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Methodology for optimizing seismic rehabilitation of pre-code reinforced concrete buildings

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This work deals with optimization of seismic rehabilitation for regular pre-code reinforced concrete buildings. Evaluation of vulnerability is performed through conventional non-linear static pushover computations, using specialized finite element software. Three limitations were considered. These include: a prescribed code seismic design load of roof displacement of the total building, the maximum inter story drift and the prevention of collapse. Four factors characterizing transverse sections of beams and columns and their respective reinforcements are introduced. A complete factorial design of experiment table having three levels has been used to define a finite set of data points where the base shear, the roof displacement and the maximum inter story drift were evaluated. Response surface models were derived then via polynomial regressions and used to write explicitly the optimization problem constraints. Optimal seismic rehabilitation of the building was carried out, with the objective of minimizing the total cost of building structural members.

Key words: Vulnerability, earthquake, 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. Vulnerability is the probability of damage exceeding certain limit state conditions at a given seismic intensity level. Seismic vulnerability of buildings varies as function of material and geometric characteristics. Analysis of vulnerability is essential for seismic diagnostic in order to program rehabilitation operations to mitigate seismic risk (Buratti et al., 2010; Kaveh et al., 2010; Quanwang 2006, Mehmet and Hayri, 2006; Shakib et al., 2011; Hasan et al., 2002; Applied Technology Council ATC-40 (1996). Seismic evaluation and retrofitting of existing buildings describe deficiency-based and systematic procedures based on performance principles to evaluate and retrofit existing buildings to withstand the effects of earthquakes. This next generation standard combines the evaluation and retrofit process and puts forth a three-tiered process for seismic evaluation based on a range of building performance level: from collapse prevention to operation that marries targeted structural performance with non-structural elements performance. The deficiency-based procedures involve focussing the evaluation and retrofit on specific potential deficiencies based on past earthquake observations, to obtain a permissible set of

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building types and heights. The systematic procedure, applicable to any building, sets forth a methodology for evaluating the entire building in a rigorous manner.

Existing vulnerability studies of buildings at seismic risk are found to be related to three major approaches: empirical, judgmental and analytical. Empirical approaches are derived mainly from observed post earthquake surveys. Judgmental approaches are obtained through experts' opinion. Analytical approaches are based on mathematical model simulations, mainly by using the finite element method. Rossetto and Elnashai (2003) have reviewed these various vulnerability approaches and have given new empirical fragility curves for European reinforced concrete buildings that were derived from a data bank of a hundred post-earthquake damage distributions. Krawinkler and Seneviratna (1998) have discussed the possibility to use pushover analysis in order to assess analytically building vulnerability. They concluded that the pushover analysis provides adequate information on seismic demands if the building structure is regular.

To perform rehabilitation of pre-code buildings, it is important to assess effects of the intervening key material and geometric factors on their vulnerability. A simplified methodology is proposed in this work. It is based on pushover analysis of the reinforced concrete building which is performed according to a full factorial design of experiment DOE table constructed by choosing key building parameters and some of their levels. Post-analysis of variance on the obtained results enables them to assess the relative influence of each factor. This is of great importance if one desires to perform proper mitigation measures for seismic rehabilitation of buildings since he could act pretty on the most important factors to reduce vulnerability.

Polynomial regression can be used also on the obtained DOE results to derive a response surface metamodel which will give explicit approximation of the considered limit state as function of the design variables. These regressions can be used to write the constraints associated to the required seismic performance level. Considering the objective function which consists generally in minimizing the cost of building structural members, a non linear mathematical program can be written and solved to find the optimal design.

This methodology is employed in the following to assess vulnerability of concrete reinforced buildings and to perform their optimal rehabilitation. The performance levels searched include the limit states as defined by the Moroccan seismic code RPS2000 (Royaume du Maroc, 2001) under the prescribed seismic design load, and the collapse prevention state under the design load augmented by 50%. Zeus Non-Linear (ZeusNL) software package is made use of (Elnashai et al., 2008). ZeusNL enables through nonlinear static pushover analysis of the building to obtain the roof displacement and inter-story drift under the prescribed seismic design load as well as the total base shear capacity at collapse. The main building parameters that are considered include columns and beams sections and their respective reinforcements.

METHODOLOGY

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 four key building design variables and fixing three levels for each of them. Post-analysis of variance conducted on the obtained results enabled the assessment of 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 of the cost of structural members to be minimized, a non-linear mathematical programming is written and solved to find the optimal design.

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 columns and beams sections and their respective reinforcements. To simulate the building response under lateral seismic loads, ZeusNL software package is used (Elnashai et al., 2008). This software provides an efficient way to run eigenvalue analysis for building structures, conventional and adaptive pushover and nonlinear dynamic time-history. The modelling takes into account both geometric and material nonlinear behaviour. 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 exclusively used here (Elnashai et al., 2008), 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 material properties, section configurations, applied loadings and analysis protocol.

