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# A REVIEW OF SOIL-STRUCTURE INTERACTION EFFECTS IN THE SEISMIC ANALYSIS OF NUCLEAR POWER PLANTS

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## SUMMARY

A survey of recent investigations of soil-structure interaction in seismic analysis of nuclear power plants is presented. This survey is divided into two major sections: analytical methods and results of investigations. After a brief outline of solutions based on spring mass models, half space models and finite element procedures, results of various investigators are presented and in some cases compared.

The most common method of analysis is based on spring-mass model of the building structure and major internal components. Soil properties are represented by equivalent springs. Since the soil response depends on frequency, the soil spring properties are based on the fundamental mode of vibration. The response of spring mass models is evaluated using either time history analyses or spectrum techniques. There is some evidence that the use of spectrum techniques leads to needlessly conservative results because of the addition of modal forces. Time history analyses indicate that the fundamental rotational mode of the containment structure and the fundamental lateral mode are out-of-phase. Thus, seismic inertia loads of the higher mass points of the containment vessel structure associated with these modes do not act in the same direction. The influence of the variation of soil-stiffness with frequency in the spring element model can be included using methods of operational calculus or normal mode procedures.

Finite element methods have also been used to analyze this phenomenon. The soil is usually modeled as a two-dimensional half space; the structure is either modeled with finite elements or a lumped mass model. Two methods have been used to input ground motion to the structure: a time history at one of the vertical boundaries and a rigid time-dependent motion over the bottom boundary of the soil elements. Differences in responses of the structure using these two models is significant. In fact, different conclusions can be made about soil-structure interaction depending upon this difference in analytical model. In the completed paper this problem is discussed in detail.

In conclusion, suggestions for additional analytical work are made. Also, procedures that can be used to confirm analytical methods are proposed.

## 1.0 INTRODUCTION

The subject of soil-structure interaction of buildings subjected to seismic excitation has received increasing attention during the past ten years. Volume 1 of the Abstract Journal in Earthquake Engineering (Wagy [1]) lists 41 papers published on this topic in 1971. Many significant papers have been published since that time and more contributions will be presented at the Fifth World Conference on Earthquake Engineering. Because of the number of investigations on this subject, it is not the objective of the authors to briefly discuss the contribution made in each investigation. Instead the objective of this paper is to review the various analytical methods used to study this phenomenon, discuss some of the significant effects on the structural response of nuclear power plants and to suggest additional experimental and theoretical investigations.

Numerical work in most studies investigate the response of conventional building structures. Containment vessels of nuclear power plants are massive and stiff relative to other structures of similar heights. Because of these two characteristics of nuclear power plant containment vessels, soil-structure interaction effects are particularly significant and must be considered in seismic analyses.

## 2.0 METHODS OF ANALYSIS

One of the most widely used methods of analysis is to model the structure as an N-mass system and the response of the soil by equivalent soil springs (Bohm [2]). If two-dimensional normal mode theory is applied to the N-mass system, the reactions to the soil by the structure can be written as a function of the transient foundation translation u(t) and rotation θ(t) as (O'Hara [3], Bailey [4], and Jennings [5]).

$$F(t) = -\sum_j M_j\omega_j \int_o^t \ddot{u}(\tau)\sin\omega_j(t-\tau)d\tau - \sum_j Z_j\omega_j \int_o^t \ddot{\theta}(\tau)\sin\omega_j(t-\tau)d\tau \qquad (1)$$

$$M(t) = -\sum_j Z_j\omega_j \int_o^t \ddot{u}(\tau)\sin\omega_j(t-\tau)d\tau - \sum_j I_j\omega_j \int_o^t \ddot{\theta}(\tau)\sin\omega_j(t-\tau)d\tau \qquad (2)$$

where $M_j$, $Z_j$, $I_j$ are the modal mass, first modal moment and second modal moment, respectively. Structural damping can be approximated by including the term $e^{-\zeta_j\omega_j(t-\tau)}$ in the integrals. The modal constants are defined as

$$M_j = \frac{(\sum_i m_i \overline{X}_{ij})^2}{\sum_i m_i \overline{X}_{ij}^2} \tag{3}$$

$$Z_j = \frac{(\sum_i m_i h_i \overline{X}_{ij})(\sum m_i \overline{X}_{ij})}{\sum_i m_i \overline{X}_{ij}^2} \tag{4}$$

$$I_j = \frac{(\sum_i m_i h_i \overline{X}_{ij})^2}{\sum_i m_i \overline{X}_{ij}^2} \tag{5}$$

