SEISMIC DESIGN CRITERIA OF FIRE PROTECTION SYSTEMS FOR DOE FACILITIES

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ABSTRACT

Fire protection systems are critical to the safety of personnel and to the protection of inventory during any kind of emergency situation that involves a fire. The importance of these fire protection systems is hightened for DOE facilities which often house nuclear, chemical or scientific processes. Current research into the topic of "fires following earthquakes" has demonstrated that the risks of a fire starting as a result of a major earthquake can be significant. Thus, fire protection systems need to be designed to withstand the anticipated seismic event for the site in question.

The majority of the commercial and DOE facilities in the United States designed their fire protection systems to their applicable building codes. These codes typically rely on the National Fire Protection Association (NFPA) standards for their seismic design requirements. The NFPA seismic requirements are very limited and do not address all of the attributes which cause fire protection systems to fail during an earthquake. The Loma Prieta earthquake which rocked the San Francisco Bay area in 1989 provided numerous instances wherein relatively newly designed/installed fire protection systems (designed to current NFPA codes) failed during the earthquake. Research into the causes of these failures reveals that a limited number of changes and enhancements to the NFPA requirements would have alleviated the majority of these failures. The purpose of this paper is to document a seismic evaluation and design procedure which addresses the failure modes which have arisen during recent earthquakes. This criteria is more extensive than the current NFPA code requirements, but is much simpler than ASME code requirements for safety related piping systems in nuclear power plant applications. Thus, the authors have attempted to bridge the two undesirable options of either using the NFPA criteria which is known to have shortcomings, or using ASME code criteria which is prohibitively expensive and time consuming.

DETERMINATION OF EFFECTIVE ACCELERATION FOR USE IN DESIGN AT THE LAWRENCE LIVERMORE NATIONAL LABORATORY SITE*

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ABSTRACT

An rms-based effective acceleration study has been conducted for the Lawrence Livermore National Laboratory. The study used real time history records with epicentral distances, magnitudes and site conditions deemed appropriate for the LLNL Livermore site. Only those records having strong motion durations, T_D', >3.0 seconds, and peak ground accelerations $\geq$ .4g were selected for determining the effective acceleration hazard curve used in design. These parameters are consistent with LLNL's use of the broad-band Newmark-Hall Spectra for design, and the high peak instrumental accelerations corresponding to the return intervals of interest. Study results were used to modify the acceleration hazard curve for facility design/evaluation at LLNL.

INTRODUCTION

The Lawrence Livermore National Laboratory (LLNL) contracted with Geomatrix, Inc. to update the seismic acceleration hazard curve for the LLNL site, [1]. As a corollary to this effort, Geomatrix was also tasked with developing "effective" accelerations for the LLNL site using the approach outlined by Kennedy *et al.*, [2], as suggested in Kennedy *et al.*, [3].

The purpose of this paper is to summarize the justification for the use of "effective" accelerations for design, the technical basis of the approach used, and the results of the Geomatrix evaluations.

BASIS OF EFFECTIVE ACCELERATION APPROACH

The most commonly specified parameter used in the design of facilities subject to earthquake ground motions is the peak ground acceleration. It is frequently used in conjunction with "standard" response spectra shapes to form design spectra. However, many factors play a significant role in determining the actual response and damage experienced. Some of the more significant of these factors include frequency content of the motion; the duration of strong ground-shaking; soil-structure interaction effects; and the inherent reserve energy capacity (ductility) of the structure.

Experience has shown that many structures located close to the epicenters of earthquakes, and subjected to ground motions which yield high peak instrumental accelerations, have sustained much less damage than would be expected. Furthermore, the reductions in foundation motions resulting from the effects of soil-structure interaction (SSI) are more dramatic when associated with near-source ground motion of short duration and high peak instrumental acceleration. These observations have led investigators to seek a more realistic approach than the use of high peak instrumental accelerations as the primary parameter for seismic design, particularly for structures whose seismic risk is dominated by earthquakes with short epicentral distances.

