REVIEW OF SOIL-STRUCTURE
INTERACTION AND SEISMIC ANALYSIS
OF CATEGORY I STRUCTURES
SOUTH TEXAS PROJECT
UNITS 1 AND 2

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REVIEW OF SOIL-STRUCTURE INTERACTION AND SEISMIC ANALYSIS OF CATEGORY I STRUCTURES SOUTH TEXAS PROJECT UNITS 1 & 2

by

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1.0 INTRODUCTION

The dynamic soil-structure interaction (SSI) analysis of the South Texas Project (STP) plant structures was performed in 1975 (Refs. 19, 20). The SSI analysis approach used was established and agreed upon at a meeting held in February 1975 among representatives of the NRC, Houston Lighting and Power Company (HL&P), Brown & Root (B&R), Woodward-Clyde Consultants (WCC), and EDS Nuclear (Refs. 19, 20). To reevaluate the appropriateness of the SSI analysis approach used, HL&P requested (Ref. 21) representatives of Woodward-Clyde Consultants (WCC), Brown & Root, Inc. (B&R), and their consultants (Drs. E. A. M. Kausel, J. Lysmer, and H. B. Seed) to review the soil-structure interaction analysis (Refs. 20, 22) and seismic structural analyses conducted (Refs. 1, 2, 3, 4) and the results used in the design of Category I structures of the STP (Refs. 1, 2, 3, 4, 20). The scope of review included the following: Applicability of the finite element method for soil-structure interaction analyses of STP site; Applicability of procedures used for the seismic analysis of Category I structure; Sources of conservatism and conservatism of results.

This report is prepared by the project team (WCC and B&R) and project consultants based on the review. The report is organized as follows:

- Applicability of the finite element method for soil-structure interaction analyses of the STP site;
- Applicability of procedures used for the seismic analysis of Category I structures;
- Sources of conservatism and conservatism of results;
- Conclusions

2.0 APPLICABILITY OF THE FINITE ELEMENT METHOD FOR SOIL-STRUCTURE INTERACTION ANALYSES OF THE STP SITE

The soil-structure interaction analysis (SSI) conducted for the STP was made by using the two-dimensional dynamic finite element method. This method of analysis is applicable to the STP site for the following reasons:

1. The soil conditions at the STP site consist of alternating layers of stiff to hard clays and dense silts and sands which extend to depths of several thousand feet (Ref. 20). The stratigraphy across the site is very uniform; the soil layers are essentially horizontal and are generally continuous. The layered soil conditions can be properly modeled by using the dynamic finite element method.

2. Two of the Category I structures, the Reactor Containment Building and the Fuel Handling Building are deeply embedded. The depth of embedment of the Reactor Containment Building is approximately 58 feet (Fig. 4.5-1 of Ref. 20) and that of the lowest foundation level of the Fuel Handling Building is approximately 61 feet (Fig. 4.5-4 of Ref. 20). The effects of embedment on the response of structures and lateral pressures on basement walls can be properly accounted for and computed by using the dynamic finite element method.

3. All Category I structures are situated in close proximity to the Reactor Containment Building. Effects of structure-structure interaction on structural response can be conservatively accounted for by the finite element method.

In the following paragraphs, the appropriateness of the basic assumptions made in the dynamic finite element analyses for the STP (Ref. 20) is examined.

1. The incident waves to the soil-structure models were assumed to be vertically-propagating body waves (i.e. vertically-propagating shear waves during horizontal excitation, and vertically-propagating compression waves during vertical excitation). As described previously, the STP site is a uniform, horizontally layered soil site which extends to depths of several thousand feet. Recent analytical and empirical data by Seed and Lysmer (1980) on the significance of site response in soil-structure interaction analyses for nuclear facilities indicate that for structures embedded in uniform, horizontally layered soil, the main source of excitation can be considered for all practical purposes to be body waves propagating in a vertical direction.

