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SOIL-STRUCTURE INTERACTION EFFECTS AT THE HUMBOLDT BAY POWER PLANT IN THE FERNDALE EARTHQUAKE OF JUNE 7, 1975

by
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J. LYSMER

A report on research sponsored by
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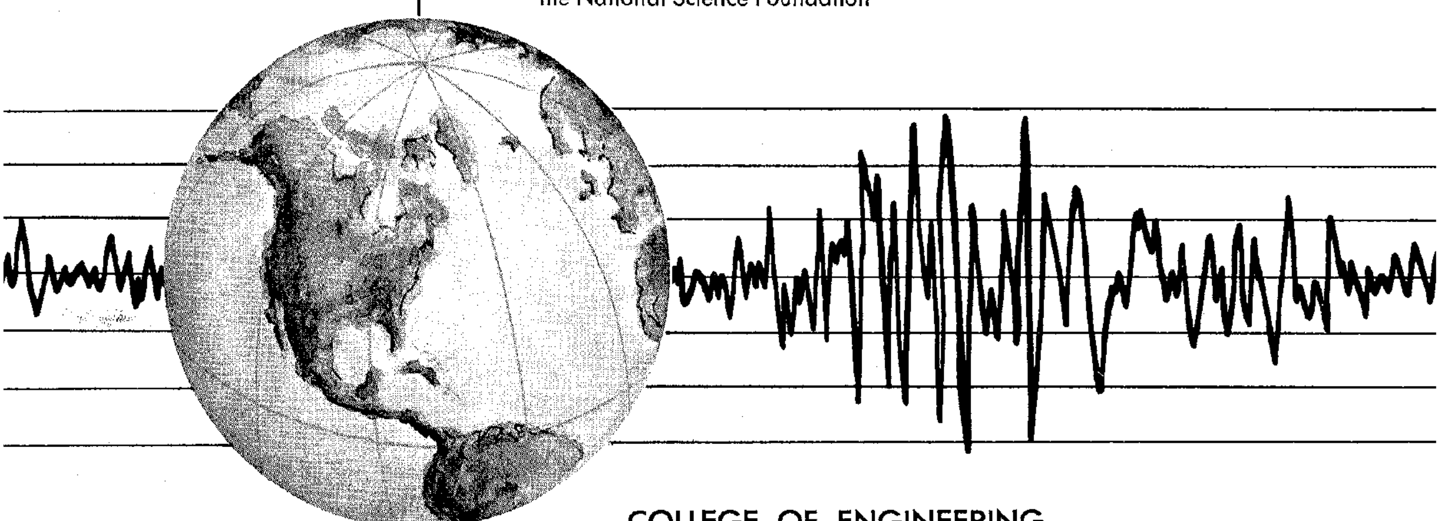

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16. Abstracts

This report presents the results of a study of the distribution of ground motions and structural response in the Humboldt Bay Nuclear Power Station during the Ferndale earthquake of June 7, 1975. Based on a knowledge of the motions developed 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 Refueling Building at a depth of 85 ft below the ground surface and within the Refueling Building at a depth of 85 ft 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 in the same earthquake. In addition, the recorded motions are compared with those computed by an analysis procedure which generally meets existing requirements of the Nuclear Regulatory Commission and it is shown that the regulatory requirements lead to an entirely adequate but not excessively conservative margin of safety based on the motions recorded in this event.

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Soil-Structure Interaction Effects at the Humboldt Bay Power Plant in the Ferndale Earthquake of June 7, 1975

by

Julio E. Valera,[1] H. Bolton Seed,[2] C. F. Tsai[3] and J. Lysmer[4]

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 which attempt to make some evaluation of the variations in earthquake motions both in the structure and 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 (Seed et al, 1975b). 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 visco-elastic layered systems. It appears to be the

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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" (ASCE Ad-hoc Committee on Soil Structure Interaction for Design of Nuclear Power Plants, 1976). 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 often provide results with sufficient accuracy for practical purposes. However as increasingly efficient and versatile computer programs are developed for finite element analyses and progressively more sophisticated forms of half space analysis are developed, which introduce the essential concepts of a complete interaction approach, it seems that both methods of analysis may ultimately develop to the point where they give similar results for embedded structures.

A major contributing factor to the continuing debate concerning the merits of any form of analytical approach has been the total absence of recorded field performance by which the adequacy of such an approach might be judged--making it necessary for engineers to adopt one approach or the other on the basis of their personal appraisals of such factors as the degree of sophistication of the analysis, the potential savings in design costs, the potential losses in overall project costs, their degree of understanding of the nature of the phenomena and principles involved, etc. In the absence of known field performance, all reasonable suggestions for design approaches must be considered potentially applicable and considerable insight, wisdom and intellectual honesty is required to select a design method which offers the greatest potential for combining adequate safety for critical structures with reasonable overall economy in the cost of the completed facility. It is for these reasons that the motions recorded at the Humboldt Bay Power Plant in the Ferndale earthquake of June 7, 1975 are of major significance.

A general view of the plant is shown in Fig. 1. Units 1 and 2 are fossil fuel units whereas Unit 3 is nuclear. The buried reactor structure within the Refueling Building of Unit 3 consists of a massive concrete caisson embedded at a depth of about 85 feet below the ground surface. The various surrounding structures are light-weight structures and are founded at or close to the ground surface. The facility was constructed in 1963 and has been operating satisfactorily since that time.

Strong motion instruments at the plant have been in operation since September 1971. These are located at elevation +12 (plant grade level) and elevation -66 in the Refueling Building, and in a Storage Building (elevation +12) some 330 feet south of the Refueling Building.

The June 7, 1975 earthquake (magnitude about 5.5) had its epicenter some 15 miles south of the plant site and triggered strong motion instruments in the surrounding area including those located at the Humboldt Bay Plant (Valera and Brady, 1976). The earthquake records obtained at the Humboldt Plant are shown in Fig. 2. Although the duration of strong shaking was only about 3 to 5 records, the baseline-corrected peak accelerations developed in the free field (Storage Building) were 0.35g and 0.26g in the transverse and

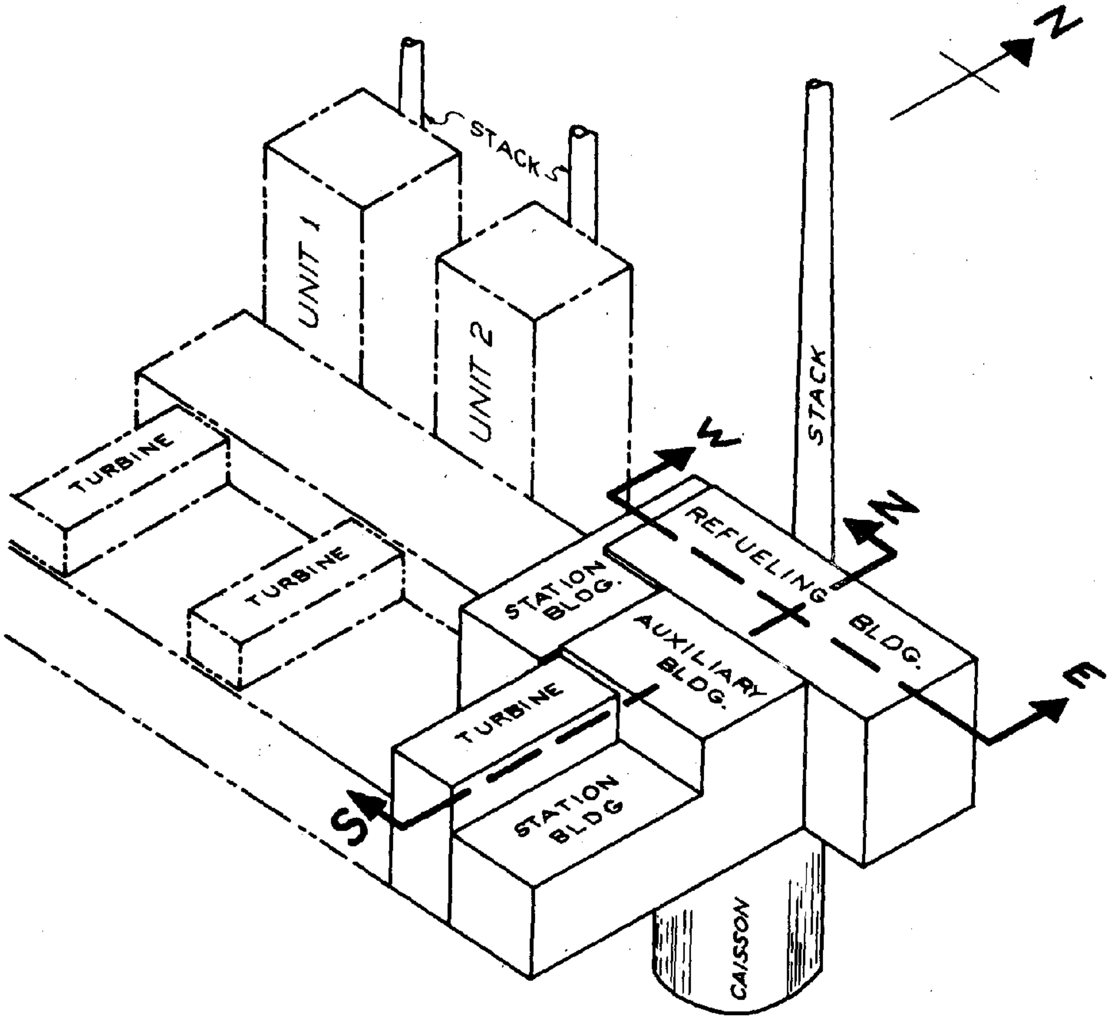


Fig. 1 GENERAL VIEW OF HUMBOLDT POWER PLANT
(After Bechtel Corp.)

