Vulnerability assessment of an existing reinforced concrete shear wall building

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Abstract

The present study focused on assessing seismic vulnerability of an existing dwelling building with reinforced concrete shear walls, designed according to previous standards without seismic design. Firstly, visual inspection of the building is performed in order to identify defects, then, qualitative seismic vulnerability is assessed using existing methods. Finally, the compliance of the structure with some requirements of the current Algerian seismic regulations is examined.

Keywords: existing building, seismic vulnerability, shear walls.

1. Introduction

The existing reinforced concrete (RC) buildings, prior to 1981, that the characteristics are not perfectly known, represent an important part of the building stock in Algeria. Their design without seismic measures, makes difficult to predict their behaviour in case of future earthquake. The seismic vulnerability of RC structures is affected firstly by the quality of their resistant system and by a number of factors, like regularity, quality and workmanship and ductility.

Seismic vulnerability, as a part of seismic risk, is defined as the intrinsic predisposition of a certain element to suffer a certain level of damage when subjected to a seismic event of defined intensity [1,2]. Seismic risk, is defined as the result of combinition between hazard and vulnerability of the exposed elements. Seismic hazard, expresses the probability of earthquake occurrence. It can be defined as the probability of occurrence of the adverse consequences to society (economic losses, deaths, physical damage…) [3]. Thus, building vulnerability assessment is an essential step in risk assessment.

A number of methods have been developed during recent years to estimate damage for a given level of earthquake. The classification of these methods is based on the criteria used in the evaluation study and on the scale of application;

individual buildings, aggregate of buildings or urban area [2]. The detailed approaches used for individual buildings, implies a detailed evaluation with a necessary and sufficient level of information regarding the analyzed structure. Calvi and al. (2006) [4] divided the various existing methods of vulnerability assessment into two main categories; empirical and analytical methods, both of which can be used in hybrid methods. Empirical methods for the seismic vulnerability assessment of buildings are essentially based on the damage observed after the past earthquakes. The selection of one of these methods depends also on the available information and the level of evaluation required, which sometimes involves a transition from a simple qualitative visual method to a detailed analytical method, [2,5].

In Algeria, following the 1980 Chelef earthquake, the first Algerian seismic regulation RPA81 was published. It was revised in 1983, 1988, 1999 and 2003 (RPA99 version 2003) [6] following the 2003 Boumerdes earthquake, which caused considerable human and material losses [7,8]. Some studies have focused on assessing the vulnerability of existing RC buildings built prior to 1981's code [9,10], however there are several types with different structural systems.

In this study, we try to assess the qualitative vulnerability of an existing reinforced concrete shear wall building, built in 1980 without seismic design. The study is performed according to the following steps: (a) vulnerability presomption is assessed according to the European Macroseismic Scale EMS-98 definitions [11], then, a vulnerability index is established according to the AFPS method (Battier group) [12]. (b) Building structural analysis is performed in order to obtain its dynamic characteristics, and verify if it meets seismic requirements of the current code.

1.1 Chronology of Algerian seismic regulations

Figure 1 shows the publication and revision dates of the Algerian seismic code (RPA), in relation with the major earthquakes occured in Algeria:

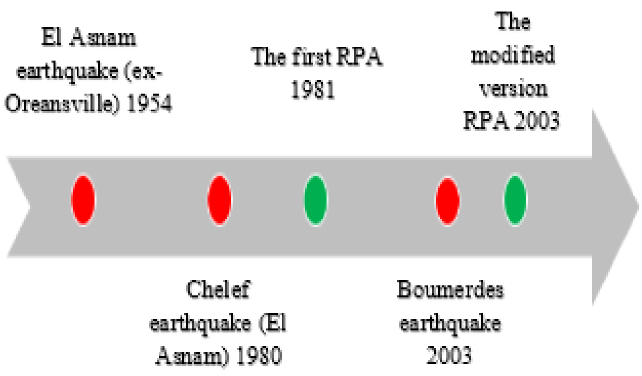


Figure 1. *Chronology of Algerian seismic regulations.*

- Before 1954 and between 1954 and 1980, there was no seismic code.
- Between 1980 and 2003 (before the 2003 Boumerdes earthquake): despite the seismic code that already existed and revised four times, the knowledge on earthquake and seismic risk remained rudimentary.
- After 2003: the seismic code was revised (RPA 99 version 2003). The earthquake knowledge has improved, and important seismic risk studies have been completed, but the seismic risk mitigation mesures remained limited.

2. Presentation of the studied building

The studied dwelling building was built in 1980. It is located in the "800 logements area" of Boumerdes town. It is a five-stories RC shear walls structure (Figures 2 and 3). On its two longitudinal facades, there are prefabricated reinforced concrete filling panels. Its plan dimensions are 17.46m ×11.58m, with a total height of 14.14m. This dwelling building was designed according to old standards (CCBA68) rules without seismic design.

