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Research Paper

Seismic Retrofitting: Reinforced concrete shear wall versus CFRP reinforced concrete using pushover analysis

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Seismic retrofitting of constructions vulnerable to earthquakes is a current problem of great political and social relevance. During the last sixty years, moderate to severe earthquakes have occurred in Morocco (specifically in Agadir 1960 and Hoceima 2004). Such events have clearly shown the vulnerability of the building stock in particular and of the built environment in general. Hence, it is very much essential to retrofit the vulnerable building to cope up for the next damaging earthquake. In this paper, the focus will be on a comparative study between two techniques of seismic retrofitting, the first one is a reinforcement using carbon fiber reinforced polymer (CFRP) applied to RC elements by bonding , and the second one is a reinforcement with a shear wall. For this study, we will use a non-linear static analysis -also known as Pushover analysis - on a reinforced concrete structure consisting of beams and columns, and composed from eight storey with a gross area of 240 m², designed conforming to the Moroccan Seismic code [1].

1 Introduction

Among the reasons beyond the intensive research activity in this field, one finds the huge need for diagnosis and rehabilitation of pre-code constructions, particularly in the case of historic monuments [2].Other reasons are associated to the emergence of new design approaches, which are founded on the concept of performance-based engineering. Normally, loads on these structures are low and result in elastic structural behavior. However, under a strong seismic event, a structure may actually be subjected to forces beyond its elastic limit. Although building codes can provide reliable indication of actual performance of individual structural elements, it is out of their scope to describe the expected

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performance of a designed structure as a whole, under large forces. Nonlinear static analysis also known as pushover analysis is a possible method to calculate structural response under a strong seismic event.

The use of the nonlinear static analysis (pushover analysis) came in to practice in 1970 but the potential of the pushover analysis has been recognized for last two decades years. This procedure is mainly used to estimate the strength and drift capacity of existing structure and the seismic demand for this structure subjected to selected earthquake. This procedure can be used for checking the adequacy of new structural design as well. The effectiveness of pushover analysis and its computational simplicity brought this procedure in to several seismic guidelines [3, 4] and design codes [5, 6] in last few years.

Pushover analysis is defined as an analysis wherein a mathematical model directly incorporating the nonlinear load-deformation characteristics of individual components and elements of the building shall be subjected to monotonically increasing lateral loads representing inertia forces in an earthquake until a “target displacement” is exceeded. Target displacement is the maximum displacement (elastic plus inelastic) of the building at roof expected under selected earthquake ground motion. The structural Pushover analysis assesses performance by estimating the force and deformation capacity and seismic demand using a nonlinear static analysis algorithm. The seismic demand parameters are storey drifts, global displacement (at roof or any other reference point), storey forces, and component deformation and component forces. The analysis accounts for material inelasticity, geometric nonlinearity and the redistribution of internal forces.

The structures that designed for gravity loading are susceptible for damage during earthquake. During a severe earthquake, the structural members are expected to perform a ductile behavior. The structure is likely to undergo inelastic deformation and has to depend on ductility and energy absorption capacity to avoid collapse.

With this seismic forces, increase in loads and lack of regular maintenance, a need to reinstate, reinforce and upgrade existing concrete structures has been seen in the construction industry in recent years.

A variety of technical solutions has been implemented for seismic retrofitting, in this study we will interest at two types of reinforcement. The first technique is reinforcement by RC shear wall. During an earthquake, the shear walls play a major role in resisting lateral loads for concrete buildings [7]; they are efficient to dissipate the induced seismic energy, and they are designed to provide not only adequate strength, but also sufficient ductility to avoid brittle failure under strong lateral loads. Shear walls provide also stiffness to buildings, which significantly reduces lateral sway of the building and thereby reduces damage to structure and its contents. The use of shear wall structure has gained popularity in high-rise building structure, especially in the construction of service apartment or office/ commercial tower. It has been proven that this system provides efficient structural system for multi-storey building in the range of 30-35 storeys [8]. In the past 30 years of the record service history of tall building containing shear wall element, none has collapsed during strong winds and earthquakes [9].

The second technique of seismic retrofitting is reinforcement of RC element (columns and beams) using CFRP. In the last decade, composite materials such as the Carbon Fibers associated with Polymeric Matrices (CFRP) applied to RC elements by bonding are proved effective for the protection and reinforcement of beams and columns. S.A. Sheikh et al. [10] have investigated the seismic behavior of concrete columns confined with steel and FRP; they concluded that the use of FRP significantly enhances strength, ductility, and energy absorption capacity of columns. R.D. Lacobucci et al. [11] (2003) investigated the retrofit of square concrete columns with CFRP for seismic resistance; they found that added confinement with CFRP at critical locations enhanced ductility, energy dissipation capacity and strength of all substandard members. In spite of the extensive work on reinforced concrete columns, very few researchers have worked on reinforced concrete columns strengthened using FRP.

The present paper, investigates the efficiency of seismic retrofitting with a RC shear wall, against a reinforcement of RC members with FRP materials (CFRP), using a Pushover analysis to estimate seismic structural deformations.

