NUREG/CP-0054
BNL-NUREG-52011

PROCEEDINGS

OF THE

WORKSHOP ON SOIL-STRUCTURE INTERACTION

BETHESDA, MARYLAND
JUNE 16-18, 1986

STRUCTURAL ANALYSIS DIVISION
DEPARTMENT OF NUCLEAR ENERGY, BROOKHAVEN NATIONAL LABORATORY
UPTON, NEW YORK 11973

PREPARED FOR
THE OFFICE OF NUCLEAR REGULATORY RESEARCH
U.S. NUCLEAR REGULATORY COMMISSION
WASHINGTON, DC 20555
CONTRACT NO. DE-AC02-76CH00016

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H.L. GRAVES* and A.J. PHILIPPACOPOULOS, EDITORS

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Office of Nuclear Regulatory Research

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FIN A-3298

Available from
Superintendent of Documents
U.S. Government Printing Office
P.O. Box 37082
Washington, DC 20013-7982
and
National Technical Information Service
Springfield, Virginia 22161

PREFACE

The subject of Soil-Structure Interaction (SSI) analysis which is addressed in part in the resolution of the Unresolved Safety Issue (USI) A-40, "Seismic Design Criteria", has often proved to be difficult and controversial in past licensing reviews. To examine some of the areas of difficulty and to review the licensing process the U.S. Nuclear Regulatory Commission, Office of Research sponsored a workshop on SSI. The workshop was held in Bethesda, Maryland, on June 16-18, 1986. More than 120 people attended the workshop inlcuding representatives from France, Italy, Japan and Switzerland as well as the NRC staff and U.S. industry.

The workshop was hosted by Brookhaven National Laboratory (BNL) under contract to U.S. NRC Principal organizers for the workshop were H. L. Graves of U.S. NRC and A. J. Philippacopoulos of BNL.

The workshop was conducted on a basis of presentations by invited panelists, and NRC staff, followed by questions and answers from other panelists and the audience. Summaries of each technical session were presented and are contained in this report along with consensus opinions of panelists on issues discussed.

These proceedings are published with the intent that they can assist the NRC staff and industry in eliminating the difficult and controversial areas of SSI and lead to an improved licensing process.

ABSTRACT

The Workshop on Soil-Structure Interaction provided an exchange of information between regulators, practitioners and researchers for the purpose of examining SSI licensing criteria in the light of recent analytical and experimental development. These proceedings contain the papers presented by panelists and summaries of the sessions along with recommendations of the panel members for each session. Technical areas covered by the panels were (1) definition of free-field motion, (2) ground motion input needed for site specific SSI analysis, (3) SSI methodology, and (4) experience and experimental observation. The summaries were derived to identify areas in the licensing criteria which could be changed to improve the licensing process.

ACKNOWLEDGEMENTS

The organizers wish to express their appreciation to Dr. Nilesh C. Chokshi, Office of Nuclear Reactor Regulation, U.S. NRC and other NRC staff for advice and support while developing the workshop program. In addition, the organizers wish to thank Ms. Joan Murray, Liz Gilbert, Diana Votruba, Jean Ramirez and Bonnie Biittner for assistance and attention to details which helped make the workshop possible.

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June 16-18, 1986
Bethesda Marriott
Bethesda, MD

Hosted by: Brookhaven National Laboratory
Sponsored by: U.S. Nuclear Regulatory Commission

Conference Organizers:
H. Graves, U.S. Nuclear Regulatory Commission
A.J. Philippacopoulos, Brookhaven National Laboratory

P R O G R A M S C H E D U L E

MONDAY, JUNE 16

8:00-9:30 a.m.	Registration
9:30-10:00 a.m.	Opening of the Workshop Harold R. Denton - U.S. Nuclear Regulatory Commission
10:00-11:30 a.m.	SESSION 1: Introduction and Overview of SSI Issues Moderator: H. Graves - U.S. Nuclear Regulatory Commission Speaker: G. Bagchi - U.S. Nuclear Regulatory Commission
11:30a.m.-1:00p.m.	L U N C H
1:15-5:00 p.m.	SESSION 2: Definition of Free-Field Motion Moderator: L. Reiter - U.S. Nuclear Regulatory Commission Panel: D. Bernreuter - Lawrence Livermore National Laboratory K. Campbell - U.S. Geological Survey C. Costantino - City University of New York G. Frazier - Scientific Applications International Corp. R. McGuire - Risk Engineering, Inc. A. Murphy - U.S. Nuclear Regulatory Commission J. King - Electric Power Research Institute

Invited Papers:

- "Empirical Prediction of Free-Field Ground Motion Using Statistical Regression Models"
  K.W. Campbell
- "Uncertainties in the Estimates of Seismic Ground Motion at a Site Made Using the Site-Specific Spectra Approach"
  D. Bernreuter, J.C. Chen
- "Random-Vibration Models of Free-Field Seismic Ground Motion for Soil-Structure Interaction"
  R.K. McGuire

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- "Numerical Simulation of Earthquake Motion-Capability Review"
  G. Frazier

- "Some Comments on Ground-Motion Aspects of the Proposed Revised Standard Review Plan"
  J.L. King

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Moderator:
L. Heller - U.S. Nuclear Regulatory Commission
Panel:
T. Cheng - U.S. Nuclear Regulatory Commission
J.R. Hall, Jr. - Stone and Webster Corp.
C. Costantino - City University of New York
J. Costello - U.S. Nuclear Regulatory Commission
I. Idriss - Woodward-Clyde Consultants
R. Kennedy - RPK Structural Mechanics Consulting
J. Roesset - University of Texas, Austin
H. Seed - University of California, Berkeley

Invited Papers

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  R.P. Kennedy, M.S. Power, C.-Y. Chang

- "Specification of Ground Motion Input for SSI Analysis"
  J. Roesset

- "Development of Site-Specific Earthquake Ground Motions"
  I. Idriss, P. Somerville

- "Uncertainties in Soil-Structure Interaction Analysis"
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11:30a.m.-1:15p.m. L U N C H

1:15-5:00p.m.

SESSION 4: SSI Methodology
Moderator:
P.T. Kuo - U.S. Nuclear Regulatory Commission
Panel:
H. Graves - U.S. Nuclear Regulatory Commission
A. Hadjian - Bechtel Corp.
J. Johnson - EQE Incorporated
E. Kausel - Massachusetts Institute of Technology
J. Lysmer - University of California, Berkeley
A. Philippacopoulos - Brookhaven National Laboratory
A. Veletsos - Rice University
J. Wolf - Electrowatt Engineering Services

Invited Papers:

- "Licensing Concerns in SSI Methodology"
  A. Hadjian

- "Some Perspectives on Dynamics of Soil-Structure Interaction"
  A.S. Veletsos

- "Site Response - A Critical Problem in Soil-Structure Interaction Analyses for Embedded Structures"
  H. Bolton Seed, J. Lysmer

- "Methodologies for SSI - Stochastic Response of Rigid Foundations"
  A. Pais, E. Kausel

WEDNESDAY, JUNE 18

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C.A. Miller - City University of New York
Panel:
J. Burns - U.S. Nuclear Regulatory Commission
C. Higgins - Applied Research Associates
P. Smith - EQE Incorporated
M. Srinivasan - Argonne National Laboratory
H.T. Tang - Electric Power Research Institute
D. Vaughan - Weidlinger Associates

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  H.T. Tang

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- "Numerical-Experimental Approach to the SSI Analysis"
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Moderator:
N. Chokshi - U.S. Nuclear Regulatory Commission
Panel:
All moderators from previous sessions
Speaker:
J.D. Stevenson - Stevenson and Associates
"Economics of SSI Effects on Nuclear Power Plants"

4:00-4:15p.m. — Closing Comments
Speaker:
J. Burns - U.S. Nuclear Regulatory Commission

SOIL STRUCTURE INTERACTION WORKSHOP
BETHESDA MARRIOTT HOTEL
BETHESDA, MD
JUNE 16-18, 1986

LIST OF ATTENDEES

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SESSION 1

INTRODUCTION AND OVERVIEW OF SSI ISSUES

INTRODUCTION AND OVERVIEW OF SSI ISSUES

Nilesh Chokshi and Goutam Bagchi

Introduction and Objective

In the last 10 years or so there has been considerable national and international research activity in the area of soil-structure interaction analysis. These research activities have not only included analytical and theoretical investigations but have also incorporated both experimental investigations and evaluation of recorded data. Also, the NRC is developing a proposed resolution for USI A-40, "Seismic Design Criteria," which include revisions to the staff guidelines in the area of SSI, i.e. the revision of the SRP sections. To complete this guidance the staff intends to utilize information developed in this workshop. It is our intention to endorse those SSI analysis methods which reflect current understanding and accommodate new developments in SSI.

The controversies and heated debates about SSI issues are not new to those familiar with past licensing reviews. It seems that we always discuss the same issues in each licensing proceeding without reaching agreement. Therefore, the main objective of this workshop is to facilitate effective communication between the NRC and the experts assembled here, and come to some common understanding about SSI issues which will form the basis for revising the SRP sections.

In particular, the objectives of this workshop can be summarized as follows: (1) to examine the SSI-related licensing concerns and various procedures and alternatives jointly by the regulators, practitioners, researchers, utility representatives and other interested groups in the light of the recent analytical and experimental development; (2) to examine the areas of greater uncertainties and means to address them; (3) to review the licensing criteria in the SSI area and discuss suggestions to improve the licensing process; and (4) to present results of recent US NRC research.

By examining the SSI procedure in its entirety, including seismological considerations and the nature of ground motion input, it is hoped that this workshop will lead to the development of more consistent requirements in the seismic analysis criteria. The impact of requirements in each individual step on the overall seismic evaluation result should then be made clear.

To what extent we achieve our objectives will depend upon how effectively we communicate with each other. It is hoped that this workshop establishes the will to communicate with each other on various aspects of SSI issues. This approach should alleviate the SSI concerns in each licensing proceeding and foster a better agreement between the utility and the staff.

Format

The format for the workshop is developed keeping in mind the main objective, i.e., discussion of issues important in licensing process. The workshop is divided into four technical sessions and one summary session as follows. Sessions 2 and 3 will discuss the topics on definition of free-field motion and ground motion input needed for site specific SSI analysis. The subject of SSI proper will be discussed in Session 4. In Session 5, we will discuss observations from experiments and actual earthquakes as they pertain to the SSI issues. The sessions will run sequentially as each session covers issues which are an integral part of the overall SSI process. Each session moderator will introduce the issues of concern, the session objectives and a set of specific questions that will be addressed by the invited panelists. In addition, each of the panelist will present their views on the issues at hand. One to two hours will be reserved at the end of each session during which the panelists and moderators will discuss the issues and attempt to answer questions raised. It is hoped that the audience will actively participate during these panel discussions providing a valuable input by expressing their views on the subject topic and its relation to overall SSI licensing criteria. In the final session, the moderator of each session will attempt to summarize the views of the respective session with the objective of providing an opportunity to examine the entire SSI procedure. At the end of each day, panels will meet to discuss the summary which is to be presented in the final session.

A Brief Discussion of SSI Licensing Criteria and Issues

Since the late 1960s, SSI effects have been considered in the seismic analysis of nuclear power plant structures. The earlier SSI analyses were carried out by representation of soil media as lumped, frequency independent spring and dashpot systems through the use of elastic half-space theory. In 1972-73, soil-column modeling (i.e. shear beam or one-dimensional shear wave propagating vertically as represented by SHAKE) was proposed and thus finite element representation of soil-media came into being. The NRC staff position in the 1975 version of Standard Review Plan Section 3.7.2 advocated the use of the finite element method (FEM) for the sites which were layered or inhomogeneous. The use of elastic half-space (EHS) was considered acceptable for sites with uniform properties. Specifically, SRP requirements stated that:

a. The design response spectra are defined for the free field and applied at the proposed finished grade level of the site.

b. Using an appropriate analysis method, with appropriate soil properties, obtain a time history at the base of the idealized soil profile. One acceptable method for deconvolution analysis of the design response spectra at a finished grade is a combined application of the SHAKE and LUSH computer codes. Use of other equivalent computer codes and analysis techniques is also acceptable. When the time history obtained from these methods is applied at the base of the idealized soil profile and the soil-structure interaction system, the resulting free field vibratory ground motion at finished grade level should give response spectra that envelop the design response spectra. This time history should appropriately account for variation in the soil properties at the site. In addition, when the time history obtained is applied at the base of the idealized soil profile, using appropriate soil properties, the vibratory motion calculated at the elevation of Category I structural foundations should, in general, give response spectra at all frequencies (0.2 cps to 50.0 cps), not less than 60% of the design response spectra. The same limitation applies to the response spectra obtained at the foundation level in the free field for the soil-structure interaction system. Response spectral values in the idealized soil profile at the foundation level and those at the foundation level of the interaction system that are less than 60% of the corresponding design response spectral values may be acceptable provided they can be justified. The justification will be reviewed on a case-by-case basis.

c. The time history developed in item b. above should be used at the base of the soil-structure interaction system, with appropriate soil properties, for subsequent soil-structure interaction analysis.

These staff requirements resulted in considerable controversy between the NRC staff and other practitioners. The staff took the position that there was enough uncertainty about both methods so that one should not rely solely on one method. The major controversy arose with regard to the input location of the control motion and the use of "deconvolution" procedures. Because of the unverifiable nature of the one-dimensional wave propagation model due to the lack of observed or experimental data, it was contended that the use of the deconvolution procedure may lead to unrealistic dips in the design ground spectrum and may underestimate the input motion to the structures. This and other issues led to an ACRS subcommittee meeting on SSI issues with experts from all over the U.S. including the NRC staff in February of 1977. No specific conclusion was reached on the validity of either method. However, the meeting appeared to conclude that for shallow embedment case, the use of deconvolution method to reduce the ground motion with the soil-column assumption was questionable and not reasonably conservative. In any case, the NRC staff implemented the following requirements as the acceptance criteria for SSI analysis after the meeting, and these criteria were incorporated into the current version (1981) of the Standard Review Plan. In summary, this position requires that Category I Structures, systems, and components should be designed to accommodate responses obtained by one of the methods listed below:

(a) Envelop the results of both EHS and FEM:

(b) Use results of one method with conservative design considerations of effects from use of the other method; and

(c) Combination of (a) and (b) with provisions for adequate conservatism in design.

Note that the Regulatory Guide 1.60 motion anchored at OBE and SSE 'g' values (or site specific motion if applicable) is to be applied at foundation level in the free-field. This position led to some difficulties in the review of OL applications of a number of plants. These difficulties resulted not only from the fact that these plants were designed based on the earlier requirements applicable at the CP stage, but the staff position was also challenged on various technical grounds. It has been contended that requiring the broad band motion to be input within a soil structure, such as at the foundation level, leads to inconsistent results (this is not a new issue - previously, discussions of this issue with the industry resulted in the 1975 version of the SRP requirements).

It is also argued that the SSI analysis techniques have advanced enough so that the use of only one method should be sufficient. NUREG/CR-1161, the document which discusses technical issues related to USI A-40 also recommends similar views. This workshop has assembled all of us here to collectively examine the NRC requirements in the SSI area in detail such that the future licensing requirements become more consistent with the state-of-the-knowledge.

Since the establishment of the current staff criteria, advances in both analytical and experimental investigations have been considerable and the examination of the fundamental technical issues is appropriate. Of course, this is not to say that in future there will be no further need for revision to SRP criteria or other workshops. We all recognize that significant advances in the state-of-knowledge must be accommodated in the SSI evaluation procedures. In fact, the currently planned large scale field studies, such as at Lotung, Taiwan should lead to further reduction in uncertainties in many SSI aspects. Since the SRP revisions are being undertaken as a part of the resolution of USI A-40, let us briefly examine the overall seismic design requirement presently being considered by the staff. Note that after this workshop the staff will finalize its draft resolution of USI A-40. This proposed resolution will be made available for public comments. The final resolution of this issue will consider all the comments.

Status of USI A-40

In mid and late seventies, as the total seismic design process evolved, two questions faced those concerned with seismic safety of nuclear reactor facilities:

a. How adequate are the plants in earlier generations with respect to current safety requirements?

b. What is the margin of safety in the overall seismic design process?

USI A-40 was initiated to address these questions. The Task Action Plan stated the objectives as "...to investigate selected areas of the seismic design sequence to determine their conservatism for all types of sites, to investigate alternate approaches to parts of the design sequence, to quantify the overall conservatism of the design sequence, and to modify the NRC criteria in the Standard Review Plan if changes are found to be justified." Although one of the studies conducted under A-40 included the quantification of conservatism in seismic design, it was concluded that the meaningful quantification of margin would require considerable resources and would be beyond the scope of A-40 studies. SSMRP and the Seismic Margin Program were conceived from this realization. Other studies under A-40 included the following: (1) elasto-plastic seismic analysis methods; (2) site-specific response spectra; (3) nonlinear structural dynamic analysis procedures; and (4) soil-structure interaction. The final task in USI A-40 was to review the results of other studies under A-40 and recommend changes in the Standard Review Plan and Regulatory Guides. Thus, NUREG/CR-1161, "Recommended Revisions to Nuclear Regulatory Commission Seismic Design Criteria," is considered to present technical findings of USI A-40.

Based on the review of the recommendations of NUREG/CR-1161, the staff has proposed changes in SRP Sections 2.5.2, 3.7.1, 3.7.2 and 3.7.3. It should be noted that the proposed revision to SRP Section 2.5.2 while incorporating some recommendations from the NUREG/CR-1161 relies primarily upon staff position developed during recent OL reviews. Table 1 indicates the summary of activities completed and to be completed in the resolution of USI A-40. The future activities and schedules will be described after the brief discussion of the proposed changes in the SRP sections.

The proposed changes in SRP Sections 3.7.1 through 3.7.3 can be grouped into the following eight categories:

1. Design Time History

2. Development of Floor Response Spectra and Effects of Parameter Variations on Floor Response Spectra

3. Percentage of Critical Damping Values

4. Soil-Structure Interaction

5. Seismic Analysis Methods

6. Seismic Analysis Methods and Combination of Modal Responses

7. Methods of Seismic Analysis of Above-Ground Tanks

8. Category I Buried Piping, Conduits and Tunnels

In Table 2, a brief summary of the proposed changes in each of the above category is presented for information. Except for the soil-structure interaction area, the detailed discussion is not presented here for other categories. These proposed changes are described in the published report on the value/impact assessment study (NUREG/CR-3480, August 1986) and proposed changes will be issued for public comments as discussed earlier.

The invited expert panel members were provided with the working drafts of the proposed SRP sections to assist them in preparing their comments for this workshop. In these SRP sections, two broad alternatives for SSI analyses were included. It must be emphasized that these broad alternatives do not represent the proposed position on the SSI requirements. These alternatives were included to initiate discussions at this workshop and to highlight the licensing issues faced by the staff. One obvious alternative is to endorse with only minor modifications the current staff requirements in the SSI area as discussed earlier. The obvious considerations here are the familiarity of the industry and the staff reviewers with the criteria; recognition of the fact that any new plants at this stage will be essentially at the CP stage and difficulties encountered in recent OL reviews will not be repeated; and also the recognition that the current criteria, in general, are conservative and the issue of uncertainties may be somewhat moot. Of course, this approach would greatly suffer from not recognizing our current state-of-knowledge, perhaps leading to inconsistent and extremely conservative results in some cases, and impeding the progress in this area. Alternative 2 is essentially to update licensing criteria to reflect current state-of-the-art. The revised review criteria may include several alternatives rather than one prescriptive method to perform SSI analysis and would require development of the specific technical requirements, explicit recognition of uncertainties and means to address them, and simplified approaches to judge the validity of results from complex analytical models. This last point is important from the point of view of a reviewer who has to judge the adequacy of complex analysis models. Thus, one can identify three major topics as indicated in Figure 1 for the development of the SSI review guidelines. In various technical sessions these elements will be discussed in detail with respect to our current understanding of the subject. It is hoped that this workshop will provide useful guidance to the staff in developing review criteria.

Future Activities and Summary

As discussed earlier, after the recommendation of the Committee for the Review of Generic Requirements (CRGR) the staff will issue the proposed revisions to SRP Sections 2.5.2, 3.7.1, 3.7.2 and 3.7.3 for public comment. The staff is also planning to publish the proceedings of this workshop which will include the comments from the invited experts and summaries prepared by the moderators of each technical session.

As indicated in the opening address, this workshop is not the only staff attempt to examine its review criteria. There is considerable research underway in this area at NRC, in universities, in industry, and by foreign governments. In particular, large shaker tests at HDR facility in Germany, shaker table tests at Japan, large-scale experiment in Lotung, Taiwan and analytical research at various institutions is likely to provide significant findings. If these findings indicate that further revision of staff review criteria is warranted, future workshops will be held. With the distinguished experts and panelists gathered here today let us bring to bear our collective knowledge and insights to improve licensing criteria in the SSI area.

SSI LICENSING REVIEW CRITERIA

Specific Requirements, such as
- ° Acceptable class of methodologies for various site conditions
- ° Specification of input motion and location
- ° Location of boundaries
- ° Analysis requirements, etc.

Recognition of Uncertainties and Means to Address Them, such as
- ° What are the important uncertainties for a given situation?
- ° Variation in properties, parameters, non-linearities, etc.

Validity of Results From Complex Analysis, such as
- ° Verification of basic behavior and results
- ° Simplified methods
- ° Simplified models, etc.

Figure 1 - Elements of SSI Licensing Review Criteria

TABLE 1 Summary of Activities Under Draft Resolution of USI A-40

Activities	Status
Draft Resolution	
1. Technical Findings of USI A-40 (Publication of NUREG/CR-1161)	Complete
2. Staff Review of Technical Findings	Complete
3. Proposed Revision to SRP Sections	Complete
4. Value/Impact Assessment Study (Publication of NUREG/CR-3480)	Complete
5. Initial ONRR Management Review	Complete
6. Conduct SSI Workshop	June 16-18, 1986
7. Finalize the Staff Requirements In SSI	July, 1986
8. Complete the A-40 package for ONRR Management Review	August, 1986
9. CRGR Review	August, 1986
10. Issue for Public Comments	September, 1986
Final Resolution	
1. Revise USI A-40 package (SRPs, Value/Impact study) based on the resolution of public comments.	
2. ONRR Management Review	
3. CRGR Recommendation	
4. Issue Revised SRP Sections	

TABLE 2 Summary of Proposed Changes to SRP Sections 3.7.1, 3.7.2 and 3.7.3

SRP Section	SRP Topic	Proposed Change in Requirements
3.7.1	Design Time History	° Additional Power Spectral Density (PSD) requirement identified for the use of single time-history in the analysis. ° Option to use multiple time histories.
3.7.2	Development of Floor Response	° Option of direct generation of floor response spectra with proper justification. ° Option to use multiple time histories and parameter variations.
3.7.1	Percentage of Critical Damping Values	° Higher damping values may be used if justified. ° Compliance with provision in RG 1.61 on stress levels highlighted.
3.7.1 and 3.7.2	Soil-Structure Interaction Analysis	° Final requirements to be developed after Workshop. Major considerations: Location of Control Motion Acceptable Methodologies Uncertainties
3.7.2	Seismic Analysis Methods and Combination of Modal Responses	° Acceptance criteria for adequacy of number of degrees of freedom modified. ° Acceptance criteria for consideration of high frequency modes given in new proposed Appendix.
3.7.3	Methods of Seismic Analysis of Above-Ground Tanks	° New topic. Fluid dynamics and tank flexibility considerations included.
3.7.3	Category I Buried Piping, Conduits, and Tunnels	° Specifically states the kinds of ground-shaking induced loadings to be considered.

SESSION 2

DEFINITION OF FREE-FIELD MOTION

EMPIRICAL PREDICTION OF FREE-FIELD GROUND MOTION
USING STATISTICAL REGRESSION MODELS

by

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INTRODUCTION

The analysis of soil-structure interaction (SSI) requires a knowledge of free-field ground motion in the vicinity of structure. The manner in which this free-field motion is estimated can have an important influence on the formulation of the SSI problem. It has particular importance in the nuclear industry where SSI is included in the licensing process.

Several methods are currently used to estimate free-field ground motion. One of the more common methods, and that discussed here, is the use of statistical regression to develop strong-motion attenuation relationships. There are several important questions regarding the development and application of this method that are addressed in this paper. These are (1) What are the capabilities, limitations and uncertainties associated with the method?; (2) How should the method incorporate source effects, propagation effects, local site effects, ground-motion parameters, and high-frequency motions in a low-magnitude event?; and (3) How should correlations between the three components of motion be treated? These questions have been addressed in considerable detail elsewhere (e.g., Campbell, 1985a; Boore and Joyner, 1982). The intent of this paper is to discuss those aspects of the problem that relate specifically to the estimation of free-field ground motion, leaving the reader to refer to these other papers for a more complete discussion of the method.

THE REGRESSION MODEL

In its most common form, the regression model is given by the expression

$$Y = b_1\ f_1(M)\ f_2(R)\ f_4(P_i)\ \varepsilon \qquad (1)$$

where Y is the strong-motion parameter being predicted; $f_1(M)$ is a function of earthquake size--usually magnitude M; $f_2(R)$ is a function of some distance measure R; $f_4(P_i)$ is a function of parameters of the earthquake, path, site, or structure; ε is a random variable representing the uncertainty in the estimation of Y; and b_1 is a constant. Eq. (1) is typically referred to as an attenuation relationship.

In its most common form, $f_1(M)$ is represented by an exponential function of magnitude,

$$f_1(M) = e^{b_2 M} \qquad (2)$$

which is derived from the basic definition of magnitude as a logarithmic measure of ground motion amplitude. Some investigators have extended this relationship to include a polynomial of M to accommodate a reduction in the scaling of Y with M at larger magnitudes, a phenomenon referred to as saturation.

Common expressions for $f_2(R)$ are given by the relationships

$$f_2(R) = [R + b_5]^{-b_3} e^{-b_4 R} \tag{3a}$$

$$f_2(R) = [\sqrt{R^2 + b_5^2}]^{-b_3} e^{-b_4 R} \tag{3b}$$

where the term in brackets represents attenuation due to geometrical spreading of the wave front, with b_3 representing the geometrical attenuation rate, and where the exponential of R represents anelastic attenuation--that is, material damping and wave scattering--with b_4 representing the coefficient of anelastic attenuation. b_5 is used by many investigators to limit the value of Y at small distances, a form of near-source saturation.

Equation (3) is also used to account for differences in magnitude scaling with distance, most commonly by replacing b_5 with an exponential function of magnitude (Campbell, 1981; Joyner and Boore, 1981),

$$b_5 = b_6 e^{b_7 M} \tag{4}$$

This function has the effect of reducing the amount of scaling between Y and M at small distances, another form of near-source saturation.

The function $f_4(P_i)$ is usually represented by an expression of the form

$$f_4(P_i) = \Pi e^{b_i P_i} \tag{5}$$

Although its form is somewhat arbitrary, this expression agrees with empirical evidence suggesting that most earthquake source, site, and structure effects are multiplicative. One should add functions of magnitude and distance to this expression if P_i are found to correlate with these parameters (Campbell, 1985a).

The random variable ε is usually assumed to be lognormally distributed, though this is not an absolute requirement of most regression procedures. Other distributions for ε have been proposed, but a lognormal distribution is preferred because of its simplicity. If Y is lognormally distributed, then the logarithm of Y has a simple Gaussian distribution, allowing one to use common analytical expressions for characterizing the statistics of the regression. Some justification for the choice of a lognormal distribution for Y comes from the observations that most of the functions in Eq. (1) are exponential and that the residuals associated with Eq. (1) are usually found to approximate a lognormal distribution.

The b_i of Eqs. (1) through (5) are estimated from a set of observations by means of regression analysis, usually using the method of least squares. However, other regression methods are available that can be used to minimize the impact of "bad" data, or so-called outliers (Mosteller and Tukey, 1977). In addition, techniques have been proposed for minimizing the bias associated with nested errors associated with within-earthquake and between-earthquake variability. The most popular of these include weighted regression (Campbell, 1981), two-step regression (Joyner and Boore, 1981), and regression based on random effects (Brillinger and Preisler, 1984).

If Y is lognormally distributed, the regression should be performed on the logarithmic transformation

$$y = \ln Y \tag{6}$$

where $\varepsilon' = \ln \varepsilon$ is defined as a Gaussian random variable with zero mean and standard deviation σ.

All phases of the analysis are important if one expects to obtain reliable estimates of ground motion (Campbell, 1985a). Especially critical are: (1) the selection of appropriate functions in Eq. (1); (2) the selection of appropriate parameters to characterize ground motion, earthquake size and other earthquake-related effects, source-to-site distance, site effects, and structural effects; (3) the selection of an appropriate data base; (4) the selection of an appropriate analysis procedure; and (5) the evaluation of the relationships. The reader is referred to Campbell (1985a) for a detailed discussion of these aspects of the analysis. The remainder of this paper will be concerned with the specific problems associated with estimating free-field ground motion.

FREE-FIELD GROUND MOTION

Strictly speaking, free-field refers to a point on the surface of the earth located far enough away from man-made structures as to be unaffected by their presence. Unfortunately, there are few strong-motion records that meet these idealistic criteria. Thus, practically all existing recordings are affected to some degree by SSI. Strong empirical evidence of SSI effects has been presented by Campbell (1983, 1984a). Two lines of evidence are offered. The first and most direct evidence comes from a comparison of nearby recordings of strong ground motion obtained during the same earthquake. The second line of evidence comes from regression analyses between strong-motion parameters and earthquake magnitude, source-to-site distance, and other site and structural parameters. Both types of evidence indicate that high-frequency ground motions recorded at the base of a building founded on soil are significantly affected by the size and embedment of the structure, with larger and more deeply embedded buildings exhibiting smaller strong-motion amplitudes at wave periods less than about 1 s.

Analysis of Nearby Recordings. The data base used by Campbell (1984a) to study nearby recordings from the same earthquake is a revision of that originally compiled by TERA (1980) and Darragh and Campbell (1981). Campbell's analysis serves as a case study of the effects of building size and embedment on recorded ground motion. Such a study essentially removes certain inherent biases that can exist in regression analyses. For example, the use of nearby recordings from the same earthquake removes potential differences in source and path characteristics, while it minimizes the effects of local site conditions. To maintain quality and consistency in his data, recordings were selected if (1) the distance between any two stations was 1300 m or less, (2) the magnitude of the earthquake was larger than 3.4 M_L, (3) the epicentral distance was 75 km or less, and (4) the peak accelerations were 0.02 g or greater.

Four types of comparisons were made. The first comparison, Type IA, was between recordings obtained in small instrument shelters and those obtained in basements of buildings; the second comparison, Type IB, was between recordings obtained in instrument shelters and those obtained at the ground level of buildings without basements; the third comparison, Type II, was between recordings obtained in buildings without basements and those obtained in buildings with basements; and the fourth comparison, Type III, was between recordings obtained in buildings of different heights. For each comparison, reductions, in percent, due to building embedment or size were computed from the expression

$$100 \left[1 - \frac{Y_1}{Y_2}\right]$$

where Y_1 represents ground motion obtained in an embedded or larger building and Y_2 represents ground motion obtained in an instrument shelter, a nonembedded building, or a smaller building.

Recordings obtained at the Differential Array site in El Centro, California, during the 1979 Imperial Valley earthquake (6.9 M_s) support the contentions of Bycroft (1978), Crouse (1983), McNeill (1983) and Crouse et al. (1984) that even small instrument shelters can significantly modify true free-field ground motion. Adjacent recordings obtained from two different instruments of similar dynamic characteristics were found to have significantly different amplitudes, especially at high frequencies. The larger recordings, with peak horizontal accelerations of 0.37 and 0.51 g, were obtained in a small instrument shed attached to a concrete pad (Dogwood Road instrument). The smaller recordings, with peak horizontal accelerations of 0.19 and 0.38 g, were obtained on an instrument buried approximately 4 ft deep in a 5-inch-diameter hole (Differential Array No. 1 instrument). Bycroft and others indicate that this latter type of installation should approximate true free-field conditions. The soils are relatively soft, having a shear-wave velocity of approximately 100-150 m/s near the surface.

The ratios of the 5 percent-damped, pseudo-relative velocity response spectra for the two stations appear in Figure 1. Substantial amplification of the Differential Array No. 1 spectra occurs at frequencies of about 5-15 Hz for the two horizontal components. This amplification is consistent with simple soil-structure interaction concepts (Bycroft, 1978; McNeill, 1983). While such effects are restricted to instrument shelters of moderate size (the shed's concrete pad is approximately 9 m^2) founded on extremely soft deposits, they indicate a potential bias associated with recordings obtained in similar installations, and may explain some of the anomlously high peak accelerations that have been recorded in similar instrument sheds.

The mean reduction in horizontal and vertical peak accelerations for Type IA, IB, and II comparisons of nearby recordings appears in Table 1. This table demonstrates that there are, on average, substantial reductions in peak acceleration due to the size and embedment of buildings. The largest reductions are found for Type Ia comparisons (39% for horizontal and 44% for vertical) with somewhat smaller reductions being observed for Type IB and Type II comparisons. Reductions in vertical accelerations are similar to those for horizontal accelerations for Type I comparisons, but they are found to be substantially less for Type II comparisons. This suggests that ground-level buildings may reduce vertical accelerations to a greater extent than they do horizontal accelerations. This may result from the higher frequencies and shorter wavelengths typical of the vertical components of acceleration. The

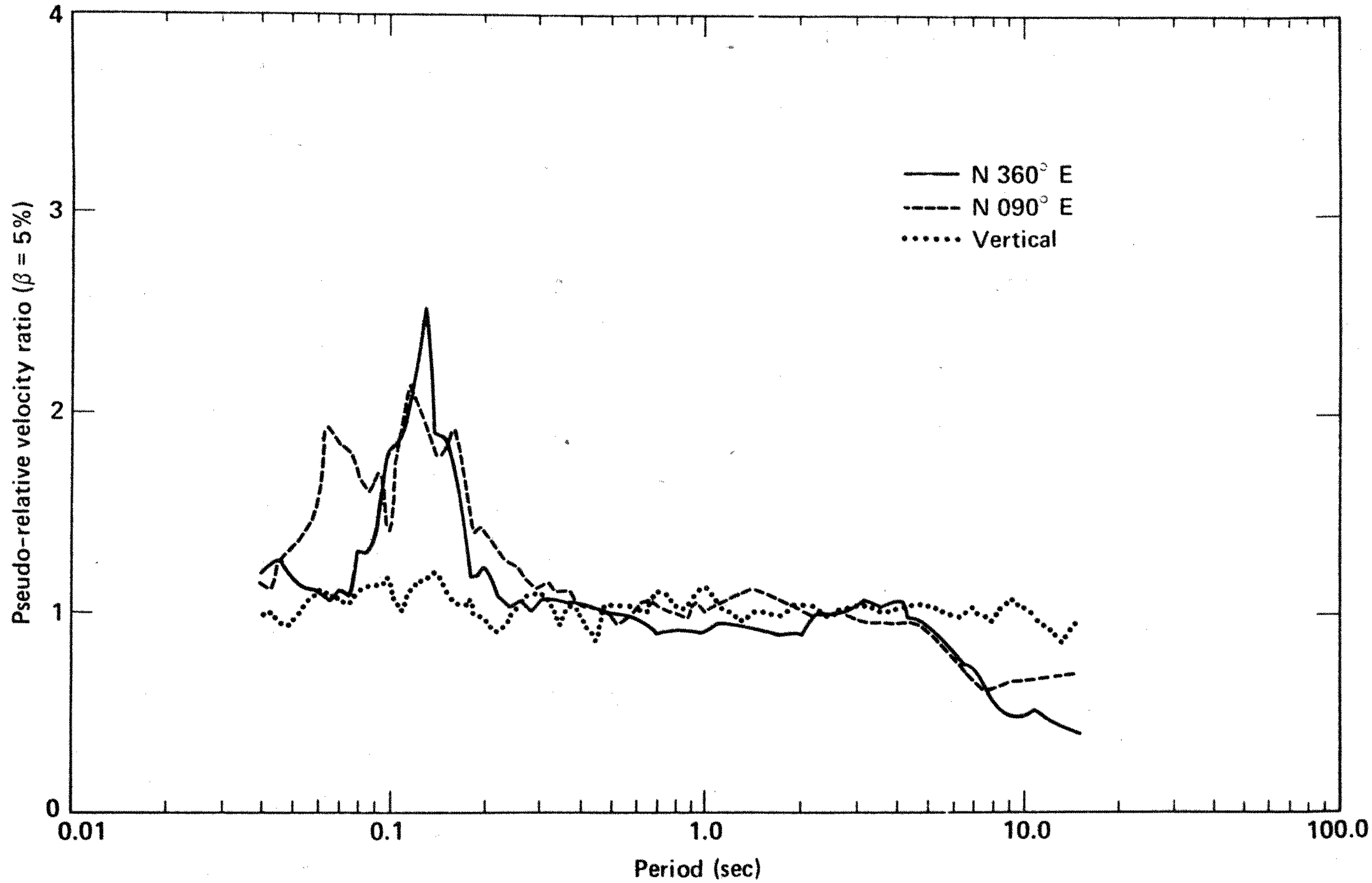

Figure 1. Ratio of 5%-damped Pseudo-Relative Velocity Spectra between Dogwood Road Instrument Shed and Differential Array No. 1 for the 15 October 1979 Imperial Valley Earthquake.

results for Type IA and Type IB comparisons may be biased somewhat by the soil-structure interaction effects of instrument sheds.

TABLE 1

Average Reductions in Peak Acceleration for Nearby Recordings

Type	Description	Reduction in Peak Acceleration (%)			
		Horiz.	No.*	Vert.	No.*
IA	Embedded Building vs. Inst. Shed	39	7	44	5
IB	Nonembedded Building vs. Inst. Shed	29	4	29	2
II	Embedded Building vs. Nonembedded Building	33	7	19	7
IA, IB	Building vs. Free-Field	35	11	40	7

*Number of comparisons used in the analysis.

Campbell (1984a) gives evidence that suggests that the observed reductions in peak horizontal acceleration may be related to earthquake magnitude. This is demonstrated in Figure 2, where reductions in peak horizontal acceleration are plotted as a function of magnitude. This figure suggests a tendency for Type IA comparisons to exhibit smaller reductions at higher magnitudes. Type IB and Type II comparisons appear to be consistent with this tendency.

As reported by Campbell (1983), TERA (1980) also investigated reductions associated with Type I and Type II comparisons for 5 percent-damped, pseudo-relative velocity (S_v). They found that for periods of 0.04 to about 0.2 s, horizontal S_v exhibited reductions equal to or larger than those observed for peak horizontal acceleration. For periods longer than about 0.2 s, reductions

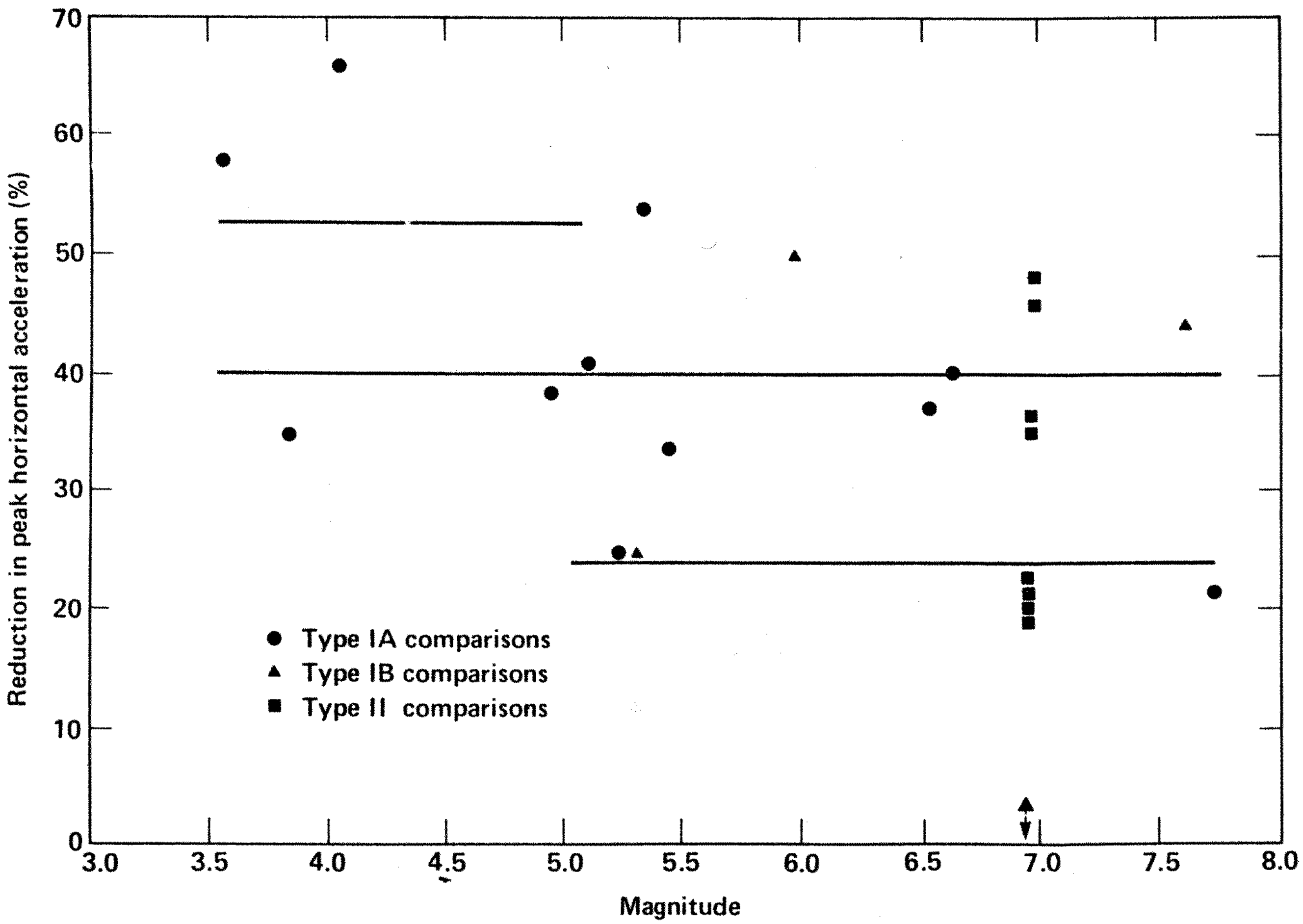

Figure 2. Reduction in peak horizontal acceleration for Types IA, IB, and II comparisons. The horizontal bars represent the reductions resulting from regression analyses of all near-source data as a function of magnitude and distance for the magnitude ranges shown.

tended to become smaller, approaching zero at around 1.0 to 2.0 s. These observations are consistent with those made by others (e.g., Crouse, 1978).

Campbell's (1984a) Type III comparisons investigated the effect of building height on the amplitudes of peak horizontal acceleration. Groups of buildings, either both with or both without basements, were investigated for systematic differences in recorded accelerations due to differences in height, determined on the basis of the number of stories. The taller buildings were consistently found to have smaller accelerations, the difference being larger for greater differences in the heights of the buildings. A simple regression analysis (Fig. 3) indicated that this reduction, in percent, was equivalent to 1.2 ΔS, where ΔS is the difference in the number of stories. The standard error of this estimate is a rather large 29 percent reduction, however, the slope of the relation was found to be significantly different from zero at a 90 percent level of confidence.

Seed and Lysmer (1980) and McCann and Boore (1983) have given evidence in support of a decrease in peak acceleration with increasing depth of embedment for buildings in Tokyo and southern California (Figs. 4 and 5). However, as McCann and Boore have shown (Fig. 6), there is a strong correlation between depth of embedment and building height which makes the separation of these two effects extremely difficult. The Type III comparisons presented above attempted to minimize embedment effects by comparing only ground-level or basement-level recordings. However, these results, as well as those of the other two studies, no doubt include, to some extent, the effects of both building size and embedment in their analyses.

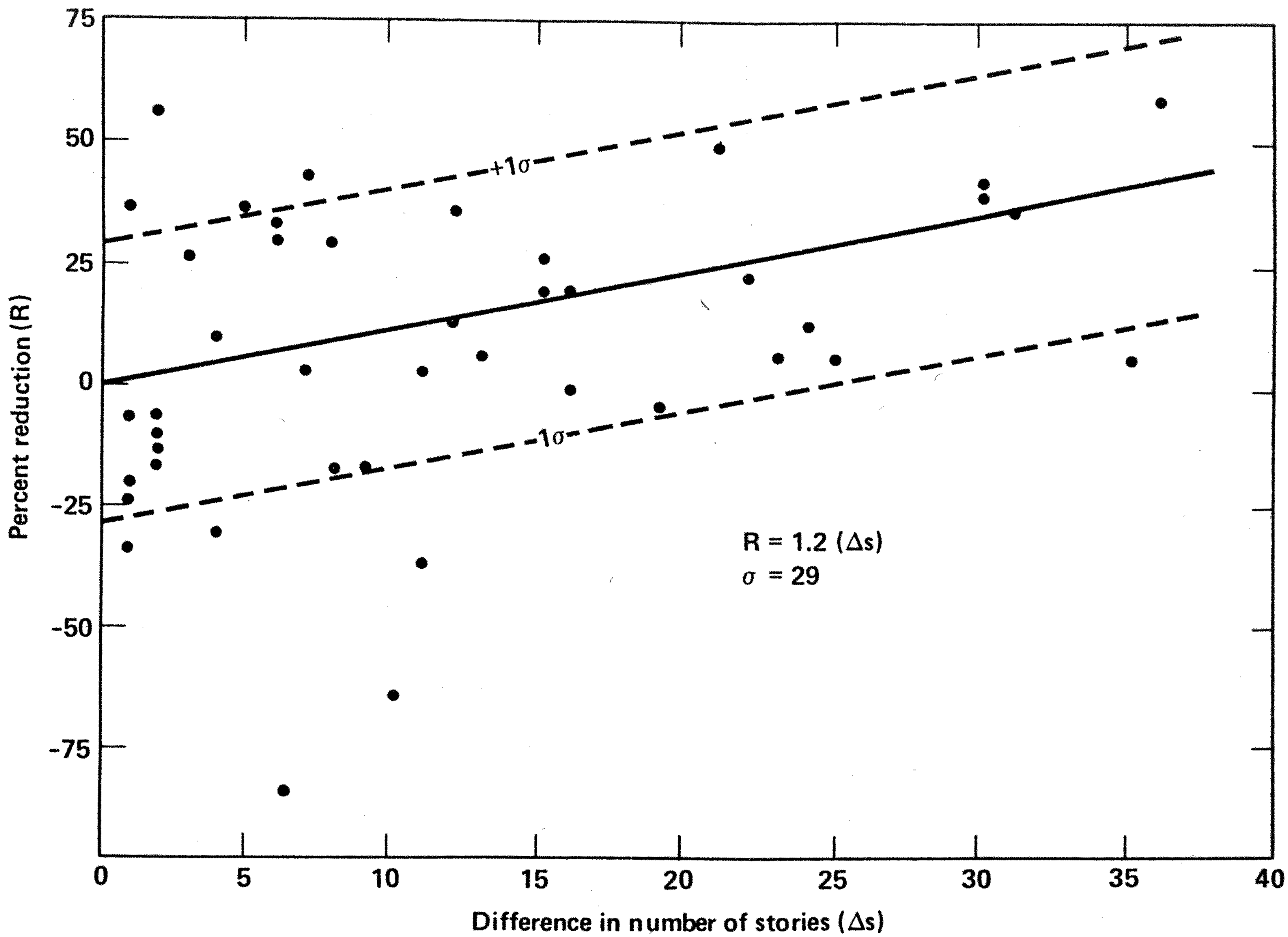

Figure 3. Reduction in peak horizontal acceleration for TypeIII comparisons. The solid line represents the results of a linear regression between the two variables plotted and the dashed lines represent the one standard error bounds

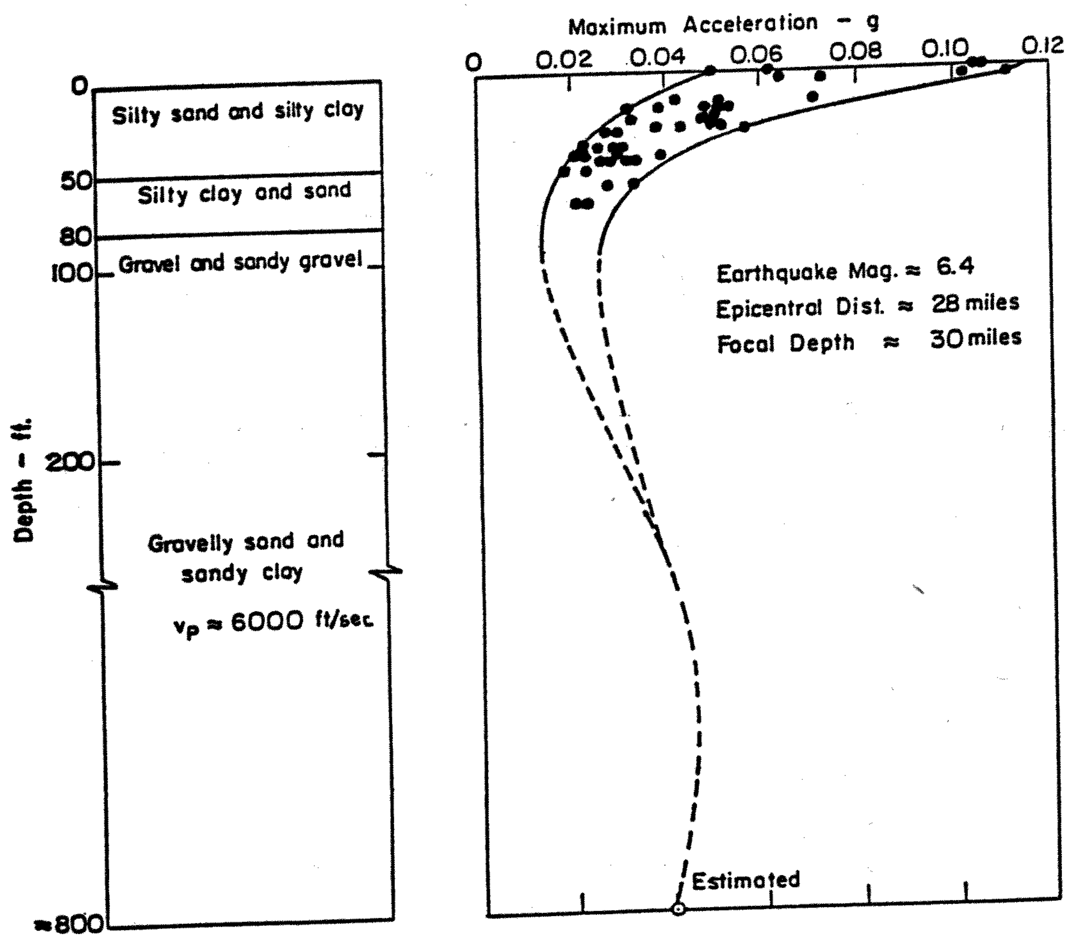

Figure 4. Peak Horizontal Acceleration as a Function of Depth for Buildings Located in Tokyo in the 1968 Tokyo-Higashi-Matsuyama Earthquake.

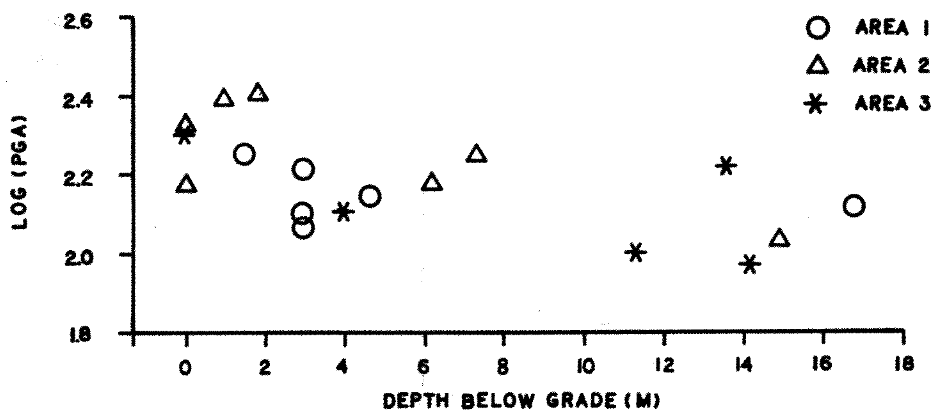

Figure 5. Peak Horizontal Acceleration versus Embedment Depth for Buildings Located in the Three Area Data Groups Used by McCann and Boore (1983).

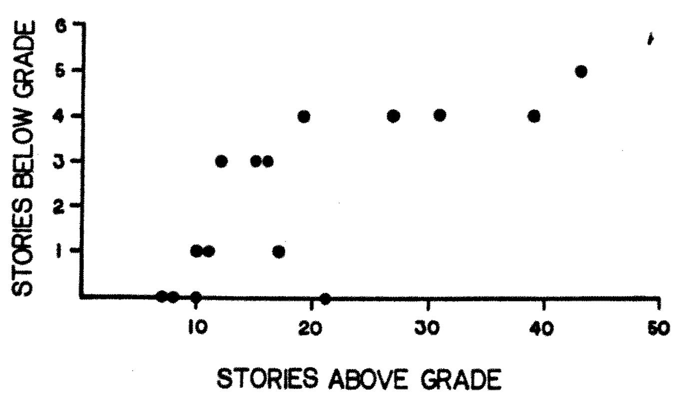

Figure 6. Stories Below Grade versus Stories Above Grade for Buildings Used in the Study of McCann and Boore (1983).

Regression Analyses. Campbell (1983, 1984a, 1984b) has used regression analyses of relevant strong-motion data to establish scaling variables for various structural parameters. The form of the attenuation relationship used to establish these variables was expressed by the equation (Campbell, 1984b)

$$\ln Y = a + bM - d \ln [R + c_1 \exp(c_2 M)] + \varepsilon' \qquad (6)$$

where Y is the ground-motion parameter to be predicted, R is the shortest distance between the site and the seismogenic zone of rupture, and M is surface-wave magnitude (M_S) when both local magnitude (M_L) and surface-wave magnitude are greater than or equal to 6.0, or local magnitude when both M_L and M_S are less than 6.0.

By replacing the expression on the right side of Eq. (6), excluding ε', by the function f(R,M), the equation used to establish structure-related scaling variables was represented by the expression

$$\ln Y = f(R,M) + \Sigma\ (e_i F_i) + \varepsilon' \qquad (7)$$

where e_i represents the scaling variables as determined from the regression analyses and F_i are "dummy variables" which take on values of either 0 or 1 depending on the classification of the data. For example, if one were to classify data as representing either embedded or nonembedded buildings, then such a classification could be identified by the variable F_1, where $F_1 = 1$ for nonembedded buildings and $F_1 = 0$ for embedded buildings.

Because of the functional form of f(M,R), a nonlinear regression analysis was used to establish the values of the scaling variables in Eqs. (6) and (7). Furthermore, in order to control the influence and possible bias associated with a few earthquakes that contributed a large number of recordings to the analysis, a weighted regression was performed, where weights were assigned to give each earthquake an equal contribution to the analysis within each of nine distance intervals. Justification for this weighting scheme and details of the analysis are discussed by Campbell (1981, 1982). Hypothesis testing techniques were used to determine the statistical significance of structural scaling variables established from the regressions. This was accomplished by performing the regression analysis omitting the scaling variable of interest, then analyzing the residuals of the various structural classifications for significant differences in their means and standard deviations. Standard t-tests could not be used because of the nonlinear form of the equation.

Using Eqs. (6) and (7), Campbell (1984b) performed a regression analysis on peak horizontal accelerations obtained from earthquakes of magnitude 5.0 M_L and larger recorded within 50 km of the fault. He classified recordings into three groups. The first group contained recordings obtained in basements of large embedded buildings (buildings 10 stories or greater in height). The second group contained recordings obtained in basements of small embedded buildings (buildings 3 to 9 stories in height). The third group contained recordings obtained at nonembedded sites, including buildings without basements. All sites were located on Quaternary alluvial and terrace deposits. The results indicated that the embedded sites were associated with horizontal accelerations significantly smaller than those associated with nonembedded sites, with the effect being a function of both building height and distance from the source. For example, at 5 km, the reduction in peak acceleration associated with large embedded buildings was estimated to be approximately 60 percent, while that associated with small embedded buildings was approximately 40 percent. At 25 km, the estimated reductions were approximately 30 percent and 5 percent, respectively.

Subsequent analyses suggest that the observed distance dependence of the reduction due to building embedment may be due, in part, to peculiarities associated with recordings obtained in downtown Los Angeles (a distance of approximately 25 km) during the 1971 San Fernando earthquake. An analysis of peak accelerations from this earthquake indicates that recordings in downtown Los Angeles have, as a whole, higher peak accelerations than other recordings from similar size earthquakes at similar distances. Yet, the relative effect of embedded versus nonembedded buildings for these downtown sites is consistent with the case studies presented earlier. The reasons for this higher-than-average ground motion are largely unknown. It could simply be due to azimuthal effects, such as radiation pattern, or be due to the effects of surface and subsurface topography.

An unpublished analysis of peak accelerations recorded during the 1968 Borrego Mountain earthquake, where the majority of sites were located greater than 200 km from the source, indicates that embedded buildings are associated with smaller-than-average accelerations even at these distances. In fact, preliminary analyses indicate that large embedded buildings have accelerations approximately 45 percent smaller than nonembedded sites and that small embedded buildings have accelerations approximately 25 percent smaller than nonembedded sites. These results also contradict the observed distance-dependent reductions observed for embedded recordings in the 1984 study, and suggest that SSI effects are important even at these large distances.

Campbell (1979, 1985b) has performed regression analyses on earthquakes of $M_L < 5$. He found peak horizontal accelerations to be approximately 30 to 45 percent lower when recorded in the basements of buildings rather than being recorded in nonembedded structures.

Discussion. Empirical analyses have clearly demonstrated the presence of SSI effects in strong-motion recordings. So, how can we estimate true free-field motions? The obvious answer is to simply exclude the recordings believed to contain SSI effects from our analyses. However, careful inspection of the recordings indicates that this is not a viable approach. The majority of the existing records have been obtained in large buildings, in basements of buildings, and near dams, where substantial modification due to SSI and embedment have been documented. In fact, few recordings have been obtained in what we could reasonably refer to as the free-field--vaults, small buried casings, and very small lightweight instrument shelters--many of which may also be affected to some degree by SSI (Bycroft, 1978; Crouse, 1983; McNeill, 1983; Crouse et al., 1984).

For small earthquakes, there are sufficient numbers of recordings in nonembedded one-story buildings to use these as a basis for developing attenuation relationships. However, it is not obvious that they too are not affected by SSI at relatively high frequencies, where base averaging may tend to attenuate the free-field motion. For the large earthquakes of greatest interest to engineers, there are not even a sufficient number of recordings from nonembedded one-story buildings to serve as an adequate data base.

Thus, we are left with the approach taken by Campbell (1983, 1984b, 1985a) as the only reasonable alternative. He proposes to exclude only those SSI-effected recordings that comprise an insignificant portion of the data base. The others he proposes to include, but parameterize, in order to accommodate their observed bias. Using this technique, there are then a sufficient number of recordings with which to statistically determine magnitude and distance scaling parameters.

A critical aspect of the above technique is the process of parameterizing the biased data. The most straightforward analysis is to include all the data in the regression without any explanatory variables, other than magnitude, distance, and any site parameters to be included, then inspect the residuals for any observed biases. This can be done by plotting the residuals versus potentially important parameters such as building height, building plan dimensions, or embedment depth, or plottting residuals as a function of magnitude and distance segregated into classes using appropriate structural parameters. The significance of observed differences can be assessed by correlation analyses and hypothesis-testing techniques. Trends with magnitude or distance should be modeled if found to be significant. Once significant parameters have been identified, they can be included in the relationships and the regression analysis repeated. A t-test can be used to determine the statistical significance of the estimated coefficients once the regression analysis is performed. Alternatively, a stepwise regression technique could be used to select significant parameters.

DISCUSSION

How should the regression technique include source effects, propagation effects, local site effects, ground motion parameters, and consideration of high-frequency motion in a low-magnitude event? The first four items are discussed in considerable detail by Campbell (1985a) and will not be repeated here. The last item is of great engineering interest. Small-magnitude earthquakes have generated some extremely large accelerations with little or no accompanying damage. The lack of observable damage presumably is a result of the relatively high frequencies and short duration of the strong phase of shaking, which contains little energy at frequencies of engineering interest. Very brittle structures and equipment having high natural frequencies are much more susceptible to damage from this type of ground motion than are typical buildings.

Because of the short durations and high frequencies characteristic of ground motions from near-source, small-magnitude earthquakes, peak acceleration, by itself, is not representative of the damage potential of the ground motion. Response spectra or Fourier spectra would be much more useful. Therefore, for such motions, attenuation relationships in terms of spectral values would be a more appropriate model. The problem is that there are very little spectral data from small-magnitude events, because there has been little interest for such data in the past.

What are the capabilities, limitations, and uncertainties associated with empirical attenuation relationships? The empirical attenuation relationship falls somewhere between the empirical and theoretical approaches to estimating ground motion. Unlike the empirical technique*, it provides a model for extrapolating beyond the available data--a capability extremely important for many engineering applications. Yet, unlike the theoretical technique*, it has a sound empirical basis.

The regression-based attenuation relationship has two major limitations. The most critical limitation is its regional dependence. Because of its empirical basis, the relationship is tied to strong-motion recordings from a specific region--primarily California. Since other regions of the United States have different crustal structure and attenuation properties than does California, one cannot simply apply attenuation relationships developed for California in these other regions without accounting for these differences. The second limitation is its use of parameters to characterize strong ground motion. In many engineering applications, such as in nonlinear analysis, the analyst requires a time series of ground motion. Though there are techniques for simulating an acceleration time series from strong-motion parameters, the resulting time series lacks many of the important features of actual accelerograms.

There are three types of uncertainty associated with predictions based on an attenuation relationship: model uncertainty, statistical uncertainty, and parameter uncertainty. Model uncertainty represents error associated with the assumed functional form of the relationship. This uncertainty is intangible, since we do not know <u>a priori</u> what the correct form of the relationship should

*The capabilities and limitations of the empirical and theoretical techniques are discussed in companion papers appearing in these proceedings.

be. Of course, the data itself can be used to choose among a proposed set of functional forms, but further winowing of this set requires some other basis. In principle, theoretical studies can be used as a basis for choosing between alternate forms, however, conflicting interpretations among seismologists typically make it difficult to absolutely eliminate many models solely on the basis of such studies. Model uncertainty is best estimated from an ensemble of attenuation relationships whose functional forms have been chosen based on both theoretical and empirical criteria. It may be calculated from the expression

$$\sigma_1^2 = \frac{1}{N-1} \sum (\hat{y}_{avg} - \hat{y}_i)^2 \tag{8}$$

where N is the number of attenuation relationships, $\hat{y}_i$ is the predicted value of ln Y for the ith relationship, and $\hat{y}_{avg}$ is the average value of $\hat{y}_i$.

Statistical uncertainty represents error associated with data dispersion. It is calculated from the expression

$$\sigma_2^2 = \frac{\sigma^2}{n_o} \tag{9}$$

where σ is the standard error of estimate associated with the regression and n_o is the number of future observations being predicted. For most applications, we are interested in a single future observation, for which $n_o = 1$.

Parameter uncertainty is error in the predicted value of y associated with error in the regression coefficients. For linear models whose coefficients have Gaussian distributions, this error is calculated from the expression

$$\sigma_3{}^2 = \sigma^2 X_o' C X_o \tag{10}$$

where X_o is the vector containing specific values of the independent variables (e.g., magnitude), X_o' is the transpose of X_o, and $\sigma^2 C$ is the variance-covariance matrix of the regression coefficients. For nonlinear models whose coefficients are non-Gaussian, Monte Carlo techniques must be used to estimate σ_3.

Assuming independence of the three types of uncertainty and assuming a lognormal distribution for Y, the α-percentile value of y is computed as

$$y_\alpha = \hat{y} + t_{\alpha,\nu} \left[\sigma_1{}^2 + \sigma_2{}^2 + \sigma_3{}^2\right]^{1/2} \tag{11}$$

where $t_{\alpha,\nu}$ is the t-statistic associated with cumulative probability α and $\nu = n - p - 1$ degrees of freedom, n is the number of observations used in the analysis, p is the number of regression coefficients including the constant, σ_1 is the standard deviation of y associated with model uncertainty, σ_2 is the standard deviation of y associated with statistical uncertainty, σ_3 is the standard deviation of y associated with parameter uncertainty, and $\hat{y}$ is the median estimate of y.

The usual, though inappropriate, application of Eq. (11) is to estimate y_α by the expression

$$y_\alpha = \hat{y} + z_\alpha \sigma \tag{12}$$

where z_α is the standard normal variable associated with cumulative probability α. This involves two assumptions: (1) $t_{\alpha,\nu}$ is assumed to be equal to z_α, valid only for a large number of degrees of freedom, and (2) σ_1 and σ_3 are assumed to be negligible compared to σ_2. While the first assumption can be satisfied if n is large, the second assumption is rarely satisfied. In fact, the second assumption is true only for estimates of y near the centroid of data where $\sigma_3 \approx \sigma/\sqrt{n}$ and $\sigma_1 << \sigma$. Both σ_1 and σ_3 are expected to become large when estimates of y represent an extrapolation of the data.

How should correlations between the three components of ground motion be treated? Correlation between the three components of ground motion can be handled with a technique known as multivariate multiple regression (Johnson and Wichern, 1982). The technique addresses the problem of modeling the relationship between several dependent variables and a single set of independent variables. Each dependent variable is assumed to follow its own regression model [e.g., Eq. (1)], but the error terms associated with different dependent variables may be correlated. Thus, the technique results in the ability to jointly estimate a set of dependent variables from a single set of independent variables.

Correlation between components has been treated in a variety of ways in the past. Some have simply ignored the need for a joint estimation by treating the errors as totally independent. Examples of such techniques are the inclusion of both horizontal components as independent observations and the treatment of one or more of the components as independent variables. Both techniques artificially increase the number of degrees of freedom of the regression and, thus, underestimate the standard error of estimate.

Other techniques have avoided the correlation between components (usually horizontal components) by performing regressions on the largest horizontal component, on the mean of the two horizontal components, on the vectoral combination of the two horizontal components, or on a random selection of horizontal components. Such techniques preserve the correct number of degrees of freedom, but do not easily allow for the estimation of more than one component.

CONCLUSIONS

If carefully developed and applied, regression-based attenuation relationships can provide a reliable approach to estimating strong-motion parameters. They combine attributes of both empirical and theoretical techniques. Their largest shortcoming is their reliance on data that, for the most part, have been modified by soil-structure interaction. The problem can be minimized by including parameters to account for the more significant effects, but the problem can only be resolved by the acquisition of a sufficient number of true free-field recordings.

ACKNOWLEDGEMENTS

I wish to thank Bernice Bender and Steve Harmsen for their review of the original manuscript. This research was funded in part by the U.S. Nuclear Regulatory Commission.

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UNCERTAINTIES IN THE ESTIMATES OF SEISMIC GROUND MOTION AT A SITE MADE USING THE SITE SPECIFIC SPECTRA APPROACH*

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ABSTRACT

This paper describes the site specific spectra (SSSP) approach used by NRC as one of its checks in the safety assessment of nuclear power plants. The SSSP approach is placed in the context of other methods for estimating the ground motion at a site. The sources of uncertainty are categorized and estimates of the contribution to the total uncertainty from each category are made. SSSP are developed for deep soil and rock sites for MM VII and MM VIII SSE levels to cover many of the cases for sites in the eastern United States.

1. INTRODUCTION

It is generally agreed that there is considerable uncertainty in estimates of ground motion at a site from any postulated earthquake. There are a number of sources contributing to the uncertainty which can be lumped into three major categories:

1) Uncertainties due to "source effects", e.g., fault geometry, and the rupture physics, etc. which varies from earthquake to earthquake.

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*This work was supported by the United States Nuclear Regulatory Commission under a Memorandum of Understanding with the United States Department of Energy.

2) Uncertainties introduced because the travel path from the fault to the site varies from earthquake to earthquake and site to site. We also include the relative geometry of the site relative to fault (radiation pattern effects) in this category.

3) Uncertainties introduced by the local site's geology. Although technically part of the travel path, it is worthwhile to single out the last few hundred meters of travel path. Also lumped into this category are recording building/pad effects.

There are several generic approaches that can be used to estimate the ground motion at any given site from some postulated earthquake:

1) Detailed numerical-theoretical modeling the fault geometry, rupture physics, travel path and site geology.

2) Empirical approach using regression analysis techniques applied to "appropriate" data sets to obtain relations between the expected ground motion and the parameters of the postulated earthquake, e.g., magnitude, distance away, etc.

3) Use of empirical/theoretical models, e.g., the random vibration (Stochastic) approach.

The use of any one of these techniques is plagued by our lack of understanding of the relative significance of each of the three main categories of uncertainties.

In this paper we want to examine, in particular, how these uncertainties impact the estimations made by a specific approach (the SSSP approach) and, in general, attempt to shed some light on the relative importance of each of the main sources of uncertainty.

2. SITE SPECIFIC SPECTRA APPROACH

The SSSP approach was suggested by Bernreuter (1979). It is a variation on the "regression analysis" approach. Typically, the regression analysis approach attempts to fit a model of the form:

$$\log y = C_1 + C_2M + C_3\log\text{ (distance metric)} + C_5\text{ (distance metric)} + C_6\text{ (site type parameter)} + E \quad (2.1)$$

where: y = ground motion parameter of interest, e.g., peak acceleration, spectra acceleration at some frequency, etc.
C_i = constants to be determined from the analysis.
E = error term, with zero mean and standard deviation σ

to some appropriate data set. If indeed one had a large unbiased data set, then the problem would be simple. But, we do not have such a data set, although for some areas it is improving rapidly. Unfortunately, nuclear facilities are located in the Eastern United States (EUS) where the data set is very limited.

The SSSP approach was developed as an attempt to overcome the lack of data in the EUS. The assumption is made that, on the average, major differences between the ground motion observed from typical EUS earthquakes and the ground motion from earthquakes of the same "magnitude" recorded elsewhere are primarily due to attenuation (travel path) and local site effects differences. The travel path differences can be minimized by use of a suite of records of the appropriate magnitude range recorded "near" the epicenter or rupture surface. The local effects can be minimized by the use of records recorded on similar site conditions. The estimated SSSP is obtained by averaging all of the response spectra computed from the selected suite of ground motion recordings. In most applications of the SSSP approach it has been assumed that the distribution of spectral amplitudes at any period is lognormally distributed. This assumption is also made in this paper. Generally, the 1-sigma spectrum resulting from the averaging process is used. Under the lognormal assumption it becomes the 84th percentile spectrum.

In selecting the suite of records a number of questions arise:

- o What is the appropriate magnitude range to use?
- o What is the appropriate distance range to use?
- o What distribution on distance is appropriate?
- o How many earthquakes/travel paths must be in the data set?
- o How "similar" must the recording sites be to the site at which the prediction is to be made?

The answers to some of these questions are, of course, regulatory issues, e.g., what earthquake is the selected suite of earthquakes attempting to model? In all cases, the answers depend on our understanding of the relative contribution to the uncertainty from the three major categories of sources of uncertainty. For example, if all earthquakes of the same magnitude M have basically the same source properties, then one would only need a few earthquakes to capture the potential variation in ground motion observed at a site from earthquakes of magnitude M.

The U.S. Nuclear Regulatory Commission (NRC) practice, Kimball (1983), Reiter and Jackson (1983), has been to utilize acceleration histories from a suite of earthquakes whose magnitudes are within $\pm$ 0.5 magnitude units of the target. For EUS sites, the safe-shutdown-earthquake (SSE) is defined in terms of Modified Mercalli (MM) epicentral intensity, in most cases either MM VII or MM VIII. NRC has generally assumed that a M_L = 5.3 earthquake models a MM VII earthquake and a M_L = 5.8 earthquake models a MM VIII earthquake.

In terms of the target distance, NRC has generally utilized acceleration time histories recorded within 25 km of the source with the average distance being 15 km. No clear criteria have been developed to define "similar" site conditions. Generally, there is little data which can be used to define the site conditions at most accelerograph sites. This lack of data makes it difficult to select records recorded at sites with "similar" site conditions. Thus one of the major objectives of this study is to attempt to determine a measure of what "similar" might entail.

3. MODEL FOR THE UNCERTAINTY

A typical example of the uncertainty resulting from a typical application of the site specific spectra approach is illustrated on Fig. 3.1a which shows an overplot of all of the spectra selected to define the site specific spectra for a deep soil site in the EUS. For this case, the SSE was specified as MM VII. As noted in Section 2 it is assumed that the spectral values at any period are lognormally distributed. Figure 3.1b shows the resultant median and 84th percentile spectra for the set of spectra shown on Fig. 3.1a. The uncertainty, under the assumption of the lognormal distribution, is measured by the size of σ_T, the standard deviation of the assumed lognormal distribution. It is seen from Figs. 3.1a, b that the uncertainty is considerable. Considering the large uncertainty in the SSSP estimate for the SSE at a typical deep soil site in the EUS, one might ask if the large uncertainty is due primarily to factors which are indeed uncertain such as: the rupture process of the earthquake; it's possible location relative to the site; it's magnitude; or if the large uncertainty is primarily due to the fact that data recorded at a too diverse set of sites was used.

To address this question we need to better understand the relative contribution of each of the categories to the uncertainty defined in Section 1.

To the first order, we can model σ_T by the relation:

$$\sigma_T^2 = \sigma_S^2 + \sigma_{TP}^2 + \sigma_{LS}^2 + \sigma_M^2 + \sigma_R^2 \tag{3.1}$$

where σ_S= Contribution to the uncertainty in the observed spectra due to variations in source parameters between earthquakes of the same magnitude.

σ_{TP}= Contribution to the uncertainty in the observed spectra due to travel path and seismic radiation pattern variations.

σ_{LS} = Contribution to the uncertainty in the observed spectra due to the fact that the records were recorded at different sites and in different buildings.

σ_M = Contribution to the uncertainty in the observed spectra because a range of magnitudes was used.

σ_R = contribution to the uncertainty because a range of distances (geometric spreading) was used.

In the application of a typical regression analysis approach, one is generally interested in σ_S, σ_{TP} and σ_{LS}, as explicit terms are included to account for neglecting the fact that earthquakes of various magnitudes and distances are used. However, in the site specific spectra approach, σ_M and σ_R could be significant.

Unfortunately, it is very difficult to study the relative contribution of each of the component σ's to σ_T. Ideally, one would like to isolate each factor, e.g., fix the site and travel path and examine the recorded ground motions from a wide variety of earthquakes. Clearly, this is not possible. The best that can be expected is a series of nearby earthquakes. To study travel path, one would like to fix the source and site and vary the location of the earthquakes. If all earthquakes were truly similar then this would be possible, but this is clearly not the case. If one knows σ and σ_{TP}, then one could estimate σ_{LS} by using data at fixed sites at which a number of earthquakes have been recorded and data at various sites recorded for the same earthquake. These questions have been addressed to some extent by Bernreuter (1979), McCann and Boore (1983) and McCann (1983). McCann (1983) and McCann and Boore (1983) used data from the 1971 San Fernanco earthquake recorded at several nearby subsets of locations to study σ_{TP} and σ_{LS}. It is difficult to draw firm conclusions because only one earthquake is involved and unfortunately a wide variety of buildings/depths of instrumentation are involved, although the building type and depth of the recording instrument were explicitly accounted for.

4.0 UNCERTAINTY INTRODUCED BY THE TRAVEL PATH

Because of the difficulty of fixing the travel path, it is worthwhile to first examine how much variation in the observed ground motion at a fixed site from a source with fixed parameters is introduced by changes in the travel path. Because the variation from source effects can be considerable (as will be shown in the next section), the data we propose to use to address this issue consists of the ground motion recorded from large underground nuclear explosions (UNE).

Figure 4.1 shows the 5 percent damped relative velocity spectra obtained from data recorded from 5 UNE at the Nevada Test Site (NTS). These 5 events each had a yield $\pm$ 4 kilotons of each other and the distance between the UNE and the recording station (station 10) was 55 $\pm$ 2 km. All of these UNE were located in Pahute Mesa area of NTS within a few kilometers of each other. It is observed from Fig. 4.1 that there is considerable variation between the spectra from event to event.

Figure 4.2 shows the envelope, median and 1-Sigma spectra based on the spectra for the five UNE shown in Fig. 4.1. As noted in Section 3 the computation is made assuming that the distribution is lognormal. The uncertainty as measured by standard deviation of the natural log of

S_v (σ_{LnSv}) varies the with the period with an average value of about 0.2. It is observed that there is as much as a factor of two difference between the computed median and the maximum observed value. It should be noted that if only the radial component is used the spread is somewhat reduced to a factor of 1.6. These results indicate that very minor variations in the travel path can induce considerable uncertainty in the observed ground motion at a site. There is some additional data, discussed in Section 6, which sheds additional light on the question of just how much variation in the observed ground motion at a site is due to only variations in travel path.

5.0 UNCERTAINTY INTRODUCED BY THE SOURCE

The question we would like to address in this section is: assuming that one could fix the site and travel path, what is the uncertainty in the observed ground motion at a fixed site from all possible earthquake source mechanisms for earthquakes of magnitude M? Note that this uncertainty could well be a function of M with it being larger for large earthquakes as compared to smaller earthquakes. There is very little data to address this issue. Thus, it is useful to first examine simple theoretical models to see what help they can give us, and use the little available data to see how well the simple theoretical models match reality.

Typically, to the first order, Brune's model (1970) is used to represent the source spectrum of an earthquake. Brune's model relates a few fundamental parameters of the earthquake to the Fourier amplitude spectrum of the displacement. The source function that would be observed at a distance R from the source in a half-space, (i.e., no complexity of layering transmission path, etc.) is:

$$FS_d(R,\omega) = \frac{K\Delta\sigma}{Rf_o} \frac{1}{\omega^2 + f_o^2} \tag{5.1}$$

where

$\Delta\sigma$ = Stress drop (source parameter)

f_o = Corner frequency (related to the rupture area)

R = Distance

K = constant

Using this simple model and a few other assumptions, Thatcher and Hanks (1973) showed that the local magnitude M_L of an earthquake is related to the parameters of Brunes' model of an earthquake by the relation

$$M_L = \log\Delta\sigma - 3/2 \log f_o + C \tag{5.2}$$

Thatcher and Hanks (1977) showed that this relation fits the data. Using Eqs. (5.1) and (5.2), it is relatively easy to show that

$$FS_a(R,\omega) = \frac{K f_o^{1/2}\, 10^{M_L}}{R} \qquad \frac{\omega^2}{\omega^2 + f_o^2} \tag{5.3}$$

where FS_a = Fourier spectrum of the acceleration

There is a close relationship between FS_a and the undamped relative velocity spectrum of interest in structural analysis. Eq. (5.3) shows that for a given magnitude we can expect the level of FS_a to vary as $\sqrt{f_o}$, Thatcher and Hanks (1973) provide measured data which suggest that f_o varies by about an order of magnitude for earthquakes in the range of 4.5 to 6.5. Thus, we would expect at a minimum up to a factor of 3 difference between the maximum and minimum levels of FS_a. If one thinks in terms of the SSSP approach, all of these earthquakes would be averaged together relative to the median prediction, we might expect that our uncertainty as measured by Sigma ($\mathrm{Ln}FS_a$) would be in the range of 0.4 - 0.5 or a factor of 1.5 or so uncertainty.

Figure 5.1 shows the 5 percent damped relative velocity spectra (both horizontal components) for two earthquakes of approximately the same magnitude

(M_L = 4.6) and within a few kilometers of each other, computed from records obtained at the Oroville Airport. It is observed that the relative level of the high frequency (short period) spectral amplitudes differ by about a factor of 3. Figure 5.2 shows a similar comparison for two M_L = 3.6 earthquakes based on records obtained at the Brawley airport. Although the two earthquakes recorded at Brawley were relatively close together, they are so close to the recording station that the radiation pattern could be a factor in the observed differences between the spectra. Once again the variation between the two earthquakes is about a factor of 3 or more.

Naturally, as the earthquakes become larger, a larger number of "barriers" become involved. Thus, we might well expect to see a wider variation in the observed ground motion at a fixed site.

6.0 UNCERTAINTY INTRODUCED BY THE SITE

In this section we want to examine how much uncertainty is introduced into our ground motion estimates from mixing data recorded at a number of different sites. If the site is fixed, i.e., if we only look at data for a fixed site, then $\sigma_{LS} = 0$, and

$$(\sigma_T)_{\text{fixed site}} = \sigma_S^2 + \sigma_{TP}^2 + (\sigma_M^2 + \sigma_R^2) \tag{6.1}$$

We can also estimate σ_{LS} from

$$\sigma_{LS}^2 = \sigma_T^2 - (\sigma_T^2)_{\text{fixed site}} \tag{6.2}$$

To do this, one needs to have recorded ground motion data at a number of sites that have experienced repeated earthquakes. The problem is that there are only a few sites which have ground motion data from a number of earthquakes. Some data sets from UNE also exist which can be very useful as travel path and source effects can be limited. Then, smaller data sets can be used to estimate the reduction in uncertainty achieved by fixing the site relative to a fixed source and travel path.

In addition to the reduction in the uncertainty in our predictions (at a fixed site), we would expect that the median prediction might increase or decrease depending on the actual conditions at any given site. Finally, we might expect the predicted spectrum to have more "character", i.e., have local maximums and minimums relative to a general data set.

As noted, ground motion recorded from UNE can shed considerable light on this question. Lynch (1971) used data recorded at thirteen stations from fourteen UNE to study this problem. Source and travel path variations for each site were minimized by restricting the data to only UNE located on Pahute Mesa. The level of ground motion was low so that the nonlinear effects in the soil at the sites should be small. The sites were located from 52 to 170 km from Pahute Mesa. Lynch (1971) analyzed this data using a <u>covariance</u> analysis that explicitly included each site rather than a typical regression analysis. Briefly, covariance analysis relates a component of spectrum SV at period i recorded at K stations to the yield of events detonated in a restricted area. The statistical model has the form

$$SV_{ik} = A_{ik}W^{Bi} \qquad \begin{cases} i = 1, 2, \ldots, \text{No. of Periods} \\ k = 1, 2, \ldots, K \end{cases} \tag{6.3}$$

Covariance analysis determines for each component at each period, i, a yield scaling exponent B_i, the standard deviation of the estimate σ_i, and K sets of amplitude coefficients A_{ik}. The amplitude coefficients A_{ik} implicitly reflect the average distance from the event area to the recording station, local station amplification and any factors associated with the transmission path. The parameter σ_i reflects variance due to neglected source parameters, transmission path factors for each station and differential changes in the average distance to each recording station.

A plot of the $\exp(\sigma_i)$ obtained from Eq. (6.3) using the restricted Pahute Mesa data set and the $\exp(\sigma_i)$ obtained from a general regression, Lynch (1969), of the form

$$SV = AW^{B_1}R^{B_2} \tag{6.4}$$

is shown in Fig. 6.1. Figure 6.1 shows that there is a considerable reduction in the uncertainty if the data is restricted to a fixed site.

The data set used by Lynch (1969) was somewhat different than the data set used by Lynch (1971); however, in both cases, the data were restricted to Pahute Mesa UNE. Thus, the reduction in uncertainty can be primarily attributed to fixing the site conditions as source and travel path variations are about the same. It is very interesting to note from Fig. 6.1 that the uncertainty associated with the model described by equation 6.4 is much larger than generally associated with typical earthquake data sets. This is surprising because we have what appears to be a rather homogenous data set compared to typical earthquake data sets. However, a closer look at the make-up of the stations involved indicates that a relatively large percentage of them consist of near-by pairs exhibiting wide variations in response (as illustrated in Fig. 6.2 for one pair of stations, one station being located on a rock outcrop and one located on a nearby soil deposit.)

Lynch and Williams (1972) performed a covariance analysis on data obtained from 22 UNE located at Yucca Flat. Figure 6.3 compares the computed $\exp(\sigma_i)$ of the Yucca Flat data set to the $\exp(\sigma_i)$ of the Pahute Mesa data set. It is seen that they are very similar.

The values of the $\exp(\sigma_i)$ shown in Fig. 6.3 primarily measure the uncertainty introduced by modest changes in travel path as source and site effects have been held constant. From Fig. 6.3 it is seen that σ_i varies as a function of the period but is approximately constant at a value of 0.4. This value of $\sigma_i \sim 0.4$ is larger than the value of σ_{LnSv} observed in Section 4.

This is not surprising because the variations in the travel paths of the data used in Section 4 were smaller than the variations in the travel paths for the data used by Lynch.

Note, however, that both data sets represent rather limited variations in travel paths. Thus for earthquakes we could expect that the uncertainty introduced by travel path alone could be much larger. In addition, the increase in the uncertainty introduced by the radiation patterns of the earthquakes could also be very significant.

If a given site has experienced a number of earthquakes for which measurements of the ground motion are available, then it is possible to determine if the site generally amplifies ground motion relative to typical correlations such as those developed by McGuire (1978) and Joyner and Boore (1981). It is also possible to determine if the data at a given site are less dispersed than the more general data set for a number of sites. The difficulty is to assess the role that site response factors play relative to source and travel path factors. Source and travel path variations are extremely important. It is difficult to account directly for these factors other than by the simple approach of grouping the available data so that these factors are minimized.

The El Centro and Ferndale sites have the most complete data sets. Both of these sites are deep soil sites, although the overall soil depth is greater at El Centro than at Ferndale, where the soil is somewhat stiffer than at the El Centro site.

Bernreuter (1979b) used the data recorded at these two sites to compare the peak acceleration to the mean predicted by typical correlations among peak accelerations, site type, earthquake magnitude, and distance. For his comparison, he used the correlation developed by McGuire (1978) as representative. For soil sites, McGuire determined that

$$a = \frac{24.5 \exp\left[0.89M\right]}{R^{1.17}} \; ; \; \sigma_{1na} = .62 \, , \tag{6.5}$$

where

M = Local Richter magnitude

R = Distance from energy release

σ = Standard deviation

Figure 6.4 shows a comparison of the recorded acceleration at the El Centro and Ferndale sites normalized by exp 0.89M as a function of R. Also shown, are the median normalized line given by Equation 6.5 and the $\pm$ one-sigma lines. It is evident from this figure that consistently higher-than-average peak accelerations are recorded at the Ferndale site during the earthquakes, suggesting some site amplification at the Ferndale site as compared to say the El Centro site.

In order to quantify the dispersion of the data, separate regression analyses were performed for the data at the El Centro and Ferndale sites. The results of these analyses are:

$$\ln(a) = 4.82 + 0.52M - 0.83 \ln(r)$$

$$\sigma_{\ln a} = 0.39$$

for Ferndale, and

$$\ln(a) = 4.12 + 0.53 = 0.85 \ln(r)$$

$$\sigma_{\ln a} = 0.67$$

for El Centro.

There is significantly less scatter to the data at the Ferndale site than to the data at the El Centro site. The data at El Centro have about the same standard deviation as the more general data set used by McGuire, which included eight El Centro records and nine Ferndale records. Although the distances R for the Ferndale events are less certain than for El Centro events, no systematic error appears to exist, and it is unlikely that the errors are large enough to significantly change the conclusion reached above.

We also examined the Oroville data set. We performed regression analysis on the data in Seekins and Hanks (1978) and used the resulting relation to normalize the recorded peak accelerations. Figure 6.5 shows the ratio between the measured and estimated peak acceleration as a function of distance. In Fig. 6.5, the ratios for the Johnson Ranch data are denoted by the symbol J and for the ratios for the Oroville Airport data by the symbol A. The $\pm$ 1-sigma limits are also shown. It is seen that the observed peak acceleration recorded at the Johnson Ranch site is consistently high (6 out of 8 data points above the +1- sigma limit) and the observed peak acceleration recorded at the Oroville Airport is consistently low (9 out of 11 data points below the median). However, there is a much greater spread to the airport data than to the ranch data.

We also examined how the error term might be reduced by going from the general data set to a fixed site. We only examined (for the fixed site cases) those sites at which eight or more earthquakes were recorded. We obtained

σ_{lna} = 0.60 General data set
0.42 Johnson Ranch Station
0.26 Station 5
0.61 Station 1
0.36 EBH Station
0.7 Airport

These results are in reasonable agreement with the results for the Ferndale, and El Centro data. There is considerable scatter to the data making it difficult to estimate σ_{Ls} . The "stiffer sites" (Ferndale, Johnson Ranch, Station 5, EBH) show lower $\sigma_{\ell na}$ value than deep than softer stations (El Centro and Station 1).

To estimate σ_{Ls} we need σ_T for fixed sites. We approximated this by defining a site category for each site for the Oroville data set and used the model

$$\ln a = C_1 + \Sigma\, CS_i\, S_i + C_2 M + C_3 \ln R \tag{6.6}$$

where

C_i and CS_i are constants to be determined by a regression analysis

S_i are site categories, S_i is one if the data point was obtained at site i and is zero otherwise

R = hypocentral distance

M = local magnitude

Our fit to the data resulted in a $\sigma_T = 0.52$ for "fixed sites". Using $\sigma_T = 0.60$ from our general fit to the data without site categories we estimate that

$$\sigma_{LS}^2 = \sigma_T^2 - (\sigma_T)^2_{\text{(fixed site)}} \sim (.60)^2 - (.52)^2$$

$$\sigma_{LS} \sim 0.30$$

McCann and Boore (1983) and McCann (1983) attempted to sort out the various contributions to the uncertainty in the prediction of ground motions using data from the San Fernando Earthquake. McCann (1983) concluded that local site effects contributes about 30% of the uncertainty, i.e. $(\sigma_{\ln a})_{LS} \sim 0.23$ based on a few sites. This result is consistent with our results.

7. UNCERTAINTIES INTRODUCED BY THE RANGE FOR M AND R USED

In a typical regression analysis approach both the magnitude and distance variations are explicitly accounted for. This is not the case in SSSP approach where a range of distances and magnitude are included. Infact it should be noted, that one of the important features of the SSSP approach is to include the potential uncertainties in magnitude and location of the SSE in the predicted spectrum. However, if the uncertainty introduced by the range for M and R are much larger than the uncertainties introduced by the other sources of uncertainty, then it would seem necessary to rethink the ranges for M and R used. Recall from Section 2 that the "target" is typically and earthquake of $M_L = 5.3$ (for MM VII SSE) and $M_L = 5.8$ (for MM VIII SSE) with an average distance of 15 km.

A simple estimate of the uncertainty introduced in our estimate of the ground motion using the SSSP approach can be obtained by using the results of typical regression analysis of data sets. e.g., Joyner and Boore (1981) found that

$$\log_{10} a = C_1 + 0.25M - \log_{10} r - 0.0026r$$

$$\text{where } r^2 = d^2 + 53$$

The term $-.0026r$ is relatively unimportant for the distance range out to 25 km. Thus the potential multiplicative error, E^* in the actual value of the acceleration at high frequency end of the estimated spectra obtained by use of the SSSP approach for distances (d) of 0 to 25 km with an average distance of 15 km is of the order

$$0.64 < E^* < 2.3$$

That is, the spectra from the most distant earthquake used will be about a factor of 0.6 smaller than the "target" and the very near fault data might be a factor of 2 larger than the target. These variations are of the same order or on the average smaller as the uncertainties introduced by the other sources of uncertainty.

The maximum potential uncertainty introduced by the range in magnitudes used ($\pm$ 0.5 units) is on the order of $10^{(.125)}$ or a factor of 1.3. Clearly the actual uncertainty introduced by the distribution of earthquake magnitudes used is smaller and less than the other uncertainties.

8. APPLICATION OF SSSP APPROACH

In Sections 4-7 we attempted to estimate the contribution to the overall uncertainty from each of the three major categories of the sources of uncertainty defined in Section 1. It can be concluded from the results presented in Section 4-7 that each of the three categories defined in Section 1 introduces significant uncertainty into our estimates of the ground motion. We now want to examine what the implication of Section 4-7 are in the application of the SSSP approach.

No single category appeared to contribute more to the uncertainty than any other. However, in the application of the SSSP approach the local site's geology must be treated differently than the other categories because the estimate is for a fixed site and hence, unlike the other parameters, the local geology is not a random variable. Thus we have two conflicting problems in the selection of data. First, because of the fixed nature of the site, we would like to limit our data selection to very similar site conditions. However, our results also show that we can expect large variations in the observed ground motion at any fixed site because of the uncertainties introduced by source travel path effects. This means that we need a large sample to be sure that we have adequately defined the estimated median and 84th percentile spectra. In general, such a sample can only be obtained by relaxing the criteria used to determine if a site is "similar" to the target site because at many accelerograph sites we only have very limited information about the parameters needed to determine if the accelerograph site matches the site for which the prediction is to be made. In fact, when trying to develop SSSP for specific sites one has problems in selecting a suite of appropriate records if the site is other than "deep" soil or weathered rock.

In Fig. 8.1a we compare the SSSP for a deep soil site to that for a weathered rock site for a SSE level of MM VII and in Fig. 8.1b for a SSE level of MM VIII. The major difference between the weathered rock case and the deep soil case is at periods longer than 0.2 seconds where the deep soil SSSP is higher than the rock SSSP. There does not appear to be much difference between the deep soil and weathered rock SSSP at short periods. Some care was taken in the selection of the records to insure that the soil records were all obtained on relatively deep (deeper than 200 ft) soil. The rock records have a range of rock types, but mostly soft weathered rock.

Figure 8.2, shows σ_T for the rock and deep SSSP plotted in Fig. 8.1a. Because of the large uncertainties in estimated SSSP, it is of some interest to see if the range of sites used had a significant impact on σ_T. One way to get a handle on this is to develop SSSP using data from only one site. Figure 8.3 shows σ_T for a SSSP suite for the Oroville Airport based on six earthquake all with $M_L = 4.7 \pm 0.5$ at an average distance of 13 km. The value of σ_T is very similar to that shown in Fig. 8.2 suggesting that the variation in sites used to develop the SSSP shown in Fig.s 8a and b has not significantly increased σ_T over the contribution for source and travel path uncertainities.

There can be considerable departure from the relatively smooth spectral shape obtained for deep soil and weathered rock sites as is illustrated in Fig. 8.4 where we plot SSSP for the Oroville Airport (deep soil) and the Johnson Ranch (shallow soil). The median SSSP for the airport and Johnson Ranch were developed using only data recorded at the respective sites. Only two earthquakes were common to both data sets, thus the average magnitude and distance of the two SSSP are different. For the Oroville Airport the average M is 4.7 and the average R is 13 km. For Johnson Ranch the average M is 4.4 and the average R is 12 km. The data shown in Fig. 8.4 suggests that there is considerable amplification of the ground motion acceleration at Johnson Ranch relative to the Oroville Airport. This is in general agreement with Fig. 6.5.

An analysis using simple one-dimensional soil column type models e.g., Bernreuter et al. (1986), shows that sites with soil depths less than 200 ft tend to amplify the ground motion as compared to weathered rock or deep soil sites. The analysis also shows that this amplification should be observed even if data from a suite of sites are used, although the uncertainty will be significant. The computed amplification varies with period with a maximum median amplification factor of approximately two relative to a rock or deep soil base case. This indicates that, until more data becomes available at shallower soil sites so appropriate SSSP suites can be formed, considerable care should be taken in the extrapolation of any data set to yield an estimate of the ground motion at such sites.

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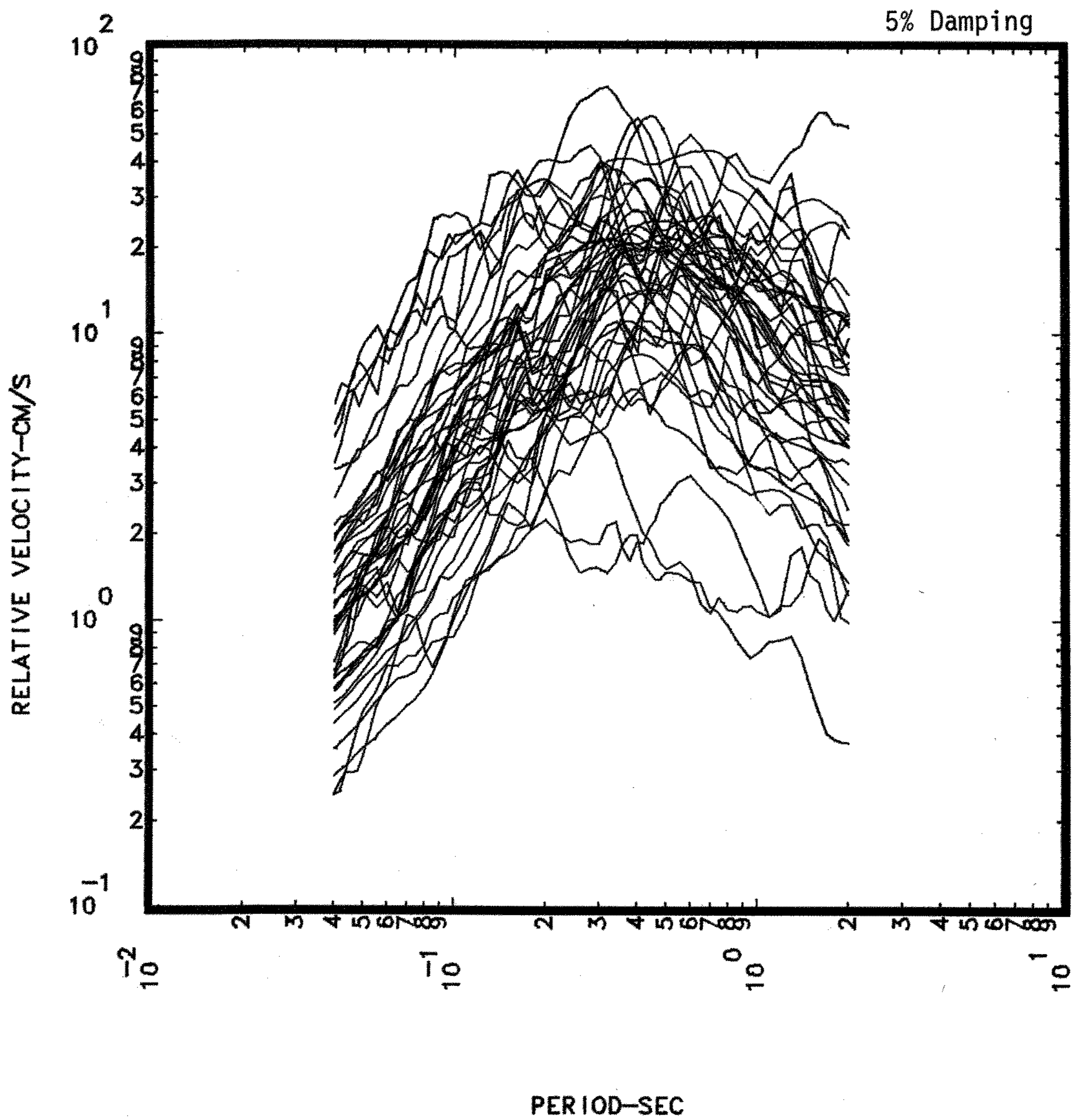

Fig. 3.1.a. Over plot of all of the suite of spectra selected to define the SSSP for a deep soil site having a SSE of MMVII.

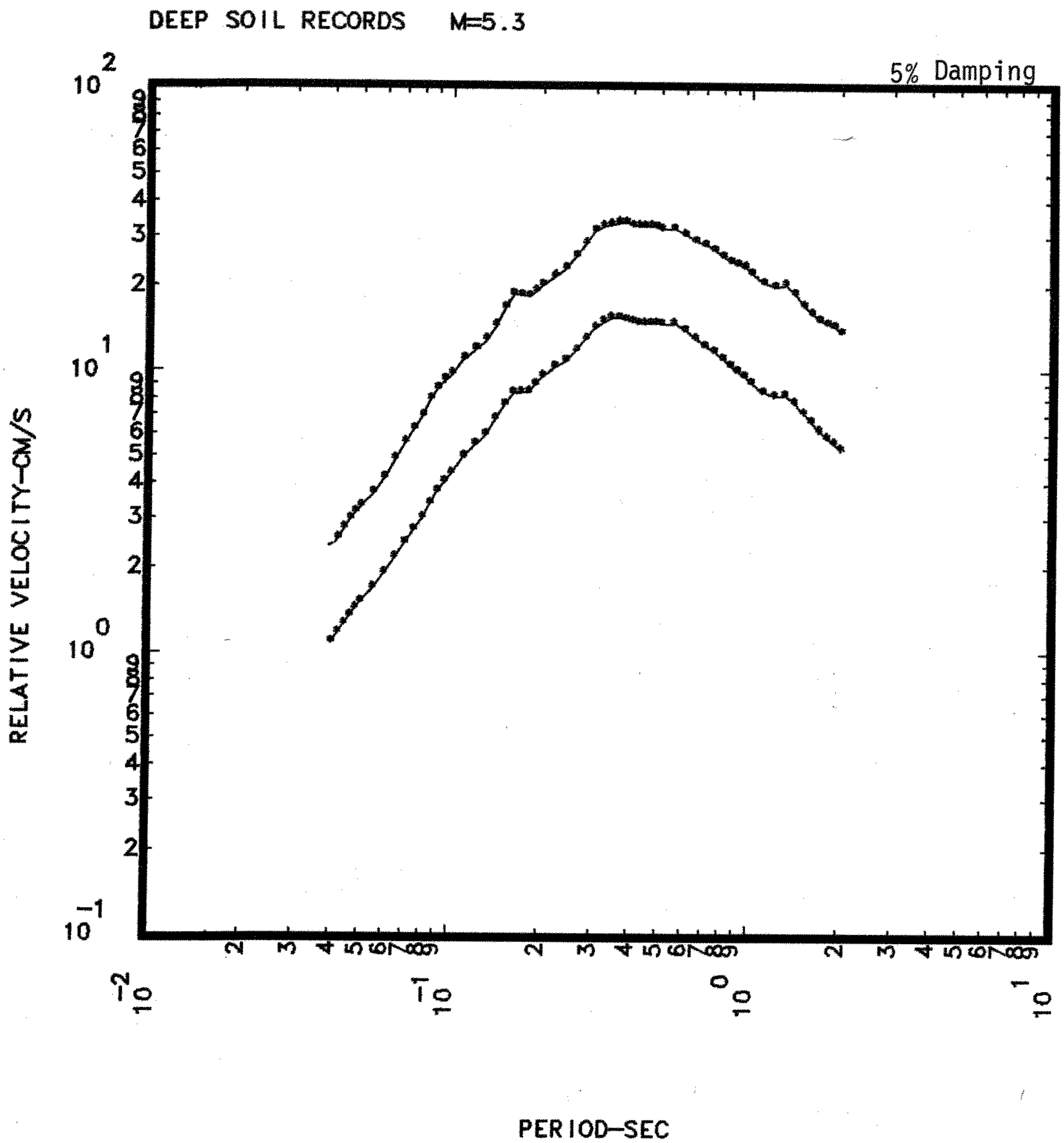

Fig. 3.1b. The Median and 84th percentile spectra of the suite of records shown in Fig. 3.1a.

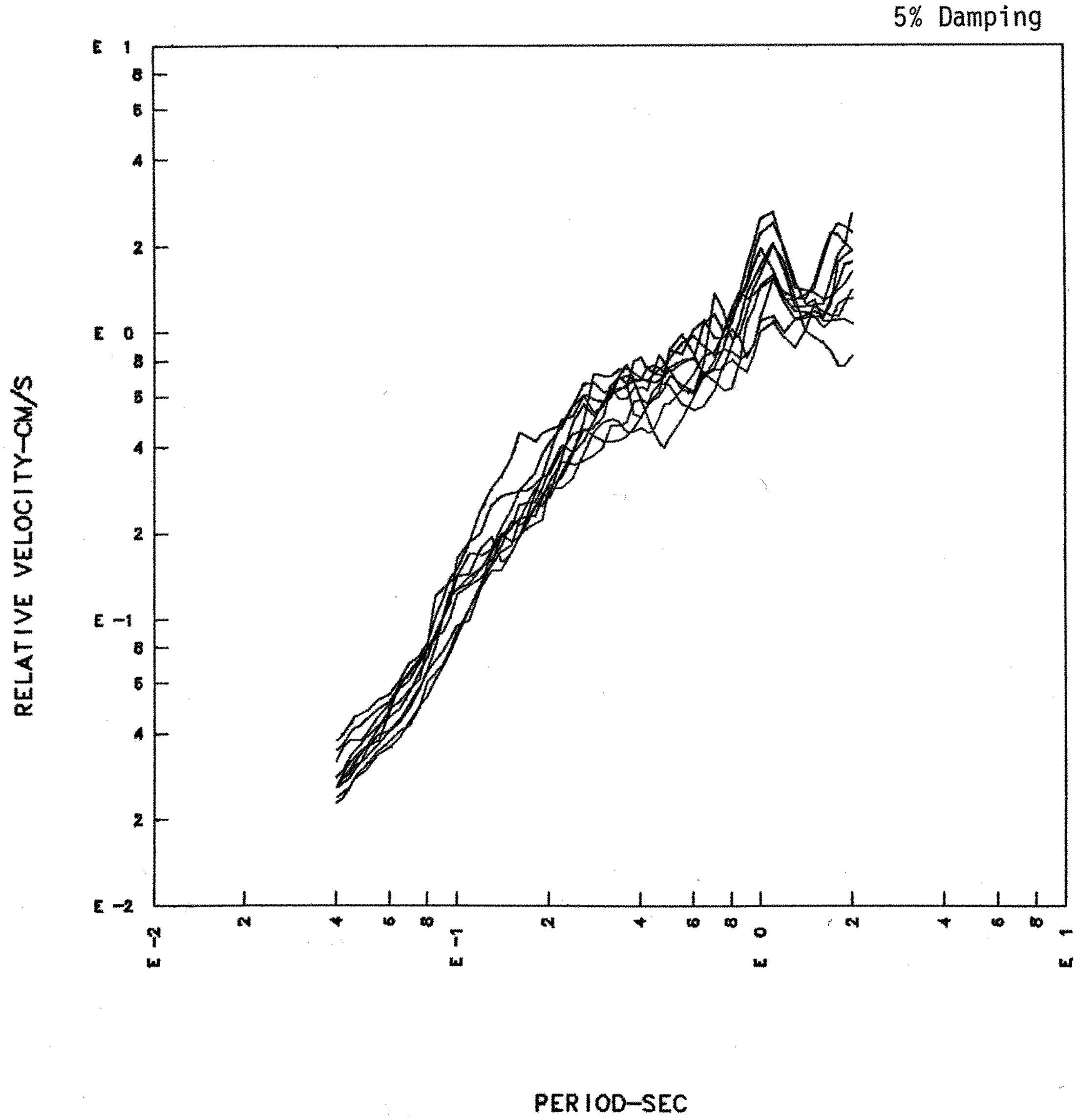

Fig. 4.1. Over plot of both radial and tangential components of 5 UNE recorded at a single deep soil station. These 5 UNE all had a yield of ±5 kilotons of each other at a distance of 55 ± 2 km from the recording station.

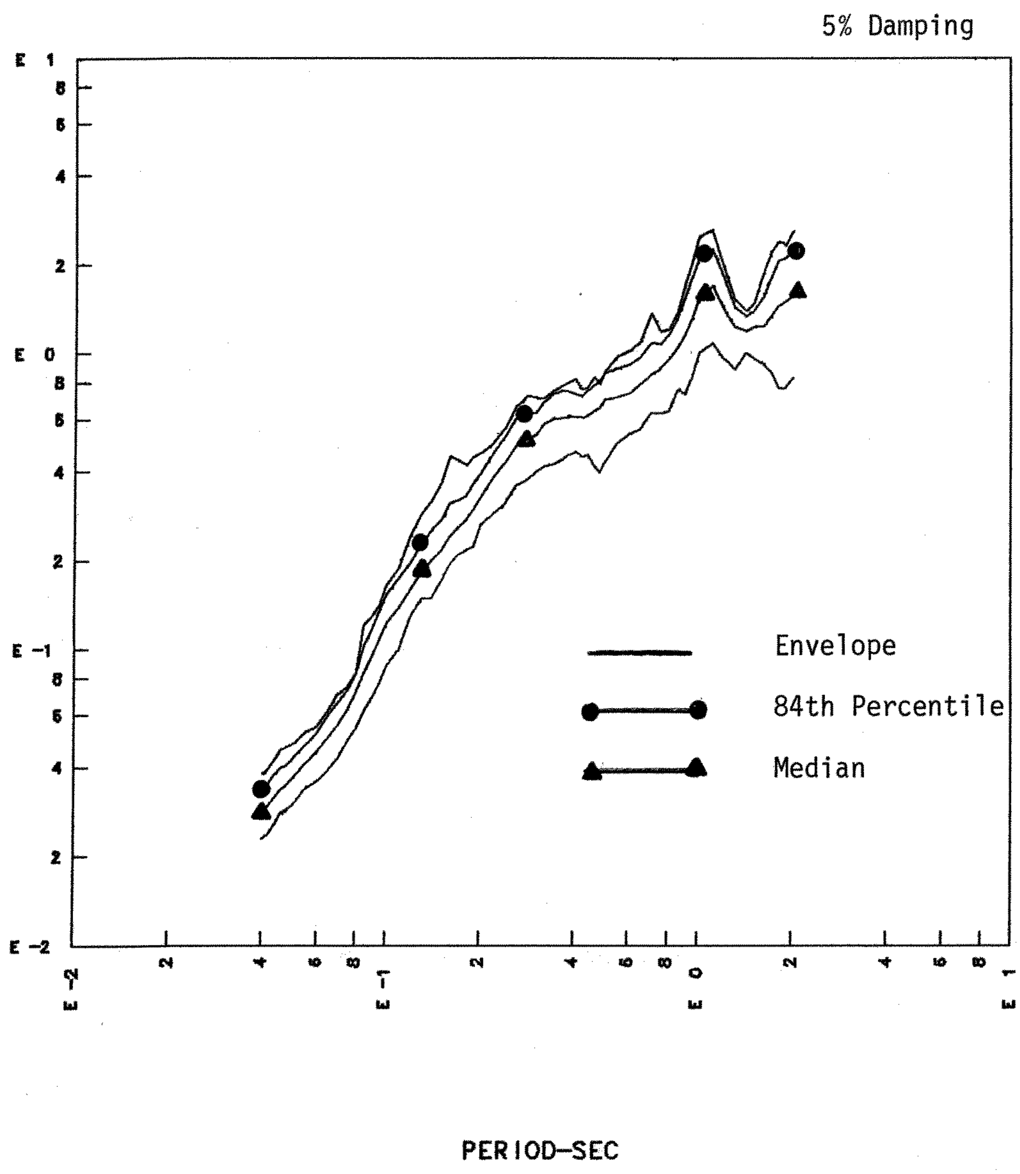

Fig. 4.2. The envelope, median and 84th percentile spectra of the suite of spectra shown in Fig. 4.1.

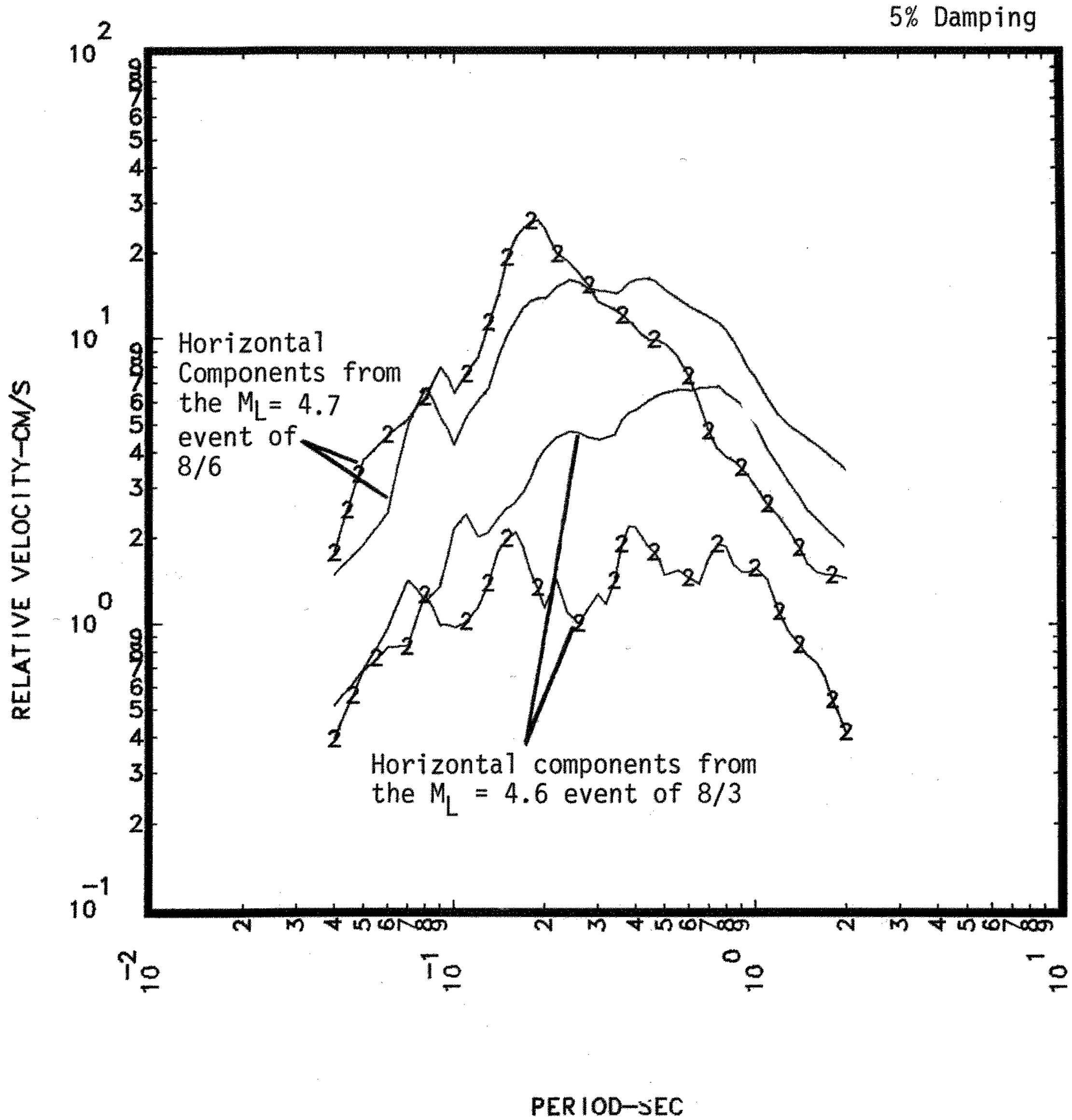

Fig. 5.1 Comparison of the spectra from two earthquakes located within a few kilometers of each other with similar M_L and recorded at the Oroville Airport.

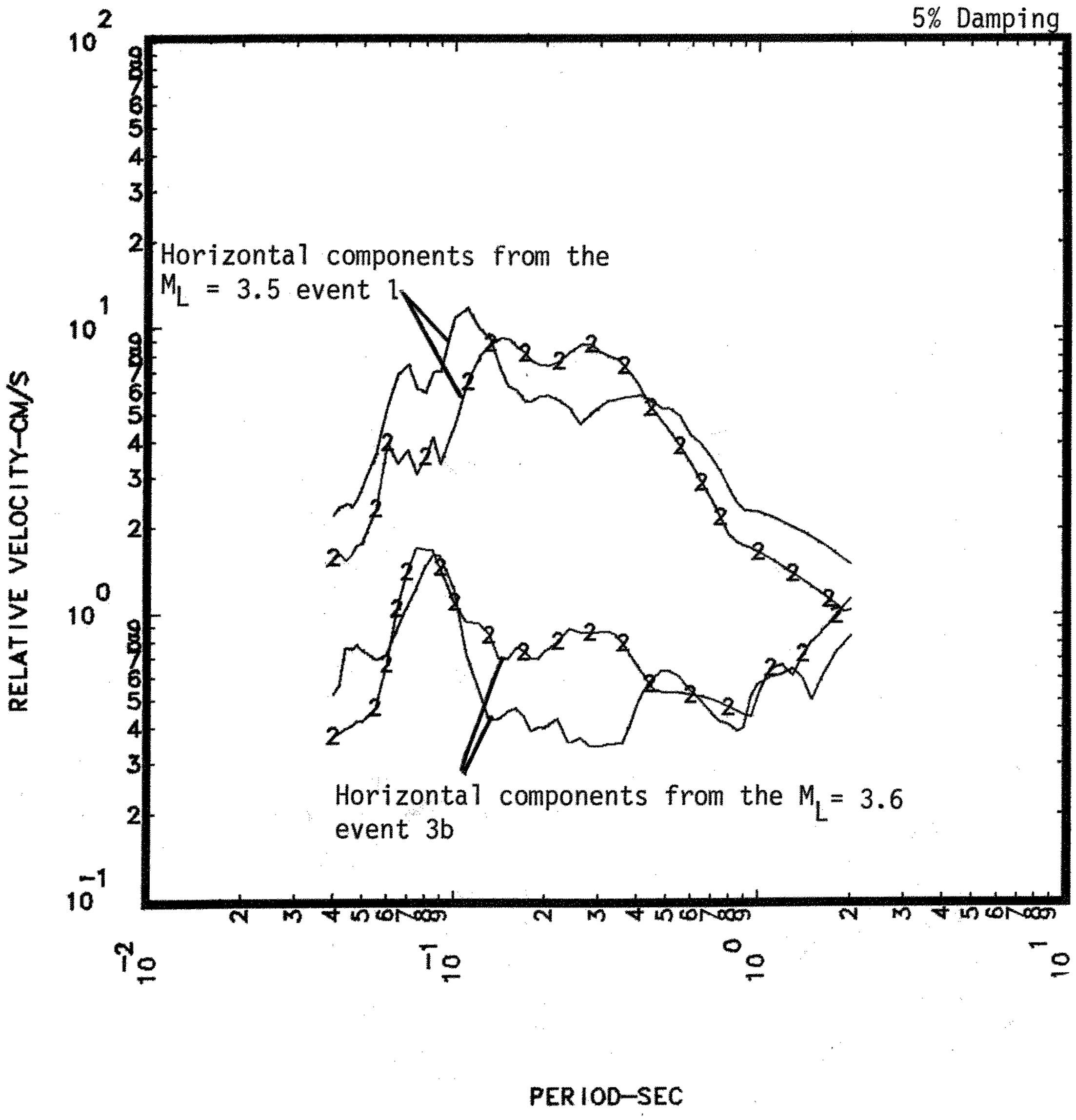

Fig. 5.2 Comparison of the spectra from two earthquakes located within a few kilometers of each other with similar magnitudes and recorded at the Brawley Airport. Event numbers and magnitudes from Johnson and Hanks (1976).

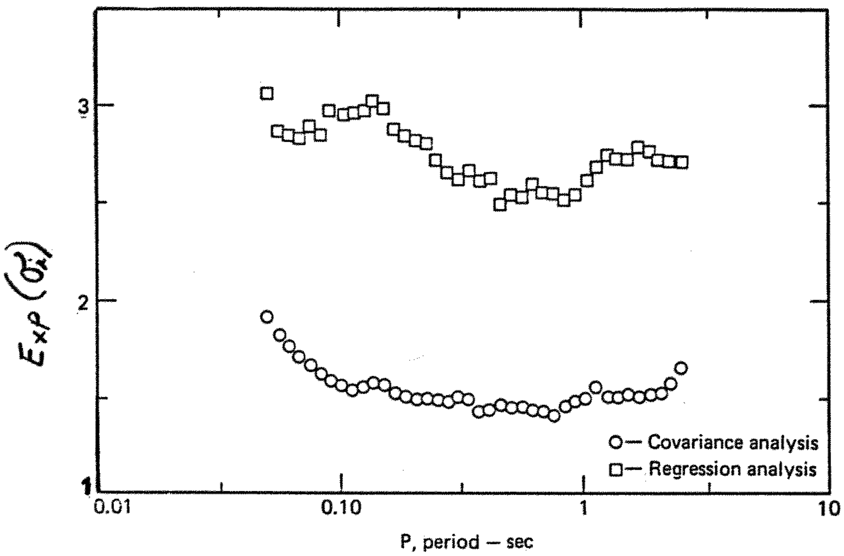

Fig. 6.1 Comparison of Exp (σ_i) obtained from a covariance analysis accounting for each site to that obtained by Lynch (1969) which does not include site factors.