2.1 Building visual inspection – Damage to the building during the 2003 Boumerdes earthquake

The building resisted well in the 2003 earthquake but still sustained some slight structural and non-structural damages. Post-seismic expertise had classified the building in the "green category" of damage (level 2), corresponding to

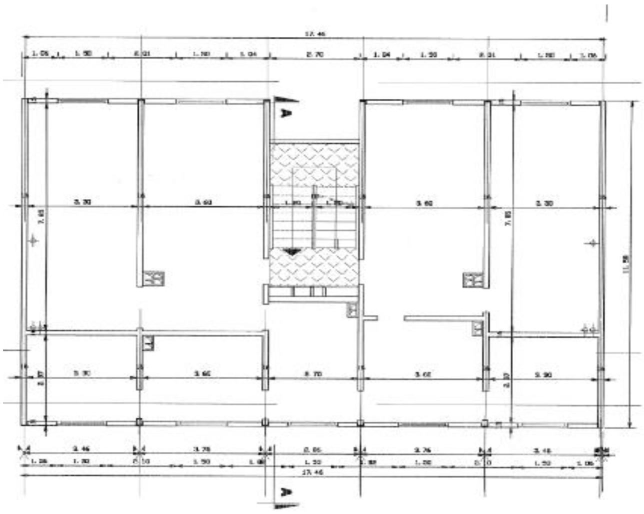


Figure 2. *Plan view of the building.*

Figure 3. *Main facade of the building.*

moderate damage of the European Macroseismic Scale EMS-98. Thus, his immediate reoccupation was authorized. The reported damages were related to its previous state, such as; construction deficiencies, the corrosion of the reinforcing steel, spalling of concrete cover, segregation of the concrete at the bottom of walls. Due to the earthquake, craks appeared at joints due to the displacement of the prefabricated facade panels.

The building was repaired promptly after the 2003 earthquake. However, in its existing state, shows the same disorders, due probably to the lack of maintenance and the exposure to marine air.

Figure 4. *Degradations observed on the building.*

2.2 Vulnerability presumption of the studied building

2.2.1 Typological classification and presumption of vulnerability according to EMS-98

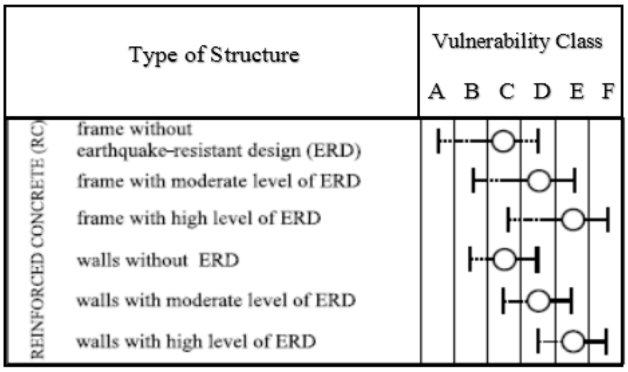
The EMS-98 scale [11], classifies RC buildings into six (06) seismic categories (typologies) and assigns them vulnerability classes (A, B, C, D, E and F) according to their type of structure and their earthquake-resistant design (ERD), as shown in the Vulnerability Table (Figure 5).

According to the Vulnerability Table, the building belongs the typology of reinforced concrete buildings with walls without earthquake resistant design (ERD), hence the typological vulnerability class of such building is class C.

Based on the good behavior of the building during the 2003 earthquake, and the definitions provided by the scale, RC structures with modern structural systems, not designed against lateral seismic loads, can still offer a certain level of earthquake resistance which can be comparable to the level integrated in RC buildings with ERD [11], so the building can be equated to class D.

2.2.2 Presumption of vulnerability according to AFPS method (Battier Group)

The Battier method was developed by the French



○ Most likely vulnerability class; ▬ probable range;

▬ ▪ ▪ Range of less propable, exceptional cases

Figure 5. *Vulnerability classes according to EMS-98 for reinforced concrete structures [11].*

Association of Seismic Engineering, AFPS (Association Française du Génie Parasismique) [12]. It allows to assess the vulnerability of RC buildings using a scoring system. A single building or a group of buildings can be considered.

To assess seismic qualitative vulnerability, a vulnerability index is calculated by summing all the scores assigned to each item in the AFPS Table (Figure 6) using equation 1. The items are essentially ; structural features, resistant system, regularity in plan and elevation, critical elements and site.

$$V = \sum K_i \qquad (1)$$

The presumption of vulnerability is: high for : $\sum K_i > 50$, moderate for : $25 < \sum K_i \leq 50$, low for : $10 < \sum K_i \leq 25$, and very low for : $\sum K_i \leq 10$.

