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February 26, 1988
C320-88-0270

Mr. A.W. Dromerick
U.S. Nuclear Regulatory Commission
Washington, DC 20555

Dear Mr. Dromerick:

SUBJECT: Information Requested by A.W. Dromerick's Letter Dated December 16, 1987 re: Oyster Creek Seismic Floor Response Spectra Methodology

This letter transmits the enclosed information as outlined below. The information was requested by your letter dated December 16, 1987 concerning Oyster Creek seismic floor response spectra. As per our telephone conversation on February 10, 1988, the information is being forwarded directly to you in order to expedite your review. Our formal submittals will be made in the near future. Although the enclosed documents are advanced copies, it is expected that there will be very few (if any) changes.

Listed below are the materials being transmitted by this letter.

Enclosure 1. Variations in soil properties - This material provides responses to Item 1 mentioned in your letter of December 16, 1987. Part A of this material provides reasonings behind choosing the soil property bounds used in the Oyster Creek SSI analyses and Part B presents a study of the effects of strain - dependent soil properties on reduction of motion in the SSI analyses.

Enclosure 2. Modeling uncertainties - This material provides responses to Item 2 mentioned in your letter of December 16, 1987. Under this item, NRC staff wanted us to provide published reports of comparisons between SUPERFLUSH analytical results and measured responses of structures under actual seismic events. This is to obtain assurance that SSI modeling and computations, as implemented in the SUPERFLUSH code, properly capture the structural responses in the field.

Enclosure 3. Computational parameters and their limitations - This information is provided in response to Item 3 of your December 16, 1987 letter. It describes the enveloping procedures that have been followed to obtain final floor response spectra.

Enclosure 4. Verification and validation of computer code with the measured results - This information is provided in response to Item 4 of your December 16, 1987 letter.

Enclosure 5. Effect of saturated soil - This information is provided in response to Item 6 of your December 16, 1987 letter. It describes the soil saturation condition at the Oyster Creek reactor building area and how the saturation effects were considered in the reactor building SSI analyses.

Enclosure 6. Torsional effects on structures - This information addresses Item 7 of your December 16, 1987 letter. It describes the study done to assess the torsional effects of Oyster Creek reactor building mathematical model on the computed horizontal floor response spectra.

Please note that information requested by Item 5 of your December 16, 1987 letter, "Power spectral density for the time history being used" is still in the process of being gathered. It will be forwarded to you as soon as the information becomes available.

If you have any questions concerning the materials being transmitted by this letter, please contact me.

Very truly yours

Yosh Nagai

Yosh Nagai
Licensing Engineer

/jbw
Enclosure

cc: Civil/Structural Manager - L. Garibian (w/o attach.)
Director - Engineering & Design - G.R. Capodanno "
Director - Engineering Projects - D.K. Croneberger "
Lic. & Reg. Affairs Director - J.R. Thorpe "
Manager BWR Licensing - M.W. Laggart "
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Manager Special Projects - E.F. O'Connor "

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VARIATIONS IN SOIL PROPERTIES

The attached packages have been developed in response to information requested by USNRC audit staff (Ref. memo from Mr. A. Dromerick (NRC) to Mr. P. Fiedler (GPU), dated December 16, 1987).

The attached material provides responses to item 1 - Variations in Soil Properties - mentioned in the NRC memo. Part A provides reasonings behind choosing the soil property bounds used in the Oyster Creek SSI analyses. Part B presents a study of the effects of strain-dependent soil properties on reduction of motion in the SSI analyses.

PART A

SOIL PROPERTY BOUNDS USED IN OYSTER CREEK REACTOR BUILDING SSI ANALYSES

Introduction

The following paragraphs describe the study done by URS/Blume to obtain the soil property bounds to be used for SSI analyses. These soil property bounds are considered to account for realistic soil property variations that may occur at the Oyster Creek site.

Measured Soil Properties at Oyster Creek Site Area

Shear (S) and compression (P) wave velocities were measured in the site area. These are presented in Figure 1. The S- and P-wave velocities are given for four layers — a 15-ft top layer overlying 25-ft, 65-ft, and 35-ft layers. Shear wave velocities increase with depth from 600 ft/sec for the top layer to 1,400 ft/sec for the fourth layer. Compression wave velocities are also measured for these layers. Poisson's ratio for each of the layers is derived from its shear and compression wave velocities. The water table varies between elevation +3 ft 6 in. to +10 ft, as monitored in the well network at the site since 1984.

The reactor building is embedded 53 ft below grade. This places the basemat of the reactor building in the third layer (dense Cohansey sand). The third layer extends below the reactor building basemat and is the most important layer for consideration of SSI effects on the reactor building. The third layer has a measured shear wave velocity of 1,200 ft/sec. Relative densities were also obtained from relative density tests performed on undisturbed tube samples. The average relative density of the sands in the third layer was determined to be 94%, with the maximum and minimum values being 101% and 78%.

Best Estimate Soil Properties

For the purposes of computing the best estimate soil shear modulus to be used in SSI analyses, the field-measured seismic shear wave velocities have been used. For the third layer, this yields the following best estimate soil shear modulus:

$$G = \rho V_s^2$$

$$= \left(\frac{0.120}{32.2}\right)\left(1200\right)^2$$

$$= 5{,}366 \text{ ksf}$$

where:

G = Shear modulus in ksf

ρ = Soil mass density, K-sec^2/ft

V_s = Soil shear wave velocity, ft/sec

We have also calculated the shear modulus for the third layer from tested relative density (D_r) values as follows:

Average value of D_r for the third layer is 94%. This is assumed to be best estimate value. Shear modulus of sand is also given by the following formula:

$$G = K_2 \sqrt{\sigma'_m}$$

where σ'_m is the effective mean principal stress and K_2 is a parameter which is a function of relative density and shear strain.

K_2 = 73 for D_r = 94% and low shear strain (10^{-4}%) from Figure 2 (Ref. Fig. 5 of reference 1)

The effective stress σ'_m is calculated at the middle of the third layer (72.5 ft below grade) as follows:

Assuming water table at elevation +10 ft (maximum level):

$$\sigma'_m = 13.5 \times 120 + 59.0 \times (120 - 62.4)$$
$$= 5{,}018.4 \text{ psf}$$

Assuming water table at elevation +3.5 ft (minimum level):

$$\sigma'_m = 20 \times 120 + 52.5 \times (120 - 62.4)$$
$$= 5{,}424 \text{ psf}$$

The average $\sigma'_m = (5{,}018.4 + 5{,}424)/2 = 5{,}221.2$ psf

The best estimate $G = 73\ (\sqrt{5{,}221.2})$
$= 5{,}274.8$ ksf
$\approx 5{,}275$ ksf

The best estimate shear modulus value of 5,275 ksf, calculated from the relative density test results, compares very well with 5,366 ksf computed from shear wave velocity measurements. So, 5,366 ksf was chosen as the best estimate shear modulus for the third layer.

Determination of Upper and Lower Bound Soil Properties

To determine realistic upper and lower bound soil properties, we have examined the actual soil property variations reported for the site soil tests. For the third layer, the relative density variations were reported to be 78% (minimum) and 101% (maximum).

The K_2 values associated with these densities are obtained from Figure 2 as follows:

$K_2 = 61$ for $D_r = 78\%$ and low shear strain ($10^{-4}\%$)
$= 78$ for $D_r = 101\%$ and low shear strain ($10^{-4}\%$)

Upper value of G = $K_2 \sqrt{\sigma'_m}$

= $78 \sqrt{5221.2}$

= 5,636 ksf

= 105% of best estimate G of 5,366 ksf

Lower value of G = $K_2 \sqrt{\sigma'_m}$

= $61 \sqrt{5221.2}$

= 4,408 ksf

= 0.82% of best estimate G of 5,366 ksf

From the above calculations, it is judged that conservative estimates of upper bound and lower bound shear moduli may be taken as 1.25 G_{be} and 0.67 G_{be}, where G_{be} is best estimate shear modulus. The K_2 values corresponding to upper bound and lower bound soil properties are 93 and 50. Seed and Idriss (Ref. 1) examined K_2 values determined from several in-situ shear wave velocity measurement tests reported in the literature. The range of values of K_2 for dense to extremely dense sands were found to be 44 to 86. Thus, a range of 50 to 93, chosen for K_2 values for the dense Cohansey sand layer at the Oyster Creek site, covers the possible variation in K_2 of such dense sands as reported in the literature.

Figure 3 presents the best estimate, upper bound, and lower bound soil profiles used in Oyster Creek reactor building SSI analyses.

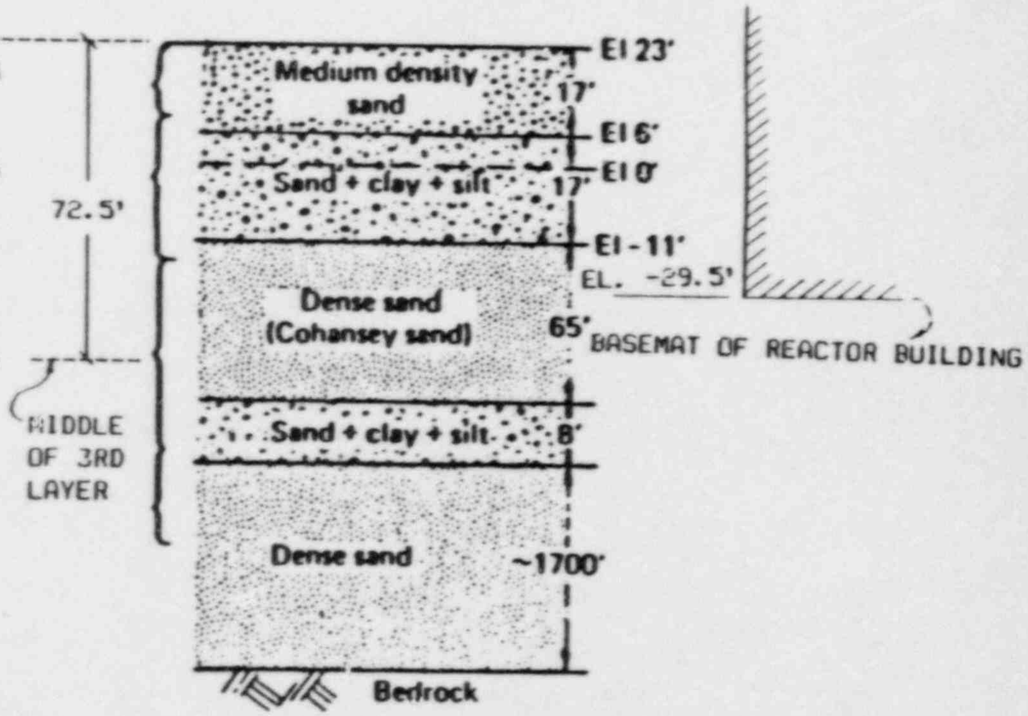
Conclusion

From the above study, it is concluded that the soil properties used in SSI analyses of Oyster Creek reactor building are based on actual soil property measurements in the site area. The soil property variations considered are realistic and conservative bounds of actual soil property variations obtained from site soil test results.

Reference

1. Seed, H. B., and Idriss, I. M., "Soil Moduli and Damping Factors for Dynamic Response Analyses," Report No EERC-70-10, University of California, Berkeley, December 1970.

LAYER NO.	MEASURED P-WAVE VELOCITY VP (FT/SEC)	MEASURED SHEAR WAVE VELOCITY Vs (FT/SEC)	POISSON'S RATIO	LAYER DEPTH (FT)
1	1400	600	0.39	15
2	5200	1000	0.48	25
3	5600	1200	0.48	65
4	5900	1400	0.47	35



NOTE: WATER TABLE VARIES BETWEEN EL. +3.5' TO +10.0'

FIGURE 1: SOIL SEISMIC WAVE VELOCITY PROFILE AT OYSTER CREEK PLANT

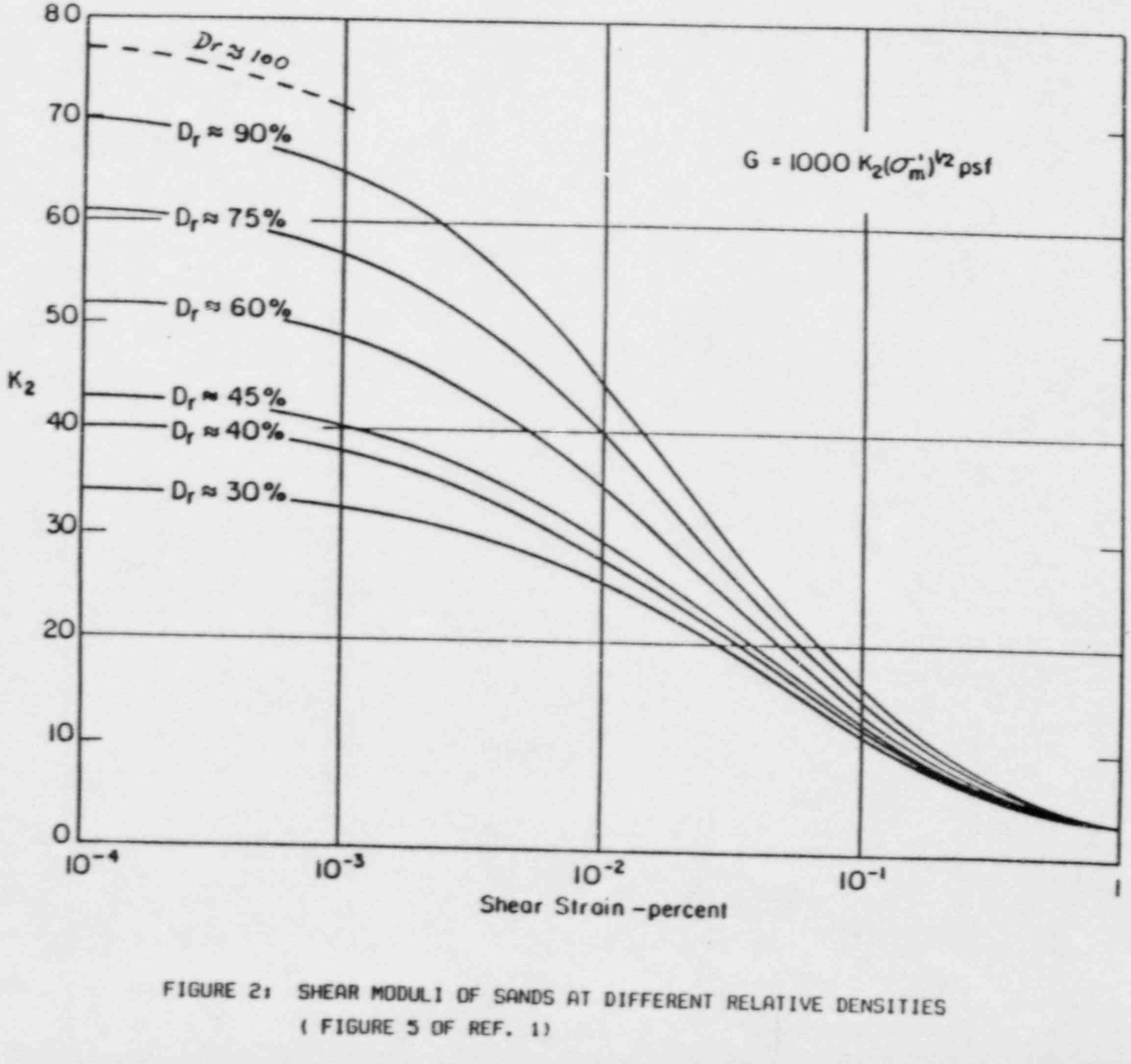
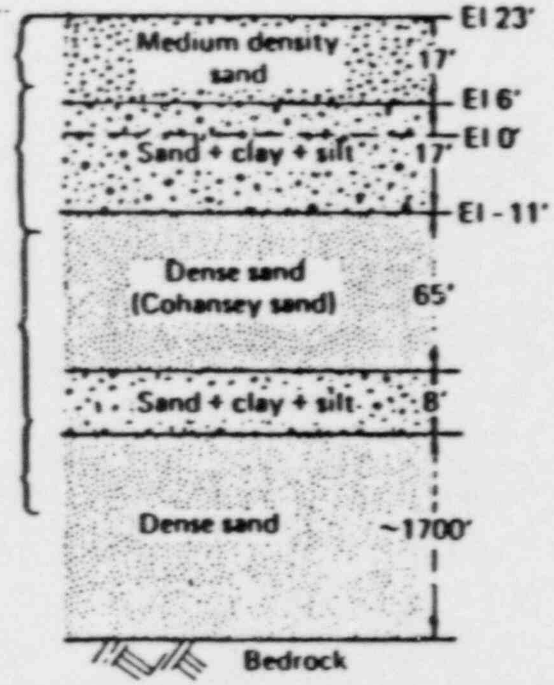


