Random Variability of Dynamic Characteristics of Nuclear Power Plant Shear Wall Structures

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SUMMARY

Sources of random variability which can contribute to the dispersion of the structural dynamic characteristics of a typical nuclear power plant shear wall structure are discussed. Random variability is considered to represent those sources of dispersion which cannot be reduced by additional analysis or more data. Uncertainty resulting from lack of knowledge of material properties and other parameters or approximations in analytical modelling are not included.

The effects of the random variability are reflected in shifts in both the modal frequencies and structural response. The structure chosen for investigation was a typical PWR auxiliary building but many of the results may be considered as representative of concrete shear wall structures constructed to nuclear power plant criteria. The results are presented in terms of median values of the important random sources of dispersion together with the corresponding coefficients of variation assuming a log-normal distribution for all variables.

Sources of random variability in the calculated modal frequencies of a lightly damped elastic structure occur in both the mass and stiffness terms. The principal sources of random variability in the structural masses are the variation in the concrete density and the member dimensions. The coefficient of variation (COV) on structure mass random effects was determined to be about 0.03. The principal sources of random variability in the stiffness terms include the member dimensions and the modulus of elasticity and shear modulus. The modulus of elasticity and shear modulus are in turn dependent on the strength of the concrete. An average logarithmic standard deviation associated with stiffness due to randomness of the concrete member properties was found to be approximately 0.02. A coefficient of variation about 0.14 results for the modulus of elasticity, and a COV of 0.14 for the shear modulus. The combined COV for stiffness for a shear wall structure for random effects is approximately 0.14. The variations due to random mass and stiffness effects can be combined to give the coefficient of variation for frequency of approximately 0.07.

Calculations were performed to determine the variation in structure response which may be expected at damping levels throughout the elastic response range. Real earthquake time history records scaled to a common peak ground acceleration were used as sources of excitation. Variations in structure response due to damping and time history input characteristics were obtained.

1. Introduction

Variations in the computed dynamic response of nuclear power plant structures can originate from a great many sources. These variations can occur in both the response frequencies and the dynamic amplification of the structures. The dispersion can result from both sources of inherent randomness and from uncertainty concerning the values of the sources. In this context, inherent randomness is considered to represent those sources of dispersion which cannot be reduced by additional analysis or more data. Modelling uncertainty, on the other hand, is considered to be the result of lack of knowledge of material properties and other parameters, and approximations in analytical modelling.

Although it is often difficult to completely separate random variability and uncertainty, this study is devoted principally to random sources. In order to develop quantitative estimates of the variability, it is ncecessary to define a mean or median value of the parameter and a measure of dispersion in terms of standard deviation depending on whether a normal, lognormal, or other distribution is used. For most sources of random variability, inadequate information exists to exactly define which statistical distribution most accurately represents the data. For most sources of structural variability, the lognormal distribution provides a good representation (References 1 and 2), so long as the extreme tails are not of primary concern. Furthermore, the central limit theorem states that a distribution consisting of products and quotients of distributions of several variables tends to be lognormal even if the individual variable distributions are not lognormal. Also, the lognormal distribution is mathematically tractable. For all variables discussed in this evaluation of the dynamic characteristics of structures, a lognormal distribution is assumed. For values of the logarithmic standard deviation less than about one-half, the logarithmic standard deviation and the coefficient of variation are approximately equal and are often used interchangeably.

Variations in frequencies should be discussed in terms of elastic and inelastic structure systems. For elastic systems, the response frequencies are essentially independent of amplitude for lightly damped systems and, hence, may be considered as functions primarily of the mass and flexibility of the system. The evaluation of the structure covered in this study is limited to elastic response levels. However, actually, very few structures respond completely elastically, even at very low response levels.

One of the important parameters in the computation of the frequencies of the system is the mass of the structure and equipment. Much of the dispersion due to the mass could be reduced or eliminated if sufficient effort were devoted to the calculations and measured weights for equipment were used. However, some inherent randomness would still exist due to variations in material densities and element tolerances, even if mass calculations were carried out in minute detail.