longitudinal directions, respectively, making these the strongest earthquake motions to which a nuclear power plant has so far been subjected. However thre 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 about 12 months before the earthquake occurred. In fact, extensive soil structure interaction analyses using finite element procedures with accompanying determinations of soil characteristics at the site, had been completed several months prior to the earthquake of June 7, 1975. These studies were carried out by Dames & Moore using analytical techniques developed at the University of California at Berkeley (Seed et al, 1975a). In this respect it is interesting to note that these analyses had predicted a peak acceleration at the base of the Refueling Building of 0.13g for a free-field ground surface acceleration of 0.25g while the subsequent earthquake produced an average peak acceleration at the base of the Refueling Building of 0.14g for an average free-field ground surface acceleration of 0.30g. This result alone, predicted in advance of the event and published in design reports, is of considerable interest.

While these facts are of major importance, perhaps the most significant feature of the June 7 event is the opportunity it provides 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.

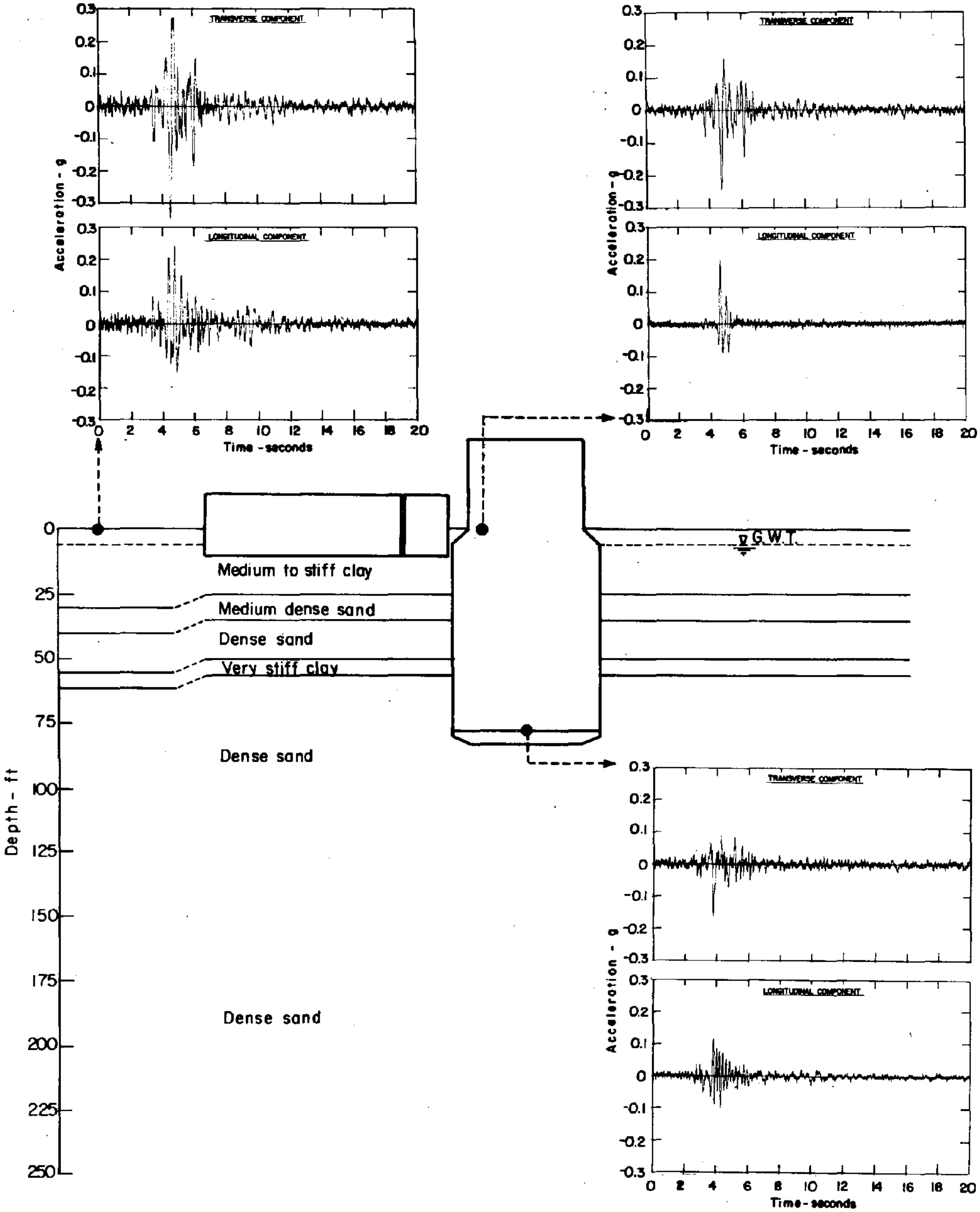
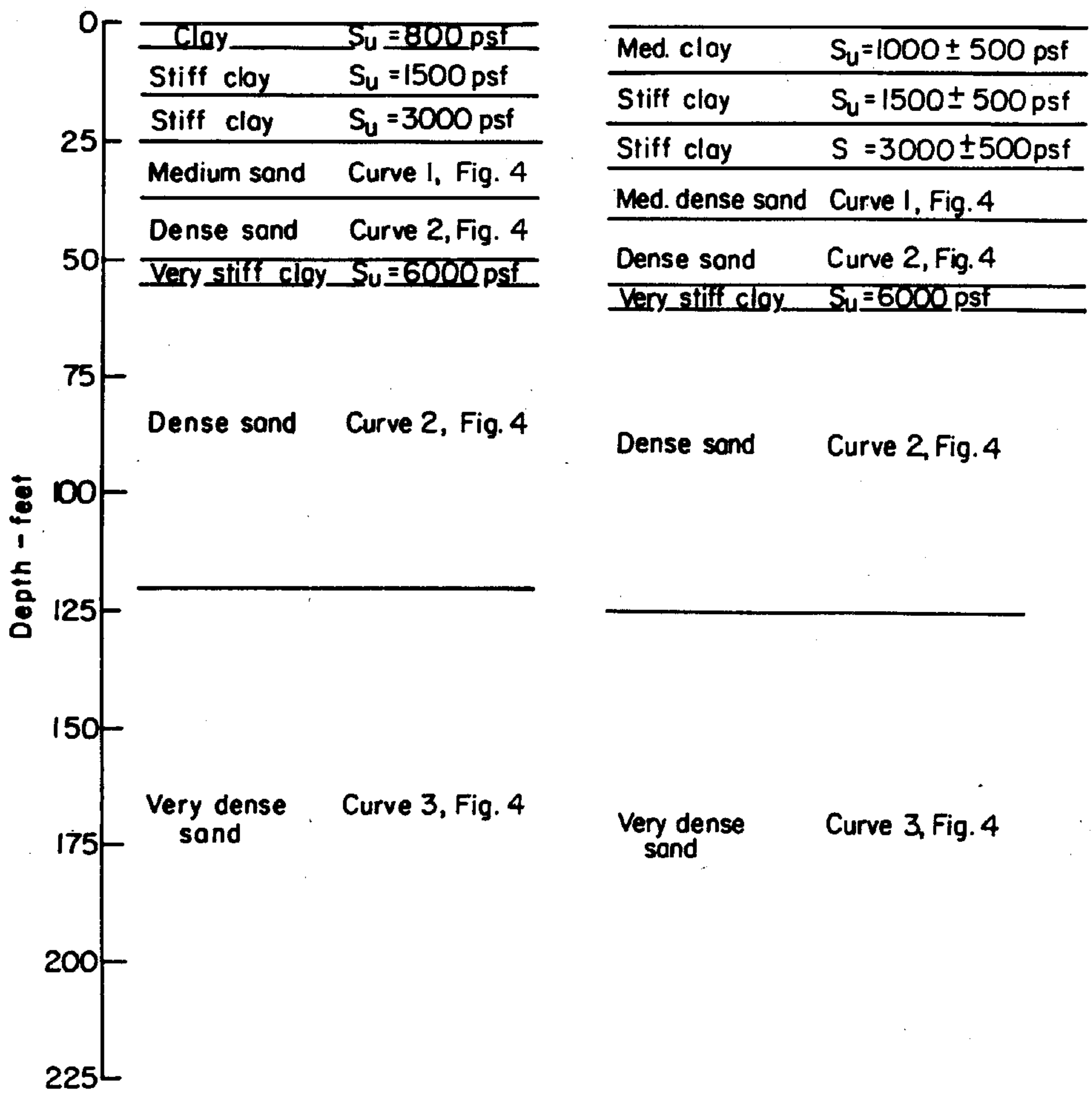


