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SEISMIC REEVALUATION OF EXISTING NUCLEAR POWER PLANTS

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1.0 INTRODUCTION

Since the need to design U.S. nuclear power plant facilities to seismic requirements was brought to focus in 1963 (1), codes and regulations governing the seismic design have been produced and continuously amended to reflect more numerous studies and more accurate understanding in the field of geology, seismicity, and systems structural responses.

The discovery of faulting zones and the recording of recent earthquakes near a site already occupied by a nuclear plant have led to ground accelerations higher than those which the plant was calculated for. (2)

Those factors result in either more stringent seismic design criteria or augmented seismic design inputs. In a few instances, they have necessitated in the United States seismic requalification of existing nuclear power plant facilities.

Most of the Western European countries Safety Regulations are based on the American requirements and thus, the designers of European Nuclear Power Plants face or will have to face the same evolution as in the U.S.

The influence of the evolution on the most significant seismic inputs used for the design of nuclear plants will be covered in Chapter 2.

The technology of seismic analysis has also advanced rapidly in recent years. This advance has been largely in the fields of numerical methods, finite element analysis and modelling techniques. The author's organization has developed and implemented modern tools of analysis with the twofold purpose of:

- Qualifying Nuclear Power Plants to the most recent safety requirements.
- Minimizing the impact on existing plants hardware of the increase in the severity of seismic requirements by reducing much of the previous methods conservatism.

The main objective of this paper is to discuss a few of these newly developed features and to describe their integration into seismic analysis procedure successfully used to requalify existing Nuclear Power Plants in the United States (2, 20).

2.0 AUGMENTED SEISMIC DESIGN REQUIREMENTS

The following discussion reviews the influence of new seismic design requirements on the most important seismic inputs used for the design of nuclear plants.

2.1 Maximum Ground Acceleration

Since 1971, Appendix A of 10CFR Part 100 (3) has outlined the geotechnical surveys and procedures necessary to obtain licensing for a nuclear power plant in the U.S. This document further describes "Procedures for determining the quantitative vibratory ground motion design basis at a site due to earthquake".

Two different magnitudes of earthquake (OBE, SSE) along with the procedure for determining their associated maximum ground accelerations are specified.

- The high magnitude, termed the "Safe Shutdown Earthquake" (SSE) corresponds to the maximum vibratory ground motion that the site can ever experience. The lower bound of the corresponding maximum ground acceleration has been set to ten percent of gravity (0.1g).
- The low magnitude termed the "Operating Basis Earthquake" (OBE) corresponds to the maximum vibratory ground motion which can reasonably be expected to affect the site during the plant lifetime.

  The lower bound of the corresponding maximum ground acceleration has been set to five percent of gravity (0.05 g). The OBE is further defined as equal to at least one half of the SSE.

During and after SSE, Nuclear Plants are only expected to shut down safety and definitely, whereas continued safe operation is expected after OBE. Consequently SSE allowables are more permissive than OBE.

OBE induced displacements of structures and equipments are presumably lower than SSE displacements. Consequently lower material and structural damping is associated to the OBE, leading to larger response amplification.

The above definitions and corollary considerations infer at least two major departures from previously used design requirements and practices:

- Previous analysis did not consider building response nor different OBE and SSE response amplifications. As a consequence, the ratio between OBE and SSE was an almost constant 0.5 wherever in the building, and the SSE responses were limiting.

  Now that OBE evaluation is conducted independently, with lower material and structural damping than those used for SSE, the relationship between the two earthquake magnitudes may become as unfavourable as:

  $$OBE = 0.8\ SSE,$$

  if transposed at the equipment levels (i.e., the Reactor Coolant Loop).

  Since OBE allowables are lower than SSE, the OBE responses have now become limiting for many of the equipments. This has been demonstrated and quantified by Stevenson (4) for:

  a) Safety Class Concrete structures other than containment.

  b) Safety Class Piping.

  Calculating the old plants OBE responses according to the new philosophy is a matter to be considered in a plant seismic requalification.

- In the past, Nuclear Plants located in extremely low seismic areas were not qualified for earthquake events.