The variability in the stiffness of the structure is the other important parameter which must be considered when treating the variability of frequencies. For concrete structures, a significant variability is introduced as a result of the inherent randomness of the modulus of elasticity and shear modulus which are in turn related to the concrete strength. Less variability results in the modulus of elasticity of structural steel. In both concrete and steel structures, some small degree of randomness is introduced as a result of variation of the member dimensions.

Variation of response levels for a given input depends to a large extent on the energy dissipation characteristics of the system. This is usually referred to as the system damping. Although there is a general lack of precise information on energy dissipation of structures, particularly at high levels of response, it is generally accepted that damping is strongly dependent on the type of construction and the level of response. One of the major sources of variation expected in the actual response of structures compared with design calculations results from actual earthquake characteristics compared to artificial earthquakes or smoothed response spectra used for design. As with frequencies, however, dispersion in the dynamic response of the structure may be expected to exist as a result of both random sources and uncertainties.

This paper concentrates on random variabilities inherent in the structures and their effects on structure response and input to equipment. Consequently, a large number of sources of uncertainties are not evaluated in this study. The random variabilities expected in a typical nuclear power plant auxiliary building are presented.

2. Variability in Structure Frequencies

Variability in linear system frequencies results primarily from variabilities in the mass and stiffness considerations. The variability in the mass terms used for nuclear power plant structural analysis results from both randomness and modelling techniques. The random variability sources in the determination of the mass terms used in structural analysis further include several considerations. Among these are material density which, in the case of concrete for instance, consists of the variation in aggregate, sand, and steel densities as well as their relative amounts. Size of the various structural elements as indicated by the dimensional tolerances also introduces a source of random variability. Typically, normal weight concrete may be expected to have a unit weight of approximately 145 pcf (Reference 3). For high strength concrete and particularly when the reinforcing steel is taken into account, a higher value is usually used. A coefficient of variation (COV) of approximately 0.026 is typical for concrete.

Data from the literature (Reference 4) provide a basis for the variabilities associated with the geometric tolerances of reinforced concrete. The data are expressed in terms of dimensional variations. These data are not shown as functions of the nominal element dimensions, so that both the mean difference from the nominal and the percentage variation in the mass of a concrete element must be determined for various size elements. Most calculations involving structure mass are based on nominal dimensions. In order to evaluate the variation of mass, several typical floor slabs and walls for a typical auxiliary building were evaluated for the random tolerance effects. The logarithmic standard deviation associated with randomness of concrete dimensions was determined to be approximately 0.014.

In order to estimate the net random effects, logarithmic standard deviations due to variations in the concrete mass density and concrete member dimensions were calculated independently assuming both effects are in the same direction. On this basis, the logarithmic standard deviation on random structure mass effects is estimated to be about 0.03.

As with the mass effects, there are a number of sources of variability in the stiffness determination from both random sources and uncertainties which affect the dynamic characteristics of structures. This study is directed to stiffness effects of concrete rather than structural steel construction. The two most important random effects are the dimensional tolerances previously discussed and variability of the modulus of elasticity and the shear modulus.

The evaluation of the stiffness of most concrete elements whether loaded in compression or shear is related to the modulus of elasticity of concrete, E_c which in turn, is typically based on the concrete compressive strength, f'_c. Concrete compressive strength used for design is normally specified as some value at a specific time from mixing (for example, 90 days). This value is verified by laboratory testing of mix samples. There are two major factors which justify the selection of a median value of concrete strength somewhat above the design strength. To meet the design specifications, the contractor attempts to create a mix that has an "average" strength somewhat above the design strength; and as concrete ages, it increases in strength. For the structure considered in this study results of 90 day concrete compression tests are available (Reference 5).

As concrete ages, its strength increases, and this must also be accounted for in determining actual median strength compared to the design strength. Data are available in the literature (Reference 7) which give typical effects of aging on the strength of concrete. No information is available on the standard deviation expected for aging. The estimated logarithmic standard deviation for aging is 0.1. For the auxiliary building in this study, the factor of median to design compressive strength is approximately 1.65 with logarithmic standard deviation of 0.13.