2 Non Linear Analysis (Pushover analysis)

2.1 Definition

The nonlinear static procedure [12] has become a standard method in structural engineering practice for performance-based seismic evaluation of structures. In the NSP or pushover analysis, the structure is subjected to monotonically increasing lateral forces with an invariant height-wise distribution until a target displacement in reached. The seismic demands are computed at the target displacement and compared against acceptability criteria. These criteria depend on the material (concrete, steel, etc.), type of member (beam, column, panel zones, connections, etc.), importance of the member (primary or secondary), and the structural performance levels (immediate occupancy, life safety, or collapse prevention).

2.2 Procedure

Pushover analysis can be performed as either force-controlled or displacement controlled depending on the physical nature of the load and the behavior expected from the structure. Force-controlled option is useful when the load is known (such as gravity loading) and the structure is expected to be able to support the load. Displacement controlled procedure should be used when specified drifts are sought (such as in seismic 2, 1 loading), where the magnitude of the applied load is not known in advance, or where the structure can be expected to lose strength or become unstable.

Some computer programs (e.g. Seismostruct, Nonlinear version of SAP2000, ANSYS) can model nonlinear behavior and perform pushover analysis directly to obtain capacity curve for two and/or three dimensional models of the structure. When such programs are not available or the available computer programs could not perform pushover analysis directly (ETABS, RISA, SAP90), a series of sequential elastic analyses are performed and superimposed to determine a force displacement curve of the overall structure.

A displacement-controlled pushover analysis is basically composed of the following steps [13]:

1. A two or three dimensional model that represents the overall structural behavior is created.
2. Bilinear or tri-linear load-deformation diagrams of all important members that affect lateral response are defined.
3. Gravity loads composed of dead loads and a specified portion of live loads are applied to the structural model initially.
4. A pre -defined lateral load pattern which is distributed along the building height is then applied.
5. Lateral loads are increased until some member(s) yield under the combined effects of gravity and lateral loads.
6. Base shear and roof displacement are recorded at first yielding.
7. The structural model is modified to account for the reduced stiffness of yielded member(s).
8. Gravity loads are removed and a new lateral load increment is applied to the modified structural model such that additional member(s) yield. Note that a separate analysis with zero initial conditions is performed on modified structural model under each incremental lateral load. Thus, member forces at the end of an incremental lateral load analysis are obtained by adding the forces from the current analysis to the sum of those from the previous increments. In other words, the results of each incremental lateral load analysis are superimposed.
9. Similarly, the lateral load increment and the roof displacement increment are added to the corresponding previous total values to obtain the accumulated values of the base shear and the roof displacement.
10. Steps 7, 8 and 9 are repeated until the roof displacement reaches a certain level of deformation or the structure becomes unstable.
11. The roof displacement is plotted with the base shear to get the global capacity (pushover) curve of the structure (Figure 1).

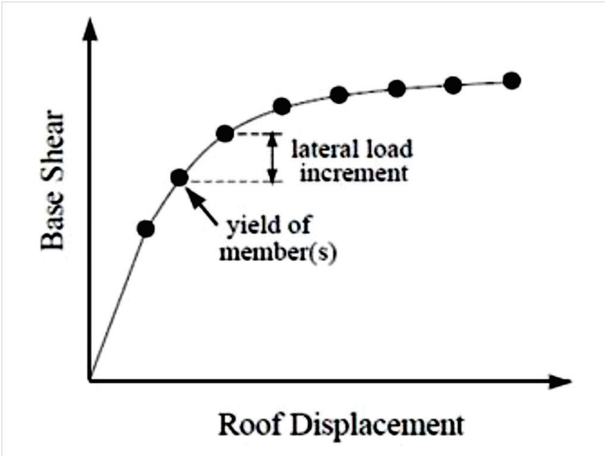


Fig. 1 –Global Capacity (Pushover) Curve of Structure [13]

2.3 Lateral Load Profile

The analysis results are sensitive to the selection of the control node and selection of lateral load pattern. In general case, the center of mass location at the roof of the building is considered as control node. The lateral load generally applied in both positive and negative directions in combination with gravity load (dead load and a portion of live load) to study the actual behavior. Different types of lateral load used in past decades are as follows:

2.3.1 "Uniform" Lateral Load Pattern

The lateral fore at any story is proportional to the mass at that story.

$$F_i = \frac{m_i}{\sum m_i} \tag{1}$$

Where:

F_i: Lateral force at i-th story

m_i: Mass of i-thstorey

2.3.2 "First Elastic Mode" Lateral Load Pattern

The lateral force at any story is proportional to the product of the amplitude of the elastic first mode and mass at that story.

$$F_i = \frac{m_i \phi_i}{\sum m_i \phi_i} \tag{2}$$

Where:

ϕ_i: Amplitude of the elastic first mode at i-thstorey

2.3.3 "Code" Lateral Load Pattern

The lateral load pattern is defined in Moroccan seismic Code [1] and the lateral force at any storey is calculated from the following formula:

$$F_n = (V - F_t)\frac{W_n h_n}{\sum_i^n W_i h_i} \tag{3}$$

$$\begin{cases} F_t = 0 & \text{si } T \leq 0.7\text{s} \\ F_t = 0.07TV & \text{si } T > 0.7s \end{cases}$$

Where:

F_n: Horizontal force applied to n-thstorey

V:Seismic Base-force

W_n: Total load of n-th floor

h_n: Height of n-thfloor measured from base

T:Fundamental period

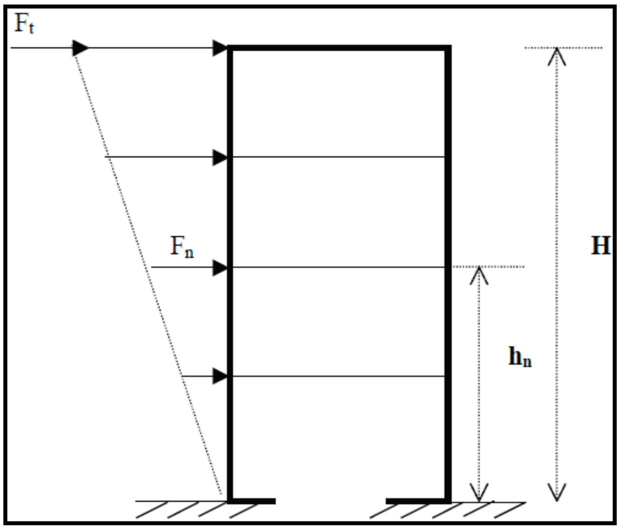


Fig. 2 – Vertical repartition of seismic forces [1]

2.3.4 "FEMA-273" Lateral Load Pattern

The lateral load pattern defined in FEMA-273 is given by the following formula that is used to calculate the internal force at any story:

$$F_i = \frac{m_i h_i^k}{\sum_i^n m_i h_i^k} \tag{4}$$

Where:

h_i: height of the i-th story above the base
K: a factor to account for the higher mode effects (k=1 for ≤0.5 sec and k=2 for >2.5 sec and varies linearly in between)

2.3.5 "Multi-Modal (or SRSS)" Lateral Load Pattern

The lateral load pattern considers the effects of elastic higher modes of vibration for long period and irregular structures and the lateral force at any story is calculated Square Root of Sum of Squares (SRSS) combinations of the load distributions obtained from the modal analysis of the structures as follows:

1) Calculate the lateral force at i-thstorey for n-th mode from equations

$$F_i = \Gamma_n m_i \phi_{in} A_n \tag{5}$$

Where:

Γ_n: Modal participation factor for the n-th mode

ϕ_{in}: Amplitude of n-th mode at i-thstorey

A_n: Pseudo-acceleration of the n-th mode Single Degree Of Freedom (SDOF) elastic system

2) Calculate the storey shears:

$$V_{in} = \sum_{j=1}^{N} F_{jn} \tag{6}$$

Where N is the total number of storeys

3) Combine the modal storey shears using SRSS rule:

$$V_i = \sqrt{\sum_n (V_{in})^2}$$

4) Back calculate the lateral storey forces, F_i, at storey levels from the combined storey shears, V_i starting from the top storey.
5) Normalize the lateral storey forces by base shear for convenience such that

$$F'_i = \frac{F_i}{\sum F_i} \tag{7}$$

The contribution of first three elastic modes of modal analysis was considered to calculate the 'Multi-Modal (or SRSS)' lateral load pattern in this study.

3 Law of behavior for FRP-confined concrete

The stress–strain behavior of FRP-confined concrete is largely dependent on the level of FRP confinement. When axial stress is low, the stress–strain response of FRP-confined concrete is similar to that of unconfined concrete. While the axial stress reaches the maximum strength of unconfined concrete, the lateral expansion of FRP-confined concrete increases drastically, accompanied with the growth of micro-cracks. If the confinement of FRP is weak, the stress will degrade beyond reaching the maximum strength until the rupture of FRP. If FRP confinement is strong enough, FRP is activated after reaching the maximum strength of unconfined concrete and applies a continuous increasing pressure on the concrete core until the rupture of FRP. A sharp softening and transition zone at the stress level of the maximum strength of unconfined concrete will be deformed, so the stress–strain curve of concrete confined with sufficient FRP displays a distinct bilinear response .Typical stress–strain responses of FRP-confined concrete are shown in Figure3.

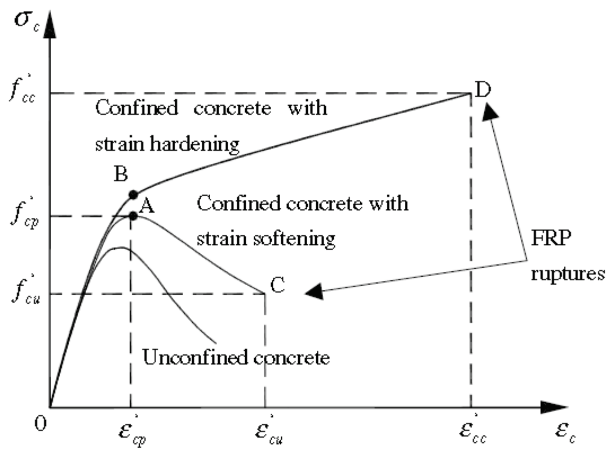


Fig. 3- Typical stress–strain responses for FRP-confined concrete [14].

In Figure3, points C and D are the ultimate status of FRP-confined concrete where FRP ruptures. Point A is the peak status of the FRP-confined concrete with a strain-softening component. Point B is within the sharp softening and transition zone, and named transition status for the stress–strain response with a strain-hardening component, but it is hard to accurately locate this point [14].