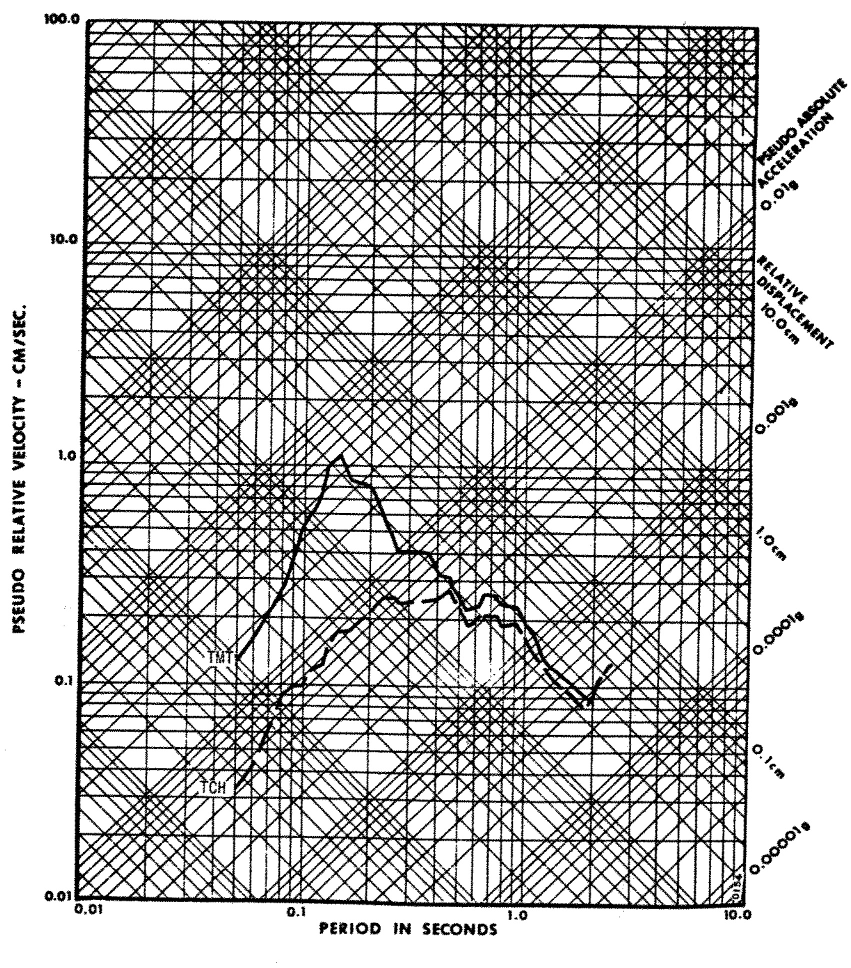

Fig. 6.2 Comparison of the radial component of the spectra from two nearby stations TMT (Tonopha motel, shallow alluvium) and TCH (Tonopah church, rock).

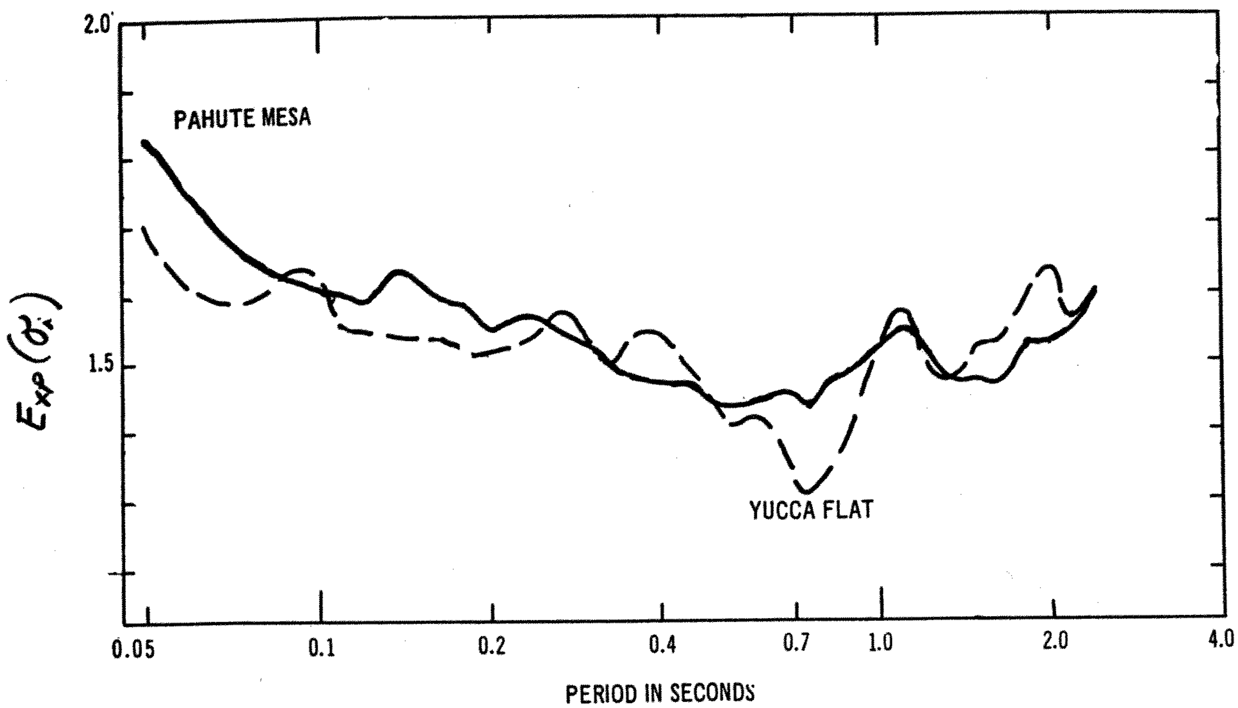

Fig. 6.3 Comparison of Exp (σ_i) for the covariance analysis for Pahute Mesa UNE (Fig. 6.1) to that for Yucca Flat UNE.

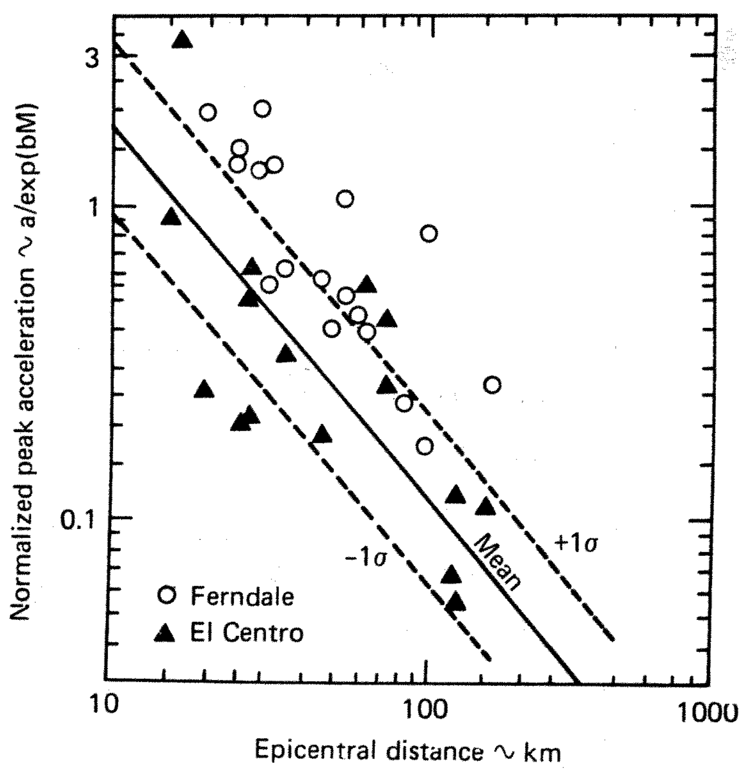

Fig. 6.4 Normalized peak accelerations recorded at El Centro and Frendale sites compared with McGuire's (1978) correlation.

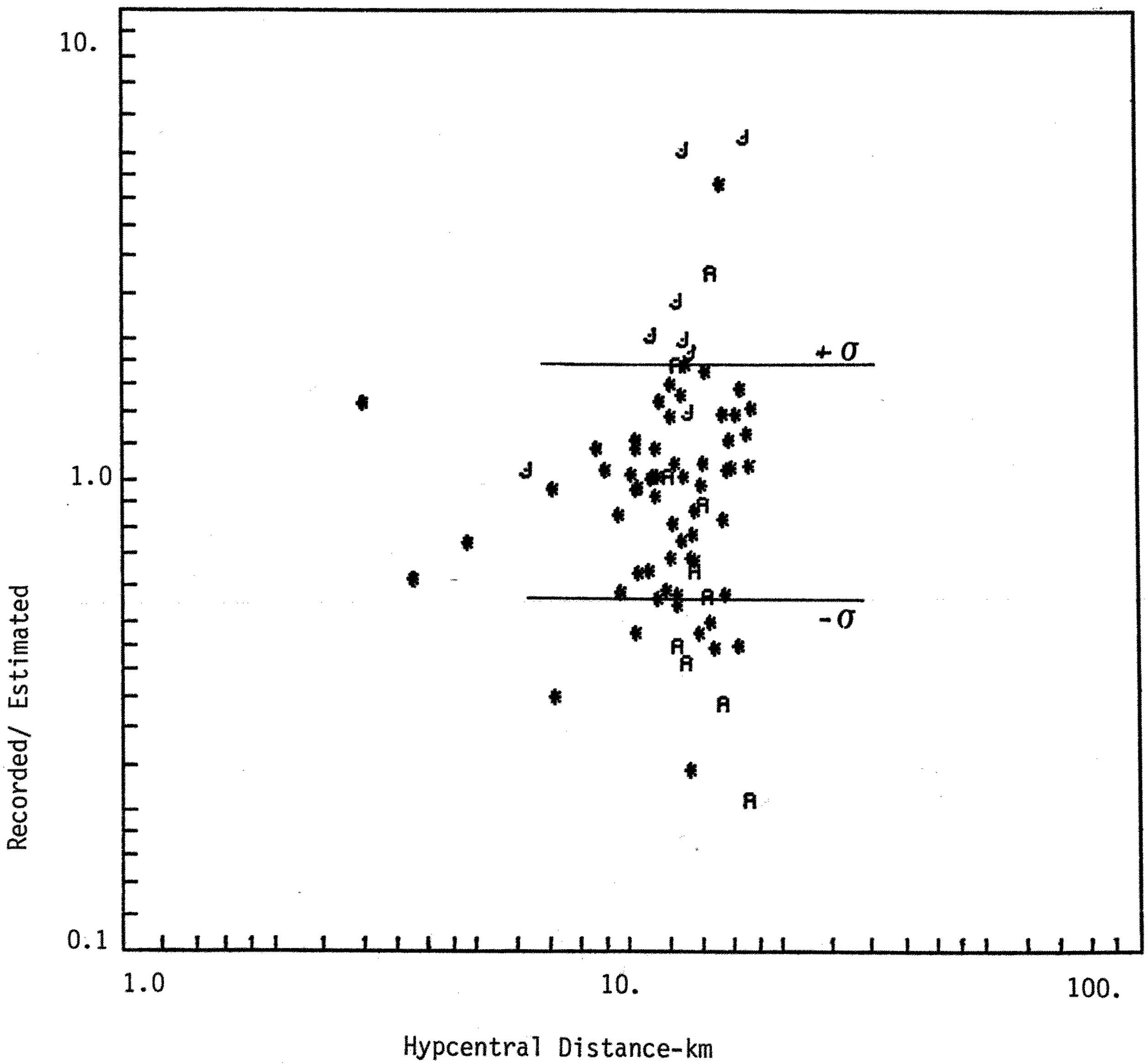

Fig. 6.5 Normalized peak horizontal acceleration for the Oroville data set. The data for Johnson Ranch are denoted by J and for the Oroville Airport by A. The data from the other stations are denoted by *.

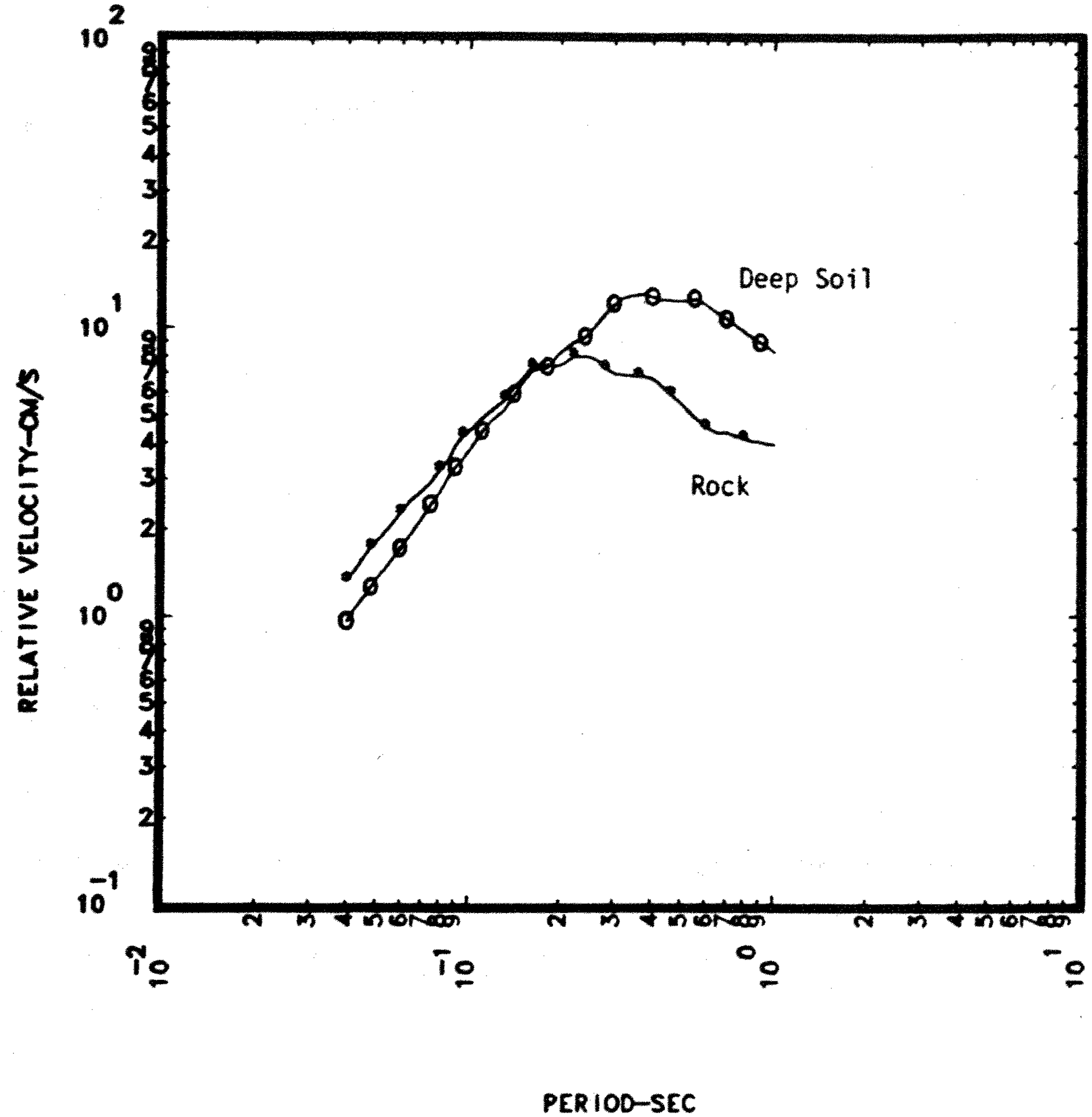

Fig. 8.1a Comparison of the median SSSP for a deep soil site to a rock site for the MM VII case.

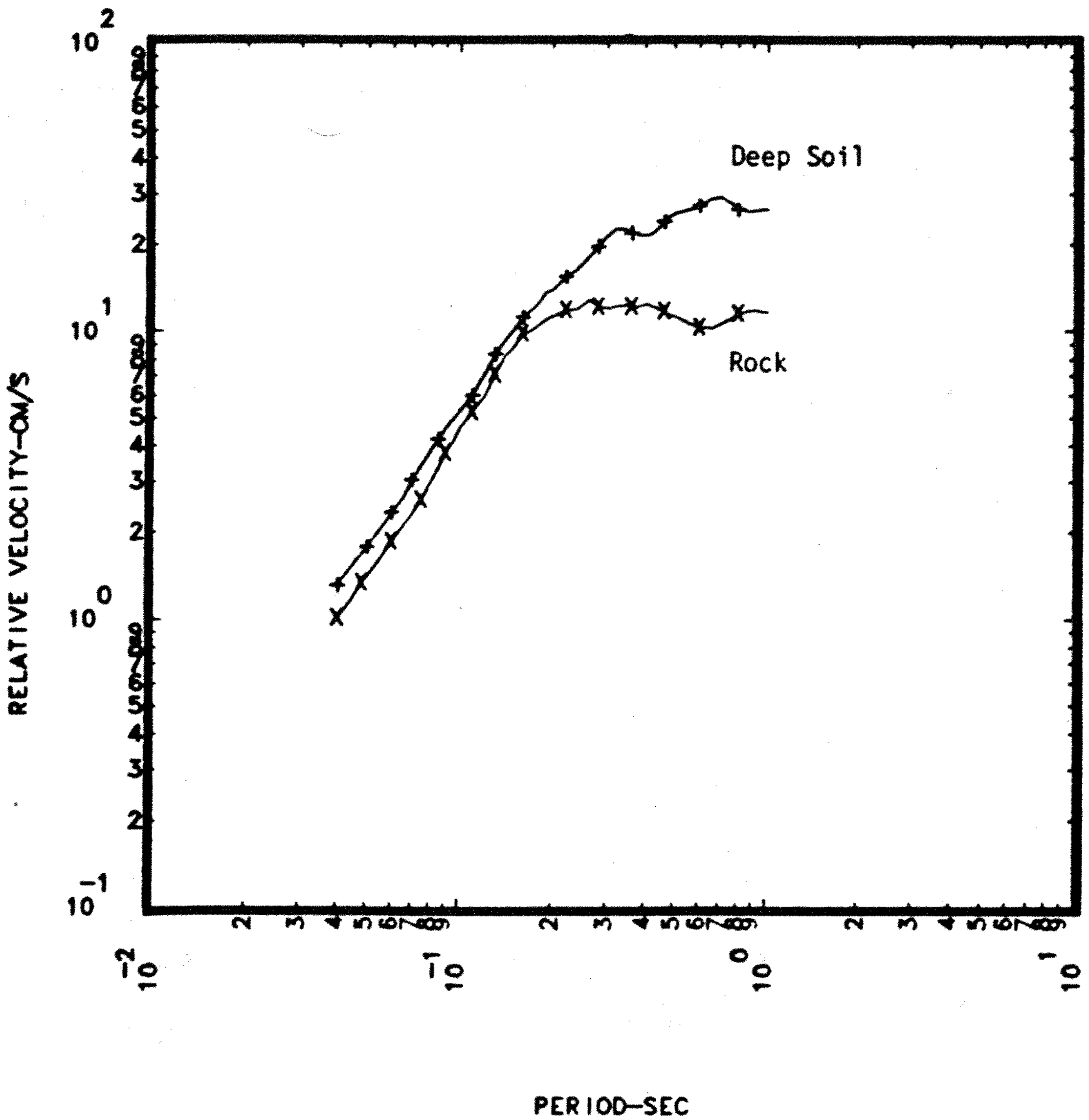

Fig. 8.1b Comparison of the median SSSP for a deep soil site to a rock site for the MM VIII case.

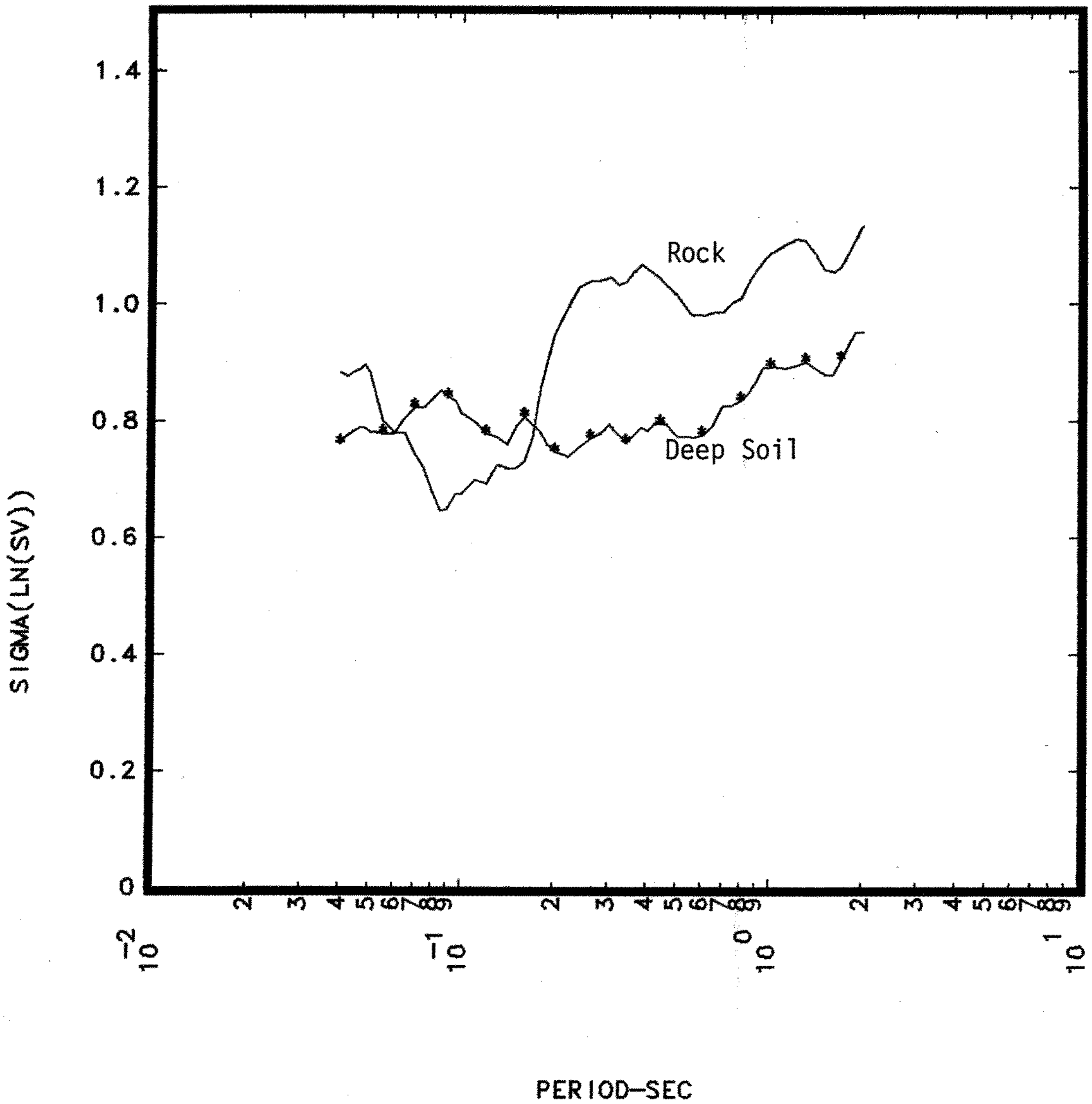

Fig. 8.2 Uncertainty of the estimated SSSP for rock and deep soil sites for a SSE of MMVII as measured by the standard deviation of the assumed lognormal distribution of the spectral velocity S_V.

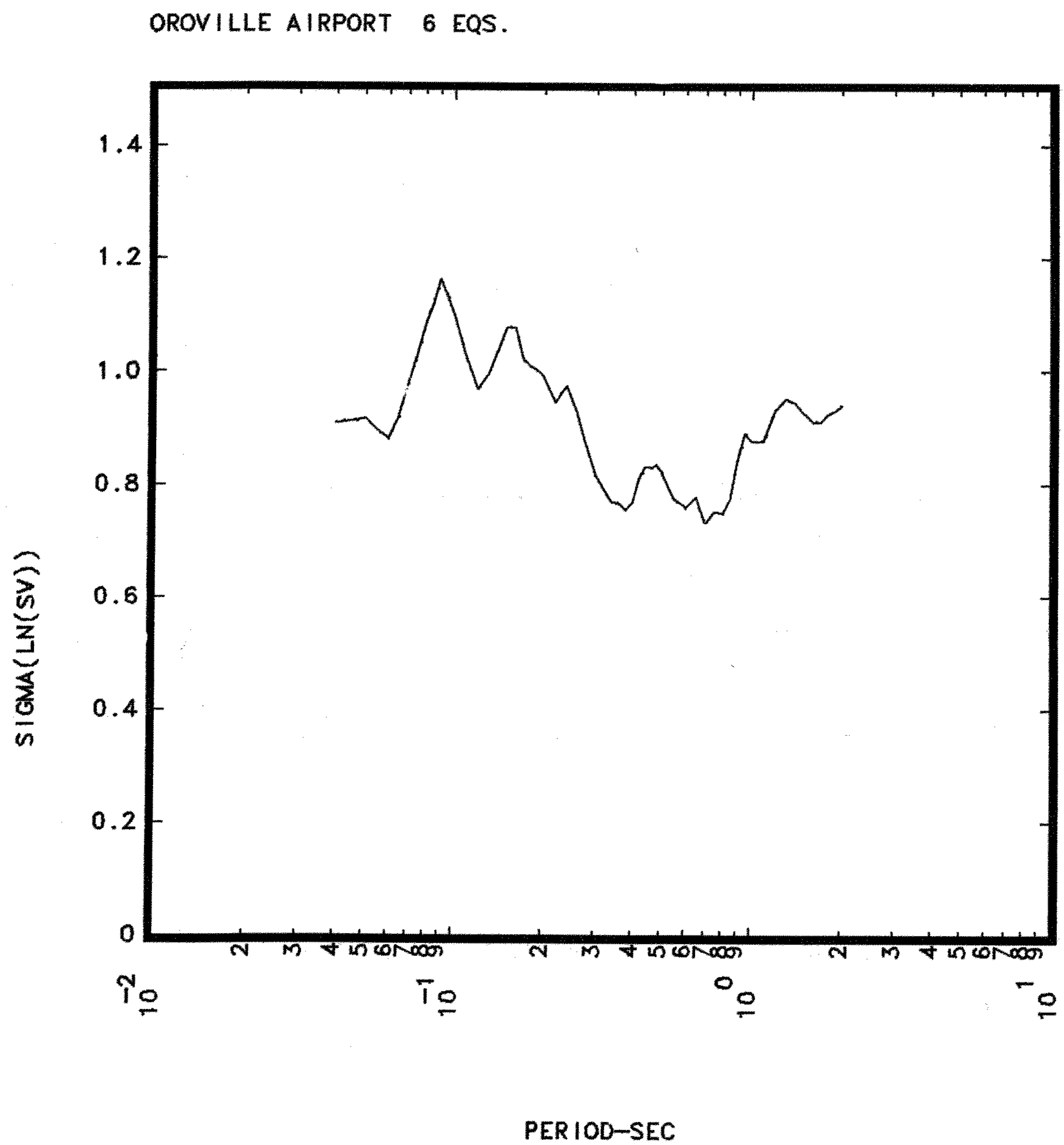

Fig. 8.3 Uncertainty of a SSSP for the Oroville Airport based on six earthquakes of $M_L = 4.7 \pm 0.5$ units recorded at the Oroville Airport as measured by the standard deviation of the assumped lognormal distribution of the spectral velocity.

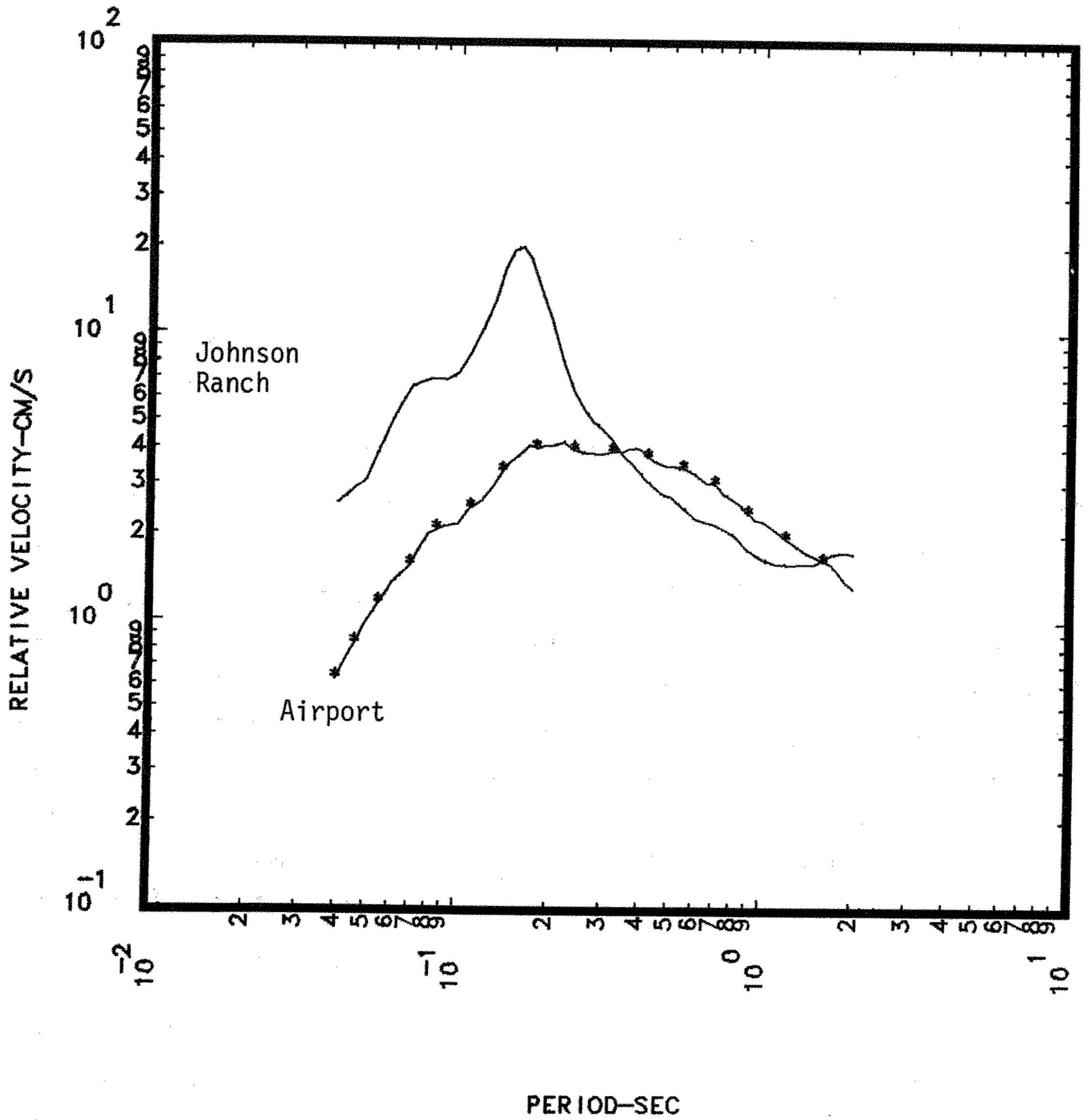

Fig. 8.4 Difference in spectral shape between a deep soil site (Oroville Airport) and a shallow soil (Johnson Ranch) using only data recorded at the respective sites. Note that only two events are common to both sites.

RANDOM-VIBRATION MODELS OF FREE-FIELD SEISMIC GROUND MOTION FOR SOIL-STRUCTURE INTERACTION

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INTRODUCTION

Various methods are used to estimate seismic ground motion for soil-structure interaction (SSI) analyses at nuclear power plants, depending on the specific application. Where data are abundant, as on the west coast of the US, empirical procedures have been applied to predict various measures of ground shaking. To estimate ground motions for magnitudes and distance ranges that are not well-documented with data even on the west coast, numerical procedures that account for characteristics of the fault rupture and the wave travel path have been developed and used. In regions such as eastern North America (ENA) where recordings of strong seismic shaking are few, other methods have been applied with some success. These include semi-theoretical procedures that combine available data and theoretical assumptions, conversions from Modified Mercalli intensity (MMI) data, and site-specific spectra based on recordings from other regions.

Both in regions where data are sparse and where they are abundant, "random vibration" (RV) representations of ground motion have been developed and applied in recent years with considerable success. This method characterizes seismic shaking as a stochastic process with a defined frequency content and specific assumptions about temporal behavior, from which all interesting measures of ground motion (peak parameter values, response spectra, and instrumental magnitude) can be derived with the application of RV concepts. In regions where data are sparse this method avoids the large uncertainties inherent in using MMI data or in extrapolating from the few empirical data; in the western US this method provides a self-consistent theoretical representation of ground motion and allows estimation of large magnitude, near-source ground motions for which recordings are rare.

This paper reviews some of the recent applications of RV methods and indicates some areas in which further work is appropriate. The method is also evaluated as a tool for specifying bedrock input motion for SSI analyses.

RANDOM VIBRATION MODEL

The basic RV model specifies the displacement Fourier spectrum of seismic shear waves at a site, assuming a long-period level proportional to the seismic moment of the earthquake, and assuming that amplitudes decrease as ω^{-2} above a corner frequency f_o (see Figure 1). For ground acceleration this means that the Fourier spectrum is essentially flat above the corner frequency up to some higher frequency that is determined by anelastic attenuation or by a maximum frequency f_{max} caused by the source or site conditions. With the earthquake source represented as a point, the ground acceleration is characterized by this Fourier spectrum, a Gaussian distribution of amplitudes, and the assumption of stationary characteristics over a finite duration equal to the duration of fault rupture. The expected maximum value of this stochastic process (the peak ground acceleration) and its derivative (peak ground velocity) are easily obtained. The relevant background and equations are given in McGuire and Hanks (1980), Hanks and McGuire (1981), and Boore (83).

With the specification of the ground acceleration characteristics, simple RV techniques are used to estimate other quantities of shaking. The spectral density function (SDF) of a linear oscillator's response, for example, is just the squared absolute value of the oscillator's transfer function times the SDF of the ground motion input. The peak oscillator response is easily calculated from its SDF and the duration of shaking, accounting for any non-stationarities in the first few cycles of response. Similarly, the peak response of seismographic instruments can be calculated, from which conventional values of magnitude can be derived (these are not a part of the specification of source characteristics, which include only seismic moment and dynamic stress drop). Equations describing these calculations are given in Boore (1983) and McGuire et al. (1984). Thus, the theory provides a self-consistent representation of all important characteristics of high-frequency strong ground motion, including Fourier and response amplitudes, duration of shaking, peak motion parameters, and magnitude of the earthquake.

APPLICATION IN CALIFORNIA

The basic point-source model has been found to be useful in explaining most recorded ground motions during California earthquakes (that is, it provides estimates that are consistent with observations). Figure 2 shows a comparison of model estimates (shown as dots) of peak acceleration and velocity, with results from empirically-based equations derived from California data using f_{max} = 15 Hz. The agreement above magnitude 5 (where the empirical equations were fit) is good. Figure 3 shows comparisons between predicted and observed response velocities for eight California earthquakes for 5 natural frequencies and 5% damping. Again the agreement is good for frequencies up to 10 Hz. For both sets of comparisons, a magnitude-independent dynamic stress drop of 100 bars was used, along with the seismic moment, to specify the seismic source for the model calculations.

For sites that might be affected by a nearby large earthquake, a point-source assumption is not appropriate; the dimensions of the causative fault and the characteristics of the rupture process as a function of time must be taken into account. Figure 4 conceptually illustrates how this can be done, using a one-dimensional representation of the energy release and accounting for the changing distance of the energy release from the site in time. Also, the change of arrival time of seismic waves at the site as a function of azimuth from the source is easily taken into account with this representation. Figure 5 shows that estimates of peak acceleration using finite source sizes agree with California data for M~6.5 at close distances for several choices of attenuation quality factor Q and energy depth d. Perhaps more interestingly, a model of this type can incorporate two corner frequencies, one related to the length of faulting and the other to the fault-displacement time function, and this representation predicts a magnitude saturation effect for high frequencies. Figure 6 shows this effect for sites very close (within 5 km) of a fault trace. The empirical verification of model predictions above the saturation level awaits the collection of data very close to large (M>7) earthquakes.

The RV model has achieved success in its ability to provide a simple, physically based explanation for high frequency, strong ground motion observations. The large majority of observations in California are in agreement with the model predictions. As with all models, improvements can be made as we understand more about the physical processes at work. In California several specific areas warrant attention for special cases. For large magnitudes the effect of magnitude saturation (e.g., Figure 6) should be further examined with alternative assumptions; model estimates are not consistent with distant observations from the Kern County earthquake, although these records may not be representative of a magnitude 7.7 shock. Alternative anelastic attenuation models (e.g. Anderson and Hough, 1985) should be examined to determine their potential effect on the predicted ground motion. The effect of a high impedence contrast in near-surface soils and rocks should also be investigated because this may have amplified most California recordings and would result in an overestimate of the dynamic stress drop (Boore, personal communication, 1985), although the predictions of ground motion would not be affected by this interpretation. The power of simple, physically based representations of ground motion such as the RV model is that these issues can be addressed directly; we need not await the collection of additional strong motion records, as is the case with empirically based techniques, to resolve these issues.

APPLICATION IN EASTERN NORTH AMERICA

In regions with few strong motion records the use of RV methods is particularly fruitful because the methods can account in a theoretical way for differences in source and crustal properties without reliance on the sparse data base. (The data are used to verify the model rather than to calibrate it.) The first application of this type in ENA (Atkinson, 1984) accounted for the low anelastic attenuation of ground motion with distance and the seismic energy at high frequencies (f_{max} ~ 40 Hz) typically associated with ENA earthquakes. More recent applications (Toro and McGuire, 1986; Boore and Atkinson, 1986) account, in addition, for the longer duration of high-frequency waves and for the slower geometric attenuation beyond 100 km. caused by the predominance of surface waves. Also, the response of the WWSSN seismograph is used to calculate an Lg magnitude for the predictions, since this is the magnitude scale commonly used in ENA.

Figure 7 shows comparisons of predictions and observations for 10 Hz response velocity for m_{Lg} = 5 and 6. The predictions are made for 50, 100, and 200 bar stress drops, and the observations are from accelerographs (primarily on soil sites--including river valleys--indicated by lower-case letters) and seismographs (primarily on rock sites--indicated by upper-case letters). All data are from ENA except those indicated by letter "D", which are from the Gazli earthquake, USSR, for which only an approximate magnitude and distance are available. Figure 7 and other comparisons presented in the above mentioned references indicate that the RV model in ENA explains most of the characteristics of ground motion observed in that region. The implication of these studies is that spectral amplitudes of seismic ground motion in ENA within 100 km of the source (the most important region for seismic hazard) attenuate in a manner very similar to California. A constant stress drop model also implies that the scaling of spectral amplitudes with source size is similar for the two regions.

As in California, an increased physical understanding of the ground motion process raises further issues that should be resolved. With respect to magnitude scaling, the RV estimates do not match all observations for midplate earthquakes, although the applicability of some of these data to ENA has been questioned. Examining the original seismograms and re-interpreting Lg magnitudes may resolve these questions. The effects of soft soils in ENA on ground motion is not well understood and deserves site-specific studies at locations where accelerograms have been obtained to further understanding of these effects. Related to the soil effects are the large observed felt areas and damage areas for ENA earthquakes; these need to be resolved in terms of the quantitative observations of ground motion that are available from numerous seismographs in ENA, as illustrated in Figure 7.

CONCLUSIONS

The RV method of estimating high-frequency strong ground motion offers advantages over empirically based alternatives in that estimation of ground motion where few data are available can be based on physically sound principles. The method is particularly appropriate for specifying the input to SSI analyses because it represents ground motion for hard rock or firm site conditions unmodified by soil response. Any amplification or de-amplification of ground motion can then be considered. Issues discussed above about the appropriateness of various assumptions used in the RV procedures point to the usefulness of the technique in isolating the critical physical phenomena that must be further studied in order to gain a better understanding of strong earthquake ground motion.

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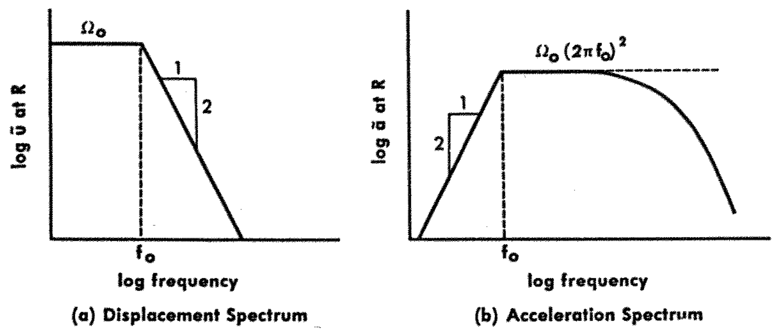

Figure 1. Fourier spectrum of ground displacement and acceleration (after McGuire and Hanks, 1980).

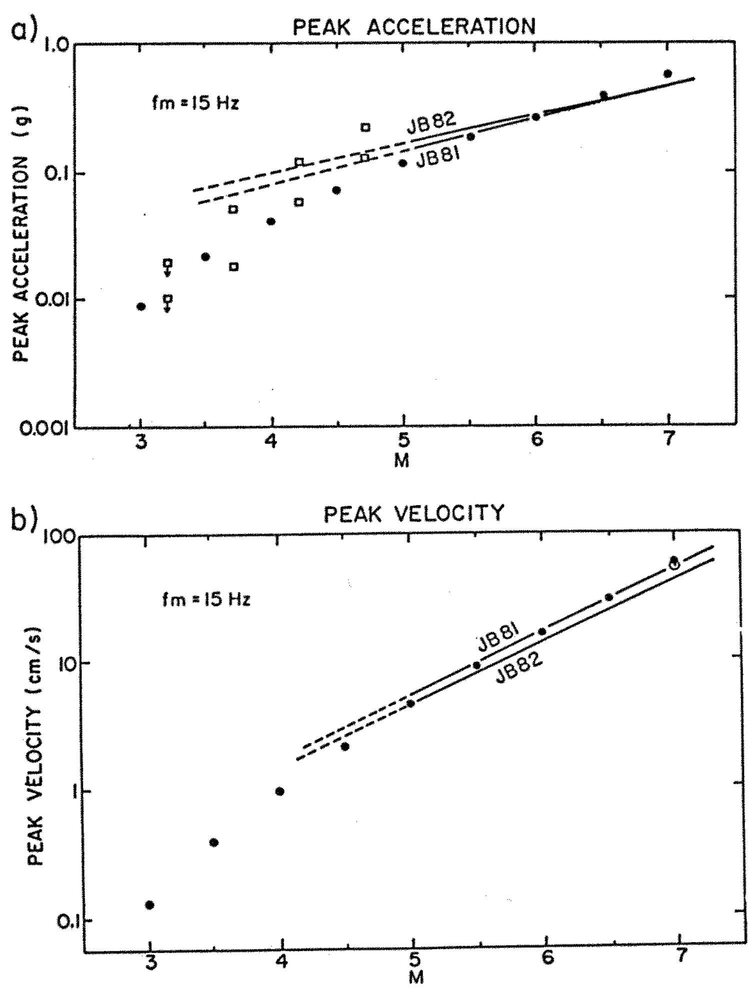

Figure 2. Comparison between random vibration simulations of peak acceleration and velocity (dots) and estimates from empirical equations (lines), at 10 km. (after Boore, 1983).

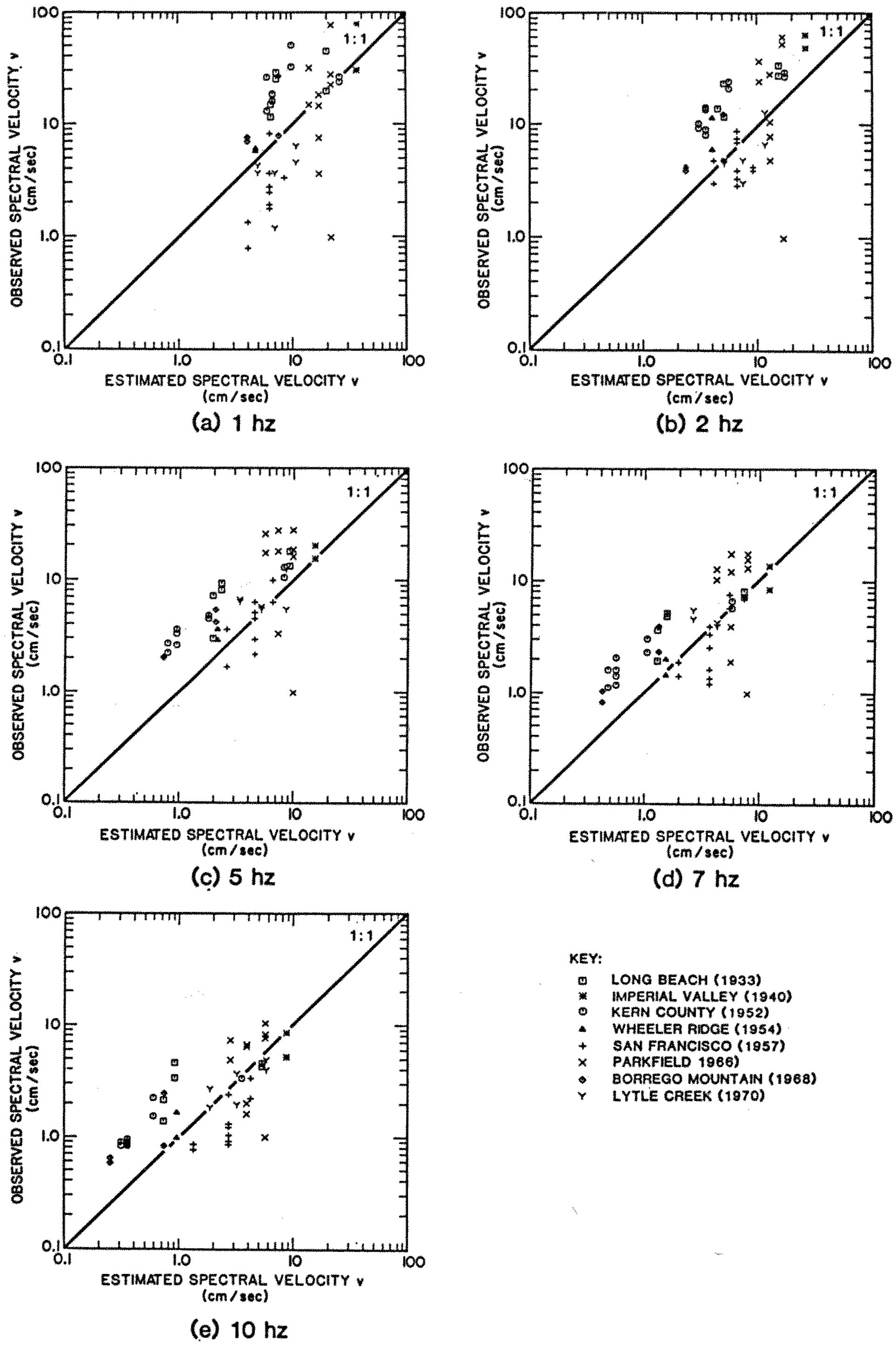

Figure 3. Estimates versus observed response velocity (5% damping) for 5 frequencies (after McGuire et al., 1984).

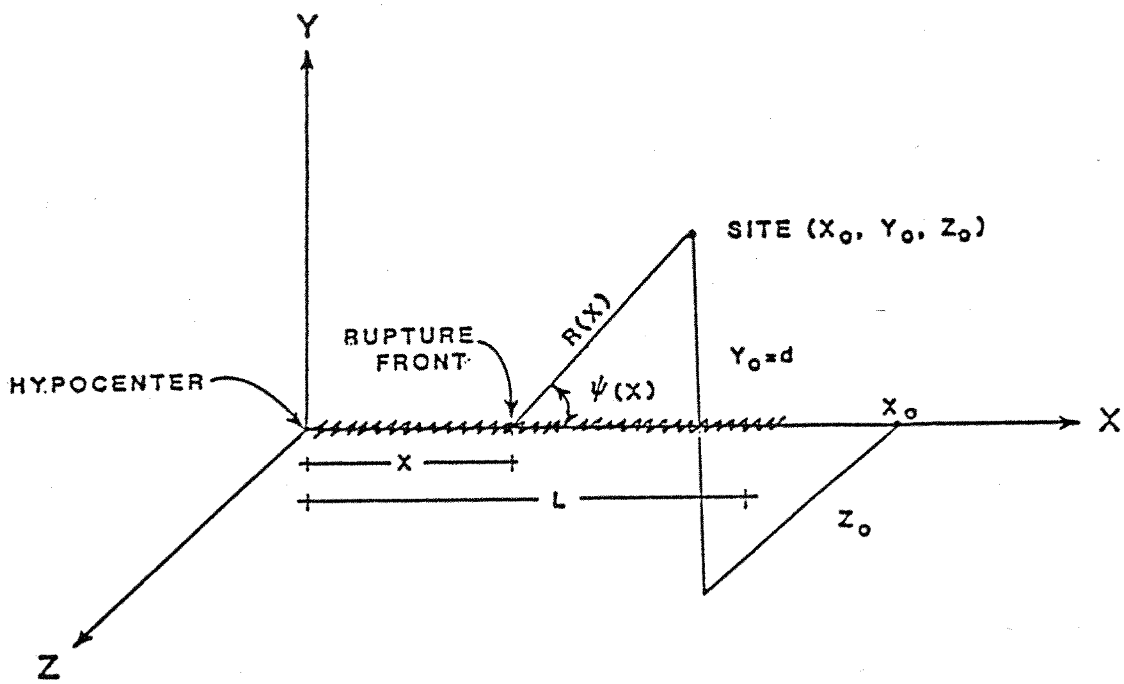

Figure 4. Three-dimensional depiction of site and fault rupture used to represent finite moving sources (after Toro and McGuire, 1985).

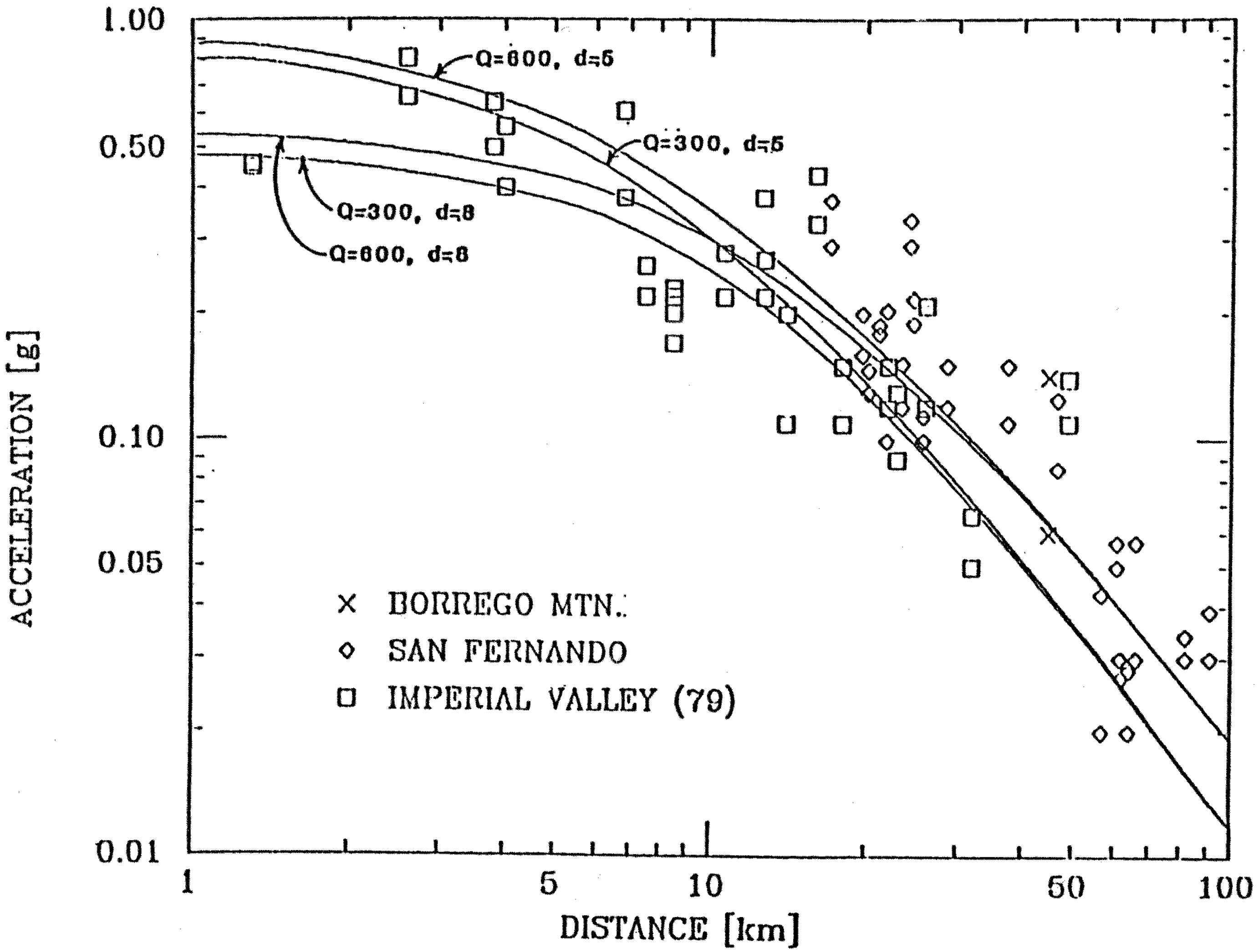

Figure 5. Predicted and observed peak accelerations versus distance for M ~ 6.5 (after Toro and McGuire, 1985).

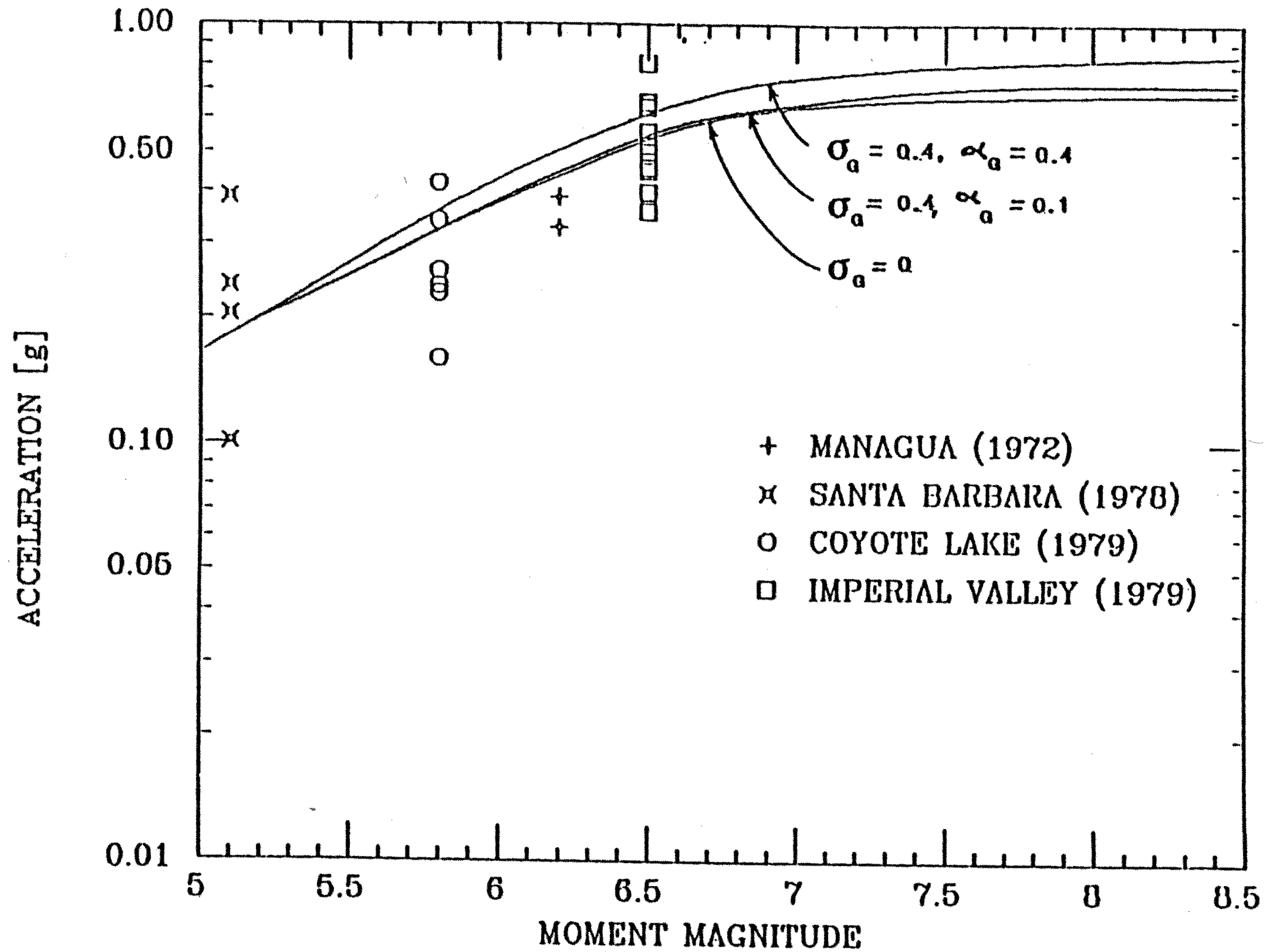

Figure 6. Predicted and observed peak accelerations versus magnitude for R < 5 km. (after Toro and McGuire, 1985).

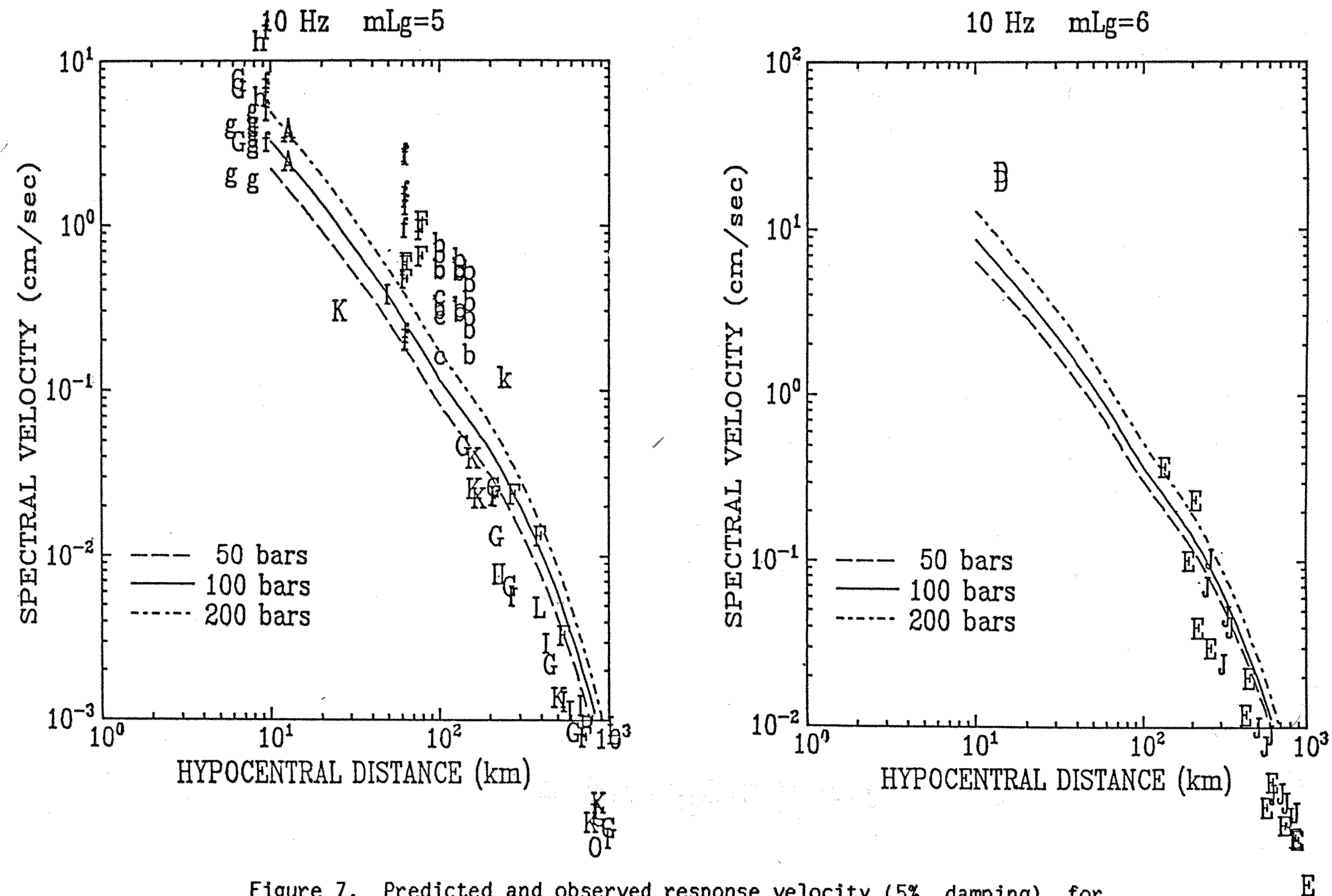

Figure 7. Predicted and observed response velocity (5% damping) for 10 Hz versus distance, m_{Lg} = 5 and 6 (after Toro and McGuire, 1986).

Numerical Simulation of Earthquake Motion-Capability Review(*)

by

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*Paper Not Available.

Some Comments on Ground-Motion Aspects of the Proposed Revised Standard Review Plan

by

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The proposed revisions to the Standard Review Plan (SRP) constitute a considerable improvement, and I would like to extend my compliments to the authors. A few comments on the specification of design ground motions as outlined in the revised SRP are given below. Some of the comments are general and are offered recognizing that, even should the authors concur, not all can be addressed because the SRP must conform to the regulation, Appendix A to 10 CFR Part 100, which itself is in need of revision.

PROBLEMS WITH RESPONSE SPECTRA

Seismological Limitations of Response Spectra

It is unfortunate that the current regulation mandates a response-spectrum representation of the SSE. While response spectra are quite useful for engineering design and analysis of linear elastic systems, they are non-unique and incomplete representations of ground motion. As is discussed further below, artificial time histories generated to have response spectra that meet or exceed a design response spectrum can have widely varying power spectral density (PSD) functions, which can lead to both overly conservative and unconservative estimates of system response. If time histories have to be generated anyway for the design process, it would seem to make sense to specify design ground motions in the form of time histories. The recent EERI Workshop on Strong Ground Motion Simulation and Earthquake Engineering Applications (1) concluded:

> "The most desirable specification of ground motion for the engineer to receive from the seismologist is an ensemble of ground-motion time histories induced in the vicinity of the structure by the design earthquake events. These time-history specifications should be accompanied by the seismologist's assessment of their sensitivity to variations in assumptions and parameters so that the degree of conservatism in the results can be evaluated.
>
> The development of ground-motion time histories for engineering design is a team effort involving the geologist (who defines the geologic and tectonic environment), the engineer (who provides a probabilistic specification and an identification of key ground-motion features), and the seismologist (who models the earthquake environment and computes the ground-response time histories)."

Alternatively, a stochastic representation of design ground motion could be utilized, given in terms of a PSD function and a probability density function (typically Gaussian) for ground motion amplitudes. PSD functions are discussed further, below.

Problems with Smoothed Design Response Spectra

The inherent seismological limitations of response spectra are compounded by the practice (again, mandated by Appendix A) of defining a single SSE spectrum that envelopes the peak response in each frequency band even though the peak responses in different bands may be produced by different maximum potential earthquakes. The resulting design response spectrum, and any time histories generated to match it, are not physically realizable. This does not matter for analyses of the peak response of linear elastic systems, but only realistic ground motions should be used in nonlinear analyses, as is pointed out in the revised version of the SRP (Section 3.7.1). The point to be made here is that time histories derived from smoothed design response spectra should not be used to analyze soil-structure interaction (SSI) when nonlinear soil behavior is involved.

The explicitly stated preference in the revised SRP of site-specific spectra over Reg. Guide 1.60 standard design response spectra is to be applauded. R.G. 1.60 spectra often significantly exceed site-specific spectra at low frequencies (< 1 Hz) and can fall below site-specific spectra at high frequencies (> 10 Hz) if short-duration but high-amplitude ground motions from small, very close earthquakes control the site-specific spectra.

Vertical Design Response Spectra

The practice of specifying vertical design response spectra to be two-thirds of the horizontal values across the entire frequency band of interest is an engineering approximation that can be improved upon by current modeling techniques and by empirical studies of available strong-motion data. The recommendation in the revised SRP (Section 2.5.2.6) to use this procedure should be dropped for this reason and also because it is inconsistent with guidance elsewhere in the SRP to develop site-specific estimates of vertical-component response spectra.

SITE RESPONSE

Amplification Functions

Section 2.5.2.5 should provide more guidance on how to present the seismic wave transmission characteristics (amplification or deamplification) of materials overlying bedrock at the site. Seismologically, Fourier spectral ratios are usually the best representation of frequency-dependent amplification. (Complex transfer functions -- ratios of complex Fourier transforms --

which describe phase response as well as amplitude response, are the ideal representation, but cannot be reliably defined for most sites.) On the other hand, response spectral ratios have obvious engineering appeal if design motions are in terms of response spectra. However, response spectral ratios (unlike Fourier spectral ratios) depend not only on the seismic wave transmission characteristics of the site, but also on the character of the input time history. Fortunately, recent work sponsored by EPRI* has shown that response spectral ratios do seem to be reasonably stable for input motions corresponding to different magnitude earthquakes, as long as the epicentral distances are less than about 50 km. The key requirement is that input motions not have very low power levels at any frequency at which response spectral ratios are to be calculated. If response spectral ratios are used to characterize the seismic wave transmission characteristics of a site, the input time histories used and their Fourier spectra or PSD functions should be presented as well. Ideally, the time histories should correspond to the SSE.

Nonlinear Analyses

The references cited in the revised SRP (Section 2.5.2.5) as examples of nonlinear analysis are out of date. Codes currently being used to model nonlinear soil response (e.g., 2 - 6) should be cited instead.

One-Dimensionality

Section 3.7.2 appropriately recognizes uncertainties in SSI analysis due to dipping strata and general lack of symmetry in the soil. Section 2.5.2.5 should be equally explicit in recognizing these uncertainties; as written, it appears to assume that a one-dimensional model of the site will be appropriate ("...material properties should be determined for each stratum under the site.") The subsurface geometry of materials overlying bedrock, as well as their properties, should be a part of the site characterization.

Input Motion

Section 3.7.2 states that, "The input motion at the base of the discrete soil model should produce the specified design spectra at the free surface of the soil profile in the free field."

While this guidance assures consistency with the specified free-field design spectrum and, at first glance, seems to make good sense, it may lead to unrealistic input motions. If the free-field design spectrum is a smoothed spectrum, and if the amplification function of the soil profile exhibits significant peaks, deconvolution to the base of the soil profile will produce an input-motion spectrum with unrealistic spectral nodes

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* EPRI Research Project RP2556-07, Woodward-Clyde Consultants.

(valleys). The point here is that a site with strong frequency-dependent amplification should have a free-surface design spectrum with peaks and valleys at the proper frequencies, if that surface motion is to be deconvolved to depth.

Deconvolution of surface motions to depth is always problematic (and theoretically, impossible) if the soil responds nonlinearly. If the controlling free-field motion were specified for outcropping bedrock instead of surface soil, then input motions at any depth as well as consistent soil-surface motions could be estimated using forward-marching-time techniques, with no need for deconvolution. The state of the art of seismology is such that ground motion can be predicted for rock or stiff sites with as much confidence (perhaps more) as for soil sites. Thus, defining the SSE for (perhaps hypothetical) outcropping bedrock near the site would seem to be both viable and advantageous.

POWER SPECTRAL DENSITY (PSD) FUNCTIONS

General Comments

The problem cited in Section 3.7.1 of power spectral variability of artificial time histories generated from design response spectra is another disadvantage, in addition to those discussed earlier, of using response spectra to characterize design earthquake ground motions. If a spectral (rather than time-history) representation of design ground motion is desired, PSD functions would be a better choice, seismologically, than response spectra, and given a design PSD function, random vibration methodology can be used to generate response spectra or to directly predict probabilistic structural response (7).

The use of PSD functions implies the idealization of an earthquake acceleration time history as a finite-duration, band-limited realization of a stationary stochastic process, which, of course, is not strictly correct -- earthquake motions are transient. However, for modeling time intervals of relatively steady strong shaking, the approximation appears to work well (e.g., 8, 9, 10). It is important to note that, in addition to the PSD function, the duration of strong motion must be specified for a complete description of ground motion. The duration is a function of both the earthquake source and the hypocentral distance (10, 11).

The Kanai-Tajimi (K-T) Formulation

Section 3.7.1 specifies a particular functional form -- the Kanai-Tajimi form (12) -- as a target PSD function for artificial time histories generated from R.G. 1.60 design response spectra. The need to demonstrate sufficient energy in artificial time histories at all frequencies of interest is certainly real, but I have several concerns regarding the particular PSD function being proposed:

1. Using the notation of the SRP, the K-T formulation corresponds to a white-noise excitation of intensity S_0 at bedrock, filtered through a surface soil layer with damping coefficient ξ_g and natural frequency ω_g(12). In contrast, the R.G. 1.60 spectral form was derived by enveloping spectra obtained from a number of different sites with different site conditions, and may, therefore, be inherently incompatible with the K-T PSD formulation.

2. The parameters ξ_g and ω_g control the shape of the K-T function and S_0 controls the level. In the revised SRP, values for all three parameters are specified. Since the level of an R.G. 1.60 spectrum varies from site to site, depending on the zero-period acceleration, it would seem that the parameter S_0 should be left free to vary.

3. The values proposed for ξ, ω_g and S_0 all differ significantly from typical of average values determined by fitting K-T functions to western U.S. strong-motion records (12). Specifically, the proposed value for ξ_g (0.9793) is much greater than the mean value (0.32) cited in (12), and the proposed value for ω_g (10.66 rad/sec) is much lower than mean values found for either rock (26.7 rad/sec) or soil (19.1 rad/sec) sites. The proposed value for S_0 (1,100 in^2/sec^3) is more than three times greater than the largest value found in (12), for a record of the Parkfield earthquake with a peak acceleration of 0.434 g.

MISCELLANEOUS COMMENTS

Operating Basis Earthquake (OBE)

The semi-quantitative probabilistic interpretation of the OBE in Section 2.5.2.7 ("...return period on the order of hundreds of years") is a small but laudable step towards probabilistic standards.

The requirement in Section 3.7.3 to assume five OBE's during the plant life for seismic subsystem analysis seems excessive in view of the OBE return period.

Site-Specific Spectra

The guidance provided in Sections 2.5.2 and 3.7.1 regarding how design motions should compare to site-specific spectra seems to be inconsistent. Section 3.7.1 states that "...OBE and SSE design response spectra... should, generally, meet or exceed amplitudes of the site-specific spectra at all frequencies." Section 2.5.2 states that "... proposed free-field spectra shall be considered acceptable if they equal or exceed the estimated 84th percentile ground motion spectra from the maximum or controlling earthquake...."

Correlation Between Three Components of Motion

Statistical independence of the components of earthquake motion in three directions is stated as a requirement in Section 3.7.2 for time-history analyses of seismic response. Actual earthquake time histories will be correlated to some degree. The two horizontal components of motion may be highly correlated during the direct S-wave arrival, which often causes the strongest shaking.

ACKNOWLEDGMENTS

Some of the thoughts expressed herein on response spectral ratios and on control-motion location stemmed from conversations with Walt Silva. Donald G. Anderson kindly provided the references for nonlinear soil-response models.

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SESSION 3

GROUND MOTION FOR SITE-SPECIFIC SSI ANALYSIS

GROUND MOTION CONSIDERATIONS FOR NUCLEAR POWER PLANT DESIGN WITH EMPHASIS ON SOIL-STRUCTURE INTERACTION ASPECTS

R.P. Kennedy[1], M.S. Power[2], and C.-Y. Chang[3]

1. INTRODUCTION

Nuclear power plant structures tend to be massive, stiff structures with fundamental frequencies in the 2 to 10 Hz range depending on soil conditions. Whereas damage to conventional structures probably correlates best with velocity content of the ground motion, the seismic vulnerability of nuclear power plants is probably more related to acceleration content. For this reason, the ground motion input for seismic design of nuclear power plants has been primarily expressed in terms of acceleration.

It has often been noted, particularly in connection with near-source motions due to low-to-moderate magnitude earthquakes, that structures have performed much better than would be predicted considering the free-field instrumental peak acceleration to which the structures were subjected. In such cases, the differences in measured ground motion, design levels, and observed behavior are so great that they cannot be reconciled with typical safety factors associated with elastic seismic analyses for design even when typical ductility considerations are included [1]. The problems with free-field instrumental peak acceleration and free-field instrumental-based elastic response spectra as measures of potential seismic damage are threefold. First, the foundation motion of massive stiff structures is

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often substantially less than that recorded by a free-field instrument due to soil-structure interaction (SSI) and spatial variation of gound motion effects. Secondly, a limited number of high-frequency spikes of high acceleration but very short duration has little effect on the elastic response spectra within the region of interest. Thirdly, structural damage is related to structures being strained into the inelastic range in which the breadth of the frequency content of the actual record plays a very important role because of frequency shifts with inelastic response. Also inelastic response and damage are likely to be influenced by the duration of strong motion or the number of cycles of strong response. A standard elastic spectrum ignores these effects.

Because of these considerations, the U.S. Nuclear Regulatory Commission (USNRC) has sponsored research work on the subject of engineering characterization of ground motion [2-6]. This paper presents a summary of this work with emphasis on SSI and spatial variation of ground motion effects. As such, this paper borrows heavily from Ref. [6]. Before discussing SSI effects, one should start with an understanding of the minimum parameters needed to provide an adequate engineering characterization of ground motion.

2. ENGINEERING CHARACTERIZATION OF GROUND MOTION

The three most important parameters providing an engineering characterization of the ground motion are: (1) breadth of the frequency content, (2) description of the amplitude, and (3) SSI and spatial variation of ground motion effects. So long as the strong motion duration exceeds about 3 seconds, strong duration has a much lesser importance. The strong motion duration is defined in Section 2.2.

2.1 Frequency Content

The breadth of the frequency content has a substantial influence on both elastic and inelastic responses [2] and on the importance of SSI effects [3,5,6]. This breadth can be described by a mean frequency Ω and the frequency range (f_{10} to f_{90}) over which 80 percent of the power of the input motion is distributed. The mean frequency can be defined in terms of the zero and second moments (λ_0 and λ_2) of the power spectral density function, G(f) by [7]:

$$\Omega = \lambda_2/\lambda_0 \quad ; \quad \lambda_i = \int_0^\infty f^i \, G(f)df \qquad (1)$$

Figure 1 shows the cumulative power spectral density function (obtained by integrating G(f) over frequency) for the 1952 Taft accelerogram. f_{10} and f_{90} are the frequencies at which 10% and 90% of the cumulative power are reached. Table I presents these parameters for some representative earthquake records as well as for an artificial Reg. Guide 1.60 record.

It has been found [2] that the Reg. Guide 1.60 spectrum when anchored to an effective acceleration, A_D, can be used to slightly conservatively predict both elastic and inelastic responses of stiff structures (2 to 10 Hz) for all those records in Table I with broad frequency content in which $f_{10} < 2.0$ Hz, $f_{90} > 5.0$ Hz, and Ω lies between 3.0 and 4.8 Hz (Records 1-3 and 5-7 in Table I). The other 5 records (Records 8-12) in Table I have frequency content sufficiently dissimilar to that of the Reg. Guide 1.60 spectrum that this spectrum cannot be used to adequately predict either elastic or inelastic response throughout the frequency range from 2 to 10 Hz no matter what value of effective acceleration is chosen for anchoring the Reg. Guide 1.60 spectrum. The concept of an effective anchor acceleration, A_D, cannot be used to compensate for severely dissimilar frequency breadth.

Figure 2 shows the 7% damped response spectra shapes for the artificial Reg. Guide 1.60 record and three real records all anchored to 1g zero period acceleration (ZPA). The El Centro #5 record is typical of the broad frequency content records in Table I. Parkfield is devoid of frequency content above about 3 Hz and has a low mean frequency. Structural responses from a record such as Parkfield could only be approximated by a design or evaluation spectrum with a mean frequency between 1.8 and 3.0 Hz and with $f_{90} < 3.2$ Hz. On the other hand, the Melendy Ranch record has high mean frequency and is devoid of frequency content below 2.5 to 3.5 Hz. Such a record could only be approximated in the range from 2 to 10 Hz by a design or evaluation spectrum with a mean frequency between 5 and 8 Hz and with $f_{10} > 2.5$ Hz.

2.2 Effective Ground Acceleration

Many investigators (e.g., [1, 2, and 8]) have pointed out that the instrumental peak acceleration, A_I, is a poor indicator of elastic response spectrum values. The spectral accelerations, S_a, are primarily influenced by the energy contained within a number of cycles of ground motion and are little influenced by a few spikes of very high acceleration. Blume [1] has shown that clipping the highest 30% off the measured acceleration-time history (using only 70% of the record, in an absolute sense, closest to the zero line) produced only about a 5% reduction in the elastic response spectrum for frequencies up to 10 Hz. Other investigators have reported similar findings which has led them to recommend that the design or evaluation response spectrum be anchored to an effective acceleration, A_D, based on the sustained or repeatable peak acceleration rather than A_I.

An approach is to define the effective acceleration in terms of the rate of energy that is fed into structures. Arias [9] and Housner [10] have demonstrated that $E(T'_D)$, as illustrated in Figure 3, can serve as a measure of the cumulative energy per unit mass fed into all single-degree-of-freedom oscillators over strong motion duration, T'_D. The average rate of energy input (power), P, and root-mean-square (rms) acceleration, a_{rms}, are then given by:

$$P = \frac{E(T'_D)}{T'_D} \quad ; \quad a_{rms} = \sqrt{P} \tag{2}$$

Several investigators, (e.g., [2, 11]) have suggested that because a_{rms} describes the average power over time, T'_D, it provides substantially more information about the influence of the record on structural response than does A_I. Use of a_{rms} enables an effective design acceleration, A_{DE}, to be selected at any desired probability of exceedance during the time history. The design acceleration is related to rms acceleration by:

$$A_{DE} = K_p * a_{rms} \quad ; \; K_p = \sqrt{2 \ell n \; (2.8 \; T'_D \; \Omega \;)} \quad \text{except } K_p \geq \; 2 \qquad (3)$$

where K_p is a factor which is a function of the acceptable exceedance probability for each individual peak of the time history. Considering the design acceleration as that which is expected to occur once on the average over the strong motion duration for a stationary random Gaussian motion, Vanmarcke and Lai [12] have determined the above expression for K_p.

For stiff structures, only the portion of the record with maximum power significantly influences either elastic or inelastic response [2]. Therefore, the duration of interest is only the strong motion duration, T'_D, associated with the duration of maximum power. Different investigators [2, 11, 13] have proposed differing techniques for defining T'_D. One method [2] for defining T'_D is the duration from $T_{0.05}$ to T_m where $T_{0.05}$ is the time at which 5% of the cumulative energy is reached and T_m is either the time $T_{0.75}$ at which 75% of the cumulative energy is reached or the time of the first zero crossing after both the maximum positive and maximum negative instrumental accelerations are reached, whichever is later in time. Table I presents duration, T'_D, and effective design acceleration, A_{DE}, estimates [2] as well as the instrumental peak acceleration, A_I for 11 real earthquake records.

Studies have been performed [2] to determine the adequacy of using a broad-frequency content design spectrum (Reg. Guide 1.60) anchored to A_{DE} to predict response of stiff structures from actual earthquake records. Table II presents the maximum, minimum, and median values of the ratio of spectral acceleration from the actual record to spectral accelerations from Reg. Guide 1.60 anchored to A_{DE} over the frequency range from 1.8 to 10 Hz. Elastic (μ = 1.0) and two levels of inelastic (μ = 1.85, and 4.27) response ratio results are presented. Inelastic results are in terms of inelastic spectral accelerations (i.e., the spectral acceleration for which the structure would have to be designed to be at the onset of yield in order to achieve a given ductility, μ, from the time history record).

A review of Table II indicates that for the 6 records (1-3, 5-7) that have frequency content similar to the Reg. Guide 1.60 spectrum (see Table I), the Reg. Guide 1.60 spectrum anchored to A_{DE} does an adequate job of predicting the required design spectral accelerations for both elastic and inelastic responses. The maximum factor of unconservatism is only 1.3 and the maximum factor of conservatism is about 2.0 (i.e., 1.0/0.49) for both elastic and inelastic responses. On the average, only slight conserva- tism is introduced. However, for the 5 records (8-12) that have frequency content substantially different than the Reg. Guide 1.60 spectrum, the Reg. Guide 1.60 spectrum anchored to A_{DE} does not provide a universally adequate prediction of responses in the frequency range from 1.8 to 10 Hz. For the low-frequency content records (Goleta and Parkfield), the use of the Reg. Guide 1.60 spectrum generally introduces excessive conservatism but in the case of inelastic response its use can introduce excessive unconservatism within narrow frequency bands. For the high-frequency content records (Coyote Lake, Gavilan College, and Melendy Ranch), the use of the Reg. Guide 1.60 spectrum produces completely inconsistent results with factors of conservatism as high as 10.0 (1.0/0.10) and factors of unconservatism as high as 1.6.

2.3 General Soil-Structure Interaction Effects

Soil-structure interaction (SSI) effects tend to reduce the input motion at the foundation of large, massive, stiff buildings below that of the free-field ground surface. Soil-structure interaction effects include: 1) lesser fundamental characteristic frequency for the soil-structure system than for the structure alone, 2) increased system damping for the combined soil-structure system, 3) spatial averaging of the input motion over the basemat width, and 4) spatial variation of the ground motion with depth for structures with embedded foundations (kinematic interaction).

Substantial reductions in embedded foundation motions, resulting from lower system frequency, increased damping, and kinematic interaction, are often computed by SSI analyses that incorporate kinematic interaction for embedded foundations [3, 4, 5, 14]. Figure 4 compares response spectra of the free-field control motion and foundation horizontal motions computed from SSI analyses for the foundation of a large, stiff building with a foundation size of about 280-feet by 140-feet and with both no embedment and 40-feet embedment [14]. The surface-founded structure foundation motion is substantially reduced from the free-field motion for frequencies between 5 and 20 Hz. With 40-feet of embedment, even greater reductions occur at all frequencies above 2 Hz. Ignoring these reductions in structural response due to SSI effects including kinematic and inertial interaction can lead to overprediction of structural damage for stiff structures. SSI effects will be discussed in more detail in subsequent sections.

3. INFLUENCE OF GROUND MOTION CHARACTERIZATION ON STRUCTURAL DAMAGE

For stiff shear wall structures, structural damage might be defined in terms of inelastic lateral drift or ductility, μ. The ratio by which the ground motion exceeds that corresponding to the linear computed capacity limit of the structure can be defined as the demand-capacity ratio, F. The relationship between demand-capacity ratio, F, and ductility, μ, is strongly influenced by both the frequency content of the ground motion and SSI effects including kinematic interaction. Thus, both characteristics of the ground motion and SSI effects must be properly modeled if one wishes to predict structural damage for stiff structures.

For fixed-base structures, the magnitude of F corresponding to a given μ is primarily influenced by the frequency content of the accelerogram relative to the elastic frequency of the structure. As the structure goes into the inelastic range during response to the scaled accelerogram, its effective frequency shifts (decreases) from the elastic frequency, f, toward a secant frequency, f_s, that corresponds to a certain ductility. As this occurs, energy is fed into the structure over this frequency range, and it is therefore the spectral content of the accelerogram over this frequency range that determines the inelastic structural response. If the accelerogram has a response spectrum that is characterized by increasing spectral accelerations as the structure softens from frequency f to f_s, then F values for a given μ will be low. On the other hand, if the accelerogram is characterized by decreasing spectral accelerations with decreasing frequency over this range, then F factors will be high for the same μ.

The importance of the frequency content of the accelerogram to the scale factors for nonlinear response is illustrated in Figure 2,, in which response spectra for the Parkfield, Melendy Ranch, El Centro #5, and artificial (Reg. Guide 1.60) accelerograms are compared (each accelerogram is scaled to a peak acceleration of 1.0g).

Based on elastic response of a fixed-base structure, one would expect the Melendy Ranch accelerogram to be substantially the most damaging to a 5.3 Hz structure and the Parkfield record to result in negligible damage potential. However, the demand-capacity ratio, F, corresponding to a ductility of 4.3 significantly differs for each of these four records. Because the Parkfield accelerogram has a response spectrum that shows increasing spectral values as the structure softens from the elastic frequency, f, of 5.3 Hz to a secant frequency, f_s, of 2.8 Hz (for μ equal to 4.3) (Figure 2), it would be expected that the scale factor, F, for this accelerogram would be relatively low. On the other hand, because the response spectrum of the Melendy Ranch accelerogram decreases in going from 5.3 to 2.8 Hz, a relatively high scale factor would be expected for this accelerogram. Scale factors for the El Centro #5 and the artificial accelerogram intermediate between those for Parkfield and Melendy Ranch would be expected based on the relatively flat spectral response for these accelerograms over the f to f_s range. The scale factors determined from nonlinear analyses are in accord with these expected results, being equal to approximately 1.3, 5.5, 2.7, and 1.9 for the Parkfield, Melendy Ranch, El Centro #5 and artificial accelerograms . For all accelerograms in Table I, it was found that the scale factor exceeded 2.7 for every case in which spectral acceleration decreased as the structure softened and was less than 1.7 for every case in which spectral acceleration increased (for of 4.3). Thus the response spectrum frequency content over the frequency range for inelastic response appears to be the dominant ground motion characteristic influencing attainment of structural deformations. This frequency content effect over the range from f to f_s is also termed herein the "spectral averaging" effect. Primarily because of this "spectral averaging" effect, the artificial Reg. Guide 1.60 accelerogram is by far the most damaging record for the 5.3 Hz structure whereas the Melendy Ranch accelerogram lies midway between the damage effectiveness of El Centro #5 and Parkfield when all records are scaled to the same peak ground acceleration.

To further study frequency content effects as well as SSI effects including kinematic interaction, inelastic response analyses were conducted on a PWR reactor building model which includes both a prestressed concrete containment and a reinforced concrete internal structure [3]. The containment and internal structure have fixed-base fundamental natural frequencies of 4.5 and 5.2 Hz, respectively. The containment has very high seismic capacity so only the internal structure is susceptible to inelastic response. This structure was sited on a rock site and was designed for a 0.2g Reg. Guide 1.60 spectrum. The building model was then subjected to the four accelerograms whose spectra are shown in Figure 2, each scaled to a peak acceleration of 0.5g or 2.5 times the design level. The internal structure is characterized by relatively high ratios of shear demand to shear capacity (i.e., elastic computed shear loads to shear strength) near its base, which results in that location being a "weak link" in which all the nonlinear, inelastic behavior occurs.

The structure was analyzed for both fixed-base and soil foundation conditions. In the latter cases, 40 feet of foundation embedment (embedment depth to foundation diameter ratio approximately equal to 0.3) was assumed in soil profiles of two stiffnesses, designated "intermediate" and "stiff". Both soil profiles consist of soil layers to a depth of 250 ft overlying rock. The shear wave velocity of the soils in the intermediate soil profile is approximately 1000 ft/sec. The soils of the stiff soil profile consist of a 40-ft layer with a shear wave velocity of approximately 900 ft/sec overlying a material with a shear wave velocity of approximately 1800 ft/sec. The shear wave velocity of the underlying rock is 3600 ft/sec. Variations of ground motions with depth and kinematic and inertial soil-structure interaction were included for the soil foundation cases. The input accelerograms were applied directly to the foundation in fixed-base cases and were applied to the ground surface in the free field in soil-structure interaction cases.

Maximum demand/capacity ratios, $(F)_M$, based on elastic analysis and maximum story drift ductility, $(\mu_s)_M$, based on inelastic analysis both occurred at the base of the internal structure for all cases. Table III (from [3]) summarizes these results. The influence of spectral-averaging effects is best seen in the fixed-base cases. For this 5.2 Hz internal structure, the Melendy Ranch accelerogram is far less damaging in terms of maximum story drift ductility than indicated by the linear elastic computed demand-capacity ratio, whereas the Parkfield accelerogram is far more damaging than indicated by linear elastic analysis. Because of its broad spectral shape and longer duration, the artificial Reg. Guide 1.60 accelerogram is far more damaging than any of the real accelerograms studied.

The influence of kinematic interaction and increased system damping due to SSI effects is best seen for artificial and El Centro #5 accelerograms. The internal structure response is primarily influenced by two modes in these SSI cases with natural frequencies of 2.6 and 4.8 Hz for the stiff soil profile and 1.8 and 4.3 Hz for the intermediate soil profile versus the one 5.2 Hz mode for the fixed-base case. Over the frequency range from 1.8 Hz to 5.2 Hz the spectral accelerations are reasonably constant for both the artificial and El Centro #5 accelerograms. For El Centro #5, for elastic responses, the base shears in the internal structure are only 50% and 30% as great for the stiff and intermediate soil profile cases, respectively, as for the fixed-base case. Correspondingly, for the artificial accelerogram, these base shears are only 60% and 40% as great for the stiff and intermediate soil profile cases, respectively, as for the fixed-base case. For the El Centro #5 and artificial accelerograms, these reductions in base shear are totally due to kinematic interaction and increased system damping. Ignoring these aspects of SSI results in excessively conservative and unrealistic damage estimates for the El Centro #5 and artificial accelerograms which are 2.5 times the fixed base design peak acceleration capacity levels. Despite exceeding the design ground motion by a factor of 2.5, these accelerograms result in either elastic response or very low levels of inelastic response for this structure when sited on intermediate soil due to the benefits of kinematic interaction and increased system damping. These influences on predicted damage are impressive.

The influence of frequency shift to lower frequencies due to SSI effects are best seen for the Parkfield and Melendy Ranch accelerograms. For Melendy Ranch, the frequency shift due to SSI shifts response from being highly inelastic to being totally elastic. For Parkfield, frequency shifts due to SSI greatly increase inelastic response particularly for the stiff soil profile. In this case, the detriment due to frequency shift from SSI overpower the benefit due to kinematic interaction and increased system damping. These cases illustrate the importance of accurately estimating the frequency content of the ground motion, especially at the fundamental characteristic frequency of the soil-structure system.

Because of the importance of spatial variation of the ground motion and kinematic interaction on structural response, these topics will be further discussed in the remaining sections of this paper.

4. CHARACTERIZATION OF VARIATIONS OF GROUND MOTION WITH DEPTH

Plane wave propagation models are used in current practice in conducting soil-structure interaction analyses for nuclear power plant structures. A wave field consisting of vertically propagating waves is typically assumed for these analyses. Figure 5 provides a comparison of the response spectrum of the horizontal input motion at the free-field ground surface (free-field control motion) with the response spectrum of the motion at a depth of 40 ft in the free field obtained from deconvolution analysis for the previously mentioned "intermediate" soil profile [6]. The spectrum of the motion at depth is significantly lower than the spectrum of the motion at the ground surface. The pronounced valley in the spectrum of the motion at depth occurs at the fundamental frequency of the overlying soil profile (approximately 6 Hz). Also shown in Figure 5 are the horizontal and rocking foundation input motions of the previously described PWR reactor building embedded at a depth of 40 ft. The foundation input motions are those resulting from kinematic interaction of a massless, rigid foundation with the free-field wave field. Due to kinematic interaction, the higher-frequency peaks and valleys of the foundation-level, free-field motion are smoothed out in the horizontal component of the foundation input motion and a rocking component of motion is introduced. The low spectral amplitudes of the horizontal component of the foundation input motion in the high frequency range relative to the free-field control motion and the introduction of a rocking component illustrate the potential significance of ground motion variations with depth.

In conducting the complete SSI analysis for the input motions in Figure 5, it was found that the horizontal component of the actual foundation response (including inertial as well as kinematic interaction) was nearly the same as the foundation input motion. However, the actual foundation rocking response motion was greatly different from the rocking component of the foundation input motion. The rocking response mainly reflected response of the soil-structure system (at a fundamental characteristic frequency of about 2 Hz) to the horizontal component of the foundation input motion. Thus, the reduction in horizontal foundation input motion due to embedment and kinematic interaction was very important to structural response, whereas the introduction of a rocking input motion due to kinematic interaction had relatively minor importance.

A review of observational data on the variations of earthquake ground motion with depth has been conducted [4,15,16]. Figure 6 presents representative results of the analysis of the Japanese downhole data from the Narimasu site [4]. This figure illustrates the substantial reductions in the amplitudes of recorded peak accelerations and response spectra with depth below the ground surface. Deconvolution analyses of the Narimasu array site were made using the recorded surface (-1m) motions as input motions. These analyses utilized the vertical plane-wave propagation technique that is typically used in practice (computer program SHAKE). The assumption of vertically incident waves is consistent with the predominant wave field estimated for the ground motion at the Narimasu site. The calculated ground motions show reductions in amplitude and changes in frequency content with depth that are generally consistent with those of the recorded motions (Figure 6). The calculated ground motions are somewhat higher than the recorded motions with differences tending to increase at greater depths, indicating that the results of the deconvolution analyses are conservative. Similar results and observations were obtained for data from the Waseda, Japan site [4].

There are two possible reasons why the recorded ground motions at depth were higher than the calculated ground motions. One reason is scattering of seismic waves in the near surface soils. As a result of scattering, the near surface motions may contain components of motion that would not be predicted by plane wave propagation theory for vertically propagating waves. When the near-surface motions containing these components of motion are deconvolved, the resulting calculated motions at depth would be higher than the recorded motions.

The other reason is that the high frequency motions may be over-damped in the theoretical calculation [17]. The motions are calculated based on the assumption of a constant, average soil damping throughout the duration of shaking, as required to be made in currently available frequency domain linear or equivalent linear techniques used for deconvolution and soil-structure interaction analysis. In reality, soil damping varies throughout the duration of shaking. The higher frequency motions during the shaking tend to be associated with smaller strains and thus with lower damping. Thus, when an average soil damping is used in calculations, high frequency motions may be overdamped, resulting in an overestimation of high-frequency ground motion at depth from a deconvolution analysis. It is expected that this effect would become more significant for high levels of excitation.

The ground motions recorded at the Humboldt Bay Power Plant during the 1975 Ferndale, California earthquake provide data on the motions at the base of a massive, deeply embedded structure relative to those at the ground surface in the free field. Figure 7 shows the response spectra of horizontal ground motions recorded at the base of the refueling building (deeply embedded at a depth of 84 feet) and at the free-field ground surface. The horizontal motions at the base of the embedded structure are significantly lower than the free-field ground surface motions. However, the vertical motion at the base of the structure is higher than the free-field ground surface motion [4]. The horizontal motions were analyzed in detail [18, 19]. These analyses, which incorporated wave propagation effects on the variation of ground motion with depth and soil-structure interaction effects, resulted in good agreement of the response spectra of the calculated motions and the recorded motions at the base of the refueling building.

Another example of data for embedded structures is provided by ground motion recordings obtained in nearby buildings with and without basements during the 1971 San Fernando, California earthquake [4, 20, 21]. The data indicate that, in general, the foundation motions of the buildings with basements are significantly lower than the foundation motions in nearby buildings without basements. Response spectra comparisons for two nearby buildings with and without basements, showing reduction in lower-period (higher-frequency) motions for the basement motions are shown in Figure 8. Analysis of this data pair [4, 22] indicated that reductions were larger than those that would be predicted considering variations in ground motion with depth and soil-structure interaction effects.

Limitations in the available observational data pertaining to variations of ground motion with depth include the fact that available data from downhole arrays analyzed to date are of relatively low amplitude (highest peak accelerations equal to or less than about 0.1 g). It is desirable to have additional data to verify trends at higher acceleration levels including assessment of the influence of nonlinear soil behavior (nonlinear effects should be small for the low excitation levels of the currently available downhole data). Also, most of the downhole array data are for relatively soft soil conditions, and more data are needed for stiffer soils. Despite these limitations, considerable data exist from which to examine empirical trends and compare with results of analyses. From a review of these data, it is concluded that the empirically observed trends are generally consistent with predictions from plane wave propaga- tion models as are typically used in practice in evaluating ground motion variations with depth and conducting soil-structure interaction analyses. It appears that the analysis methods tend to result in somewhat conservative estimates of the variations of ground motion with depth.

5. EFFECTS ON STRUCTURAL RESPONSE OF NEGLECTING GROUND MOTION VARIATIONS WITH DEPTH

Often, soil-structure interaction analyses for embedded structures have applied the control motion, defined at the free-field ground surface, as the translational foundation input motion and have neglected the rocking components of the foundation input motion. To assess the effects of this practice of neglecting ground motion variations with depth and kinematic interaction, a series of comparative analyses was performed [5] on the previously described reactor building using two embedment depths (20 ft and 40 ft), four soil profiles, and the four seismic excitations whose spectra are shown in Figure 2. The four soil profiles have shear wave velocities in the upper 250 ft ranging from 3600 ft/sec for the stiffest (rocklike) profile I to 1000 ft/sec for the softest profile IV. Profiles III and IV correspond approximately to the previously described "stiff" and "intermediate" profiles, respectively. Structural responses from the analyses that excluded effects of ground motion variations with depth and kinematic interaction (referred to subsequently herein as "analyses excluding kinematic interaction") were compared with analyses in which these effects were included [5, 6, 16]. The structural responses examined included base shear forces and moments and peak accelerations and floor response spectra at various locations in the containment shell and internal structure. It was found that for all parameters and for all cases analyzed, excluding kinematic interaction increased the response. The effects of excluding kinematic interaction on the peak base shear force in the containment shell for the 40 ft-embedment-depth cases are summarized in Table IV. The effects increase as the profile stiffness decreases, and they are largest for high-frequency excitations such as the Melendy Ranch record. Thus, for the Melendy Ranch excitation, the ratio of peak base shear force excluding kinematic interaction to that including kinematic interaction increased from a value less than 1.1 for a very stiff (rocklike) soil profile (profile I) to as much as 1.7 to 1.8 for softer soil profiles (profiles III and IV). On the other hand, excluding kinematic interaction had a relatively small effect for the Parkfield excitation, which has a low content of high frequency motion.

Effects of excluding kinematic interaction on floor response spectra are illustrated in Figure 9 for the case of the artificial (Reg. Guide 1.60) excitation, soil profile III, and 40-ft embedment. It can be seen that substantial increases in floor response spectra occurred when kinematic interaction was excluded from the analysis.

The results obtained from these analyses indicate that excluding kinematic interaction can lead to significant overestimation of structural responses. The overestimation increases with decreasing soil stiffness, increasing high-frequency content of the free-field control motion, and increasing embedment depth.

It should be noted that the analytical effects of not including variations of ground motion with depth examined in this section pertain to the substructure method of carrying out soil-structure interaction analyses. The effects would be even greater using the finite element method when the control motion is input at the foundation level in the free field rather than at the finished grade. With such a practice, wave propagation analysis leads to motions at the finished grade that are generally greatly amplified above the control motion specified for the site. Furthermore, induced foundation rotations due to kinematic interaction are automatically included in the finite element analyses.

6. CHARACTERIZATION OF VARIATIONS OF GROUND MOTION IN A HORIZONTAL PLANE

Spatial variation of input motion in a horizontal plane results from either horizontal traveling wave effects or statistical incoherence of the ground motion. Both are briefly considered below.

6.1 Horizontal Traveling Wave Effect

Phase differences in ground motion in a horizontal plane result from the apparent horizontal velocity of the seismic waves. Newmark [8] proposed that traveling wave effects could be conservatively approximated by reducing the effective design acceleration by a factor R given by:

$$R = (1-5\ \tau) \geqq 0.67 \qquad (4)$$

where τ is the travel time across the basemat. Available data, summarized in [4], have indicated apparent horizontal wave speeds on soil sites ranging from 2.5 to 5.5 km/sec. Such wave speeds are much greater than had been envisioned by Newmark. With a wave speed of 2.5 km/sec, the τ-effect from Equation 4 becomes negligible in reducing the foundation motion.

Soil-structure interaction analyses using non-vertically incident waves having propagation velocities of at least 2 km/sec for SH waves and at least 3 km/sec for P-SV waves show less than 10% reductions in translational and rocking responses [5, 6, 23]. It appears that non-vertically-incident wave effects on translational and rocking responses are small enough to be neglected as long as apparent horizontal wave speeds exceed 2 km/sec.

The transverse component of non-vertically-incident waves induces a torsional response of a structure due to the transverse motions being out of phase across the foundation. The torsional response is approximately inversely proportional to the apparent wave speed and is zero for vertically propagating waves input to a symmetric structure. Torsional responses due to non-vertically-incident waves may potentially increase horizontal motions toward the perimeter of a structure relative to the motions due to vertically incident waves.

A 5 percent accidental eccentricity of the peak base shear force has often been assumed in practice to incorporate possible torsional moments induced by non-vertically-incident waves. It appears that such a value of accidental eccentricity would generally cover response for cases in which the free-field input motion has a broad-banded response spectrum, such as Reg. Guide 1.60 and that torsional effects on perimeter response motions are small enough for this type of input motion to be neglected in most cases [6].

It also appears, however, that more significant torsional effects due to non-vertically-incident waves may occur with some other types of free-field motions. Specifically, analyses [5, 6] indicate that a significant torsional effect may be associated with narrow-banded, high-frequency, short-duration input motions, such as Melendy Ranch, that produce a strong torsional response but a weak translational response due to the spectral peak of the motion occurring at frequencies significantly higher than the fundamental characteristic frequency of the soil-structure system. In such cases, the accidental eccentricity can substantially exceed 5 percent and effects on perimeter motions can be significant. Further studies are desirable to better define the range of practical conditions, including types of seismic excitations, wave fields, and foundation dimensions and depths, for which non-vertically-incident wave effects should be considered.

6.2 Statistical Incoherence Effects

Due to the lack of complete coherence of the free-field motion, mat foundations of large structures may be subject to an average motion that is lower than the free-field motion. This reduction, which has been termed a "base-averaging effect", increases with an increase in 1) foundation dimension, 2) input motion frequency, and 3) inhomogeneity of the subsurface geological conditions. The effect may also depend on the relative rigidity of the foundation and underlying soil deposit, increasing as the relative rigidity increases. The base averaging effect could be incorporated by a frequency-dependent reduction in translational foundation input motions. The amount of the reduction is not well quantified at the present due to limited data. For a relatively homogeneous soil site for a foundation width of 165 feet, it was inferred that the base averaging effect resulted in reductions in response spectral accelerations of foundation horizontal motions of about 15 to 25 percent in the 20 to 30 Hz range, 10 percent at 10 Hz, and a negligible amount below 5 Hz [4, 6]. Incoherence could also introduce rotational components of motion in the foundation input motions. Data are not presently available to assess potential rotational motions.

7. CONCLUSIONS

7.1 Characterization of Free-Field Control Motion

The free-field control motion should be specified at the ground surface. In order to do a realistic job of predicting structural response, it is essential that the control motion provide a realistic estimate of the frequency content of earthquake ground motion accelerograms expected at the ground surface of the site being considered. Reasonable strong motion duration estimates are also important.

Both studies of inelastic structural response and soil-structure interaction point to the importance of adequately characterizing the frequency content of ground motion at frequencies lower than the structural elastic frequency, as well as at the structural elastic frequency. Moreover, frequency content over a broad frequency range is important for floor response spectra. Because frequency content as well as duration of ground motion are strongly dependent on site-specific factors, including earthquake source characteristics, source-to-site wave propagation characteristics, and local soil conditions, one should emphasize the importance of site-specific ground motion characterizations, rather than using standard, non-site-specific characterizations such as the Reg. Guide 1.60 response spectra shapes.

The Reg. Guide 1.60 response spectrum appears to provide an adequate characterization of the ground motion in those cases where one expects broad frequency content ground motion in which $f_{10} < 2.0$ Hz and $f_{90} > 5.0$ Hz (see Figure 1 for definition of f_{10} and f_{90}) and the mean frequency lies between 3.0 and 4.8 Hz. Many accelerograms with strong motion durations greater than 3 seconds have these characteristics and for stiff structures, the effect of these accelerograms can be adequately and slightly conservatively predicted by the Reg. Guide 1.60 spectrum.

However, for narrower frequency accelerograms characteristic of records with strong durations less than 3 seconds, the Reg. Guide 1.60 spectra do not do an adequate job of predicting structural response particularly in the cases of soil-structure interaction or inelastic structural behavior. Even in these cases, the standard Reg. Guide 1.60 spectral shapes provide a generally conservative design basis. In some cases, the Reg. Guide 1.60 motion provides an overwhelmingly conservative response. In other cases, it provides a moderately unconservative response.

In developing a site-specific ground motion characterization, it is essential to recognize and allow for the uncertainty in ground motion characteristics. Uncertainty in site-specific spectra is appropriately incorporated by specifying a smooth design response spectrum at a reasonably conservative level (typically 84th percentile level). The smooth spectrum is intended to cover a reasonable range of ground motion spectral characteristics, in terms of both the amplitudes and the frequencies of the spectral peaks, that could occur at a site for a given design earthquake. Such spectra may be developed based on statistical analysis of ground motion data recorded under similar conditions, supplemented as appropriate by analytical studies of site-specific factors (i.e., earthquake rupture, source-to-site wave propagation, and/or local site response). Having developed a smooth design response spectrum, acceleration time histories compatible with this spectrum and of realistic duration for the design event may be selected or developed. For a given design earthquake and component of motion, the time histories may consist of either a single artificial time history whose spectrum envelops the design spectrum, or multiple recorded time histories whose spectra differ individually but collectively envelop the design spectrum. For a realistic appraisal of nonlinear structural response, the use of recorded time histories is preferable to use of an artificial time history.

7.2 Characterization of Variations of Ground Motion with Depth

Observational data indicate that, in general, both peak accelerations and response spectra decrease significantly with depth in the depth range of typical embedment depths of nuclear power plant structures. Comparisons of data and analysis indicate that deconvolution procedures assuming vertically propagating shear waves provide reasonable and apparently somewhat conservative estimates of the variations of ground motion with depth. The practice of excluding ground motion variations with depth, as has been done in a number of instances in nuclear power plant design practice, is not founded on a physical basis and appears to uniformly lead to additional conservatism and overestimation of structural response. Therefore, it is concluded that it is appropriate to allow for variations of ground motion with depth in characterizing foundation input motions and carrying out SSI analyses for embedded structures, and that current wave propagation analysis techniques may appropriately be used.

In practice, soil-structure interaction analyses typically include analyses for parametric variations in soil properties about best-estimated values. Soil property variations also result in variations in the calculated ground motions with depth in deconvolution analyses. It is believed that incorporating such soil property variations is an appropriate way not only to include effects of uncertainties in the properties on foundation stiffness and inertial soil-structure interaction, but also to reasonably include effects of uncertainties in the characterization of ground motion variations with depth.

If one takes credit for the reduction of translational foundation input motion with depth, then rotational foundation input components should be included. If one places the control motion at the foundation level, then rotational components may be ignored. Again note, the latter option often leads to excessive conservatism.

7.3 Characterization of Variations of Ground Motion in a Horizontal Plane

For apparent horizontal velocities in excess of 2.5 km/sec, it appears that, in most cases, effects of phase differences in ground motion in a horizontal plane on structural translational and rocking responses are small enough to be neglected and effects on torsional response are adequately incorporated by the common design practice of assuming a 5 percent accidental eccentricity of the induced base shear force. Thus, in general, it would appear to be satisfactory to analyze assuming vertically propagating waves except for providing a nominal 5 percent accidental eccentricity. However, a more significant torsional response due to non-vertically incident waves may occur for the case of a high-frequency, narrow-banded, short duration ground motion input to a structure on a soil site.

Effects of incoherence on foundation input motions could be incorporated as a frequency-dependent reduction in the translational input motions. Although the amount of reduction is not well quantified at present due to limited data, it appears that for a relatively homogeneous site and foundation width of 165 feet, the averaging effect is negligible below 5 Hz and results in reduction of about 10% at 10 Hz, and 15% to 25% in the 20 to 30 Hz range. These reductions increase with increasing foundation size and increasing foundation rigidity relative to the underlying soil and would be larger for less homogeneous sites. Data are not currently available to ascertain effects of incoherence on rotational foundation input motions.

ACKNOWLEDGMENTS

The USNRC-sponsored research work on which this paper is primarily based was conducted by Woodward-Clyde Consultants, Walnut Creek, California as prime contractor and by Structural Mechanics Associates, Inc., Newport Beach, California, Structural and Earthquake Engineering Consultants, Inc., Sierra Madre, California, and Interpacific Technology, Inc., Oakland, California as primary subcontractors. Project consultants were W.J. Hall of the University of Illinois, Champaign, J.E. Luco of the University of California, San Diego, J.M. Roesset of the University of Texas, Austin, H.B. Seed of the University of California, Berkeley, and N.C. Tsai of NCT Engineering, Inc., Lafayette, California. J.F. Costello was the technical representative of the USNRC for this research project.

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TABLE I COMPARISON OF FREQUENCY DOMAIN, DURATION, AND ACCELERATION PARAMETERS

	EARTHQUAKE RECORD (COMPONENT)	FREQUENCY RANGE (Hz)		MEAN FREQ. Ω(Hz)	T'_D (sec)	A_I (g)	RMS BASED A_{DE} (g)
		f_{10}	f_{90}				
1	Olympia, WA., 1949 (N86E)	1.20	6.10	3.90	15.6	0.281	0.202
2	Taft, Kern Co., 1952 (S69E)	1.10	5.50	3.61	10.3	0.180	0.155
3	El Centro Array No. 12 Imperial Valley, 1979 (140)	0.55	7.50	4.52	9.6	0.142	0.133
4	Artificial (R.G. 1.60)	0.60	6.55	3.91	—	—	—
5	Pacoima Dam, San Fernando, 1971 (S14W)	0.75	6.70	4.19	6.1	1.170	0.795
6	Hollywood Storage PE Lot, San Fernando, 1971 (N90E)	0.75	7.90	4.68	5.4	0.211	0.213
7	El Centro Array No. 5 Imperial Valley, 1979 (140)	0.80	6.75	4.12	3.4	0.530	0.404
8	UCSB Goleta Santa Barbara, 1978 (180)	0.80	3.05	2.34	3.0	0.347	0.332
9	Gilroy Array No. 2 Coyote Lake, 1979 (050)	2.70	6.90	5.12	2.2	0.191	0.202
10	Cholame Array No. 2, Parkfield, 1966 (N65E)	1.20	2.75	2.34	1.4	0.490	0.514
11	Gavilan College Hollister, 1974 (S67W)	2.55	11.35	7.67	1.1	0.138	0.106
12	Melendy Ranch Barn, Bear Valley, 1972 (N29W)	3.55	8.20	6.11	0.8	0.520	0.435

TABLE II COMPARISON OF ACTUAL SPECTRAL ACCELERATIONS TO R.G. 1.60 SPECTRAL ACCELERATIONS ANCHORED TO "EFFECTIVE ACCELERATION", A_{DE}

	Earthquake Record (Component)	$(SA_{\mu_a}/SA_{\mu_{1.60}})$*								
		μ = 1.0			μ = 1.85			μ = 4.27		
		MAX.	MEDIAN	MIN.	MAX.	MEDIAN	MIN.	MAX.	MEDIAN	MIN.
1	Olympia, WA., 1949 (N86E)	1.21	0.98	0.76	1.15	1.01	0.88	1.18	1.04	0.67
2	Taft, Kern Co, 1952(S69E)	1.18	0.86	0.59	1.05	0.86	0.64	1.05	0.91	0.82
3	El Centro Array No. 12 Imperial Valley,1979(140)	1.05	0.83	0.59	1.00	0.85	0.68	1.03	0.84	0.71
5	Pacoima Dam San Fernando, 1971(S14W)	1.16	0.90	0.59	1.06	0.97	0.51	1.06	0.94	0.56
6	Hollywood Storage PE Lot, San Fernando, 1971(N90E)	1.25	0.94	0.49	1.14	1.02	0.57	1.20	1.03	0.59
7	El Centro Array No. 5 Imperial Valley, 1979(140)	1.29	1.05	0.70	1.24	1.15	0.65	1.32	1.06	0.67
8	USCB Goleta Santa Barbara, 1978(180)	1.13	0.70	0.59	1.35	0.73	0.62	1.63	0.77	0.64
9	Gilroy Array No. 2, Coyote Lake 1979(050)	1.33	0.89	0.29	1.24	0.95	0.32	1.22	0.93	0.32
10	Cholame Array No. 2, Parkfield 1966(N65E)	1.27	0.52	0.40	1.41	0.53	0.46	1.40	0.60	0.51
11	Gavilan College Hollister, 1974(S67W)	1.64	0.57	0.26	1.20	0.80	0.17	0.96	0.50	0.10
12	Melendy Ranch Barn, Bear Valley 1972(N29W)	1.50	0.91	0.11	1.49	1.16	0.13	1.44	0.87	0.14

*Ratios of Actual to Reg. Guide 1.60 Spectral Accelerations are reported for the frequency range of 1.8 to 10 Hz.

