

Effective Ground Motion Considerations for Nuclear Power Plant Design

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Abstract

A brief overview of U.S. research on the characteristics of earthquake ground motion which cause structural damage is presented. Areas covered include; 1) breadth of frequency content; 2) effective ground acceleration; 3) number of strong nonlinear response cycles and strong motion duration and; 4) soil-structure interaction. Instrumental peak acceleration and corresponding elastic response spectra are not suitable measures of structural damage and the above effects should be considered in defining design earthquake parameters.

1. Introduction

Nuclear power plant structures tend to be massive, stiff structures with fundamental frequencies in the 2 to 10 Hz range depending on soil conditions. Whereas damage to conventional structures probably correlates best with velocity content of the ground motion, the seismic vulnerability of nuclear power plants is probably more vulnerable to acceleration content. For this reason, the ground motion input for seismic designs and seismic margin reviews, as well as for seismic probabilistic risk assessments, is generally expressed in terms of ground motion acceleration.

It has often been noted, particularly in connection with near-source motions due to low-to-moderate magnitude earthquakes, that structures have performed much better than would be predicted considering the free-field instrumental peak acceleration to which the structures were subjected. In such cases, the differences in measured ground motion, design levels, and observed behavior is so great that it cannot be reconciled with typical safety factors associated with elastic seismic analyses for design even when typical ductility considerations are included [1]. The problems with free-field instrumental peak acceleration and free-field instrumental-based elastic response spectra as measures of potential seismic damage are threefold. First, the foundation motion of massive stiff structures is often substantially less than that recorded by a free-field instrument due to soil-structure interaction (SSI) and spatial variation of ground motion effects. Secondly, a limited number of high-frequency spikes of high acceleration but very short duration have little effect on the elastic response spectra within the region of interest. Thirdly, structure damage is related to structures being strained into the inelastic range in which the breadth of the frequency content of the actual record plays a very important role because of frequency shifts with inelastic response. Also, inelastic response and damage is likely to be influenced by the duration of strong motion or the number of cycles of strong response. A stand-

and elastic spectrum ignores these effects.

This paper presents a brief overview of U.S. research on the above-mentioned problems concentrating on recent U.S. NRC-sponsored work [2-6]. Many important references are omitted and the reader is advised to consult the above references for a broader reference list.

2. Importance of Breadth of Frequency Content

The breadth of the frequency content has a substantial influence on both elastic and inelastic response [3] and on the importance of SSI effects [4, 5]. This breadth can be described by a mean frequency Ω and the frequency range (f_{10} to f_{90}) over which 80 percent of the power of the input motion is distributed. The mean frequency can be defined in terms of the zero and second moments (λ_0 and λ_2) of the power spectral density function, $G(f)$ by [8]:

$$\Omega = \sqrt{\lambda_2/\lambda_0} \quad ; \quad \lambda_i = \int_0^{\infty} f^i G(f) df \quad (1)$$

Figure 1 shows the Cumulative Power Spectral Density function (obtained by integrating $G(f)$ over frequency) for the 1952 Taft accelerogram. f_{10} and f_{90} are the frequencies at which 10 and 90% of the cumulative power are reached. Table I presents these parameters for some representative earthquake records as well as for an artificial R.G. 1.60 record.

It has been found [3] that the R.G. 1.60 spectrum when anchored to an effective acceleration, A_D , can be used to slightly conservatively predict both elastic and inelastic response of stiff structures (2 to 10 Hz) for all those records in Table I with broad frequency content in which $f_{10} < 2.0$ Hz, $f_{90} > 5.0$ Hz, and Ω lies between 3.0 and 4.8 Hz (Records 1-3 and 5-7 in Table I). The other 5 records (Records 8-12) in Table I have frequency content sufficiently dissimilar to that of the R.G. 1.60 spectrum that this spectrum cannot be used to adequately predict either elastic or inelastic response throughout the frequency range from 2 to 10 Hz no matter what value of effective acceleration is chosen for anchoring the R.G. 1.60 spectrum. The concept of an effective anchor acceleration, A_D , cannot be used to compensate for severely dissimilar frequency breadth.