Fig. 2 GROUND MOTION RECORDS AT HUMBOLDT BAY POWER PLANT

Site Conditions and Soil Properties

A general description of the subsurface soil conditions at the plant site has been presented by Valera and Brady (1976). A cross-section through Unit 3 in the N-S direction is shown in Fig. 2. Basically the soils around the Refueling Building consist of about 25 ft of medium to stiff clay (increasing to about 30 ft at the Storage Building), underlain successively by about 30 ft of medium dense to dense sand, 10 ft of very stiff clay and then a deep bed of dense sand containing some clay lenses extending to a depth of about 400 ft. All of the soils surrounding the Refueling Building are overconsolidated with an average overconsolidation ratio of at least 6 to 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 presented in Figs. 3(a) and 4, respectively. The soil profiles and soil properties used in the present study are presented 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.

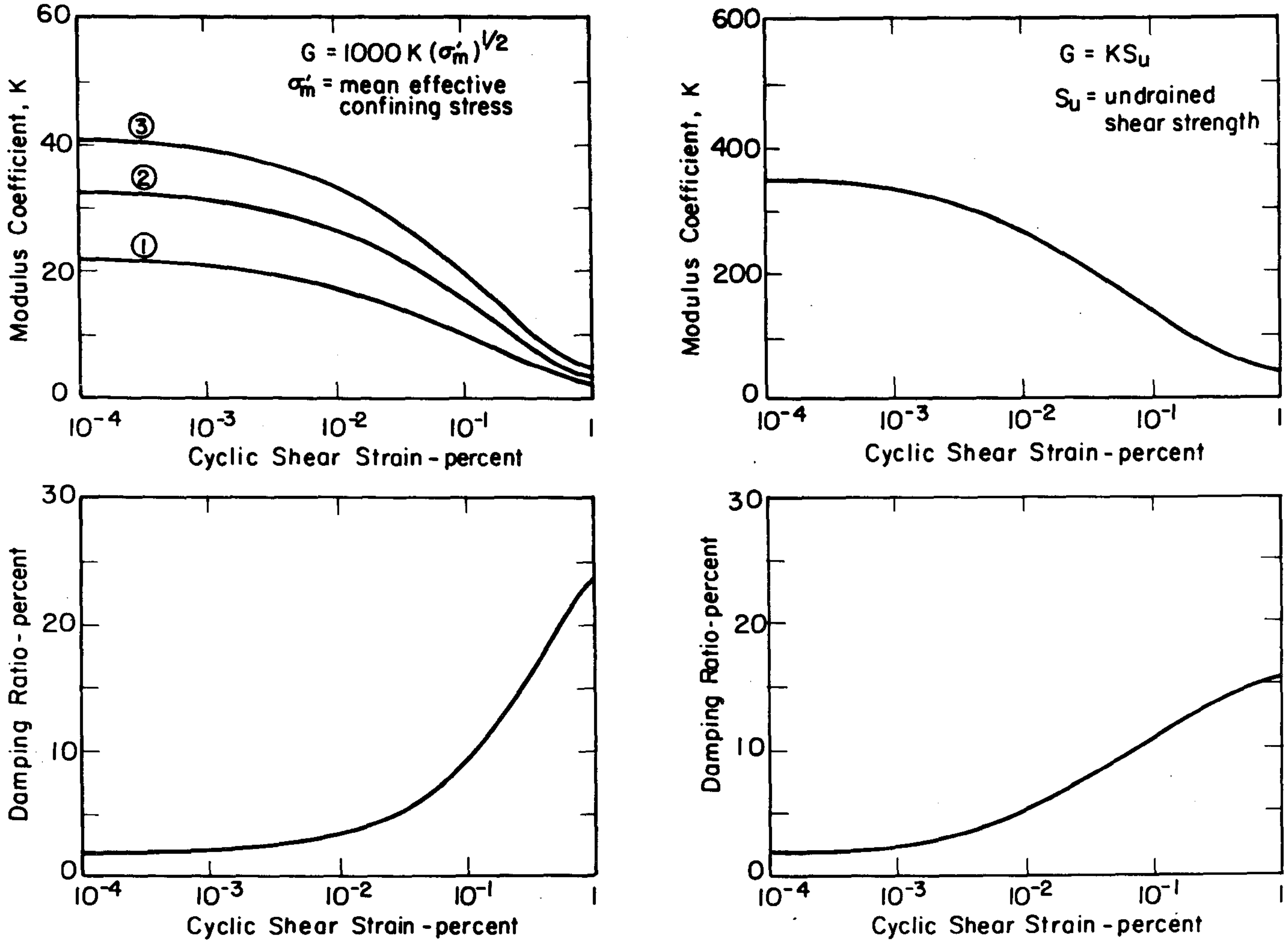
At the site of the Storage Building itself where the free-field records were obtained, there is some uncertainty about the actual strength of the top 30 ft of clay as the closest boring is at least 100 feet away and there is considerable scatter in the measured values of shear strength for undisturbed 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, as illustrated by soil profiles A, B and C in Fig. 5. Results for all profiles investigated fell within the range represented by profiles A and B.



