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CONSIDERATIONS ABOUT SEISMIC PERFORMANCES UPGRADING OF RC BUILDINGS USING CFRP WRAPPING.

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Abstract

Knowing that the rehabilitation practice in Algeria is not codified yet, the principal aim of this paper is to evaluate seismic performance of retrofitted reinforced concrete buildings by CFRP wrapping using a Pushover analysis to estimate the structural capacity in both strength and ductility. A nonlinear analysis is conducted on a series of low-rise buildings with poor concrete strength of their vertical components where the CFRP wrapping is adopted as a rehabilitation solution. An explanation of the simulation work before and after the retrofitting operation is given through the modeling approach of the nonlinear sectional behavior based on a combination of confining effects, helping in Moment-Curvature analysis and user defined plastic hinges modeling. Finally, seismic performance enhancement is evaluated in local and global behavior considering several structural criteria.

K DROUNA [1]
N DJEBBAR [2]

1 Civil Engineering Department, University Freres Mentouri Constantine, Algeria.

2 Civil Engineering Department, LMDC, University Freres Mentouri Constantine, Algeria.

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INTRODUCTION

The primary objective of this paper is to evaluate seismic performance of vulnerable low rise reinforced concrete buildings having poor concrete compressive strength for their well reinforced columns by using nonlinear static procedure. For that, a series of typical regular building has been considered. This work is conducted in order to situate how much such buildings are safe against earthquake attack. After evaluating the seismic capacity of buildings, the resulting damage is assessed. The obtained results showed that all the columns of the considerred buildings will lose their structural stability since their resistance capacity was found insufficient comparatively with what is required by the Algerian seismic regulation (RPA99 version 2003). Because of this, a retrofit solution is needed in order to enhance both resistance and ductility capacities. The use of jackets around existing deficient columns induces lateral confining stresses in the concrete as it expands laterally in the compression zone as a function of axial compression strains or in tension zone as a function of dilatation of lap splices under incipient splice failure. Since CFRP wrapping of vulnerable elements partially or completely present a preferable solution for seismic performances upgrading regarding its simplicity, rapidity of execution and immediate performances enhancement, this retrofitting technique is used. The needed clarifications related to the numerical modelling of the doubly confined concrete of the rehabilitated buildings are introduced. A particularity is addressed to the influence of the axial load intensity which can be satisfied despite the low compressive strength leading by the way to an under-estimation of the severity of the problem if the verification is focussed on the resistance capacity only. The importance of the axial load strength cannot be ignored while considering the local and global ductility since it has a more defining role in failure mechanism and energy dissipation under seismic action.

1. BACKGROUND FOR NONLINEAR ANALYSIS

In recent years, a breakthrough of simplified Nonlinear Static Procedures has been observed. These nonlinear static methodologies were initially proposed and verified for seismic assessment and retrofitting of buildings. Owing to their simplicity compared to the inelastic dynamic analysis, they were implemented into the modern guidelines and codes (ATC-40, FEMA 273, FEMA 356 and CEN, 2005). Well known as a simplified method for buildings seismic behaviour modelling, inelastic static analysis commonly referred to as the pushover analysis is widely adopted by researcher's community [1,2] where nonlinear behaviour of buildings is represented by plastic hinges assigned to critical sections.

The pushover analysis can be conducted using finite element software that take into account both geometrical and material nonlinearity, but the result's precision is directly influenced by the input data especially in post-elastic behaviour modeling. Sectional analysis must represent the most possible realistic state of the elements since shear and flexural behaviour of critical sections can change the global response of the building hence its performance. Shear effects are generally neglected and the nonlinear behaviour of buildings is represented only by moment hinges. Also assigned plastic hinges conditioning the nonlinear global response of the building needs more caution and precision. Differences of pushover analysis results due to default and user defined plastic hinges have been studied which results implement the use of user defined plastic hinges [3]. The important number of plastic hinges to generate varies depending on material properties, reinforcement and axial load level, based on Moment-Curvature analysis of each element. For simply reinforced concrete section, it can be directly issued from one of the finite element software depending on geometrical, mechanical properties and axial load intensity.

