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Seismic Behavior of Reinforced Masonry Structure: Relation between the Behavior Factor and the Ductility

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Abstract

The present work concerns a numerical study of the behavior of reinforced masonry (RM) structures under seismic loading. These structures are made of small hollow elements with reinforcements embedded in the horizontal joints. They were dimensioned according to the rules and codes commonly used. They are subject to vertical loads due to their own weight, and to horizontal loads due to seismic forces introduced by the accelerograms. The software used is the non-linear analysis program Drain2D, based on the finite element method, where the shear panel element was introduced. A series of calculations was performed on a number of structures at different levels, excited by three major accelerograms (El Centro, Cherchell, and Kobe). Throughout the study, our main interest is to evaluate the behaviour factor, the ductility, and the failure mode of these structures while increasing the intensity of earthquakes introduced. The results of this present study indicate that the average values of the behaviour factor and the global ductility are of the order of $q \approx \mu \approx 3.00$. The reinforced masonry structures studied have been broken by interstage displacement. The results given by the study are comparable to those given in the literature and in Eurocode 8. The behavior of reinforced masonry under a seismic load is similar to the behavior of reinforced concrete; it is a ductile behavior that allows the dissipation of the energy transmitted by the earthquake. These numerical studies confirm and complete the experimental work carried out by other researchers.

Keywords: Reinforced Masonry; Elastic Inelastic; Earthquake; Failure Mode; Behavior Factor; Ductility.

1. Introduction

The idea for this work comes from the fact that northern Algeria has practically exhausted its building potential. Knowing that, the south of the country presents opportunities to build in order to lodge the populations, especially when the exploration is still relevant and the future is there. In addition, recent events prove that even this area of Algeria is prone to earthquakes such as the BISKRA earthquake. However, it is then necessary to build and take the measures to deal with this natural hazard.

The first concern that the design engineer must have is to provide provisions ensuring the general stability and especially the bracing of all the buildings. The purpose of these requirements is not only to ensure the resistance to the horizontal forces taken into account in calculations, such as those resulting from the action of the earthquake, but also to enable buildings to withstand, without undue damage, the effects of certain stresses, such as localized explosions [1, 2]. The damage encountered in structures under seismic loading differs according to the type of bracing chosen [3, 4], the metal bracing, reinforced concrete, or reinforced masonry cited in the seismic code [5-7], which is characterized by

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its coefficient of behavior [8, 9], its energy dissipation capacity, and its ductility [10]. When seismic solicitations are important, the lack of ductility makes the structures vulnerable to rupture. In this work, we study the bracing of reinforced masonry [11].

In the literature, masonry is defined as "the art of tidying brick and stones with mortar or other bonding". As is known, masonry is a good material for facing compressive forces, and bad compared to tensile forces. From where, the idea of arming the masonry as the reinforcements have very good resistance to tensile forces [10-12]. Masonry in general and reinforced masonry in particular have very complex mechanical behaviour due mainly to their heterogeneity. Its behaviour is a function of the mechanical characteristics of the masonry elements, the mortar that composes them, and possibly reinforcements [13, 14], as well as the interaction of these three constituents [15]. In addition, the mechanical behaviour of the masonry is greatly influenced by the applied solicitations, mainly by the conditions of their implementation [11, 16].

The brittleness of the failure of unreinforced masonry shear walls, which is more remarkable with high axial loads, may be reduced by the use of steel reinforcement. The role of the horizontal reinforcement on the shear resistance of masonry walls has been investigated in the perspective of the development of novel solution for reinforced masonry walls [10, 17].

There are two types of reinforced masonry: The first type is horizontally reinforced masonry, where the armature is embedded in the horizontal joints. The second type is vertically reinforced masonry, where the armature is placed in the hollow elements vertically and then filled with concrete or cement slurry [18, 19]. The method of placing reinforcement in mortar either horizontally or vertically is described by Fódi (2011) and Araya-Letelier et al. (2019) [20, 21]. Husain et al. [22] carried out documentary and comparative research on the behavior of unreinforced masonry and reinforced masonry based on modeling strategies and methods, mechanical behavior, and influencing factors available in the technical literature.

Most of the work done in the field of reinforced masonry is based on experimentation, such as; Kumar et al. [23] in their experimental analysis of reinforced and unreinforced brick masonry walls with specified dimensions and different properties. Koutras et al. [24] studied the seismic behavior of reinforced masonry structures, and Cheng et al. [25] evaluated the collapse resistance of reinforced masonry by shake-table tests. Ahmadi et al. [26] and Banting et al. [27] studied the displacement of reinforced masonry structures under seismic parameters. Tomazevic et al. [20] studied the behaviour of reinforced masonry structures. Potro et al. [28] worked on the behaviour of a new reinforced masonry system In-plane cyclic.

The present paper focuses on the determination of the reinforced masonry structures' behavior, their modes of failure, and the relationship between the behavior factor q and the overall ductility. These structures are made of products of small elements hollow with reinforcements embedded in the horizontal joints, excited by several earthquakes. These structures have two, three, four, and five levels.

Our research is based on a numerical study of reinforced masonry structures, made possible by a non-linear analysis program called Drain 2D based on the principles of the finite element method [15, 29, 30]. This numerical study completes and approves the experimental work carried out by other researchers based on the behavior of reinforced masonry under seismic loads, the behavior factor, and the global ductility [15, 20, 21, 31].

1.1. Behavior Factor

The role of a bracing is to dissipate the energy transmitted by the seismic action. This quality is described in the literature by a coefficient called of global behaviour factor of the structure. Each calculation code defines it by a name specific to the country of origin. The Algerian paraseismic regulation defines it by the coefficient "R" and Eurocode 8 by the coefficient "q".

The behavior factor "q" of the structure [2, 4] is defined by Equation 1, given below:

$$q_i = \frac{\lambda_{max}}{\lambda_e} \tag{1}$$

- A multiplier λ_e such that the displacement inter-stages D_e is achieved.
- A multiplier λ_{max} such that the maximum displacement inter-stages D_{max} is reached.

