# EXPERIMENTAL NEEDS FOR GEOTECHNICAL EARTHQUAKE ENGINEERING

Report of a Workshop held in Albuquerque, New Mexico
on November 4 and 5, 1991,
under the sponsorship of the National Science Foundation.

NSF Grant No. BCS-9017661

Organizing Committee:

Cornelius J. Higgins, Chairman
Applied Research Associates, Inc.

Clifford J. Astill
National Science Foundation

Koon Meng Chua
The University of New Mexico

Paul F. Hadala
U.S. Army Waterways Experiment Station

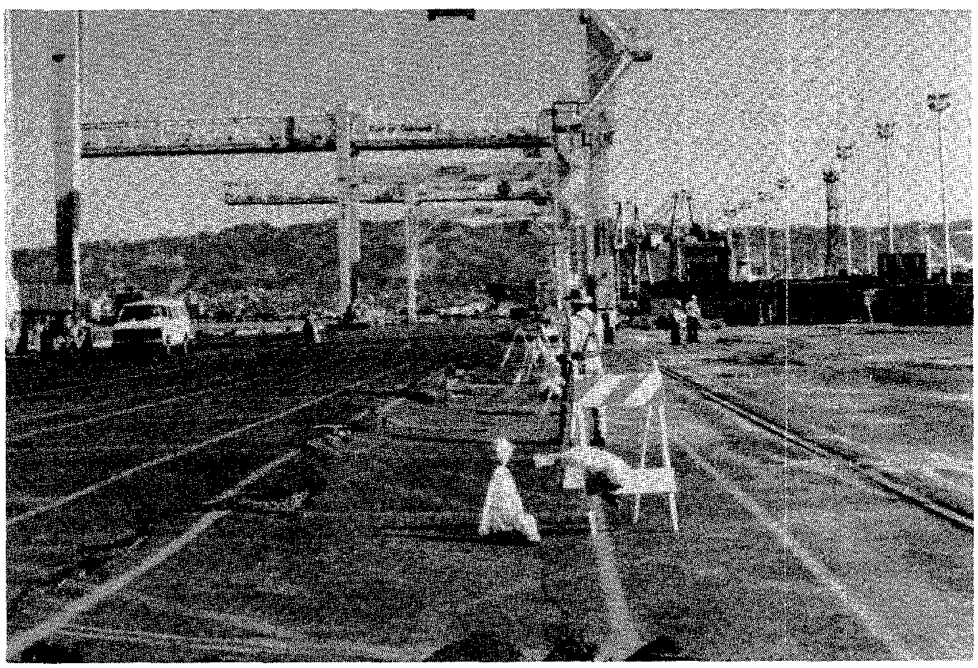
Ronald F. Scott
California Institute of Technology

H.T. Tang
Electric Power Research Institute

Richard D. Woods
University of Michigan

T. Leslie Youd
Brigham Young University

Rio Vizcaya Bridge: Liquefaction Failure During 1991 Costa Rica Earthquake (Ref. 7, fig. 6-21, pg. 74, reprinted by permission of Earthquake Engineering Research Institute).



Settlement of Fill Supporting a Rail for Bridge Crane at Port of Oakland,
Loma Prieta Earthquake (Photograph by Marshall Lew) (Ref. 5, fig. 4.22,
pg. 100, reprinted by permission of Earthquake Engineering Research Institute).

## EXECUTIVE SUMMARY

Geotechnical factors have a major influence on the performance of almost every civil engineering structure in an earthquake. This importance has been demonstrated by catastrophic or near catastrophic failures during major earthquakes over the past 30 years. Significant attention to geotechnical issues began after the extensive liquefaction in the 1964 Niigata, Japan earthquake, the catastrophic landslides and ground failures in the great Alaskan earthquake of the same year, and the near-failure of two critical earthdams in the 1971 San Fernando earthquake. Soil-structure interaction effects on the behavior of nuclear power plants in earthquakes have been a major source of uncertainty.

Our lack of understanding of dynamic geotechnical behavior continues to be confirmed time and again as new earthquakes occur. Each new earthquake serves to dispel long held misunderstandings and to reveal important new insights. The 1979 Imperial Valley earthquake demonstrated that horizontal accelerations on level ground above 0.5g were physically possible; vertical accelerations above 1g occurred in the epicentral region. The Mexico earthquake of 1985 demonstrated significant amplifications of peak motions, duration, and spectra due to local site conditions involving deep, soft soil layers. The 1989 Loma Prieta earthquake in California, probably the most relevant from a United States perspective, was a major geotechnical earthquake which caused widespread damage due to soil- and rock-related failure or phenomena. The earthquake was moderately large in magnitude ($M_s = 7$) but the duration was relatively short. There was substantial damage in San Francisco and in Oakland and Alameda, over 50 miles from the epicenter. Significant geotechnical responses/failures included ground motion amplification, liquefaction in loose, sandy hydraulic fills, a large number of landslides, settlement of highway bridge approach and abutment fills, and bridge failures due to liquefaction. The overall damage in the Loma Prieta earthquake is estimated at $7 to $10 billion. It is probably reasonable to estimate that at least one-half this amount was due to geotechnical related failures.

Post-earthquake observations have given excellent documentation and insight into geotechnical hazards. Yet, there are large remaining uncertainties due to lack of measurements at locations of interest and lack of data on pre-earthquake insitu conditions. The removal of these uncertainties will require direct experimental programs under carefully controlled conditions. This need for direct experiments led to the workshop reported here.

The overall objective of this workshop was to provide an evaluation of current experimental needs for geotechnical earthquake engineering and current capabilities to support those needs. Some specific objectives were to:

- Determine current needs for experimental data in support of geotechnical earthquake engineering issues.
- Establish the level(s) of faithfulness with which test methods must represent earthquake ground motion fields to meet geotechnical testing needs.
- Document past experience and the current status of various testing techniques including: actual earthquakes and after-shocks, shake tables, explosive simulation, external shaker and pulser excitation, and dynamic tests on centrifuges.

- Evaluate the fidelity and suitability of each test method for meeting current needs including: cost-benefit factors for each method, integration of the methods into a hierarchy of testing for cost effective achievement of experimental objectives, and compatibility of the methods with university research approaches.

Workshop participants included over 50 researchers and practitioners from academia, government, and private industry. Their objective was to develop recommendations on overall experimental needs, requirements for enhancement/improvement/evaluation, and test programs which will lead to improved safety of geotechnical designs.

The format for the workshop included state-of-the-art presentations on geotechnical earthquake engineering issues, and experimental needs and methods; panel/writing sessions covering specific topics; and plenary discussions. The topics and speakers were:

- Observational Methods: Ralph B. Peck
- Earthdams and Natural Slopes: Larry Von Thun
- Soil-Structure Interaction (covering shallow and deep foundations, retaining structures, and underground facilities): Stuart Werner
- Ground Motion (covering amplification, ground stability, and site improvement): Geoffrey Martin
- Tests of Full Size Facilities by Actual Earthquakes and After-Shocks, and by Shakers and Pulsers: T. Leslie Youd
- Explosive Simulation Tests: Cornelius J. Higgins
- Dynamic Tests on Shake Tables and Centrifuges: Hon Yim Ko

Panels, organized around geotechnical topics, considered the needs in more detail, as well as the applicability, fidelity, and cost effectiveness of the various experimental methods. There were six panels: 1) Earthdams, 2) Natural Slopes, 3) Foundations, 4) Retaining and Underground Structures, 5) Ground Motion Response, and 6) Ground Instability and Site Improvement.

Six categories of knowledge gaps were identified by the panels: 1) Input Ground Motion, 2) Site Characterization, 3) Fundamental Behavior and Properties of Soil and Rock, 4) Behavior/Response/Failure Modes of the Geotechnical System, 5) Analytical Models (First Principle Models, Simple Models for Practice, Model Validation, Model and Parameter Selection) and 6) Mitigation/Soil Improvement.

