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Effective Performance based Seismic Retrofit of Pre-code Reinforced Concrete Buildings

[1]Ahmed El Haouzi, [1]Abdellatif Khamlichi, [1]Mohammed Bezzazi, [2]Francesc Lopez Almansa

[1]Modelling and simulation of Mechanical Systems Laboratory, Faculty of Sciences at Tetouan 93002, BP. 2121, M'hannech, Tetouan, Morocco
[2]Technical University of Catalonia, Architecture Structures Dept., Av. Diagonal 649, 08028 Barcelona, Spain

Abstract: Seismic retrofit of reinforced concrete pre-code buildings is considered in this work. Nonlinear static pushover analysis is performed by means of specialized finite element software to obtain building response under the code prescribed seismic design load. Performance states that were considered include limitations of the total building roof displacement and of the maximum inter story drift. Three factors characterizing material, geometric and reinforcement of structural members were introduced. A full factorial design of experiment table had been used to define a finite set of data points where the base shears associated to drift displacements thresholds were evaluated. Response surface based models were derived then to write explicit expressions for the constraints related to the performance limit states. Optimization of seismic retrofit was finally carried out with the objective to minimize the total cost of retrofit operation.

Key words: Seism, Retrofit, Reinforced concrete, Pushover, Regression, Analysis of variance

INTRODUCTION

Since pre-code buildings have been engineered without taking into account seismic code regulations, they are seismic vulnerable (Buratti, 2010; Kaveh, 2010; Quanwang, 2006; Mehmet and Hayri, 2006; Hasan, 2002; Applied Technology Council, ATC- 40, 1996). Diagnostic of existing structures with insufficient seismic resistance is of critical importance in order to carry out appropriate retrofitting to mitigate seismic risk in a cost effective fashion. Seismic retrofitting can be performed through several methods in order to increase the structure capacity. These include various objectives (FEMA, 2000) such as increasing rigidity, strength, energy dissipation, isolation, regularity, etc…

Conventional as well as emerging retrofit methods are available. Conventional retrofitting methods include column and beam jacketing, or addition of new structural elements to the system and enlarging the existing members. Structural members that are likely to be overstressed under the action of seismic event are strengthened through concrete or steel jacketing. Emerging retrofit techniques include fibre reinforced polymer wrap bonded on the structural members.

The main research need associated with conventional strengthening methods is optimization of the retrofit design to achieve a satisfactory structural performance level at a minimum cost based on reliably characterized seismic demand and structural capacity. Thus, it is important to assess building vulnerability and to know how it could change by modifying key material and geometric factors. Rossetto et al. (Rossetto, and Elnashai, 2003) have reviewed various vulnerability approaches. Krawinkler et al. (Krawinkler and Seneviratna, 1998) have discussed the possibility to use pushover analysis in order to assess analytically building vulnerability. They have concluded that pushover analysis provides adequate information on seismic demands if the building structure is regular.

Optimization of seismic retrofit by means of column and beam jacketing is considered in the following. A simplified methodology is proposed for this purpose. It is based on pushover analysis of the reinforced concrete building performed according to a full factorial design of experiment (DOE) table. This last was constructed by choosing three key building design variables and fixing three levels for each of them. Post-

Corresponding Author: Ahmed El Haouzi, Abdellatif Khamlichi, Mohammed Bezzazi Modelling, simulation of Mechanical Systems Laboratory, Faculty of Sciences at Tetouan 93002, BP. 2121, M'hannech, Tetouan, Morocco
E mail haouzi1982@hotmail.com

analysis of variance conducted on the obtained results enabled then to assess the relative influence of each factor. Polynomial regression was performed after that on the obtained results to derive response surface metamodels which give explicit approximations of the considered limit states as function of the design variables. These regressions are used to write the constraints associated to the required seismic performance levels through the retrofitting operation. Considering the objective function the cost of structural members to be minimized, a nonlinear mathematical programming is written and solved to find the optimal design.