The concrete behaviour was chosen to be described by the nonlinear concrete model with constant (active) confinement modelling (con2). 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. Improved cyclic rules were included to enable the prediction of continuing cyclic degradation of strength and stiffness, as well as better numerical stability under large displacements analysis (Elnashai et al., 2008). To enter this concrete model during simulations, four parameters are required: compressive strength, tensile strength, crushing strain and confinement factor.

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

Static pushover analysis was conducted under ZeusNL by taking the most adverse seismic direction and 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

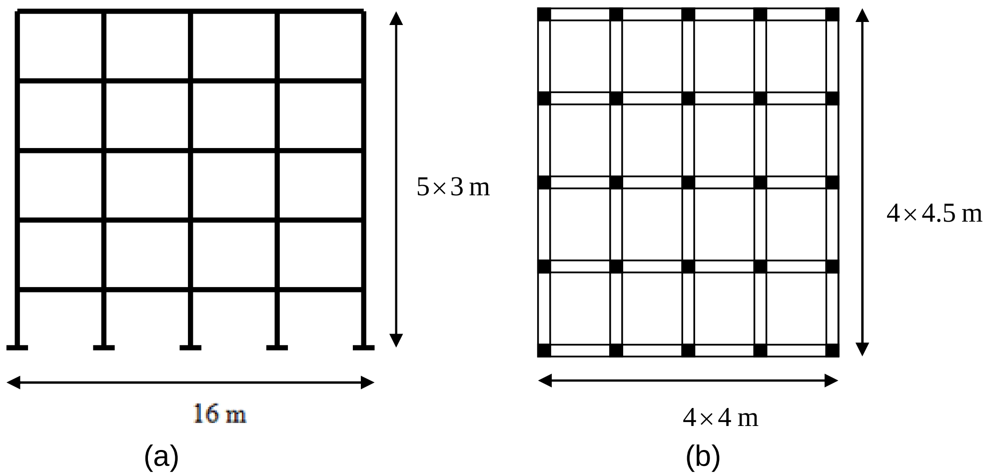

Figure 1. Five-storey four-bay reinforced concrete structure; (a) Vertical elevation, (b) Plane view.

Table 1. Beams and columns sections with their reinforcements.

Factor	Section width (cm)	Section depth (cm)	Reinforcements at section bottom	Reinforcements at section top	Reinforcements at mid section
Columns	20	40	$2\phi 12$	$2\phi 12$	$3\phi 12$
Beams	30	50	$6\phi 8$	$9\phi 8$	0

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.

Presentation of the case study

Pushover simulations have been conducted on a typical modern Moroccan reinforced concrete building. The structure is a five-story regular building that lays on a rectangular horizontal surface of $16\,\mathrm{m}\times 18\,\mathrm{m}$. The inter story height is $3\,\mathrm{m}$. The most severe seismic direction corresponds to the building width and for which the building could be represented by a five-story four-bay frame where the bay length is $4\,\mathrm{m}$. Figure 1 depicts the building elevation and plane view.

The permanent and variable loads per unit surface of the stories are respectively $G = 5.3\,\mathrm{kN.m^{-2}}$ and $Q = 1.5\,\mathrm{kN.m^{-2}}$. The active gravity loads are computed by taking the standard combination: $P = G + 0.2Q$, according to the Moroccan seismic code RPS2000 (Royaume du Maroc, 2001).

Using the reinforced concrete buildings code, Eurocode 2 with common material properties (concrete resistance $f_{c28} = 25\,\mathrm{MPa}$; steel characteristic resistance $f_e = 500\,\mathrm{MPa}$ and soil capacity resistance: $0.22\,\mathrm{MPa}$) a coarse design of this building can be obtained. In this coarse design no seismic code has been considered and the building is termed then pre-code. Table 1 displays the obtained dimensions of beams and columns as well as their reinforcements.

Quantities given in Table 1 are those of a pre-code building for which the problem is now to diagnose vulnerability and to perform rehabilitation operations with regard to some given performance criteria. The pursued objective is then to know how to make optimal rehabilitation of this building in order to minimize the cost.

In this work, reference is made to limit states introduced by the Moroccan seismic code RPS2000, which intends to limit the building roof displacement ratio and the maximum inter story drift under the action of the prescribed design seismic load. To these, limit states and the collapse prevention state will be considered under the RPS2000 design load augmented by 50%.

Other performance criteria introduced to distinguish performance-based engineering states with regards to earthquake events could be used as, for example, those defined according to the Federal Emergency Management Agency (FEMA, 2000). In this case, the performance states include: operational performance for which the event does not affect the occupants or functioning of the building; immediate occupancy performance for which the occupants can immediately return to the building after the seismic event; life safety performance and collapse prevention performance.

