EXPLORING DIFFERENT SCALED GROUND MOTIONS AND A PROPOSED APPROACH TO COMPUTE SEISMIC RELIABILITY

A.D. García-Soto[(1)], M. Jaimes[(2)]

[(1)] *Researcher, Department of Civil Engineering, Universidad de Guanajuato, adgarcia@ugto.mx*
[(2)] *Researcher (Associate), Institute of Engineering, UNAM, mjaimest@iingen.unam.mx*

Abstract

This study explores the seismic reliability of engineering systems related to scaled ground motions and a proposed approach. The scaled records are commonly used in the standard IDA (Incremental Dynamic Analysis) approach to compute the seismic fragility of engineering systems. In this method, at a given site, a set of realistic or simulated motions are simply scaled at several fixed values of each intensity measure (IM, e.g., PGA, spectral acceleration, etc.); scaling may be as high as 20 times to evaluate seismic reliability of engineering systems at large return periods. On the other hand, to compute the seismic reliability, a proposal based on the idea of a method termed the combined approach (CA) is used. The reliability analysis is carried out for the maximum displacements of yielding 2DOF structures.

For the described approaches, the seismic reliability is discussed for two-degrees-of-freedom (2DOF) structures with and without control system for seismic protection at their first story during earthquakes in Mexico City. Two types of configurations with nonlinear behavior are examined: 1) yielding 2DOF structure simply supported on solitary columns and 2) 2DOF structure retrofitted with supplemental damper at its first story.

Key conclusions:

- The research demonstrates that seismic reliability of engineering systems with IDA-based approach lead to congruent results.
- The proposed approach is easy to implement and could be a simple alternative for estimating the seismic reliability.
- Finally, this investigation shows that the reliability of 2DOF structures retrofitted with supplemental damper at its first story, in terms of the maximum displacement of the first story, can be higher or lower than that of 2DOF structures simply supported on yielding solitary columns depending on the considered case.

Keywords: seismic reliability; reliability approach; scaled ground motions; 2DOF structures

1. Introduction

The structural response of systems subjected to seismic activity is a complex phenomenon which depends not only on the seismic hazard originated from several source mechanisms, magnitude of earthquake, site-to-source distance including path effects, site conditions among others, but also on the structural characteristics as type of structure, material, seismic protection systems, etc. If on top of all these related parameters, the uncertainty of each variable is taken into account, the computing of the seismic reliability of structures could prove to be quite a challenging task.

Methods to calculate the seismic reliability of complex systems are available in the literature; many of them are variants of the simulation technique (Monte Carlo simulation), carried out in such a way, that the simulations can be significantly reduced without compromising the precision too much. Other classical approaches can be adequate if certain conditions are met; for instance, when an explicit limit state function (LSF) can be stated, the first-order reliability method (FORM) can be quite efficient [1]. When an analytical closed-form expression cannot be used to represent the LSF, the combined approach (CA) can be employed [2].

On the other hand, to evaluate the seismic response associated to a set of return periods, scaling of the records may be required. The scaled records are used in the standard IDA (Incremental Dynamic Analysis) approach developed by Vamvatsikos and Cornell [3], which researchers often resort to, as a mean to build the seismic fragility of engineering systems. In the IDA, a given site is considered, and a set of realistic or simulated motions are simply scaled at given values of an intensity measure (IM) of interest to apply the scaled input signal to structural systems (e.g., two-degree-of-freedom systems, 2DOFs).

The main objective of this study is to compute the seismic reliability of 2DOF systems subjected to scaled ground motions associated to different return periods by using a proposed simple approach.

2. Ground motions used

The dynamic response of two-degree of freedom (2DOF) structures with seismic protection systems is very sensitive to the spectral pseudo-acceleration (SA), to the duration of the intense phase of the ground motion (i.e., the time window encompassing 5% to 95% of the Arias intensity) and to the frequency content, among other intensity measures (IMs). To capture the influence of the narrow band frequency content and the high spectral amplification of the ground motions on the 2DOF structures, a set of ground motions recorded at accelerometric stations installed in soft soils in Mexico City is used in this study.

