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EARTHQUAKES AND NUCLEAR POWER PLANTS
IN THE USA

by

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Earthquakes and Nuclear Power Plants in the USA

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1. Introduction

Nuclear power plants differ from conventional structures in that they must be designed to withstand extreme loads having very low probabilities of occurrence. Even though the seismic threat is unevenly distributed throughout the USA, the law—as expressed in the Code of Federal Regulations 10 CFR 100, Appendix A—establishes that earthquake loads must be considered for every nuclear power plant site in the country. Thus, even power plants erected within seismically inactive regions, such as Florida, must be designed for potential earthquakes having peak ground accelerations of at least 10% of gravity (0.1g). In fact, current regulatory thinking considers increasing this minimum requirement to 20% of gravity, except for a few sites at which somewhat smaller values would be acceptable. Hence, load combinations involving seismic forces will typically dominate in the design of nuclear power plant facilities.

2. Codes and Regulations

A relatively large number of codes, standards and regulations govern, or at least affect, the seismic analysis and design of nuclear power plant components in the USA. Of the more than 75 different documents that have influence in the analysis and design of safety class structures and equipment, the most important are:

a) The Code of Federal Regulations (10 CFR 100, Appendix A and 10 CFR 50, Appendices A & B.) This is a U.S. federal law. It defines, among other things, the safety categories for nuclear structures and components, the Safe-Shutdown Earthquake (SSE) and the Operating Basis Earthquake (OBE), capable faults and the minimum design ground acceleration, etc. It also sets forth general design criteria that must be used in the design of safety class facilities, i.e., those structural systems and components important to safety which in the event of a Safe Shutdown Earthquake must remain functional.

b) Nuclear Regulatory Commission, Regulatory Guides (NRC, RG). These are standards issued by the NRC that delineate criteria used by the staff in
evaluating specific problems. While compliance with these guides is not required, they exert a regulatory power approaching that of federal law. Examples of Regulatory Guides that address issues in earthquake engineering are: RG 1.12 (Instrumentation for Earthquakes), RG 1.60 (Design Response Spectra), RG 1.61 (Damping Values for Seismic Design), RG 1.92 (Combination of Modes and Spatial Components of Earthquakes) and others.

c. NRC Standard Review Plan (SRP). The SRP is a document in the format of a Safety Analysis Report (FSAR) that contains NRC "branch" position papers with analysis methods that reflect current technical standards. Neither is compliance with the SRP required, nor is admissibility by the NRC of calculations performed by the applicant in accordance with the SRP guaranteed.

d. Other important codes, standards and regulations: ANSI (American National Standards Institute), ASME Boiler and Pressure Vessel Code (American Society of Mechanical Engineers), ACI (American Concrete Institute), ANS (American Nuclear Society), IEEE Seismic Standards (Institute of Electrical and Electronic Engineers) etc.

It should be noted that, while a large number of codes and standards regulate the analysis and design of nuclear power plant facilities, the actual methods, procedures or standards used in a particular project are proposed and detailed by the applicant in the (Final) Safety Analysis Report (FSAR), and decided in (public) negotiation between the NRC and the applicant. Thus, technical standards change from project to project and improvements are incorporated as the state of the art evolves. On the other hand, the NRC may impose or disallow methods of analysis even in the absence of specific regulations; an example of such "unwritten laws" occurs in the seismic evaluation of category I structures, which may be performed only with deterministic, linear analyses, even though ductility requirements will ensure the safety of the structure well beyond the elastic limit, as the experience with conventional buildings demonstrates.
3. **Design Earthquakes**

Both the SSE and OBE earthquakes are defined in the "free field" at the free surface. The earthquakes must consist of three statistically independent components (East-West, North-South, and Vertical), each having a peak acceleration equal to or greater than the design acceleration provided by the seismic risk evaluation report for the site. It is of interest to observe that the peak acceleration values are prescribed for each principal direction rather than for the motion obtained by vectorial addition of the components; this is consistent with the statistical information derived from the components of actual past earthquakes, which ignores (potentially larger) accelerations in directions not coinciding with the principal axes.