The basic idea about performance-based seismic engineering is that the client or code regulations determine at first an acceptable hazard level for the required performance criteria. Degree of the desired performance state will increase in general when decreasing the probability of an earthquake event over a time period that is fixed apriori. As an example, for an earthquake event considered with probability 50% in 50 years, FEMA immediate occupancy performance may be demanded. On the other hand, for an earthquake event with probability 2% in 50 years only FEMA life safety performance may be wanted.

To reach the optimal rehabilitation of the building, the following methodology will be used.

Table 2. Levels of the considered factors during pushover simulations.

Factor	A		B			C		D		
	Beams sections		Beams reinforcements			Columns sections		Columns reinforcements		
	Height (cm)	Width (cm)	Top	Mid	Bottom	Height (cm)	Width (cm)	Top	Mid	Bottom
Level 1	40	20	9 ϕ 8	0	6 ϕ 8	40	20	2 ϕ 12	3 ϕ 12	2 ϕ 12
Level 2	50	30	9 ϕ 10	0	6 ϕ 10	50	30	2 ϕ 14	3 ϕ 14	2 ϕ 14
Level 3	60	40	9 ϕ 12	0	6 ϕ 12	60	40	2 ϕ 16	3 ϕ 16	2 ϕ 16

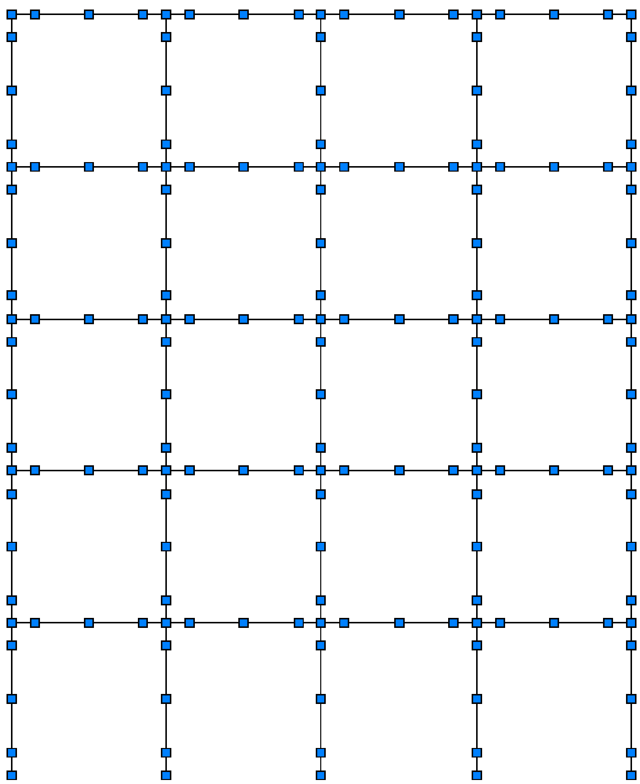

Figure 2. Finite element model built under Zeus NL software package.

First, the study is restricted to only four factors: beams sections (A), beams reinforcements (B), columns sections (C) and columns reinforcements (D). Secondly, low threshold levels (Level 1) are chosen to those corresponding to data given in Table 2. The intermediate design (Level 2) is obtained by incrementing these data. Then, they are incremented again to obtain high threshold levels (Level 3). The tree levels are like those obtained for each factor. Table 2 recalls the three levels associated to each intervening factor.

Using a full factorial design of experiment table and performing simulations by means of Zeus NL software the roof displacement and the maximum inter story drift under the RPS2000 prescribed design are obtained. The collapse load is also determined from the obtained pushover curve for each combination of factors; it corresponds to the maximum of this curve. This enables performing an analysis of variance in order to determine the relative influence of the intervening factors. Response surface metamodels can be derived after that in order to evaluate limit states explicitly in terms of the intervening factors.

The response surface based models could now serve to write the constraints of a mathematical program for which minimizing the building structural members total cost is searched. Standard optimization software could be used to track the optimal solution.

RESULTS AND DISCUSSION

Based on Table 2, a full factorial design of experiment table including 81 combinations can be constructed. Table A1 of Appendix A gives summarises the obtained results in terms of roof displacement and maximum inter story drift under the prescribed seismic design load as well as the total base shear at collapse.

Figure 2 gives the finite element model build under Zeus NL. Figure 3 gives the deformed shape just before collapse for the particular combination number, 26. Figure 4 gives total base shear (seismic load) as function of the roof drift displacement for combinations 1, 9, 26 and 81. Figure 5 gives total base shear as function of the maximum inter-story drift displacement. All the curves are between the envelope curves corresponding to combinations of 1 and 81.