(a) Soil Profile Adjacent to Refueling Building

(b) Soil Profile Adjacent to Storage Building

Fig. 3 SOIL PROFILES AT HUMBOLDT BAY POWER PLANT SITE



COHESIONLESS SOILS

COHESIVE SOIL

Fig. 4 AVERAGE DYNAMIC SOIL PROPERTIES

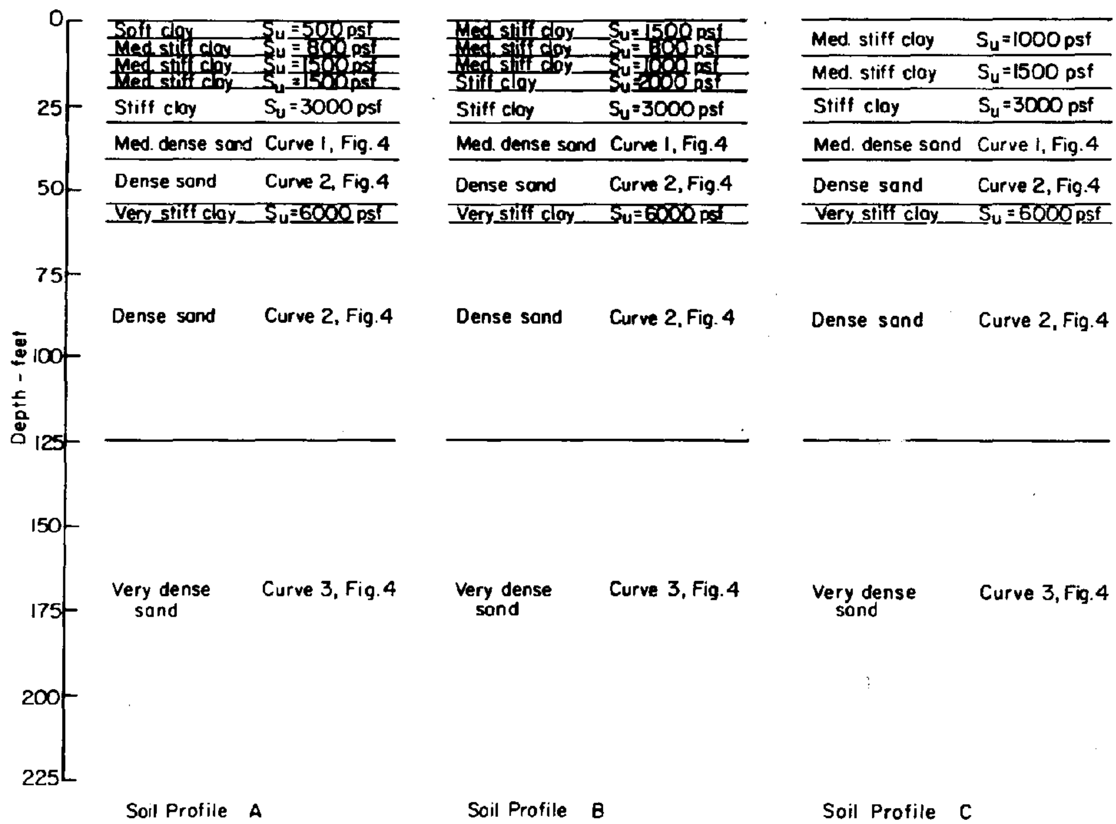


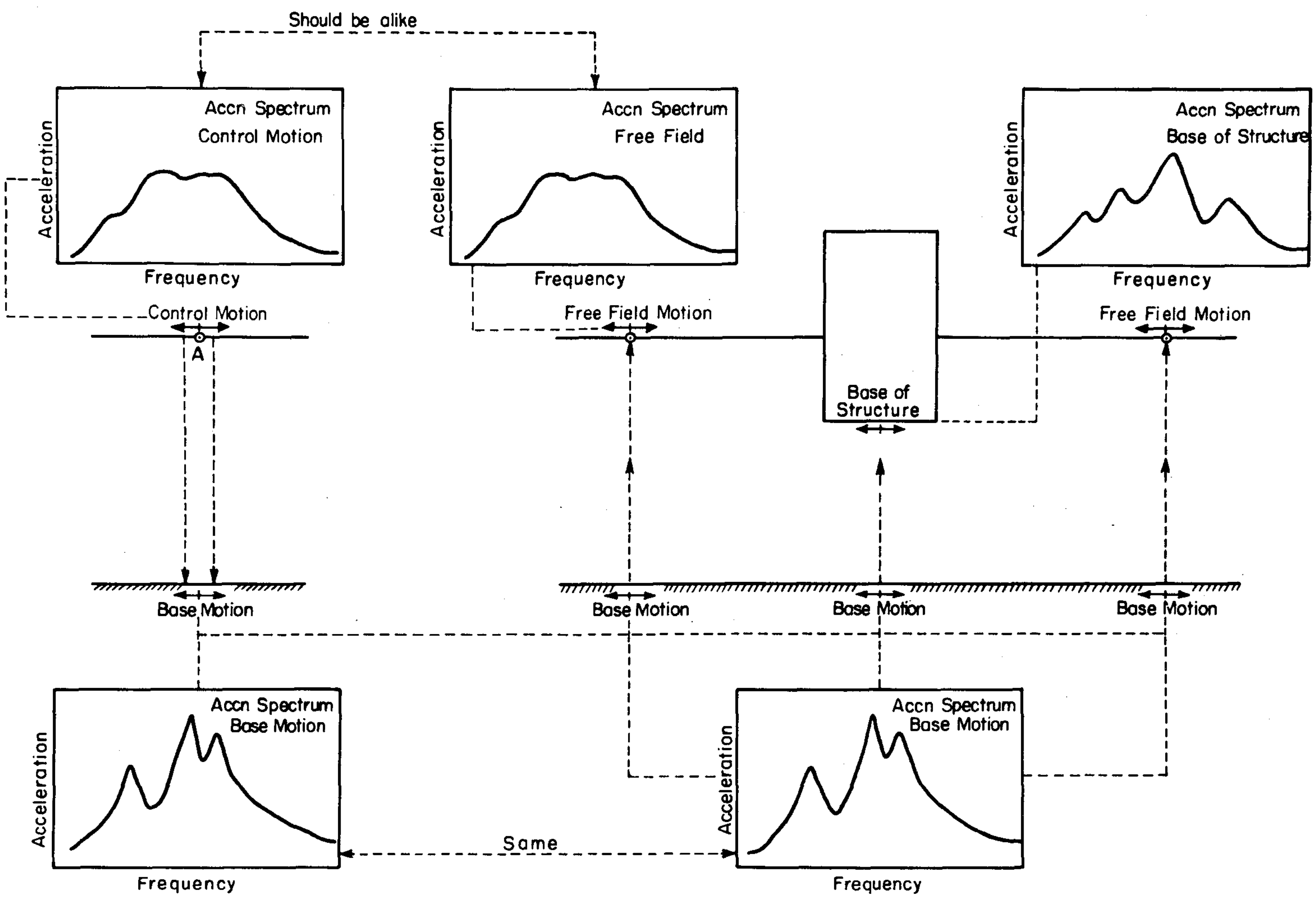
Fig. 5 TYPICAL SOIL PROFILES AT STORAGE BUILDING USED FOR DECONVOLUTION STUDIES

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 results were determined and filed with the Nuclear Regulatory Commision 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 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 (Seed et al, 1974) is illustrated schematically in Fig. 6. The known ground surface motions developed in the free-field are first 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 (say 150 to 200 ft) 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 (Schnabel et al, 1972).

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 free-field motions are indeed developed in the free field. The basic



(a) Soil Deposit Model

(b) Finite Element Model of Soil-Structure System

Fig. 6 SCHEMATIC REPRESENTATION OF SOIL-STRUCTURE INTERACTION ANALYSIS USING FINITE ELEMENT MODEL requirements of a suitable analysis and computer program (Seed et al, 1975b) 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 where this is appropriate,

(4) The variation of soil characteristics with depth,

and (5) The non-linear 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 (Lysmer et al, 1974). Analyses were carried out for cross-sections in the N-S and E-W 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 0.25g and 0.4g, respectively.

Table 1. Structural Properties of Reactor Caisson

Depth Below Ground Surface, ft	Shear Modulus $\times 10^6$ psf	Density pcf	Poisson's Ratio
0-15	289	0	0.2
15-31	86	0	"
31-44	80	0	"
44-71	76	0	"
71-78	83	0	"
78-87	4160	158	"

Table 2. Masses Lumped at Center Line of Reactor Caisson

Depth Below Ground Surface, ft - - -	0	15	25	37	51	57	71
Weight of Mass (kips) - -	82	82	76	44	43	47	54

Table 3. Structural Properties of Refueling Building

Depth Above Ground Surface, ft	Shear Modulus x 10^6 psf	Density pcf	Poisson's Ratio
0-17.5	5	25	0.2
17.5-35	5	10	0.2

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 (Hwang, 1973) 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 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 (over 250 ft) bodies of soil or including layers of soft to medium stiff clay and sand (Seed et al, 1974).

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

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 NRC design spectra stipulated in Regulatory Guide 1.60. In these studies it was necessary to use a cutoff frequency of 15 to 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 presented 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.

Soil-Structure Interaction Analyses

Using the base motions computed at a depth of 150 ft 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