The performance states to be attained include two limit states as defined by the Moroccan seismic code RPS2000 (Royaume du Maroc, 2001). Use is made of Zeus NonLinear (ZeusNL) software package (Elnashai , 2008). ZeusNL enables through nonlinear static pushover analysis to obtain the roof displacement and inter-story drift under the prescribed seismic design load. The main building parameters that are considered include concrete resistance at the age of 28 days, columns and beams sections and their respective reinforcements.

***Static Nonlinear Pushover Analysis by Means of ZeusNL Software*:**

ZeusNL is an open source software package (Elnashai, 2008) which provides an efficient way to run structural analyses such as conventional and adaptive pushover and nonlinear dynamic time-history. The modelling takes into account both geometric and material nonlinear behaviour. Common concrete and steel material models are available, together with a large library of elements that can be used with a wide choice of typical pre-defined steel, concrete and composite section configurations. The applied loading can include constant or variable forces, displacements and accelerations.

In the conventional pushover analysis which is used in the following, the applied loads vary proportionally according to a predefined pattern. The post-peak response is obtained with a displacement control procedure. Modelling static pushover under ZeusNL software requires entering configuration of members sections, material properties, applied loadings and analysis protocol.

In the present analysis, the concrete behaviour was chosen to be described by the nonlinear concrete model with constant active confinement modelling (con2), figure 1. This enables accurate uniaxial concrete behaviour description where a constant confining pressure is assumed in order to take into account the maximum transverse pressure from confining steel. This is introduced on the model through a constant confinement factor, used to scale up the stress-strain relationship throughout the entire strain range. To enter this concrete model during simulations, four parameters are required: compressive strength, tensile strength, crushing strain and confinement factor.

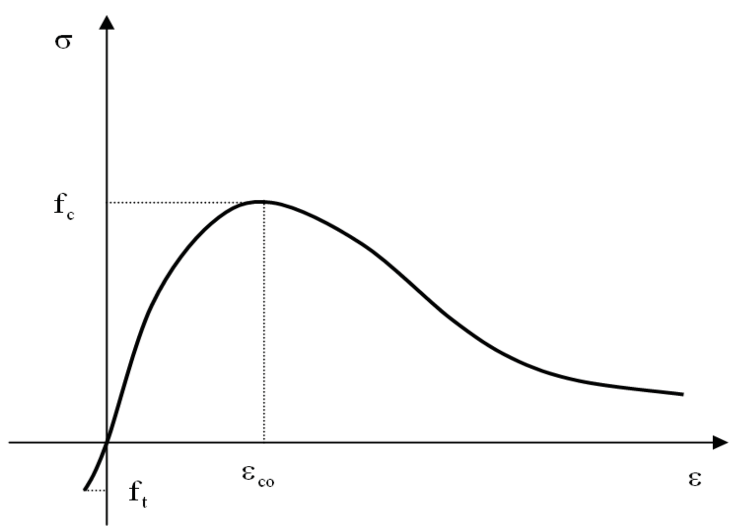

Fig. 1: Uniaxial constant confinement concrete model

The reinforcement steel behaviour was assumed to be a bilinear elastic plastic model with kinematics strain-hardening (stl1), figure 2. This model is applied for the uniaxial modelling of mild steel. To enter this model during simulations, three parameters are required: Young's Modulus E , yield strength σ_y and kinematic strain-hardening μ.

Static pushover analysis was conducted by taking the most adverse seismic direction when the building structure is assumed to be a plane gateway frame. Response control protocol was chosen to monitor the nonlinear analysis. This refers to the situation where the displacement of the building roof is specified by the user and is incrementally increased. The loading applied as well as the deformations of the other nodes are determined by the solution of the program.