1.2. Ductility

Ductility by definition is the ability of a material or an element of a structure to elongate in the plastic domain, without breaking or losing strength [2, 32]. In the present study, we consider structural elements whose characteristic under cyclic action is of the perfectly plastic elastic type. The ductility is defined as follows:

$$\mu = \frac{D}{D_e} \tag{2}$$

The research flow chart is shown in Figure 1.

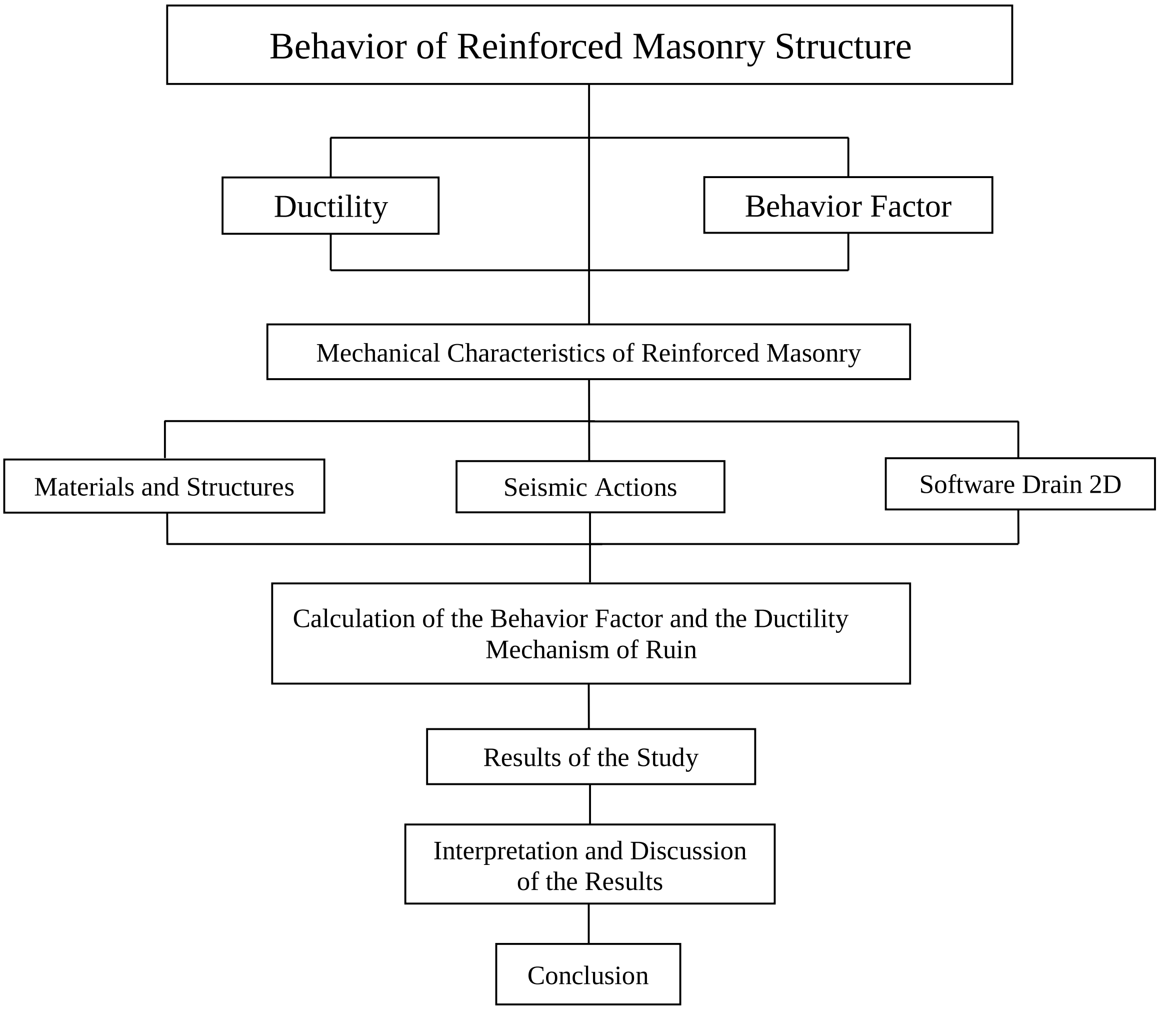

Figure 1. Flow chart of the study

2. Behavior Law and Mechanical Characteristics of Reinforced Masonry

According to Eurocode 6 [6] and Ghassan [15] project, the calculation is based on the same principles as for reinforced concrete element.

It is assumed that the masonry achieved:

$$F_{Mua} = 0,85\ F_k \tag{3}$$

$$F_k = K\ .f_b^{0,65}\ .f_m^{0,25} \tag{4}$$

When steel reaches its ultimate value F_{su} = 400 N/mm^2, the equilibrium equation of forces gives:

$$F_{masonry\ armed} = F_{steel} \rightarrow 0,8\ .b\ .y\ .F_{Mua} = A_s\ .F_{su} \text{ and } y = \frac{A_S\ .F_{su}}{0,8\ .b\ .F_{Mua}} \tag{5}$$

$$\frac{\varepsilon_S}{d-y} = \frac{\varepsilon_{Ma}}{y} \text{ and } \varepsilon_{Ma} = \frac{y}{d-y}\ \varepsilon_S \tag{6}$$

Such as:

$$F_{Mua} = \varepsilon_{Ma}\ .E_{Ma} \tag{7}$$

$$E_{Ma} = \frac{F_{Mua}}{\varepsilon_{Ma}} \tag{8}$$

since f_{ma} and ε_{ma} are defined, we can have E_{ma}. It only remains to define f_k.

The mechanical characteristics of the studied structures are summarized in Table 1.

Table 1. Mechanical characteristics of the masonry studied

Mechanical characteristics	f_m N/mm²	f_b N/mm²	f_k N/mm²	δ	γ_m	f_{vk} N/mm²	f_{vd} N/mm²	E N/mm²	G N/mm²
Values	10	4.60	2.03	1.15	2.2	0.3	0.136	1.83×10^5	0.732×10^5

3. Materials and Method

3.1. Constituent Materials and Structures

The structures studied are reinforced with two bars of diameter (8mm) welded in zigzags and embedded in the horizontal joints, made of hollow bricks of dimension ($200\times200\times500mm^3$).The mortars used are of high strength which ensures a great adhesion between the elements of the armed masonry. The structures are respectively two, three, four and five levels. They are shown in Figure 2.