1) Specific ground motion concerns included coherency and spatial variation of motions, the effects of wave type (P, S, R, etc.), vertical motions, local site and topographic amplification, fault rupture, especially beneath dams, and overall prediction uncertainties.

2) Detailed site characterization in three dimensions was especially important for large geotechnical systems where the site can be variable over the dimensions of the system and site details can dominate response. Although site characterization is clearly important for earthquakes and most other geotechnical problems, dynamic and otherwise, it was not the focus of this workshop.

3) Soil and rock behavior is a basic input into understanding and predicting the overall behavior of geotechnical systems. As with site characterization, the behavior of

geotechnical materials was not a direct topic in this workshop. Fundamental behavior is of paramount importance, however, and must be pursued by NSF and other research agencies.

4) Major knowledge gaps which require experimental evaluation for their resolution/illumination begin to emerge in the consideration of the response modes of geotechnical systems. Response and failure modes are major uncertainties because either earthquakes have not occurred in the vicinity of major geotechnical systems and/or post-earthquake observation and information are inadequate.

5) First principle finite element or finite difference models can provide some insight into response and failure modes. However, they are limited by their ability to handle large strain, nonlinear and inelastic behavior, three dimensional input, site conditions and system geometry, and details of boundary and interface conditions. Several panels pointed out the lack of validated models. Panels also commented on the need for simple models for use in practical design and analysis.

6) There is a general lack of information on the effectiveness of mitigation and soil improvement measures, including the use of geotextiles and density improvements. There is also significant uncertainty on the spatial extent and depth of improvement required.

The experimental methods recommended by the panels included: 1) Post-Earthquake Observations, 2) Instrumented Sites in Seismically Active Regions (and Mobile Instrumentation for Aftershock Monitoring), 3) Artificially-Induced Ground Shaking, 4) Centrifuge Tests, 5) Shake Table Tests, and 6) Field Shaker and Snapback Tests.

All panels supported the continued conduct of post-earthquake observation of geotechnical systems.

Instrumented sites in seismically active regions were recognized by all panels, without controversy, as the ideal experimental condition. Experimental fidelity factors such as wavefields, boundary conditions, and scaling are not issues in actual earthquakes. All panels recommended permanent instrumented geotechnical projects at active sites.

Although instrumented sites in seismically active regions provide ideal experimental conditions, the expected results from such experiments are limited by lack of knowledge of precise time and location of the event, as well as restrictions of numbers of measurements and limited geotechnical site information. More use should be made of rapid instrumentation of sites immediately after earthquake occurrence. Controlled, artificially-induced ground shaking to simulate earthquake motion would permit concentration of instrumentation in spatially optimized locations. Knowledge of the time, location, and source characteristics would permit pretest predictions, better experimental planning, and higher instrumentation reliability. Candidates for artificially-induced ground shaking include explosives and water pulse devices (being used by the Russians as a seismic source). A limited amount of geotechnical information at low levels of excitation can be obtained from the installation of shaking machines on instrumented structures.

Centrifuges have the advantage of permitting small scale experiments to be performed in a properly scaled gravity field.

Only passing interest was expressed in the use of shake tables for geotechnical studies. This was due mainly to concerns about scaling and boundary influence because of the limited size of shake tables in the U.S. Japanese participants at the workshop stated that geotechnical studies are conducted in Japan on their large shake tables.

Technical barriers were defined as those issues which must be resolved before adequate experimental programs can be conducted. Three significant technical issues were raised relating to instrumented sites in seismically active regions. First and most important was the issue of instrument reliability/longevity, second was site aging effects, and third was the uncertainty associated with time and location of the earthquake. This is really the origin of the instrument reliability and site aging issues.

Explosive simulation is the main artificially-induced ground shaking method that has been used to date. Since the primary motion from explosions in the near-field is from P-waves, there is concern that the response will be different than S-wave induced response, thought to be the main component in earthquakes. Concerns were also expressed about non-uniformity of motion, higher frequency content, shorter duration, and fewer cycles of motion in explosions. It was recommended that experiments and analyses be pursued to examine and resolve these issues.

The existence of several technical barriers was raised regarding the use of centrifuges for geotechnical earthquake engineering studies. One barrier was the size of even miniature transducers and instrumentation cables at the scales used in centrifuges. Another significant barrier for studies of saturated soils, especially liquefaction, is the fact that time for water flow (diffusion) scales as $N^2$ while time for dynamic response scales as N. Other technical barriers include size limits and resulting boundary reflections and in-flight property determination. The major technical barrier to progress with centrifuges is the lack of high capacity shaker systems for the larger centrifuges in the U.S. One-dimensional shakers are an immediate requirement.

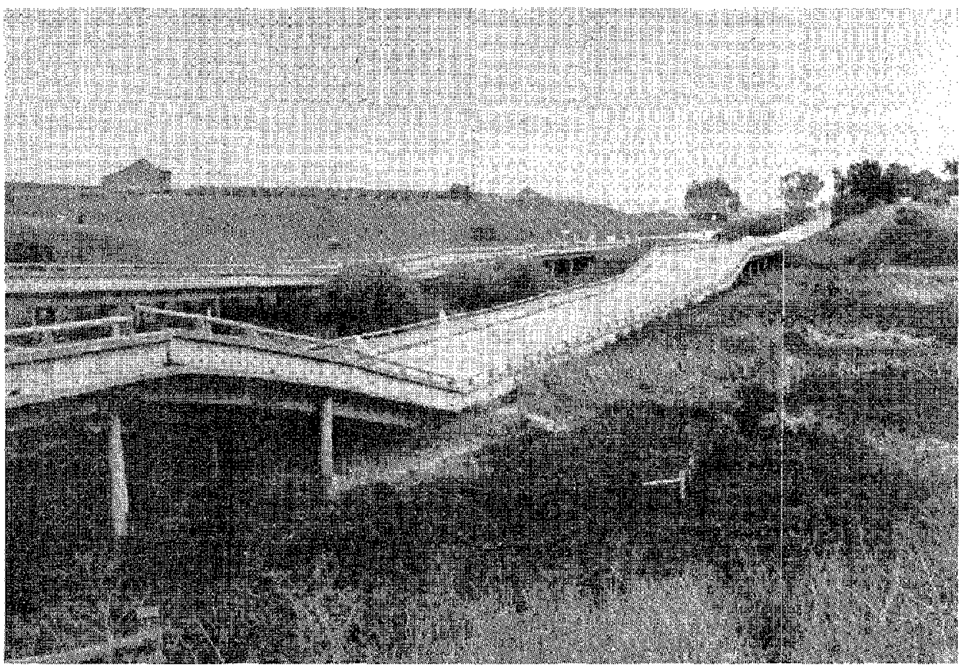
The major technical barrier to meaningful shake table studies is the limited size and capacity of U.S. shake tables. Larger shake tables, similar to the large tables in Japan, would enable some geotechnical studies to be conducted under satisfactory conditions.

The lack of sufficient funding was noted at the workshop as an overriding non-technical barrier. This leads immediately to the non-technical barriers associated with a lack of equipment, and inability on the part of the community to aggressively pursue large size projects. Serious experiments cost significant amounts of money. Progress in the area of geotechnical earthquake engineering has been significantly retarded by lack of funding. Other non-technical barriers included the lack of central organizations tasked with the deployment and long term maintenance of permanent instrumentation at geotechnical test sites. The USGS, Army Corps of Engineers, and California SMIP instrument and maintain their own sites, but it is difficult for geotechnical researchers to gain support from these organizations for deployment or maintenance of geotechnical test sites.

It is the conclusion of the organizing committee that research funding to support the needs identified by this workshop is too low in proportion to past and continuing investment in geotechnical infrastructure and structures influenced by geotechnical behavior. Loma Prieta damage, mainly due to geotechnical causes, is estimated at $7 to $10 billion. A 1 percent investment in research of say $100 million might have reduced the damage by as much as 25 percent. Considering a ten-year investment period (perhaps roughly on the order of the return period for a significant damage causing earthquake), one could easily justify an investment of an added $10 million per year for geotechnical earthquake engineering. There are many other ways to derive similar estimates. The conclusion is simply that the nation must find ways to invest additional resources into this research area.