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Historically, however, the development of site-specific seismic design criteria has concentrated on defining peak instrumental accelerations representing the absolute maximum peak acceleration, A_{max}, recorded by strong motion accelerometers at the free ground surface. Many studies (e.g., Hoffman, [4]; Page *et al.*, [5]; Housner, [6], [7]; Housner and Jennings, [8]; Newmark, [9]; Blume, [10]; Nuttli, [11]; Kennedy, [12]; Kennedy *et al.*, [2]) have demonstrated that this parameter is a poor measure of the damage potential of earthquakes, and can be excessively conservative when used to scale broad-band spectra for use in predicting elastic response, particularly in connection with near-source motions (less than 20 km) having high peak instrumental accelerations.

Peak instrumental accelerations fail to adequately account for the fact that a limited number of high-frequency, high-acceleration spikes, of limited energy content, have little effect on the amplified elastic response spectra within the frequency region of 1.8 to 10 Hz (Kennedy *et al.*, [2]). Thus simply scaling a broad-band design spectrum (such as RG1.60 or Newmark-Hall) by the peak instrumental acceleration can lead to excess conservatism in this frequency range. To eliminate this excess conservatism, many investigators have developed a variety of approaches for defining an effective acceleration which is then used to anchor a broad-band design spectrum in place of the peak instrumental acceleration. The intent of the effective ground acceleration approach is to scale down the broad frequency design spectrum such that, "on the average", the broad frequency design spectrum will predict about the same elastic response as would be predicted by the actual spectrum for structures in the frequency range of 1.8 to 10 Hz. This frequency range is also the frequency range of interest for the majority of facilities at the LLNL site.

TECHNICAL APPROACH USED

A great deal of research (e.g., Blume, [10]; Schnabel and Seed, [13]; Ploessel and Slosson, [14]; Newmark, [15]; Kennedy, [12]) and a wide variety of techniques have been suggested to define effective accelerations for use in anchoring elastic design spectra. This research has led to a number of recommendations for defining effective accelerations, including the use of sustained or repeatable peak accelerations (Nuttli, [11]); equivalent cyclic motion (Whitman, [16]); filtered time histories (Page *et al.*, [5]; Ploessel and Slosson, [14]); and the "third highest peak" approach (Kennedy, [12]).

Other approaches have looked at energy content and strong motion duration parameters for defining effective accelerations (e.g., Arias, [17]; Housner, [6]). Still others have suggested the root-mean-square acceleration, rms, as the parameter of interest in defining effective accelerations (Mortgat, [18]; McCann and Shaw, [19]).

Use of the rms acceleration as the basis for determining an effective acceleration, A_{eff}, has advantages from a computational and statistical point of view. First, the rms acceleration is an easily computed quantity once a definition for strong motion duration, T_D, is accepted. Second, Mortgat, [18], has shown that use of an rms acceleration parameter, A_{rms}, enables an effective acceleration to be selected at any desired probability of exceedance.

The approach employed is probabilistic in nature and is based on the premise that an approximate relationship must exist between A_{max} and A_{rms}. Using this premise, the effective acceleration for design is related to the rms acceleration by:

$$A_{eff} = (K_p)\, A_{rms} \qquad (1\text{-}1)$$

where K_p is a function derived from the theory of stationary Gaussian random motions which provides a prediction of the most probable value of the factor A_{max}/A_{rms}, during a known time interval, for a specified probability of exceedance level. Thus, the computed value of A_{eff} from Eq. 1-1 is equivalent to the expected or most probable value of the maximum acceleration for the time history record over the time interval of interest. Vanmarcke and Lai, [20], have shown that considering the effective acceleration as a median estimate of the most probable maximum acceleration over the duration of strong motion, T_D, for a stationary Gaussian random motion, K_p is:

$$K_p = \sqrt{2 \ln (2.8\, T_D/T_O)} \qquad (1\text{-}2)$$

except K_p is not less than $\sqrt{2}$, and T_O is the predominant period of the ground motion. Kennedy, [12], has reported that Eqs. 1-1 and 1-2 work well for defining an effective acceleration to which elastic response spectra can be anchored.

In a study of effective acceleration parameters for use with broad-band design spectra, Kennedy *et al.*, [2], modified Eqs. 1-1 and 1-2 as follows:

$$A_{eff} = \sqrt{2 \ln (2.8 T_D' \Omega')}\,(A_{rms}) \tag{1-3}$$

where A_{eff} is an effective elastic rms-based anchor acceleration for scaling broad-band design spectra.