TABLE III MAXIMUM STORY DRIFT DUCTILITIES, $(\mu_S)_M$, IN INTERNAL STRUCTURE VERSUS ELASTIC DEMAND/CAPACITY RATIO, $(F)_M$

EARTHQUAKE	FIXED BASE		STIFF SOIL PROFILE		INTERMEDIATE SOIL PROFILE	
	$(F)_M$	$(\mu_S)_M$	$(F)_M$	$(\mu_S)_M$	$(F)_M$	$(\mu_S)_M$
ARTIFICIAL	2.67	11.9	1.62	9.2	1.06	1.2
EL CENTRO #5	2.19	5.6	1.12	1.7	0.64	ELASTIC
PARKFIELD	1.29	3.2	1.85	12.9	1.44	6.3
MELENDY RANCH	2.97	4.7	0.60	ELASTIC	0.65	ELASTIC

TABLE IV EFFECT OF EXCLUDING KINEMATIC INTERACTION ON BASE SHEAR FORCE IN CONTAINMENT SHELL

Input (Control) Motion	Ratio of Peak Base Shear Force in Containment Shell without Kinematic Interaction to Peak Base Shear Force with Kinematic Interaction*			
	Soil Profile I	Soil Profile II	Soil Profile III	Soil Profile IV
R.G.1.60 Artificial	1.03	1.07	1.22	1.08
Melendy Ranch Comp. N61°E	1.07	1.11	1.67	1.79
Comp. N29°W	1.07	1.20	1.54	1.23
El Centro Sta. No. 5 Comp. N140°E	1.05	1.12	1.32	1.07
Comp. N230°E	1.04	1.08	1.18	1.10
Parkfield Sta. No. 2 Comp. N65°E	1.02	1.06	1.17	1.09

*from soil-structure interaction analyses for a reactor building embedded at a depth of 40 ft (Luco et al., 1986, Vol. 4 of NUREG/CR-3805)

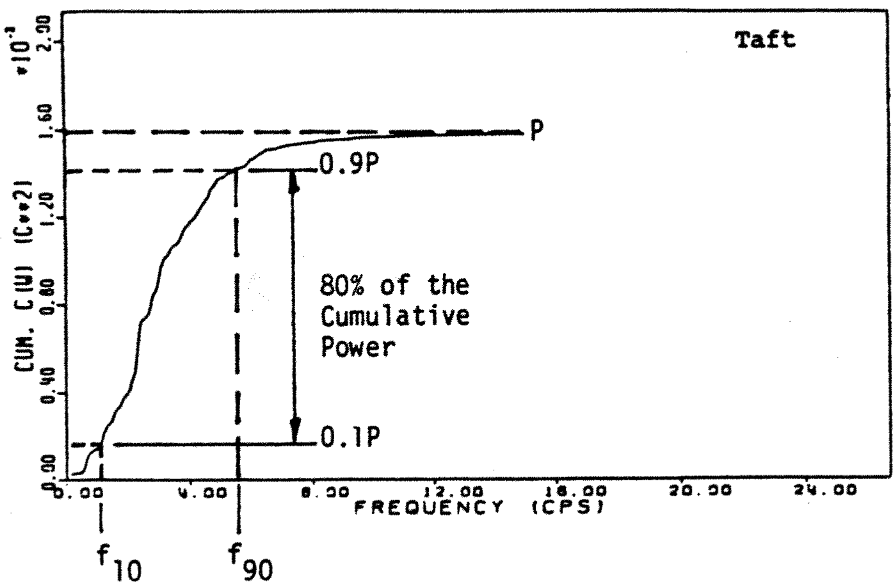


FIGURE 1. CUMULATIVE POWER SPECTRAL DENSITY FUNCTION OF THE TAFT ACCELEROGRAM

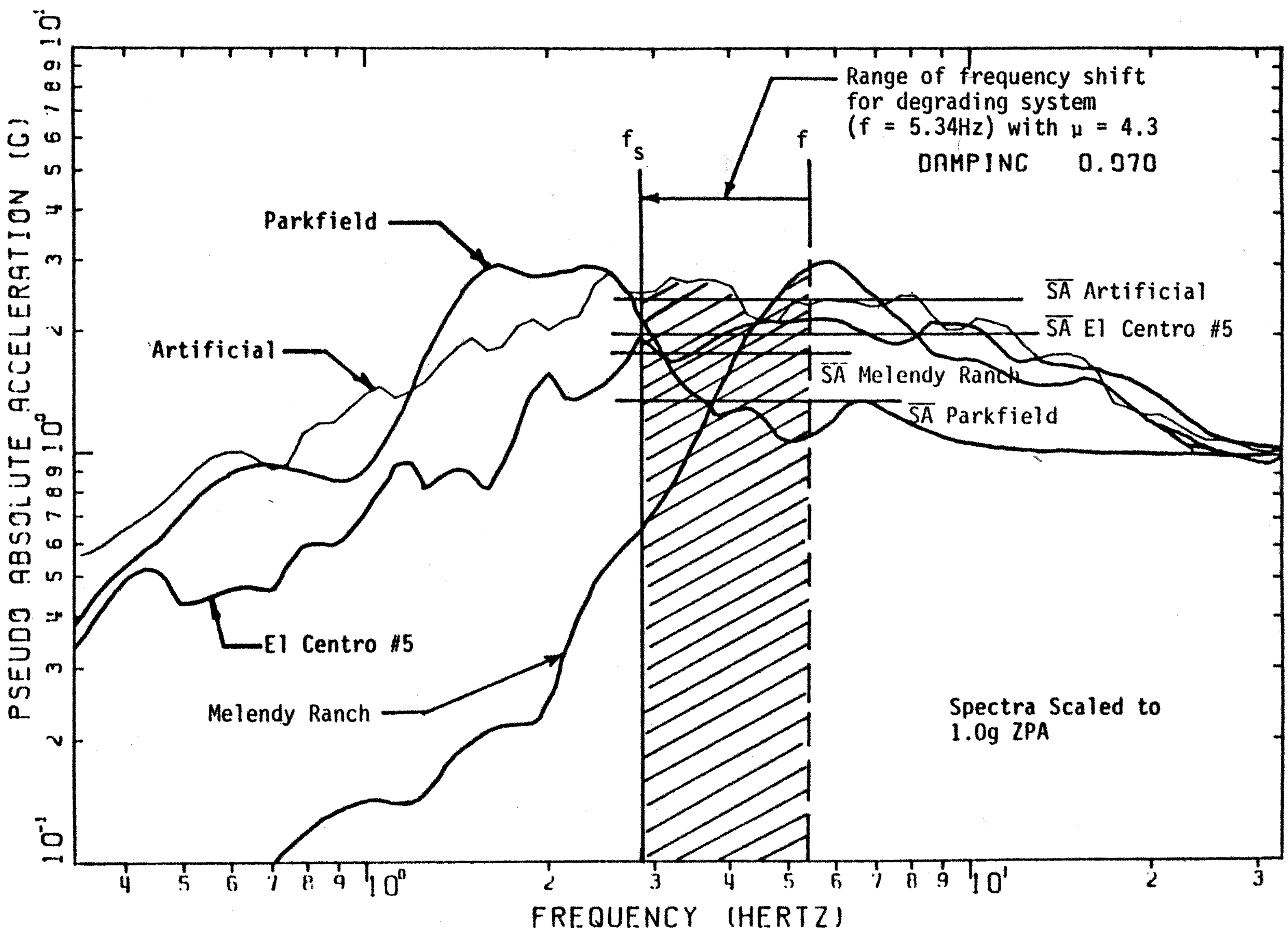


FIGURE 2. COMPARISON OF ELASTIC RESPONSE SPECTRUM FOR ARTIFICIAL, EL CENTRO #5, PARKFIELD, AND MELENDY RANCH EARTHQUAKES

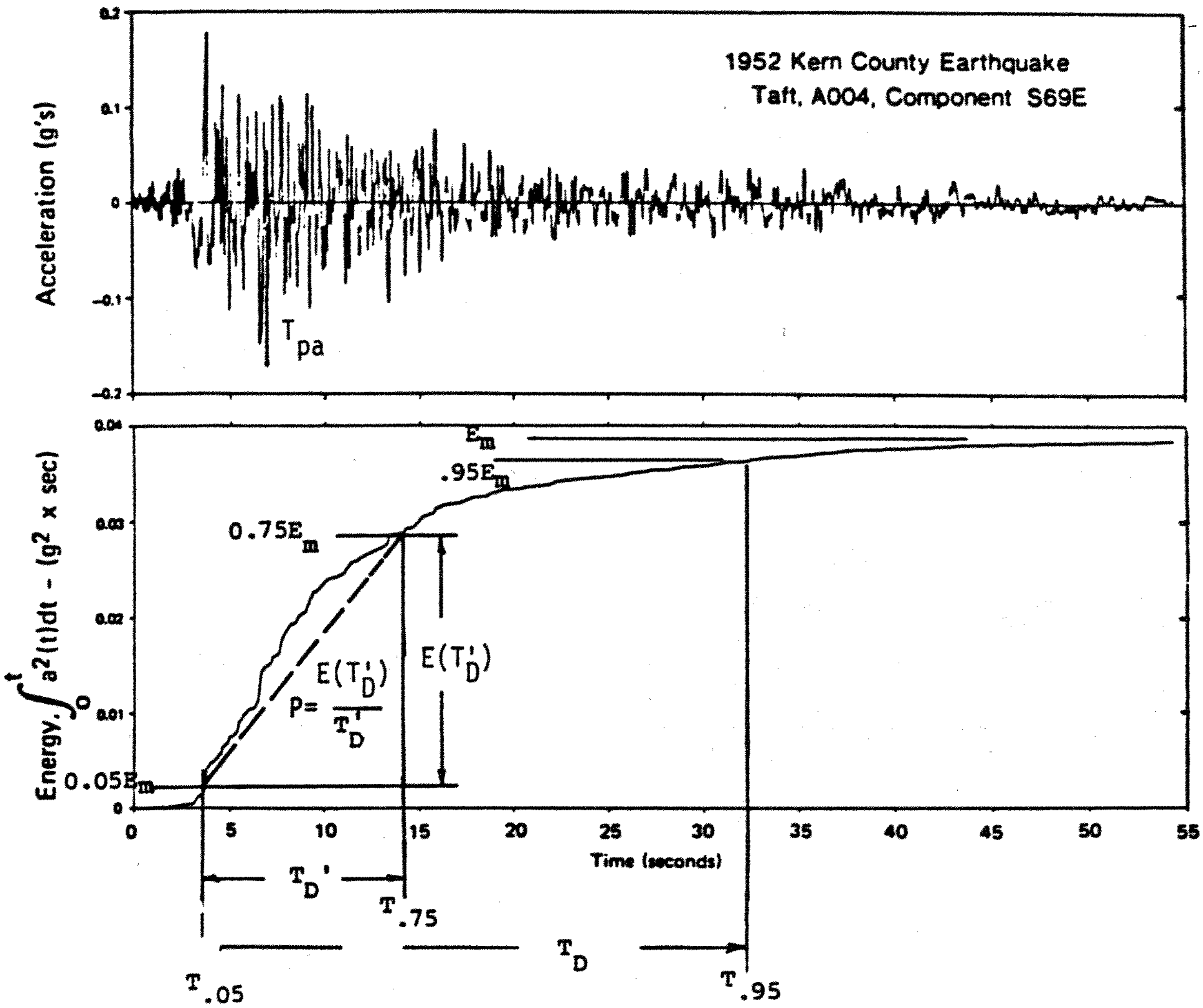


FIGURE 3. ACCELEROGRAM AND VARIATION OF CUMULATIVE ENERGY WITH TIME FOR THE 1952 KERN COUNTY EARTHQUAKE RECORDING AT TAFT

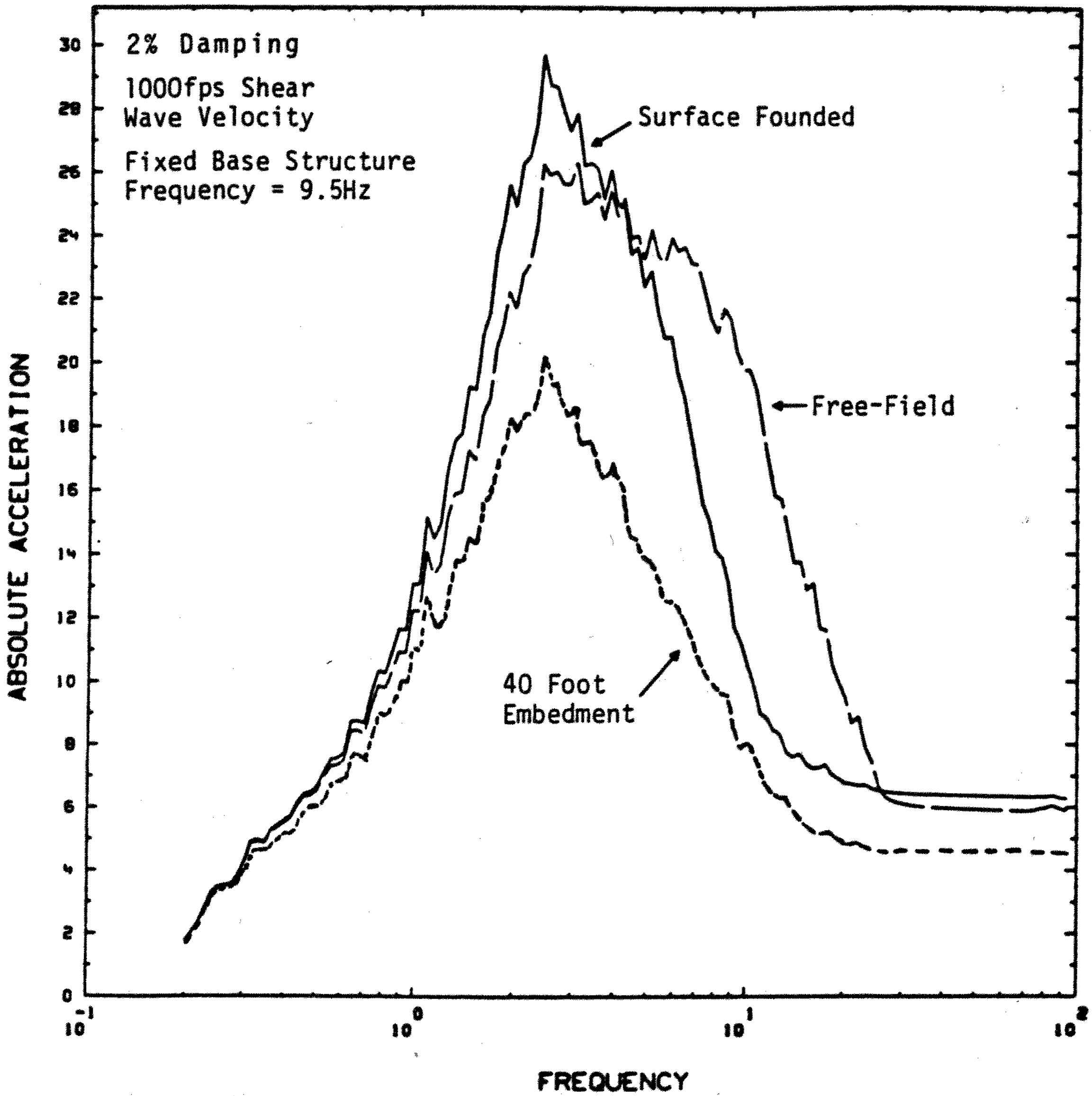


FIGURE 4. ANALYTICALLY COMPUTED INPUT SPECTRA AT BASE OF STIFF STRUCTURE DUE TO SSI EFFECTS (FREQUENCY SHIFT & INCREASED DAMPINGS) & KINEMATIC INTERACTION

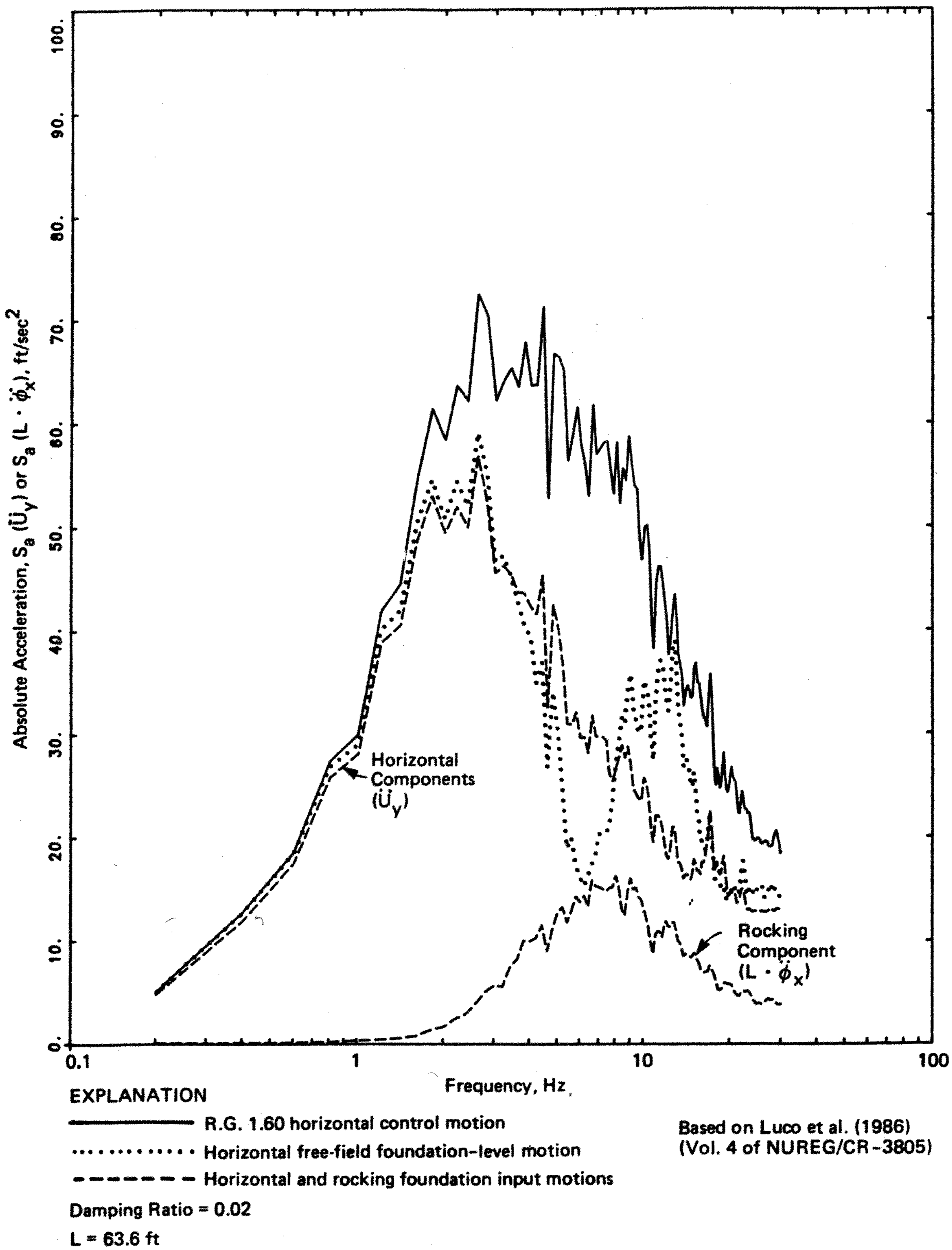
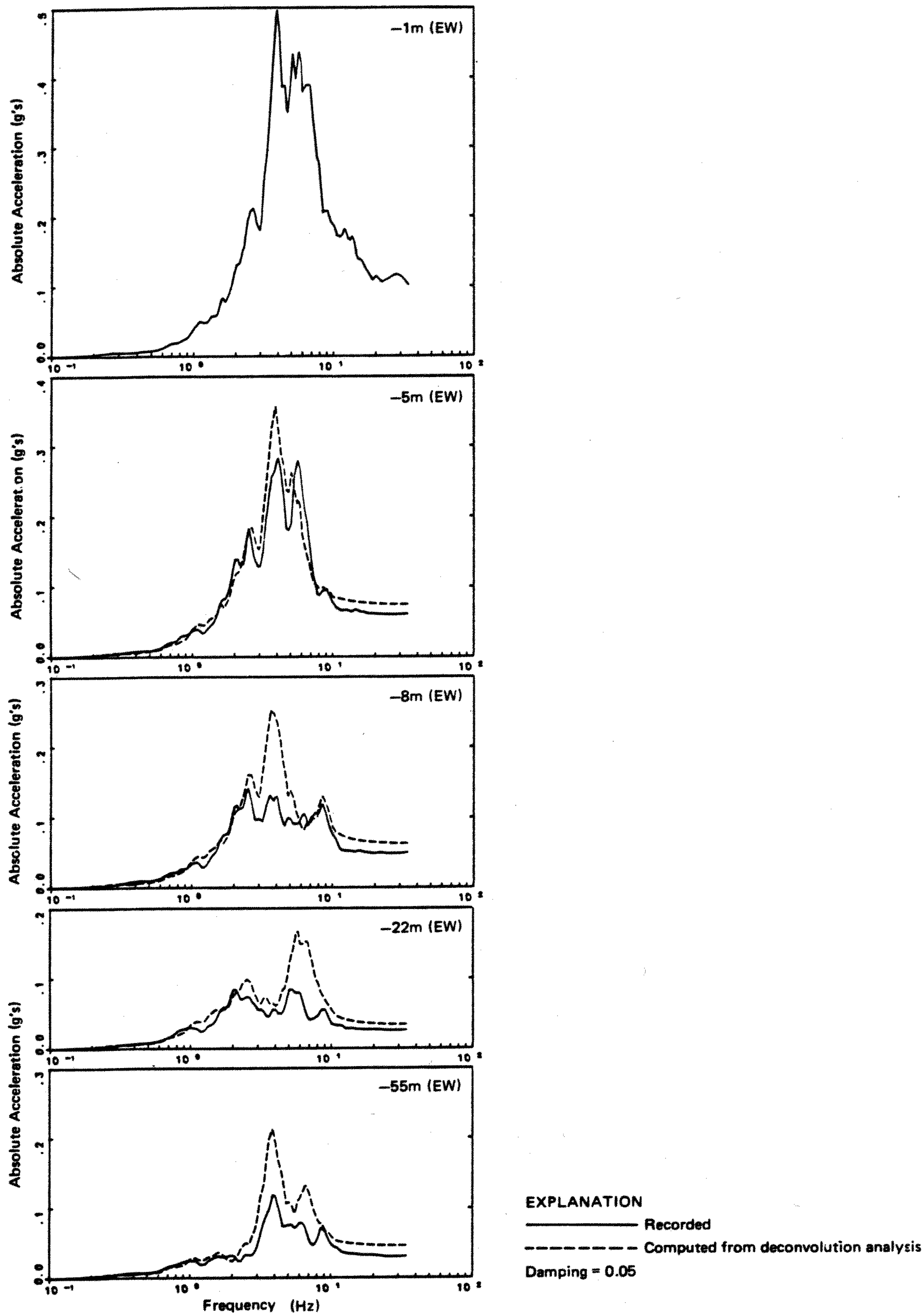


FIGURE 5. COMPARISON OF RESPONSE SPECTRA OF REG. GUIDE 1.60 HORIZONTAL CONTROL MOTION, HORIZONTAL FREE-FIELD FOUNDATION-LEVEL MOTION, AND FOUNDATION INPUT MOTIONS, (REACTOR BUILDING, 40 FT EMBEDMENT, SOIL PROFILE IV, VERTICALLY INCIDENT WAVES)



From Chang et al. (1986)
(Vol. 3 of NUREG/CR-3805)

FIGURE 6. COMPARISON OF RESPONSE SPECTRA OF RECORDED AND COMPUTED MOTIONS, DECONVOLUTION ANALYSIS, EW COMPONENTS, NARIMASU SITE

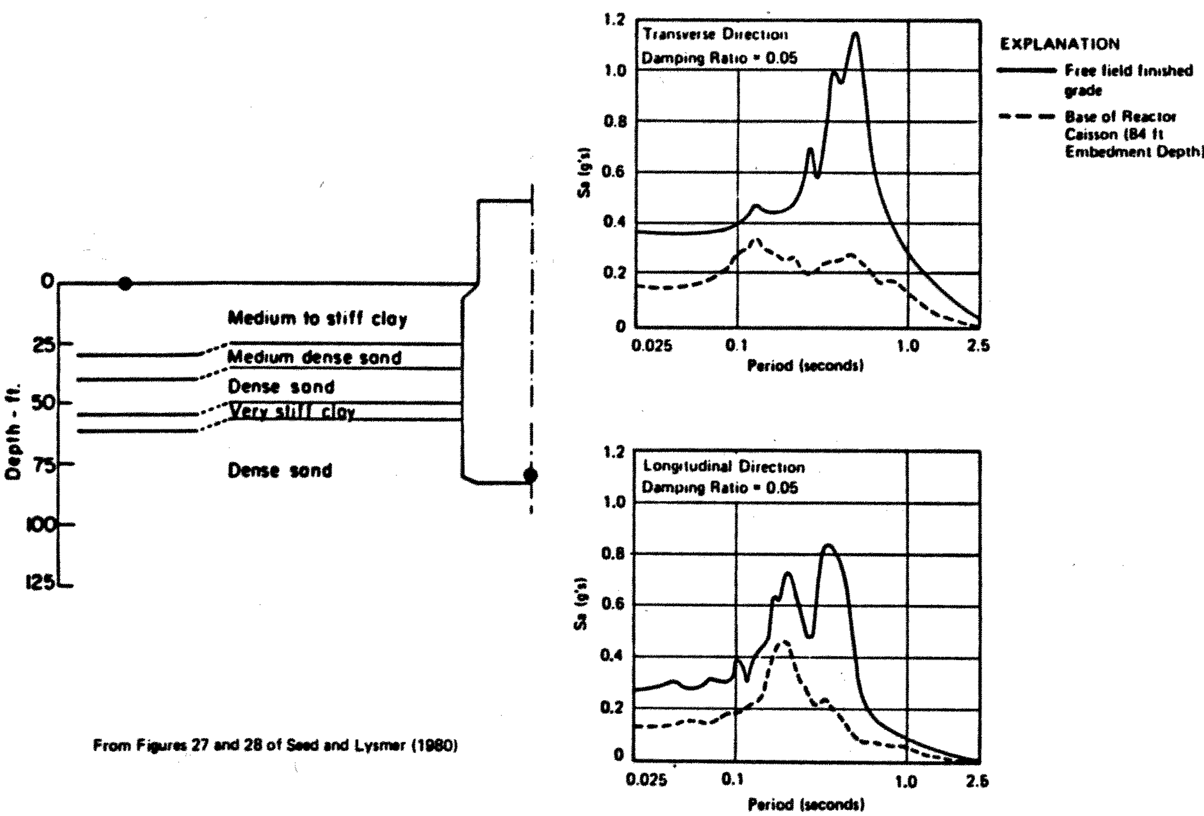
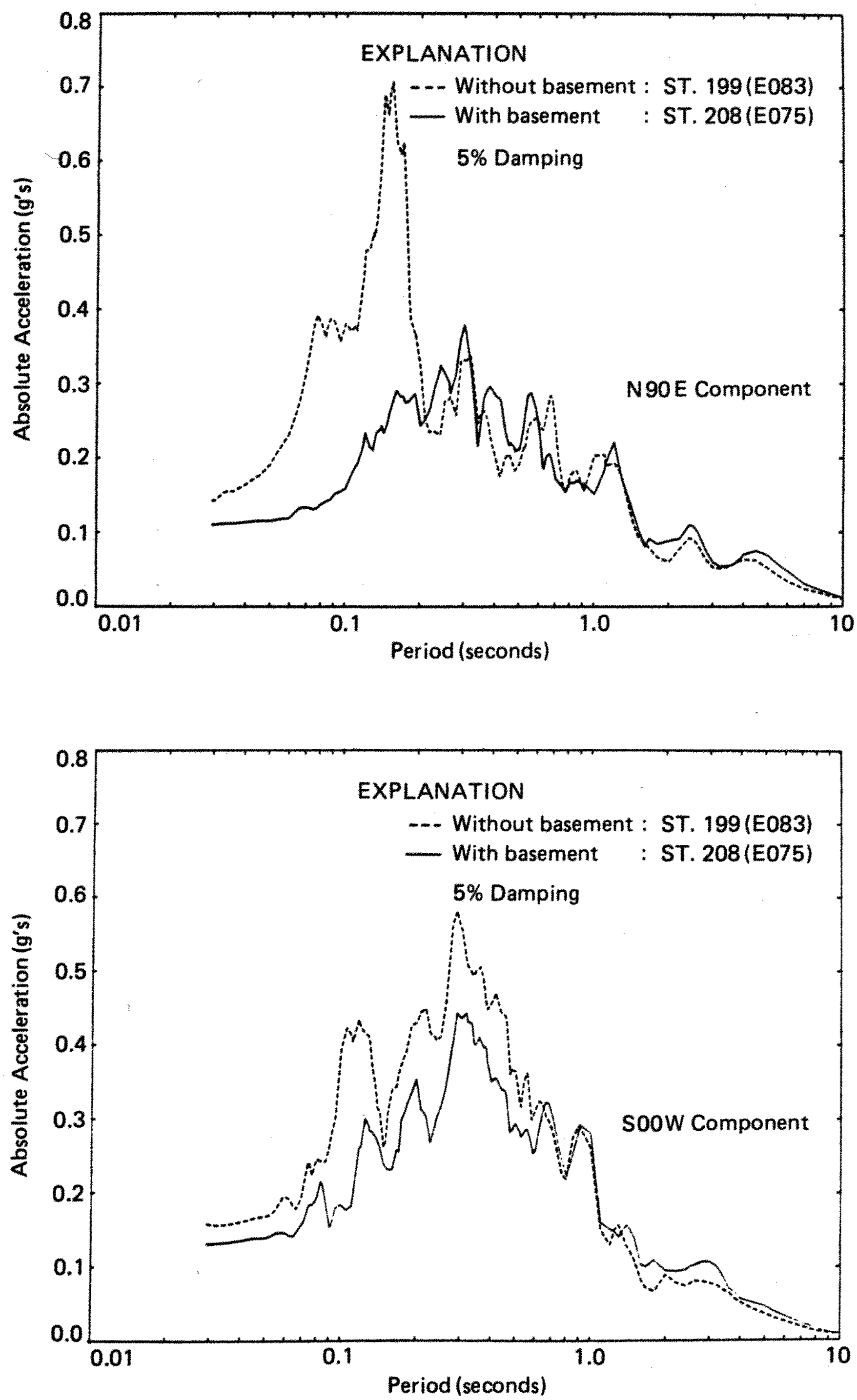
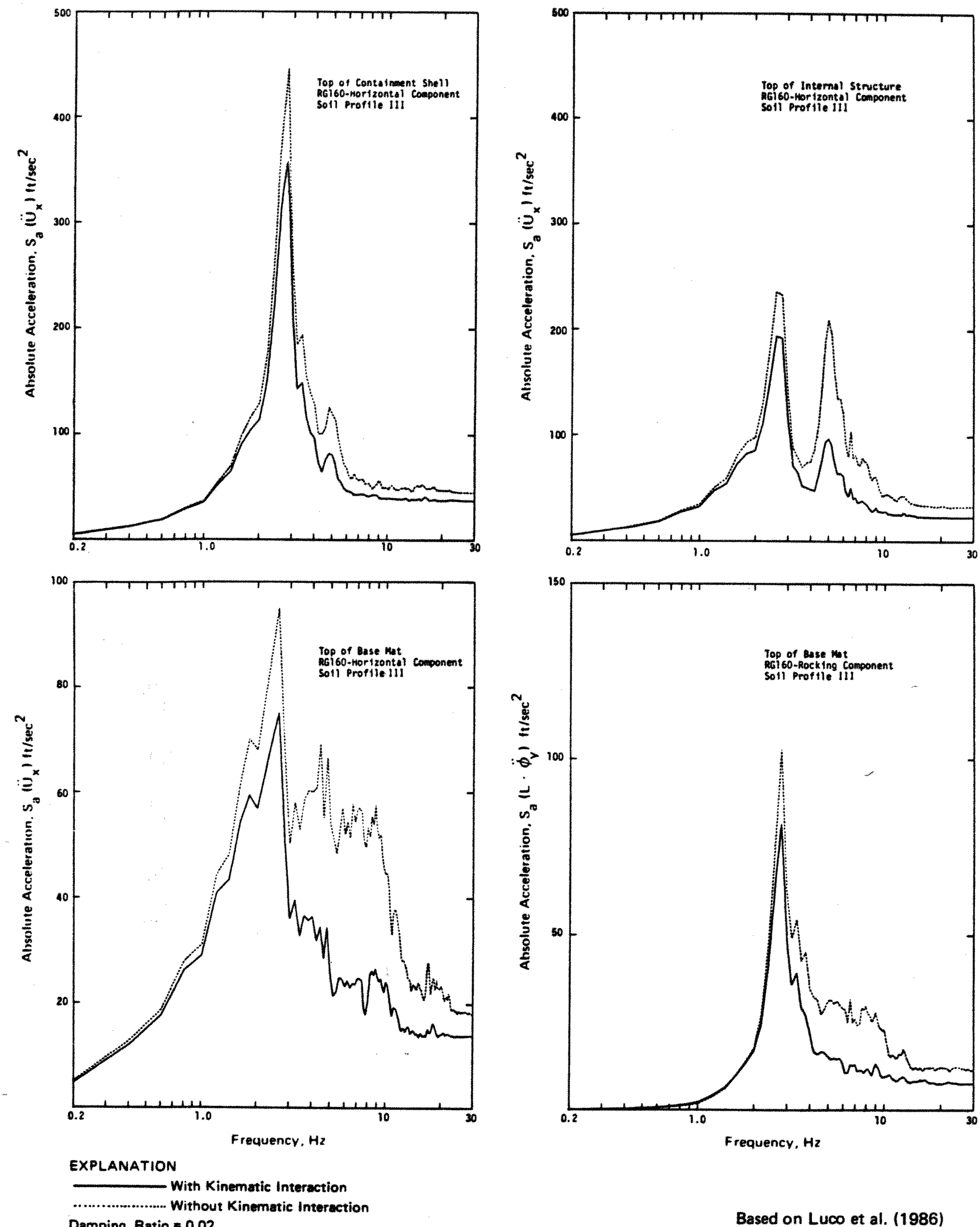


FIGURE 7. COMPARISON OF RESPONSE SPECTRA OF ACCELEROGRAMS RECORDED AT FINISHED GRADE IN THE FREE FIELD AND AT THE BASE OF THE REACTOR CAISSON AT THE HUMBOLDT BAY PLANT DURING THE JUNE 6, 1975, FERNDALE, CALIFORNIA EARTHQUAKE (HORIZONTAL COMPONENTS)



From Chang et al. (1986)
(Vol. 3 of NUREG/CR–3805)

FIGURE 8. COMPARISON OF RESPONSE SPECTRA OF MOTIONS RECORDED AT STATIONS 199 (WITHOUT BASEMENT) AND 208 (WITH BASEMENT) DURING THE 1971 SAN FERNANDO EARTHQUAKE



Based on Luco et al. (1986)
(Vol. 4 of NUREG/CR-3805)

FIGURE 9. COMPARISON OF FLOOR RESPONSE SPECTRA OF REACTOR BUILDING FROM ANALYSES WITH AND WITHOUT CONSIDERATION OF KINEMATIC INTERACTION, ARTIFICIAL R.G. 1.60 EXCITATION, SOIL PROFILE III (40 FT EMBEDMENT, VERTICAL INCIDENCE)

SPECIFICATION OF GROUND MOTION INPUT FOR SSI ANALYSES

by

Jose M. Roesset[1]

The effects of local soil conditions on the characteristics of the earthquake motions to which a structure may be subjected are normally studied in three separate phases:

- the amplitude and the frequency content of the seismic motion at the free surface of a soil deposit, before any structure is built, are functions of the soil properties in the linear elastic and the inelastic ranges. This effect is commonly known as soil amplification although the name may be misleading, since there is in fact amplification over certain ranges of frequencies and deamplification over others.
- the motion of a massless foundation, before any structure is built or if the structure had no mass, would be different from that recorded at the free surface of the soil. The differences will consist in general of a filtering of the translational motions (a decrease of their amplitude in the high frequency range) and the occurrence of rotational components of motion. The only case where this effect would not be present is for a surface foundation and vertically propagating seismic waves.
- once the structure is built, or when its mass is taken into account, the inertia forces generated by the structural vibrations will give rise to base shears, an axial force and overturning and torsional moments. Unless the soil is extremely stiff these base forces will result in additional deformations which will alter the motion of the foundation. The acceleration at the base of the structure will thus be different from the one that would be recorded on the free surface of the soil and the one that the foundation would experience by itself.

These last two effects are commonly known as soil structure interaction. The first one is called sometimes kinematic interaction (function of the geometry of the foundation and the types of seismic waves propagating through the soil), while the second is referred to as inertial interaction (caused by the inertia forces in the structure). It appears that in some cases, however, the term soil structure interaction is used only in reference to the second phenomenon.

While this division into three separate phases is convenient from a conceptual point of view and to check and interpret the results of seismic SSI analyses, it is important to remember that the effects must take place simultaneously and are very closely related. If separate analyses are performed for two or all three of these effects, consistent assumptions should be used to obtain results which are physically reasonable.

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This paper is concerned primarily with the effect of the soil on the seismic motions in the free field (the first effect discussed above). Some discussion of kinematic interaction effects and the way to model them is, however, necessary in order to address the following questions which have been asked of the presenters in this session:

- Where are the best locations for ground motion input to SSI? How does the use of site specific or generic spectra (e.g. R.G. 1.60) affect the choice of location?
- How should the design ground motion be characterized for input to SSI in relation to the free field motion?
- To what extent do localized conditions have to be reflected in the input (e.g. variation of soil properties, sloping bedrock)?
- How good are the available techniques in modifying the free field input (e.g. one dimensional wave propagation vs. inclined waves)? What is the importance of rotational components and how should they be addressed?

The first two questions are closely related and will be addressed by discussing the ways in which the input can be defined in the free field, the location where this input should be specified and the procedures that should be used to obtain the motions at the foundation level for the inertial interaction analysis. The last two questions refer to the state of the art in soil amplification and kinematic interaction studies, or more generally wave propagation analysis, and they will be discussed next.

Specification of the Seismic Input in the Free Field

There are six different ways in which the seismic input can be defined in the free field, as shown schematically in Table 1.

1. It is common practice today to specify the seismic input in terms of a peak ground acceleration (or an effective acceleration) and a standard (generic) family of response spectra for various values of damping. Artificial earthquakes are then generated to match the response spectra within certain tolerances. Three statistically-independent motions are generated: two with the same intensity for two perpendicular horizontal directions, and a third one, scaled by a factor of 2/3 for the vertical component. These synthetic records can then be used for time history analyses.

This procedure ignores clearly the effects of the earthquake mechanism, magnitude and distance, the general topographic and geologic features between the fault and the site, local soil conditions and orientation of the building with respect to the fault on the frequency content of the motions. Because the R.G. 1.60 rules to construct design spectra are based on a statistical analysis of a large number of actual earthquake records, corresponding to different conditions (magnitude, distance, soil profiles) they are supposed to provide, within a certain probability level, a conservative upper bound to the motions that can be expected at any site. If this conservatism applied not only to the design spectra but also to the results of SSI analyses, while the soil structure interaction effects might be substantially different for actual or site specific records, the use of the synthetic accelerograms based on the R.G. 1.60 spectra would be a reasonable choice, when more information is not available, particularly for soil deposits that can be classified within the all encompassing and conveniently vague term of firm ground. The design spectrum should then apply at the free surface of the soil deposit. Some recent studies have shown that results of soil structure interaction analyses for some actual records may be larger than those obtained for artificial earthquakes based on the R.G. 1.60 generic spectrum with the same effective acceleration. The reasons for these larger values need further clarification.

Apart from this question the main problem with this procedure is that the degree of conservatism involved is hard to quantify: it may be very large in some occasions and nearly nonexistent in others, leading to a lack of uniformity in safety requirements.

2. A variation from the previous approach that represents some improvement is to use more than one standard spectrum for a given level of acceleration. Following, for instance, the recommendations of the ATC-3 one could specify a spectrum (or a family of spectra for various values of damping) for rock, another one for firm ground and a third one for very deep soil deposits. The ATC-3 recommends in fact two different spectra for the latter case, depending on the effective acceleration. While this procedure still ignores the effects of magnitude and distance (as well as overall topographic features and building orientation) on the frequency content of the motions, it includes, at least, in some average sense, the effect of local soil conditions. Additional and significant improvements could be easily introduced through some further research, which makes it an attractive and relatively simple alternative. Again in this case the spectra should be specified at the free surface of the soil.

3. A third alternative would be to specify the design earthquake in terms of more than one parameter; so, for instance, instead of using only the effective ground acceleration to characterize the earthquake, one could use peak ground acceleration, velocity and displacement. For nuclear power plants acceleration and velocity might be sufficient for practical purposes since the range of frequencies over which the value of the ground displacement is important is rarely of interest. Smooth design response spectra based on these three quantities could then be derived, for different values of damping, using rules like those suggested by Newmark and Newmark and Hall among others. This approach has the potential to include many of the effects neglected in the previous two if one could find appropriate functional relationships between peak ground acceleration, velocity and displacement and earthquake mechanism, magnitude, distance, soil conditions etc. Unfortunately this information is not available at present and existing expressions relating these three parameters to magnitude and distance (neglecting other effects) give sometimes unrealistic results. Research in this area using both empirical data and analytical earthquake models could greatly improve the state of the art and make this approach very attractive. The corresponding earthquake parameters and the resulting spectra would apply, once more, at the free surface of the soil.

4. An even simpler approach would be to use real earthquake records corresponding to magnitudes, distances and general and local soil conditions similar to those of the situation at hand. These records could then be used directly, performing the analyses for various earthquakes instead of a single one, or could serve as a basis to construct appropriate design spectra. In both cases the motions would apply again at the free surface of the soil. The main problem with this approach at the present time is the availability of a sufficient number of actual records to provide an adequate data base. As the network of instruments around the world expands and the number of recorded earthquakes increases steadily this kind of information may become available. Synthetic accelerograms based on physical models of the rupture along a fault may also help to complement recorded data. On the other hand scaling site specific records to a desired acceleration does not provide a very accurate or reliable means to increase the data base.

5. All four of the above approaches would specify the design earthquake in terms of response spectra or time histories of acceleration at the free surface of the soil. None of them requires an explicit definition of the type of seismic waves that generate the motions, although, as will be discussed later, this information would be necessary always for the kinematic interaction analyses (to determine the motions around the foundation). A slightly different alternative that has been suggested and used in the past is to specify the design spectrum at the free surface of a hypothetical rock outcropping (using for instance rock spectra as those suggested by Mohraz). For a soil profile where bedrock can be clearly identified at a finite (and not too large) depth, wave propagation analyses can then be conducted to determine compatible motions at any point in the free field and at the free surface in particular. This approach allows therefore to construct site specific spectra for each individual case. It requires, however, some indication as to the type of waves that constitute the earthquake. At present this method is used in practice for shallow or only moderately deep soil deposits assuming vertically propagating body waves (shear waves for the horizontal motions and compressional waves for the vertical component). When the depth to bedrock is not clearly defined, or is very large, the use of this procedure is less attractive and the definition of an alternative spectrum for deep soil deposits, as in the second approach discussed earlier would be more appropriate.

6. A final alternative would be to specify the design motions in the free field in terms of wave amplitudes and angle of incidence as a function of frequency. While this would be the most desirable approach for soil structure interaction analyses a complete definition of the earthquake this way may be extremely difficult and cumbersome. It would be necessary to define where these waves and amplitudes apply; the most logical choice would be in rock but this would require again, as in the previous approach, the existence of bedrock at a known depth, with the same problems and limitations. Moreover if there is internal dissipation of energy in the soil, as would be normally the case, the horizontal location would also have to be specified since the amplitudes would decay with horizontal distance (this may be an important consideration when analyzing simultaneously various buildings). In spite of these difficulties some simplified form of this approach is

possible and is in fact needed in practice for kinematic interaction analyses, to study the effect of adjoining structures and for the seismic analysis of bodies with large dimensions (pipes and conduits, bridges, very large mats supporting more than one building etc.).

Location of the Control Motion

Six different alternative have been suggested in the past for the location where the design motion is specified. They are:

1 - Free surface of the soil deposit at the site.
2 - Free surface (outcropping) of rock.
3 - Foundation level accounting for possible excavation but without structure (or for a massless structure).
4 - Foundation level of structure (including structure and its mass).
5 - Bedrock (interface between soil and rock at the site).
6 - Foundation level in the free field.

Of these six possibilities the first two are the most logical ones. As discussed in the previous section the design motion should be specified at the free surface of the soil when the input is defined by a generic (R.G. 1.60) type family of response spectra, by a set of standard site specific spectra, by the values of the peak ground acceleration, velocity and displacement at the site (including effects of local soil conditions) or by a collection of real earthquake records corresponding to similar conditions (the first four approaches). The second alternative corresponds to approaches 5 and 6 in the specification of the design motion. It is particularly appropriate for soil deposits of moderate thickness where the location of the bedrock can be clearly identified and it allows to conduct wave propagation studies to determine compatible site specific free field motions.

The third alternative would ignore entirely kinematic interaction effects due to embedment and shape of the foundation. For surface foundations and vertically propagating waves it is clearly the same as the previous two. For embedded foundations in rock with normal embedment ratios these effects may not be too important and this solution would still be reasonable. For embedded foundations in soil the approach will normally result in larger translational motions than could be realistically expected but in the absence of rotational components that would actually occur. Its use in this case is only justified when the motion is specified in terms of a generic (R.G. 1.60) type spectrum and the waves are assumed to propagate vertically (it makes little sense to follow this approach and to consider travelling waves). The main objections to this alternative are the same ones discussed in part 1 of the previous section; not only does it ignore theoretical knowledge and physical evidence but it provides a degree of safety or conservatism which is very hard to quantify; it can be very large (and excessive) for some situations and nonexistent for others. In addition it gives rise to serious inconsistencies when dealing with several buildings founded at different depths (or with different foundation types) and attempting to study the effects of adjoining buildings or the behavior of connecting elements (pipes, conduits etc.). Clearly an analysis that would model together the different structures and the surrounding soil would be meaningless with the motions specified independently for each foundation.

The fourth alternative would ignore all soil structure interaction effects (both kinematic and inertial interaction) and would represent a return to the approach of the past where structures were assumed to be on a rigid base. Neglecting soil structure interaction effects may be reasonable for structures founded on rock and horizontal excitation (for vertical motions inertial interaction effects may still be important due to the higher frequencies of interest and the large values of radiation damping). For other conditions and for very stiff and massive structures such as nuclear power plants ignoring SSI effects is equivalent to negating the research accomplishments of the last twenty years. Moreover, while the approach would be generally conservative when using generic R.G. 1.60 spectra (and the conservatism may be extreme in some cases), it could be unconservative when using site specific spectra or real earthquake records.

The fifth and sixth alternatives correspond to specification of the seismic input at points within the soil profile in the free field. Since the motions at these points are a function of the soil properties these approaches can only be applied rigorously when the input is specified in terms of the amplitudes and angles of the seismic waves (sixth procedure of the previous section), and wave propagation analyses are conducted for the site. Specification of the motion at bedrock (fifth alternative) would also be a reasonable approximation if the input was specified in terms of a rock spectrum and the modulus of the soil was substantially smaller than that of the rock. It would then be equivalent to specifying the motion at rock outcropping (second alternative). For all other cases, when the input is defined in terms of design spectra (generic or site specific for soils) or in terms of actual earthquake records these two approaches lack any logic and should be avoided. They would imply highly unreasonable motions at the free surface of the soil deposit and at other levels than the one where the motion is specified and would make again meaningless the study of more than one building simultaneously.

Determination of Foundation Motions

The input for an inertial soil structure interaction analysis must be the motions that would occur at the interface between the foundation and the soil before the structure is built. For a flexible foundation there are three translational components of motion at each interface point (normally the points at which the interface is discretized). For a rigid foundation, when these interface motions are condensed by imposing the constraints of a rigid body, there will be in general three translational and three rotational components of motion. It should be noticed that the foundation motions will generally be different from those that occur at any point in the free field. The differences are due to wave scattering and correspond to the kinematic interaction effect. The only exception is that of a surface foundation subjected to vertically propagating seismic waves. In this case the foundation motions will be the same as those on the free surface of the soil.

An explicit determination of the foundation motions is not necessary in a direct soil structure interaction analysis (when the structure and the soil are modelled together) or even in a substructure analysis (if the right-hand side of the equation of motion is expressed in terms of the motions at the foundation soil interface in the free field, without any excavation, and a stiffness matrix for the interface points also in the free field). These motions constitute however a set of intermediate results in the analysis that can provide an important insight into the nature and magnitude of interaction effects. They are extremely useful in the interpretation and assessment of the accuracy and reasonableness of the analysis procedure and the final results (as may be for instance the natural frequencies and mode shapes of the structure).

The foundation motions before the structure is built can be determined with the same techniques used to compute foundation stiffnesses or perform direct soil structure interaction analyses. There is a persistent misconception that discrete formulations (such as finite element models) can only be used for shallow soil profiles underlain by rigid rock, while the substructure, impedance or soil springs approach, is only applicable when dealing with an elastic half space. This mistake is apparently caused by the identification of each one of these procedures with past studies or applications and their simplest form rather than a clear understanding of the fundamentals and general potential of these methods. Whether the discretization extends over the soil domain with consistent lateral boundaries (as in the finite element models) or it is applied only to the interface between the foundation and the soil using appropriate Green's functions and an integral equation formulation (with weighted residual techniques, the direct or the indirect boundary element method) it is possible to obtain solutions to any dynamic soil structure interaction problem with any desired degree of accuracy. The only differences between these two approaches, when properly implemented, are those of convenience, availability of computer codes and economics. A solution based on the use of the continuous Green's functions is particularly appropriate when the soil can be considered as an elastic half space or can be reproduced by a small number of horizontal layers. Finite element type formulations or discrete Green's functions tend to be advantageous when a large number of layers is needed to reproduce properly the variation of soil properties with depth. Both approaches become more complicated and expensive when dealing with nonlinear problems (nonlinear soil behavior or nonlinear effects at the foundation soil interface), soil deposits where the properties vary in the horizontal direction as well as with depth, or when the stratum is not horizontally layered (sloping layers or bedrock, two or three dimensional geometries), although a finite element model is less affected by some of these variations.

Whatever the method used the determination of the foundation motions requires the specification of the types of waves that give rise to the motions. When the earthquake is not defined directly in terms of its wave content it will be necessary to make explicit or implicit assumptions in this respect. The most common assumption is that the seismic waves are propagating vertically, although special analyses are also conducted at times for horizontally travelling waves (or surface waves). The consideration of any wavetrain (or a combination of various wavefronts) offers no special difficulty.

The first partial studies on the effect of travelling waves were presented in 1970 by Yamahara, who pointed out the filtering effect of long rigid foundations on the translational motions, and by Newmark in 1969, who pointed out the existence of torsional components. The motions of embedded foundations caused by vertically propagating waves were studied by Elsabee and Morray who suggested approximate expressions for the transfer functions for the horizontal translation and the rotation of the foundation (assumed to be rigid) for a specified motion at the free surface of the soil deposit (Figure 1). As can be seen the translational motion decreases in amplitude with increasing frequency up to a frequency which is about 0.7 times the natural frequency of the embedment layer and remains almost constant (but with some fluctuations) for larger frequencies with an average value which is about 45% of the amplitude at the free surface. For relatively flexible structures on stiff soils or with a small embedment the reduction in the motion at the frequency of main interest (fundamental frequency of the soil structure system) will be very small and often negligible. For a stiff structure with significant embedment (in relation to the base dimension) and a soft soil the frequency of interest can be in the range of maximum reduction and the effect would be significant. It should be noticed, however, that the reduction in the translational motion is accompanied by the creation of a rocking component. The amplitude of the base rotation increases with frequency up to about the natural frequency of the embedment layer, then remains nearly constant with some fluctuations (over the range of frequencies studied). The average amplitude of the vertical motion at the edge of the foundation due to this rotation in the nearly constant range is about 25% of the amplitude of the horizontal motion at the free surface. It is important to keep in mind that these two effects occur always simultaneously. One should never allow for one and neglect the other. Considering the reduction in the translation without rotation would be unconservative; including the rotation but disallowing the decrease in the translation would be unreasonably conservative. Near the base of the structure (and near the axis when there are torsional components) kinematic interaction effects tend to be beneficial since there is a reduction in the amplitude of motion. Towards the top of the structure (or the edge) the combination with the rotational components may lead to higher amplitudes of motion than if the effects were neglected.

Additional and more extensive studies on foundation motions (both surface and embedded) under trains of body waves at different angles as well as surface waves have been conducted by Iguchi, Scanlan, Luco, Dominguez, Luco and Wong and Pais and Kausel among others. The results show in all cases coupled reductions in the translational motions and occurrence of rotational components (both rocking and torsional terms) with a reduction of some effects as the others increase. Figure 2 shows some typical results of the studies by Dominguez on rectangular foundations embedded in an elastic half space.

In spite of all these studies and the uniformity of their conclusions there is a continued reluctance to accept the results of kinematic interaction analyses. It appears that this resistance may be due in part to an unfortunate confusion between the foundation motions obtained from a proper interaction study and those that would occur at the foundation level in the free field (for a one dimensional wave propagation study). The latter would not have any rotational components. Moreover the transfer function for the free field motion at the foundation level will exhibit pronounced valleys at the natural frequencies of the embedment layer, while those in the transfer function for the foundation (at slightly different frequencies) are much less drastic. This behavior is further illustrated in Figure 3 that shows the response spectra at the free surface of the soil deposit, the foundation accounting for kinematic interaction and the foundation level in the free field.

A more legitimate source of concern is the fact that the details of the motions of the foundation and their frequency content (in particular the valleys in the transfer functions and the response spectra) are a function of the types of waves and the soil properties. Clearly if the analyses are performed for only one earthquake record, a unique type of waves and a single set of soil properties the results could be unconservative. On the other hand when performing analyses for three sets of soil properties (as is often required) the location of the valleys will be shifted and the envelope of the results should eliminate potential unconservatisms. Various approaches have been suggested in the past to account for the uncertainties in the types of waves, earthquake characteristics and soil properties:

1. to limit the amount of reduction in the translational motions of the foundation with respect to those at the free surface at each frequency. While this is not an optimum solution because it does not address the rotational (rocking or torsional) motions, it is reasonable in the absence of more detailed information.

2. to conduct the analyses not only for three different sets of soil properties, but also for more than one train of waves (if a small number of logical trains can be defined) and for more than one earthquake (particularly when dealing with actual records) and to envelop the results. If the results to be enveloped are the motions at the foundation the approach does not offer significant advantages over the previous one: it would be possible to envelop the response spectra of the translational motions and then to generate an artificial earthquake to match the smoothed spectra for the inertial interaction analyses. It is not as easy to envelop the rotational components of motion and still get reasonable results since the phases between the different components, which are important, would be lost. The most sensible approach would be to envelop (or interpret statistically) the results of the final interaction analyses. The main objection to this procedure is the large number of analyses that would have to be conducted and the associated cost. While this cost may not be as large as claimed if one uses a modal synthesis of the structure it is clear that some compromise must be reached as to the total number of combinations of soil properties, motions and types of waves that should be studied.

In spite of these difficulties an explicit (or implicit) determination of the effects of kinematic interaction, accounting for the effects of embedment and foundation shape to determine the foundation motions which will serve as input to the inertial interaction analyses is the most rational and sensible approach. While it is impossible to eliminate entirely the uncertainties in this phase, as in all other phases, of seismic analysis an improved knowledge of the types of waves that can be reasonably expected at a site and some engineering judgment can help to reduce considerably the number of variations that must be considered to have confidence in the final results. For most cases it would appear that consideration of vertically propagating waves (the assumption most commonly used today) in combination with smooth design spectra at the free surface of the soil (or at rock outcropping) will provide a reasonable solution. The torsional components of motion created by travelling waves can be accounted for, particularly when dealing with an R.G. 1.60 type spectrum, by imposing an accidental eccentricity.

Adequacy of Wave Propagation Analyses

The first studies on the effect of local soil conditions on earthquake motions were conducted by Kanai in 1961 and Donovan and Matthiesen in 1968 considering the one dimensional problem of shear waves propagating vertically through a horizontally stratified soil deposit and using the analytical solution of the governing differential equation. A discrete model consisting of lumped masses and shear springs, which corresponds to a finite difference discretization of the continuous differential equation, was proposed by Idriss and Seed (1968) and Seed and Idriss (1969). While this model did not account originally for the elastic properties of the rock underlying the soil, assuming a rigid base and corresponding therefore to a specification of the input motion at bedrock rather than rock outcropping, it incorporated treatment of nonlinear soil behavior. A comparison between these formulations showing their equivalence and discussing the effect of the rock properties on the surface motions were presented by Roesset and Whitman in 1969.

Most studies on soil amplification, either to obtain site specific motions when the input is specified at rock outcropping, or to derive compatible motions at depth when the input is specified at the free surface (deconvolution process), have been carried out under the simplifying assumption of vertically propagating shear waves for horizontal motions and compressional waves for vertical motions. Some studies have considered, however, SH-waves at arbitrary angles of incidence (Roesset 1969), combinations of SV- and P-waves at various angles (Jones 1970) and generalized surface waves (Michalopoulos 1976). As pointed out before the effects of travelling waves on the motions of rigid, massless foundations have been studied by a number of authors. All these studies indicate that the overall shape of the amplification function for a soil profile is very similar for all types of waves. The actual details, such as the amplitudes or relative importance of the different peaks (corresponding to the natural frequencies of the deposit), are, however, strongly affected by the angles of incidence and relative amplitudes of the waves. Figure 4 shows for instance the amplification function from the outcropping of rock to the free surface of the soil for SH-waves propagating at various angles of incidence with respect to the vertical ($\alpha = 0$ is the case of vertically propagating waves, $\alpha = 90$ would correspond to horizontally travelling waves). It should be noticed that α is the angle of incidence in the rock. The corresponding incidences in the soil would be in all cases nearly vertical, invalidating the argument presented at one time that the assumption of vertically propagating waves is justified because the incidence near the surface is almost normal. It can be seen that the curves have similar shapes for the different angles but that the amplitudes of the peaks and the predominance of the peak at the fundamental frequency of the stratum over the others tend to decrease as the angle of incidence increases.

Figure 5 shows the amplification functions for a combination of SV- and P-waves assuming an amplitude of the shear waves 2.5 times that of the compressional waves. For this combination of waves (α_p is the angle of incidence of the P-waves in the rock) it can be seen that there is not only a decrease in the amplitudes of the first peak as the angle of incidence increases but also a clear coupling between the shear and dilatational modes of the soil stratum. This coupling gives rise to a small undulation at the first dilatational frequency of the stratum, between the first and second shear frequencies, and to a third peak which is larger than the second for $\alpha_p = 60$. This is due to the fact that the third shear frequency of the stratum is close to the second dilatational frequency. These effects get accentuated as the amplitudes of the two types of waves become more similar.

These general conclusions based on theoretical considerations are substantiated by the analysis of motions recorded at different depths in various places. These analyses show that the measured transfer functions are well reproduced in general terms by one dimensional wave propagation models (particularly when the soil properties are adjusted slightly) but that the details cannot be entirely duplicated.

A second consideration, even more important, is the fact that the strains induced in the soil by the seismic waves will generally exceed the range over which the material can be assumed to be linearly elastic. As the seismic intensity increases the effective soil modulus will decrease resulting in a decrease in the natural frequencies of the stratum (lengthening of the periods) and a corresponding shift in the peaks of the amplification function. The internal soil damping will increase, on the other hand, due to a loss of energy by hysteresis, resulting in a decrease of the peaks of the amplification. Figure 6 shows the response spectra of the motions at the free surface of a soil deposit corresponding to the same motion at the base rock but different acceleration levels (0.05 and 0.35 g). In order to facilitate the comparison the spectra have been normalized dividing their values at each period by the corresponding values of the spectrum of the base motion. It can be seen that for the higher intensity of motion the maximum amplification is smaller and that it takes place at a longer period. It is not possible, therefore, to characterize the effect of a soil deposit by a single amplification function independent of the earthquake intensity and wave content and computed using the soil properties corresponding to very low levels of strain. This explains also why different motions recorded at a given site will not have always the exact same frequency content (an argument that had been presented in the past to negate the existence of site amplification effects and which is clearly based on an oversimplification of the problem).

The importance of nonlinear soil behavior was already recognized, at a very early stage, in the work of Idriss and Seed. The procedure they suggested to derive strain compatible amplification functions was an iterative linear analysis in which moduli and damping values were adjusted at the beginning of each cycle on the basis of characteristic strains at various depths obtained from the previous cycle. This is the procedure which is still most commonly used today not only for amplification studies (to determine site specific motions) but also, when the input is specified at the free surface of the soil, to determine equivalent soil properties for soil structure interaction analyses. The approximations introduced by this scheme were studied by Constantopoulos (1973) who showed that for soil deposits which are not too deep and for moderate earthquakes the accelerations at the free surface are overestimated by 10 or 20% while the strains are more seriously underestimated. For deep, soft soil deposits and large accelerations the results tend to be less reliable. The iterative procedure tends to filter high frequency components in amplification studies (for a motion specified at rock outcropping or bedrock) and to exaggerate them in deconvolution analyses (for motions specified at the free surface). This is due to the assumption of a frequency independent damping. The alternative is to conduct the dynamic analyses in the time domain using a discrete model with nonlinear soil springs or nonlinear constitutive equations for the soil. This procedure does not allow, however, to determine easily equivalent soil properties for the remaining analyses.

The iterative procedure has also been used in two dimensional studies, normally with finite element discretizations of the soil, to determine the response to more general trains of waves. Its accuracy is even more questionable in this case. Nonlinear analyses with other types of waves and more general constitutive equations for the soil are yet to be performed, and there is some degree of inconsistency at present when analyses are conducted for travelling waves using soil properties corresponding to vertically propagating waves.

Most of the work conducted to date has considered a horizontally layered soil profile where soil properties vary only with depth. Wave propagation studies for more general and realistic geometries have been scarce and limited to the consideration of a homogeneous soil deposit on sloping rock or simple elliptical, cylindrical or spherical bowls, often under SH-wave excitation. Numerical techniques are, however, available to study more complicated geometries. All of them involve discretization either of the domain (finite elements) or of the boundaries. The discretization must be sufficiently fine in all cases to be able to reproduce the wave passage in the frequency range of interest. Thus the studies become computationally expensive. The cost is particularly high for three dimensional studies. The main problem, however, is the large number of variables that would have to be considered (angle of the sloping layers, relative position of the structure with respect to the surface expression of the rock or harder layers, overall geometry of the soil deposit, etc.) for meaningful parametric studies. Without these studies and on the basis only of the available results it is very hard to reach general conclusions on the magnitude of the effects of localized conditions, sloping layers or soil heterogeneities on the characteristics of the seismic motions, particularly when dealing with the details of their frequency content.

If the type of waves propagating through the soil, their amplitudes as a function of frequency, their angles of incidence and the soil properties were known it would be possible with the techniques available today to predict with accuracy the characteristics of the motions at the free surface or at any point within the soil mass, as well as the motions (translational and rotational components) of embedded or surface foundations. Because of the incomplete knowledge of these data and the precise geometry of the soil deposit as well as the approximations introduced in modelling the nonlinear soil behavior the derivation of site specific spectra for each individual soil profile may not be yet justified. On the other hand the use of more than one generic spectrum (for various categories of soils) or of several real motions corresponding to the same general characteristics would be an improvement over present practice. Uncertainties will exist not only in soil structure interaction analyses but in all phases of seismic design, including the modelling of the structure, and reasonable provisions, based on judgment, such as smoothing of design spectra or broadening of the peaks of instructure spectra, must be imposed to guarantee that the results are not unconservative. One should avoid, however, extreme conservatism or negating sound theoretical procedures and physical evidence.

Summary

The seismic input should be specified at the free surface of the soil when it is defined in terms of generic R.G. 1.60 type spectra for average soil conditions, a collection of spectra for various soil categories (rock, firm ground, deep soil deposits) or actual records corresponding to the same general conditions (magnitude, distance, soil deposits). For a direct analysis modelling together the structure and the soil compatible motions (and stresses) should be computed at the bottom and lateral boundaries of the model. For a substructure approach the input to the inertial interaction analysis should be the foundation motions resulting from a kinematic interaction analysis. In both cases an assumption has to be made as to the types of waves. In the absence of more precise information the assumption of vertically propagating waves is a reasonable one. For a substructure approach the foundation motions (for a rigid foundation) should include rotational components. In addition some provision must be made for torsional motions in both approaches. For generic R.G. 1.60 spectra an accidental eccentricity as normally assumed seems to provide sensible results. For site specific motions larger eccentricities may be required.

The alternative is to specify the input in terms of a rock spectrum at rock outcropping. One would have to compute again compatible motions at the lateral boundaries of the soil domain for the direct solution or the foundation motions for a substructure approach. The same comments apply in this case.

In all cases the soil properties and layering should be modelled as accurately as possible within the available data.

Improvements over this situation will be possible when additional knowledge is obtained about the types of waves (amplitudes, angles of incidence and frequency content) that can be expected as a function of the earthquake mechanism, magnitude, distance and overall topographic conditions.

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Table 1

Specification of the Seismic Input in the Free Field

	Mode	Location
1.	R.G. 1.60 Generic Spectrum	Free surface of soil deposit for average soil profiles
2.	Collection of standard smooth spectra for various soil categories (rock, firm ground, deep soil deposits)	Free surface of soil profile
3.	Peak ground acceleration, velocity and displacement as a basis to construct smooth spectra	Free surface of soil
4.	Real earthquake records corresponding to similar magnitudes, distances and soil conditions	Free surface of soil
5.	Rock spectrum	Outcropping of rock
6.	Amplitudes and angles of the seismic waves as a function of frequency	Could be anywhere, normally on rock

Table 2

Location of the Control Motion

1.	Free surface of soil deposit	For smooth response spectra or earthquake parameters such as peak acceleration velocity and displacement and for real motions
2.	Outcropping of rock	For rock spectrum
3.	Foundation level accounting for geometry of foundation but without structure	Ignores kinematic interaction and rotational components
4.	Foundation of structure	Ignores all interaction effects
5.	Bedrock	Only valid for rock spectrum if rock is much stiffer than soil
6.	Foundation level in the free field	Never to be used

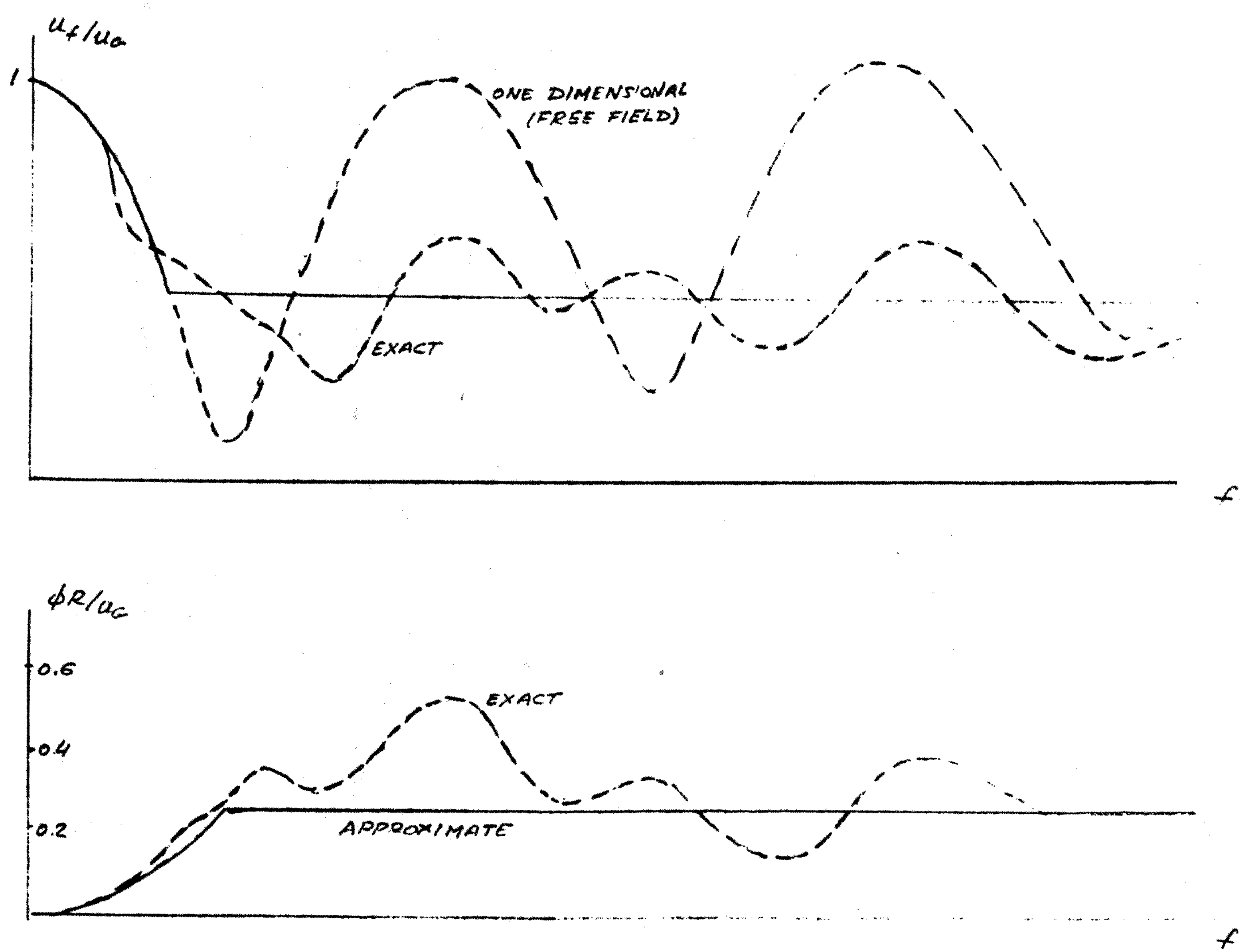

FIGURE 1. TRANSLATION AND ROCKING OF EMBEDDED FOUNDATION

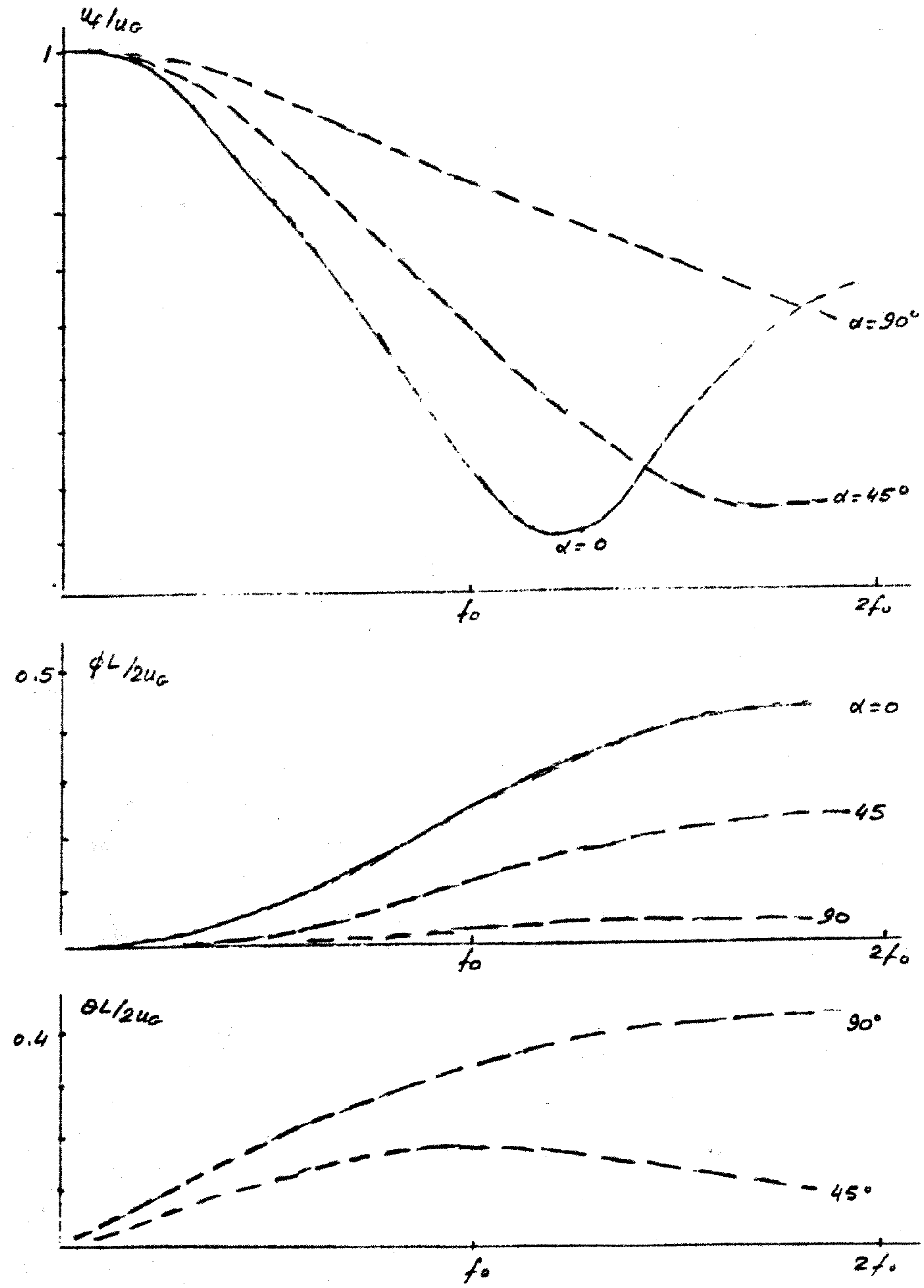

FIGURE 2. TRANSLATION, ROCKING AND TORSION OF EMBEDDED FOUNDATION

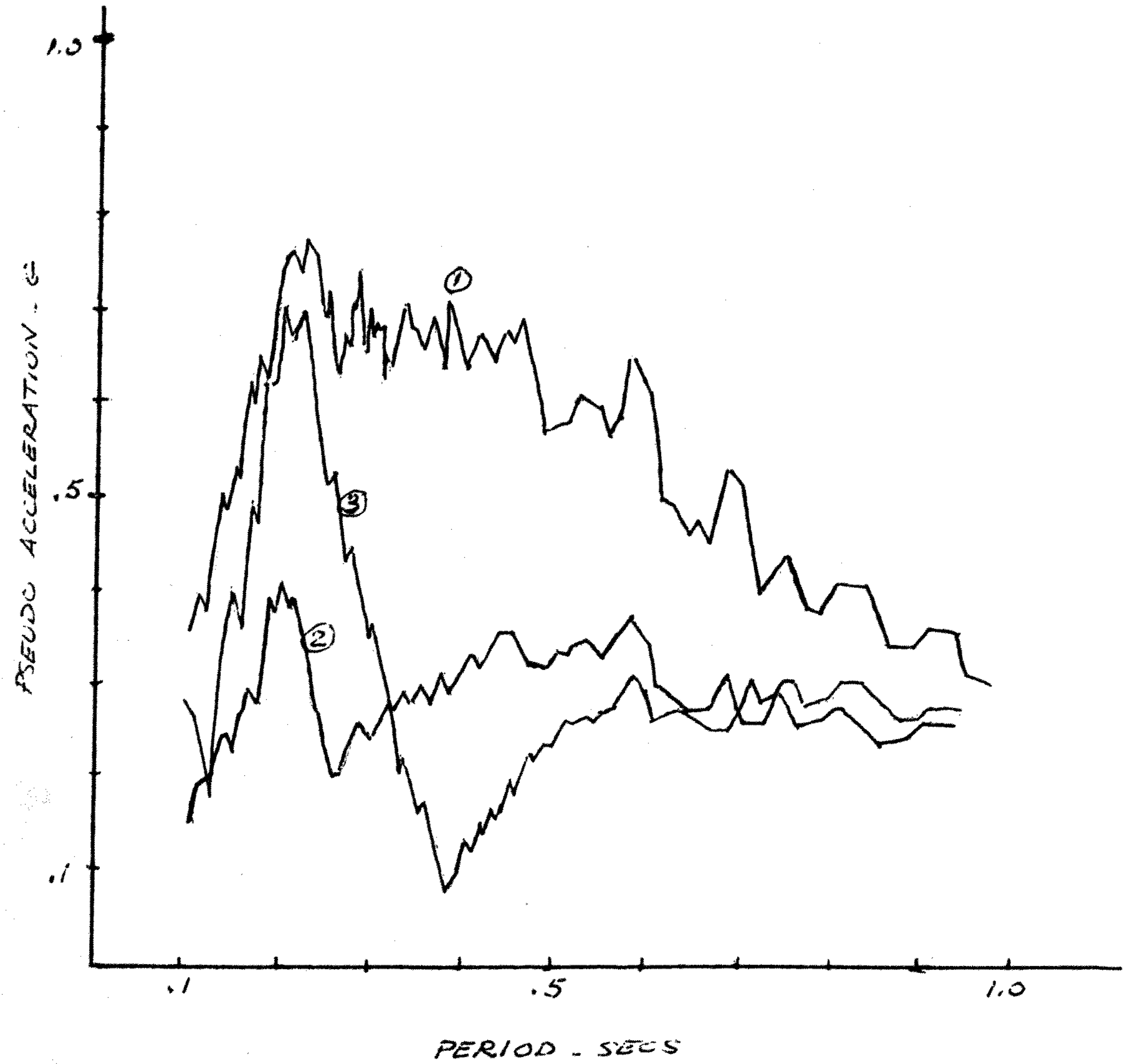

FIGURE 3- RESPONSE SPECTRA OF MOTIONS AT

①. FREE SURFACE OF SOIL

②. FOUNDATION (WITH KINEMATIC INTERACTION)

③ FOUNDATION LEVEL FREE FIELD

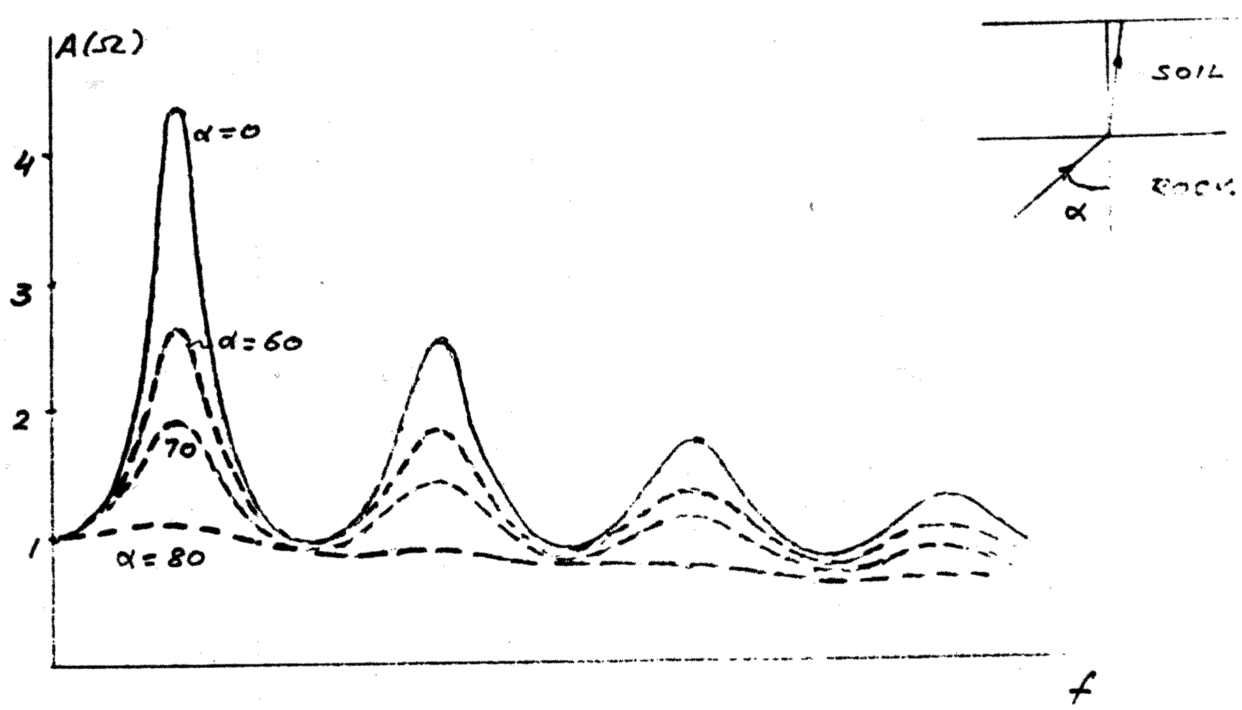

FIGURE 4 EFFECT OF ANGLE OF INCIDENCE - SH WAVES

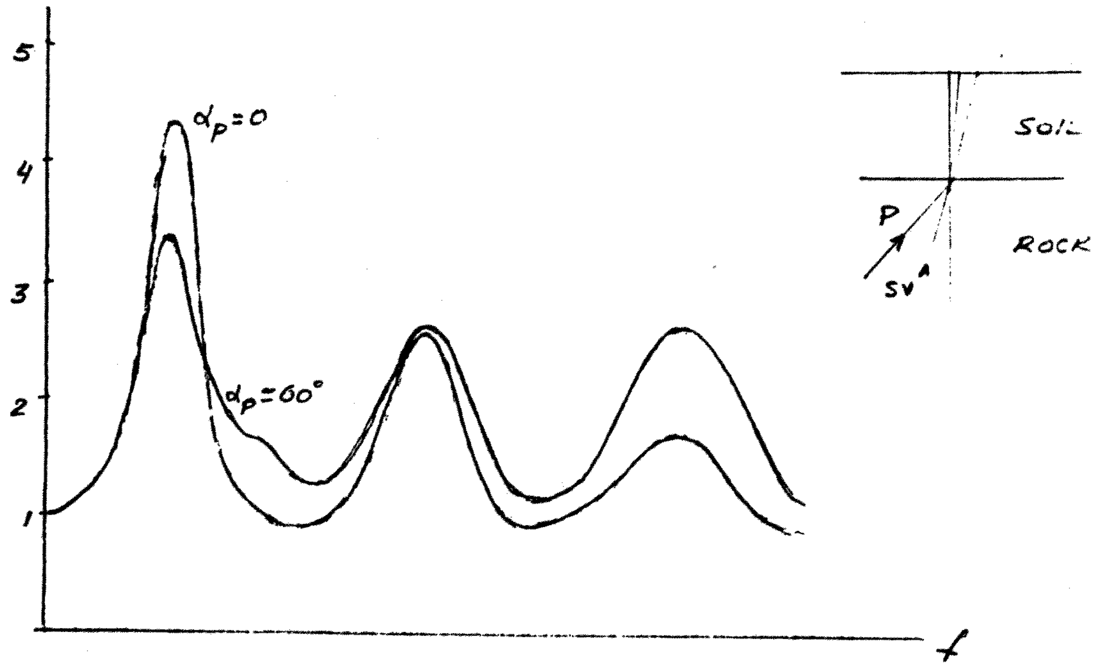

FIGURE 5. AMPLIFICATION OF SV AND P WAVES

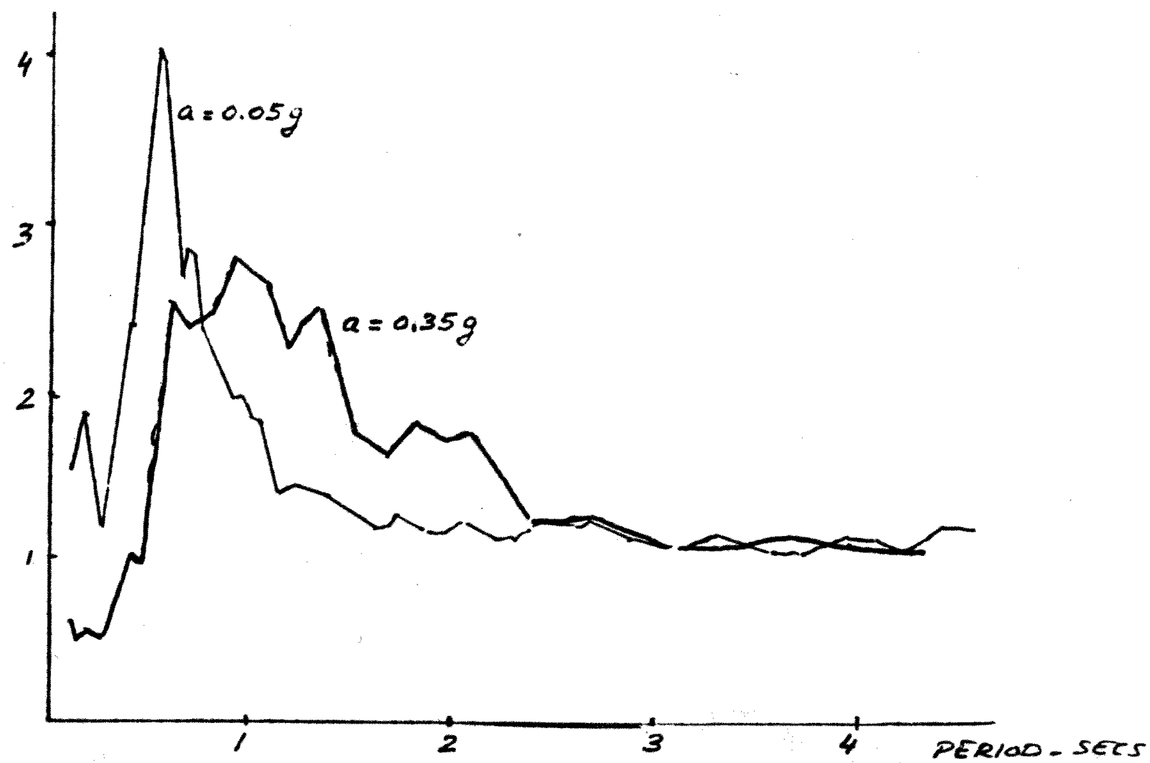

FIGURE 6. RATIO OF RESPONSE SPECTRA

Development of Site-Specific Earthquake Ground Motion*

by

I. Idriss, P. Somerville
Woodward-Clyde Consultants

*Paper Not Available.

UNCERTAINTIES IN SOIL-STRUCTURE INTERACTION ANALYSES

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1.0 INTRODUCTION

In view of the severe consequences that will develop from a failure of a major component of a nuclear power plant, seismic analyses have long been included in the evaluation of the safety of the plant. The first criteria document to establish the general requirements for seismic criteria was TID 7024 published by the USAEC (Ref. 20) in 1963. It primarily called for a proper determination of a maximum ground acceleration associated with an existing earthquake record and a ground response spectrum. A variety of available acceleration records and response spectra were then adopted, among the most frequently used earthquake records being the El Centro 1940 N-S component, the Taft 1952 and the Helena 1935 EW records. The average strong motion spectrum obtained from four major events (El Centro 1934 and 1940, Olympia 1949 and Taft 1952) was published by Housner (Ref. 10) and incorporated into the USAEC document. A modified design spectrum was recommended by Newmark and Hall (Ref. 19) in 1969, a modified version of which has been used on many nuclear facility evaluations.

During the early years, typical seismic studies made use of lumped parameter analyses, with frequency independent soil springs/dampers to account for soil-structure interaction effects. As computer capability increased into the early seventies, a variety of finite element analyses were developed using convolution concepts of upward traveling body waves (Ref. 24, 25). Phase differences due to nonvertical propagation were neglected as well as the effects of any incoherence in ground motions. Alternate finite element approaches were then developed, such as the three step methods (Ref. 14),

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which incorporate separate kinematic and inertial interaction analyses. Development of impedance approaches began in the late seventies, the primary goals of which are to generate interaction and scattering matrices, so as to overcome some of the known inadequacies of some of the finite element approaches.

1.1 Current SRP Procedures

The current procedures used to evaluate soil-structure interaction effects on Category I nuclear facilities are based upon the two separate approaches mentioned above, namely, (a) the lumped parameter method using interaction coefficients obtained from analytic formulations, and (b) a consistent finite element analysis suitably accounting for three-dimensional effects, soil layering, depth of burial, nonlinear soil strains, etc. In both approaches, the design motion history, suitably broad-banded, is input to the structural models at the foundation level. The final design response spectra are then obtained by enveloping the results from the two approaches. If the foundation level is at the ground surface, and if the half-space is reasonably uniform, it would be expected that the calculated motions of the system would be approximately the same from either approach, assuming of course that the finite element calculation is properly performed. However, if the structure is embedded within the soil, and if the soil system is significantly layered, it would be anticipated that significant differences would arise.

1.2 Issue of Concern

It has been well known for a number of years, primarily from convolution studies utilizing the concepts of the upward traveling shear wave, that the motion history at the ground surface is significantly different from that at depth. Comparison of computed free-field motions at and below the surface show major differences, primarily at the high frequency end of the response spectra. Thus, if one would consider the design motion history as a surface definition, and apply this design motion at the surface of a finite element model of the halfspace with an embedded structure, the comparison of response with that of the lumped-parameter approach (with the motion input at the foundation level) would show major differences. Just such a comparison of foundation motions obtained by the two methods is shown in Figure 1, which was a calculation performed for an embedded containment structure situated at a relatively shallow soil site. The spectrum from the lumped parameter analysis with the design motion input at the foundation level is seen to be significantly higher than that obtained from the finite element calculation over the entire frequency range of interest. In the finite element calculation, the design motion was input at the ground surface.

The difference in the two approaches is then clear. If interaction coefficients are obtained from realistic frequency dependent analyses, suitably treating soil layering, depth of embedment, kinematic interaction, etc., the lumped parameter approach should lead to essentially the same solution as the finite element method. Only minor differences should occur, if the analyses are performed properly, due to differences in the numerical methods used in each approach. This assumes of course that the input control motion is the same in each calculation. Any major differences in computed responses between the two methods would then be attributable primarily to the level at which the design input motion is applied (the ground surface or the foundation level). For most problems, it is this issue which is of overriding inportance.

The decision that must be made then is which analysis is more appropriate. Should both methods be used and should enveloping the two methods be a requirement for the seismic design of Category I structures? On the surface of the discussion, it would appear that for a site in which embedment plays such a significant role, inputting the criteria motion at the foundation level is clearly inappropriate, if the criteria motion is defined as a surface motion. However, for the vast majority of cases investigated, the simplified approach (presuming interaction coefficients are properly chosen) would at least offer a comfortable amount of conservatism in the design. The principal effect of inputting the design motion at the ground level in the finite element approaches is to reduce the input seimic motions into the structure, effectively reducing the safety factor in the final design. The question that must now be answered is how much confidence do we have in our predictive capability to allow ourselves the freedom to reduce this input, and do we still have a significant safety factor in our design.

In an attempt to arrive at an answer to this question, we present in this paper a series of experiences that have been obtained over the years in reviewing specific plant analyses, performing research into the SSI area, and assembling some of the conclusions obtained by other researchers in the field. The purpose of this summary is to enable us to focus on the degree of confidence that we can place on our predictive capability, and therefore to try to assess the safety inherent in seismic designs. We have assembled these results into three separate categories of uncertainties, namely those resulting from observations of field measurements, those resulting from analytic simplifications and those arising from numerical approximations.

2.0 FIELD OBSERVATIONS

An extensive storehouse of information is beginning to be amassed, based primarily upon recordings taken during relatively strong events at the various instrument array sites located around the world. These data, both surface and downhole, provide a backdrop against which the various analytic approaches can be tested. In particular, these arrays can be used to assess the adequacy of the concept of upward traveling body waves to describe the input motions for SSI calculations. For example, recordings from the array at the Chiba Experiment Station in Japan (Ref. 13) clearly indicate a strong decrease in the higher frequency components (above 4 hz) at a depth of 20m as compared to those of the surface motions (Figure 2), while the corresponding peak soil strains and displacements which are more low frequency dependent do not show as significant a decay with depth.

At the Iwaki downhole array near Tokyo (Ref. 22), recorders were located at the rock site to a depth of 330 m, as shown in Figure 3. The uniformity in shear wave velocity can be noted from this data. Some of the results measured at this site, as well as at the Tomioka site nearby, are shown in Figure 4. It can be noted that a significant decrease in peak accelerations occurs within the first 20m of depth, after which peak accelerations remain fairly constant. These generalities are, of course, all in keeping with the behavior associated with the concept of the upward traveling shear wave. It is the accuracy of these predictions, however, that is of concern in the study of nuclear reactor facilities.

When applied to a very soft site, such as Mexico City , the convolution method has been found to provide reasonable estimates of both the primary and secondary harmonics at the site (Ref. 9). However, from strong motion data amassed from some 20 other sites, it was noted in Ref. 9 that no specific correlation could be found between the frequencies of the peak responses in the two horizontal directions. In addition, it is well known that many destructive earthquakes involve significantly differing patterns of damage (Ref. 11) within narrow regions with ostensibly the same soil profile. These facts can be interpreted as an indication that this concept of upward propagating shear waves, although attractive from a computational point of view, may in fact be inadequate for many sites of interest, particularly when considering the entire frequency range of interest for the design of nuclear power plants. The results obtained from such a calculational model must then be used with caution.

A study recently completed for the NRC (Ref. 3) presents a review of a significant amount of array data with the goal of trying to assess and improve the general state of knowledge on spatial variations of ground motion. As an example, some records are available from the Waseda and Narimasu array sites in Japan, both soil sites consisting essentially of uniform sands and gravels to significant depths. The Waseda site exhibits a relatively uniform shear wave velocity of about 500 m/s throughout the top 300 feet of soil overburden (Figure 5) while the Narimasu site shows a gradually increasing wave velocity to 500 m/s. Both sites show relatively consistent and high values of SPT sample blow counts, typical of good foundation soils.

An example of the recorded spectra from the Waseda site is shown in Figure 6 for both the NS and EW directions. Near the ground surface, the measured acceleration spectra are very similar in both directions, both in magnitude and shape. The spectra with depth, however, differ by factors of about 2. Spectral ratios shown in Figure 6 not only confirm this discrepancy, but also show that the discrepancies are different in the two directions. The corresponding transfer functions for the Waseda site are shown in Figure 7. It is striking to note the dissimilarity in the observed transfer functions in the two horizontal directions, and differences with the theoretical predictions. The shear wave speed was adjusted in the calculations of Ref. 3 in an attempt to match predominant frequencies. It is quite clear that although the concept of the upward traveling shear wave is a convenient calculational tool, it does not appear to adequately predict near surface behavior, particularly at the higher freqencies of interest (above 2 to 3 hz) for the design of structural members, equipment and piping associated with nuclear power plants.

Similar data is shown in Figure 8 for the Narimasu soil site (Ref. 3). Again, it is interesting to note the striking discrepancies with the theoretical predictions, particularly at the higher frequencies of interest. To match the measured principal frequency of the soil column, the measured shear wave velocity had to be artificially adjusted by changing the effective shear modulus of the soil overburden. In addition, if convolution from the bottom of the colum upward were performed, the results at the higher frequencies of interest would not agree with the measured data since the theoretical amplification factors are lower than actual for the site. Although general trends can be noted in the results, the variability in the details leaves much room for improvement.

A detailed analysis was conducted for the NRC (Ref. 16) on the array output determined from the Fukushima Nuclear Power Plant during a strong motion event in June, 1978. A sketch of the reactor building and the accelerometer locations is shown in Figure 9. The reactor building is founded on a mudstone with a relatively uniform shear wave velocity of about 530 m/s. Although no specific dynamic rock test data were available, rock damping was quoted to be approximately 10%. A detailed parametric study was undertaken for this plant using the standard convolution approach. To provide a reasonable match of the measured structural response, artificially high rock damping values of 20% had to be used in the calculations. If deconvolution from the actual surface record were performed using the more realistic value of 10% for the rock damping, larger structural motions than measured would be predicted. This again serves to indicate the difficulties encountered in the applicability of this approach.

3.0 ANALYTIC APPROXIMATIONS

In assessing the adequacy of the calculations typically employed in seismic studies, several different factors must be evaluated. Various theoretical simplifications are typically used to make the problem tractable. Each of these in turn leads to errors in the seismic evaluations. Among the primary assumptions used are the following.

3.1 Equivalent Linear Soil Constitutive Model

The only practical analytic/numerical tools currently used to investigate seismic effects in soil are based upon simplified linear viscoelastic soil constitutive models. However, for real soils, the levels of strains induced by strong seismic events are associated with significant nonlinear soil behavior. The concept of equivalent soil damping has evolved over the years in an attempt to somehow include energy dissipation into the soil system to account for this nonlinear behavior during cyclic loadings. Estimates of damping ratios are typically determined from cyclic shear tests (torsional, triaxial or direct shear) conducted at a relatively low frequency of 1 or 2 cps. Although no other approach currently appears reasonable to attack the seismic problem, several concerns immediately come to mind in trying to assess the adequacy of the linear approximations for predicting detailed motion histories.

Firstly, several one-dimensional convolution studies have been made using simplified nonlinear soil models (Ref. 4, 21,26), usually a form of the Ramberg-Osgood shear model. Comparing the results between the nonlinear and linear approximations with similar energy loss indicates that the linear approximation consistently underestimates peak column displacements and overestimates peak accelerations as compared to the nonlinear models. The differences, of course, will increase with seismic levels since the degree of nonlinearity developed in the soil column increases.

Secondly, in most of the calculations conducted for nuclear reactor facilities, the linear damping approximation is applied to the total stress strain relation, although experimental data clearly indicates that the hysteretic behavior under shear and hydrostatic stress components are significantly different. This is particularly true for the case of soils which are fully saturated. The effective damping for these soils will be less than included in the normal calculation. This effect is particularly important for the vertical and rocking modes of structural response since they are so dependent upon soil compressibility under the foundation.

A third important factor which is currently unevaluated is the fact that the equivalent damping ratio used in seismic calculations is typically maintained as constant with frequency. It is more probable, however, that damping ratios decrease as frequency increases (Ref. 3) since strains associated with these higher frequencies are low. This fact could have a significant impact on calculations of peak accelerations in the SSI problem, associated with the higher frequency ranges of interest.

3.2 Influence of Soil Saturation

Numerical results were recently obtained (Ref. 5) for interaction coefficients for the case of saturated soils. Numerical results were obtained considering the two-phased Biot formulation for solids and pore water. The problem of particular interest in that study is shown schematically of Figure 10. As an initial calculation, frequency dependent interaction coefficients were generated for the two dimensional plane problem of a rigid surface footing resting on a fully saturated linear soil, with the ground water table located at the ground surface. The results indicate that interaction coefficients are significantly modified by the presence of the pore water. This is particularly true for the vertical and rocking modes of response. A comparison of vertical interaction coefficients for the dry and fully saturated cases is shown in Figures 11. As may be noted, significant differences occur, particularly at the higher frequency end of the spectrum. The specific impact of this effect on actual plant response has not as yet been determined.

3.3 Horizontal Soil Layering

All analyses currently performed in SSI studies are based upon the assumption of boundless horizontal soil layers of constant thickness and elastic properties. This assumption is obviously necessary to simplify the problem formulation and make any solution possible. Generally speaking, many soil profiles encountered in practice are intuitively assumed to be reasonably approximated by horizontal layers, although it is clear that no specific data is available to adequately judge the effect of this assumption. At still other sites, the nonhorizontal effect is obviously significant, particularly for the case of sloping and/or variable bedrock configurations. Little if any information is currently available on which to base reasonable assessments of these effects.

A summary of the studies performed in this area is presented in Reference 23. Some numerical evaluations have been attempted using FLUSH type analyses (Ref. 2, 7) applied to a site consisting of a uniform soil deposit overlying a sloping bedrock (Figure 12). Transmitting boundaries were not used in these calculations since their formulation is tied to the horizontal layer assumption. Rather than deconvolving from the surface, the El Centro accelerogram was input directly to the bottom of the finite element mesh. The results obtained indicated that for slope angles of less than 20°, the assumption of horizontal bedrock was considered adequate, with coupling between horizontal and vertical motions unimportant. For higher slope angles, however, the entire free-field problem formulation is suspect.

An evaluation of the effects of undulating soil layers (Figure 12) was also attempted using the FLUSH approach in References 2 and 7, but with little success. Based upon the work in Reference 12, however, it was concluded that such undulating layers can significantly affect the results as compared to the case of horizontal layering. Again, no easy rules are available to allow for assessment of these effects. Other attempts at analytic formulations of the dipping layer problem have to date met with little success, due to the obvious complexity of the problem. Although the FLUSH work mentioned gives some insights into the sloping bedrock problem, it is not clear what its impact would be on the inertial and kinematic interaction problems.

A recent publication (Ref. 1) makes use of an analytic solution to the problem of elliptical soil layering, such as would occur with soils residing in a synclinal rock valley. Some of the results indicate that this layering type can have significant a impact on interaction coefficients. This is particularly true for the case of the radiation damping coefficient, since the "saucer" effect of the rock basement tends to trap energy within the softer soils.

4.0 NUMERICAL APPROXIMATIONS

There are various numerical considerations that must also be considered when evaluating the acceptability of large (usually finite element) numerical calculations. Each of the problems mentioned below can play a significant role in modifying the calculational accuracy of the seismic predictions, and must obviously be carefully assessed in each case.

4.1 Element Size Criteria

It is well known that finite element solution methods (as all lumped mass numerical solutions) have inadequate transmission capability above a certain mesh cutoff frequency (Ref. 8) which in turn leads to errors in computed solutions at frequencies near and above this cutoff frequency. This feature can play a significant roll when generating solutions typically used in seismic studies. From numerical results generated for the one-dimensional convolution problem (Ref. 6), it was shown that the effective cutoff frequency of a finite element mesh can be approximately determined from the relation

$$f_{max} = [V_S/B]/4.4 \tag{1}$$

where V_S is the shear wave velocity and B is the dimension of the element.

An example of the error generated from this phenomenon is shown in Figure 13. The response spectra from an exact convolution solution for a specific layered soil site is compared with those obtained using finite element meshes of varying transmission capability. As may be noted, the spectra from the finite element solutions for this particular case lie above the exact spectrum, although from other calculations, it was found that the results from the finite element solutions can lie either above or below the exact solution. What is important, however, is the fact that the discrepancies can be significant, with the discrepancy increasing as the mesh becomes coarser. In addition, the error is still found to be significant at frequencies below the theoretical cutoff frequency of Equation 1.

As an example of an application of the finite element approach to the two and three dimensional SSI problem, a mesh that has been used for the response analysis for a deep soil site is shown in Figure 14. As may be noted, elements are used which have an aspect ratio of as much as five to one, although it is well known that such large aspect ratios are not desireable for finite element calculations. Associated with this concern is the fact that the element possesses different transmission capability in the horizontal and vertical directions. The degree of error as a function of frequency is essentially unknown for such problems, as only intuitive rules of thumb are generally available for the analyst. In most seismic calculations submitted for review, such concerns are rarely addressed by the applicant.

4.2 Mesh Size Requirements

The individual element size requirements for a finite element mesh are governed by the criteria discussed above. The lateral and depth extent of the mesh, however, must be chosen on the basis of relatively loose criteria. Since the bottom boundary of the mesh is a nontransmitting boundary in those calculations associated with the convolution method, the depth to this artificial interface must be chosen judiciously. If an actual rock interface exists which has a large impedance mismatch with the soil above, the choice of this bottom boundary is simple. However, for many of the problems of interest, such a choice is not always evident, and may have an impact on structural calculations. As may be noted from Figure 14, extremely large meshes may be generated which in turn can lead to expensive computer calculations.

Of even more concern for the particular site of Figure 14 is the fact that the ground water level is near the ground surface, leading to the situation of having to simulate the response of a two-phase material (soil-water) by means of a single phase elastic analysis. To accomplish this objective, attempts are made to match the relatively soft shear wave velocity of the soil material, while trying to maintain the high compressional velocity of the pore water, leading to the necessity of using values of Poisson's ratio near 0.5. As can be appreciated, this in turn can lead to serious stability problems in the calculation and lack of convergence of the iteration process on peak soil shear strains. Again, the adequacy of such calculations are seldom adressed.

4.3 Lumped Mass Structural Stick Models

For most plant analyses, the structures sitting atop the foundation mat are typically represented by lumped mass stick models, a typical example of which is shown in Figure 15. As can be appreciated, the methods used for selecting the number of nodes and elements for a particular stick are based more upon intuition designer rules-of-thumb than criteria based in fact. Suffice it to say that for most systems of interest, a relatively few degrees of freedom are used in SSI calculations. The question of adequacy of the model over the entire frequency range of interest is one of serious concern, but one which is not often addressed, primarily due to the numerical difficulties involved.

However, by studying measured responses at sites subjected to actual low-level seimic events, some idea of the adequacy of the structural models can be determined. A comparison of the amplification functions for a real nuclear reactor structure subjected to a low level seismic event is shown in Figure 16. The transfer functions determined from the analysis (using the stick model of Figure 15) and measured during the seismic event compare well at low frequencies (below about 6 hz), but show significant differences above this frequency. The measured spectra show significant energy content at the higher frequencies which do not appear in the analytic solution.

This is more clearly shown by computing the coherence of the motions at the two locations. The coherence between the motions is defined as

$$\gamma_{xy}(f_j) = | S_{xy}(f_j)|^2 / S_x(f_j)\, S_y(f_j) \tag{2}$$

where γ_{xy} = the coherence of the motion at a given frequency f_j

S_x = the power spectral density of the measured accelerogram at point x and frequency f_j

S_y = the corresponding PSD for the accelerogram at point y

S_{xy} = the cross spectral PSD

For the analytic solution using the stick model, the coherence of the motion is obviously unity. The actual measured accelerograms, however, show that above a frequency of about 6 hz, the two motions are not coherent. Therefore, it would appear that the numerical models being used to represent the response of the superstructures at higher frequencies of interest may in fact be inadequate at these higher frequency ranges.

5.0 CONCLUSIONS AND RECOMMENDATIONS

Based upon the summary presented herein, a variety of conclusions may be drawn which all have applicability to the SSI problem of interest. On the one hand, it may be stated with confidence that the approaches which are currently being used provide amazing insight into the dynamic response of a complex soil-structure system, much more than would normally be expected considering the amalgam of assumptions made. However, providing insight and having confidence in the degree of safety inherent in specific predictions used for design are two different issues. It is this latter aspect that is of concern.

The current SRP procedures used, which require enveloping of results from both a simplified and a detailed SSI calculation, almost always leads to a conservative design. For the case of embedded facilities, the requirement that the analyses incorporate the design motion at the foundation level can lead to overly conservative results. In addition, the use of broad-banded site independent response spectra may lead to analytic results which in fact cannot exist in actual situations.

To simply eliminate these current requirements, however, cannot be condoned unless all those concerned with the analysis and design of nuclear facilities are specifcally aware of the consequences. One can view the current design requirements as a way of ensuring high (but unknown) factors of safety in design. If we are to eliminate these requirements, the industry must then more properly address the question of suitable safety in design. Based on the results presented herein, two primary conclusions can be reached, which are:

(a) For the majority of sites and structures associated with reactor facilities, the current SRP procedures lead to conservative (overly so in some cases) seismic designs.

(b) The analytic tools currently being used, if not properly exercised, cannot adequately predict seismic response and vulnerability of complex soil-structure systems in the primary frequency range of interest (above 2 to 3 cps).

If we are to improve seismic design by using less conservative and (hopefully) more realistic analysis methods, a greater emphasis must be placed upon procedures to more rationally evaluate the adequacy of SSI analyses. The current cookbook approach to SSI studies typically endorsed by the industry must be replaced by a procedure in which all participants (analysts, designers and reviewers) play complementary roles to ensure that adequate engineering effort is brought to bear on the problem on a site by site basis. The current adversarial approach taken between applicant and reviewer must be removed.

Based on our previous discussion, we recommend that the current SRP procedures be modified to allow the applicant the ability to take either of two approaches to SSI analysis, the first a simplified cookbook approach, or the second a more detailed realistic analysis requiring extensive study to ensure that safety factors are adequately met. If the applicant decides not to invest a major effort in a seismic evaluation, it should be allowed to follow what are essentially the current procedures which can be summarized as follows:

(a) Use the current RegGuide 1.60 broad-banded response spectra recommendations together with (suitably enhanced) enveloping time histories;
(b) Perform both lumped parameter and half-space (either finite element or substructure method) seismic response analyses, using current enveloping and spectra spreading methods.

The one modification to the current procedures that we would endorse which is applicable to the case of embedded structures is to allow some nominal reduction in the high frequency content of the foundation spectra to account for the known reduction currently being measured at the various instrument arrays. The specific magnitude and form of the reduction should be arrived after study of the data currently available.

If the applicant decides to invest a greater effort in the seismic program, it should be allowed the privilege of incorporating site specific behavior in the design, but at the same time responding to the many questions inherent in these studies, some of which are mentioned herein. Thus, we would recommend that the following procedures be used:

(a) Perform suitably detailed analyses to arrive at a proper set of design motions accounting for specific site properties and realistic hazard estimates;
(b) Perform detailed computational studies using different methods of analysis to ensure that a complete set of structural responses is determined for use in design;
(c) Perform enough parametric studies so as to adequately assess primary uncertainties in all aspects of the computations.

This more realistic approach will obviously require that all parties concerned, both analysts and reviewers, be intimately involved with all decisions made throughout the seismic evaluation period.

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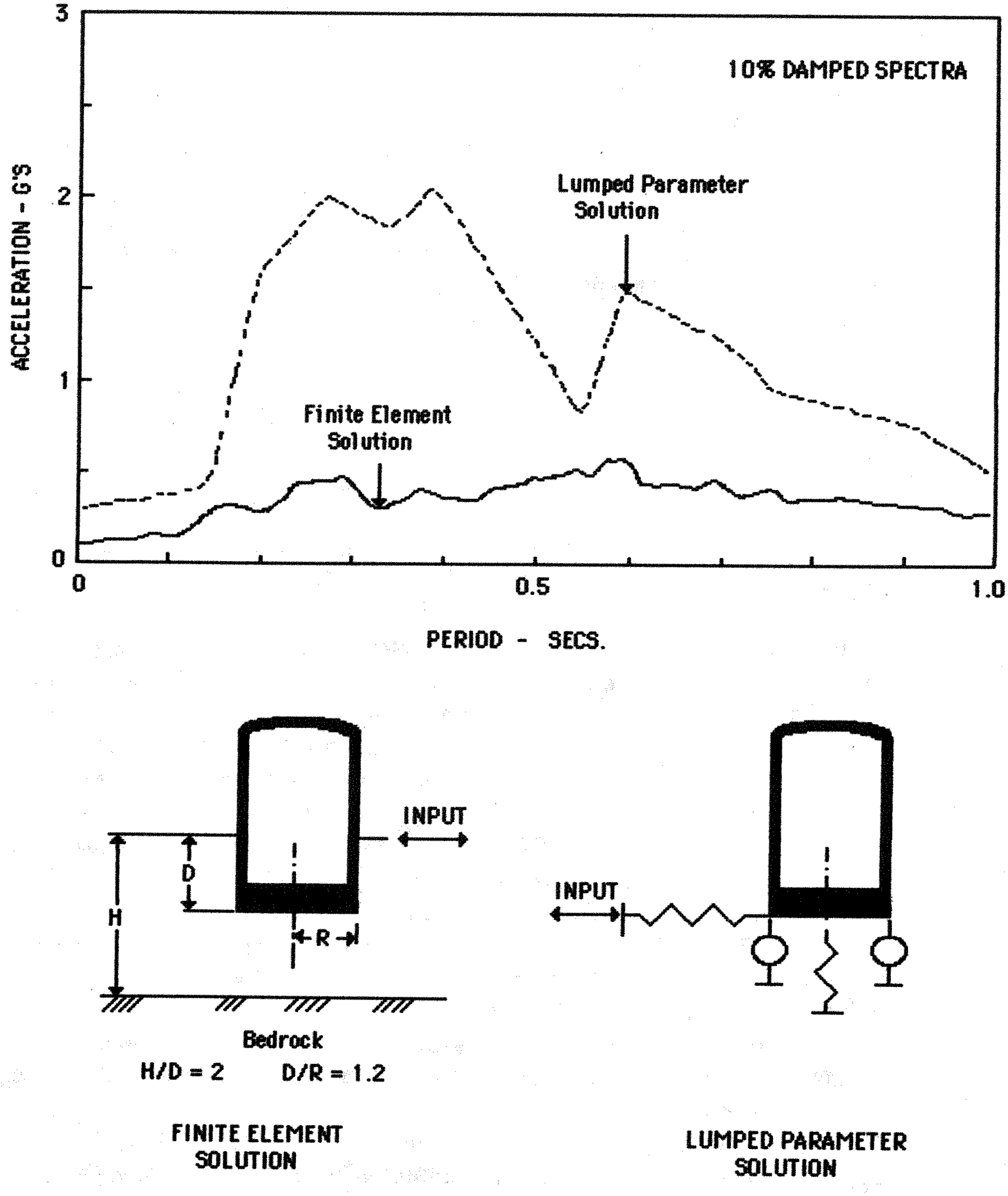
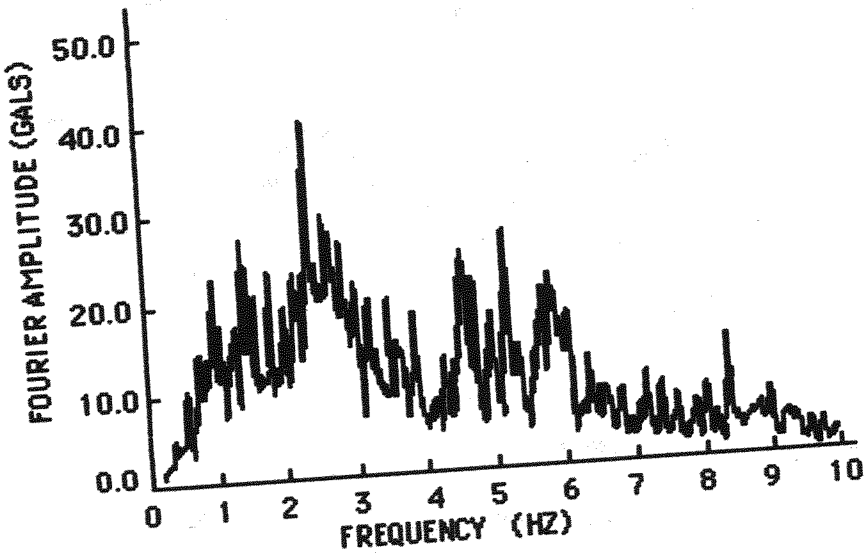
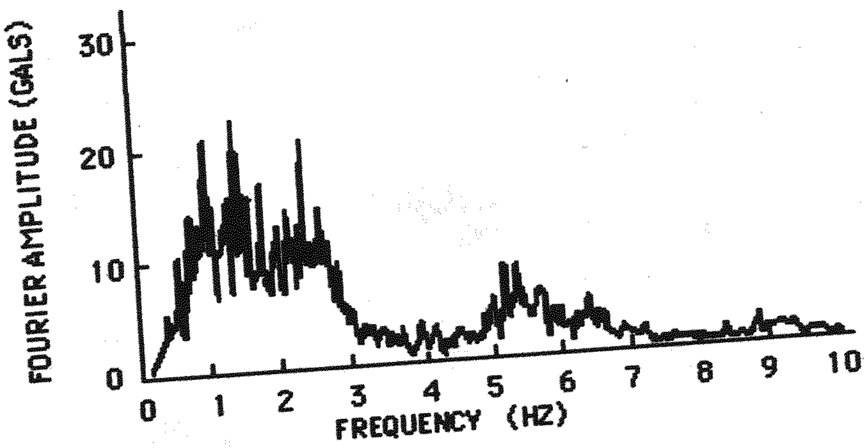


FIGURE 1 MAT RESPONSE COMPUTED BY FINITE ELEMENT AND LUMPED PARAMETER ANALYSES WITH INPUT AT DIFFERENT LEVELS, HORIZONTAL SSE ANALYSES



a) MOTION AT DEPTH OF 1M



b) MOTION AT DEPTH OF 20M

FIGURE 2 RESULTS FROM CHIBA EXPERIMENT STATION
(REF. 13)

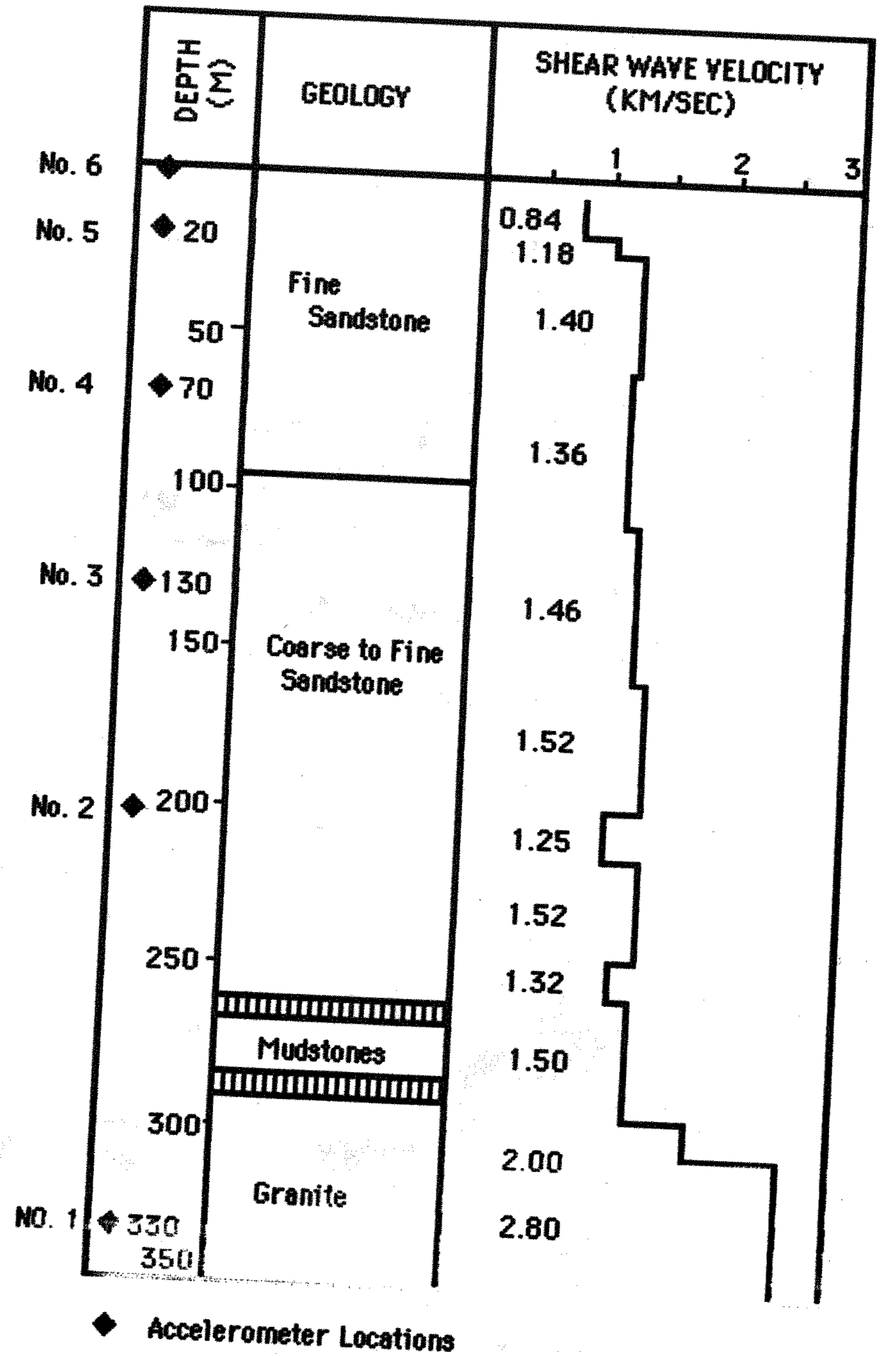


FIGURE 3 ROCK SHEAR WAVE VELOCITY PROFILE AT IWAKI DOWNHOLE ARRAY SITE (REF. 22)

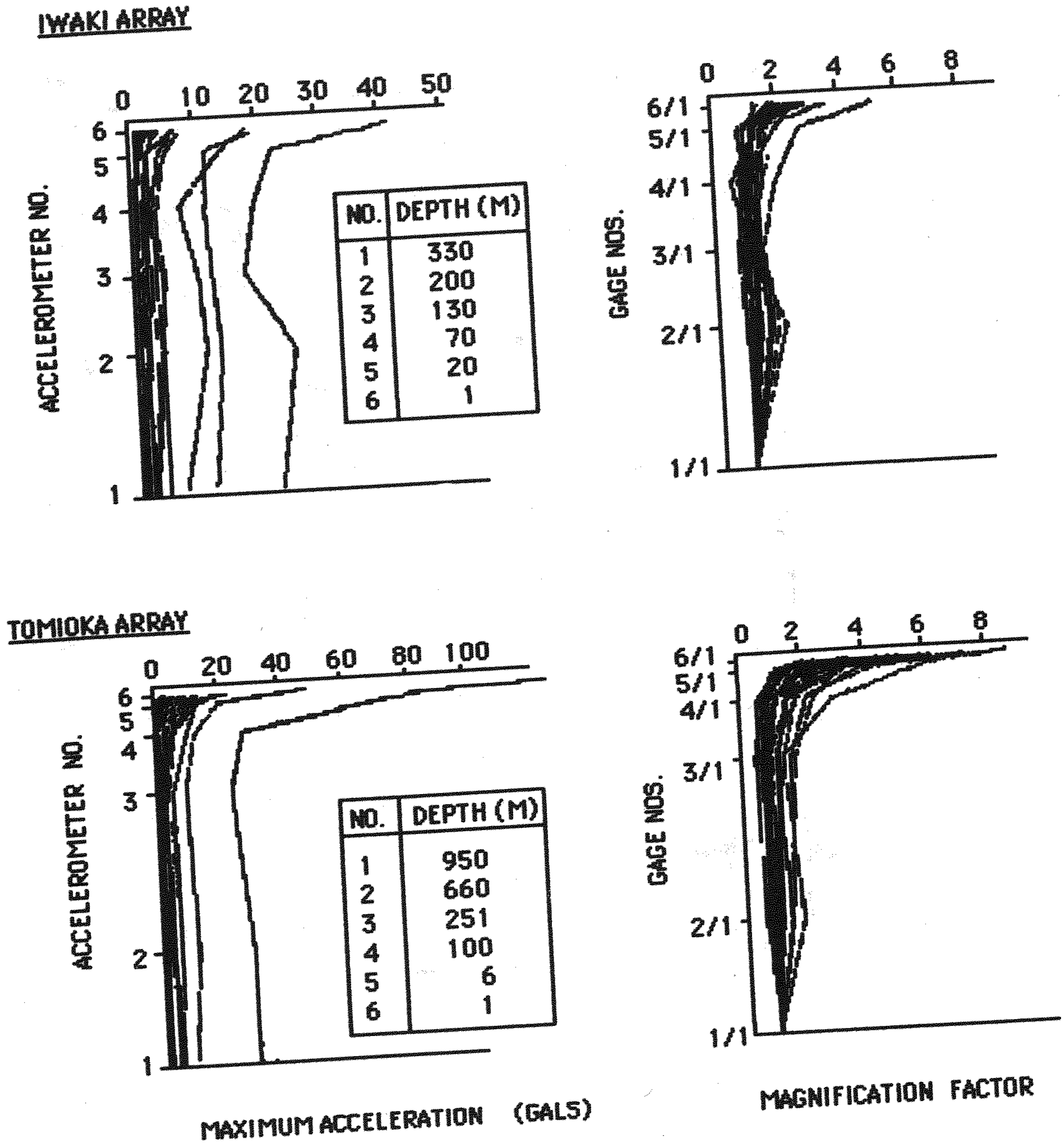


FIGURE 4 VERTICAL DISTRIBUTION OF PEAK ACCELERATION AT IWAKI AND TOMIOKA SITES (REF. 22)

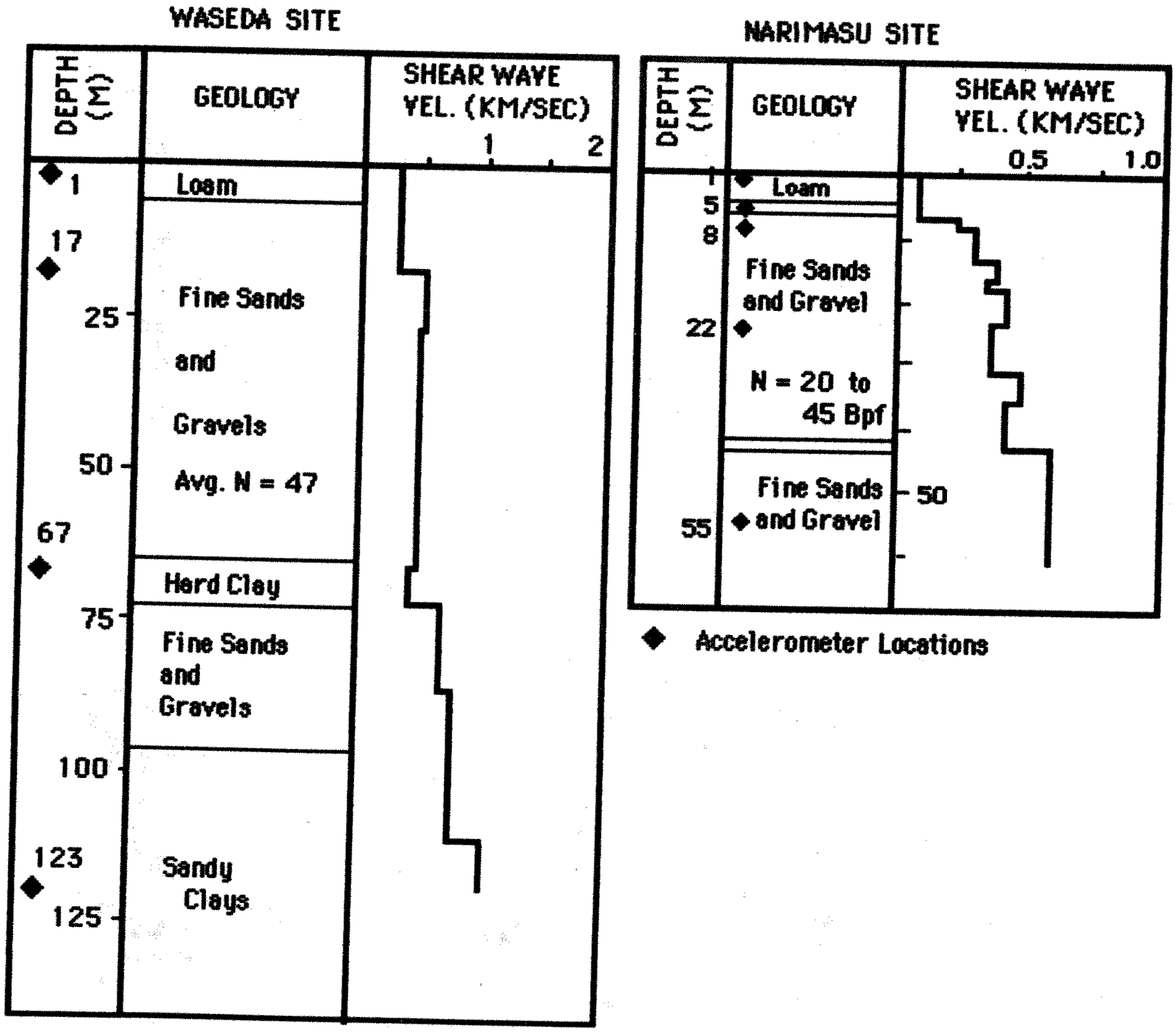


FIGURE 5 SOIL SHEAR WAVE VELOCITY PROFILES AT THE WASEDA AND NARIMASU DOWNHOLE ARRAY SITES (REF. 3)