Some combination of government and industry must address these funding requirements. The construction industry in the U.S. has little incentive to innovate and support building research. More incentive must be provided. The adversarial relationship that often exists between owner, engineer, and constructor, the high level of litigation in the U.S., and constraining government regulations lead to uncreative, traditional solutions only because they have been accepted in the past. As a result, research leadership and the bulk of building research funding has been and, apparently, must continue to be provided by the Federal Government.

TABLE OF CONTENTS



Liquefaction Damaged State Highway 1 Bridge Over Struve Slough
(Photograph by Marshall Lew) (Ref. 5, fig. 4.33, pg. 109, reprinted
by permission of Earthquake Engineering Research Institute).

# CHAPTER 1
# INTRODUCTION

## 1.1 BACKGROUND

Geotechnical earthquake engineering issues began to receive serious attention in the U.S. in the 1970's in the wake of catastrophic or near catastrophic failures during earthquakes in the 1960's and 1970's. The main events which illuminated the importance of geotechnical issues were the extensive liquefaction in the 1964 Niigata, Japan earthquake, the catastrophic landslides and ground failures in the great Alaskan earthquake of the same year, and the near-failure of two critical earthdams in the 1971 San Fernando earthquake. In June 1977, the National Science Foundation (NSF) and National Bureau of Standards (now National Institute of Standards and Technology) sponsored a workshop at the University of Texas on "Research Needs and Priorities for Geotechnical Earthquake Engineering Applications" (Ref. 1).

The University of Texas workshop resulted in recommendations for research in four major areas:

(1) Fundamentals
- analytical methods
- stress strain relation
- soil properties

(2) Improved Design Methods

(3) Evaluation of Design Procedures by:
- observations during and after earthquakes
- dynamic excitation of large structures
- studies on large shaking tables
- studies on large centrifuges

(4) Technology Transfer
- financial support for graduate research
- research-user interaction

Substantial research progress has been made in these areas since the 1977 workshop. However, the one area that has not received significant attention nor seen much progress has been the evaluation of design procedures by experimental means. There have been continuing post-earthquake observations, some small centrifuge studies, and a recent larger program to evaluate liquefaction on centrifuges, but there has been very little activity in the area of dynamic excitation of large structures and studies on large shaking tables.

There is no question that post-earthquake observations are critical to our understanding of the behavior of geotechnical materials and systems during earthquakes, but the evaluation of the observations is limited by the lack of experimental control, limited understanding of the pre-earthquake properties of the system, and limited instrumentation.

Our lack of full understanding of geotechnical behavior has been confirmed time and again in earthquakes since 1977. Some of the most important earthquakes are:

- 1979 Imperial Valley, California (Ref. 2)
- 1985 Mexico City, Mexico (Ref. 3)
- 1988 Armenia (Ref. 4)
- 1989 Loma Prieta, California (Ref. 5)
- 1990 Philippines (Ref. 6)
- 1991 Costa Rica (Ref. 7)

Substantial damage occurred in these earthquakes due to the behavior or failure of geotechnical systems, and each earthquake served to dispel long held misunderstandings and to reveal important phenomena. For example, the Imperial Valley earthquake demonstrated that horizontal accelerations on level ground above 0.5g were physically possible. Furthermore, vertical accelerations above 1g occurred in the epicentral region. This result cast in doubt the assumption of an upper limit to earthquake accelerations in performing analyses and writing building codes. This led to consideration of spectral acceleration in damped spectra and "effective" peak accelerations. The response of the Imperial County Services Building also gave strong indications of the importance of soil-structure interaction in earthquakes.

The Mexico earthquake of 1985 demonstrated, among other things, significant amplifications of peak motion, duration, and spectra due to local site conditions involving deep soft soil layers. Esteva (Ref. 8) notes that 5% damped spectra at some sites reached nearly 1g while the maximum design spectra was 0.06g. There were also substantial reductions in shear capacity of clay soils after many cycles of loading. This affected friction pile behavior.

The 1988 Armenia earthquake gave further evidence of the amplification of ground motion at sites with relatively deep soft sediments. There also were a great number of landslides, as well as concrete retaining wall and bridge abutment failures.

The 1989 Loma Prieta earthquake in California is probably the most relevant from a United States perspective. It was a major geotechnical earthquake which caused widespread damage due to soil- and rock-related failure or phenomena. The overall damage in the Loma Prieta earthquake is estimated at $7 to $10 billion. It is probably reasonable to estimate that at least one-half this amount was due to geotechnical related failures. The earthquake was moderately large in magnitude ($M_s$ = 7.) but the duration was relatively short. Still, there was substantial damage in San Francisco and the east bay area of Oakland and Alameda, over 50 miles from the epicenter. This damage was due to two major geotechnical factors: (1) ground motion amplification on deep fills and cohesive soils (bay muds), and (2) liquefaction in loose, sandy hydraulic fills. The damage in the Marina district of San Francisco was due to large amplitudes of shaking and/or large ground movement due to liquefaction. The east bay area experienced damaging liquefaction at the Oakland International Airport, which suffered cracking and settlement of the northern 9,000 ft of the main runway and taxiway, and at the Port of Oakland where some paved areas were cracked and settled. In addition, the large cranes at the Port were made inoperable due to misalignment from settlement and lateral spreading. (See accompanying photographs.)

Some local site conditions in the Bay Area caused amplification of peak acceleration estimated at up to three times that on nearby rock sites. There were no strong motion measurements in the vicinity of the Cypress Viaduct failure, but measurements at other locations suggest the free-field ground motions were at least a factor of 2 over what would have been predicted on firm ground. Current code provisions do not consider such a large amplification (Ref. 9).

Other examples of geotechnical failures in the Loma Prieta earthquake include a large number of landslides in the Santa Cruz mountains, settlement of highway bridge approach and abutment fills, and bridge failures due to liquefaction and subsequent settlement, loss of bearing capacity, and lateral spreading around the bridge foundation. Contrary to these failures, engineered fills and dams which explicitly considered geotechnical earthquake engineering factors performed well.

Geotechnical response was also important in the 1990 Philippine and the 1991 Costa Rica earthquakes. In the Philippines, there was significant damage related to liquefaction, landslides, and ground motion amplification. In Costa Rica, substantial liquefaction occurred in alluvial plains, in turn causing damage to roads, bridges, railways, ports, water systems, and plantations.

In summary, our experience with earthquakes over the past twelve or more years suggests that geotechnical factors have had a major effect on the amount, location, and types of damage. Geotechnical hazards associated with large earthquakes include:

- Site amplified ground motions
- Landslides and rockfalls
- Widespread liquefaction with resulting settlement and lateral spreading affecting highways, bridge abutments, and wharves, in particular
- Damage to levees and dams
- Soil strength reductions affecting foundation performance
- Soil-structure interaction

- Damaged utilities
- Retaining wall movements

Post-earthquake observations have given excellent documentation and insight into these hazards. Analyses of the behavior and failures will lead to improved prediction and design methods. Yet, there will be large remaining uncertainties due to lack of measurements at locations of interest and lack of data on pre-earthquake insitu conditions. The removal of these uncertainties will require direct experimental programs under carefully controlled conditions. This need for direct experiments led to the workshop reported here.

Testing, including large scale testing, for structures has been in progress for a long period of time. Structural testing is accomplished with shake tables, dynamic shakers, and quasi-dynamic testing. This testing, however, does not address soil-structure interaction which requires incorporation of the foundation material.