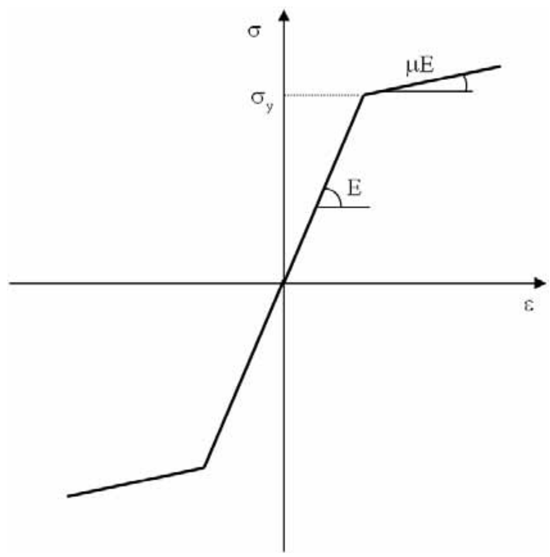

Fig. 2: Uniaxial bilinear elasto-plastic law with kinematic strain-hardening modelling mild steel

Presentation of the Case of Study:

A typical Moroccan pre-code reinforced concrete building is considered. The structure is a five-story regular building that lays on a rectangular horizontal surface of 16m×18m. The inter story height is 3m. The most severe seismic direction corresponds to the building width. The building behaviour for this seismic direction can be represented by a five-story four-bay frame with 4m bay length. Figures 3 and 4 depict the building elevation and plane view.

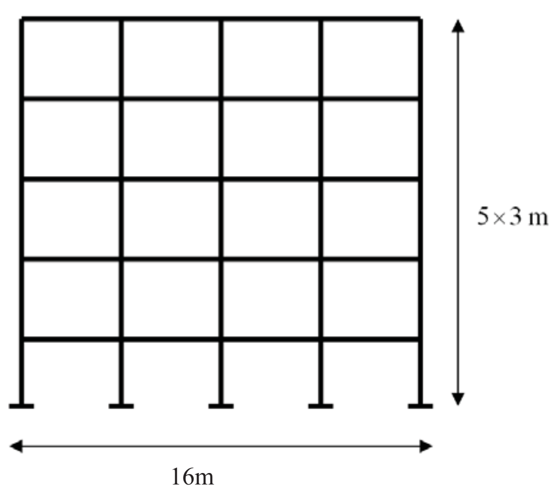

Fig. 3: Vertical elevation of the five-storey reinforced concrete structure in the seismic direction

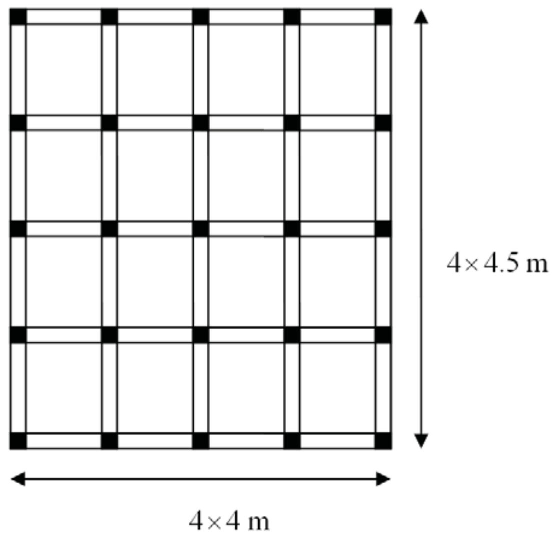

Fig. 4: Plane view of the five-storey reinforced concrete structure

The permanent and variable loads per unit surface are respectively $G = 5.3\ \mathrm{kN.m^{-2}}$ and $Q = 1.5\ \mathrm{kN.m^{-2}}$

The active seismic gravity loads are computed by taking the combination: P=G+0.2Q, defined according to the Moroccan seismic code RPS2000, (Royaume du Maroc, 2001).

This building was designed according to a traditional reinforced concrete code with material properties: concrete resistance f_{C28}=20MPa and steel characteristic resistance f_e=500 MPa. In this design no seismic regulations have been considered, so the building is a pre-code one. Table 1 displays the obtained dimensions of beams and columns as well as their reinforcements.