FIGURE 2: SHEAR MODULI OF SANDS AT DIFFERENT RELATIVE DENSITIES
(FIGURE 5 OF REF. 1)

LOW STRAIN SHEAR MODULUS (KSF)			POISSON'S RATIO	LAYER DEPTH (FT)
BEST ESTIMATE	UPPER BOUND	LOWER BOUND		
1342	1678	895	0.39	15
3727	4659	2485	0.48	25
5366	6708	3577	0.48	65
7304	9130	4869	0.47	35



NOTE: WATER TABLE VARIES BETWEEN EL. +3.5' TO +10.0'

FIGURE 3: SOIL PROPERTY PROFILE USED FOR SSI ANALYSES OF REACTOR BUILDING

PART B

EFFECT OF STRAIN-DEPENDENT SOIL PROPERTIES ON REDUCTION OF MOTION OYSTER CREEK REACTOR BUILDING SSI ANALYSES

Introduction

In implementing the SSI analyses for Oyster Creek reactor building, URS/Blume has used strain-dependent soil properties in the computer code SuperFLUSH. The strain dependence of soil shear modulus and damping with shear strain are given in Figures 1 and 2. These are the well-known Seed and Idriss curves for sand (Ref.1). The soil shear modulus reduces with increased strain, and this is given as a ratio of low-strain (10^{-4}%) shear modulus, G_{max}. Usually, the shear moduli calculated from shear wave velocities measured in the field from downhole and crosshole data are assumed to be low-strain shear moduli, i.e., G_{max}.

The best estimate, upper bound, and lower bound low-strain shear moduli for the Oyster Creek site are shown in Figure 3. The best estimate low-strain shear moduli are calculated from the shear wave velocities measured in the Oyster Creek site area. The upper bound and lower bound moduli are 1.25 and 0.67 times the best estimate values. Three SSI analyses were conducted with SuperFLUSH for the three low-strain soil profiles, and the soil layer properties (shear modulus and damping) were iterated to correspond to the shear strain state in the soil as per Figures 1 and 2. The floor response spectra, computed from these three SSI analyses, were enveloped to obtain the final spectra for the Oyster Creek project.

Parametric Study

To investigate the effect of strain-dependent properties on the computed seismic responses, we have used deconvolution procedures to compute the motion at the basemat level of the reactor building (el. -29 ft 6 in.) for strain-independent soil profile (i.e., soil properties were not varied with shear strain). The shear moduli of the soil layers were assumed to remain constant at the low-strain best estimate shear modulus values as given in Figure 3. The corresponding soil damping was assumed to be 1%. The best

estimate soil profile was chosen because these properties correspond to the field-measured shear wave velocities. The motion at the ground surface (El. +23 ft 6 in.) was assumed to be the design time history (0.165g ZPA) for the deconvolution computation.

The response spectrum for the deconvolved motion at el. -29 ft 6 in. for the above-mentioned strain-independent best estimate soil profile was computed. This was compared with the response spectra at the same level, computed for the project using the envelope of spectra from strain-dependent best estimate, upper bound, and lower bound soil profiles. The ratio of the spectral amplitudes of the project spectrum to the strain-independent response spectrum is plotted as a function of frequency in Figure 4. A ratio value of less than 1.0 implies that consideration of strain-dependent soil properties and enveloping results from the three soil cases give lower response spectral amplitudes at that frequency than a strain-independent analysis with best estimate soil case would yield. As shown in Figure 4 (shaded area), in most frequency ranges, the spectral amplitudes are increased (by as much as 50% to 70%) due to consideration of strain-dependent soil properties and enveloping the three soil profiles. In the rest of the frequency ranges, there is a reduction of less than 10%, except in a very narrow range, where it goes up to about 20% reduction in a frequency range of 2.5 hz to 4 hz.

To further investigate the effects of soil strain-dependent properties on soil motion, we considered a case where soil shear strain is assumed to consistent with Seed and Idriss curves at a strain of 10^{-3}%. This is a very conservative estimate of the possible soil strain during an SSE event. The best estimate soil modulus at 10^{-3}% strain is 93.4% of the low-strain (10^{-4}%) value. Corresonding damping is 1.6%. Deconvolved motion at this soil condition was computed, and spectral ratios similar to Figure 4 are plotted in Figure 5. As shown in Figure 5 (shaded area), in most frequency ranges, the spectral amplitudes are increased due to consideration of strain-dependent soil properties. In the rest of the frequency ranges, there is less than 10% reduction in spectral amplitudes.

Conclusions

It is concluded from the above study that the consideration of strain-dependent soil properties in Oyster Creek SSI analyses does not lead to significant reduction in basemat motion. In most frequency ranges, spectral amplitudes are increased due to the strain-dependent soil cases when compared to strain-independent, best-estimate soil case . In other frequency ranges, the differences are less than 10% when compared to responses obtained assuming very conservative soil strains (10^{-3}%) during seismic motion and only 7% reduction in soil shear modulus and 1.6% damping in soil. Even for the extreme case of no reduction in soil shear modulus and 1% soil damping, the reductions are mostly less than 10% in all frequency ranges of interest.

References

1. Seed and Idriss (1970), "Soil Moduli and Damping Factors for Dynamic Response Analyses," EERC Report 70-10, University of California, Berkeley.

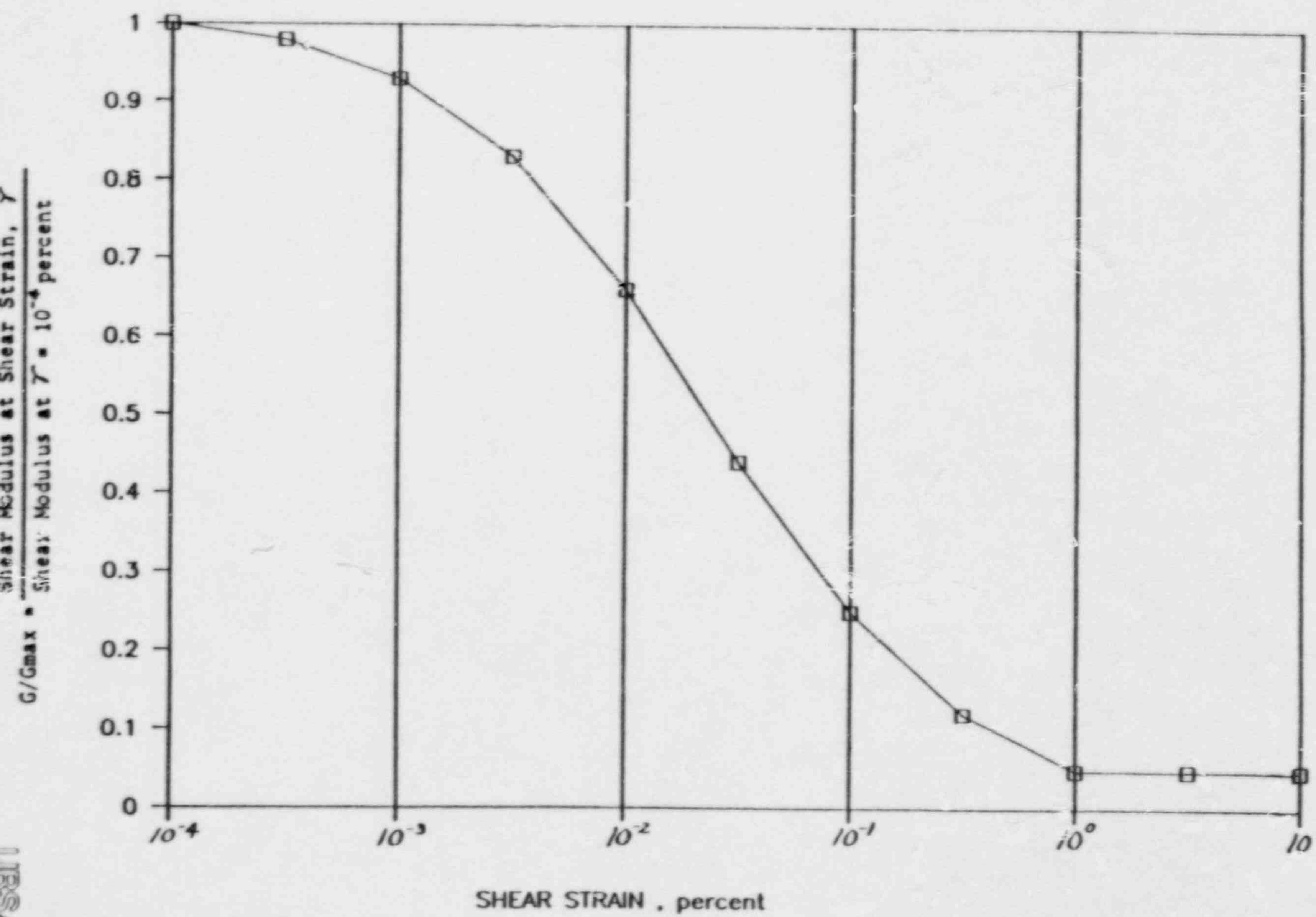


Figure 1. Variation of Shear Modulus with Shear Strain for Sands as Input in SuperFLUSH

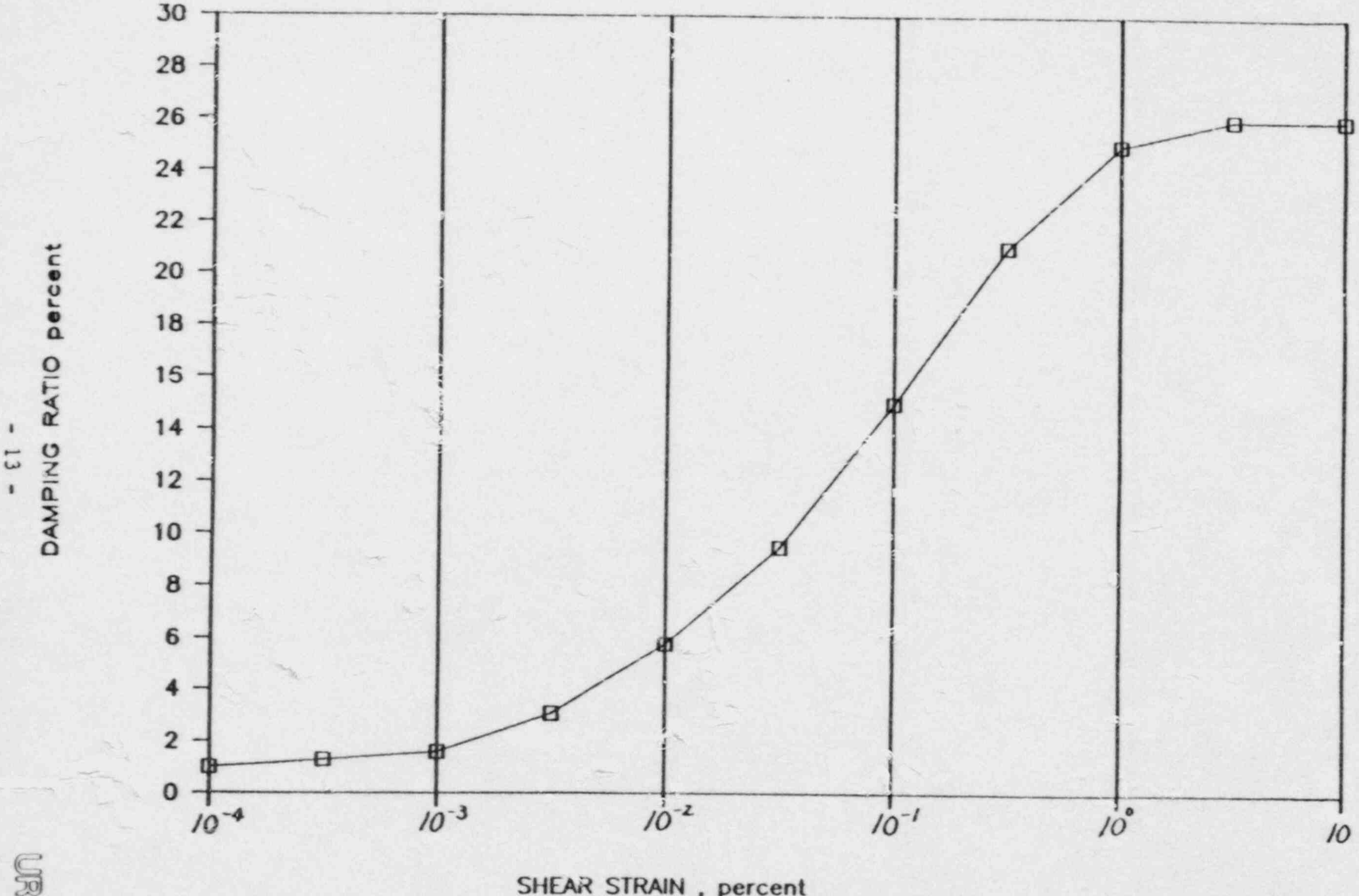
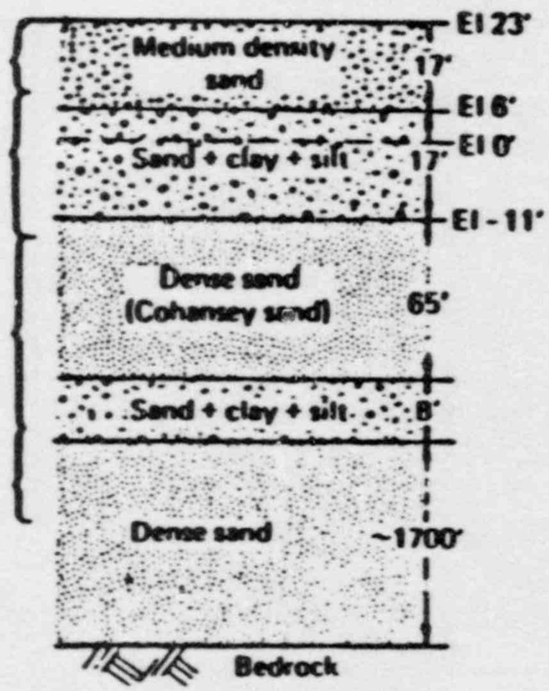