Geotechnical experimentation has not progressed as much. Geotechnical testing is more demanding because the material which composes the structure (soil and rock) is the same material through which the earthquake waves propagate. Hence, there is the need to test large samples. Furthermore, the geotechnical materials are more variable and have complex properties. There have been a few explosive simulation tests. Also, some facilities have been constructed in active earthquake regions. In both cases, soil-structure interaction was the objective. However, there is a need for testing to evaluate a wider range of geotechnical problems including site improvement. This workshop was intended to evaluate and clarify this need and establish an agenda for future experimental research to support geotechnical earthquake engineering.

### 1.1.1 Recent Related Workshops and Meetings

Major workshops conducted over the last fifteen years related to experimental requirements in support of earthquake engineering are given in Table 1. Every workshop emphasized the need for significant experimentation. Many placed high priority on large scale testing. The major workshop which covered experimental requirements for geotechnical earthquake engineering was that conducted in 1977 at the University of Texas. Although testing was only one of several topics at the workshop, the workshop recommendations emphasized prototype testing and the further development of a wide range of test methods. The workshop documented here, almost fourteen years later, focused on experimental requirements, examined progress since the 1977 workshop, and developed current research recommendations.

Table 1. Major Workshops on Experimental Requirements in Earthquake Engineering Research.

1. Workshop on Simulation of Earthquake Effects on Structures, National Academy of Engineering, 1974. Recommended study and development of explosive methods for earthquake simulation and recommended the establishment of a national test site for explosive simulation of earthquake ground motions.

2. Workshop on the Research Needs and Priorities for Geotechnical Earthquake Engineering, National Science Foundation and National Bureau of Standards, 1977. Recommended instrumentation of free-field and structure motion in earthquake environments and development of the use of explosives and mechanical shakers for testing prototypes or field models and for tests using centrifuges and shake tables.

3. Workshop on the Potential Utilization of the NASA/George C. Marshall Space Flight Center in Earthquake Engineering Research, National Science Foundation and the National Aeronautics and Space Administration, 1978. Recommended large-scale tests using static-cyclic testing towers, medium- or large-size shake tables, large centrifuges and high explosives, and instrumentation of existing structures in earthquake prone areas.

4. Workshop on the Earthquake Resistance of Highway Bridges, National Science Foundation, 1979. Recommended improved cooperation and communication between researchers and professionals, the development of means for verifying complex and sophisticated analysis methods, and the development of procedures to determine the seismic resistance and acceptable damage levels of existing bridges.

5. Workshop on Dynamic Excitation for Geotechnical Centrifuge Model Testing, National Science Foundation, 1979. Noted that accurate simulation would be difficult and expensive but that modeling systems with external vibration excitation (e.g., shakers) required little development.

6. Workshop on Experimental Research Needs for Improving Earthquake-Resistant Design of Buildings, National Science Foundation, 1984. Recommended strengthening experimental research capabilities in the U.S. by modernizing existing facilities, establishing four to six regional facilities with a variety of capabilities, and establishing a major national facility with a large (20m x 20m) shaking table. Recommended further research on high-explosive excitation methods for insitu testing and field tests for soil-structure interaction studies.

7. Workshop to Review and Identify the Earthquake Engineering Research Needs for Bridges and to Identify the Required Experimental Facilities, National Science Foundation, 1984. Recommended a national bridge laboratory to permit analytical and experimental studies of large scale models and full scale component. Recommended field experiments on full size bridges insitu, including investigation of explosive testing techniques.

8. Workshop on Designated Sites for Geotechnical Experimentation in the United States, National Science Foundation, 1988. Identified the existence, availability, and a high degree of interest in having access to documented field test sites.

9. Workshop on Dynamic Soil Property and Site Characterization, National Science Foundation and Electric Power Research Institute, 1989. Identified a wide range of requirements for improvement in laboratory and insitu testing, test sites, field monitoring, and analytical studies.

## 1.2 WORKSHOP OBJECTIVES

The overall objective of the workshop was to provide an evaluation of current experimental needs for geotechnical earthquake engineering and current capabilities to support those needs. Experiments on physical models and/or prototype scale geotechnical systems were the main issues. Laboratory testing of soils and rocks for determination of properties was not covered in the workshop. Some specific objectives were to:

- Determine current needs for experimental data in support of geotechnical earthquake engineering issues.
- Establish the level(s) of faithfulness with which test methods must represent earthquake ground motion fields to meet geotechnical testing needs.
- Document past experience and the current status of various testing techniques including: actual earthquakes and after-shocks, shake tables, explosive simulation, external shaker and pulser excitation, and dynamic tests on centrifuges.
- Evaluate the fidelity and suitability of each test method for meeting current needs including: cost-benefit factors for each method, integration of the methods into a hierarchy of testing for cost effective achievement of experimental objectives, and compatibility of the methods with university research approaches.

Workshop participants included researchers and practitioners from academia, government, and private industry. Their objective was to develop recommendations on overall experimental needs, requirements for enhancement/improvement/evaluation, and test programs which will lead to improved safety of geotechnical designs. A list of participants is given in Appendix A.

The workshop was organized by a committee comprised of Cornelius J. Higgins (Applied Research Associates, Inc.), Chairman, Clifford J. Astill (U.S. National Science Foundation), H.T. Tang (Electric Power Research Institute), T. Leslie Youd (Brigham Young University), Koon Meng Chua (The University of New Mexico), Ronald F. Scott (California Institute of Technology), Paul F. Hadala (U.S. Army Corps of Engineers), and Richard D. Woods (The University of Michigan).

## 1.3 WORKSHOP FORMAT

The format for the workshop included state-of-the-art presentations on geotechnical earthquake engineering issues, and experimental needs and methods; panel/writing sessions covering specific topics; and plenary discussions. Participants were invited to submit 2 - 3 pages of comments in advance of the workshop on any topic related to the workshop. These submittals were then given to the state-of-the-art speakers for consideration in their presentations.

The workshop was opened with a presentation on Observational Methods by Dr. Ralph B. Peck. Three state-of-the-art speakers then considered major geotechnical earthquake engineering issues organized by application. The topics and speakers were:

- Earthdams and Natural Slopes: Larry Von Thun
- Soil-Structure Interaction (covering shallow and deep foundations, retaining structures, and underground facilities): Stuart Werner
- Ground Motion (covering amplification, ground stability, and site improvement): Geoffrey Martin

State-of-the-art speakers focused on current methods, uncertainties in these areas, and experimental needs to reduce the uncertainties.

The workshop then turned to experimental methods with state-of-the-art presentations on:

- Tests of Actual Full Size Facilities by Actual Earthquakes and After-Shocks, and by Shakers and Pulsers: T. Leslie Youd
- Explosive Simulation Tests: Cornelius J. Higgins
- Dynamic Tests on Shake Tables and Centrifuges: Hon Yim Ko