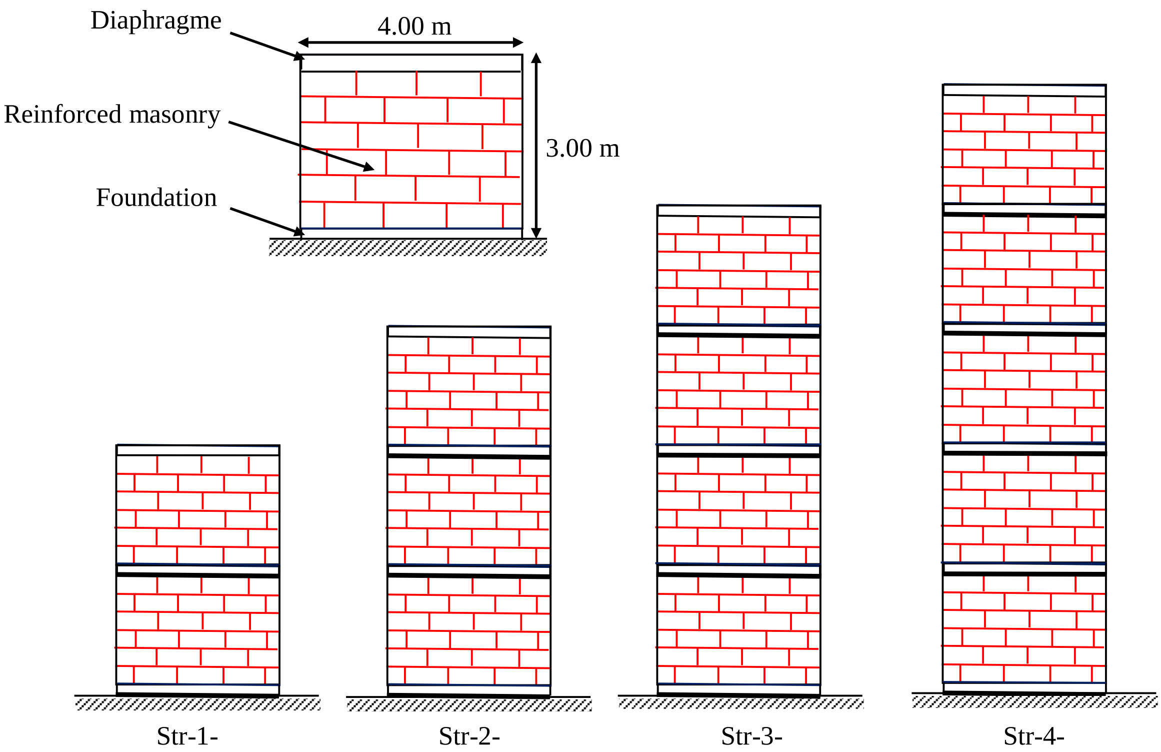

Figure 2. Elevation view of the different structures studied

3.2. Study Software

The software used is the Drain2D program developed by A. Kanaan and G.H Powel at the University of California (Berkeley) [33].

It has the following elements:

a) A beam element that flexes plastically.

b) A column beam element that plasticizes by forming plastic hinges with rigid nodes.

c) A semi - rigid joining element.

d) A shearing sail member that has shear stiffness only.

e) A bar element that plasticizes in tension and flames elastically in compression.

In the present study, a shear web element is taken into account (element (d)).

3.3. Seismic Action

Pre-sizing of these structures was done according to the requirements of the regulatory codes. A vertical descent of loads was made as well as horizontal loads which are given by seismic forces introduced with three accelerograms:

Seismic actions are defined by three accelerograms that were used in this study:

- The earthquake accelerogram El Centro, California from May 18, 1940 with a magnitude of 6.9 on the Richter scale.
- The earthquake accelerogram Cherchell in Algeria from October 29, 1989 a magnitude 6.2 to 6.8 on the Richter scale.
- The accelerogram of the Kobe earthquake in Japan on January17, 1995 with a magnitude of 7.2 on the Richter scale.

3.4. Calculation of the Behavior Factor and the Ductility

The seismic action is increased by multiplying accelerogram *a(t)* by the coefficient λ, ([$a(t)\ x\ \lambda$]), so as to find the particular values of each state of the different structures, and reach an inter-stage displacement corresponding [7, 34, 35]. To:

$$\delta_{(i+1)}-\delta_i=1\%H \tag{9}$$

$$\delta_{(i+1)} - \delta_i = 3\%\ H \tag{10}$$

The particular values of each state of the various structures characterized by the behavior factor values and the responses in terms of displacement are defined as to $\frac{D_i}{D_e}$ [2, 7]. Thus the value of q corresponding to the intersection on the curve with the bisector (Figure 3) identifies the maximum value of the behavior factor of the structure.

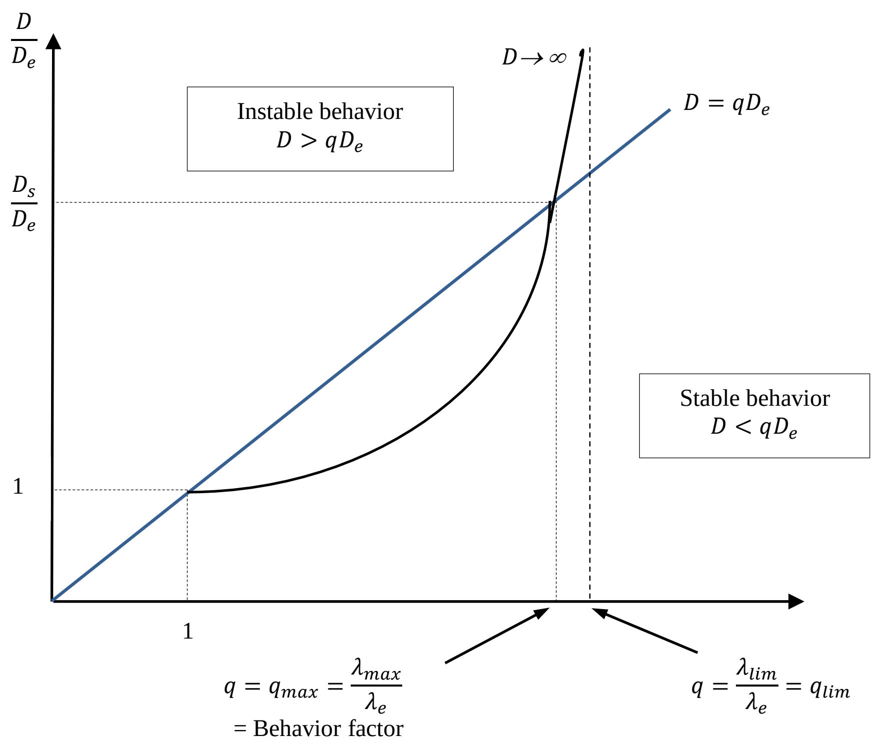

Figure 3. Property of the behavior factor q depending on the ductility [2]