Figures 6, 7 and 8 give, based on the function of the combination number, the obtained results in terms of the base shear at collapse V_{base}; the roof drift displacement, δ_{roof}, and the maximum inter story drift, $\delta_{inter-story}$, under the RPS2000 prescribed seismic design load. These displacements are to be compared to limit state displacements as stated by RPS2000 code (in m).

Limit- state 1

$$\bar{\delta}_{roof} = 0.004H=0.06 \quad (1)$$

Limit -state 2

$$\bar{\delta}_{inter-story} = 0.01h/K=0.015 \quad (2)$$

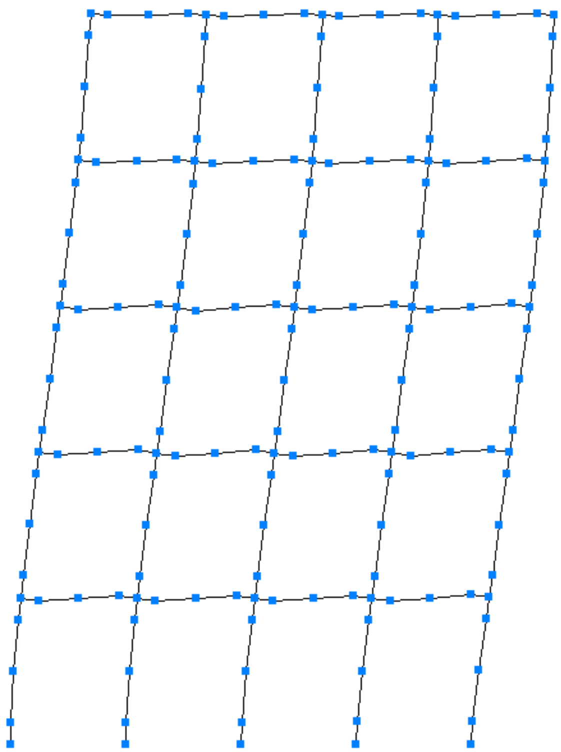

Figure 3. Deformed shape of the building just before collapse for combination number 26.

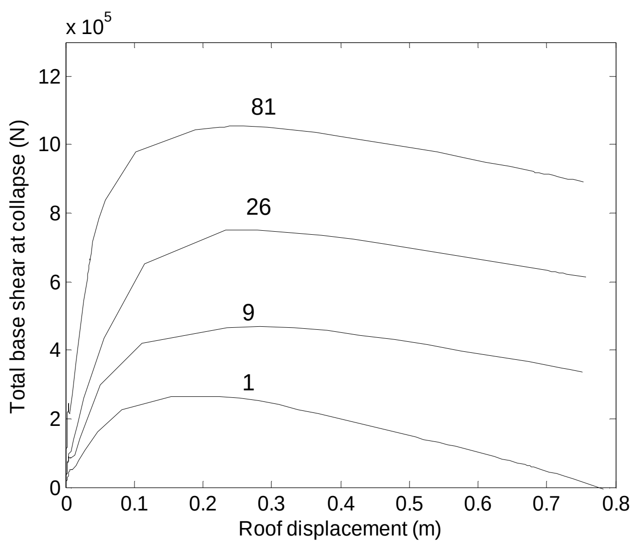

Figure 4. Total base shear for combinations 1, 9, 26 and 81 as function of roof displacement.

Where H is the total height of 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 $\delta_{inter-story}$ from a given pushover curve if the collapse load is less than the RPS2000 prescribed seismic design load, rather than considering the roof displacement and maximum inter-story drift, it is pertinent to reason on base shears associated to the limit-states displacements. These are obtained from pushover curves as

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

$$V_{inter-story} = V(\overline{\delta}_{inter-story}) \tag{4}$$

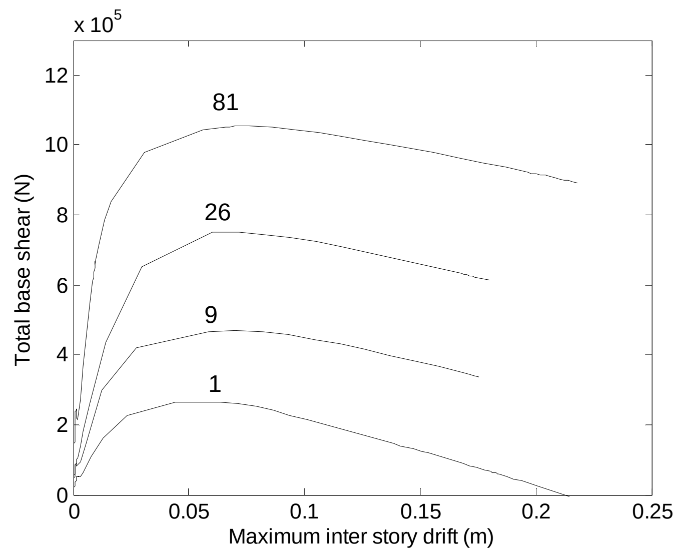

Figure 5. Total base shear for combinations 1, 9, 26 and 81 as function of maximum inter-story drift.