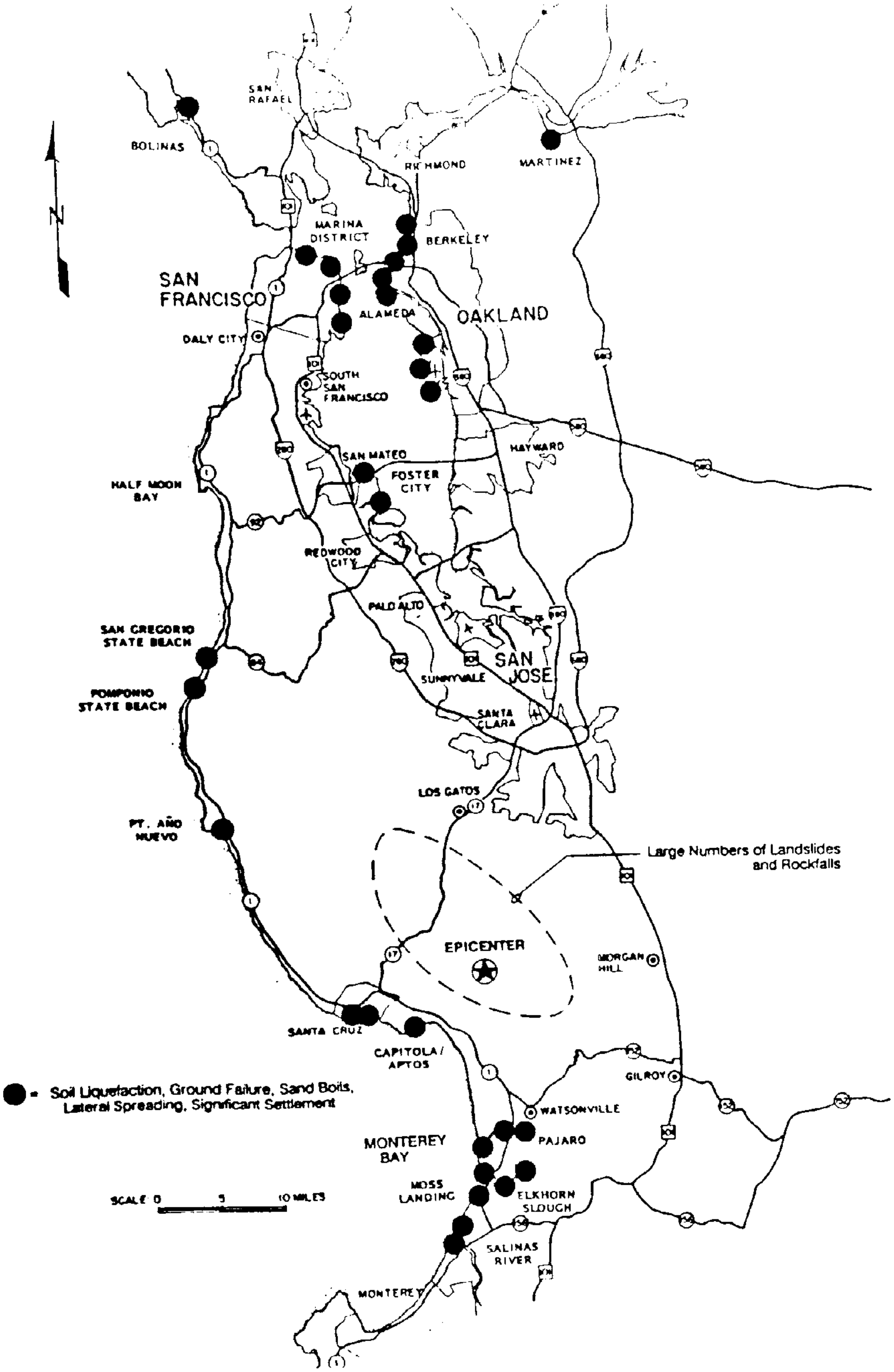
Panels, organized around the geotechnical topics, considered the needs in more detail, as well as the applicability, fidelity, and cost effectiveness of the various experimental methods. There were six panels as follows: (1) Earthdams, (2) Natural Slopes, (3) Foundations, (4) Retaining and Underground Structures, (5) Ground Motion Response, and (6) Ground Instability and Site Improvement.

These panels interacted during plenary sessions to permit exchange of information and critique of panel conclusions.

## 1.4 REPORT OUTLINE

The remainder of this report contains 4 chapters. Chapter 2 contains the state-of-the-art papers prepared for the workshop. Chapter 3, written by the Organizing Committee, attempts to synthesize and summarize the recommendations common to all of the workshop panels. The specific panel reports are presented in Chapter 4. They vary in format and detail. They are necessarily brief and sketchy due to the limited time available for their preparation during the workshop. Finally, the Organizing Committee members provide some summary and concluding remarks in Chapter 5.

Appendix B gives the agenda for the workshop. Appendix C gives the panel assignments.

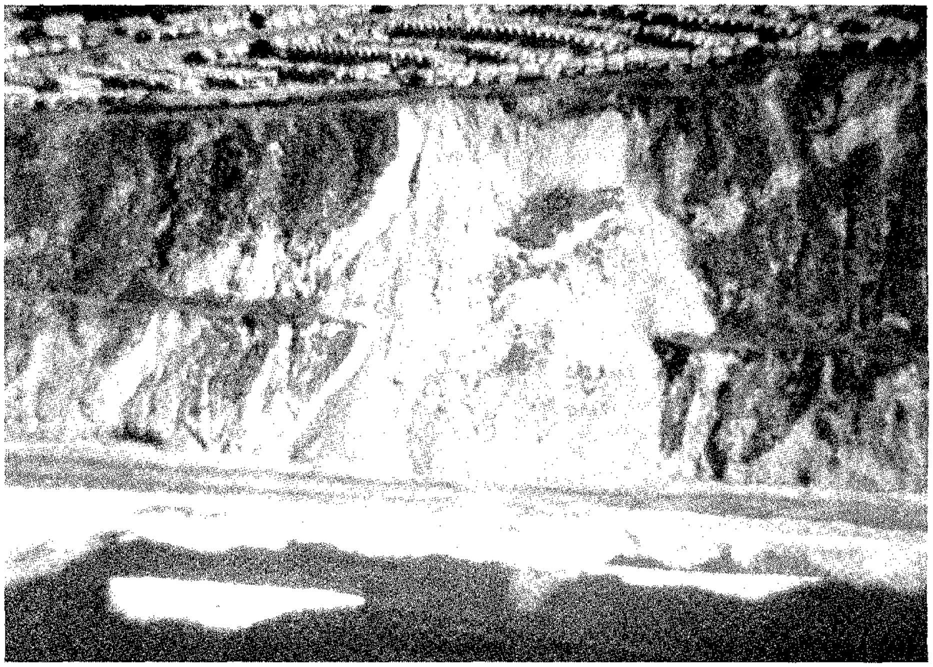



Map Showing Regional Extent of Significant Geotechnial Observations in Loma Prieta Earthquake (Courtesy R.B. Seed) (Ref. 5, fig. 4.1, pg. 82, reprinted by permission of Earthquake Engineering Research Institute).

# CHAPTER 2
# STATE-OF-THE-ART PAPERS

## 2.1 EARTHQUAKE ENGINEERING OF EARTH DAMS AND SLOPES - EXPERIMENTAL NEEDS IN 1991

Larry Von Thun
U.S. Bureau of Reclamation
Denver, Colorado



Coastal Bluff Failure at Daly City in Loma Prieta Earthquake (Photograph by University of California, Berkeley, Geotechnical Engineering Group) (Ref. 5, fig. 4.44, pg. 120, reprinted by permission of Earthquake Engineering Research Institute).

Preceding page blank

# Earthquake Engineering of Earth Dams and Slopes - Experimental Needs in 1991

by
Larry Von Thun
U.S. Bureau of Reclamation, Denver Colorado

## I. Introduction

The purpose of this paper is to serve as an aid to NSF workshop participants seeking to identify the most important, current, experimental needs in the field of geotechnical earthquake engineering of earth dams and slopes. To accomplish that objective in a systematic way, the following approach will be taken:

A. Summarize the elements of geotechnical earthquake engineering analysis of earth dams and slopes and the current procedures used in these analyses.

B. Present a framework for evaluating the relative importance and merits of needs being considered.

C. Identify difficulties and uncertainties presently being experienced in the solution of geotechnical earthquake engineering problems. Also, concurrently review the status of design and analysis capabilities and applications relative to the identified need. This identification will provide an initial list of experimental needs.

## II. Earthquake Engineering Analysis and Design of Earth Dams and Slopes

Evaluation of static slope stability of earth dams and slopes consists of four basic elements:

1. Definition of potential failure modes and associated failure geometry

2. Estimation of the pore pressure acting on the potential failure planes

3. Estimation of the shear strength of materials along the potential failure slide planes

4. Selection of an analysis method, execution of the analysis, and evaluation of the results

Two elements are added for the dynamic problem:

5. Estimation of the earthquake loading and the attendant ground motions to be applied

6. Determination of any dynamic response of the slope or earth dam to the earthquake motions

It is noted that the workshop separately addresses the question of ground motions. However, in the interest of completeness and the desire to focus on earth dams and slopes, consideration will be given to needs related to ground motions.