In eqs. (3) to (5), $m_i$ are the concentrated masses, $\overline{X}_{ij}$ are the modal displacements of $m_i$ for mode j and $h_i$ is the height of $m_i$ above the foundation. Interaction equations are often developed by relating foundation forces through a compliance matrix (Jennings [5], Hradilek [6])

$$\begin{Bmatrix} F \\ M \end{Bmatrix} = \begin{bmatrix} K_{ff} & K_{fm} \\ K_{fm} & K_{mm} \end{bmatrix} \begin{Bmatrix} u - u_g \\ \theta \end{Bmatrix} \tag{6}$$

where $u_g$ is the free-field motion

$$K_{ff} = k_{ff} + ia_o\, C_{ff}$$

$$K_{fm} = k_{fm} + ia_o\, C_{fm}$$

$$K_{mm} = k_{mm} + ia_o\, C_{mm}$$

and $k_{ij}$ and $C_{ij}$ can be interpreted as the effective stiffnesses and effective viscous dampers of the frequency - dependent soil response (Matthiesen [7], Arnold [8]). One of the theoretical difficulties with this method of analysis is that the elements of the matrix are frequency dependent. Often, interaction effects are dominated by forces associated with the fundamental mode (Jennings [5]). As a result, the values of these elements are often approximated by the fundamental frequency. Also it is found that the effects of the off-diagonal elements are not always significant (Jennings [5]). Thus, by neglecting these elements the interaction equations can be statically uncoupled. These simplified soil spring models have been used for both time-history studies and spectrum analysis. A parametric study of nuclear reactor buildings using spectrum methods is presented by Whitman [39].

Much attention has been given to the elimination of approximations employed in the solution to eq. (6). Bielak [14] derived the Laplace transform of the compliance matrix.

$$\begin{bmatrix} \dfrac{\overline{F}(s)}{\mu a^2} \\ \\ \dfrac{\overline{M}(s)}{\mu a^3} \end{bmatrix} = \begin{bmatrix} K_{ff}(s_o,\sigma) & K_{fm}(s_o,\sigma) \\ \\ K_{mf}(s_o,\sigma) & K_{mm}(s_o,\sigma) \end{bmatrix} \begin{bmatrix} \dfrac{\overline{u}(s)}{a} - \dfrac{\overline{u}_g(s)}{a} \\ \\ \overline{\theta}(s) \end{bmatrix} \qquad (7)$$

Using this approach the frequency characteristics of the transient motion will not affect the accuracy of the solutions. Lee and Wesley [9] make use of Fourier transform methods to solve the transient problem of a three dimensional structure. Thus a 6 x 6 compliance matrix is obtained. It was also concluded in this investigation that only the diagonal terms are significant. Rainer [10] also employed Fourier transform technique in a study of an equivalent single-degree-of-freedom system. Fast Fourier transformation computations are often employed to obtain the inverse transformation. Lee and Wesley [11] extended the methods of Ref. [9] in order to investigate the effects of through ground coupling between adjacent structures. This solution is based on the work of Warburton [12].

The use of Fast Fourier Transform techniques in the analysis of the soil-structure interaction phenomenon are outlined by Liu and Fagel [13]. For a general soil-structure system with frequency-dependent coefficients the equations of motion can be expressed as

$$[M]\{\ddot{u}(i\omega)\} + [C(i\omega)]\{u(i\omega)\} + [K(i\omega)]\{u(i\omega)\} = \{F(i\omega)\} \qquad (8)$$

Fourier amplitude spectra of strong motion earthquakes have been tabulated in Volume II of Ref. [38]. One of the difficulties with this formulation is that the product of the transformed stiffness matrix times the arbitrary motions must be evaluated using the convolution theorem or from the governing equations as indicated by Jennings and Bielak [5, 14].

The authors [15, 16] made use of the response of a plane-strain half space to an arbitrary surface shear forces F(t) in order to eliminate the frequency dependent effects of the soil springs. The displacement of the surface can be expressed as

$$u(t) = -\frac{b}{\mu}\int_0^t \frac{F(\tau)}{A}\,d\tau - \frac{b^2}{2\pi c\mu}\int_0^t\int_0^{t-\tau} \frac{F(\zeta)}{A}\, I_m g\left\{\frac{b\tau}{c}\right\} d\zeta\, d\tau \qquad (9)$$

Using eq. (1), the surface force F(t) can be eliminated from eq. (9) and an integro-differential equation developed. Free-field ground motion is introduced by adding $u_g(t)$ to eq. (9). In this approach rocking of the foundation is not considered. However, the combined lateral-rotational problem has been formulated (Bailey [4]) using plane-strain solutions of the Lamb problem. In addition to studying the laterally excited interaction problem using this method, vertical ground structure action has also been studied (Little et al [40]).

Duke [17, 38] applied linear systems techniques to evaluate the soil-structure interaction phenomenon. Amplification between bedrock and the free-field surface is also included in the formulation. Transfer functions of the soil-structure interaction are based on the work Luco [6, 18]. Analytical results are compared to data recorded in the Hollywood Storage Building during the 1952 earthquake.