Figure 2. Damping Ratios for Sands as Input in SuperFLUSH

MEASURED P-WAVE VELOCITY Vp (ft/sec)	MEASURED SHEAR WAVE VELOCITY Vs (ft/sec)	POISSON'S RATIO	SHEAR MODULUS (KSF)			LAYER DEPTH (ft)
			BEST ESTIMATE	UPPER BOUND	LOWER BOUND	
1400	600	0.39	1342	1670	895	15
5200	1000	0.40	3727	4659	2485	25
5600	1200	0.40	5366	6708	3577	65
5900	1400	0.47	7384	9130	4869	35



Note: Water table varies between El. +3.5' to +10.0'

Figure 3. SOIL SHEAR WAVE VELOCITY PROFILE AT OYSTER CREEK PLANT

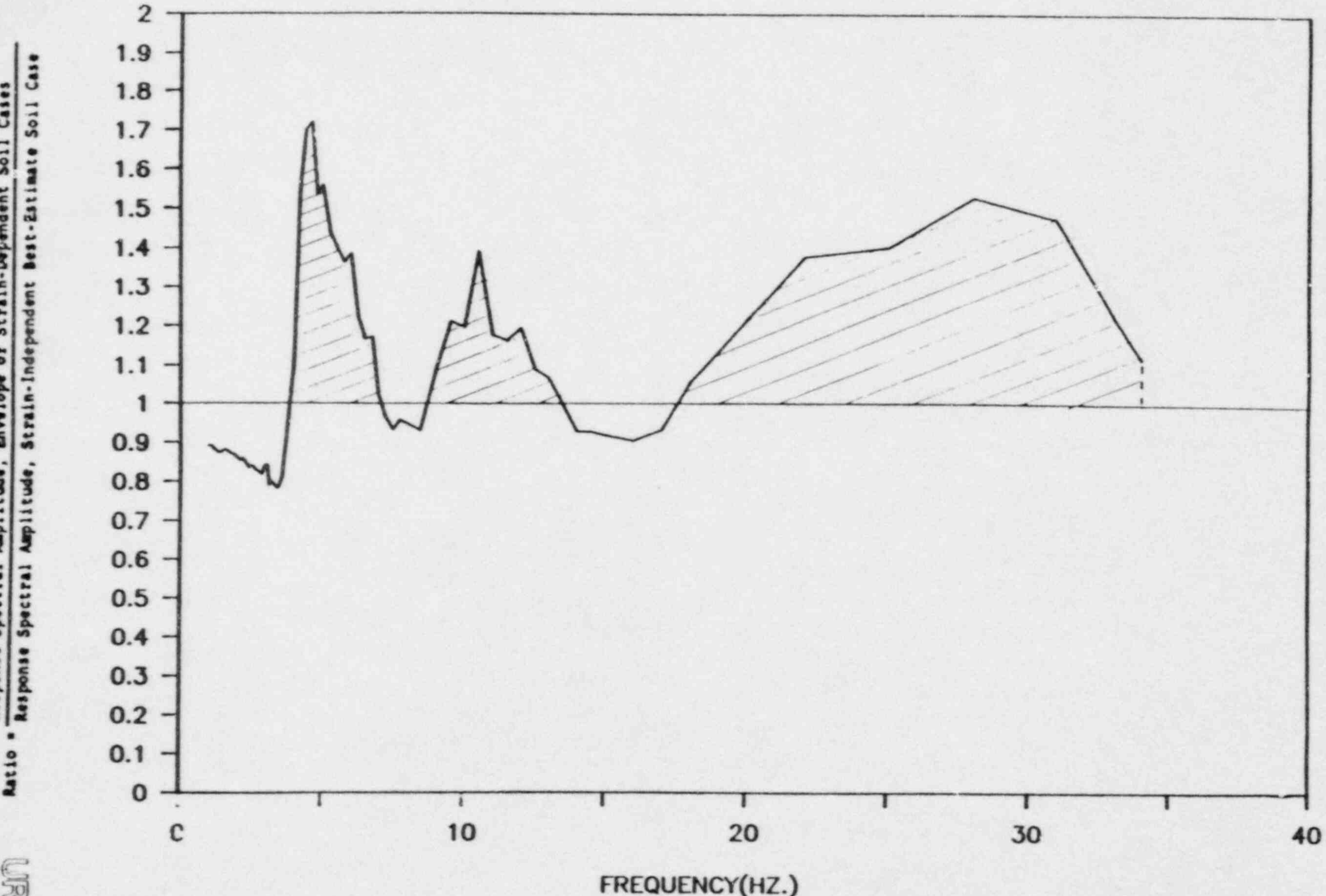


Figure 4. Effect of Strain-Dependent Soil Properties on Reduction of Motion at Basemat Level (El. -29'-6"), Oyster Creek Reactor Building

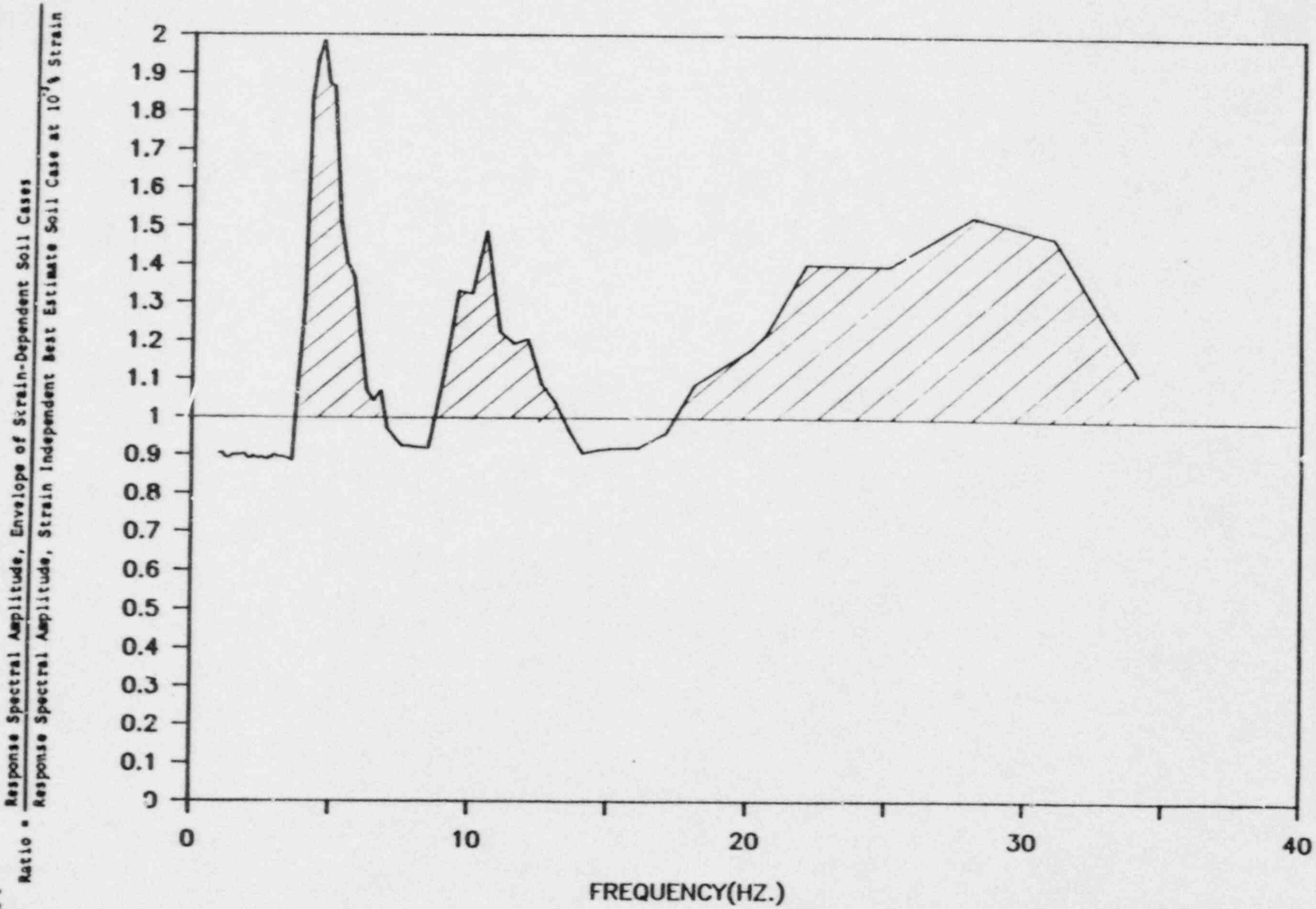


Figure 5. Effect of Strain-Dependent Soil Properties on Reduction of Motion at Basemat Level (El. -29'-6"), Oyster Creek Reactor Building

1A

Seismic soil-structure interaction effects at Fukushima Nuclear Power Plant in the Miyagiken-oki earthquake

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1 INTRODUCTION

The finite element method provides a powerful tool for the solution of seismic soil-structure interaction problems especially with complicated properties, geometries, and boundary conditions (EET 1983). However, only limited field observations of strong excitation at nuclear power plants are available at present. This paper describes soil-structure interaction analyses of the Fukushima Daiichi Nuclear Power Plant complex using motions recorded at the plant during the Miyagiken-oki earthquake and evaluates the adequacy of the analytical methods used in current design practice.

2 DESCRIPTION OF THE REACTOR BUILDING

The reactor building, which houses a 1100 MWe BWR Mark II reactor is constructed of reinforced concrete and is structually isolated from the adjacent turbine building and radwaste building. The reactor building is about 73m in height from the base of the foundation mat and is 68.5m x 68.3m in plan at the basement level and 45.5m x 42.5m in plan at the roof level. The reactor building is partially embedded and is founded on a mudstone at a depth of 17m below ground surface.

3 MIYAGIKEN-OKI EARTHQUAKE OF 1978 AND OBSERVATION SYSTEM

On June 12, 1978, the Fukushima Daiichi Nuclear Power Plant experienced a strong earthquake. The earthquake was assigned a Richter magnitude of 7.4 with the epicenter being approximately 140km from the plant and the focal depth being 40km. Four accelerometers of the moving coil type were installed at the time of the earthquake and all four recorded motions. The accelerometers are located under the roof (P01), on the refueling floor (P02), at the top of the base mat (P03) and in the mudstone at a depth of 31m below ground surface (P04). The duration of strong motion was more than 30 seconds. The observed peak accelerations were 145 to 208 Gal at the plant roof level and 60 to 84 Gal inside the mudstone. Following the Miyagiken-oki earthquake, the total number of observation points has been increased. Cross-sectional view of the locations of the seismographs are shown in Fig. 1 and more explanations are available in reference (Narikawa 1987).

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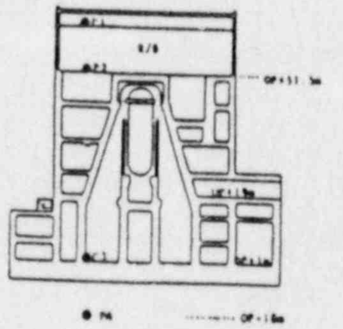

Fig. 1 Seismograph Locations

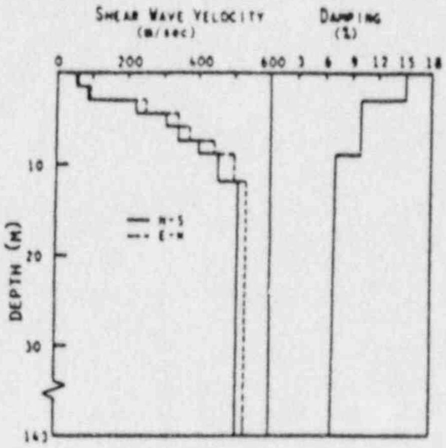

Fig. 2 Soil Properties

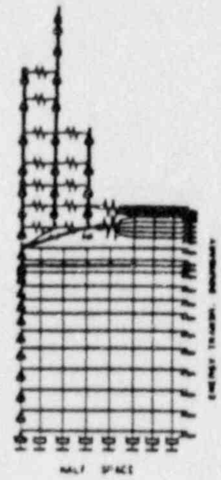

Fig. 3 Model

4 SOIL-STRUCTURE INTERACTION ANALYSES

4.1 Outline of analytical approach

The computer program "SuperFLUSH" (EET 1983, Lysmer 1975) has been used in order to analyze the response of the Unit 6 reactor building during the earthquake. This program uses the complex response method of computing the response of a finite element model. In this study, seismic soil-structure interaction analyses have been carried out using a special deconvolutional procedure. Commonly, in soil-structure interaction analyses the control motion is specified at the surface of the free field away from the structure. However, the locations P01 to P04 are too close to the reactor building to avoid soil-structure interaction effects, and may not be considered to be free field motions. In this series of studies, the control motions were considered to be at two of the observed locations in the finite element model and a deconvolutional procedure has been used to estimate the free field ground surface motion. The deconvolutional procedure was applied to the recorded accelerations at both P03 and P04. However, since the ratio of vertical excitation to horizontal excitation was negligible, only the horizontal component was considered at this location. The results obtained from a previous study (Narikawa, 1987) based on a multi-point observation system were also utilized to obtain better understanding of the soil-structure behavior at Unit 6 reactor building site and the motion recorded at Unit 1 site was also used as a control motion for simulation of the response of Unit 6.