Figure 2 shows the 7% damped response spectra shapes for the artificial R.G. 1.60 record and three real records all anchored to 1g zero period acceleration (ZPA). The El Centro #5 record is typical of the broad frequency content records in Table I. Parkfield is devoid of frequency content above about 3 Hz and has a low mean frequency. Structural responses from such records could only be approximated by a design or evaluation spectrum with a mean frequency between 1.8 and 3.0 Hz and with $f_{90} \leq 3.2$ Hz. On the other hand, the Melendy Ranch record has high mean frequency and is devoid of frequency content below 2.5 to 3.5 Hz. Such records could only be approximated in the range from 2 to 10 Hz by a design or evaluation spectrum with a mean frequency between 5 and 8 Hz and with $f_{10} > 2.5$ Hz.

3. Effective Ground Acceleration

Many investigators (e.g., [1, 2, 3, 12]) have pointed out that the instrumental peak acceleration, A_I , is a poor indicator of elastic response spectrum values. The spectral accelerations, S_a , are primarily influenced by the energy contained within a number of cycles of ground motion and are little influenced by a few spikes of very high acceleration. Blume [1] has shown that clipping the highest 30% off the measured acceleration-time history (using only 70% of the record, in an absolute sense, closest to the zero line) produced only about a 5% reduction in the elastic response spectrum for frequencies up to 10 Hz. Other investigators have reported similar findings which has led them to recommend that

the design or evaluation response spectrum be anchored to an effective acceleration, A_D , based on the sustained or repeatable peak acceleration rather than A_I .

An approach is to define the effective acceleration in terms of the rate energy fed into structures. Arias [13] and Housner [14] have demonstrated that $E(T'_D)$, as illustrated in Figure 3, can serve as a measure of the cumulative energy per unit mass fed into all single-degree-of-freedom oscillators over strong motion duration, T'_D . The average rate of energy input (power), P , and root-mean-square (rms) acceleration, a_{rms} , are then given by:

$$P = \frac{E(T'_D)}{T'_D} \quad ; \quad a_{rms} = \sqrt{P} \quad (2)$$

Several investigators (e.g., [3, 7]) have suggested that because a_{rms} describes the average power over time T'_D , it provides substantially more information about the influence of the record on structural response than does A_I . Use of a_{rms} enables an effective design acceleration, A_{DE} to be selected at any desired probability of exceedance during the time history. The design acceleration is related to rms acceleration by:

$$A_{DE} = K_P * a_{rms} \quad ; \quad K_P = \sqrt{2 \ln(2.8 T'_D \Omega)} \text{ except } K_P \geq \sqrt{2} \quad (3)$$

where K_P is a factor which is a function of the acceptable exceedance probability for each individual peak of the time history. Considering the design acceleration as that which is expected to occur once on the average over the strong motion duration for a stationary random Gaussian motion, Vanmarcke and Lai [9] have determined the above expression for K_P .

For stiff structures, only the portion of the record with maximum power significantly influences either elastic or inelastic response [3]. Therefore, the duration of interest is only the strong motion duration, T'_D , associated with the duration of maximum power. Different investigators [3, 7, 10] have proposed differing techniques for defining T'_D . One method [3] for defining T'_D is the duration from $T_{0.05}$ to T_m where $T_{0.05}$ is the time at which 5% of the cumulative energy is reached and T_m is either the time $T_{0.75}$ at which 75% of the cumulative energy is reached or the time of the first zero crossing after both the maximum positive and maximum negative instrumental accelerations are reached, whichever is later in time. Table I presents duration, T'_D , and effective design acceleration, A_{DE} , estimates [3] as well as the instrumental peak acceleration, A_I for 11 real earthquake records.