T_O of Eq. 1-2 has been replaced in Eq. 1-3 by $T_O = 1/\Omega'$ where Ω' represents the central, or mean, frequency (Hz) of the time history record, and T_D' is defined as:.

$$T_D' = T_M - T_{0.05} \tag{1-4}$$

$$\text{where } T_M = \max \begin{Bmatrix} T_{0.75} \\ T_{pa} \end{Bmatrix}$$

and $T_{0.75}$ and $T_{0.05}$ represent the time at which 75% and 5%, respectively, of the total energy, as measured by $\int a^2 dt$, have been reached. If the time T_{pa} of the maximum positive or negative ground acceleration occurs after $T_{0.75}$, then the time Tpa of the first zero crossing after the time of the peak acceleration is used in lieu of $T_{0.75}$. The remaining factor, A_{rms}, is the square root of the Housner power (average rate of energy input) over the time interval T_D':

$$\text{i.e., } A_{rms} = \sqrt{\frac{\Delta E}{T_D'}} \tag{1-5}$$

where ΔE is the cumulative energy between time $T_{0.05}$ and T_M.

Equation 1-4 was used in the 1984 study by Kennedy *et al.*, [2], as it provided a better definition of the relative length of strong duration for stiff structures than the more commonly accepted definition of strong motion duration as given by Trifunac and Brady, [21]:

$$T_D = T_{0.95} - T_{0.05} \tag{1-6}$$

Equation 1-6 can result in strong motion durations from 2 to 3 times as long as that given by Eq. 1-4. This is due to the fact that many records contain long durations of motion with small accelerations at the end of the record. This portion of the record continues to input energy but at a much reduced rate than earlier portions of the record, but is nevertheless a significant contributor in the strong motion duration calculation when Eq. 1-6 is used.

The determination of A_{rms} is strongly influenced by the selection of the strong motion duration interval, and use of Eq. 1-6 can result in unconservatively low values of A_{rms}, which in turn leads to unconservatively low spectral amplification values in the frequency range of 1.8 to 10 Hz for broad band frequency spectra scaled to A_{eff}, as given by Eq. 1-3.

In Kennedy's 1984 study, [2], 11 real time history records were evaluated for effective accelerations using Eq. 1-3. It was shown that Eq. 1-3, when used to scale the broad-band RG1.60 spectrum, provided a reasonably good fit to the 6 real ground motion records having a strong motion duration, T_D', greater than 3.0 seconds.

However, the use of Eq. 1-3 as a scaling parameter for RG1.60 to the remaining 5 real time history records having strong duration times, T_D', < 3.0 seconds, did not provide a good fit (generally, excessive conservatism is introduced). The primary reason for these differences appear to relate to the lack of sufficient frequency content and cumulative power in the five records having a $T_D' < 3.0$ seconds.

The conclusion of the study was that broad-band frequency content spectra anchored to an "effective" acceleration, as given by Eq. 1-3, provided an acceptable basis to characterize ground motion records which contain significant power between at least 1.2 to 5.5 Hz (i.e., 80% of the cumulative power of the strong motion portion, T_D', of the record), and are appropriate for use with stiff structures (1.8 to 10 Hz). For flexible structures (<1.8 Hz), the approach given is conservative. Furthermore, a strong correlation was shown to exist between T_D' and the breadth of the frequency band of significant power. All records with $T_D' > 3.0$ seconds contained significant power throughout the frequency range from 1.2 to 5.5 Hz, while none of the records with $T_D' \leq 3.0$ seconds had significant power in this frequency range.

EFFECTIVE ACCELERATIONS FOR THE LLNL SITE

For the LLNL Livermore site, an effective acceleration scaling factor, A_{eff}/A_{max}, was determined using the approach developed by Kennedy *et al.*, [2], where A_{eff} is as given in Eq. 1-3 and A_{max} is the unmodified peak instrumental acceleration. The study to determine this factor was conducted for LLNL by Geomatrix, Inc., [22].

The peak instrumental accelerations for the design return intervals of interest (500, 1000, and 5000 yrs.) are quite high (i.e., .557, .682, and .983 g, respectively). As such, a subset of all potential time history records was selected. The records used were from corrected and uncorrected recordings obtained at small source-to-site distances from earthquakes in the range of magnitude 5.6 to 7. These records reflect deep firm alluvial sites, similar to conditions at LLNL, and are consistent with the data used to derive the empirical attenuation relationships used in the probabilistic seismic hazard analysis for LLNL. Furthermore, the effective acceleration scaling factor was derived using only records having peak ground accelerations (corrected or uncorrected) $\geq .4g$ and strong motion duration intervals, T_D', >3.0 seconds, as these two additional parameters are considered appropriate in view of the use of the broad-band Newmark-Hall response spectra shape for design at LLNL, and the aforementioned high peak instrumental accelerations. Twelve records were found which satisfied all of the above parameters (Table 1).