4.2 Material properties

The soil properties used in the analyses are shown in Fig. 2. The same shear wave velocities as used by Narikawa (1987) were used for the soil-structure interaction studies. The damping ratios used were higher than those used in the previous study because of the higher level of excita-

tion. The reactor building itself was represented by three flexible beams and the foundation was modeled by a series of rigid beams. The structure properties are also identical to those used by Narikawa (1987) and were determined by computing effective shear areas, mass moments, moments of inertia and masses based on the original blue prints of the reactor building. A damping ratio of 2% was assigned to all structural components to be compatible with the previous analyses. The finite element model used in the analyses is shown in Fig. 3. A semi-infinite half space (EET 1983, Joyner 1975) is assumed at a depth of 65m below ground surface and energy transmitting boundaries (EET 1983, Waas 1972) are attached at the vertical boundaries in order to transmit energy beyond the finite element mesh.

4.3 Analyses based on the motions recorded at P03 and P04

Comparisons of the recorded response spectra and computed response spectra which assume P03 as a control point at the foundation mat are shown in Figs. 4(a) through 4(c). The solid lines and dotted lines in the figures are recorded and computed response spectra, respectively. The comparisons show excellent agreement with each other. Similar comparisons of recorded and computed response spectra are shown in Figs. 5(a) through 5(c), taking the control point to be P04. The discrepancy between the observed and computed motions at P04 in the range of 0.10 to 0.30 seconds that is shown in Fig. 4(c) significantly affected the structural response. This discrepancy may be due to the irregularity of the geometry and material properties of the real soil and rock layers.

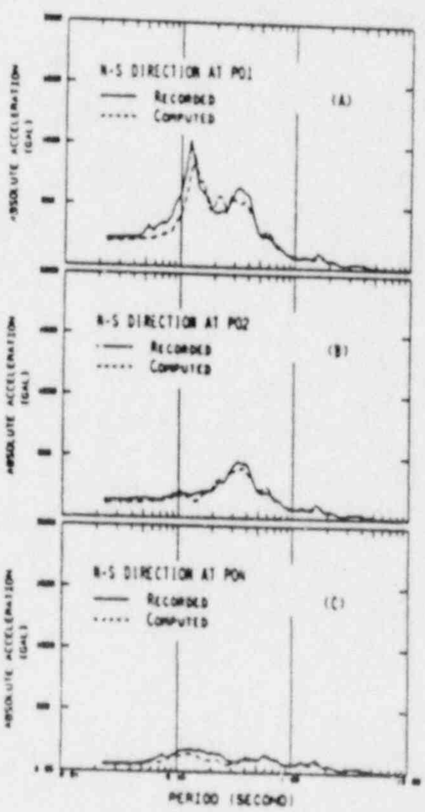

Fig. 4 Comparisons of Response Spectra for Control Motion at P03

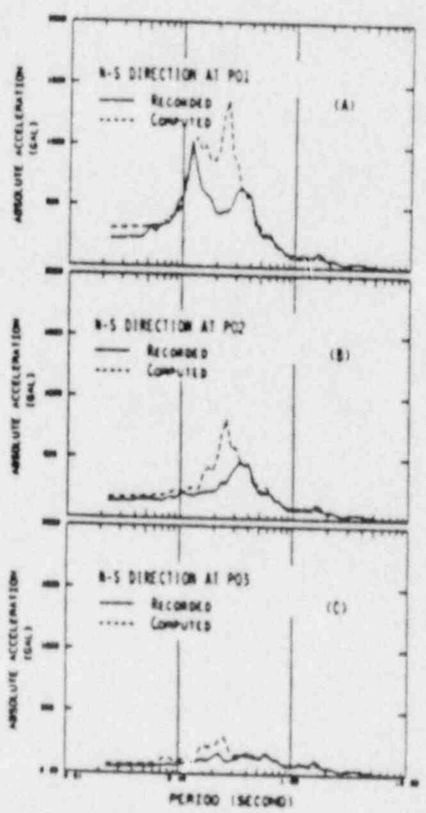

Fig. 5 Comparisons of Response Spectra for Control Motion at P04

Seismic Soil-Structure interaction effects at Fukushima Nuclear Power Plant

4.4 Analyses assuming a local irregularity at P04

In order to compensate for the discrepancy observed at location P04, local irregularity of the material properties has been introduced to the analyses. The properties in the vicinity of locarion P04 were assumed to be slightly softer for the upper layers and slightly stiffer below 30m as compared to those used in the free field. The incident wave component of the recorded motions was then computed at a depth of 143m below ground surface and used as the control motion in further analyses. This procedure is illustrated schematically in Fig. 6. Comparisons of recorded and computed response spectra and transfer functions in N-S direction are shown in Figs. 7(a) through 7(d), and Figs. 8(a) through 8(c), respectively. The transfer functions obtained are relative to the motion at P04. Figs. 9(a) through 9(d) show comparisons of the recorded and computed response spectra in E-W direction. The computed values are in good agreement with the recorded values.

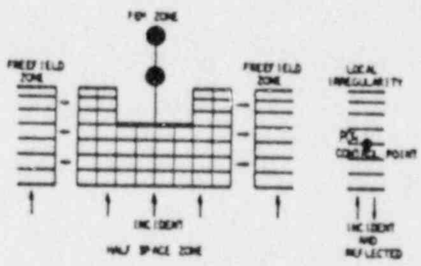

FIG. 6 SCHEMATIC REPRESENTATION OF THE PROCEDURE INTRODUCING LOCAL IRREGULARITY

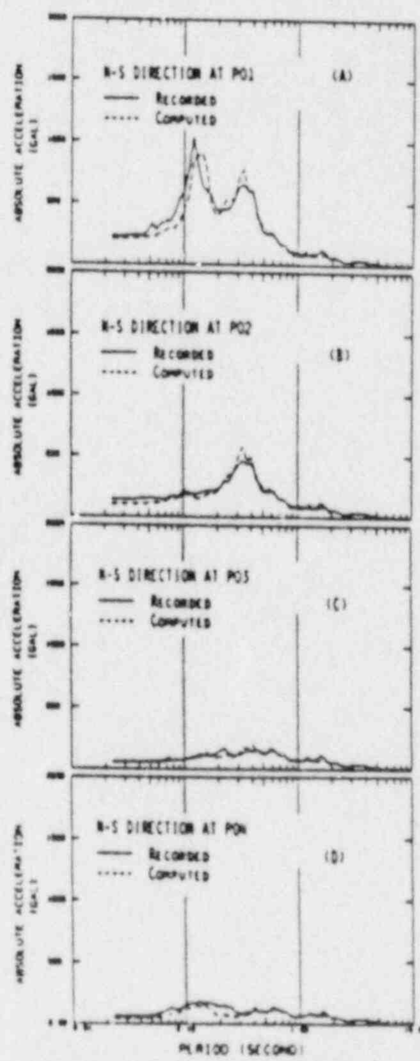

FIG. 7 COMPARISONS OF RESPONSE SPECTRA ASSUMING A LOCAL IRREGULARITY AT P04 (N-S)

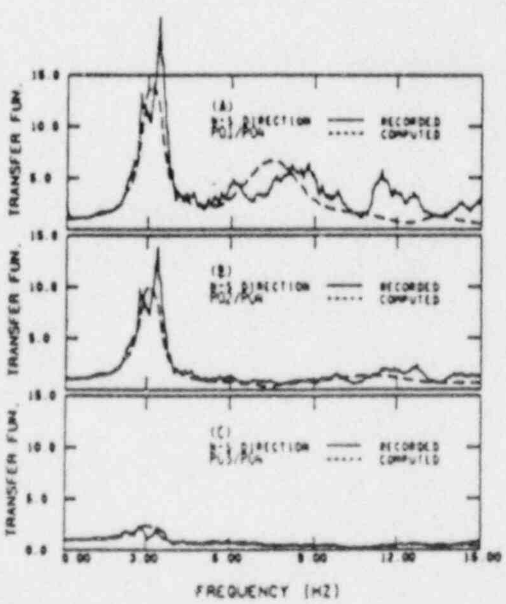

FIG. 8 COMPARISONS OF TRANSFER FUNCTIONS

Seismic soil-structure interaction effects at Fukushima Nuclear Power Plant in the Miyagiken-Oki earthquake

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4.5 Analyses based on the motion recorded at Unit 1

The motion recorded at Unit 1 at a depth of 14m below the ground surface was also used as an input motion and the corresponding incident wave component was obtained at a depth of 143m using the mudstone properties given by Tanaka (1980). Figs. 10(a) through 10(d) show comparisons of the recorded and computed response spectra at Unit 6. Good agreement is again obtained.

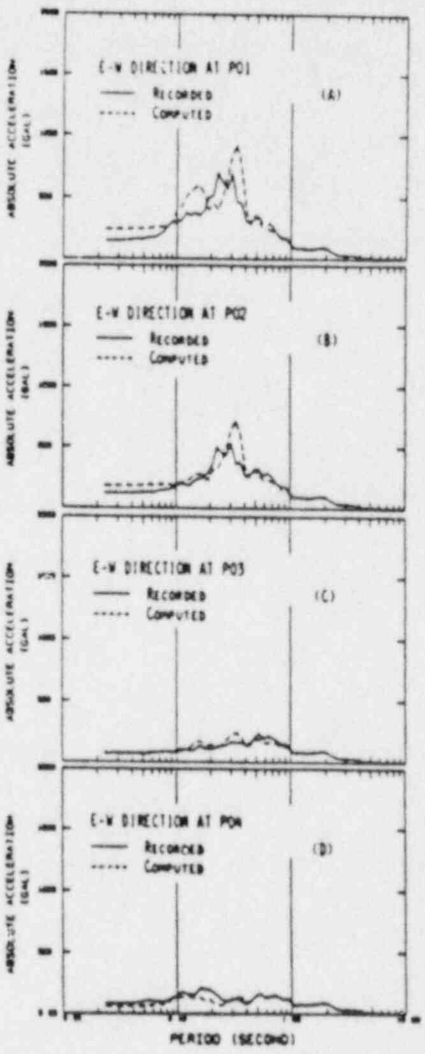

FIG. 9 COMPARISONS OF RESPONSE SPECTRA ASSUMING A LOCAL IRREGULARITY AT P04 (E-W)

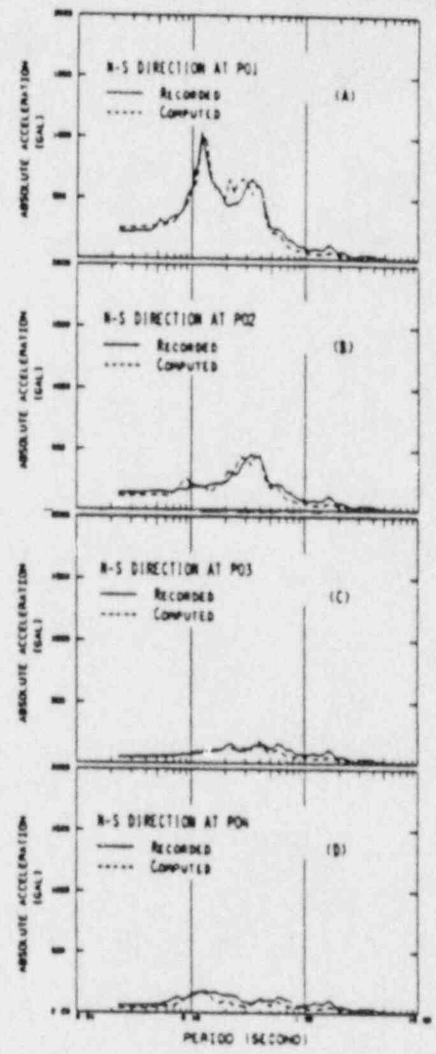

FIG. 10 COMPARISONS OF RESPONSE SPECTRA BASED ON THE MOTION RECORDED AT UNIT 1 (AN9)

Seismic soil-structure interaction effects at Fukushima Nuclear

5 CONCLUSION

In order to obtain a better understanding of the behavior of the reactor building, a number of dynamic soil-structure interaction analyses have been performed using the motions recorded during the Miyagiken-oki earthquake of 1978. The results of the analyses show good agreement with the recorded responses. The results presented herein along with those presented by Narikawa (1987) indicate that this class of soil-structure interaction analyses can be very useful in evaluating the observed responses of reactor buildings and should give increased confidence to their use in design analyses.

ACKNOWLEDGEMENT

Many valuable suggestions from Dr. H. Tanaka are deeply appreciated.

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Seismic soil-structure interaction effects at Fukushima Nuclear Power Plant

(1B)

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Seismic soil-structure interaction behavior at Fukushima Nuclear Power Plant based on multi-point observations

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1 INTRODUCTION

Evaluation of soil-structure interaction effects is one of the most important processes in the seismic design of nuclear power plants. In order to better understand soil-structure interaction effects and to accomplish more safely designed structures it is therefore very desirable to obtain observations of field performance and to perform analytical simulations for both large and small earthquakes. On September 14, 1982, a minor earthquake occurred off the coast of the Tohoku district of the Main Island of Japan. The earthquake was assigned a Richter magnitude of 5.0 with the epicenter being approximately 37km from the Fukushima Daiichi Nuclear Power Station of The Tokyo Electric Power Company and the focal depth being approximately 60km. Acceleration records were obtained at thirteen locations both within and outside the Unit 6 reactor building. Analytical simulations have subsequently performed to study the dynamic response of the reactor building and to examine the adequacy of the analytical methods used in current design practice.

2 DESCRIPTION OF THE REACTOR BUILDING

The reactor building of Unit 6 of the Fukushima Daiichi Nuclear Power Plant (a BWR Mark II type, 1100 MWe) is approximately 73m high from the bottom of base mat (OP-4.0m) to the top of the structure (OP+68.6m). The building is partially embedded and is founded on a mudstone at an elevation of 17m below ground surface. The plan dimension of the reactor building is 68.5m x 68.3m at the bottom and 45.5m x 42.5m at the top. The reactor building is constructed of reinforced concrete and is structually isolated from the adjacent turbine and radwaste buildings.

3 MULTI-POINT OBSERVATION SYSTEM

Cross-sectional view of the approximate locations of the seismographs are shown in Fig. 1. Two seismographs are installed at the roof level (Op+65.5m, P01 and P11), two at the refueling floor level (OP+51.5m, P02 and P10), one at OP+19m level (P08), and two in the basement floor (OP+1m, P03 and P05), resulting in total of seven seismographs inside the reactor building. Five seismographs are installed outside the reactor building: two (P04 and P13) in the mudstone at a depth of

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-31m below ground surface near the reactor building, one (P14) at a depth of -143m below ground surface, one at the surface (P07) and one at depth of -17m (P12). The seismographs P07 and P12 are located 130m north of the reactor building, and are considered to be located in a nearly perfect free field environment. The observed peak acceleration during the earthquake on September 14, 1982 were 26 to 28.6 Gal at the plant roof level and 20.5 to 26.5 Gal at the free field ground surface. The approximate duration of motions was 40 seconds.

