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Reliability of lattice dome with and without the effect of using small number of ground motion records in seismic design

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ABSTRACT

Linear and nonlinear seismic responses can be estimated using the time history analysis for given ground motion records. To reduce the computing time, design codes prescribe guidelines to select a small number of ground motion records to perform the analysis and to estimate the seismic design demand. However, the assessment of the statistics of the seismic design level by using a small number of record components and the evaluation of the failure probability of the designed structures in such a manner are unavailable. The assessment and evaluation focused on the lattice dome are presented in this study. The results indicate that the use of average response from seven ground motion record components for design can lead to over- or under-estimation of the seismic design effect, the frequency of underestimation is about 50%. The variability of the seismic design effect estimated by using seven selected record components is considerable. By carrying out simulation analysis with 1000 trials, the minimum underestimation and the maximum overestimation of the seismic design demand are 35% and 64%, respectively; the estimated failure probabilities of the dome designed by using average response from seven record components can be about half an order of magnitude greater or smaller than that of the dome designed without the effect of small sample size. This suggests that to reduce the observed relative differences in the failure probabilities, an increased number of ground motion record components needs to be used in seismic design.

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1. Introduction

Seismic response of linear and nonlinear structural systems can be estimated using the time history analysis for selected ground motion records. The responses of interest may include the maximum displacement, drift ratio, base shear, stress and ductility demand. Since the seismic ground motion is a stochastic process, the responses of multi-degree-of-freedom (MDOF) system cannot be predicted deterministically even for the records that match specified spectral acceleration (SA) or peak ground acceleration (PGA) value given in design codes. To reduce the computing time but without sacrificing significantly the accuracy in estimating seismic design demand, the seismic design codes (e.g., [1–5]) prescribe general guidelines to select a limited number of ground motion records required to perform time history analysis for calculating the responses. Although the codes do not provide very specific characteristics of the records, loosely speaking, they require that the selected records should "match the design spectrum", and the average or envelop of responses for the selected records are to be used to check the adequacy of the designed structures. The record selection is further complicated by the consideration of the record scaling [6–9], and the combination of magnitude, site-to-source distance, and record-to-record variability identified through the deaggregation of seismic hazard and risk [10,11].

According to GB50011 [1,2], if three records are used, the envelop responses of the time history analysis results or the larger responses obtained by using the modal combination and spectrum analysis method need to be considered. If seven or more than seven records are considered, the average of the responses obtained from the time history analysis for all the selected records is to be used. For lattice shell structures, the Chinese code [3] follows the recommended practice in GB50011 [2], and the records need to be adjusted to match the design spectrum which is defined based on a "standard" response spectrum multiplying the peak ground acceleration (PGA) that corresponds to 50-year return period value. This design spectrum is employed for design considering the linear elastic behaviour alone and without reduction due to

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ductile behaviour or overstrengthening. The designed structure must be checked for collapse using the design spectrum which is scaled by the peak ground acceleration corresponding to 2475-year return period value.

The commentary to NBCC [4] recommends that typically seven or more records which are compatible with the design response spectrum should be used to accommodate the uncertainty associated with selecting ground motions for analysis. The design spectrum is defined using the uniform hazard spectrum with 2% of exceedance probability in 50 years (i.e., 2475 years return period values). The ASCE/SEI-7-10 [5] stipulates that at least three appropriate ground motions shall be used in the analysis, and that the historical records should be selected from records of events having magnitudes, source-to-site, and source mechanisms that are consistent with those that control the maximum considered earthquake.

In all cases, the imposed minimum number of records is based on engineering experience [12–14]. The number of records required to obtain an estimate of the median response of interest to within a defined the response due to the use of scaled records [6]. The investigation in [12] attempted to investigate how to select and scale the records, and how many records need to be used for the nonlinear time history analysis. For their analysis, three reinforced concrete frames designed according to Mexico City's design code are considered and 14 Mexican ground motion records are employed. The criterion adopted for selecting the number of records considers that the estimated median responses of interest are within a specified confidence interval for a considered estimation error. However, the application of confidence interval in such a manner to select the number of records does not ensure or imply reliability consistency. The analysis carried out in [14] is for single-degree-of-freedom (SDOF) systems using 30 records from seven shallow crustal earthquakes in California and Japan. Again, the effect of using a small number of records on the structural reliability is not estimated.