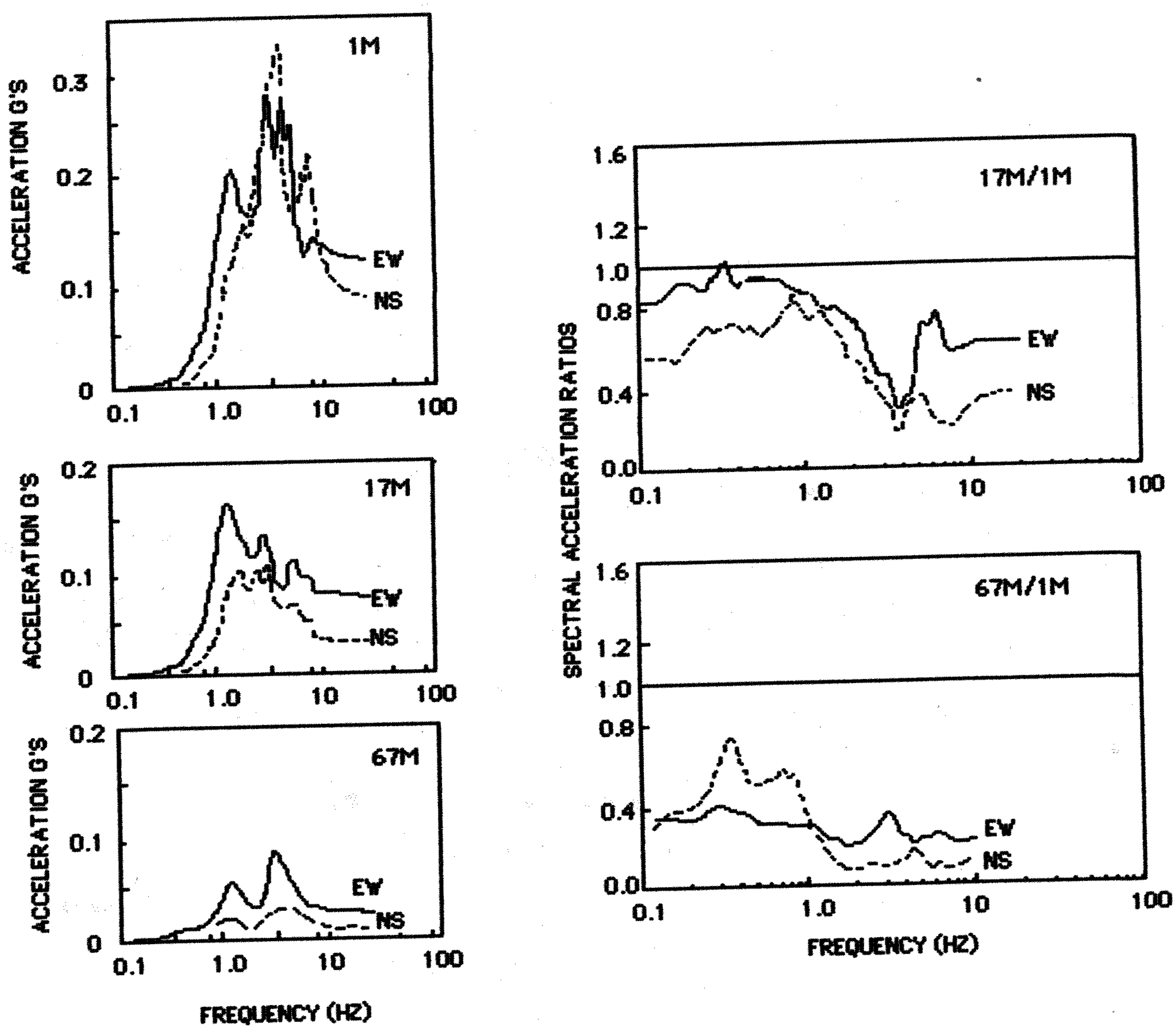


FIGURE 6 RECORDED MOTIONS AT WASEDA DOWNHOLE ARRAY (REF. 3)

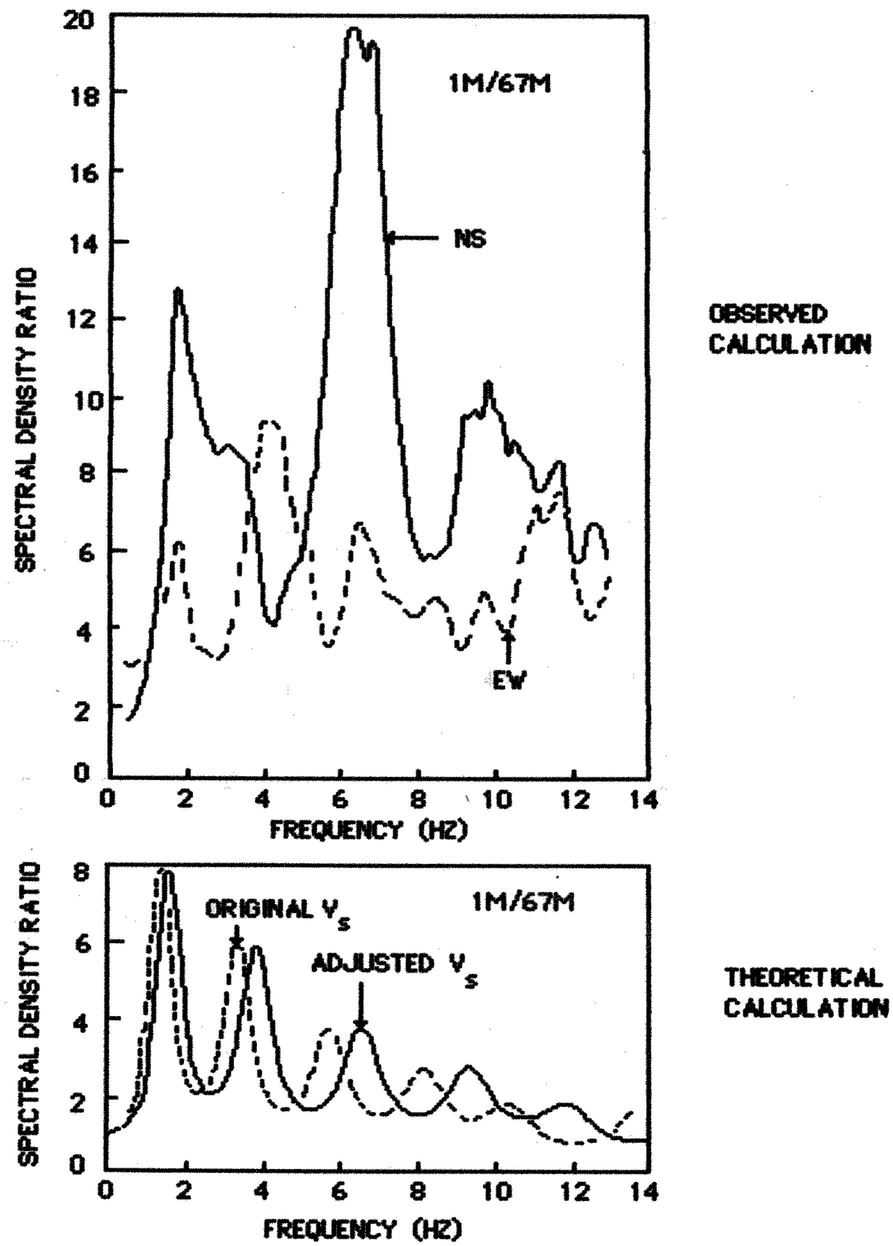


FIGURE 7 COMPARISON OF OBSERVED AND THEORETICAL TRANSFER FUNCTIONS AT WASEDA DOWNHOLE ARRAY (REF. 3)

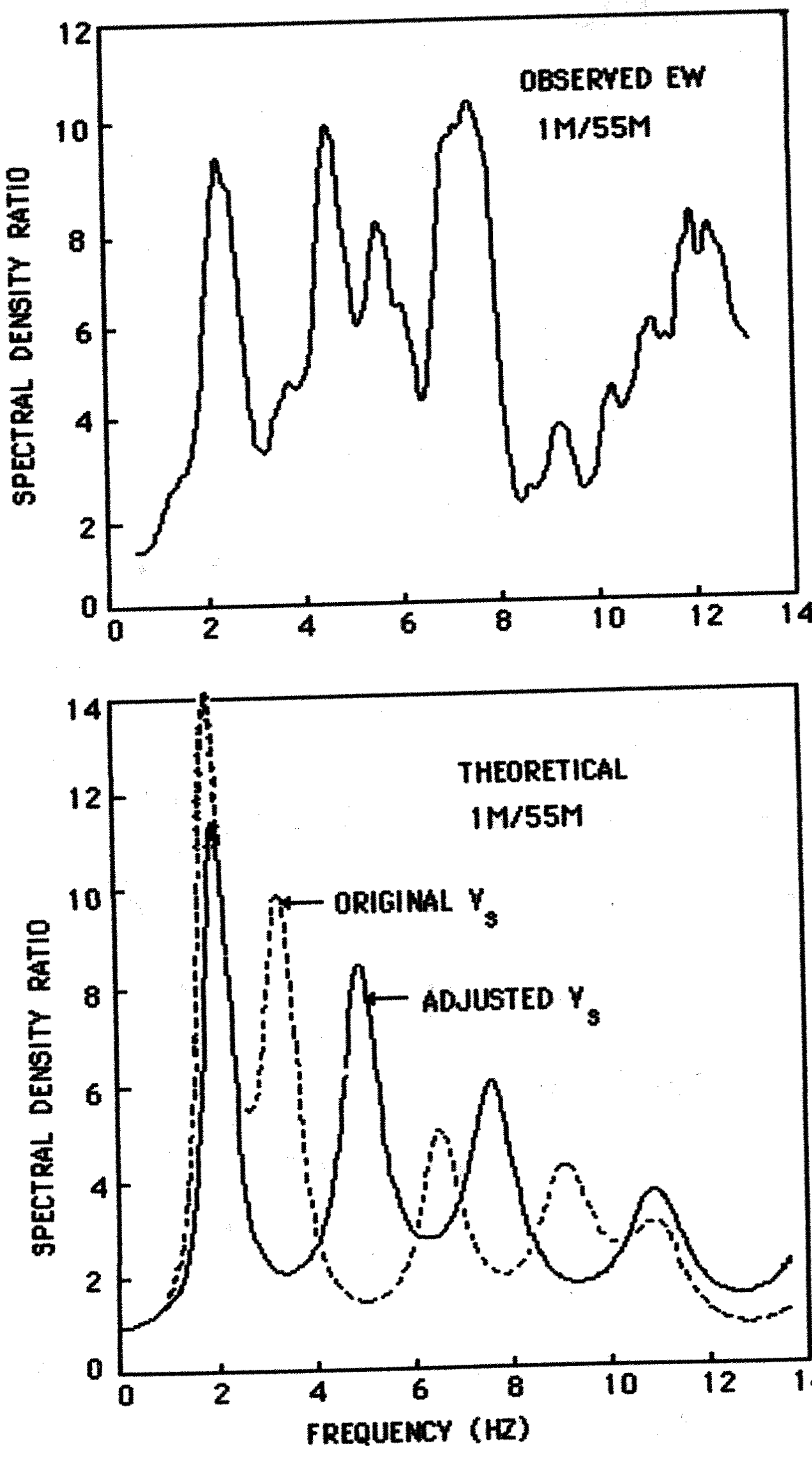


FIGURE 8 COMPARISON OF OBSERVED AND THEORETICAL TRANSFER FUNCTIONS AT NARIMASU SITE (REF. 3)

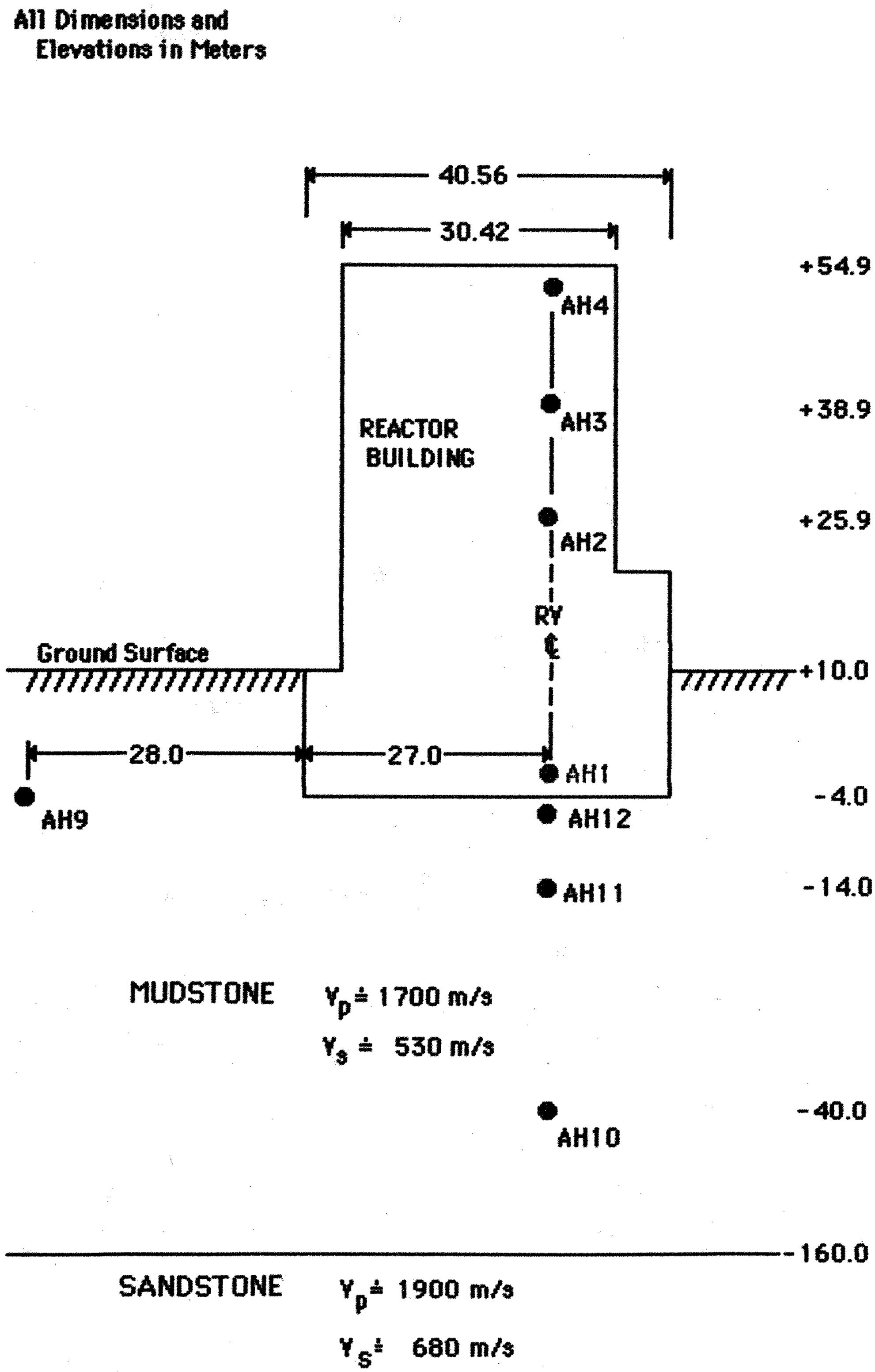


FIGURE 9 EW SECTION OF REACTOR BUILDING
FUKUSHIMA SITE

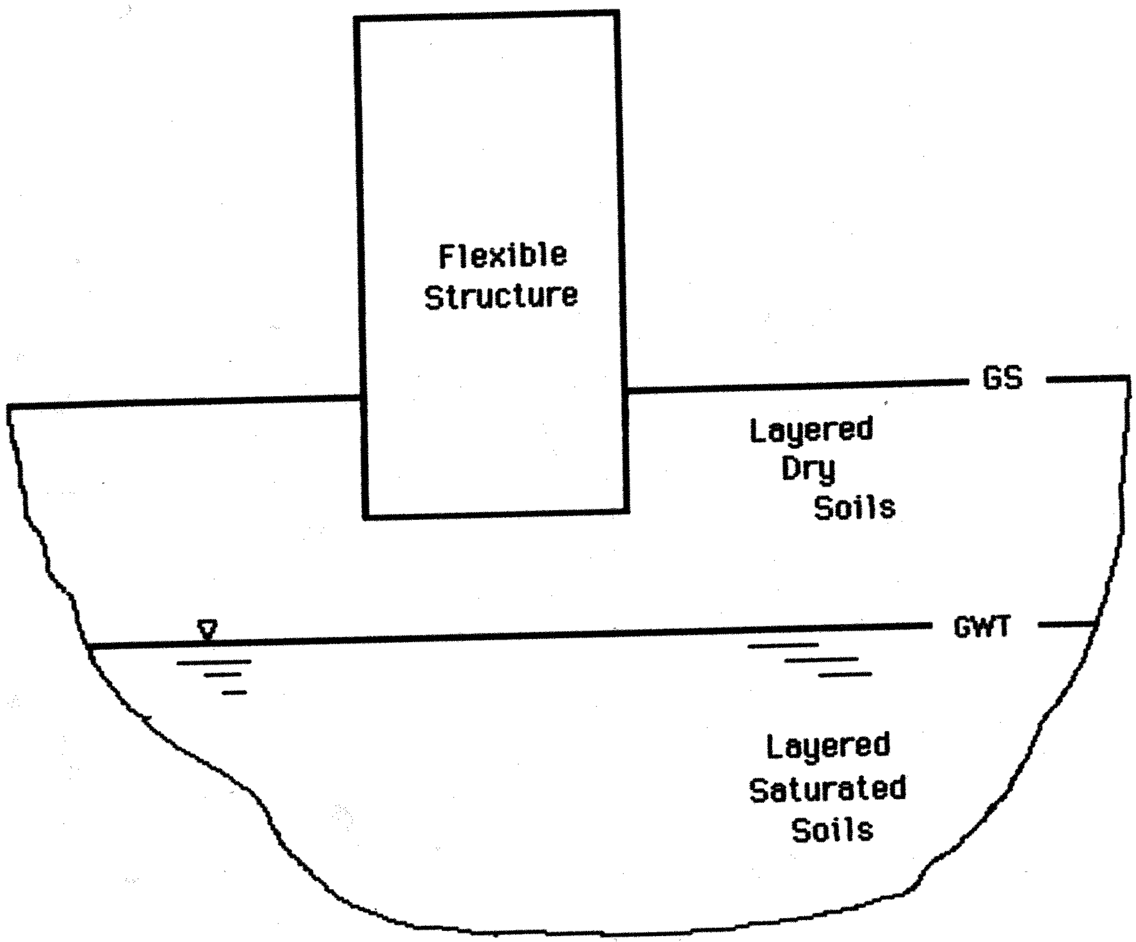


FIGURE 10 TYPICAL CONFIGURATION OF SATURATED SOIL SITE

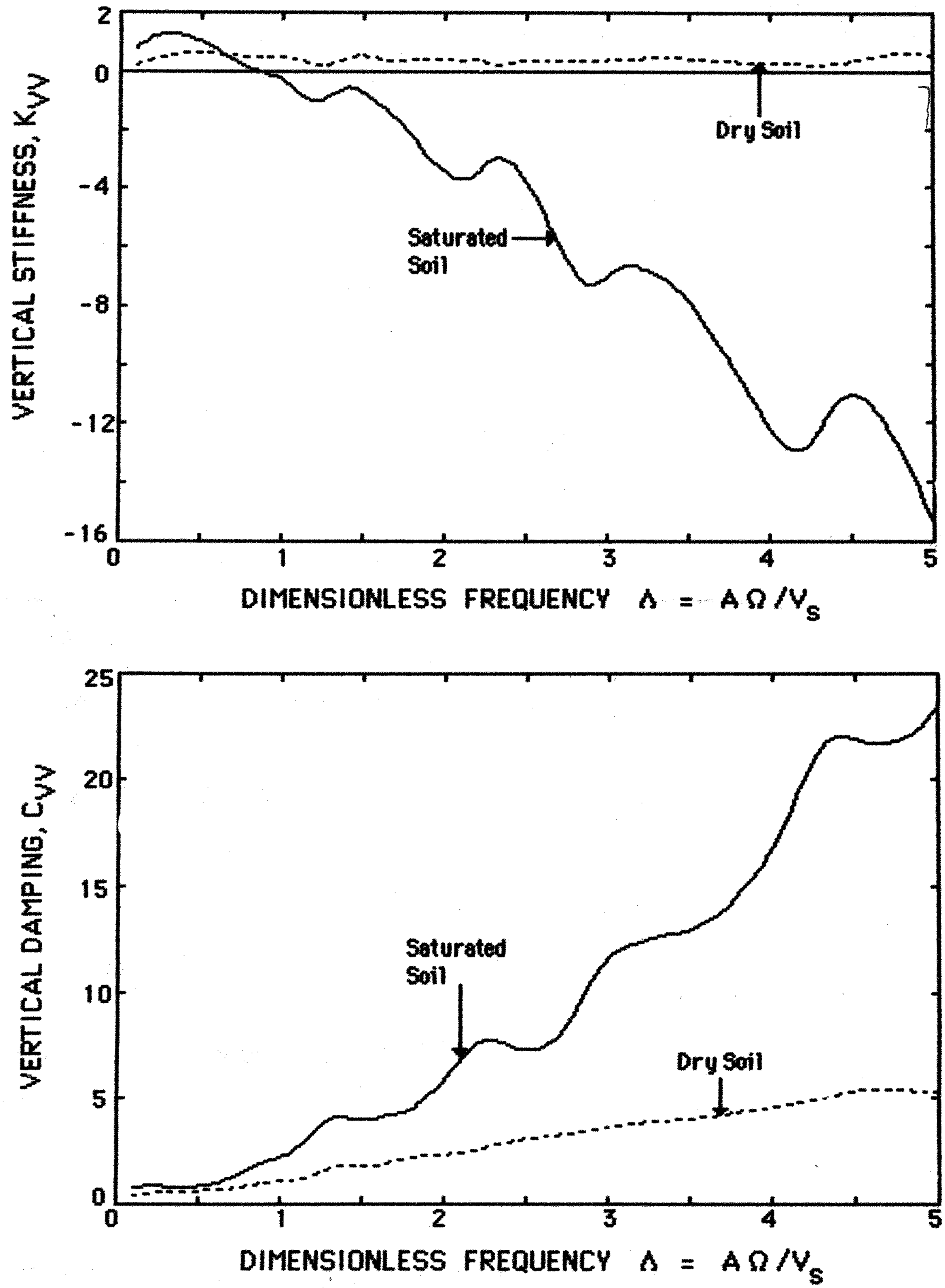
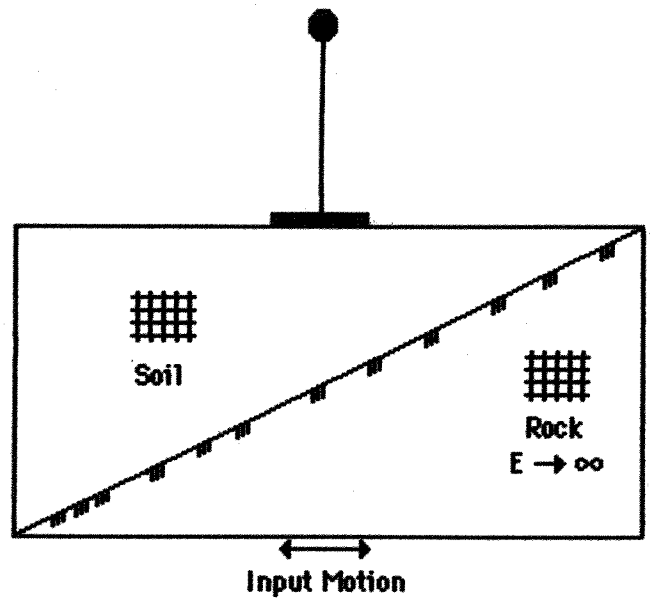
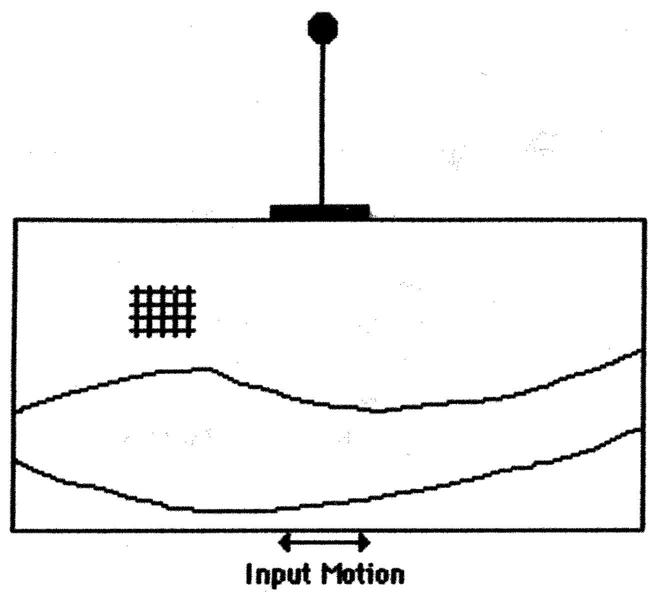


FIGURE 11 COMPARISON OF DRY AND FULLY SATURATED INTERACTION COEFFICIENTS (REF. 5)



(a) SLOPING BEDROCK PROBLEM



(b) UNDULATING LAYERS PROBLEM

FIGURE 12 NONHORIZONTAL LAYER PROBLEMS USING FLUSH TYPE ANALYSES

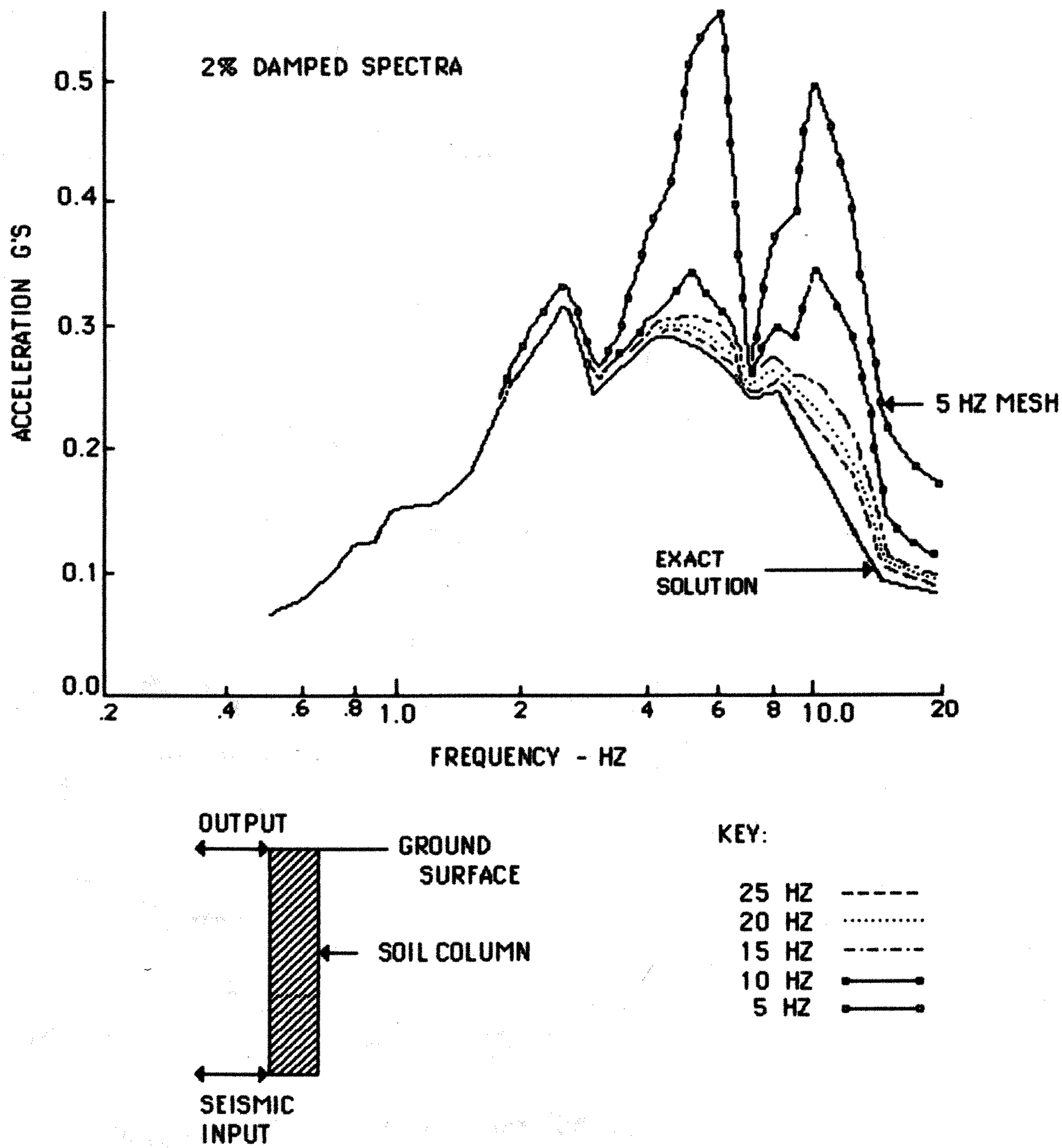


FIGURE 13 ERRORS IN FINITE ELEMENT CALCULATIONS OBTAINED FROM 1D CONVOLUTION STUDIES (REF. 6)

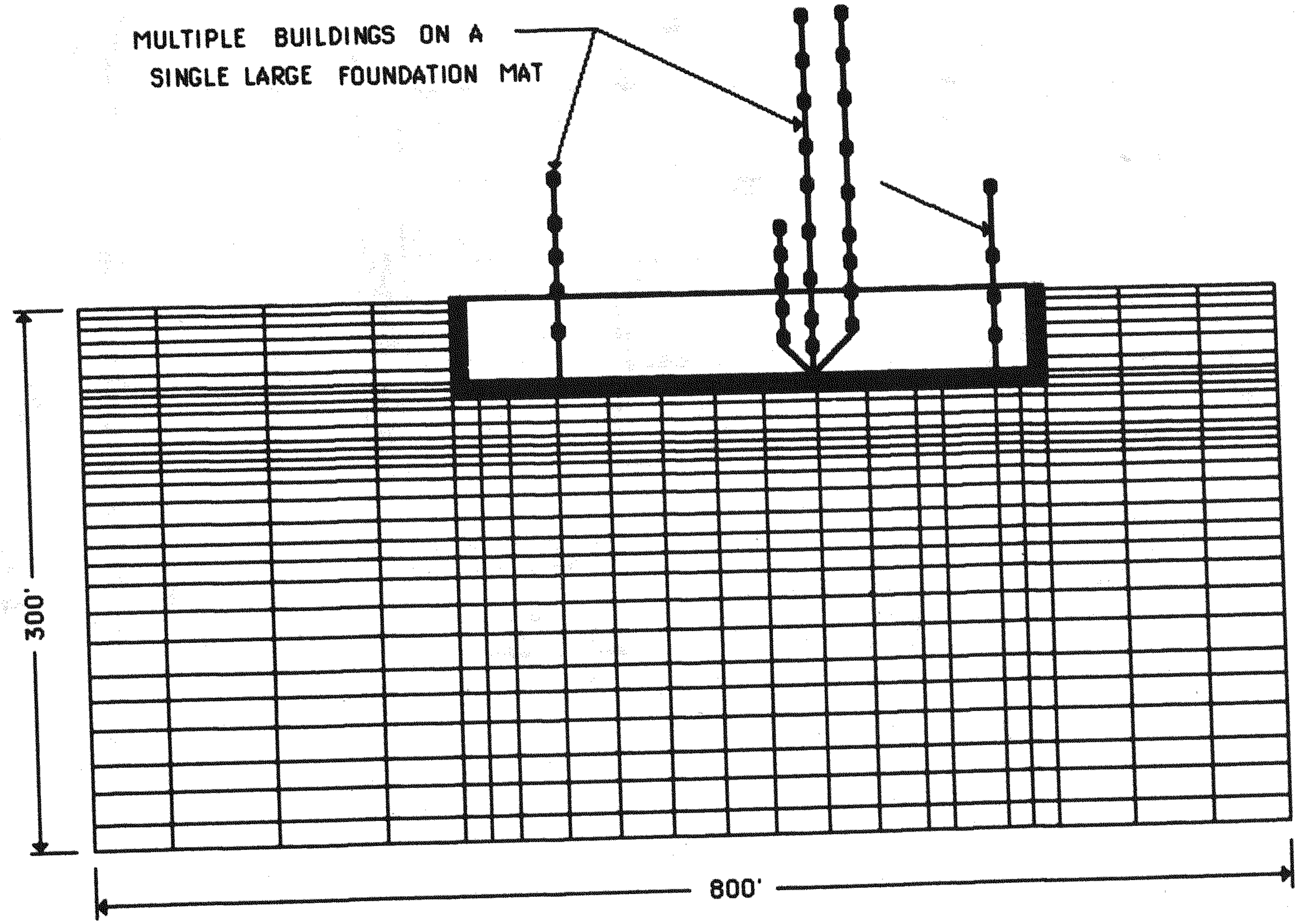


FIGURE 14 LARGE FINITE ELEMENT MESH USED FOR DEEP SOIL SITE

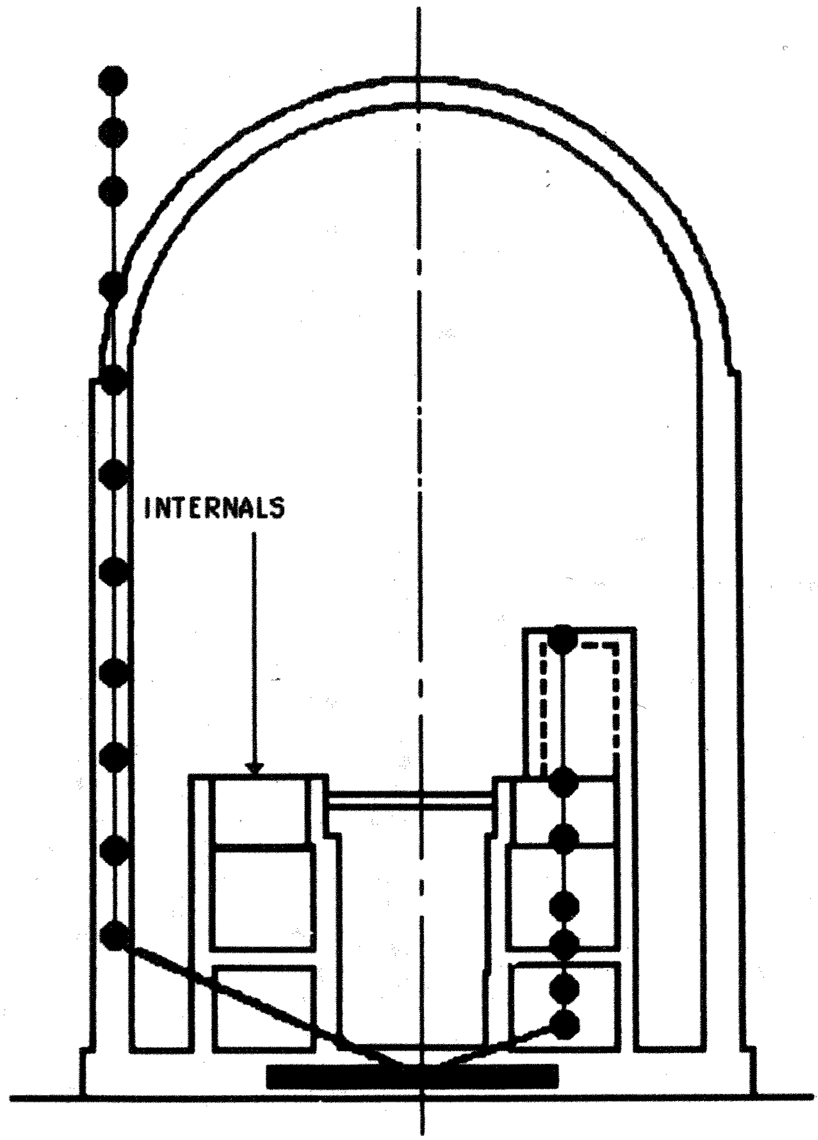


FIGURE 15 TYPICAL REACTOR BUILDING STICK MODEL

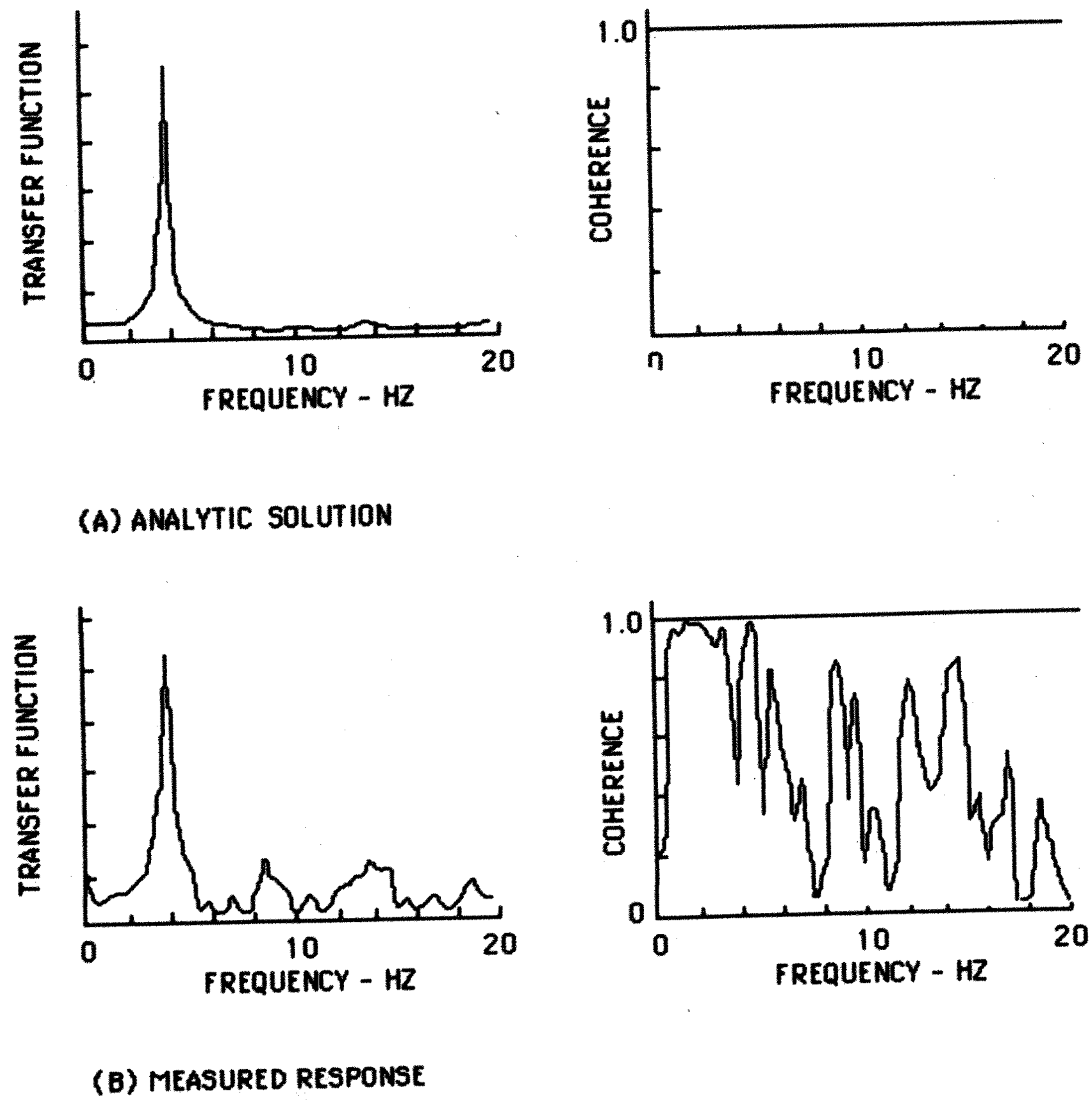


FIGURE 16 COMPARISON OF MEASURED COHERENCE WITH THEORETICAL RESULTS

SESSION 4

SSI METHODOLOGY

LICENSING CONCERNS IN SSI METHODOLOGY

A. H. HADJIAN*

INTRODUCTION

Earlier solutions of the Soil-Structure Interaction (SSI) problem were concerned with the development of lumped spring and dashpot representations of the subgrade for machine foundations. Later, the impedance (substructure) and finite-element (direct) methods for earthquake induced motions began to be developed in parallel and often in a confrontational atmosphere. The hybrid solution followed suit, which treats the subgrade in the immediate vicinity of the structure as a finite-element model and the far-field as the half-space. Boundary element solutions to the problem are being published. The more obvious limitations in the basic methods were immediately recognized and solutions found. Nevertheless, inherent difficulties still remain in both basic approaches which would require significant efforts for their resolution (Hadjian, 1981).

Soil-Structure Interaction has been a controversial subject in US practice from the start. The existing NRC licensing requirements were drawn up when the confrontation between the two basic approaches was at its height. Even though significant improvements have been made to-date, a complete resolution of the problem is not yet possible as will be explained below. However, enough has been learned to-date that existing licensing requirements can be modified and possibly simplified. Based on earlier published studies, the present presentation addresses some of the more immediate NRC concerns.

CHOICE OF SOLUTION METHOD

Given the same model, e.g. a structure founded on a visco-elastic layer overlying a relatively rigid half-space, both solutions, whether two or three dimensional, would produce essentially similar results. Moreover, computationally speaking, both the substructure or the direct methods, continuum or finite element models and time-history or frequency domain solutions would produce equally acceptable results. As the problem becomes more complex however, differences between the solutions begin to appear due to modeling limitations. The outstanding issue therefore relates not to the mathematical manipulations of the solution techniques but primarily to the modeling constraints. For example, extensive studies were carried out (Hadjian et al, 1981) on the reduction of 3D SSI problems to 2D models. Problems that arise in structural modeling, modeling of isolated circular and rectangular foundations, modeling of the complete plant structures and structure-structure interaction were studied in detail. Guidelines are now available to minimize the impact of the two-dimensional simplification of the three-dimensional problem.

Since real situations are more complex than the above simple example, it should not be expected that results from the two basic solution methods be similar in all cases. This is primarily a consequence of the modeling

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constraints of both methods. Insisting that results from both methods be similar for all classes of problems would simply continue the artificial arguments relative to the intrinsic merits of the solutions. Enveloping of responses when one method gives inadequate results does not lead to the correct solution either. One or the other method would be the preferred solution for a given specific problem. The characteristics of the problem under consideration should dictate the use of this or that method. Recognition of the modeling constraints may in certain situations even lead to the use of both methods at different stages of the solution. The licensing emphasis should therefore shift to the appropriateness of this or that method for the solution of a given class of problems. A set of basic requirements for each of the methods can be drawn up to avoid the misuse of the available computer codes for SSI. Moreover, a set of validation problems can be prepared by which all methods and users could be checked for their basic modeling capabilities and user skills. These precautionary steps can be prepared by the experts and viewed as technology transfer to the uninitiated.

NEED FOR SOPHISTICATED ANALYSES

A corrollary to the above is the usefulness of detailed and sophisticated analyses in view of the fact that the details of the actual ground motion to be experienced at the site would, in all likelihood, be different from the one used in design. Reproducing the response of an experiment given the actual input motions is a far cry from the design process objectives. What must be aimed for in design is an acceptable estimate of the response, and the objective should be to endow the structure with a prudent level of resistance, rather than argue about the details of a deterministic response.

Interaction effects on structural response quantities fluctuate from 0.67 to 1.5 of the fixed base response (Hadjian, 1984). Fig. 1 shows the results of a parametric study where the site shear wave velocity was varied to cover a wide spectrum of potential sites and the fundamental frequency of the sample structure was assigned a wide range of frequencies (1/16, 1/4, 1, 4, 16 and 64 Hz). Thus a total of 30 structure-soil systems were analyzed for base shear and overturning moment. The ratio of the system response to its fixed base counterpart value was obtained and plotted against a dimensionless period, $T_o = V_s/1R$. As expected the ratios are both above and below unity, indicating that SSI can both amplify and reduce response of structures. The important point to be made is that the increase is limited only to a factor of 1.5. (Incidentally, the site factor, S, of the UBC code is also limited to 1.5). The use of fixed base results should be encouraged as a check on the adequacy of the structural response quantities. The incremental effort to obtain fixed base results is usually small.

With regards to the response of equipment housed in nuclear plant structures, the system frequency characteristics are more critical to safety than the variation of response amplitude. The latter will vary anyway when the real earthquake happens, as a result of the peaks and troughs that show up in the response spectra of real events. However, the correct prediction of the interacted system frequencies is of paramount importance given the peakedness of floor spectra which occur at the system frequencies. The issue of frequency variation is discussed next.

FREQUENCY VARIATION

To account for uncertainties in material properties, modeling assumptions, analysis simplifications, non-linear behavior and a host of other unknowns, the use of the "worst case" analyses technique is usually advocated. In this approach arbitrary high- and low-parameter values are assumed irrespective of a) the nature of the variability, b) the probability of achieving the high and low values of the parameters, and c) the care and expense by which parameter values have been determined. A worst case parameter by parameter variation can lead to innumerable combinations and extremely conservative results. Using a single parameter, such as the soil modulus, to encompass all other variabilities may not be a physically defensible procedure. Nothing much can be said about the end results except that they are different from the best estimate results. A better alternative is to first determine the probability density functions of the significant parameters in question and then to calculate the probabilistic frequency variations of structure-soil systems.

Assuming that the dominant variability in concrete structures stems from variations in the modulus of elasticity E and the (cracked) moment of inertia, I, it has been shown that all modes of a fixed base structure have the same frequency distribution (Hamilton and Hadjian, 1976). The results of such a calculation are summarized in Table 1. With E-only effect considered, the frequencies are overestimated more than they are underestimated for the same probability fractile; that is, the mean value of the frequency is larger than the nominal frequency. This is expected since the mean value of the compressive strength of concrete is larger than its nominal value by design. However, when the EI variation is considered, the mean value of the frequency is less than the nominal, indicating that the impact of the variation of the moment of inertia (due to cracking) is greater than that of the variation in E. For the more severe EI variation an underestimate of the frequency by 15% occurs with a probability of 0.1 and an overestimate of the frequency by 15% occurs with a probability of only 0.05. Moreover, there is only a one percent chance that the frequency will vary by about $\pm 23\%$. Obviously this is the better way to describe the variability of structure frequencies.

Figs. 2 and 3 show SSI results when the variation of the impedances are introduced. For lack of actual soil data similar to concrete, the impedances are assumed to be normally distributed with the nominal value set at the mean. The variability of the impedances is characterized by the

Table 1. Frequency Fractiles in a Typical Beam
(Expressed as Percentage Variations With Respect to the Nominal Frequency)

P	.01	.05	.10	.90	.95	.99
E-only Effect	-6.6%	-3.1%	-1.2%	+12.1%	+14.0%	+17.6%
Joint EI Effect	-23.5%	-18.0%	-14.9%	+10.7%	+14.9%	+23.2%

coefficient of variation. The salient points from the curves are the following:

1) The impedance variability impacts only on the interacted frequencies, as it should.

2) SSI does not necessarily increase the variability in the system frequencies as compared to the fixed-base frequencies. In fact, it may serve to reduce this variation for certain frequencies if the impedances have relatively small variabilities. A parameter variation (worst case) procedure cannot lead to such a reduction.

3) As the variability in the impedances increases from s = 0.0 to s = 0.5, the frequency variation increases but not by the same amount for all frequencies.

4) The variability in the impedances should be large to begin impacting the system frequency over and above the fixed base variability.

The licensing emphasis should be on rational principles rather than arbitrary procedures. The latter could lead to untenable conclusions when applied to diverse situations.

GROUND MOTION REDUCTION WITH DEPTH

The issue of ground motion reduction with depth for embedded structures has not been properly considered, primarily because the reduction could be very large or very small depending on the size of the excavation. With proper modeling, the problem should take care of itself and there is absolutely no need to introduce artificial constraints. The significant troughs at foundation level reported in the literature are simply due to the use of smooth design surface spectra.

Referring to Fig. 4, and adopting primarily a designer's perspective, both buildings in the top half of the figure (a) should be designed for the same motion, notwithstanding differences in motion recorded in close proximity. The excavation is large enough that design motions at A and B must be assumed to be the same. In the lower half of the figure (b) the situation is different and the motions at A and B, in the absence of the buildings, must be different; and, in the limit, as the width of the excavation tends to zero, the motion at A and C would become the same. Thus, reduction of motion could vary from none to that at point C in the free-field, depending on the size of the excavation. Both analytic (Hadjian et al, 1986) and earthquake recorded motions will be presented in support of these arguments. In the past these effects were invariably evaluated by the response of embedded structures, thus mixing the effects of the variation of ground motion with depth and inertial interaction. A more appropriate investigation of this problem would be through the elimination of inertial SSI effects.

The basic study model is shown in Fig. 5, where the model is expected to represent three lateral excavation sizes in an elastic half-space. This discussion is strictly applicable to those cases that do not have an identifiable hard stratum close to the foundation. The following two variations of the basic model are also used either separately or together: a) the top layer in the excavation is replaced with concrete to simulate the presence of a rigid basemat and b) the bottom boundary is contoured to follow the surface excavation. The purpose of the concrete slab is to average out the motions in the excavation and that of the bottom contour, to eliminate the highly sensitive length parameter of a finite soil column from unduly impacting the results. This important latter effect is shown in Fig. 6 where the ratio of the response of two soil columns relative to a 300 ft soil column is presented. Large amplitude and frequency shift effects are to be noted.

Fig. 7 shows a set of typical results from this study for a 300 ft wide excavation without concrete slab or bottom contour. Response ratios greater than unity indicate that the response in question is higher than the corresponding free-field response. The first number in the frame designation refers to the node as defined in Fig. 5 and the second number to the free-field motion: 9 refers to the free-field at 30 ft below the free-surface and 10 refers to the free-surface motion. The results in Fig. 7 indicate that the response ratio in the excavation relative to the surface (node 10) is, in general, larger than unity. Additionally, the response in the excavation is not uniform. As the excavation size is reduced to 30 ft (Fig. 8), the response ratio in the excavation relative to the surface becomes less than unity and that relative to node 9 is drastically reduced.

Fig. 9 is a summary of the results at the center of the 300 ft excavation. The combined effect of both the basemat concrete and contouring does achieve, in a remarkable fashion, the premise of the study: namely, that ground motion characteristics should not, at least from the viewpoint of design, be dissimilar in and out of large shallow depressions. For all practical purposes the response ratio in Fig. 9d fluctuates, only slightly, about 0.93. It is expected that as the excavation width becomes even larger, the average response ratio will tend towards unity with only minor fluctuations.

The above expectation is evident in Fig. 11 which shows the response spectra comparisons of earthquake ground motions recorded at the Fukushima Nuclear Power Plant during the Magnitude 7.4 Miyagi-ken-Oki, Japan Earthquake of June 12, 1978. The basic data is reproduced from Miller et al, (1985). Fig. 10 shows the relative locations of the recorded data around the Reactor Building (RB).

Fig. 11a is a comparison of the recorded motion above and below the basemat. The differences in the spectra are minor and it can be concluded that the rigidity of the slab is such that there is no amplification through the slab and the motion at El -4.0 can be considered to be the free-surface motion in the excavation. Obviously interaction effects to some unknown level are included in both of these signals. Fig. 11c shows the modification of motion in a length of 26 meters. El -14M being away from the free-surface does not show significant variation from the motions at El -40M, except in the frequency range of 1.5-3.0 Hz. In Fig. 11b, which compares El -14M with El -4M, there is further amplification in this same frequency range. It thus can be concluded that response spectra reduction occurs with depth in a given frequency range and this reduction can be significant; about 3:1 on the average in a depth of 36 meters. Starting with a Reg. Guide 1.60 type smooth surface motion, significant troughs at the foundation level are therefore expected. More importantly though in the high frequency regime, greater than about 4-5 Hz, the motion attenuates as it propagates upwards. The more significant comparison is shown in Fig. 11d that compares the motions at the same elevation, one inside the foundation medium and the other on the free-surface of the excavation. It is not surprising that the motion to the side of the reactor building is significantly smaller than the motion at the free-surface inside the excavation.

Three points have been made: a) Design ground motion should be more appropriately specified at the free ground surface. b) ground motion reduction with depth is controlled by the extent of the free-surface at a given excavation. The larger the width of the excavation the more the motion tends to be similar to the free ground surface. The smaller the width of the excavation the more the motion tends to be similar to the free-field motion in the medium itself at the same elevation. Thus ground motion reduction could vary from none to a maximum (that in the free-field at depth) as a function of the extent of the excavation. c) if properly modeled (i.e., bottom contouring and averaging by a rigid slab), the reduction phenomenon will take care of itself and no special constraints are needed. In the absence of the above modeling recommendations, the response could take on some undefined characteristics and conflicting results have been often reported regarding this issue.

BURIED STRUCTURES

The emphasis in buried structures should be on design rather than analysis. Underground structures survive earthquakes if proper design considerations are employed. More than inertial effects, buried structures are subjected to strains and therefore the thrust should be on minimizing interaction. The Mexico City underground rail system was back on track again as soon as power was restored. The rail support system was such that strains were not transmitted (that is interaction was minimal), between foundation and rails. In high seismic regions we have adopted the use of electrical duct banks embedded in sand to minimize the interaction that would otherwise take place if a more rigid construction material, such as concrete, is used. Another design consideration for buried structures is articulation. If interaction cannot be minimized the structure should be articulated. This concept was also employed in the Mexico City underground metro tunnel construction where 6M long segments of the tunnel were "hinged" to allow the strains to accumulate at the hinges without causing tensile cracking or compressive buckling.

CONCLUSIONS

Several current issues in soil-structure interaction of concern to the staff of the Nuclear Regulatory Commission have been reviewed and the following recommendations are presented for consideration by the staff.

1. The calculational methodologies have matured to a point where any one of the presently available methods can be used. Obviously, as the complexity of the problem increases, one or the other of the basic methodologies may be the more appropriate tool to be used simply due to the modeling constraints. The licensing emphasis

should therefore shift to the modeling constraints of each of the methods commonly used. To avoid misuse a set of basic requirements for each of the methods can be drawn up. Moreover, a set of validation problems can be prepared by which all methods and users could be checked for their basic modeling capabilities and user skills. A technology transfer from the experts to the uninitiated must be made.

2. Sophistication in analysis may be necessary but should not be a substitute for good engineering. Since the structural response quantities may vary from 0.67 to 1.5 times the fixed base response, emphasis in unnecessary detailed calculations is not warranted. Given the peaks and troughs of actual recorded ground motions, the response during the earthquake could vary from the design values by similar factors. Therefore the emphasis should shift to an acceptable estimation of system frequencies as this may have important effects on the response of supported systems (by identifying a wrong peak of the floor spectra).

3. To account for uncertainties a single parameter variation (usually that of the soil) to account for several important uncertainties does not seem to be justified. A more rational approach to calculate the mean and the variability of system frequencies can be had if all important input parameters are described in a probabilistic format. Depending on the relative severity of the variabilities of the structure and foundation properties, soil property variability may not even be a controlling parameter.

4. Design ground motion should be more appropriately specified at the free ground surface and its subsequent reduction with depth will then depend on the extent of the excavation for the structure embedment. For large embedment width this reduction could be only very minor. For excavations with small lateral widths the reduction could be larger. These variations in ground motion reduction are expected and should be allowed without insisting on artificial constraints. The more important concern should be the details of the soil modeling that impacts on this phenomenon.

5. The emphasis in buried structures should be on design rather than analysis. Design should strive to minimize interaction where possible. Where not possible, design should strive to pre-select those locations where strains should accumulate without causing tensile cracking or compressive buckling to occur at other undesirable locations.

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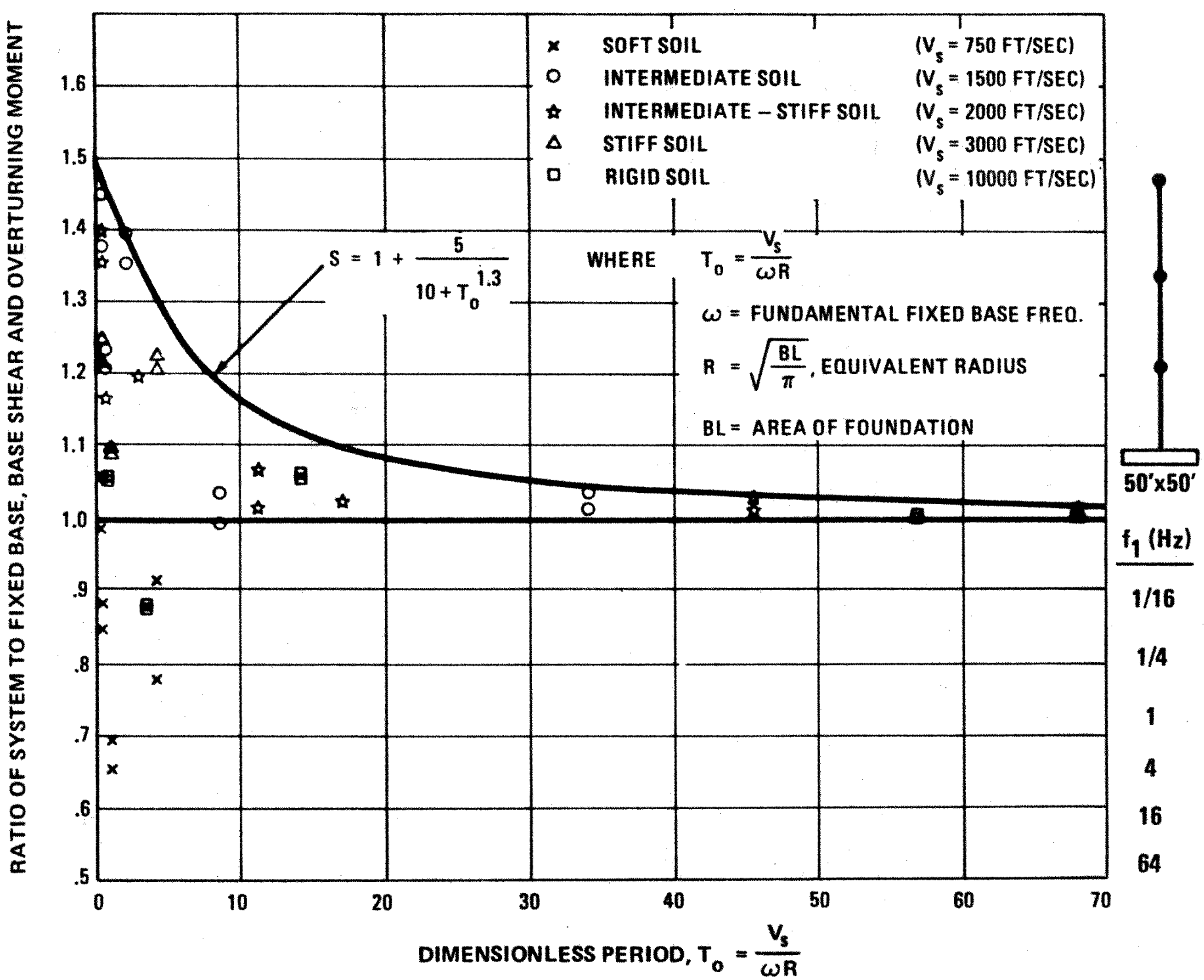

FIG. 1 IMPACT OF SOIL-STRUCTURE INTERACTION ON BASE SHEAR AND OVERTURNING MOMENT

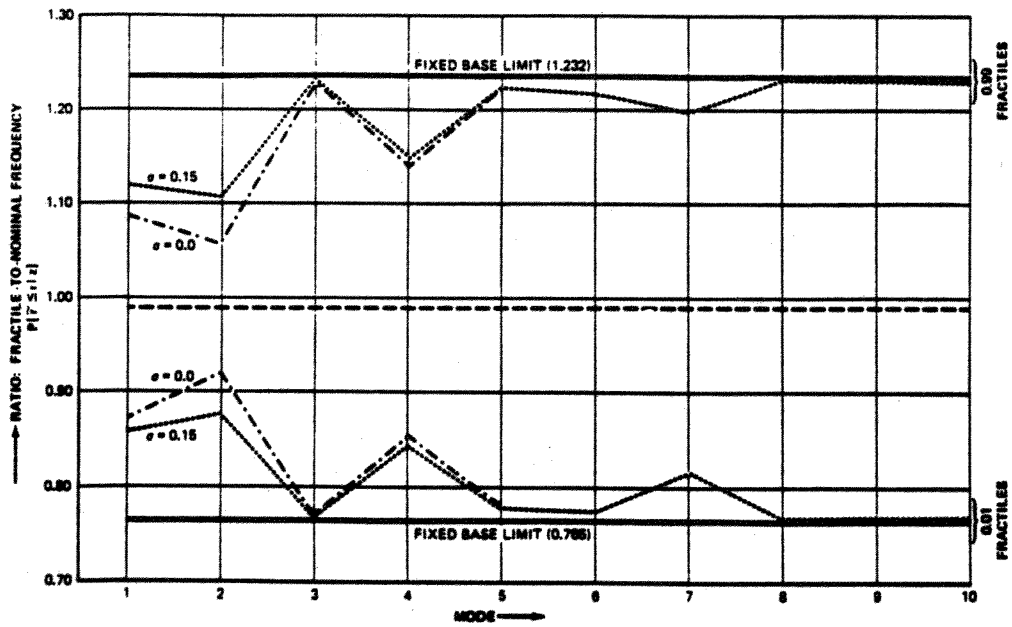

FIG. 2 EFFECT OF SOIL VARIABILITY AND MODE LEVEL ON0.01 AND 0.99 FREQUENCY FRACTILES: CONTAINMENT BUILDING.

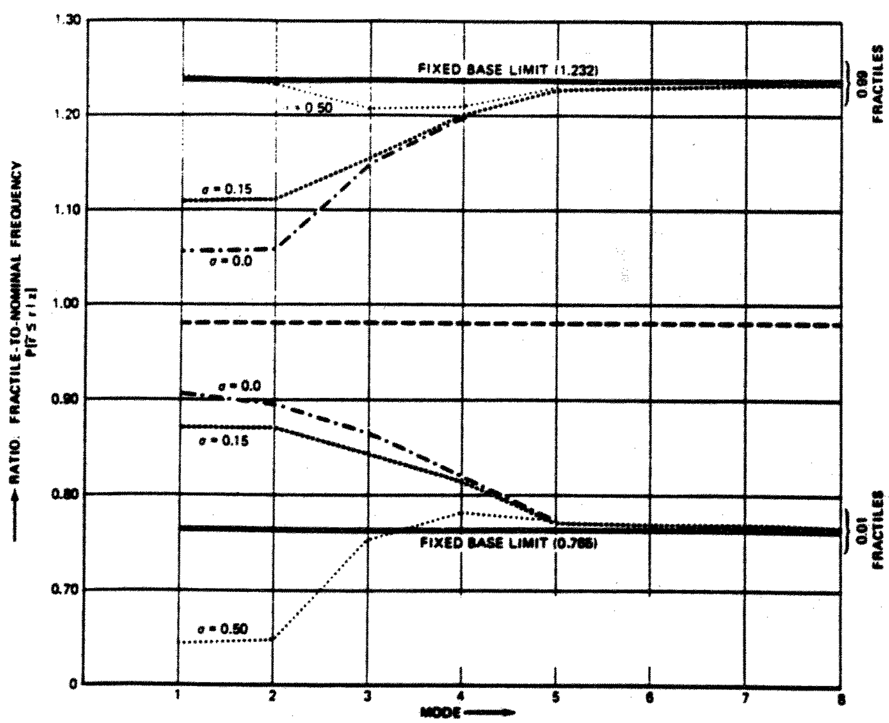

FIG. 3 EFFECT OF SOIL VARIABILITY AND MODE LEVEL ON 0.01 AND 0.99 FREQUENCY FRACTILES: AUXILIARY BUILDING.

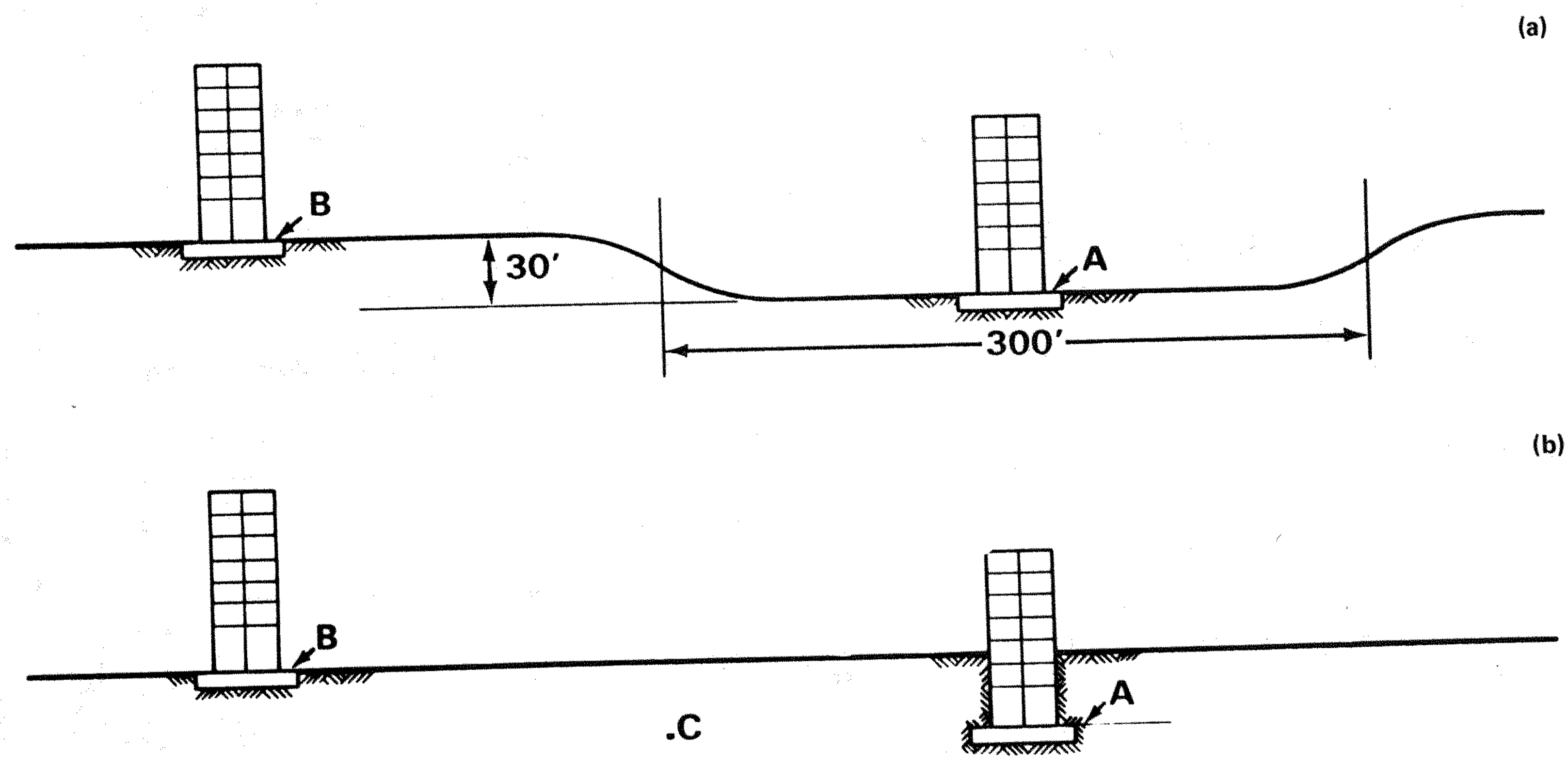

FIG. 4 GROUND MOTION VARIATION WITH EXTENT OF EXCAVATION

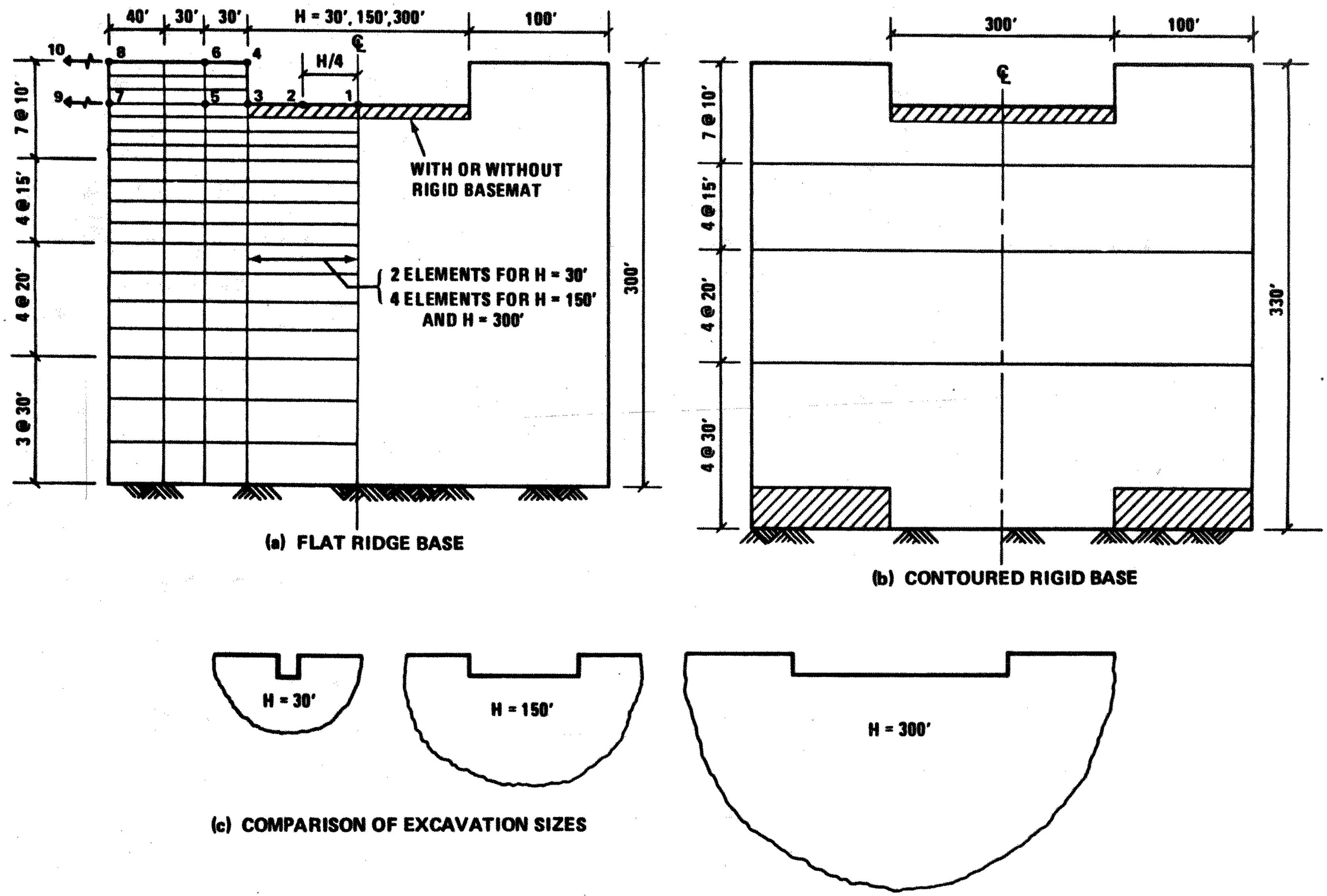

FIG. 5 DETAILS OF MODELS USED IN STUDY

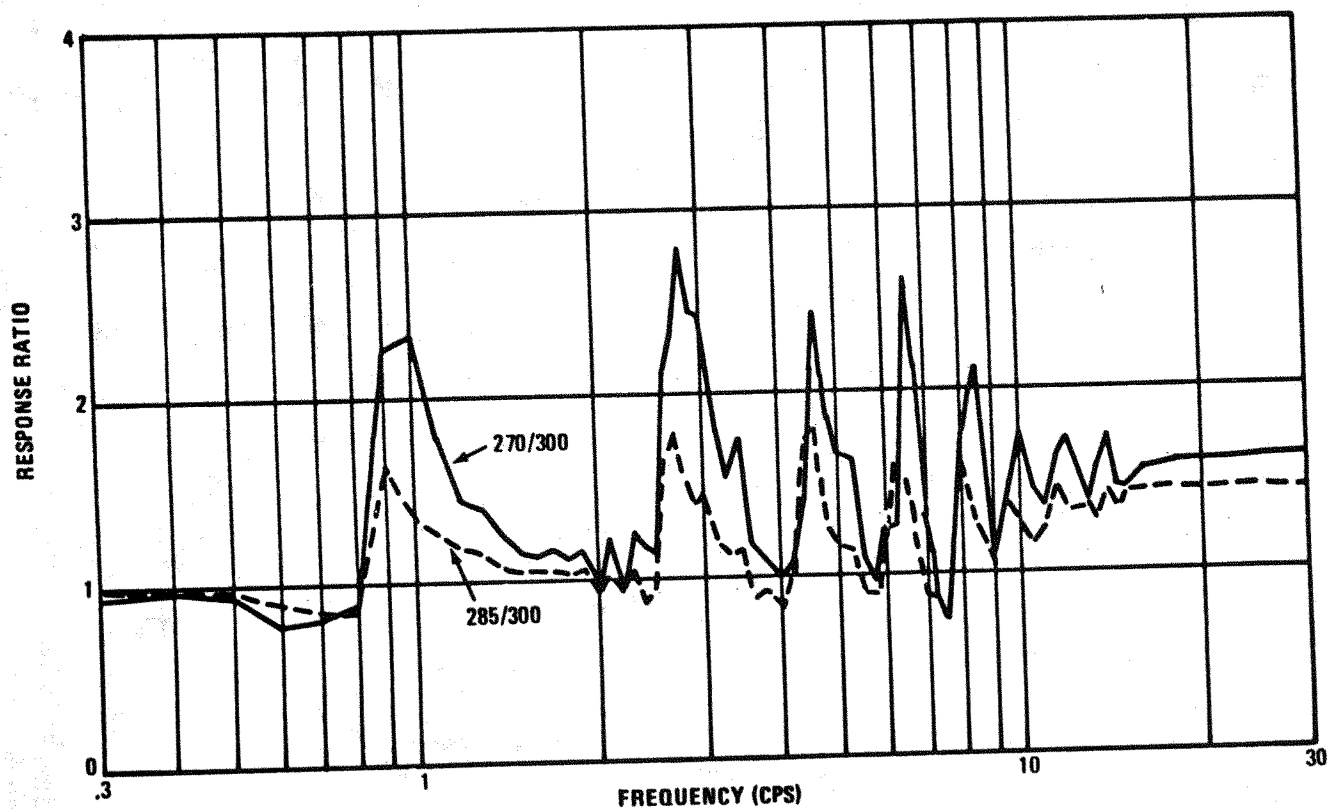

FIG. 6 RESPONSE RATIO OF 270 FT AND 285 FT SOIL-COLUMNS RELATIVE TO 300 FT SOIL-COLUMN

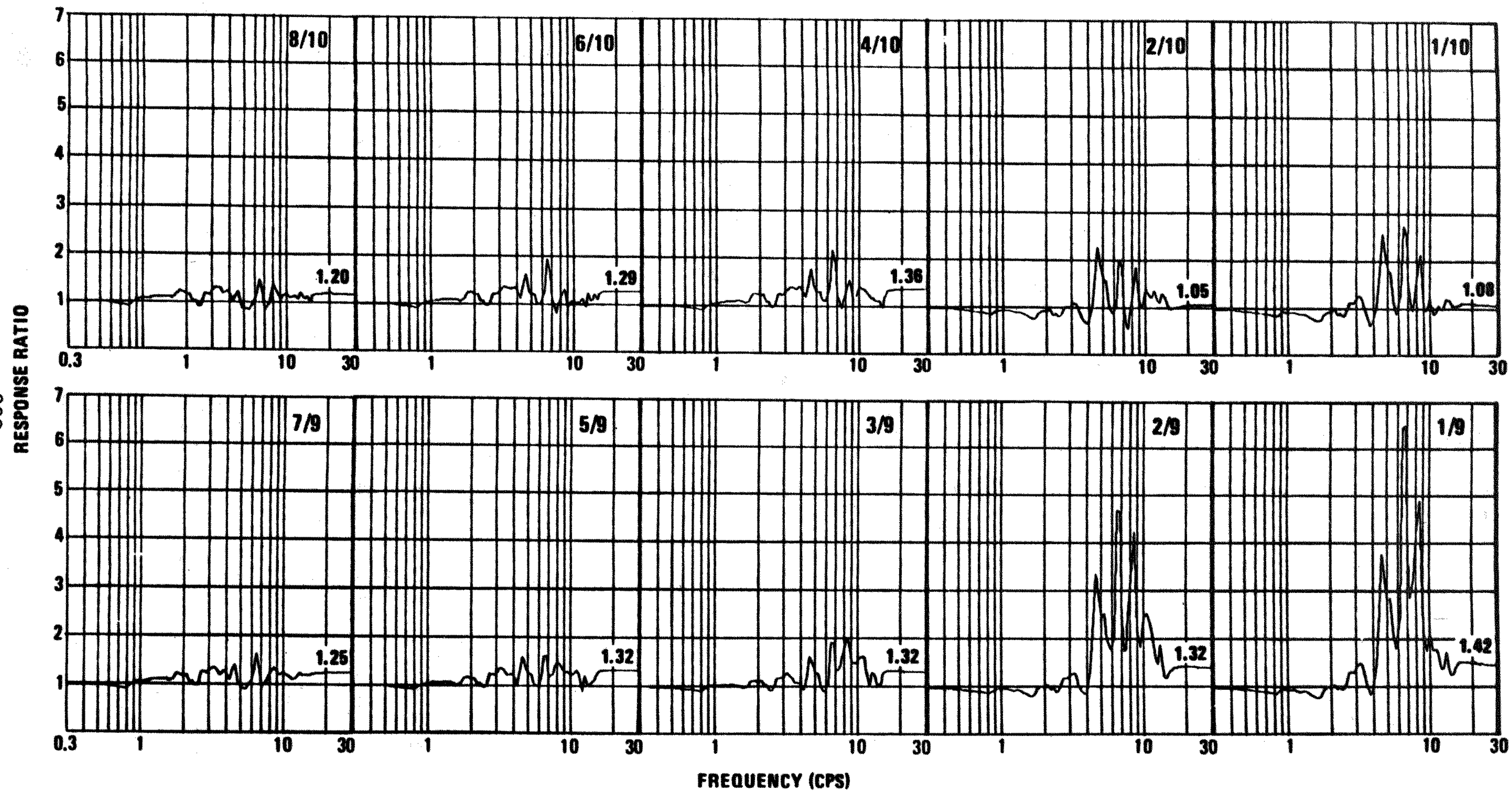

FIG. 7 RESPONSE RATIOS AT NODES 1 THROUGH 8 FOR MODEL 4

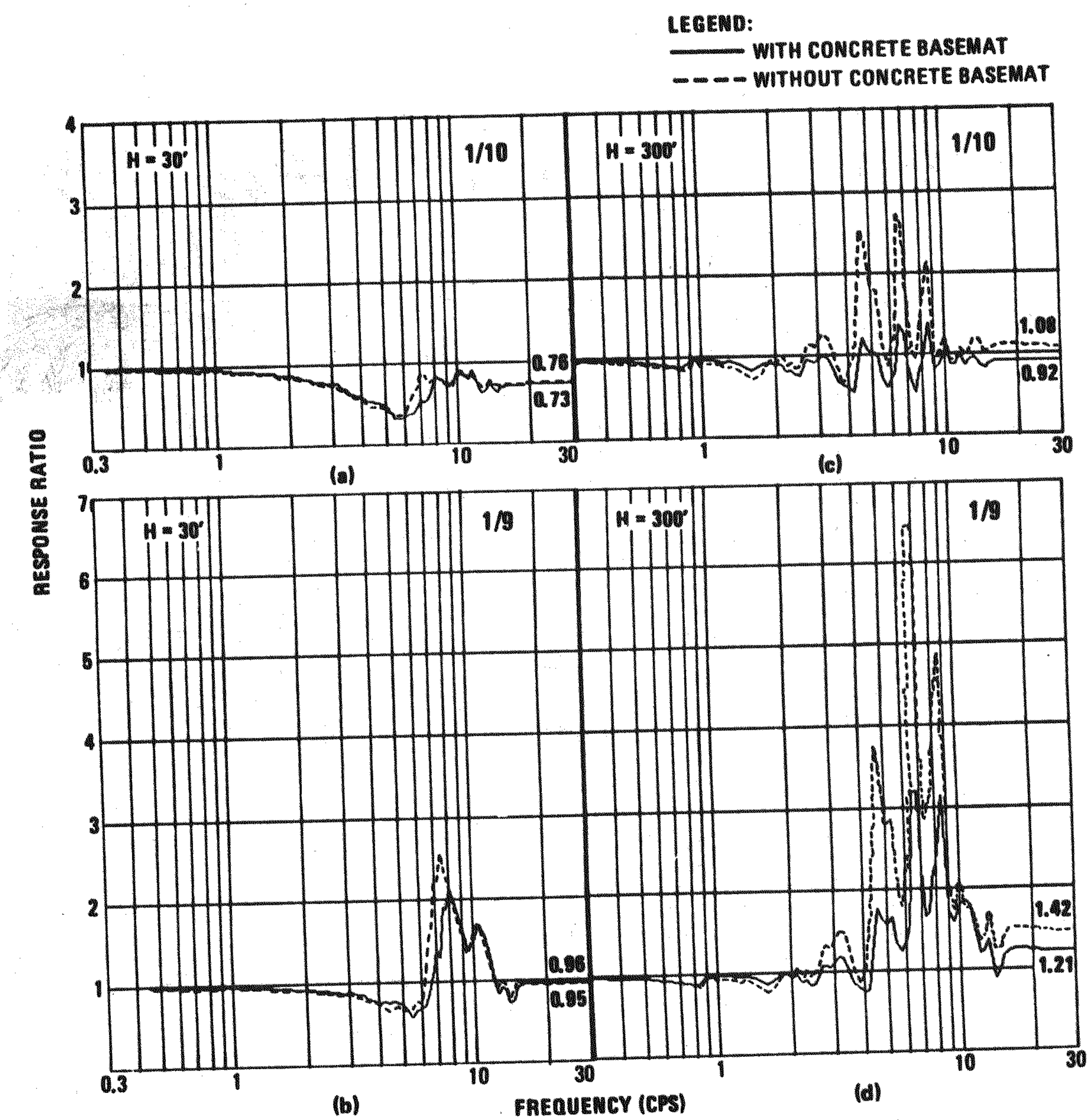

FIG. 8 COMPARISON OF RESPONSE RATIOS AT CENTER OF EXCAVATION

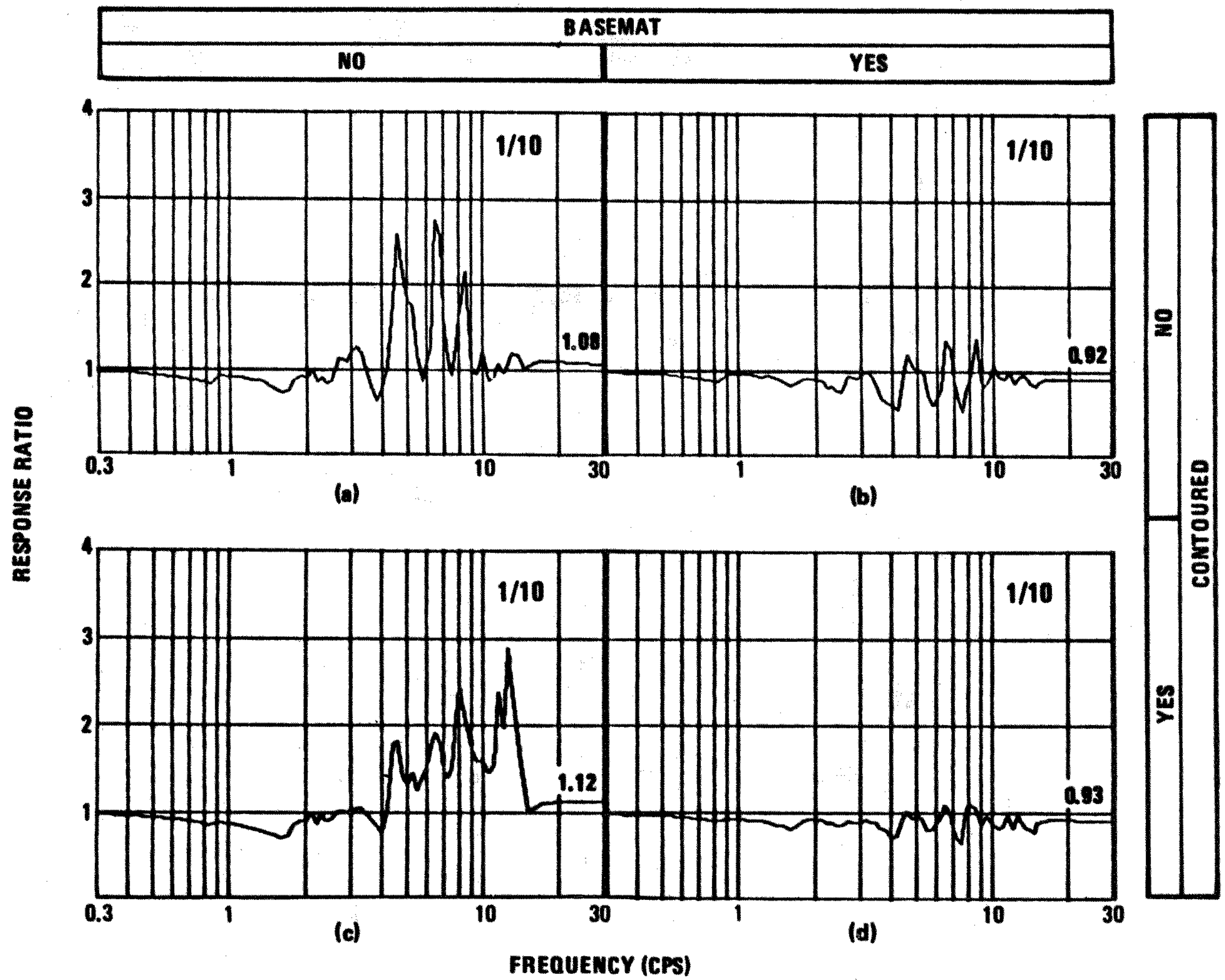

FIG. 9 RESPONSE RATIOS FOR 300FT EXCAVATION

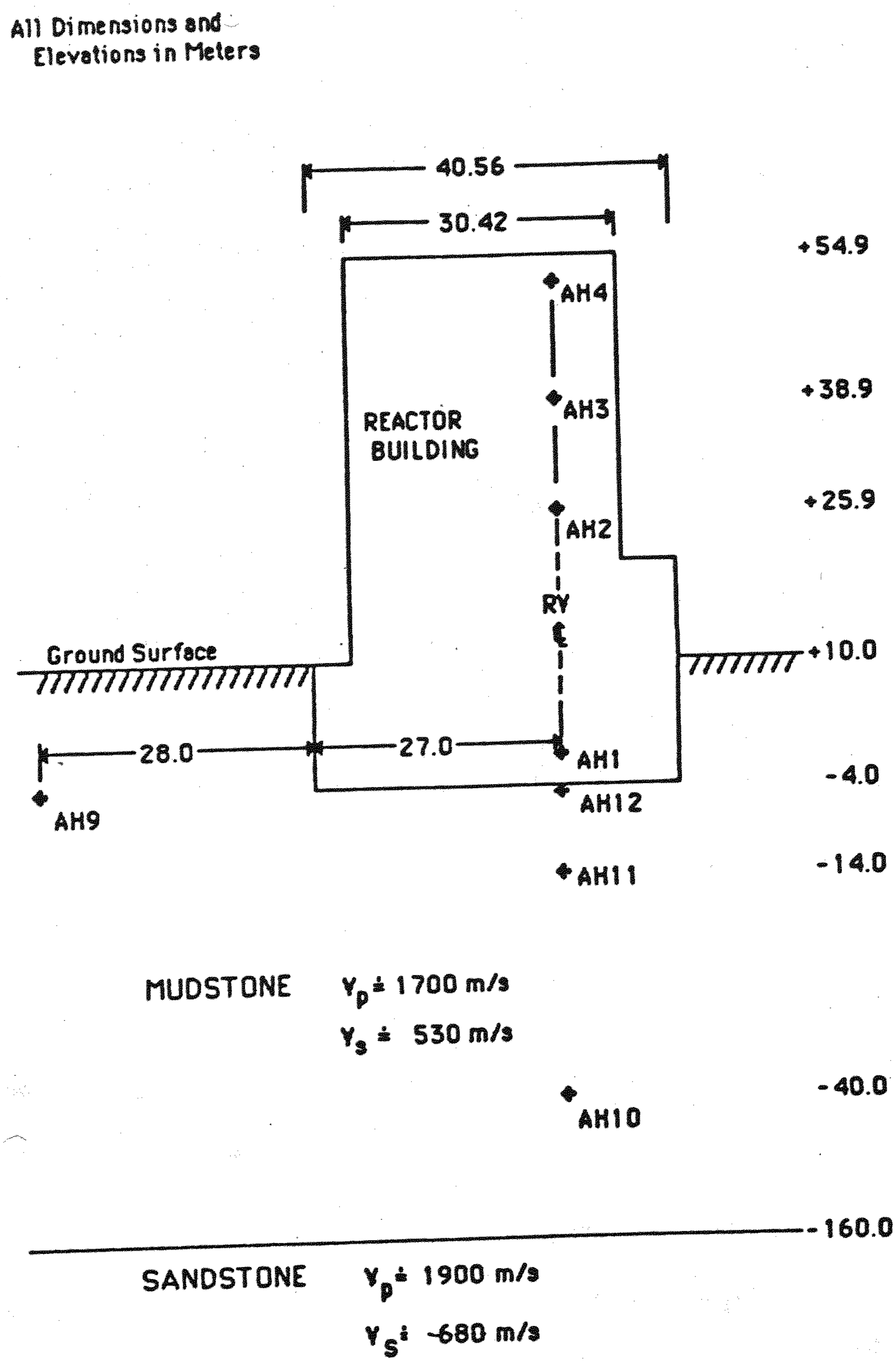

FIG. 10 E-W SECTION OF REACTOR BUILDING

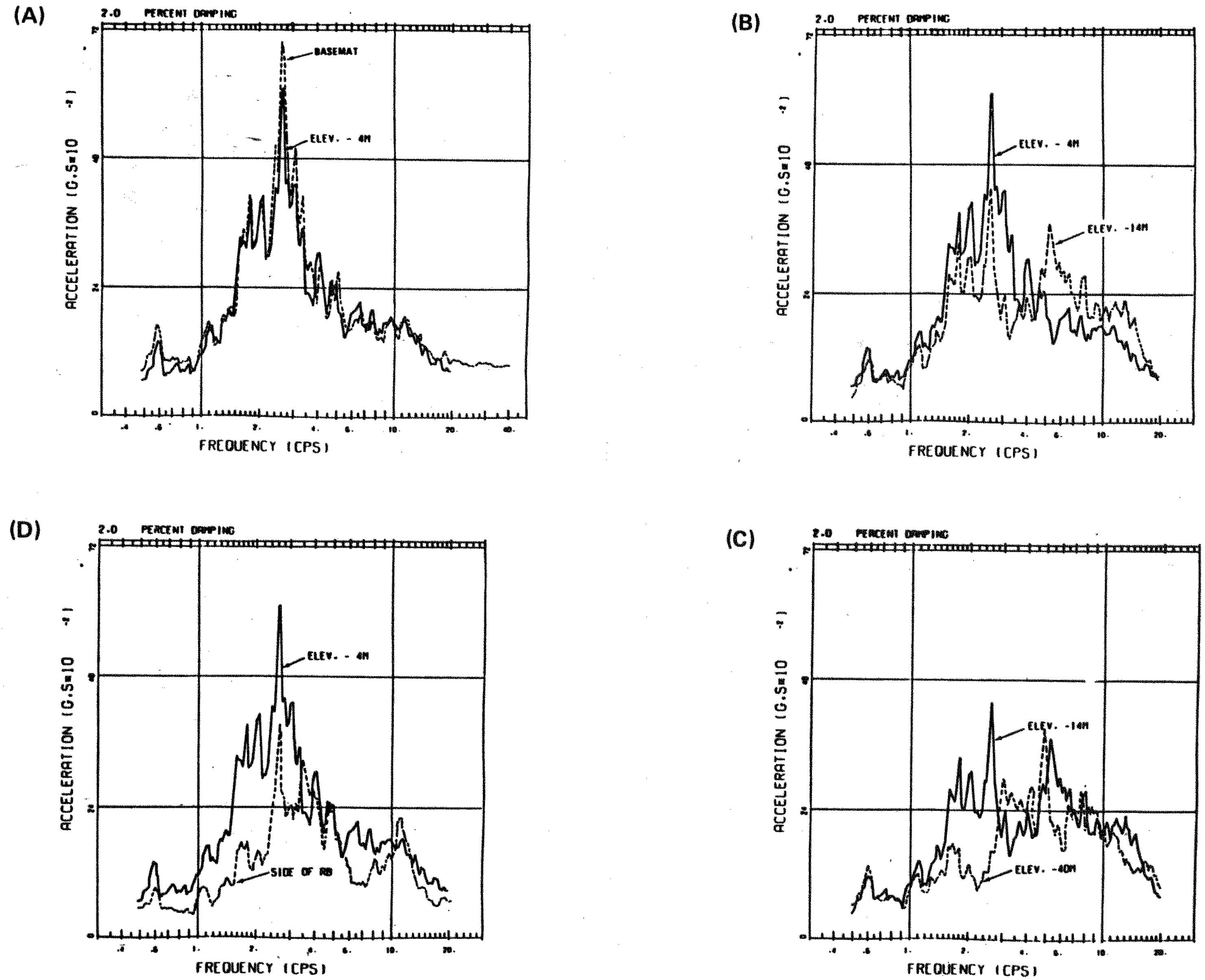

FIG. 11 ACCELERATION SPECTRA FOR FUKUSHIMA PLANT, JUNE 1978

SOME PERSPECTIVES ON DYNAMICS OF SOIL-STRUCTURE INTERACTION

by

A. S. Veletsos[1]

INTRODUCTION

The primary objective of this paper is to indicate the value of simple approaches in studies of soil-structure interaction, and to review two examples of such procedures. Additional objectives are to identify what is believed to be a source of current controversies on the subject, and to present a brief account of areas of needed research.

The first of the examples considered concerns the interaction effects for certain classes of building structures, whereas the second deals with the hydrodynamic effects in above-ground liquid storage tanks, and with the consequences of soil-structure interaction on the design of such systems.

NEED FOR SIMPLE APPROACHES

With the methods of analysis and the high-speed computers now available, it is, in principle, possible to evaluate the dynamic response of any structure-foundation-soil system to any prescribed excitation of the base. It is self-evident, however, that the results of such analyses can be no better than the assumptions on which they are based.

Central to any analysis of the seismic response of a structural system are assumptions and approximations concerning the physical properties of the structure and the supporting soils, the characteristics of the excitation, and the manner in which the ground motion reaches the site and is transmitted to the structure. Normally, assumptions also are involved in the analysis of the idealized system itself, and there is the ever present possibility of improper use of the method. It is important, therefore, that such analyses, and the computer programs which are needed for their implementation, be used with due appreciation for the underlying simplifications, and that the results obtained therefrom be interpreted with due recognition for the sensitivity of the response to possible deviations from the presumed conditions.

Even with the high-speed computers now available, mathematically precise analyses of complex structure-foundation-soil systems require extensive computational effort and are generally too costly for preliminary design purposes. This is particularly true if account is to be taken, as it must in most cases, of the effects of inelastic and other nonlinear actions.

Compounding the difficulty is the fact that, because of the multitude of factors that influence the dynamic response of such systems and because of the sensitivity of the response to many of these factors, it is generally quite difficult, if not impossible, intelligently to interpret the results of a small number of complex analyses. Budgetary and/or time constraints

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often limit the sets of possible conditions that may be considered with such methods, and design decisions often are reached with inadequate assessment of the effects upon the final design of the numerous uncertainties that are inherent in the problem and its idealization. The end product under such conditions may be no better than, and may in fact be inferior to, what could be achieved with less effort and at a lower cost with simpler approaches, with which the effects of variations in the more important factors can be assessed readily. To be acceptable, the simpler procedures must, of course, be rationally based and must be capable of capturing the essential aspects of the problem.

The highly refined, complex methods also are more likely to misinterpretation and misuse than the simpler methods. Furthermore, they tend to provide a false sense of security and accuracy to their users, and may discourage the exercise of the educated judgement which is essential to a satisfactory design. Expressed differently, increased complexity and sophistication in analysis do not necessarily ensure improved design.

There is a pressing need at this time for the development of simple procedures and concepts of structural action with which the effects of the principal factors that control the response of soil-structure systems can be assessed reliably and cost-effectively.

In addition to being ideally suited for preliminary design studies, such simple approximate methods may, in many cases, be adequate for final design purposes as well, and may provide the framework needed for the planning of the more elaborate analyses that may be required and for the interpretation of the results of such analyses. Finally, the simpler approaches are essential to the formulation of design regulations and code provisions, and to the proper planning of the experimental programs which are needed to guide the analytical developments and to assess the reliability of the analytical predictions. Two examples of such procedures are presented in subsequent sections.

SOURCE OF SOME CURRENT CONTROVERSIES

Some of the current controversies regarding dynamic soil-structure interaction are believed to stem from a lack of an agreed upon definition of the effects of interaction. This statement is amplified in the following paragraphs by reference to the simplest possible system that may be considered.

Consider the linear structure shown in Fig. 1, which has mass, m, height, h, lateral stiffness, k, and coefficient of viscous damping, c, and is supported through a foundation of mass, m_o, on the surface of a homogeneous, elastic halfspace. The foundation mat is idealized as a rigid disk of negligible thickness which is bonded to the halfspace so that no uplifting or sliding may occur, and the columns of the structure are presumed to be massless and axially inextensible. The supporting medium is characterized by its mass density, ρ, shear wave velocity, v_s, and Poisson's ratio, ν. This structure may be viewed either as the direct model of a single-story building frame, or more generally, as the model of a multistory, multimode structure that responds in its fundamental mode when fixed at the base. In the latter case, h must be interpreted as the distance from the base to the centroid of the inertia forces associated with the fundamental mode of vibration of the fixed-base structure; and m, k, and c must be interpreted as the generalized structural mass, generalized stiffness, and generalized damping coefficient for that mode, respectively.

The system is presumed to be excited by a ground motion which in the absence of the structure and the foundation is a uniform horizontal motion. The displacement of the free-field ground motion at any time, t, is denoted by y(t), and the associated velocity and acceleration are denoted by $\dot{y}(t)$ and $\ddot{y}(t)$, respectively. The maximum values of these quantities will be denoted by y_o, $\dot{y}_o$, and $\ddot{y}_o$.

Under the influence of this excitation, the foundation of the structure will displace horizontally by an amount x(t) which is generally different from y(t), and will rotate by an amount $\theta(t)$. The configuration of the system can then be specified in terms of x(t), $\theta(t)$, and the structural deformation, u(t). For a rigidly supported structure, $\theta(t) = 0$ and $x(t) = y(t)$.

The response of this sytem is governed by a set of three coupled second order differential equations, of which only one is needed for our purposes here. Equilibrium of the forces acting on the top mass leads to

$$m\ddot{u} + c\dot{u} + ku = -m(\ddot{x} + h\ddot{\theta}) \tag{1}$$

in which a dot superscript denotes differentiation with time.

From the similarity of Eq. 1 to that governing the response of a base-excited, single-degree-of-freedom system, it is concluded that the deformation, u, of the actual, interacting system is the same as that of a non-interacting, rigidly-supported system subjected to a base acceleration

$$\ddot{x}_e = \ddot{x} + h\ddot{\theta} \tag{2}$$

The quantity $\ddot{x}_e$ will be referred to as the effective acceleration of the foundation. Note that not only is $\ddot{x}_e$ different from the free-field acceleration, $\ddot{y}$, but it also differs by the term $h\ddot{\theta}$ from the horizontal acceleration of the foundation, $\ddot{x}$. The latter term, which represents the contribution of the rocking action of the foundation, would be expected to be particularly important for tall, slender structures.

The interrelationships between $\ddot{y}$, $\ddot{x}$ and $\ddot{x}_e$ are shown in Fig. 2 for a short, squatty structure with h/r = 1. A relatively simple free-field ground motion is considered, for which the acceleration trace is represented by a sequence of three triangular pulses, as shown by the dashed lines at the top figure. The time scale in these plots is normalized with respect to t_1, the half-duration of the excitation.

In an effort to accentuate the interaction effects, the shear-wave velocity for the halfspace is taken as $v_s = 2h/T$, in which T = the fixed-base natural period of the structure; and the duration of the excitation, $2t_1$, is considered to be such that $ft_1 = 0.6$, in which f = 1/T = the fixed-base natural frequency of the system. The damping factor for the structure in its fixed-base condition, β, is considered to be $\beta = 0.02$; the foundation mass, m_0, is taken as zero; and the mass of the structure, m, is taken as 15 percent of the value it would have if the structure were filled with soil, i.e, $m = 0.15\pi r^2 h\rho$. These parameters are the same as those used in Ref. 12.

As would be expected, the horizontal acceleration of the foundation, $\ddot{x}$, is different from the free-field acceleration, $\ddot{y}$. However, the difference is not particularly great, and the two traces have the same general appearance. It is important to realize, however, that the use of $\ddot{x}$ as the base motion in a rigid-base idealization of the structure will not yield the correct structural deformation, u. To obtain the correct response, the base motion must be taken equal to the effective acceleration, $\ddot{x}_e$, represented by the sum of the solid curve in the upper part of the figure and the curve in the middle diagram. Note that even for this rather short structure, for which the foundation rocking motion would be expected to be minimal, the contribution of that motion to the effective base motion is quite large, and $\ddot{x}_e$ differs dramatically both from $\ddot{y}$ and from $\ddot{x}$.

Different Measures of Interaction

The consequences of soil-structure interaction displayed in Fig. 2 may be expressed by:

1. The difference in the peak values of $\ddot{y}$ and $\ddot{x}$;
2. The difference in the histories of $\ddot{y}$ and $\ddot{x}$;
3. The difference in the peak values of $\ddot{y}$ and $\ddot{x}_e$; or
4. The difference in the histories of $\ddot{y}$ and $\ddot{x}_e$.

Although used extensively in practice to infer interaction effects from records of ground motions obtained during actual earthquakes, the first option is the least satisfactory. There are two reasons for this: First, the peak acceleration of a foundation motion is generally not a good index of the effect of that motion on the response of the structure; and second, the response of the interacting system, as already noted, is governed by the effective acceleration of the foundation motion, $\ddot{x}_e$, rather than merely by the horizontal component of that motion, $\ddot{x}$.

The fourth option is clearly the most desirable, but even this provides only an **indirect** measure of the effects of interaction on the response of the system. It may be noted in passing that, to evaluate $\ddot{x}_e$ from field measurements, it is necessary to have continuous and simultaneous recordings of $\ddot{x}$ and $\ddot{\theta}$. Such records are seldom obtained in field test programs.

Of primary interest in design practice are the consequences of soil-structure interaction on the response of the structure itself, particularly the maximum values of such global quantities as the base shear and base moment. It follows that the interaction effects must be evaluated directly for these response quantities. Clearly, these responses depend on the characteristics of the structure, and different structures would be affected differently by a prescribed change in the effective foundation motion. Furthermore, different response quantities, such as deformations, absolute displacements, or absolute and relative velocities and accelerations, also would be affected differently.

Let RI be the **desired** structural response computed with due regard for soil-structure interaction, and RO be the corresponding response computed by considering the system to be rigidly supported. The most direct and valuable definition of soil-structure interaction is then provided by the difference between RI and RO. Incidentally, this difference cannot be determined directly from field measurements, as it is unlikely to have two identical structures, one supported on rigid ground and the other on flexible soils, with both subjected to precisely the same earthquake ground shaking. Even for the most intelligently planned field test programs, a combination of field test data and analytical studies are required to evaluate the interaction effects for structural response.

<u>Kinematic and Inertial Effects</u>

For the system examined so far, the free-field motions for all points of the ground surface beneath the foundation were presumed to be same. In general, the motions of different points are likely to be different due to differences in the times of arrival of the individual wave trains and/or due to lack of coherence in these waves. As a result, the free-field foundation input motion, FFF, which is defined as the motion that the foundation would experience if both it and the superimposed structure were massless, would be different from the corresponding motion at some reference or control point, denoted herein by the symbol FFC. The FFF would generally include torsional and rocking components of motion in addition to translational components, and depending on the plan dimensions of the foundation and the angle of incidence and/or coherence of the impinging waves, it may be substantially different from the FFC.

For the conditions presumed in the analysis of the system shown in Fig. 1, the FFF is identical to the FFC, and the interaction effects for structural response in this case are represented by the difference of RI(FFC) and RO(FFC). The terms in parentheses in the latter symbols define the input motion for which the response of the system is evaluated. For a system for which the FFF differs from the FFC, the response must naturally be evaluated for the FFF. The interaction effects in this case are given by the difference between RI(FFF) and RO(FFF). These facts also are shown schematically by the entries on the first and second rows of the diagram in Fig. 3.

Provided it is computed for the foundation input motion which is appropriate to the problem under consideration, the difference between RI and RO represents the total or actual interaction effect for the particular response quantity being examined. Such differences, however, are commonly referred to as the inertial interaction effects, and in conformity with this practice, they are identified by the symbols IIC and IIF in Fig. 3. The first symbol stands for "inertial interaction computed for the **control** motion", and the second symbol stands for "inertial interaction computed for the **foundation** input motion".

Notwithstanding the fact the RI(FFF) is controlled by the FFF, this response is frequently compared to that computed for the control motion, and soil-structure interaction is often defined as the difference between RI(FFF) and RO(FFC). This difference is identified in Fig. 3 as SSI, but the appropriateness of this generalized interpretation and of the designation itself is questioned. This is not to suggest that the difference between FFF and FFC is not an important factor that should be considered in the analysis, but rather to emphasize that the response of the structure is defined by the FFF, and that the FFC may be a poor index of this response.

The difference between the FFF and FFC is normally referred to as the kinematic interaction, and it is identified in Fig. 3 by the symbol KI. It is important to note, however, that this difference refers to the input motions and not to the corresponding responses. The difference in the responses computed for the FFF and the FFC without regard for interaction is denoted by KIO, and the difference in the responses computed considering the interaction effects is denoted by KII. For a linear system, the SSI may then also be expressed by the sum of IIF and KIO, or by the sum of IIC and KII.

Considering the variety of interpretations that are possible for the effects of soil-structure interaction, it is important that the nature of the effects considered in a particular study be defined clearly and completely.

Determination of Free-Field Control Motion

Implicit in the material presented so far has been the assumption that the free-field control motion of the ground is known. The determination of this motion is probably the most difficult and uncertain step in analyses of the seismic response of structures.

The characteristics of the free-field control motion depend on such factors as the magnitude of the earthquake; the distance from the earthquake source to the site under consideration; the source mechanism, which refers to the details of the fracturing process and to the orientation and direction of propagation of the break; the characteristics of the travel path, which include the size, orientation and physical properties of the surface and subsurface strata through which the waves must travel to reach the site; and the topography, geology, and local soil conditions of the site.

The soil-structure interaction problem is sometimes defined in a generalized sense to include all, or at least some, of the factors involved in the definition of the free-field control motion. Local site conditions represent the factor most commonly considered in this regard. Such generalized interpretations of soil-structure interaction are believed to be unduly broad; they tend to diffuse the issues involved and to complicate their resolution. A more desirable approach consists in breaking the problem into its component parts and examining each part separately and critically.

EXAMPLES OF SIMPLE PROCEDURES

Two examples are presented to illustrate the type of simple, design-oriented procedures that are believed to be needed. The first concerns the interaction effects for certain classes of building structures, whereas the second deals with the hydrodynamic pressures and forces induced in liquid containing upright cylindrical tanks and with the consequences of soil-structure interaction on the response of such systems.

Interaction Effects for Building Structures

Two different approaches may be used to assess the effects of soil-structure interaction on the response of a ground-excited system (12). The first involves modifying the stipulated free-field ground motion (along the lines indicated in the discussion of Fig. 2) and evaluating the response of the superstructure to the modified base motion; whereas the second involves modifying the dynamic properties of the superstructure and evaluating the response of the modified structure to the prescribed free-field ground motion, considering the structure to be rigidly supported. Although less direct, the second approach is the more convenient of the two for design purposes, as it permits the use of the specified free-field ground motion and of the associated response spectra for rigidly supported systems. This approach is described in the following paragraphs by reference to the simple system shown in Fig. 1.

The interaction effects in this approach (e.g., Ref. 12) may be expressed by an increase in the fixed-base natural period of the structure, and by a change (generally an increase) in the associated damping. The increase in period results from the flexibility of the supporting medium whereas the increase in damping results from the capacity of the medium to dissipate energy by radiation of waves and by hysteretic action.

Modified Period and Damping. Let T be the natural period of the structure in its fixed-base condition, and $\tilde{T}$ be the corresponding period of the modified structure which approximates the actual, flexibly supported system. The two periods are interrelated by the equation (see, for example, Ref. 12):

$$\tilde{T} = T\sqrt{1 + \frac{k}{K_x}\left(1 + \frac{K_x h^2}{K_\theta}\right)} \tag{3}$$

in which K_x = the lateral translational stiffness of the foundation, defined as the horizontal force necessary to displace the foundation by a unit amount; and K_θ = the rocking stiffness of the foundation, defined as the moment necessary to rotate the foundation by a unit amount about a horizontal centroidal axis. For a circular foundation supported at the surface of a homogeneous soil deposit, the translational stiffness is given by

$$K_x = \frac{8}{2 - \nu} G r \tag{4}$$

and the rocking stiffness is given by

$$K_\theta = \frac{8}{3(1 - \nu)} G r^3 \tag{5}$$

in which $G = \rho v_s^2$ = the shear modulus of the soil. Equations 4 and 5 are strictly valid for massless foundations and soils, and make no provision for the effect of the small coupling between translational and rocking actions. Although the effects of the neglected factors can be taken into account in the manner indicated in Ref. 12, this is generally an unnecessary refinement for most building structures.

For mat foundations of arbitrary shape, the quantity r in Eq. 4 must be interpreted as the radius of a disk which has the area of the actual foundation, and in Eq. 5 it must be interpreted as the radius of a disk the static moment of which about a horizontal centroidal axis is equal to that of the actual foundation in the direction in which the response is being evaluated.

It can further be shown (e.g., Ref. 12) that if β represents the percentage of critical damping for the fixed-base structure, and $\tilde{\beta}$ represents the corresponding damping of the modified structure that approximates the interacting system, the two quantities are interrelated by

$$\tilde{\beta} = \beta_o + \frac{\beta}{(\tilde{T}/T)^3} \tag{6}$$

in which β_o represents the contribution of the foundation damping, including radiation and soil material damping. Note that β_o and β are not directly additive, and that the effectiveness of the structural damping is reduced by soil-structure interaction, the reduction being substantial when $\tilde{T}/T$ is large. In fact, unless the reduced contribution of structural damping is compensated by the foundation damping, the overall damping of the interacting system will be less than that of the rigidly supported structure.

The three most important parameters that affect the value of β_o are: the period ratio $\tilde{T}/T$, which is a measure of the relative flexibilites of the foundation medium and structure; the ratio h/r of the height of the structure to the radius of the foundation; and the hysteretic capacity of the soil itself. The latter capacity is defined by the factor

$$\tan\delta = \frac{1}{2\pi}\frac{\Delta W_s}{W_s} \tag{7}$$

in which ΔW_s represents the area of the hysteresis loop in the stress-strain diagram for a soil specimen undergoing harmonic shearing deformation, and W_s represents the strain energy stored in a linearly elastic material subjected to the same maximum values of stress and strain (i.e., the area of the triangle in the stress-strain diagram between the origin and the point of the maximum induced stress and strain). This factor depends on the magnitude of the imposed peak strain, increasing with increasing intensity of excitation or level of straining.

The variation of β_o with $\tilde{T}/T$ is shown in Fig. 4 for two values of $\tan\delta$. The dashed lines, which refer to systems supported on a purely elastic medium, represent the effect of radiation damping only, whereas the solid lines, which refer to a viscoelastic medium with $\tan\delta = 0.10$, represent the combined effects of radiation and hysteretic soil action. It can be seen that the contribution of the foundation damping may be quite substantial, and that the component due to hysteretic soil action may be particularly significant for tall structures for which the radiational effects are generally quite small.

The particular data presented in Fig. 4 are for systems with negligible foundation mass and a structural mass equal to 15 percent of the mass of the structure when filled with soil. The latter value is representative of that for building structures. Similar results have been obtained for other mass ratios as well (12).

Summary of Procedure. With the information that has been presented, the analysis of the interacting system may be implemented as follows:

1. Evaluate the fixed-base natural period of the structure, T.
2. By application of Eq. 3 and Eqs. 4 and 5, evaluate the modified natural period, $\tilde{T}$.
3. Estimate the structural damping factor, β, and the soil material damping factor, $\tan\delta$, which would be appropriate for the anticipated strains in the supporting medium, and by application of Eq. 6 and of the data presented in Fig. 4 and/or Ref. 12, determine the effective damping factor, $\tilde{\beta}$.
4. From the response spectrum for single-degree-of-freedom systems subjected to the stipulated free-field ground motion, evaluate the pseudo-acceleration, A, corresponding to $\tilde{T}$ and $\tilde{\beta}$.
5. The maximum value of the base shear for the interacting system, $\tilde{Q}$, may then be determined from

$$\tilde{Q} = m\tilde{A} \tag{8}$$

and the maximum displacement of the structure relative to the ground, Δ_m, may be determined from

$$\Delta_m = \frac{\tilde{Q}}{k} + \frac{\tilde{Q}h^2}{K_\theta} = u_m\left(1 + \frac{kh^2}{K_\theta}\right) \tag{9}$$

The second term on the right side of the last expression represents the contribution of the foundation rotation.

Consequences of Interaction. A recurring question is whether soil-structure interaction may increase or decrease the maximum response of a structure. The answer is a function of the response quantity under examination, and of the characteristics of the ground motion and the system itself. More specifically, soil-structure interaction may increase, decrease, or have no effect on the peak response of a system depending on the characteristics of the relevant response spectrum, and the regions of the spectrum to which the fundamental natural periods of the fixed-base and the interacting systems fall.

The various possibilities are illustrated in the following paragraphs by reference to the base shear induced by a free-field ground motion, the pseudo-acceleration spectrum of which is represented by the piecewise linear diagram shown in Fig. 5. Note that the response spectrum is displayed on logarithmic scales.

1. If both T and $\tilde{T}$ fall in the extremely small period range of the

spectrum (to the left of point b), then soil-structure interaction will have no effect on the response, as the pseudo-acceleration value in this case is unaffected by changes in either period or damping.

2. If T falls to the right of point c, soil-structure interaction will reduce the maximum response, the magnitude of the reduction being a function of the values of T, $\tilde{T}$, β and $\tilde{\beta}$. An increase in damping under these conditions decreases the pseudo-acceleration, whereas an increase in period either does not change or further decreases the pseudo-acceleration.

3. If T falls in the intermediate period range (between points b and c in Fig. 5), or if T falls to the left of point b and $\tilde{T}$ falls to the right of this point, soil-structure interaction may increase or decrease the response depending on the values of $\tilde{T}/T$ and $\tilde{\beta}/\beta$. An increase in period in this case increases the response, whereas an increase in damping has the opposite effect.

Generalization of Procedure and its Use in Seismic Design Provisions. For building structures that must be analyzed as multi-degree-of-freedom systems in their fixed-base condition, a reasonable approximation to the maximum response of a structure may be obtained by assuming that soil-structure interaction affects only the response component contributed by the fundamental mode of vibration. This component may be evaluated by the procedure outlined using the generalized interpretations for h and m noted previously. The contributions of the higher modes may then be computed disregarding the interaction effects. The rationale for this approach is explained in Ref. 12.

The concepts summarized herein have provided the basis of the design provisions for soil-structure interaction for building structures that have been recommended by the Applied Technology Council (1) and have been adopted recently by the Building Seismic Safety Council (3) in connection with the National Earthquake Hazard Reduction Program (NEHRP).

The pseudo-acceleration response spectrum for fixed-base systems in these provisions is a non-increasing function of the fundamental natural period of the system. As a result, consideration of soil-structure interaction reduces the design values of the lateral forces, shears and overturning moments below the levels applicable to a rigid-base condition, and it is conservative in this case to neglect the interaction effects. Because of the influence of foundation rocking, however, the horizontal displacement of the structure relative to the moving base may increase due to interaction, and this will, in turn, increase both the required spacing between buildings and the secondary design forces associated with the P-Δ effects. The latter increases are generally small and have only a minor influence on the final design.

In the form presented here, the simplified design procedure for assessing the consequences of soil-structure interaction is applicable to structures that are supported through mat foundations at the surface of a homogeneous soil deposit. With suitable adjustments, however, the procedure can also be applied to mat foundations that are embedded or supported on layered media, and to structures that are supported on spread footings or piles. Some of these extensions are considered in Refs. 1, 3 and 12.

The ATC-NEHRP design provisions for soil-structure interaction are definitely limited to building structures. However, the concepts underlying these provisions are of much broader applicability and can be, and have been, used in preliminary designs of a variety of structures, including nuclear power plants. As a matter of fact, only for unusual structures of extreme importance, and only when the simple procedure summarized herein indicates that the interaction effects are indeed of definite consequence in design, would the use of more refined and elaborate analyses be justified.

Hydrodynamic Effects in Liquid Containing Tanks

A ground-supported, upright, circular cylindrical tank of radius, a, height, H, and uniform wall thickness, h, is considered. The tank is presumed to be filled with liquid, anchored at the base, and be excited by a horizontal component of ground shaking. The liquid is assumed to be incompressible, inviscid and free at its upper surface, and only linear effects are examined.

In early studies of this problem (e.g., Refs. 6,7,8), the tank was considered to be rigid. These studies revealed that the resulting hydrodynamic pressures and tank forces can conveniently be expressed as the sum of two components: (1) The so-called impulsive component, which represents the effects of that part of the contained liquid which may be considered to move in synchronism with the tank wall as a rigidly attached mass; and (2) the so-called convective component, which represents the effects of the remaining liquid which undergoes sloshing or rocking motion about a horizontal axis normal to the direction of excitation. In mathematical terms, the impulsive component represents the solution of the differential equation for the oscillating liquid, obtained by satisfying the actual boundary conditions along the tank wall and the base and the condition of zero hydrodynamic pressure at the free liquid surface. Similarly, the convective component represents a corrective solution which accounts for the difference between the true hydrodynamic pressure at the still free liquid surface and that considered in the development of the impulsive solution. For tanks of proportions most frequently encountered in practice, the impulsive effects represent the dominant response component.

Effects of Tank Flexibility. Subsequent studies of the problem (e.g., Refs. 4,11,13,17) revealed that the impulsive component of the response may be influenced significantly by the flexibility of the tank wall, and that the peak values of the hydrodynamic pressures and associated forces for flexible tanks may well be two or three times as large as those induced by the same ground shaking in rigid tanks of the same dimensions.

In recognition of this fact, the latest draft of the Nuclear Regulatory Commission's Standard Review plan for above-ground tanks considers it "unacceptable to assume the tank to be rigid unless the assumption can be justified", and further states that "a minimum acceptable analysis must incorporate at least two horizontal modes of combined fluid-tank vibration" (the fundamental lateral natural mode of the tank-liquid system and the fundamental mode of the contained liquid).

Inasmuch as the seismic resistances of most of the tanks now in service were evaluated by the procedure described in Ref. 10, which considers the tank to be rigid, the adequacy of these designs would have to be demonstrated in light of the improved understanding of the problem now available.

Effects of Soil-Structure Interaction. The studies which provided the basis for the recent NRC design provisions referred to rigidly supported tanks, for which the base motion may be considered to be the same as the free-field ground motion, and did not account for the feedback or interaction effects between the tank-liquid system and supporting medium. Although the effects of soil-structure interaction for liquid containing tanks have been the subject of several exploratory recent studies (2,5,9,13,15,16), additional research is needed before these effects can be quantified and provided for reliably and cost-effectively in design.

The most comprehensive of the studies conducted for far are those reported in Refs. 15 and 16. The tanks in these studies were presumed to be supported through circular rigid mat foundations at the surface of a homogeneous, elastic halfspace, and to be excited by a **vertical** component of ground shaking.

These studies have shown that, as for the building systems examined earlier, the effects of soil-structure interaction can be evaluated with reasonable accuracy from a fixed-base analysis, by modifying the fundamental natural period of the system and the associated damping. Let f be the fundamental natural frequency of the system in an axisymmetric, breathing mode of vibration with the tank-liquid system considered to be rigidly supported at the base, and let $\tilde{f}$ be the corresponding frequency of the modified system which incorporates the effects of the foundation flexibility. Similarly, let β_0 be the foundation damping of the system, in percent of critical damping.

In Fig. 6 are given the values of $\tilde{f}/f$ and β_0, computed for a group of steel tanks filled with water. Reproduced from Ref. 15, these data are plotted as a function of the height-to-radius ratio, H/a, for two values of thickness-to-radius ratio, h/a, and several different values of shear-wave velocity for the supporting medium, v_s. Note that the foundation damping factor may be quite large for soft soils and stiff tanks (tanks with large values of h/a), particularly when H/a is close to unity. Large increases in system damping, with small changes in natural frequency, would naturally be expected to lead to significant reductions in response.

As an indication of the magnitude of the reduction that may result, it is noted that for a steel tank with H/a = 1 and h/a = 0.001, supported on a soil deposit with v_s = 700 ft/sec, the peak values of the hydrodynamic effects computed considering the effects of soil-structure interaction are about one-half of those computed without regard for such interaction, and approximately 30 percent in excess of those computed considering both the tank wall and the foundation soils to be rigid (15). The damping value of the tank-liquid system in its fixed-base condition was taken as 2 percent of critical damping in these solutions.