Table 1: Beams and columns sections with their reinforcements

	Sectin width (cm)	Section depth (cm)	Reinf. Section (cm^2)	Reinf. at section bottom	Reinf. at mid section	Reinf. at section top
Columns	20	40	A_{s1}=7.92	2 φ12	3 φ12	2 φ12
Beams	20	40	A_{s2}=7.5	6 φ8	0	9 φ8

The problem is now to retrofit this pre-code building with respect to the required performance criteria. The basic idea about performance-based seismic engineering is that the client or code regulations determine at first an acceptable hazard level for the desired performance criteria. Caution degree of the performance state will increase in general when decreasing the probability of an earthquake event to occur over a time period that is fixed in advance.

In this work, reference is made to limit states proposed by the Moroccan seismic code RPS2000, which limit the building roof displacement ratio and the maximum inter story drift under the action of the prescribed seismic design load. Other performance criteria that were introduced to distinguish performance-based engineering states with regards to earthquake events could be used such as those defined according to the Federal Emergency Management Agency (Federal Emergency Management Agency, FEMA- 356, 2000).

The roof displacement hreshold, $\overline{\delta}_{\mathrm{roof}}$, and the maximum inter story drift threshold, $\overline{\delta}_{\mathrm{is}}$, (in m)

according to RPS2000 write: Limit-state 1:

$$\delta_{\mathrm{roof}} \le \overline{\delta}_{\mathrm{roof}} = 0.004\mathrm{H}=0.06 \tag{1}$$

Limit-state 2:

$$\delta_{\mathrm{is}} \le \overline{\delta}_{\mathrm{is}} = 0.01\mathrm{h/K}=0.015 \tag{2}$$

where H is the total height of the building, h the inter-story height and K the coefficient of ductility. Since there does not exist always the possibility to evaluate δ_{roof} and δ_{is} from a given pushover curve if the collapse load is less than the RPS2000 prescribed seismic design load, it is pertinent to reason in terms of base shears associated to the limit-states displacements rather than considering the roof displacement and the maximum inter-story drift. The base shears are obtained from pushover curves as

$$\bar{V}_{roof} = V(\bar{\delta}_{roof}) \tag{3}$$

$$\bar{V}_{is} = V(\bar{\delta}_{is}) \tag{4}$$

The limit states are then: $V_{roof} \leq \bar{V}_{roof}$ and $V_{is} \leq \bar{V}_{is}$

In order to minimize the retrofitting cost, the rehabilitation operation should be performed in an optimal fashion. The following optimization methodology will be used. Assuming that beams and columns widths as well as reinforcements are modified proportionally, the considered design variables are restricted to only three factors: reduced concrete resistance f_C(MPa)/20 (factor A), members width multiplier (factor B), members reinforcements total section multiplier (factor C). Considering low threshold levels (Level 1) to be those corresponding to data given in table 1, intermediate design levels (Level 2) and high threshold levels (Level 3) are defined according to table 2.

Pushover simulations of the building were performed by using ZeusNL finite element software. Figure 5 gives the finite element model that was built under ZeusNL.

Table 2: Levels of the considered factors during pushover simulations

Factor	A	B	C
	f_c (Mpa)/ 20	Members width multiplier	Reinforcement multiplier
Level 1	1	1	1
Level 2	1.25	1.5625	1.5625
Level 3	1.5	2.25	2.25

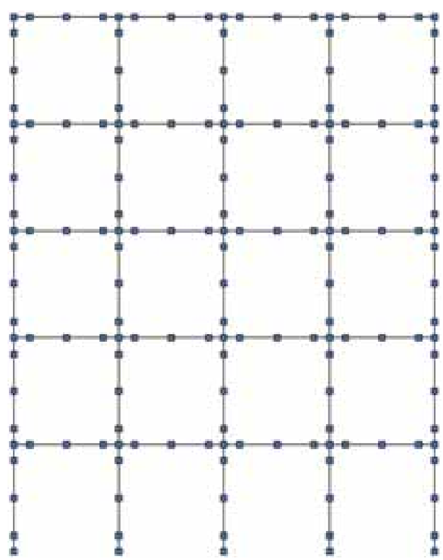

Fig. 5: Finite element model built under Zeus NL software package

Material properties that were used for concrete are: compressive strength f_C which varies according to table 2, tensile strength f_t= 2.2×10^6 Pa, crushing strain ε_{co}= 0.002, confinement factor for confined concrete K=1.2 and confinement factor for unconfined concrete K=1.02. For reinforcement steel material properties, input data were: Young's modulus E_s=2×10^{11} Pa, yield strength $\sigma_{y=}5\times10^8$ Pa and strain-hardening parameter μ=0.005.