Seed and Lysmer (1980) examined site response due to horizontally propagating waves (i.e. surface waves and inclined body waves) and vertically propagating waves. Analyses of the Rayleigh waves in layered systems show that the high frequency components (frequencies higher than 1 to 2 Hz) of the Rayleigh modes including the fundamental Rayleigh mode and some higher modes decay rapidly in the direction of wave propagation; other high-order modes may decay less rapidly than the fundamental mode. However, studies by Chen and Lysmer (1979) have shown that the low-decay modes, when they occur, are associated with energy propagation in deeper high-velocity layers and that they cause near surface motions which are similar to those caused by vertical or slightly inclined body waves. Thus, Seed and Lysmer concluded that for structures on soil deposits and cases where the frequencies of concern are greater than 1 or 2 Hz, there seems to be no realistic basis for considering that Rayleigh waves make any significant contribution to the site response.

Analyses of surface response to inclined body waves by Joyner, et al. (1976) and by Udaka (1975) indicate that for angles of incidence up to 45 degrees, there is a negligible difference between the motions computed for inclined waves and for

vertically propagating waves. Thus, it is reasonable to conclude that the variation of horizontal motions with depth within a soil deposit is for all practical purposes the same, whether they are computed for vertical or inclined directions of propagation, within the embedment depth of the structures. On the basis of the above discussions, the assumption of vertically propagating incident body waves to the SSI analyses conducted for the STP site is reasonable and appropriate.

Based on analyses of site response and empirical data on variations of peak accelerations and frequency characteristics of ground motions with depth, Seed and Lysmer (1980) concluded that if a broad-band design spectrum such as NRC Regulatory Guide 1.60 is used, the control point should be located at the ground surface to have a reasonable variation of ground motion with depth. Thus, it is concluded that the specification of the control motion at the finished grade level in the free-field in the SSI analyses for the STP is appropriate.

2. In the SSI analyses, the nonlinear behavior of soil was approximated by the equivalent linear method. Strain-compatible shear modulus and damping values were used for the soils. Considerations of variations of shear modulus and damping ratio with shear strain in the SSI analyses conducted for the STP site are appropriate and justified. Stress-strain behavior of soils subjected to dynamic cyclic loadings is nonlinear and hysteretic. The State-of-the-Art paper by Hardin (1978) summarizes the nonlinear stress-strain behavior and stress-strain relations for soils under static or dynamic loads. It has been shown that values of shear modulus and damping of soils are highly strain-dependent (Seed and Idriss, 1970; Hardin and Drnevich, 1972a and 1972b; SWA-AJA, 1972; SW-AA, 1976). In the analysis of the apparent system frequencies and of the apparent damping ratio for the Millikan Library building during the San Fernando earthquake of 1971, Luco (1980) was able to explain most

of the observed behavior by considerations of a combination of a permanent change in structural stiffness together with nonlinear behavior of the soils. In that study, Luco considered that shear modulus of the soil decreased and damping ratio increased due to straining induced by the earthquake shaking and the soil-structure interaction during the San Fernando Earthquake. Variations of shear modulus and damping ratio with shear strain developed by Seed and Idriss (1970) were used in that study.

The SSI studies were conducted for wide parametric variations in dynamic shear modulus (Section 4.6.5 of Ref. 20). The average soil properties were defined based on data from in-situ measurements of shear wave velocity at low strain levels and data from laboratory dynamic tests at higher strain levels. Upper-bound soil properties were defined by increasing values of maximum shear modulus at low strain levels by 50 percent; lower-bound soil properties were defined by decreasing values of maximum shear modulus by 40 percent. These large variations in modulus result in envelopes of response spectral values at the foundation levels in the free-field, from analyses using average, upper-bound and lower-bound soil properties, which not only satisfy the criterion of enveloping the 60 percent of the Reg. Guide 1.60 spectral values specified at the finished grade level in the free-field, but also are significantly higher (50 to 100 percent) than the 60 percent of the Reg. Guide 1.60 spectral values in the frequency range higher than 3 Hz (Fig. 5.6.2 of Ref. 20). Considerations of variations in dynamic soil properties are reasonable and appropriate to account for uncertainties in the material properties of the soil. However, the parametric variations used in the analyses were very conservative with respect to the actual measured values of shear modulus at the site.

3. In the SSI studies, the actual three-dimensional soil-structure system was approximated by two-dimensional plane-strain models.

Thus, energy dissipation in the third direction perpendicular to the direction of excitation was not accounted for. Seed, et al. (1977) have shown that plane-strain idealizations with no energy dissipation in the third direction generally result in slightly higher response. In addition, the plane-strain idealizations may result in conservative response computed due to structure-structure interaction as discussed by Hadjian (1976).