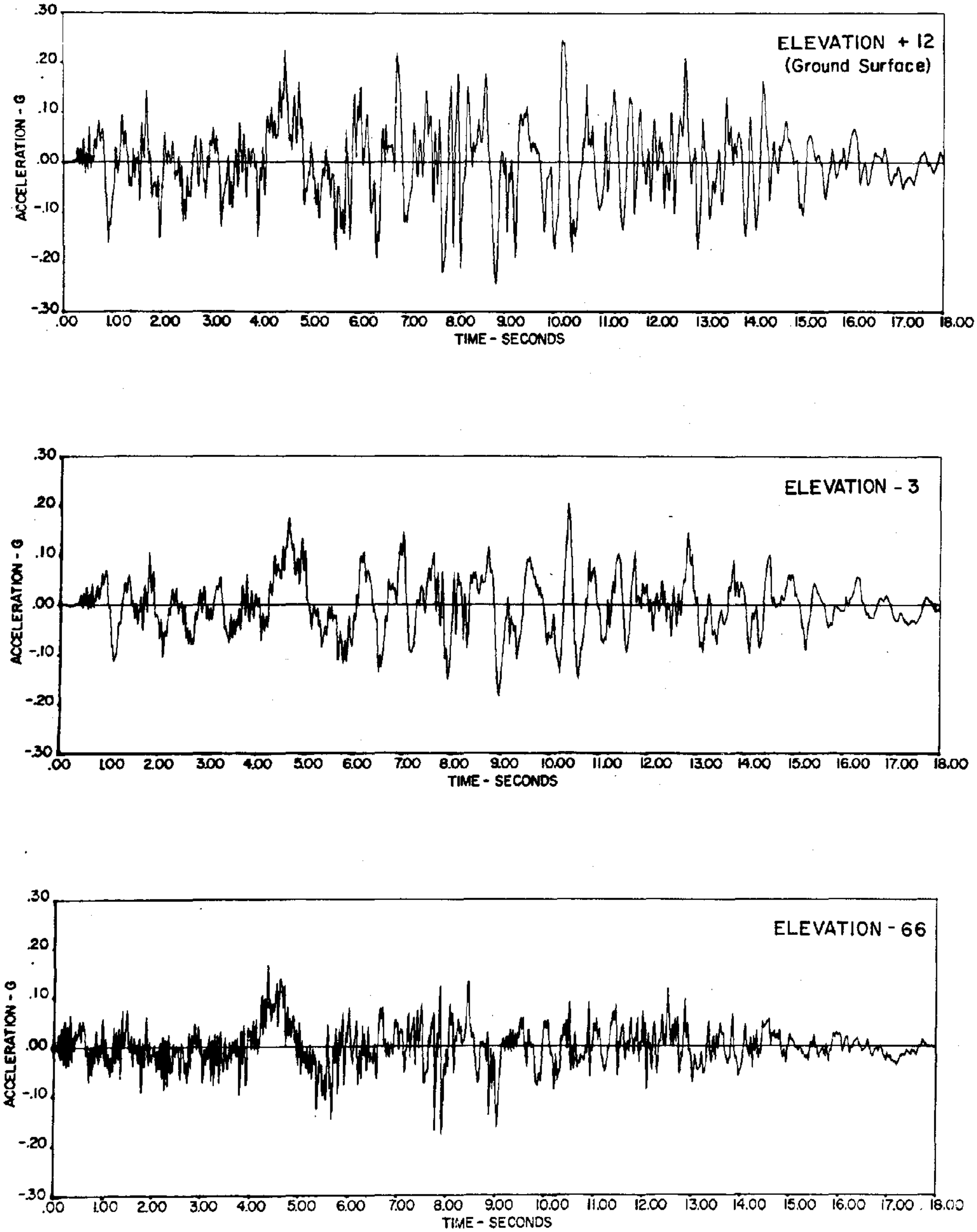


Fig. 7 HORIZONTAL ACCELERATION HISTORIES COMPUTED BY DECONVOLUTION OF SURFACE MOTIONS FOR FREE FIELD CONDITIONS (a_{max} = 0.25 g) IN PRE-EARTHQUAKE ANALYSES

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.

In the course of these studies, analyses were made for ground surface motions having peak accelerations of 0.4g and 0.25g. 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 analysis. Nevertheless the good agreement in these values is an encouraging aspect of the analytical procedure used in the studies.

Table 4. Comparison of Recorded and Computed Accelerations

Location	Elevation	Max. Accelerations for Recorded Motions: Transverse	Long.	Max. Accelerations for Computed Motions	
Free-field (Storage Building)	+12	0.35g	0.26g	0.40g	0.25g
Refueling Building	+12	0.25g	0.20g	0.23g	0.15g
Reactor Caisson	-66	0.16g	0.12g	0.22g	0.13g

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 above but using the computer programs SHAKE and FLUSH (Lysmer et al, 1975) 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 the recorded surface motions, using filtering or cut-off frequencies of 20, 15 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, 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 about 30 ft and below. In fact the peak accelerations computed to develop in the free-field at the level of the base of the Refueling Building (about 85 ft) is in the range of 0.10g to 0.14g or less than 60 percent of the maximum acceleration at the ground surface.

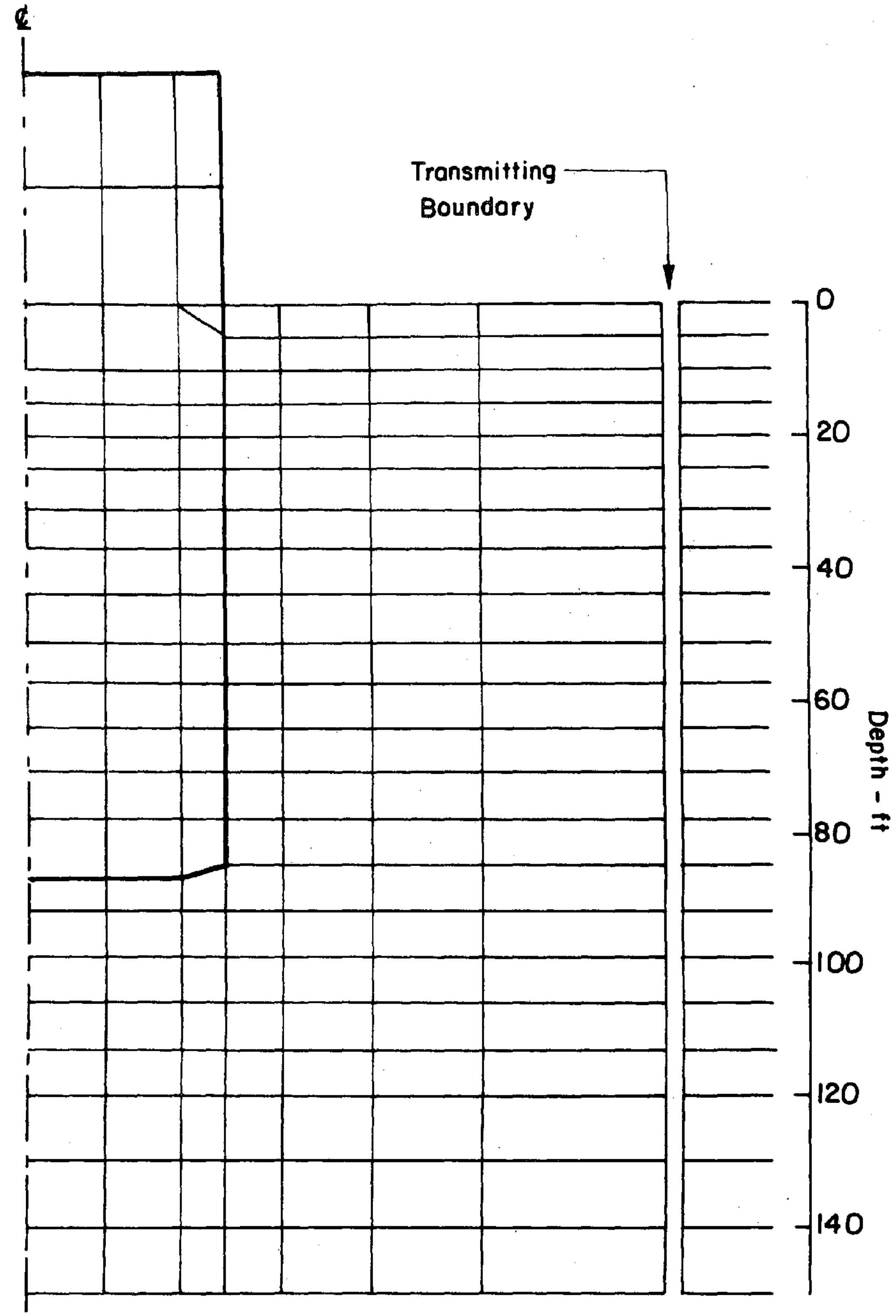


Fig. 8 FINITE ELEMENT MESH USED FOR ANALYSIS

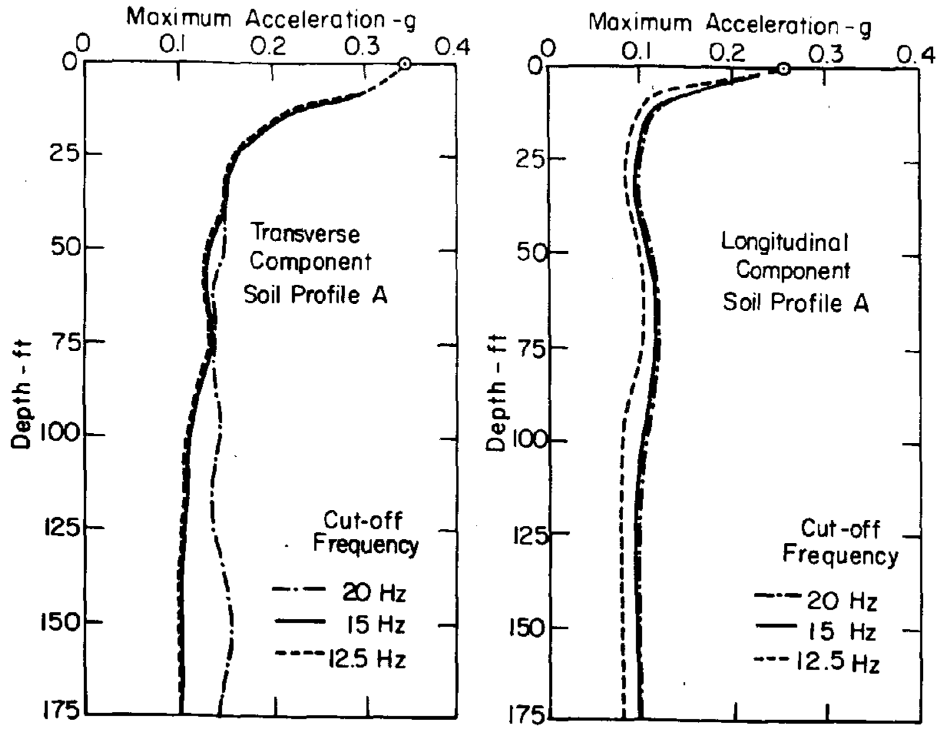


Fig. 9 ACCELERATION DISTRIBUTIONS COMPUTED BY DECONVOLUTION OF RECORDED SURFACE MOTIONS – HUMBOLDT BAY POWER PLANT, JUNE 7, 1975

