Seismic performance reliability assessment for irregular reinforced concrete buildings

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ABSTRACT

Reliability analysis assessment of seismic performance for irregular reinforced concrete buildings was investigated in this work. A limit state defined in terms of the total building roof displacement was considered. The seismic behavior of the building was examined by using dynamic transient analysis through finite element computations. Four random variables characterizing seismic acceleration, building mass, concrete members sections and steel reinforcements were introduced. Analysis of reliability was achieved by using surface response approximation and first order reliability method. A complete factorial design of experiment table having three levels was used to define a finite set of data points where the limit state function was evaluated. The influence of each factor was then determined by performing analysis of variance on the obtained results. Two factors were found to govern largely the considered performance state. Identification of the building response surface model via quadratic polynomial regression was achieved with a good correlation. Finally, discussion was carried out about the effect on reliability resulting from the distributions of probability modelling parameters uncertainties.

Keywords - **Seism, Reinforced concrete buildings, Uncertainties, Reliability, Response surface, FEM**

I. INTRODUCTION

Considering buildings at risk of earthquake events, seismic demand loads and structural capacity are random variables and vary as function of uncertainties governing the intervening basic design parameters. In particular, the ground motion acting on the building structure includes stochastic variables like the peak intensity, the frequency content as well as the seism duration. On the other hand, the nonlinear dynamic response of the structure depends on the configuration geometry of the building and the uncertain features of material properties such as stiffness, strength or damping.

Taking into consideration all the uncertainties during seismic performance assessment with regards to a given limit state enables to render in a more realistic way the risk the building is undergoing. This is carried out in terms of probabilities within the framework of reliability analysis which makes it possible to get estimation of the probability of failure through uncertainty propagation [1-4].

Unlike the simple case where the building is regular, the general problem of irregular buildings at risk of earthquake events is characterized by the strong nonlinear dynamic behavior for which transient time-history analysis is needed. Since in this case explicit relationships between the input basic design variables and the structural responses can not be established, it is not possible to derive explicit forms for performance limit states. The calculation of failure probabilities relies then on either a lot of recurring simulations or on formulating a priori assumptions about the form of failure surfaces.

The simulation based methodology is essentially that which relies on Monte-Carlo process [5]. It does not require any assumption about the shape of the failure surface. However, the calculation of maximum responses necessitates obtaining the complete nonlinear dynamic history for each given state event. This is a rather severe limitation as dynamic transient analysis of building structures is known to be in general computationally demanding. Furthermore, to get acceptable confidence in probabilities with required orders of accuracy a large number of runs are needed.

To reduce the computational cost associated with Monte-Carlo simulation based methods, approaches using response surface approximation in order to represent the dynamic analysis results were introduced. In conditions where the explicit approximation can be made to be sufficiently accurate, reliability estimation and the optimization task of performance based design can then be simply conducted. However, the response surface needs to be adjusted to data, and these have to be developed by a priori runs of dynamic analysis where relevant set of combinations of the intervening variables should be used. Although this condition may be, to some extent, computationally challenging, the obtained response surface given in terms of the basic design variables permits definitely quick and efficient reliability estimation.

The objective of this work is to investigate feasibility of the response surface approach methodology [5-11] to assess the reliability of performance based seismic design for irregular reinforced concrete buildings. The limit state regarding limitation of the maximum building roof displacement is considered and four factors are introduced in the analysis. Two of them are associated to the seismic demand while the two others are related to the building structural capacity. These factors include the seismic acceleration, the building mass, concrete members sections and steel reinforcements sections. A data set of results was obtained by using deterministic dynamic nonlinear computations which were performed according to a full factorial design of experiment table built on the four factors by choosing three levels for each one of them. Analysis of variance was conducted on the obtained results and had shown that seismic acceleration and concrete members sections govern to large extent the considered performance limit state in the range of parameters that were studied. The results were used after that to derive a global quadratic polynomial surface interpolating the building roof displacement over the entire chosen range of design variables. This surface was used to examine the effect resulting from a particular choice of distributions of probabilities to represent the random variables.

A case study consisting of a 3-story reinforced concrete irregular building was considered. The building was subjected to permanent loads and to seismic excitation having intensities that are likely to occur in the northern zone of Morocco.