4 Description of the building

4.1 Geometry

The building is a reinforced concrete eight storey building with a gross area of 240 m². The building height is 27 m with 3m in each storey. The RC structure is composed from three bays with a 4 m in each one. Fig.3 shows the general geometric arrangement of the structure. The slabs thickness is 25 mm (20+5). The beams size is 25x25mm. For the columns, there is four types with size “40x40”, “35x35”, “30x30”, “25x25”.

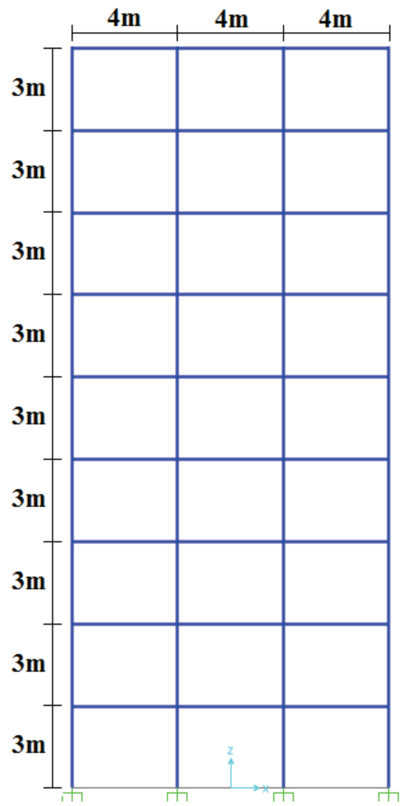


Fig. 4 –Dimensions of the building (Model realized on SAP2000)

4.2 Material properties

Table 1 - Concrete properties

Weight per unit volume (t/m^3)	Modulus of Elasticity E (MPa)	Poisson’s ratio U	Coefficient of thermal expansion A	Concrete compressive strength f’$_c$(MPA)
2.5	32164	0.2	$1.2.10^{-5}$	25

Table 2 - Steel properties

Weight per unit volume (t/m^3)	Modulus of Elasticity E (MPa)	Poisson’s ratio U	Coefficient of thermal expansion A	Concrete compressive strength f’$_c$(MPA)
7.85	$2.1.10^{5}$	0.29	$1.1.10^{-5}$	500

Table 3–Carbon Fiber properties

Weight per unit volume (t/m^3)	Modulus of Elasticity E (MPa)	Poisson's ratio U	Coefficient of thermal expansion A	Concrete compressive strength f'$_c$(MPA)
17.5	$4.5.10^5$	0.2	$0.02.10^{-5}$	2500

4.3 Seismic Data

Table 4 – Seismic data of the site

Poids total de la structure W (t)	Acceleration coefficient A	Soil factor S	Amplification factor D	Priority factor I	Behavior factor K
329.94	0.16	1.2	2.31	1	2

5 Result and Discussion

The lateral loads applied to the structure are calculated with Moroccan seismic Code (RPS2000). The following table resume the results of the lateral force applied to each floor.

Table 5 – Lateral loads result

Level	Base-force (V)	Height of each storey h_i (m)	Lateral force for each storey F_i (kN)
1		3	16.3
3	731.8	9	146.4
6		18	585.4
9		27	1317.2

The pushover analysis of the structure is performed using SAP2000, a structural calculator software that allow us to perform analysis and seismic design of buildings. The hinges placed on columns is defined to P-M3 type, for beams the hinges are all type M3.

5.1 Deformed shape of the structure and plastic hinge formation

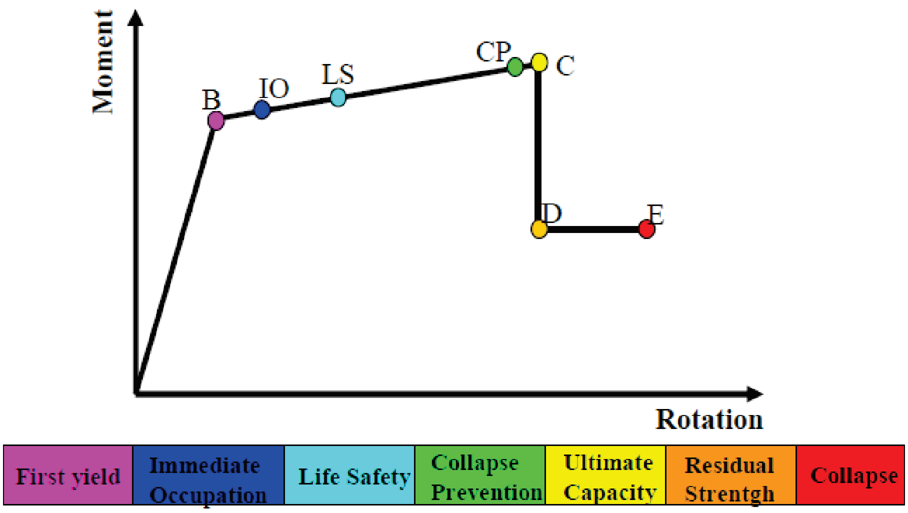


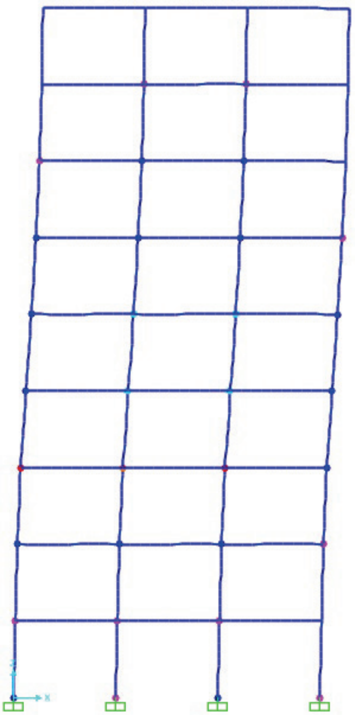
Fig. 5 –The plastic hinge behavior [15]