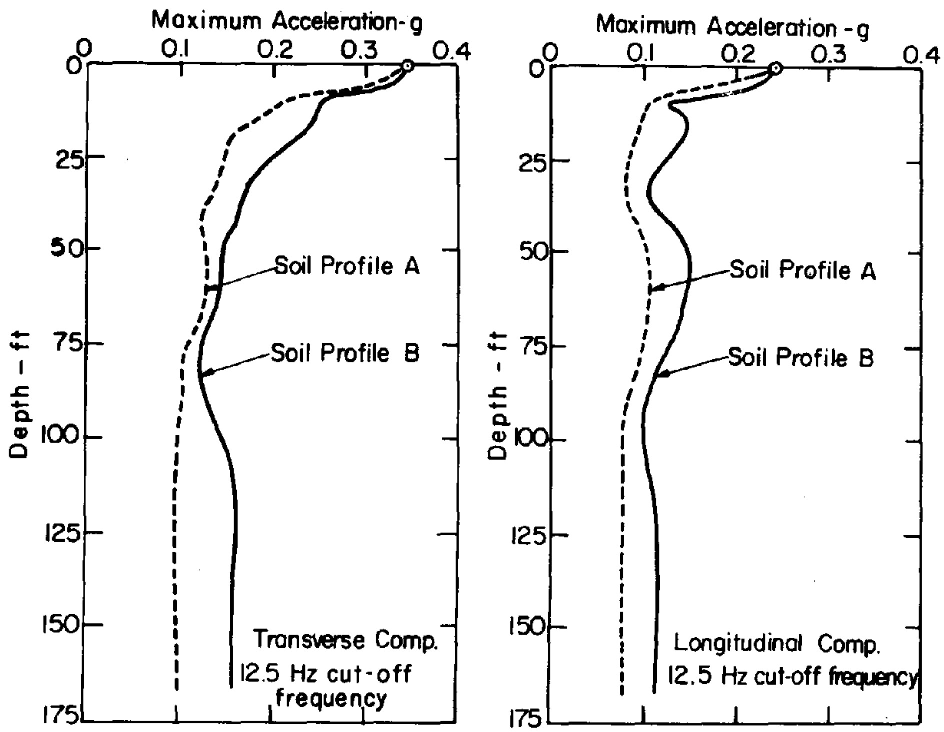


Fig. 10 EFFECT OF SOIL PROFILE ON VARIATION OF MAXIMUM ACCELERATION WITH DEPTH COMPUTED BY DECONVOLUTION OF GROUND SURFACE MOTION RECORDS

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 to produce the recorded motions at the ground surface under free field conditions, the same motions 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 presented in Fig. 11 in the form of response spectra, where they 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 about 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 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 use for checking the adequacy of procedures

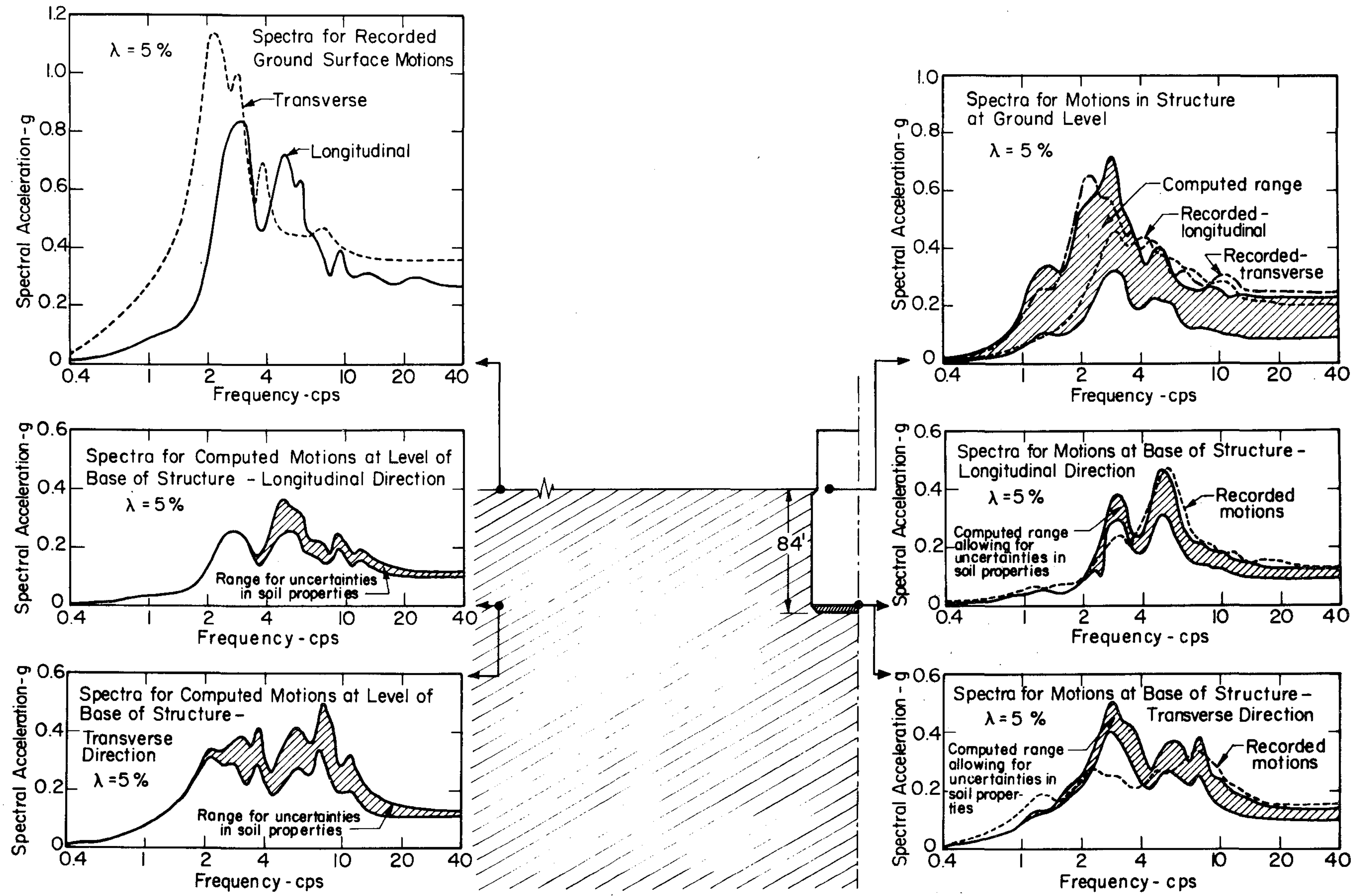


Fig. 11 COMPARISON OF RECORDED AND COMPUTED SPECTRA IN REFUELING BUILDING – HUMBOLDT BAY POWER PLANT

for analyzing soil-structure interaction, the records obtained at the Humboldt Bay Power Plant can also be used to investigate 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 Nuclear Regulatory Commission. Since the average peak acceleration recorded in the free-field at the Humboldt Bay plant was 0.3g, 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 0.3g 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 time history having this spectrum and having a duration of about 16 seconds 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:

1. that 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. that 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 should be not less than 60 percent