II. NONLINEAR DYNAMIC REPSONSE ANALYSIS

Let u designate the global vector of generalized nodal displacements, then the dynamic equilibrium equation for the finite element discrete system modelling the building structure writes

$$M\ddot{u} + C\dot{u} + K_T \Delta u = R - F \qquad (1)$$

in which M, C, K are, respectively, the mass, the damping and the tangent stiffness matrices; $\ddot{u}$ and $\dot{u}$ are the global acceleration and velocity vectors at time $t+\Delta t$; Δu is the global displacement increment between times t and $t+\Delta t$; R is the vector of external actions at time $t+\Delta t$; and F is the vector of internal forces at t.

In the following, nonlinear dynamic response analysis of the building is performed by means of ZeusNL software package [12-14]. ZeusNL is open source software 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. The ZeusNL element library includes various element types that can be used to model structural elements such as beams or columns, non-structural elements like mass and damping and boundary conditions for supports and joints. Common concrete and steel material models are available, together with a wide choice of typical pre-defined steel, concrete and composite section configurations. The applied loading can include constant or variable forces, displacements or accelerations.

In dynamic analysis, non-structural mass and damping elements are added to the finite element model, and the dynamic equation of motion is solved by using one of the following explicit time integration schemes: Newmark (default) and Hilber-Hughes-Taylor. Modelling of seismic action in ZeusNL is achieved by introducing acceleration loading at the supports. The ability to employ different loading curves at each support allows for representation of eventual asynchronous ground excitation. Gravitational loads are included in the analysis as static initial forces, taking into account their effect on the internal stresses in each element and their corresponding plastic behavior in critical cross-sections.

In the present analysis, the 3D cubic elastic plastic beam-column element is used. This element gives a detailed inelastic modelling. It accounts for the spread of plasticity along the member length and across the section depth. To disretize masses, lumped mass element is used and no damping was considered.

The concrete behaviour was chosen to be described by the nonlinear concrete model with constant active confinement modelling (con2) [12]. 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 denoted k and 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 f_c, tensile strength f_f, crushing strain ε_{c0} and confinement factor k.

The reinforcement steel behaviour was assumed to be a bilinear elastic plastic model with kinematics strain-hardening (stl1) [12]. 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.

The iterative strategy employed for dynamic analysis during the solution procedure is determined by specifying:

- The maximum number of iterations to be performed at each increment.
- The number of initial reformations of the tangent stiffness matrix to be performed at each increment.
- The step reduction factor when convergence is not achieved.
- The iteration, at which checks of divergence are performed.
- The reference tolerance value used to check for divergence of the solution.

There are two different convergence criteria in ZeusNL. The first is based on the norm of the out-of-balance forces. Convergence is attained when the norm is smaller than the tolerance defined in Settings. The second criterion, which is the default, is based on the maximum iterative increment of displacements. These should be lesser than the specified displacement and rotation reference values.

III. PROBABILISTIC MODEL FOR RELAIBILITY PERFORMANCE SEISMIC ASSESSMENT

The dynamic response of a structure subjected to seismic excitation depends on a large number of variables. Some of these variables are associated to seismic demand in terms of the ground motion action while the others represent the part of the building structural behavior. All these variables exhibit uncertainty and are random in nature. However, all the parameters have not an equal effect on results. There are some variables which have strong effect on the limit state while the others have only rather marginal effects. To simplify the analysis in the context of seismic performance reliability assessment, only four factors are considered in the following. In fact, only two factors among them are found to monitor predominantly the performance state associated to limiting the building roof displacement.

Let us denote x the vector of basic design random variables. This vector include the peak ground motion acceleration denoted a, the mass factor denoted m, the concrete members section depth denoted c, and the steel reinforcement bars area denoted s. A single performance function $g(x)$ is considered in this work under the following form

$$g(x) = \delta_{lim} - \delta(x) \qquad (2)$$

where $x = [a \quad m \quad h \quad s]^t$ is the vector of design variables with the exponent t designating the transpose, δ_{lim} is the limit threshold displacement of the considered response being here the maximum building roof displacement $\delta(x)$.

The actual structural response $\delta(x)$ is the building roof maximum displacement as obtained through a complete nonlinear dynamic analysis performed by ZeusNL for some given design variables vector x. The structural response will be here approximately represented by an explicit polynomial function of x. The approximation is obtained through regression of the obtained results according to a full factorial design of experiment table constructed on the input variables x. A quadratic polynomial will be chosen.

The reliability probability for the limit state considered is obtained through evaluating the probability of the corresponding failure event $g(x) < 0$. This probability results from the multiple integral over the failure domain.