Based on the results of soil-structure studies for building structures, similar, though somewhat smaller, reductions are expected for laterally excited tanks as well. Unless this estimate proves erroneous (which is deemed unlikely), it should be clear that analyses which take due account of the flexibility of the tank wall but neglect the effects of soil-structure interaction may lead to overly conservative designs, or may suggest the need for modifications in existing designs that are unwarranted. Clearly, this is a matter for further research. It should also be noted that, because of the importance of base rocking for laterally excited tanks supported on soft soils, soil-structure interaction may increase the surface sloshing motion of the contained liquid and hence the freeboard that must be provided to prevent the liquid from impacting the roof. This is also a matter for further study.

RESEARCH NEEDS

A brief summary is given in this section of the research that is believed to be needed to improve the reliability of soil-structure interaction analyses for structures subjected to earthquakes.

1. There is a need to define with greater precision than is now possible the seismic environment to which a structure in a given site is likely to be subjected. This requires improved insight into the factors that control the composition, angle of incidence, and coherence of the seismic waves, and improved understanding of the effects of the source mechanism and travel path characteristics. The satisfactory resolution of these issues will require the coordinated efforts of engineers and seismologists, and it is hoped that workshops such as this, which provide the opportunity for improved appreciation of the viewpoints, requirements and capabilities of each group, may be held more frequently in the years ahead. Of special value in such studies are field data from instrumental arrays concerning the spatial variation of high-intensity earthquake ground motions.

2. The second need concerns the characterization of the properties of soil deposits. There is a need to clarify the interrelationship of data obtained from laboratory and field tests, and to develop improved nonlinear constitutive models for soils under states of stress representative of those encountered in practice.

3. Existing methods and computer programs for evaluating the dynamic response of soil-structure systems should be expanded to accomodate the improved characterizations of the seismic environment and soil properties referred to above. In addition, further efforts should be made to improve the efficiency and cost-effectiveness of existing programs.

4. Comprehensive parametric studies are needed to assess the effects and relative importance of the numerous factors that characterize the structure-soil system, and to evaluate the sensitivity of its response to the uncertainties that are inherent in the definition of these factors. The results of these studies should then be used to develop simplified methods of analysis and design. Of special importance in this regard are techniques for defining the free-field foundation input motion and assessing the importance of the kinematic interaction effects. These studies may be carried out with existing methods of analysis and computer programs, and need not await the development of the expanded capabilities referred to under item 3.

5. Finally, there is a need for carefully planned and properly executed experimental programs to test the adequacy of the analytical preditions and to guide future analytical developments.

CONCLUDING REMARKS

Emphasis in this presentation has been placed on the role and value of simple, approximate methods in studies of the dynamics of soil-structure interaction. There has been no intent, however, to deemphasize the utility of more sophisticated and complex methods, or to suggest that such methods are not important or necessary; they are, of course, both important and necessary. Rather, the intent has been to indicate how such methods can be used to the greatest possible advantage in engineering practice and research.

The more elaborate, refined methods are certainly needed in design studies for final verification purposes, but above all, for the development of the simple approximate methods. And lest the reader may think that simple approaches are necessarily unsophisticated, it may be noted that simplicity which is founded on rationality is indeed the ultimate sophistication in analysis.

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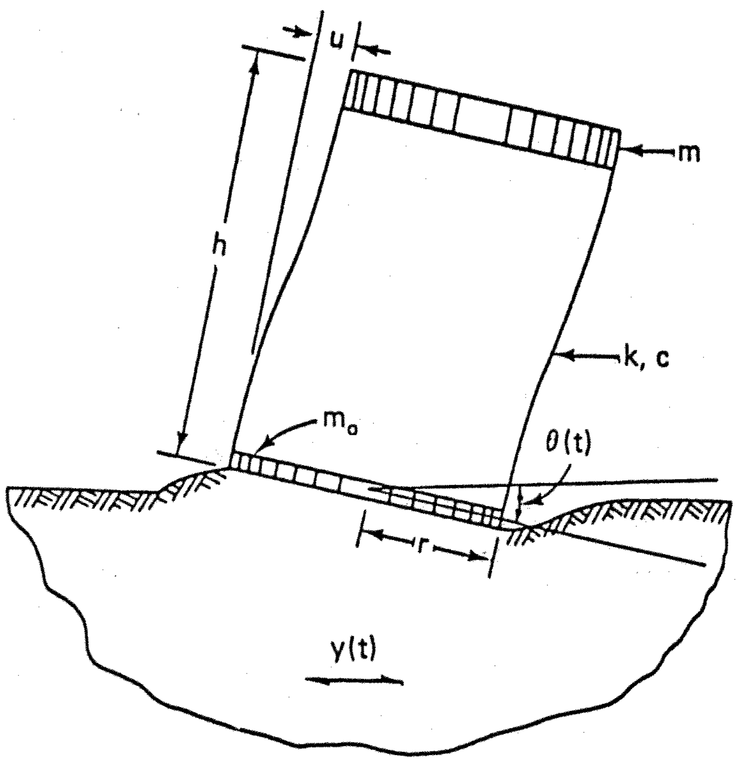

FIG. 1 Simple System Considered

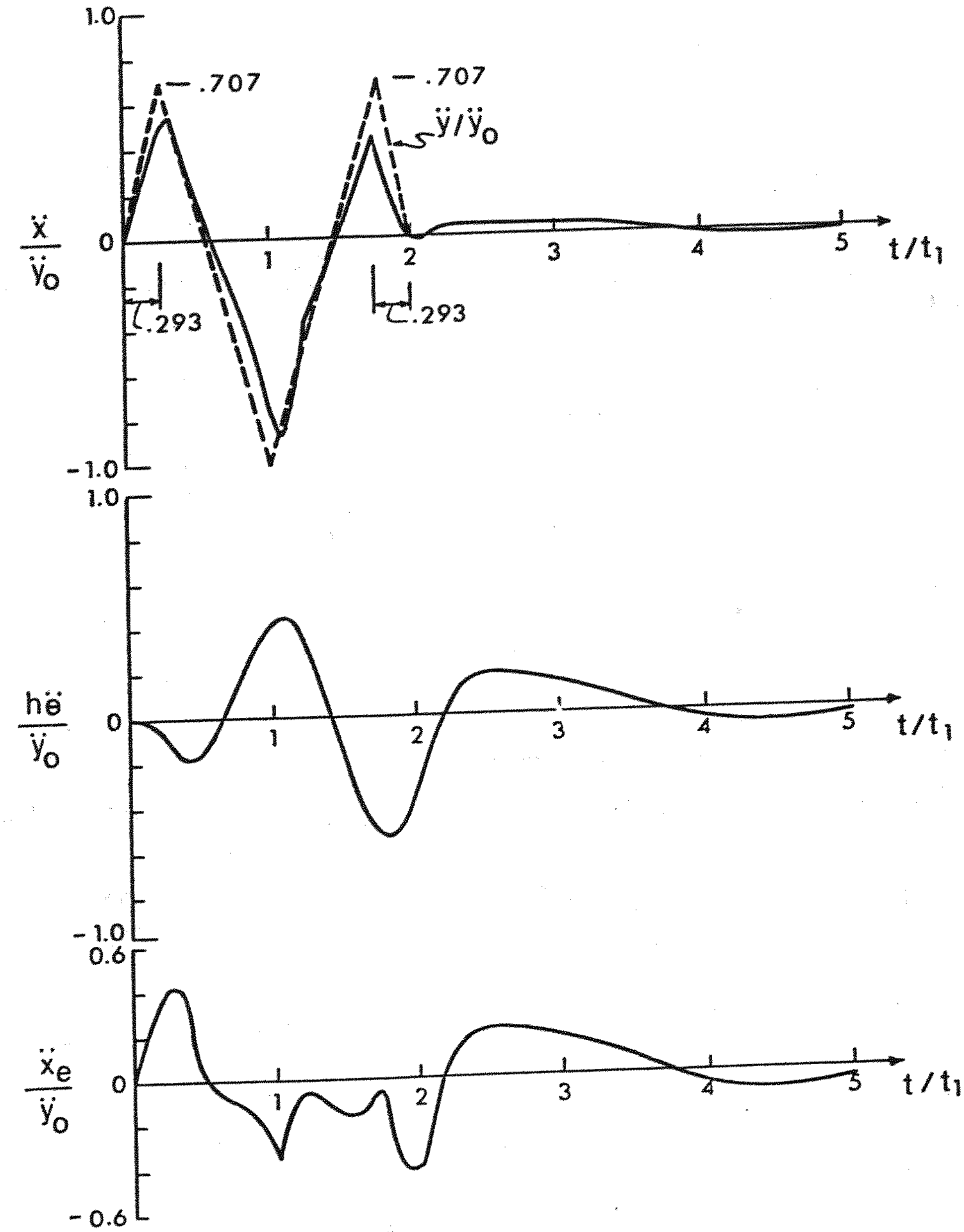

FIG. 2 Base Acceleration for a Squatty Structure with h/r = 1

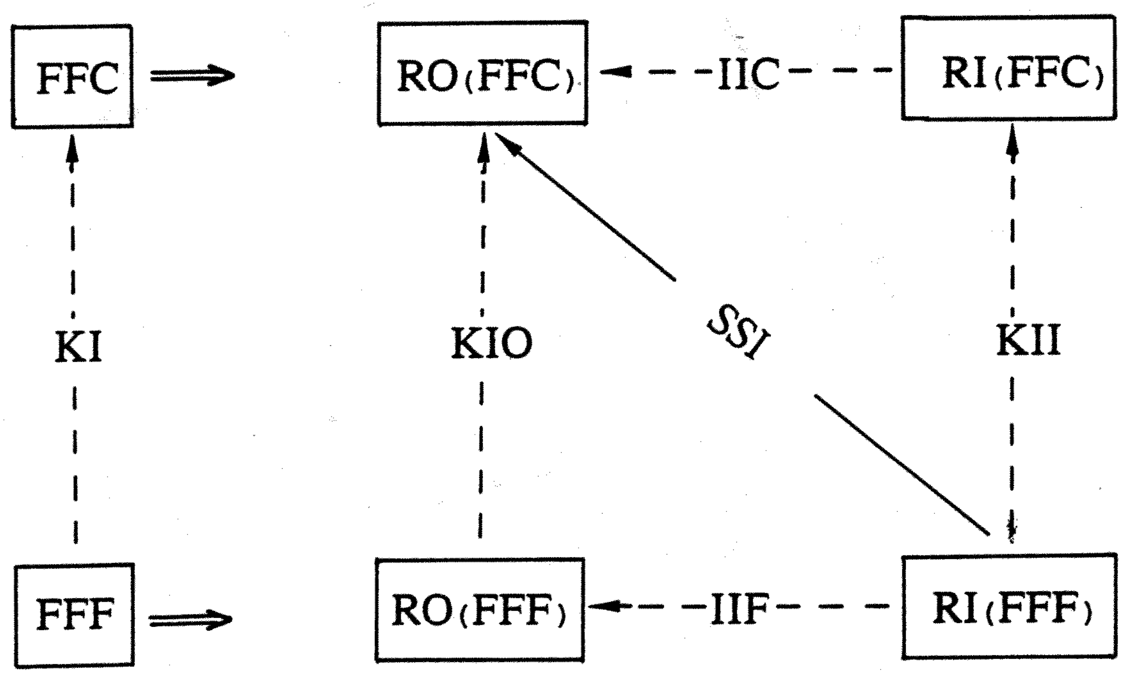

FIG. 3 Possible Definitions of Soil-Structure Interaction

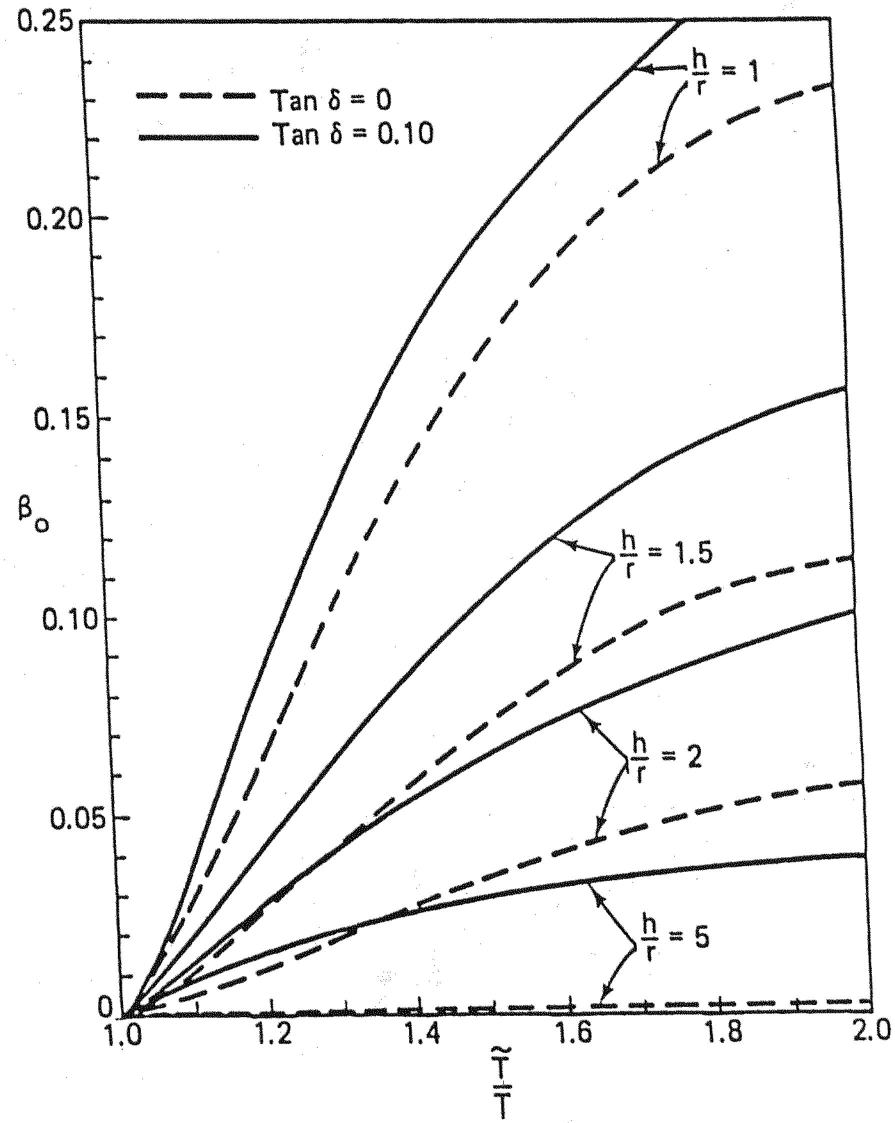

FIG. 4 Foundation Damping Factor, β_o, for Simple Building Systems

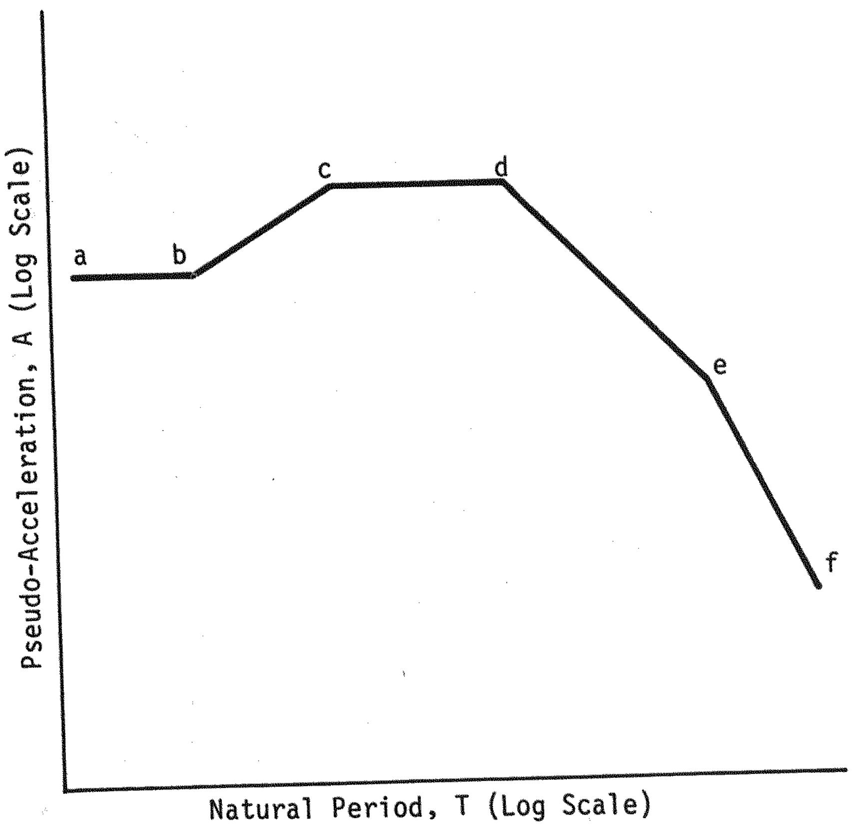

FIG. 5 Representative Pseudo-Acceleration Design Spectrum

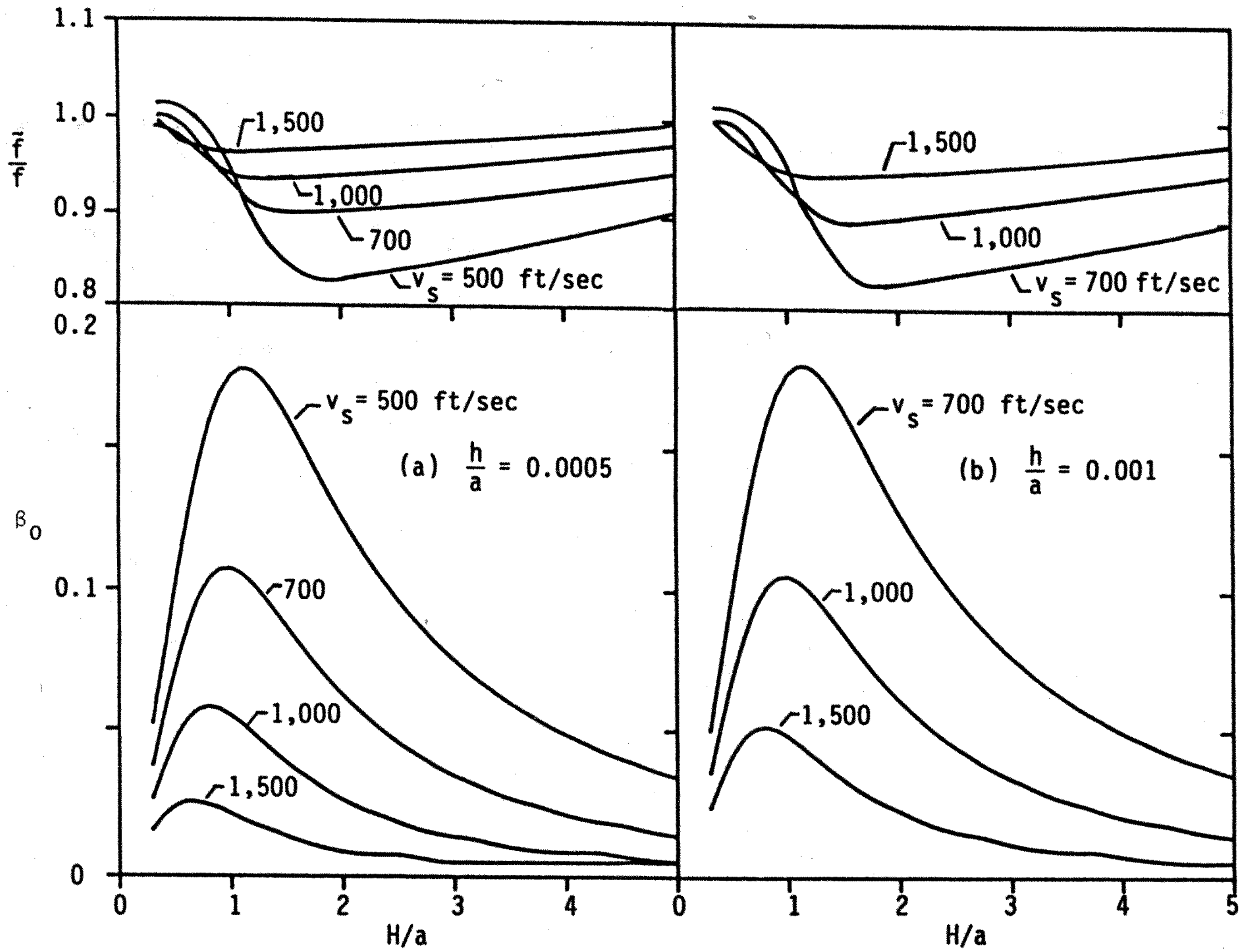

FIG. 6 Effective Natural Frequency and Foundation Damping Factor for Elastically Supported Steel Tanks Filled with Water

H. Bolton Seed and John Lysmer

Site Response--A Critical Problem in Soil-Structure Interaction Analyses for Embedded Structures

ABSTRACT Soil-structure interaction analyses for embedded structures must necessarily be based on a knowledge of the manner in which the soil would behave in the absence of any structure--that is on a knowledge and understanding of the spatial distribution of motions in the ground within the depth of embedment of the structure. The nature of these spatial variations is discussed and illustrated by examples of recorded motions. It is shown that both the amplitude of peak acceleration and the form of the acceleration response spectrum for earthquake motions will necessarily vary with depth and failure to take these variations into account may introduce an unwarranted degree of conservatism into the soil-structure interaction analysis procedure.

GENERAL CONSIDERATIONS

The basic problem of soil-structure interaction is illustrated in Fig. 1. It involves the determination of the motions of one or more structures at a given site from a knowledge of a given motion (the control motion) at a specified point (the control point) of the site prior to construction (the free field).

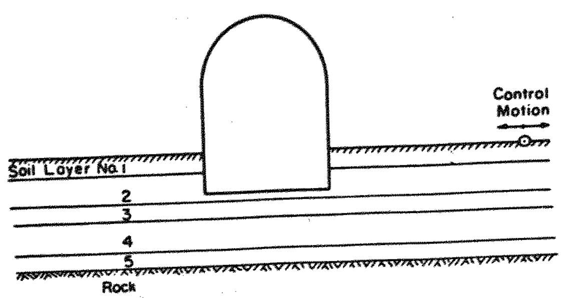

FIG. 1 SOIL STRUCTURE INTERACTION PROBLEM

A complete soil-structure interaction analysis for any structure must necessarily consist of two distinct parts; a site response analysis and an interaction analysis. Unless the nature of the seismic wave field into which the structure is being placed is known with a reasonable degree of accuracy, there is no way in which the resulting interaction of the structure with the soil deposit and the wave field can be determined.

The site response analysis involves the determination of the temporal and spatial variations of the free-field motions. The interaction analysis involves the determination of the motions of a structure placed in the above seismic environment. These are different types of problems but each needs to be addressed to determine a solution to the soil-structure interaction problem.

Each of the above problem types can in principle be formulated in terms of continuum models or discretized models, and it is not possible here to describe all of the possible forms of equations of motion which have been proposed. It is, however, useful for a better understanding of the nature of and the connection between the two problem types to consider the equations of motion for the three linear models shown in Fig. 2. The models are identical in the sense that all are of the finite element type and all are spanned by the same finite element mesh. Also, all masses and stiffnesses are the same, except that the structural part of the model shown in Fig. 2(b) has no stiffness and mass, and that for this model the structural nodes above ground level are assumed to be fixed in space (actually these points can be given any specified motion without loss of generality).

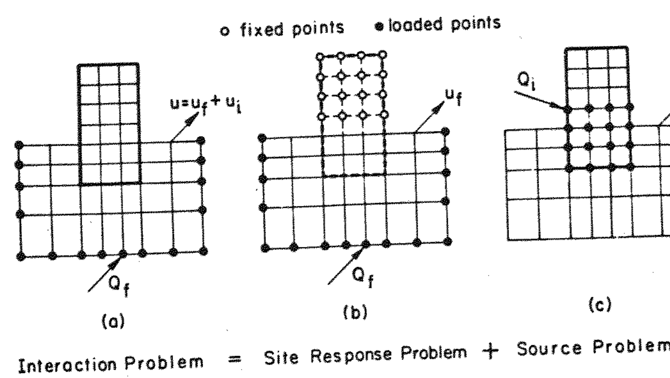

FIG. 2 SUPERPOSITION THEOREM FOR INTERACTION PROBLEMS

Since the fixed nodes have no influence on the motion of the ground, Fig. 2(b) represents a free-field site response problem. It has the equation of motion

$$[M_f]\{\ddot{u}_f\} + [C_f]\{\dot{u}_f\} + [K_f]\{u_f\} = \{Q_f\} \tag{1}$$

where $[M_f]$, $[C_f]$, $[K_f]$ are the mass, damping, and stiffness matrices, respectively, for the free field, and $\{u_f\}$ is a vector containing the nodal point displacements. Since the source of excitation is outside the model the load vector $\{Q_f\}$ has non-zero elements on the external boundary only. Solutions to the equation of motion, Eq. (1), can be obtained by standard methods, see Desai and Christian (1976).

It will here be assumed that a free-field solution is available. Thus $\{u_f\}$ and $\{Q_f\}$ are known.

Figure 2(a) represents the corresponding interaction problem. The total displacements can be written

$$\{u\} = \{u_f\} + \{u_i\} \tag{2}$$

where $\{u_f\}$ are the known free-field displacements and $\{u_i\}$ are the interaction displacements. Assuming that the external boundary is very far away from the structure the equation of motion for the interaction problem is

$$[M]\{\ddot{u}\} + [C]\{\dot{u}\} + [K]\{u\} = \{Q_f\} \tag{3}$$

where $\{Q_f\}$ is the same load vector as in Eq. (1) and [M], [C], and [K] are the total mass, damping, and stiffness matrices, respectively. Substitution of Eqs. (1) and (2) into Eq. (3) yields

$$[M]\{\ddot{u}_i\} + [C]\{\dot{u}_i\} + [K]\{u_i\} = \{Q_i\} \tag{4}$$

where

$$\{Q_i\} = ([M_f] - [M])\{\ddot{u}_f\} + ([C_f] - [C])\{\dot{u}_f\} + ([K_f] - [K])\{u_f\} \tag{5}$$

The load vector, $\{Q_i\}$, in Eq. (5), can be computed from the known free-field displacements. It depends only on the difference in properties between the structure and the excavated soil. Thus $\{Q_i\}$ has non-zero elements only at the structure and Eq. (4) is the equation of motion for the source problem illustrated by Fig. 2(c). This problem is well-posed and can be solved for the interaction displacements, $\{u_i\}$. The total displacements for the soil-structure interaction problem can be found by superposition as indicated by Eq. (2).

Equations (2) and (4) remain valid even as the distance to the boundary goes to infinity and the mesh size shrinks to infinitesimal dimensions. Hence, the above formulation can be extended to continuum mechanics and three dimensions.

The above formulations reveal three important characteristics of the soil-structure interaction phenomenon:

1. The only free field ground motions which are of importance for the interaction phenomena are those within the volume to be excavated for the embedded part of the structure.

2. For an embedded structure the amount of interaction depends on the difference in mass and stiffness between the structure and the volume of excavated soil, see Eqs. (4) and (5).

3. Soil-structure interaction analysis implies in many cases the use of superposition, see Eq. (2). Thus true nonlinear analyses may, for many types of motion specification, not be possible.

The first observation has far reaching consequences; especially for embedded structures on relatively soft sites since, for such sites, both theory and observation indicate that the free-field motions vary significantly with depth. This implies that the site response analysis is an important, and in the opinion of the writers perhaps the most important, part of a soil-structure interaction analysis. The significance of site response with regard to the design of nuclear facilities is discussed in the following pages.

THE SITE RESPONSE PROBLEM

Site response problems involve the determination of the temporal and spatial variation of motions within a site. In principle, these motions can be determined from a large model which includes the source of the earthquake. However, in practice the source parameters and the regional geology cannot be determined in sufficient detail to solve this problem with a high degree of accuracy in the frequency range of interest for design. Thus, current methods of site response analysis normally attempt to predict the above variation of motions from a single specified control motion at some control point within the site. This problem is mathematically ill-posed and unique solutions can be obtained only by the introduction of restrictive assumptions regarding the geometry of the site and the nature of the wave field causing the control motion. In practice, consistent solutions can be obtained only for horizontally layered sites. Possible wave patterns include: vertically propagating or inclined plane body waves, and horizontally propagating surface waves. Only the case of vertically propagating waves can currently be solved by truly nonlinear methods.

CONTROL MOTION AND CONTROL POINT

The inherent problem in site response analysis is the choice of wave field to be used in the analysis and it is therefore natural to classify and discuss the different available methods according to the type of wave field assumed. However, before doing so it should be mentioned that the choice of an appropriate control motion and control point is just as, and in many cases, more important than the choice of wave field.

The control motion should be chosen with due respect to observed relations between earthquake magnitude, epicentral distance, maximum acceleration, duration, frequency content, see Idriss (1978) and Ref. 1, and the site-dependent characteristics established by Hayashi et al. (1971), Seed et al. (1976a, 1976b), Faccioli (1978), and Ref. 1.

Except for the obvious case in which the control motion is an observed record at the control point, the preferable control point is a point either at the ground surface or, as discussed below, at an assumed rock outcrop in the vicinity of the site. This is so because most of our data base of strong motion earthquake records from which the control motion has to be estimated was obtained at surface stations and, even more important, because the frequency content of motions at points below the ground surface is strongly influenced by reflections at the free surface. Thus the specification that the control motion at depth should be a broad-band spectrum or a motion recorded at another site or depth may result in completely unrealistic computed motions for the site.

With the control motion, the control point and the site properties fixed, the solution to the site response problem depends entirely on the nature of the wave field producing the ground surface motions. This wave field may consist of many components including:

(1) some Rayleigh waves
(2) some Love waves
(3) some plane vertically propagating waves
(4) some plane body waves inclined at an angle to the vertical
and (5) some other wave types such as spherical and cylindrical waves which are usually not considered.

At the present time seismologists often cannot advise engineers in sufficient detail on the relative contents of the different possible wave forms which make up the surface motion. Thus, even though soil-structure interaction analyses can be performed for an arbitrary wave field, in practice, such analyses cannot always be made due to lack of data on the characteristics of the wave field involved.

Under these conditions, a typical engineering approach is to make analyses for extreme cases of possible wave fields, i.e., for a motion represented by all Rayleigh waves or for a motion represented entirely by a system of body waves, and to determine the influence of the motion specification on the results of the analysis. If the differences are small, then precise specification of the components of the wave field is considered unnecessary. If the differences are large, then increasing efforts must be made to determine even a crude assessment of the relative components of different wave types, or alternatively, conservative choices of wave components may have to be made for different parts of the analysis. It is important therefore to examine the characteristics of the different wave forms which might contribute to the surface control motion.

HORIZONTALLY PROPAGATING WAVES

For horizontally layered sites it is relatively easy to set up linear methods of analysis for horizontally propagating waves. However, many possible choices of wave patterns exist (inclined body waves at different angles of incidence, different modes of surface waves, etc.) and it is currently impossible to determine from available seismological data the exact contributions of each wave type to earthquake motions near the surface in the frequency range of interest to earthquake engineers. However, some estimates have been made (Trifunac and Brune, 1970; Randall, 1971; Chandra, 1972; Nair and Emery, 1975; Liang and Duke, 1977; and Toki, 1977). These estimates involve considerable uncertainties however; in view of these uncertainties analyses of site response for motions represented only by horizontally propagating waves are mainly of scientific interest and represent an extreme bound of the probable motions. Nevertheless, as will be discussed below, some practical conclusions can be drawn from such analyses.

The free-field motions caused by horizontally propagating waves will be discussed in three parts: Surface waves are discussed immediately below, inclined body waves are discussed after the section on vertically propagating waves and, finally, the three wave types are discussed together in a section on motions at shallow depths.

Surface Waves

Rayleigh waves in a perfect elastic half-space are well-known and the theory for these are given in standard textbooks, e.g., Richart et al., (1970). However, for obvious reasons soil dynamics analysts are much more interested in surface waves in multi-layered systems, Thomson (1950), Haskell (1953), Ewing et al. (1957). Two types of waves may occur in such systems: Love waves, in which the motions are horizontal and perpendicular to the direction, x, of wave propagation, and Rayleigh waves which involve both vertical and horizontal motions in the vertical xz-plane. For plane harmonic waves the displacement fields are of the form:

$$\text{Love waves:} \quad u_y = \sum_{s=1}^{\infty} L_s \cdot h_s(z) \cdot e^{i(\omega t - c_s x)} \tag{6}$$

$$\text{Rayleigh waves:} \quad \left\{ \begin{aligned} u_x &= \sum_{s=1}^{\infty} R_s \cdot f_s(z) \cdot e^{i(\omega t - k_s x)} \\ u_z &= \sum_{s=1}^{\infty} R_s \cdot g_s(z) \cdot e^{i(\omega t - k_s x)} \end{aligned} \right\} \tag{7}$$

where ω and t are the frequency and time, respectively, and L_s and R_s are unknown mode participation factors. The infinite sets of wave numbers, c_s and k_s, and mode shapes, $h_s(z)$, $f_s(z)$, and $g_s(z)$, may in principle be determined by methods developed by Thomson (1950) and Haskell (1953). The wave numbers are directly related to the phase velocities of the different wave modes through $V_L = \omega/c$ and $V_R = \omega/k$. Thus the fundamental problem of site response analysis with surface waves is to determine the infinite set of factors L_s and R_s from a single given amplitude at the control point. This is clearly an ill-posed problem and solutions can only be obtained by further assumptions, the most common of which is to assume that only the fundamental Rayleigh or Love mode, corresponding to s = 1, exists. For undamped systems the frequency-dependent phase velocity and mode shape can be found by the Thomson-Haskell method and the amplitude L_1 or R_1 may be determined from the control motion.

Continuum analyses are possible for the case of viscoelastic layers over an undamped half-space (Ewing et al. (1957), Boncheva (1977)). However, for this case it is more practical to first discretize the semi-finite system by the use of finite elements as proposed by Lysmer (1970) and Waas (1972) for Rayleigh waves and Waas (1972, Lysmer and Waas (1972) for Love waves. Only Rayleigh waves will be discussed here. The theoretical model is shown in Fig. 3. It involves the assumption of a linear variation of displacements between layer interfaces and the existence of a stationary rigid base at some finite depth. If this depth is chosen to be considerably larger than the wave length of the Rayleigh waves of interest, a half-space is simulated by this model.

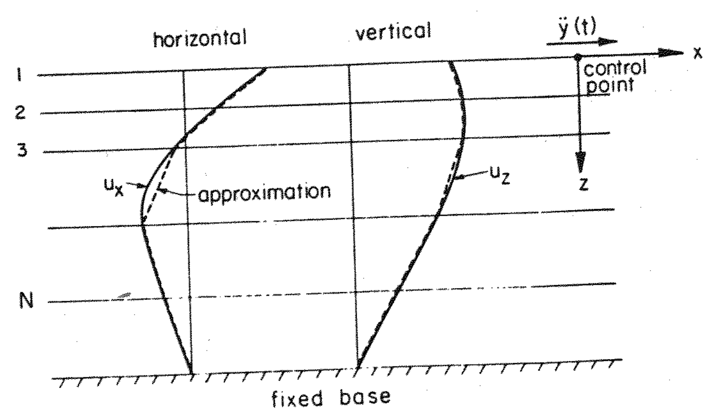

FIG. 3 TYPICAL SOIL PROFILE AND RAYLEIGH WAVE MODE SHAPE

For an N-layer system these assumptions reduce the equation of motion for the layered system to a quadratic eigenvalue problem:

$$([A]k^2 + [B]k + [C] - \omega^2[M])\{v\} = 0 \tag{8}$$

where [A], [B], [C] and [M] are simple 2N x 2N matrices which can be formed from the stiffnesses, damping ratios and mass densities of the layered system, and $\{v\}$ is an eigenvector (mode shape) which contains the 2N displacement amplitudes of the layer interfaces. The mode shape represents the functions $f_s(z)$ and $g_s(z)$ in Eq. (7). For a given frequency, ω, Eq. (8) can be solved by methods developed by Waas (1972). The solution consists of 2N possible wave numbers, k_s, and associated mode shapes $\{v\}_s$ and, in analogy with Eq. (7), the general solution to the equation of motion may be expressed in the form:

$$\{u\} = \sum_{s=1}^{2N} R_s\{v\}_s \cdot e^{i(\omega t - k_s x)} \tag{9}$$

For a damped system all the wave numbers will be complex with negative imaginary parts. Hence Eq. (9) can also be written

$$\{u\} = \sum_{s=1}^{2N} e^{im(k_s)} \cdot R_s\{v\}_s \cdot e^{i(\omega t - Re(k_s)x)} \tag{10}$$

which represents a system of generalized Rayleigh Waves (modes) which propagate in the positive x-direction, each with its own mode shape, $\{v\}_s$, phase velocity, $\omega/Re(k_s)$, and decay factor, exp $[2\pi Im(k_s)/Re(k_s)]$, per wave length, $\lambda = 2\pi/Re(k_s)$. Experience with the method has shown that most of the Rayleigh modes decay extremely rapidly in the frequency range of interest to earthquake engineers and only a few terms of Eq. (9) need therefore be considered. If it is assumed that only the fundamental mode (defined as the mode with the largest value of $Re(k_s)$) is present, Eq. (9) reduces to

$$\{u\} = R_1 \cdot \{v\}_1 e^{i(\omega t - k_1 x)} \tag{11}$$

and the mode participation factor, R_1, can be determined at each frequency from the amplitude of the control motion at say the surface at $x = 0$. This method has been used by Chen and Lysmer (1979) to determine possible Rayleigh motion fields for several sites.

High-order Surface Waves

The fundamental Rayleigh mode defined above is, as shown by Lysmer (1970), identical to the fundamental mode considered by seismologists in layered systems overlying a deformable half-space. The rest of the terms of Eq. (9) represent higher-order Rayleigh modes (in the terminology of seismologists) and body waves. These modes will have longer wavelengths and will propagate faster than the fundamental mode which by definition has the shortest wavelength and thus the lowest phase velocity. While most of these higher modes can be neglected, since they decay rapidly in the direction of wave propagation, others may decay less rapidly than the fundamental mode. This phenomenon occurs only at relatively high frequencies on sites with a marked increase in stiffness with depth; say a sand profile over rock. These low-decay modes could conceivably contribute significantly to the motion at a surface control point. However, studies by Chen and Lysmer (1979) have shown that such modes, when they occur, are associated with energy propagation in deeper high-velocity layers and that they cause near surface motions which are similar to those caused by vertical or slightly inclined body waves. That this is so is not surprising when one considers the propagation mechanism of these modes. The very facts that the waves travel at high velocities and decay slowly indicate that the major part of the energy propagation occurs in deeper layers with high body wave velocities and low damping. This immediately implies that insignificant amounts of energy are propagated horizontally in the softer surface layers or, in other words, that the higher frequency motions in the surface layers are maintained through nearly vertical energy propagation through a mechanism similar to that of slightly inclined body waves. The result is that the upper parts of the mode shapes, i.e. the variation of displacements with depth, are virtually identical to those found in analyzing vertically propagating or slightly inclined body waves.

Thus, in practical calculations the effects of higher-order surface wave components can be considered by assuming a certain content of slightly inclined or vertical body waves in the control motion. In view of this observation only Rayleigh wave fields consisting of fundamental modes will be considered in the following sections.

Effect of Layering and Distance of Propagation

The importance of using layered system theory, rather than the simpler half-space theory, in dealing with structures constructed in a layer of soil overlying rock has been illustrated in numerous studies. Failure to correctly represent a layered system in a dynamic analysis can lead to erroneous conclusions concerning the variation of vertical motions with depth in a soil deposit.

In dealing with horizontally propagating Rayleigh waves, it is also important to consider the manner in which the motions will vary with distance of propagation. The results of such a study are shown in Fig. 4. In this analysis a typical NRC control motion with a peak acceleration of 0.25g was represented by a system of fundamental Rayleigh waves which were then allowed to propagate horizontally from the control point location across a horizontal deposit of sand overlying rock. The figure shows the changes in surface motion characteristics with distance of propagation. It is readily apparent that in such a system, the high frequency components of the fundamental Rayleigh waves are rapidly damped out as a result of the relatively high damping characteristics of the soil and in fact, at a distance of a few hundred feet, virtually all motions with frequencies higher than 1 to 2 Hz have decayed to insignificant values. Since most soil deposits extend horizontally for thousands of feet, it is thus unrealistic to expect that the high frequency components of motions in such deposits could result from horizontal propagation of fundamental Rayleigh waves.

As to the higher-order surface waves discussed in a previous section, some of these modes may not decay as rapidly as the fundamental modes. However, as discussed, if such waves do occur, they will, at shallow depths, be similar to slightly inclined body waves and they can be treated as such. They will, because of their long wavelengths produce motions which are essentially in phase at any shallow depth within reasonable horizontal distances. Thus they are unlikely to produce significant rocking and/or torsion in nuclear structures.

Thus, for structures on soil deposits and cases where the frequencies of concern are greater than 1 or 2 Hz, there seems to be no realistic basis for considering that Rayleigh waves make any significant contribution to the site response. The same logic would also apply to Love waves.

However there are two cases where this argument would not necessarily apply:

(1) for surface motions propagating in rock where the much lower damping and much higher wave velocities would lead to very low rates of horizontal attenuation of motions

and (2) for structures whose response is primarily

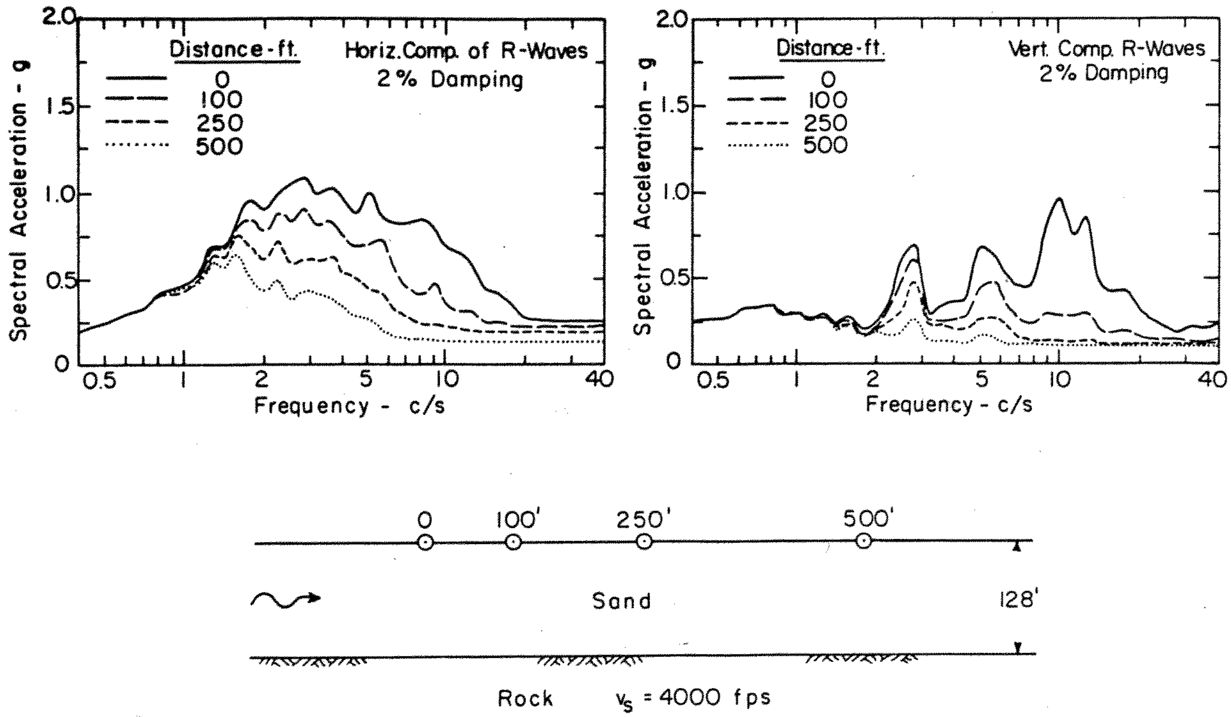

FIG. 4 ATTENUATION OF RAYLEIGH WAVE CHARACTERISTICS WITH DISTANCE TRAVELLED THROUGH SOIL DEPOSIT

dependent on the long period components of motion (say below 1 Hz) since such components are apparently not damped out readily even in soil deposits.

The above observations are apparently consistent with seismological observations of earthquake ground motions which do not indicate any significant contributions of Rayleigh waves in the high frequency range.

With regard to the design of nuclear power plants, the above discussion leads to two important conclusions:

(1) For structures located on soil deposits, there is no need to consider Rayleigh or Love waves as making any significant contribution to the ground motions for the purpose of evaluating soil-structure interaction effects.

and (2) For structures located on rock, surface waves may contribute to the ground motions but not at the highest frequencies required to be considered in design. Never-the-less consideration of the possible effects of some components of surface waves may be warranted in design.

Additional evidence supporting these statements has been presented in Gomez-Masso et al. (1979).

VERTICALLY PROPAGATING WAVES

The great majority of methods for site response analysis use Kanai's (1952) assumption of vertically propagating waves. This assumption leads to simple one-dimensional mathematical models for horizontally layered systems and has, partly because of the similarity between motions caused by different wave fields, led to remarkable success in predicting the major features of site response during earthquakes, especially since the analytical procedure was modified by introduction of the equivalent linear method by Seed and Idriss (1969).

Linear site response problems with vertically propagating waves can be solved by a multitude of numerical techniques which are described in texts on soil dynamics, e.g., Desai and Christian (1976). The most efficient method for computing free-field motions from a specified surface control motion appears to be the complex response method used in the program SHAKE, Schnabel et al. (1972). With these methods it is currently possible to analyze any layered viscoelastic soil system overlaying a viscoelastic half-space. The control motion can be specified at the ground surface, at any depth in the soil deposit or as an outcrop motion. Nonlinear effects can be approximated by the equivalent linear method.

Recent efforts have been directed towards the development of true nonlinear methods of analysis. Several methods have been proposed for performing nonlinear total stress analysis of site response problems with vertically propagating shear waves. The most important of these are: The method of characteristics, Streeter et al. (1974), Idriss et al. (1976), Taylor and Larkin (1978); the finite difference method, Joyner (1977); and implicit integration schemes, Martin (1975). In addition several methods of effective stress analysis have been proposed, Ghaboussi and Dikmen (1978), Zienkiewicz et al. (1978), Finn et al. (1977), Liou et al. (1977), and Martin and Seed (1979), which can predict the pore pressure build-up in saturated sands during seismic excitation.

Comparative studies of ground motion characteristics computed by the equivalent linear method and non-linear methods show relatively small differences except where motions are very strong and soils relatively weak--a situation not likely to occur at a nuclear plant site. Thus the development of non-linear analysis techniques has further confirmed the fact that equivalent linear methods are sufficiently accurate for virtually all practical purposes in evaluating the response of nuclear power plant sites.

INCLINED BODY WAVES

Some energy may be arriving at the control point in the form of non-vertically propagating body waves. There is in fact evidence to suggest that most of the energy approaching the ground surface results from body waves inclined within about 30° of the vertical. This includes the effect of the high-order surface wave modes discussed in a previous section.

The response of horizontally layered sites to plane harmonic body waves arriving at a specified incident angle through an underlying elastic half-space has been investigated by several researchers. The fundamental work was done by Thomson (1950) and Haskell (1960, 1962) who developed an efficient matrix method for computing the frequency-dependent transmission coefficients in a layered continuum for incident SH, SV and P-waves. Efficient computer codes for the Thomson-Haskell method were developed by Hannon (1964) and Teng (1967). Silva (1976) extended the Thomson-Haskell method to include damping in the soil layers. With this method it is possible to solve linear or equivalent linear site response problems with inclined body waves for systems consisting of viscoelastic soil layers overlying a uniform undamped half-space, provided the incident angle in the half-space is known. Since no damping is included in the half-space and the resulting surface motions do not decay in this horizontal direction. More recently, Chen and Lysmer (1979), have developed a method which includes damping in the underlying half-space.

Analyses of surface response to inclined body waves have also been made by Joyner et al. (1976), who determined the transfer functions from bedrock to the soil surface for a soil deposit 186 m in depth for shear waves propagating at various angles to the vertical. The results of this study are shown in Fig. 5, and it is apparent that for angles of incidence up to 45°, there is a negligible difference between the motions computed for inclined waves and for vertically propagating waves. Similar conclusions have been reached by Udaka et al. (1978).

It is reasonable to conclude therefore that the variation of horizontal motions with depth within a soil deposit are for all practical purposes the same, whether they are computed for vertical or inclined directions of propagation within the depth range of interest to engineers. On this basis it is appropriate to use analyses for vertically propagating waves because of their greater simplicity and availability of solutions.

MOTIONS OF SHALLOW DEPTH

As discussed early in this paper, only the motions within a relatively shallow depth (the projected depth of embedment) of the free field will influence the motions of structures. The same discussion also indicates that both the spatial and temporal variation of the free-field motions within this depth are of importance in evaluating soil-structure interaction effects. It is therefore appropriate to discuss in more detail how the amplitude and frequency content of free-field motions vary with depth and, in particular, how they vary near the free ground surface.

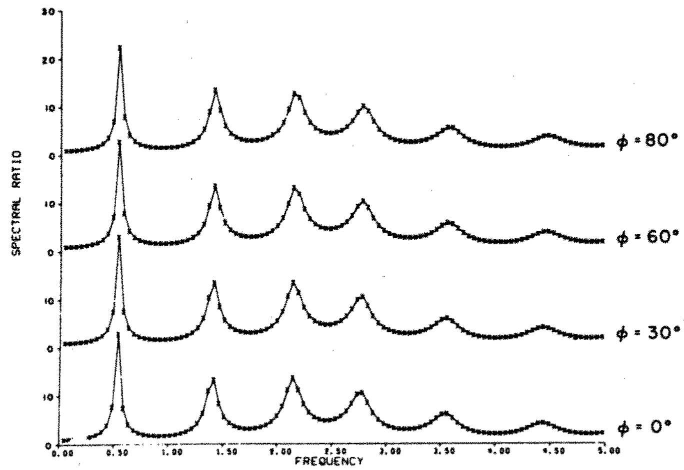

FIG. 5 INFLUENCE OF ANGLE OF SHEAR WAVE INCIDENCE ON COMPUTED SURFACE RESPONSE

(after Joyner et al. 1976)

If motions could not vary significantly within relatively shallow depths, there would be little point in pursuing this matter further. Not to consider this variation would be equivalent to assuming that the soil mass behaves like a rigid body in which case no interaction would take place and an analysis of the problem would be unnecessary. The belief that it is necessary to analyze soil-structure interaction therefore implies the tacit assumption that motions will vary with depth and appropriate consideration of this fact must be included in any analytical procedure.

The presence of the strong discontinuity represented by the free ground surface imposes predictable and observable limitations on how horizontal amplitudes and frequency contents of motions vary with depth near the ground surface and gives reason to believe that these variations may be significant. Thus the subject merits careful consideration in soil-structure interaction studies.

Theoretical Considerations

The potential effects of the free ground surface on the amplitude and frequency content of waves at various depths in a uniform deposit is shown in Figs. 6 and 7. Both figures show the variation of amplitude with the dimensionless depth z/λ_s in a perfect half-space, where $\lambda_s = V_s/f$ is the wavelength of shear waves at the frequency, f[Hz], considered.

Figure 6 corresponds to the case of vertically propagating shear waves for which the horizontal amplitude is

$$U = U_o \cos 2\pi \frac{z}{\lambda_s} \tag{12}$$

and Fig. 7 corresponds to the case of horizontally propagating Rayleigh waves.

The two types of wave fields are obviously quite different. It is remarkable, however, that both the shear wave field and the Rayleigh wave field produce monotonically decreasing horizontal displacements within the approximate depth

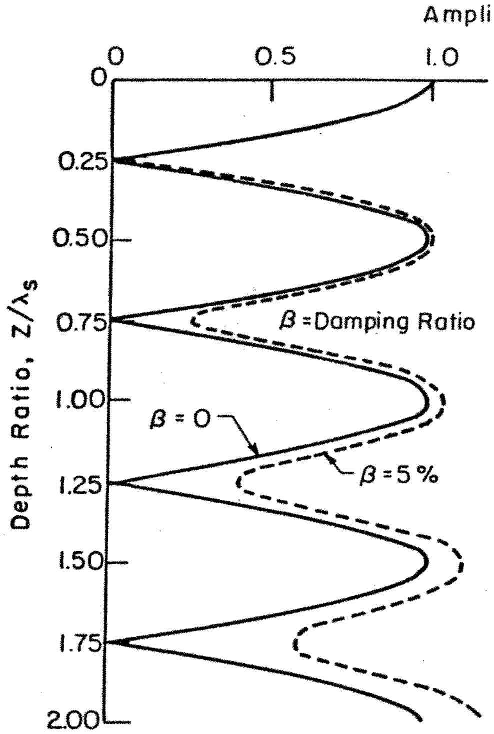

FIG. 6 SHEAR WAVES

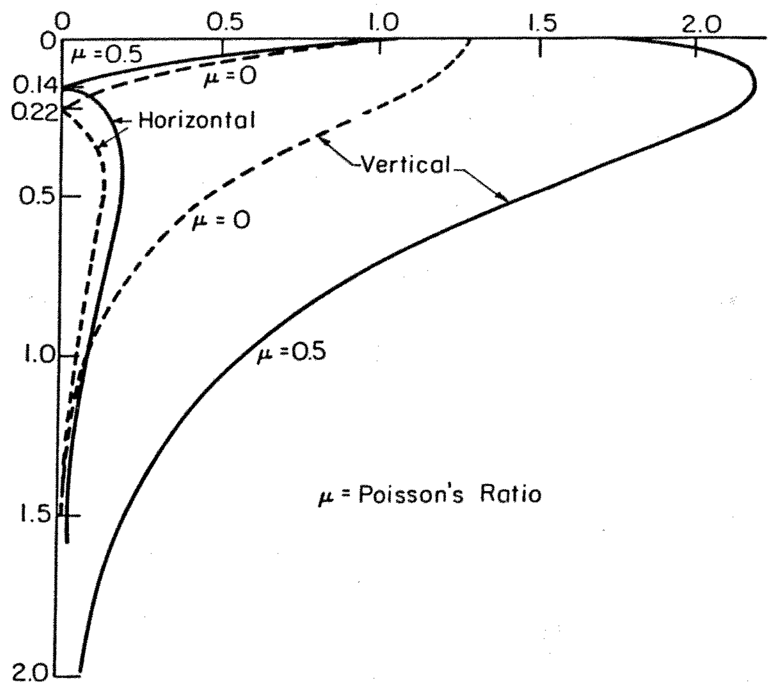

FIG. 7 RAYLEIGH WAVES

$$z \simeq \frac{1}{4}\lambda_s \quad \text{to} \quad \frac{1}{5}\lambda_s$$
$$\simeq \frac{V_s}{4f} \quad \text{to} \quad \frac{V_s}{5f} \qquad (13)$$

and that all horizontal displacements vanish at this depth. A similar phenomenon occurs for inclined shear waves and for layered soil systems where V_s in Eq. (17) can be replaced by the average shear wave velocity, $\overline{V}_s$ above the depth z. As can be seen from the dotted curve shown in Fig. 9 the existence of material damping does not change the substance of these observations.

Two important conclusions can be drawn from these analyses:

(1) Any horizontal motion computed (or observed) at the depth z must be deficient of components of the frequency

$$f \simeq \frac{\overline{V}_s}{4z} \quad \text{to} \quad \frac{\overline{V}_s}{5z} \qquad (14)$$

i.e., its response spectrum will have a dip at the approximate frequency f, which incidentally is equal to the fixed base natural frequency of the soil column above the depth z. Thus, the only level at which a smooth spectrum can exist is at a free surface, and specifying a control motion with a smooth spectrum at any other depth will, as experience has shown, lead to completely unrealistic results. This free surface can be the actual ground surface or a real or imaginary rock outcrop; however a smooth spectrum cannot exist within a soil deposit, whether the motions be due to near-vertically propagating body waves or to horizontally propagating Rayleigh waves.

(2) In a deposit with uniform properties, seismic motions will decrease with depth below the ground surface at least down to the depth

$$z \quad \frac{\overline{V}_s}{4f_{max}} \quad \text{to} \quad \frac{\overline{V}_s}{5f_{max}} \qquad (15)$$

where f_{max} is the highest frequency present in the motion.

This follows directly from Eq. (12) which shows that all components decrease in amplitude within the above depth. Because of variations in soil characteristics with depth this predicted reduction will often extend to depths greater than those indicated by Eq. (15). For a typical soil site, with say $\overline{V}_s$ = 1000 fps, and a seismic environment, with say f_{max} = 20 Hz, the above formula shows that a significant reduction in the free field motion may occur within the upper 10 ft (or deeper if the predominant frequency is lower) of the site. Thus in view of the discussion in connection with Eq. (4), even relatively shallow embedment may significantly influence the seismic response of structures on soft sites and both the embedment and the reduction in the amplitude of the seismic environment with depth should be considered in a rational interaction analysis.

Substitution of realistic values of V_s and f into Eq. (13) will show that z is typically larger than 20 ft for soil sites and 60 ft for rock sites. Thus typical structures experience only the upper part ($z/\lambda_s < 0.2$) of the motions shown in Figs. 6 and 7. In this "shallow" depth range horizontal motions produced by any seismic environment with the same horizontal surface control motion will be quite similar. It is therefore to be expected that the horizontal motions produced at points below the ground surface during earthquakes will be relatively independent of the type of wave field producing the motions.

The above observations were made for motions in a uniform half-space. For layered systems, the stiffness of which usually increases with depth, calculations have shown that the similarity between motions produced at shallow depth by different types of wave fields is even more pronounced.

An interesting example of these effects in a 600 feet deep soil deposit overlying a rigid half-space is shown by the analytical results presented in Figs. 8, 9, and 10. To study the response at different depths in this deposit, analyses were made using vertically propagating shear wave theory for 15 different excitation records. In eight of the analyses, existing records obtained on deep soil deposits were scaled to have a peak acceleration of 0.20g and considered to be developed at the ground surface. The distribution of acceleration with depth and the frequency characteristics of the motions developed at depths of 40 and 76 ft were then determined by deconvolution analyses.

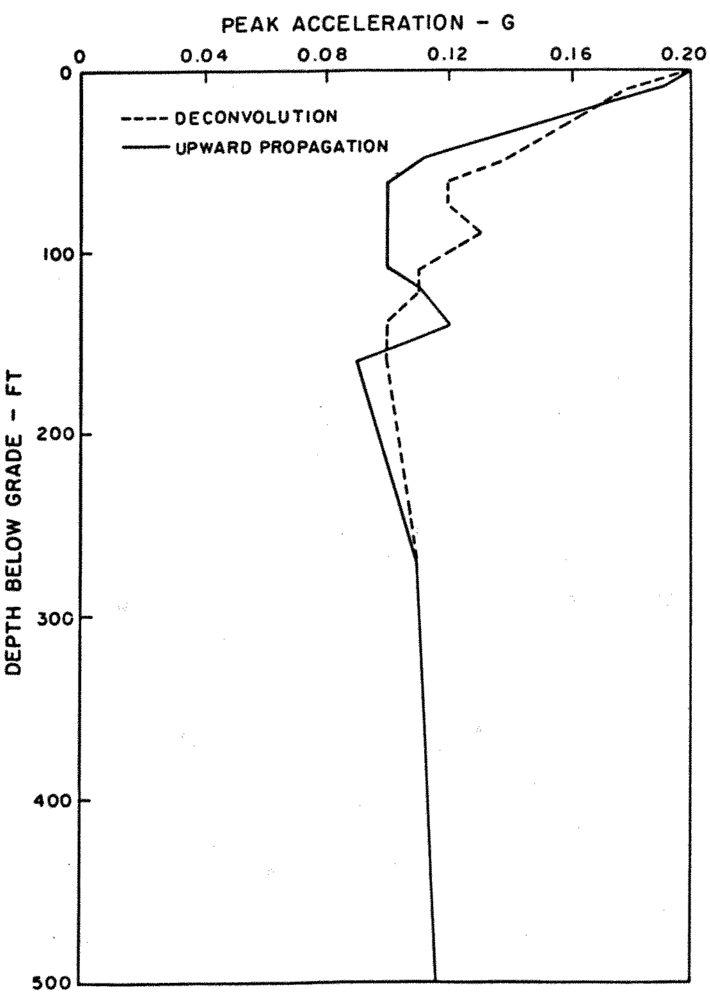

FIG. 8 VARIATION OF MEAN PEAK ACCELERATION WITH DEPTH OF DECONVOLUTION AND UPWARD PROPAGATION ANALYSES

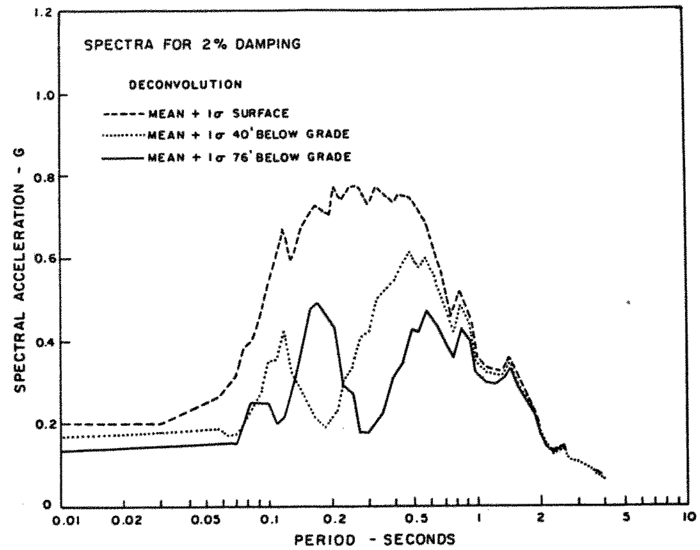

FIG. 9 STATISTICAL ANALYSIS OF COMPUTED SPECTRAL SHAPES AT DIFFERENT DEPTHS IN SOIL DEPOSIT

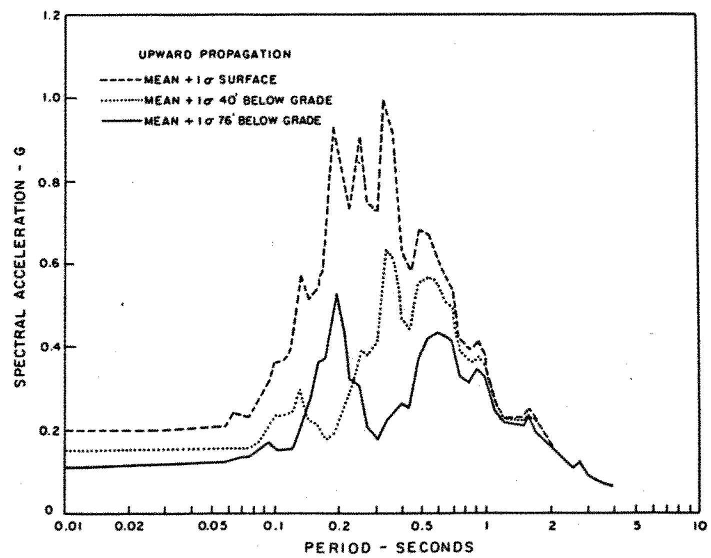

FIG. 10 STATISTICAL ANALYSIS OF SPECTRAL SHAPES AT DIFFERENT DEPTHS IN SOIL DEPOSIT

For the same soil deposit, a second study was made in which seven records representative of rock motions were used as base excitation and the base motions were scaled in each case to produce a peak acceleration of 0.20g at the ground surface.

There was surprisingly little difference in the computed distribution of motions whether the excitation was applied at the ground surface or whether it was applied at the base of the soil deposit. The results of the two sets of studies were analyzed statistically to determine the mean acceleration distribution separately for the deconvolution analyses and for the base input analyses. The results of this analysis are shown in Fig. 8. On the whole the results are remarkably similar, all showing a marked drop in peak acceleration within the upper 100 ft.

The response spectra for the motions developed at depths of 40 and 76 ft were also computed and analyzed statistically for the two different groups. The 84 percentile spectra for surface motions, motions at 40 ft depth and motions at 76 ft depth for the deconvolution analyses are shown in Fig. 9. It may be seen that while the spectrum for the surface motions is of the broad band type, the spectrum for motions at a depth of 40 ft contains a marked suppression of frequencies corresponding to a period of about 0.18 second while that for motions at a depth of 76 ft shows a marked suppression of frequencies corresponding to a period of about 0.3 second. The fixed base natural periods of this deposit for 40 ft of soil and 76 ft of soil were about 0.18 and 0.3 seconds respectively. Similar results are shown in Fig. 10 for the base excitation analyses. Thus it may be seen that the frequency suppression effect, as predicted by Eq. (14), is mainly a feature of the geometry and material characteristics of the deposit and depends only slightly on the assumed source of the wave motions involved.

In a deposit 600 ft deep extending to substantial distances in all directions there would not be expected to be any substantial contribution of surface waves to the motions in the frequency range of 1 to 20 Hz, the types of motions primarily investigated in this study, and thus the use of vertically propagating shear waves as the primary wave field is appropriate. Never-the-less the effect of the discontinuity provided by the ground surface on the amplitudes of motions and the frequency characteristics of motions at different depths is clearly illustrated by this example.

Field Evidence

Despite the fact that no concerted effort has been made to date to obtain field data to confirm the above theoretical predictions, a substantial body of field data does in fact exist.

Variation of peak acceleration with depth

The best data to show the variation of peak accelerations with depth is that obtained from vertical arrays of instruments, which only in recent years have been installed at a number of locations to record earthquake motions. Probably the most successful array has been that installed by the U.S. Geological Survey near Menlo Park, California, Joyner et al. (1976). Instruments are located at depths of 0, 12 m, 40 m, and 186 m below the ground surface in a soil profile with rock at a depth of 186 m. A number of records of small earthquakes were obtained from the instruments in this array during the period 1972 to 1977. A typical record is shown in Fig. 11. The marked decrease in amplitude of the recorded motions with depth is readily apparent.

Similar decreases of motion amplitude with depth have been observed in four earthquakes recorded in a similar array at Richmond, California, by the University of California Seismological Laboratory.

For somewhat stronger motions, an excellent set of data was obtained by records obtained in the basements of buildings in Tokyo in the Tokyo-Higashi-Matsuyama earthquake of July 1, 1968, Ohsaki and Higawara (1970). The recorded values of maximum acceleration for different basement depths are shown in Fig. 12. Although there is considerable scatter in the data, peak accelerations at a depth of 70 ft are typically only about 25% of those recorded at the ground surface. It may be argued that these results are influenced by soil-structure interaction effects, but such effects are likely to be small and in any case would tend to minimize the variation of peak acceleration with depth rather than amplify the effect.

Yet another source of data can be obtained from records obtained in nearby pairs of buildings, each pair involving one constructed at the ground surface and the other at a depth of about 15 ft below the ground surface, in the San Fernando, California earthquake of 1971. Such records for seven sets of buildings are listed in Table 1. It may be seen that in all seven cases, the peak acceleration recorded in the building with a basement was substantially less than that in the building constructed on the ground surface. While some variation of motions would be due to different spatial locations of these buildings, a statistical study of this data clearly shows that the substantial decrease in acceleration with depth is not a chance phenomenon but a pattern attributable to deterministic effects.

Finally, for very strong motions, an excellent set of records was obtained at the Humboldt Bay Power Station in the 1975 Ferndale earthquake. One of these records was obtained at a free-field ground surface location and another at the base of a caisson structure at a depth of 80 ft. The full set of records is shown in Fig. 13. The average maximum acceleration at the ground surface was 0.30 g while the average at a depth of 80 ft was 0.13 g. Clearly this difference needs to be taken into account if the effects of soil-structure interaction are to be analyzed in a meaningful way in this case.

Other data are available to show similar affects to those discussed above but it is believed that the cases presented provide sufficient validation that variations in ground motion with depth are not randomly variable, but characteristically decrease in the range of engineering interest, except for sites with unusual variations in soil characteristics with depth.

Variation of frequency characteristics with depth

Analytical considerations show not only a variation of peak horizontal accelerations with depth but also a deterministic variation of frequency content, and therefore of spectral shape at different depths below the ground surface. Specifically it is to be expected that at any depth z below the ground surface, frequencies of the order of $f = \overline{V}_s/4z$ (hz] will be suppressed due to ground surface reflection effects. (For Rayleigh waves the suppressed frequency would be approximately $\overline{V}_s/5z$ but it has already been shown that Rayleigh waves with frequencies above about 1 Hz could not persist in an extensive soil deposit due to the rapid attenuation of high frequencies in relatively short horizontal distances).

Corroborative evidence of this effect is provided by the data obtained from the Menlo Park array for the recorded motions shown in Fig. 11. The acceleration response spectra for the motions recorded in these events are plotted in Fig. 14, and normalized spectra, obtained by dividing the spectral ordinates for any period by the spectral ordinate for the surface motions at that frequency are shown in Fig. 15. It may be seen that using this technique, the normalized surface spectrum becomes a broad band spectrum and the spectra at other depths are scaled proportionally. It may also be seen that the normalized spectra for motions at a depth of 12 m show a marked suppression of frequencies (evidenced by a dip in the spectrum) corresponding to a period of 0.5 sec, which corresponds to the value $4z/\overline{V}_s$ for this deposit. Similarly the normalized

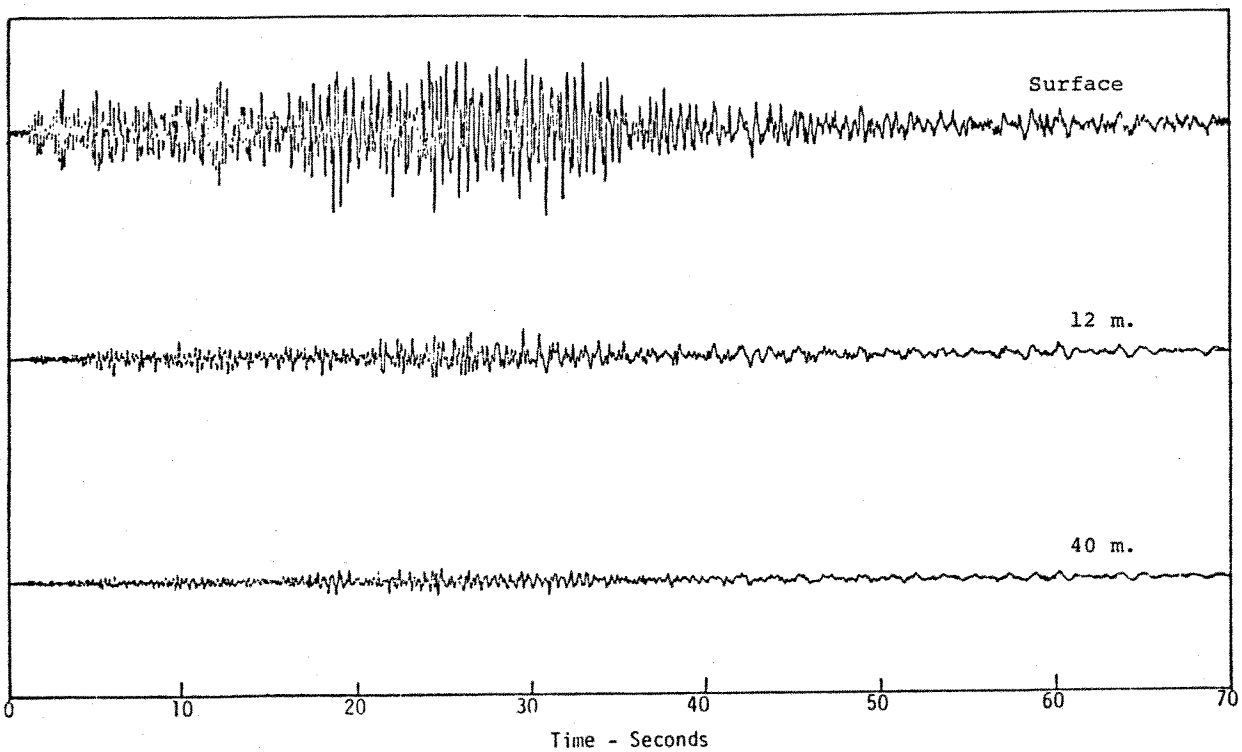

FIG. 11 VARIATION OF GROUND MOTIONS WITH DEPTH IN U.S.G.S. ARRAY DURING BEAR VALLEY (CALIFORNIA) EARTHQUAKE OF APRIL 9, 1972

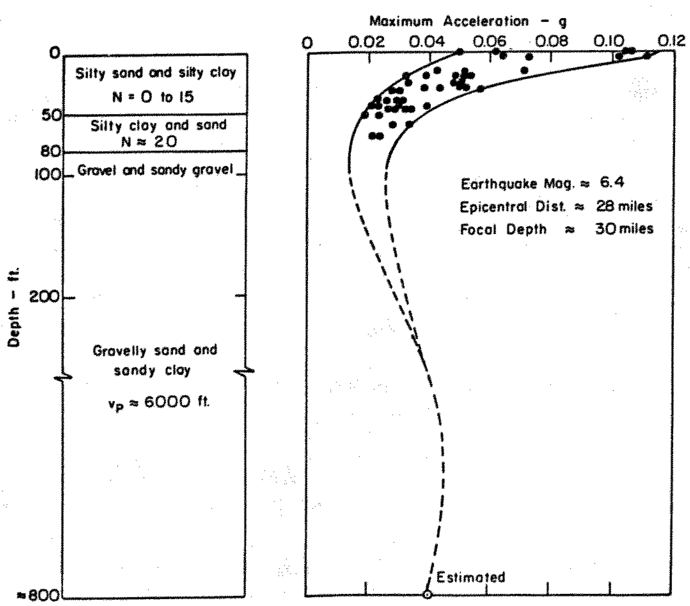

FIG. 12 VARIATION OF RECORDED MAXIMUM ACCELERATION WITH DEPTH FOR BUILDING IN TOKYO-HIGASHI-MATSUYAMA EARTHQUAKE, JULY 1, 1968

spectra for a depth of 40 m show a suppression of frequencies corresponding to a period of 0.75 seconds, which corresponds to the value of $4z/\overline{V}_s$ for the same deposit. Similar results were obtained in two other earthquakes recorded at this site. The deterministic value of the frequency suppression effect is clearly evident from this data.

Similar results were also obtained from an analysis of spectra for the motions recorded at Humboldt Bay Power Station (Fig. 13). The spectra for the transverse and longitudinal records of horizontal ground motions at the ground surface and at a depth of 80 ft are shown in Figs. 16 and 17 and the normalized spectra for the same motions are shown in Figs. 18 and 19. Again it is apparent that there is a strong frequency suppression for both transverse and longitudinal motions at a period of about 0.5 sec, which corresponds closely to the computed value of $4z/\overline{V}_s$ for the soil deposit at this site. The great similarity in normalized spectra for transverse and longitudinal motions is shown more clearly in Fig. 20, where the spectra are plotted together and show almost identical characteristics, again illustrating the deterministic nature of this effect.

Hays et al. (1979) and Gazetas and Bianchini (1979) and Chang et al.(1986) have recently reported data showing similar effects for motions recorded at depths below the ground surface.

In summary it seems apparent that the frequency suppression effect in soil deposits is not merely an analytical concept but that it is also apparent in recorded data. This agreement between analytical concepts and observational data clearly indicates the desirability of

TABLE I

CHANGE IN MAXIMUM ACCELERATION BETWEEN GROUND LEVEL AND BASEMENT LEVEL

Location	Maximum Acceleration: Ground Surface	Maximum Acceleration: Basement	Percent Change in Ground Surface Acceleration at Basement Level
8244 Orion Blvd. 15107 Vanowen Blvd.	0.26g	0.12g	-54%
14724 Ventura Blvd. 15250 Ventura Blvd.	0.26g	0.23g	-12%
Hollywood Storage Bldg.	0.22g	0.15g	-32%
6430 Sunset Blvd. 6466 Sunset Blvd.	0.19g	0.12g	-37%
1880 Century Park East 1800 Century Park East	0.13g	0.10g	-23%
222 South Figueroa 234 South Figueroa 445 South Figueroa	0.15g 0.20g	0.14g	-20%
3407 West Sixth 616 S. Normandie 3470 Wilshire 3550 Wilshire	0.18g	0.12g 0.14g 0.17g	-33% -22% - 6%

considering this phenomenon in evaluating site response or soil-structure interaction effects for embedded structures.

Summary and Conclusions

At the outset of this section it was shown that any analyses of soil-structure interaction must necessarily be based on a knowledge of the seismic environment to which the structure will be subjected. This requires an understanding of the spatial distribution of motions in the ground within the depth of embedment.

In the light of the discussion of this subject presented in the preceding pages it seems reasonable to draw the following conclusions concerning the role of the seismic environment in soil-structure interaction analyses.

1. On rock sites structures are likely to be founded near the surface. For such sites, earthquake motions may consist of an unknown mixtures of Rayleigh waves, Love waves and near-vertically propagating body waves. Because of the low damping in the rock, attenuation of Rayleigh and Love waves will be small within the general area of the site. It would be expected that the presence of such waves would tend to increase the rocking and torsional excitation on the base of a structure due to out of phase effects as the waves pass across the base. Thus structures located on rock should be analyzed for these motions to determine the potential severity of their contributions to the total response of the structure. However, since a substantial part of the response is likely to be due to the effects of near-vertically propagating body waves, the final evaluation may well show the influence of Rayleigh and Love waves to be small.

   In analyses using vertically or near-vertically propagating waves, however, it should be noted that because of the fact that these waves will in reality be inclined at different angles to the vertical and will be out-of-phase at different points on the base of the structure due to nonhomogeneities in the rock through which they must travel, some allowance could be made for the "base-slab averaging effect" which will cause the average motions developed in a stiff base slab to be somewhat less than those developed at individual points on the rock surface.

2. For soil sites, structures are likely to be embedded at some depth (say 20 to 80 ft) below the ground surface. The effects of fundamental Rayleigh and Love waves need not be considered at such sites in the design of nuclear plants because the high frequency components of these waves (greater than 1 Hz) will have been damped out by the soil if it extends to any significant distance (say 1000 ft) around the location of the plant. Higher order Rayleigh modes can be simulated by inclined body waves. Thus the main source of excitation will be inclined body waves and for all practical purposes, these can be analyzed as if they propagated in a vertical direction. However in soil deposits there will be an important variation in motion characteristics with depth and this should be considered in the analysis if meaningful results are to be obtained. The assumption of uniform motions in the upper layers of a soil deposit is inconsistent with the physical nature of wave mechanics and observations in the field and can only lead to misleading results unless the specified control motion is intended to take the natural variations in motion characteristics into account in some way. Without knowing something about the variations in motion it is difficult to see how this can be done realistically

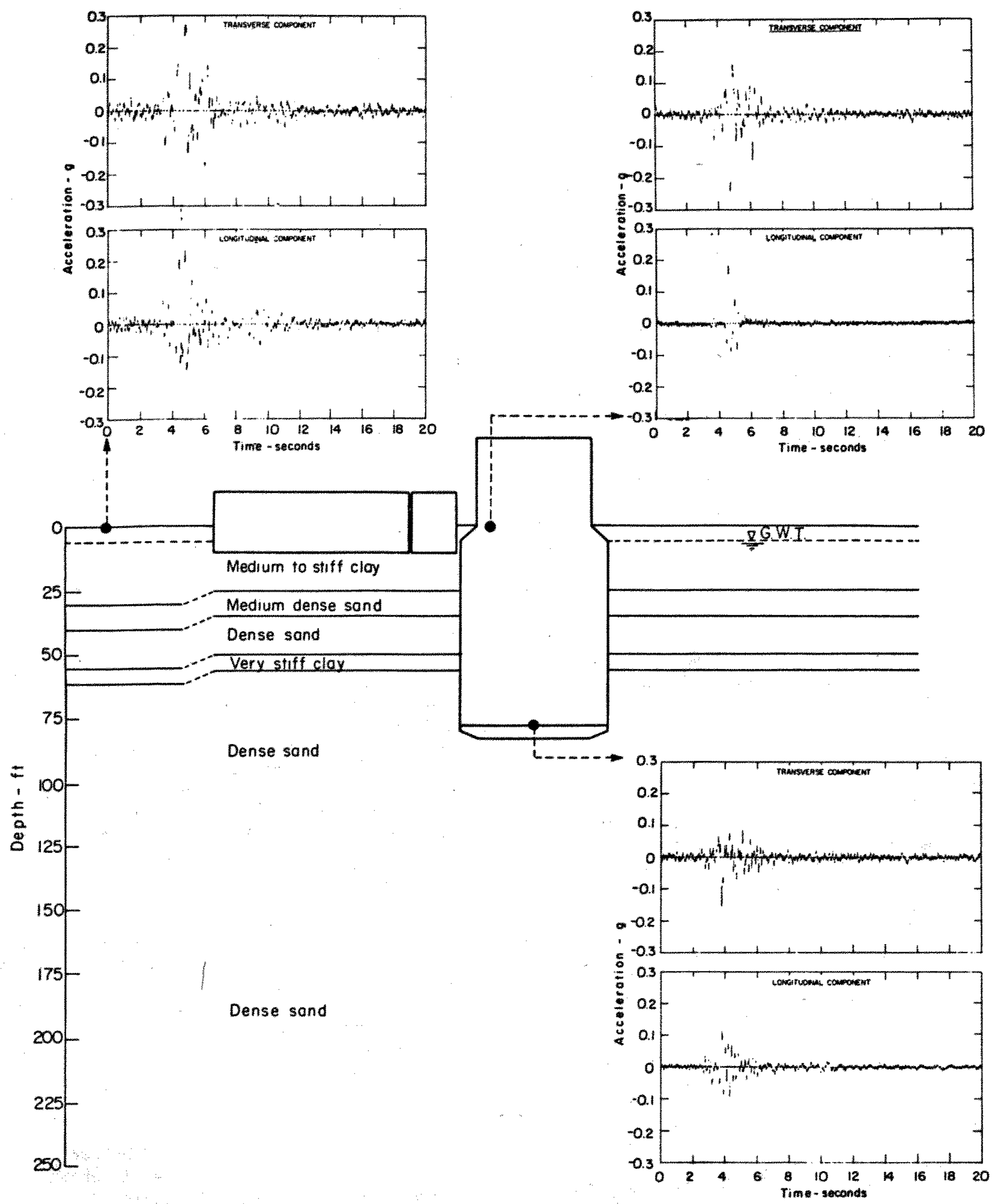

FIG. 13 GROUND MOTION RECORDS AT HUMBOLDT BAY NPS

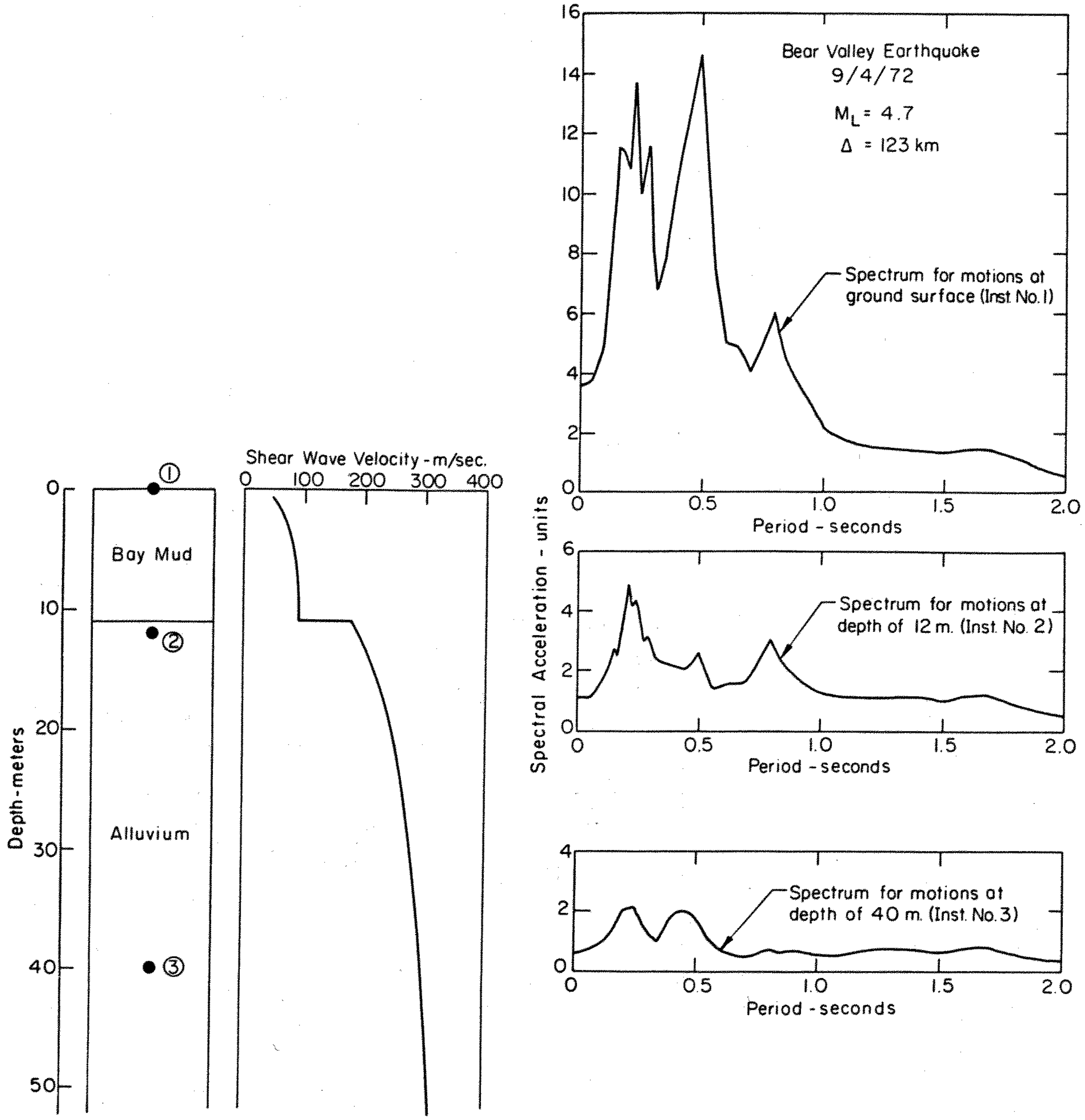

FIG. 14 SPECTRA FOR MOTIONS RECORDED IN BEAR VALLEY EARTHQUAKE, 1972

without introducing an unwarranted degree of conservatism into the soil-structure analysis procedure. The fact that the waves are slightly inclined and out of phase will also introduce a torsional input to the base of the structure and thus should also be considered.

It should not be construed from the above statements that the assumption of vertically propagating waves at soil sites is appropraite for all types of structures. The long-period components of horizontally propagating waves may be extremely important for the design of buried pipelines, tunnel linings and earth retaining structures. However, except for increased stresses in the walls of buried or embedded structures the change in the stress field due to these waves appear to have little effect on the overall horizontal motions of such structures.

The propagating nature of the displacement field may also induce additional displacements and stresses in long aboveground structures such as bridges, Bogdanoff et al. (1965), Johnson and Galletly (1972), Abdel-Ghaffar and Trifunac (1976); and rocking and torsional motions in long period single structures, Wong (1975), Scanlan (1976) and Wong and Luco (1976).

Finally, and perhaps most important, control motions should be chosen with due respect to site conditions and, if a broad band design spectrum is used, the control point should be located at the ground surface or, alternatively, at an imaginary outcrop, where it could conceivably exist, and not at some arbitrary depth below the ground surface where the boundary conditions resulting simply from the existence of a ground surface preclude this possibility.

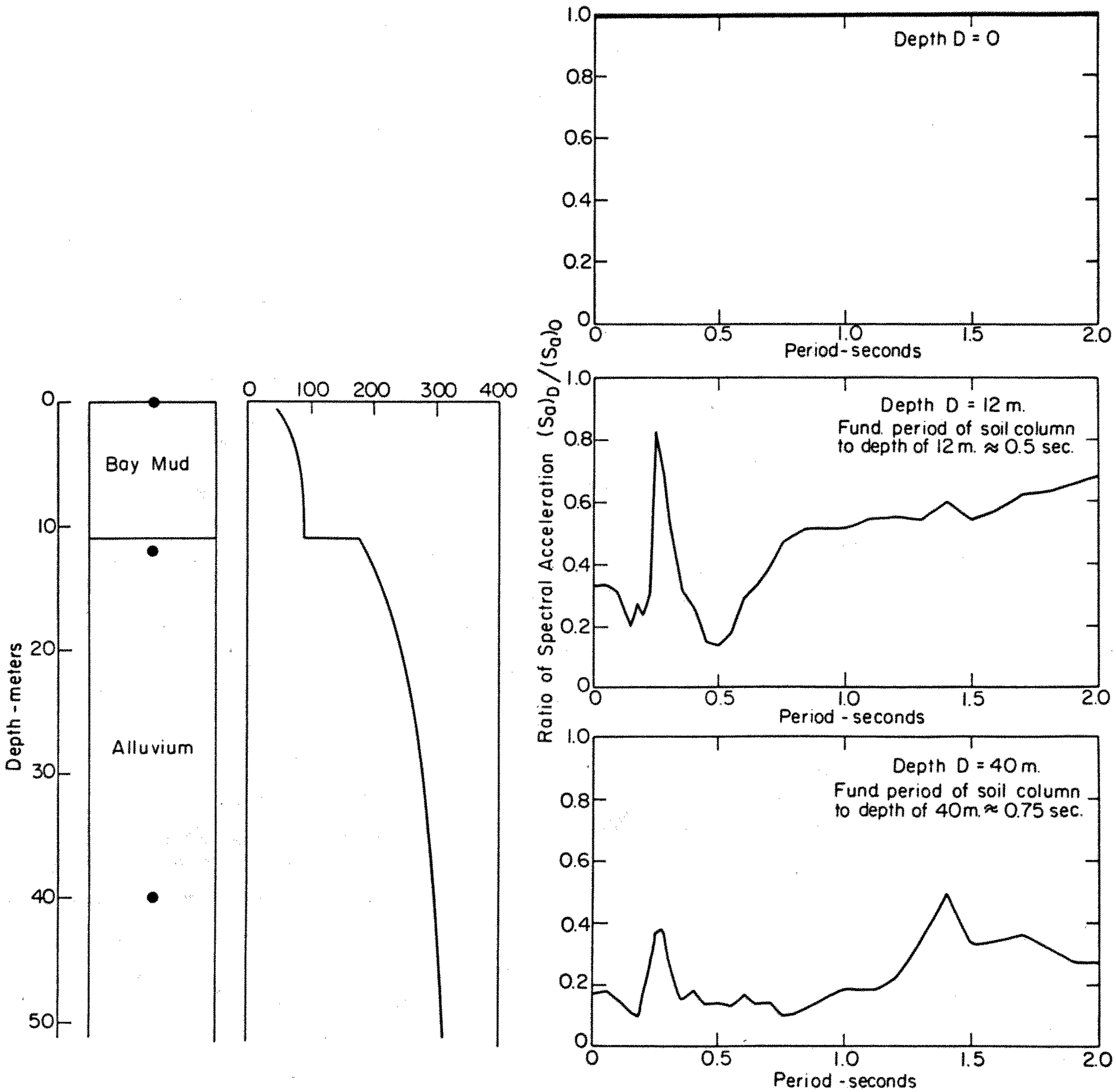

FIG. 15 RELATIVE SPECTRAL ACCELERATIONS AT DIFFERENT DEPTHS - BEAR VALLEY EARTHQUAKE, 1972

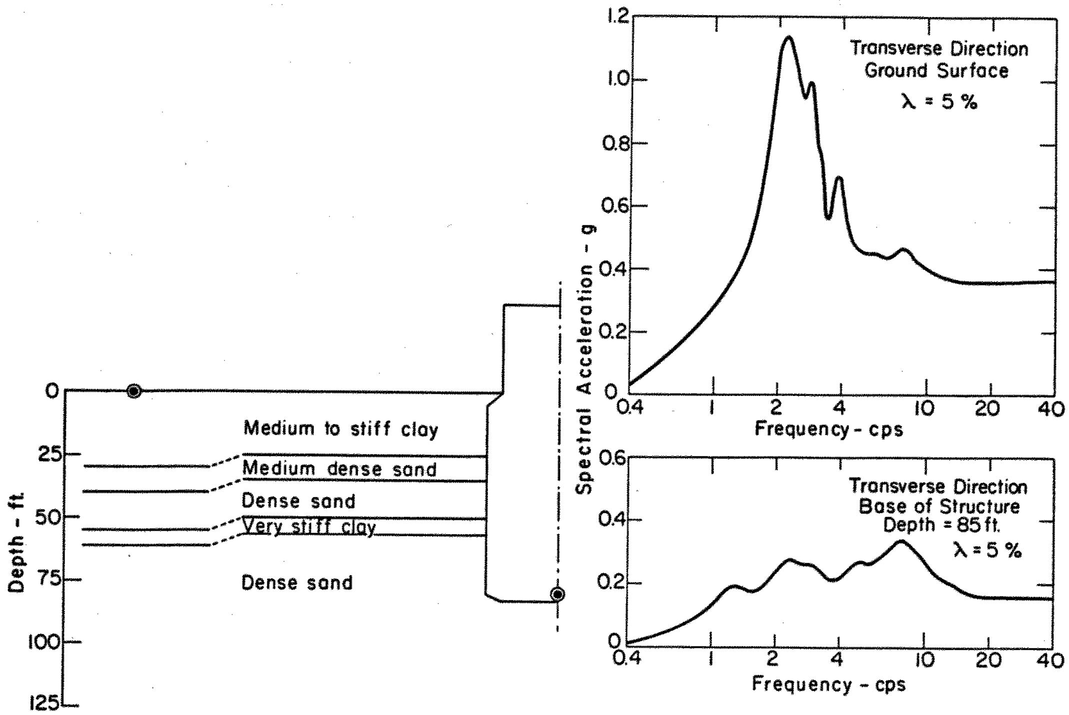

FIG. 16 SPECTRA FOR TRANSVERSE MOTIONS RECORDED AT HUMBOLDT BAY IN 1975 FERNDALE EARTHQUAKE

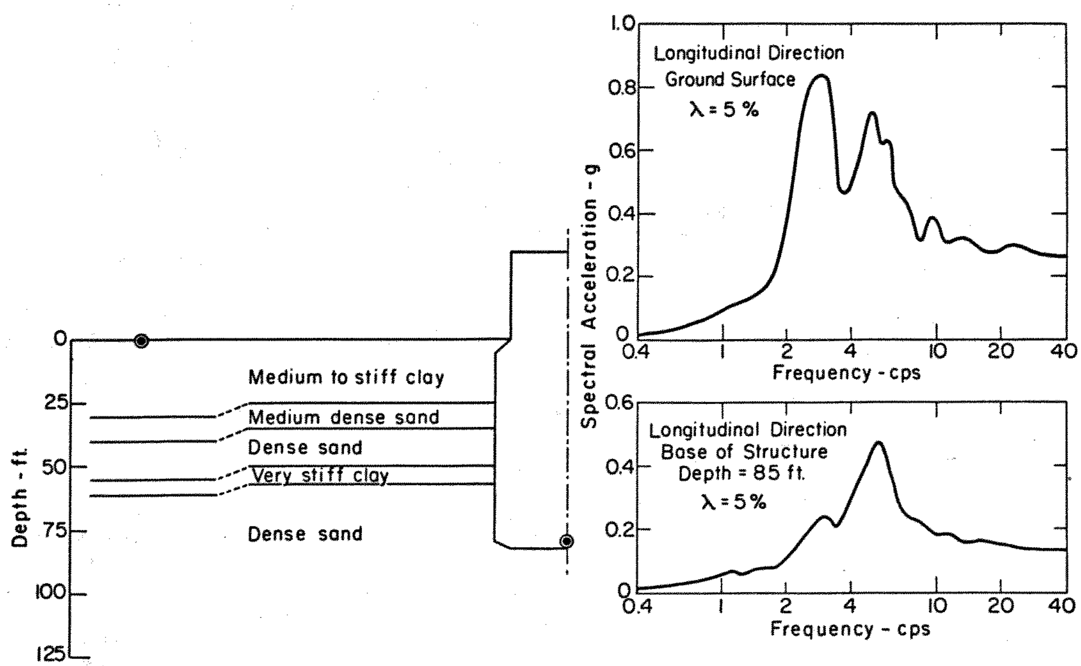

FIG. 17 SPECTRA FOR LONGITUDINAL MOTIONS RECORDED AT HUMBOLDT BAY IN 1975 FERNDALE EARTHQUAKE

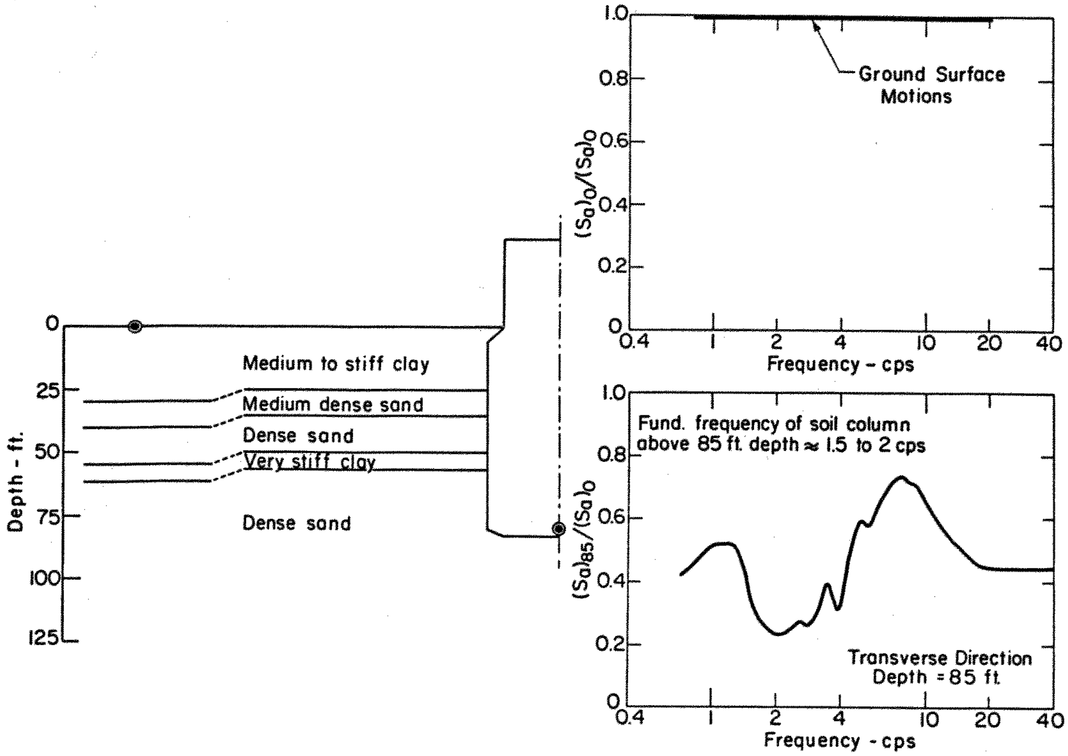

FIG. 18 RELATIVE SPECTRAL ACCELERATIONS AT DIFFERENT DEPTHS AT HUMBOLDT BAY (FERNDALE EARTHQUAKE, - 1975 - TRANSVERSE DIRECTION)

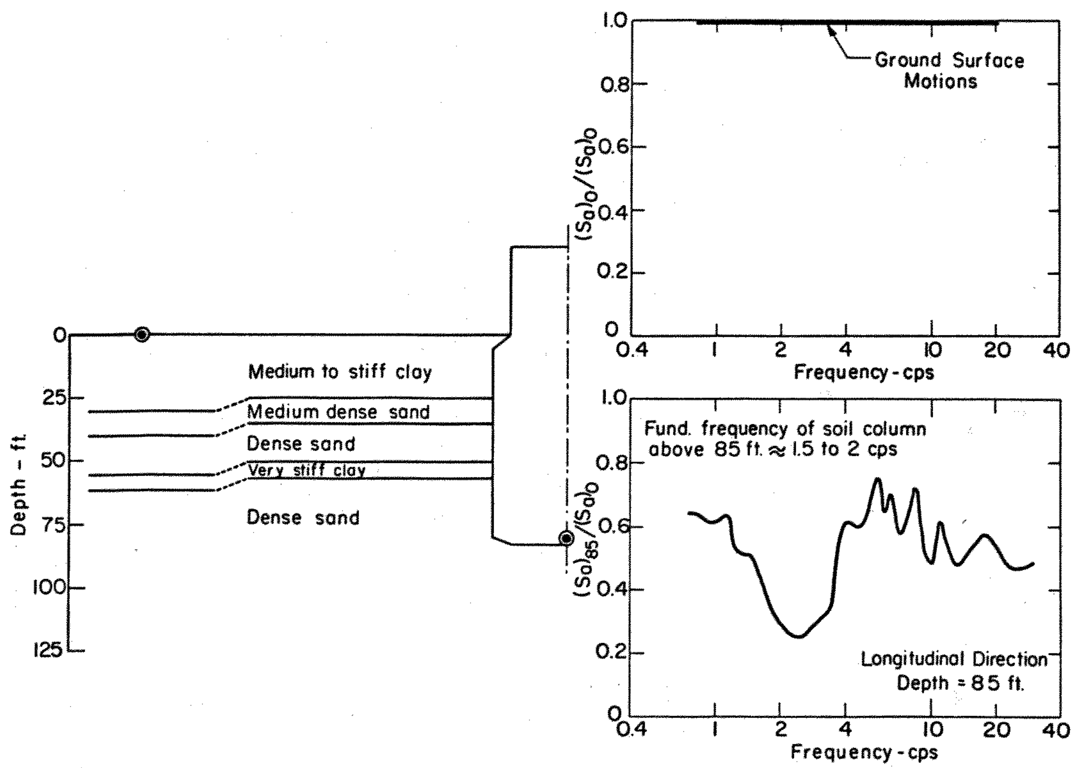

FIG. 19 RELATIVE SPECTRAL ACCELERATIONS AT DIFFERENT DEPTHS AT HUMBOLDT BAY (FERNDALE EARTHQUAKE, 1975 - LONGITUDINAL DIRECTION)

NORMALIZED SPECTRA AT 85 FT DEPTH CORRESPONDING TO BROAD BAND SPECTRUM AT GROUND SURFACE HUMBOLDT BAY POWER STATION, FERNDALE EQ. JUNE 7, '75

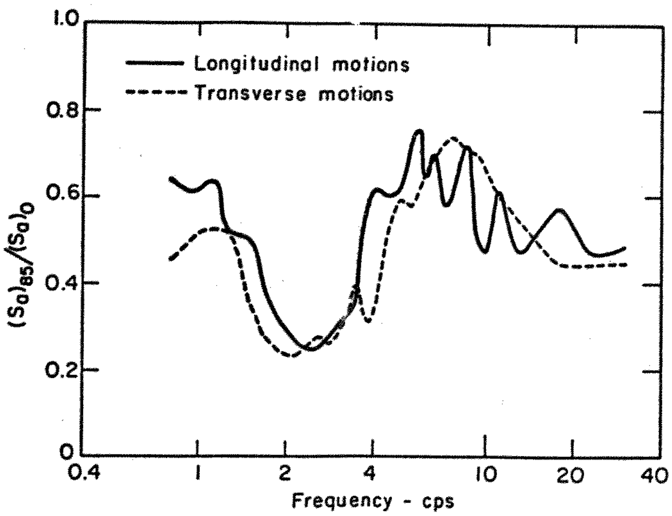

FIG. 20 COMPARISON OF NORMALIZED SPECTRA AT 85 FT DEPTH FOR TRANSVERSE AND LONGITUDINAL MOTIONS

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STOCHASTIC RESPONSE OF RIGID FOUNDATIONS

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SUMMARY

While the study of Kinematic Interaction effects calls, in general, for advanced analytical and numerical techniques, an excellent approximation was proposed recently by Iguchi. This approximation was used by the authors to analyze embedded foundations subjected to spatially random SH-wave fields, i.e., motions that exhibit some degree of incoherence. The wave fields considered ranged from perfectly coherent motions (resulting from seismic waves arriving from a single direction) to chaotic motions resulting from waves arriving simultaneously from all directions. Additional parameters considered were the shape of the foundation (cylindrical, rectangular) and the degree of embedment. It was found that kinematic interaction usually reduces the severity of the motions transmitted to the structure, and that incoherent motions do not exhibit the frequency selectivity (i.e., narrow valleys in the foundation response spectra) that coherent motions do.

INTRODUCTION

Although the variability in time of earthquake ground motions is easily quantified from seismograms, less is known about their spatial variability. For extended structures, this spatial variability is an important consideration and cannot, in general, be neglected.

On a dimension scale comparable to the size of the structure, the seismic motion can be thought of as the superposition of several waves travelling in the soil along different directions. Assuming linearity (valid for small ground deformations) the total response of the structure can then be obtained as the sum of the effects produced by each single wave. In principle, the earthquake motion can be thought of as the superposition of many body and surface waves. For simplicity, however, the seismic fields considered in this paper are restricted to simple SH wave models.

The exact solution for the kinematic interaction problem is very complex; analytical solutions are available only for very simple configurations. For other geometries, numerical methods such as finite elements and boundary elements must be used. Although such methods can solve the kinematic interaction problem with enough accuracy, their use is relatively expensive, and an approximate solution is often more desirable. One such method, which was recently proposed by Iguchi (1), provides good approximations to the kinematic interaction problem, even for embedded foundations. However, this method requires knowing a priori the dynamic stiffnesses of the foundation, and this restricts its use, at the moment, to cylindrical and rectangular foundations.

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However, z_i depends on the various modes of the system as well as on the transfer functions due to kinematic interaction. Hence, the total characteristics of the system must be known before computing S_{yi}.

A simplified case arises when the system has well separated natural frequencies, so that the product $h_j h_{j'}$ can be neglected for pairs of different modes $j \neq j'$; $|z_i|^2$ can then be written as

$$|z_i|^2 = \sum_{j=1}^{n} \phi_{ij}^2 \; |x_j|^2 \tag{9}$$

where from eq. 6a

$$|x_j|^2 = |h_j|^2 \Big[\sum_{\ell=1}^{6} \gamma_{j\ell}^2 |t_{\theta\ell}|^2 + \sum_{\ell=1}^{6} \sum_{k=\ell+1}^{6} 2\gamma_{j\ell}\gamma_{jk} \mathrm{Re}(t_{\theta\ell} t_{\theta k}^*) \Big] \tag{10}$$

$t_{\theta k}^*$ being the complex conjugate of $t_{\theta k}$ and Re designating the real part of the complex quantity.

Until now it has been assumed that the dynamic excitation is due to a single SH-wave, arriving from a defined direction, and having a prescribed spectral density function S_ω. A more real situation is to consider the seismic waves arriving from several directions simultaneously. Thus, the above formulation must be extended to this case. Waves arriving at different angles will have experienced different travel times and travel paths, so that their cross spectral density functions cannot be significant. Hence, one can assume that the cross spectral density functions between two waves will be zero and that the same will be true for the motions that they induce.

If there are m uncorrelated waves arriving at different angles $\theta_1 \ldots\ldots \theta_m$, then for a one dof system

$$S_y = \sum_{i=1}^{m} S_{\theta i} \tag{11}$$

Introducing eqs.8 and 10 into eq.11, interchanging summations and assuming for simplicity that the waves coming at the various angles have spectral density functions exhibiting the same variation with frequency, although not necessarily the same amplitude ($S_{\theta i} = g_{\theta i}^2 \; S_\omega$), then

$$S_y = |h|^2 \Big[\sum_{\ell=1}^{6} \gamma_\ell^2 \sum_{i=1}^{m} |t_{\theta_i \ell}|^2 g_{\theta i}^2 + \sum_{\ell=1}^{6} \sum_{k=\ell+1}^{6} \gamma_\ell \gamma_k \sum_{i=1}^{m} g_{\theta i}^2 \; 2\mathrm{Re}(t_{\theta_i \ell} t_{\theta_i k}^*) \Big] S_\omega \tag{12}$$

The quantities $\sum_{i=1}^{m} |t_{\theta_i \ell}|^2 g_{\theta i}^2$ and $\sum_{i=1}^{m} g_{\theta i}^2 2\mathrm{Re}(t_{\theta_i \ell} t_{\theta_i k}^*)$ can be computed for all possible values of ℓ and k, independently of the characteristics of the structure. They depend only on the intensity and angle of arrival of the SH-waves and the geometry of the foundation.

Interpretation of Some Results

The formulation above was applied to the case of an embedded cylindrical foundation with a degree of embedment E/R=1, E being the height of the sidewalls and R the radius of the foundation. Referring to eq.12 , the values of the transfer functions due to kinematic interaction $t_{\theta i \ell}$ are computed using Iguchi's approximation. It will be assumed that the waves arrive continuously between two extreme directions and that their travel paths are in the same vertical plane. Waves arriving from any azimuth could easily be considered by referring the resulting motion of the foundation, for each wave, to the global axis; however, this would introduce too many variables in the problem and obscure its understanding.

In order to compare results for each case, it is convenient to normalize them in such a way that the sum of the squares of each wave's intensity equals

By using such a method, the approximate response of the foundation can be computed, and the results compared, for different wave patterns. The stochastic response of the foundation can then be computed by assuming each single wave to have a prescribed spectral density function. In the more general case, cross spectral density functions between pairs of different waves must be taken into account as well.

STOCHASTIC RESPONSE OF STRUCTURES

Formulation

Assuming the foundation to be rigid, its motion can be described by a vector, $\underset{\sim}{u}_f$, with 6 components, namely 3 displacements and 3 rotations :

$$\underset{\sim}{u}_f^T = [u_{fx}\ u_{fy}\ u_{fz}\ u_{fR_x}\ u_{fR_y}\ u_{fR_z}] \tag{1}$$

For a structure with lumped masses, the dynamic equation of motion can be written in matrix form as

$$\underset{\sim}{M}\ddot{\underset{\sim}{Y}}+\underset{\sim}{C}\dot{\underset{\sim}{Y}}+\underset{\sim}{K}\underset{\sim}{Y} = -\underset{\sim}{M}\underset{\sim}{E}\ddot{\underset{\sim}{u}}_f \tag{2}$$

where $\underset{\sim}{M}$, $\underset{\sim}{C}$ and $\underset{\sim}{K}$ are the mass, damping and stiffness matrices of the structure; $\underset{\sim}{E}$ is a geometric transformation matrix representing the rigid body displacements in the structure due to unit displacements of the foundation (small rotations assumed); $\underset{\sim}{Y}$ represents the relative displacements of the structure with respect to the ground; and $\ddot{\underset{\sim}{u}}_f$ is the foundation acceleration vector. In a solution in the frequency domain, it is assumed that $\underset{\sim}{u}_f$ is harmonic, of the form

$$\underset{\sim}{u}_f = \underset{\sim}{T}_\theta u_{g0} e^{i\omega t} \tag{3}$$

in which u_{g0}=1= the displacement at the free surface in the free field; and $\underset{\sim}{T}_\theta=\{t_{\theta\ell}\}$=the transfer function vector relating the free-field motion, produced by wave-trains propagating along direction θ , and the foundation motion (i.e., the solution to the kinematic interaction problem). Substituting eq.3 into eq.2, one obtains (with $\underset{\sim}{Y} = \underset{\sim}{Z}\, e^{i\omega t}$)

$$(-\omega^2\underset{\sim}{M}+i\omega\underset{\sim}{C}+\underset{\sim}{K})\ \underset{\sim}{Z} = \omega^2\underset{\sim}{M}\underset{\sim}{E}\underset{\sim}{T}_\theta \tag{4}$$

Using the modal transformation $\underset{\sim}{Z} = \underset{\sim}{\Phi}\underset{\sim}{X}$ (with Φ being the modal matrix of the undamped structural system , normalized so that $\Phi^T\underset{\sim}{M}\Phi = \underset{\sim}{I}$ = the identity matrix), and assuming a proportional damping matrix, it follows that

$$(-\omega^2\underset{\sim}{I} + 2i\underset{\sim}{B}\underset{\sim}{\Omega} +\underset{\sim}{\Omega}^2)\ \underset{\sim}{X} = \omega^2\underset{\sim}{\Gamma}\underset{\sim}{T}_\theta \tag{5}$$

in which Ω = diag(ω_j) = the natural frequencies of the system; $\underset{\sim}{B}$ = diag(β_j) = modal damping matrix; and $\underset{\sim}{\Gamma} = \{\gamma_{j\ell}\}$ = matrix of participation factors. For each modal component, the transfer functions are then obtained from eq.5 as

$$x_j = h_j \sum_{\ell=1}^{6} \gamma_{j\ell} t_{\theta\ell} \tag{6a}$$

in which

$$h_j = \frac{\omega^2}{\omega_j^2+2i\beta_j\omega\omega_j-\omega^2} \tag{6b}$$

while the transfer functions for relative displacements are

$$z_i = \sum_{j=1}^{n} \phi_{ij} h_j \sum_{\ell=1}^{6} \gamma_{j\ell} t_{\theta\ell} \tag{7}$$

with n= total number of degrees of freedom.

Generalizing the formulation to the case of a SH-wave with a prescribed spectral density function S_ω, the spectral density function of y_i can be computed as

$$S_{yi} = |z_i|^2\ S_\omega \tag{8}$$

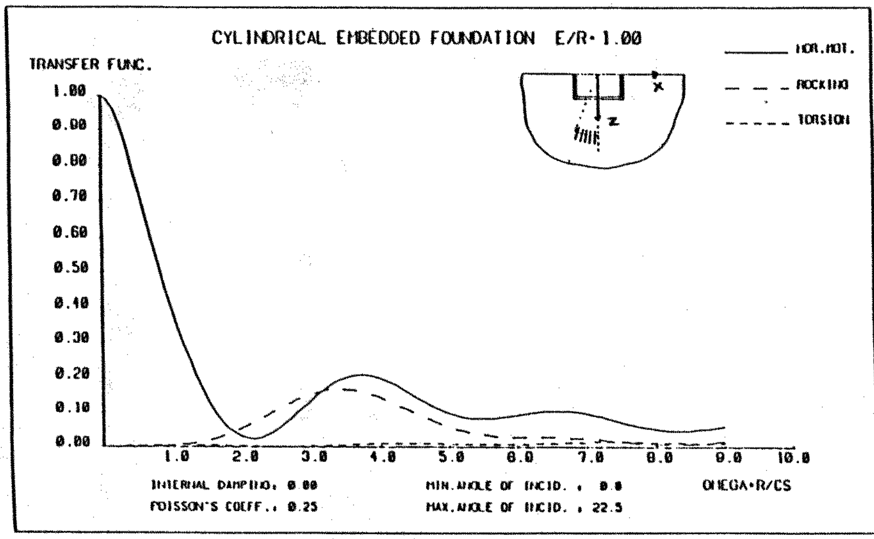

Fig. 1

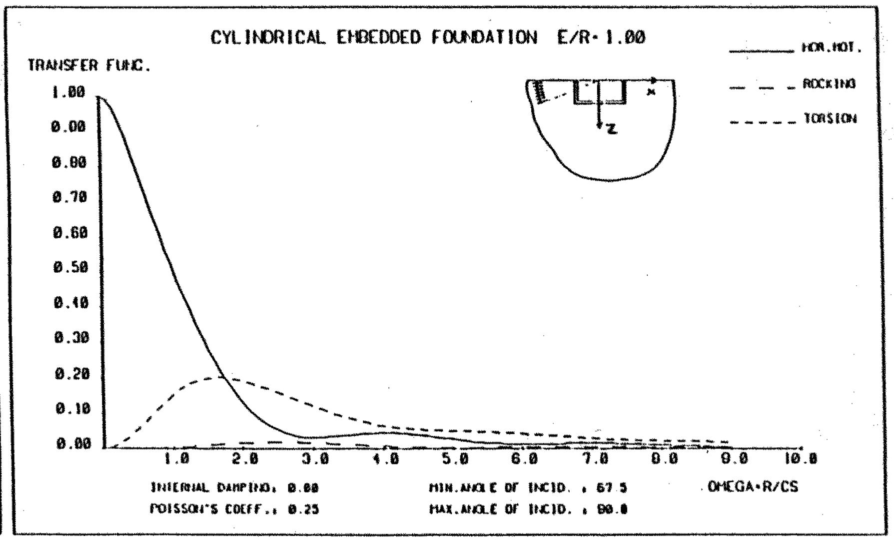

Fig. 2

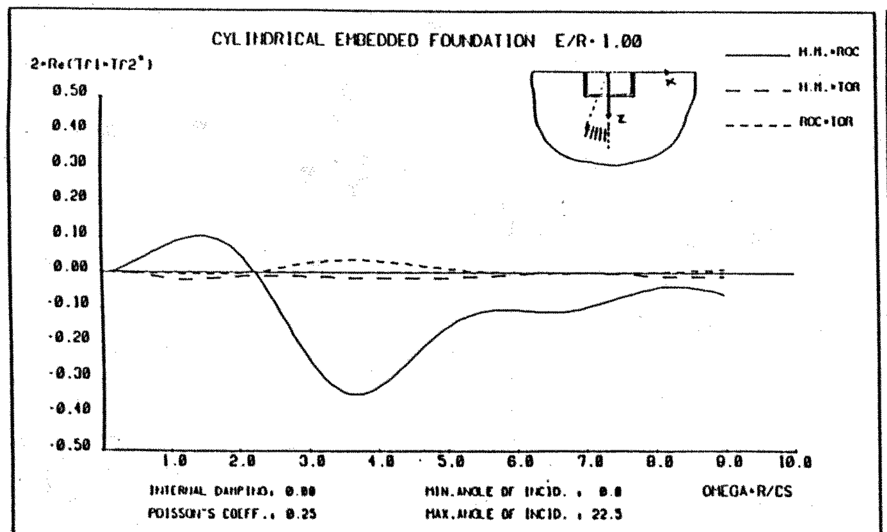

Fig. 3

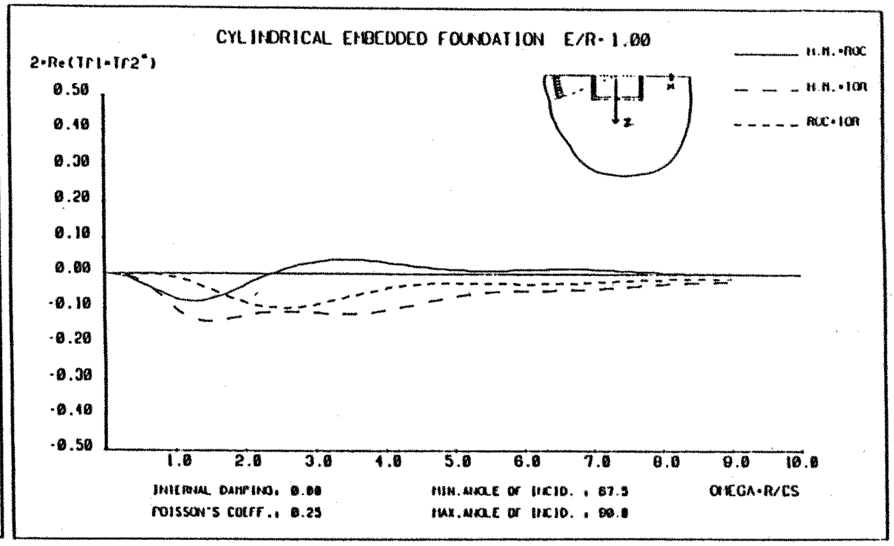

Fig. 4

unity, $\sum_{i=1}^{\omega} g_{\theta i}^2=1$. Also it is assumed that the waves arrive uniformly within a chosen sector, which is discretized into 100 different directions to model approximately waves arriving continuously along all the sector. A Poisson's coefficient of 0.25 is chosen, as well as zero internal damping in the soil. The motions are referred to the center of the bottom of the foundation. The rocking and torsional response functions have been multiplied by R to express them in dimensionless form.

Figures 1 through 4 display the results for the various terms present in eq. 12 as a function of the nondimensional frequency a_o.($a_o = \omega R/Cs$; ω=angular frequency of the motion, R=radius of the foundation, Cs=shear wave celerity in the soil). Comparing Figs. 1 and 2 it can be seen that shallowly arriving waves induce higher torsion and less rocking in the foundation than steep waves, as could be expected by looking at the free field motion. The rate of decay in the low frequency range of the mean square transfer function for the horizontal displacement of the foundation does not depend significantly on the direction of arrival of the waves. In the high frequency range the horizontal translation is mainly due to the waves arriving vertically.

Analysing Figs. 3 and 4, it can be seen that only the cross terms between horizontal motion and rocking (full line) presents important values, explained by the coupling between their stiffnesses and by the fact that the reference axes are placed at the bottom of the foundation. Vertically propagating waves are the ones that contribute the most to this cross term, as shown in Fig. 3. Shallow waves give higher values for the cross terms involving torsion, because they induce more torsion than steep waves. It should be noted that, even though negative values are found for these cross terms of eq. 12, the overall result will always be positive, as expected. It is important to note that these cross terms cannot, in general, be neglected even for high frequencies and should not be discarded from eq. 12 to obtain the Spectral Density function S_y.