For example, if \( x = x(t), y = y(t) \) are the time histories of the earthquake records in the East-West and North-South directions, then the resultant \( r = r(t, \theta) \) in the direction with azimuth \( \theta \) is

\[
r = x \cos \theta + y \sin \theta
\]

(and a resultant \( s = -\sin \theta x + \cos \theta y \) at 90° from this direction).

The maximum value occurs in the direction \( \theta \) for which \( \frac{\partial r}{\partial \theta} = 0 \); that is,

\[
\frac{\partial r}{\partial \theta} = -x \sin \theta + y \cos \theta = 0
\]

implying

\[
\left\{
\begin{align*}
\cos \theta &= \frac{x}{\sqrt{x^2 + y^2}} \\
\sin \theta &= \frac{y}{\sqrt{x^2 + y^2}}.
\end{align*}
\right.
\]

Substituting into the equation for \( r \), we obtain

\[
r = \sqrt{x^2 + y^2}, \quad r^2 = x^2 + y^2.
\]

Since \( x, y \) are (by definition) statistically independent variables,

\[
E(r^2) = E(x^2) + E(y^2),
\]

which shows that a direction \( \theta \) exists for which the peak ground acceleration is expectedly larger than for the components.
The seismic risk evaluation for the site normally involves investigations on the geographic location; the geologic conditions and tectonic structure; the static and dynamic characteristics of the underlying materials (soil conditions); the location and nature of existing faults in the vicinity; historical listings of all earthquakes which may have affected the site, and identification of the effects that they may have caused; correlation of epicenters, etc.

Because the characteristics of earthquakes are uncertain, and they may change from event to event—even for a given site—it is customary to design synthetic (artificial) earthquakes rich in all frequencies, i.e., that exhibit broad-band response spectra. While such records provide a conservative basis for the analysis and design of linear single-degree-of-freedom systems, and are probably conservative for linear multiple-degree-of-freedom systems as well, they should not be used in nonlinear response calculations (for example, in soil amplification studies by the iterative method or in analyses of the soil liquefaction potential). The reason is that use of those records may grossly exaggerate degradation of the soil properties, leading to numerical difficulties and prediction of impossible motions at some points in the soil mass and the structures.

Specification of the response spectra or time histories for the design earthquakes at the control point in the free field (i.e., near the ground surface and unperturbed by the presence of structures) is not sufficient to fully describe the seismic environment: it is necessary to provide the wave contents as well. Earthquake motions result from complex wave patterns whose characteristics depend on many factors. Actual seismic ground motions are of a probabilistic nature which can be described as stochastic processes. They depend mainly on the magnitude of the seismic event, site location, travel path of the stress wave, local geology, and soil conditions. At present, it is customary to assume that the motions in the free field are the result of plane waves that propagate vertically, and that the soil deposits are horizontally stratified. Empirical evidence seems to provide some support to the shear beam theory, at least from a qualitative point of view. Theoretical investigations of more complex wave types, on the other hand, show that the amplification factor is not very sensitive to the angle of incidence of the seismic stress waves. Even studies for surface waves predict a decrease in the motion intensity and frequency selectivity (the suppression of certain frequencies) with
depth surprisingly similar to that predicted by the elementary shear beam model. However, amplification ratios and response spectra are not the only indicators of the accuracy of a theory. Waves that are propagating not vertically cause phase differences in the motions within the soil mass (the so-called wave passage phenomenon) which can become an important factor in the assessment of soil-structure interaction effects. Results of other studies suggest that the strain levels in the soil and, therefore, the magnitude of material nonlinear effects could be altered if more general wave types are considered. It should be realized, however, that any arbitrary motion at the control point in the free field can be caused by infinitely many different wave patterns. At the present time it is not practical or even feasible to deterministically assign relative weights to the individual wave components. Furthermore, analyses performed with only a single wave component (for instance, horizontally polarized shear waves travelling at an incidence angle of, say 45°) suffer from the same shortcoming as vertically propagating stress wave models, and they are more complicated and equally arbitrary. Also, since material damping attenuates the amplitude of the waves, particularly the high frequency components, the motion intensity will decrease in the horizontal direction for non-vertically incident waves, and it is not clear where in the free field the control motion should be defined. Current research and field instrumentation programs are aimed at providing an understanding of the effects of non-vertically propagating body and surface waves on the seismic environment, as well as on the influence of neighboring rock outcroppings, dipping layers, and narrow alluvial valleys (basin effects) on the actual predicted motions.