4 SOIL-STRUCTURE INTERACTION ANALYSIS

4.1 Outline of analytical approach

Dynamic soil-structure interaction analyses have been performed utilizing the computer program "SuperFLUSH" (EET 1983), which uses the complex response method of computing the response of a finite element model (Lysmer 1975). The recorded motion at the ground surface in the free field was taken as control motion for the analysis. A semi-infinite half space was assumed at depth of 65m at the bottom of the finite element model. A technique for separating incident and reflected components of the incoming motion was used and only incident component is applied at the bottom of the finite element model. The reflected component, including the disturbance due to the reactor building, is absorbed by the half space (Joyner 1975, Udaka 1981). This approach eliminates any uncertainty caused by the assumption of any arbitrary depth for the finite element model. Energy transmitting boundaries (EET 1983, Lysmer 1975, Waas 1972) were attached at the vertical boundaries of the finite element model to simulate the existence of semi-infinite soil layers beyond the finite element model. Soil-structure interaction analyses were performed in both N-S and E-W directions and rocking motion of the reactor building along an axis approximately 45 degrees from the N-S direction was computed by taking the resultant accelerations of the N-S and E-W excitations. Torsional behavior of the structure was also simulated by use of a traveling SH wave concept.

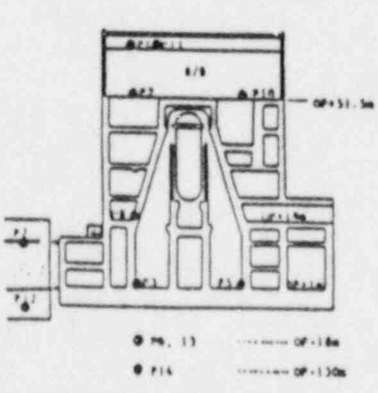

FIG. 1 SEISMOGRAPH LOCATIONS

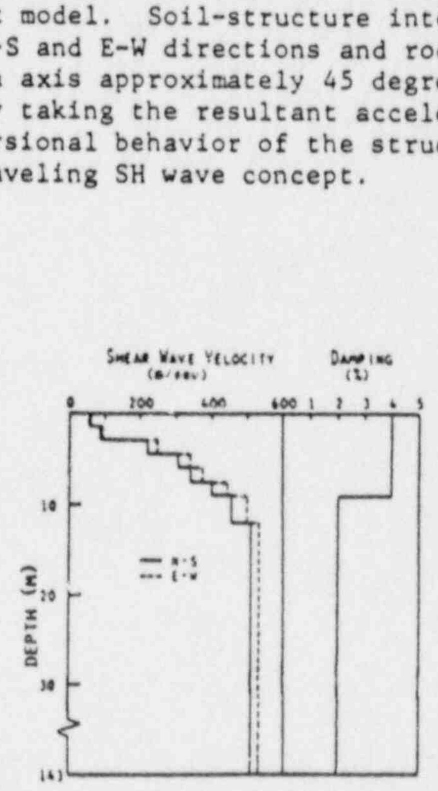

FIG. 2 SOIL PROPERTIES

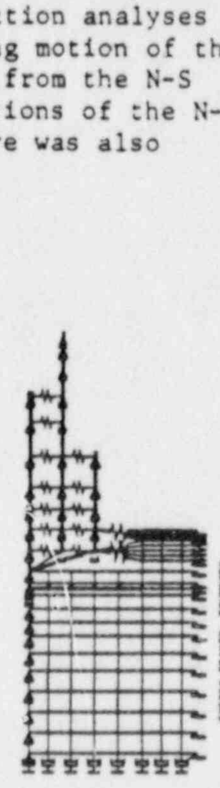

FIG. 3 MODEL

Seismic soil-structure interaction behavior at Fukushima Nuclear Power Plant based on multi-point observations

4.2 Material properties

The soil properties used in the analyses are shown in Fig. 2. The reactor building was modeled in the N-S direction by three shear beams whose properties were obtained by computing effective shear areas, mass moments, moments of inertia, and masses based on the original blue prints for the reactor building. The base mat was modeled as a rigid beam. A damping ratio of 2% was assumed for all structural components. Fig. 3 shows the finite element model used for soil-structure interaction analyses.

4.3 Comparison of recorded and computed responses

4.3.1 Horizontal motions

Fig. 4 shows comparisons of the recorded and computed responses in the N-S direction at P10, located on the refueling level, at P08, located on OP+19m level, and at P05, located in the basement. Similarly, Fig. 5 shows comparisons of the recorded and computed responses in the E-W direction. All the response spectra are computed for a damping ratio of 5%. The transfer functions shown in Fig. 6 for the N-S direction and in Fig. 7 for the E-W direction are relative to the free field ground surface motions (P07). A comparison of the recorded and computed maximum accelerations is tabulated in Table 1. The computed values are in extremely good agreement with the recorded values.

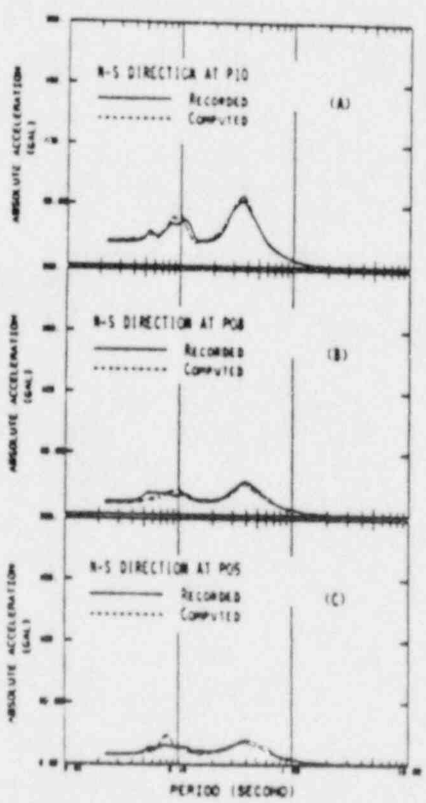

Fig. 4 Comparisons of Response Spectra (N-S)

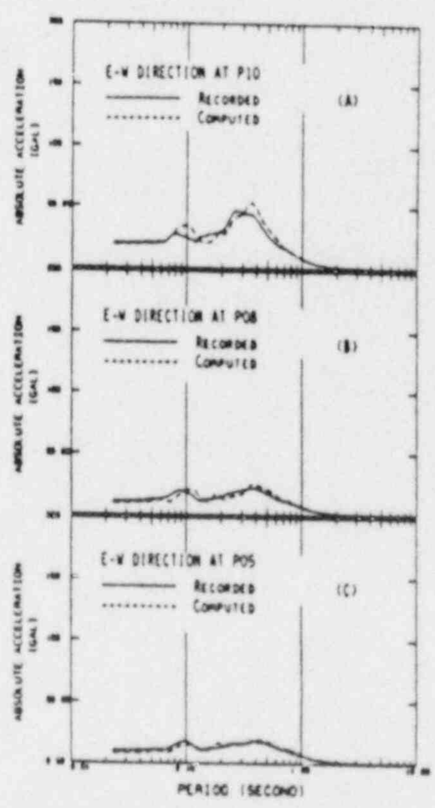

Fig. 5 Comparisons of Response Spectra (E-W)

Seismic soil-structure interaction behavior at Fukushima Nuclear Power

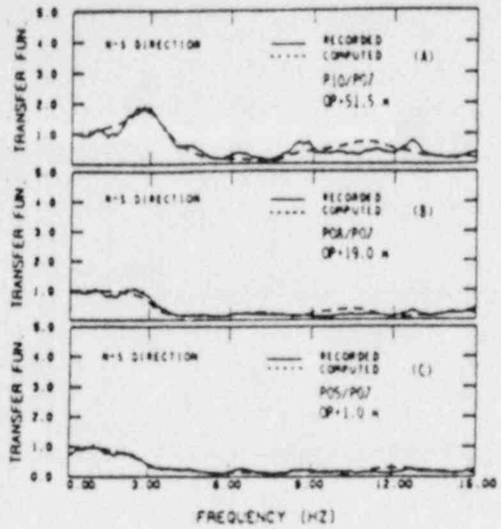

Fig. 6 Comparisons of Transfer Functions (N-S)

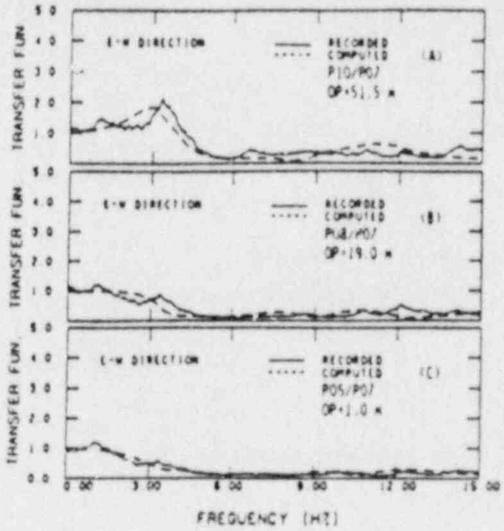

Fig. 7 Comparisons of Transfer Functions (E-W)

Table 1 Comparison of Maximum Accelerations

		P01	P02	P10	P08	P03	P05
N-S	RECORDED	26.0	23.3	18.8	10.1	8.7	7.8
	COMPUTED	25.6	20.4	20.4	9.2	8.1	8.1
E-W	RECORDED	28.6	21.8	20.2	11.0	11.1	9.7
	COMPUTED	32.7	19.3	19.3	10.2	8.5	8.5

Unit Gal

4.3.2 Rocking

The seismographs P03 and P05 are located on a line which is nearly 45 degrees from the N-S axis. Therefore, the accelerations obtained by the subtraction of the observed vertical components at P03 from P05 can be assumed to be twice the rocking component of the foundation along this axis. Likewise, the computed vertical responses due to both N-S and E-W horizontal excitations at the foundation level are simply combined to obtain the rocking response on the axis 45 degrees from the N-S direction. A comparison of the recorded and computed rocking response spectra is shown in Fig. 8. The computed response is again in excellent agreement with the recorded response. The minor differences that can be seen may be due to the assumption in the analysis that the structure is perfectly symmetric when this is not actually true.

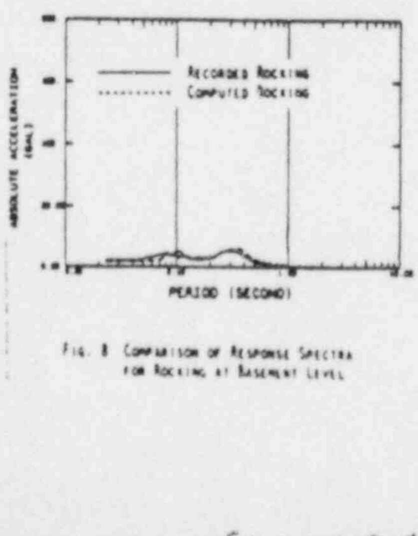

Fig. 8 Comparison of Response Spectra for Rocking at Basement Level

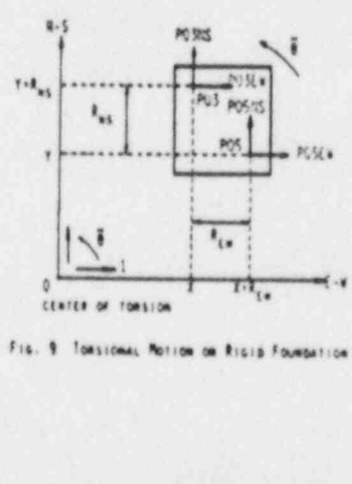

Fig. 9 Torsional Motion on Rigid Foundation

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Seismic soil-structure interaction behavior at Fukushima Nuclear Power

4.3.3 Torsional behavior

In order to evaluate the torsional response of Fukushima Unit 6, the base foundation was assumed to be perfectly rigid as shown in Fig. 9. The torsional components with respect to the N-S and E-W directions can then be expressed as follows:

$$\ddot{\theta}_{NS} = (P03NS-P05NS)/R_{EW}$$
$$\ddot{\theta}_{EW} = (P05EW-P03EW)/R_{NS} \qquad \text{Eq. (1)}$$

If $\ddot{\theta}_{NS}$ is equal to $\ddot{\theta}_{EW}$, because of the assumption of rigidity, the foundation has a torsional movement with angular acceleration of $\ddot{\theta}_{NS}$.

The observed torsional motions were then computed using Eq. (1) and the response spectra for $\ddot{\theta}_{NS}$ and $\ddot{\theta}_{EW}$ multiplied by a factor R_{NS} as shown in Fig. 10. It is apparent that the reactor building has a rigid body torsional component in the order of 10% of the translational component.

The torsional components $\ddot{\theta}_{NS} R_{EW}$ and $\ddot{\theta}_{EW} R_{NS}$ can also be calculated using the recorded responses at P03 and traveling SH wave concepts (Udaka 1979) with phase velocities of 2000m/sec in N-S and 3000m/sec in E-W directions, respectively. A phase velocity of 2000m/sec corresponds to an incident angle of approximately 15 degrees to the vertical while a phase velocity of 3000m/sec corresponds to an incident angle of 10 degrees. Comparisons of response spectra based on the observed motion computed using Eq. (1) and the traveling SH wave concept are shown in Fig. 11. The results shown suggest that torsional behavior at the site may be well explained by the traveling SH wave concept.

Finally $\ddot{\theta}_{NS}$ and $\ddot{\theta}_{EW}$ were computed using the responses computed in the soil-structure interaction analysis and traveling wave concepts with phase velocities of 2000m/sec for the N-S and 3000m/sec for the E-W directions, respectively. Comparison of response spectra for recorded and computed torsional components are shown in Fig. 12. The good agreement that can be seen between recorded and computed results should increase confidence in the concept of traveling SH waves for evaluating the torsional response of structures.

The computed torsional component $\ddot{\theta}_{NS} R_{EW}$ was then added to the computed translational components. The computed response spectra with the torsional component included are shown in Fig. 13. It may be seen that the torsional behavior has no significant effect on the total translational behavior in this case.

5 CONCLUSION

The results of the analyses of horizontal and rocking motions show excellent agreement with the recorded responses. The torsional movement of the reactor building was also studied and was found to have no significant effect on the horizontal responses. These results indicate that the methods used herein are both useful and suitable for evaluation of soil-structure interaction effects and also that analysis based on motions generated by small magnitude earthquakes can be very informative in evaluating the adequacy of analysis and design procedures.

ACKNOWLEDGEMENT

Many valuable suggestions from Dr. H. Tanaka are deeply appreciated.