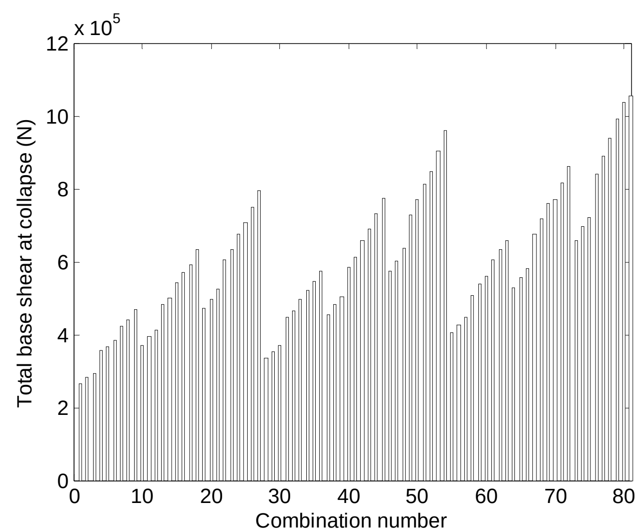

Figure 6. Pushover results in terms of the total base shear as function of the combination number.

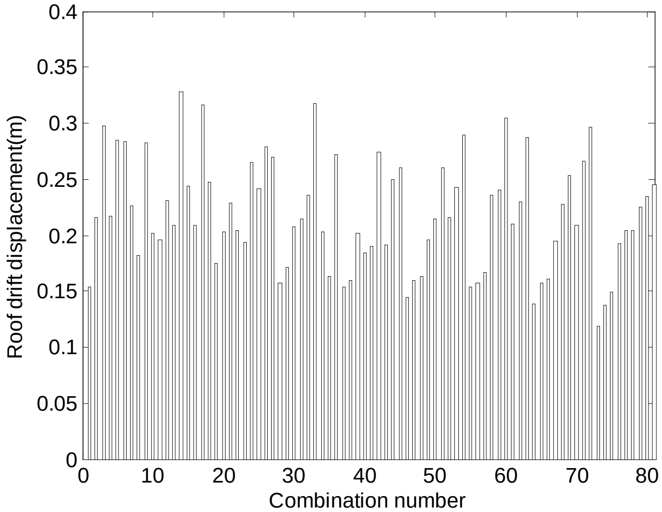

Figure 7. Pushover results in terms of the roof drift as function of the combination number.

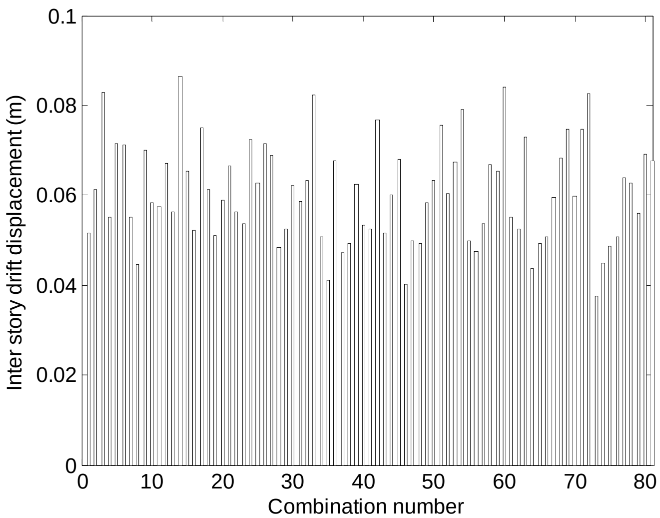

Figure 8. Pushover results in terms of the maximum inter story drift as function of the combination number.

Analysis of variance is performed by means of Matlab command *anovan* on the obtained results: V_{base}, V_{roof} and $V_{inter-story}$ according to the full factorial design of experiment table.

Table 3 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 is due to all factors and their interactions since the p-values are minimal. Interaction between beams sections and columns reinforcements has smaller influence in comparison with the others. For the base shear associated to limit-state 1, the mean factors are beam sections and their reinforcements, followed by columns sections. This result is not obvious, since one expects that columns sections have the most important effect on roof displacement. For the base shear associated to limit-state 2, all the intervening factors are important. The most significant interactions are those of beams sections with their reinforcements and with columns sections, and also interaction between columns sections and beam reinforcements.

Within the range of variation of the selected parameters, the results show that the intervening factors have no equal effect on performance criteria since their influence is not the same when one considers the roof drift displacement limit-state or the inter-story drift limit-state. They show also that maximising the total base shear at collapse is not equivalent to minimizing the displacements limit-states.