4. Seismic Analysis of Safety Class Structures

Most—or perhaps all—dynamic analyses for seismic effects in nuclear power plants currently performed in the USA are both linear and deterministic.

The analysis and design of the safety class structures, secondary systems, equipment and piping is controlled at present by the requirement that the dynamic loads induced by the design earthquake be resisted in the linear range. Possible exploitations of the reserve capacity in the nonlinear range on the basis of ductility considerations, are not permitted in the analysis stage, since a safe shutdown must be assured. In this respect, design of nuclear structures differs from the design of conventional structures in
seismic areas, in that design ground accelerations which are relatively low in comparison to the peak accelerations of some recorded earthquakes are admissible for the latter in recognition of their inherent strength and ductility.

On the other hand, structural reliability concepts involving consideration of the probabilities of exceedance of the design loads, system behavior and probability distributions for the strength of the structural components, are not yet used in an explicit manner to assess the overall reliability of the system to earthquake loads. At most, these concepts have been used to define rationally the factors of safety that are necessary in the design of the structures for given deterministic loads. Some tentatives have also been made to obtain amplified response spectra directly from the ground motion input, without the need of going through a response analysis in the time domain. These methods are basically more consistent in themselves because the input motion is described by power spectral density functions from which the amplified spectra are estimated directly, using the theory of random vibrations. It must be realized, however, that no accurate solutions exist yet for this problem, particularly when the earthquake input is considered a general nonstationary process. Partially for this reason, the NRC has not yet approved these methods.

Both nonlinear and probabilistic concepts, however, may yet be required to assess, post-design, the reliability of the structures dimensioned with the results of the deterministic, linearly elastic analyses.

Uncertainties in material properties are typically handled by performing suites of linear analyses with varying properties. For example, three soil stiffnesses are typically considered: normal, soft and hard soil. Also, the containment structure is usually analyzed for both an uncracked and for a fully cracked condition. The results are then enveloped (rather than statistically averaged!), and the floor spectra broadened. Needless to say, the determination of the response parameters (accelerations, response spectra, etc.) at the various points (and directions) of interest, and the proper interfacing of the various analyses for both SSE and OBE events, create a data handling problem of paramount importance; this problem needs to be addressed properly, if errors are to be avoided.
Heavy equipment, such as the nuclear steam supply system, are normally included, at least qualitatively, in the dynamic model for the structure and the soil. The motions obtained at the base of these systems are then supplied to the equipment vendors, in the form of time histories or response spectra, for their use in qualifying the equipment for the seismic loads. It can be shown, however, that this practice may lead to excessive conservatism in the design of the equipment, if the dynamic models used by the vendors are not consistent with the models used by the contractors in the overall seismic analyses. Similar consistency problems may also develop in analyses for soil-structure interaction effects, when two-dimensional finite element models of the soil-structure systems are first used to determine the motions at the base of the structures, and then applied as support excitations to more elaborate (albeit inconsistent!) three-dimensional idealizations of the superstructure. The reason for this inconsistency is that the frequency spectrum of the motion at the support point of the equipment (or the motions of the foundation in the soil-structure interaction example) must exhibit valleys at the natural frequencies of the equipment (or structure); refinement of the model for the equipment (or structure) in the second step effectively changes (if ever so slightly) these frequencies, causing the amplification peaks not to coincide exactly with the valleys in the support motion. As a result, spurious amplification develops.