  Now that lower bound have been set to OBE and SSE maximum vibratory ground acceleration, all plants in all countries following U.S. regulations must be seismically qualified. This requirement also generates a matter for requalification. Its recent application has even produced gross inconsistency in low seismic areas where newly licensed plant designed for 0.1g SSE neighbors an older facility which was not seismically qualified.

2.2 Design Ground Response Spectra

The response spectrum developed for a historical earthquake consists of peaks and valleys. Shown in Figure 1 is a 2 percent damping response spectrum for the El-Centro 1940 earthquake E-W component, normalized to 0.2g maximum ground acceleration. The peaks and valleys of the response spectrum are different for each different earthquake. Since each ground motion may be viewed as a sample from a population of random functions, it is meaningful to obtain a statistical average of the response spectra for a number of strong earthquakes. Generally, such an average response spectrum is a smooth curve. One of the earliest attempts in creating such spectra was made by Housner (5). The response spectra generated by Housner were averaged over four strong earthquakes using two components of each (that is, a total of eight seismic accelerograms).

Because they are the average of four strong earthquakes that have occurred in the Western United States, however, there are concerns that Housner's Response Spectra may not be adequate to represent all typical strong earthquakes, which may occur in the future. Consequently, more research was carried out (6), (7), (8) and (9), leading to the proposal made by Newmark Blume and Kapur (10).

Regulatory Guide 1.60 (11) which describes a procedure for defining response spectra is tailored to NBK's recommendations (7).

Figure 2 shows the comparison of the Housner spectrum (5), the NBK spectrum (10) and the RG 1.60 spectrum (11). Large differences can be noticed, between the response spectra recommended by Housner and those recommended by Newmark and Regulatory Guide 1.60. While the Housner spectra were averages of four strong earthquakes with two components each, the Regulatory Guide 1.60 spectra were obtained using the one sigma plus the mean value for a total of 33 recorded earthquakes. Consequently, the latter corresponded to a much higher confidence level, that is, their probability of exceedance is much less. In fact, it can be calculated that the probability for future earthquakes not to exceed the Regulatory Guide 1.60 is approximately 84 percent rather than 50 percent for the Housner spectra.

Housner response spectra have been used as the design basis for a number of nuclear power plants.

Going to Regulatory 1.60 spectra might be a matter for requalification for those plants.

2.3 Three Components of Earthquake

The earlier philosophy was to consider one of the horizontal response spectra component in conjunction with a vertical response spectrum equal, in most cases, to two-thirds of the horizontal.

Instead, the Regulatory Guide 1.92 (12) required that three orthogonal components of the response spectra be applied in the nuclear power plant design and analyzed on a simultaneous basis. The magnitude of the vertical component is now equal to the horizontal in the frequency range where most of the significant systems respond.

Figure 3 shows the comparison of the horizontal and vertical components of the Regulatory Guide 1.60 spectra.

3.0 SEISMIC REQUALIFICATION

Cloud (13) has indicated the method preferably used in the author's organization to analyse nuclear plants for seismic events. This approach is sketched in Figure 4. Each block of that diagram represents a major analysis task the description of which has been covered by Cloud (13). They will be summarized in further paragraphs 3.2 to 3.5 for completeness.

3.1 Need for Time History Analysis

The design response spectra (see 2.2) are suitable inputs only for an analysis where the response can be represented by a combination of normal modes such that each modal response is a function of its response spectral value at the modal frequency.

This analysis procedure, termed responsed spectrum modal analysis, although largely used in the past, has two major drawbacks:

- For systems which possess high degree of nonlinearity such as nonlinear material property, friction, gaps, one-way active supports, it is difficult to conduct a response spectrum modal analysis without the extensive use of linearizations and iterations.
- It would not be economical nor reasonable to perform in one single run the total seismic analysis of a nuclear plant from the free field motion to the piping and equipment response. That is why the procedure of Figure 4 exhibits several steps. This approach necessitates however, the development of seismic excitations, either floor accelerograms or floor response spectra, at the various elevations of the building structure where equipment is supported.

  Empirical method (14) exists to derive response spectra at a point of structure from the response spectra applied at the base and the modal responses of the structure. However, the application of that method is restricted by many conditions and its repeated use leads to overly conservative results.

The need for more realistic and reliable seismic responses combined with the development of capable structural computer codes (3.5) has favored the implementation of Time history analysis technique in the nuclear industry.