Finite element methods have also been employed in the study of this phenomenon. Chiapetta [19] used the SLAM program which is a stress wave propagation program to study the influence of a lumped mass structure on seismic motion. In a more extensive study, Isenberg [20] investigated the effects of various soil stiffnesses and input motions with a similar dynamic model of a nuclear power plant. In both cases these analyses are based on two-dimensional plane strain elements. A typical model is indicated in Fig. 1. More recently three dimensional finite element theories have been applied to the analysis of the combined problem of soil-amplification and soil-structure interaction problems. Wilson [24] has used an axisymmetric finite element with a quadrilateral cross-section. By expanding the displacements of the elements in N terms of sine and cosine series

$$
\begin{aligned}
u_r(r,z,\theta) &= \sum_{m=o}^{N} U_{rm}(r,z)\cos m\theta \\
u_z(r,z,\theta) &= \sum_{m=o}^{N} U_{zm}(r,z)\cos m\theta \qquad (10) \\
u_\theta(r,z,\theta) &= \sum_{m=o}^{N} U_{\theta m}(r,z)\sin m\theta
\end{aligned}
$$

and applying the theorem of minimum potential energy, the resulting equilibrium equations are developed for N sets of uncoupled two-dimensional problems. Brandow [22] uses three-dimensional rectangular elements to study the combined problems of soil-amplification between bedrock and the surface and the soil-structure interaction phenomenon. Options in the program make it possible to reduce the number of degrees of freedom of the stiffness matrix in order to reduce computational costs. In some recent work nonlinear linear finite elements are being employed in seismic analyses. A numerical algorithm is described by Wilson [24]. Some applications of non-linear analyses to nuclear design problems are presented by Bohm [2].

Two methods have been used to input seismic motion to the finite element mesh. In studies by Chiapetta [19] and Isenberg [20], time history motion is prescribed along the vertical side. In Studies by Wilson [21], Brandow [22], and Isenberg [23] horizontal seismic input motion is specified on the bottom boundary of the mesh. Moreover, the bottom boundary is usually taken to be rigid. In the first method, an attempt is made to simulate seismic waves propagating along the surface of the earth; in the second case, the bottom boundary of the mesh is assumed to be bedrock with wave propagation velocities that greatly exceed those of the covering earth. Calculated forces in the structures are dependent on the method of input.

Numerous vibration tests have been conducted on conventional building structures (i.e. Jennings [25, 26]). Recently shaker tests have been conducted on full-sized nuclear power plants (Matthiesen [27], Smith [28], Ibanez [29], Matthiesen [30], Shanman [31], and Olsen [32]). Ambient vibration as well as shaker tests have been employed to study the dynamic characteristics of the CVTR [33]. Also, some testing has been conducted using explosives to simulate earthquake ground motion (Chrostowski [34]). There are a few studies based on actual earthquake motions recorded in nuclear power foundations (Matthiesen [30], Muto [35], Housner [36]). Muto and Omatsuzawa [35] have used motion recorded at the foundation to calculate the response of the building structure and power plant components. Housner [36] and Matthiesen [30] have considered response motions at the San Onofre nuclear plant. Similar studies have been conducted on building structures (Muto [37]). The San Onofre nuclear generating station has been instrumented to record free-field motion and foundation motion simultaneously after measurements were made during the 1968 Borrego Mountain Earthquake (Housner [36]). Subsequently, data has been recorded during the 1971 San Fernando earthquake and the 1970 Lytle Creek earthquake. Emphasis in the analysis of the data has been placed on the study of the response of major components of the primary system. However, some preliminary results have been obtained on the ground-structure interaction phenomenon (Vasudevan [41]).

## 3.0 DISCUSSION OF RESULTS

Both experimental and analytical results show that ground-structure interaction effects may alter lateral and vertical free-field motions and initiate rocking of the structure. Before summarizing analytical results, a summary of experimental work will be presented.

### 3.1 EXPERIMENTAL OBSERVATIONS

Data from the Hollywood storage building and the adjacent parking lot provides some of the best experimental data of the soil-structure interaction phenomenon. The plan of this 14-story building is rectangular (51' x 217') with the long dimension in the east-west direction. Ground motion recorded in a parking lot (112 feet west of the structure) is used as free-field input motion; simultaneous response measurements are made in the basement and roof. Response spectra of the lateral east-west basement motion and the lateral north-south basement motion have been calculated by Housner [42] and compared to free-field values. In the east-west direction the fundamental frequency is approximately twice the north-south value. The fundamental period in the east-west direction is approximately 0.5 seconds. In this frequency range the east-west spectrum response of the basement motion is reduced by approximately 40%. In the more flexible north-south direction, lateral interaction affects are not significant. However, interaction causes foundation rocking which must be considered in order to predict the dynamic north-south building response. The difference between the north-south and east-west lateral interaction can be explained by the frequency dependence of the building inertia forces (eq. (1)). The reaction of the structure to the foundation increases with the fundamental frequency.