INFLUENCE OF KINEMATIC INTERACTION IN A SIMPLE STRUCTURE

Filter Functions

A simple structural system is studied to assess the influence of kinematic interaction and angle of arrival of the seismic waves on the dynamic response of the system.

The structure is idealized as a single degree of freedom system, i.e., a point mass connected to a cylindrical embedded foundation by a massless column (see Fig. 5). The mass of the foundation is also neglected so that the only dynamic degree of freedom of the system is the horizontal displacement of the structural mass. In this case the torsion induced on the foundation will have no effect on the motion of the mass; hence the terms associated with torsion in eq. 12 vanish. Also, due to the axisymmetry of the structure, the rocking and horizontal translation have only one component in the direction of the free-field motion.

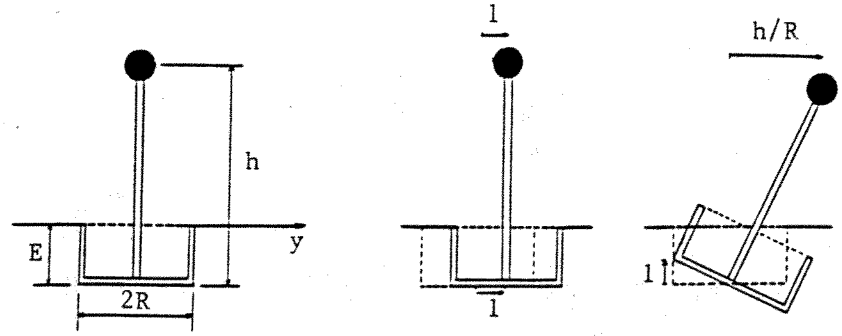

Fig. 5 - Structural system and induced motions

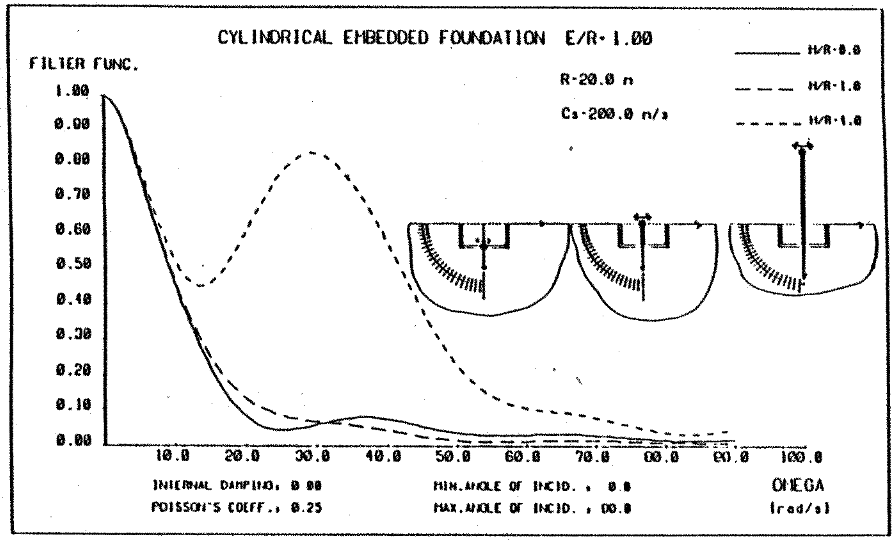

Fig. 6

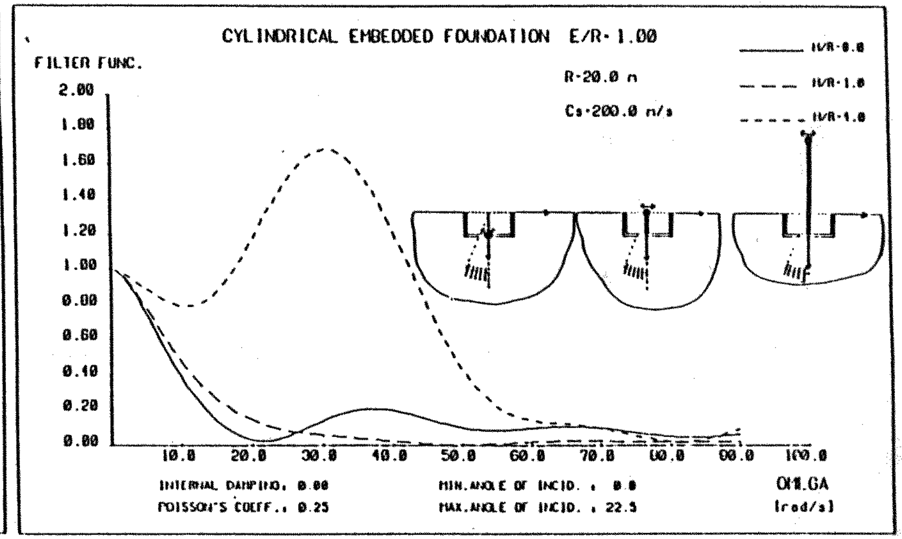

Fig. 7

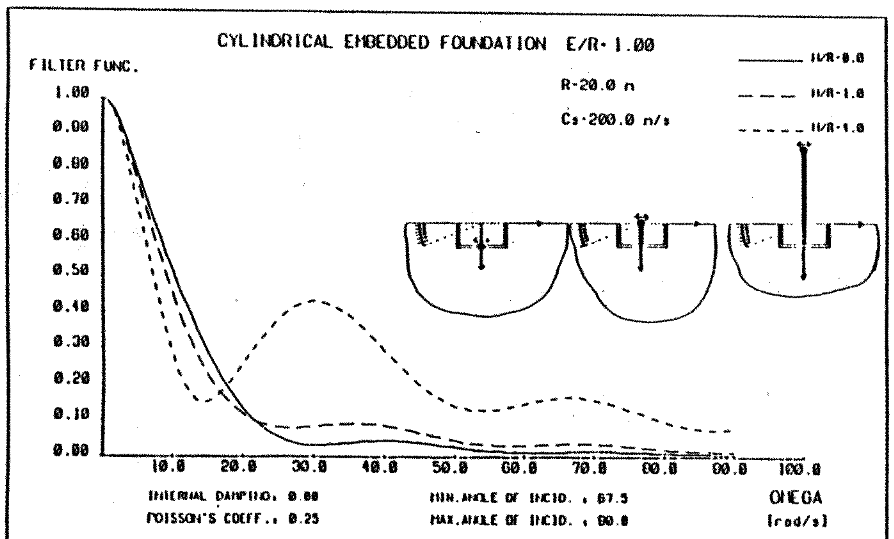

Fig. 8

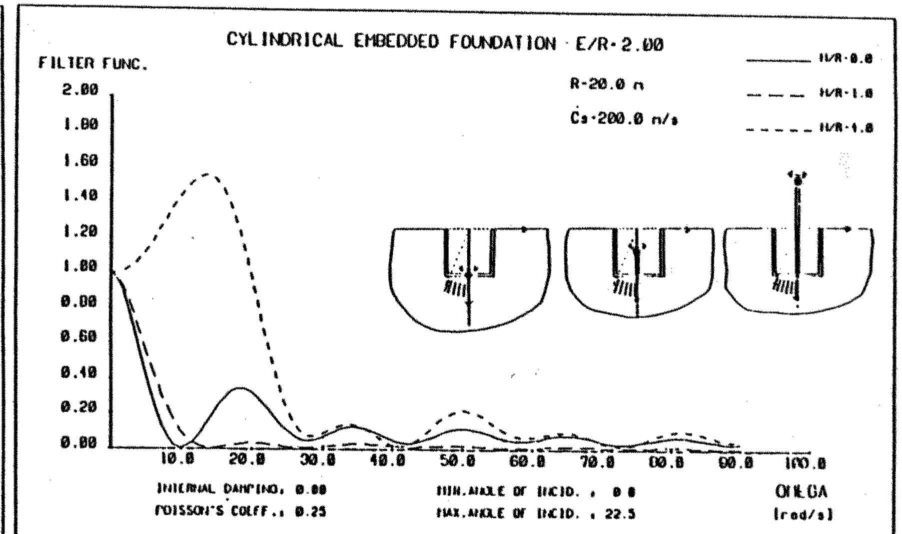

Fig. 9

In the examples analysed it is assumed that the foundation has a radius of 20 m, and the celerity of the SH-waves, Cs, is 200 m/s. However, the same results would have been obtained for any other combination of physical parameters satisfying the ratio R/Cs=0.1. Three wave patterns are considered: a broad incidence range with waves arriving uniformly within the first quadrant; a steep incidence range; and a shallow incidence range. Two different degrees of embedment are analysed: E/R=1 and E/R=2.

From Figs. 6-9, it is concluded that for h/R=4 the influence of the rocking is very pronounced especially for vertically propagating waves. Other interesting conclusions, comparing Figs. 7 and 9, are that the frequency at which the motion is more amplified decreases with the degree of embedment, as well as its amplification value. However, these two cases cannot be compared directly because the height h is taken from the bottom of the foundaiton and not from the ground surface. As taller structures usually have lower natural frquencies and their degree of embedment is more important, then kinematic interaction is likely to magnify the dynamic response of such systems and should, therefore, be taken into account. For high freequencies, the dynamic response, in all cases decreases substantially, due to kinematic interaction.

<u>Response Spectra</u>

In practice the Spectral Density function of the response is not a meaningful quantity to the designer, who, in general may want to know the maximum expected value of that same reponse.

Response Spectra are widely used in Seismic Design, and they are more readily understandable by the design engineer than Spectral Density functions - the maximum response induced in simple oscillators by the motion considered. Response Spectra are normally displayed as a function of the natural freauency of the oscillator and for a given amount of modal damping. It is, therefore, interesting to study also the influence of kinematic interaction on the Response Spectra.

Using approximate formulas developed by Der Kieureghian (3,4) (and based on the work by Vanmarcke (5)) for the maximum expected value of a Gaussian random process during a given interval, the Response Spectra of the 1-dof oscillator in fig. 5 was obtained for a realistic spectral density function of the free field ground motion. A duration of motion of 100 sec. was chosen as well as 5% of critical damping for the oscillator.

The Response Spectra for the relative displacement of the oscillator S_d, are plotted in Figs. 10-13. The full line is obtained neglecting kinematic interaction, while the dashed line represents the Response Spectra derived from the spectral density function, S_y. Comparing both curves, the influence of kinematic interaction can be evaluated. S_v and S_a represent the pseudo-spectra of velocity and acceleration. For lightly damped systems, S_a can be interpreted as the Reponse Spectrum of the absolute acceleration of the oscillator.

From Figs. 10-13, it can be seen that kinematic interaction usually reduces substantially the maximum response of the systemexcept when the natural freauency of the oscillator is less than ≈0.5 Hz (for this situation), in which case the difference between the two Response Spectra is small. However, for tall structures with deeply embedded foundations and vertically propagating waves, the Response Spectra may increase as a result of kinematic interaction (see Fig. 13), as could be anticipated from the results in Figs. 6-9.

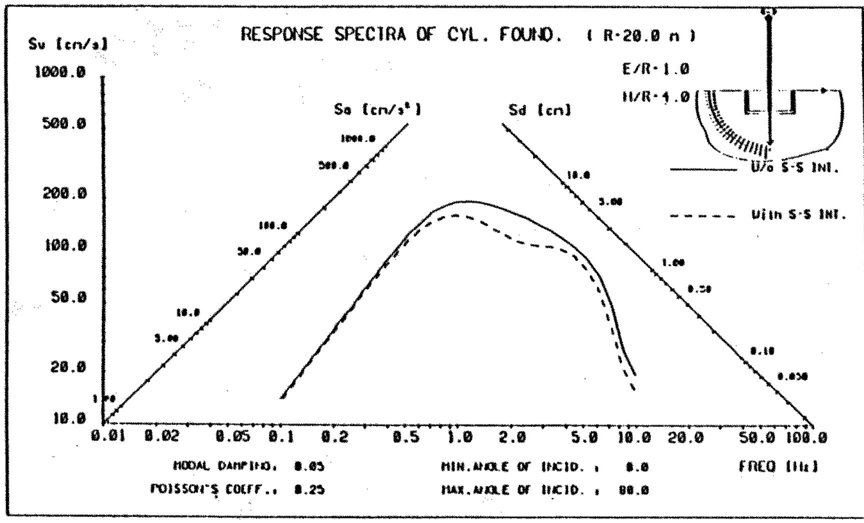

Fig. 10

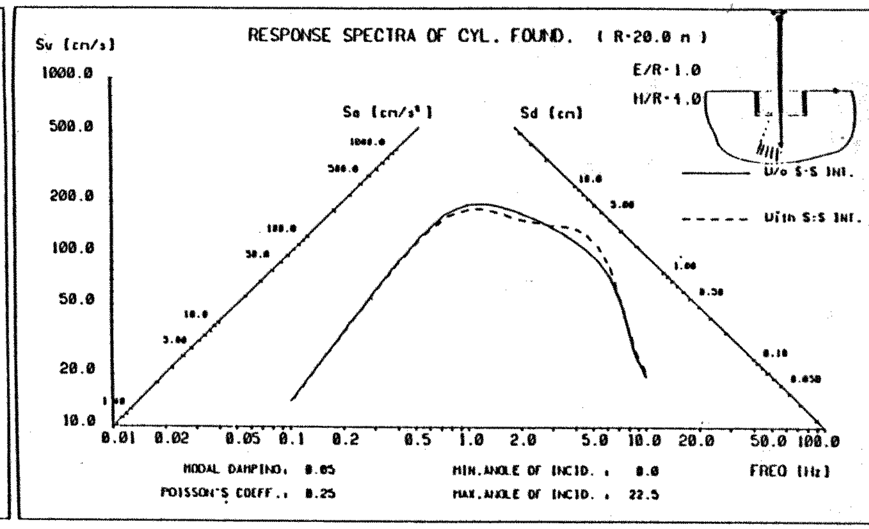

Fig. 11

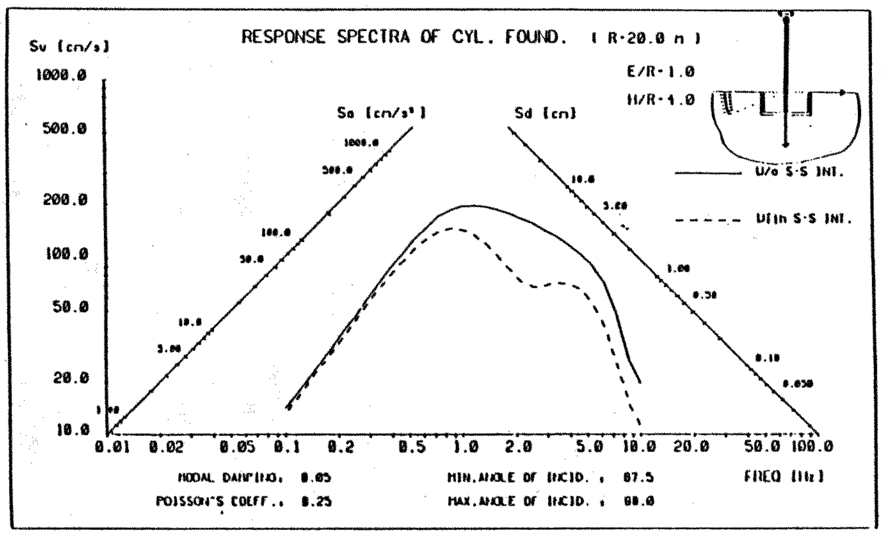

Fig. 12

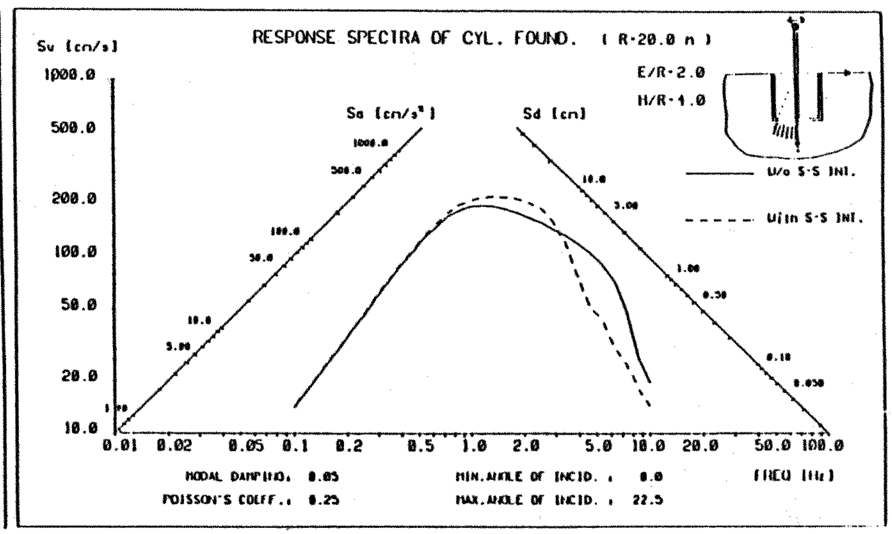

Fig. 13

CONCLUSIONS

By using approximate procedures, the influence of kinematic interaction was evaluated for structures with rigid foundations when subjected simultaneously to several obliquely incident SH-waves. For numerical simplicity it was assumed that waves arriving from different directions are completely uncorrelated as well as the response in the various natural modes of the structure. However, the formulation permits the incorporation of some degree of correlation between the seismic waves and also between different modes.

It is seen that the kinematic interaction reduces the translation transmitted to the structure, and amplification can be observed in tall structures due to the rocking. These conclusions are also valid for the influence of kinematic interaction on the maximum response of the structure.

Although only SH-waves and very simple examples are used in this study to illustrate the possible implications of kinematic interaction, this formulation can easily be extended to other types of waves. It is shown how simple approximations can be of help in the understanding of the complex kinematic interaction problem and, at least for a preliminary design, be used to estimate the effective seismic input.

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SESSION 5

EXPERIENCE AND EXPERIMENTAL OBSERVATION

EPRI Research on Soil-Structure Interaction

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3412 Hillview Avenue
Palo Alto, CA 94303

Proceedings
Workshop on Soil-Structure Interaction

June 16-18, 1986
Bethesda, Maryland

Sponsored by

U.S. NRC Office of Research

EPRI Research on Soil-Structure Interaction

H. T. Tang*

Background

When seismic waves impinge on an embedded structure, two physical effects occur simultaneously: (1) seismic waves incident on the structure are diffracted and reflected back into the foundation medium, and (2) motion is transmitted to the structure. The first effect, referred to as kinematic interaction, is independent of the inertial properties of the structure. The second is the dynamic response of the coupled structure-foundation system which is governed by the inertial properties of the structure and foundation impedances. These two effects together are nomally referred to as soil-structure interaction (SSI).

There are currently two accepted analytical approaches to predicting SSI; half-space (or substructure) and direct. Each approach has known limitations. In the absence of comprehensive large-scale data, a preferred or better approach cannot be conclusively identified.

In SRP 3.7.2, NRC requires that SSI computations be carried out using both the half-space approach and the finite element method. The accepted results must envelop the results of separate analysis. Alternatively, an acceptable demonstration of conservatism must be given if enveloping is not used. However, in the absence of and data of clearly defined criteria for an acceptable alternative, most applicants choose to use the enveloping

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analysis. This is clearly conservative.

Strategic Need

Strategically, the nuclear utility industry needs a realistic, experimentally qualified SSI analysis methodology for nuclear plant design to reduce excessive conservatism and stabilize the licensing process, and which is acceptable to NRC.

To achieve the goal of having well-defined, realistic guidelines and acceptance criteria for SSI in SPR 3.7.2, a coherent SSI analysis practice needs to be developed. In order to test alternative SSI models and to validate an acceptable model, field earthquake data (on structure, free-field, downhole, etc.) must be collected.

EPRI Research

EPRI sponsored research to satisfy the needs in the SSI area can be divided into three phases. In the first phase, simulated experiments using explosive induced ground motions in a medium soft soil environment were conducted [1, 2]. Detailed discussions of this series of testing, SIMQUAKE I and II, is given in a separate paper presented in this workshop by C. J. Higgins. Other phases of research and their significance are described in the following.

Niagara Mohawk/EPRI Research

The second phase of research was conducted jointly with Niagara Mohawk Power Cooperation [3]. Both forced vibration ("shaker") and ground motion tests were performed on four test structures. Three sequentially detonated planar arrays of buried explosives were used to create an "earthquake like" ground motion input for the test structures, Figure 1. The structures and surrounding rock were instrumented, Figures 2 and 3, so that the experiment could be numerically simulated either as a base motion input at a structure model or by including an "island" of rock with known (measured) boundary inputs.

The major difference between the SIMQUAKE I and II and the Niagara Mohawk testing is the soil foundation property. In the Niagara Mohawk testing, model structures were constructed with their foundations set in rock sockets. The material properties based on boring holes at the test site are given in Table 1. The two rectangular structures tested (1/10- and 1/20-scale) were designed based loosely on Niagara Mohawk Power Corporation's Nine Mile Point Unit 1 Containment building and the two cylindrical models were similar in design to those used in the SIMQUAKE I and II experiments. The latter two were tested to provide data for different site conditions.

Unlike the SIMQUAKE I and II test results, which show significant frequency variations, the structure frequencies observed during the explosive test (strong ground motion) correspond reasonably well with those reported for the forced vibration tests (low level exciation) as shown in Table 2. The explosive test frequencies tend to be on the low end or slightly lower than the ranges of sine dwell test frequencies. With the exception of the 1/20-scale rectangular structure, all of the structures exhibited approximately 10% damping on the ring-down of the explosive test as given in Table 3. This is somewhat higher than the 3.5% to 7% damping obtained from the forced vibration tests. The 1/20-scale rectangular structure damping was found to be approximately 3.5% for both the explosive test and most of the forced vibration tests.

Overall speaking, the Niagara Mohawk test results show small SSI effect for the rock sites. The data base supplements the SIMQUAKE I and II data base for providing variations needed in benchmarking computer simulation procedures for SSI analysis. Table 4 summarizes the major findings of the three simulated earthquake experiments.

EPRI/Taipower Research

In the simulated earthquake experiments discussed in the above, the detonation of vertical arrays of explosives propagated wave motions through the ground to the model structures. Although such a simulation can provide information about dynamic SSI characteristics in a strong motion environment, it lacks seismic wave scattering characteristics for studying seismic input to the soil-structure system and the effect of different kinds of wave composition to the soil-structure response.

To supplement these simulated earthquakes, EPRI, with the cooperation of the Taiwan Power Company, designed and constructed a 1/4-scale and a 1/12-scale model containments in Lotung, Taiwan. The location is a seismically active one where the University of California at Berkeley under grant to National Science Foundation and the direction of professors B. Bolt and J. Penzien has deployed a two-dimensional strong motion array (SMART-1), Figure 4, to collect seismological data. The soil in Lotung is relatively soft which is of particular importance in view of the strong SSI in this environment.

The 1/12-scale model constructed has a cylindrical configuration and is the same as the ones tested in the SIMQUAKE and Niagara Mohawk experiments. The purpose of installing this model is to obtain actual earthquake-induced data that can be directly compared with the simulated earthquake experimental results.

In designing the 1/4-scale model, one of the major considerations was to have the maximum amplification in the actual earthquake-induced strong motion environment. Since the dominant frequencies of earthquakes in Lotung based on the SMART-1 data were in the range of 3 to 8 Hz, the 1/4-scale model was designed to have the fundamental frequency around 6 Hz. To obtain data on internal component response, a mocked-up steam generator and a pipe run were designed within the 1/4-scale model.

Both the free field and the models were instrumented as shown in Figures 5-7. The free-field instrumentation has three linear surface arrays radiating from the 1/4-scale model with a 150-foot radius and two downhole arrays to a 150-foot depth with one beneath the model and the other in line with the outer edge of the surface array. Strong motion accelerometers were installed on the basemat and near the top of the 1/12- and 1/4-scale models to record the foundation basemat motion and dynamic structural amplification. Also installed underneath the models were pressure gages to monitor uplifting and bonding-debonding between soil and structure during strong motion earthquakes. Since the models are located within the U.C. Berkeley strong motion array, valuable seismological and geotechnical information for the Lotung region are available to benefit the SSI study.

Forced vibration tests sponsored by NRC have been conducted. Five major earthquakes ranging from Richter magnitude 5.3 to 6.5 with the maximum peak ground acceleration about 0.27 g were recorded from September 1985 to May 1986. The Lotung experiment has yielded a data base significant for SSI analysis verification and development. Currently, EPRI, Taipower and the U.S. NRC have separately sponsored SSI analysis using the Lotung data for analysis method verification and more quantified SSI understanding. These three efforts will be coordinated with the ultimate goal of coming up with recommended procedures for SRP revision consideration.

Soil properties and their effect on SSI will also be studied in the Lotung project. It has been recognized that laboratory testing for materials such as soil has many deficiencies. However, is in-situ characterization available and dependable or is development needed? Is there enough data to make an assessment between laboratory testing and in-situ testing? Only when these questions are answered, can one further address the sensitivity of soil property in the complete SSI analysis. At the Lotung site, EPRI will cooperate with the U.S. NSF through U.C. Davis under Professor C. K. Shen to obtain soil properties by using the state-of-the-art laboratory and in-situ procedures. Also included will be soil settlement and pore pressure monitoring in the field.

<u>EPRI/CRIEPI Research</u>

Under the technical exchange agreement with the Japanese Century Research Institute for Electric Power Industry, EPRI obtained the forced vibration test data of a full scale reactor building, Figure 8, for SSI investigation [4, 5].

The reactor building was subjected to horizontal harmonic forced vibrations from 1 to 20 cps in the North-South (N-S) direction generated by shakers placed on the refueling floor (El. 46.5m). The steady-state displacement frequency response functions obtained from the test were used as the bases for comparison with the corresponding analytically calculated displacement frequency response functions for validating the analytical SSI models. The lumped parameter representation were constructed following the U.S. industry practice. The lumped parameter SSI models adopted were basically two-dimensional models consisting of a lumped-mass-beam-stick model for the structure and a set of foundation impedances for the foundation medium. The foundation impedances were either frequency-dependent impedance functions or frequency-independent (constant) impedance values. Embedment effect and the coupling between horizontal translation and rocking due to embedment were also considered. About twenty parametric studies covering variations of both the soil and structure parameters were performed. Within the range of uncertainty of each parameter, judged to be reasonable, the best correlation as illustrated in Figures 9 and 10 shows that analysis overpredicted the response amplitudes with conservative margins. The case with the embedment coupling impedance produced responses closer to the test results, indicating the importance of including the coupling impedance for an embedded foundation.

It is interesting to note that for a similar study performed on a 1/15-scale model [5] analysis underpredicted the response for the surface foundation case and provided reasonable prediction for the embedded foundation if the CLASSI type impedances were used.

The phenomenon that the analytical impedances seemingly overestimated the foundation radiation damping for the scaled model and underestimated for the full scale reactor building indicates that model scaling has an effect on the test results. The close similarity between the shapes of test and analytical displacement frequency response amplitudes for both tests indicates that the analytical lumped parameter SSI models considered herein are capable of capturing the SSI dynamic response characteristics with reasonable accuracy, and with conservative margins for the case of prototype structure.

Concluding Remarks

Despite the research and development effort in the past 15 years, the seismic SSI analysis remains as one of the major sources of uncertainty in the design and licensing of nuclear power plants. Although a variety of analysis techniques are presently available, different analysis techniques often result in significantly different responses for the same SSI problem, especially for structures embedded in soil foundations. Due to the lack of methodology validation, the prudent engineering approach often chosen is to perform more than one SSI analysis using different techniques, and envelop the analysis results for design. However, such an approach is deemed to be very conservative. Since the seismic SSI response provides a direct input to the design of plant structures, piping, components, and equipment, the reduction of uncertainties associated with the SSI analysis is a task that should be assigned a high priority.

In order to reduce the uncertainties, the mathematical models and analysis assumptions employed in various SSI analysis techniques must be carefully sorted and validated through systematic correlations with controlled test and/or field observed data. Until recently, very limited work has been done in the U.S. on experimental verification of SSI analysis techniques due to lack of data. Some progress in this direction has been and is being made as evidenced in this workshop. However, the result has not yet been conclusive because a systematic verification of various assumptions has not been performed.

The SIMQUAKE and Niagara Mohawk experiments showed the feasibility of using the explosive induced strong motion to study the dynamic soil-structure system characteristics in particular the part associated with the inertial aspect of the SSI problem. The data base generated certainly can be used to benchmark SSI analysis methodology if it is used in a proper manner. Questions have been raised regarding the large frequency shifting due to bonding-debonding between structure and soil in the SIMQUAKE I and II tests. Whether this behavior has significance for prototypical plant subject to actual earthquakes is an ongoing effort as reported in this workshop and may need further investigation.

The lumped parameter approach following the U.S. industry practice produced conservative simulated test responses for the prototypical reactor building but underpredicted the test response for the reduced scale model structure. An overestimation of foundation radiation damping which appeared for the small-scale model test was not applicable for the full-scale test. It is apparent that the results of a small-scale model test should be carefully studied before applying the results to full-scale structures.

With the availability of the actual earthquake data obtained in the Lotung project, a systematic verification of various assumptions can be performed and one should be able to qualify and improve current SSI analysis methods to provide confidence on the accuracy of the analysis results.

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Table 1

Bulk Density, Ultrasonic Wave Velocities, Maximum Axial Stress, Young's Modulus, and Poisson's Ratio of Samples TH-1 and TH-2

Boring No.*	Depth (ft)	Bulk Density (g/cc)	Ultrasonic Wave Velocities (ft/sec)		Maximum Axial Stress at Failure (psix10^3)	Young's Modulus (psix10^6)	Poisson's Ratio
			Shear	Compression			
TH-1	27.3-27.9	2.64	8591	12,150	20.3	4.5	0.12
TH-1	41.3-42.2	2.54	8536	12,096	22.2	4.9	0.19
TH-2	10.3-11.4	2.56	8641	12,447	29.6	5.7	0.15
TH-2	36.0-36.6	2.59	8186	11,246	25.4	4.4	0.17

Table 2

COMPARISON OF FIRST MODE FREQUENCIES FROM EXPLOSIVE AND FORCED VIBRATION TESTS

Structure	Frequency (Hz) Explosive Test(1)	Frequency (Hz) Forced Vibration Tests(2)
1/10-Scale Rectangle	36	37.8 - 45.0
1/20-Scale Rectangle	74	83.0 - 90.0
1/12-Scale Cylinder Without Rock Bolts	36	34.4 - 36.0
1/12-Scale Cylinder With Rock Bolts	39	40.0 - 43.6

(1) Derived from transfer functions of explosive test data.

(2) The range of frequencies derived from the various tests with the same backfill conditions as reported in Section 4.4.

Table 3

LOG DECREMENT DAMPING VALUES DERIVED FROM THE EXPLOSIVE TEST DATA

Structure	1st Mode Response Amplitude (g)	Damping (%)
1/10-Scale Rectangle	2.5	10
1/20-Scale Rectangle	6	3.5
1/12-Scale Cylinder Without Rock Bolts	4	10
1/12-Scale Cylinder With Rock Bolts	1.5	10

Table 4

SIMQUAKE AND NMPC TEST COMPARISON

	Site Condition (V_x)	Model Size & Shape	Explosive Array Design	Peak Response (g) at 1/12 models		Frequency Variations of 1/12 models (Hz)		Comments
				Free Field	Top Structure	Forced Vibration	Explosive	
SIMQK IA, IB	Med/soft sandy soil (2000 fps)	1/48, 1/24, 1/12 cylinders	Single array	1.1	2	12	3.5	Significant frequency downshift due to SSI
				3.1	5.5	15	6.5	
SIMQK II	Med/soft sandy soil (2000 fps)	1/24, 1/12, 1/8 cylinders	Double array (1/2 sec. apart)	1.4-2.3	4-6	13-16	6.0	
NMPC	Stiff sandstone (8500 fps)	1/12 cylinders 1/20, 1/10 rectangles (all with rock sockets)	Triple array (each 0.075 sec. apart)	7 *	11 *	40-43 *	39 *	Minor SSI effect
				10 **	19 **	34-36 **	35 **	Insignificant SSI effect

* With rock bolts and backfill.

** Without rock bolts and with backfill.

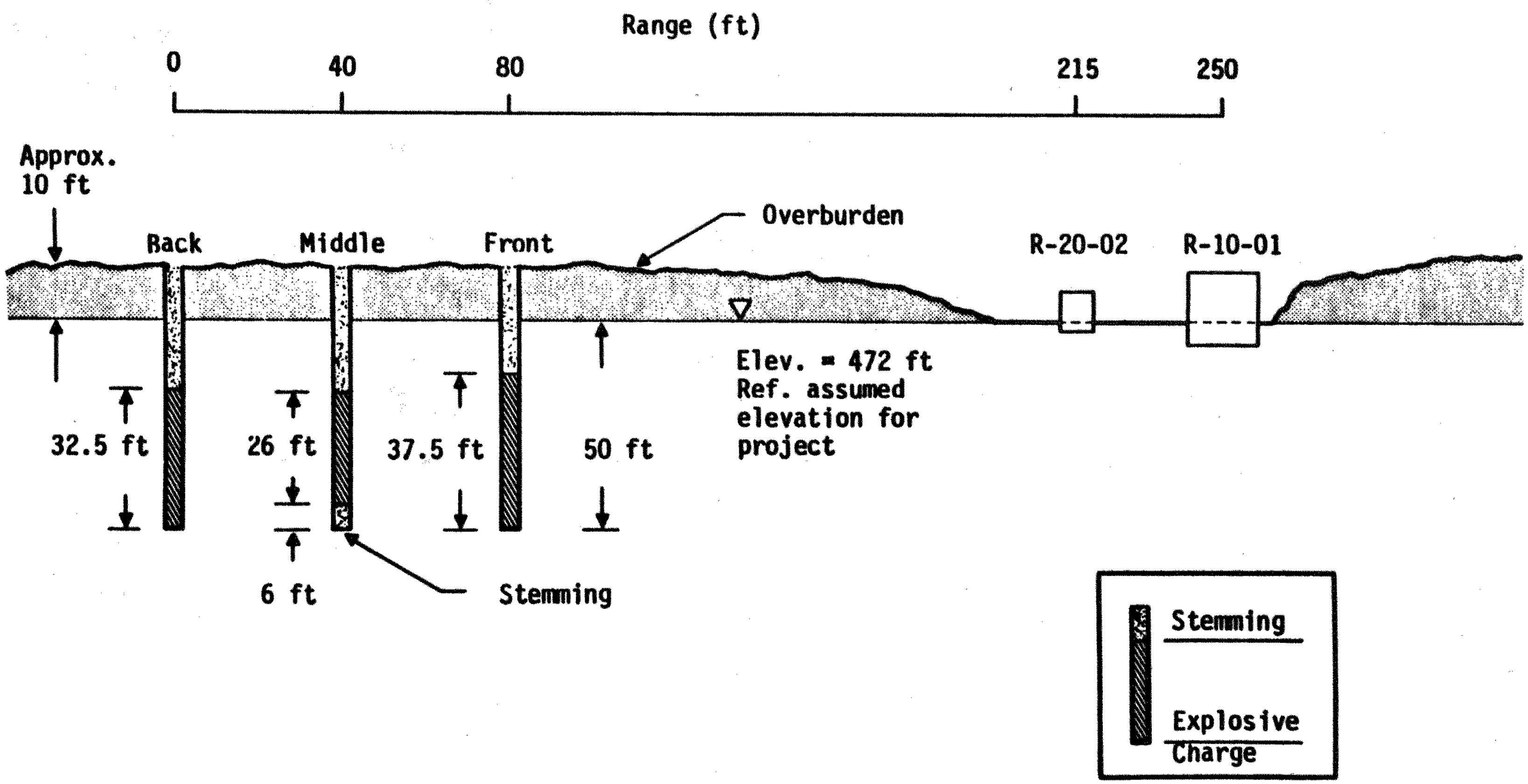


Figure 1. Profile View of the Explosive Arrays As-Built

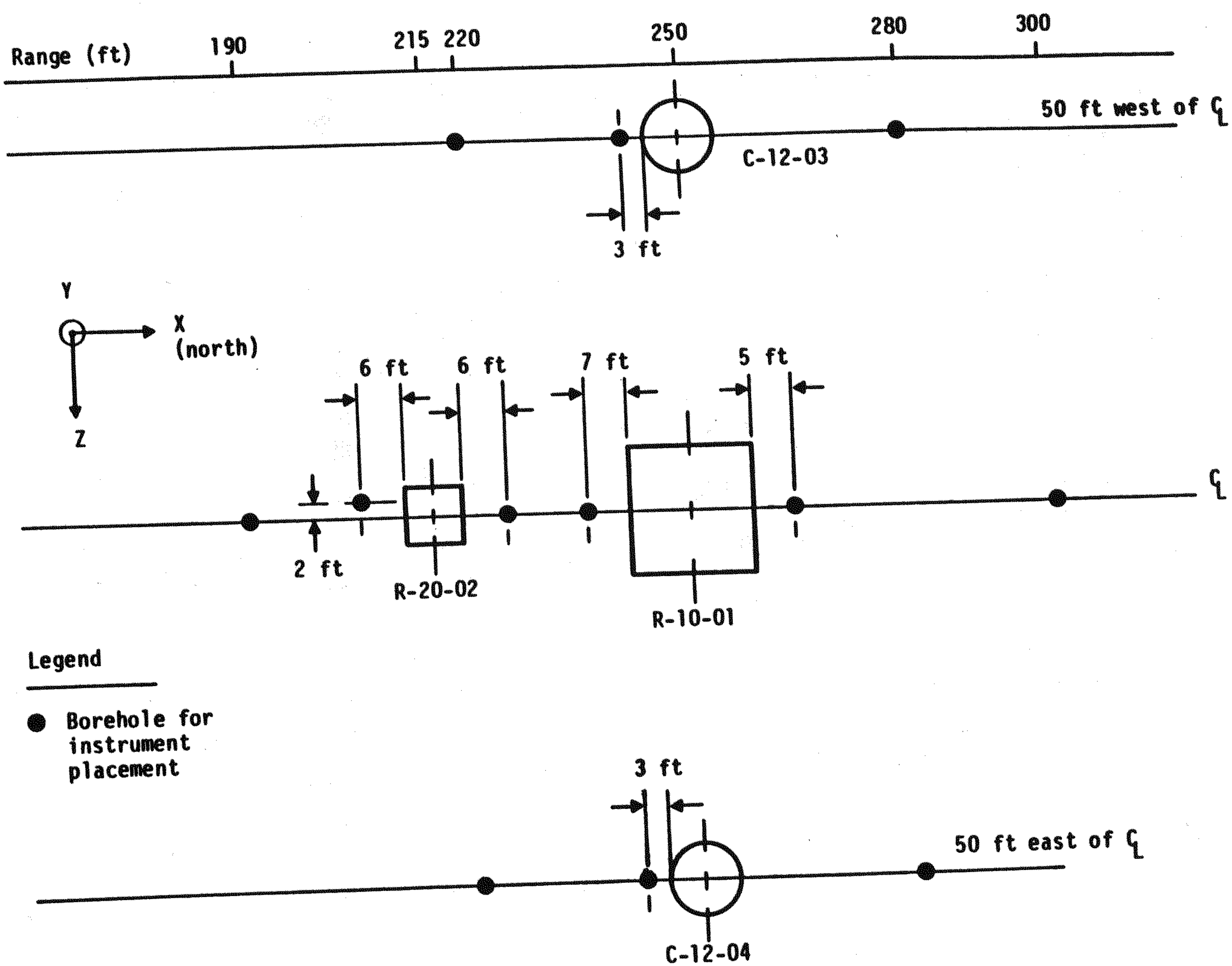
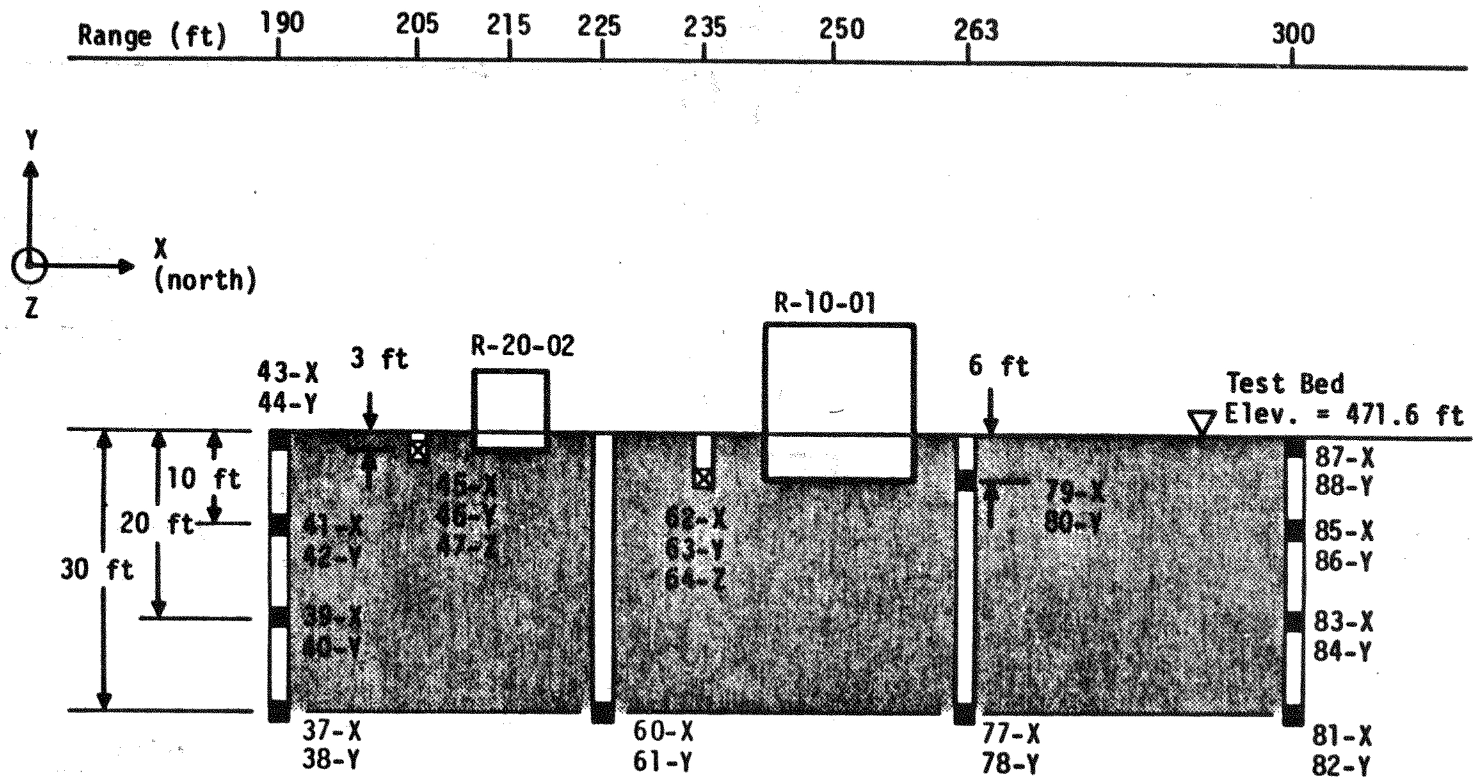


Figure 2. Plan View of Borehole Locations for Field Instruments



Legend

■ Biaxial accelerometer array (X, Y)

⊠ Triaxial accelerometer array (X, Y, Z)

Note: The labels on each instrument symbol denote the channel number followed by the direction.

Figure 3. Profile View of the Field Instrument Locations Along the Rectangular Structure Line

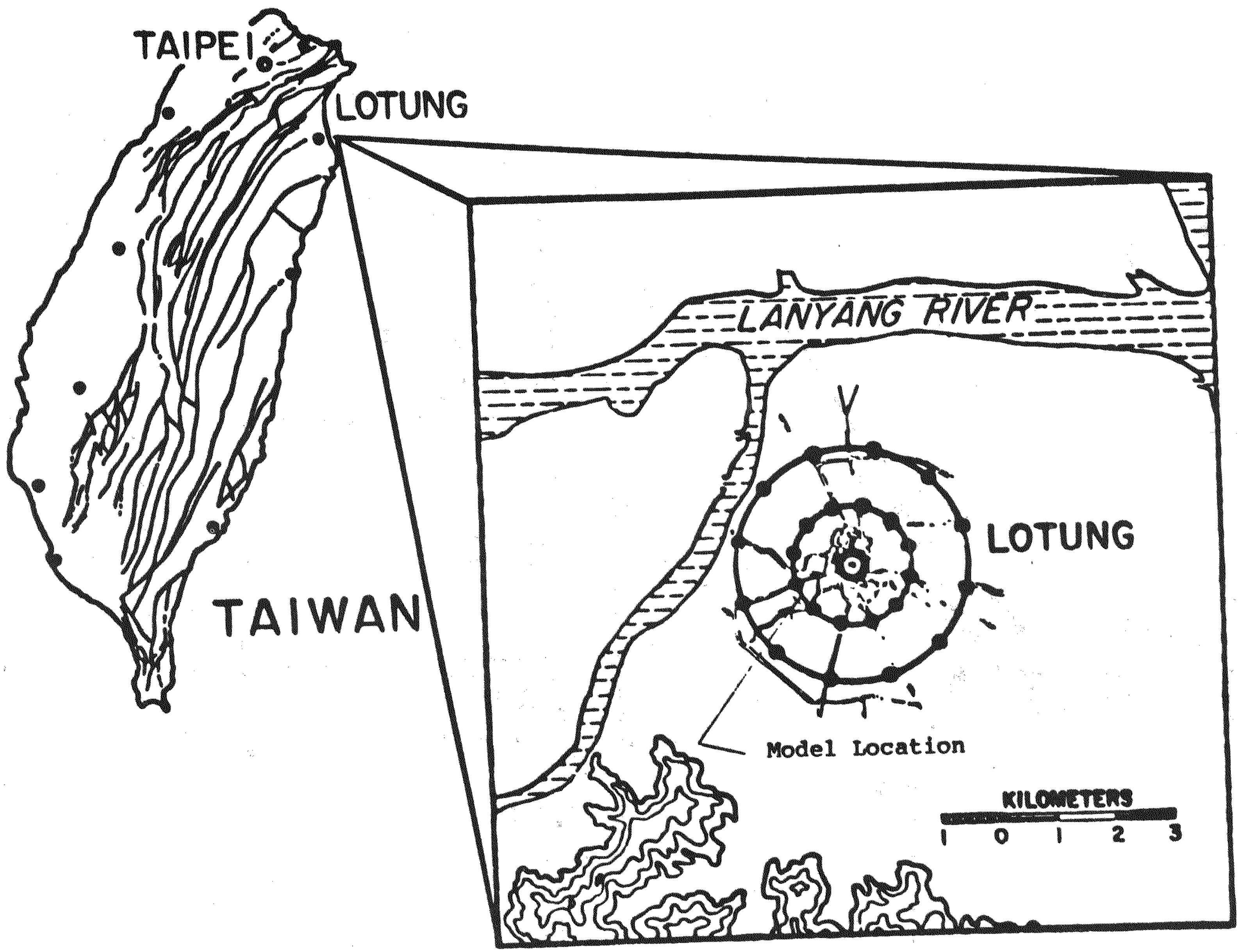


Figure 4. Strong Motion Array

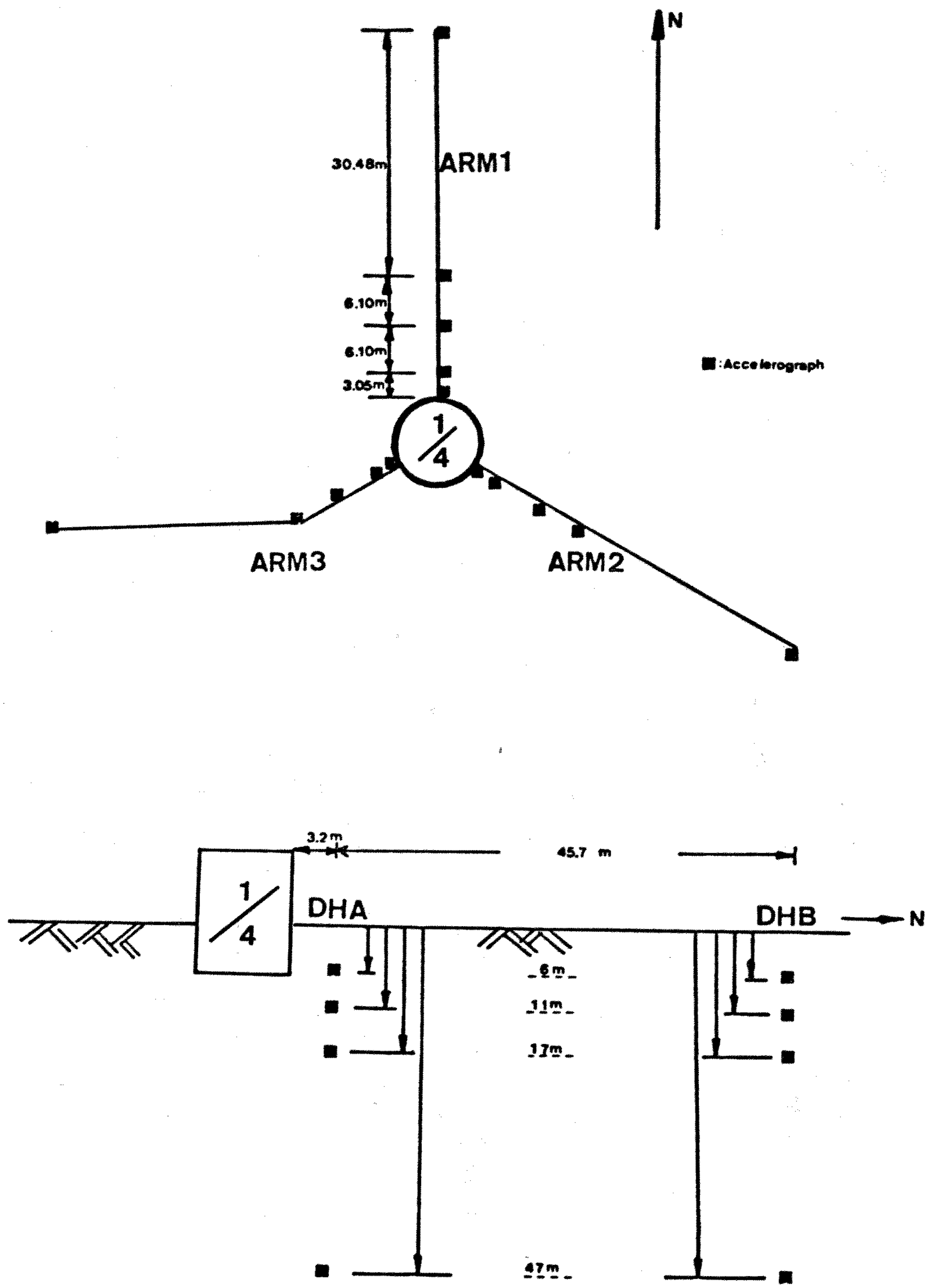


Figure 5. Surface and Downhole Instrumentations for the Lotung SSI Project

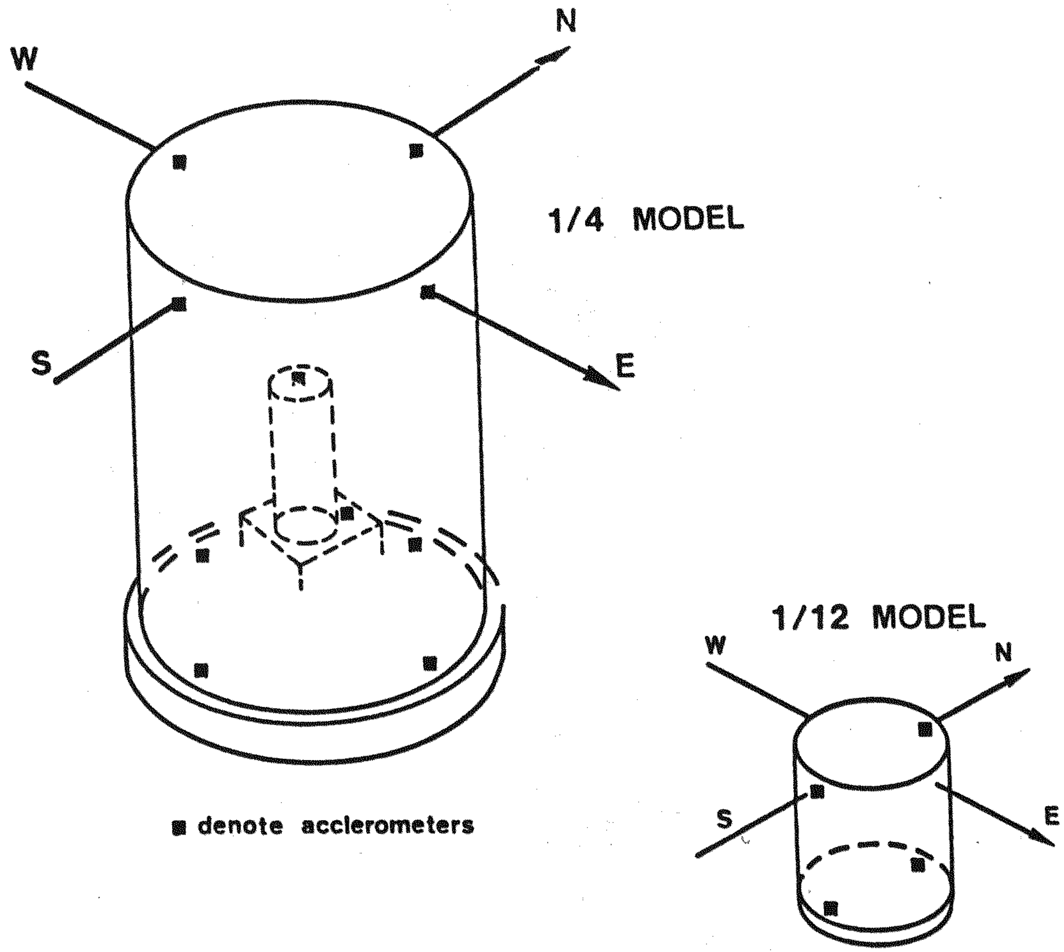


Figure 6. Accelerometers on Structures and Components for the Lotung SSI Project

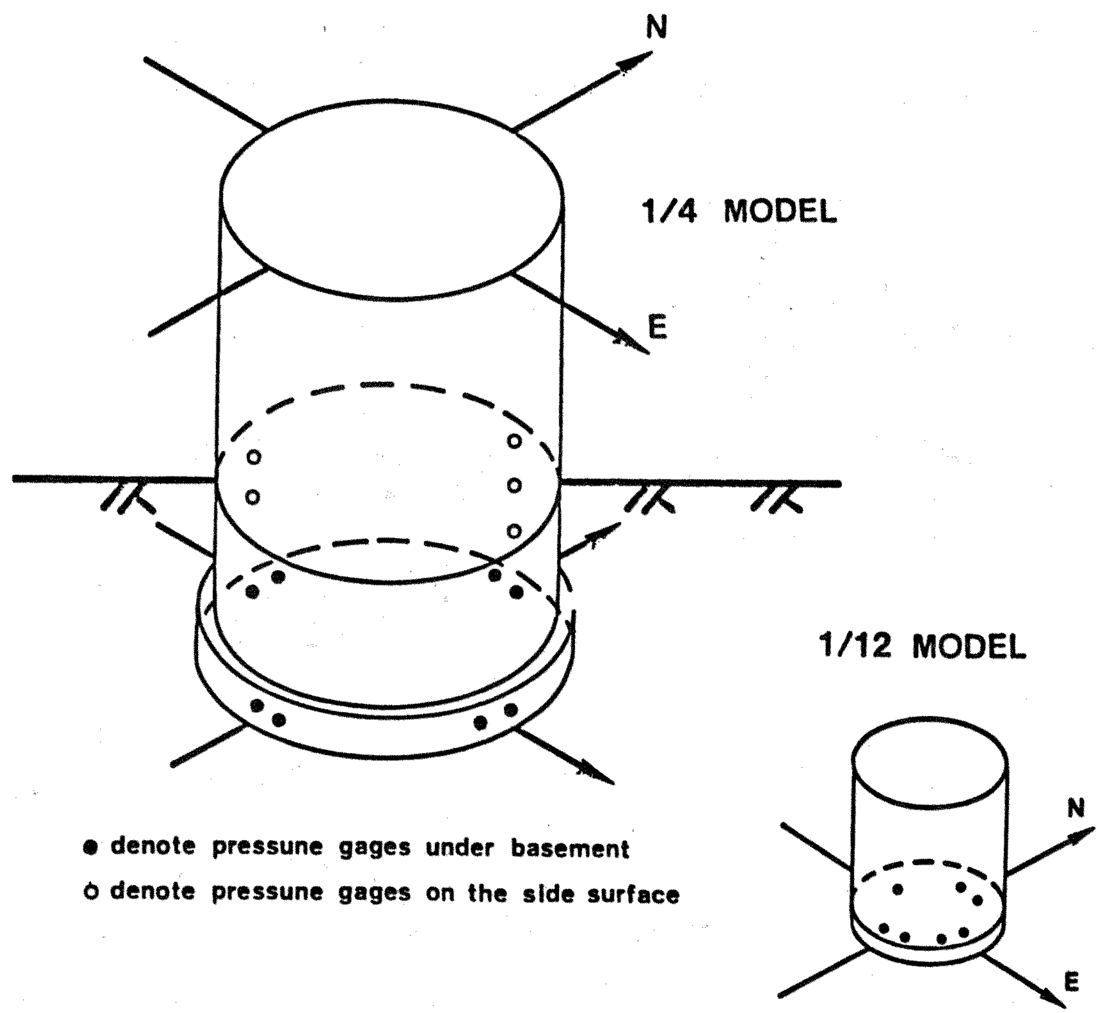


Figure 7. Pressure Gages Between the Structure and Soil for the Lotung SSI Project

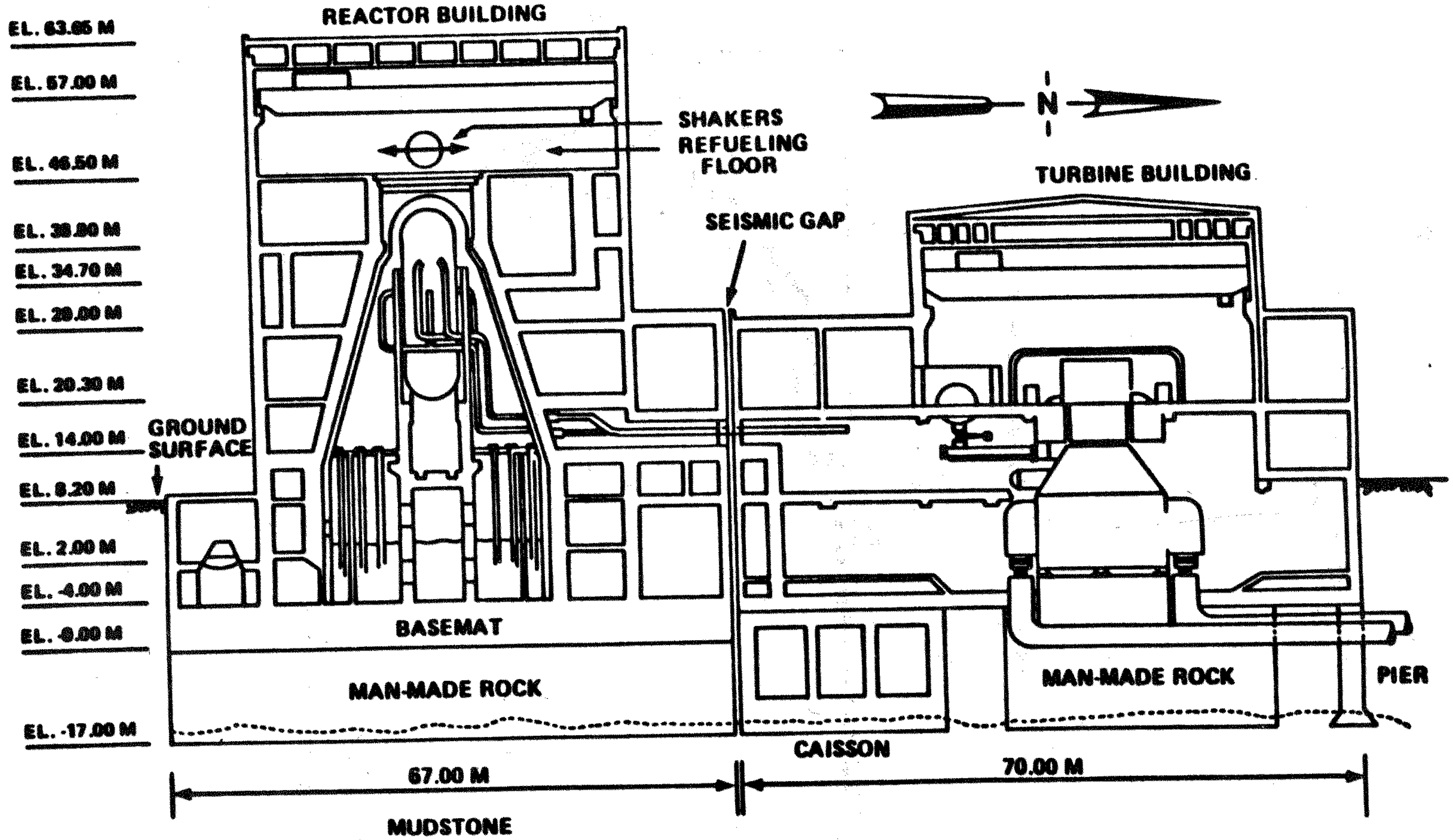
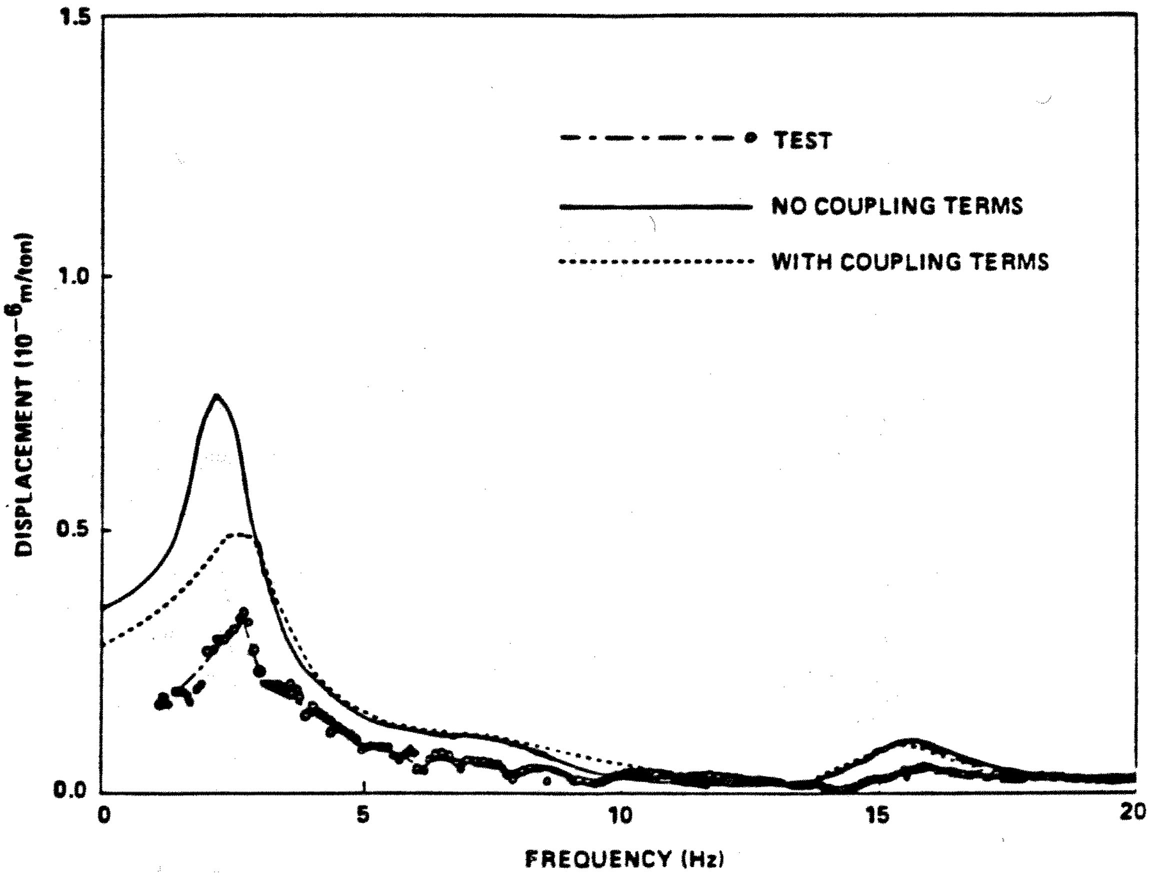
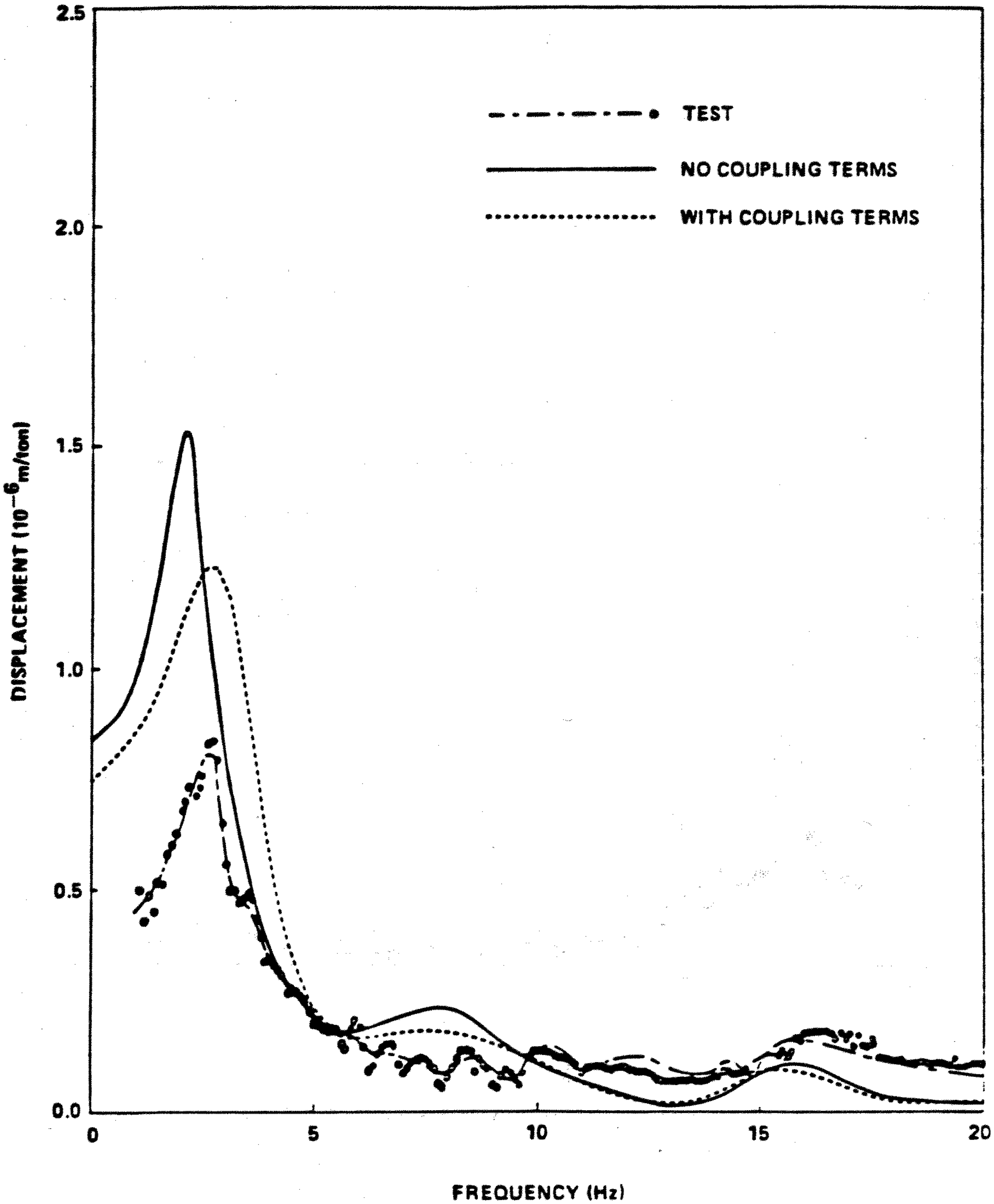


Figure 8. Reactor Building and Turbine Building Section



(d) PROTOTYPE REACTOR BUILDING, EL. 14 M

Figure 9. Normalized Steady-State Displacement Frequency Response Amplitude



(c) PROTOTYPE REACTOR BUILDING, EL. 46.5 M

Figure 10. Normalized Steady-State Displacement Frequency Response Amplitude

EARTHQUAKE SIMULATION EXPERIENCE

and

EXPERIMENTAL OBSERVATIONS

by

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Prepared for

Workshop on Soil-Structure Interaction
Sponsored by U.S. Nuclear Regulatory Commission
Office of Research

Bethesda, MD

June 16-18, 1986

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1.0 INTRODUCTION

Earthquake occurrences are widely separated in space and time. Although it is possible to observe the gross post-earthquake condition of engineered systems, there is rarely sufficient data to define the input ground motions and specific responses of important structures. A more complete data base for development and verification of models must come from well instrumented experiments. This paper discusses several topics associated with past and future experiments related to evaluating the earthquake response of engineered systems. These topics include:

- Desirable Data
- Sources of Available Data
- The SIMQUAKE Experience
- Important Structural Response Observations from SIMQUAKE
- A Simple Phenomena-Based SSI Model

2.0 DESIRABLE DATA

In order to judge the applicability of a particular set of test data, it is first necessary to identify the key aspects of the nuclear power plant (NPP) SSI problem. These may be arbitrarily separated into two general categories: primary features, which define the structural and geometrical nature of a NPP subjected to an actual earthquake, and secondary features, which describe some fundamental aspects of SSI behavior under general dynamic loadings.

Primary features are the most identifiable features of the NPP problem and involve the nature of the structure and the input to the structure caused by an earthquake. They are features which directly involve the response of an NPP to an earthquake. They may be evaluated directly without the need to invoke assumption and models regarding the structure or the dynamic input. Figure 1 illustrates some of the primary and secondary features that are desirable in an ideal experiment.

A plant is usually composed of several structures in close proximity which may or may not be joined at foundation level. The containment structure itself is necessarily massive for safety against radiation leakage. It is usually embedded in the surrounding medium. These two physical aspects have several important consequences:

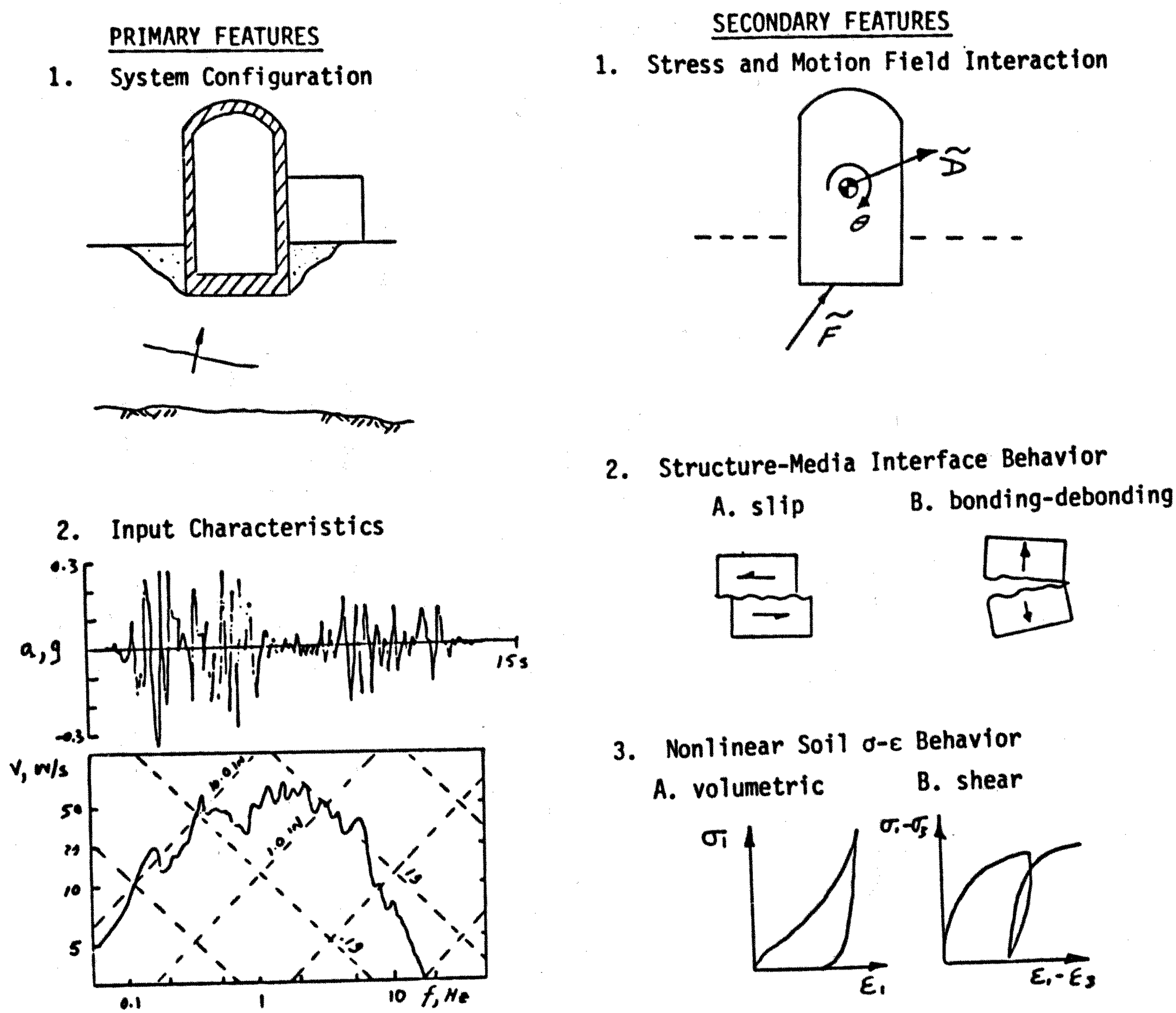


Figure 1. Primary and Secondary Features of Ideal Experiments.

- There are several potential loading surfaces (i.e., sides and bottom), and both vertical and horizontal load components may be significant. Resistance to vertical loads is due to both structure weight and friction along the sides of the embedded portions of the structure.

- The center of structural mass is typically offset from the center of applied force. As a result, there is high potential for rocking in addition to horizontal and vertical translation.

- Because the structure is massive and relatively stiff, rigid body modes of deformation are usually important.

The geometry of containment structures varies, and building shape in plan is more important than above-ground (elevation) shape. Unusual or irregular shapes and connected buildings can create significant three-dimensional effects and make overall response difficult to predict.

The input to the NPP seismic problem is a ground motion and stress field resulting from an earthquake. The characteristics of the disturbance depend on the transmission path of the seismic energy and the properties of the geologic media at the site. The source of loading is underneath the structure. The primary characteristics of the input are: peak amplitudes of acceleration, velocity, displacement, and stress, frequency content, and duration.

Secondary features are the fundamental behaviors of interest involving interaction of the structure with the surrounding medium. Basic relations between free field and structure motions describe the mutual influence which the structure and medium have on each other. Interface behavior includes mechanisms of stress transfer, bonding-debonding, and relative slip. The nonlinear nature of the dynamic behavior of geologic materials is also important. On a basic level, these behaviors together constitute the SSI problem.

The most desirable data for verification and benchmarking purposes would be data from a well-instrumented full-scale nuclear power plant which had been subjected to an earthquake of significant magnitude. This would match the primary features of structure geometry and input characteristics, as well as all the secondary features of fundamental SSI behavior. There are a few cases of actual measurements of NPP response to an actual earthquake, but there are no cases of comprehensive unambiguous measurement of all relevant inputs and responses. Because of the lack of sufficient actual earthquake data, the earthquake engineering community must take advantage of relevant data taken under less than ideal conditions.

3.0 SOURCES OF AVAILABLE DATA

Data which are relevant to the NPP earthquake response problem can be obtained from at least four sources:

- Earthquake Simulations with Explosives
- Department of Defense Blast Tests on Structures
- Field Vibration Tests
- Laboratory Scale Tests

Earthquake simulations with explosives involve the loading of full scale or model nuclear power containment structures by setting off carefully designed arrays of high explosives in their vicinity. It is possible to create a ground motion field which resembles that due to an earthquake. Test results of this type are limited but quite useful because they satisfy at least one (and approximately both) of the primary features of ideal data: structure configuration and input characteristics. The SIMQUAKE tests are discussed later.

Since the Limited Test Ban Treaty of 1963, the US Government has sponsored a number of field test series which have evaluated the response of various configurations of hardened and nonhardened structures under simulated nuclear blast and shock environments. The loading on these structures is transient and high in amplitude compared with earthquakes.

The structures in DOD tests are usually scale models of a prototype personnel or equipment shelter. Basic generic shapes which have been tested include slabs, horizontal and vertical cylinders, domes, arches, and boxes. They may be situated either on the ground surface, shallow-buried, or deeply-buried. Backfill conditions range from no backfill to combined backfill/native soil to completely backfilled. Structural materials vary depending on the purpose of the structure, but the majority of test models have been constructed from reinforced concrete. Although the environment differs from that of an earthquake, the tests do reveal some relevant aspects of SSI behavior.

Field vibration tests, especially of footings and other foundations of various configurations and under various loads, are the basis for many aspects of half-space modeling of SSI. Their main shortcoming is the low strains created in the soil.

Various static and dynamic SSI data are available from small to intermediate scale tests performed under laboratory conditions. Some experiments have investigated the overall response of small scale structures, using shake tables and, more recently, centrifuges. Some experiments have dealt with the more fundamental aspects of SSI behavior (e.g., interfaces). An advantage of smaller scale tests is that they are less expensive than larger scale experiments and therefore a greater number can be done to evaluate a particular response of interest. A disadvantage of smaller scale tests is the uncertainty involved in establishing correct similitude between behavior at full scale and behavior on a laboratory scale. Extrapolating results developed on the basis of laboratory tests to larger structures is usually difficult.

Shake table tests have provided much useful information on structural motion under highly controllable earthquake-like motions. However, the investigation of SSI behavior with shake tables has been inhibited by payload and performance limitations. Centrifuge test results are interesting because simulated gravity effects can be properly induced at small scales. Much of this work is still developmental. Other work has been done in the lab to investigate various aspects of SSI behavior, such as dynamic concrete-soil interface properties.

A data sources bibliography (Ref. 1) which identifies SSI data with potential applications to the nuclear power plant earthquake SSI problem was recently prepared. Data from structures with measured earthquake response, earthquake simulation with explosives, Department of Defense field tests, and laboratory scale tests are included. A consistent format is used in presenting each data source so that their comparison and evaluation can be more easily accomplished. A preliminary assessment was made of the applicability of each data source to the particular case of analyzing soil-structure interaction of nuclear power plant structures under earthquake excitation.

4.0 RECENT EARTHQUAKE SIMULATIONS USING HIGH EXPLOSIVES

One of the best methods for evaluating SSI phenomena under conditions similar to those experienced in earthquakes is through the use of specially designed arrays of high explosives. The general method, developed under NSF sponsorship, is described in detail in Reference 2. The following paragraphs briefly describe the explosive simulation approach.

High explosive methods are particularly useful for two major applications:

- Soil-Structure Interaction
- Excitation of Large Structures to High Response Levels

Explosive simulation allows soil-structure interaction problems to be investigated with the excitation coming from the soil medium, and without boundary effects. Also, explosive simulation provides sufficient output energy to bring large structures to high levels of nonlinear response.

A typical experimental layout is shown in Figure 2. It has been found that a planar explosive array configuration is most advantageous for earthquake simulation applications because it provides a wider range of control over motion amplitude, frequency content and attenuation rates than available with spherical or cylindrical explosive arrangements. In addition, a planar array provides relatively uniform motions across a region whose width is about the same as the width of the array. A planar array is approximated by placing the explosives in closely spaced vertical drill holes. The explosive array is overburdened to ensure maximum coupling of the explosive energy.

Ground motion prediction methods which cover the prediction of acceleration, velocity, displacement, frequency content and duration have been developed from dynamic wave propagation analyses and previous data. These methods are used to select array designs and structure locations to achieve responses which meet some desired simulation criteria. Multiple, time-sequenced arrays are used to lengthen motion time durations, if necessary, for a particular application.

Downhole and surface vertical and horizontal accelerometers and velocity gages are used to measure free-field response and near-field response adjacent to structures. Structure responses are also measured with accelerometers and

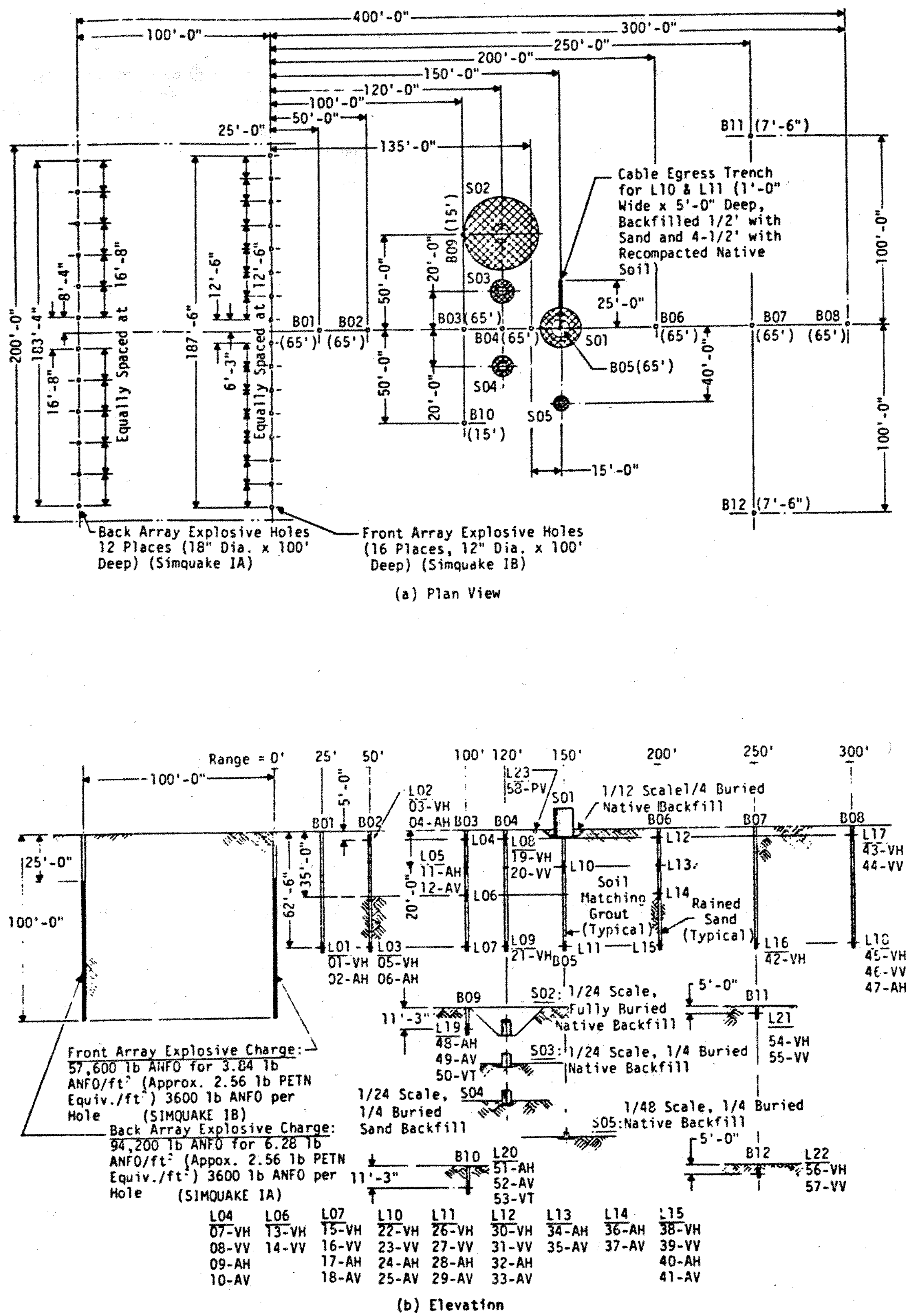


Figure 2. SIMQUAKE I Layout and Instrumentation. (Ref. 3)

velocity gages, as well as strain gages. Interface stresses on embedded structure faces are measured with interface structure-media interaction gages of various types. Up to 120 active channels of data have been measured on previous experiments. The data are recorded on magnetic tape in a mobile field recording van. Data are later digitized and processed on a digital computer for analysis.

The word "simulation" generally implies that the prototype environment cannot be generated at will, and that some characteristics of the prototype environment will not be reproduced exactly in a simulation. In developing simulation criteria, it is necessary to consider the two primary aspects of the prototype problem: (1) the characteristics of the prototype earthquake environment, and (2) the dynamic characteristics of the engineering system. Understanding of the prototype earthquake environment may be the weakest link in the process. Earthquake mechanisms and prediction are still incompletely understood.

The important characteristics of earthquakes in terms of structural response and criteria for a good simulation are not universally agreed upon. Many American engineers and most US design codes focus on peak acceleration. There are some applications where particle velocity is important. Frequency content and duration also play a role. The approach to simulation criteria used to date focuses on system response.

At one extreme, simulation criteria could require that the simulated environment contain a precise duplication of the prototype waves, and their stress and motion time histories. This would ensure precise duplication of structure response for full size prototype structures. Such a severe criterion would be economically impractical and technically difficult to achieve. In fact, it is inconsistent with the state of knowledge of the prototype environment and understanding of the inelastic interactions which occur. A more realistic approach is to consider the type of structure, its dynamic characteristics, anticipated response in the prototype environment and the major uncertainties in the anticipated response. Simulation criteria should then be specified to ensure similar response, especially excitation of the structure in such a way that the major uncertainties can be evaluated.

The criteria will probably vary from structure to structure and may include any or all of the following: (1) wave types (P, SV, SH, or R), (2) stress-time history associated with the waves, (3) motion-time history at a point or points, (4) some level and type of response in the structure. If below-ground stresses are important, then simulations may be required to reproduce specific earthquake waves and their associated stress fields. If only the ground motion is important, then there is flexibility in the methods that can be used to create the ground motion. In many instances, it appears that a good method for evaluating (and, inversely, designing) the ground motion is through the use of response spectra. If a ground motion produces system response at levels similar to those produced by an earthquake, then the specific details of the motion are important only to the extent that they influence the response spectrum. If modeling is used, appropriate scaling of both the system and the input environment is required.

The first series of experiments which utilized the method for the simulation of earthquake effects was the SIMQUAKE series of experiments (Refs. 3 and 4) supported by the Electric Power Research Institute (EPRI). The objective of the experiments was the establishment of a controlled three-dimensional data base on soil-structure interaction phenomena of the type that is important for nuclear power plant containment structures.

The experimental series focused on obtaining data at high soil strain levels and under multiple cycles of motion with amplitudes on the order of those that might be expected in a strong earthquake. The SIMQUAKE series consisted of four separate events: Mini-SIMQUAKE (MSQ), SIMQUAKE IA (SQIA), SIMQUAKE IB (SQIB), and SIMQUAKE II (SQII). Parameters varied included structure size (1/8, 1/12, 1/24, and 1/48 nominal sizes), backfill materials, and depth of embedment, as well as motion amplitude variations.

Although every attempt was made to do as good a job as possible in achieving an earthquake-like simulation, it was not intended that the experiments be interpreted as direct simulations of any earthquake. Nor was it intended that any attempt be made to directly scale the results from the small size structures to a prototype size structure. The project was directed toward providing a comprehensive and self-consistent group of data which could form the basis for evaluations and improvements of current analysis methodologies.

The SIMQUAKE experimental series demonstrated that it is possible to generate ground motions with amplitudes and frequency content similar to those encountered in moderate to strong earthquakes. It also demonstrated that it is feasible to extend the duration and number of cycles in the ground motions from explosive simulations by using sequentially fired arrays. The entire SIMQUAKE series of experiments yielded over 2500 channels of active data on free-field and near-field ground motions and structure responses. A more recent experiment in rock, supported by the Niagara-Mohawk Power Corporation and Electric Power Research Institute, will be described by others at this workshop.

An elevation of SIMQUAKE I, including free-field instrumentation, was shown in Figure 2. The SIMQUAKE I event was designed to be a large scale experiment involving two arrays containing a total of 63,490 kg (70 tons) of ANFO explosive. The back array was designed to fire first, followed by the front array about 1.5 seconds later. Due to a firing system malfunction, only the back array (36,200 kg [40 tons] of ANFO) fired at the scheduled test time. The instrumentation was then refurbished and the front array (27,210 kg [30 tons] of ANFO) was fired a few weeks later. The two firings resulted in two complete sets of data under two different levels of excitation. The back array event has been designated SIMQUAKE IA and the front array event SIMQUAKE IB.

The SIMQUAKE II explosive design was essentially a repeat of the SIMQUAKE I design, but with special precautions taken to ensure that the two explosive arrays fired in sequence, so that the effect of multiple ground motion cycles and longer duration could be evaluated. Based upon the SIMQUAKE I experience, the firing delay was changed to 1.2 seconds. In addition, the experiment included a larger structure and additional parameter variations on structure configuration and motion amplitudes at the structure locations. References 3 and 4 provide more detailed information on the experiment designs and results.

4.1 Structure Descriptions

A generic, full-sized structure would have been prohibitively expensive to build for the experimental program. Also, a single structure would not have permitted variations in parameters. Therefore, it was decided to utilize subsized structures to investigate the responses of interest. The scale could not be too small because of uncertainties in modeling in inelastic materials, gravity influences, and the need for a practical structure size for instrumentation purposes. In addition, it was believed to be important to use a range of structure sizes that were sufficiently large to provide a data base suitable for validating analytical techniques. If the dynamic behavior of the variously sized sub-scale structures could be analyzed and understood, then the engineering community would have greater confidence in the application of the same methods to prototype (full-scale) earthquake problems.

Initially, three structure sizes (1/2, 1/24, 1/48) were selected for investigation. In SIMQUAKE II, a fourth size (1/8) was included. These size designations do not mean the test structures were precisely scaled models of an actual prototype nuclear power plant structure. Rather, the scale designations are to indicate approximate relations between the models and an average generic structure. Among themselves, the structures were precise, dimensionally scaled models of each other (the 1/8 size had different reinforcing). As a result, the test structures could logically have been designated prototype (1/8), 2/3 scale (1/12), 1/3 scale (1/24), and 1/6 scale (1/48), for the purposes of this program.

In order to ensure that translation/rocking behavior would be emphasized and not confused with deformational modes of behavior, the structures were designed to preclude structural damage and to have structural mode frequencies at least twice the small strain translation/rocking frequency. The structures were constructed as right circular cylinders of reinforced concrete with open tops. The 1/8 size structure was reinforced with conventional reinforcing bars. The smaller structures were reinforced with steel plate on the inner and outer surfaces, because of difficulty in scaling reinforcing bar sizes.

The SIMQUAKE IA and IB events contained five model structures (designated S01 through S05 in Fig. 2). The largest structure (S01) was a nominal 1/12 size model of a generic prototype facility. It was embedded to 25% of its height in recompacted native soil. Figure 3 shows the configuration and instrumentation details for the 1/12 size structure. Three of the structures were nominal 1/24 scale. One was fully buried (S02). The other two were embedded to 25% of their height, one in native soil (S03) and the other in sand backfill (S04). The fifth structure (S05) was a nominal 1/48 scale embedded to 25% of its height in native soil.

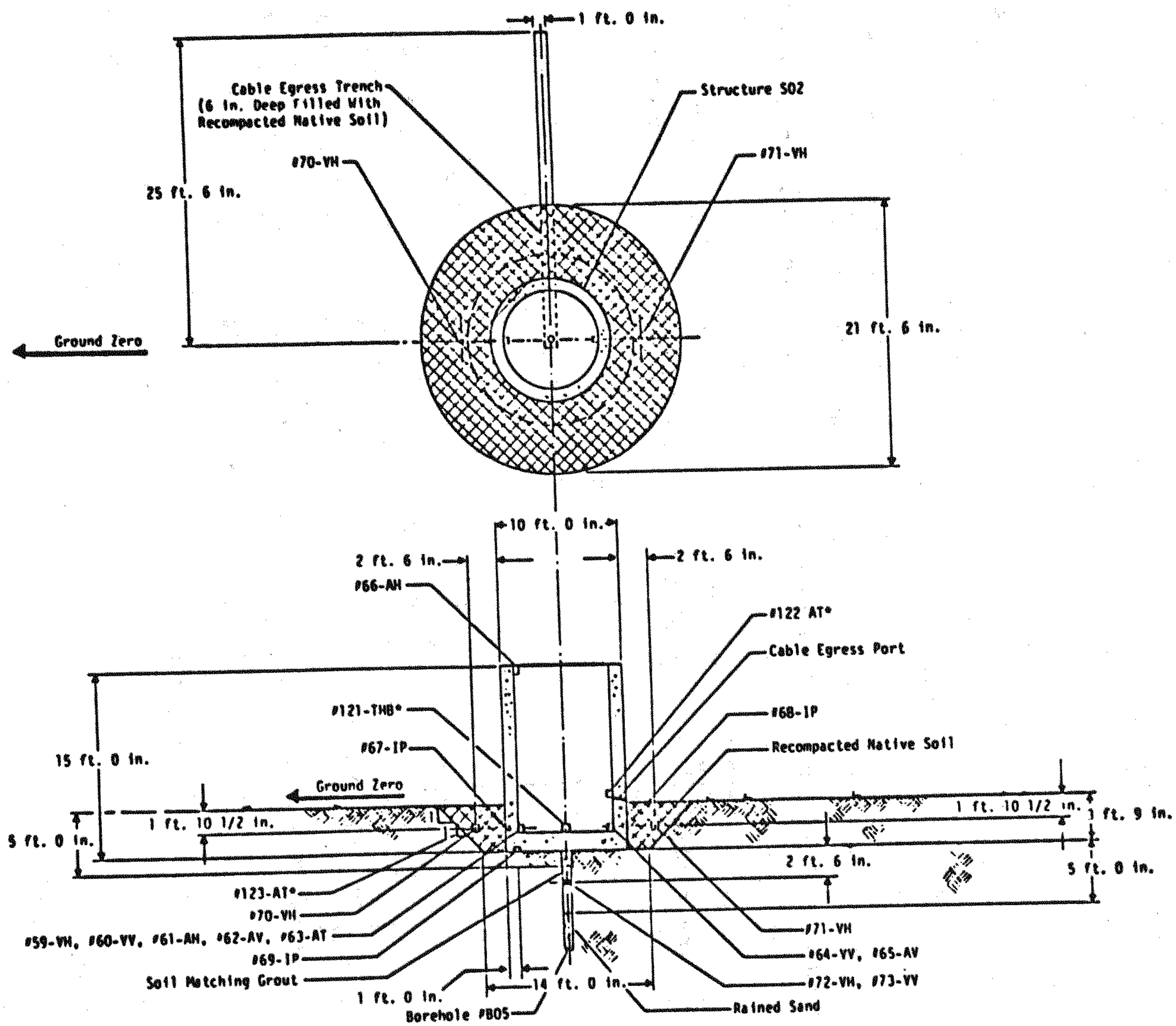


Figure 3. Configuration and Instrumentation Details for the 1/12 Size Structure (S01) in SIMQUAKE I.

Six structures (S01 through S06) were included on SIMQUAKE II. Four structures (S03 through S06) were used previously in SQIA and SQIB. They were the 1/12 size structure and the three 1/24 size structures. The 1/48 size structure was not reused because its translation/rocking response frequencies were too high to be excited by the low frequency ground motions in the experiment. A second 1/12 scale structure (S02) was constructed so that the behavior of the same structure could be observed at two different motion amplitude levels. In addition, a nominal 1/8 scale structure (S01) was placed on SIMQUAKE II. The 1/8 size, both 1/12 size, and one 1/24 size structures were embedded to 25% of their height in native soil backfill. One of the 1/24 size structures was placed flush with the ground surface (0% embedded). It contained an inner tank filled with water so that fluid/structure interaction could be observed. Another 1/24 size structure was mounted on a slab which was isolated from a ground slab through an isolation system. Table 1 provides the dimensions of each of the structures while Table 2 provides details on the configurations of the structures in each experiment. Results in later paragraphs are given only for the structures in the 25% embedment configuration in native backfill.

TABLE 1

STRUCTURE DIMENSIONS

Size	Height		Diameter		Wall Thickness		Base Thickness		Weight	
	ft	m	ft	m	ft	m	ft	m	tn.	T
1/8	22.5	6.86	15.0	4.57	1.5	0.46	2.25	0.69	126.4	114.9
1/12	15.0	4.57	10.0	3.05	1.0	0.30	1.50	0.46	38.9	35.4
1/24	7.5	2.29	5.0	1.52	0.5	0.15	0.75	0.23	4.9	4.5
1/48	3.75	1.14	2.5	0.76	0.25	0.076	0.375	0.114	0.61	0.55

tn. = English tons
T = Metric tons

TABLE 2

STRUCTURE CONFIGURATIONS

Test	Structure	Size	Embedment (%)	Backfill	Other
MSQ	S01	1/48	25	Native	---
SQI and IB	S01	1/12	25	Native	---
	S02	1/24	Buried	Native	---
	S03	1/24	25	Native	---
	S04	1/24	25	Sand	---
	S05	1/48	25	Native	---
SQII	S01	1/8	25	Native	---
	S02	1/12	25	Native	---
	S03	1/24	0	Native	Water Filled (1.49 tons total)
	S04	1/24	0	---	Isolated
	S05	1/24	25	Native	---
	S06	1/12	25	Native	---

Active instrumentation was placed in the free-field, in the structures, and in the vicinity of the structures. These later measurements are called near-field measurements. The free-field measurements consisted of horizontal, vertical, and transverse accelerations and velocities. Structure measurements included similar motion measurements plus angular displacement and interface stress on the front, rear, and base faces of some of the structures.

4.2 Test Site and Backfill Characteristics

All the test events were conducted in the same region of McCormick Ranch. The site consists mainly of silty, clayey fine sand (SM and SC classification) with a few layers of plastic silts and clays. Cementation of varying amounts occurs throughout. The ground water table is below 152 m (500 ft). Seismic wavespeeds, derived from refraction, uphole and crosshole measurements, and estimates of in situ weight are given in Table 3.

TABLE 3

SEISMIC PROPERTIES OF SIMQUAKE I TEST SITE

Depth m (ft)	P-Wave Speed m/s (ft/s)	S-Wave Speed m/s (ft/s)	Total Unit Weight kg/m^3 (lb/ft^3)
0-1.83 (0-6)	366 (1200)	244 (800)	1522 (95)
1.83-7.62 (6-25)	594 (1950)	290 (950)	1810 (133)
7.62-27.5 (25-90)	762 (2500)	335 (1100)	1922 (120) Estimates
27.4- (90-)	975 (3200)		

All structures were in recompacted native backfill, except one of the 1/24 scale structures in SIMQUAKE IA and IB, which was in a rained sand backfill. Native backfill wet densities ranged from 1810 to 2060 kg/m^3 (113 to 129 lb/ft^3) with 1970 kg/m^3 (123 lb/ft^3) being average. The seismic velocity in the backfill was about 392 m/s (1300 ft/s).

Dynamic uniaxial strain dynamic oedometer tests on recompacted native material with a total unit weight of about 1570 kg/m^3 (98 lb/ft^3) yielded near-linear loading behavior in the range of 0 to 0.69 MPa (0 to 100 psi). Moduli ranged from 22.7 to 69 MPA (3300 to 10,000 psi) corresponding to dilatational wave-speeds of about 120 to 210 m/s (400 to 700 ft). The average modulus was about 43 MPa (6200 psi) corresponding to an average wavespeed of about 165 m/s (540 ft/s). The rained sand backfill density ranged from 1590 to 1730 kg/m^3 (99 to 108 lb/ft^3) with 1670 kg/m^3 (104 lb/ft^3) being the average.

Ground motion characteristics from the explosive simulators can best be described by referring to specific measurements. Figure 4 shows the free-field horizontal acceleration and its integrations at the 61 m (200 ft) range and 1.52 m (5 ft) depth on SIMQUAKE II. The double array explosion caused approximately four cycles of excitation in both the acceleration and the velocity time histories. The ground motions duration is about 2.5 s. The peak ground motions are 2.2 g, 0.95 m/s (37 in/s) and 0.14 m (5.5 in). The major frequency content is in the range of 1 to 2 Hz.

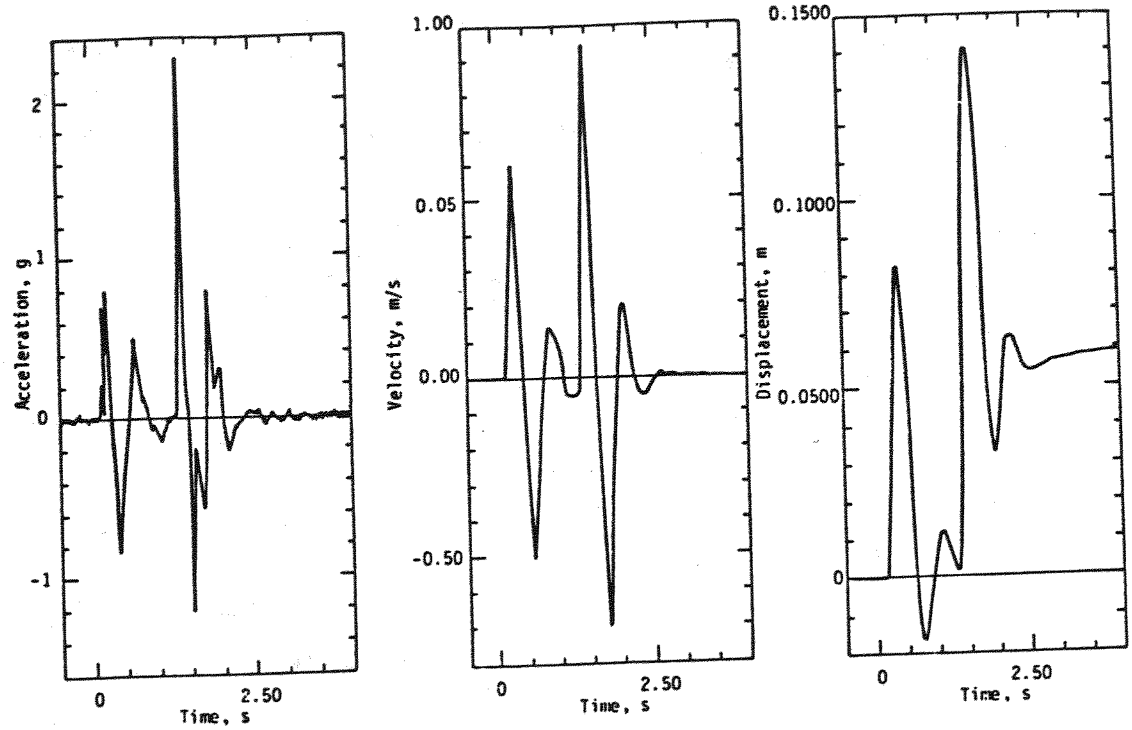


Figure 4. Horizontal Motions at 61 m (200 ft) Range and 1.53 m (5 ft) Depth on SIMQUAKE II.

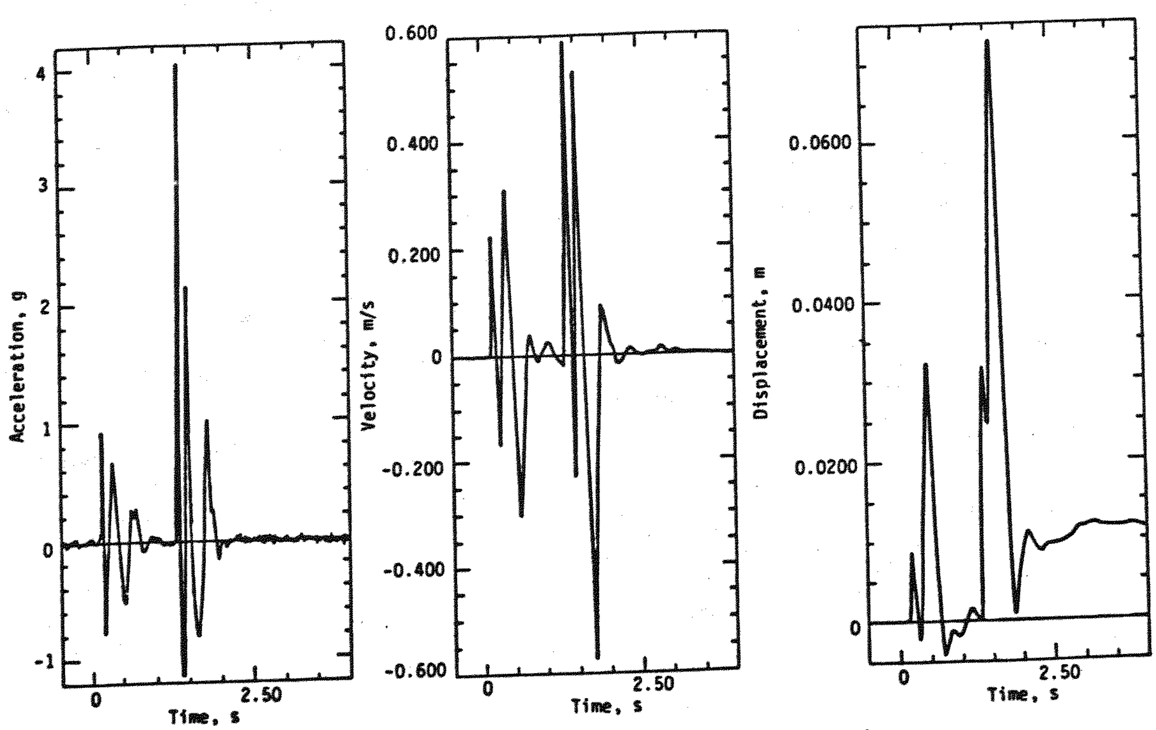


Figure 5. Vertical Motions at 61 m (200 ft) Range and 1.52 m (5 ft) Depth on SIMQUAKE II.

The vertical acceleration and its integrations at the same location are shown in Figure 5. The peak upward vertical motions are about 4.1 g, 0.58 m/s (23 in/s), and 0.073 m (2.9 in). The vertical velocity and displacement are about 1/2 to 2/3 the horizontal values, while the vertical acceleration is about 75% greater. The frequency associated with the maximum vertical acceleration is fairly high and, as a result, the vertical velocity and displacement remain lower than the horizontal values. The high vertical acceleration is apparently due to vertical relief toward the free surface.

Table 4 summarizes the peak near-surface free-field ground motions in the vicinity of the large (1/12 and 1/8) structures in SIMQUAKE IA, SIMQUAKE IB, and SIMQUAKE II. The peak acceleration and velocity amplitudes in SQII are about the same as in SQIB because they correspond to the same explosive array. The displacements in SQII are higher because of the superposition effects of the sequenced arrays. The peak horizontal accelerations, velocities and displacements range from 0.5 to 3.8 g, 0.3 to 1.2 m/s, and 4 to 27 cm, respectively. For a full sized structure, the accelerations tend to the high side, but the velocities and displacements correspond to those which might be encountered in a moderate to very strong earthquake. For smaller structures, the accelerations are too low from a scaled viewpoint, but this is not believed important for overall structure response. The major frequency content of the ground motions in both experiments is in the range of 1 to 2 Hz.

The frequency content of the ground motions is exhibited more explicitly in the response spectra. Figure 6 compares the SQII horizontal spectra at the 200 ft (61 m) range and 5 ft (1.52 m) depth with 1/8 and 1/12 scaled prototype spectra based on Reference 5. These comparisons suggest that the 1/12 scale structure at 200 ft (61 m) was excited at levels above those corresponding to a 1/2 g earthquake for frequencies below about 3 Hz. The 1/8 scale structure was excited at levels above those of a 12 g earthquake for frequencies below about 4 Hz.

TABLE 4

SUMMARY OF PEAK FREE FIELD GROUND MOTIONS AT 1.52 m (5 ft) DEPTH IN VICINITY OF 1/12 AND 1/8 SIZE STRUCTURES

Range m (ft)	Event	Horizontal Acceleration g	Horizontal Velocity m/s (in/s)	Horizontal Displacement cm (in)	Upward Vertical Acceleration g	Upward Vertical Velocity m/s (in/s)	Upward Vertical Displacement cm (in)	Downward Vertical Acceleration g
45.7(150)	IA* IB* II	1.1* 3.1 3.8	0.7(28) 1.2(47) 1.16(46)	10(4) 20(7.9) 27(10.7)	1.1 4.4 6.8	0.37(15) 0.88(35) 1.07(42)	4.4(1.7) 11(4.3) 18.8(7.4)	0.7 1.1 1.3
61.0(200)	IA IB II	0.75 2.1 2.3	0.44(17) 0.95(37) 0.90(35)	6.2(2.4) 11(4.3) 14(5.5)	0.81 2.8 4.1	0.34(13) 0.50(20) 0.51(20)	2.2(0.87) 5(2.0) 7.3(2.9)	0.8 1.0 1.1
76.2(250)	IA* IB* II	0.55 1.5 1.4	0.3(12) 0.6(24) 0.72(28)	4.2(1.7) 7.4(2.9) 10.9(4.3)	0.55 1.0 2.0	0.17(6.7) 0.4(16) 0.33(13)	1.7(0.67) 3(1.2) 3.6(1.4)	1.0

*Interpolated or Extrapolated From Adjacent Measurements

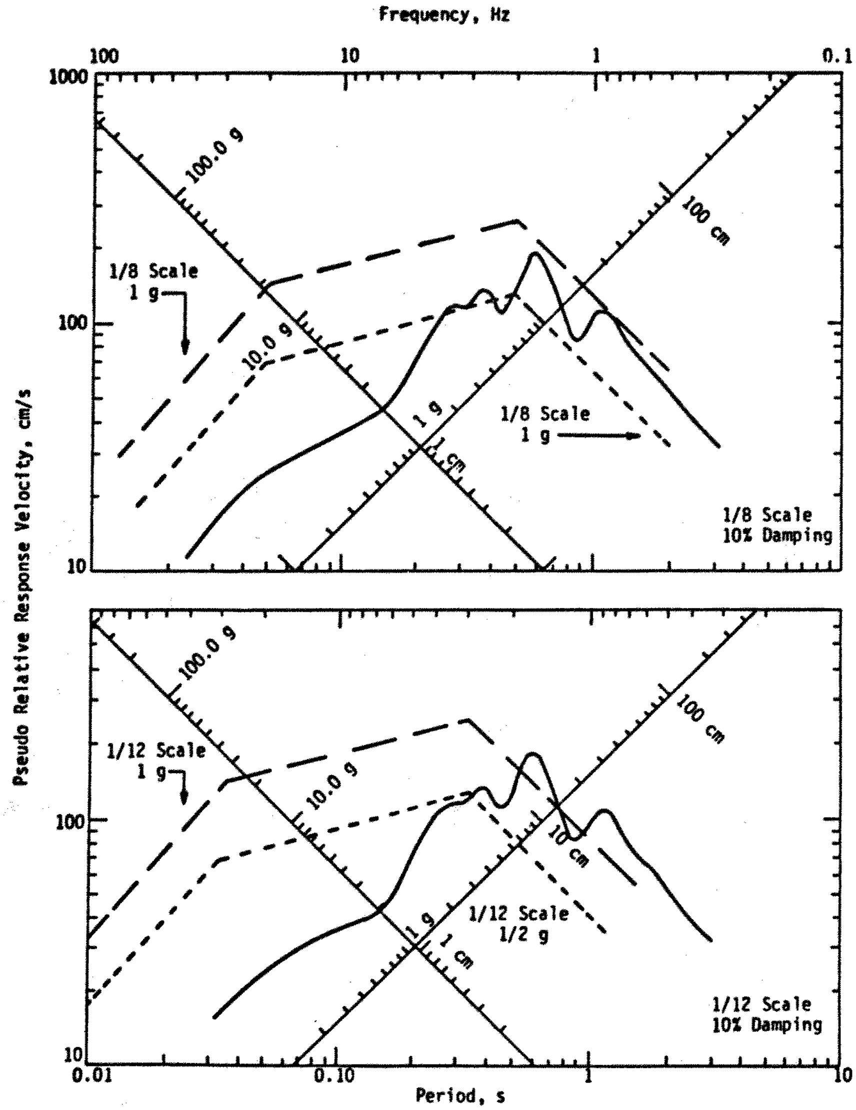


Figure 6. **Comparison of SQII Response Spectra at 61 m (200 ft) Range and 1.52 m (5 ft) Depth with Appropriately Scaled Prototype Spectra Based on Reference 5.**

Four significant simulation fidelity issues have been recognized as potentially important in evaluating the usefulness of explosive simulation. They are:

- Horizontal Acceleration Amplitude
- Ground Shaking Duration
- Ground Motion Wave Field
- Vertical Motion Environment

In general, explosive simulations produce horizontal accelerations which are relatively high for a given level of velocity and displacement than occurs for earthquakes. For example, the SQII motions at one range of interest were 2.2 g, 0.95 m/s and 0.14 m. The importance of this difference depends upon the importance of peak acceleration to overall structural response and the scale of the structure. Generally, the peak acceleration is associated with a high frequency spike. Velocity, displacement and overall frequency content are expected to be the major contributors to response. It should also be noted that at less than full scale the accelerations generated by explosives for simulating strong earthquake-like ground motion effects is the relatively short time duration of the excitation. The time durations in SIMQUAKE IA and IB were on the order of 1 s and the ground motions contained 1½ significant cycles of particle velocity. Two explosive arrays were sequentially detonated in SIMQUAKE II to lengthen the ground motion duration. The arrays performed successfully. As a result the motion duration from the double array explosion was about 2.5 s. It appears that multiple, sequentially fired arrays can be used to extend the duration of ground shaking to achieve adequate simulation times.

The ground motion wave field issue relates to the type of wave in the earth causing the ground motion. In the recent past, some members of the earthquake engineering community held the position that the major part of earthquake ground motions could be attributed to vertically propagating shear waves. Very little attention was given to vertical motions or the effect of compression waves. With the high vertical accelerations experienced in the Imperial Valley earthquake of 1978, this limited view has lost some credibility. It is now clear that earthquake wave fields are much more complex than represented by simple models.

To a large extent, explosive ground motions are generated by compression waves. However, detailed analysis of the motions in the SIMQUAKE series indicates that there are substantial shear wave associated motions, especially in the near-surface region. The importance of the type of wave generating the ground motion is not yet fully understood, but there is some evidence that stiff structures, with dimensions that are small compared with the wavelengths of the ground motions, respond primarily to the motion time history and are insensitive to the details of the wave field causing the motion. This is especially the case for structures whose foundation mass and stiffness are sufficiently high compared with the soil so that the foundation behaves essentially as a rigid body. Horizontal motions, regardless of the incoming wave field, are transmitted to the structure by shear on the structure base and compression on the embedded faces. Even shear particle motion is converted to compressions against the structure walls through reflections.

The characteristics of the vertical motions are also important for evaluating simulation fidelity. In general, the amplitudes of the vertical velocities and displacements of explosive simulations are about 1/2 to 2/3 the horizontal values, corresponding very well to what is understood about prototype earthquakes. In addition, the frequencies associated with the vertical behavior were somewhat higher than the horizontal frequencies, also in agreement with prototype earthquakes. However, the vertical accelerations can be equal to or as much as 75% greater than the horizontal accelerations. This latter difference has been pointed to as a possible fidelity shortcoming in explosive simulation. Whether or not this is the case depends, first and most importantly, upon the contribution of the vertical acceleration to overall response.

The frequency associated with the maximum vertical acceleration is very high and the acceleration duration is very short. The high acceleration is apparently related to vertical relief toward the free surface. The recent measurements in the Imperial Valley earthquake of 1978 indicate that very high vertical accelerations, in some cases on the order of 50% greater than the horizontal accelerations, may occur in the source regions of earthquakes, perhaps for the same reason. In any case, the effect of the vertical environment in both simulations and earthquakes should be thoroughly investigated in the design of any simulation experiment.

5.0 STRUCTURE RESPONSE IN SIMQUAKE

The SIMQUAKE experiments provide information on the response of the cylindrical structures in various embedment configurations under various levels and durations of excitation. The structures were designed to ensure that the responses would be of a rigid body type. The measured structural strains confirmed that structural deformations were trivial. Hence, the measured responses are all soil-structure interaction responses due to rigid body motion. The following paragraphs discuss the main characteristics of the response of the structures partially embedded in native backfill.

The major evidence of structure rocking and, therefore, significant soil-structure interaction, comes from the differences between the horizontal response of the top and base of the structures. Figure 7 shows the horizontal response at the base and top of the 1/12 size structure in SQIB. The peak outward acceleration at the structure top is only about 60% of that at the base. The inward acceleration, however, exceeds the base response. More importantly, the top acceleration has little similarity to the base acceleration after the first outward acceleration pulse and undergoes significant free-vibration as the base acceleration approaches quiescence. These differences between top and base response indicate significant structure rocking.

Structural rocking is exhibited directly in the differential acceleration, velocity, and displacement between the top and base of the structure. These differentials are shown in Figure 8. The peak differential motion amplitudes are on the order of the peak transient free-field amplitudes. The rocking frequency, as interpreted from the differential velocity shown in Figure 8(b), is initially about 1.8 Hz. The frequency increases to about 3.3 Hz as the rocking amplitude diminishes. The peak rocking angle is about 2°, as shown in Figure 8(c). The residual tilt of about 1° is in excellent agreement with field survey measurements.

The 1/12 scale structure also rocked in SQIA. The rocking amplitudes as reflected in the differential response, however, were less compared to the free-field than in SQIB. In addition, the rocking frequency in SQIA was higher than in SQIB.