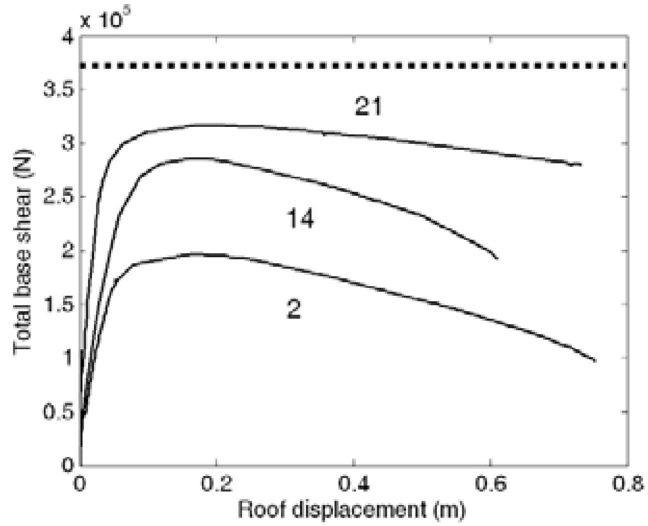

Fig. 6: Total base shear as function of the roof displacement for combinations 2, 14 and 21

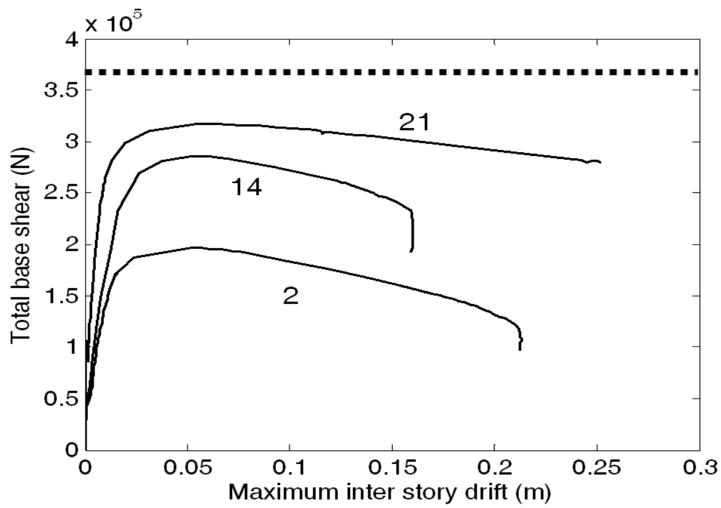

Fig.7: Total base shear as function of the maximum inter-story drift for combinations 2, 14 and 21

Based on table 2 a full factorial design of experiment table including 27 combinations can be constructed. Figures 6 and 7 give the total base shear (seismic capacity) as function respectively of the roof displacement and the maximum inter-story drift for combinations 2, 14 and 21. Table 3 summarises the obtained results in terms of the roof displacementδ_{roof}, the maximum inter story drift δ_{is} and collapse displacementδ_{ult}. The base shears, V_{roof} , V_{is} and V_{ult} associated to these displacements are also given.

The prescribed seismic design load is defined according to the Moroccan seismic code RPS2000 under the following assumptions: seismic zone: 3; site type: S2; priority class: 2; ductility coefficient: 2 and damping coefficient: 0.05. The RPS2000 seismic load acting on one frame of the building, as shown in figure 1, is given by

$$F = \frac{1}{5}\frac{A_{max}SDI}{K} W = 3.871 \times 10^5 \text{ N} \tag{5}$$

where $W=8.064\times10^6$ N is the total seismic load, $A_{max}=0.16$ the seismic acceleration for zone 3, D=2.5 the dynamic amplification factor, l=l the coefficient of priority and K= 2 the coefficient of ductility.