Duke [17] compared the field data obtained from the lateral east-west motion of the Hollywood Storage building to analysis using the Fourier transform method. A comparison in the frequency domain is presented in Fig. (2). Overall agreement is shown to be good.

Recently, free-field and structural response data has been recorded at the San Onofre nuclear power plant (Vasudevan [41]). A complete analysis of this structure-ground system has not been reported to date. However, a preliminary comparison of free-field data measured 500 feet south-east of the containment vessel and basement response data using the Fourier modulus indicates an amplification of ground motion at the basement foundation level (-10ft elevation). Thus, these data are not in agreement with observations made at the Hollywood storage building or of the analyses of this phenomenon. It must be emphasized that this ground structure system has not been analyzed in detail. Torsional effects or through ground coupling between the containment vessel and the adjacent turbine building structure could affect results. Also, the reported free-field measurement is measured adjacent to the cliff of the Palisades. Undoubtedly, the location of this instrument affects the "free-field" measurement. A thorough analysis of this nuclear power plate including site conditions would lead to a better understanding of this phenomenon.

Dynamic characteristics of the containment vessels of pressurized water reactors have been obtained from shaker tests. Typical fundamental frequencies of containment vessels vary from 4 cps to 6 cps. (Matthiesen [27], Smith [28], Ibanez [29], Matthiesen [30], Shanman [31], Olsen [32]). Measured values of viscous damping is found to be high (15% to 20% of critical). This damping is caused by radiation of energy from the structure to the ground as well as soil friction losses. Fundamental frequencies of major components of the primary system are also in this same frequency range (2 to 6 cps).

The containment structure of the Fukushima Nuclear Power Plant studied by Muto [35] is different from most United States designs. The primary containment vessel is much smaller. However, the fundamental frequency of the five story building structure which contains this , structure is also approximately 4 cps. Effective damping constants associated with fundamental modes of the building were found to be approximately 30% of the critical value.

In addition to the shake tests and the ambient vibration survey conducted on the CVTR, an accurate mathematical model of the plant was developed [33]. Originally, the fundamental building frequency determined from the model was above experimental values. In order to obtain agreement, it was necessary to neglect a lateral soil spring, $K_s$, in the model that was intended to account for embedment (Fig. 3). Also, the stiffness of the rotational and torsional effective soil springs had to be reduced. However, these high stiffness values are probably associated with the high shear wave velocity (3300 ft/sec) used in the analysis. Subsequent studies measured shear wave velocities of 2300 ft/sec for the underlying soil and 800 ft/sec near the surface, (Matthiesen [27]). Results of this investigation also indicate through-ground coupling between adjacent structures. The superheater located approximately 75 feet from the containment vessel affected the response of the containment vessel. Excitation of torsional vibration modes caused by the lack of symmetery was also found to be significant and a three-dimensional model is recommended.

3.2 ANALYTICAL RESULTS

Investigations of soil-structure interaction effects on the seismic response of the nuclear containment building structure show that inertia forces acting on the building are reduced. The responses of structures internal to the building are also altered; seismic forces may be either increased or decreased and the change depends upon the fixed base frequencies of the structure and its location in the containment vessel (Isenberg [20]). One method of calculating the influence of interaction effects is to compare spectrum response curves of the structure with and without soil-structure interaction included in the model. Some investigators have used building shear loads to show changes; others have tabulated relative displacement of a single degree-of-freedom system (which is often defined as the displacement spectrum).

There seems to be some confusion in the literature concerning the use of the true natural frequency of the structure (which includes the effects of soil flexibility) and the fixed-base natural frequencies. If the free-field motion is used with a mathematical mode which includes effective soil-springs it is the characteristics of the motion at the true natural frequencies that are significant in the spectrum response curves. On the other hand, if the foundation or basement motion is employed the fixed-base frequencies (structural frequencies obtained with a rigid foundation) are the significant values in the foundation response curves. The fixed-base frequency has been used in this manner for a number of years in naval shock design methods (Belsheim [46]). These conclusions are based on normal mode equations (O'Hara [3]).

The fundamental mode of vibration of a structure is reduced by interaction effects. Typical results are presented by Parmelee [43, 44] and Rainer [45]. Parmelee [44] also investigated the effects of interaction by calculating the shear load. Results were obtained for 5, 10, 15 and 20 story buildings subjected to the 1940 El Centro earthquake and the 1949 Olympia earthquake. Shear loads are both increased and decreased by interaction effects. Variations also depended upon the input earthquake motions. In general, the shear loads are increased with the taller structures and reduced for the 5 story structure. Fundamental frequencies of the structures are not tabulated.