However strengthened sections (RC jacketing, FRP wrapping, steel casing,…) needs more focus because of the lack of experimental results or programs to get Moment-Curvature curve in a reinforced concrete element with another confining layer [4]. In this area, a modeling approach for the considered CFRP wrapped sections has been performed in two steps to get Moment-Curvature curves for these composite sections.

2. NONLINEAR ANALYSIS OF THE FRAMES

Plastic behavior of the elements was characterized using two nonlinear hinges at the ending member sections based on the lumped plasticity concept. A simplified bilinear moment-curvature curve was used for each plastic hinge. The nonlinear static (pushover) analysis was performed in order to estimate the seismic response of the structures. A finite element analysis program, SAP 2000 [5], commonly used by structural engineering professionals, was utilized to run the nonlinear static analysis.

2.1 Nonlinearity section analysis

The moment-curvature properties of the plastic hinges were determined using fiber analysis while considering section properties, reinforcement details and a constant axial load. Axial loads on the columns resulted from dead loads plus 20% of live loads as recommended by the Algerian code of practice for seismic resistant design of buildings [6]. However, beams are purely flexural elements. For the original frames the commonly used confined concrete model proposed by Mander *et al.* [7], was implemented while an elastic perfectly plastic model with parabolic strain hardening was considered for steel. For the steel confined concrete core values of f_{cc} , ε_{cc} and ε_{cu} according to Mander's model for simply confined RC sections. The ultimate strain of the confined concrete core $\varepsilon_{cu} = 0.004 + \frac{1.4\rho_{sh} f_{yh} \varepsilon_{su}}{f'_{cc}}$ is limited to ε_{cu}= 6%, f_{yh} = 400Mpa.

For CFRP wrapped sections, the superposition of the analytical effects of external and internal confinement judges reasonably the experimentally obtained response of doubly confined concrete [8]. The Moment-Curvature state of CFRP wrapped sections was obtained through the integration of the nonlinear (M-Ø) response of the individual fibers in which the section has been subdivided. The doubly confined section was analyzed in two steps:

First, considering the doubly confined effect of both internal transverse reinforcement and external CFRP wrapping for the hole section, material properties f_{cc} and ε_{cc} obtained according to below formula [9] for monotonic loading:

$$\frac{f_{cc}}{f_c} = 1 + 0.7\frac{\alpha_f \rho_f E_f \varepsilon_{uf}}{f_c} + \frac{\rho_w f_{yw}}{f_c} + 0.85\frac{\rho_{tot} f_y}{f_c} \qquad (1)$$

$$\varepsilon_{cc} = 0.0035 + x\left(\frac{10}{h(mm)}\right)^2 + 0.57\alpha_f . a_{eff.j} min\left[0.5,\ \frac{\rho_f f_{uf}}{f_{cc}}\right] \qquad (2)$$

Second: the effect of transverse reinforcement for the well reinforced initial section applies to the inside concrete core, the outside concrete cover is assumed to be crushed instantly with CFRP layers; because if the CFRP gives a light confinement and the steel ties a heavy one, the column may survive rupture of the CFRP and reach later an ultimate curvature controlled by the inside concrete core confined by ties [10, 11].

The Moment- Curvature curve for both simply and doubly confined sections has been idealized using the first yield and the ultimate state points corresponding to the performance levels as a fraction of the hinge plastic capacity [3, 12]:

- 10% for the immediate occupancy level corresponding to minor damage,
- 60% for the life safety level
- 90% for the collapse prevention level where non-reparable damage is envisaged [13].

Thus, defining user defined plastic hinges to introduce for the nonlinear analysis.

2.2 Plastic hinge length

Plastic hinges occur in RC columns when they are overloaded, particularly under an earthquake excitation. Performance-based design of RC columns also requires the knowledge of plastic hinge length for displacement calculation [14]. Plastic hinge failure can be mitigated by confining columns with a jacket. The length of the plastic hinge zone should be known because flexural jacketing needs to cover the confining region which is function of the plastic hinge length. For ordinary RC columns, the plastic hinge length depends on many factors:
1) axial load intensity; 2) moment gradient; 3) level of shear stress in the plastic hinge region;
4) mechanical properties of reinforcement; 5) concrete strength; 6) level of confinement in the potential hinge region [14].