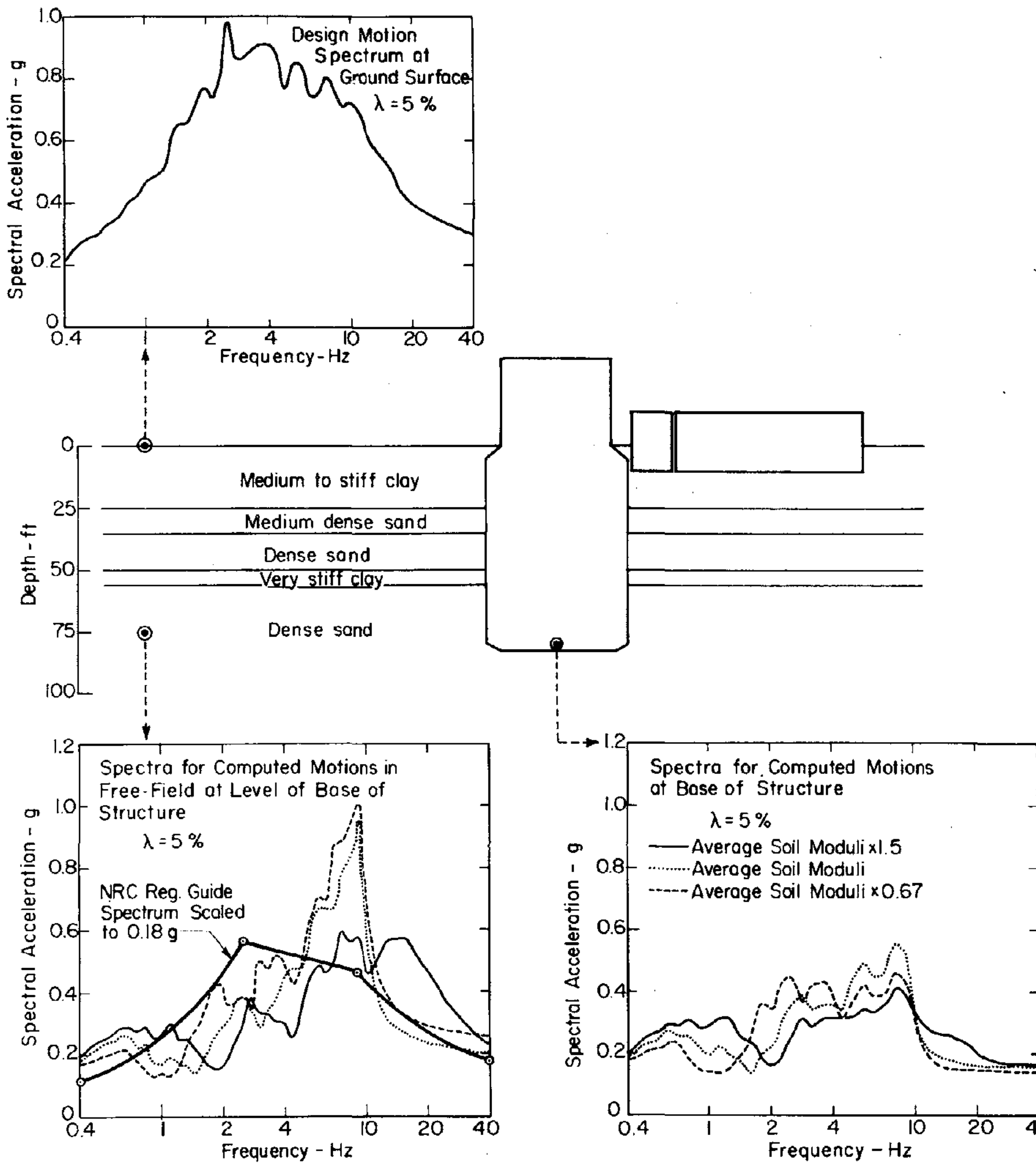


Fig. 12 SUMMARY OF SPECTRA FOR COMPUTED MOTIONS IN FREE-FIELD AND AT BASE OF STRUCTURE – MAXIMUM GROUND SURFACE ACCELERATION = 0.3g

of the spectral accelerations for the ground surface control motion;

and 3. that 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) above.

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 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 about 2 to 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 to 5.5 Hz does the spectrum for the recorded motions come close to that for the computed base motion envelope spectrum.

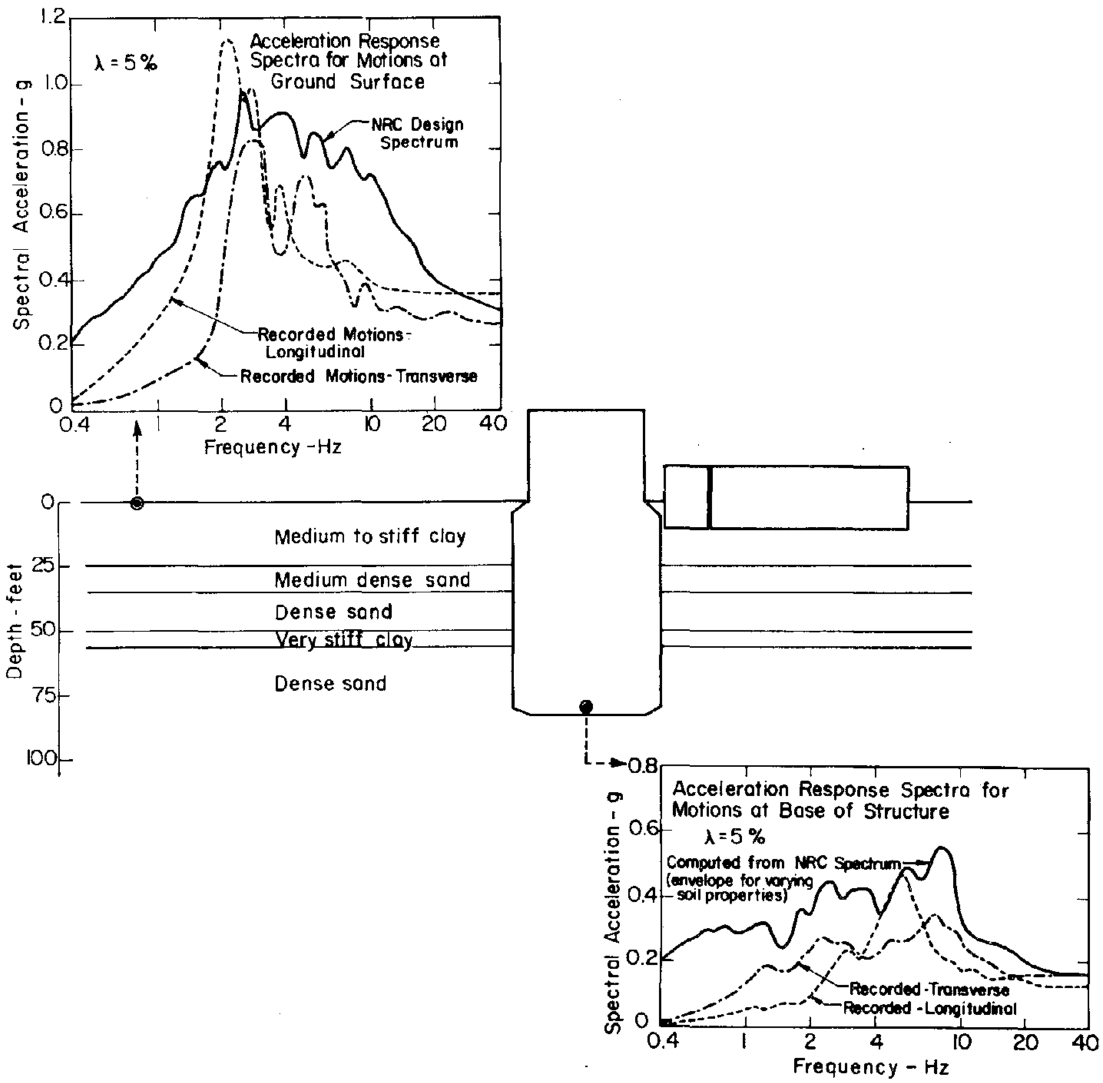


Fig. 13 COMPARISON OF SPECTRA FOR DESIGN AND RECORDED MOTIONS

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 the reduced values of soil moduli by 30 percent. With a satisfactory degree of accuracy, this leads to corresponding increases of 30 percent in both the free field spectrum at a depth of 85 ft 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 above.