Rainer [10, 45] studied the response of a base-mass-plus-dynamic mass structural model to motions the 1940 El Centro Earthquake. The effect of interaction on the spectrum responses is shown to be reduced by as much as a factor of eight. The effect on the fundamental frequency and the overturning moment is also studied. Numerous parametric studies were conducted. A portion of the results are presented on Table I. For all cases studied the relative motion between the dynamic mass and base mass decreased as the fixed-base frequency increased. The effect on the overturning moment is more complex. For low structures (20 ft.) the moment decreases with $\omega$ and for tall structures (80 ft.) the value increases with $\omega$. Rainer [45] concluded from this work that "For massive structures with low-height-to-base ratio, the relative base displacement as well as rocking motion influence the frequency reduction and consequently the ground-structure interaction effects are also influenced significantly."

The authors [15, 16] also studied the influence of the lateral foundation motion of a nuclear power plant by comparing the spectrum response curves to the free-field and the foundation motion. Most of the parametric studies were conducted with a dynamic model similar to that employed by Rainer; a single dynamic mass represented the response of the containment vessel and a rigid base mass. Other studies were conducted with multi-mass systems. Typical response curves for the two-mass system subjected to the east-west ground motion of the 1957 San Francisco earthquake recorded at Golden Gate Park are presented on Figures 4 and 5. The fixed-base frequencies of the containment vessel mass are 4.06 cps and 4.44 cps, respectively, on these graphs. It should be noted that the spectrum of the foundation motion is affected significantly at the fixed-base frequency of the containment vessel. For the system with the lower fixed-based frequency, the spectrum response of the foundation motion peaks near 4.5

cps. However, when the stiffness of the system is increased so that the fixed-base frequency is 4.44 cps the spectrum of the foundation motion is suppressed at higher frequencies. This is a general characteristic of the interaction phenomenon [16]; reductions occur at the fixed-base frequency. The effect of soil-structure interaction on the lateral foundation motion was found to depend upon the mass of the structure, the fixed-base frequency and the frequency characteristics of the free-field input. For typical nuclear power plants, seismic forces on the containment vessel and other major components are reduced by a factor of approximately two by interaction effects. These results are in agreement with those obtained by Lee and Wesley [9].

Vertical interaction effects were examined by Little [40]. Vertical fixed-base frequencies of the containment vessel were varied from 12 cps to 20 cps. These frequencies exceed lateral values by approximately a factor of 3. As a consequence, the calculated reduction in the spectrum response of the vertical foundation motion exceeds lateral effects; the percentage reduction for soil stiffness with shear wave velocities which vary between 500 ft/sec and 3000 ft/sec are approximately 80% to 95%.

Through-ground coupling has been studied by Lee and Wesley [11]. Results of this study show that response of the system depend on the soil stiffness, the inertia of the structures, the natural frequencies of the structures and the separation distances. Calculations of the response of structures indicate that maximum response of an equipment structure can be changed by a factor of two by through-ground coupling.

In the finite element studies of the soil-structure interaction phenomenon in which seismic motion is input along the vertical boundary (Chiapetta [19], Isenberg [20]) lateral ground-structure interaction effects are found to be similar to analytical studies. The model employed is similar to the base-mass plus dynamic mass model studied by the authors. In this case an internal structure mass is added to the model. Results are tabulated on Tables II to IV. Ratios of lateral foundation spectrum to lateral free-field spectra at the fixed-base fundamental frequency are tabulated. The ratios indicate a substantial reduction in seismic forces because of ground-structure interaction.

In finite element studies which input seismic motion at bedrock, an amplification of bedrock acceleration at the structure foundation is often calculated. For example, Wilson [24] inputs motion with a peak bedrock acceleration of 10 ft/sec$^2$. Maximum total acceleration at the foundation is approximately 40 ft/sec$^2$ (Fig. 6). These results are stated to be exploratory in nature. Similar amplification was obtained by Brandow [22]. Brandow concluded that this amplification was caused by resonance between the soil medium and the structure.

Furthermore, the soil responded in a manner similar to a single-degree-of-freedom oscillator. Amplification of the response of the soil could be interpreted as a resonance of the soil excited by waves of a particular period close to the fundamental period of the soil.

In a recent study by Isenberg and Adham [23], bedrock motions were input to a finite element mesh in which the element equations of motion included proportional damping (15% in the first mode and 6% in the second mode). This finite element program was derived from programs developed by Wilson. Free-field surface motion (mesh without structure) did not produce the amplification effects observed by Brandow [24]. Results of this finite element analysis are also compared to a soil-spring lumped mass model. The containment structure of the nuclear power plant is embedded in the soil approximately 40 feet. Free-field acceleration spectra are calculated at the surface and at a depth of 40 feet. These values as well as acceleration spectra calculated from the foundation of the structure are listed in Table V. At the fixed-base frequency, there is a reduction in the spectra from free-field surface values. However, there is an amplification of the free-field foundation spectra. Spectrum calculations from the soil-spring model were less than either of these values at the fixed-base frequencies. However, at low frequencies, the spectra calculated using this simple model exceed finite element results. Thus, amplified seismic loads would be predicted for light internal structures with these low frequencies using the soil-spring model.