Many formulas based on analytical or experimental investigations under monotonic or cyclic loading have been proposed. In this context, Park and Priestley [7] suggested the following expression for cantilever columns subjected to an axial load and a lateral force at the top:

$$L_p = 0.08L + 6d_b \qquad (3)$$

Paulay and Priestley [15] ameliorated the above equation to account for different grades of flexural reinforcement:

$$L_p = 0.08L + 0.022d_b f_y \qquad (4)$$

f_y: yield strength of longitudinal reinforcement.
d_b: is the diameter of longitudinal reinforcement.

Many researchers provided other formula [16, 17] but equation (4) is still the most used regarding its simplicity and good accuracy to experimental results. In this study for the calculation of plastic hinges length for initial structures equation (4) has been adopted. Plastic hinge length for CFRP wrapped members has not been widely mathematically expressed; Gu et al [18] provided the following expression:

$$L_p = L_{p0} + L_{pc} \qquad (5)$$

$$L_p = 0.08\,L + 0.022 f_y\, d_b + \begin{cases} 3.28\lambda_f & when\ 0 \le \lambda_f \le 0.1 \\ (0.51 - 2.30\lambda_f + 2.28{\lambda_f}^2)L & when\ 0.1 < \lambda_f < 0.5 \end{cases} \qquad (6)$$

The performance of their model using the previous equation compared to the test results collected from the literature presents acceptable approximation to test results [14]. For that, it is adopted for the calculation of the plastic hinge length for CFRP wrapped columns in this study.

3. VERIFICATION OF THE MODELLING ASSUMPTIONS

Before implementing user defined hinges in the push over analysis for the selected structures of the case study, a checking phase is envisaged. Since the effect of the sectional analyses is a primordial step within nonlinear analysis. The obtained modeled plastic hinges are implemented in the pushover using SAP2000 [5] for tested retrofitted or non-retrofitted frames.

These frames consisted of one floor and one span presenting poor concrete strength and low reinforcement ratio, subjected to a pseudo static test before and after CFRP wrapping [4]. The obtained numerical results were compared to test results show acceptable accuracy when simulating the nonlinear static response of RC frames retrofitted for both considered cases. It is to note that frame 2 was retrofitted with partially CFRP wrapping of the column (700mm) in both extremities.

A reduction factor r was assigned to elements stiffness to consider the antecedent of deformation caused by pushing frame 2 to a displacement of 1% of the total height before applying the CFRP wrapping.

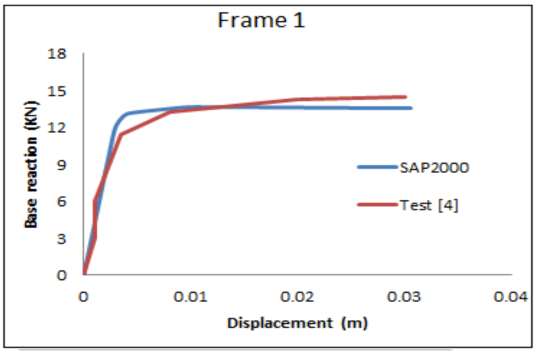


Figure1. Initial frame: simulations/test results

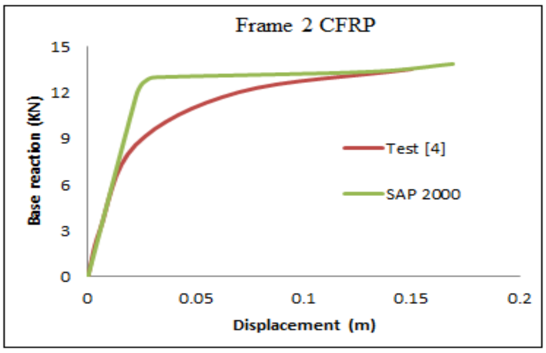


Figure2. Strengthened frame: simulation /test results

- The obtained results show that NL analysis RC frames can give good approximation to test results if more caution and precision are given to different stages of the simulation process especially for sectional analysis, plastic hinges definition and the consideration of cracked stiffness.
- The user defined plastic hinges of CFRP wrapped RC elements obtained by an equivalent Moment-Curvature curve based on two steps sectional analysis gives the possibility to the pushover analysis to approximate reasonably test results.