Studies have been performed [3] to determine the adequacy of using a broad-frequency content design spectrum (R.G. 1.60) anchored to A_{DE} to predict response of stiff structures from actual earthquake records. Table II presents the maximum, minimum, and median values of the ratio of spectral acceleration from the actual record to spectral accelerations from R.G. 1.60 anchored to A_{DE} over the frequency range from 1.8 to 10 Hz. Elastic ($\nu = 1.0$) and two levels of inelastic ($\nu = 1.85$, and 4.27) response ratio results are presented. Inelastic results are in terms of inelastic spectral accelerations (i.e., the spectral acceleration for which the structure would have to be designed to be at the onset of yield in order to achieve a given ductility, μ , from the time history record).

A review of Table II indicates that for the 6 records (1-3, 5-7) that have frequency content similar to the R.G. 1.60 spectrum (see Table I), the R.G. 1.60 spectrum anchored to A_{DE} does an adequate job of predicting the required design spectral accelerations for both elastic and inelastic response. The maximum factor of unconservatism is only 1.3 and the

maximum factor of conservatism is about 2.0 (i.e., 1.0/0.49) for both elastic and inelastic responses. On the average, only slight conservatism is introduced. However, for the 5 records (8-12) that have frequency content substantially different than the R.G. 1.60 spectrum, the R.G. 1.60 spectrum anchored to A_{DE} does not provide a universally adequate prediction of responses in the frequency range from 1.8 to 10 Hz. For the low-frequency content records (Goleta and Parkfield), the use of the R.G. 1.60 spectrum generally introduces excessive conservatism but in the case of inelastic response its use can introduce excessive unconservatism within narrow frequency bands. For the high-frequency content records (Coyote Lake, Gavilan College, and Melendy Ranch), the use of the R.G. 1.60 spectrum produces completely inconsistent results with factors of conservatism as high as 10.0 (1.0/0.10) and factors of unconservatism as high as 1.6.

4. Influence of Number of Strong Nonlinear Response Cycles and Strong Motion Duration

Structural damage correlates with the maximum inelastic deformation reached and is a function of the number of strong response cycles to this level of deformation. Most structural elements can undergo one nonlinear response cycle to a greater ductility level than would be permissible for five to ten equal nonlinear response cycles. For this reason, some investigators (e.g., [10]) have suggested the use of total hysteretic energy absorbed as a measure of damage rather than ductility level reached or maximum deformation. To maintain an approximately constant level of total hysteretic energy absorbed would require:

$$N(\mu-1) = \text{Constant} \quad (4)$$

where N represents the number of strong nonlinear response cycles.

The number of strong nonlinear response cycles, N, correlate with the strong motion duration T'_D . Kennedy [3] reports N of 1 to 2 for records with $T'_D < 7$ seconds, and 3 to 4 for $T'_D > 9$ seconds. Zahrah [10] reports N generally ranges from 2 to 3 for $T'_D < 2$ seconds and 3 to 5 for $T'_D > 5$ seconds. Thus, short duration records produce significantly less strong nonlinear response cycles than do long duration records and based upon eq. (4) one would expect that a structure could undergo at least 1.5 times greater ductility for an equal amount of damage from these short duration records than from the long duration records.

In addition to the above, duration also tends to indirectly influence the amount of nonlinear deformation that a structure undergoes. Nonlinear response depends significantly on the average spectral acceleration over a frequency range to the soft side of the elastic frequency because the structure gradually softens during nonlinear response [3]. For stiff structures with a broad frequency content input spectrum, spectral averaging has little influence since the spectral accelerations do not radically change as the structure softens (see Figure 2). However, short duration records tend to have narrow frequency content (see Table I). For short duration records with high-frequency content, this spectral averaging effect can dramatically reduce the nonlinear response because the average spectral acceleration is likely to be much less than the elastic spectral acceleration. On the other hand, short duration records with low frequency content show average spectral accelerations as stiff structures soften which are greater than the elastic spectral acceleration so that nonlinear response is increased.

5. Soil-Structure Interaction Effects

Soil-structure interaction (SSI) effects tend to reduce the input motion at the foundation of large, massive, stiff buildings below that of the free-field ground surface. These effects include: 1) lesser frequency for the soil-structure system than for the structure

alone, 2) substantially increased system damping for the combined system, 3) horizontal spatial averaging of the input motion over the mat dimensions, and 4) vertical spatial variation of the ground motion for structures with embedded foundations (kinematic interaction).