The formula recommended in Reference 6 is believed to give the best estimate of secant concrete modulus of elasticity for the stress levels of interest.

$$E_c = 57{,}000 \sqrt{f'_c} \quad \text{(psi)}$$

Reference 7 reports that the coefficient of variation of the constant term (57,000) is 0.124. This value may be combined with the dispersion due to concrete strength to calculate the total coefficient of variation. Most references report that Poisson's ratio for concrete is typically between 0.15 and 0.20. It is estimated that the coefficient of variation is 0.10. For the typical structure investigated in this study, the modulus of elasticity of the concrete was found to be approximately 4.63×10^6 psi with a coefficient of variation of 0.14. A corresponding shear modulus of approximately 1.98×10^6 psi with a coefficient of variation of approximately 0.14 results.

As in the case of the variability of the structure mass, variability on the concrete member dimensions will cause variability of stiffness. The variability of stiffness introduced by given concrete tolerance depends on the geometry of the structure and also on whether the shear stiffness which is a function of the member area or the bending stiffness which is a function of the member section modulus is controlling. For the shear wall structure considered here, the stiffness is only slightly affected by bending stiffness. Based on shear stiffness properties computed for a number of typical storey shear stiffnesses for this structure, coefficients of variation of from 0.13 to 0.24 are calculated. From these values, an average logarithmic standard deviation associated with stiffness due to randomness of concrete member properties is estimated to be approximately 0.018. As in the case of the random variability of the mass, the coefficient of variability for the stiffness due to the combined shear modulus and dimensional tolerance effects can be calculated as approximately 0.17.

The variability due to mass and stiffness randomness can be combined to determine the variation in frequency. For the auxiliary building used in this example, the random variability of the stiffness is significantly greater than that for the mass terms. The logarithmic standard deviation of frequency for this structure due to randomness is approximately 0.072. It is expected that the frequencies of the other shear wall concrete structures constructed to nuclear power plant criteria will have similar variability due to randomness.

A large amount of the variability in the mass and stiffness terms in the analysis of nuclear power plant structures typically results from modelling uncertainties as well as from the random effects. For instance, rather than perform very large numbers of detailed calculations to determine all the equipment weights, it is common practice to assume a uniform increase in the weight of the floor slabs to cover equipment weight and perform mass calculations only for a few specific items of significant size. Similarly, it is often usual practice to neglect the effects of doorways and other cutouts in the walls or floor slabs. Significant uncertainties in the stiffness effects are also introduced by lack of knowledge of the material properties and different modelling techniques are present. The design analysis of nuclear power plant structures is normally completed well before any construction. Consequently, no test data from concrete tests are available at the time the analysis is conducted. Concrete compressive strength is almost inevitably assumed as the minimum specified design strength, and corresponding values for modulus of elasticity are used. This introduces a systematic bias which may be expected to result in lower calculated stiffness parameters and corresponding lower structure frequencies than actually exist.

Other modelling uncertainties which can affect both the mass and stiffness formulations and hence the structure frequencies include finite element techniques compared to lumped mass or distributed mass models, and methods of formulating member stiffness including treatment of diaphragm flexibility, the degree of refinement in modelling wall cutouts and treatment of the form factor for shear walls. All these as well as many other important uncertainties such as treatment of soil structure interaction exist in the analyses of nuclear power plant but are not evaluated here. It should be expected, however, that the variation due to uncertainty will exceed that due to the random structural effects. A number of aspects concerning modelling uncertainties are discussed in References 8 and 9.