Based on the full factorial results presented in Table A.1 (Appendix), it is possible to derive simplified regression models or surface response models (Roux et al., 1998). They are however valid on the domain of parameters investigated in this work and no extrapolation could be made with the guarantee of accuracy. 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, C and D, the obtained regressions are :

$$\begin{aligned}V_{base}(A,B,C,D) &= -1.7900\times10^{5}+1.3365\times10^{6}A+2.0621\times10^{8}B\\ &+1.6509\times10^{6}C-4.4803\times10^{6}D+4.8625\times10^{8}AB+2.2391\times10^{6}AC\\ &+1.2904\times10^{8}AD+7.0525\times10^{8}BC+5.9507\times10^{10}BD+2.4974\times10^{8}CD\\ &-3.6219\times10^{6}A^{2}-5.6614\times10^{10}B^{2}-5.0628\times10^{6}C^{2}-1.1529\times10^{10}D^{2}\end{aligned} \tag{5}$$

$$\begin{aligned}V_{roof}(A,B,C,D) &= -1.3682\times10^{5}+1.5345\times10^{6}A+1.1626\times10^{8}B+9.4332\times10^{5}C\\ &+1.3462\times10^{7}D+7.8566\times10^{8}AB+3.0347\times10^{6}AC+1.903\times10^{8}AD+4.9059\times10^{8}BC\\ &+2.9316\times10^{10}BD+1.5531\times10^{8}CD-4.9766\times10^{6}A^{2}-6.2195\times10^{10}B^{2}-3.3006\times10^{6}C^{2}\\ &-3.0395\times10^{10}D^{2}\end{aligned} \tag{6}$$

$$\begin{aligned}V_{inter-story}(A,B,C,D) &= -1.571\times10^{5}+1.8923\times10^{6}A+2.2213\times10^{8}B+6.1484\times10^{5}C\\ &-8.6357\times10^{7}D+6.3718\times10^{8}AB+2.3745\times10^{6}AC+1.6683\times10^{8}AD+4.3137\times10^{8}BC\\ &+2.4523\times10^{10}BD+3.9291\times10^{8}CD-5.5534\times10^{6}A^{2}-9.7068\times10^{10}B^{2}-2.2086\times10^{6}C^{2}\\ &+6.6582\times10^{9}D^{2}\end{aligned} \tag{7}$$

The associated R-square values are $R^2=99.8\%$, $R^2=99.3\%$ and $R^2=97\%$ for the total base shear at collapse and base shears corresponding to limit-states 1 and 2. These indicate that the interpolations are quite good for all the base shears and that the quadratic surface response models can be used to predict these efforts within the intervals of interpolation.

Table 3. ANOVA statistics for the base shear at collapse and base shears associated to limit states 1 and 2.

Factor	Base shear at collapse		Base shear for limit-state 1		Base shear for limit-sate 2	
	F	p- value	F	p- value	F	p- value
A	4504.94	0	3.71	0.0317	501.62	0
B	10104.55	0	5.32	0.0082	270.11	0
C	6494.19	0	1.38	0.2616	362.69	0
D	432.31	0	0.61	0.5456	7.4	0.0016
A*B	51.2	0	1.09	0.3718	8.83	0
A*C	35.37	0	1.51	0.2147	4.62	0.0031
A*D	3.18	0.0213	1.09	0.3716	0.84	0.5046
B*C	109.24	0	1.09	0.3735	4.7	0.0028
B*D	12.09	0	1.04	0.3974	0.83	0.5119
C*D	6.69	0.0002	1.07	0.3822	1.62	0.1854

Optimizing rehabilitation

Optimization of the rehabilitation process can be formulated as a straightforward numerical programming problem in which an objective function must be minimized with respect to the design variables chosen here to be the total cost of structural members. The problem constraints are given by the required conditions to meet the previous three limit states (Paolo et al., 2012).

As a real case study, where the developed methodology is expected to be of considerable help, the pre-code building is considered in the following to be rehabilitated with minimum cost. The prescribed seismic design load is evaluated 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. Taking one frame of the building, the RPS2000 seismic design load is given by:

$$F = \frac{1}{5}\frac{A_{max}SDI}{K}W = 3.871\times10^5\ \mathrm{N} \tag{8}$$

Where $W = 8.064\times10^6\ \mathrm{N}$ is the total seismic load, $A_{max} = 0.16$ the seismic acceleration for zone3, $D = 2.5$, the dynamic amplification factor, $I = 1$, the coefficient of priority and $K = 2$ the coefficient of ductility.