The second part of the study consists on a numerical investigation conducted on the efficiency of carbon fibre reinforced polymers (CFRP) in improving the seismic performance of RC frames. The evaluation of seismic performance is established through structural criteria to a series of vulnerable low rise buildings before and after the strengthening with full CFRP wrapping.

4. CASE STUDY

A series of typical vulnerable low rise buildings constructed after 1999 with poor concrete compressive strength for their vertical components ($f_{c0} = 15MPa$), against well reinforcement according to RPA 2003[6] with steel reinforcement FeE 400 type; presents a particular casual case in construction site of buildings in Algeria. Inelastic static pushover analysis is used for the post-elastic response simulation.

It should be noted that the obtained results while conducting an early investigation [19], aiming to compare inelastic pushover analysis to inelastic dynamic analysis show that the triangular load shape is adequate to predict the global response of low rise frames and almost an identical response is observed between the dynamic analysis best-fit envelopes and the static response obtained from the triangular and multimodal distributions.

The initial structures are modeled and analyzed by using Sap2000 nonlinear analysis program. A first estimation of seismic performances is conducted through static pushover analysis since the first structural mode of vibration in dominating. A re-evaluation of the performances using the same technical analysis of the retrofitted considered structures by CFRP strengthening of all columns, using three layers of CFRP SIKA wrap 230C/45 product unidirectional wrap with: E= 34Gpa, tensile strength 450 Mpa, thickness (per ply)1mm and ultimate strain 14‰ is also presented and discussed.

Structures	**Properties**										
	Height (m)	Width (m)	Beam (m)	Column (m)	f_{c0} (MPa)	f_{cc} (MPa)	$f_{cc,CFRP}$ (MPa)	ε_{c0}	ε_{cU}	ε_{cCFRP}	ε_s
3stories	09.18	12	0.3x0.35	0.3 x0.3 8Ø14	15	19.64	39.95	2 ‰	3.5‰	14‰	0.5 ε_{su}
4stories	12.24	12	0.3x0.35	0.3 x0.3 8Ø14	15	19.64	39.95	2 ‰	3.5‰	14‰	0.5 ε_{su}
5stories	15.3	12	0.3x0.35	0.3 x0.3 8Ø14	15	19.64	39.95	2 ‰	3.5‰	14‰	0.5 ε_{su}

Table1. Geometrical, mechanical properties of materials for studied structures

Cracked stiffness was assigned to structural elements according to Paulay- Priestley [16], columns and beams are modeled as nonlinear frame elements with lumped plasticity regions in both extremities and P-M-M hinges were assigned to columns and flexure hinges to beams. The beam-column joints were assumed to be rigid [20].

4.1 Sectional analyses:

Results of sectional analyses before and after CFRP strengthening are presented in figure 3.

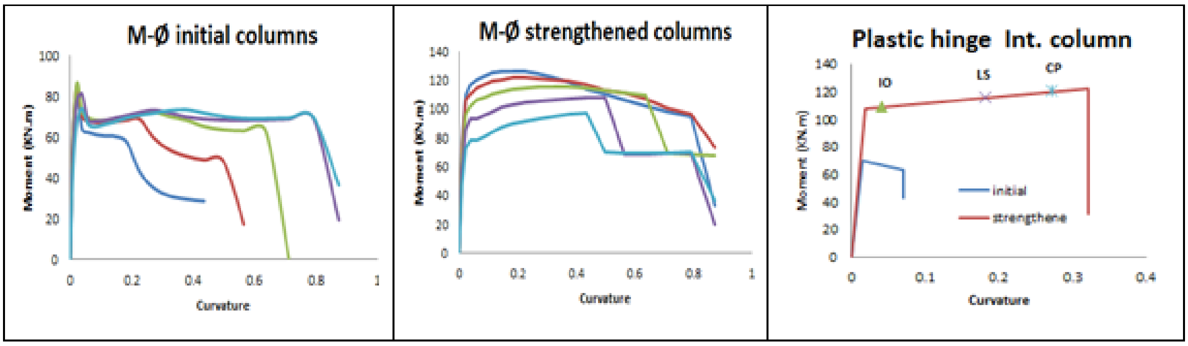


Figure 3. Moment-curvature curves and user defined plastic hinge presentation for internal column.

