longitudinal direction X-X   (b) Transversal direction Y-Y

Fig. 13 Displacement capacity and demand for strengthened structure **Amax=0.25 g**

(a) Longitudinal direction X-X   (b) Transversal direction Y-Y

Fig. 14 Ductility capacity and demand for strengthened structure **Amax=0.25 g**

Figs. 13(a)-(b) and 14(a)-(b) show clearly, that in terms of both displacement and ductility for a major earthquake (Amax=0.25 g), the new capacity of the strengthened building increased significantly and overcome the demand caused by the three selected accelerograms. This strengthening is just a proposal. If the owner of the building opted for this proposal, a detailed study of the proposed method will be executed by considering the cost and feasibility of the proposed retrofitting.

4. Conclusions

The quantitative evaluation of seismic vulnerability on a regional level indisputably constitutes the necessary condition to an objective perception of the seismic risk. In this study, a methodological approach of seismic vulnerability assessment of the strategic buildings which constitute the Algerian inheritance has been implemented. A case study of one of these strategic buildings located in the city of Constantine, east part of Algeria, was presented.

The original structural system showed an important lack and deficiency in capacity criteria in terms of forces, displacements and ductility at each level of the building, due the fact that this structure was build without any seismic regulations, which led us to suggest a strengthening solution to the building. One of the most difficult problems of strengthening of an existing building is how to find the most adequate solution that satisfies both economical and technical aspects. In our case, many solutions were carried out to get the best and feasible solution. Eight additional RC walls placed at the external frames were inserted to the existing system. This retrofitting method showed a great improvement in the capacity of the building, and satisfied the criteria of the methodology in terms of displacements and ductility.

The approach developed in this context is a basic tool for risk reduction in Algeria. Although this study does not take into account certain basic parameters in the quantification of the vulnerability, it enables us nevertheless to take certain preventive measures in order to attenuate possible consequences of a future earthquake.

Acknowledgments

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