The two first limit states require that the roof and the maximum inter-story drift displacements are less than the limit thresholds given by equations (1) and (2). These can be transformed by using Equations (3), (4), (6) and (7) under the following form,

$$g_1(A,B,C,D) = F_{roof}(0.06) - F \le 0 \tag{9}$$

$$g_2(A,B,C,D) = F_{inter-story}(0.015) - F \le 0 \tag{10}$$

One must add to these equations the collapse limit state under the prescribed seismic design load augmented by 50%. This last constraint is,

$$g_3(A,B,C,D) = 1.5F - V_{base} \le 0 \tag{11}$$

Where V_{base} is the total base shear at collapse.

Calculating the total weight of beams and columns members, assuming that the cost of steel reinforcement is 10 times higher than that of concrete, the objective function is,

$$f(A,B,C,D) = 2.72\,A + 84.85B + C + 31.2D \tag{12}$$

Matlab command *fmincon* based on the sequential quadratic programming algorithm (SQP) can be used to solve the mathematical program defined by Equations (9), (10), (11) and (12). Upon choosing the lower and upper bounds of the solution, the following vectors are taken,

$$lb=[0.08,\ 7.54\times10^{-4},\ 0.08,\ 7.92\times10^{-4}],\quad ub=[0.24,\ 1.70\times10^{-3},\ 0.24,\ 1.41\times10^{-3}] \tag{13}$$

and initializing the algorithm by $(A_0,B_0,C_0,D_0)=[0.08,\ 7.54\times10^{-4},\ 0.08,\ 7.92\times10^{-4}]$, the obtained optimal solution is $A = 0.08\,m^2$, $B = 1.70\times10^{-3}m^2$, $C = 0.104m^2$ and $D = 1.41\times10^{-3}m^2$. The value of the objective function at this minimum is: $fval = 0.509$.

If the *fmincon* command and the genetic algorithm based command *ga* were used, then the obtained solution is $A = 0.08\,m^2$, $B = 1.61\times10^{-3}m^2$, $C = 0.146m^2$ and $D = 0.792\times10^{-3}m^2$. The value of the objective function in this last case is: $fval = 0.525$.

This shows that the *ga* algorithm gives a good approximation of the optimal solution but with a different solution.

Conclusion

Optimal rehabilitation design of reinforced concrete buildings was performed in this work. This was done by means of response surface based models obtained by pushover analysis conducted with respect to a full factorial design of experiment table constructed on four key design variables of the building. Relative effects of the intervening parameters have been determined by using analysis of variance on the obtained results. Quadratic regressions were derived for the total base shear at collapse, the base shears associated to the displacement limit-states as specified by the Moroccan seismic code. Exact optimization was conducted after solving a nonlinear program in order to minimize the cost of structural members under the constraints that the building meets the various performance states. This methodology has proved to be applicable through this conclusive case study.

Conflict of Interests

The author(s) have not declared any conflict of interests.

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Appendix A: Pushover results as function of the considered combination.
Table A1. Roof displacement and inter story drift as function of the simulation number.