3.5. Mechanisms of Ruin

Three modes of failure can be identified in reinforced masonry are given by multiple searches [36-39].

a) Out shear panels.

b) Failure by bending the panels.

c) Ultimate interstate Displacement.

In our case the mechanism (c) is used.

4. Results of the Study

In our research, we are interested in two aspects: the behavior factor and overall ductility. The study consists in finding a relative inter-stage displacement equal to *1% H*= 3.00cm. This state corresponds to global ductility value at $D_{max}/D_e = 1$ or to a global behavior factor $q = 1$ (1). This is achieved by increasing the excitation using the parameter *λ,* until finding an inter-stage displacement equal to *3% H*=9.00cm. This state corresponds to the ruin of the structures.

4.1. Behavior Factor

The values of the behavior factor are summarized in Tables 2 to 5; three accelerograms were used for the structures.

Table 2. Overall displacement of structure 1 for the three accelerograms used

Structure	Accelerograms	λ_i	Levels H_i	δ_i (cm)	δ_i-δ_{i-1} (cm)	D (cm)	D/D_e	$q_i = \lambda / \lambda_e$
1	El Centro	$\lambda_e = 4.80$	2	0.055	-	0.055	1	1
			1	0.025	0.03			
		$\lambda_l = 14.40$	2	0.164	-	0.164	2.98	3.00
			1	0.074	0.09			
	Cherchell	$\lambda_e = 6.80$	2	0.065	-	0.065	1	1
			1	0.035	0.03			
		$\lambda_l = 20.00$	2	0.193	-	0.193	2.93	2.94
			1	0.103	0.09			
	Kobe	$\lambda_e = 1.60$	2	0.059	-	0.059	1	1
			1	0.029	0.03			
		$\lambda_l = 4.70$	2	0.175	-	0.175	2.97	2.94
			1	0.085	0.09			

Table 3. Overall displacement of structure 2 for the three accelerograms used.

Structure	Accelerograms	λ_i	Levels H_i	δ_i (cm)	δ_i-δ_{i-1} (cm)	D (cm)	D/D_e	$q_i = \lambda / \lambda_e$
2	El Centro	$\lambda_e = 1,45$	3	0.043	-	0.043	1	1
			2	0.043	0			
			1	0.013	0.03			
		$\lambda_l = 4.40$	3	0.131	-	0.131	3.04	3.03
			2	0.131	0			
			1	0.041	0.09			
	Cherchell	$\lambda_e = 1.15$	3	0.051	-	0.051	1	1
			2	0.051	0			
			1	0.021	0.03			
		$\lambda_l = 3.37$	3	0.149	-	0.149	2.92	2.93
			2	0.149	0			
			1	0.059	0.09			
	Kobe	$\lambda_e = 0.43$	3	0.048	-	0.048	1	1
			2	0.048	0			
			1	0.018	0.03			
		$\lambda_l = 1.28$	3	0.143	-	0.143	2.98	2.98
			2	0.143	0			
			1	0.053	0.09			

Table 4. Overall displacement of structure 3 for the three accelerograms used

Structure	Accelerograms	λ_i	Levels H_i	δ_i (cm)	δ_i-δ_{i-1} (cm)	D (cm)	D/De	$q_i = \lambda / \lambda_e$
3	El Centro	λ_e = 1,50	4	0.051	-	0.051	1	1
			3	0.051	0			
			2	0.051	0			
			1	0.021	0.03			
		λ_l = 4.40	4	0.150	-	0.150	2.94	2.93
			3	0.150	0			
			2	0.150	0			
			1	0.060	0.09			
	Cherchell	λ_e = 1.00	4	0.051	-	0.051	1	1
			3	0.051	0			
			2	0.051	0			
			1	0.021	0.03			
		λ_l = 2.90	4	0.149	-	0.149	2.92	2.90
			3	0.149	0			
			2	0.149	0			
			1	0.059	0.09			
	Kobe	λ_e = 0.35	4	0.049	-	0.049	1	1
			3	0.049	0			
			2	0.049	0			
			1	0.049	0.03			
		λ_l =1.00	4	0.140	-	0.140	2.85	2.86
			3	0.140	0			
			2	0.140	0			
			1	0.05	0.09			

Table 5. Overall displacement of structure 4 for the three accelerograms used

Structure	Accelerograms	λ_i	Levels H_i	δ_i (cm)	δ_i-δ_{i-1} (cm)	D (cm)	D/De	$q_i = \lambda / \lambda_e$
4	El Centro	λ_e = 1,30	5	0.047	-	0.047	1	1
			4	0.047	0			
			3	0.047	0			
			2	0.047	0			
			1	0.017	0.03			
		λ_l = 3.90	1	0.140	-	0.140	2.98	3.00
			4	0.140	0			
			3	0.140	0			
			2	0.140	0			
			1	0.140	0.09			
	Cherchell	λ_e = 0.65	1	0.046	-	0.046	1	1
			2	0.046	0			
			3	0.046	0			
			4	0.046	0			
			5	0.046	0.03			
		λ_l = 2.06	5	0.145	-	0.145	3.15	3.15
			4	0.145	0			
			3	0.145	0			
			2	0.145	0			
			1	0.055	0.09			

Kobe	$\lambda_e = 0.33$	5	0.047	-	0.047	1	1
		4	0.047	0			
		3	0.047	0			
		2	0.047	0			
		1	0.047	0.03			
	$\lambda_l = 0.98$	5	0.138	-	0.138	2.94	2.97
		4	0.138	0			
		3	0.138	0			
		2	0.138	0			
		1	0.048	0.09			

The results of each Table are illustrated by following the figures, which represent the value of the behavior factor *q* as a function of ductility *D/De* of the structure. This last is shown in Figures 4 to 7 for each structure and accelerograms.