The main objective of this work is to compare reliability estimates for different distribution of probabilities describing the intervening random variable while the building dynamics is assumed to be represented by the response surface approximation. Three different distributions of probabilities are considered: Normal, Gumbel and Uniform. They are assumed to have the same means and the same standard deviations.

Reliability analysis is performed by using FORM analysis [2]. This method is based on two mean operations. Firstly, the design point in the transformed uncorrelated standard normal space is located. Secondly, the limit-state surface at this point is approximated by a hyper-plane and use is made of the properties of the standard normal space to obtain the probability estimate. The distance from the origin of the standard normal space to the design point is termed the reliability index and is denoted β. The first-order probability estimate is then found as $P_f = \Phi(-\beta)$ where Φ is the standard normal cumulative distribution of probabilities.

IV. Case Study

The 3D model of the building as built under ZeusNL software is depicted in figure 1. The building is a three-story irregular structure with three vertical plane frames at each side. It lays on the rectangular surface $9.2\ \text{m} \times 11.7\ \text{m}$. The height of stories is $3\ \text{m}$. To enhance irregularity of the building a steel column, represented in red colour on figure 1, was introduced in the first story. Its cross section is square and have the dimensions: $25\ \text{cm} \times 25\ \text{cm}$. All the other columns have square cross sections of depth $h \times h$. They are reinforced according to figure 2a. Figure 2b shows the cross section of beams having the T-form. They are all assumed to be equal.

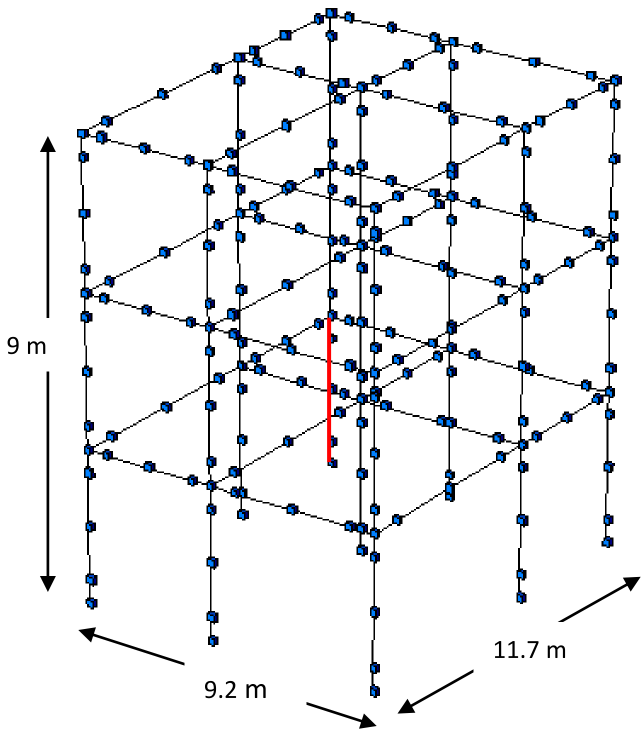

Figure 1: The finite element model of the studied irregular building

The material properties are assumed to be deterministic. The buildings columns and beams cross sections are assumed to be random and perfectly correlated among all the members. The depth of building structural members is assumed to be the same. Three levels were selected for this variable: $h = 25\,\text{cm}$; $h = 30\,\text{cm}$ and $h = 35\,\text{cm}$.

The steel reinforcement sections are also assumed to be random and perfectly correlated. They are proportional to the same reference section denoted s. This last was varied according to the following values: $s = 250\,\text{mm}^2$, $s = 300\,\text{mm}^2$ and $s = 350\,\text{mm}^2$.

The confined concrete properties are: $f_c = 25\,\text{MPa}$, $f_t = 2.5\,\text{MPa}$, $e_{co} = 0.002$ and $k = 1.2$. The unconfined concrete properties are the same but $k = 1.02$. Steel characteristics are: $E = 2 \times 10^{11}\,\text{MPa}$, $\sigma_y = 500\,\text{MPa}$ and $\mu = 0.01$.