The obtained vulnerability index value is V=30, consequently the building can be classified as moderately vulnerable against earthquake loads.

3.1 Dynamic characteristics of the structure

The dynamic analysis of the structure allows to obtain, among others, the periods and vibration modes. The first mode of vibration is a translation on x-x direction, with a fundamental period T_1 = 0.244 sec. The second mode of vibration is a translation on y-y direction, with a period T_2 = 0.097 sec. The third mode of vibration is a torsion on z-z

Critère	N°	Description	Pénalité
A Implantation du bâtiment	1	Pente générale du terrain >40%	5
	2	Proximité d'un changement de pente D<2H du bâtiment	15
B Environnement du bâtiment	1	Bâtiments accolés : joint = 0 ou rempli d'un matériau	25
	2	Joints entre blocs adjacents: <2cm / 2 à 4 cm / >4 cm	25 / 10 / 5
C Type de structure	1	Murs en maçonnerie de blocs	15
	2	Murs en béton non armé	10
	3	Murs en béton armé	5
	4	Ossature poteaux-poutres sans remplissage	20
	5	Ossature poteaux-poutres avec remplissage	25
	6	Système mixte murs en maçonnerie et ossature	20
	7	Panneaux de façade BA préfabriqués porteurs	10
	8	Ossature BA préfabriquée porteuse	50
D Forme en plan	1	Irrégulière	5
	2	Elancement en plan L/l>4	5
	3	Parties saillantes ou rentrantes	5
E Forme en élévation	1	Etages en encorbellement >2m	15
	2	Retraits en façade >40%	20
	3	Planchers d'un même étage situés à des hauteurs différentes	10
	4	Présence d'un plancher lourd ou d'une toiture lourde	10
	5	Absence de diaphragme horizontal en toiture	20
F Contreventement	1	Variation verticale croissante des rigidités	0 à 100 (voir formule 1)
	2	Dissymétrie : torsion	faible : 5 accusée : 50
	3	Absence de contreventement dans le sens des x ou y	100
	4	Densité de voiles de contreventement sens x et/ou y	0 à 100 (voir formule 2)
G Zones ou éléments critiques	1	Descente de charge en balonnette	25
	2	Présence de poteaux courts ou partiellement bridés participant au contreventement	50
	3	Présence de poteaux élancés	10
	4	percements inserts dans les poteaux e>d/3	25
	5	percements inserts dans les poutres e>d/3	10
	6	percements inserts dans les nœuds e>d/3	50
	7	Présence d'un angle de façade affaibli	15
	8	Axes poteaux et poutres non concourants e>c/2	10
	9	Diaphragmes horizontaux avec grandes ouvertures s>10%S	10
	10	Absence de chainages encadrant les murs de contreventement en MAC	verticaux : 25 horizontaux : 75
H Divers	1	Etat de conservation du gros œuvre	médiocre : 10 mauvais : 25
	2	Risque de chute d'éléments non structuraux	5
	3	Façade BA préfabriquée non porteuse	10
Total des pénalités partielles :			

Formule 1 $K = 50 (\mu^{1/3}-1)$ avec $\mu = \Sigma I$ supérieur / ΣI inférieur (cf. figures données en page suivante)

Formule 2 $K = 25 (1000 \lambda - 5)^2 / 4$ avec $\lambda = \Sigma I / S\,H$ (cf. figure donnée en page suivante)

dans ces formules : Σ I = somme des inerties des segments de voile dans la direction de calcul (m4)
S = surface du plancher courant (m2)
H = hauteur totale du bâtiment (m)

Figure 6. *AFPS Table of Scores [12].*

direction, with a period T_3 = 0.087 sec.

In the x-x response direction, for the first four (04) modes, 90% of the masses participate in the sollution. But, in y-y response direction, we need more than twelve modes to reach 90% of mass participation.

According to the seismic code, RPA 99 version 2003 [6], the fundamental period (T) of the structure is estimated using the following empirical formula (2) :

$$T = C_T h_N^{3/4} \qquad (2)$$

With :

C_T: coefficient, function of the lateral force resisting system and of the type of infill.

h_N: building height measured in meters from the basis of the structure to the top of the last level (N).

In the case of partially or totally RC shear walls, braced frames and masonry walls, the formula (3) can also be used:

$$T = 0.09 h_N / \sqrt{D} \quad (3)$$

D is the dimension of the building measured at its base in the considered direction.

In this case study, for each considered direction, the smaller of the two values obtained using respectively equations (2) and (3) is :

$$\begin{cases} T_x = \min(0.365, 0.304) = 0.304 \sec \\ T_y = \min(0.365, 0.374) = 0.365 \sec \end{cases}$$

According to the RPA99 version 2003 [6], the numerical value of the fondamental period must not exceed that estimated using empirical formula by more than 30%. So, we have: $\begin{cases} T_x^{num} = 0.244 \sec < T_x^{RPA} = 0.395 sec \\ T_y^{num} = 0.097 sec < T_y^{RPA} = 0.473 sec \end{cases}$

Results show that the condition is satisfied.