Point A corresponds to the unloaded condition. Load deformation relation shall be described by the linear response from A to an effective yield B. Then the stiffness reduces from point B to C. Point C has a resistance equal to the nominal strength then a sudden decrease in lateral load resistance to point D, the response at reduced resistance to E, final loss of resistance. The slope of the BC line is usually taken between 0 and 10% of the initial slope. The CD line corresponds to an initial failure of the member. The DE Line represents the residual strength of the member. These points are specified according to FEMA to determine hinge rotation behavior of RC members. The points between B and C represent acceptance criteria for the hinge, which is Immediate Occupancy (IO), LS (Life Safety), and CP (Collapse Prevention).

Table 6 – Performance level of building

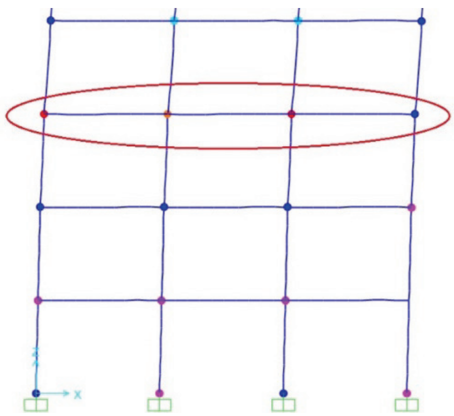
Level	Description
Operational	Very light damage, no permanent drift, structure retains original strength and stiffness, all systems are normal
Immediate Occupancy	Light damage, no permanent drift, structure retains original strength and stiffness, elevator can be restarted, Fire protection operable
Life Safety	Moderate damage, some permanent drift, some residual strength and stiffness left in all stories, damage to partition, building may be beyond economical repair
Collapse Prevention	Severe damage, large displacement, little residual stiffness and strength but loading bearing column and wall function, building is near collapse

The mode of failure of this type of structure under seismic excitations is plastics hinges formation. The figure below show the plastics hinges formation for the lateral load and the deformed shape of the three structures: unreinforced, reinforced with shear wall and finally reinforced with CFRP.



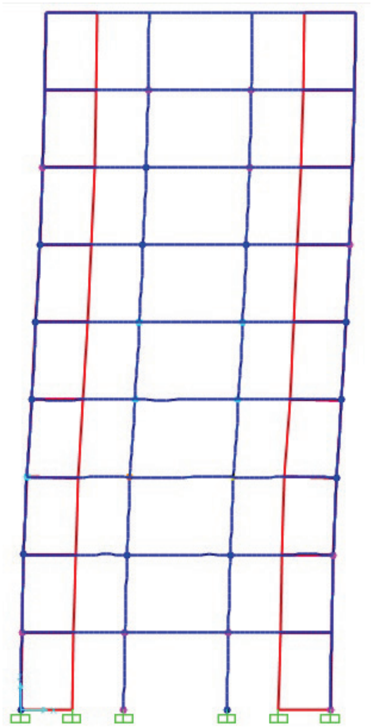
(a)

Fig. 6.a –Plastic hinge formation in the unreinforced structure



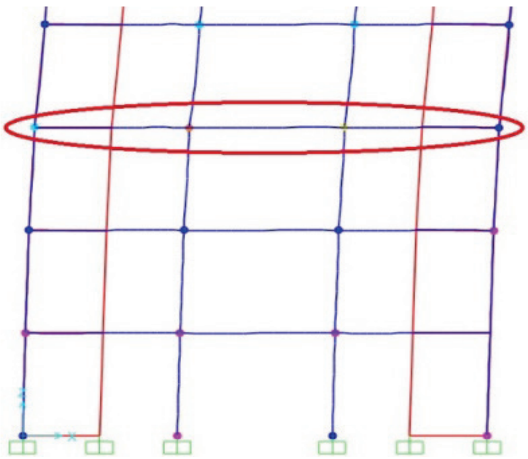
(b)

Fig. 6.b –Zoom on the lower part of unreinforced structure



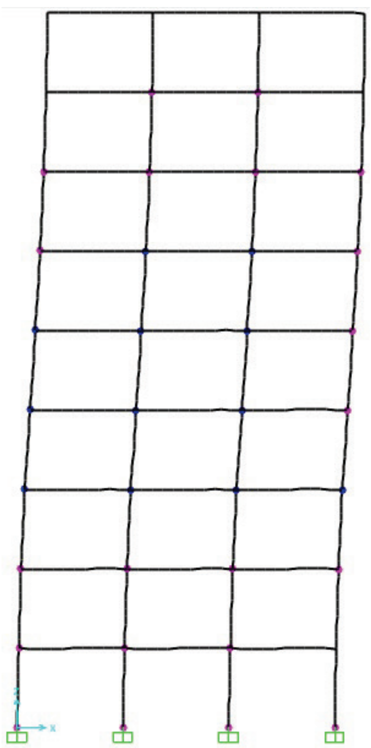
(a)