3. Variability in Structure Response

Once the structure response frequencies together with their variability have been established, the random variability expected in the response can be determined by a consideration of the system energy dissipation characteristics and seismic inputs to the structure. Energy dissipation in structures subjected to dynamic loading can occur by several means such as friction, heat generation, and micro scale yielding and cracking etc., even in the linear range of response. For structures with response levels in the nonlinear range, the energy absorption characteristics include additional mechanisms such as gross yielding and cracking. For structures, little precise information exists on damping, even at low levels of response. As the response levels increase to include ranges of significant nonlinearity, even less data are available. Damping levels have been determined for the most part by observation and measurement and almost invariably have been limited to low response levels. Virtually no damping tests of representative structures have been conducted at response levels approaching collapse. Also, no data are available to allow separation of structure and geometric damping for actual structures.

Damping levels have been observed to show a fairly wide spread, and for design purposes, low values are typically used. Damping is known to vary with the type of construction or material used, the stress levels, and the configuration of the structure. Thus different damping occurs in concrete and steel structures and further gradations may be expected in prestressed versus reinforced concrete and welded compared to bolted or riveted steel structures. Although difficult to define precisely, damping also is known to increase for increased stress levels. For a system as complex as a nuclear power plant structure with many different elements with different stress levels, connection details, interacting equipment, etc., no simple relationships adequately define the phenomena of damping for different structures for varying response levels.

Finally, the geometric configuration results in different damping values for different structures even though they are constructed in a similar manner. For instance, a structure where a higher percentage of the structural material is stressed to a given level may be expected to dissipate more energy and hence exhibit higher damping than one where only a small percentage of material is stressed. Thus, for a given maximum stress level, low rise shear wall structures may be expected to have higher damping levels than high rise structures where much of the structure in the upper stories is more lowly stressed. This can be verified by a breakdown along these categories of the structures included in Reference 10 which contains a survey of most known damping results for actual buildings.

Other physical conditions which can affect the damping levels in structures include such items as rate of loading effects, stress history, combination of steady state and cyclic stress levels, thermal condition, and interaction of nonstructural elements. Essentially no information is available concerning such effects for structures.

Based on a survey of existing information available in the literature (References 10, 11 and 12), representative median viscous damping levels can be established. For ground acceleration levels which develop stresses below approximately half the yield stress of both reinforced and prestressed concrete, a median value of structure damping of 3% of critical is expected. For acceleration levels producing stresses approximately equal to yield and above, 10% of critical damping is expected for reinforced concrete. A coefficient of variation of approximately 0.4 to 0.5 for all stress levels is indicated. Wide variability on damping of nuclear power plant structures is expected, not only from structure to structure but also within a given structure at different response levels. In order to determine the variation on structure response resulting from different assumed values of damping, a preliminary study was conducted. To evaluate the effect of various damping levels on the response of a typical nuclear power plant shear wall structure, a series to time history analyses was conducted with different damping values. Constant properties for the mass and stiffness parameters were used throughout the analysis, and no variation in frequencies were considered in the determination of structure response due to damping variation.

The input into the structure consisted of a series of natural earthquakes. These earthquakes were selected as being representative of what could be expected for strong motion excitation. Earthquakes with relatively broad band response spectra were chosen which for the most part consisted of deep and layered alluvium sites. All earthquakes were scaled to a common peak ground acceleration of 1g for the maximum horizontal component. The structure was analyzed with both horizontal directional components applied simultaneously, and the phasing of the two components was the same as was originally recorded in all cases. The structure was then analyzed with the earthquake directional components rotated 90 degrees. In effect, the response of the structure to ten earthquake records was thus obtained.

The dispersion in the structural response can be summarized in terms of the coefficients of variation for the four assumed damping values as shown in Table 2 for E-W response. Coefficients of variation for story shear-response in the range of approximately 0.33 to 0.23 occur when the results from the total of ten analyses are considered. Coefficients of variation for bending moments are similar. In general,the coefficients of variation do not show major shifts with changes in damping for either shears or moments. Slightly smaller coefficients of variation are uniformly indicated for increased damping levels and in no case was the COV found to decrease with higher damping. Also, in general, the observed trend is for somewhat smaller coefficients of variation to occur at the lower elevations. Structural damping is seen not to be a controlling parameter on dispersion but rather the natural earthquake characteristics.