Special problems are also posed by the design of equipment and piping with multiple support points. Current practice is to take the most severe motions for all supports, and consider as well the situation in which the supports move relative to each other (i.e., have antiphase motions). Such procedures, while practical, may result in an unknown degree of conservatism (or even unconservatism).

5. **Load and Response Combinations**

Earthquake loads are normally considered in combination with dead loads, live loads and thermal loads. Other extreme loads, such as those produced by aircraft impact or tornadoes, are not considered in conjunction with earthquakes, since their joint probability of occurrence is vanishingly small. For some structural systems, however, one may have to consider also equipment reactions, and internal missiles that could be generated by pipe ruptures. The containment structure may also be subjected to internal pressure, resulting from pipe breaks and subsequent steam release.
While the three (statistically independent!) components of the earthquake motion could, in principle, be prescribed simultaneously to a three-dimensional model of the soil-structure system, this is not normally done in practice; the effects for the various components are studied instead separately. Unless all response time histories computed in this manner are stored in full, however, it is not possible to overlap the various effects in time, giving rise to the problem of how to combine the maximum response values and response spectra computed with each earthquake component separately.

For example, consider the combination of the response quantities \( S_x(t) \) and \( S_y(t) \) (column shears, floor displacements, etc.) in directions \( x, y \). If the dynamic model comprises all earthquake components, then \( S_x, S_y \) can be combined to find the response \( S_\theta \) in some arbitrary direction:

\[
S_\theta = S_x \cos \theta + S_y \sin \theta
\]

Performing computations similar to those in section 3 of this paper, we obtain the maximum response at time \( t \) as

\[
S_\theta(t) = \sqrt{S_x^2 + S_y^2}
\]

Defining \( \overline{S}_x = \max [S_x(t)] \), \( \overline{S}_y = \max [S_y(t)] \), it can be concluded from the above that

\[
S_{\theta_{\text{max}}} \leq \sqrt{\overline{S}_x^2 + \overline{S}_y^2}
\]

since the maxima \( \overline{S}_x, \overline{S}_y \) do not necessarily coincide in time. In practice, this discrepancy is ignored, and the above equation is used with the equal sign. This has the advantage that only the maximum values need to be stored. A similar argument applies to the combinations of response spectra that correspond to orthogonal directions.

On the other hand, suppose that the dynamic analysis was performed for each earthquake component separately; then

\[
S_x = S_{xx} + S_{xy}
\]

\[
S_y = S_{yx} + S_{yy}
\]
in which $S_{\theta x} = S_{xx}(t)$, etc. are the responses in the directions identified by the first subindex, caused by the earthquake acting in the directions indicated by the second. Again,

$$S_{\theta} = S_x \cos \theta + S_y \sin \theta$$

$$(S_{xx} + S_{xy}) \cos \theta + (S_{yx} + S_{yy}) \sin \theta$$

$$(S_{xx} \cos \theta + S_{yx} \sin \theta) + (S_{xy} \cos \theta + S_{yy} \sin \theta)$$

$$= S_{\theta x} + S_{\theta y}$$

in which $S_{\theta x}, S_{\theta y}$ are the responses in direction $\theta$ due to the earthquakes in directions $x$ or $y$. Since these earthquakes are statistically independent, the maxima of the responses, $\bar{S}_{\theta x}$ and $\bar{S}_{\theta y}$ are not likely to occur simultaneously; hence, the maximum response $\bar{S}_{\theta}$ can be estimated by the SRSS rule

$$\bar{S}_{\theta} = \sqrt{\bar{S}_{\theta x}^2 + \bar{S}_{\theta y}^2}$$

On the other hand, the maxima $\bar{S}_{\theta x}, \bar{S}_{\theta y}$ follow from the component maxima $\bar{S}_{xx}, \bar{S}_{xy}, \bar{S}_{yx}, \bar{S}_{yy}$ as