A moment curvature analyse is carried out considering section properties and constant axial loads. The obtained moment values for different building column sections are gathered in table 2. We can notice that the resistance criterion is not satisfied since the column's resistance capacity is less than its resistance demand leading to a structural collapse. It is evident that the considered building columns must be retrofitted.

The use of CFRP allowed the enhancement of the sectional state for the initial columns not responding to seismic requirements, from fragile (ν > 0.3) ,characterized by a brittle failure, to a highly ductile state by increasing the compressive resistance capacity, thus lightening the reduced axial load ν and increasing the ultimate concrete strain capacity ε_{cu}.

Structure	Reduced axial load $P/A_g * f_c$			Moment (KNm)						Curvature ductility $\mu_\varphi = \dfrac{\varphi_u}{\varphi_y}$		
				M_Y			M_u					
	ν Initial (1)	ν Wrap (2)	**Gain**	Initial (1)	Wrap (2)	**Gain** (2)/(1)	Initial (1)	Wrap (2)	**Gain** (2)/(1)	Initial (1)	Wrap (2)	**Gain** (2)/(1)
3 Stories	0.43	0.16	**2.69**	83.10	87.73	**1.06**	69.75	110.49	**1.59**	2.5	15.40	**6.16**
4 Stories	0.57	0.21	**2.71**	75.90	103.09	**1.36**	66.75	112.84	**1.62**	2.26	18.04	**7.98**
5 Stories	0.70	0.26	**2.69**	71.78	107.96	**1.50**	63.86	113.32	**1.74**	2.5	17.21	**6.88**

Table 2: Sectional analyses results.

The increase of columns carrying capacity enhances significantly the local ductility μ_φ, as presented in table 2, which has a positive influence on the overall behavior of the structures.

4.2 Interaction curves:

The examination of the P-M curve interaction for internal column (fig.4), illustrates the enhancement of axial load and moment capacity for the strengthened elements. This is due to the use of unidirectional CFRP tissue that increased the capacity of radial deformation and the flexure strength of the section without influencing the initial stiffness [21].

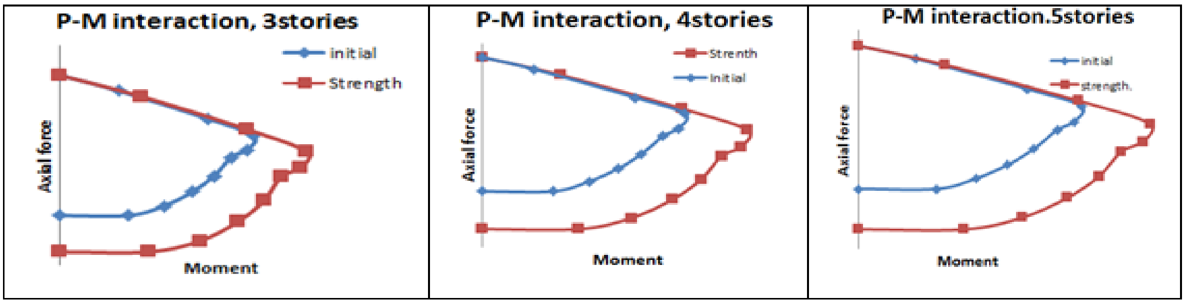


Figure 4. P-M interaction curves before and after intervention

4.3 Structural behavior- Pushover analysis results

a- Capacity curves:

Three different damage levels [6] are incorporated within the idealized user defined plastic hinge:

- Minimum damage limits (MN): $\varepsilon_c = 0.0035$ $\varepsilon_s = 0.01$
- Safety limits (SL): $\varepsilon_c \leq 0.0135$ $\varepsilon_s = 0.04$
- Failure limits (FL) $\varepsilon_{cu} \leq 0.0180$ $\varepsilon_s = 0.06$

Relating base shear to roof displacement is presented in figure 5. The CFRP wrapping confers a valuable improvement on the global behaviour through deformation and strenght capacities; thus, satisfying seismic performances criteria. The initial stiffness is conserved so the original conception and ductility class remains the same while a great benefit in displacement capacity is registered.