3.2 Generation of Synthesized Time History Motions

For the purpose of carrying out Time history seismic analysis, Time history motions consistent with the design ground response spectra (2.2) are required.

The generation of the spectra consistent time history motions is achieved by the repeated modification of an actual earthquake motion using the spectrum suppression and spectrum raising techniques (15). Spectra raising is accomplished by adding to the original time history a harmonic function at the frequency of interest. On the other hand, in order to lower the amplitude of the response spectrum computed from the time hsitory motion, the motion is passed through frequency suppressing filters.

Figure 5 shows a typical example obtained using this approach.

Since Regulatory Guide 1.92 (12) requires that the analysis of a nuclear power plant be conducted using three components of the design earthquake motion, the synthesized time history must possess characteristics similar to the actual recorded earthquake.

The two techniques described do not alter major features of the time history record. Figure 6 compares the auto-correlation functions of the modified time history components and the original records. Moreover, the normalized cross correlation functions shown in Figure 7 are small. The synthesized time histories, therefore, are uncorrelated or statistically independent.

3.3 Soil Structure Interaction

The design response spectra (2.2) and the spectra consistent time histories (3.2) are defined at the free field. With the presence of a massive structure, such as a nuclear power plant, the free field motion cannot be applied directly at the base mat to determine the structural response. Instead, a soil structure interaction analysis has to be performed. It has been found, for instance (16) that the response motion at the base mat of the structure may have large reductions at the natural frequency of the structure on a fixed base. This reduction is typically due to the fact that the motion surrounding the base mat is greatly affected by the dynamic shear force resulting from the structural response.

The soil structure interaction analysis method indicated in Figure 4, blocks 2 and 3, is based on finite element analysis and uses the computer programs SHAKE and FLUSH.

In its original formulation, developed at the University of California by Seed and Lysmer (17), (18), the theory is based upon the notion that the nuclear plant is some distance removed from the epicenter of the earthquake, and further that the earth is composed of reasonable stiff bedrock overlaid by a layer or layers of softer strata or soils. The theory is that the earthquake energy and primary motion is propagated through the bedrock, which forms the foundation for the soil. The motion in turn reaches the earth's surface (and the nuclear plant) by vertically propagating shear waves. The method has been generalized somewhat since the original inception, but the basic idea remains as described.

Two sets of analysis models are utilized. The first model consists of a soil column from the location of the free field or control motion to a depth of sufficiently competent strata to be considered bedrock. By performing an iterative time-history dynamic analysis of this soil column, accelerograms at the bedrock can be found which produce the desired design control motion. An iterative solution is required because soil properties are a nonlinear function of strain and the properties for each finite element must be consistent with the strain range in that element. The process of establishing a dynamic motion in bedrock that corresponds to a given spectra at the surface is referred to as deconvolution (Block 2). A second set of models are developed. These finite element models consist of at least two dimensional (2-D) mutually perpendicular slices through the nuclear plant, including simplified building representations and extending down to the bedrock. A simplified representation of such a model is shown in Figure 8.

Dynamic time histories performed on these models with the bedrock motion produce for each building a five component base motion time history.

A more than satisfactory experimental verification of the method was obtained when the Ferndale earthquake of 1975 shook the Humboldt Bay Nuclear Plant (19). Full scale experimental correlations are not available for other soil structure interaction analysis methods.

3.4 Building and Equipment Evaluation

Models used in soil structure interaction analysis are much too simplified to furnish adequate information for detailed evaluation of buildings.

Therefore two analyses using the previously established base mat excitations are performed:

- One analysis is made on a model involving a simplified representation of the reactor coolant loop and a very detailed modeling of the building and interior concrete. This model is mainly composed of building elements, slabs, floors, walls of concrete and is designated as building dominant (Bloc 4 of Figure 4).
- The other analysis is made on a model involving a detailed representation of the reactor coolant loop and a simplified representation of the building. This model is composed of primary equipment, piping, supports and will be nonlinear to account for one way restraints, gaps, etc. It is designated as equipment dominant (Block 6 of Figure 4).

With this approach the following benefits are achieved:

- Models have reasonable size
- Detailed output are available
- Building to equipment coupling is represented as well as interfloor differential excitation
- Cross check on the analysis is made possible

The analysis performed on these models are time history dynamic analysis considering full three-dimensional input and response.