The available strong ground motion data are grouped in two subsets of 32 ground motions depending on the soil period Ts for each accelerometric station: i) Group 1 (G1) – 2.21% (9 of 408 stations) with $1<Ts\leq1.5s$ (i.e., dominant motion frequency approximately of 0.8 Hz) and ii) Group 2 (G2) – 41.91% (171 of 408 stations) with $1.5<Ts\leq2s$ (i.e., dominant motion frequency approximately of 0.57 Hz). The G1 and G2 groups are representatives of the zones with higher observed damages during the 19 September 2017 Mw7.1 and 19 September 1985 Mw8.1 earthquakes, respectively. Figure 1 shows the SA response spectra for both groups (i.e., G1 left or G2 right, depending on soil period Ts). In Figure 1 average soil periods in the ranges of 1.04-1.49s (i.e., high-frequency content) and of 1.51-2s (i.e., low-frequency content) for groups G1 (left) and G2 (right), respectively, are shown.

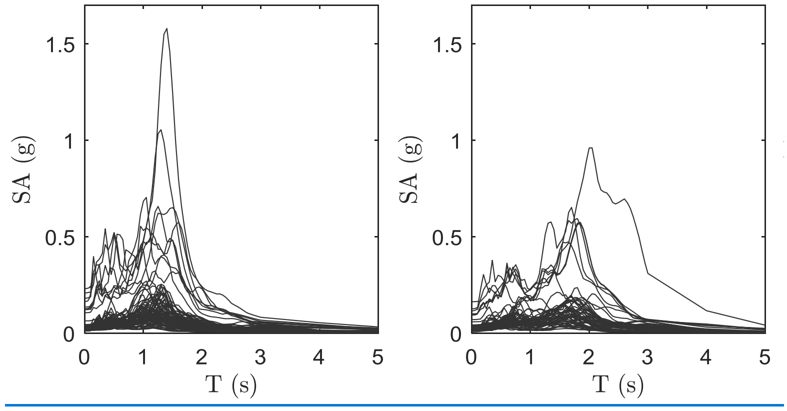

Figure 1. Pseudo-acceleration spectra (SA) of the input ground motions considered for the reliability analysis in Mexico City for two groups depending on the soil period Ts. Group G1 (left) and Group G2 (right).

3. Case study: 2DOF Structures

In this study two structural configurations are considered and shown in Figure 2: 1) a 2DOF structure simply supported on solitary columns (Figure 2a) and 2) a 2DOF structure with a supplementary damper in the first floor (Figure 2b). For all cases, the structure is excited along the horizontal axis. For the 2DOF supported on solitary columns (i.e., weak first floor) and 2DOF structures with supplementary damping in the first floor, the structural periods were adjusted to 1 s and 2 s. Also, for all the analysis the structures mass ratio was set to $\gamma=m_2/(m_1+m_2)=0.5$, m_1 and m_2 are the masses of the first and second story, T_1 and T_2 are the nominal periods, $\xi_1=0.05$, y $\xi_2=0.05$ denote nominal damping ratios; in this study $T_2=0.2$ s and T_1 takes different values.

The 2DOF structure with supplementary damping at the first floor can be considered as a retrofitted structure (e.g., the systems supported on solitary columns retrofitted) which has been added a damper in the first floor. For such a system, it is established that the supplemental damping provided by the damper is $\xi_d=0.2$. This means that $\xi_1 = \xi_c + \xi_d = 0.05+0.2=0.25$.