Simulation number	A (m^2)	B $(10^{-4}\,m^2)$	C (m^2)	D $(10^{-4}\,m^2)$	V_{base} $(\times 10^5\,N)$	V_{roof} $(\times 10^5\,N)$	$V_{inter-story}$ $(\times 10^5\,N)$
1	0.08	7.5	0.08	7.92	2.657	1.8631	1.7422
2	0.08	7.5	0.08	10.78	2.811	1.9898	1.8477
3	0.08	7.5	0.08	14.07	2.922	1.927	1.8241
4	0.08	7.5	0.15	7.92	3.568	2.386	2.3366
5	0.08	7.5	0.15	10.78	3.67	2.5209	2.4416
6	0.08	7.5	0.15	14.07	3.856	2.5693	2.4845
7	0.08	7.5	0.24	7.92	4.226	3.0795	3.1299
8	0.08	7.5	0.24	10.78	4.42	3.0706	3.097
9	0.08	7.5	0.24	14.07	4.686	3.16	3.1785
10	0.08	11.85	0.08	7.92	3.703	2.3699	2.2034
11	0.08	11.85	0.08	10.78	3.93	2.4212	2.2222
12	0.08	11.85	0.08	14.07	4.136	2.478	2.3148
13	0.08	11.85	0.15	7.92	4.836	3.134	3.0317
14	0.08	11.85	0.15	10.78	5.008	3.1525	3.0506
15	0.08	11.85	0.15	14.07	5.405	3.2244	3.0961
16	0.08	11.85	0.24	7.92	5.71	3.6277	3.6295
17	0.08	11.85	0.24	10.78	5.906	3.8796	3.9761
18	0.08	11.85	0.24	14.07	6.331	3.9385	3.9613
19	0.08	16.95	0.08	7.92	4.717	2.8515	2.7279
20	0.08	16.95	0.08	10.78	4.962	2.9205	2.7953
21	0.08	16.95	0.08	14.07	5.239	2.9846	2.8725
22	0.08	16.95	0.15	7.92	6.044	3.7178	3.5691
23	0.08	16.95	0.15	10.78	6.335	3.7883	3.6223
24	0.08	16.95	0.15	14.07	6.772	3.8538	3.6805
25	0.08	16.95	0.24	7.92	7.075	4.4313	4.4468
26	0.08	16.95	0.24	10.78	7.506	4.4643	4.4669
27	0.08	16.95	0.24	14.07	7.951	4.6035	4.6153
28	0.15	7.5	0.08	7.92	3.348	2.7121	2.5788
29	0.15	7.5	0.08	10.78	3.514	2.7992	2.6058
30	0.15	7.5	0.08	14.07	3.716	2.8934	2.7139
31	0.15	7.5	0.15	7.92	4.489	3.5307	3.3911
32	0.15	7.5	0.15	10.78	4.651	3.5995	3.4443
33	0.15	7.5	0.15	14.07	4.956	3.6271	3.4991
34	0.15	7.5	0.24	7.92	5.204	4.0848	4.0472
35	0.15	7.5	0.24	10.78	5.44	4.3431	4.3134
36	0.15	7.5	0.24	14.07	5.723	4.2821	4.2955
37	0.15	11.85	0.08	7.92	4.547	3.6288	3.3806
38	0.15	11.85	0.08	10.78	4.827	3.6211	3.342
39	0.15	11.85	0.08	14.07	5.044	3.787	3.529
40	0.15	11.85	0.15	7.92	5.844	4.6262	4.4102
41	0.15	11.85	0.15	10.78	6.134	4.7126	4.5086
42	0.15	11.85	0.15	14.07	6.586	4.7949	4.5268
43	0.15	11.85	0.24	7.92	6.887	5.3785	5.2461
44	0.15	11.85	0.24	10.78	7.327	5.5016	5.3414
45	0.15	11.85	0.24	14.07	7.744	5.7374	7.5184
46	0.15	16.95	0.08	7.92	5.73	4.2327	3.8902
47	0.15	16.95	0.08	10.78	6.029	4.2937	4.0207

Table A1. Contd.

48	0.15	16.95	0.08	14.07	6.367	4.383	4.1096
49	0.15	16.95	0.15	7.92	7.281	5.3858	5.0702
50	0.15	16.95	0.15	10.78	7.694	5.5119	5.1775
51	0.15	16.95	0.15	14.07	8.136	5.6318	5.2469
52	0.15	16.95	0.24	7.92	8.463	6.4024	6.2352
53	0.15	16.95	0.24	10.78	9.024	6.534	6.2575
54	0.15	16.95	0.24	14.07	9.61	6.6237	6.3349
55	0.24	7.5	0.08	7.92	4.05	3.5282	3.3253
56	0.24	7.5	0.08	10.78	4.258	3.6107	3.3946
57	0.24	7.5	0.08	14.07	4.485	3.6948	3.4606
58	0.24	7.5	0.15	7.92	5.062	4.3143	4.1073
59	0.24	7.5	0.15	10.78	5.374	4.3985	4.2111
60	0.24	7.5	0.15	14.07	5.58	4.4772	4.2644
61	0.24	7.5	0.24	7.92	6.07	5.1649	5.0491
62	0.24	7.5	0.24	10.78	6.335	5.273	5.1265
63	0.24	7.5	0.24	14.07	6.592	5.3469	5.2503
64	0.24	11.85	0.08	7.92	5.285	4.4601	4.2186
65	0.24	11.85	0.08	10.78	5.568	4.6003	4.337
66	0.24	11.85	0.08	14.07	5.798	4.6622	4.4612
67	0.24	11.85	0.15	7.92	6.768	5.6262	5.3552
68	0.24	11.85	0.15	10.78	7.173	5.7068	5.4599
69	0.24	11.85	0.15	14.07	7.6	5.7955	5.499
70	0.24	11.85	0.24	7.92	7.694	6.5648	6.3523
71	0.24	11.85	0.24	10.78	8.153	6.7233	6.4443
72	0.24	11.85	0.24	14.07	8.632	6.8588	6.5818
73	0.24	16.95	0.08	7.92	6.572	5.134	4.7067
74	0.24	16.95	0.08	10.78	6.976	5.341	4.9283
75	0.24	16.95	0.08	14.07	7.198	5.488	5.0378
76	0.24	16.95	0.15	7.92	8.394	7.0273	6.6828
77	0.24	16.95	0.15	10.78	8.893	7.062	6.6302
78	0.24	16.95	0.15	14.07	9.389	7.3934	6.8208
79	0.24	16.95	0.24	7.92	9.918	7.3934	6.8208
80	0.24	16.95	0.24	10.78	10.357	8.3869	7.9779
81	0.24	16.95	0.24	14.07	10.541	8.4579	8.0873