The two-dimensional (2-D) structural models used in the SSI studies were developed from detailed three-dimensional (3-D) models (Ref. 20). The global inertia properties (mass, mass moment of inertia, and location of center of gravity and the fundamental frequency of the 2-D finite element models are similar to those of the 3-D lumped mass models. However, the dynamic characteristics of the 2-D and 3-D models were not identical. Kausel (1980) indicates that the differences in the structural idealizations result in conservative response at the resonant frequencies of the structure computed in the detailed 3-D structural analyses using the basemat motions from the 2-D SSI analyses. Further discussions of conservatism of the results are presented in Section 4.

3.0 APPLICABILITY OF PROCEDURES USED FOR THE SEISMIC ANALYSIS OF CATEGORY I STRUCTURES

The seismic analysis of the superstructure was performed using the time history modal analysis method. In this analysis, modal responses were combined exactly taking into account the phase angle relationship between modes. Thus, no approximation method such as the square root of the sum of the squares (SRSS) was used in modal combination.

For this analysis a detailed three-dimensional lumped mass model was constructed for each Category I structure. In this model, the torsional soil spring was introduced at the foundation mat level and the eccentricities between centers of mass and centers of rigidity were properly incorporated. Therefore, the torsional effects due to eccentricities were automatically taken into account.

The input motions applied at the base of the 3-D superstructure models, at the interface between the structures and the soil, were obtained from the dynamic finite element analysis of the integrated model including adjacent structures. Therefore, the structure to structure interaction effects were included in the structural seismic responses.

The sloshing effect of fluid was also considered in the analysis. The subsystems and components such as the reactor vessel, steam generator etc. were included in the model. The dynamic decoupling of systems from subsystems is based on the following criteria (Ref. 3):

$R_m \leq 0.01$ or

$R_m \leq 0.1$ for $R_f \geq 1.25$ or $R_f \leq 0.8$

Where:

R_m = Effective mass ratio of subsystem to system

R_f = Natural frequency ratio of subsystem to system

This assures that the interaction of structures and essential components was properly taken into account.

The damping values that were used in the design were obtained from regulatory guide 1.61 which were lower than the test values.

An adequate number of masses and degrees of freedom in each dynamic structural model has been taken into consideration. In all cases, either the number of degrees of freedom has been chosen more than twice the number of modes with frequencies up to 33 cps, or the inclusion of additional modes will not result in more than a 10% increase in responses. Therefore, contributions from all significant modes were properly included.

In the development of floor design response spectra, the regulatory guide 1.122 was strictly adhered to. The analysis was performed separately for each of the three directions and the ordinates of the floor response spectra at the location of interest for a given direction were obtained by combining the ordinates of the three codirectional floor response spectra according to the SRSS criterion. In actual calculations, conservative coefficients were used to increase the primary response spectra to cover the cross coupling effects. This introduced conservatism beyond that required by the code and regulations.

In addition to the above conservative measure of varying the soil properties over a wider range than test data indicated, the peaks of computed floor response spectra were shifted with respect to the natural frequencies. The frequency variation, $\pm \Delta f_j$, was determined by taking the SRSS of a minimum variation of .05 f_j and the individual frequency variation $(\Delta f_j)_n$, that is (Ref. 3):

$$\Delta f_j = \sqrt{(0.05\, f_j)^2 + \Sigma(\Delta f_j)^2{}_n}$$

As a minimum, a value of 0.10 f_j was used if the actual computed value of Δf_j was less than 0.10 f_j. With this peak widening provision, the chances for safety related equipment to be subjected to the peak load was much greater in our calculations than in reality.

The dynamic peak responses such as shears and moments that were calculated at an instant of time were treated as static loads occurring at all times in the design. This approach provides added conservatism and further increases the ability of these Category I structures to withstand the effect of the design earthquake.

The industry practice and related regulations were comparatively well established in the seismic analysis of Category I structures. Conservatism beyond that required by the regulations was provided in the analysis of the Category I structures of the STP. Therefore, the analysis results used in the design are conservative.