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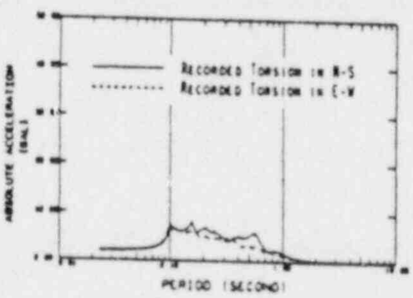

Fig. 10 Comparison of Response Spectra for Torsion Recorded at Basement Level

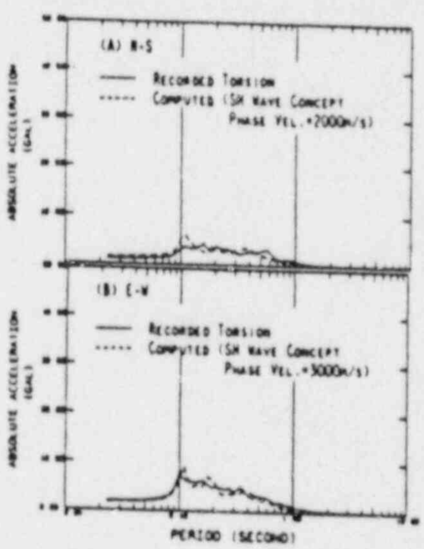

Fig. 12 Recorded and Computed Response Spectra for Torsion at Basement Level

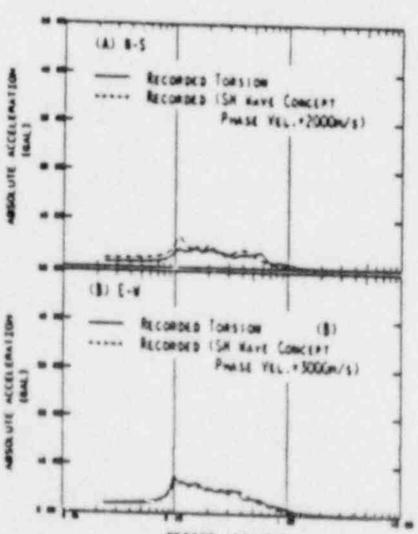

Fig. 11 Response Spectra for Torsion Obtained from SH Wave Concept

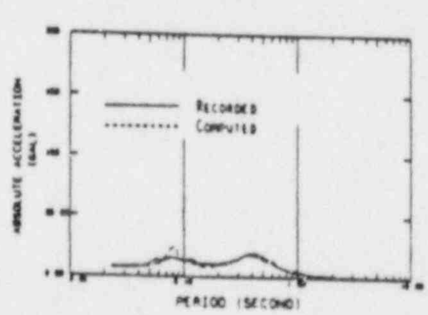

Fig. 13 Response Spectra at Basement Level with Torsional Component

MODELLING UNCERTAINTIES

The following paragraphs describe our response to the modelling uncertainties item mentioned by USNRC audit staff (Ref. Memo from Mr. A. Dromerick (NRC) to Mr. P. B. Fiedler (GPU), dated December 16, 1987, item 2, page 1). Under this item, NRC staff wanted us to provide published reports of comparisons between SuperFLUSH analytical results and measured responses of structures under actual seismic events. This is to obtain assurance that SSI modelling and computations as implemented in the SuperFLUSH code properly capture the structural responses in the field.

The following documents are submitted herewith in response to the abovementioned USNRC request:

1. Validation of SuperFLUSH code with Measured Earthquake Results: The following papers present comparisons of SuperFLUSH analytical results and actual measured earthquake responses at Fukushima Unit 6 plant (BWR MK.II, 1100 MWe) in Japan. The earthquakes were Miyagiken-Oki (June 12, 1978, 7.4 magnitude, 140 km epicentral distance) and Tohoku (September 14, 1982, magnitude 5, 37 km epicentral distance).

   1a. Narikawa, Udaka, and Okumura, "Seismic Soil-structure Interaction Effects at Fukushima Nuclear Power Plant in Miyagiken-Oki Earthquake," Paper K5/7, SMiRT-9, Lausanne, 1987.

   1b. Narikawa, Udaka, and Okumura, "Seismic Soil-structure Interaction Behavior at Fukushima Nuclear Plant Based on Multi-point Observations," Paper K5/8, SMiRT-9, Lausanne, 1987.

2. Comparison of Analysis Results from FLUSH Group of Programs with Measured Earthquake Results: The following papers compare results from SSI analyses using FLUSH group of programs with measured results from actual earthquakes in nuclear plant structures or scaled models.

2a. Valera, Seed, Tsai, and Lysmer, "Soil-structure Interaction Effects at the Humboldt Bay Power Plant in the Ferndale Earthquake of June 7, 1975," J. Geot. Engrg. Div., ASCE, Vol. 103, No. GT10, October 1977.

2b. Berger, Fierz, and Kluge, "Predictive Response Computations for Vibration Tests and Earthquake of May 20, 1986, Using an Axisymmetric Finite Element Formulation Based on the Complex Response Method and Comparison with Measurements--A Swiss Contribution," EPRI/NRC/TPC Lotung SSI Workshop, 1987.

2c. Sato, Nakai, Yamazaki, Mita, and Ishii, "Soil-structure Interaction Analysis of Quarter-Scale Model Using AXERA Code," EPRI/NRC/TPC Lotung SSI Workshop, 1987.

2d. Tseng, et al., "Soil-structure Interaction Analyses of Quarter-Scale Containment Model Experiment in Lotung, Taiwan; Part 3: FLUSH Method," EPRI/NRC/TPC Lotung SSI Workshop, 1987.

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JOURNAL OF THE GEOTECHNICAL ENGINEERING DIVISION

Seismic Soil-Structure Interaction Effects at Humboldt Bay Power Plant

By Julio E. Valera,[1] M. ASCE, H. Bolton Seed,[2] F. ASCE, C. F. Tsai,[3] and John Lysmer,[4] M. ASCE

Introduction

One of the many controversial aspects of nuclear power plant design in the past several years has been that of evaluating the seismic soil-structure interaction effects during design levels of earthquake shaking. Basically, two methods of approach are available for determining these effects: (1) Complete interaction analyses that attempt to make some evaluation of the variations in earthquake motions, both in the structure and in the soil in which it is embedded; and (2) inertial interaction analyses in which the motions in the soil surrounding the structure are considered to be some representative average motion having the same characteristics at all points (10). The former approach has usually been applied through the use of finite element methods of analysis while the latter, although it can be performed using finite element techniques, has usually been associated with half-space analyses of elastic or viscoelastic layered systems. It appears to be the prevailing opinion "that for near surface structures, good results can be obtained by a well performed analysis of either type. However, for embedded structures, the complete interaction analysis approach comes closest to representing in a rational way all the important aspects of the problem" (1). The principal limitation of this approach at the present time is usually considered to be the cost of the analysis and, in some cases, the less expensive inertial interaction approach may provide results of sufficient accuracy for practical purposes. However, as increasingly efficient and versatile computer programs are developed for finite element analyses, and as progressively more

2A

Note.—Discussion open until March 1, 1978. To extend the closing date one month, a written request must be filed with the Editor of Technical Publications, ASCE. This paper is part of the copyrighted Journal of the Geotechnical Engineering Division, Proceedings of the American Society of Civil Engineers, Vol. 103, No. GT10, October, 1977. Manuscript was submitted for review for possible publication on February 8, 1977.
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field (Storage Building) were 0.35 g and 0.26 g in the transverse and longitudinal directions respectively, making these the strongest earthquake motions to which a nuclear power plant has so far been subjected. However there was no observable damage to the facility resulting from these motions.

A fortuitous aspect of the records obtained from the Humboldt Bay Plant was the fact that the soil conditions at the plant site had been determined by a comprehensive field investigation only approx 12 months before the earthquake occurred. In fact, extensive liquifaction and soil-structure interaction analyses using finite element procedures, with accompanying determinations of soil characteristics at the site, had been carried out prior to the earthquake of June 7, 1975. These studies were performed by Dames & Moore using analytical techniques developed at the University of California at Berkeley (8). In this respect, it is interesting to note that these analyses had predicted a peak acceleration at the base of the Refueling Building of 0.13 g for a free-field ground surface acceleration of 0.25 g, while the subsequent earthquake produced an average peak acceleration at the base of the Refueling Building, of 0.14 g for an average free-field ground surface acceleration of 0.30 g. This result alone, predicted in advance of the event and published in design reports, is of considerable interest.

Though these facts are of major importance, perhaps the most significant feature of the June 7 event is the opportunity it provided to check the adequacy of seismic design procedures against the known performance of a prototype structure under known field conditions of considerable intensity. The results of such an evaluation are presented in the following pages.

Site Conditions and Soil Properties

A general description of the subsurface soil conditions at the plant site has been presented by Valera and Brady. A crosssection through Unit 3 in the north-south direction is shown in Fig. 2. Basically, the soils around the Refueling Building consist of approx 25 ft (8 m) of medium to stiff clay [increasing to approx 30 ft (9 m) at the Storage Building], underlain successively by approx 30 ft (9 m) of medium-dense to dense sand, 10 ft (3 m) of very stiff clay and then a deep bed of dense sand containing some clay lenses extending to a depth of approx 400 ft (120 m). All of the soils surrounding the Refueling Building are overconsolidated, with an average overconsolidation ratio of at least 6–8, indicating that the coefficient of earth pressure at rest in the sands would be on the order of one or more. The soil profile and soil properties used in the pre-earthquake soil-structure interaction studies are shown in Figs. 3(*a*) and 4, respectively. The soil profiles and soil properties used in the present study are shown in Figs. 3, 4, and 5. The profile for the conditions adjacent to the Refueling Building was identical to that used in the pre-earthquake analyses.

At the site of the Storage Building itself, where the free-field records were obtained, there is some uncertainty about the actual strength of the top 30 ft (9 m) of clay, because the closest boring is at least 100 feet (30 m) away and there is considerable scatter in the measured values of shear strength for undisturbed samples of clay taken from three borings surrounding the building. This uncertainty is reflected by the ranges of strength values for these soils indicated in Fig. 3(*b*). To allow for this uncertainty, analyses were made for

a number of soil profiles involving clay strengths varying considerably in the upper 20 ft, (6 m) as shown in soil profiles A, B, and C in Fig. 5. Results for all profiles investigated fell within the range represented by profiles A and B.

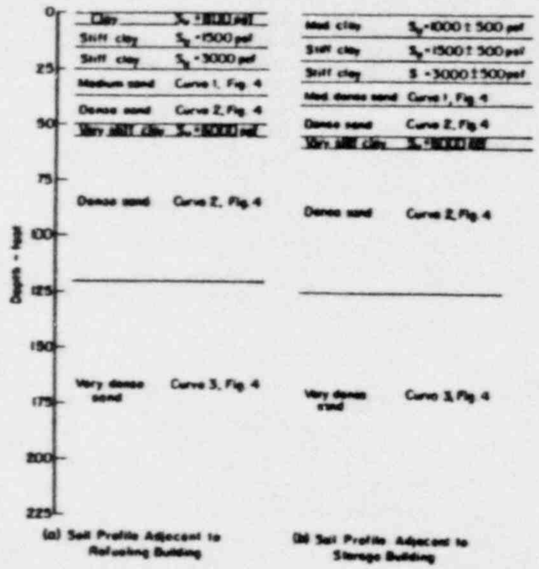

FIG. 3.—Soil Profiles at Humboldt Bay Power Plant Site (1 ft = 0.305 m; 1 psf = 47.9 N/m²)

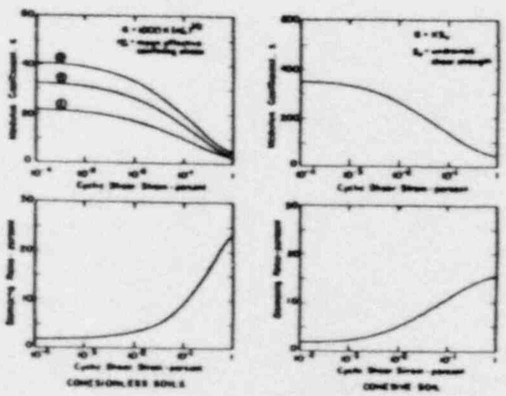

FIG. 4.—Average Dynamic Soil Properties

The dynamic shear moduli and damping characteristics of the soils were determined by standard soil testing procedures using resonant column tests and cyclic-triaxial tests on undisturbed samples reconsolidated under the in-situ confining pressures. These are shown in Fig. 4. It is pertinent to note that

these dynamic properties were determined and filed with the Nuclear Regulatory Commission before the earthquake of June 7, 1975. At the time the studies were initiated (early 1973) it was not considered necessary to make determinations of field shear wave velocities since it was clear from preliminary studies that

FIG. 5.—Typical Soil Profiles at Storage Building Used for Deconvolution Studies (1 ft = 0.305 m, 1 psf = 47.9 N/m²)

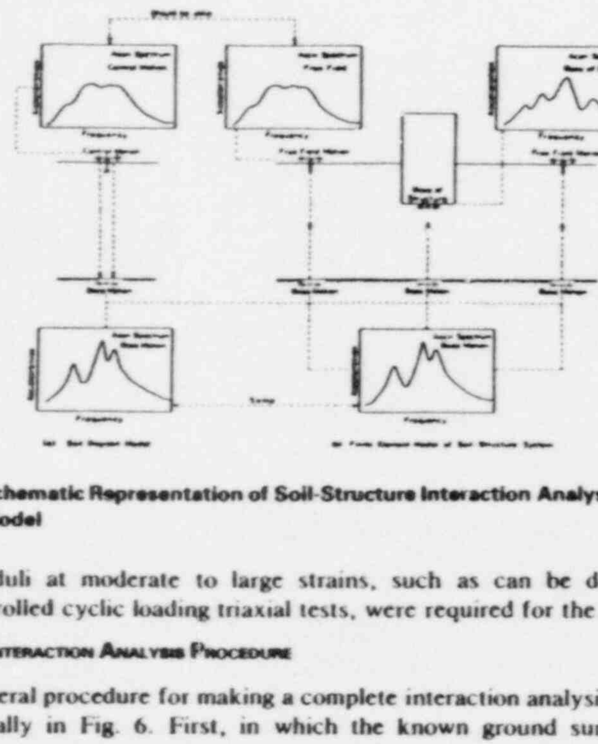

FIG. 6.—Schematic Representation of Soil-Structure Interaction Analysis Using Finite Element Model

shear moduli at moderate to large strains, such as can be determined by strain-controlled cyclic loading triaxial tests, were required for the analysis.

Complete Interaction Analysis Procedure

The general procedure for making a complete interaction analysis (7) is shown schematically in Fig. 6. First, in which the known ground surface motions

developed in the free-field are analyzed by a deconvolution procedure for the soil deposit alone to determine the motions which would have to be developed at a considerable depth below the ground surface [e.g., 150 ft–200 ft (46 m–61 m)] in order to produce the actual ground surface motions by transmission of body waves (vertical shear waves) through the soil deposit. This can be accomplished through the use of a computer program such as SHAKE (6). These same base motions are then used to analyze the response of a finite element model of the soil-structure system and the results of this latter analysis are checked by ensuring that the required freefield motions are indeed developed in the free field. The basic requirements of a suitable analysis and computer program (10) are that it should be capable of considering: (1) The variation of ground motions with depth; (2) the three-dimensional nature of the problem; (3) the effects of adjacent structures on each other, if appropriate; (4) the variation of soil characteristics with depth; and (5) the nonlinear stress-strain and energy-absorbing characteristics of the soil.