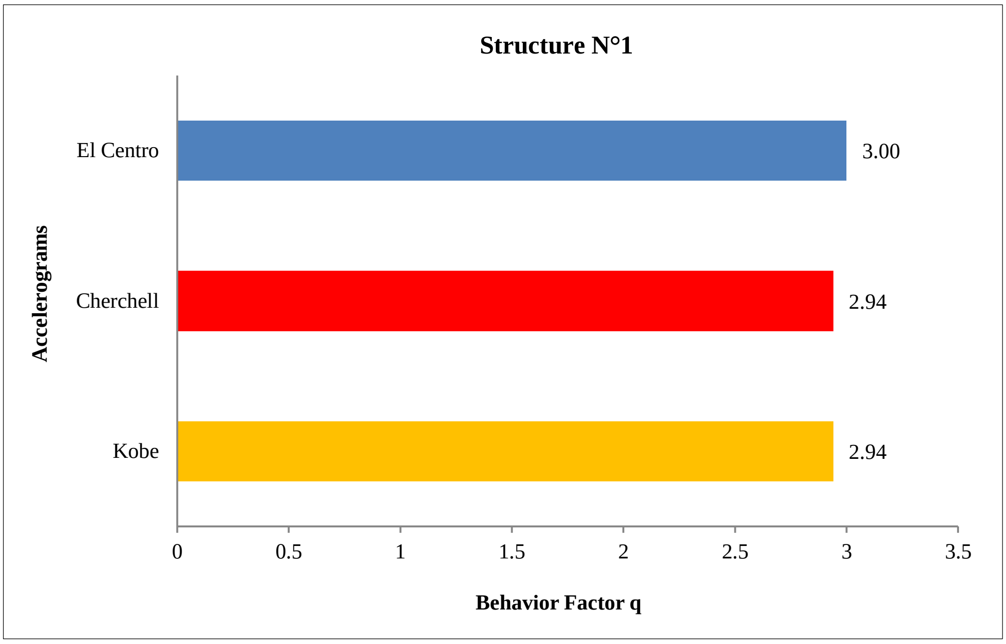

Figure 4. Variations of the q global behavior factor values for the structure 1 to different accelerograms

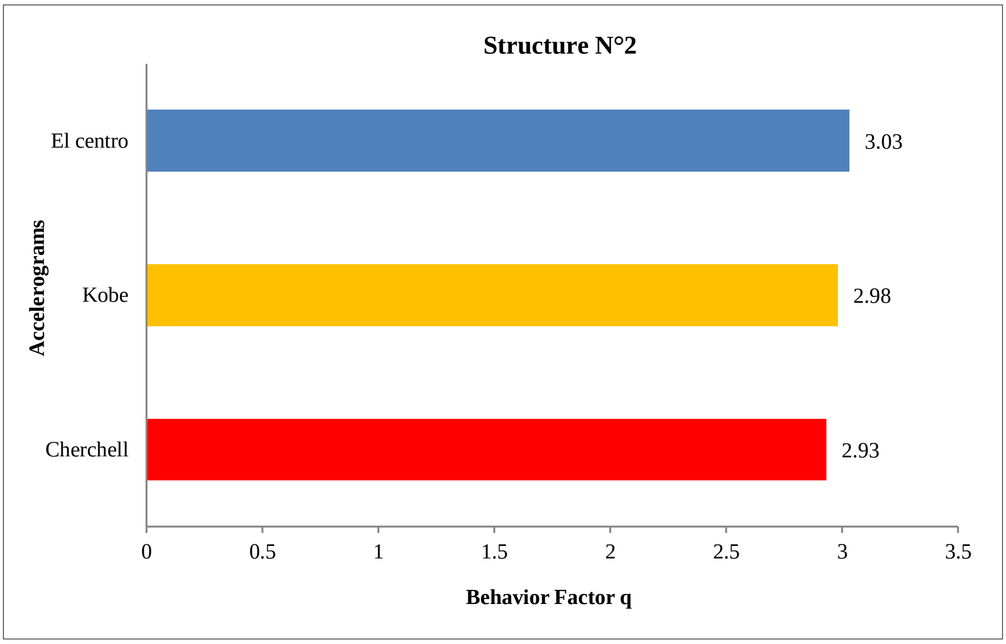

Figure 5. Variations of the q global behavior factor values for the structure 2 to different accelerograms

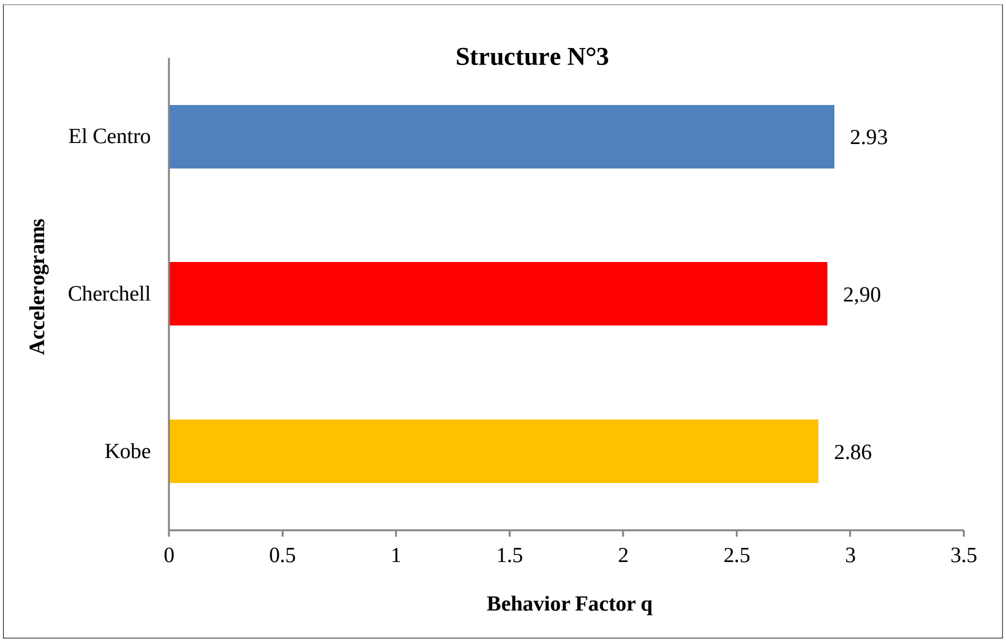

Figure 6. Variations of the q global behavior factor values for the structure 3 to different accelerograms