Fig. 7.a – Plastic hinge formation in the reinforced structure with shear wall



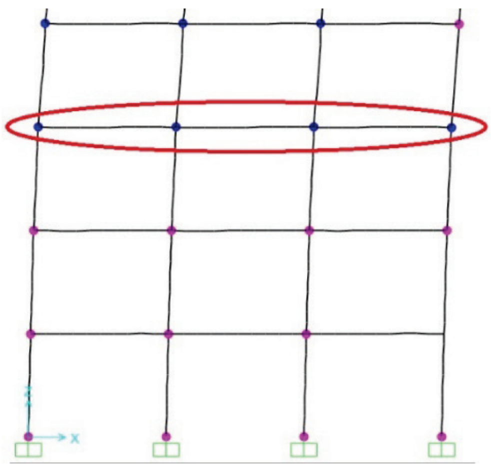
(b)

Fig. 7.b – Zoom on the lower part of the structure reinforced with shear wall



(a)

Fig. 8.a – Plastic hinge formation in the structure reinforced with CFRP



(b)

Fig. 8.b – Zoom on the lower part of the structure reinforced with CFRP

The following tables show the type of the hinges for each element in the three structures:

Table 7 –type of hinges for unreinforced structure

Step	A to B	B to IO	IO to LS	LS to CP	CP to C	C to D	D to E	Beyond E	Total
0	63	0	0	0	0	0	0	0	63
1	62	1	0	0	0	0	0	0	63
2	39	16	8	0	0	0	0	0	63
3	31	14	13	5	0	0	0	0	63
4	31	10	15	6	0	1	0	0	63
5	31	10	15	6	0	1	0	0	63
6	31	10	15	5	0	2	0	0	63
7	31	10	15	5	0	1	1	0	63
8	31	10	15	5	0	0	2	0	63
9	31	10	15	4	0	0	2	1	63
10	31	10	15	4	0	0	1	2	63
11	31	10	15	4	0	0	1	2	63

Table 8 – Formation hinges for reinforced structure with shear wall

Step	A to B	B to IO	IO to LS	LS to CP	CP to C	C to D	D to E	Beyond E	Total
0	63	0	0	0	0	0	0	0	63
1	62	1	0	0	0	0	0	0	63
2	39	16	8	0	0	0	0	0	63
3	31	12	15	5	0	0	0	0	63
4	31	11	14	6	0	1	0	0	63
5	31	11	14	5	0	2	0	0	63
6	31	11	14	5	0	1	1	0	63
7	31	11	14	5	0	1	1	0	63

Table 9 – Formation hinges for reinforced structure with CFRP

Step	A to B	B to IO	IO to LS	LS to CP	CP to C	C to D	D to E	Beyond E	Total
0	63	0	0	0	0	0	0	0	63
1	61	2	0	0	0	0	0	0	63
2	31	24	8	0	0	0	0	0	63
3	30	21	12	0	0	0	0	0	63
4	30	21	12	0	0	0	0	0	63
5	30	21	12	0	0	0	0	0	63

From the table7, at every step, the unreinforced structure loss in stiffness we can see in the last step, there is some element beyond the point E, which mean that our structure cannot resist to this lateral force, in other way, we have to reinforce it.

In table 8, we applied the first technique of reinforcement; it is a reinforcement using shear wall. The degree of damage is reduced compared with the first table, as shown in the second table we have improved the resistance of the structure, there is zero element beyond point E and just one element between D and E even. If it is the case, the structure may suffer from several damage.

As consequence, we have applied another technique of reinforcement. It is the one using the carbon fiber reinforced polymer (CFRP), and the result shown in the table 9 is very satisfying. We have all element under the Life Safety zone, which mean that our structure have an elastic behavior, in other way it can resist without any problem to the seismic force.