Horizontal spatial variation of input motion results from horizontal traveling wave effects or statistical incoherence of the ground motion. Newmark [12] demonstrated that reductions of about a factor of 2 between foundation and free-field motions were consistent with the assumption that the foundation size reduced the motion by averaging traveling wave effects over its dimension. He proposed that traveling wave effects could be conservatively approximated by reducing the effective design acceleration by a factor R given by:

$$R = (1-5\tau) \geq 0.67 \quad (7)$$

where τ is the travel time across the mat. However, subsequent research [6] using data from differential arrays has indicated much higher apparent horizontal wave speeds even on soil sites than had been envisioned by Newmark for which this τ -effect becomes negligible in reducing the foundation motion.

Due to the lack of complete coherence of the free-field motion, mat foundations of large structures would be subject to the average motion over their dimension which would be reduced from the free-field motion. The reduction due to base-averaging effects increases with an increase in 1) foundation dimension, 2) input motion frequency, and 3) inhomogeneity of the subsurface geological conditions. At a relatively homogeneous site for a foundation width of 165 feet, this averaging effect is negligible below 5 Hz and results in reductions of about 10% at 10 Hz, and 15 to 25% in the 20 to 30 Hz range [6]. Note that there is only limited data pertaining to the evaluation of statistical incoherence.

Greater reductions appear to result from SSI effects which lower system frequency and increase damping plus kinematic interaction for embedded foundations. A comparison of foundation motions in a large number of buildings with and without basements during the 1971 San Fernando earthquake shows consistently less motion in buildings with basements, often by a factor of 2 to 3, particularly at frequencies above 8 Hz [8]. Similarly, large reductions of foundation motion were reported at the Humboldt Bay Nuclear Power Plant during the 1975 Ferndale earthquake, the Pleasant Valley Pump Station during the 1983 Coalinga earthquake, the Fukushima Nuclear Power Plant, and a large belowground LNG tank in Japan. Japanese, Richmond Field Station, and USGS Menlo Park downhole array data also show reduction of ground motion with depth [6]. Such reductions in foundation motion can be computed by SSI analyses which incorporate kinematic interaction for embedded foundations [4, 5, 6, 11]. Figure 4 compares computed free-field and foundation horizontal response spectra for the foundation of a large, stiff building with a foundation size of about 280-feet by 140-feet and with both no embedment and 40-feet embedment [11]. The surface founded structure input motion is substantially reduced from the free-field motion for frequencies between 5 and 20 Hz. With 40-feet of embedment, even greater reductions occur at all frequencies above 2 Hz.

6. Conclusions

Realistic and not excessively conservative estimates of structural response require an appropriate characterization of the motion input to stiff structures. Such a characterization should incorporate the beneficial reduction in foundation motion due to soil-structure interaction and kinematic interaction effects. Excessively conservative design evaluations which indicate such effects might substantially increase foundation motions over the free-field motion do not agree with observations and should be avoided. At frequencies greater

than about 8 Hz, horizontal spatial averaging effects should also be considered. The design or evaluation input motion must adequately represent the frequency content (breadth and central frequency) of real input motions which might be expected. An appropriate frequency content design or evaluation spectrum should be anchored to an effective design acceleration, A_{DE} rather than the instrumental peak acceleration. This effective design acceleration should be based on the rms-acceleration, a_{rms} , in order to account for the energy content of actual records. Lastly, the design or evaluation input motion should contain strong motion duration, T'_D , associated with the maximum power of the record.