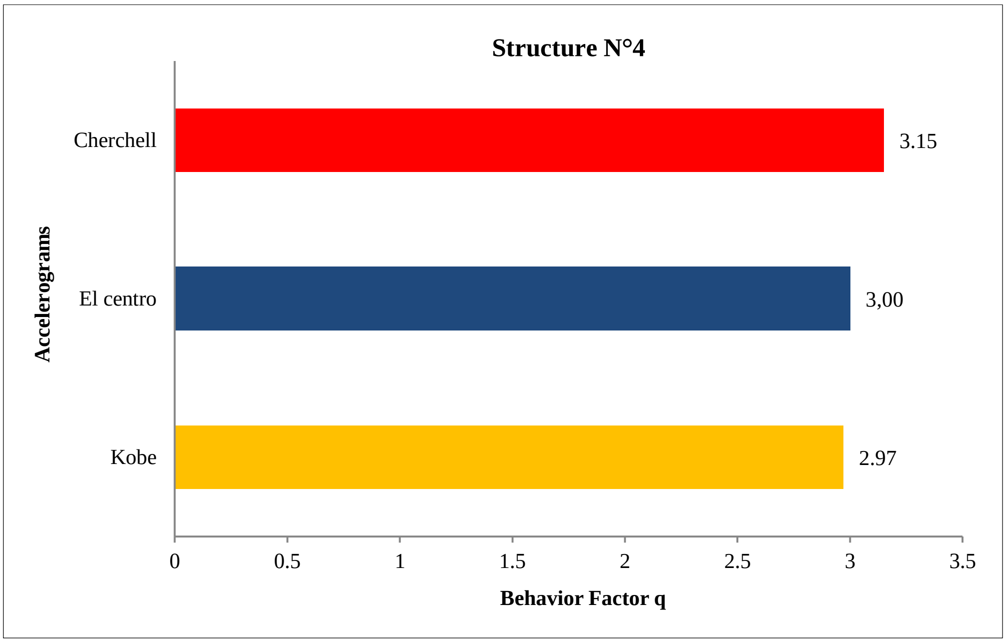

Figure 7. Variations of the q global behavior factor values for the structure 4 to different

4.2. Overall Ductility

The distribution of the overall ductility μ_i for each structure according to the height of the different structures and for each accelerogram is given in Tables 6 to 9 and in Figures 8 to 11.

Table 6. Distribution of the overall ductility required μ for structure 1 as a function of height

Structure	Accelerograms	$\frac{H_i - H_1}{H_t - H_1}$	$\mu = \frac{D}{D_e}$	μ_{moy}
1	El Centro	1.00	2.98	2.97
		0.00	2.96	
	Cherchell	1.00	2.92	2.865
		0.00	2.81	
	Kobe	1.00	2.97	2.95
		0.00	2.93	

Table 7. Distribution of the overall ductility required μ for structure 2 as a function of height

Structure	Accelerograms	$\frac{H_i - H_1}{H_t - H_1}$	$\mu = \frac{D}{D_e}$	μ_{moy}
2	El Centro	1.00	3.04	3.076
		0.50	3.04	
		0.00	3.15	
	Cherchell	1.00	2.92	2.883
		0.50	2.92	
		0.00	2.81	
	Kobe	1.00	2.98	2.966
		0.50	2.98	
		0.00	2.94	

Table 8. Distribution of the overall ductility required μ for structure 3 as a function of height

Structure	Accelerograms	$\frac{H_i - H_1}{H_t - H_1}$	$\mu = \frac{D}{D_e}$	μ_{moy}
3	El Centro	1.00	2.94	2.92
		0.66	2.94	
		0.33	2.94	
		0.00	2.94	
	Cherchell	1.00	2.92	2.892
		0.66	2.92	
		0.33	2.92	
		0.00	2.81	
	Kobe	1.00	2.86	2.802
		0.66	2.86	
		0.33	2.86	
		0.00	2.63	

Table 9. Distribution of the overall ductility required μ for structure 4 as a function of height

Structure	Accelerograms	$\frac{H_i - H_1}{H_t - H_1}$	$\mu = \frac{D}{D_e}$	μ_{moy}
4	El Centro	1.00	2.98	2.972
		0.75	2.98	
		0.50	2.98	
		0.25	2.98	
		0.00	2.94	
	Cherchell	1.00	3.15	3.208
		0.75	3.15	
		0.50	3.15	
		0.25	3.15	
		0.00	3.44	
	Kobe	1.00	2.94	2.918
		0.75	2.94	
		0.50	2.94	
		0.25	2.94	
		0.00	2.83	