The response of the 1/8 size structure typifies the rocking response of the structures in the SQII multiple array experiment. Figure 9 presents time

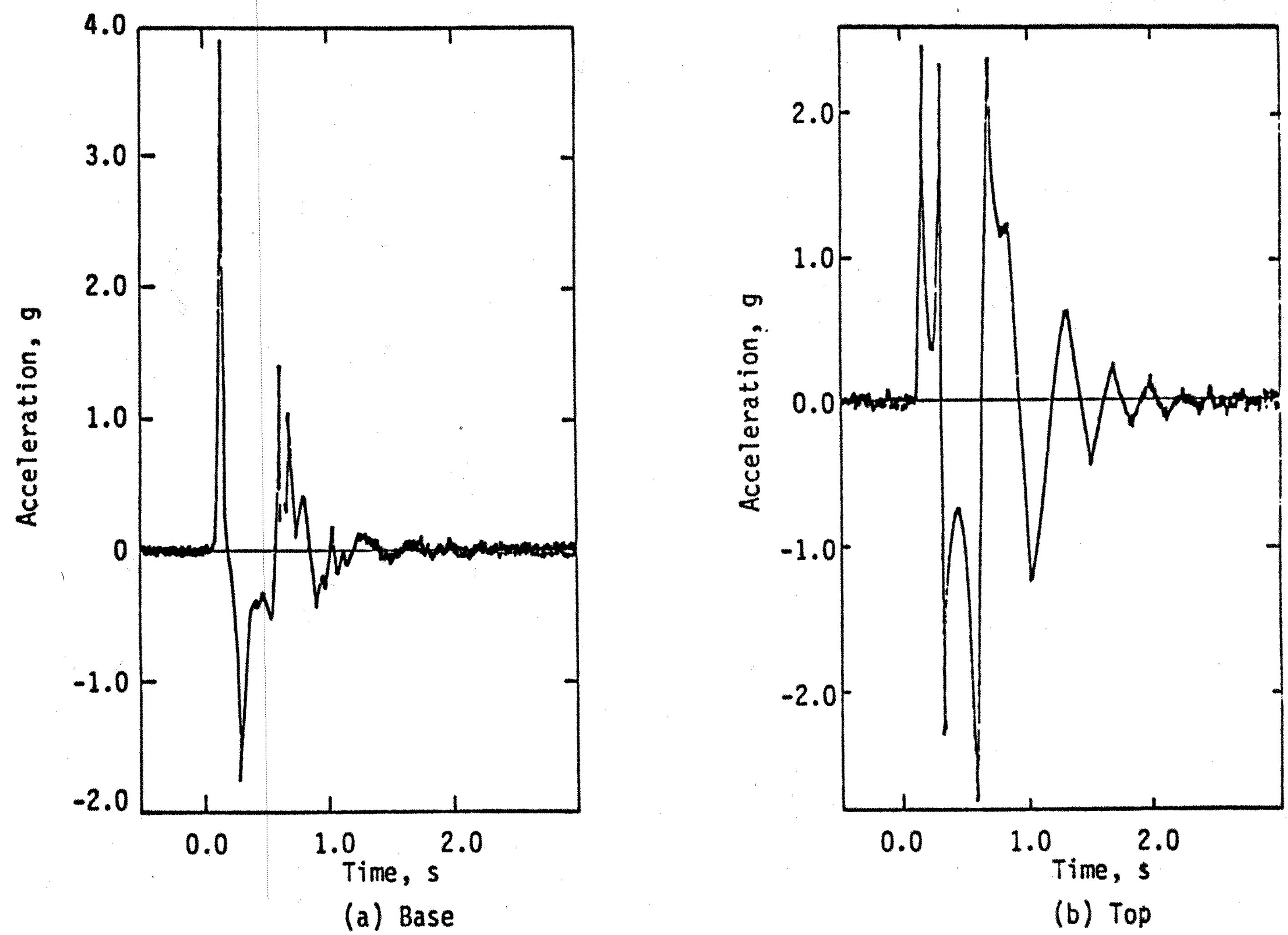
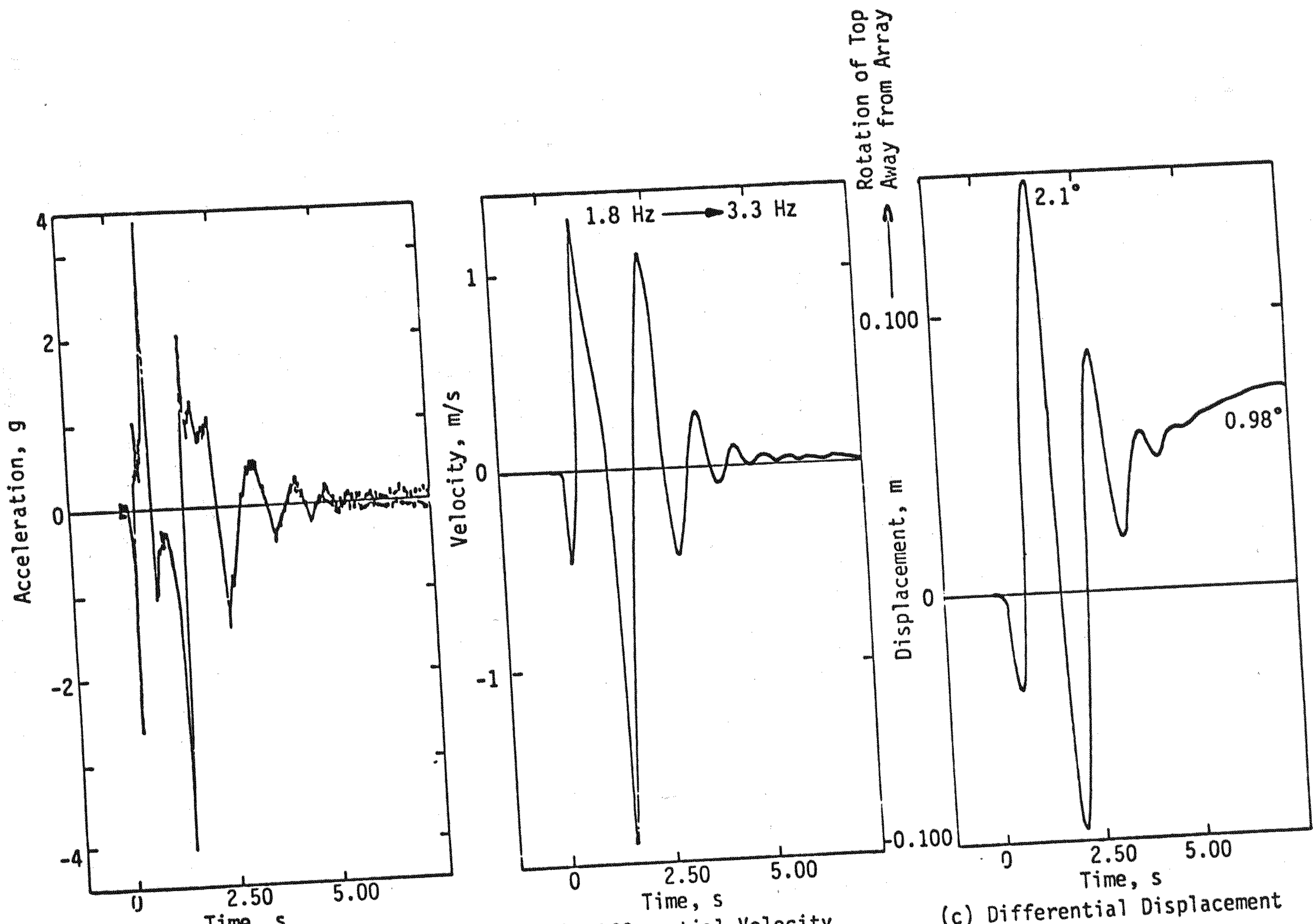


Figure 7. Horizontal Accelerations in 1/12 Size Structure (S01) in SIMQUAKE IB.

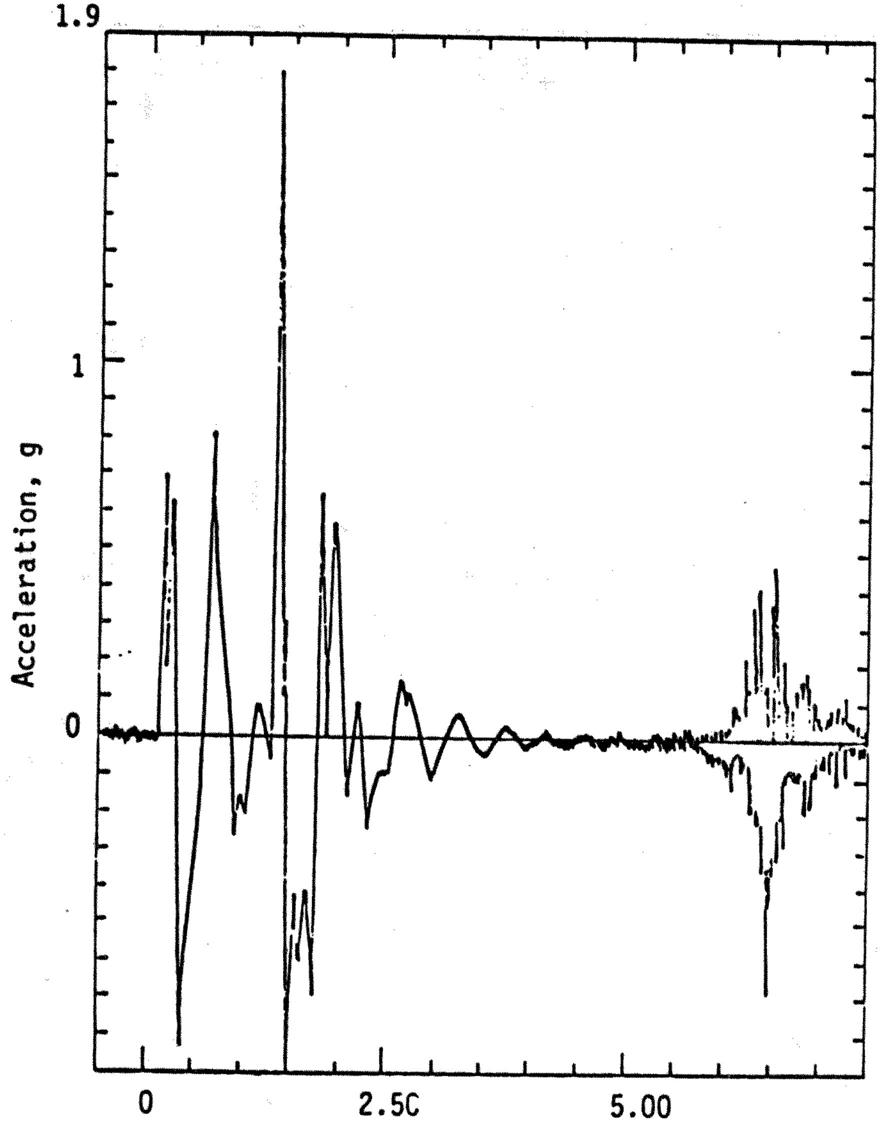


(a) Differential Acceleration

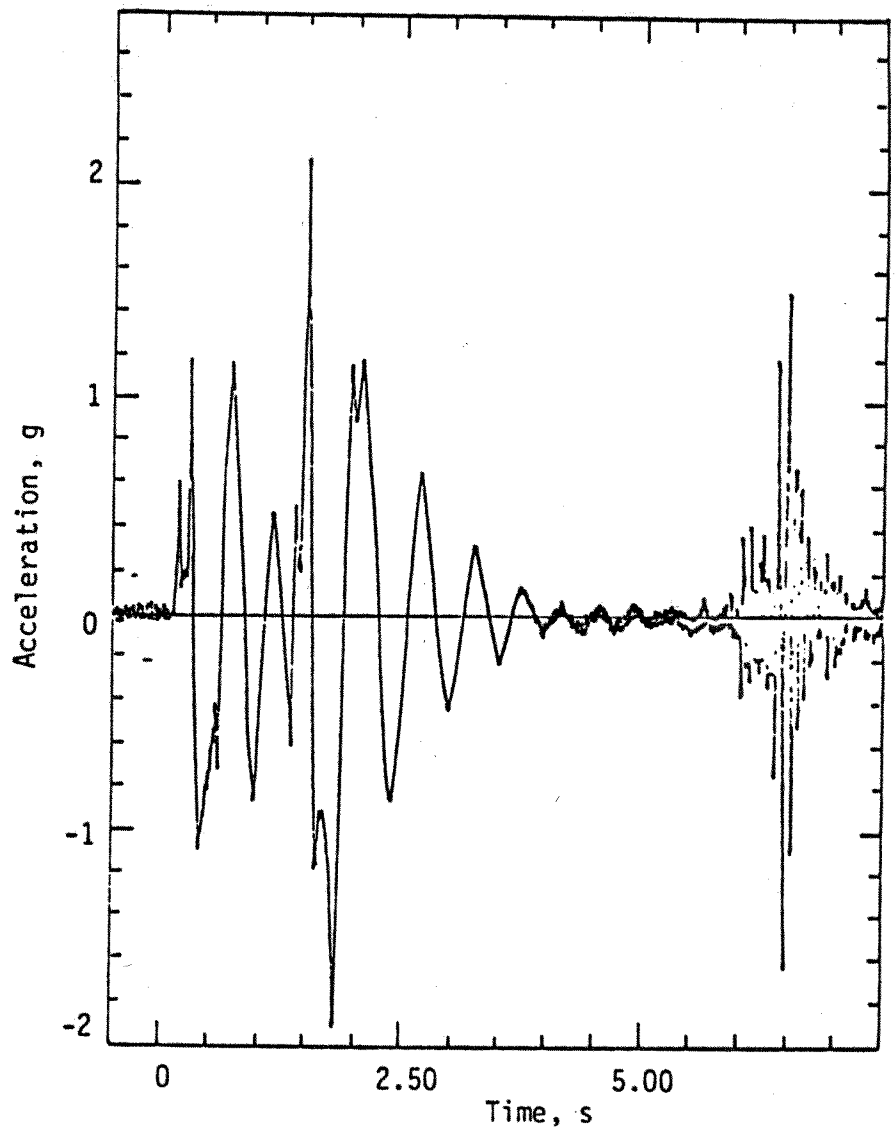
(b) Differential Velocity

(c) Differential Displacement

Figure 8. Differential Horizontal Motion Between the Top and Base of the 1/8 Size Structure on SIMQUAKE IB.



(a) Base Acceleration



(b) Top Acceleration

Figure 9. Horizontal Accelerations in the 1/8 Size Structure in SIMQUAKE II.

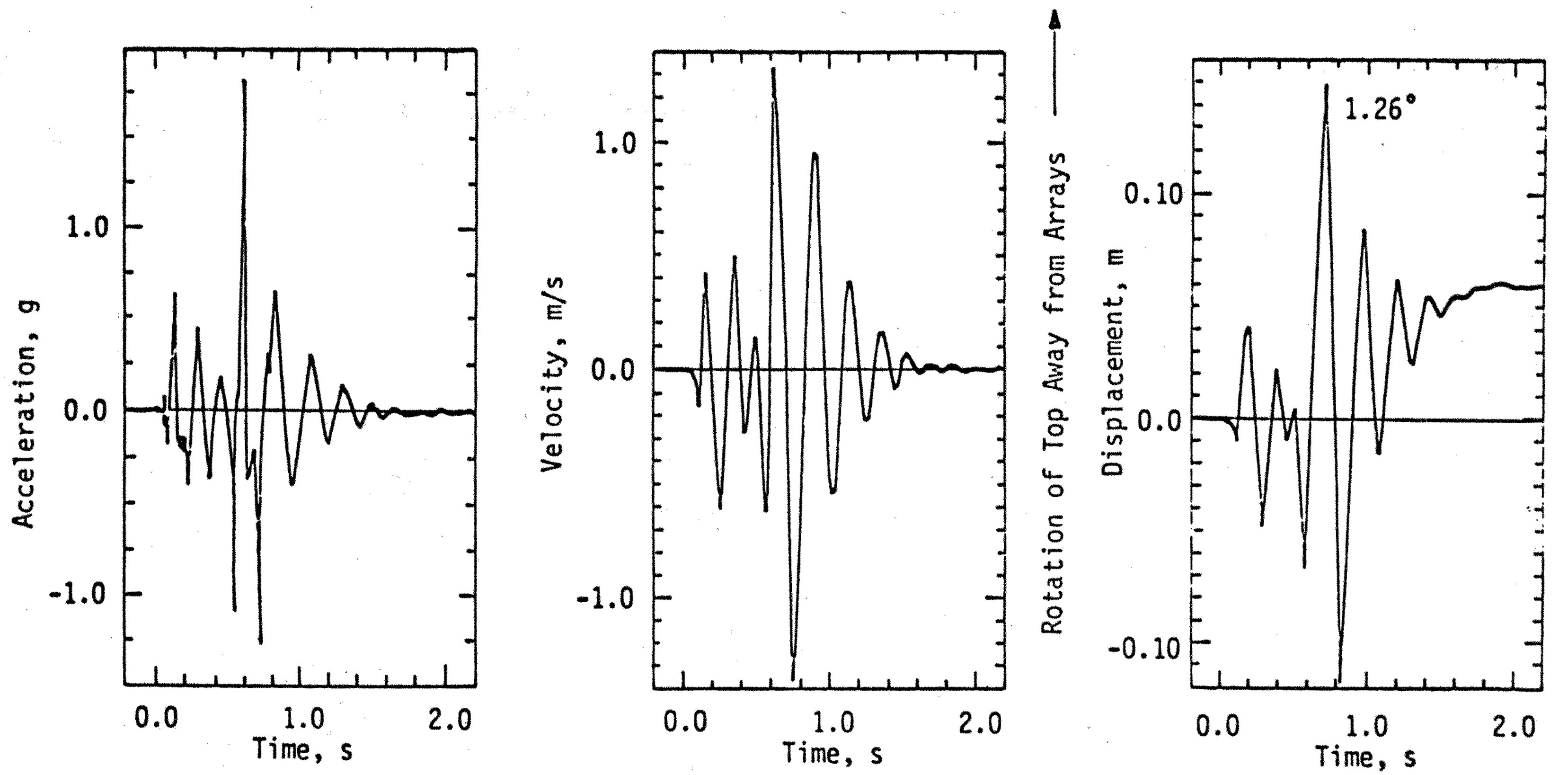
histories of acceleration at the base and top of the 1/8 scale structure. The marked difference in the quantitative and qualitative features of top and base responses indicates clear rocking behavior. The top of the structure overshoots the base response throughout the time history. In addition, while the free-field (see Fig. 4) is quiescent after about 3 s, the structure exhibits a ring down behavior out to 5 or 6 s. It is important to note that both the structure top and structure base exhibit rocking, although the rocking amplitudes at the top are the highest. This indicates that the center of rotation is beneath the structure. This differs from the behavior of the 1/12 scale structure in SQIA and SQIB where the structure base corresponding very closely with the free-field.

Rocking of the 1/8 scale structure is exhibited directly in the differential motions between the base and top of the structure shown in Figure 10. The peak rocking amplitude of 1.25° occurs after the ground motion from the front array arrives. Two ring down frequencies appear in the data. After excitation by the first pulse (back array) the structure rocks at about 3.0 Hz. The ring down after the second pulse (front array) is about 1.9 Hz.

Horizontal response spectra in the top and base of the 1/8 scale structure are compared with each other and with the free-field spectra at the same range in Figure 11. It can be seen that the base response follows the free-field below about 2.5 Hz, and is below the free-field spectra above this frequency. The top response is well above both the base and free-field for all frequencies below about 20 Hz. The pseudo-velocity peak at the structure top is about a factor of two higher than at the base or in the free-field.

All of the data for the 1/8 scale structure indicate that significant rocking behavior was induced by SIMQUAKE II. This behavior was apparently due to a dramatic reduction in the fundamental frequencies associated with rocking behavior due to nonlinear soil-structure interaction in the large amplitude ground motion experiment.

All of the structures exhibited some level of rocking behavior in the SIMQUAKE tests. Table 5 summarizes the rocking responses of all the structures except the SQI structure embedded in sand, and the isolated structure in SQII, which was in a different configuration. The amount of rocking is revealed by the rocking ring down frequency and the values of the rocking parameters. The



(a) Differential Acceleration (b) Differential Velocity (c) Differential Displacement

Figure 10. Differential Horizontal Motion Between the Top and Base of the 1/8 Size Structure on SIMQUAKE II.

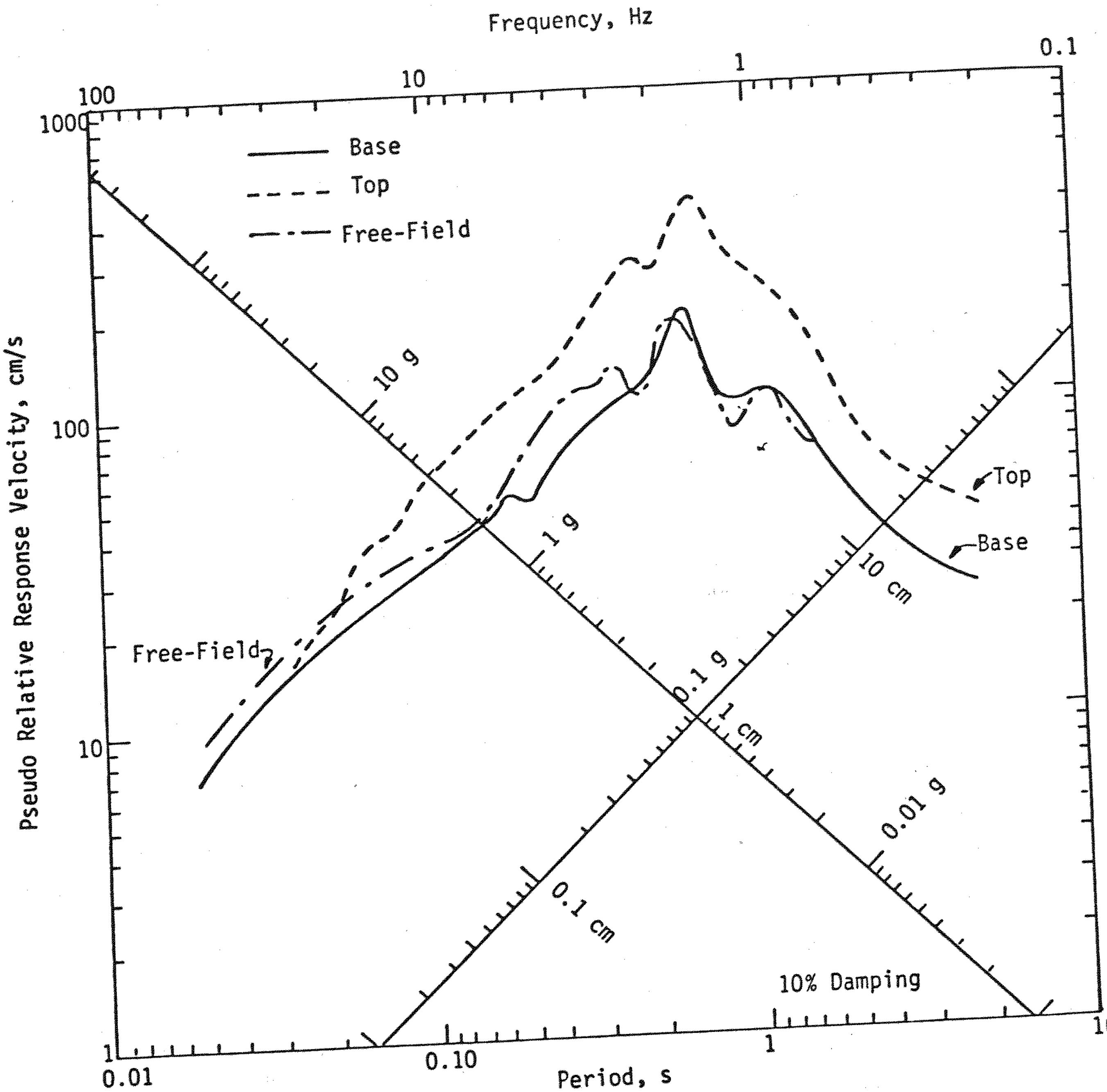


Figure 11. Comparison of 1/8 Size Structure (S01) Horizontal Spectra in the Free-Field, Structure Base and Structure Top.

TABLE 5

ROCKING RESPONSES OF STRUCTURES EMBEDDED IN OR RESTING ON NATIVE BACKFILL

Size	Event	Portion of Event	Small Strain Fundamental Rocking Frequency[1] Hz	Ring Down Frequency Hz	Ratio of Ring Down to Small Strain Frequency	$\ddot{\theta}_{max}$ (deg/s^2)	$\dot{\theta}_{max}$ (deg/s)	θ_{max} (deg)	Free-Field A_h (g)	Free-Field v_h (m/s)
1/8	II	1st Pulse	11.0	3.0	0.27	101	4.63	0.45	0.80	0.57
	II	2nd Pulse		1.8	0.16	300	11.7	1.26	2.3	0.92
1/12	IA	---	15.5	6.5	0.42	207	4.9	0.33	1.1	0.76
	IB	---		3.5	0.23	550	26.5	2.10	3.1	1.29
	II(S02)	1st Pulse	15.5	---	---	163	2.11	0.11	0.80	0.57
	II(S02)	2nd Pulse		4.9	0.32	402	10.0	0.43	2.3	0.92
	II(S06)	1st Pulse	16.3	6.4	0.39	67	2.15	0.11	0.50	0.42
	II(S06)	2nd Pulse		4.8	0.29	224	7.34	0.40	1.40	0.72
1/24	IA(S03)	---	30	9.2	0.31	220	5.3	0.29	1.4	1.0
	IB(S03)	---		5.0	0.17	1145	24.0	1.95	4.2	1.4
	II(S05)	1st Pulse	33.0	---	---	229	2.49	0.08	0.60	0.48
	II(S05)	2nd Pulse		9.8	0.30	312	5.76	0.29	1.70	0.80
1/24*	II(S03)	1st Pulse	13.1	3.8	0.29	279	5.15	0.87	0.60	0.48
	II(S03)	2nd Pulse		2.3	0.18	671	22.11	3.36	1.7	0.80
1/48	MSQ	2nd Pulse	50 Hz	7 Hz	0.14	Not Computed	Not Computed	1.30	5.0	0.90
	IA	---	50 Hz	---	---	---	---	---	---	---
	IB	---	50 Hz	---	---	---	---	---	---	---

*Surface Flush

1. Determined by Impulse Tests Performed by ANCO Engineers, Inc.

rocking amplitudes vary depending upon the size of the structure, the amount of embedment, and the amplitude of the input ground motion. In general, however, the data clearly indicate ring down rocking frequencies well below those estimated using Reference 6 and measured pre-test small strain levels. The larger structures (1/8, 1/12) experienced the largest rocking responses in SQIA, IB, and II, while the 1/24 size structures responded at moderate levels. The 1/48 size structure experienced very little rocking in SQIA and IB, however, it experienced significant rocking in MSQ, where the frequency of the input ground motions was better tuned to the fundamental rocking frequency of the structure.

The SIMQUAKE experiments excited significant rocking behavior in several model structures which were designed to be representative of nuclear power plant containment structures. The rocking behavior occurred at frequencies well below the pre-test frequencies estimated and measured at small amplitudes of excitation. The frequencies in the experiments were in the range of 25-35% of the small amplitude values. Large rocking amplitudes occurred in the experiments because the reduced rocking frequencies (especially for the 1/8 and 1/12 scale structures) were relatively near the frequency of the input ground motions. Comparisons of measured spectra with scaled prototype design spectra indicate that the rocking response may have been on the order of that expected in a large earthquake.

The 1/12 scale structure responses in SQIA, SQIB, and at two ranges in SQII indicate two effects. First, there is a clear correspondence between motion amplitude and the degree of nonlinearity as there appear to be increased rocking amplitudes associated with increased number of motion cycles for the same peak ground motion amplitudes. The experiment also verified differences in the fundamental rocking frequencies which are related to embedment. These differences occur at small and large strain.

Some potential contributors to the nonlinear rocking response are:

- Large ground motion amplitudes leading to large nonlinear soil strains
- Debonding of the soil-structure interface

Debonding can occur due to loss of embedment from irreversible "beat back" of the surrounding soil, and from loss of contact between the structure base and underlying soil. Loss of embedment during the early period of structure response may be at least partially responsible for the significant reduction in rocking frequencies and resulting large rocking amplitudes. Dilation of the near-surface soil may also lead to reduced soil stiffness. Some data in and near some of the structures in SQIB suggest the possibility of tensile failure and resulting dilation of the near-surface soil.

6.0 A SIMPLE PHENOMENA-BASED SSI MODEL

A simple model for calculating SSI of an embedded rigid body was developed from our observations on SIMQUAKE. The model allows evaluation of the influence of various geometric and material parameters on SSI. The method can also be used to rapidly and inexpensively determine structure motions given a free-field ground motion input.

The geometry of the problem considered by the simple model is shown in Figure 12. The structure is assumed to be rigid and may be embedded to any depth (d). The motion of the rigid body is completely described by the three components of motion about its center of gravity (CG). The equations of motion at the CG are:

$$\ddot{V} = \frac{1}{m}(F_V) - g$$

$$\ddot{U} = \frac{1}{m}(F_H)$$

$$\ddot{\theta} = \frac{1}{mr_o^2}(M)$$

where $\ddot{V}$ = vertical acceleration of CG
$\ddot{U}$ = horizontal acceleration of CG
$\ddot{\theta}$ = rotational acceleration about CG
m = total mass of structure
r_o = radius of gyration
g = acceleration of gravity
F_V = total vertical force acting on structure
F_H = total horizontal force acting on structure
M = total moment acting on structure

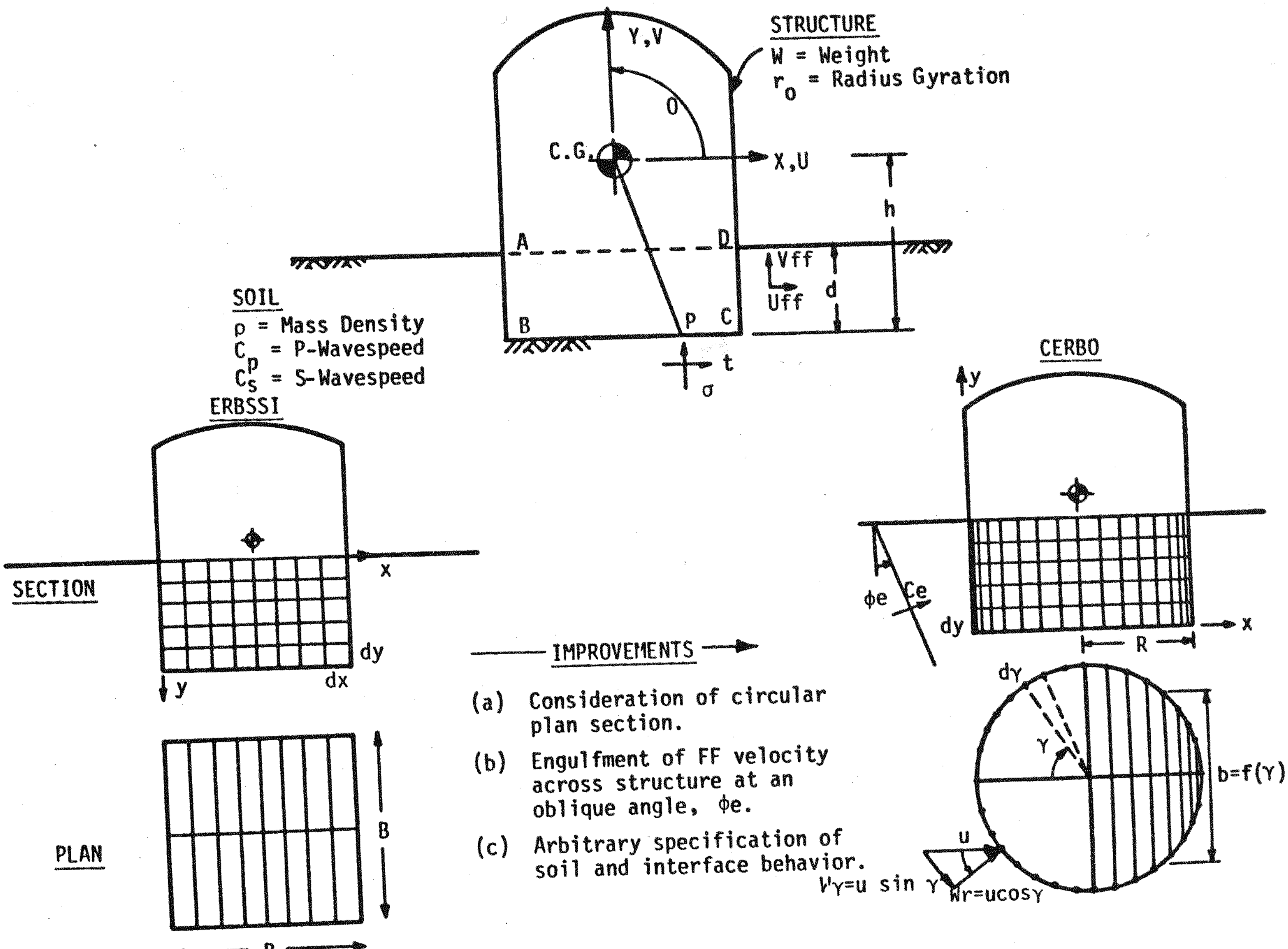


Figure 12. Notation and Geometry for Simple Model Analysis.

The radius of gyration, r_o, is calculated from:

$$r_o = \sqrt{\frac{I}{m}}$$

where I is the total mass moment of inertia about the CG. The external forces acting on the structure are transmitted through the surrounding medium at the sides and base (i.e., the embedded portion). The total forces are determined by discretizing the embedded portion as shown in Figure 12, calculating incremental forces on each element and summing these over the entire embedded surface area. Two stresses are calculated at each element: σ, the normal stress, and τ, the shear stress. Normal stress is not allowed to be tensile, approximating physical separation between soil and structure. Shear stress is limited by a Mohr-Coulomb shear failure condition:

$$|\tau| \leq c + \sigma \tan\phi$$

where c and ϕ are the cohesion and friction angles of the soil-structure interface, respectively. This condition approximates sliding between soil and structure along their interface. Elemental forces are calculated by multiplying each elemental stress by element area.

The stress at any point on the interface consists of the stress in the free-field surrounding the structure plus (or minus) the stress caused by momentum interchange between the free-field and structure as it moves (Ref. 7), or

$$\sigma_p = \sigma_{ff} \pm \sigma_m$$

Normal stress may be expressed in terms of particle velocity by assuming

$$\sigma = M\epsilon$$

where M is constrained modulus and ϵ is strain. Further substitution of

$$M = \rho c^2{}_\ell$$

where ρ = mass density

c_ℓ = compressional wavespeed

and

$$\epsilon = \frac{\mathring{u}}{c_\ell}$$

where $\mathring{u}$ = particle velocity

yields

$$\sigma = \left[\rho c^2_\ell\right]\left[\frac{\mathring{u}}{c_\ell}\right] = \rho c_\ell \mathring{u}$$

then the stress at a point on the interface is:

$$\sigma = \rho c_\ell \mathring{u}_{ff} \pm \rho c_\ell (\mathring{u}_{ff} - \mathring{u}_s)$$

where ff denotes free-field and s denotes structure. The sign of the momentum exchange term depends on the direction of the outward normal to the interface, and is positive for the upstream face and negative for the downstream face. The expression for shear stress is assumed to be entirely analogous to normal stress:

$$\tau_p = \rho c_s \mathring{v}_{ff} \pm \rho c_s (\mathring{v}_{ff} - \mathring{v}_s)$$

using c_s, the shear wavespeed, and particle velocities parallel to the interface in question, $\mathring{v}$.

These expressions appear to be adequate for calculating the stresses during the transient phase of motion, and are consistent with the SIMQUAKE observation that interface stresses are related more to velocity than to acceleration or displacement (Ref. 8). At later times, in order to model rocking ring down, a term considering particle displacement becomes necessary. The expression for stress is then

$$\sigma = \sigma_{ff} \pm \rho c_\ell (\mathring{u}_{ff} - \mathring{u}_s) \pm K (\mathring{u}_{ff} - \mathring{u}_s)$$

where K is a stiffness term with units of stress/length.

In summary, the calculation procedure is:

(1) Discretize embedded portion of structure

(2) Determine free-field velocity (and therefore free-field stress) at locations corresponding to each element

(3) Calculate σ and τ for sidewall and base elements based on current interface and soil properties

(4) Check σ for tensile failure and τ for shear failure

(5) Integrate over embedded surface for total horizontal and vertical forces and moment at the CG

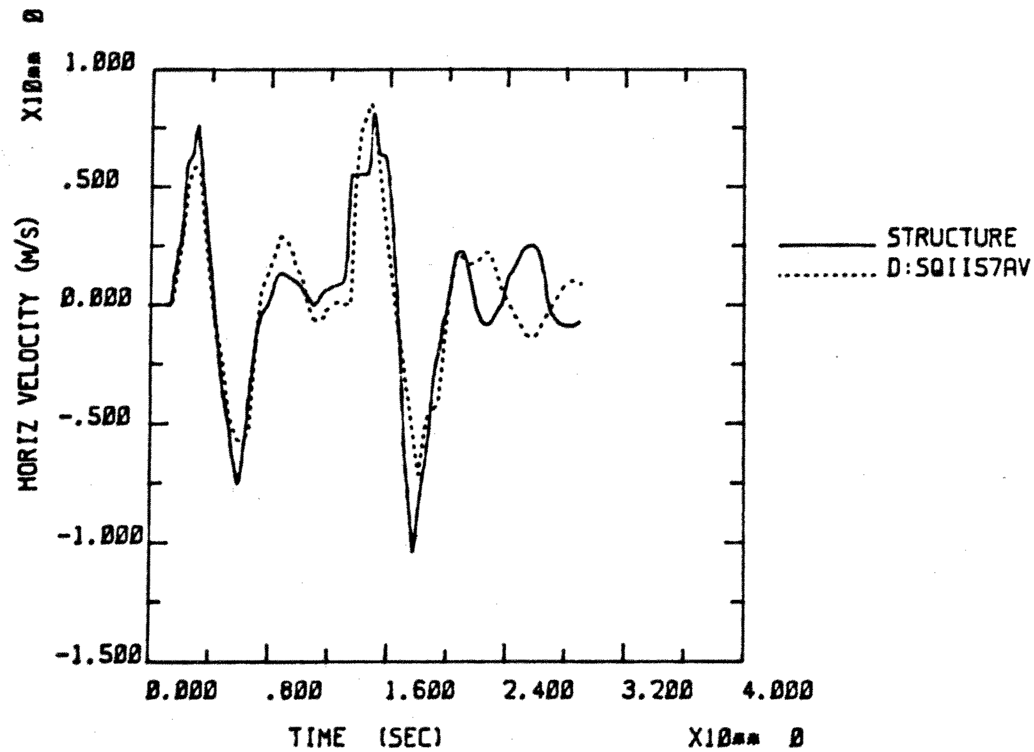
(6) Compute CG motions

(7) Increment time and return to step (2)

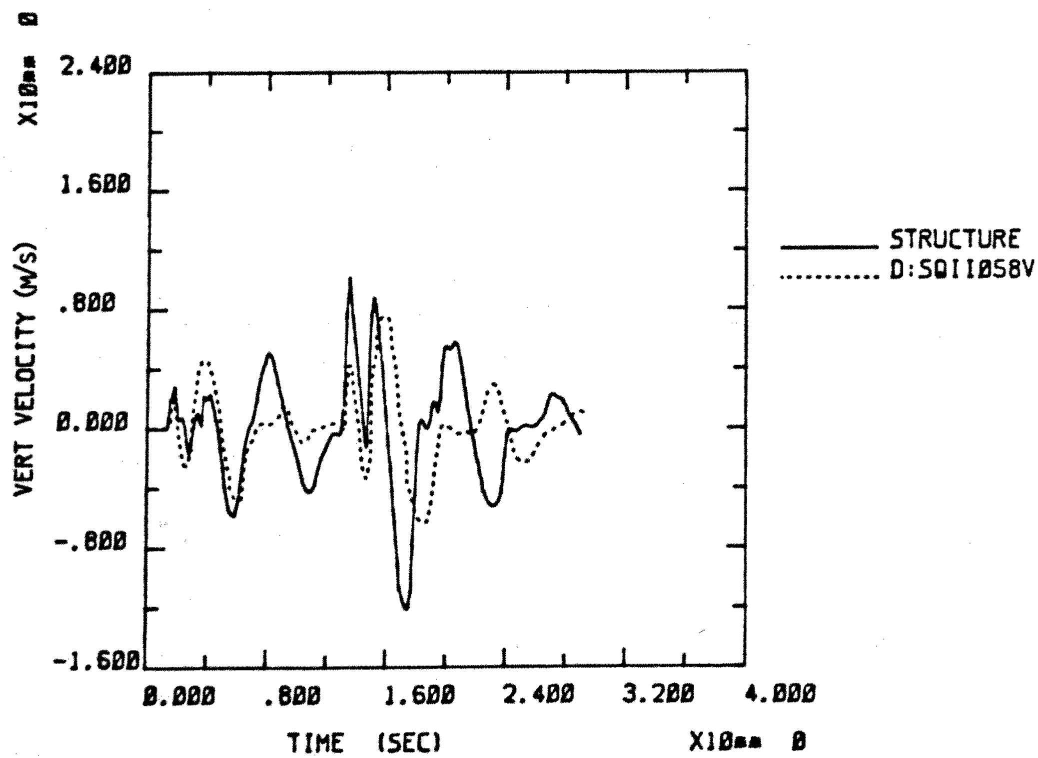
The model features illustrated in Figure 12 include consideration of a circular plan section, engulfment of the free-field velocity field across the structure at an arbitrary angle, and arbitrary specification of soil and interface behavior to include nonlinear effects and stress-history dependence.

Calculations performed with the model to date have focused on the SIMQUAKE I and II 1/8 and 1/12 scale model containment structures, although the model is applicable to full size containments as well. In all cases, the input velocity history which drove the calculation was chosen to be the free-field data closest to the base of the structure of interest. Figure 13 shows the calculated response of the 1/8 scale structure in SQII at the base, compared with the measured base response. A better measure of calculational adequacy, shown in Figure 14, is the rocking response, as represented by subtracting horizontal velocity at the base from that at the top of the structure. The comparison of calculation and data here is fairly good, with some significant differences in the later part of the first array pulse motion. Figure 15 shows calculated and measured interface stresses at the upstream base of the structure. Both the spring and free-field velocity (momentum interchange) components of calculated stress are broken out in the figure. Again, correspondence between calculation and data is fairly good.

Various parameters can be varied in the model to study SSI phenomena. For example, the effect of embedment on rocking behavior is shown in Figure 16. Increased embedment causes lower rocking velocity and a shift toward higher frequency of motion. One hundred percent embedment greatly inhibits rocking, as would be expected.



(A) Horizontal Velocity



(B) Vertical Velocity

Figure 13. Calculated Base Response of SQII-S01 vs Data.

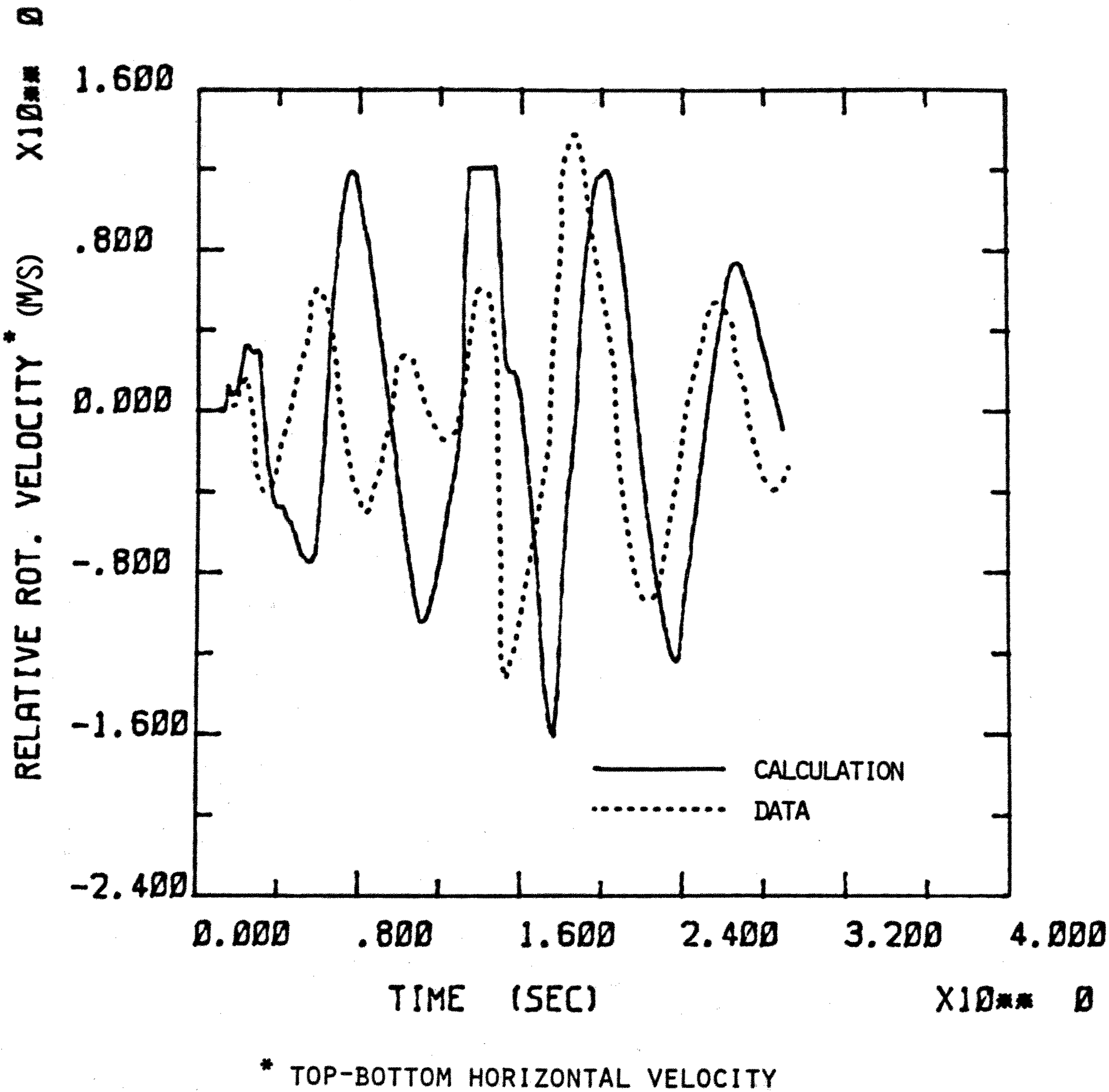


Figure 14. Calculated Rocking Response of SQII-S01 vs Data.

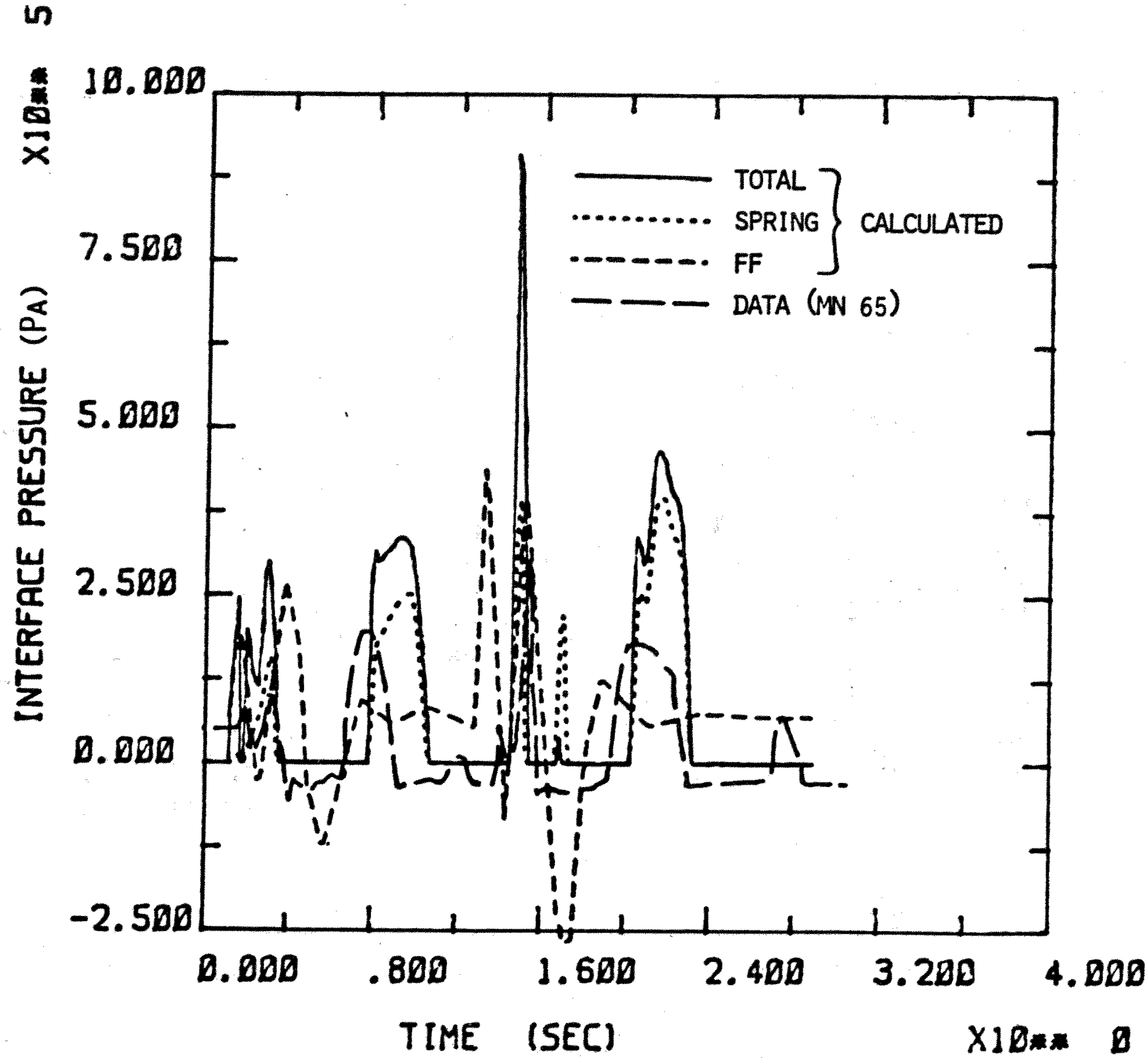


Figure 15. Calculated vs Measured Interface Pressures.

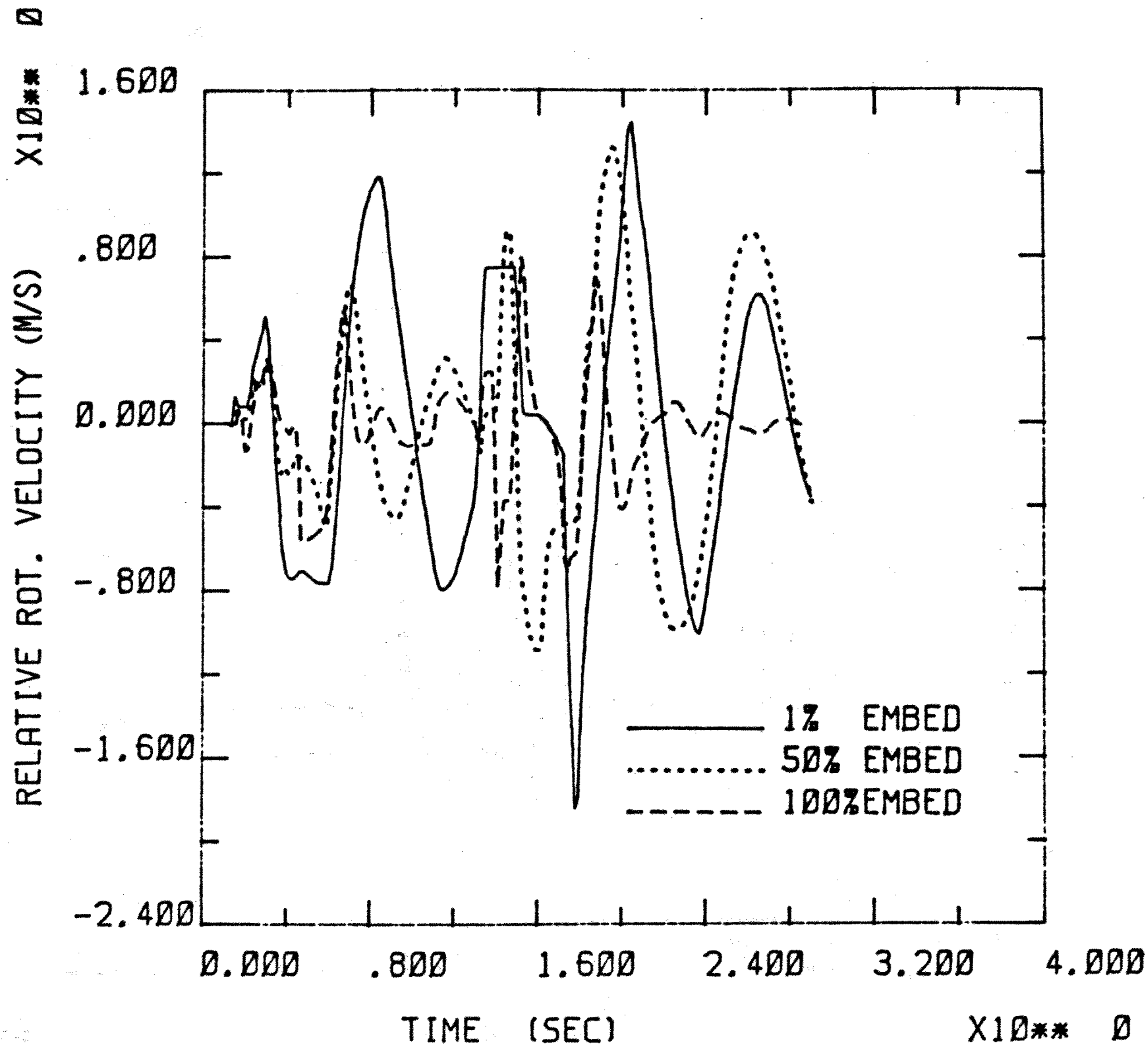


Figure 16. Calculated Effect of Embedment.

Finally, Figures 17 and 18 compare the response of the 1/8 scale model in SQII with the simple model and with a DYNA3D nonlinear three-dimensional calculation. The simple model does a better job than DYNA in the transient phase. Since DYNA was not run into the ring down phase, it cannot be evaluated there. The simple model does not model the ring down well. As noted earlier, a displacement term (spring) is needed in this phase to model the ring down response.

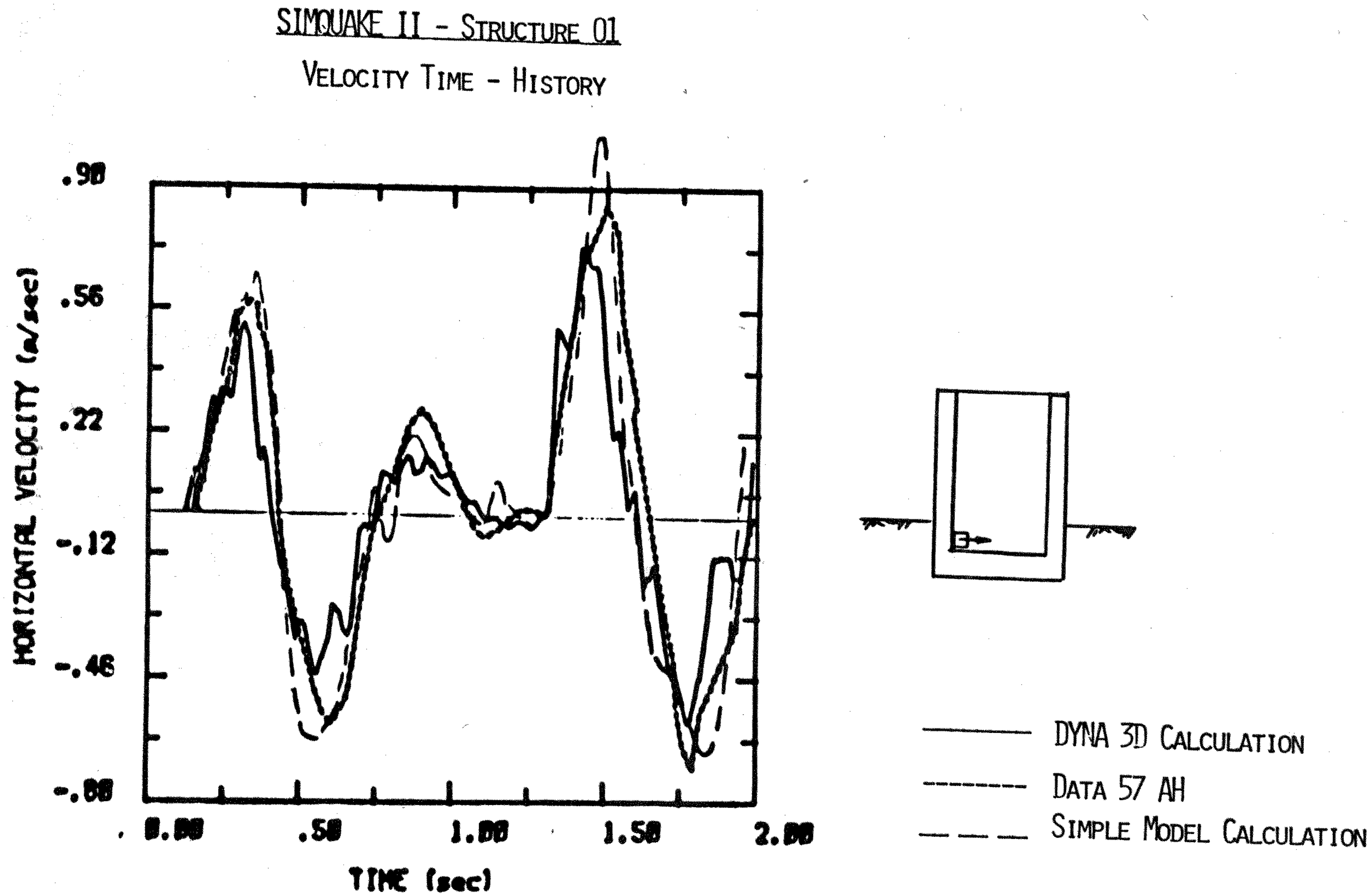


Figure 17. Comparison of Simple SSI Model, DYNA 3D, and Base Data for 1/8 Scale Structure in SQ II.

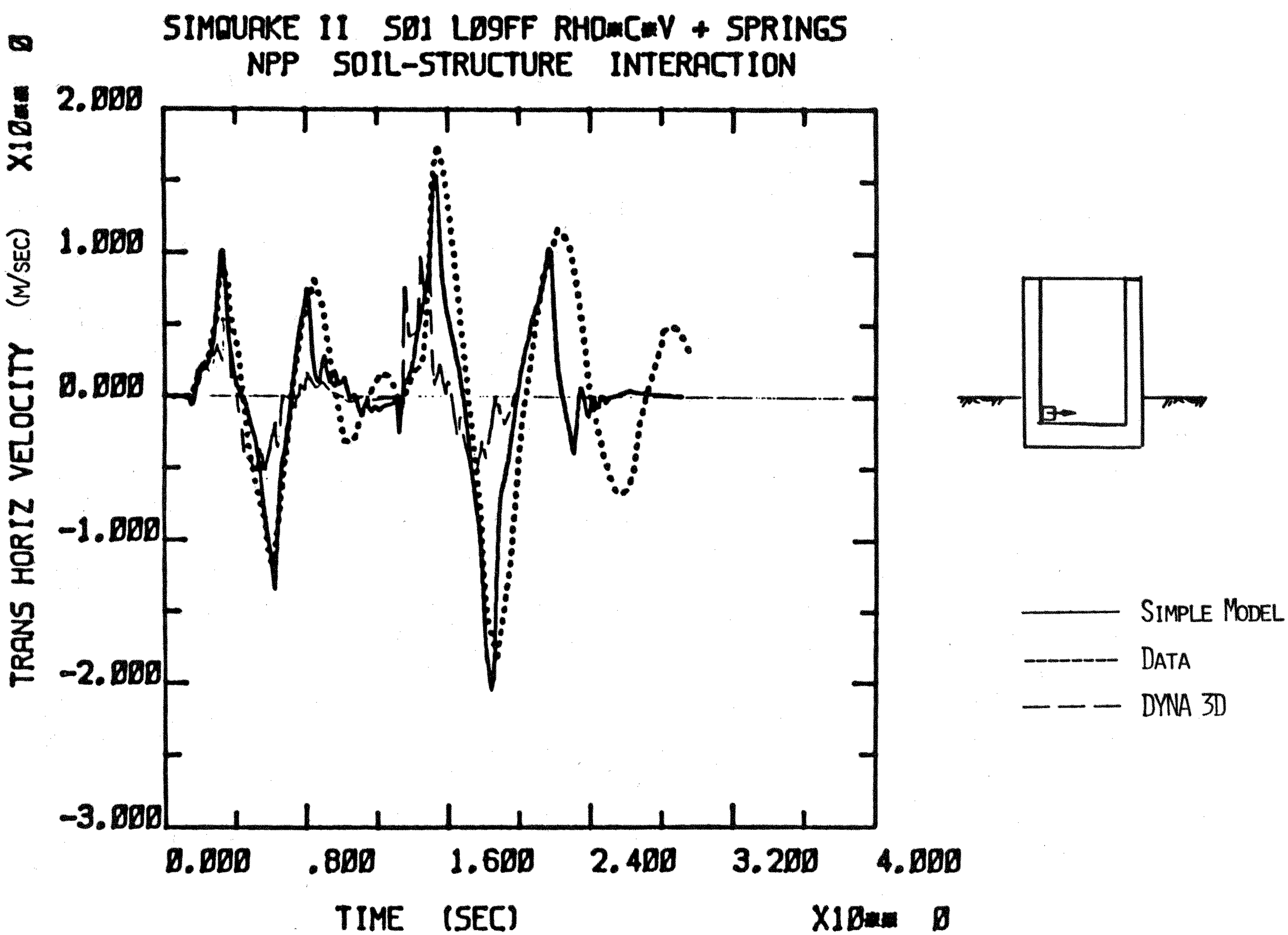


Figure 18. Comparison of Simple SSI Model, DYNA 3D, and Top Data for 1/8 Scale Structure in SQ II.

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Role of Experiments in Soil-Structure Interaction Methodology Verification

by

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The draft of proposed NRC revisions to the Standard Review Plan (SRP) notes regarding Soil-Structure Interaction (SSI) analyses that such analyses must recognize the uncertainties prevalent throughout the phenomenon. The present paper discusses the role of experimental data that might help resolve or reduce the uncertainties in the modeling of SSI. Though there is a generic aspect to some of these uncertainties, most of them arise predominantly from lack of knowledge of soil behavior and are thus site-specific in character. Therefore, it is obvious that data from experiments performed at one site cannot be directly extrapolated to resolve particular SSI issues pertaining to a different site. The value of experiments primarily lies in their utility for verifying and improving modeling techniques and for establishing a database on parameters affecting SSI. The main criterion here in evaluating the utility of experimental data is how the data relates to the SSI issues noted in the proposed revisions to the SRP.

Experimental data includes recorded building/site response of full scale (as-built) structures and scale models founded on soils to naturally occuring ground motions or artificial dynamic excitations. We may classify the experiments into five categories and list them in the order of decreasing utility, (from the SSI analysis point of view) as follows:

1. Recorded structural and site response of large as-built structures to natural earthquakes.

2. Recorded structural and site response of large scale-model structures founded on soil subjected to natural earthquakes.

3. Recorded structural and site response of large as-built structures to artificial dynamic excitation.

4. Recorded structural and site response of properly scaled model structures founded on soil subjected to artificial dynamic excitation.

5. Recorded response of scale models of soil-structure systems to artificial excitation in laboratories. (Scaled structures that are not founded on soil are not included).

The first category, the most useful for verifying analytical techniques, is perhaps the scarcest of all. This is in spite of the fact that a large number of buildings and other structures have been instrumented to record their response to natural earthquakes. Many instances of such recorded response were briefly reviewed by the present authors in a study sponsored by the USNRC.[1] As noted in that review, the greatest inadequacy of much of this data from the present point of view is the lack of free field records of ground motions. The norm in most tests was two response records in the structure, one at the base and the other at the top. The utility of the single response record of motion of the base (even if one presumes it to be purely translatory) for defining the input motion in SSI analysis is questionable. Only an indirect and partial verification of SSI analyses through a comparison of analytical modal parameters with those determined from recorded data may be feasible in most such cases. Even when feasible, such partial verification does not help resolve uncertainties in specific SSI aspects. Extensive ground motion measurements in the free field - both surface and downhole - together with structural response measurements are necessary for such data to be useful for SSI analysis verification. The existing data may, however, be of some value in adding to the database on the dynamic characteristics such as composite modal damping of as-built structures.

A notable exception to the above is the Fukushima data which includes structural and site response of the containment building of Unit 1 of Fukushima nuclear power plant to the Miyagiken-oki earthquake of 1978.[2] Even the somewhat limited data available to USNRC led to a successful effort on verification of SSI methods.[3] As such data enables us to verify the entire methodology of SSI analysis, more such data from different sites, representing a variety of soils, is needed.

The second category of experiments, i.e., earthquake response of large scale models (e.g., 1/2 or 1/4 scale) built on typical sites, can generate data almost as useful as those from the first. They have the added advantage that they can be built at sites that are seismically very active, thus ensuring the generation of required data. The main difficulty here is in the proper 'scaling' of the model so that it will have measurable response at the frequency ranges containing most of the energy of the seismic excitation. The only example of existing data that the present authors are aware of are those obtained from a scale model built on site in Lotung, Taiwan in an EPRI/TAIPOWER cooperative project. The 1/4-scale model of a prototype nuclear power plant containment building was designed such that its lower natural frequencies lie within the frequency range of the local earthquakes. The model is located within the SMART-1 array of Taiwan. The structure and the free field (surface and downhole) are extensively instrumented. Earthquake response data is being collected beginning from late 1985. The response data collected will be used in a NRC/EPRI cooperative program on the verification of SSI analysis methodologies. It is expected that this data will be very appropriate for verifying the two major aspects of SSI analyses, i.e., spatial variation of ground motion and the response of the soil-structure system.

The third category of experiments is perhaps the most widespread. A large number of as-built structures have been subjected to dynamic testing with a variety of excitations. The present authors have assessed dynamic testing of large structures including nuclear power plant containment buildings[1,4] in previous NRC-sponsored studies. Unlike in the first category of experiments, the excitation in this category (with the exception of the use of buried explosives for excitation) is directly applied on the structure thus reversing the interaction path in a ground-motion induced structural-response situation. This imposes limitations on the use of data from this category of experiments for verifying SSI analysis methodologies. In addition, the fact that most of the dynamic tests produce structural responses that are very low compared to earthquake induced motions have also raised questions about the applicability of these data to SSI analysis verification.

In spite of these limitations, rejection of all data from this category of experiments for SSI analysis verification is not warranted because at least some selected data are useful for verifying specific aspects of SSI analyses. First, dynamic tests on large foundation slabs (prototype or scale model) can be used for verifying analytical modeling of soil-springs or impedance functions. With only the basemat of the 1/4-scale containment model (built by EPRI/TAIPOWER in Taiwan) complete, NRC-sponsored vibration tests were performed on the basemat in the NRC/EPRI cooperative program. The response of the basemat, measured at five locations on it, will be used to evaluate response predicted by different SSI-analysis methods including those incorporating soil-spring or impedance function formulations. Alternatively, experimental impedance functions may be obtained from test data and can be used to evaluate analytical predictions of these functions. The authors are aware of at least one Japanese project to do this with many different foundation sizes. However, the information is proprietary. Specific effort is needed to collect information on other sources of similar data that may be available for verification purposes.

Secondly, data from past dynamic tests on as-built structures are useful for establishing a database on (a) composite modal damping values, (b) variabilities of experimentally determined design parameters, and (c) modeling uncertainties. Not all published information/data from the numerous tests in the past may be used for this purpose. For instance, in many tests, the identification of modal parameters from the test response was not performed through a rigorous analysis. In most cases, the damping estimates were not as reliable as natural frequency estimates. Despite these caveats, a large body of good data exists - some of which is noted in Ref. 1 - and a systematic effort could generate a useful database from this.

The specific case of the dynamic tests of the HDR containment building will be briefly discussed here. The containment building of the HDR (a decommissioned nuclear power plant in Germany) was subjected to dynamic testing at relatively low levels of excitation with a variety of means of loading during 1975-79. Currently, in the second phase of testing, relatively high-level excitation tests are being conducted.

In the first phase[5,6], the containment was excited in separate tests with (a) rotating eccentric mass shakers installed on the operating floor (steady-state sinusoidal forcing), (b) blasting of small buried explosive charges of up to 10 kg (impulsive forcing), and (c) reaction rockets mounted on the dome of outer concrete structure (impulsive forcing). Tests at different force levels were performed with each type of excitation. Associated with these tests, a large number of pre- and post-test analyses were performed. Analytical models of different levels of complexity were exercised to make predictive calculations. The test data was analyzed to identify modal parameters and was used to evaluate many predictive analyses. A vast data bank exists that contains the test data and information on the analytical models and results.

Phase II of the HDR testing again involves different excitation methods. Impact tests, using a large pendulum (20,000 kg), on the concrete shield building of the HDR were recently completed, the objective being to study the effects of severe local loading and load transmission. In experiments that are now underway, a large shaker (capable of a peak force of 2000 tons) is installed on the operating floor of the HDR containment building. Coastdown tests, in which the shaker is brought to a desired maximum frequency in a balanced condition and then allowed to coastdown with an eccentric mass, will be performed with many starting frequencies. Peak responses in the containment building are expected to reach about 0.5 g. These high-level tests have also given rise to a number of pre-test predictive analyses and many post-test analyses are planned. When completed, Phase II data will augment Phase I data and enhance the utility of the data for establishing a database.

Systematic selection and analyses of Phase I and Phase II data could provide information on (a) modal damping values, their variability with level of excitation, and the range of random variations at a given level of excitation (b) variability of other design parameters - e.g., soil material properties (from the different soil tests) - both random and systematic, and (c) modeling uncertainties, i.e., the deviation of different predictions from measured responses and the correlation of the deviations with modeling assumptions.

The fourth category of experiments consists of artificial excitation of scale models of structures founded on soil in the field. It has been shown by Krawinkler[7] that it is not possible to simulate gravitational, inertial, and restoring forces simultaneously using true replica models in a 1-g environment. This means that buildings and soils cannot be scaled properly in the field. The models of the SIMQUAKE series of tests[8] in which 1/8 to 1/48th scale model containment structures, built on soil in the field, were subject to blast-excitation thus did not satisfy all the similitude requirements. The SIMQUAKE tests have been criticized[9] also because the soil experienced very high ground motions in the tests due to the attempt to scale prototype accelerations. The severe deformations imposed on the soil by these high accelerations are noted to have invalidated conclusions regarding SSI. It is apparent that these tests are difficult to perform if one attempts to simulate earthquakes.

There is another type of scale model, - built in the field on soil and subjected to direct excitation of the building, - that may yield data as useful as those from full-size, as-built structures subjected to dynamic forcing. These scale models would be large structures in their own right and it is not unreasonable to assume that the SSI phenomena in the tests are qualitatively the same as those in the dynamic excitation of prototype systems, though no special effort is made to satisfy similitude laws in performing the tests. The vibration testing of the 1/4-scale containment model in Taiwan, sponsored by USNRC, has yielded data that will be used to verify some of the modeling techniques that are employed in SSI analyses. The tests on the scale model consisted of radial and tangential steady-state excitation of the structure in the frequency range of 1-30 hz by an eccentric-mass shaker located on the roof of the building. Twenty channels of response acceleration of the structure was recorded in the test. The data from these tests will be used for partial verification/improvement of methodology by many of the participants in the NRC/EPRI program on verification of SSI methods.

The fifth category of experiments are scale-model tests of soil-structure systems in laboratories. For these tests to be useful, either all the similitude laws must be satisfied or the violations of such laws must be taken into account in interpreting the data. Krawinkler[7] has noted the use of large centrifuges in the USSR to augment gravitational acceleration for performing model studies. Hadjian[9] has advocated greater use of centrifuge testing compared to scale models built on soils in the field, though he notes possible difficulties with boundary conditions in the former. The use of a centrifuge to augment gravity to 100 g to study a 1/100 scale model of a frame structure founded on piles in a soil body (of 1.3 m x 8 m x 4 m depth) was reported by French authors.[10] The earthquake signal was simulated by air pressure from a series of programmed small explosions. The flexibility matrix for the foundation was determined from experiments. To satisfy similitude laws, the accelerations had to be 100 times the prototype acceleration. The authors have not discussed the effect of such high accelerations on the soil behavior. They state that such centrifuge tests are very useful for obtaining foundation flexibility matrices.

Summary and Conclusions

Different kinds of experimental data may be useful for partial or full verification of SSI analysis methods. The great bulk of existing data comes from earthquake records and dynamic testing of as-built structures. However, much of this data may not be suitable for the present purpose as the measurement locations were not selected with the verification of SSI analysis in mind and hence are too few in number or inappropriate in character. Data from scale model testing that include the soil in the model - both in-situ and laboratory - are relatively scarce. If the difficulty in satisfying the requirements of similitude laws on the one hand and simulating realistic soil behavior on the other can be resolved, scale model testing may generate very useful data for relatively low cost.

The current NRC sponsored programs, - in HDR together with PHDR/KfK and in Taiwan together with EPRI - is expected to generate data very useful for verifying analysis methods for SSI.

A systematic effort to inventory, evaluate and classify existing data - both domestic and foreign (available through exchange programs) - is first necessary. This effort would probably show that more data is needed for the better understanding of SSI aspects such as spatial variation of ground motion and the related issue of foundation input motion, and soil stiffness. Collection of response data from in-structure and free field (surface and downhole) through instrumentation of selected as-built structures in seismically active regions may be the most efficient way to obtain the needed data. Augmentation of this data from properly designed scale model tests should also be considered.

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Experimental Verification of Soil/Structure Interaction Methods

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Abstract

This paper discusses some of the sources of experimental data that are available to be used on soil/structure interaction (SSI) verification studies. The data is discussed in relation to its applicability to the primary issues of SSI. Results of verification studies using two of the data sources are also presented. One data source is data collected at the Fukushima nuclear power plant during an earthquake. The second data source is the SIMQUAKE experiments during which earthquake loading was simulated with high explosive charges.

Introduction

The work reported in this paper is based on a study conducted at Brookhaven National Laboratories (BNL) under the sponsorship of the Structural Engineering Research Branch of the Nuclear Regulatory Commision. The results of that study have been reported in NUREG/CR-4182 (Ref.1). The objectives of the study were to locate and utilize data sources to validate soil/structure interaction (SSI) methods. Applied Research Associates (ARA) were a subcontractor on this program, and their efforts were focused on identifying the potential sources of data. Dr. Higgins of ARA will present a paper at this workshop describing their results.

The methods of performing SSI computations are first reviewed with the objective of identifying the major sources of uncertainty. These uncertainties are then correlated with available data sources so that a general view of the degree of validation that is currently possible may be made. This correlation is also useful in establishing a "wish list" of experiments that are required to resolve the major issues.

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Two data sources were used to validate the SSI methods. Data collected at an operating nuclear power station (Fukushima in Japan) during an earthquake, and data collected at the SIMQUAKE high explosive simulation tests were used to correlate with SSI methods. These studies are summarized in this paper.

SSI Uncertainties

There are two generally accepted methods of performing SSI computations, the substructure and finite element methods. The substructure method should be divided into two. The lumped parameter method has historical significance and is still the simplest method to use. The half space method is a new refinement of the lumped parameter method and is rather complicated to use. Each of these substructuring methods and the finite element method are discussed with the goal of identifying the major uncertainties.

The lumped parameter method is based upon a finite element (or lumped mass "stick") model of the structure coupled to the free field with springs and dampers intended to represent the SSI process. A general discussion of this method is given in Ref. 2. Both vertical and horizontal free field motions are input to the base of the interaction spring-damper models. A very fundamental assumption of this method is that motion of the structure does not effect the free field motion (i.e., the effect of scattering of the incident waves by the rigid foundation is neglected). It is argued that the structural motion may influence the very near field motion but will not seriously alter the free field motion at reasonable distances from the structure. This is an uncertainty of the lumped parameter method. The heart of this method lies in the selection of the parameters of the SSI model. Six parameters are required representing the interaction spring and damper in the vertical, horizontal, and rocking directions. The values used to represent these parameters are usually based on solutions to the steady state vibration of a rigid foundation on an elastic half space. The applicability of these solutions to real site properties (e.g., non linear soil stiffness and damping, layering, water table effects) represent uncertainties which should be considered. The interaction coefficients are functions of frequency. Several studies have been performed however to show that, over the frequency ranges of interest in the design of nuclear power plant facilities, average frequency independent solutions may be used. Other uncertainties of the method deal with the effect of embeddment and foundation flexibility.

The half space method is identical to the lumped parameter method except that it treats the scattering of the seismic waves by the structure. A three step method was proposed by Kausel and Roesset (Ref.3). The first step requires a solution for the motion of a rigid massless foundation subjected to the free field motion. This can be carried out with a finite element model of the free field which contains the rigid foundation. The finite element model is subjected to some control free field motion. This is the solution for the scattering problem which is neglected in the lumped parameter method. The second step involves the generation of the SSI interaction coefficients as was done for the lumped parameter problem. The third step involves the computation of the structural motion when it is connected to the free field with the impedance functions generated in step 2 and subjected to the foundation motion generated from step 1. The total structural motion is the sum of that calculated in steps 1 and 3. Uncertainties of this method are similar to those dicussed above which are based on the soil and interface conditions.

The finite element method is based on the use of a single model to represent both the free field and the structure (Ref.4). The free field is modeled with finite elements and the structure may be represented with either a finite element or a "stick" model. Because of the size limitations of current computer hardware and software, these models are usually taken as two dimensional slices through the facility and free field. The free field model is then bounded by the surface and three artificial boundaries, a horizontal boundary at bedrock or at some reasonable depth for very deep sites, and two vertical boundaries at some distance from the facility. The input seismic motion is specified at the bottom boundary. The major uncertainties with this method lie in selecting appropriate mesh sizes and boundary conditions along the artificial boundaries. Of course there are also uncertainties arising from nonlinear soil properties.

The uncertainties discussed above are summarized in Table 1. This table includes a list of some of the available experimental data that may be used for correlation studies to resolve the uncertainties. A more complete discussion of the available data sources is given by Higgins in another paper at this session of the workshop. The Fukushima data is attractive because it represents a power plant subjected to a high intensity earthquake. The site at Fukushima is rather stiff and uniform however, so that it represents an ideal situation in which SSI errors may be less significant that at other sites.The HDR is a power plant but the loading for which data is available is only vibratory.The SIMQUAKE

Table 1
UNCERTAINTIES IN SSI METHODOLOGY

Uncertainty	Experiment				
	Fukushima	HDR	SIMQUAKE	Taiwan	DOD
Scattering Effects [1]	X			X	X
Frequency Dependence [1]	X	X	X	X	
Embeddment Effects [1,2]			X		X
Layering [1,2,3]					X
Water Table [1,2,3]				X	X
Non Linear Soil [1,2,3]	X	X	X	X	X
Liftoff/Separation [1,2,3]			X		X
Mesh Size [2,3]	X	X		X	
Boundary Effects [2,3]	X	X		X	
Convolution [1,2,3]	X			X	

Notes: 1 Applies to lumped parameter method
2 Applies to half space method
3 Applies to finite element method

experiment was conducted using scale model containments. The loading was induced with high explosive and has temporal characteristics which are quite different from an earthquake. A scale model containment has been constructed in Taiwan at a site that is subjected to frequent earthquakes. This will represent an excellent source of data. The Department of Defense (DOD) conducts many tests in support of their missile ground facilities program. Their tests are conducted in a variety of sites so that this data could be useful in assessing the effect of site properties on SSI. Of course their structures are different from nuclear power plant structures and the level of input is significantly higher than a seismic input. It may be possible to "tag along" on future DOD tests and place models at the DOD test sites in regions where the level of the input has been sufficiently attenuated to be representative of a seismic input.

The (X) in Table 1 indicate experiments that may be useful in evaluating the uncertainty. Certainly many of the correlations will not be complete. It is clear that additional test data are required where site conditions are not uniform (e.g., layering including non horizontal, water table effects) and where non linearities in site properties are important.

Correlations with Fukushima Data

The Fukushima nuclear power station contains two BWR reactors, and it is located on the coastline in northern Honshu about 100 km south of Sendai, Japan. The reactor building is founded on mudstone (shear wave velocities vary from 180 mps at the surface to 530 mps at a depth of 10 meters). The building is about 40 meters square in plan and has a height of 59 meters. It is embedded 14 meters. The building is reinforced concrete up to the operating floor (the lower 43 meters). The upper part of the building is steel frame. The building and surrounding free field contain eight accelerometers located as shown in Figure 1. These accelerometers give sufficient data so that one may obtain a good description of both the free field and in-structure motions.

The Miyagiken-Oki earthquake occured on June 12,1978. No damage was reported at the plant but there was considerable structural damage reported at Sendai which was close to the epicenter. The earthquake had a magnitude of 7.4 as measured on the Richter scale. The data measured at the plant for this earthquake were obtained on a magnetic tape from Dr. H. Tanaka of Tokyo Electric Power. Response spectra for the measured free field histories are shown on Figures 2-5. A comparison of the -14 M and -40 M spectra show the motions to be quite similar for frequencies above 5 cps. The -14 M spectra has a significant peak at 2.6 cps which is not evident in the spectra for the motion at the deeper depth. The spectra for the motion at the basemat level shows an additional amplification of this 2.6 cps content. Note that the peak spectral value at 2.6 cps for the gage located to the side of the facility does not show this amplification. It is concluded that the 2.6 cps amplification in the vicinity of the structure is due to the interaction process. The "rigid structure" interaction freuencies are 3.6 cps and 11.0 cps. One would expect amplification of spectral peaks below 3.6 cps.

The spectra for the four in-structure measurements are shown on Figure 6.The peak measured accelerations (ZPA) are ,from the basemat to the top: 0.08 G's; 0.14 G's; 0.20 G's; and 0.60 G's. Note that the first three measurements are in the concrete portion of the structure with the large amplification occurring in the steel portion. The free field ZPA diectly under the basemat is 0.08 G's while the free field ZPA measured to the side of the structure is 0.07 G's. The structural spectra shown peaks at 2.0 cps, 2.6 cps, 3.5 cps, 4.8 cps, and 11 cps.

In-structure amplifications at these peaks based on the free field input to the side of the structure are shown on Table 2.

Table 2
AMPLIFICATION OF FREE FIELD MOTION

Frequency (cps)	Amplification at Structural Location Basemat	Elev. 25.9 M	Operating Floor	Top
2.0	2.2	2.8	3.3	4.4
2.6	1.8	2.8	3.3	6.0
3.5	1.2	3.3	5.2	13.0
4.8	1.0	1.5	1.5	8.5
11.0	1.0	1.0	1.0	5.0
ZPA	1.3	2.1	2.9	8.9

The first eight coupled interaction/structural frequencies vary from 2.5 to 14.8 cps. The interaction translation mode is primarily at 2.5 cps while the primary rocking interaction coupled mode is at 12.6 cps. It therefore appears from the amplification data in Table 2 that the rocking interaction has an effect on the steel portion of the structure but a much smaller effect on the lower concrete portion.

A lumped parameter solution was found for the facility using the measured pulses to the side and directly beneath the basemat. A comparison of the spectra from the calculated and measured responses are shown on Figures 7 and 8. The best correlation between measured and calculated spectra, with the input to the side of the structure, was found when the standard interaction springs and 75% of the standard interaction dampers were used. The comparison shown on Figure 7 indicates a fairly good correlation. It should be noted that this example is most representative of the manner in which the lumped parameter method is used. The required reduction in the interaction damping coefficients indicates that the standard method would be unconservative. The best result, for the input pulse at the basemat, was found using the standard interaction parameters. The correlation between measured and calculated spectra (see Figure 8) is not as good as for the first case. There are significant frequency shifts that are not as significant in the comparisons shown on Figure 7.

Correlation with SIMQUAKE Data

The SIMQUAKE II experiment was conducted on June 2, 1977 at the University of New Mexico's McCormick Ranch test site, which is located south of Alberquerque New Mexico. The test was conducted by the Civil Engineering Research Facility of the University for EPRI. The data utilized in this paper was obtained from a magnetic tape containing all of the recorded data This tape was obtained from EPRI through Applied Research Associates.

Scale (1/8 to 1/24) model containments were located at the test facility and subjected to ground motions induced with high explosives. The site conditions are rather uniform. The surface shear wave velocity is about 800 fps while the shear velocity at 25-90 feet is about 1100 fps. The loading was induced.with two plane, high explosive arrays each covering a vertical plane about 200 feet wide and 75 feet deep. The arrays were separated by about 100 feet and began about 25 feet below the surface. The structural models were placed along a line normal to the plane of the arrays.. The closest structure was 200 feet from the front array while the furthest structure was 250 feet from the arrays. The arrays were fired in sequence so that the free field motions consisted of two peaks separated by about 1.5 seconds. The peak free field accelelerations were rather high with the peak ground acceleration being about 4 G's at a range of 200 feet.

A considerable amount of liftoff was noted during the experiments. This may be seen in the recorded pressure measurements at the soil/structure interface and by observing the permanent rotation of the test structures. Lumped parameter models of the test structures were made and solutions found using the SIMQUAKE free field motion as input. Two interaction models were used, and comparisons of response spectra made with spectra based on the measured structural motions. The first interaction model is the standard method and neglects any effect of liftoff (results for this model are marked with circles on the spectra). The second is based on a nonlinear interaction model where the pressures acting at the soil/structure interface are restricted to compression. This model also incorporates additional damping to account for energy dissipation as the structure impacts the soil after it has separated. Details of this model are discussed in Ref.5. Spectra for this model are marked with a triangle on the spectral comparisons.

Since the containment models are effectively rigid ,comparisons of calculated and measured spectra are made for the rigid body motions (at the base in the hortizontal direction, at the top in the horizontal direction, and vertically). The three spectra are shown on Figures 9 through 11 respectively. Spectra of the measured motions are marked with crosses. There is reasonable correlation for the base-horizontal motion indicating that liftoff has little effect on the horizontal motion. Significant differences are noted, however, for the top-horizontal motion, indicating that liftoff does have an effect on the rocking motion of the structure. A comparison of the measured spectra with the standard interaction parameter model spectra (triangles) indicate that the standard model greatly underestimates the low frequency (1-4 cps) response and overestimates the mid frequency (4-10 cps) response. The interaction model which includes a liftoff characteristic (circles) greatly improves the low frequency comparisons, has little effect on the mid frequency response, and makes the high frequency comparison worse. There is some indication that a better treatment of the impact energy dissipation mechanism would improve the liftoff model in this mid-high frequency range. In any event the liftoff model results in conservative spectral accelerations throughout the frequency range while the standard interaction parameter model is unconservative in the low frequency range. A similar situation exist for the vertical spectral comparisons. Neglect of liftoff effects tend to give unconservative results in the low frequency range and the model including liftoff gives better correlations with the spectra of the measured motions.

The free field accelerations of the SIMQUAKE experiment were quite large and it may be questioned whether the above conclusions would carry over to actual earthquake loadings on power plant structures. A series of numerical solutions were obtained (Ref.5) using the liftoff interaction model discussed above. Properties of an actual containent were used and the El Centro earthquake used as the input motion. The El Centro pulses were scaled by multiplying the vertical and horizontal accelerograms by a factor to vary the input motion from a moderate to a severe earthquake. The peak acceleration of the free field accelerogram to cause liftoff to first occur was evaluated. Typical results are shown on Figure 12. The parameter (ζ) represents the ratio of height of the structure's CG to its radius, and the parameter (ε) is a nondimensional interaction damping. The value ($\varepsilon = 10$) corresponds to a vertical damping of 19% and a rocking daming of 1%. The parameter (ψ) is the ratio of depth of burial to foundation height of the CG above the base. As may be seen on Figure 12 reasonably small peak accelerations initiate liftoff.

Two sets of solutions were next generated. The first used the standard interaction parameters neglecting liftoff effects while the second used an interactioin model accounting for liftoff. Solutions were found for input accelerograms 1.33, 1.67, and 2.00 times that required to cause liftoff. Horizontal, rotational, and rocking spectra were generated for each of the three levels of input using each of the models. No difference was found for the horizontal spectra. For the other two spectra, the spectral value for the model including liftoff effects was divided by the spectral value for the model neglecting liftoff effects. The results are plotted on Figures 13 and 14 for the rocking and vertical spectra respectively. It can be seen that the neglect of liftoff effects results in underpredictions of the rocking spectral values by factors up to 3 times in significant frequency ranges (1-4 cps) when the input is twice that required to cause liftoff. The difference is not significant when the input is 1.33 times that required to cause liftoff while factors of 1.5 occur for input motions 1.67 times that required to cause liftoff. There are significant differences in the vertical spectra shown on Figure 14. Comparisons with the SIMQUAKE data however indicated that these predictions using the liftoff model may not be real. Further work is required to establish the validity of the liftoff model in this frequency range.

Conclusions

The following conclusions may be drawn from the results of the studies reported in this paper:

1. There is not a large source of experiemntal data which may be used to validate SSI methodologies. Much of the data that is available is deficient in that either the structures or the input motions are not representative of those found in nuclear power plants subjected to earthquake loadings.

2. A serious shortcoming is the lack of data for sites which have significant non uniform and non linear properties The SSI problem is expected to be more severe for such sites than for the uniform sites for which data is available. Comparisons of analytical predictions with data collected at uniform sites may not be sufficient to validate the method.

3. Comparisons of the lumped parameter method with data collected at the Fukushima nuclear power station indicate that scattering effects are not significant for uniform sites.

4. The standard method of computing the interaction springs and dampers

result in structural motion predictions which correlate reasonably well with the Fuhushima data. There is some indication that the standard interaction dampers may be a little high however. The best correlations between measured and calculated motions were found when 75% of the standard interaction dampers were used.

5. Comparisons of measured and calculated spectra for the SIMQUAKE data indicate that the standard lumped parameter method is not able to predict liftoff effects. The neglect of liftoff effects result in rocking and vertical spectral predictions which are unconservative in the 1-4 cps region.

6. A modification of the lumped parameter method so that liftoff is included appears to give results which correlate reasonably well with the measured data. This modification restricts the spring/damper SSI model to transmit compressive pressures and includes damping which occurs as the foundation impacts the soil after separation ahs occurred.

7. The liftoff lumped parameter model is used to perform computations for a typical containment structure subjected to an eathquake loading. Liftoff is found to occur at reasonably low inputs (0.2-0.4 G peak horizontal accelerations). Comparison studies are performed for in-structure spectra using the lumped parameter model both including and neglecting liftoff. Significant non conservative predictions are found when liftoff is neglected if the peak acceleration is 1.67 times that required to cause liftoff.

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Acknowledgements

The authors wish to express their appreciation to Mr. Herman Graves for his advice and support during the research programs upon which this paper is based. Mr. Graves, of the Mechanical and Structural Engineering Branch of the Office of Nuclear Regulatory Research, U.S. Nuclear Regulatory Commission, served as technical monitor. In addition, the authors wish to thank Dr. James Costello of the same office for his interest in the project and for information provided during the course of the study.

All Dimensions and
Elevations in Meters

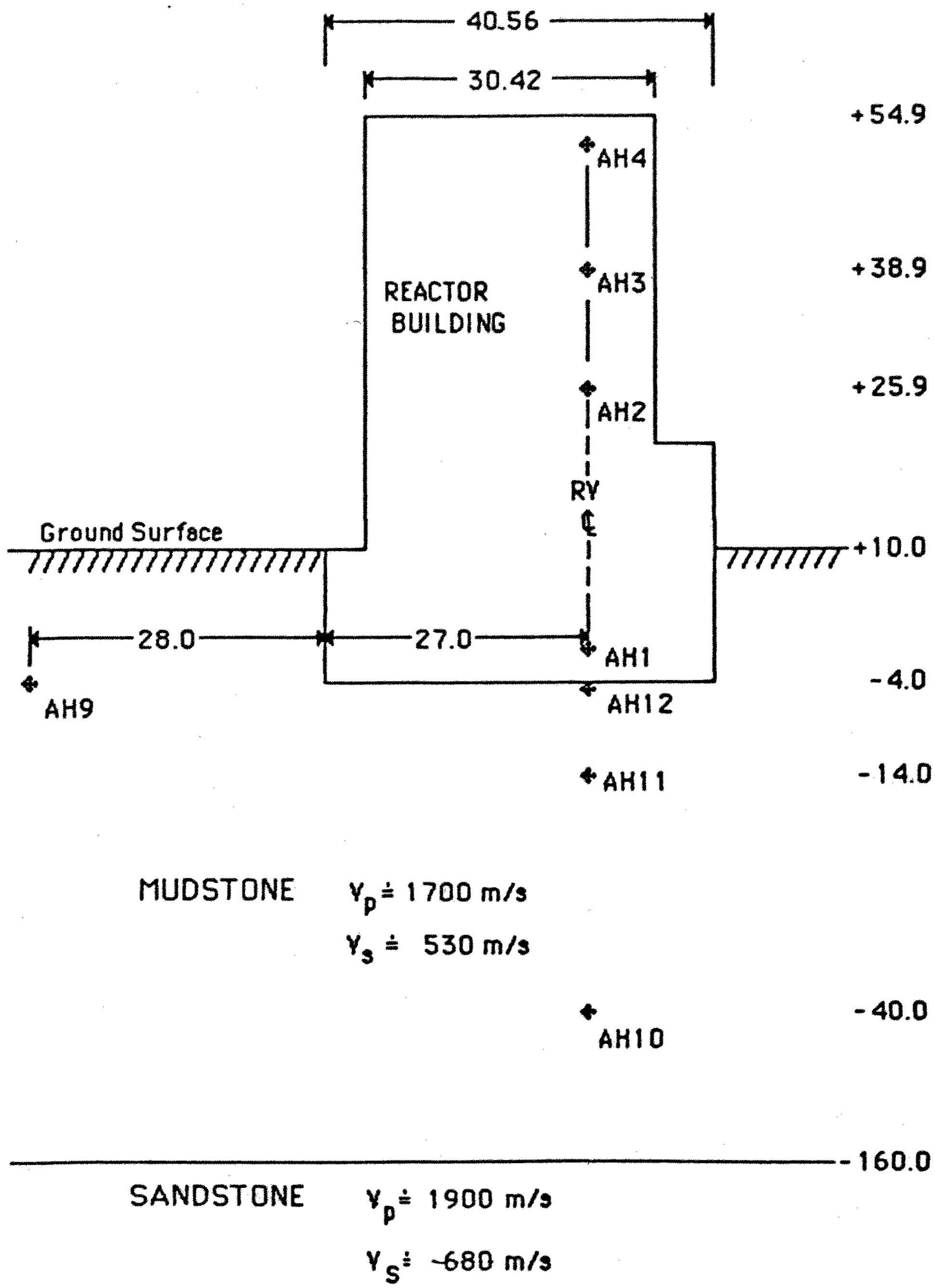


Fig. 1 Accelerometer Locations at Fukushima

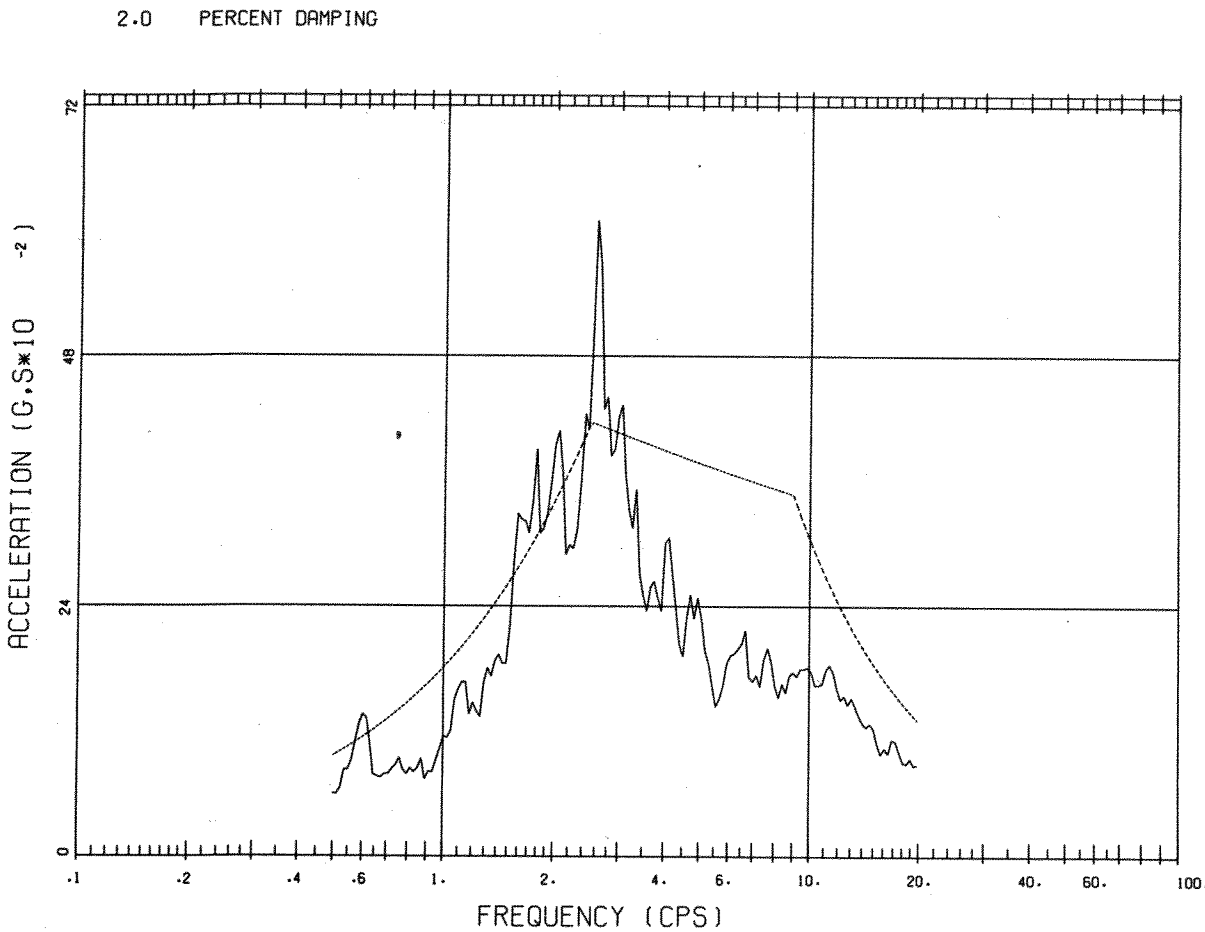


Fig. 2 -4M Recorded Spectra Under Fukushima Reactor Building

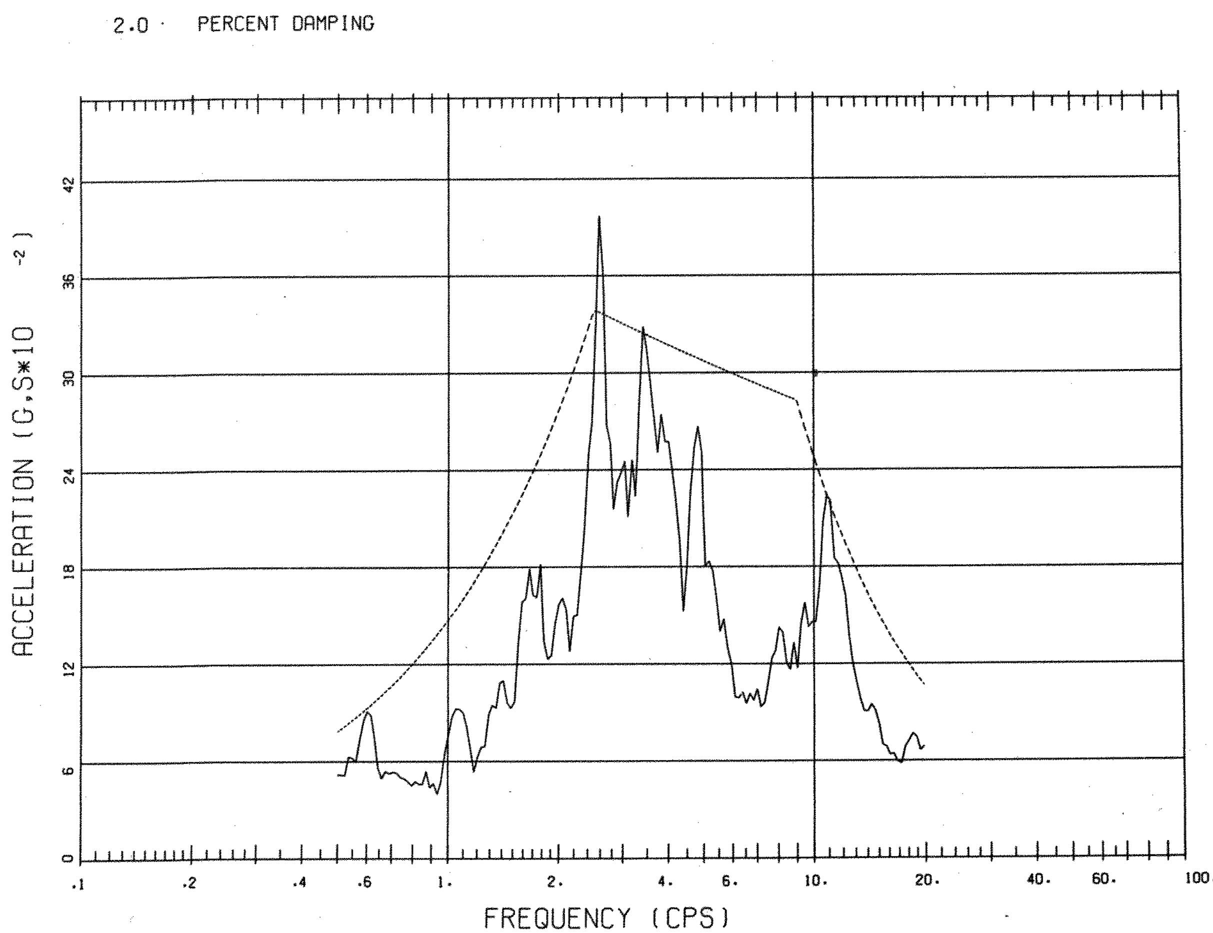


Fig. 3 -4M Recorded Spectra At Side of Fukushima Reactor Building

2.0 PERCENT DAMPING

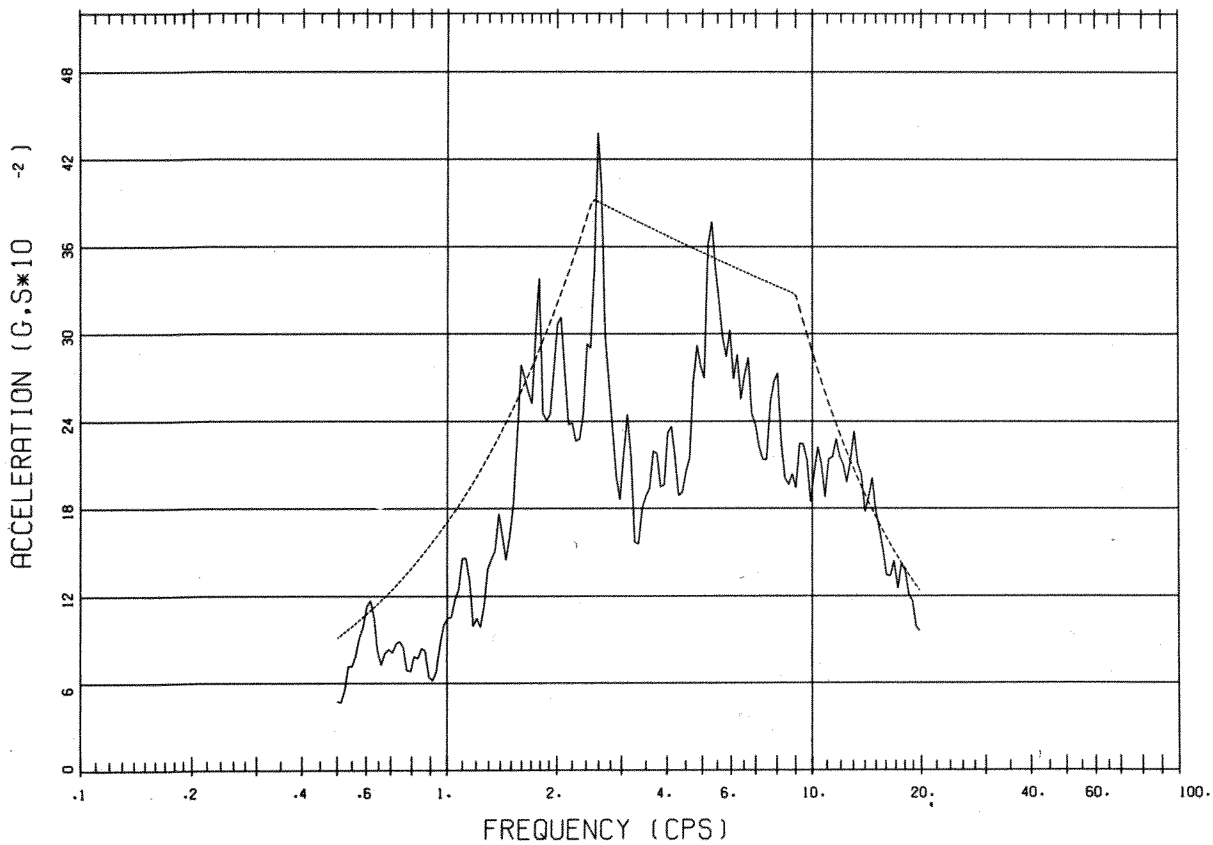


Fig. 4-14M Recorded Spectra Under Fukushima Reactor Building

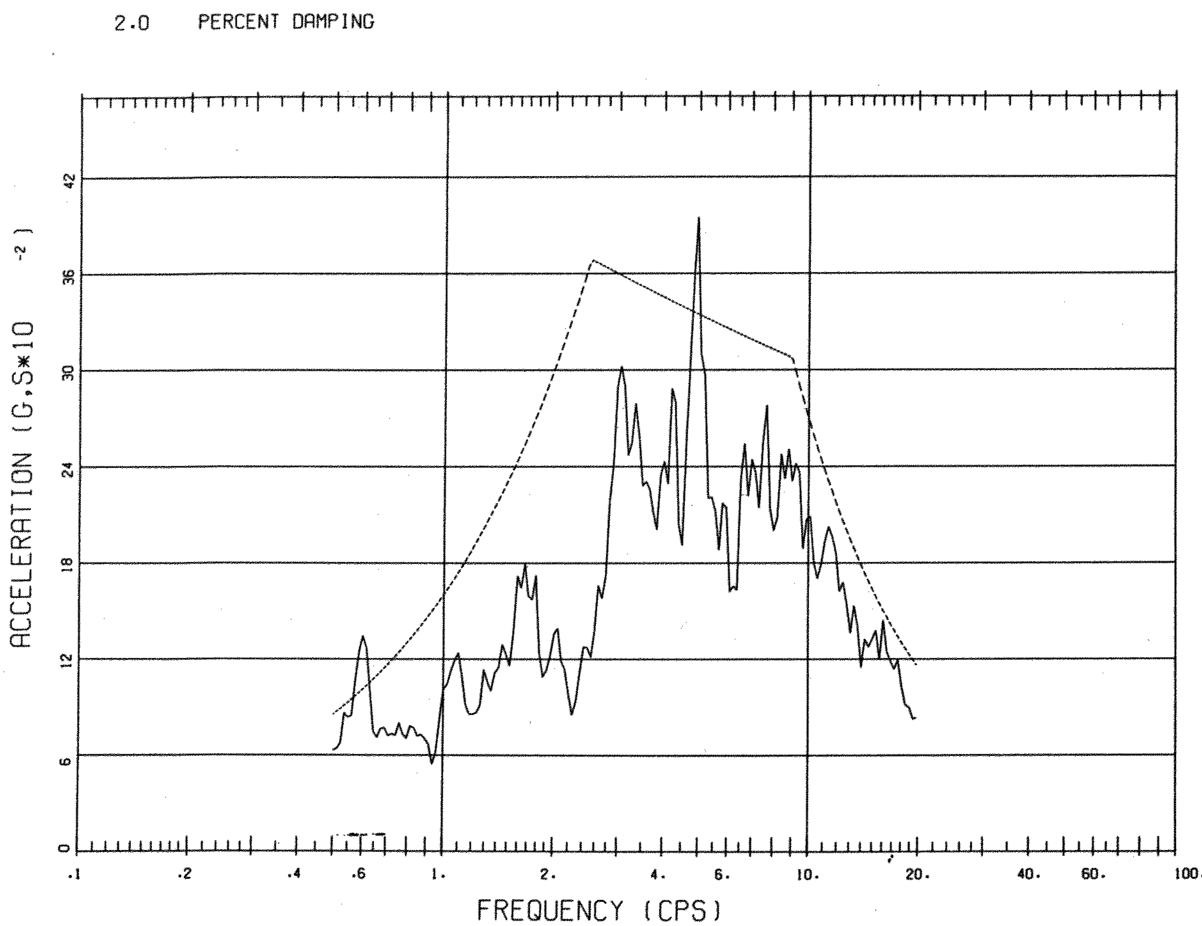
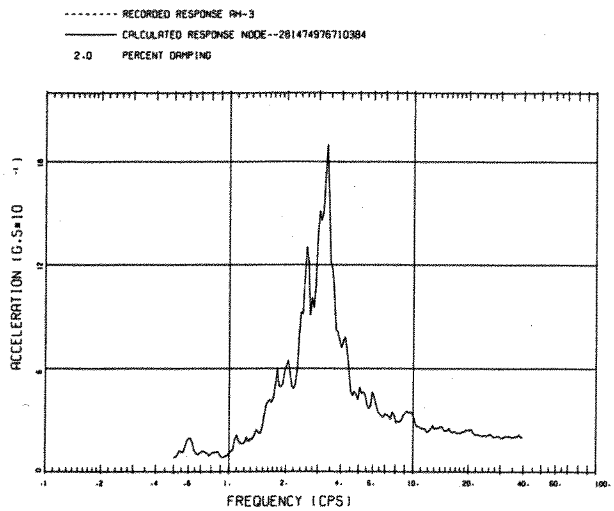
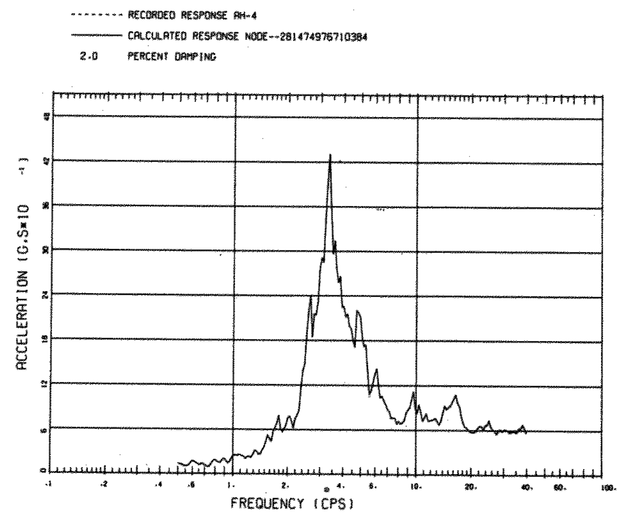


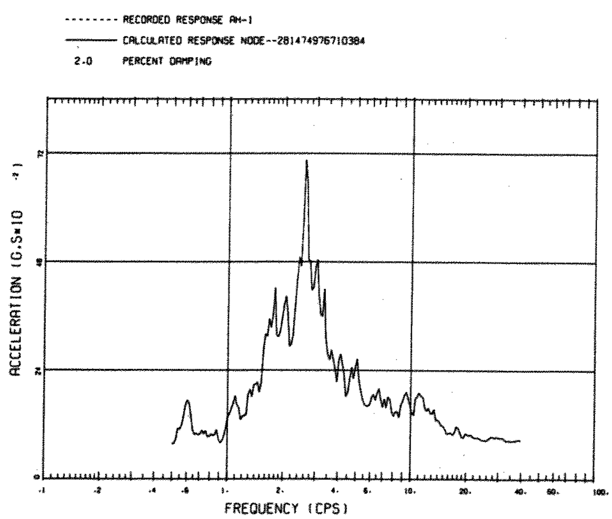
Fig. 5 -40M Recorded Spectra Under Fukushima Reactor Building



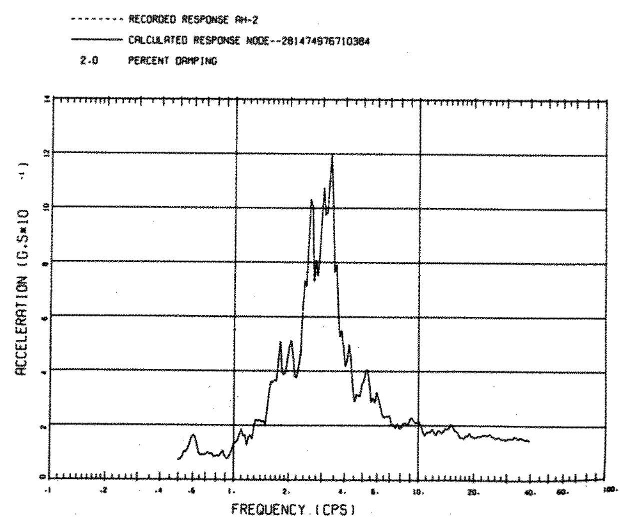
(c) Operating Floor



(d) Top of Structure

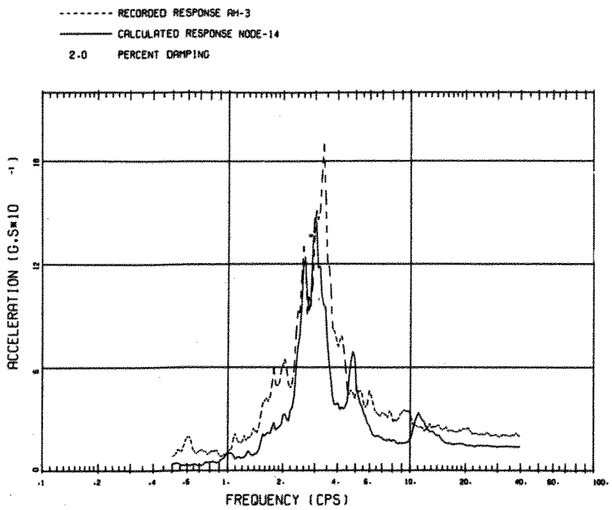


(a) Basemat

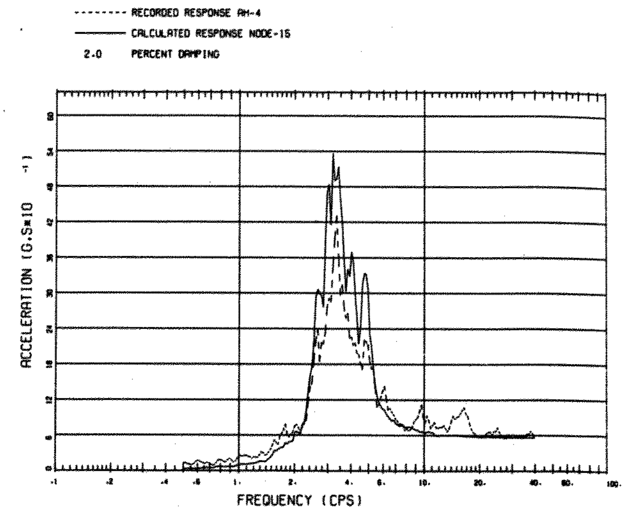


(b) Elev. 25.9 M

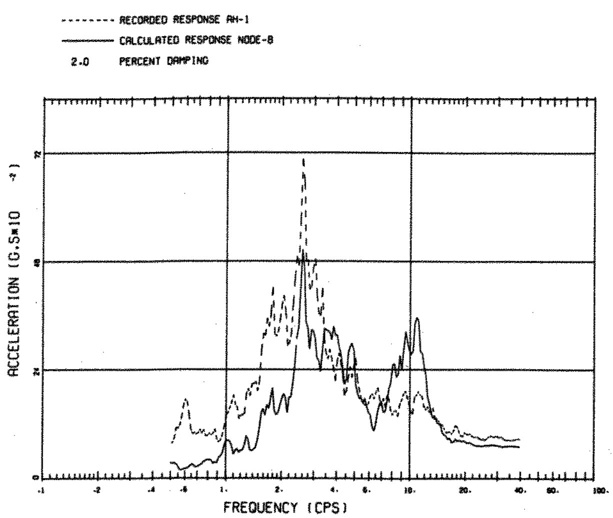
Fig. 6 Recorded Spectra in Fukushima Reactor Building



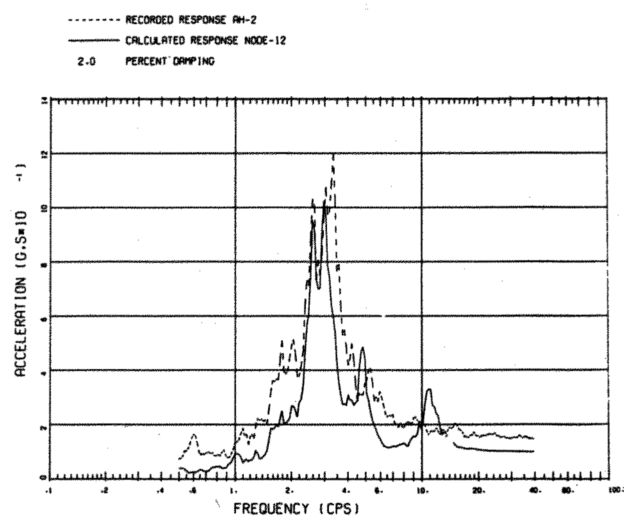
(c) Operating Floor



(d) Top of Structure

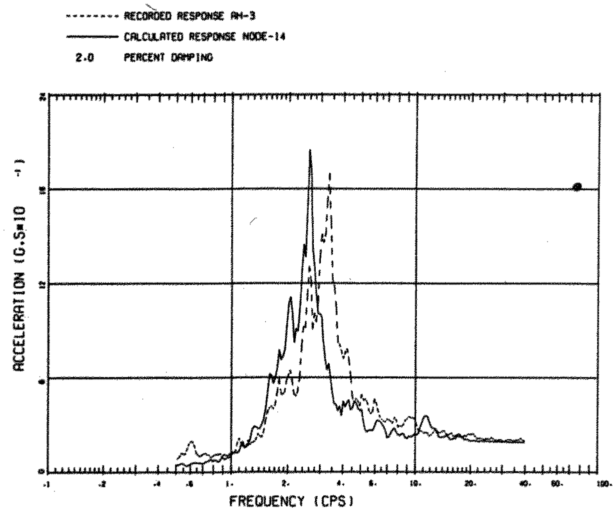


(a) Basemat

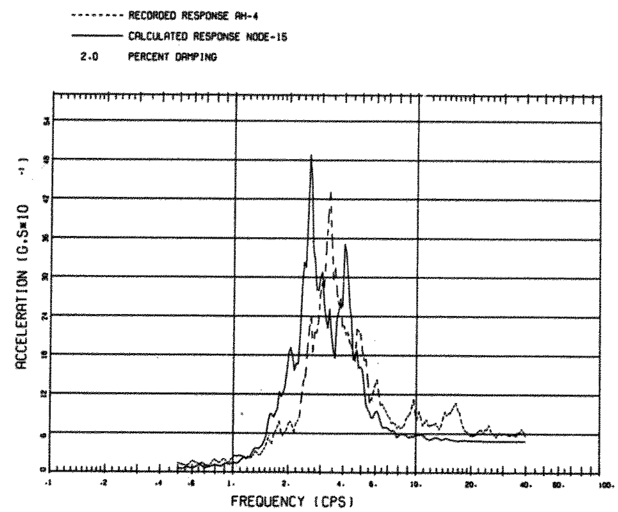


(b) Elev. 25.9 M

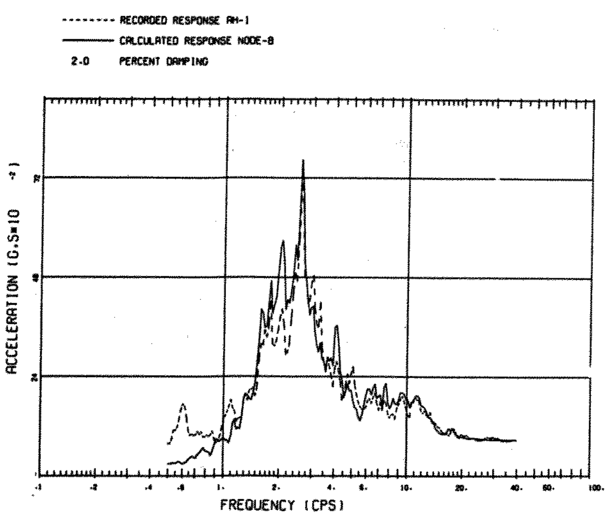
Fig. 7 Comparison of Measured and Calculated Spectra Using Input to Side of Building (100% Interaction Springs and 75% of Interaction Dampers)



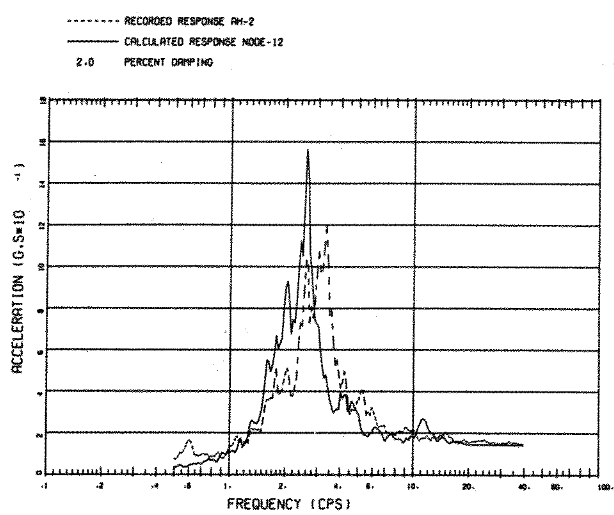
(c) Operating Floor



(d) Top of Structure



(a) Basemat



(b) Elev. 25.9 M

Fig. 8 Comparison of Measured and Calculated Spectra Using Input Under Building (100% Interaction Parameters)