The code requires that the resultant of the seismic forces V_t at the base of the building obtained by combination of the modal values should not be less than 80% of the resultant of the seismic forces V determined by the equivalent static method for a value of the fundamental period given by the empirical formula. V is calculated successively in both horizontal and orthogonal directions, according to formula (4):

$$V = \frac{A \cdot D \cdot Q}{R} W \qquad (4)$$

A: zone acceleration coefficient

D: average dynamic amplification factor

Q: quality factor

R: behaviour coefficient of the structure

W : total weight of the structure

The results summarized in Table 1, show that the resultants of the seismic forces satisfy the previous condition required by the seismic code.

Tableau 1. *Static and dynamic seismic forces.*

Sens	V^{sta} (KN)	$0.8V^{sta}$ (KN)	F_x (KN)	F_y (KN)	$V=\sqrt{F_x^2+F_y^2}$ (KN)
X-X	1961.97	1569.58	1845.22	36.66	1845.59
Y-Y	1961.97	1569.58	36.66	1954.46	1954.46

3.2 Story relative lateral displacement:

The story relative displacement at level "k" in comparison with level "k-1" is obtained using equation (5).

$$\Delta = \delta_k - \delta_{k-1} \;\; ; \;\; \delta_k = R\delta_{ek} \quad (5)$$

δ_k : horizontal displacement at each level "k" of the structure.

δ_{ek} : displacement due to seismic forces F_i (including torsional effect).

R: behaviour coefficient.

The displacement between two adjacent stories must not exceed 1.0% of the height of the story, as required by the seismic code.

As shown in Table 2 and Figure 7, on y-y direction, the stories relative displacements are not exceeding 1.0% story height, however, on x-x direction the condition is not verified. It is probably due to the insufficient RC walls in this direction.

Tableau 2. *Story relative displacement.*

Stories	X-X		Y-Y	
	Δ_k (cm)	Δ_k / h_k (%)	Δ_k (cm)	Δ_k / h_k (%)
1	0,2191	0,0777	0,0339	0,012
2	0,3552	0,1259	0,0571	0,0202
3	0,3889	0,1379	0,0686	0,0243
4	0,3675	0,1303	0,071	0,0252
5	0,3255	0,1138	0,069	0,0241

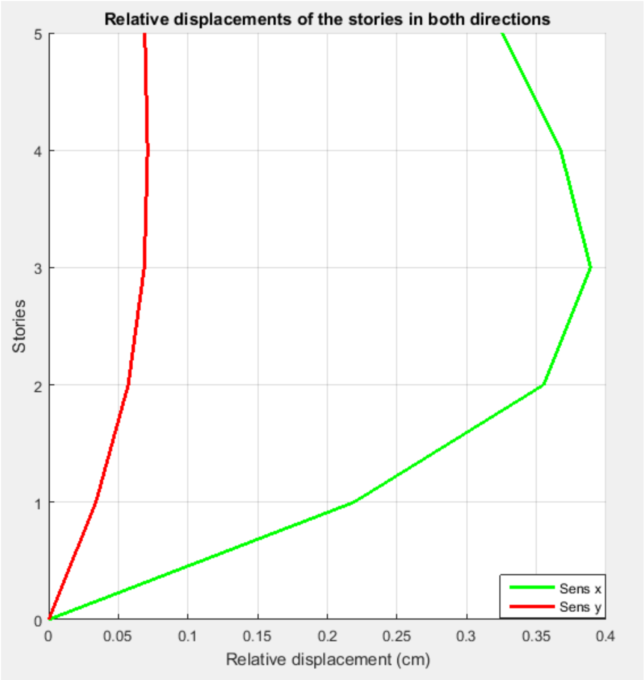


Figure 7. *Relative story displacement.*

4. Conclusion

The vulnerability assessment of an existing dwilling RC shear wall building is conducted according the EMS-98 guidelines and the AFPS method. The vulnerability class assigned to the building and the AFPS vulnerability index obtained allowed to classify the building as moderately vulnerable to seismic loads. The building was not designed against lateral seismic forces, but it behaved well in the 2003 Boumerdes earthquake and provided a certain level of ERD comparable to building belonging vulnerability class D.

The structural dynamic analysis of the building, show that its structural design complies with some requirements of the current Algerian seismic regulations. Despite the few non-structural and structural degradations shown, the building can be regarded as having inherent earthquake resistance. This behavior is related to its structural system that provide horizontal stiffness and therefore improve performance. The strength of the floors of a building and the horizontal stiffening elements, often plays a key role in deciding the vulnerability of a structure.

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