For each of these records, the effective acceleration scaling factor, A_{eff}/A_{max}, was determined considering both the corrected and uncorrected values of A_{max} for each record. Using this approach, a total of 23 scaling factors were determined that conformed to the requirement that $A_{max} \geq .4g$ and $T_D' >$ 3 sec. (one record had a corrected value for A_{max} of <.4g and as such, this value was not used). The average value of the uncorrected and corrected scaling factors of these records was 0.83.

Both uncorrected and corrected values were considered since two of the attenuation relationships used in the probabilistic hazard analysis (Geomatrix, [1]) are based on "uncorrected" data (Idriss, [23], and Sadigh *et al.*, [24]) and two are based on "corrected" data (Campbell, [25], and Joyner and Boore, [26]). The average effective acceleration scaling factor of 0.83 was used to modify the acceleration hazard curve developed for the LLNL Livermore site (Geomatrix, [1]), to produce an effective acceleration hazard curve (Fig. 1). The appropriate method to do this is to adjust each of the attenuation relationships by the appropriate A_{eff}/A_{max} scaling factor and recompute the combined hazard. However, it was found that an accurate estimate of this curve could be obtained by simply multiplying the abscissa of the instrumental acceleration hazard curve by the average A_{eff}/A_{max} scaling factor of 0.83.

As a check on the reasonableness of the computed effective accelerations, plots of the response spectra of the real time histories were compared to the Newmark-Hall spectra anchored at the "effective" acceleration. This comparison showed the fits to be quite good.

CONCLUSIONS

The Lawrence Livermore National Laboratory has selected the effective seismic hazard curve of Fig. 1 for use in facility design/analysis at the Livermore site conforming to the guidelines presented in UCRL-15910.

For active operational components of equipment (e.g., relays, breakers, motors, etc.) which must remain functional during a seismic event to maintain safe operating conditions or safe shut-down in moderate hazard and high hazard facilities, an additional factor of 1/.83 will be applied to seismic loads produced by use of the effective accelerations. This additional factor reflects the fact that these active components might be more sensitive than structures to peak instrumental accelerations.

Table 1. Selected Free Field Strong Motion Recordings.

Earthquake/Date/Station	Mw	R	Comp	A1	A2	T_D'	Ae	Ae/A1	Ae/A2
Imperial Valley, CA 1979/10/15	6.5								
5054 Bonds Corner		2.5	S50W	0.778	0.785	4.66	0.685	0.880	0.872
			S40E	0.596	0.587	6.11	0.507	0.851	0.864
5165 Differential Array		5.3	NOOE	0.487	0.486	3.02	0.477	0.979	0.980
942 El Centro Array #06		1.0	S50W	0.456	0.436	3.91	0.406	0.891	0.931
			S40E	0.459	0.376	5.27	0.338	0.737	0.900
952 El Centro Array #05		1.0	S40E	0.560	0.530	3.40	0.404	0.721	0.762
955 El Centro Array #04		4.2	S40E	0.489	0.493	3.40	0.389	0.797	0.790
958 El Centro Array #08		3.8	S40E	0.619	0.610	3.01	0.433	0.700	0.710
Westmorland, CA 1981/04/26 11369 Westmorland Fire Sta.	5.6	13.3	SOOE	0.500	0.475	3.64	0.430	0.860	0.906
Whittier Nar., CA 1987/10/01 24400 LA: Obregon Park	6.0	13.9	S90W	0.450	0.407	3.06	0.351	0.780	0.863
Loma Prieta, CA 1989/10/17 57382 Gilroy Array #04 58065 Saratoga:14675 Aloha	7.0	16.1 13.0	360 360	0.416 0.504	0.416 0.504	3.80 3.74	0.323 0.373	0.776 0.740	0.776 0.740