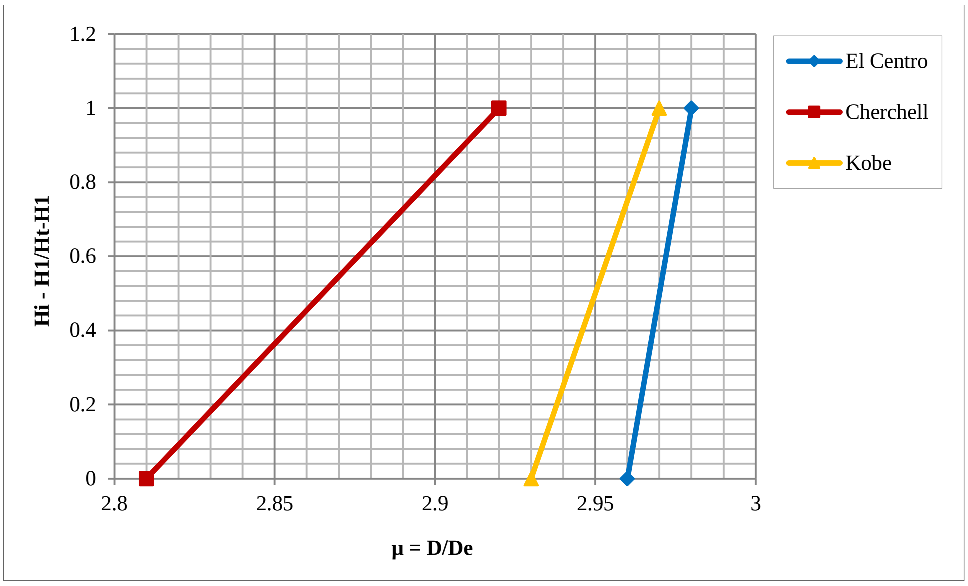

Figure 8. Distribution of the overall ductility of the structure 1

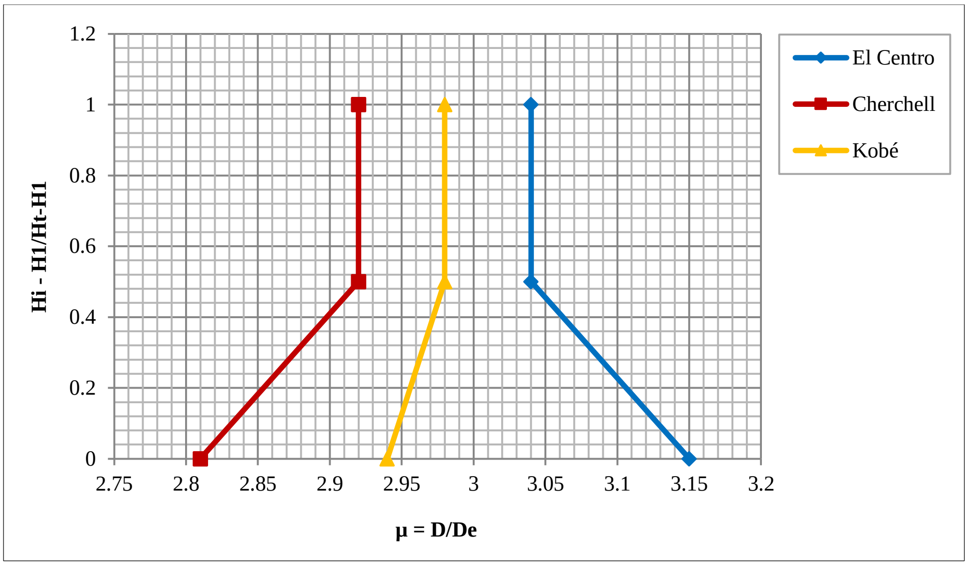

Figure 9. Distribution of the overall ductility of the structure 2

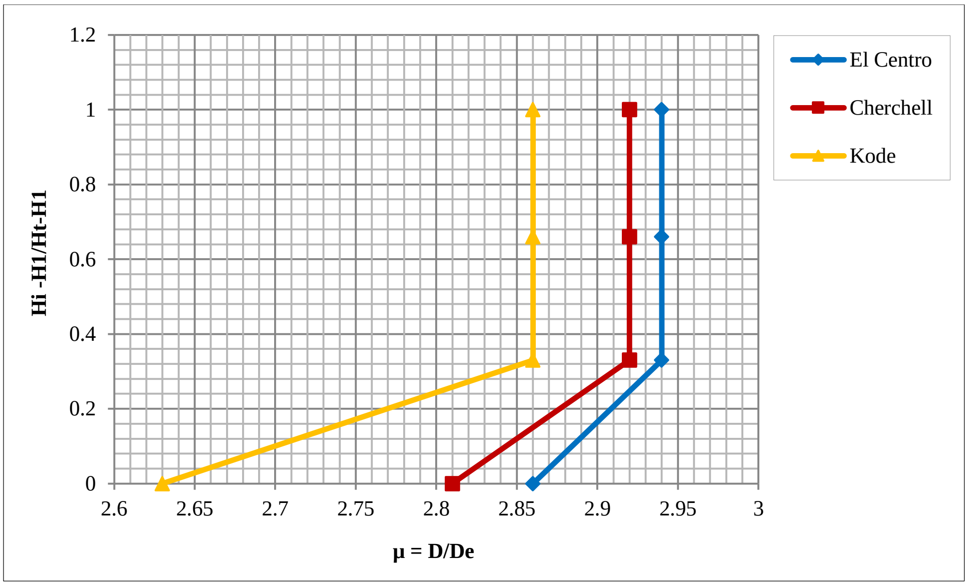

Figure 10. Distribution of the overall ductility of the structure 3

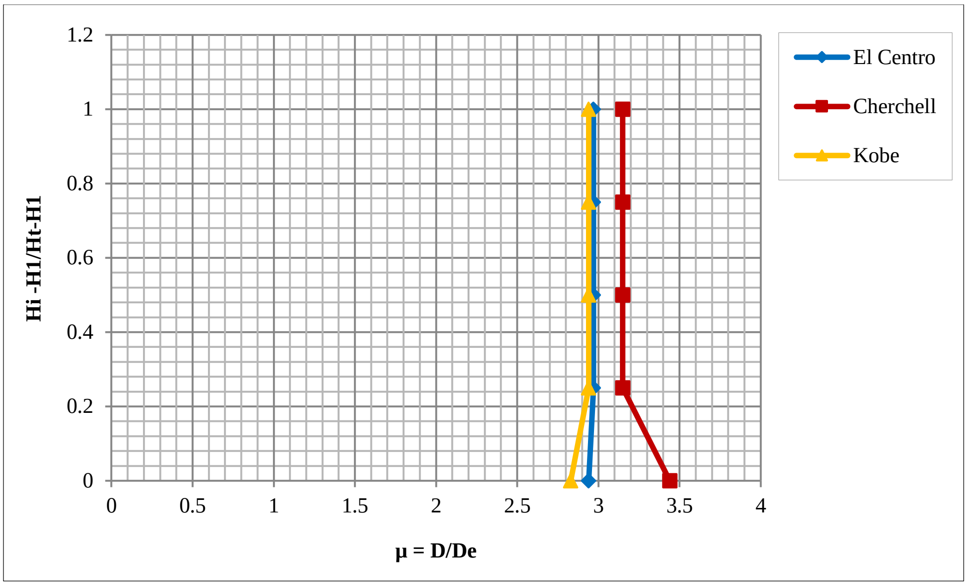

Figure 11. Distribution of the overall ductility of the structure 4

5. Interpretation of the Results

The reinforced masonry structures studied are made of products of small elements hollow with reinforcements embedded in the horizontal joints. The mortars used are of high strength, which ensures great adhesion between the elements of the reinforced masonry. The reinforcement was placed at each interface of the brick. Four structures at different levels were excited by three major accelerometers. The software used is the non-linear analysis program Drain2D, based on the finite element method, where the shear panel element was introduced. We have kept the same height dimensions for all the structures, and the same vertical and horizontal loads have been applied. Thus, we considered that the reinforced masonry consists of two elements: the reinforced mortar joints on the one hand and the hollow element on the other. The results of the study are summarized in Table 10:

Table 10. Average values of ductility and behavior factor for reinforced masonry

Accelerogram	μ	q	μ_{moy}	q_{moy}
El Centro	2.985	2.99		
Cherchell	2.98	2.98	2.967	2.968
Kobe	2.935	2.935		

The average value of the behavior factor is of the order of $q \approx 3.00$. This value is comparable to that given in the literature for reinforced concrete structures with flexural failure [5, 20, 35]. The accepted values for the overall behavior factor of structures through the main research established by the European Union are recorded in Eurocode 8 for reinforced masonry and are of the order of 2.5 to 3.

The value of the behavior factor found in the study is confirmed by tests on a vibrating table realized by Tomazevic et al.; the average value given in this experimental study is in the order of 3.74. The ductility value is of the order of $\mu \approx 3.00$. This value was confirmed by the three accelerograms and the different structures studied. The experimental study was carried out by Sandoval et al. [22], which the ductility result value ranges between 2.5 to 5.5.

The ductility is distributed uniformly, it can be concluded that these structures have good behavior in seismic zones. The failure of the structure studied occurred by exceeding the displacement between stages. There is no bending or shearing appearing during the excitation of the structures by the three accelerograms used: El Centro, Cherchell, and Kobe. Akbarzade & Tasnimiin [34] their study of the Behavior of Reinforced Masonry Walls showed that the consequent displacement of the panels proving a good ductility.

The mode of failure by shearing did not appear because the sections of reinforcement arranged in the mortar are important; they are placed at each interface of the bricks. Load-deformation response and failures of the masonry were affected by the following factors: reinforcement; the ratio of reinforcement steel bars played an important role in the behavior of reinforced masonry [22]. Sandoval et al. [19] found the increase in horizontal reinforcement ratio led to a large increase in shear capacity. Alcocer & Meli [17] found that the horizontal steel bars increase the shear capacity of brick walls by up to 30% compared with unreinforced walls. The behavior of reinforced masonry was quite similar to reinforced concrete structural elements [22, 40].

6. Conclusions