Results of Pre-Earthquake Analysis

The pre-earthquake studies performed by Dames and Moore were made using the computer programs SHAKE and LUSH (4). Analyses were carried out for cross sections in the north-south and east-west directions (Fig. 1) and for various levels of peak ground surface acceleration.

The soil properties shown in Figs. 3(*a*) and 4 together with the structure characteristics shown in Tables 1, 2, and 3 were assigned to the finite element model. Damping values of 4% and 7% were used for the structures for analyses conducted using peak ground surface accelerations of 0.25 g and 0.4 g, respectively.

From the results of the initial studies it was found that the effects of the adjacent structures on the response of the buried reactor caisson were relatively minor. Thus, the adjacent structures were not included in the finite element model used for the later studies. Since transmitting boundaries are not included in the computer program LUSH, it was necessary to use an extensive mesh in the horizontal direction to ensure that the computed response of the Reactor Caisson and Refueling Building was not influenced by the Boundary conditions of the analytical model. However, previous studies (2) have shown that it is only necessary to consider the response of the soil deposit to a depth of about one-half the structure width below the base of the structure; consequently, the base of the analytical model was taken at a depth of 150 ft (46 m) below the ground surface.

Deconvolution Studies.—In performing a deconvolution analysis of a ground surface motion to determine a corresponding base motion for use in a soil-structure interaction analysis, it is often necessary to filter out the high frequency components of the ground surface motion in order to obtain meaningful results. There are two reasons for this requirement:

1. The specified ground surface motion may contain high frequency components which would not, in reality, be developed for the site conditions under consideration. This is particularly true for sites consisting of deep [250-ft (76-m)] bodies of soil or including layers of soft to medium-stiff clay and sand (7).

2. Deconvolution by a wave propagation analysis using equivalent-linear properties to represent the nonlinear stress-strain characteristics of the soil inevitably leads to an excessive amplification, with depth, of high frequency motions.

TABLE 1.—Structural Properties of Reactor Caisson

Depth below ground surface, in feet (1)	Shear modulus, in pounds per square foot × 10^6 (2)	Density, in pounds per cubic foot (3)	Poisson's ratio (4)
0-15	289	0	0.2
15-31	86	0	0.2
31-44	80	0	0.2
44-71	76	0	0.2
71-78	83	0	0.2
78-87	4160	158	0.2

Note: 1 ft = 0.305 m; 1 psf = 47.9 N/m^2; 1 pcf = 16.0 kg/m^3.

TABLE 2.—Masses Lumped at Center Line of Reactor Caisson

Depth below ground surface, in feet (1)	Weight of mass, in kips (2)
0	82
15	82
25	76
37	44
51	43
57	47
71	54

Note: 1 ft = 0.305 m; 1 kip = 453.6 kg.

TABLE 3.—Structural Properties of Refueling Building

Depth above ground surface, in feet (1)	Shear modulus, in pounds per square foot × 10^6 (2)	Density, in pounds per cubic foot (3)	Poisson's ratio (4)
0-17.5	5	25	0.2
17.5-35	5	10	0.2

Note: 1 ft = 0.305 m; 1 psf = 47.9 N/m^2; 1 pcf = 16.0 kg/m^3.

In the pre-earthquake deconvolution analyses, the acceleration time history shown at the top of Fig. 7 was used as the free-field ground surface motion.

The spectra for this time history closely match the Nuclear Regulatory Commission (NRC) design spectra stipulated in Regulatory Guide 1.60. In these studies, it was necessary to use a cutoff frequency of 15 Hz-20 Hz in order to ensure that the accelerations at depth did not become excessive.

Acceleration time histories computed at various depths within the free-field soil profile are also shown in Fig. 7. It may be seen that there is both a decrease in the amplitude of the motion and an increase in the frequency content, with an increase in depth within the profile.

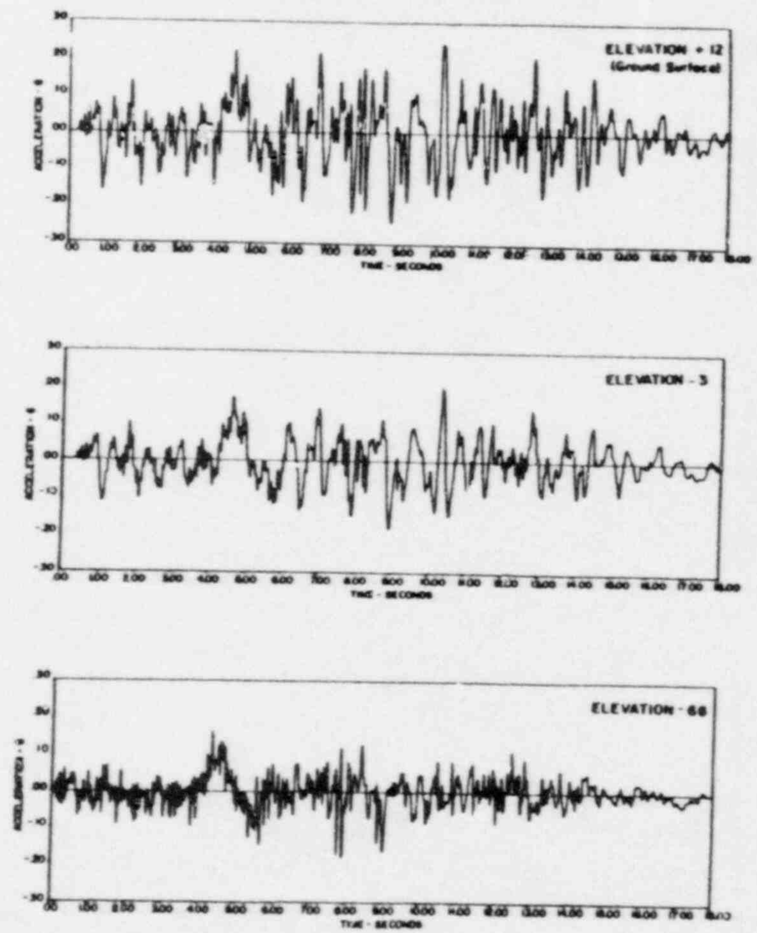

FIG. 7.—Horizontal Acceleration Histories Computed by Deconvolution of Surface Motions for Free Field Conditions (a_{max} = 0.25 g) in Pre-Earthquake Analyses

Soil-Structure Interaction Analyses.—Using the base motions computed at a depth of 150 ft (46 m) in the deconvolution studies, analyses were then made, using the program LUSH and a suitably fine but extensive mesh, to compute the response of the soil-structure system. Computations were made for a variety

of soil properties, and envelope spectra for motions at various levels within the structures were finally selected for design, based on the range of computed results supplemented by engineering judgment.

TABLE 4.—Comparison of Recorded and Computed Accelerations

Maximum Accelerations for Recorded Motions				Maximum accelerations for computed motions
Location (1)	Elevation (2)	Transverse (3)	Longitudinal (4)	(5)
Free-field (Storage building)	+12	0.35 g	0.26 g	0.40 g 0.25 g
Refueling building	+12	0.25 g	0.20 g	0.23 g 0.15 g
Reactor caisson	−66	0.16 g	0.12 g	0.22 g 0.13 g

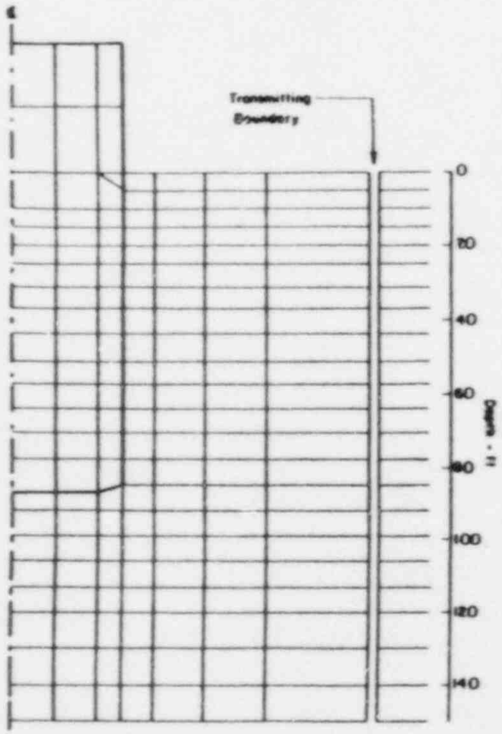

FIG. 8.—Finite Element Mesh Used for Analysis (1 ft = 0.305 m)

In the course of these studies, analyses were made for ground surface motions having peak accelerations of 0.4 g and 0.25 g. Since these are in the range of peak accelerations developed in the transverse and longitudinal directions during the June 7 earthquake, it is of interest to compare the values of computed and recorded peak accelerations at instrument locations in the structure. Such a comparison is shown in Table 4. It may be seen that the values show a remarkably high degree of agreement although there is some indication that the actual stiffness of the structure was somewhat less than that used in the

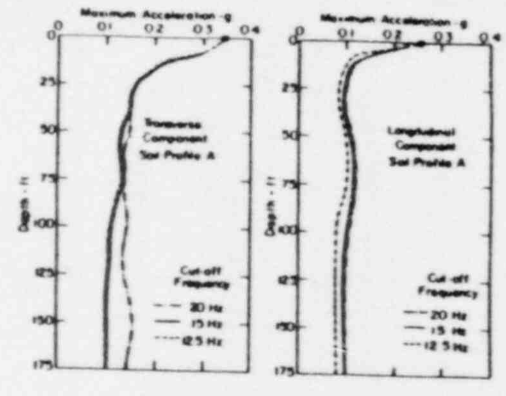

FIG. 9.—Acceleration Distributions Computed by Deconvolution of Recorded Surface Motions (1 ft = 0.305 m)

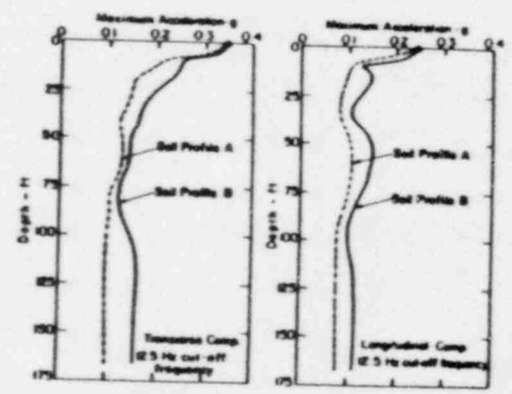

FIG. 10.—Effect of Soil Profile on Variation of Maximum Acceleration with Depth Computed by Deconvolution of Ground Surface Motion Records (1 ft = 0.305 m)

analysis. Nevertheless, the good agreement in these values is an encouraging aspect of the analytical procedure used in the studies.

Results of Post-Earthquake Analyses Using Recorded Motions

Post-earthquake studies of soil-structure interaction effects were performed following the same basic procedure as that described in the aforementioned, but using the computer programs SHAKE and FLUSH (4), since the latter provides a more versatile capability than LUSH and is also more economical. Advantage was taken of the results obtained in the earlier studies, and the

effects of the adjacent structures were therefore neglected in the analyses. Because the program FLUSH uses transmitting boundaries, it was only necessary to use the finite element mesh shown in Fig. 8 for the soil-structure interaction analyses.

Deconvolution Studies.—As stated previously, there are valid reasons why some filtering of a given ground surface motion is required in performing a deconvolution analysis to determine motions at various depths. To determine the significance of such effects for the recorded motions at the Humboldt Bay site, deconvolution analyses were made for Soil Profile A in Fig. 5 at the Storage Building site and for the recorded surface motions, using filtering or cut-off frequencies of 20 Hz, 15 Hz, and 12.5 Hz. The results of these studies, in terms of the computed variation of maximum acceleration with depth in the soil profile, are shown in Fig. 9. It may be seen that the cut-off frequency,

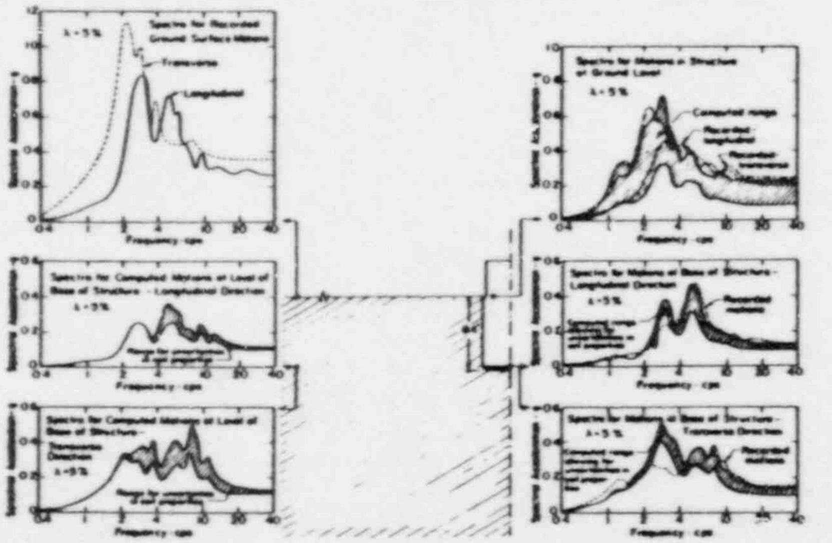

FIG. 11.—Comparison of Recorded and Computed Spectra in Refueling Building—Humboldt Bay Power Plant

within the range investigated, had little influence on the results of the analysis, all of the studies for both the longitudinal and transverse recorded motions showing a marked decrease in magnitude of the peak acceleration from the ground surface to a depth of approx 30 ft (9 m) and below. In fact, the peak accelerations computed to develop in the free-field at the level of the base of the Refueling Building [approx 85 ft (26 m)] is in the range of 0.10 g–0.14 g, or less than 60% of the maximum acceleration at the ground surface.

It may also be seen, from Fig. 10, that generally similar results are obtained whether Soil Profile A or B is used for the analysis. Although they are not shown, results for Soil Profile C fell within the range shown for Soil Profiles A and B. Thus, it would seem reasonable to conclude from these results that:

1. The recorded ground surface motions have no significant content of very high frequencies as might be expected for a deep soil condition such as that at the Humboldt Bay Plant site.

2. The results of soil-structure interaction analyses made with a cut-off frequency of 12.5 Hz will be comparable to those made using higher cut-off frequencies. Since there is a marked reduction in computer costs associated with the use of a lower cut-off frequency, the soil-structure interaction studies described in the following section were made for these conditions.