Additional dynamic aspects regarding pore pressure, shear strength and specific dynamic analysis techniques must also be considered.

| | |
|---|---|
| Pore Pressure | - build up of pore pressure within materials |
| | - pore pressure response within discontinuities as they respond (open and close) to shaking |
| Shear Strength | - variation, if any, due to the nature of dynamic loading |
| | - post earthquake resistance |
| Analysis Methods | Need to account for: |
| | - deformation during shaking |
| | - change in strength properties during loading |
| | - change in geometry |
| | - changes in dynamic response during the earthquake |

The above represents a summary of the basic elements and special factors that need to be considered when examining the experimental needs of the geotechnical engineer solving design and analysis earthquake engineering problems of earth dams or slopes. In addition to earth dam slopes, two other categories of slopes need to be considered: (1) Natural slopes either unfailed or failed (landslide slopes) which are being <u>evaluated</u> for stability under earthquake loading and (2) Cut slopes, either earth or rock, which are being <u>designed</u> to withstand earthquake loading.

The problem of dynamic analysis of earth dams is illustrated in Table 1. The table also provides a summary of the most common dynamic analysis procedures used to address each part of the problem.

Analytical procedures that are most commonly used in practice can be categorized as:

Table 1. - Dynamic analysis of embankment dams.

Given: Earthquake magnitude and distance; site ground motion parameters (at bedrock or ground surface)

Find: 1. Potential for liquefaction of dam or foundation deposits (including amount of pore pressure rise if liquefaction does not occur).
2. Permanent displacement of dam as a result of earthquake shaking (including effects of No. 1 where applicable).

**Solution:**

| Two basic types of analyses | Two basic steps in each analyses | Two basic alternative approaches |
|---|---|---|
| Liquefaction analyses | Dynamic response computation | Empirical |
| Deformation analyses | Pore pressure and deformation response computations | Analytical |

| In preparation for a liquefaction analysis | Approach type | In preparation for a deformation analyses |
|---|---|---|
| **DYNAMIC RESPONSE COMPUTATIONS** | | |
| Seismic potential | Empirical | Performance of embankment dams |
| $\tau_{avg}/\sigma_v' \approx 0.65\ (A_{max}/g)\ R_d \cdot (\sigma_v/\sigma_v')$ | Simplified | Makadisi and Seed (1977) |
| SHAKE, SHAKEM | 1-D-analytical | SHAKE, SHAKEM |
| FLUSH, TLUSH | 2-D-analytical | FLUSH, TLUSH |
| DESRA, extended FEM | Analytical coupled | DESRA, extended FEM |
| 3-D FEM, MASH | Other analytical | 3-D FEM, MASH |

**PORE PRESSURE AND DEFORMATION RESPONSE COMPUTATIONS**

| Liquefaction analysis | Approach type | Deformation analyses |
|---|---|---|
| Seismic potential | Empirical | Performance of embankment dams |
| Seed (1981-86) | SPT simplified empirical | Makdisi, Seed, and DeAlba (1977) |
| Post liquefaction stability (SPT correlations for strength) | Analytical math model | Newmark-site specific - USBR-DYNDSP |
| **(a)** Dilative - contractive | Field and lab testing for in situ void ratio vs. steady-state line | |
| **(b)** Stability under steady state | Field and lab testing for steady-state strength | |
| **(c)** Triggering analysis | Stress-strain properties under undrained cyclic loading | |
| Seed simplified, 1971 | Cyclic triaxial testing | Seed-Idriss (strain potential) |
| Expanded FEM | Coupled procedures | Expanded FEM |

- dynamic response analyses employing a one dimensional procedure (e.g. SHAKE)[1]

- deformation analyses employing the Newmark procedure[2]

- liquefaction analyses employing the empirical SPT procedure[3]

- liquefaction analyses employing steady state of deformation concepts[4]

Although there are many variations and refinements to these basic methods and approaches, they represent the fundamental status of existing practice. Verification of each of these methods is anchored to case history data. A more elegant and comprehensive solution has been developed by Finn, TARA-3[5], which combines all of the analytical procedures into a single solution. Dynamic response, liquefaction (pore pressure development) and deformation are combined into a single comprehensive analysis. This procedure has also been compared to case histories, however, its use has, to date, been limited and thus general familiarity, acceptance, and application by the profession has not been achieved.

Byrne[6] has recently developed a procedure for estimation of post liquefaction deformation. Application of this procedure has also been limited and is, therefore, not yet in general use. Certainly estimation of post liquefaction deformation is a great uncertainty in the profession.

## III. Framework for Needs Prioritization

The exercise being undertaken at the NSF workshop should identify many desired improvements in information, analysis procedures, data bases, and the like. Each, when introduced in context, has merit but it seems appropriate to consider in advance a means of determining the relative importance of the identified needs. The factors that need to be considered in formulating prioritization criteria are given below with descriptions of lower to higher priority ranking statements given for each.

| FACTOR | PRIORITY RANKING STATEMENT | |
|---|---|---|
| | LOW | HIGH |
| How great is the deficiency? | Current procedure works well but could use improvement | No good procedure is available |
| How common is the application? | Problem rarely encountered | Problem routinely encountered |
| How important is the application? | Applications have minor effects on practical application | Application can affect major decisions on projects (feasibility, cost, remediation) |
| How likely is success of meeting needs. | Remote chance - no particular lead on how to approach solution | Very likely - data or information is apparently available or new procedure is apparent |
| | | |

## IV. Failure Mode and Model Uncertainties

### A. Earth Dams

In formulating the analysis of an earth dam under dynamic loading three basic types of failure are considered:

1. Generalized deformation including spreading, raveling, settlement, slumping and sliding as primary effects, and transverse or longitudinal cracking as secondary effects. There is considerable uncertainty and attendant variation in the methods used to analyze these deformation responses. The Newmark procedure, settlement analysis formulas, and the finite element method are all used but there is little knowledge or guidance as to which method should be used under which circumstances. Further, there is considerable uncertainty in employing and interpreting the results of each method. In the Newmark procedure, a decision on the "critical" surface for dynamic analysis needs to be made. Should this surface represent the lowest factor of safety under dynamic load regardless of location or impact or should it represent a surface whose failure can potentially result in loss of the reservoir.

Finite element analyses have a good deal of uncertainty in the selection of material properties and constitutive model characteristics as well as in the interpretation of the actual response based on stress and deformation patterns.

One of the greatest unknowns and deficiencies in terms of failure potential determination and available analysis models relates to longitudinal and transverse cracking, the most commonly observed effects of dynamic loading of earth dams. This problem is currently only qualitatively evaluated on the basis of observing discontinuities in geometry, sharp variations in stress, and total amount of predicted settlement. A better empirical guideline or procedure and a more rigorous method for analyzing the possibility and extent of this occurrence is needed.

2. Liquefaction is a special case of generalized deformation in that in its most extensive expression, flow failure, it can be considered an independent failure mode while its limited occurrence can simply be a contributor to settlement, slumping and sliding. Considerable uncertainty resides in the liquefaction analysis data base in that most of the empirical data base relates to observations of liquefaction phenomenon (e.g. sand boils), rather than flow failure. Formulation of failure modes and models which attempt to account for portions of a slip surface undergoing liquefaction or for a three dimensional effect due to narrow failure zones are often regarded as experimental and non-definitive with respect to a final decision on a course of action.

The basic model or framework of understanding of the mechanism and effects of a soil undergoing liquefaction is now considered to be understood in terms of a soil's stress-strain diagram. However, there is considerable uncertainty in how to account for the relatively larger strain that must occur for less brittle, more plastic soils. Laboratory tests can readily illustrate the strain dependent effect but remain problematic with respect to being reliably able to predict whether or not a specific soil in situ under a specific overburden stress will liquefy under a given earthquake ground motion.