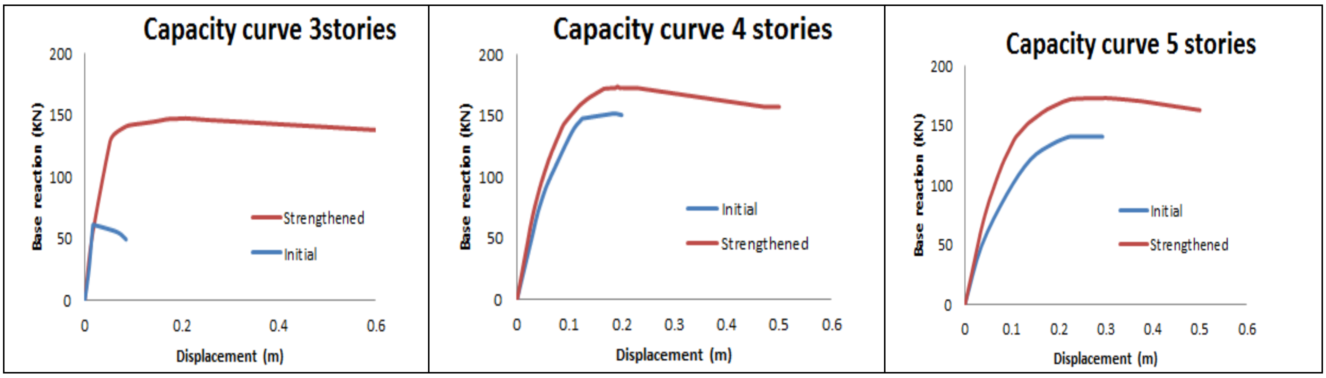


Figure5. Pushover curves for initial and strengthened structures.

b- Roof displacement:

Structure	Δ.max			Δ. 1st LS hinge			Δ. 1st CP hinge		
	before	after	Gain	before	after	Gain	before	after	Gain
3 Stories	0.005	0.20	/	0.0022	0.13	/	0.00225	0.16	/
4 Stories	0.08	0.217	**2.71**	0.04	0.15	**3.75**	0.05	0.175	**3.5**
5 Stories	0.10	0.242	**2.24**	0.05	0.189	**3.78**	0.065	0.219	**3.37**

Table3. Registered displacement in different stages of energy dissipation

For all cases, the displacement of 1% building height related to life safety performance level [6] supposed to be satisfied by the initial conception was not achieved. Failure mechanisms appeared before that consideration is attained. The CFRP strengthening permits to satisfy largely the previous condition while delaying the appearance of the collapse prevention level. (for the 5 stories building the first CP hinge appears after major displacement corresponding to 1.5% drift of building height).

C - Resistance capacity:

Base shear demand is calculated according to RPA 99, version 2003 (eq. 10). The base shear capacity is issued directly from the pushover analysis. The obtained results are gathered within table 4.

$$V = \frac{A.D.Q}{R} * W \qquad (10)$$

A: zone acceleration coefficient, D: dynamic amplification factor, Q: factor related to materials quality and execution, R: behavior factor and W: total weight of the building. The initial structures present a base shear capacity lower than what is required by the code. The adopted strengthening solution has enhanced the global resistance capacity.