Mw Moment magnitude
R Source-to-site distance (km)
Comp Component
A1 Uncorrected peak acceleration (g)
A2 Corrected peak acceleration (g)
T_D' Effective duration
Ae RMS-based effective acceleration (g)
Ae/A1 Ratio of effective acceleration to uncorrected peak acceleration
Ae/A2 Ratio of effective acceleration to corrected peak acceleration

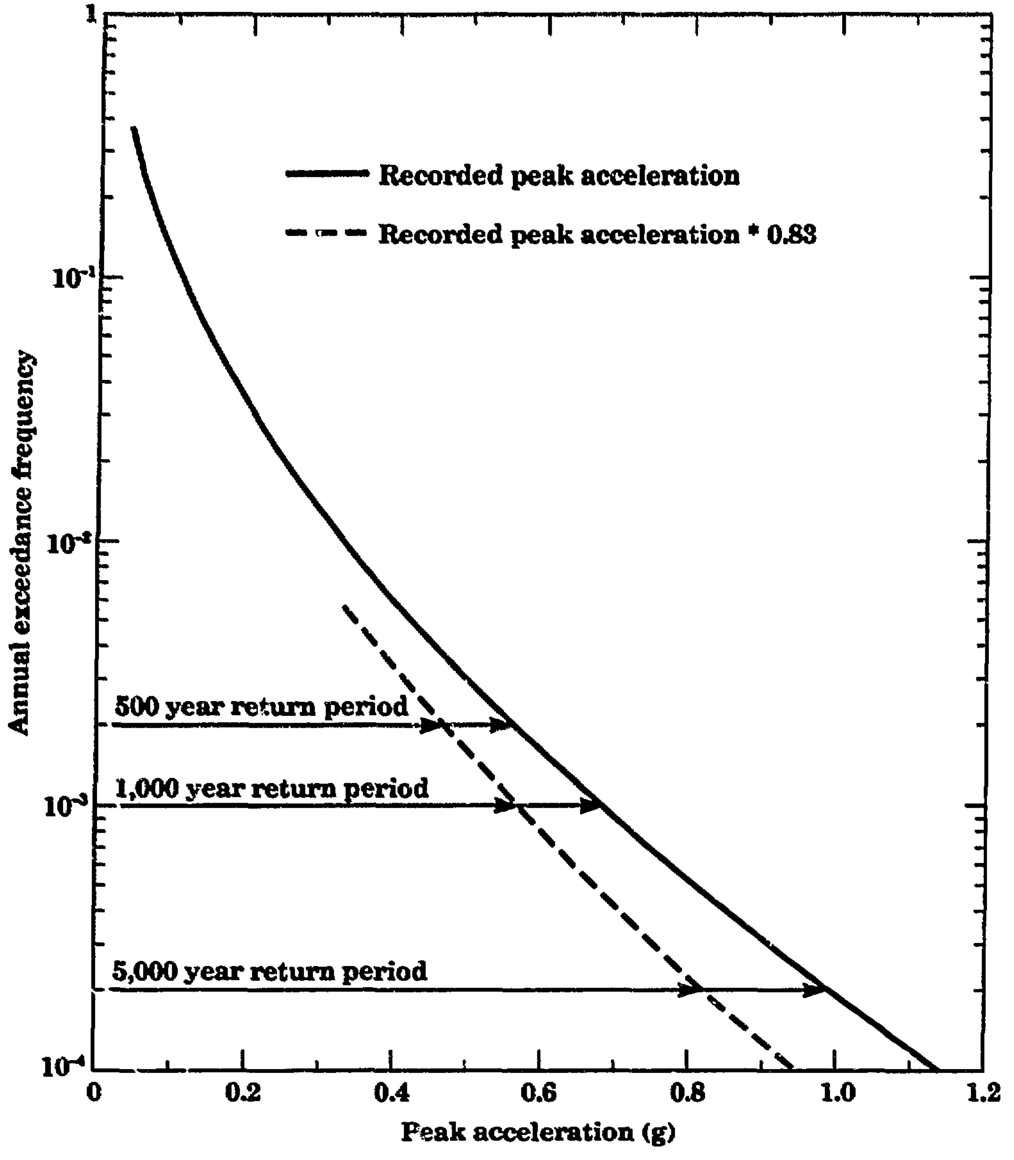

Fig. 1. Peak acceleration hazard curve and effective acceleration hazard curve (i.e., peak acceleration × 0.83), for the LLNL Livermore site.

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