The effect of the phase relationships between rotational motion and lateral motions on the seismic forces of the containment vessel has been subjected to limited study by the authors [47]. This work is based on the finite element investigations of Isenberg [19] and Chiapetta [20]. Typical results of this study are listed on Table VI. Accelerations of the containment vessel associated with the lateral motion, $\ddot{u}(t)$, are compared to those caused by the foundation rotation $\ddot{\theta}(t)$. Since the maximum structure acceleration $\ddot{X}(t)$ is less than either value, these results indicate that these motions are out-of-phase. Time history calculated motions are presented by Scavuzzo [47]. The implication of these results is that spectrum techniques will lead to unnecessary conservation in the calculation of the seismic forces of the containment structure.

## 4.0 CONCLUSIONS AND RECOMMENDATIONS

It has been established by numerous investigators that ground-structure interaction must be included in the analysis of nuclear power plants. Seismic forces acting on the containment vessel caused by the lateral foundation motion are reduced from surface free-field values and rotational motion are introduced. Forces on internal components mounted in the building structure are altered by changes in the response of the building. A number of analytical methods have been employed in the analysis of this phenomenon. A few investigators have compared results of different methods of computation. Additional study is needed in order to evaluate the significance of employing these different analytical models.

Finite element studies are usually conducted using a plane strain finite-element mesh even though some three dimensional methods of analysis have been developed. Two-dimensional analysis may be adequate. However, the significance of this limitation should be investigated by studying the response of a similar structure with two and three dimensional foundations. Also, the significance of the type of input (bedrock or vertical boundary) on the seismic response of the structure should also be studied. Analytical models should also be compared with data such as that obtained in the Hollywood storage building or at the San Onofre plant.

At present, there is very little experimental data that can be used to evaluate the accuracy of soil-structure interaction mathematical models of nuclear power plants. Priority should be given to obtain these data and evaluating various analytical models. Additional nuclear power plants and conventional building structures should be instrumented so that free-field motions and response motions are recorded simultaneously. There is some indication that torsional modes may be excited in some structures from seismic inputs. Also through-ground coupling may also be significant. Analyses of these phenomenon must also be confirmed with experimental data.

Measurements made at the San Onofre nuclear power plant during the San Fernando earthquake provided a unique opportunity to study the response of a complex plant site. Since free-field motion was recorded adjacent to the palisades, it may not be possible to develop a simple mathematical model and finite element techniques may have to be employed. Modal characteristics of the plant have been established from shake tests. A thorough study of this plant and surrounding terrain should be conducted.

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Table I. Relative Displacement and Overturning Moment Calculated by Rainer [10]

| Fixed-Base Frequency $\omega_o$, rad/sec | Fundamental Frequency $\omega$, rad/sec | Height of Dynamic Mass, h | Shear Wave Velocity, ft/sec | Amplitude of relative displacement at $\omega$ | Overturning Moment at Frequency, $\omega$ |
|---|---|---|---|---|---|
| 5 | 4.75 | 20 | 300 | 21.80 | 24.7 |
| 10 | 8.26 | 20 | 300 | 10.28 | 16.1 |
| 15 | 10.32 | 20 | 300 | 4.29 | 10.2 |
| 20 | 11.43 | 20 | 300 | 2.17 | 7.7 |
| 5 | 3.41 | 80 | 300 | 30.18 | 65.4 |
| 20 | 4.51 | 80 | 300 | 6.17 | 123.5 |
| 10 | 9.71 | 20 | 800 | 23.72 | 25.47 |
| 20 | 17.87 | 20 | 800 | 15.98 | 20.58 |

Base Mass - 1000 lb-sec$^2$/in.

Top Mass - 4000 lb-sec$^2$/in.

Base Radius - 15 ft.

Interstory Damping - 2% of critical

Table II. Effect of Soil-Structure Interaction on Response of Containment and Internal Structure (Chiapetta [19])

| Reactor Model | Lumped Mass | Fixed-Base Frequency of Mass, cps | Horizontal Acceleration Ratio (Structure Accel. / Free Field Accel.) |
|---|---|---|---|
| Surface Model I | Internal Support Structure | 5 | 0.176 |
| | Containment Structure | 4 | 0.380 |
| Surface Model II | Internal Support Structure | 7 | 0.278 |
| | Containment Structure | 4 | 0.391 |
| Embedded Model | Internal Support Structure | 5 | 0.224 |
| | Containment Structure | 4 | 0.482 |