The dashed line in figures 6 and 7 indicates the seismic design load. It is seen from these figures that the three combinations 2, 14 and 21 fail to satisfy performance requirements associated to limit states 1 and 2. Referring to table 3, only the combinations 9 and 18 satisfy these requirements. To optimize seismic retrofit design of the building analysis of variance is performed in the following and response surface models for V_{roof} and V_{is} will be derived.

***Analysis of Variance and Response Surface Models*:**

Analysis of variance is performed by means of Matlab command *anovan* on the obtained results V_{roof}, V_{is} and V_{ult} that are given in table 3.

Table 4 gives the obtained results in terms of F and p-value statistics. It could be seen that variability of building performance in terms of the base shear at collapse and the shears associated to limit states 1 and 2 is due essentially to concrete sections, their reinforcements and interaction between these two factors. Concrete resistance has a small effect on results.

Based on the full factorial results presented in table 3, it is possible to derive simplified regression models or surface response models, (Roux, 1998). They are however valid only on the domain of parameters investigated and no extrapolation could be made of these metamodels without further analysis. Three response surfaces are derived in the following by using the Matlab command *regstats*: the base shear at collapse, the base shear associated to limit-state 1 and the base shear associated to limit-state 2.

By performing polynomial quadratic interpolation of results in terms of factors A, B and C the obtained regressions are

$$\begin{aligned}\overline{V}_{roof}(A,B,C) = (&1.3642-53.992A+8.1636B+28.18C-3.8576AB \\ &-6.122AC+8.1545BC+27.605A^2-2.4278B^2-1.89C^2)\times 10^4\end{aligned} \tag{6}$$

$$\begin{aligned}\overline{V}_{is}(A,B,C) = (&1.7443-45.424A+3.7871B+25.042C-3.4618AB \\ &-7.146AC+8.2023BC+24.309A^2-1.3696B^2-0.742C^2)\times 10^4\end{aligned} \tag{7}$$

$$\begin{aligned}V_{ult}(A,B,C) = (&-5.468-39.27A+4.3263B+31.243C-1.912AB \\ &-6.84AC+9.6517BC+21.383A^2-1.2336B^2-3.6263C^2)\times 10^4\end{aligned} \tag{8}$$

The associated R-square values, for respectively the total base shear at collapse and base shears corresponding to limit-states 1 and 2, are: R^2=97.6%, R^2=97.8% and R^2=97.9%. These indicate that the quadratic interpolations are quite good for all the base shears and that the surface response based models can be used to predict the efforts within the range of parameters values that are inside the polyhedron of interpolation.

Linear models can also be used at confidence level of 95% as their correlation factors are respectively: R^2=94.9%, R^2=95% and R^2=95% .

Retrofit Optimization:

Table 3 shows that the limit state 2 is more severe than limit state 1 and the collapse limit state, thus there is no need to consider these last in optimizing seismic retrofit. Limit state 2 requires that the maximum inter-story drift displacement is less than the limit threshold given by equation (2). This can be written under the following form

$$g_2(A,B,C) = \overline{V}_{is} - F \leq 0 \tag{9}$$

As the total weight of concrete is 245×10^3A(Kg) and the total weight of reinforcement is 7289B(Kg), assuming that the cost of steel reinforcement per unit mass is 10 times higher than that of concrete, the objective function writes

$$f(A,B,C) = 24.5B+7.289C \tag{10}$$

Now it is possible to apply any kind of minimisation technique to optimize seismic retrofit. In this work Matlab command *fmincon* which is based on the sequential quadratic programming algorithm (SQP) is used to solve the mathematical programming defined by equations (9) and (10). Upon choosing as lower and upper bounds of the solution the following vectors lb=[l; l; l] and ub=[1.5; 2.25; 2.25], and initializing the algorithm with (A_o,B_o,C_o)=[1; 1; 1], the obtained optimal solution is A=1, B=1.2653 and C=2.25. This yields that the optimal solution in terms of real design variables is obtained for fc= 20MPa, column width 25.3cm, column height 50.6cm, A_{s1}=17.82cm^2 and A_{s2}=16.88cm^2.