7. References

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TABLE I: COMPARISON OF FREQUENCY DOMAIN, DURATION, AND ACCELERATION PARAMETERS

| | EARTHQUAKE RECORD (COMPONENT) | FREQUENCY RANGE (Hz) | | MEAN FREQ. $\bar{\omega}$ (Hz) | T'_D (sec) | A_I (g) | RMS BASED A_{DE} (g) |
|----|--|-------------------------|----------|-----------------------------------|-----------------|--------------|---------------------------|
| | | f_{10} | f_{90} | | | | |
| 1 | Olympia, WA., 1949 (N86E) | 1.20 | 6.10 | 3.90 | 15.6 | 0.281 | 0.202 |
| 2 | Taft, Kern Co., 1952 (S69E) | 1.10 | 5.50 | 3.61 | 10.3 | 0.180 | 0.155 |
| 3 | El Centro Array No. 12 Imperial Valley, 1979 (140) | 0.55 | 7.50 | 4.52 | 9.6 | 0.142 | 0.133 |
| 4 | Artificial (R.G. 1.60) | 0.60 | 6.55 | 3.91 | — | — | — |
| 5 | Pacoima Dam, San Fernando, 1971 (S14W) | 0.75 | 6.70 | 4.19 | 6.1 | 1.170 | 0.795 |
| 6 | Hollywood Storage PE Lot, San Fernando, 1971 (N90E) | 0.75 | 7.90 | 4.68 | 5.4 | 0.211 | 0.213 |
| 7 | El Centro Array No. 5 Imperial Valley, 1979 (140) | 0.80 | 6.75 | 4.12 | 3.4 | 0.530 | 0.404 |
| 8 | UCSB Goleta Santa Barbara, 1978 (180) | 0.80 | 3.05 | 2.34 | 3.0 | 0.347 | 0.332 |
| 9 | Gilroy Array No. 2 Coyote Lake, 1979 (050) | 2.70 | 6.90 | 5.12 | 2.2 | 0.191 | 0.202 |
| 10 | Cholame Array No. 2, Parkfield, 1966 (N65E) | 1.20 | 2.75 | 2.34 | 1.4 | 0.490 | 0.514 |
| 11 | Gavilan College Hollister, 1974 (S67W) | 2.55 | 11.35 | 7.67 | 1.1 | 0.138 | 0.106 |
| 12 | Melendy Ranch Barn, Bear Valley, 1972 (N29W) | 3.55 | 8.20 | 6.11 | 0.8 | 0.520 | 0.435 |

TABLE II: COMPARISON OF ACTUAL SPECTRAL ACCELERATIONS TO R.G. 1.60 SPECTRAL ACCELERATIONS ANCHORED TO "EFFECTIVE ACCELERATION", A_{DE}

| | Earthquake Record (Component) | $(\frac{S_A}{S_{A_{1.60}}})^*$ | | | | | | | | |
|----|---|--------------------------------|--------|------|--------------|--------|------|--------------|--------|------|
| | | $\mu = 1.0$ | | | $\mu = 1.85$ | | | $\mu = 4.27$ | | |
| | | MAX. | MEDIAN | MIN. | MAX. | MEDIAN | MIN. | MAX. | MEDIAN | MIN. |
| 1 | Olympia, WA., 1949 (N86E) | 1.21 | 0.98 | 0.76 | 1.15 | 1.01 | 0.88 | 1.18 | 1.04 | 0.67 |
| 2 | Taft, Kern Co, 1952(S69E) | 1.18 | 0.86 | 0.59 | 1.05 | 0.86 | 0.64 | 1.05 | 0.91 | 0.82 |
| 3 | El Centro Array No. 12 Imperial Valley, 1979(140) | 1.05 | 0.83 | 0.59 | 1.00 | 0.85 | 0.68 | 1.03 | 0.84 | 0.71 |
| 5 | Pacoima Dam San Fernando, 1971(S14W) | 1.16 | 0.90 | 0.59 | 1.06 | 0.97 | 0.51 | 1.06 | 0.94 | 0.56 |
| 6 | Hollywood Storage PE Lot, San Fernando, 1971(N90E) | 1.25 | 0.94 | 0.49 | 1.14 | 1.02 | 0.57 | 1.20 | 1.03 | 0.59 |
| 7 | El Centro Array No. 5 Im- perial Valley, 1979(140) | 1.29 | 1.05 | 0.70 | 1.24 | 1.15 | 0.65 | 1.32 | 1.06 | 0.67 |
| 8 | USCB Goleta Santa Barbara, 1978(180) | 1.13 | 0.70 | 0.59 | 1.35 | 0.73 | 0.62 | 1.63 | 0.77 | 0.64 |
| 9 | Gilroy Array No. 2, Coyote Lake 1979(050) | 1.33 | 0.89 | 0.29 | 1.24 | 0.95 | 0.32 | 1.22 | 0.93 | 0.32 |
| 10 | Cholame Array No. 2, Parkfield 1966(N65E) | 1.27 | 0.52 | 0.40 | 1.41 | 0.53 | 0.46 | 1.40 | 0.60 | 0.51 |
| 11 | Gavilan College Hollister, 1974(S67W) | 1.64 | 0.57 | 0.26 | 1.20 | 0.80 | 0.17 | 0.96 | 0.50 | 0.10 |
| 12 | Melendy Ranch Barn, Bear Valley 1972(N29W) | 1.50 | 0.91 | 0.11 | 1.49 | 1.16 | 0.13 | 1.44 | 0.87 | 0.14 |