Results from the building dominant model are the stress resultants, moments, shears, etc., at all key points to be used for building evaluation and amplified floor response spectra throughout the building for response spectra analysis of piping and equipment. A similar building analysis is performed for all seismic category buildings on the site.

Results from the equipment dominant model are moments, forces, etc., for piping evaluation and time history or response spectra for a more in-depth evaluation of each of the primary components.

As an example, Figures 9 and 10 show the impact forces on two opposite lateral supports of one of the steam generators and figure 11 shows the steam generator displacement. By superposition of these figures, one can see the displacement and support impact forces are absolutely consistent (considering also that a 0.059 in. gap was assumed at each support).

3.5 Computer Codes

There would be no implementation of modern seismic analysis method without capable computer codes and computer code development would be unconceivable without the steady urge from the industry for always more sophisticated analytical tools. This discussion would be uncomplete if it failed to describe one of these codes.

The computer program called WECAN used in the author's organization is based on the finite element method of analysis. It is used to efficiently solve a large variety of nonlinear static and dynamic structural analysis problems which can be one, two or three-dimensional in nature.

WECAN is equipped with a system of pre- and post-processors, called WAPPP, developed to provide efficient input and output processing.

For the purpose of qualifying complex structural systems such as reactor coolant systems subjected to earthquake ground motions, several features of the code can be used depending on the need:

- Modal and transient dynamic analysis
- Response spectra generation and calculation of total response for a 3D-earthquake input.

The five basic types of structural analysis relevant to the seismic analysis capability are:

- Static analysis
- Modal and Response Spectrum Analysis
- Harmonic Analysis
- Linear Dynamic Transient Analysis
- Non-linear Dynamic Transient Analysis

The code is also furnished with a comprehensive library of finite element types for structural analysis. These elements have been verified and qualified for use in analysis of large and complex structure systems.

WECAN has also been verified and qualified for static and dynamic analysis to determine forces, stresses and deformations of systems. For the seismic analysis, those procedures have even been reinforced by extensive documentation in the form of verification and demonstration manuals.

4.0 CONCLUSION

An analytical procedure (Figure 4) of seismic qualification of nuclear power plant has been discussed. The advantages of using that procedure are several:

- Time history analysis is more realistic and accurate. In fact, as demonstrated when superposing Figures 9, 10 and 11, it is an analytical experiment since all the major parameters throughout the system can be checked for physical consistency.
- By following the same time history motion from the free field to the equipment, the conservatism associated with the use of response spectra vanishes. That is:
  - Combination of directional responses
  - Consideration of closely spaced modes
  - Approximate method for considering differential support motion (21)
  - Empirical method for calculating response of subsystems (14)

  The application of such procedures, implementing verified and qualified modern techniques, is one of the key items to the successful requalification of existing nuclear power plants.

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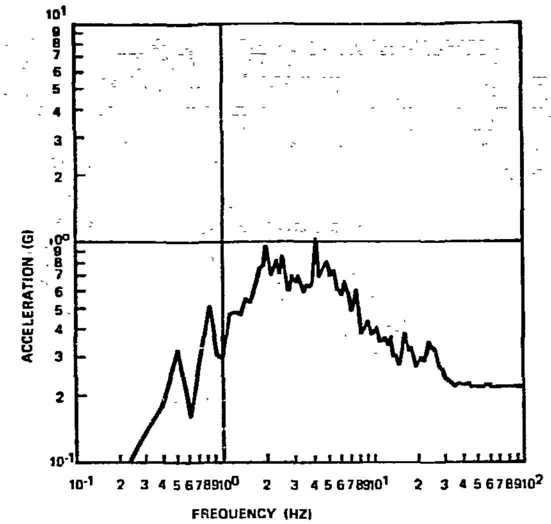