3. Ground rupture beneath an earth dam is the third potential dynamic failure mode to consider. The amount and possible locations of fault rupture beneath an earth dam are extremely difficult to predict. The orientation of the faulting and the potential

secondary effects are critical to failure mode definition. Considerable uncertainty currently exists when such a problem is faced. New designs can be formulated to accommodate a variety of movements and a variety of magnitudes, however, some existing dams must be evaluated for safety given the possibility of fault displacement.

B. Natural Slopes

Generalized deformation failure modes for natural slopes, both rock masses and earth masses, are postulated similarly to those for earth dams. However, the key question in these formulations, which is a source of great uncertainty, is whether or not the slope behavior will be slip (along a preexisting slide plane) or stick-slip (requiring a rupture prior to sliding). The difference between these two behaviors is critical to the hazard determination of the slope as well as its analysis formulation. Slopes that could rupture and accelerate during sliding can pose a reservoir wave generation hazard, and can in some cases, pose a direct or indirect threat to life.

Rock falls or rock slides constitute a second natural slope failure mode/modeling uncertainty. The current state of interlocking of a rock mass is extremely difficult to estimate, as is the effective ground motion, which might be transmitted, in shear across a potential slide plane. Extremely rapid pulses of high frequency earthquake motion may not last long enough to produce any deformation of the rock mass.

V. Earthquake Input Motion and Dynamic Response Uncertainties

A. Earth Dams

An accelerogram is required for a Newmark type deformation analysis (unless the simplified Seed-Makdisi[7] approach is used). Currently, there is considerable uncertainty in the selected accelerogram regardless of the procedure used to develop it. The uncertainties include:

1. Selection of representative earthquake motion.

2. Source to site attenuation estimate.

3. Estimation of transmission of motion from bedrock through foundation materials (if applicable) and through the dam.

4. Estimation of representative motion (frequency characteristics) along the slip surface.

5. Scaling of representative motion to represent the average level of shaking along the slip surface.

Comparative studies between 1-D and 2-D and between uncoupled and early coupled versions of earthquake transmission programs (SHAKE, LUSH, FLUSH, vs DESRA) indicate relative insensitivity to the method used under most circumstances. Variations in subsurface conditions, however, can result in great sensitivity regarding results. Guidance is needed in deciding how to appropriately account for subsurface features that may indicate a base isolation effect. In addition, guidance is needed in determining ground motions for sites with moderate depth (150-300 ft.) of alluvial cover and great depth of alluvial cover (> 300 ft).

The input for typical empirical liquefaction evaluations only requires the earthquake magnitude and the earthquake induced shear stress. This shear stress for the empirical data base formulation was derived on the basis of a peak ground surface acceleration ($A_{max}$). Currently, a much better estimate of the induced shear stress can be obtained using subsurface information and a dynamic response analysis program (e.g. SHAKE). However, since the data base was formulated on a different basis, it is uncertain whether or not the improved earthquake input actually improves the analysis. Thus, a need exists to either adjust the data base or determine under what circumstances subsurface shear wave velocity information along with SHAKE should be used.

### B. Slopes

Earthquake input for slope analysis is obtained similarly as for earth dams. However, as discussed under failure mode uncertainties, a need exists for determining effective peak motions for rock slope stability problems. This problem becomes even more uncertain for sharply varying rock geometry. Such geometry has been seen to produce extremes of earthquake motion (e.g. Pacoima Dam abutment).

## VI. Uncertainties in Pore Pressure Estimation and Liquefaction Analyses

### A. Earth Dams

Dynamic stability and deformation analyses are made assuming that pore pressures in the embankment materials either build up due to contractive soil behavior (reducing shearing resistance), are dissipated through drainage, or are not built up because the compacted materials are dilative. Deformation studies are very sensitive to these assumptions. There is a need to provide more definitive information on what compactive effort or relative density can be assumed to indicate that materials will behave in a dilative manner.

The preponderance of liquefaction related failures are reported to have occurred in fine sands and non-plastic sandy silts. Uncertainty exists about the flow failure potential of coarser materials such as gravel. A need exists to determine under what conditions that materials other than sands and non-plastic silts can liquefy.

Shear wave velocity measurements serve as a backup procedure to SPT testing when unreliable or questionable SPT results are obtained. A more definitive relationship between liquefaction potential and shear wave velocity is required.

## VII. Uncertainties Related to Shearing Resistance

Despite the recent attempts (since 1985) to relate SPT blow count to residual or steady state strength, (post liquefaction) by Seed and more recently by Harder, considerable uncertainty remains. Likewise, use of in situ sampling and laboratory test procedures suggested by Castro and Polous result in considerable uncertainty in estimation of steady state strengths. Variations in steady state or residual strength are seen to cause sharp variations in post-earthquake stability analyses indicating that they are critical to determining whether or not flow failure will or will not occur. The need exists for improvements or refinements in determining the strength/behavior of liquefied deposits.

Hadala suggests, in his notes for this conference, that there is a need for experiments to verify/quantify non-level ground liquefaction resistance. I wholeheartedly support that suggestion.

## VIII. Model/Method Verification

Empirical liquefaction analyses using SPT are, in effect, only partially empirical. Significant adjustments and correction factors are applied on the basis of cyclic triaxial test results. New methods of investigation (CPT, Becker Hammer, Shear Wave Velocity) are used but all are tied to correlation with SPT. P.K. Robertson, in notes prepared for this workshop, describes the need for full scale test facilities. Such facilities may offer a means for accommodating advances in the field of liquefaction evaluation in the areas of dynamic response, constitutive modeling, field testing, in situ sampling, and analysis methods. Large scale test facilities may also offer the opportunity to quantify the adverse effects of a clay overlying a sand or the potential positive effects of gravel deposits surrounding a narrow sand seam.

Refinements to liquefaction analysis procedures, such as those suggested by C.K. Shen who notes that higher pore pressures are developed under multi-directional earthquake loading, are needed. Such a refinement needs to be quantitatively tied to the basic empirical data base of sites which did or did not result in liquefaction, or a new procedure needs to be developed that also can be directly tied to case histories. Perhaps a new data base comprised only of flow failures would be an appropriate starting point.

## IX. Evaluation Criteria

Evaluating the dynamic stability of an earth dam or slope is primarily judgmental. Even when quantitative calculations are produced (intervals indicating liquefaction or feet of horizontal deformation during shaking), a judgment is rendered as to whether or not the structure or slope will render a satisfactory performance. The only criteria somewhat commonly in use is a factor of safety of about 1.25 against liquefaction failure under maximum earthquake loading. It is possible to use this criteria if tests and information relate to a specific layer. If, however, a wide, variable interval is being examined, the evaluation becomes considerably more judgmental.

Deformation criteria generally first relate to the question of freeboard and then to secondary effects (cracking, slumping, etc.). More definitive criteria in this regard would be very helpful.

Unless large scale testing that provides a more rigorous basis for establishing criteria is developed, then the current practice of leaving the overall evaluation to the analyst for reaching a conclusion on dynamic performance of earth dams and slopes is appropriate.

## X. Summary Matrix of Problems, Categories, and Uncertainties.

Table 2 presents a matrix consisting of the uncertainties related to five problem categories and six solution stages in earthquake engineering for earth dams and slopes. The problem categories are:

1. Liquefaction flow failure of dams

2. Dynamic deformation (but no flow slide) of dams

3. Dynamic stability of slopes

4. Dynamic stability of landslides

5. Ground rupture

The Solution Stages are:

1. Investigation Failure Mode/Model

2. Earthquake Loading/Response

3. Parameter Modeling

4. Analysis Procedure

5. Design/Treatment

6. Criteria

## XI. Conclusions

Dynamic analysis of earth dams and slopes is primarily accomplished in the profession in one of three ways:

1. Deformation analysis using Newmark's procedure or a derivative thereof.

2. Liquefaction analysis using Seed's empirical SPT procedure or a derivative. Variations using CPT, Becker Hammer testing and in situ sampling and testing are also used.