Soil Shear Wave Velocity = 802 fps

Table III. Summary of Input Parameters used in Finite Element Interaction Studies by Isenberg[20]

| Case | Type of Structure | Input Motion | Soil properties | | |
|---|---|---|---|---|---|
| 3 | Embedded reactor | Golden Gate S80E (0–4 sec) | Homogeneous elastic | $C_p$ = 1,510 fps, | $C_s$ = 875 fps |
| 4 | Embedded reactor | Golden Gate S80E (0–4 sec) | Layered elastic | $C_p$ = 1,510 fps,<br>$C_p$ = 2,500 fps, | $C_s$ = 875 fps (surface to 45 ft)<br>$C_s$ = 1,440 fps (below 45 ft) |
| 5 | Embedded reactor | Golden Gate S80E (0–4 sec) | Homogeneous elastic | $C_p$ = 2,500 fps, | $C_s$ = 1,440 fps |
| 6 | Embedded reactor | 0.2 × Olympia N10W (7–11 sec) | Homogeneous elastic | $C_p$ = 1,510 fps, | $C_s$ = 875 fps |
| I–1 | Embedded reactor | 3 × Golden Gate S80E (0–3 sec) | Homogeneous (except for overburden, which increases with depth); elastic-plastic; same elastic properties as case 3. | | |
| I–2 | Embedded reactor | 2 × Olympia (7–10 sec) | Same as case I–1 | | |

Table IV. Horizontal Acceleration Spectra $\frac{\text{Structure (Foundation}}{\text{Free-Field (Surface)}}$ on the Response of Containment and Internal Structure (Isenberg [20])

| Frequency, cps | Case | | | | | |
|---|---|---|---|---|---|---|
| | 3 | 4 | 5 | 6 | I-1 | I-2 |
| 1 | 1.06 | 1.05 | 1.09 | 1.04 | 0.63 | 0.67 |
| 2 | 0.73 | 0.91 | 1.09 | 0.65 | 0.59 | 0.42 |
| 2.5 | 0.54 | 0.63 | 0.76 | 0.45 | 0.32 | 0.25 |
| 3 | 0.51 | 0.55 | 0.76 | 0.39 | 0.25 | 0.24 |
| 4* | 0.50 | 0.34 | 0.59 | 0.55 | 0.23 | 0.26 |
| 5* | 0.63 | 0.44 | 0.51 | 0.72 | 0.31 | 0.42 |
| 6 | 0.51 | 0.61 | 0.61 | 0.43 | 0.28 | 0.34 |
| 8 | 0.48 | 0.29 | 0.84 | 0.50 | 0.33 | 0.41 |
| 10 | 0.65 | 0.31 | 0.50 | 0.58 | 0.20 | 0.84 |

* Fixed-base frequency of containment vessel (4cps) and internal structure (5cps)

Table V. Acceleration Spectra, G, Determined from Finite Element Model and Soil-Spring Model (Isenberg [23]

| | Free Field | | Foundation | |
|---|---|---|---|---|
| Frequency, cps | Surface | Foundation Level (2) | Finite Element 2-D Analysis | Soil-Spring Analysis with Interaction |
| 0.1 | 0.00018 | 0.00016 | 0.00011 | 0.00162 |
| 0.25 | 0.0013 | 0.0011 | 0.0011 | 0.0046 |
| 0.5 | 0.0081 | 0.0077 | 0.0079 | 0.0142 |
| 1.1 | 0.099 | 0.089 | 0.098 | 0.225 |
| 1.5 | 0.073 | 0.070 | 0.067 | 0.233 |
| 2 | 0.127 | 0.107 | 0.093 | 0.123 |
| 2.5 | 0.30 | 0.18 | 0.21 | 0.081 |
| 3.3 | 0.175 | 0.78 | 0.105 | 0.055 |
| 4 (1) | 0.37 | 0.107 | 0.165 | 0.061 |
| 5 (1) | 0.14 | 0.068 | 0.081 | 0.056 |
| 6.7 | 0.1 | 0.065 | 0.064 | 0.057 |
| 10 | 0.055 | 0.043 | 0.038 | 0.044 |

(1) Fixed-base frequencies

(2) Calculated from element motion at the foundation depth without structure.

Table VI.

Comparison of structure accelerations caused by rotational and lateral foundation motions. (Scavuzzo [48])

| Frequence [cps] | Structure discription | Height [ft] | $(\omega \int_0^t \ddot{u}(\tau) \sin \omega(t-\tau) d\tau)_{max}$ | $(\omega \int_0^t l\ddot{\alpha}(\tau) \sin \omega(t-\tau) d\tau)_{max}$ | $[\ddot{x}(t)]_{max}$ | Free-field ‡ acceleration |
|---|---|---|---|---|---|---|
| 4.06 | Surface model I | 113 | 0.83 | 0.95 | 0.12 | 2.53 |
| 4.06 | Surface model I | 66.5 | 0.83 | 0.47 | 0.37 | 2.53 |
| 5.0 | Surface model I | 55 | 0.26 | 0.21 | 0.23 | 1.48 |
| 5.0 | Surface model I | 27.5 | 0.26 | 0.10 | 0.22 | 1.48 |
| 4.06 | Surface model II | 133 | 0.85 | 0.97 | 0.22 | 2.53 |
| 4.06 | Surface model II | 66.5 | 0.85 | 0.48 | 0.36 | 2.53 |
| 5.0 | Surface model II * | 55 | 0.48 | 0.40 | 0.72 | 1.48 |
| 5.0 | Surface model II * | 27.5 | 0.48 | 0.20 | 0.62 | 1.48 |
| 4.06 | Embedded model | 133 | 0.99 | 0.87 | 0.35 | 2.53 |
| 4.06 | Embedded model | 66.5 | 0.99 | 0.44 | 0.56 | 2.53 |
| 5.0 | Embedded model | 55 | 0.33 | 0.15 | 0.28 | 1.43 |
| 5.0 | Embedded model | 27.5 | 0.33 | 0.07 | 0.27 | 1.43 |