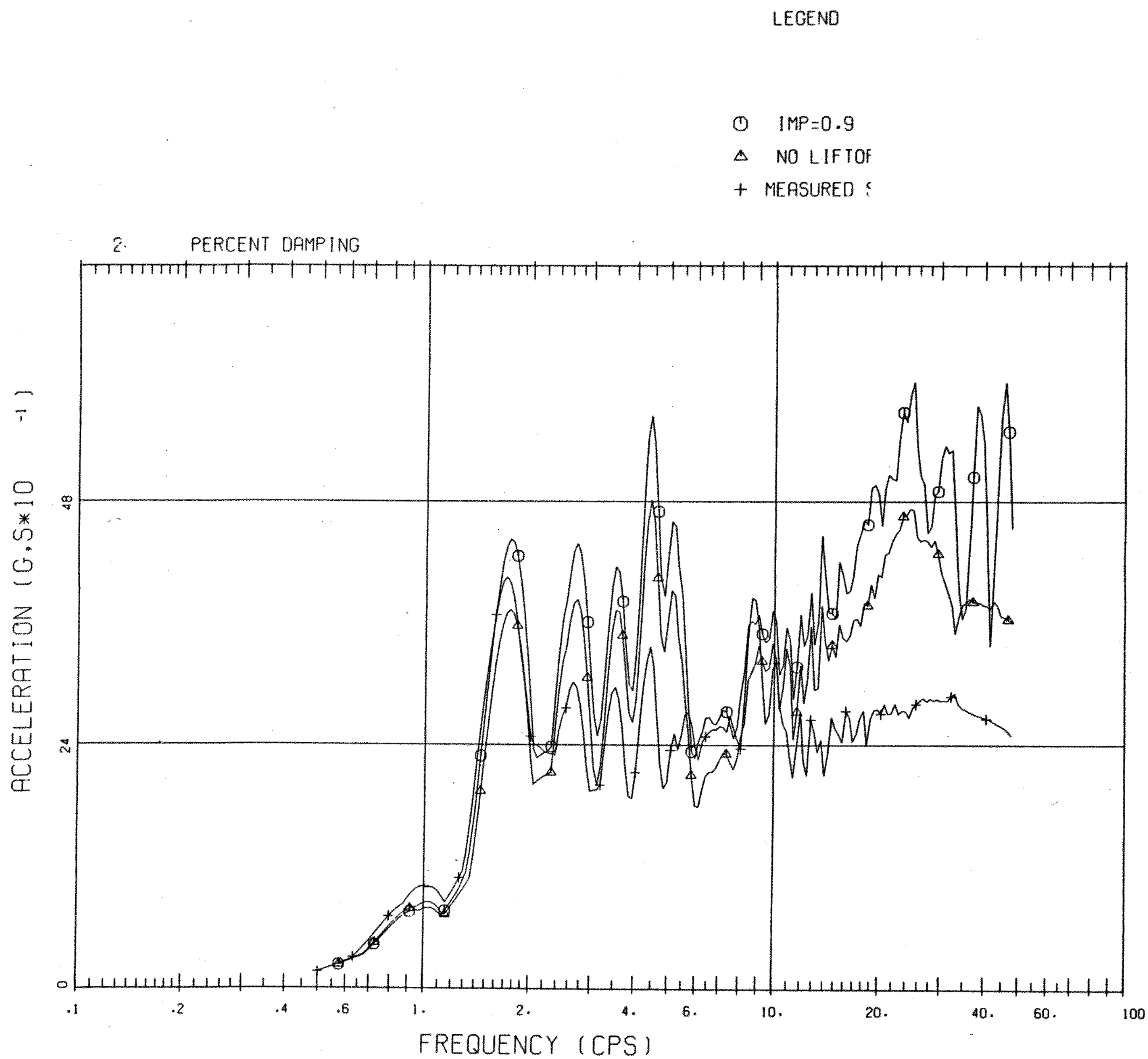


Fig. 9 Effect of Neglecting Liftoff on SIMQUAKE Horizontal Base Spectra

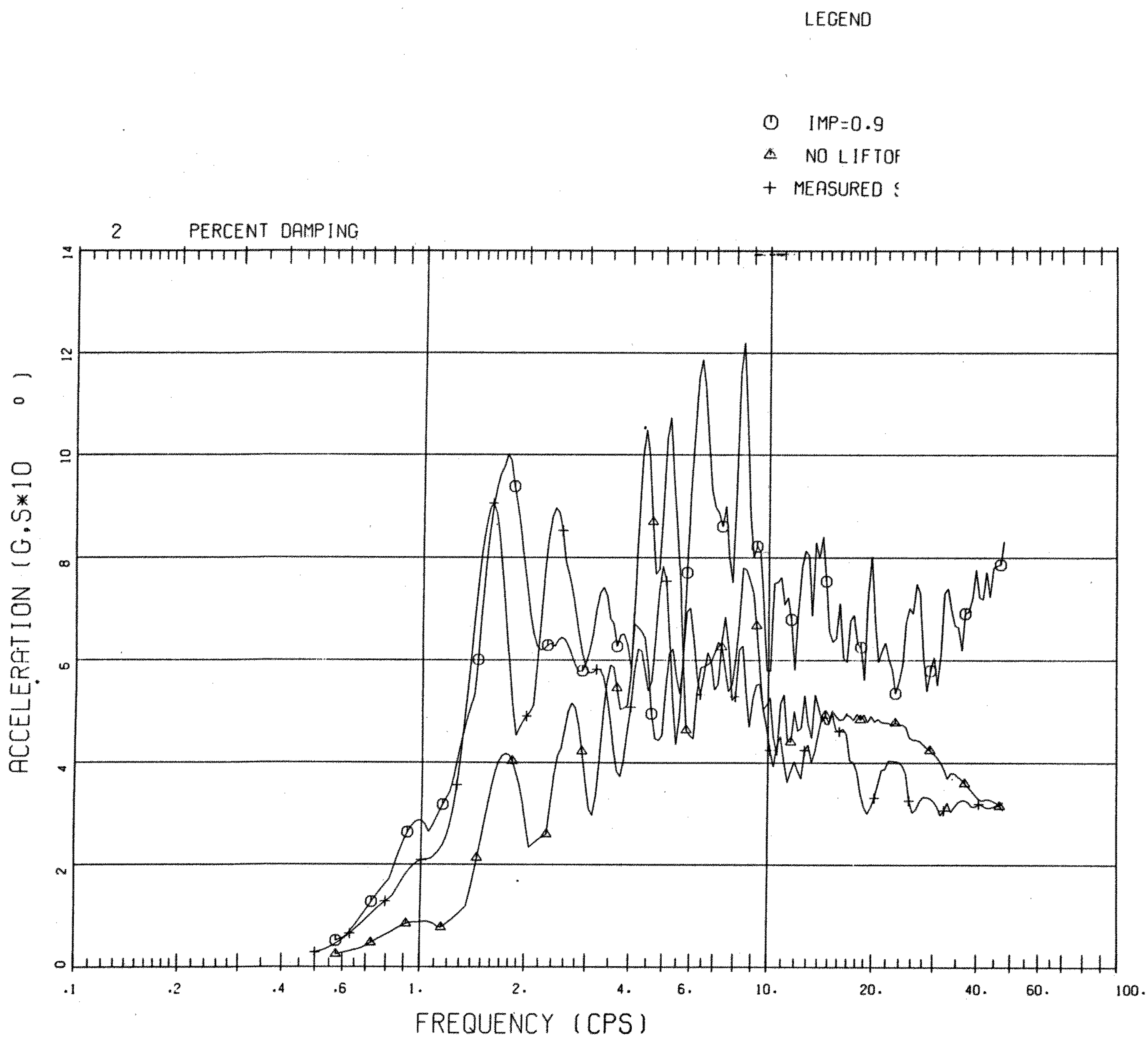


Fig. 10 Effect of Neglecting Liftoff on SIMQUAKE Horizontal Top Spectra

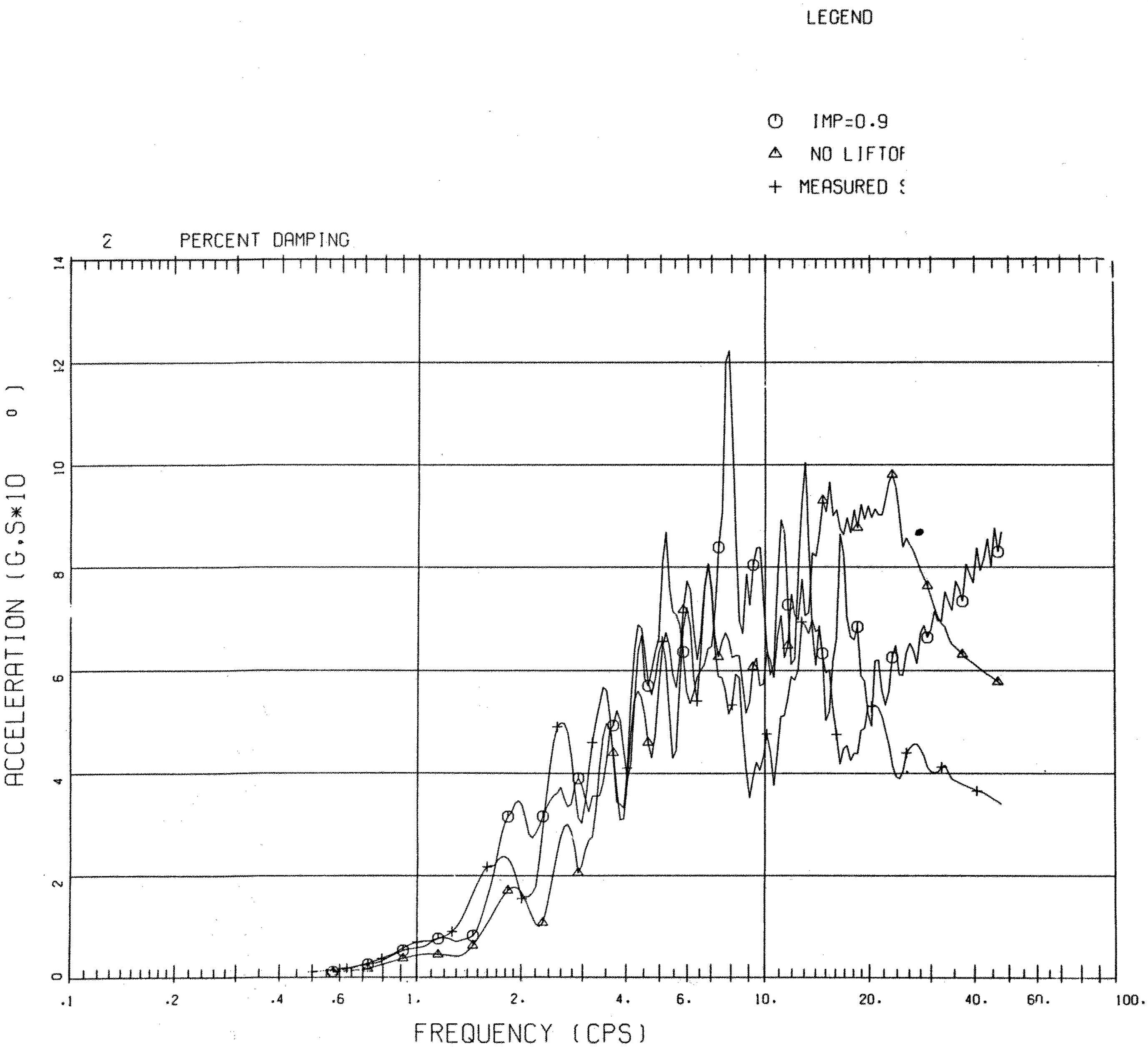


Fig. 11 Effect of Neglecting Liftoff on SIMQUAKE Vertical Spectra

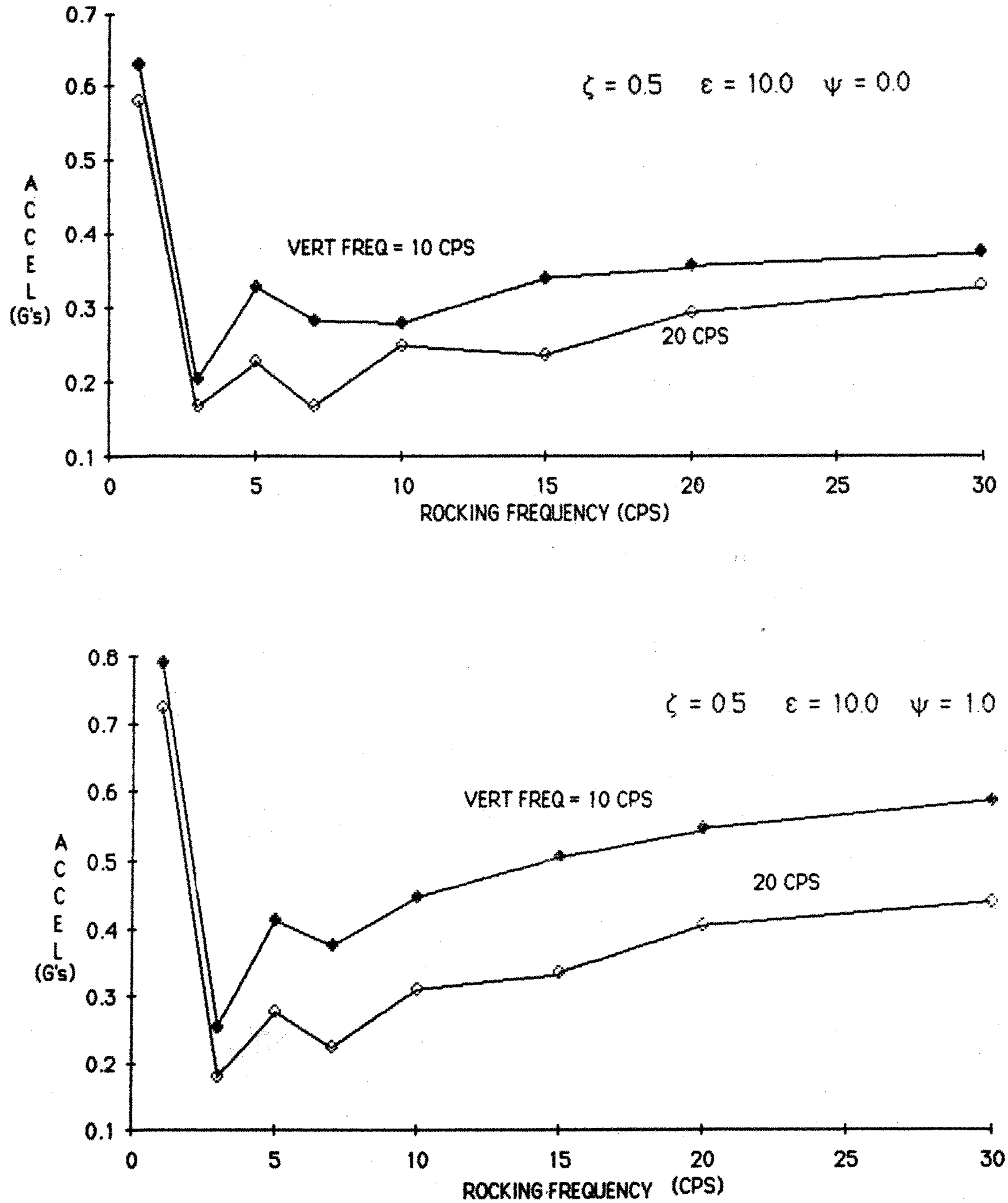


Fig.12 Peak Acceleration Required to Cause Liftoff

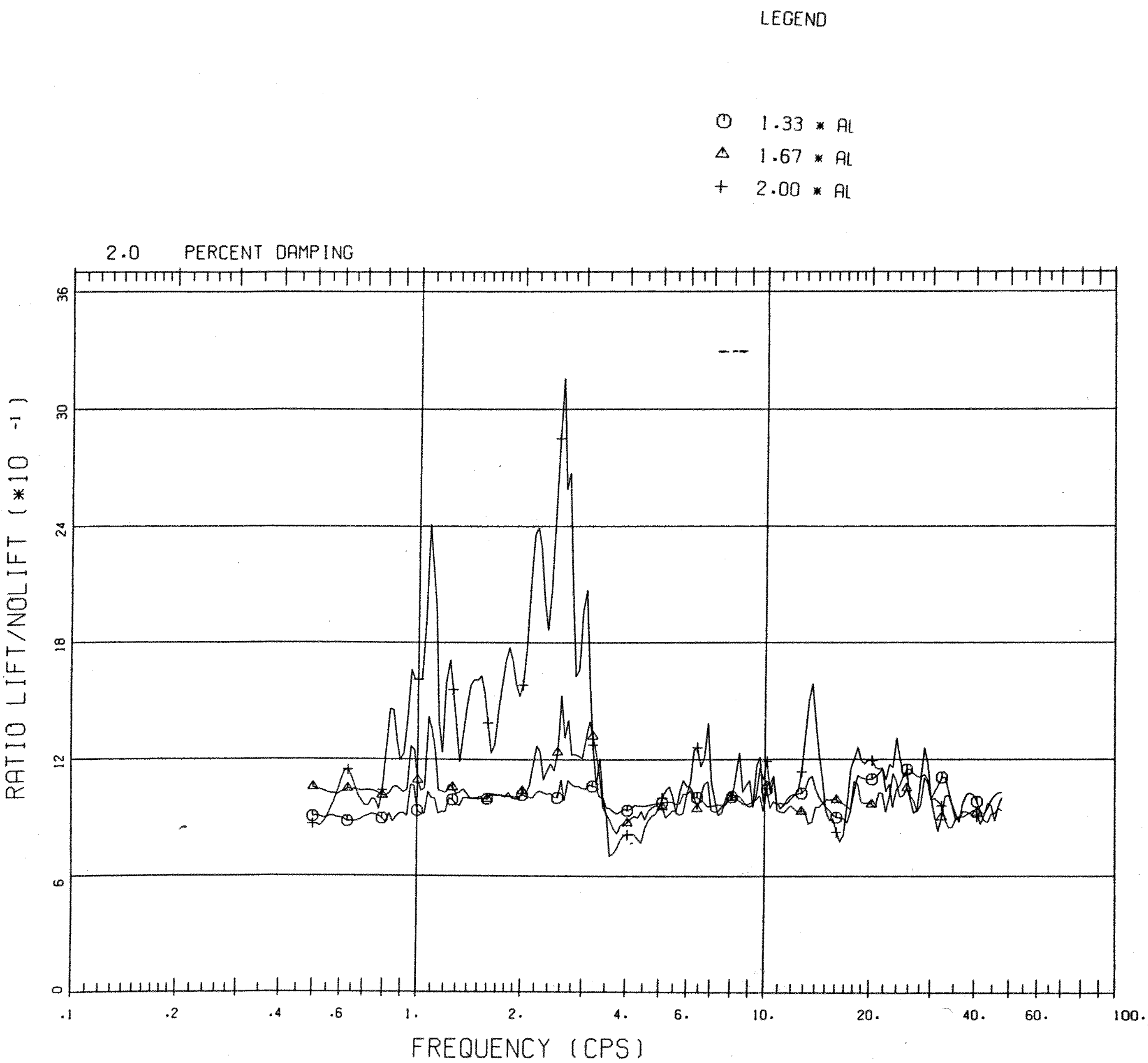


Fig. 13 Ratio of Liftoff/No Liftoff Rotational Spectra

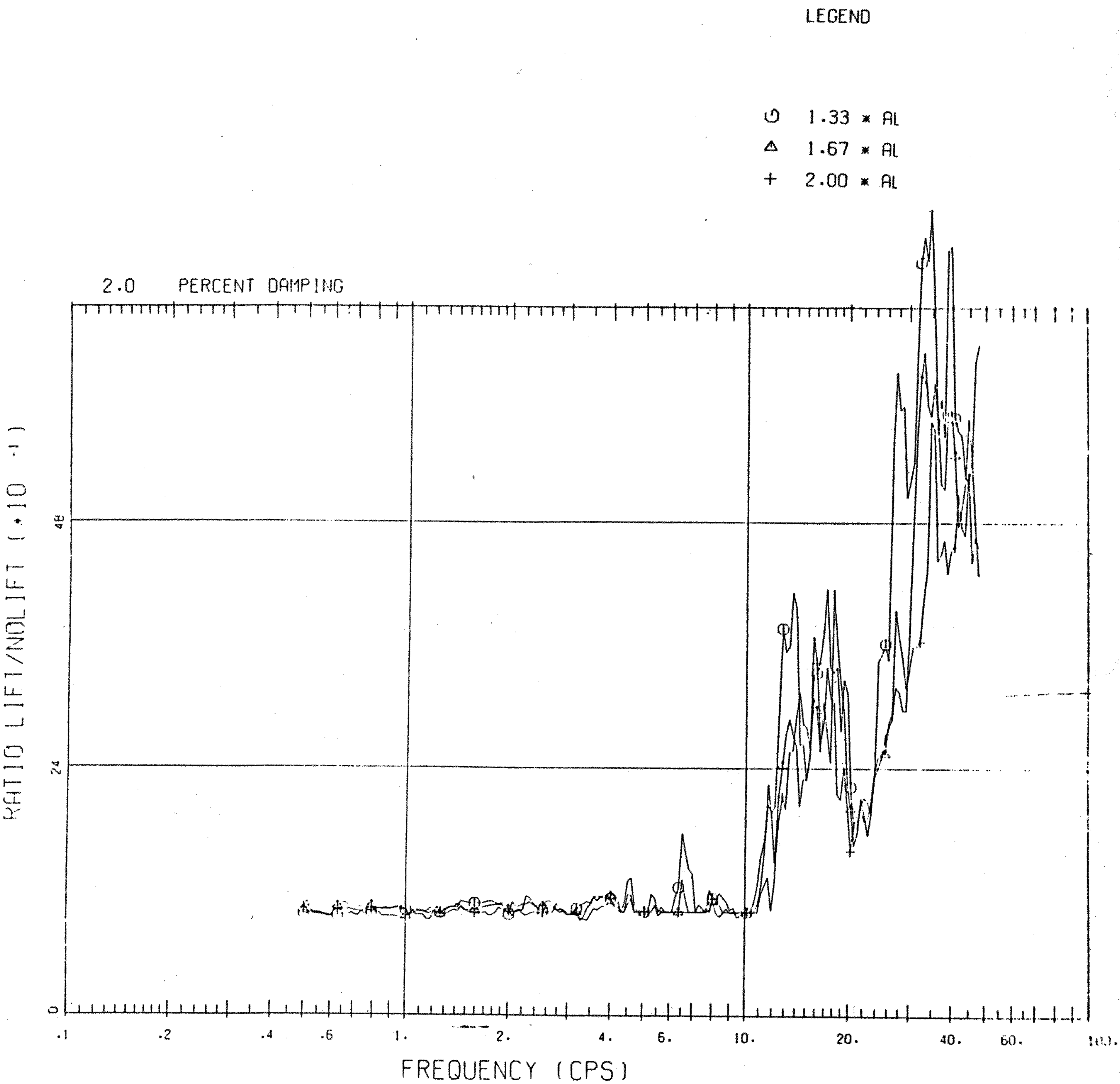


Fig. 14 Ratio Liftoff/ No Liftoff Vertical Spectra

WORKSHOP ON SOIL-STRUCTURE INTERACTION

Bethesda Marriot, MD - June 16-18, 1986

NUMERICAL - EXPERIMENTAL APPROACH TO THE S.S.I. ANALYSIS

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Abstract

This paper describes forced excitation tests carried out on the PEC fast reactor building, as well as several aspects of a complete 3D seismic response analysis. These studies focus on the soil-structure interaction problems. They have been performed by both mathematical procedures and processing of on-site experimental data. The mathematical models have been calibrated using the data obtained through the mentioned on site experimental dynamic tests on the reactor building, in order to develop a very reliable numerical model to compute the vibration levels induced by the design earthquakes and near-field earthquakes on the components and the building itself.
In this way, the safety margins of the design analysis, carried out by use of simplified methods, will be estimated.

1. INTRODUCTION

In situ tests on the reactor building aim at achieving experimental data on its actual dynamic behaviour, so as to obtain a reliable appraisal of vibration levels at the various floors caused by applied forces.

In addition, experimental results make it possible to calibrate a 3D numerical model of the reactor building, defining some significant and a-priori uncertain parameters.

Using the calibrated model, the structure response to the design earthquakes and near-field earthquakes can be reliably evaluated, and the margins present in the design calculations, performed with semplified methods and conservative assumptions, can be estimated

/1/. Indeed, to be able to compare the numerical results to the experimental values, the SSI effects must be taken into account in assessing the mathematical models. More precisely, there are two different ways to evaluate SSI: the mathematical and the experimental procedures.

The mathematical procedure consists in evaluating the soil impedence functions by means of an appropriate design soil profile under reactor building.

The experimental procedure makes use of the determination of the impedence matrix, which connects the soil movements under the foundation of the reactor building to the forces applied to the soil itself.

Both procedures are based on simplified equivalent schemes of the detailed finite element models, which are capable to transmit to the soil the same dynamic forces which are transmitted by the real structure during seismic excitation.

Finally, it is worth noting that the experimental data, obtained by forced vibration tests, allow a general characterization of the structure to be established which could be useful for a comparison with the results of possible subsequent investigations, such as those which might be performed on the basis of data obtained through the seismic monitoring system.

2. DESCRIPTION OF THE STRUCTURE /2/

The PEC reactor building has a height of about 28 m and a diameter of about 23 m, and is housed in a steel cylindrical container resting on a reinforced concrete foundation raft. Heavy concrete solid blocks were poured inside the container up to about half container height.

Thus the building structure supporting components is rather massive. It should be noted that the building structure is far from being axisymmetric and that several others structures of huge mass have been built around the reactor building (a schematic plan of the power plant buildings is shown in Figg. 1,2).

3. TESTING TECHNIQUES

3.1 Excitation methods

To excite the reactor building use has been made of three different excitation methods /3-4/:

a) a two eccentric back-rotating-mass mechanical vibrator, capable of delivering sinusoidal forces of slowly adjustable frequency within the seismic range up to a maximum value of 100 kN. The equipment were rigidly connected with the structure at the concerned floors;

b) hydraulic actuators, installed between the foundations of the reactor building (ER) and those of the adjacent fuel element handling building (EMC), used to apply sinusoidal forces up to a maximum value of 2500 kN (a sketch of ER and EMC foundations in the excitation area is shown in Fig. 3, while a diagram of the hydraulic actuator is shown in Fig. 4);

c) blasting in bore-hole by explosions of two charges of 300 kg TNT at about 2 km from the reactor.

In situ tests have been performed from 1983 to 1985. Within this time span the construction of the reactor building was completed as for as the civil works and some important mechanical components were installed.

Preliminary forced vibration tests were performed in September 1983 and March 1984 (after some recordings of ambient vibrations). The final tests were carried out in 1985, ending in November.

The preliminary investigations were conducted by exciting the structure with a series of horizontal and vertical forces produced by a mechanical shaker installed time by time in seven different positions (from A to G in Fig. 5) at elevation + 6.0 m (plane of working area). Some of these tests were also carried out at about 1,0 m above the foundation (el. - 5,6 m) to check the significancy of an excitation at the base of the building (positions H and I). During the preliminary tests no use was done of hydraulic excitation at the foundation.

The first tests which have been carried out in 1985, after completion of the reactor building, concerned blasting in bore-hole. These tests took advantage of a geophysical investigation programme performed in the neighborhood of the site.

The vibrations were induced by two explosions of 300 kg TNT at about 2 km South-West from the site and a depth of 60 m.

The final tests were conducted in November 1985 by both exciting the reactor building at the working area level by use of a mechanical vibration generator (installed, time by time, in six different positions, from A to G of Fig. 5, except position B) and by means of hydraulic actuators located at the reactor building foundations.

This last kind of investigation was limited to a single direction, due to accessibility problems for the actuators installation; in fact, a suitable area for the installation of the hydraulic actuators had to be created on purpose, by omitting the construction of a part of the structural "tooth" of the fuel element handling building (Fig. 3), which should have been placed on the reactor building foundations. In this way, it was possible to create a joint for the installation of the hydraulic actuators (Fig. 6).

3.2 Response measurements

During preliminary tests the measuring instrumentation of relevant vibrations was carried out by means of a 25-unit seismometer network installed inside the building, and accelerometers set on the steel container. In the final tests, the dynamic response of the reactor building was measured by means of a 42-unit seismometer network (some positions are shown in Figg. 5,7).
In addition 8 accelerometers were located on the steel container (Fig. 8), and 5 transducers (3 accelerometers and 2 seismometers) were used to check vibration levels reached during the tests on the rotating plug, the fuel charging channel and the primary sodium piping (some transducers are shown in Fig. 9).
Furthermore, the dynamic responses of the structures which have been built around the reactor building were measured by means of a 18unit seismometer network inside the fuel handling building (as shown in Fig. 10) and a 3-unit seismometer network at the top of the sodium building as well as at the top of the general services and control building (Fig. 2).

3.3 Recording and analysis instrumentation

Recording and analysis, partly in real-time, of the results were performed by means of an automatic sistem mainly composed of an analog/digital converter, a computer and mass memories.
By this system, it has been possible to correlate, in the frequency domain, the excitation with response (transfer functions).
Figg. 11, 12 show, as an example, some transfer functions obtained during the three testing campaigns carried out.
In particular Fig. 11 concerns dynamic responses obtained by a mechanical shaker, while Fig. 12 concerns transfer functions at different elevations on the same sensitivity axis obtained by hydraulic actuators in the two orthogonal planes taken as reference (North-South and East-West).

4. COMMENTS ON THE EXPERIMENTAL DATA

From the analysis of recorded data, following considerations may be drawn:

- Investigations carried out have indicated the possibility of getting reliable measurements of trasfer functions, even if the exciting equipment were not particularly powerful, by transducers of marked sensitivity. Tests carried out in March 1984 and November 1985 by means of a mechanical shaker were characterized by forces at low frequencies which were higher than those applied in September 1983 (see Fig. 13): this made it possible the response curves to be more accurately defined at these frequency values also.

The excitation at the foundations by means of hydraulic actuators first of all provides the possibility of using such dynamic loads to measure the transfer functions of the building. This type of excitation tools, if installed permanently, could be used to test, for verification purposes, the behaviour of specific components, restrains, anchors, supports after their installation. Such type of analysis is foreseen for the PEC reactor.

Fig. 14 shows the dynamic force applied during the excitation at the foundations.

Structural responses resulted to be considerably different (both in terms of natural frequencies and amplifications) in the three successive periods in which the structure was tested. Indeed, while in September 1983 pours were being completed up to the working area level, in March 1984 the fuel transfer cell was being poured (concrete structure sited above the aformentioned elevation) and in November 1985 all the casts inside the reactor building as well as inside the adjacent structures were finished. According to both the experimental data and the numerical evaluations, the fuel transfer cell has a remarkable effect on the dynamic response of the building.

The analysis of collected functions shows first amplification peaks in the range included between 14 and 15 Hz in September 1983, between 12,5 and 13,5 Hz in March 1984, and between 9,0 and 9,2 Hz in November 1985.

Comparing the response at the various monitored elevations, we come to the conclusion that these amplifications correspond to structural deformation modes of the building , that is not solely ascribable to rigid body movements on the foundation soil. The dynamic response in the two orthogonal planes, particularly on the basis of results of the final tests, seems to be enough similar as far as the first two amplification peaks are concerned, while it is considerably different for the third and fourth amplification peaks which lie at about 17 Hz in the plane N-S, and at about 22 Hz in the place E-W, respectively.

Finally, it is worth noting that the nodal point of the second modal shapes, which correspond to the third and fourth peaks, lies on the fuel transfer cell in both orthogonal planes, as shown in Fig. 12. In particular, fig. 15 shows schematically the first two experimental modal shapes that correspond to deformations of the structure, in the planes N-S (direction X) and E-W (direction Y), respectively.

It is worth noting that the reported modal shapes concern direct and interpolated measures at different elevations on the central vertical axis of the reactor building, along directions X, Y.

Less marked amplification peaks are found close to the modal frequencies of the structure, some of them are due to interactions between the operas built around the reactor building and the reactor building itself.

- The tests which have been performed by blasting in bore-hole took advantage of a geophysical investigation programme which were underway in the neighborhood of the site. The vibrations induced by explosions have been recorded at different locations in the building, at the surface free field conditions, and in depth at the site. The fundamental vibration levels which have been recorded are given hereafter:
  - at the surface in the free field conditions the maximum horizontal velocity was equal to 0.005 cm/s and the maximum vertical velocity was equal to 0.004 cm/s;
  - at the depth of 30 m in the free field conditions the maximum horizontal velocity was equal to 0.002 cm/s;
  - at the elevation + 6.0 m of the reactor building the maximum velocity was 0.015 cm/s;
  - the maximum horizontal acceleration of the steel container at the elevation + 15.0 m was about 6 cm/s/s.

5. DETERMINATION OF THE EXPERIMENTAL "SOIL IMPEDANCE MATRIX"

The soil behaviour can be described by the "soil impedance matrix" evaluated from the results of the final experimental tests, according to the following procedure:

- the soil is considered an elastic,linear system.

- $\{X_f(f)\}$ is the column matrix that represents the six independent movements of the reactor building foundation which has been considered as a rigid body. No relative movement can exist between soil and reactor building in the contact point at the bottom of the structure foundation. That is to say the soil movement in the contact point with the reactor building is equal to the foundation movement in that point.

- $\{N(f)\}$ is the column matrix that represents the forces acting on the soil if external forces are applied to the reactor building. This column matrix has the following expression:

$$\{N(f)\} = [T]^T (\{F(f)\} - [M_s] \{\ddot{Q}_A(f)\})$$

where $\{F(f)\}$ are the external forces and $[M_s]\{\ddot{Q}_A(f)\}$ are the inertia forces (more precisely $[M_s]$ is the mass-matrix of the structure and $\{\ddot{Q}_A(f)\}$ represents the absolute accelerations of the lumped masses by which the reactor building has been described).

- In this way the soil impedence matrix is related to vectors $\{N(f)\}$ and $\{X_f(f)\}$ by the following relation:

$$\{N(f)\} = [Z(f)] \{X_f(f)\}$$

where:

$[Z(f)]$: it is the (6x6) symmetric "soil impedence matrix". In general it is a fully populated matrix.

- therefore, for each excitation test carried out, the following equation can be written:

$$\{X_f(f)\} = [Z(f)]^{-1} [T]^T(\{F(f)\}-[M_s] \{\ddot{Q}_A(f)\})$$

Obviously, since in general the unknown functions are 36 (21 if the symmetry of the matrix $[Z(f)]$ is considered), at least six independent excitation force vectors are needed.
At the moment no result can be reported, since the processing of the experimental data is still in progress.

6. NUMERICAL ANALYSES

6.1 Main steps of the numerical analyses

The complete numerical analysis is articulated in the following main steps /5/:

a) Fixed base analysis of the reactor building using a very detailed 3D finite element model; computation of the fundamental frequencies and modal shapes.

b) Setting up, using the results of the fixed base analysis, of a simplified mathematical model suitable for soil-structure interaction analysis.

c) Preliminary soil-structure interaction analysis in the frequency domain and calibration of the parameters of the model (impedence functions of the soil) by comparing experimental and numerical results; evaluation of the non-linearities in case they are present in the soil behaviour and definition of the best estimate soil properties compatible with the deformations induced by the design earthquake.

d) Setting up of a mathematical model of the main vessel and calibration of the dynamic stiffness of the supporting structure using the results of experimental tests performed on the vessel.

e) Seismic response analysis of the reactor building subjected to the design earthquakes, taking into account the soil-structure interaction by means of the calibrated model; computation of the motion at various floors obviously including the vessel supporting floor.

At the present, activities a) and b) have been completed. The subsequent activities are at present in progress.

6.2 Fixed base analysis of the reactor building

A very detailed 3D mathematical model of the reactor building (see Figg. 16 and 17) was set up using solid and plate finite elements. Solid elements were used to model the foundation raft and the central core; the remaining structural elements were reproduced by plate elements.

Adopting the real densities of the materials, the total mass of the model revealed to be practically identical to the mass of the real structure, thus confirming the goodness of the modeling.

The modal analysis at fixed base was performed using "ABAQUS" code.

The following fundamental frequencies were computed :

Mode	1	2	3	4	etc.
Frequency (Hz)	12.5	13.4	24.0	27.5	etc.

The most significant mode shapes (see figg. 18 and 19) correspond to flexural deformations of the structure in the vertical orthogonal main planes.

It is worth noting that from the computed mode shapes useful information were obtained for the selection of the most significant

points where measurement instruments ought to be located in the final experimental tests.
Using a model with linear finite elements (3096 d.o.f. in total) or a model with isoparametric finite elements (7974 d.o.f.), the maximum differencies in frequencies of interest showed to be less than 10%, thus confirming the reliability of the results.
In order to reproduce the construction corresponding to the first preliminary experimental tests (1983), a partial construction condition (structure built up to the supporting floor of the vessel, at elevation + 6.00 m) was also examined.
For this situation notably higher frequencies were computed:

Mode	1	2	3	4	etc.
Frequency (Hz)	20.9	23.1	32.1	38.6	etc.

These results showed that the highest portion of the building (transfer cell) has a strong influence on the fundamental frequencies. Such results were computed assuming as fixed base both the bottom and the lateral cylindrical surface of the concrete foundation raft: this means that a perfect contact between these surfaces and the rock foundation was assumed.
To investigate the influence of this hypothesis, the fundamental frequencies were computed also considering as fixed base only the bottom of the concrete raft.
The following results were obtained:

Mode	1	2	3	4	etc.
Frequency (Hz)	17.6	18.9	29.0	35.9	etc.

6.3 Soil-structure interaction model

Mathematical models simpler than the detailed model used for fixed base analysis are required for the computation of the dynamic response taking into account soil-structure interaction.
Such models should allow the motion of the rigid base of the structure to be reliably computed.
To this purpose, the structure can be represented by semplified equivalent schemes, capable to transmit the same dynamic forces to the foundations which are transmitted by the real structure during seismic excitation.

6.3.1 Representation of the structure

By separately analyzing the behaviour along the two main horizontal directions, a simplified model consisting in a cluster of simple oscillators has been derived from the results of the fixed base analysis for each of the two directions.
From each computed modal shape, a corresponding simple oscillator was derived.
The characteristics of each oscillator were so defined:

- the mass and the bending moment of inertia were derived from the modal mass and the frequency of the corresponding modal shape of the reactor building;
- the height of the oscillator was computed equalizing the resultant moment transmitted by the oscillator to the foundation and the modal shape.

In this way, the dynamic shear force and moment transmitted to the foundation by each oscillator are equal to those transmitted by the real building in each vibrating mode.
The described procedure is schematically illustrated in fig. 20. A more complete and refined approach can also be used, that allows the coupling effects between orthogonal directions to be taken into account.
In this approach the mass and the bending moment of inertia of each oscillator are replaced by complete 6x6 mass and stiffness matrices; thus the complete set of the six global forces acting on the foundation is fully reproduced. Forces and moments exerted by the cluster of oscillators are equal to those of the original structure; hence the base "does not know" the difference between the cluster of oscillators and the actual structure.
It is important to note that the seismic motion at each point of the detailed 3D model can be easily obtained through a simple linear combination from the seismic response computed for the equivalent oscillators.

6.3.2 Representation of the soil foundation

The soil foundation representation will be derived from the results of the final experimental tests.
By analyzing the experimental structural response with a "system identification" technique, a fully populated 6x6 matrix of the impedence functions of the soil will be computed.
The complete soil-structure interaction model will be obtained by assembling this soil matrix and the cluster of oscillators previously described.
Theoretical impedence functions will also be computed by computer codes, using the geotechnical design profile derived from the available data on the dynamic properties of the foundation.

This will be done for useful and interesting comparisons with the impedence functions matrix derived from experimental results.

6.4 Future development

As soon as the results of the final on site experimental tests will be available, the remaining steps of the numerical analysis will be performed.
In addition, the feasibility of a complete threedimensional soil-structure interaction analysis is being evaluated and more sophisticated analysis (in which the structure and the interaction phenomena are represented in all their completeness and complexity) is planned, in order to verify the reliability of the simplified model approach. Anyway, the completion of the study is foreseen by October 1986.

7. COMPARISON BETWEEN NUMERICAL AND EXPERIMENTAL RESULTS

Both the experimental and numerical results indicate the relevant effect on the dynamic responses produced by the structures of the highest portion of the transfer cell located above elevation +6.0 m. Due to the non symmetric nature of the structure of the transfer cell structure there are also some differences on modal shapes and frequencies in the orthogonal directions. This has been verified both numerically and experimentally.
With regard to comparisons between numerical and experimental results, we note that, for the building construction stage of 1983, the first experimental frequency was 14-15 Hz against the numerical value of about 20 Hz; on the contrary for the building at the construction level of 1984 the first experimental frequency was about 12.5-13.5 Hz in good agreement with the values computed for the reactor building with complete fuel cell structure (12.5 - 13.4 Hz).
Finally, the tests carried out in March and November 1985, showed first frequency values of about 9 Hz.
The frequencies computed in 1983 conditions appear to be higher than the measured ones: this is partly due to the fact that it was impossible to exactly reproduce the situation in which the experimental tests were carried out and partly to the fact that soil-structure interaction effects were not taken into account in the numerical analysis. On the other hand, due to the rock type of foundation soil, soil-structure interaction is not expected to give a very important contribution to the frequency content of the overall dynamic responses, at least for the design earthquakes (Housner spectra cut at 7 Hz).

For the 1984 conditions, even if the soil structure interaction is again not accounted for, the experimental and numerical results were in better agreement: this is probably due to the fact that the transfer cell construction was not completed yet, while the calculation referred to the complete structure.
With regard to the interpretation of the last experimental results, it is necessary to introduce soil-structure interaction effects in the numerical model and identify the most correct contact condition between soil-rock and concrete raft. This work is going to be performed in order to obtain numerical results suitable for the final comparison with the experimental ones.

8. CONCLUSIONS

As already mentioned, the processing of the experimental data has not been completed yet and the numerical analyses are still in progress.
Nevertheless, the first results indicate the ability of the approach to allow a mathematical model of a complex and heavy structure to be developed calibrated by experimental tests conducted on the prototype at different construction levels, taking into account soil-structure interaction effects.
The preliminary investigations have indicated the adequacy of the vibration generators used to excite the reactor building and of the measuring instrumentation employed to evaluate relevant vibrations.
The experimental data of the preliminary tests have been processed in order to evaluate the first modal parameters for the definition of the best excitation and measuring positions to be adopted in the final tests. Furthermore, this approach can be helpful in the development of the design and for verification.
The final experimental tests have shown that the frequency values of the building computed in the previous simplified design analysis (about 10 Hz) are quite close to the experimental ones: this is adequate, taking into account the spectrum enlargement which has been adopted in the design analysis. Generally speaking these tests have confirmed the high value of the first resonance frequency for massive reactor building constructed on rocky soils.
The results of the numerical and the experimental activities, which are still in progress, are going to be presented at the specialist meeting on the experimental on-site seismic verification of nuclear plants to be held in Italy at the ENEA Center of BRASIMOME (which is the PEC reactor site) in the first half of May, 1987.

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April 21-25, 1985.

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/5/ P. Bonaldi, et al.:
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August 19-23, 1985.

GENERAL VIEW OF THE POWER PLANT BUILDINGS

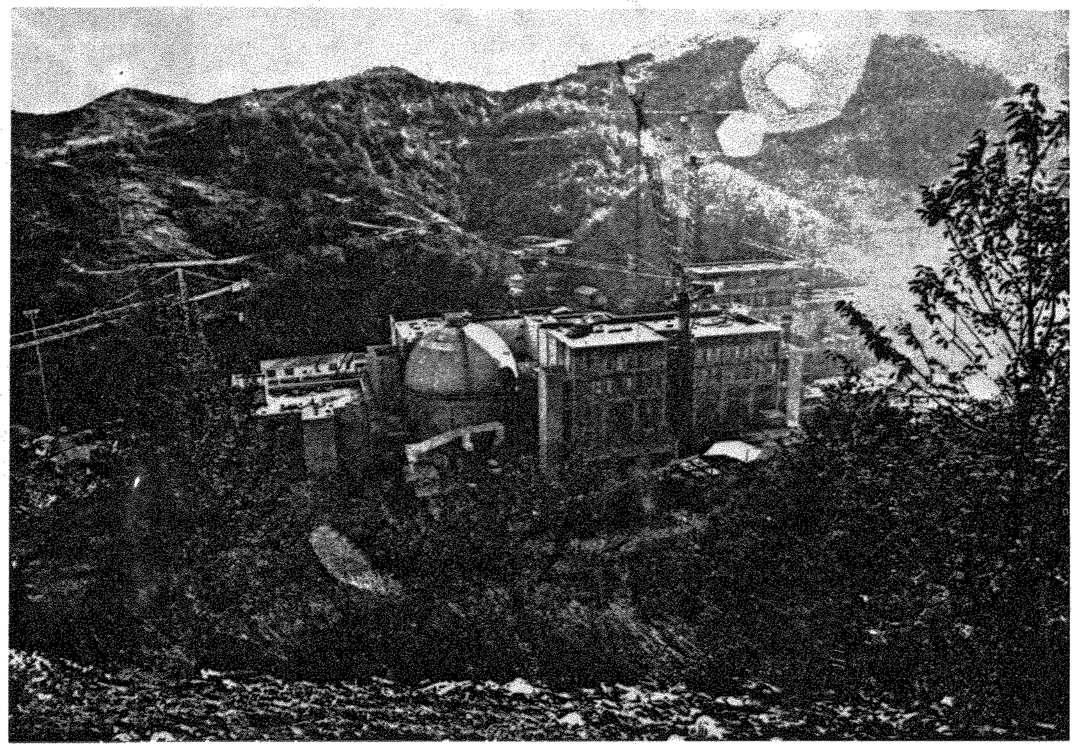


FIG. 1

SCHEMATIC PLAN OF THE POWER PLANT BUILDINGS

MEASURING INSTRUMENTATION ON THE SODIUM BUILDING AND ON THE GENERAL SERVICES AND CONTROL BUILDING

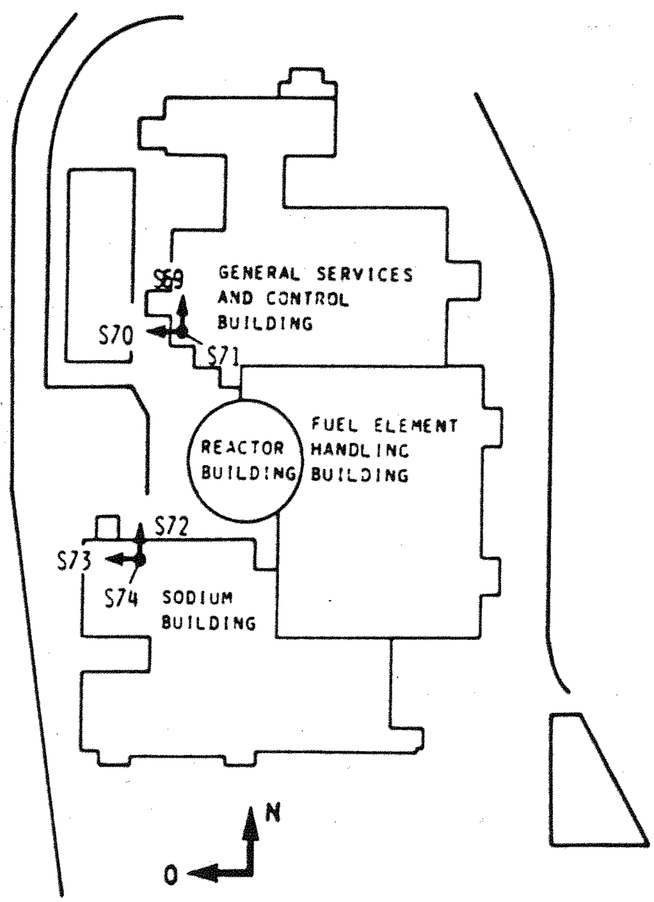
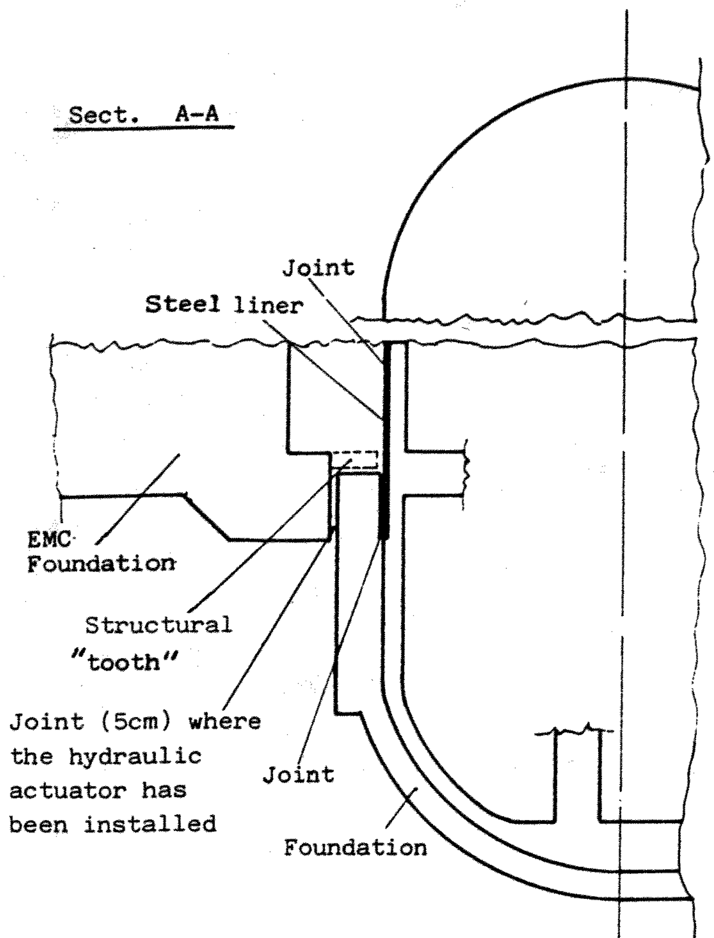


FIG. 2



SCHEMATIC REPRESENTATION
OF THE ER AND EMC FOUNDATION
AT THE EXCITATION AREA

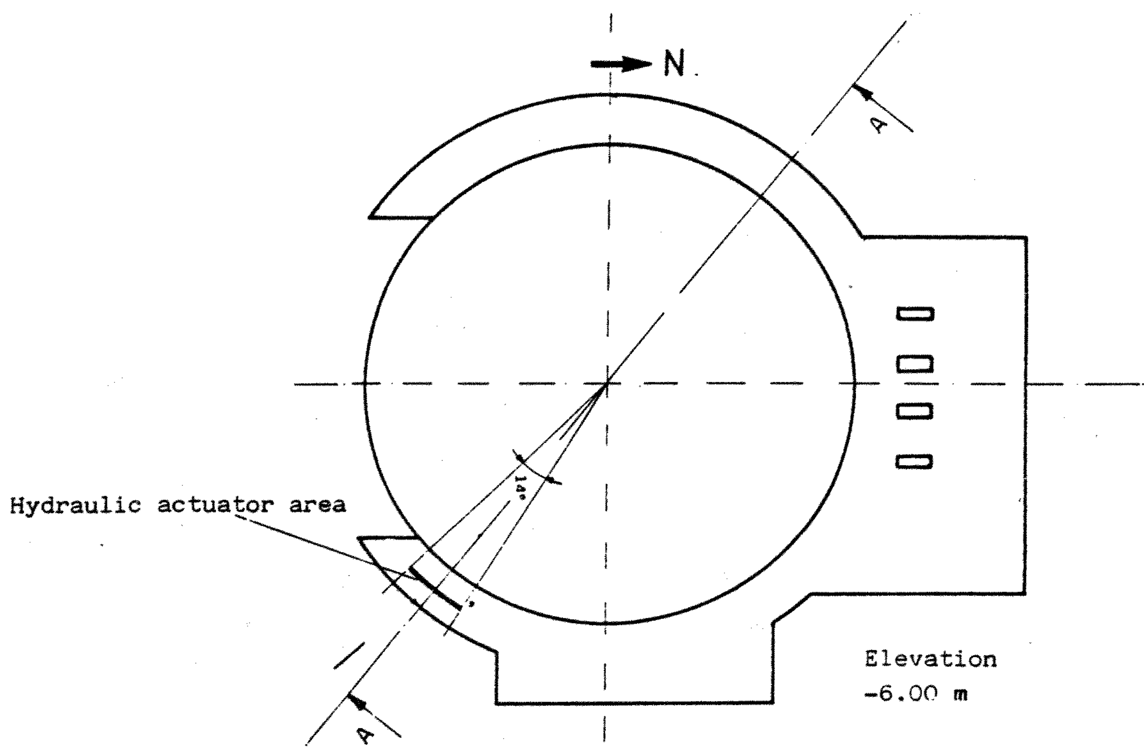


FIG. 3

DIAGRAM OF THE HYDRAULIC ACTUATOR

MAIN FEATURES

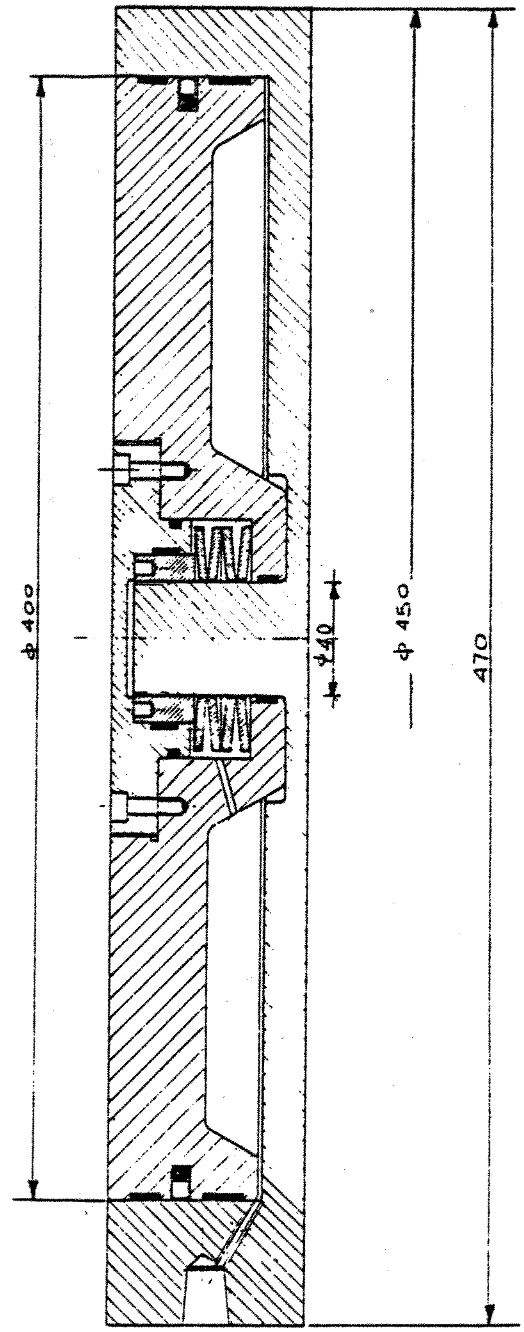
Thickness : 70 mm

Diameter : 470 mm

Design area : 1244 cm^2

Design pressure : 220 atm

Max displacement : 5 mm



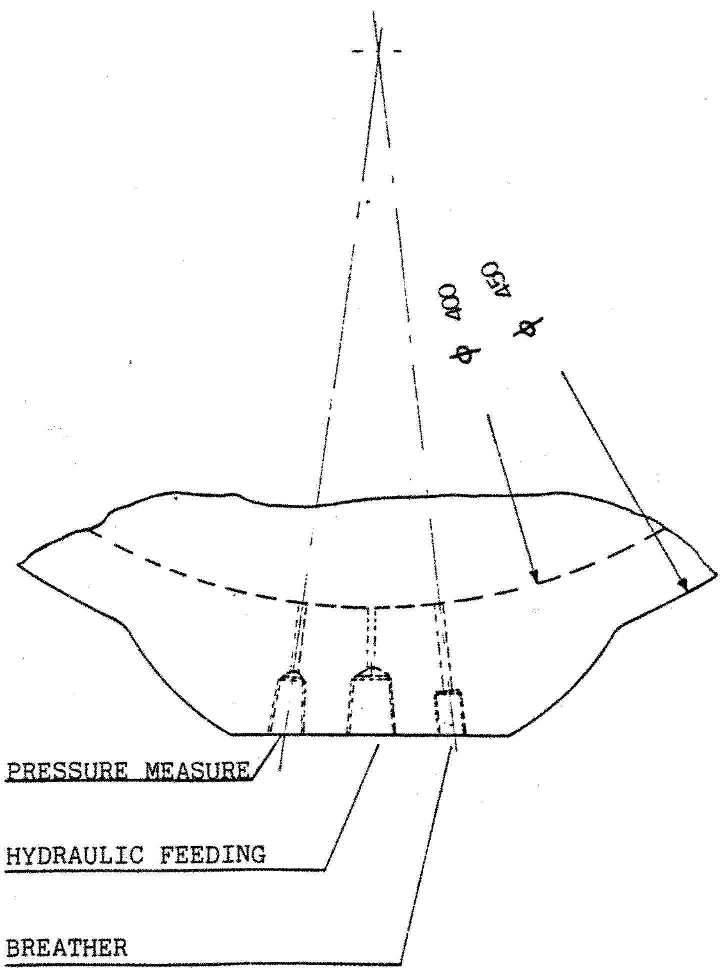


FIG. 4

SKETCH OF THE PEC REACTOR BUILDING. EXCITATION POSITIONS AND DIRECTIONS OF MECHANICAL SHAKER IN THE IN-SITU TESTS. INSTRUMENTATION AXIAL POSITIONS.

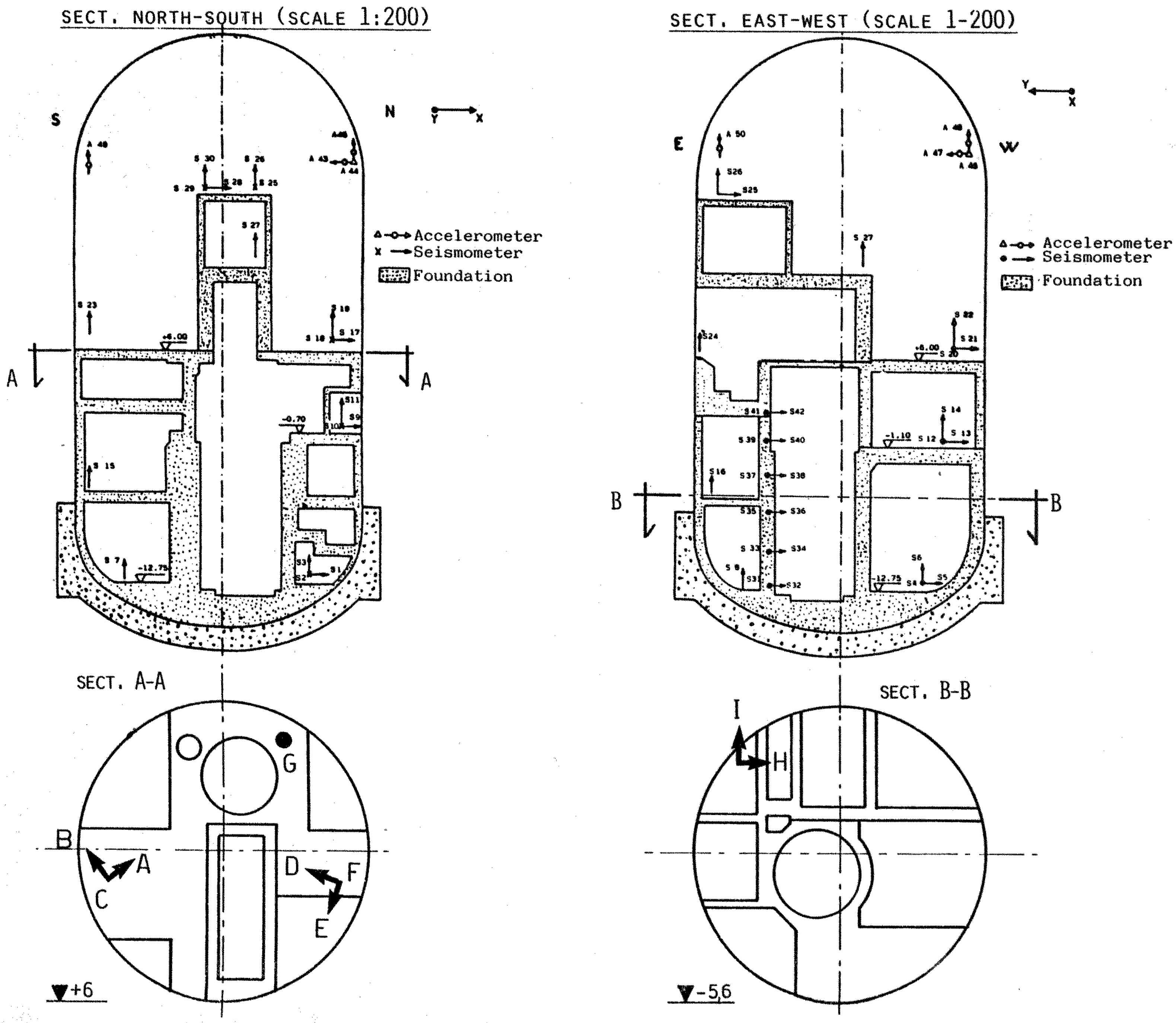


FIG. 5

INSTALLATION OF THE HYDRAULIC ACTUATORS

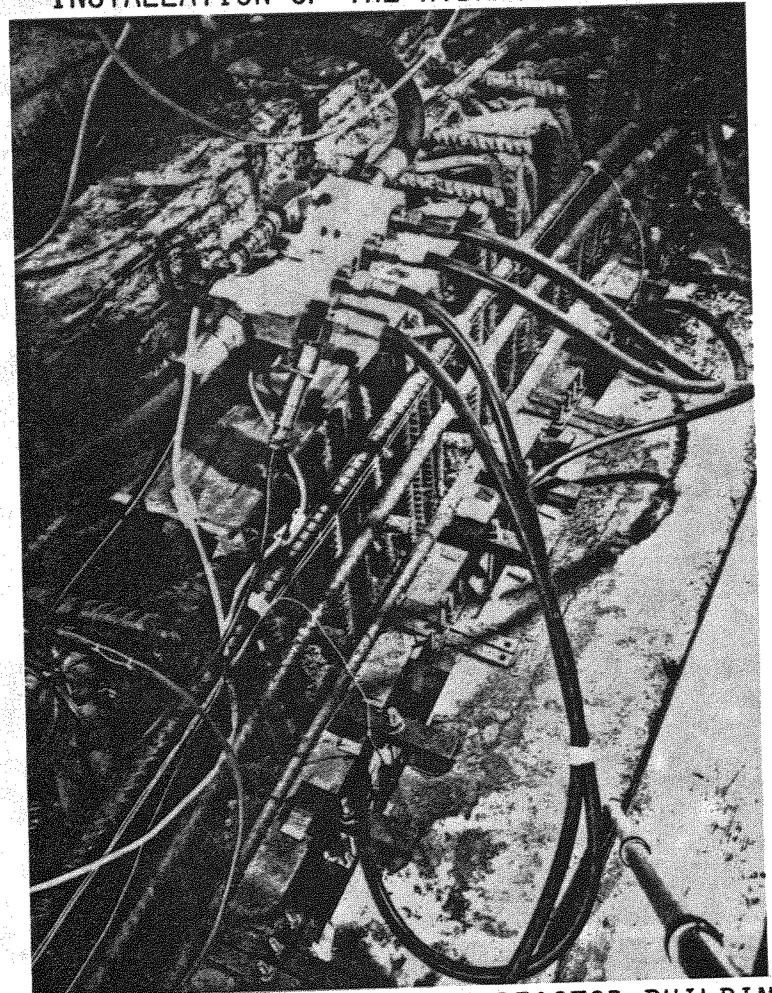


FIG. 6

SEISMOMETERS INSIDE THE REACTOR BUILDING

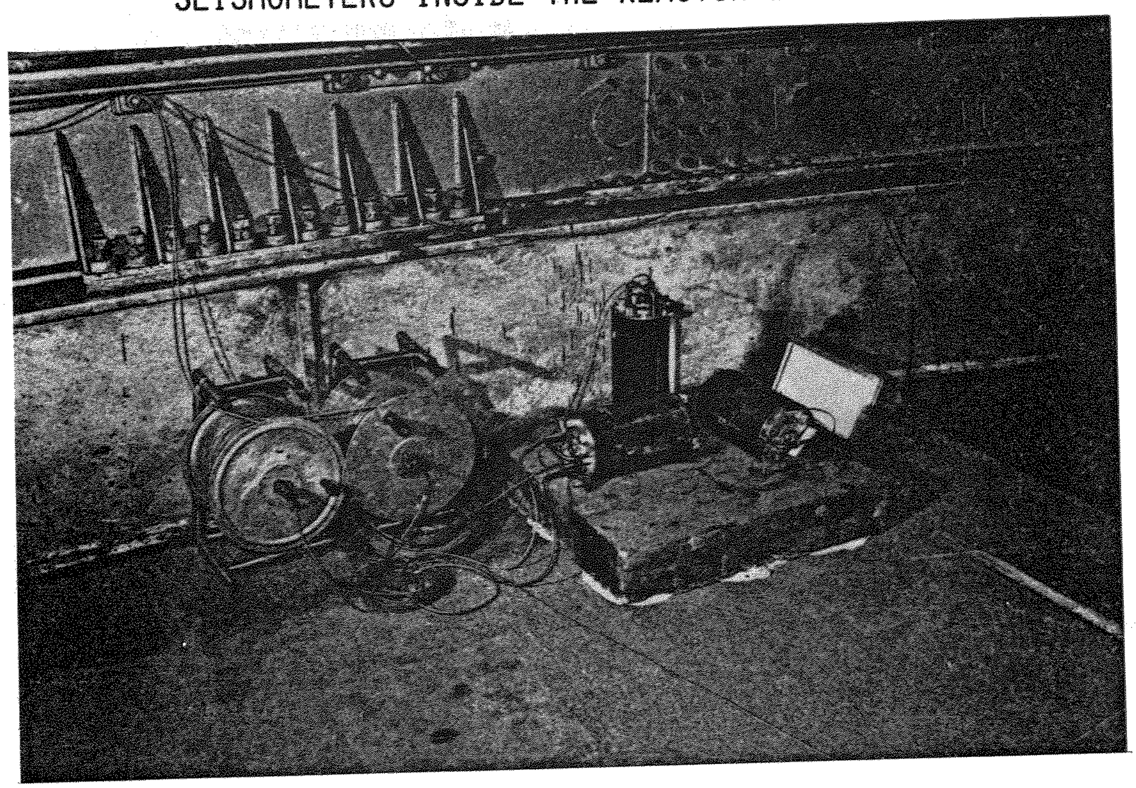


FIG. 7

ACCELEROMETER ON THE STEEL CONTAINER

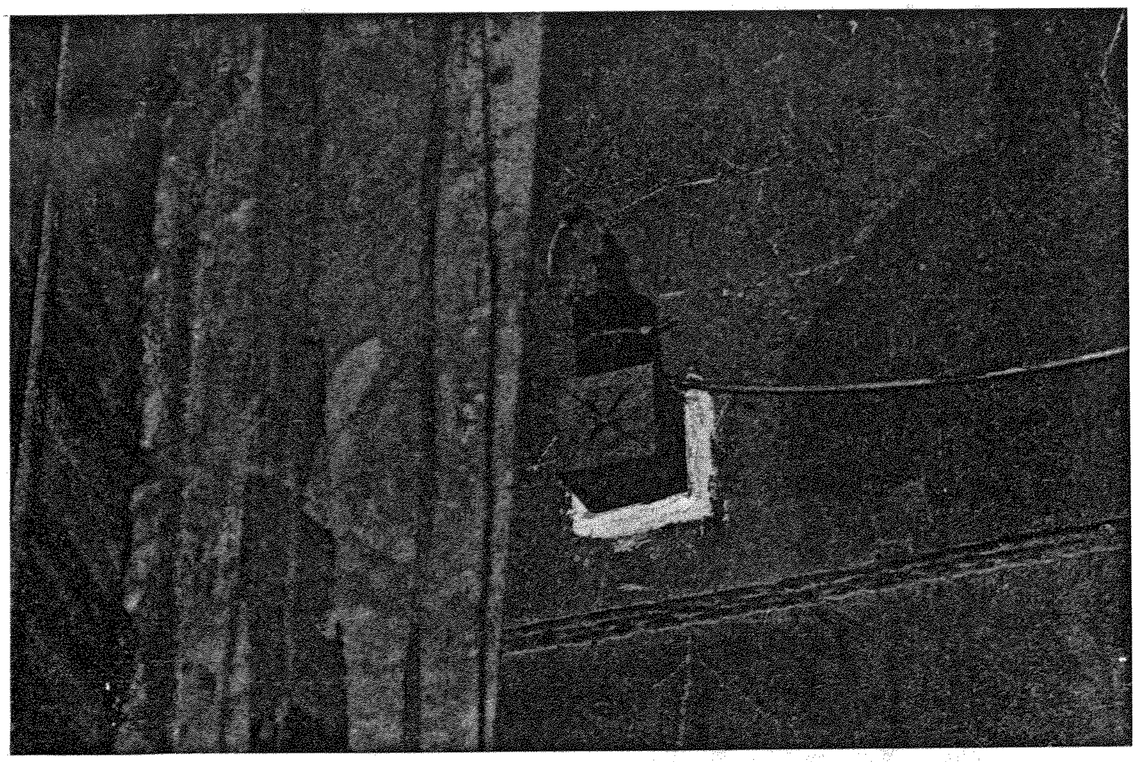


FIG. 8

TRANSDUCERS ON THE ROTATING PLUG

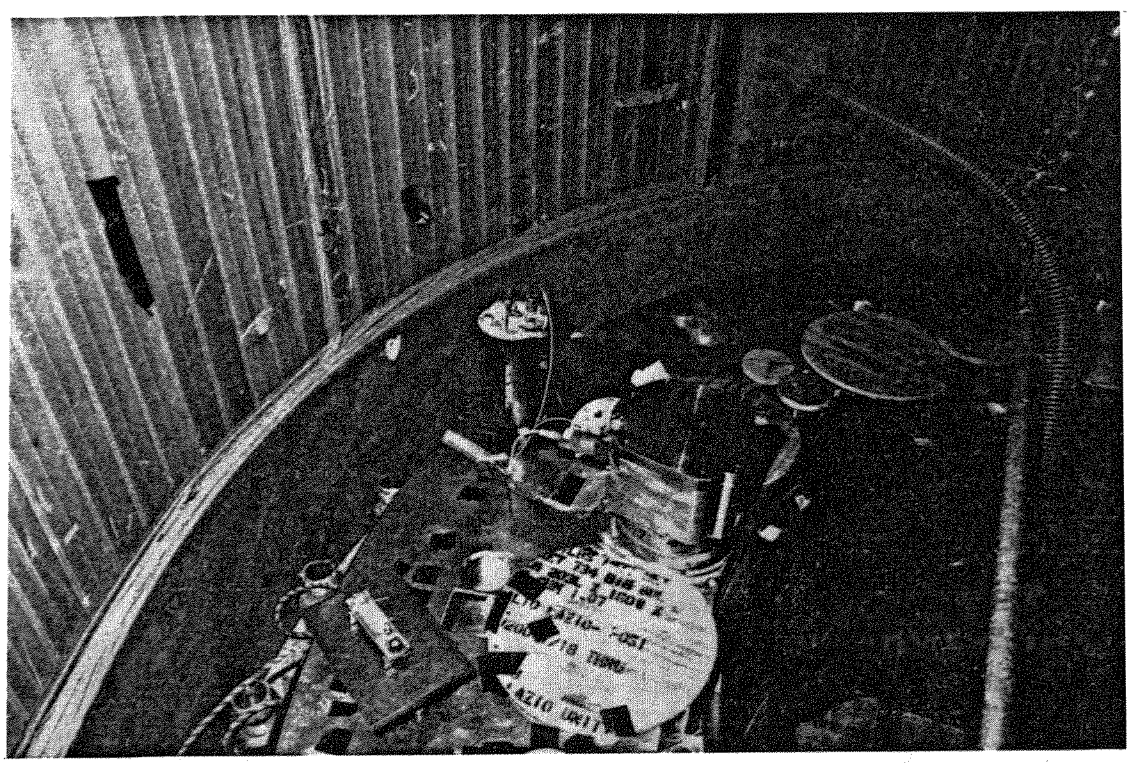


FIG. 9

EIGHTEEN - UNIT SEISMOMETER NETWORK INSIDE THE FUEL ELEMENT HANDLING BUILDING (EMC)

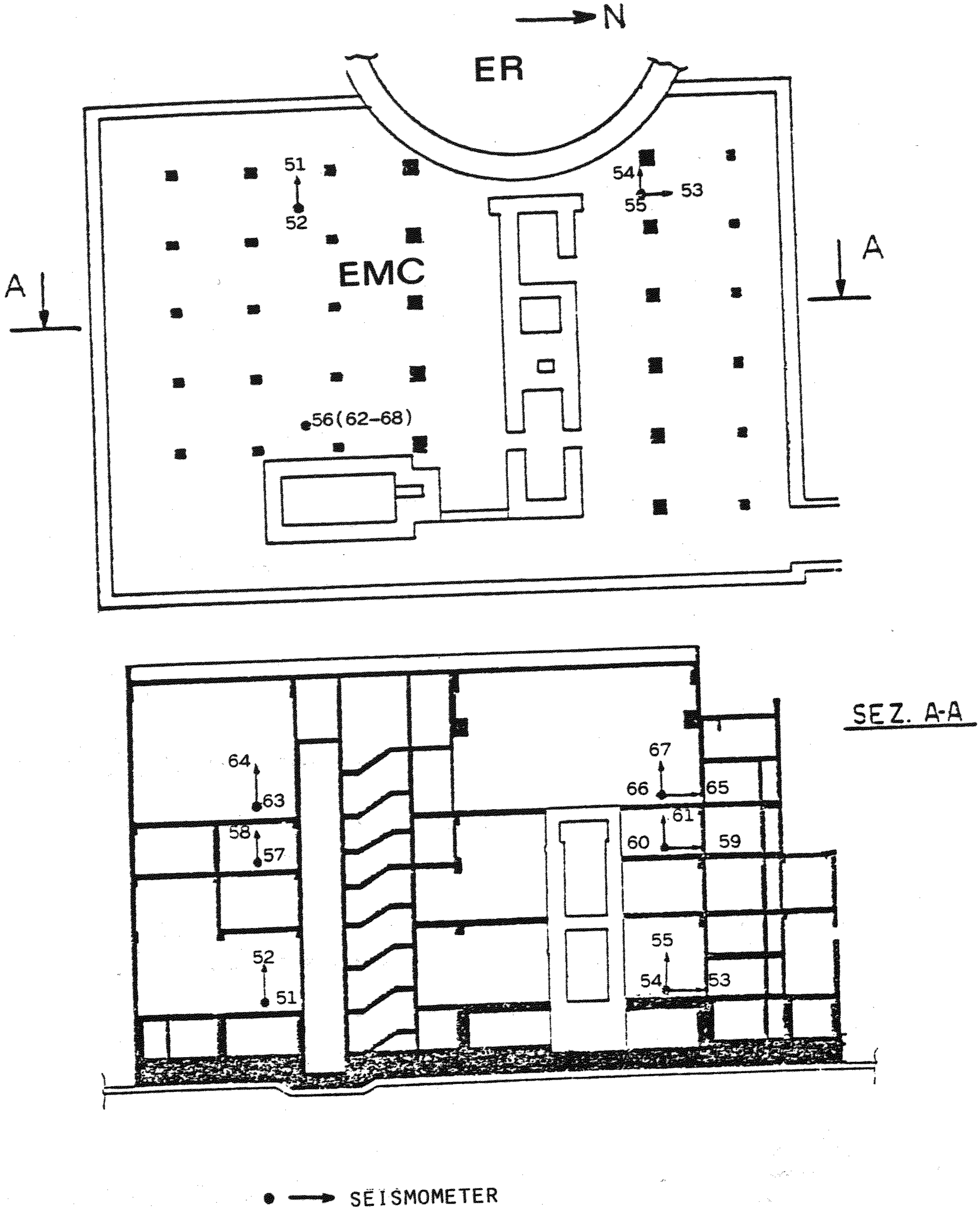


FIG. 10

EXAMPLES OF TRANSFER FUNCTIONS MEASURED DURING THE THREE IN-SITU TESTS BY A MECHANICAL EXCITER

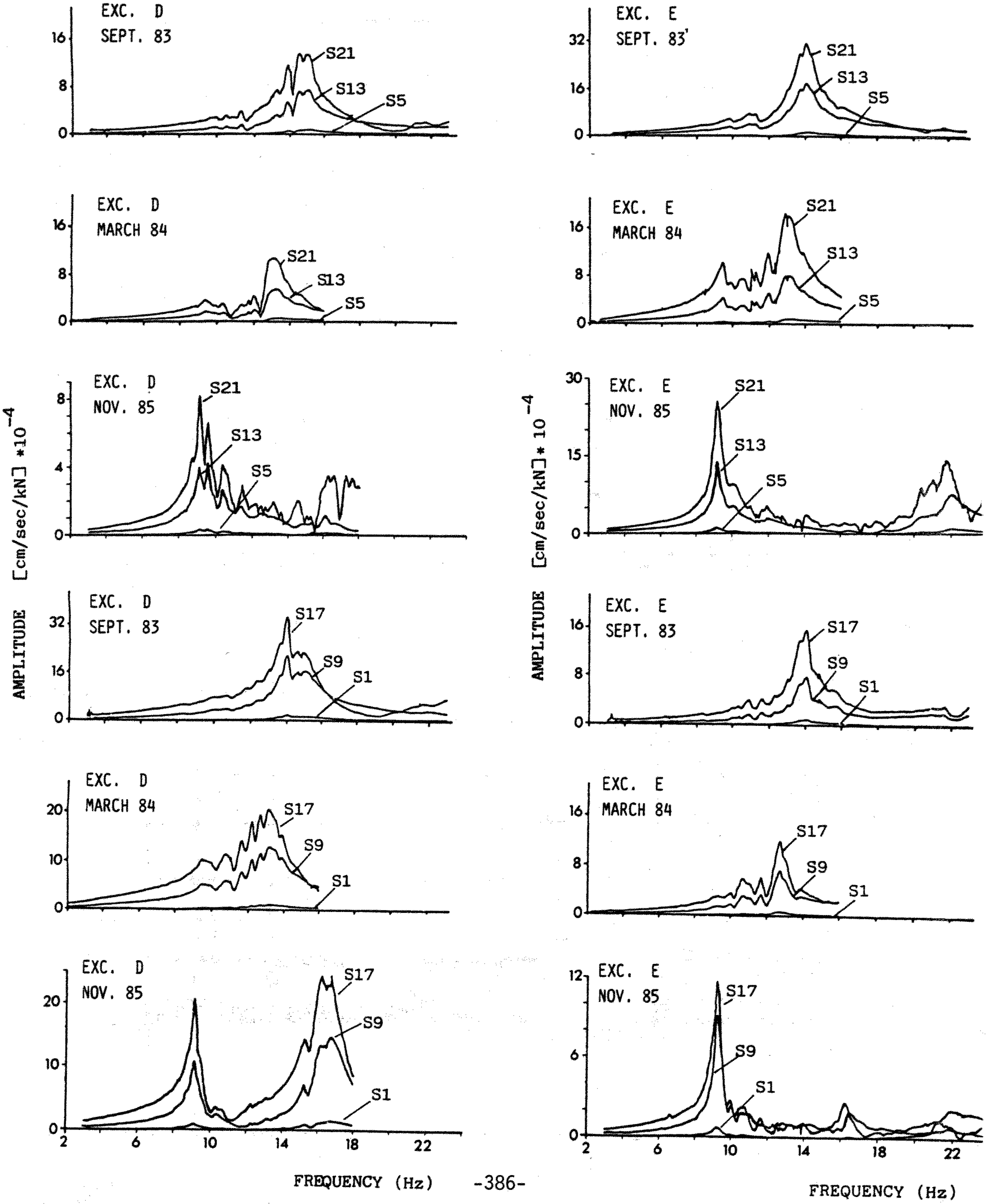


FIG. 11

EXAMPLES OF TRANSFER FUNCTIONS MEASURED DURING THE FINAL IN SITU TESTS(NOVEMBER 1985) BY HYDRAULIC ACTUATORS

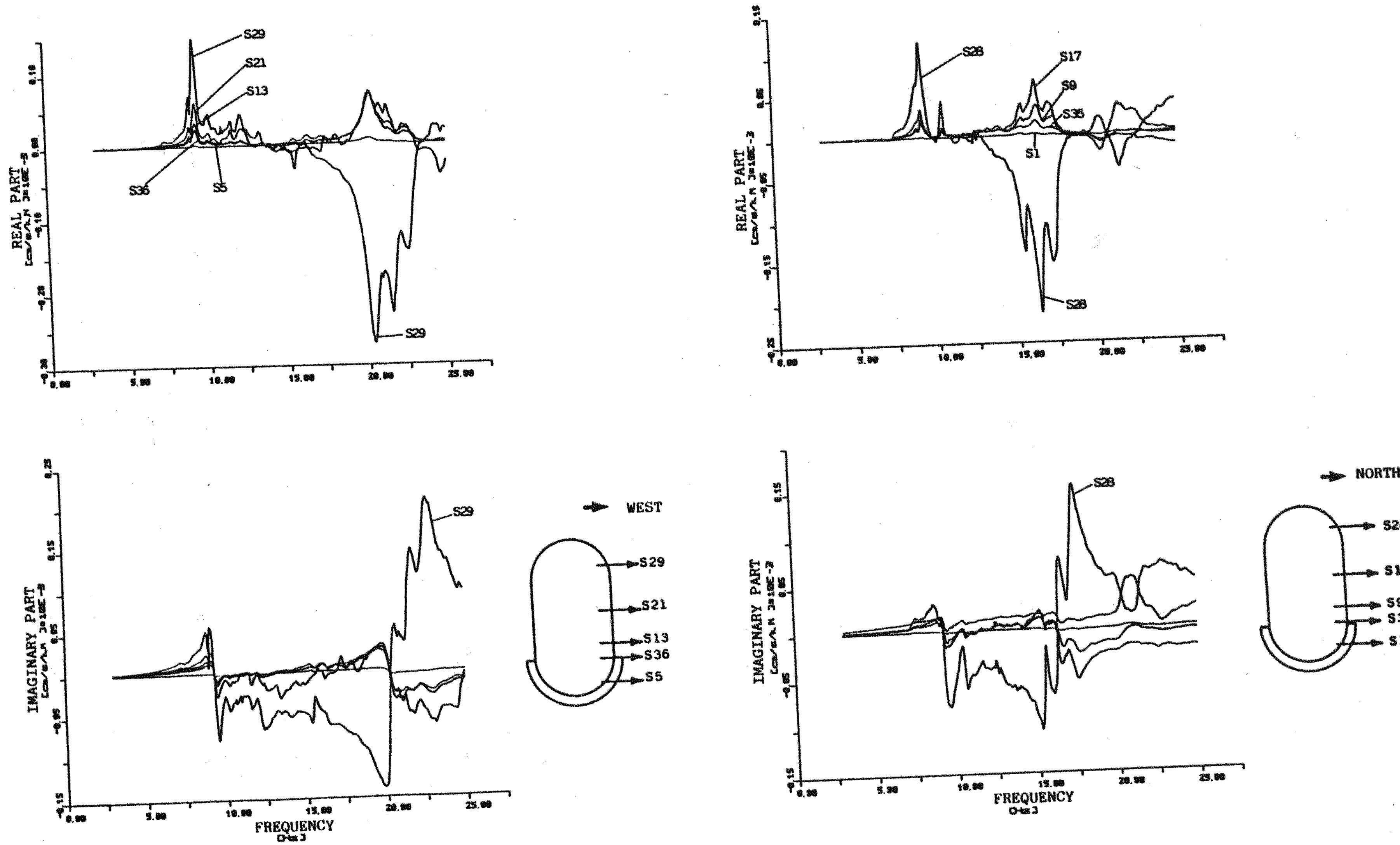


FIG. 12

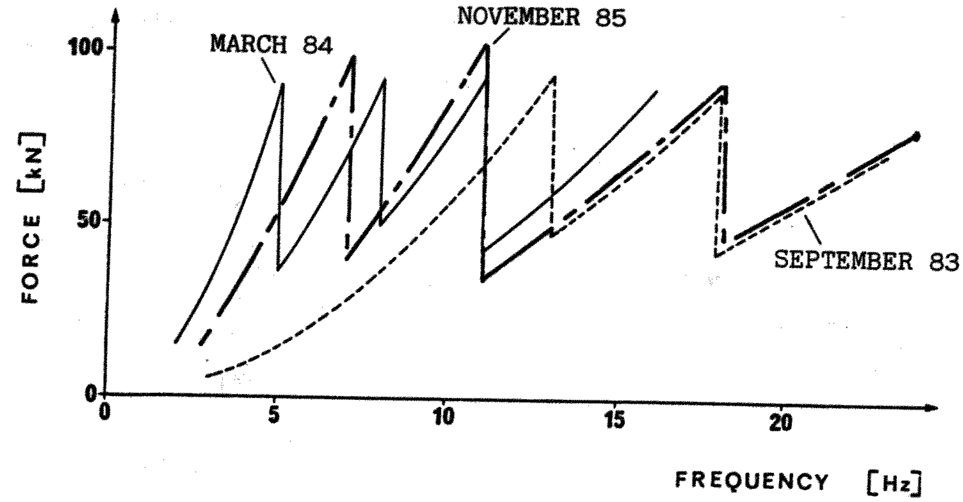


FIG. 13 DYNAMIC FORCES APPLIED DURING THE TEST BY MECHANICAL EXCITER

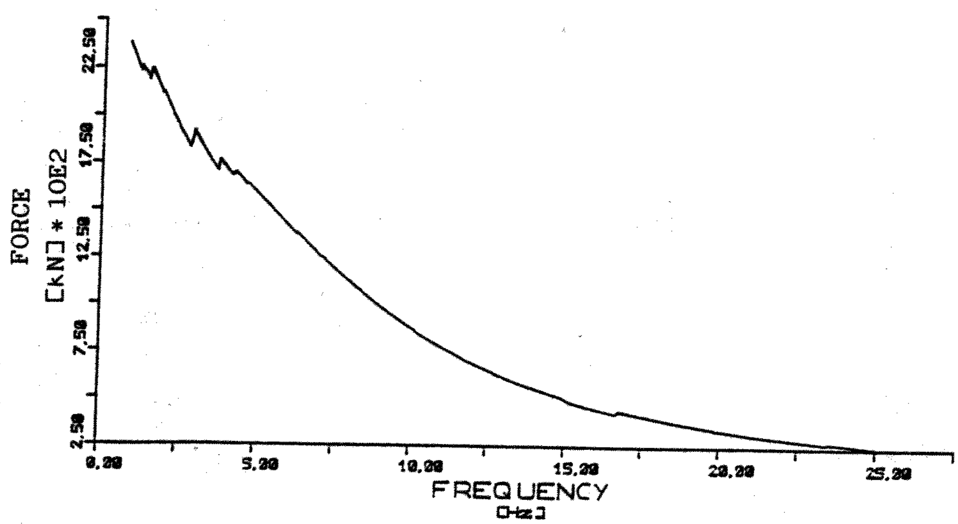
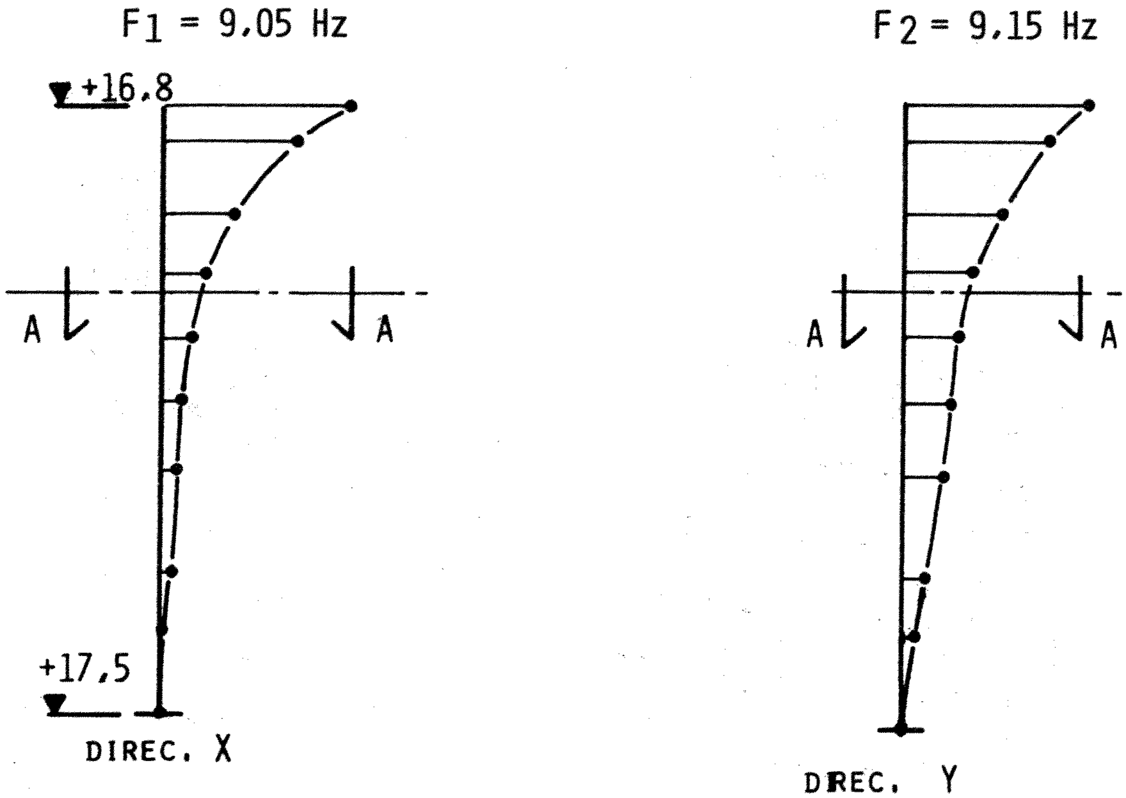


FIG. 14 DYNAMIC FORCE APPLIED DURING THE EXCITATION AT FOUNDATION.

FUNDAMENTAL MODE SHAPES



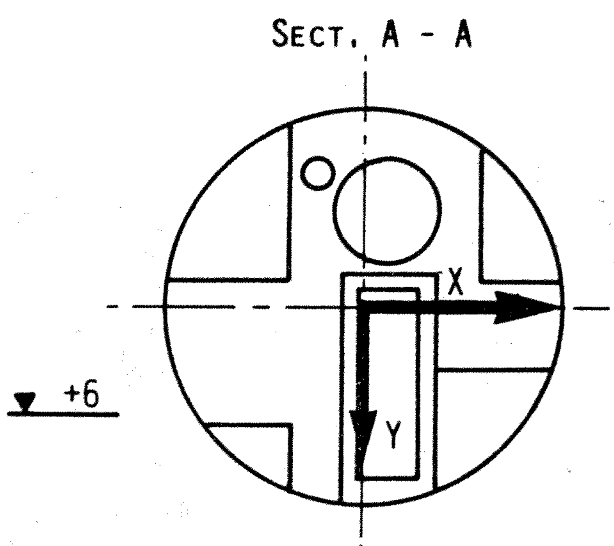


FIG. 15

3D FINITE ELEMENT MODEL

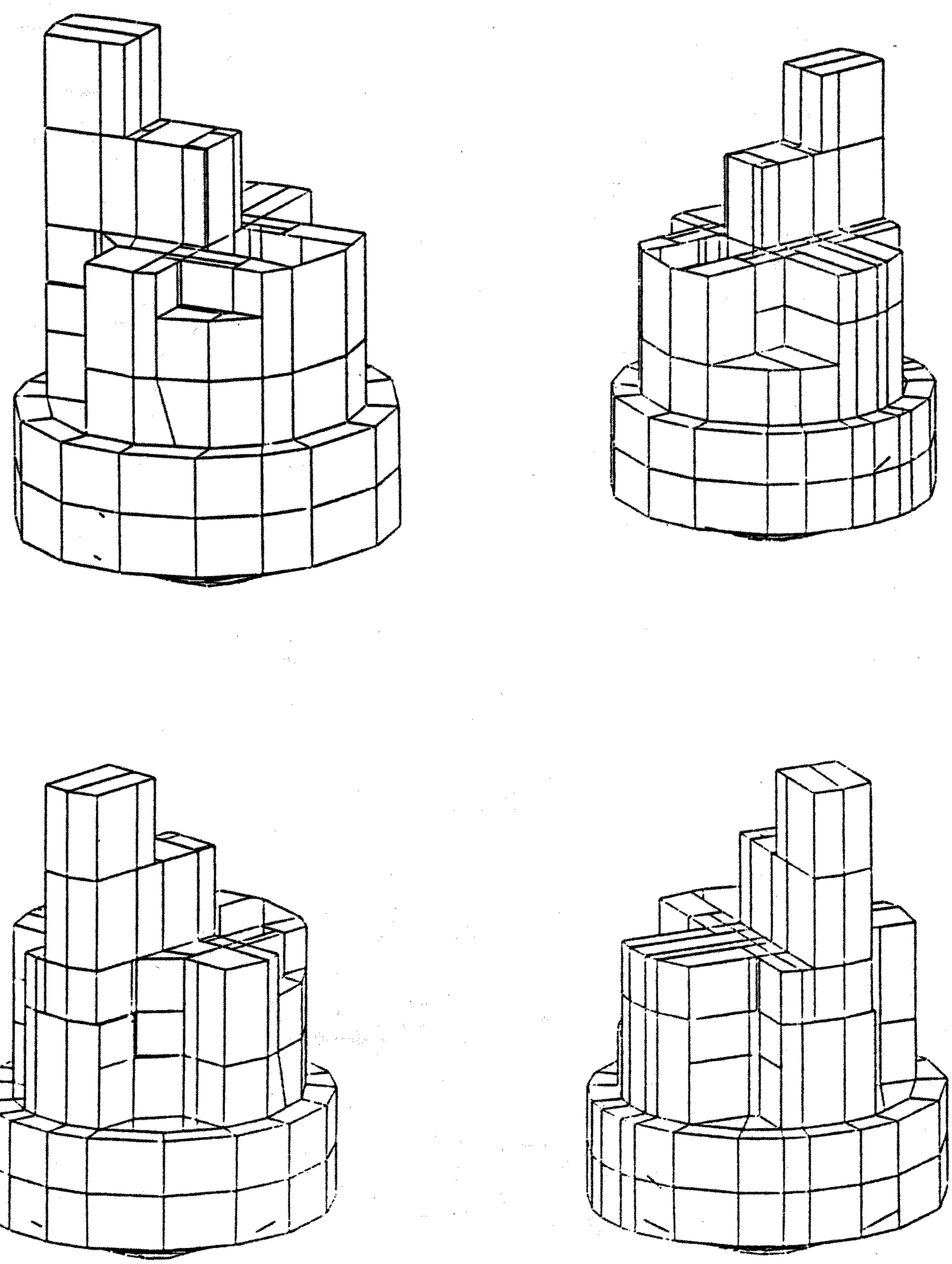


FIG. 16

3D FINITE ELEMENT MODEL

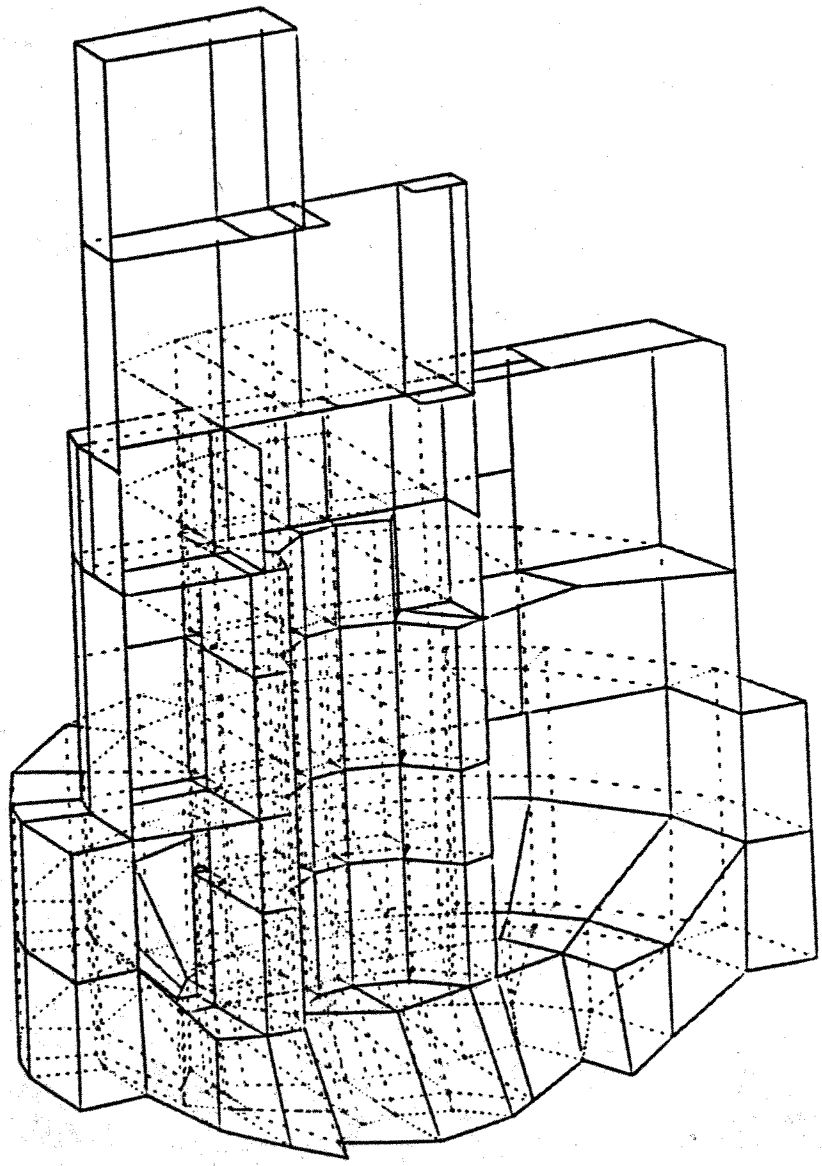


FIG. 17

FUNDAMENTAL MODE SHAPES

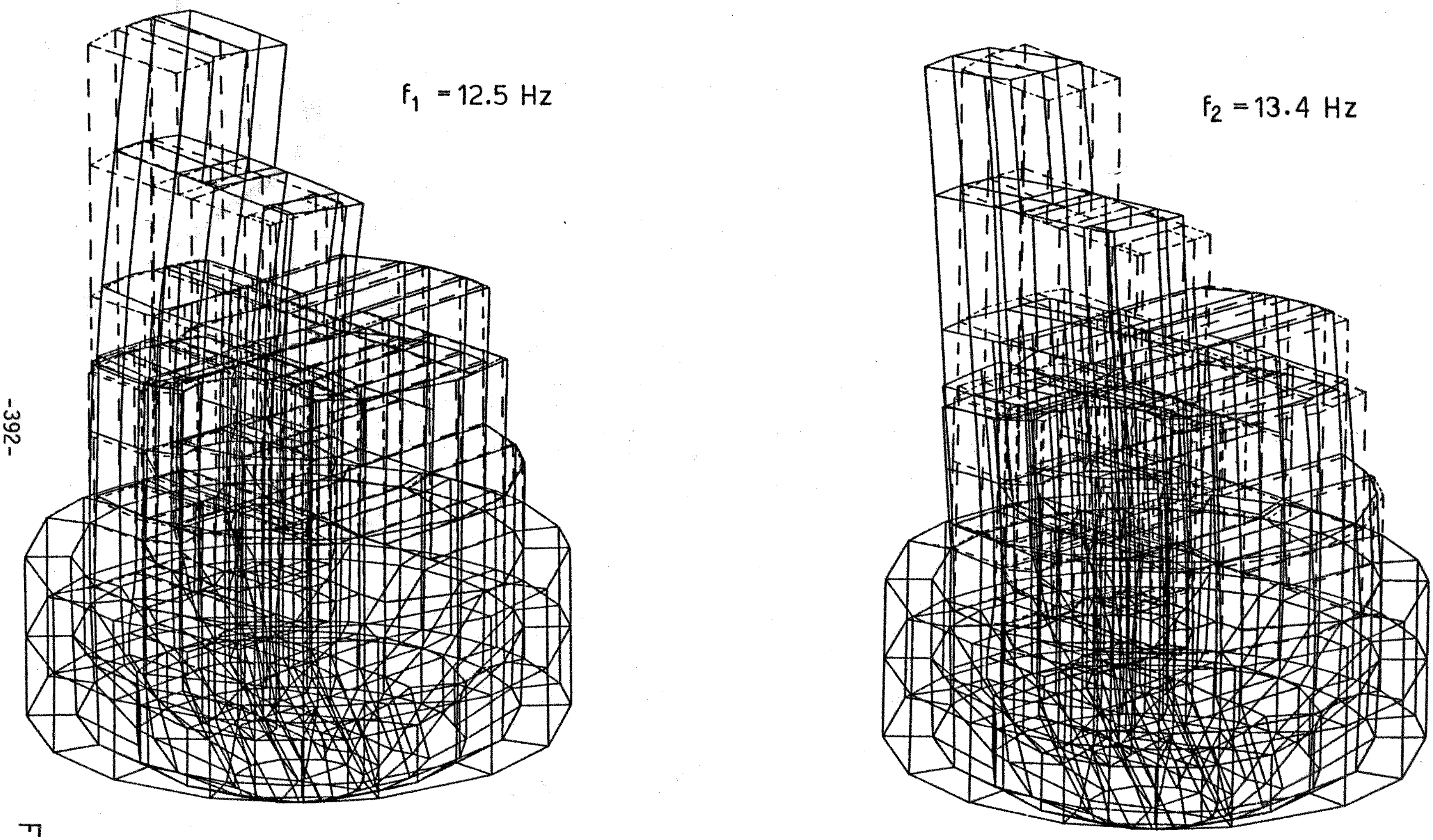


FIG.18

FUNDAMENTAL MODE SHAPES

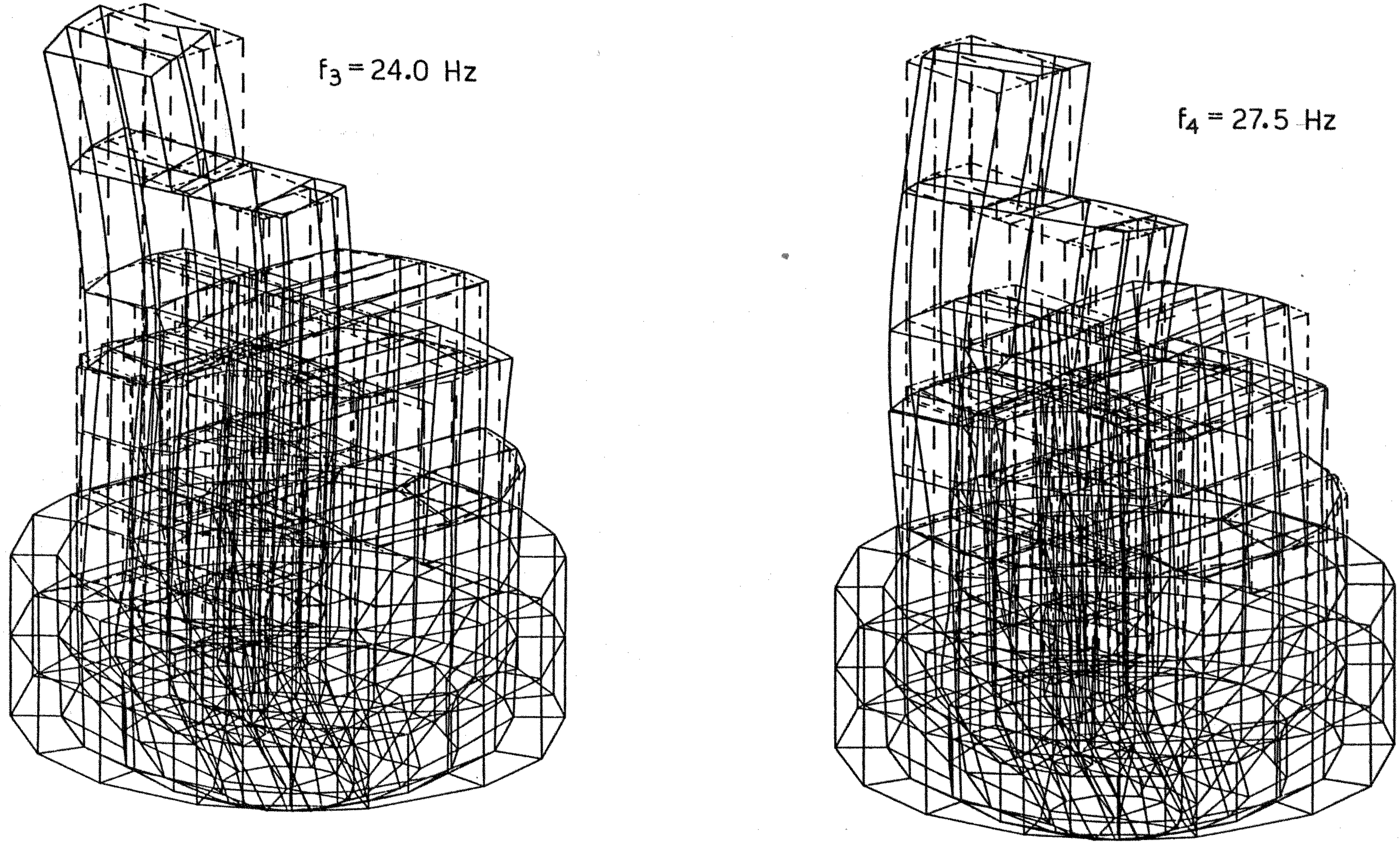


FIG. 19

SIMPLIFIED EQUIVALENT MODEL FOR SSI ANALYSIS

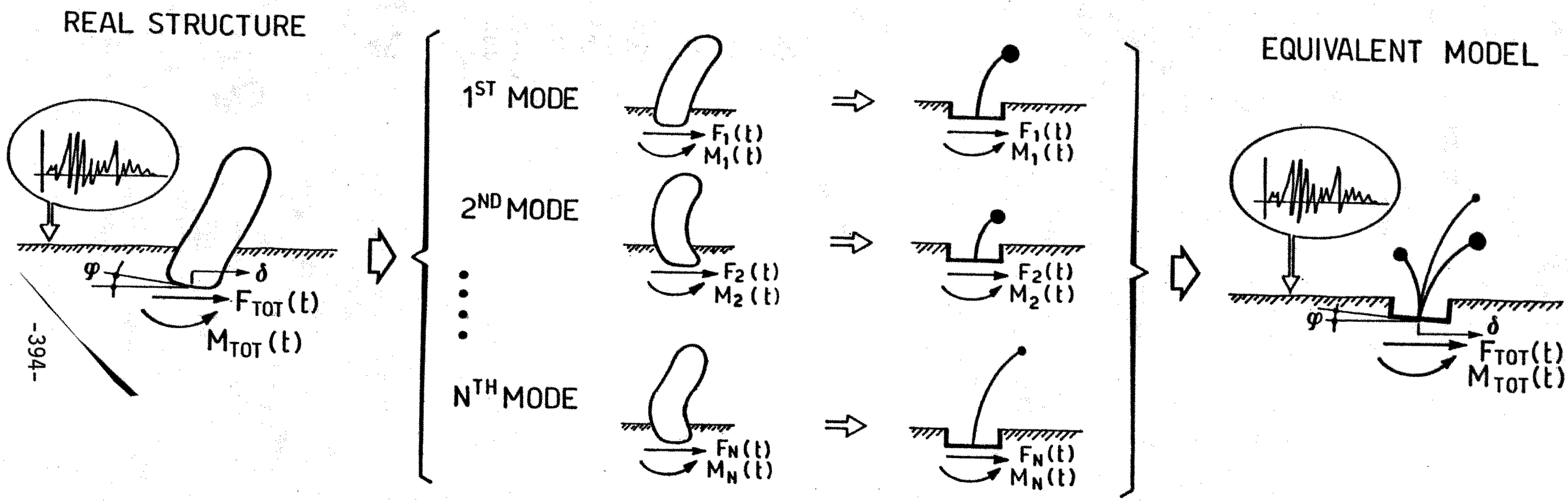


FIG. 20

SESSION 6

SUMMARY SESSION

SUMMARY AND CONCLUSIONS

N. C. CHOKSHI, H. L. GRAVES, A. J. PHILIPPACOPOULOS

As discussed in the program of the workshop, the objective of this workshop was to discuss licensing review criteria and related issues in Soil-Structure Interaction (SSI) area and obtain some guidance to assist the NRC staff in the upcoming revision of the SSI related Standard Review Plan (SRP) sections. Further, the specific objective of the last workshop session, Summary Session, was to present summaries prepared by each of the panels of the previous four sessions. This workshop was successful in achieving both of the objectives. Discussions throughout the workshop remain focused on the broader review criteria related issues. The summaries presented by moderators of technical sessions were prepared after lengthy discussions among panel members. These summaries not only provided definite guidance useful for the development of licensing review criteria but also addressed the specific issues and questions identified in the workshop program. The summaries of technical sessions (sessions 2, 3, 4, and 5) are presented here as prepared by the moderators of each of these sessions. However, before turning to the summaries of technical sessions, some general observations, recommendations, and ideas discussed at the workshop are presented first.

Dr. Harold Denton in his opening address, noted that it is necessary to avoid the compounding of conservatism in each step (seismic input, site analysis, SSI analysis, etc.,) of the seismic analysis/design process such that final results do not contain unquantifiable conservatism. In order to do so, the effective interaction between different desciplines, such as seismology, geotechnical, and structural, involved in the seismic design process is very crucial. The effective interaction should lead to clear understanding of needs of different desciplines and assumptions used in various steps, thus providing consistancy between various steps. The interaction between the regulatory agency and the design organization was also discussed. Some of the foreign participants indicated that the licensing review process in their countries was more interactive and started early enough to establish acceptable technical approaches prior to the detailed designs. The concern with unquantifiable conservatism of the current design process also led to a discussion of performance versus prescriptive requirements. Desirability of specifying a performance criterion over prescriptive requirements were expressed by a number of participants. A truly performance criterion may allow one to perform an analysis using best available techniques and then adding known margin to the final design. However, it was noted that a true performance criterion, in terms of probability of exceedance of some critical parameter, may not be feasible at this time. The degree of prescriptiveness could also vary depending on how explicitly and systematically one accounts for uncertainties in each step of the analysis. The staff's proposed revision in the SSI area is very likely to be less prescriptive than the current version.

The objective of Session 2 was to apprise the audience of the state-of-the-art approaches for ground motion estimations. In this light, four currently used approaches were discussed. The ensuing discussion, again, greatly emphasized interactions between engineers and seismologists to best define the ground motion parameters to achieve the desired design/analysis objectives. A need for revising Appendix A to 10 CFR Part 100 was also discussed. It was noted that in the U.S. the regulatory process is primarily geared to satisfy the legal requirements and technical considerations, in many cases, are secondary.

In Session 3, with respect to the seismic input to be used in the SSI analysis, the importance of the site effects and the use of the site-specific spectra (defined at finished grade or rock surface) were emphasized. The past experience of using a broad-banded site-independent spectra in a SSI analysis has led to some inconsistent results. The discussion in Session 3 centered on the most controversial element of the current licensing criteria, that is the location of input motion for a SSI analysis. The current requirement of defining the control motion at the foundation level in the free-field was examined and discussed at great length. The panel's recommendations on this issue are presented in the session summary.

With respect to the current requirement for SSI methodologies, it was noted that licensing review criteria should not try to regulate "misuse" or "abuse" certain SSI methods or procedures; however, the proper implementation of these procedures should be assured through some other means. In this light, a number of participants pointed out a need for developing simple models to identify critical parameters and to judge the adequacy of the results obtained from a complex analysis. Sensitivity studies and systematic parametric variation should be a part of a SSI analysis.

Four invited papers were presented in Session 5, these papers described some of the more pertinent sources of data (both currently available and planned) that can be used to verify soil-structure interaction methods. These papers also described some correlation studies that were performed using the data to validate various aspects of the SSI problem. A fifth paper was presented by the staff of ENEA-Fast Reactor Department (Italy) describing an experimental program being conducted as an integral part of the construction of a breeder reactor. The data sources discussed were: (1) Fukushima, (2) Humboldt Bay, (3) Lotung experiments in Taiwan, (4) SIMQUAKE experiments , (5) HDR shaker tests in Germany, and (6) Japanese shaker and high explosive tests. Much of the discussion regarding correlation studies centered on the liftoff problem. It was generally agreed that this was only important for the higher level earthquakes.

The NRC staff, with consideration of the above discussed general observations and specific conclusions and recommendations of each of the panel technical sessions and is currently preparing a draft revision of the affected SRP sections. These sections will be issued for public comment prior to finalizing the staff acceptance criteria.

Session 2, Definition of Free-Field Motion (Compiled by Dr. L. Reiter, Moderator)

1. It is important to recognize the large uncertainty associated with ground motion estimation, that it is perhaps irreducible and that we need to express this uncertainty and devise ways of living with it.

2. Ensembles of time histories convey the most seismological information, however, there are practical and regulatory considerations that may limit their use.

3. Site conditions are extremely important in ground-motion estimation and they have to be conveyed in much more sophisticated terms than "rock" or "soil." Lateral variations in site properties pose great difficulties that we may have only very limited means of dealing with.

4. SSI effects may already be present in so-called free-field records. It is important to utilize true free field (as close as possible) records if formal SSI is to be applied.

5. Magnitude is a limited, narrow frequency band description of the earthquake source and multiparameter descriptions of source strength are more useful.

6. While the state-of-the-art of earthquake ground motion estimation has improved greatly over the past years, so has our realization that the earthquake process is more variable than previously thought.

7. The various estimation techniques can provide different parameters including time-histories, however, these parameters may be associated with large and varying degrees of uncertainty. Engineers need to indicate which parameters are needed and work with seismologists in a manner to determine which are most practical.

8. There is a great need that the whole earthquake engineering process, from geologist and seismologist to engineer, be carried out in a way that facilitates maximum communication between disciplines.

Session 3, Ground Motion Input Need for Site Specific SSI Analysis (compiled by Dr. L. Heller, Moderator)

1. Ground motions, including peak ground accelerations and spectral accelerations decrease with depth and these changes should be taken into account in some way in SSI analyses.

2. Ground motions are site-specific, and site-specific motions should be specified for SSI analysis purposes.

3. Wave propagation analysis, incorporating appropriate parameter variations, can provide considerable insight into the variation of ground motions with depth and can be used, together with good judgment, to evaluate the possible magnitude of these effects.

4. For analysis purposes, it is desirable to use realistic time histories of motion, based on actual records, rather than an artificial time history which is developed to fit a specified spectral shape.

5. For study purposes it is desirable that an ensemble of realistic time histories (recorded or synthetic) be used for SSI analysis, and the results evaluated in some meaningful way (probably probablistically) but this should not be done where ample conservatism is already included in the analysis.

6. Free-field ground motions should be specified at the ground surface - either at the surface of a rock outcrop (real or imaginary) in the vicinity of the site or at the ground surface (finished grade) at the plant site.

7. Where analyses are based on limited data and very simplified or incomplete models of SSI effects, a conservative methodology should be adopted. However, where analyses are based on good field data and relatively sophisticated modeling techniques, no special conservatism needs to be incorporated to allow for deficiencies in property determinations and analysis procedures (i.e., alternative analyses).

Session 4, SSI Methodology (Compiled by Dr. P. T. Kuo, Moderator)

1. With regard to the need of ground motion specification for SSI analysis, the panel has the following recommendations:

   The control point should always be defined to be on a free surface. Two cases are identified depending on the soil characteristics at the site. For relatively uniform, sites of soil or rock with smooth variations of properties with depth, the control point should be specified on the soil free surface at the top of grade. The control motion should be consistent with the properties of the soil profile.

   For sites composed of one or more thin soil layers overlying a competent material, the control point is specified on an outcrop or a hypothetical outcrop at the location on the top of the competent material. The control motion should be consistent with the properties of the competent material.

2. In answering the question of whether there is enough confidence in current methods being used in the nuclear industry, the panel's answer is yes. However, the problem is with implementation. The confidence in proper implementation of any SSI methodology as we currently know by the industry is uncertain. Much work should be done in the area of implementation. The panel agrees that sensitivity studies for important parameters should be performed before the adequacy of the final results can be judged. The panel further recommends that some benchmark problems should be established to verify the user's capability of proper implementation of any SSI methodologies.

3. In answering the question of whether sophisticated methodologies are warranted in view of considerable uncertainties, the panel felt rather uncomfortable with the word "sophisticated." Sophistication is in the eyes of the beholder. The panel agrees that any SSI methodologies, simple or complicated, should be capable of taking into account effects of all

major parameters in the SSI analysis. The panel further agrees that a linear or equivalent linear analysis should always be performed. Non-linear analysis should be judged on the basis of a linear analysis.

4. In answering the question of how uncertainties should be addressed in the licensing process, the panel agrees that the following two tasks should be included in SSI analysis:

   a. Well-founded and properly-substantiated simple model can be used to perform certain sensitivity studies for major parameters.
   b. Varying soil properties of ± 50% should be considered.

5. In answering the question of how buried structures should be analyzed to account for SSI effects, the panel agrees that a better way to deal with this problem is to design the structure to be able to accommodate the strains imposed by ground motion with the objective to eliminate the SSI effects.

Session 5, Experience and Experimental Observation (compiled by Dr. C. Miller, Moderator)

1. There is not a sufficient data base to resolve most of the uncertainties in the SSI problem.

2. There is a need for data collected on actual power plants subjected to earthquake. At present, the plants have insufficient instrumentation, especially in the free field.

3. Data is lacking for sites where non-homoginieties and nonlinear site effects (e.g., water table, layering, soil properities) are important. The Department of Defense is conducting and has planned high explosive tests in a great variety of soil conditions. Perhaps tests could be planned as "tag along" on the DOD tests that would provide useful data.

4. A more complete evaluation of all available data sources is needed.

5. Standard problems should be established for verification of SSI methodologies. Care must be taken, however, to assure that these problems cover the range of parameters of interest. They should not be restricted to uniform sites and low levels of excitation so that a false sense of security is not developed by comparing analytical methods with the simplest of the SSI problem.

6. Gravity effects do not scale most SSI related tests. Care should be taken in the methods used to extrapolate scale model test data to full scale.

7. Data collected at the recent earthquake at the Perry plant should be reviewed as to its applicability to the SSI problem.

8. Current SSI methodology works well for moderately sized earthquakes acting on uniform sites.

9. Uncertainties in SSI may not be as important as those in the specification of the hazard. There was a discussion regarding the lack of evidence showing that improper treatment of SSI led to unexpected damage. There was a wide difference of opinion, however, over whether the normal mechanisms used to evaluate earthquake damage allows for an indentification of the reason for the damage.

10. It was generally agreed that SSI methodologies should ultimately be verified with experimental data. The data should be applicable (in terms of site properties and level of input) to the problem to which the method is to be applied.

NRC FORM 335 (2-84) NRCM 1102, 3201, 3202

U.S. NUCLEAR REGULATORY COMMISSION

BIBLIOGRAPHIC DATA SHEET

SEE INSTRUCTIONS ON THE REVERSE

1. REPORT NUMBER (Assigned by TIDC, add Vol. No., if any)
NUREG/CP-0054
BNL-NUREG-52011

2. TITLE AND SUBTITLE
Proceedings of the Workshop on Soil-Structure Interaction

3. LEAVE BLANK

4. DATE REPORT COMPLETED

MONTH	YEAR
November	1986

5. AUTHOR(S)
H.L. Graves and A.J. Philippacopoulos

6. DATE REPORT ISSUED

MONTH	YEAR
December	1986

7. PERFORMING ORGANIZATION NAME AND MAILING ADDRESS (Include Zip Code)
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8. PROJECT/TASK/WORK UNIT NUMBER

9. FIN OR GRANT NUMBER
A-3242

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11a. TYPE OF REPORT
Formal

b. PERIOD COVERED (Inclusive dates)

12. SUPPLEMENTARY NOTES

13. ABSTRACT (200 words or less)

The Workshop on Soil-Structure Interaction provided an exchange of information between regulators, practitioners and researchers for the purpose of examining SSI licensing criteria in the light of recent analytical and experimental development. These proceedings contain the papers presented by panelists and summaries of the sessions along with recommendations of the panel members for each session. Technical areas covered by the panels were (1) definition of free-field motion, (2) ground motion input needed for site specific SSI analysis, (3) SSI methodology, and (4) experience and experimental observation. The summaries were derived to identify areas in the licensing criteria which could be changed to improve the licensing process.

14. DOCUMENT ANALYSIS – a. KEYWORDS/DESCRIPTORS

b. IDENTIFIERS/OPEN ENDED TERMS

15. AVAILABILITY STATEMENT

16. SECURITY CLASSIFICATION
(This page) unclassified
(This report) unclassified

17. NUMBER OF PAGES

18. PRICE

U.S. Government Printing Office: #711-035 PO#40007