$$\bar{S}_{\theta x} = \bar{S}_{xx} \cos \theta + \bar{S}_{yx} \sin \theta$$

$$\bar{S}_{\theta y} = \bar{S}_{xy} \cos \theta + \bar{S}_{yy} \sin \theta$$

since the maxima in orthogonal directions ($\bar{S}_{xx}$ and $\bar{S}_{yx}$, or $\bar{S}_{xy}$ and $\bar{S}_{yy}$) due to the same earthquake (either $x$ or $y$) are likely to occur simultaneously. Hence

$$\bar{S}_{\theta}^2 = (\bar{S}_{xx} \cos \theta + \bar{S}_{yx} \sin \theta)^2 + (\bar{S}_{xy} \cos \theta + \bar{S}_{yy} \sin \theta)^2$$

$$= (\bar{S}_{xx}^2 + \bar{S}_{xy}^2) \cos^2 \theta + 2(\bar{S}_{xx} \bar{S}_{yx} + \bar{S}_{xy} \bar{S}_{yy}) \sin \theta \cos \theta$$

$$+ (\bar{S}_{yx}^2 + \bar{S}_{yy}^2) \sin^2 \theta$$

$$= \frac{1}{2} \left\{(\bar{S}_{xx}^2 + \bar{S}_{xy}^2 + \bar{S}_{yx}^2 + \bar{S}_{yy}^2) + (\bar{S}_{xx}^2 + \bar{S}_{xy}^2 - \bar{S}_{yx}^2 - \bar{S}_{yy}^2) \cos 2\theta$$

$$+ 2(\bar{S}_{xx} \bar{S}_{yx} + \bar{S}_{xy} \bar{S}_{yy}) \sin 2\theta \right\}$$
Taking derivative with respect to \( \theta \), and equating to zero to find the azimuth at which \( \bar{s}_{\theta_{\text{max}}} \) occurs,

\[
\tan 2\theta = \frac{2\bar{s}_{xx} \bar{s}_{yx} + \bar{s}_{xy} \bar{s}_{yy}}{\bar{s}_{xx}^2 + \bar{s}_{xy}^2 - \bar{s}_{yx}^2 - \bar{s}_{yy}^2}
\]

from which also \( \cos 2\theta \) and \( \sin 2\theta \) can be deduced. Substituting into the expression for \( \bar{s}_{\theta} \), we find

\[
\bar{s}_{\theta_{\text{max}}}^2 = \frac{1}{2} \left\{ \bar{s}_{xx}^2 + \bar{s}_{xy}^2 + \bar{s}_{yx}^2 + \bar{s}_{yy}^2 + \sqrt{(\bar{s}_{xx}^2 + \bar{s}_{xy}^2 - \bar{s}_{yx}^2 - \bar{s}_{yy}^2)^2 + 4(\bar{s}_{xx} \bar{s}_{yx} + \bar{s}_{xy} \bar{s}_{yy})^2} \right\}
\]

\[
= (\bar{s}_{xx}^2 + \bar{s}_{xy}^2 + \bar{s}_{yx}^2 + \bar{s}_{yy}^2) \frac{1}{2} (1 + \sqrt{1 - D^2})
\]

in which

\[
D = \frac{2(\bar{s}_{xx} \bar{s}_{yy} - \bar{s}_{xy} \bar{s}_{yx})}{\bar{s}_{xx}^2 + \bar{s}_{xy}^2 + \bar{s}_{yx}^2 + \bar{s}_{yy}^2}
\]

Notice that in the above expression, \( 0 < \sqrt{1 - D^2} < 1 \). Hence,

\[
\bar{s}_{\theta_{\text{max}}}^2 < \bar{s}_{xx}^2 + \bar{s}_{xy}^2 + \bar{s}_{yx}^2 + \bar{s}_{yy}^2
\]