4.0 SOURCES OF CONSERVATISM AND CONSERVATISM OF RESULTS

4.1 Conservatism of Input Motion

The SSE producing the maximum vibratory ground acceleration at the STP site was conservatively assigned an intensity of VI modified Mercalli. Intensity-peak acceleration correlations resulted in a peak acceleration of 0.07g for an earthquake of intensity VI (Ref. 22). However, a peak acceleration of 0.10g was selected for the SSE to satisfy the minimum requirements for peak acceleration given in Appendix A to 10CFR100, "Seismic and Geologic Siting Criteria for Nuclear Power Plants" (Ref. 22). Similarly, to comply with Appendix A to 10CFR100, an OBE of 0.05g was selected (Ref. 22). Thus, the peak accelerations selected for the SSE and OBE are 43 percent higher than the values conservatively estimated for the STP site.

In the SSI studies, the actual control motions used were further scaled up in order that the peak accelerations that resulted at the free-field foundation level after deconvolution were not less than the values of 0.10g for the SSE and 0.05g for the OBE for horizontal excitation and two-thirds of these values for vertical excitation. The amount of scaling depended on the soil properties assigned for a particular analysis case; the maximum amount of scaling was about 30 percent (Fig. 5.6-1 of Ref. 20).

In addition to the conservatisms mentioned previously, spectral values of the synthetic time histories used in the SSI analyses, in general, are significantly higher than the design response spectra over the entire frequency range of interest (Appendix A of Ref. 20).

4.2 Other Sources of Conservatism

Conservatisms in base motions for the Diesel Generator Building and the Fuel Handling Building (Ref. 20) - In the soil-structure interaction studies, site cross-section 3 (i.e. the North-South cross-section) was

analyzed only for the case of average soil properties (Ref. 20). However, it was necessary to evaluate the effect of soil property variations on the response of the structures in this cross-section, which are the Mechanical-Electrical Auxiliary Building and Diesel Generator Building. For that purpose, a procedure (Section 5.7.1 of Ref. 20) was developed to estimate the interaction spectra for the upper-bound and lower-bound soil properties based on the computed spectrum for the average soil properties. After the estimated interaction spectra for the upper-bound and lower-bound soil properties were obtained, together with the computed spectrum for the average soil properties, conservative smooth envelopes were constructed and raised by approximately 10 percent to take into account the possibility of additional amplification (Secrion 5.7.1 of Ref. 20). For the Fuel Handling Building, the base motion during horizontal East-West excitation was conservatively estimated to be a smooth envelope of the response spectra computed at the three foundation levels of the Fuel Handling Building in the analyses for cross-section 2 (i.e., North-South excitation) (Section 5.7.2 of Ref. 20). In the seismic structural analysis of the Fuel Handling Building in the East-West direction and the Diesel Generator Building by Brown & Root, the control motion scaled to a peak acceleration equal to 0.07g was used for the OBE (Section 5.7 of Ref. 20). The base motion used is 40 percent higher than the free-field control motion and is very conservative in the short period range compared to the estimated smooth envelopes of the interaction spectra.

Parametric Variations in Soil Properties - Parametric variations in dynamic shear moduli are conservative with respect to the actual measured values of shear modulus at the site. Large parametric variations result in conservative estimates of effects of soil property variations on the soil-structure interaction.

Conservative Floor Response Spectra Computed from the Detailed Structural Analyses - As described in Section 2, differences in the dynamic characteristics between the detailed 3-D structural models used in the seismic structural analyses and the 2-D plane-strain structural models

used in the SSI studies result in conservative floor response at the resonant frequencies of the structures computed in the detailed seismic structural analyses. These high floor responses would not be present if a consistent model was used in both the SSI analyses and the seismic structural analyses. Use of these computed floor response spectra in the design of the structures and their subsystems is therefore conservative.

Conservative Envelopes of Total Floor Response Spectra - As described in Section 3 the total floor response spectra were obtained by taking the SRSS of the codirectional responses at each frequency point. The envelopes of the total floor response spectra were constructed by shifting of the peaks with respect to natural frequencies by a minimum percentage to account for the uncertainties associated with computed natural frequencies of the structure. This enveloping procedure results in conservative floor response spectra when used in design of the structures and subsystems.

5.0 CONCLUSIONS

The finite element method used in the analysis of soil-structure interaction is an applicable and appropriate method for assessing soil-structure interaction effects at the STP. Based on examinations of various sources of conservatism, it is concluded that the results of the SSI analyses and the seismic structural analyses are very conservative for the design of the Category I structures and the subsystems at the STP site.

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