This work is based on a numerical study of the behavior of reinforced masonry structures (RM) under seismic loading and the relation between the behavior factor and the ductility. The behavior law and mechanical characteristics of reinforced masonry are defined, and the method for calculating the behavior factor and the ductility has been described. The study consists of finding a relative inter-stage displacement equal to 3.00 cm. This state corresponds to global ductility value at $\mu = 1$ or to a global behavior factor $q = 1$. This is achieved by multiplying accelerogram $a(t)$ by the coefficient λ, until finding an inter-stage displacement equal to 9.00cm. This state corresponds to the ruin of the structures.

As a result, the value of the behavior factor and the global ductility found for reinforced masonry structures at horizontal joints is of the order of 3.00. This value is the same for all the structures and is confirmed by the three used accelerograms. It is comparable to that given in the literature for reinforced concrete structures' bending failure. The breaking of all structures is reached by the displacement of the stages. There is no shearing of the structures or of flexion appearing during excitations by the accelerograms. The fracture of the studied structures by displacement between stages indicates ductile behavior, which allows the dissipation of the energy transmitted by the earthquake. The results found in the present numerical study confirm and complete the experimental studies carried out in other research works.

In this study, the reinforcement of the structure is placed at each brick interface. It is interesting to study the same structures by placing the reinforcements at 2, 3, or 4 different brick interfaces (at different heights, namely 40, 60, and 80 cm) and carry out a comparative study on the behavior of these structures in the face of seismic actions and analyze their modes of ruin at each change in the quantity of reinforcement. In this case, the shear failure mode may occur. Since the raw materials that constitute reinforced masonry, whether hollow concrete blocks or terracotta bricks that are found in abundance in nature, the manufacture of these materials remains an easy job and does not require sophisticated equipment. Since this type of construction does not require a lot of material resources or a large skilled workforce, this type of bracing remains advantageous in terms of completion times because the formwork and complicated reinforcements are avoided and the construction is done in a single part. In addition, the realization of reinforced masonry constructions remains economical because it avoids the maximum amount of steel, which is enormously expensive at the present time, and a maximum of types of concrete and expensive admixtures.

According to the results found in our study, which are confirmed by other research, the codes, and the advantages of reinforced masonry, it is interesting to introduce this type of construction into the Algerian seismic code and put it into practice. The use of reinforced masonry in construction remains interesting from the points of view of resistance, stability, and economy.

7. Declarations

7.1. Author Contributions

Conceptualization, O.H.S., M.O.M. and A.B.; methodology, O.H.S., software, O.H.S. and M.O.M., validation, O.H.S., A.B., H.A.A. and M.O.M.; formal analysis, O.H.S., A.B., H.A.A., and M.O.M.; investigation O.H.S.; writing—original draft preparation, O.H.S.; writing—review and editing, O.H.S, A.B., H.A.A. and M.O.M. All authors have read and agreed to the published version of the manuscript.

7.2. Data Availability Statement

The data presented in this study are available in the article.

7.3. Funding

The authors received no financial support for the research, authorship, and/or publication of this article.

7.4. Conflicts of Interest

The authors declare no conflict of interest.

8. References

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