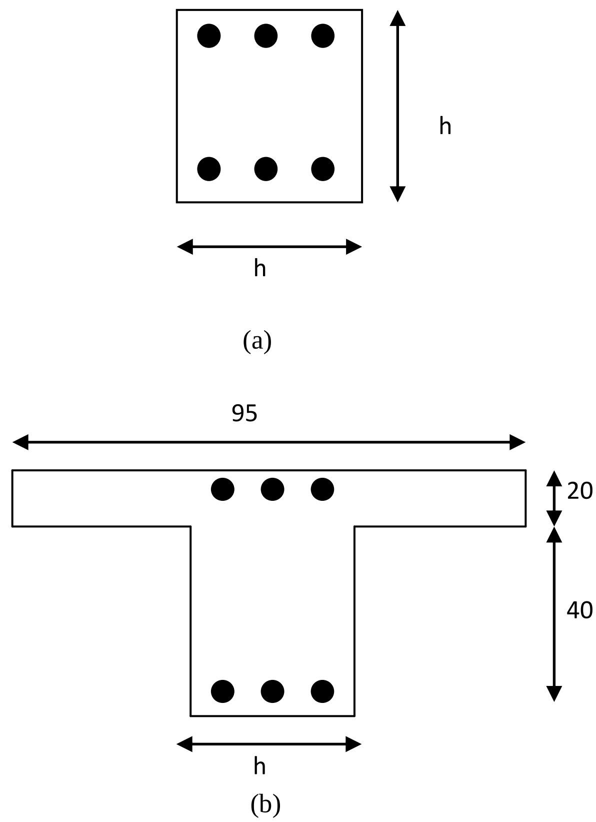

Figure 2: Members cross section (dimensions are in cm)

The seismic excitation is introduced in the analysis through using a scaled historical ground motion record: the Loma Prieta earthquake ground motion. This record is scaled to give the maximum seismic action expected to happen in the northern region of Morocco with a probability of exceedence higher than 50 years. Figures 3 and 4 give the ground motion acceleration scaled to the maximal acceleration $a = 1.57\ m/s^2$. During simulations intended to assess the influence of factors and to derive the response surface model, the maximum acceleration is assumed to have the following values $a = 1.18\ m/s^2$, $a = 1.57\ m/s^2$ and $a = 1.96\ m/s^2$. For Loma Prieta, the mean frequency value of ground motion accelerogram is $4.7\ Hz$.

To perform reliability analysis, the peak acceleration is assumed to be a random variable having the mean value $1.57\ m/s^2$ and the standard deviation $0.39\ m/s^2$.

The mean value of the total mass is obtained by considering the following combination $G + 0.2Q$, where G is the permanent load and Q the service load. The masses for all stories are assumed to be perfectly correlated. The mass scaled levels that were considered are: $m = 8$, $m = 10$ and $m = 12$. The total mass is assumed to be proportional to a random variable having the mean value 10 and the standard deviation 2.

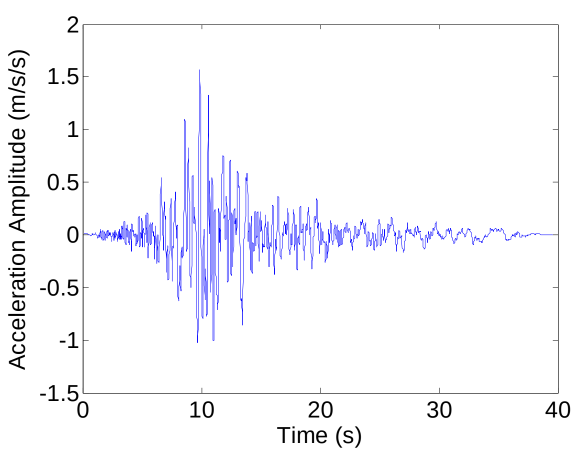

Figure 3: Loma Prieta scaled ground motion

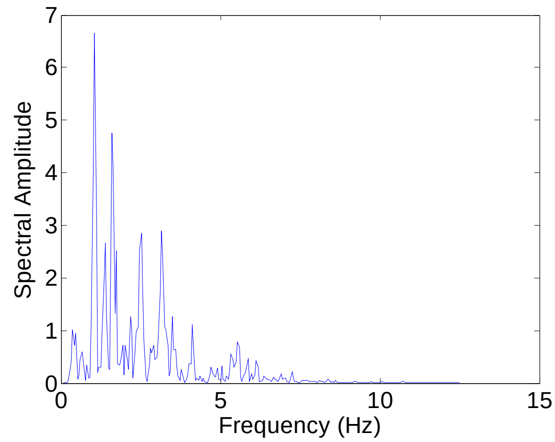

Figure 4: Loma Prieta frequency content

The limiting value δ_{lim} is considered to be deterministic. In this work, the threshold limit is fixed at the value $\delta_{lim} = 0.14\,m$. Other performance criteria can be introduced to distinguish performance-based engineering states with regards to earthquake events such as those defined according to the Federal Emergency Management Agency [15].