(The naive attempt to use the local maxima \( \bar{s}_{\theta x_{\text{max}}} = \bar{s}_{xx}^2 + \bar{s}_{yx}^2 \),
\( \bar{s}_{\theta y_{\text{max}}} = \bar{s}_{xy}^2 + \bar{s}_{yy}^2 \) to obtain the maximum \( \bar{s}_{\theta_{\text{max}}} = \bar{s}_{\theta x_{\text{max}}} + \bar{s}_{\theta y_{\text{max}}} = \bar{s}_{xx}^2 + \bar{s}_{yy}^2 + \bar{s}_{xy}^2 \)) fails, because \( \bar{s}_{\theta x_{\text{max}}}, \bar{s}_{\theta y_{\text{max}}} \) occur along distinct directions: \( \tan \theta_x = \frac{\bar{s}_{yx}}{\bar{s}_{xx}}, \tan \theta_y = \frac{\bar{s}_{yy}}{\bar{s}_{xy}} \).
When analyzing the dynamic system for the various earthquake sources separately, and only the maximum values are of interest, it is obviously not necessary to use different (statistically independent) earthquakes for the east-west and north-south directions.

6. Soil-structure Interaction

In the past decade, an unnecessary confrontation and polarization of opinions evolved between the proponents of the lumped parameter method (continuum, spring or impedance method) and the finite element method. This was unfortunate, because numerical and analytical approaches have traditionally played complementary roles in engineering mechanics. Each method has its own particular advantages and disadvantages, and they share many common difficulties. More recently, it has become evident that both techniques should give similar results if comparable models are used and consistent assumptions are made. Structures with little or no embedment are best handled by the impedance method (whether the soil is a shallow stratified deposit or a deep alluvium), while the finite element method is more appropriate for cases of deep embedment and/or irregular foundation geometries (although programs are now available that can compute the dynamic stiffnesses for an embedded foundation). Of particular interest is the possibility of using the finite element method only to determine the static "springs," and performing the interaction analysis in combination with the impedance method.

While some studies have addressed and commented on the interaction through the soil between neighboring structures (the so-called two-body problem), the effects of structure-to-structure interaction are not routinely considered in practice. Analyses of structure-to-structure interaction through soil coupling would seem to call for the finite element approach. Currently, most attempts to evaluate this effect make use of idealized plane-strain models. However, for a two-dimensional analysis, the actual three-dimensional problem has to be grossly simplified. Also, any plane model is likely to significantly exaggerate the interaction of two neighboring structures, because the relative decays of radiating waves in the two- and three-dimensional spaces are very different. Therefore, the significance of structure-to-structure interaction cannot be assessed until the three-dimensionality of the problem is recognized and incorporated in the analysis, either using numerical procedures, or evaluating
cross-impedance functions. Intensive efforts are currently being devoted to analyze this problem using the "exact" (albeit numerical) solution for two three-dimensional foundations without embedment which are typically (although not necessarily) rectangular in plan view. While this solution is of great theoretical and practical interest (particularly in machine foundation design), it should be kept in mind that the lack of embedment is likely to overemphasize possible interaction effects, and is therefore of limited applicability in the seismic design of nuclear containments which are usually deeply embedded.

7. Earthquake Protection and Isolation

Over the years, many papers have been published in the technical literature on the subject of earthquake protection and isolation of nuclear power plants. The proposed systems (some of dubious effectiveness) range from surrounding the plant with screening trenches, or floating them, to placing the structures on smooth surfaces (neoprene or teflon), etc. While some of these systems are actually being implemented elsewhere in the world, they have not been seriously considered in the USA. The main obstacles seem to lie in the cost (in)effectiveness of the available solutions, and above all, in the perceived difficulties in obtaining an operating license for a plant employing such unproved systems. Thus, these isolation systems are not likely to be used in the USA in the foreseeable future.