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

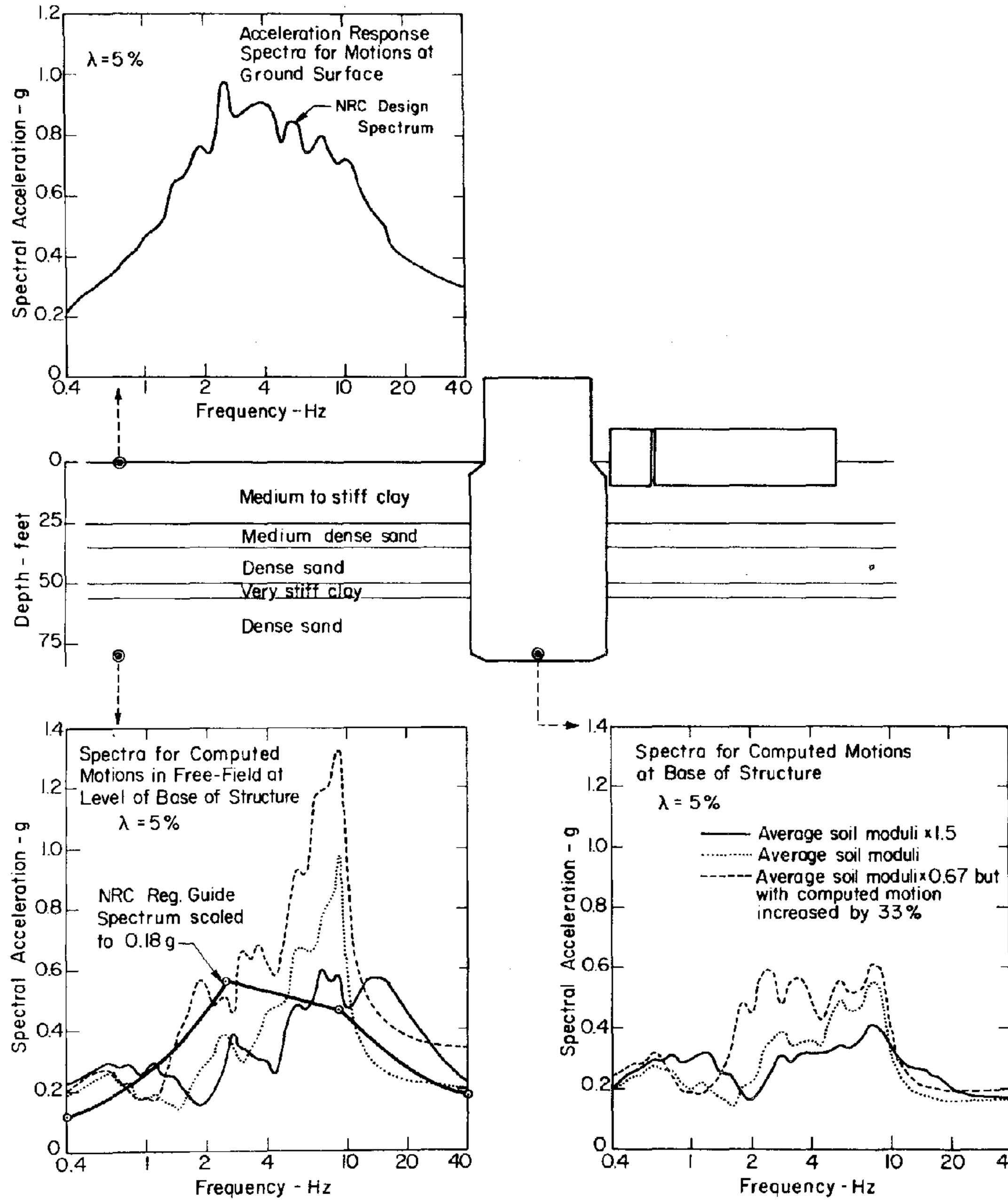


Fig. 14 SUMMARY OF SPECTRA FOR COMPUTED MOTIONS IN FREE-FIELD AND AT BASE OF STRUCTURE USING NRC DESIGN PROCEDURES

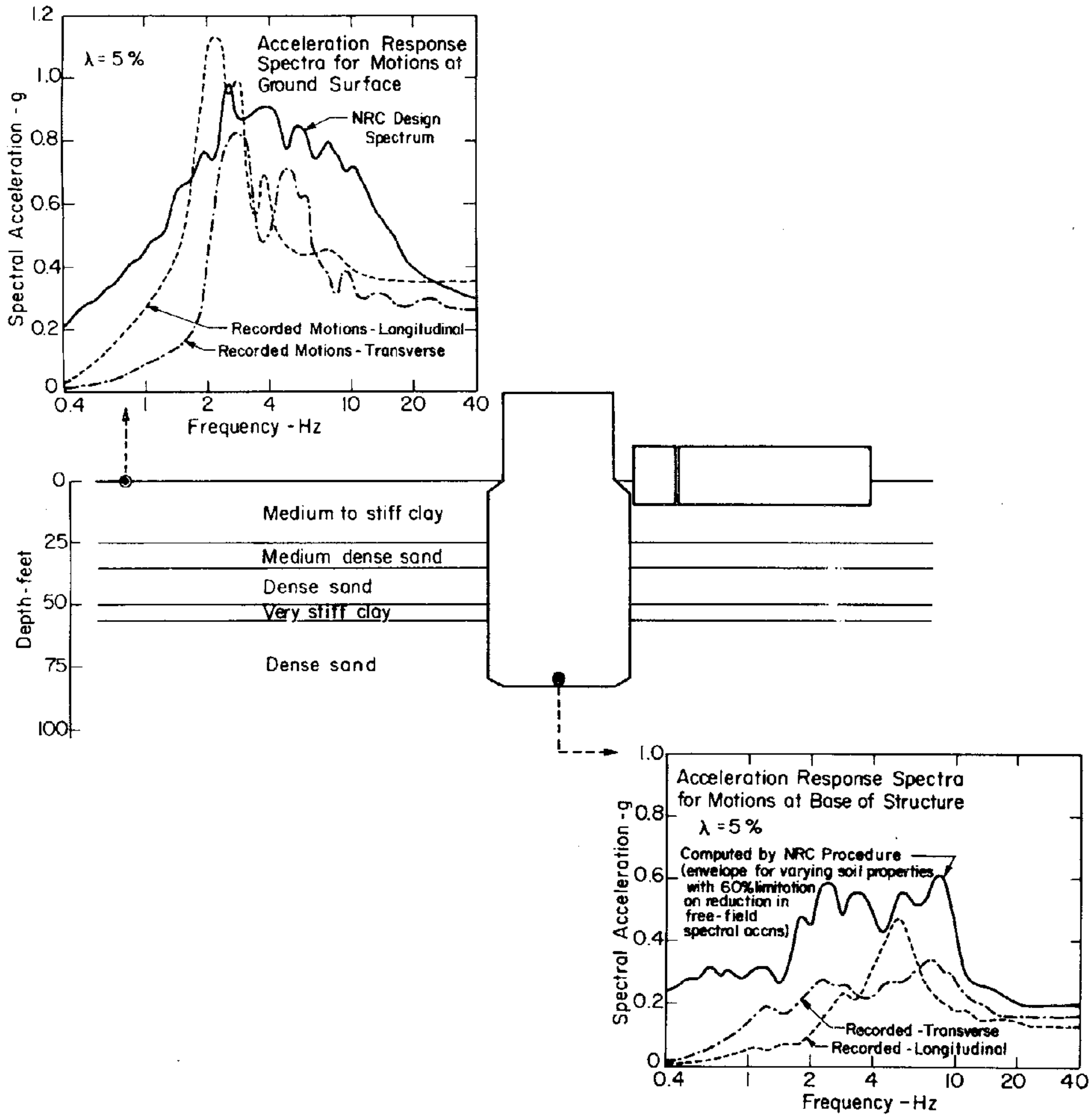


Fig. 15 COMPARISON OF SPECTRA FOR DESIGN AND RECORDED MOTIONS AT BASE OF STRUCTURE

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 Station during the Ferndale earthquake of June 7, 1975. Based on a knowledge of the motions developed 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 Refueling Building at a depth of 85 ft 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 in 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 entirely adequate but not excessively conservative margin of safety based on the motions recorded in this event.

It is of interest to note that Lambe (1973) has recently made a study of the accuracy of engineering predictions of soil behavior under static loading conditions. For this purpose he classified predictions into five groups as follows:

Type A	Prediction made before the event
Type B	Prediction made during the event but before the results are known
Type B1	Prediction made during the event but with results known at the time
Type C	Prediction made after the event but before the results are known

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 helpful 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 0.25g and 0.40g in the free field, was clearly a class A prediction using Lambe's terminology, in that the report describing this study by means of an idealized complete interaction analysis using finite element techniques was submitted to the Nuclear Regulatory Commission before the event of June 7, 1975 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.

and 3. Fixed surface motions established by the event.

Nevertheless the authors 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 of the method of analysis used for all cases. 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 clearly give some degree of support to the method used. They might also in due course give an equal degree of support to other methods which might be used for analyzing soil-structure interaction effects. These are significant facts in a field where 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.

Acknowledgements

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