4. Variation in Input to Equipment

In order to determine the dispersion in the input to equipment caused by variation of the structure damping and different earthquakes, in-structure response spectra were generated at two typical locations within the auxiliary building. These are representative of locations both relatively high and low in the structure. In-structure spectra were generated for two orthogonal directions for equipment damping of 3% of critical for each of the four values of structure damping. Figure 1 shows a plot of some typical in-structure response spectra for the case of 3% of critical structure damping. Figure 2 shows some in-structure response spectra for the same location for the case of 7% of critical structural damping. Other structural damping values result in similar plots. Although some of these earthquakes possess sufficient energy to excite the structure strongly at its fundamental frequency, others do not and significant dispersion results.

5. References

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TABLE 1

VARIABILITY OF RANDOM STRUCTURAL PARAMETERS

Item	Coefficient of Variation
Concrete Density	0.026
Concrete Dimensions	0.014
Structure Mass	0.03
Concrete Aging	0.1
Concrete Compressive Strength	0.13
Young's Modulus (Concrete)	0.14
Shear Modulus (Concrete)	0.14
Poisson's Ratio (Concrete)	0.1
Structure Stiffness (Shear)	0.14
Structure Frequency	0.072
Structure Damping	0.4 to 0.5

TABLE 2

COEFFICIENTS OF VARIATION FOR E-W STOREY SHEAR

STOREY	DAMPING			
	3%	5%	7%	10%
El. 642 to 668	0.33 (0.22)	0.33 (0.22)	0.33 (0.23)	0.32 (0.22)
El. 630 to 642	0.30 (0.16)	0.29 (0.15)	0.29 (0.15)	0.27 (0.14)
El. 617 to 630	0.29 (0.14)	0.27 (0.12)	0.27 (0.12)	0.26 (0.10)
El. 592 to 617	0.27 (0.12)	0.26 (0.074)	0.25 (0.070)	0.25 (0.058)
El. 579 to 592	0.26 (0.090)	0.25 (0.048)	0.25 (0.031)	0.25 (0.036)
El. 560 to 579	0.26 (0.072)	0.25 (0.035)	0.25 (0.034)	0.25 (0.045)
El. 542 to 560	0.25 (0.066)	0.25 (0.031)	0.25 (0.037)	0.25 (0.047)

Note: Values in parentheses are coefficients of variation for time histories with maximum acceleration components in the E-W direction only.

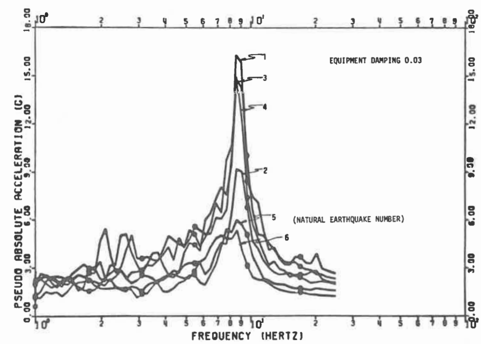

FIGURE 1. INSTRUCTURE RESPONSE SPECTRA ELEVATION 642 FT., N-S DIRECTION 3% STRUCTURE DAMPING

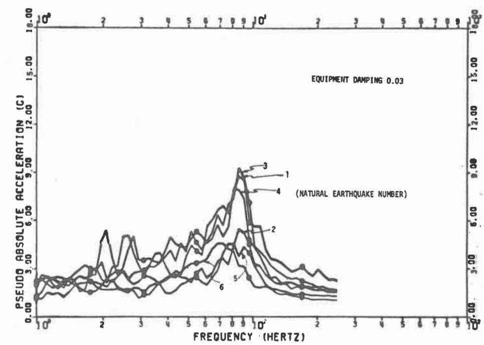

FIGURE 2. INSTRUCTURE RESPONSE SPECTRA, 642 FT., N-S DIRECTION, 7% STRUCTURE DAMPING