Table 3: Pushover simulation results as function of the combination number

Comb.	A	B	C	δ_{roof} (m)	δ_{is} (m)	δ_{ult} (m)	V_{roof} (N)	V_{is} (N)	V_{ult}(N)
1	1	1	1	0.1275	0.03818	0.2404	62710	57940	93060
2	1	1	1.5625	0.1711	0.05458	0.5066	175340	170370	196110
3	1	1	2.25	0.1752	0.05585	0.7976	288700	278530	308760
4	1	1.5625	1	0.1425	0.04205	0.3380	79390	72980	130860
5	1	1.5625	1.5625	0.1439	0.04495	0.7096	226630	211630	274670
6	1	1.5625	2.25	0.1720	0.06713	1.1117	390680	372750	430350
7	1	2.25	1	0.1532	0.04489	0.4311	96130	88340	166870
8	1	2.25	1.5625	0.1474	0.04843	0.9299	272310	256520	359970
9	1	2.25	2.25	0.1780	0.06570	1.4652	491140	476140	567190
10	1.25	1	1	0.1288	0.03803	0.2539	65800	61510	98300
11	1.25	1	1.5625	0.1665	0.05332	0.5095	157140	150170	197210
12	1.25	1	2.25	0.2018	0.06451	0.8095	291760	281290	313370
13	1.25	1.5625	1	0.1400	0.04143	0.3610	84020	77930	139740
14	1.25	1.5625	1.5625	0.1738	0.05656	0.7373	233880	219250	285410
15	1.25	1.5625	2.25	0.2102	0.07606	0.8988	305270	288810	347910
16	1.25	2.25	1	0.1591	0.04685	0.4688	101720	93370	181460
17	1.25	2.25	1.5625	0.1579	0.05134	0.9740	284040	268020	377040
18	1.25	2.25	2.25	0.2111	0.06571	1.2568	396250	387350	486510
19	1.5	1	1	0.1480	0.0445	0.2659	68780	64920	102920
20	1.5	1	1.5625	0.1495	0.05026	0.4946	170520	154350	191470
21	1.5	1	2.25	0.2014	0.06427	0.8192	296710	286300	317120
22	1.5	1.5625	1	0.1496	0.04517	0.3769	87600	81490	145890
23	1.5	1.5625	1.5625	0.1967	0.06559	0.7522	241360	225010	291190
24	1.5	1.5625	2.25	0.2228	0.08993	1.1450	402090	373370	443220
25	1.5	2.25	1	0.1576	0.04622	0.4972	105830	97930	192480
26	1.5	2.25	1.5625	0.1683	0.05401	1.0035	290530	274670	388450
27	1.5	2.25	2.25	0.2382	0.09279	1.2758	403870	385640	493860

Table 4: ANOVA statistics for the base shear at collapse and base shears associated to limit states 1 and 2

	V_{roof}		V_{is}		V_{ult}	
Factor	F	p- value	F	p- value	F	p- value
A	1.52	0.2767	1.3	0.3239	1.12	0.3735
B	295.82	0	289.43	0	324.64	0
C	35.05	0.0001	33.06	0.0001	104.65	0
A*B	1.6	0.2648	1.54	0.2796	1.89	0.2054
A*C	0.59	0.6805	0.52	0.7227	0.59	0.6792
B*C	3.68	0.0552	3.8	0.0513	5.99	0.0157

Conclusions:

Within the framework of conventional jacketing technique, optimization of seismic retrofitting of pre-code reinforced concrete buildings had been conducted in this work. Response surface based models that express performance limit states were derived through pushover analysis performed with respect to a full factorial design of experiment table where three design variables had been considered. Relative effects of these design variables had been determined by using analysis of variance on the obtained results. Optimization had been conducted after that through solving a nonlinear mathematical programming with the objective to minimize the cost of structural members under the constraints that the building meets the required performance states.

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