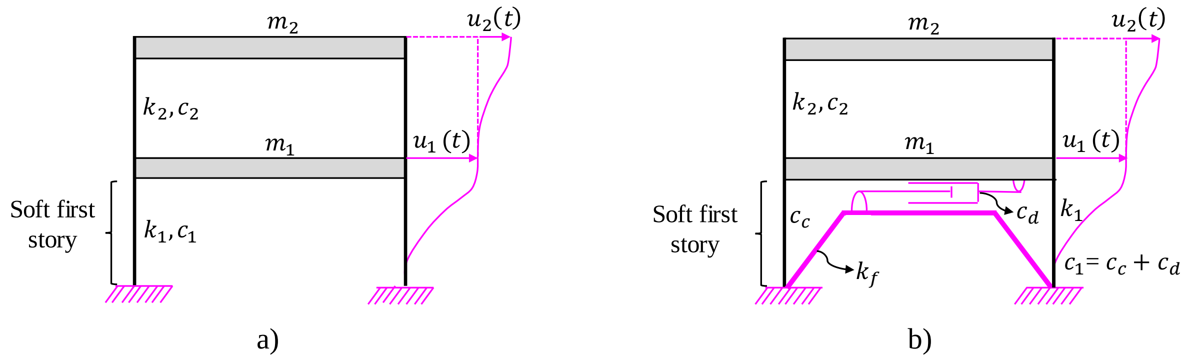

Figure 2. 2DOF structure configuration subjected to ground motions: (a) System A - 2DOF structure simply supported on solitary columns and (b) System B - 2DOF structure with a supplementary damper in the first floor.

In the following sections the structures in Figures 2a and 2b are referred to as System A and System B, respectively.

4. Reliability analysis

4.1 Characterizing the demand and capacity in probabilistic terms

To characterize the demand the following main simplification is assumed: the structural and dynamic properties of the considered systems are deterministic, being the randomness in the demand only given by a set of considered seismic records that the systems are subjected to. In other words, the response of the system (displacements, shears, etc.) resulting from the dynamic analysis for a given record is considered as a sample point. Then, the set of demands imposed to the system by each seismic record is treated as a random variable, which is probabilistically characterized using probability papers and a fitting method (e.g., least squares method, method of maximum likelihood, etc.) [4]. Moreover, the set of maximum responses are considered as sample points from independent and identical distributed random variables. This is performed for an intensity measure (IM) given as a fraction of the gravity acceleration, *g*, which is the scaling factor used in the IDA to obtain the responses. Depending on the considered site (characterized in terms of the soil period, Ts), the IM is associated to an exceedance rate, which can be translated into a return period in years, T-yr, to which the reliability of the system is related to.

To illustrate the previously described strategy, consider the systems and ground motions of the previous sections. In particular, for a system on solitary columns (System A) with a structural period T=1 s and a scaling factor of 0.2g (i.e., IM=0.2g), the maximum displacements, d_{max}, for each of the 32 considered records are plotted in several probability papers and shown in Figure 3. For the scope of this study, d_{max} corresponds to the first-floor maximum displacement. The selected site has a soil period Ts=1s; this means that the structure is highly vulnerable, since the structural period matches the soil period.