Structure	Initial structures			Strengthened structures		
	Vdemand	Vcapacity	Remarque	Vdemand	Vcapacity	Remarque
3 Stories	110.97	60.876	**C < D**	110.97	146.96	**C>D**
4 Stories	150.03	140.80	**C < D**	150.03	172.29	**C>D**
5 Stories	179.17	146.35	**C < D**	179.17	188.86	**C>D**

Table 4. Base shear demand /capacity for studied structures

d- Failure mechanisms:

1. Initial structures

The registered failure mechanism for initial structures is qualified by the philosophy of seismic design as the most dangerous. It is characterized by concentration of plastic hinges in the columns of the first story with migration of hinges to the top of columns. These later attain the CP performance level (in yellow) for all studied cases before beams start to dissipate energy. This is a clear illustration of strong beam- weak column state. This severity is much more pronounced when the 3 story structure is considered where the collapse is straightforward (plastic hinges in red). Its great rigidity (squat structure) does not enable it to be dissipative structure energy.

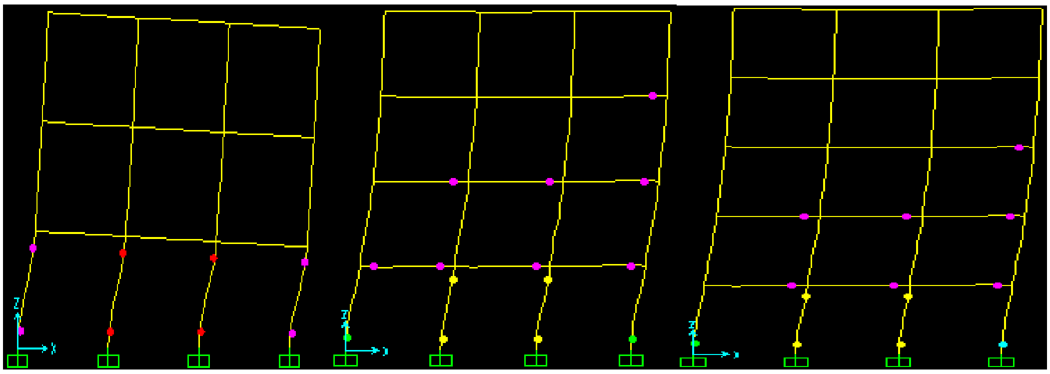


Figure6. Sequence of plastic hinge apparition before intervention.

2. Strengthened structures

For CFRP strengthened structures, general improvement in the global behavior where a participation of more members in the energy dissipation process is noticeable. The life safety performance level related to the Algerian seismic code (characterized by a 1% drift displacement) is achieved without any plastic hinge in columns. Pushing to collapse, the participation of upper floors beams in the dissipating process is remarkable (immediate occupancy level in blue and life safety in light blue). The failure mechanism is reached by appearance of CP hinges in both extremities of beams while some hinges at the top columns, exceed slightly the yielding limit (hinges in pink).

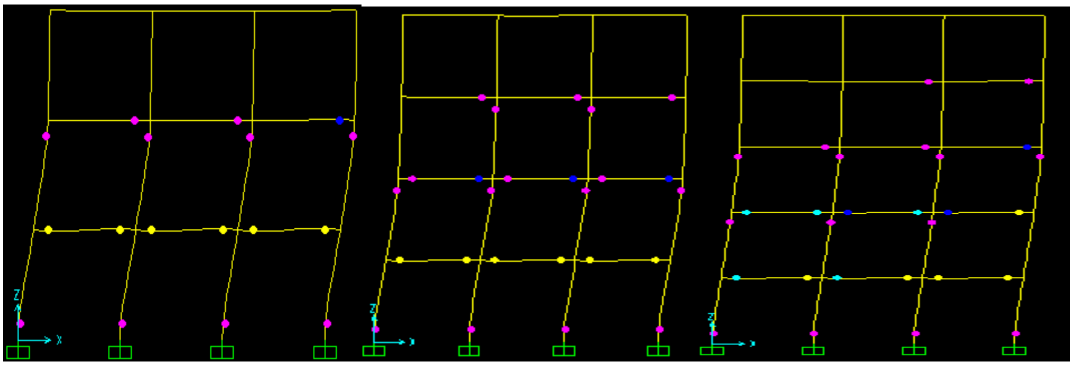
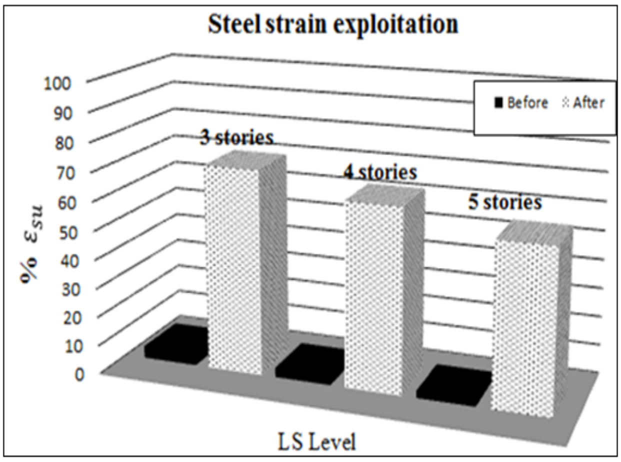