FIGURE 1
EL CENTRO E-W TIME HISTORY RESPONSE SPECTRUM
FOR 2% DAMPING

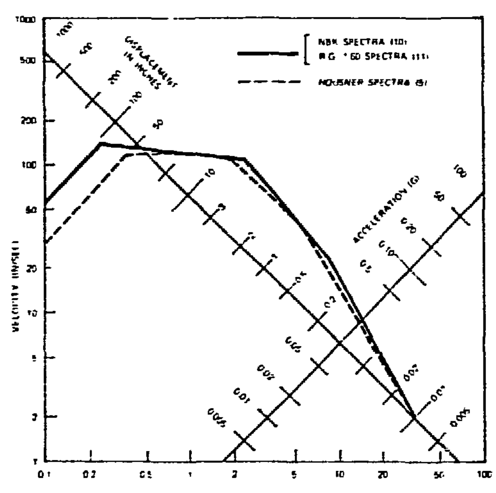

FIGURE 2
COMPARISON OF DESIGN SPECTRA
FOR 2 PERCENT DAMPING

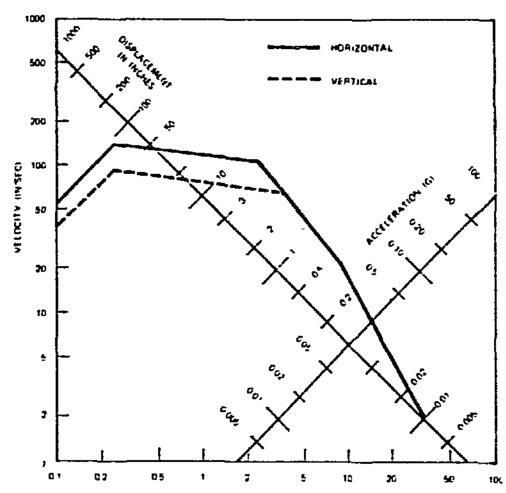

FIGURE 3
COMPARISON OF R.G. 1.60 HORIZONTAL
AND VERTICAL DESIGN SPECTRA
– 2 PERCENT DAMPING

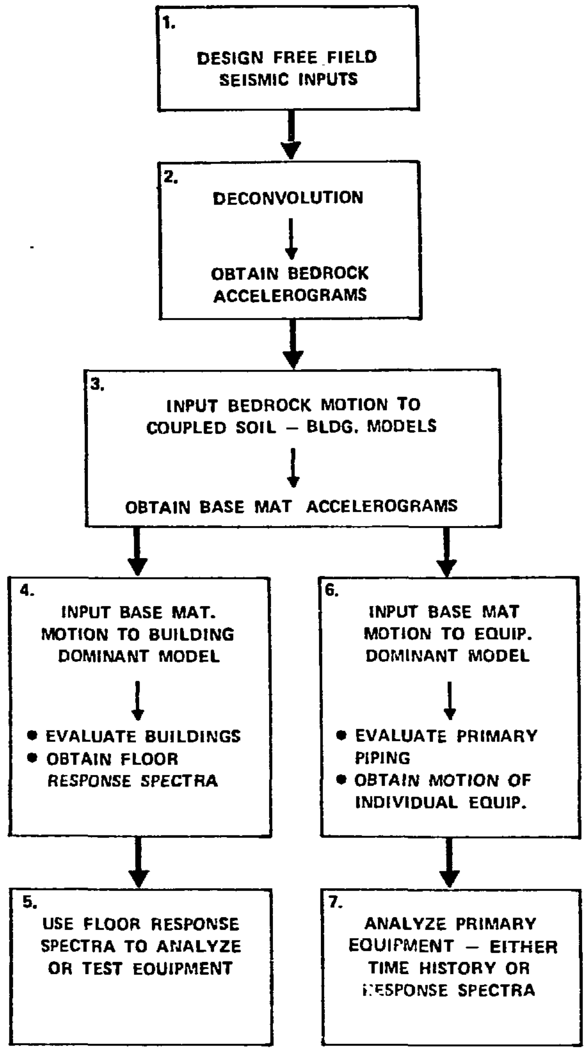

FIGURE 4

PROCEDURE FOR NUCLEAR PLANT SEISMIC ANALYSIS

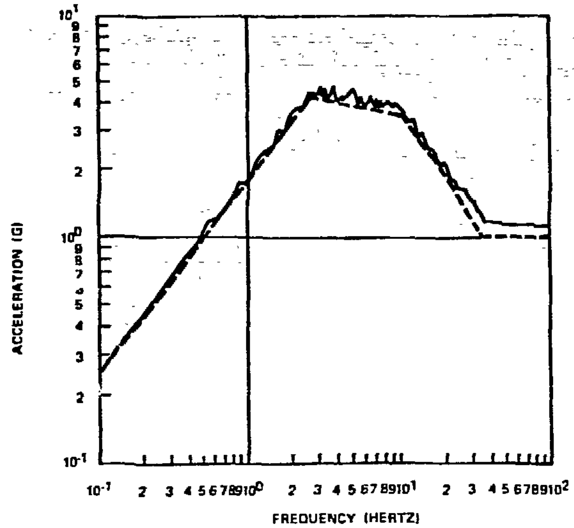