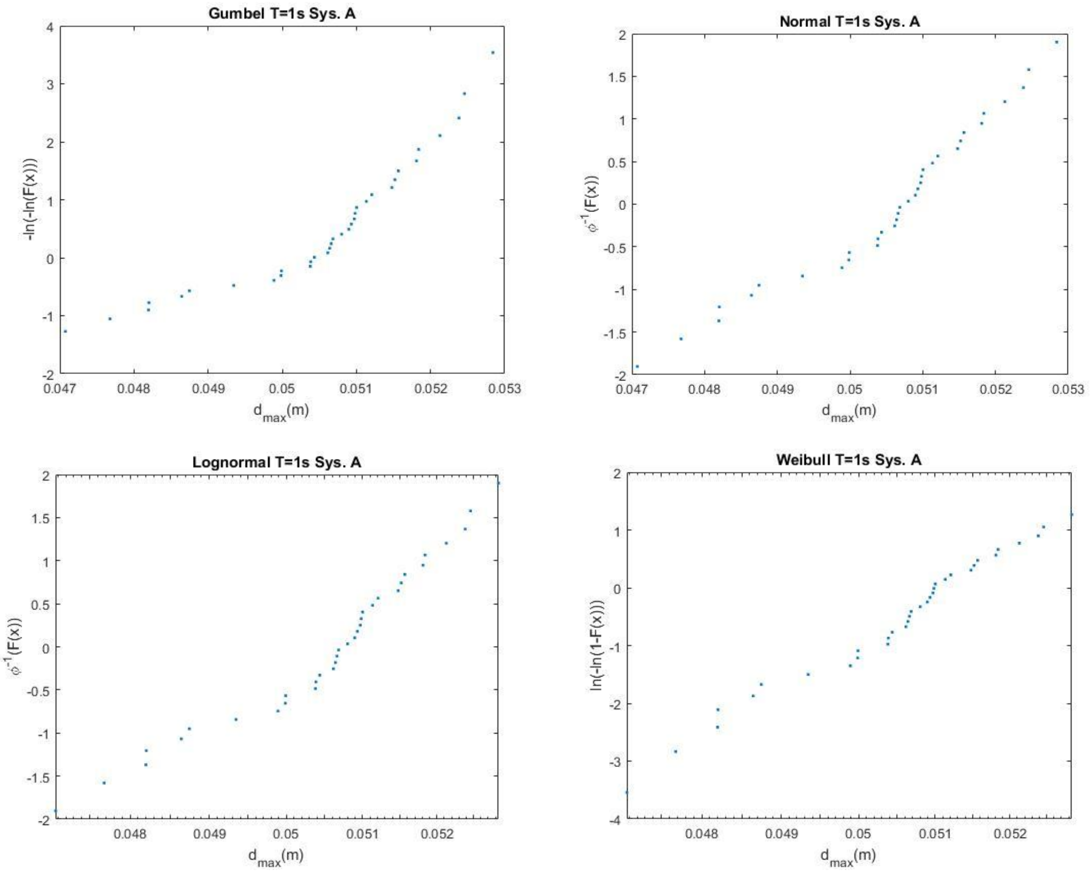

Fig. 3 – Probability papers for d_{max} (System A, T=1s at site with soil period Ts=1s)

By applying the method of maximum likelihood (MML) and the Akaike Information Criterion (AIC) [5], the lognormal distribution is the best fit for the data in Figure 3. Nevertheless, other distributions are competing options, especially the Weibull distribution but also the normal distribution. For the scope of this paper, the Lognormal distribution is adopted and used, whose probability and cumulative density functions (PDF and CDF, respectively) are defined as

$$f_Y(y) = \frac{1}{\sqrt{2\pi}\sigma_x y} e^{\left(-\frac{1}{2}\left(\frac{lny - ln\,m_y}{\sigma_x}\right)^2\right)} \quad (1)$$

$$F_Y(y) = \Phi\left(\frac{lny - m_x}{\sigma_x}\right) \quad (2)$$

where the two parameters of the distribution are determined by using the MML. Using the data in Figure 3, Eq. (1) and the MML, the fitted Eq. (2) is shown in Figure 4 in lognormal probability paper. The fitted distribution in Figure 4 is to be considered as the demand (in probabilistic terms) for the reliability analysis shown later.

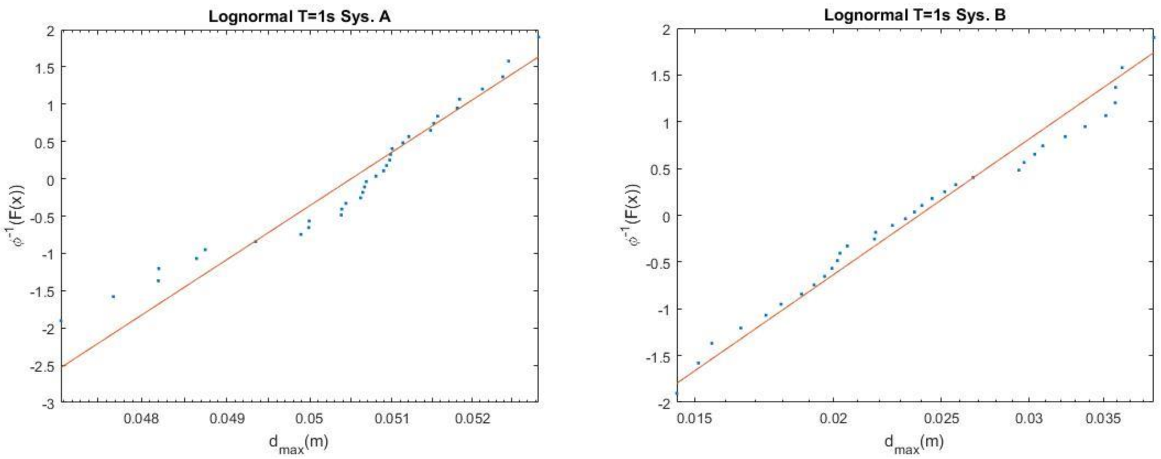

Fig. 4 – Fitted Lognormal distribution for d_{max}. Left plot: System A, T=1s. Right plot: System B, T=1s.