One of the most efficient space structures that can experience seismic excitations is the light weight single-layer reticulated dome. It is often built for sports stadiums, gymnasiums, and auditoriums; its dynamic characteristics differ from those of tall building and bridges because of its geometry. For example, its natural vibration frequencies are closely spaced, and dynamic analyses of this type of domes under uniform and nonuniform (i.e., spatially varying) seismic excitations are given in several studies [15–20]. By considering the options of three or seven records recommended in [2,3], the analysis results shown in [19,20] showed that the use of the average response estimated from seven records is preferable than the use of the envelop response from three records since the former is associated with less bias and lower scatter. However, this conclusion is based on linear elastic response only and the implied reliability of the design structure was not assessed.

This study is focused on the single-layer reticulated dome. The main objectives are to estimate the reliability of the dome designed using average response of many records so the small sample effect can be ignored, to evaluate the reliability of the dome designed using seven ground motion record components as stipulated by codes, and to compare the estimated reliability of the dome with and without the effects of using small number of ground motion records for design. The comparison can be used to judge the adequacy of using seven ground motion record components in design practice. Although the responses of the dome could be sensitive to spatially varying incoherent three-directional excitations, only unidirectional uniform excitation is considered for it is the first attempt to address the problem of the structural reliability with/without the effect of using small number of ground motion records for design.

2. Seismic hazard, records and structural model

2.1. Seismic hazard, and intensity measure

The most influential seismic hazard assessment procedure was presented in [21,22] (see also [23]). The analysis basically incorporates the information on seismic source zones, magnitude-recurrence relations and ground motion attenuation relations (i.e., ground motion prediction equations (GMPEs)) to estimate the seismic hazard in terms of the ground motion intensity measures such as the PGA and SA. One of the outcomes of the analysis is the probability distribution of the PGA and/or SA, which can be considered to be lognormally distribution [11,24,25], at least in the upper tail region. By assuming that the annual maximum PGA is lognormally distributed, and by considering that the PGA values tabulated in [2] for the "frequent earthquake" and "rare earthquake" correspond to probability of exceedance of 63% and 2% $\sim$ 3% in 50 years, the estimated coefficient of variation (cov) of the PGA for different "seismic fortification intensity" defined in [2] is shown in Table 1. Note that the "seismic fortification intensity" is related to the seismic hazard zonation. The table shows that the cov ranges from 1.67 to 3.77 and varies for different locations since the seismic fortification intensity varies from region to region. Since the design SA by considering linear elastic behaviour is obtained by multiplying the standardized response spectrum, a factor of 2.25, and the PGA for frequent earthquake, it is considered that the cov shown in Table 1 is also applicable to the SA. As will be seen, the knowledge of the probabilistic characterization of the ground motion intensity measure such as the PGA and SA are of importance for estimating structural reliability under seismic excitations.

The procedure described in [26], which is referred to as the SAC procedure, uses the displacement or drift ratio in establish limit state function, and considers that the displacement and drift ratio can be expressed as functions of a ground motion intensity measure, where the most appropriate intensity measure in terms of sufficiency and efficiency seems to be the SA [27]. The SA is also used in [28] as the intensity measure but the limit state is based on the ductility demand instead of displacement or drift ratio; they also established the equivalence between the SAC procedure and the ductility based procedure. In all cases, it is considered that the SA is lognormally distributed. The consideration that the SA and PGA can be modeled as lognormal variates is also made in the present study with a cov within the values shown in Table 1.

2.2. Selected historical ground motion records

Since the available ground motion records in the Chinese ground motion database for a specified combination of magnitude and source-to-site distance ranges are very limited, the records in

Table 1
Inferred coefficient of variation of annual maximum PGA.

Seismic fortification intensity	6	7	8	9
PGA for Frequent earthquake (cm/s^2)	18	35	70	140
PGA for rare earthquake (cm/s^2)	125	220	400	620
Estimated cov, if the value of the PGA for rare earthquake is considered to corresponds to 2% in 50 years	2.93	2.57	2.28	1.67
Estimated cov, if the value of the PGA for rare earthquake is considered to corresponds to 3% in 50 years	3.77	3.25	2.84	2.00

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