* For structural model II, the frequence of the internal structure was increased to 7 cps.

‡ Free-field accelerations are the lateral spectrum value at the indicated frequence (4 cps or 5 cps)

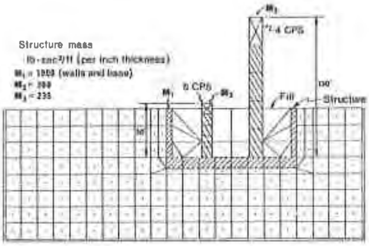


Fig. 1. Finite element representation of embedded nuclear reactor structure for Cases 3 through 6. [Ref (20)]

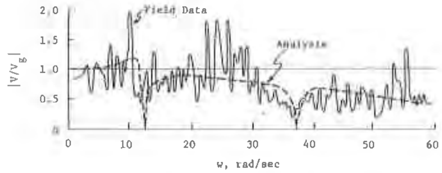


Fig. 2. Hollywood storage building east-west foundation motion, V, divided by free-field motion, $V_g$, in frequency domain taken from Ref (6) and (7).

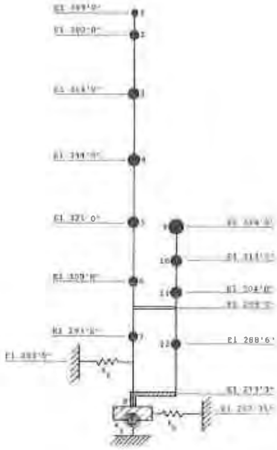


Fig. 3. Dynamic model of the CVTR [Ref (33)] showing the lateral soil springs $K_s$ and $K_b$, rotational spring $K_\phi$. The torsional spring $K_\phi'$ is not indicated.

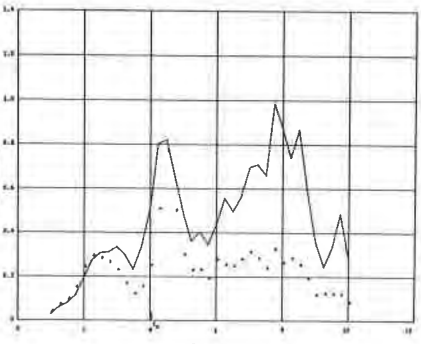

Fig. 4. The acceleration spectra of the input earthquake ground motion is shown as a solid line. Output spectra calculated from the foundation motion of a containment-vessel-plus-base-mass model with a fixed-base frequency of 4.06 cps are also plotted. The soil shear-wave velocity is 1,000 ft/sec. [Ref (16)]

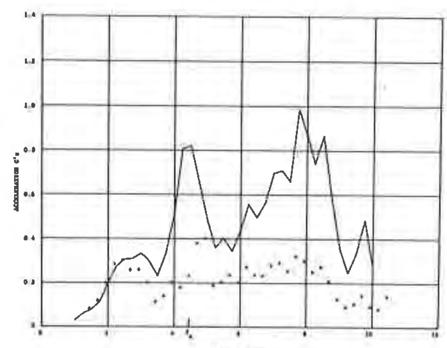

Fig. 5. The acceleration spectrum of the input earthquake ground motion is shown as a solid line. Output spectra calculated from the foundation of a containment-vessel-plus-base-mass model with a fixed-base frequency of 4.44 cps are also plotted. The soil shear-wave velocity is 1,000 ft/sec. [Ref (16)]

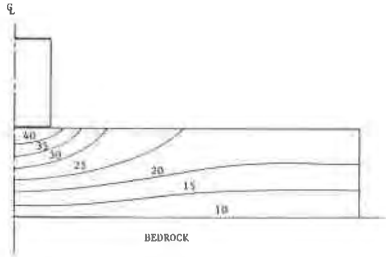


Fig. 6. Plot for maximum total acceleration (ft/sec$^2$). Base acceleration El Centro N-S 1940. $V_S$ = 1,500 ft/sec, $T_R$ = 0.44 sec, $M_T = M_B$ = 189,000 lb-sec$^2$/ft. Taken from Ref. [21]