In Figure 4 it can be observed that for System B (also for IM=0.2g), T=1s and d_{max} the Lognormal distribution does not fit as well the upper tail as for the analogous case of System A.

Although not shown, other PDFs could be used instead (e.g., the Weibull PDF); this should be investigated in future studies, since the fitting could be sensitive to the system, the site, the demand and other parameters. For the contained cases within the present study, the Lognormal distribution is used to characterize the demand; future research is recommended to assess the impact of using different PDFs in the reliability analysis.

The capacity is extracted from codified design, in terms of the allowable drift in RDCF-2017 [6] (i.e., the Mexico City seismic provisions); assumptions include that the code value is considered as the mean value, that this parameter is defined by a Normal PDF with coefficient of variation (cov) equal to 0.02 and that this drift capacity is considered time-independent and uncorrelated to the demand for the reliability analysis in the next section.

It should be noted that the structures considered here could be assigned a seismic performance factor (RCDF-2017) Q=3. Note also that in the RCDF-2017 two allowable lateral distortions are stipulated; one is related to the maximum allowable drift to avoid a high probability of collapse (an ultimate limit state) and is equal to 0.020 for Q=3; the other one is linked to frequent seismic events to avoid damage in non-structural members and is equal to 0.004 (this may be considered as a serviceability limit state); only the former is considered for the scope of this study using an allowable drift D_{allow}=0.020. In this paper the structures in Figure 2 have heights of 3 m and 2.4 m for the first and second floors, respectively; this leads to a maximum allowable displacement to avoid a high probability of collapse Δ_{allow}=0.06 m; as mentioned, this value is assumed as the mean value of the capacity for the reliability analysis, which together with the stated limit state function (LSF) are used to report the results in the next section. Therefore, the results could be considered as the implicit reliability levels if the structures are seismically designed as per the RCDF-2017.

5. Limit State functions and results

Even though the reliability analysis of dynamic systems can be quite complex, our proposal borrows an idea from a simplified method termed as combined approach [2], with some modifications. Unlike the originally proposed CA, instead of using the point estimate method (PEM; e.g. [7, 8, 9 and 10]) to define the capacity of the system (which is often a function of several parameters implicitly defined), here the same idea is followed for the demand, omitting for the scope of the present paper the PEM (future studies are planned to used it for the parameters of the demand), and just assuming that the demand (e.g., the maximum displacement, d_{max}) is characterized by the fitted distributions in Section 4.1. This allows to set the LSF for d_{max} as

$$g = \Delta - D \tag{3}$$

where Δ is a normally distributed lateral displacement with the statistical moments defined in the previous section (i.e., the capacity), and D represents the demand characterized for the previously defined Lognormal distribution of d_{max}. This means that the LSF is explicitly represented by Eq. (3); thus, several reliability approaches can be used. In this study the first order reliability method (FORM) [1] is used.

Results for the systems in Figure 2 indicate that the reliability index, β, is equal to 5.00 for System A and IM = 0.2 *g*, while it is equal 3.43 for System B and the same IM level, and which are associated to T-yr equal to approximately 22 and 11 years for System A and System B, respectively (i.e., to exceedance rates for the corresponding considered sites of around 1/22 and 1/11, respectively). For IM > 0.2 *g* System B has much larger βs than system A; however, reliability indices for increasing IMs are too low, which implies the systems would collapse under the imposed seismic demand from larger T-yr. This is expected for System A, since it was deliberately chosen to be highly vulnerable. However, the supplemental damping provided for System B does not very effectively improves the seismic response, but in few cases. It is also noteworthy, that for these (and actually all) cases the cov for System B is significantly larger than that of System A (cov=0.280 and cov=0.077, respectively); this is paradoxical, since the intention of providing supplemental damper is to mitigate the response and improve the reliability of the system, which may be achieved in average, but additional uncertainty in introduced, which in turn usually leads to higher probabilities of failure. Naturally, this conclusion must be considered with caution, since in practice the selection of passive energy dissipation devices depends on several factors. It is also pointed out that from the statistical analysis it was found that, while the mean response is shifted to increasing values for increasing IM, the cov remains practically constant when the IDA is considered; consequently, it would be desirable in future studies to inspect whether other ground motion selection criteria could lead to variations in cov as a function of IMs.