5.2 Pushover curves

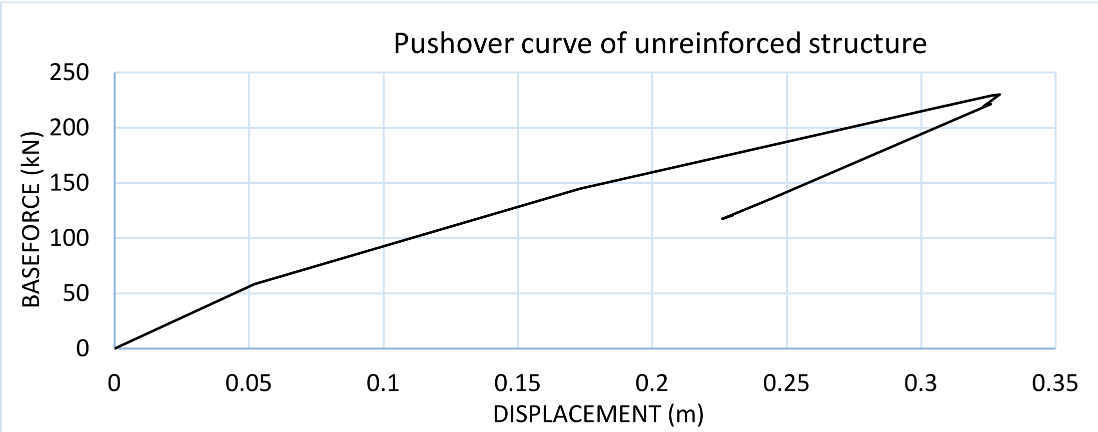


Fig. 9 – Pushover curve of unreinforced structure

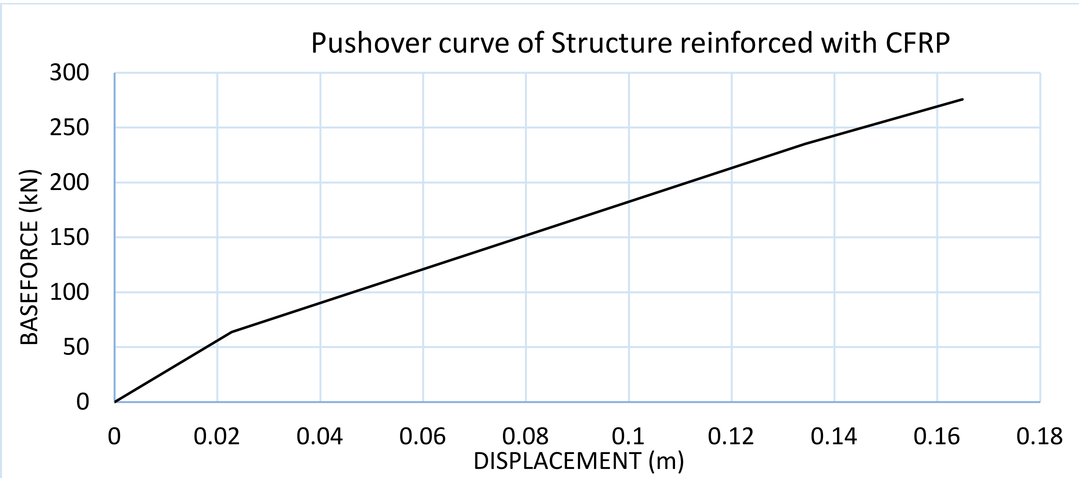


Fig. 10 – Pushover curve of reinforced structure with CFRP

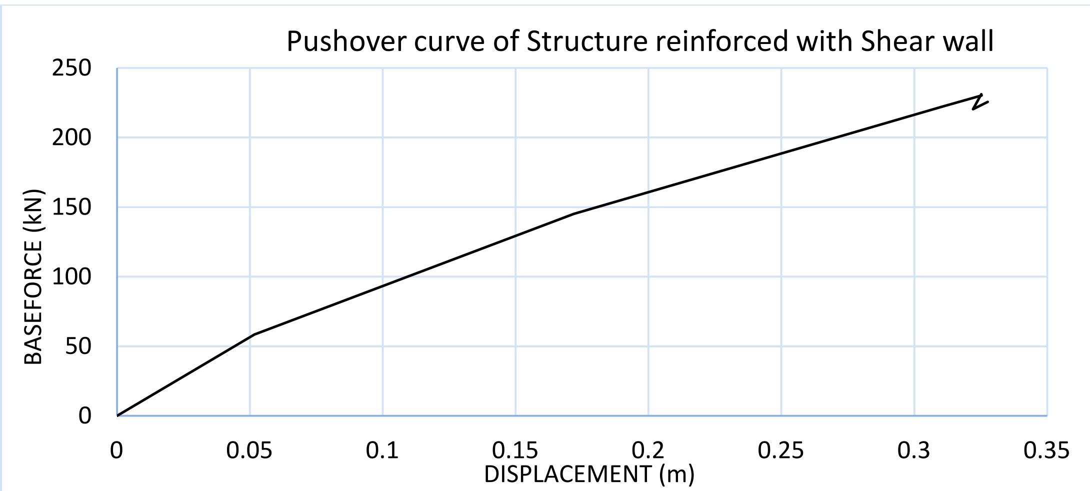


Fig. 11 – Pushover curve of reinforced structure with Shear wall

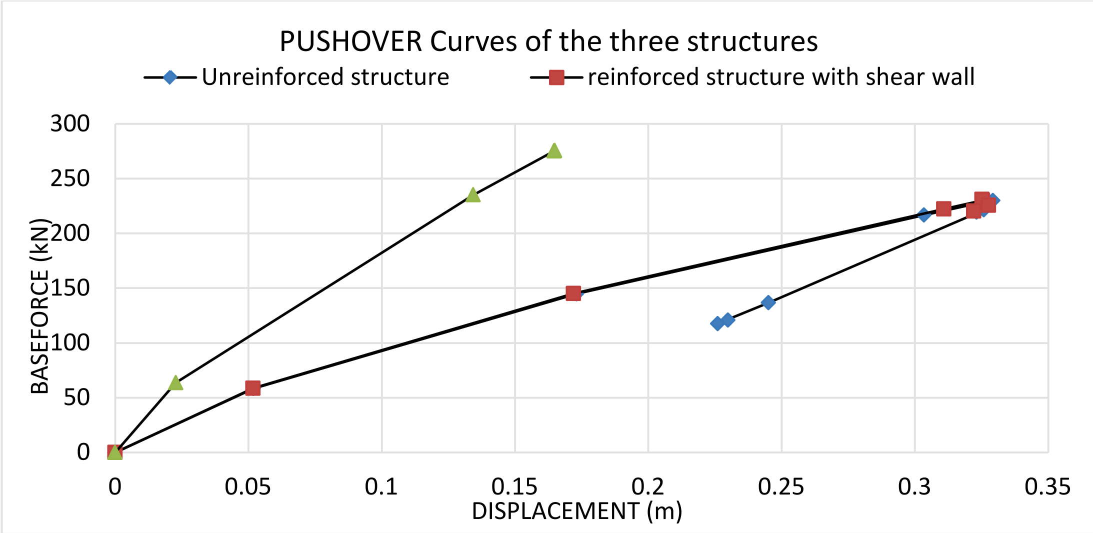


Fig. 12 – Pushover Curves of the three structures

Roof displacement versus base-force is present in the pushover curves for each structures on figures 9, 10 and 11. As mentioned before, lateral load patterns are calculated with Moroccan seismic Code (RPS2000).

In view of the results obtained in the analysis, the comparison of the pushover curves (Figure 12) shows that the reinforced structure with CFRP is stiffer than the reinforced structure with shear wall. We had conclude this by; Firstly the storey displacements obtained for the reinforced structures by CFRP is lower than the one obtained for the structure reinforced by shear wall, and secondly the ductility for structures reinforced by CRFP is higher than that for structures reinforced by shear wall.

The result of displacement and the base-force is shown in the follow tables:

Table 10 – Displacement and Base-force of unreinforced structure

Displacement	0	0,0518	0,1730	0,3033	0,3263	0,3264	0,3292	0,3231	0,3258	0,2450	0,2260	0,2296