FIGURE 5

FIGURE 5

NCR REGULATORY GUIDE 1.60 DESIGN RESPONSE SPECTRUM AND SYNTHESIZED TIME HISTORY RESPONSE SPECTRUM FOR 2 PERCENT OAMPING

FIGURE 6

COMPARISON OF THE TEMPORAL AUTO-CORRELATION FUNCTIONS, E-W COMPONENTS

FIGURE 7

NORMALIZED CROSS-CORRELATION FUNCTIONS FOR MODIFIED N-S AND E-W COMPONENTS

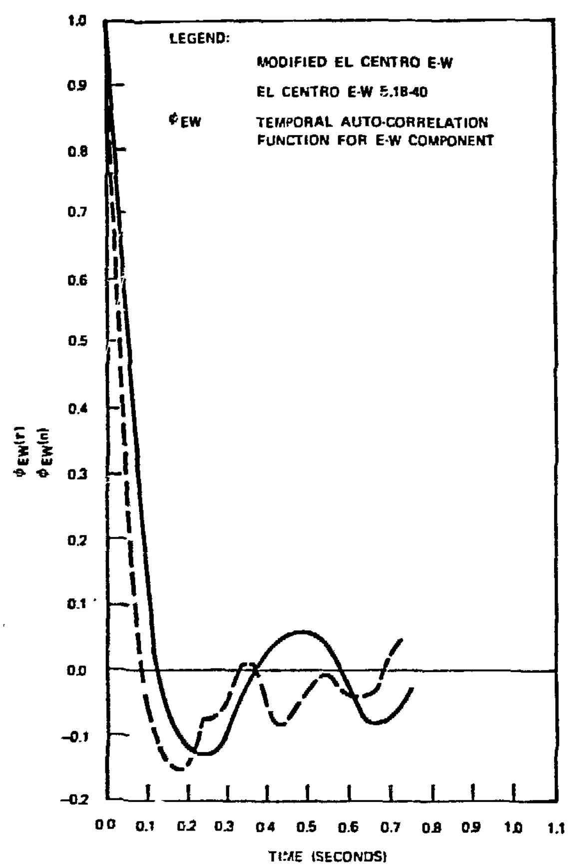

FIGURE 6

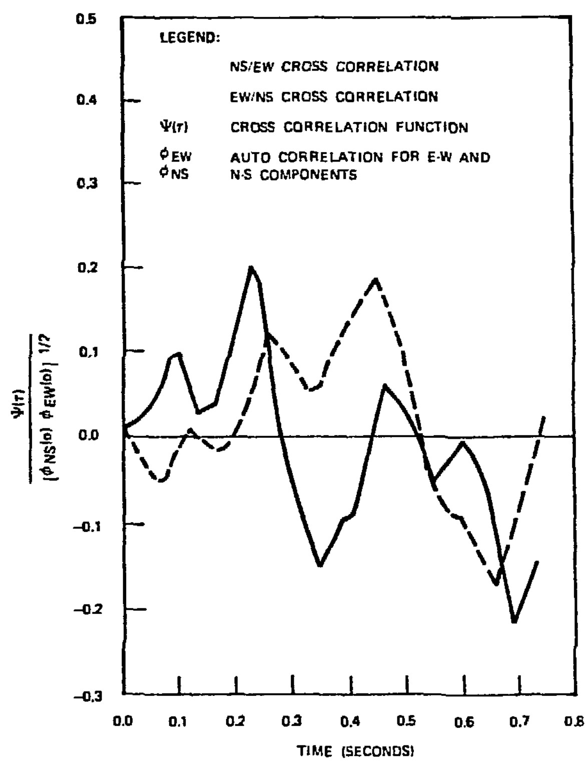

FIGURE 7