For site with soil period Ts=2, both, System A and System B, lead to so low reliability levels, that probability of collapse is very likely; this is also expected for System B, since the structural period is T=2s for this system.

In order to inspect whether a change in T would impact the reliability levels, the reliability index is computed for Systems A and B, but this time using structural periods T=0.5 s and 1.5 s. For structural period T=1.5s some improvement in the response (i.e., larger β) is gained, but only for IM ≤ 0.15 g, especially for System B.

Results for structural period T=0.5s are reported in Table 1, together with associated T-yr; only meaningful β values (as compared with usual values for codified design) are shown; i.e., too large or too small reliability indices are excluded.

Results in Table 1 indicate that moving the structural period away from the matching soil period Ts is effective in decreasing the response (and increasing β), that for IM ≤ 0.55 *g* higher reliability index is obtained for System A and the opposite occurs for IM > 0.55 *g* and that, as expected, β decreases with increasing T-yr.

If the reliability associated to a given T-yr was of interest, the IM could be simply matched to such a reference period for the IDA, and the corresponding reliability can be computed with the procedure described here. It is reminded that if different sites are considered (with different Ts), the IM associated to T-yr will differ. As an example, consider that RCDF-2017 specifies T-yr of 250 years for collapse; by inspecting Table 11, an approximate IM=0.55 *g* will be required for the return period considered in the regulations, leading to reliability indices of around 5.6 and 5.2 for systems A and B, respectively. These βs can be though as the implicit reliability level in the Mexico City standards for the described conditions.

If a β=3.5 is desired (reference value commonly used for code calibration tasks), approximate values of IM = 0.65 *g* and 0.75 *g* would be required to meet the target for System A and System B, respectively.

Although not carried out in this study, an analogous procedure to that described for d_{max} can be easily extended to other demands (e.g., base shear) and/or serviceability limit states. If the global system reliability is desired, at least the lower bound can be obtained by extending the proposal here to several ultimate LSFs and taking the minimum obtained reliability index.

Table 1 – Reliability indices and return periods for d_{max}, both systems and soil period Ts = 0.5s

Scaling factor (IM) (g)	**Associated T-yr (Years)**	**Reliability index β**	
		System A	**System B**
0.5	209.6	6.59	5.73
0.55	249.7	5.60	5.23
0.6	294.5	4.70	4.77
0.65	344.3	3.87	4.35
0.7	399.5	3.11	3.96
0.75	460.5	2.39	3.60
0.8	527.8	1.73	3.26
0.85	601.8	1.10	2.94
0.9	683.0	0.51	2.64

6. Conclusions

In this study the seismic reliability of two-degree-of-freedom systems with and without seismic protection, located at differed types of soils, is computed. The ultimate limit state is casted in terms of the maximum displacements of the systems. The employed records cover a wide range of frequencies and are scaled using the incremental dynamic analysis; therefore, the reliability associated to certain return periods can be obtained. A simple method to compute the reliabilities is proposed; it is based in characterizing the demand as a single random variable, and so is the demand, but based on codified design; in this way, the limit state function was stated in explicit form and the reliability index computed by applying the so-called first-order reliability method. The main findings of this study include that:

1) When the structural period matches the soil period the systems are highly vulnerable, implying collapse, as the reliability index is too low in such cases.
2) Results indicate that moving the structural period away from the matching soil period leads to a decrease in the response (and in the probability of failure), that higher or lower reliability index is obtained for the system without additional damper depending on the considered case and that the probability of failure increases with increasing return period.
3) In all cases investigated, the coefficient of variation of the maximum displacement associated to the system with additional damping, is significantly larger than that of the system supported by solitary columns. This implies a reduction in the average response but the introduction of additional uncertainty.
4) From the statistical analysis was also found that the mean response is shifted to larger values for larger intensity measures, but the coefficient of variation remains constant when the incremental dynamic analysis is used; future research is recommended to inspect whether other selection criterion may lead to a varying coefficient of variation as a function of return period.
5) If reliabilities for given return periods are of interest, the proposed method can be used; consequently, it could be used as an aid in code calibration tasks. It is noted that the reliability associated to a return period differs from site to site.
6) An analogous procedure to that described in this paper can be easily extended to other demands and limit state functions.

7. Acknowledgements

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