Unreinforced structure	0	58,327	144,779	216,759	229,094	229,122	229,991	219,42	221,493	136,704	117,542	120,797

Table 11– Displacement and Base-force of reinforced structure with shear wall

Displacement	0	0,051746	0,171956	0,310843	0,325369	0,325174	0,322068	0,327612
reinforced structure with shear wall	0	58,59	145,114	222,328	230,173	231,046	220,416	225,693

Table 12– Displacement and Base-force of reinforced structure with CFRP

Displacement	0	0,0228	0,134342	0,164673	0,164719	0,164904
Reinforced structure with CFRP	0	63,702	235,288	275,324	275,428	275,683

The graph below gives the comparison between displacements for each structure:

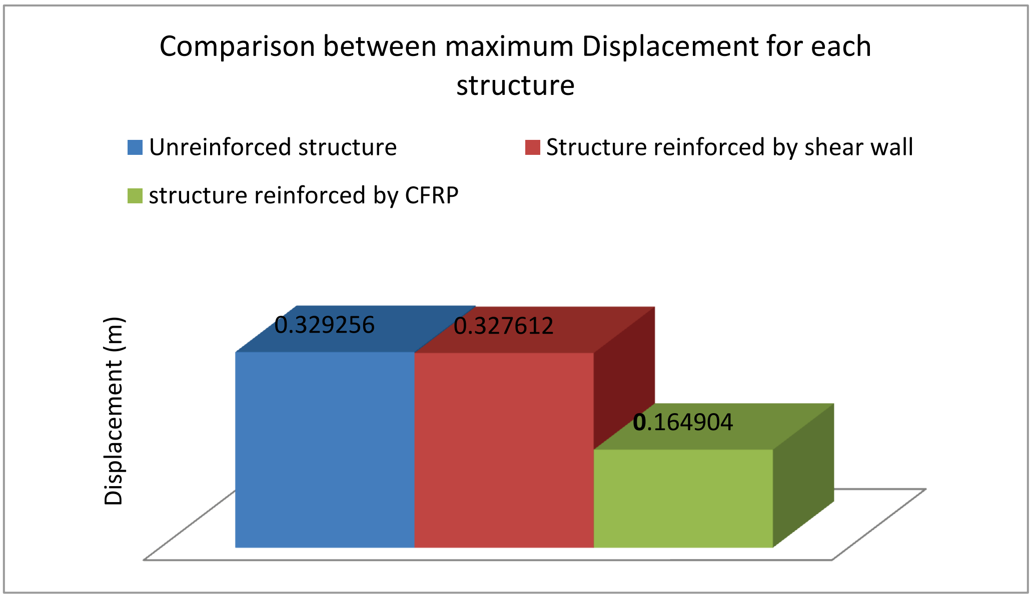


Fig. 13 – Comparative graph of the maximum displacement for the three structures

6 Conclusion

Based on this study, the more suitable technique for the retrofitting of is the one using the CFRP. In addition to the gain in ductility, capacity bearing and considerable decreasing in displacement provide by carbon fiber, this kind of fibers have excellent properties for structural members as high strength, high elastic modulus, high durability, and light weight. Unlike the reinforcement by shear wall, it increase considerably the weight of the structure and take a lot of time to realize it and it's very expansive compared with CFRP.

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