*Ratios of Actual to Reg. Guide 1.60 Spectral Accelerations are reported for the frequency range of 1.8 to 10 Hz.

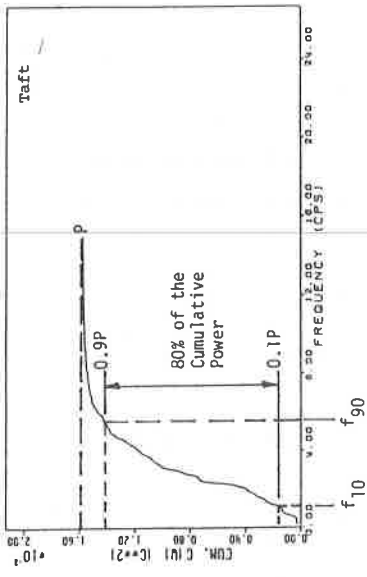


Figure 1. Cumulative Power Spectral Density Function of the Taft Accelerogram

Figure 3. Accelerogram and Variation of Cumulative Energy with Time for the 1952 Kern County Earthquake Recording at Taft

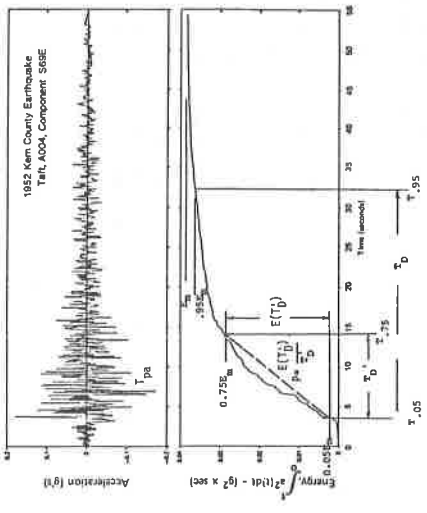


Figure 2. Comparison of Elastic Response Spectrum for Artificial, El Centro #5, Parkfield, and Melendy Ranch Earthquakes

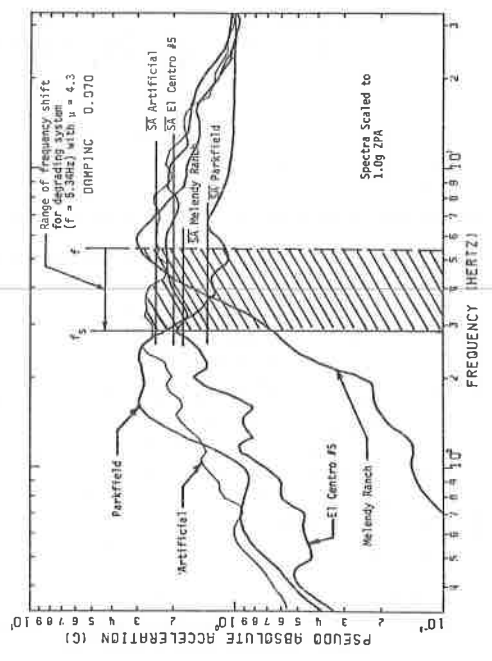


Figure 4. Free-Field versus Foundation Response Spectra