Using three levels for each design variable, 81 design points can be formed. Interpolating, in the mean square sense, the obtained maximum roof displacement results enables to determine the response surface which can be used to substitute the real dynamics of the building by a more simple metamodel. Accuracy of this model is tested by means of correlation coefficient, the obtained value of this coefficient in the present analysis is $R^2 = 0.84$. It should be noticed that the obtained response model is valid only within the intervals used to derive the model.

V. RESULTS

Table 1 shows the results obtained by performing analysis of variance on the 81 maximum displacements. It can be noticed that the seismic acceleration and concrete section depth are the major factor. They are followed by the interaction between mass and concrete section depth.

Using regression of the obtained results, the building roof displacement can be interpolated as:

$$\begin{aligned}\delta(a,m,h,s)=&2.3175\times10^{-2}+1.5532\times10^{-1}a\\ &-9.2465\times10^{-3}m-3.7348\times10^{-3}h+5.7147\times10^{-4}s\\ &+1.4993\times10^{-3}am-2.208\times10^{-3}ah-1.7236\times10^{-5}as\\ &+3.4472\times10^{-4}mh-4.5333\times10^{-5}ms-1.6333\times10^{-6}hs\\ &-8.6079\times10^{-3}a^2+3.6991\times10^{-4}m^2+1.6741\times10^{-5}h^2\\ &-2.6370\times10^{-7}s^2\end{aligned} \quad (3)$$

Giving the design point $a=1.57; m=10; h=30; s=200$ and the uncertainties means and standard deviations shown in table 2, reliability analysis can be performed by using Phimecasoft software [16].

Source	Sum Sq.	d.f.	Mean Sq.	F	Prob>F
a	0.044516	2	0.022258	126.278	0
m	0.0007242	2	0.000362	2.0544	0.1393
c	0.012779	2	0.006389	36.2503	0
s	0.0005506	2	0.000275	1.5619	0.22022
a*m	0.0001545	4	0.0000386	0.2191	0.92651
a*c	0.0009543	4	0.0002386	1.3535	0.26406
a*s	0.0003293	4	0.0000823	0.4671	0.75954
m*c	0.0024614	4	0.0006153	3.4911	0.01397
m*s	0.0009748	4	0.0002437	1.3827	0.25401
c*s	0.0003766	4	0.0000941	0.53413	0.71126
Error	0.0084605	48	0.0001763		
Total	0.072281	80			

Table 1: Analysis of variance of the obtained results

	a	m	h	s
Mean	1.57	10	30	200
Standard deviation	0.39	2	5	50

Table 2: Characteristics of the chosen random variables

The obtained results in terms of the Hasofer-Lind reliability index and probability of failure as function of the chosen distributions of probabilities for each factor are as follows:

a: Normal; m: Normal, c: Normal, s: Normal

$\Rightarrow \quad \beta=3.6414, \quad P_f=1.36\times10^{-4}$

a: Uniform; m: Uniform, c: Normal, s: Normal

$\Rightarrow \quad \beta=8.66, \quad P_f=2.36\times10^{-18}$

a: Normal; m: Normal, c: Gumbel, s: Gumbel

$\Rightarrow \quad \beta=1.31\times10^{10}, \quad P_f=0$

a: Gumbel; m: Gumbel, c: Normal, s: Normal

$\Rightarrow \quad \beta=6.3266, \quad P_f=1.25\times10^{-10}$

a: Normal; m: Normal, c: Uniform, s: Uniform

VI. Conclusion

Seismic performance of irregular buildings as affected by seismic action and building structural members geometry was investigated. Reliability analysis of the performance function associated to limiting the building roof displacement was assessed. This was performed by means of surface response approach and FORM method. Four factors were considered. The obtained results in terms of probabilities of failure had shown that these last depend hugely on the chosen distributions of probabilities that describe the uncertainties. It is not sufficient to know the mean and the standard deviation, identification of the density of probabilities is also required. The Normal distributions of probabilities had provided the most severe case.

AcknowledgmentsThe authors would like to thank the Spanish AECI Agency for its financial support of this research work under project grant

A/030882/10. They thank also the authors of ZeusNL who have provided freely this software package.

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