Figure7. Sequence of plastic hinge apparition for strengthened structures

4.4- Reinforcement exploitation:

	Steel strain. before			Steel strain. after		
Performance level	LS	CP	Observation	LS	CP	Observation
3 stories	2.13E-03	8.26E-03	$<\varepsilon_{sy} = 10‰$	0.0355	0.0424	$\varepsilon_{sy}<\varepsilon_s<\varepsilon_{su}$
4 stories	1.67E-03	6.71E-03	$<\varepsilon_{sy} = 10‰$	0.0317	0.0443	$\varepsilon_{sy}<\varepsilon_s<\varepsilon_{su}$
5 stories	1.25E-03	4.99E-03	$<\varepsilon_{sy} = 10‰$	0.028	0.0434	$\varepsilon_{sy}<\varepsilon_s<\varepsilon_{su}$

Table5. Steel strain exploitation.



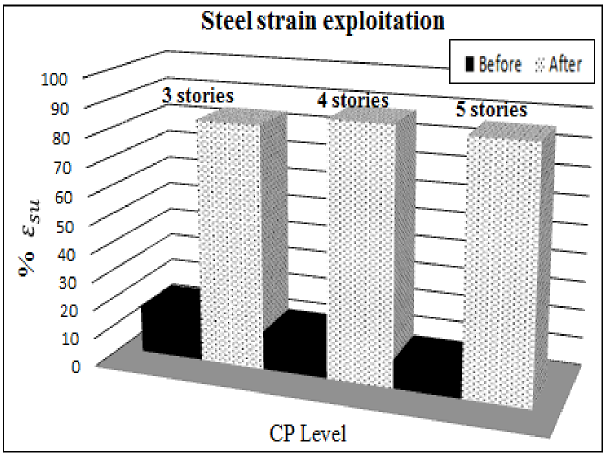


Figure8. Steel strain exploitation.

The poor concrete compressive strength for initial structures limits the exploitation of reinforcement strain capacity. The steel yielding value was not attained because of the fragile behavior of the elements. That's why the selection of CFRP jackets is desirable in such situation where the objective is to increase the elements flexural strength while maintaining the initial stiffness and permitting a full exploitation of the strain capacity of longitudinal reinforcement [22].

5- CONCLUSION:

This study confirms the possibility of nonlinear analysis to simulate reasonably structural behavior under seismic loading if more caution is afforded to sectional analyses in post-elastic stage and the attribution of particular user defined plastic hinges for each structural component. A well reinforced column against poor concrete compressive strength doesn't reduce the severity of the problem of brittle failure and the dangerous failure mechanism under seismic excitation.

In fact, it limited the exploitation of the deformation capacity of longitudinal reinforcement. This can be solved using external CFRP wrapping which conserves initial stiffness by limiting concrete cracking and enhancing its strength, leading to a better exploitation of steel strain. The proposed modeling approach of CFRP wrapped RC sections gives acceptable estimation of the sectional capacity thus; acceptable plastic hinges can be defined based on it. In seismic rehabilitation, if the objective is maintaining or marginally enhancing the flexure strength without changing the initial stiffness, CFRP wrapping is the more accommodate solution particularly for low-rise buildings braced by beam-column system.

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