Soil-Structure Interaction Studies.—Having determined the base motions required in the soil profile at a depth of 150 ft (46 m) to produce the recorded motions at the ground surface under free-field conditions, the same motions

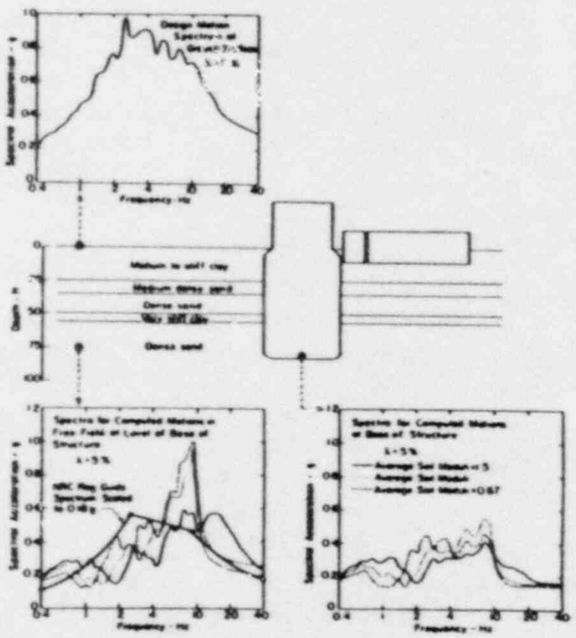

FIG. 12.—Summary of Spectra for Computed Motions in Free-Field and at Base of Structure—Maximum Ground Surface Acceleration = 0.3 g (1 ft = 0.305 m)

were used as excitation at the base of the soil-structure model shown in Fig. 8 to compute the motions developed: (1) At the base of the structure; and (2) in the structure at the level of the ground surface, where motions were recorded during the earthquake of June 7. Separate analyses were made for the longitudinal and transverse records of free-field motion and for the various soil profiles. The ranges of analytical results are shown in Fig. 11 in the form of response spectra, and are also compared with the spectra for the recorded motions.

It may be seen that for both longitudinal and transverse motions, the recorded motions at the base of the structure are in reasonably good agreement with those computed using the finite element procedure for implementation of an

"idealized" complete interaction analysis. For both components of motion, the analysis procedure indicates a higher peak in the response spectrum at a frequency of approx 3 Hz than actually developed, but considered overall, the agreement between computed and recorded base motion spectra is both gratifying and encouraging.

Similarly, the recorded motions in the structure at ground level fall essentially within the range computed by the interaction analysis procedure, providing further confirmation of the ability of a complete interaction analysis to compute the structural response with an adequate degree of accuracy in this case.

It is recognized, of course, that one such test of the applicability of any analytical procedure does not necessarily provide proof that it will always lead

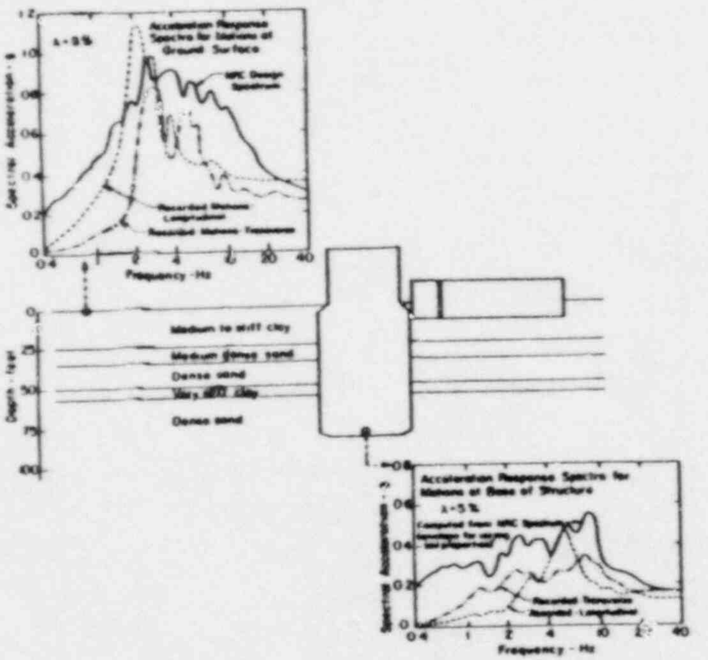

FIG. 13.—Comparison of Spectra for Design and Recorded Motions (1 ft = 0.305 m)

to good evaluations of field performance. Nevertheless, in the current absence of any other opportunity to check analytical methods for computing response under strong shaking of prototype structures, the results obtained in even this single case can give designers increased confidence in the usefulness of the analytical tools at their disposal.

Applicability of NRC Design Procedure

In addition to their function as a check of the adequacy of procedures for analyzing soil-structure interaction, the records obtained at the Humboldt Bay Power Plant are also useful in investigating the adequacy of required design

practice. At the present time, regulatory requirements for determining soil-structure interaction effects for embedded structures such as the Refueling Building require the specification of a design or control motion at the ground surface having a designated maximum acceleration and a time history whose spectrum closely matches a standard design spectrum shape specified by the NRC. Since the average peak acceleration recorded in the free-field at the Humboldt Bay plant was 0.3 g, it would seem reasonable to compare the motions recorded at the base of the Refueling Building with those computed following an approved design procedure consistent with a peak free-field ground surface acceleration of 0.3 g and the standard design spectrum shape. This is, in fact, the motion whose spectral shape is shown in the upper left corner of Fig. 12. An acceleration

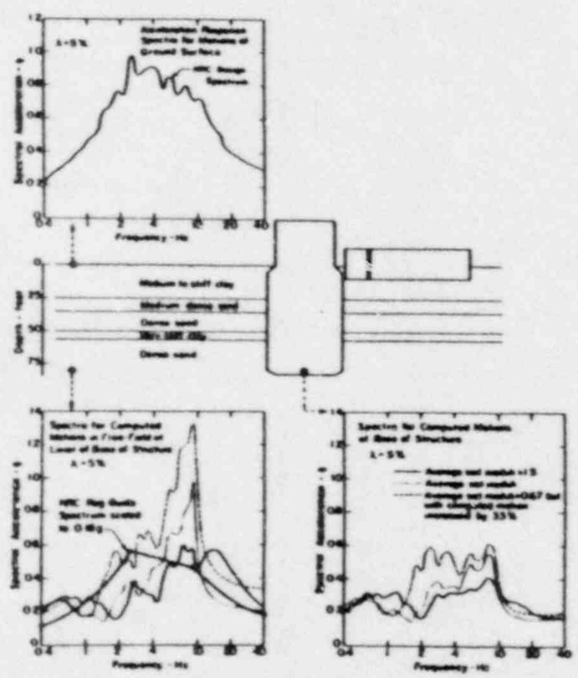

FIG. 14.—Summary of Spectra for Computed Motions in Free-Field and at Base of Structure Using NRC Design Procedures (1 ft = 0.305 m)

time history having this spectrum and having a duration of approx 16 sec was used in the following analyses.

Regulatory practice permits the deconvolution of this motion and the analysis of soil-structure interaction effects using finite element methods as previously described but it also requires that: (1) Analyses be made for the most likely values of soil moduli and for values of soil moduli which are increased and reduced by a factor of 1.5 to allow for possible uncertainties in soil property determinations; (2) the envelope of the resulting spectra for motions computed for a point in the free-field at the level of the base of the structure be not less than 60% of the spectral accelerations for the ground surface control motion;

and (3) the structural response be evaluated for motions having a spectral shape enveloping those computed at the base of the structure for free-field motions meeting the requirements of (1) and (2).

A typical set of calculations for the same ground surface control motion, but for the three different values of soil moduli, are shown in Fig. 12. In this figure, the control motion is shown in the upper left-hand corner, the spectra for the computed motions in the free-field at the level of the base of the structure are shown in the lower left-hand corner and the spectra for the computed motions at the base of the structure are shown on the lower right-hand corner. For the analysis conducted with the most likely values of soil moduli and the reduced

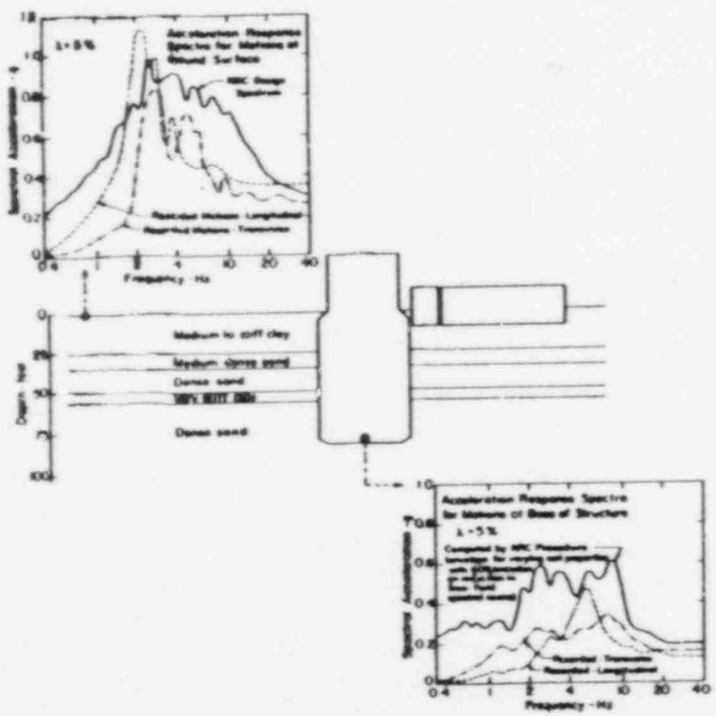

FIG. 15.—Comparison of Spectra for Design and Recorded Motions at Base of Structure (1 ft = 0.305 m)

soil moduli, the control motion was filtered at 10 Hz, while for the analysis with increased soil moduli, the control motion was filtered at 20 Hz. The envelope of the computed spectra for the motions at the base of the structure is compared with the motions recorded at the base of the structure in Fig. 13.

It may be seen that although the free-field motions fail to meet the NRC design spectral acceleration requirements in the frequency range from approx 2 Hz-5 Hz, the envelope spectrum for the computed motions at the base of the structure is nevertheless higher than the spectra for the recorded base motions at all frequencies. In fact, only at frequencies of about 4.5 Hz-5.5 Hz does

the spectrum for the recorded motions come close to that for the computed base motion envelope spectrum.

One means of increasing the free-field spectra to meet the 60% of surface control motion requirement is to increase the ground surface acceleration for the control motion for one or more of the analyses, so that after deconvolution it meets the free-field requirements. In the present case, this could be achieved by increasing the control motion for the analysis performed using values of soil moduli reduced by 30%. With a satisfactory degree of accuracy, this leads to corresponding increases of 30% in both the free-field spectrum at a depth of 85 ft (26 m) and the spectrum for motions at the base of the structure.

The superimposed spectra for the three analyses with this modification are shown in Fig. 14, and the envelope of the spectra for computed motions at the base of the structure is compared with the spectra for the motions recorded at the base of the structure in Fig. 15. It may be seen from Fig. 14 that the envelope of free-field spectra now comes very close to meeting the design spectral requirements at this location; thus the envelope of spectra for motions developed at the base of the structure as shown in Fig. 15 would be essentially acceptable for design purposes. This envelope provides a comfortable margin of safety above the spectra for the recorded base motions and would seem to indicate that, at least for these strong motion records, the current design requirements provide an adequate but not excessively conservative margin of safety for analyses conducted in the manner described in the aforementioned.

Similar studies for other methods of evaluating soil-structure interaction effects would presumably throw some light on the degree of conservatism they introduce into the design procedure.

Conclusions

The preceding pages present the results of a study of the distribution of ground motions and structural response in the Humboldt Bay Nuclear Power Plant during the Ferndale earthquake of June 7, 1975. Based on a knowledge of the motions recorded at the ground surface in the free-field, computations are made using an idealized complete interaction procedure based on finite element analysis, to determine the characteristics of the motions likely to develop at the base of the buried reactor caisson at a depth of 85 ft (26 m) below the ground surface and within the Refueling Building at the ground surface level. The computed motions are shown to be in reasonably good agreement with those recorded at these locations during the same earthquake. In addition, the recorded motions are compared with those computed by an analysis procedure which generally meets existing regulatory requirements, and it is shown that the regulatory requirements lead to an adequate but not excessively conservative margin of safety based on the motions recorded in this event.

It is of interest to note that Lambe has recently made a study of the accuracy of engineering predictions of soil behavior under static-loading conditions (3). For this purpose he classified predictions into five groups as follows:

1. Type A: Prediction made before the event.
2. Type B: Prediction made during the event but before the results are known.
3. Type B1: Prediction made during the event but with results known at the time.

4. Type C: Prediction made after the event but before the results are known.
5. Type C1: Prediction made after the event but with results known at the time.

He concluded that "Type C predictions are autopsies. Our professional literature contains the results of more Type C1 predictions than any other type. Autopsies can of course be very halpful in contributing to our knowledge. However, one must be suspicious when an author uses a Type C1 prediction to prove that any prediction technique is correct". Lambe also concluded that predicted results within a factor of two of observed field performance constitute very good predictions. It would seem optimistic to expect any better success in predicting dynamic behavior of soil or soil-structure systems.

However, the prediction of the base motion peak accelerations shown in Table 4, based on the assumption that the ground surface motions with peak accelerations of 0.25 g and 0.40 g in the free field was clearly a Class A prediction using Lambe's terminology in that the study describing these results was carried out before the event of June 7, 1975 had occurred; nevertheless, the degree of similarity between peak acceleration values assumed and developed in the free field and those predicted and developed at the base of the Reactor Caisson would seem to show that the prediction was highly satisfactory.

Similarly, although the more detailed analyses described in the preceding pages using the same general procedure were made after the event, it might reasonably be claimed that they represent a Class A prediction since they permitted virtually no latitude for manipulation of the results, in that they were based on:

1. A method of analysis developed prior to the event.
2. Soil properties established and filed with the Nuclear Regulatory Commission prior to the event.
3. Fixed surface motions established by the event.

Nevertheless, the writers would be the first to agree that the good agreement in this one case between predicted and developed motions at the base of the structure does not necessarily prove the adequacy, for all cases, of the method of analysis used. Clearly, compensating errors might be involved whose effects have not been fully appreciated. On the other hand, it is an encouraging start and the results obtained obviously give some degree of justification for the method used. They might also, in due course, given an equal degree of justification for other methods which might be used for analyzing soil-structure interaction effects. These are significant facts in a field in which no other data exists by which the adequacy of analytical procedures can be checked. At the same time, it is clear that any method of analysis which provides a poor prediction of the results obtained, based on the known values of soil and structural properties and the motions recorded at the ground surface, must be considered of dubious validity for future predictions of probable building response.

Acknowledgments

The study described in the preceding pages was conducted as part of an investigation on "Analysis of Soil-Structure Interaction Effects for Massive Embedded Structures During Earthquakes" sponsored by the National Science Foundation. The support of the foundation and of the Pacific Gas and Electric Company in providing the basic data required for the study and in encouraging the investigation is gratefully acknowledged.

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