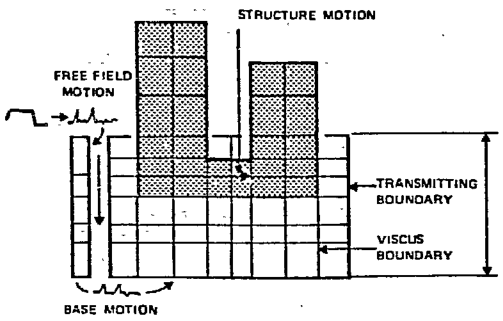

FIGURE 8

SOIL-STRUCTURE INTERACTION

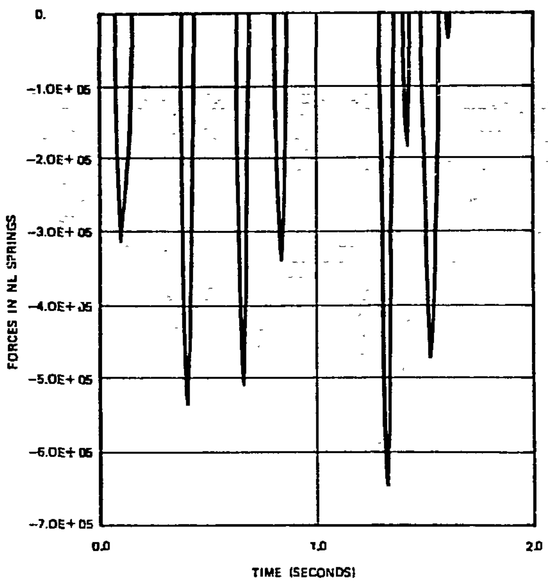

FIGURE 9

FORCES IN STEAM GENERATOR UPPER SUPPORT (−Z) BUMPER LOOP–2

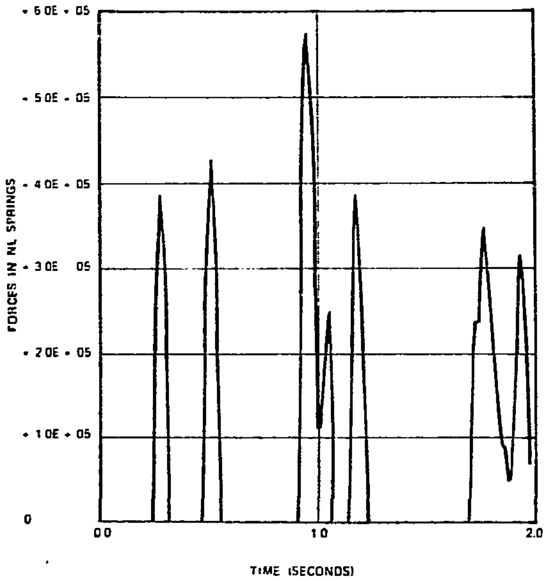

FIGURE 10

FORCE IN STEAM GENERATOR UPPER SUPPORT (+Z) BUMPER LOOP–2

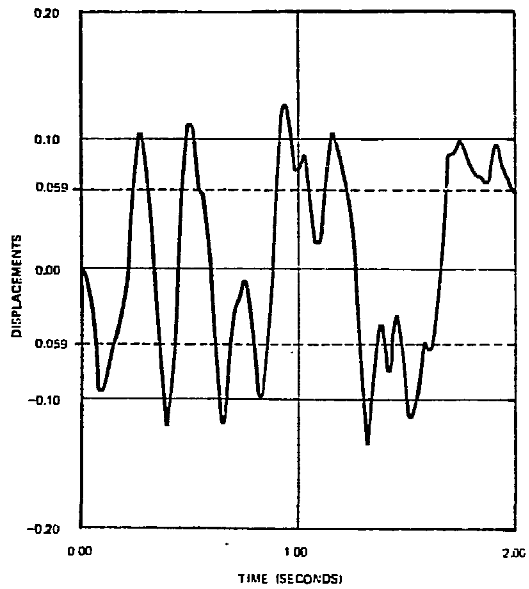

FIGURE 11

Z DISPLACEMENT OF LOOP 2 STEAM GENERATOR OF UPPER SUPPORT ELEVATION