3. A coupled finite element approach linking dynamic response, pore pressure development and deformation. (e.g. TARA-3)

For the most part these procedures, especially the Seed liquefaction analysis, are linked to case histories for verification. Although the empirical procedure is well established in the profession, there is a general unrest among analysts who have proposed or wish to develop improved liquefaction evaluation procedures, but are frustrated because definitive decisions almost always lead back to the empirical SPT data base. For changes in accepted procedures to take place, a new verification base must be developed. Large scale or full scale test facilities or development of a flow failure data base linked to in situ field testing methods would seem, at present, to offer the most promise for advances in the discipline.

TABLE 2 - UNCERTAINTIES IN EARTHQUAKE ENGINEERING FOR EARTH DAMS AND NATURAL SLOPES

| | SOLUTION STAGE | | | | | |
|---|---|---|---|---|---|---|
| PROBLEM CATEGORY | INVESTIGATION FAILURE MODE/MODEL | EARTHQUAKE LOADING/ RESPONSE | PARAMETER MODELING | ANALYSIS PROCEDURE | DESIGN/ TREATMENT | CRITERIA |
| Liquefaction flow failure | Classifying an alluvial deposit with "mixed soils and blow counts" as potentially liquefiable<br><br>When does a gravel deposit qualify as liquefiable?<br><br>Lack of case histories of failure of compacted fill dams on liquefied foundation<br><br>How much geology is required to verify problem existence or lack of it<br><br>Incorporation of geologic judgement | Attenuation relations Mean vs 84 percentile<br><br>Seed simplified vs dynamic response anal. to get cyclic shear stress ratio<br><br>Motion for site with no shear wave data- soils 100' deep or greater<br><br>Waves other than vertically propagating shear waves<br><br>Effect of deep, weak layer | Residual shear strength estimates<br><br>Loss of resistance due to shaking in compacted materials<br><br>Pore pressure buildup<br><br>Avg. vs. 35 percentile<br><br>Effects of gravel on SPT results<br><br>Extreme flows witnessed vs lab testing showing limited displacement<br><br>Effect of loading direction on steady state strength | Correction factors (SPT proc)<br><br>Effects of gradation<br><br>Triggering analysis- ever valid<br><br>Cross hole data- limitation<br><br>Three dimensional effect<br><br>Deformation calculation under flow failure condition<br><br>Incorpora- tion of lack of continuity | Stiff vs flexible systems<br><br>Harm from some treatments? such as:<br><br>Dyn Compac- tion<br>Compaction grouting<br><br>Filter placement to avoid liquefac- tion<br><br>Effects of aging on treated materials<br><br>Effective- ness/applic ability of various treatment methods | -F.S. against lique- faction<br><br>F.S. for post earth- quake condi- tion<br><br>"Accept- able" flow slide deforma- tion<br><br>Treatment verifica- tion - when required and when not |

TABLE 2 - UNCERTAINTIES IN EARTHQUAKE ENGINEERING FOR EARTH DAMS AND NATURAL SLOPES

| | SOLUTION STAGE | | | | | |
|---|---|---|---|---|---|---|
| PROBLEM CATEGORY | INVESTIGATION FAILURE MODE/MODEL | EARTHQUAKE LOADING/ RESPONSE | PARAMETER MODELING | ANALYSIS PROCEDURE | DESIGN/ TREATMENT | CRITERIA |
| Liquefaction flow failure | Layer thickness effects | | | Excess pore pressure - transmission to other areas<br><br>Validity of cyclic triaxial based analysis | Permeability testing/ design | Permeability to preclude liquefaction problem |

TABLE 2 - UNCERTAINTIES IN EARTHQUAKE ENGINEERING FOR EARTH DAMS AND NATURAL SLOPES

| | SOLUTION STAGE | | | | | |
|---|---|---|---|---|---|---|
| PROBLEM CATEGORY | INVESTIGATION FAILURE MODE/MODEL | EARTHQUAKE LOADING/ RESPONSE | PARAMETER MODELING | ANALYSIS PROCEDURE | DESIGN/ TREATMENT | CRITERIA |
| Dynamic deformation<br><br>Some portion of deposit may be liquefiable but flow slide is not possible-i.e., post liquefaction F.S. > 1.0 | Discrete failure plane(s)<br><br>Which surfaces should be analysed<br><br>Generalized settlement<br><br>When is three dimensional model required | Attenuation relation to use<br><br>Vert & Horiz<br><br>What motion for discrete surface | Strength of material (drained or undrained strength if dilative)<br><br>When does liquefaction take place<br><br>Pore pressure generation<br><br>Strain dependent properties, site specific determination required, large strain consideration | Is Newmark procedure adequate<br><br>Seed-Makdisi Simplified Method<br><br>Lateral Spreading Analysis Methodology | Provide adequate freeboard<br><br>Defensive design measures | What deforma-tion consti-tutes a problem for secondary effects<br><br>Accept-ability of deforma-tion criteria |

TABLE 2 - UNCERTAINTIES IN EARTHQUAKE ENGINEERING FOR EARTH DAMS AND NATURAL SLOPES

| | SOLUTION STAGE | | | | | |
|---|---|---|---|---|---|---|
| PROBLEM CATEGORY | INVESTIGATION FAILURE MODE/MODEL | EARTHQUAKE LOADING/ RESPONSE | PARAMETER MODELING | ANALYSIS PROCEDURE | DESIGN/ TREATMENT | CRITERIA |
| Slopes- natural or cut slopes - earth or rock | Continuity of joints<br><br>Interaction of discontinuity and treatment U/S or D/S of block<br><br>Identification of failure planes<br><br>Toppling vs sliding mode identification<br><br>Identification of limits and boundaries | Effective peak acceleration<br><br>Dynamic response above shear surface<br><br>Vertical and horizontal (both needed)<br><br>Ground motion along shear surface | Some slides not able to be explained<br><br>Future pore pressure conditions<br><br>Effects of drainage | Newmark Method -Does it account for stiffness o.k.<br><br>Finite element -Yield criterion<br><br>Toppling procedures<br><br>Proper accounting for strain compatibility | Longevity of rigid treatment Anchors/ tendons<br><br>Plugging of drains | Accept- ability of deformation criteria<br><br>F.S. criteria not realistic unless used with effective peak accelera- tion |

TABLE 2 - UNCERTAINTIES IN EARTHQUAKE ENGINEERING FOR EARTH DAMS AND NATURAL SLOPES

| | SOLUTION STAGE | | | | | |
|---|---|---|---|---|---|---|
| PROBLEM CATEGORY | INVESTIGATION FAILURE MODE/MODEL | EARTHQUAKE LOADING/ RESPONSE | PARAMETER MODELING | ANALYSIS PROCEDURE | DESIGN/ TREATMENT | CRITERIA |
| Landslides - existing - active or dormant | Base plane location<br><br>Identification of any resistant bodies along failure plane<br><br>Limits for analysis<br><br>Post earthquake landslide potential | Variation along base<br><br>Transmission across base | Current strength of dormant slide<br><br>Current pore pressure<br><br>Pore pressure in past and future<br><br>Variation with movement and speed of movement | O.K. for deformation analysis Newmark<br><br>Highly sensitive to speed of failure (empirical)<br><br>Highly sensitive to shore absorption (LS wave)<br><br>Run out and crest setback distances | Calculation of beneficial effects of drains | Displacement<br><br>Factor of safety<br><br>Velocity |
| Ground rupture | Direction of faulting | Amount of movement | | | Response of crack stopper | Probability of occurrence |

## References


[1] Schnabel, B., et al., "Shake, A Computer Program for the Earthquake Response Analysis of Horizontally Layered Sites," Report No. 72-12, Earthquake Engineering Research Center, University of California, Berkeley, 1972.

[2] Newmark, N. M., "Effects of Earthquakes on Dams and Embankments," Geotechnique, vol. 15, No. 2, pp. 139-160, 1965.

[3] Seed, H. B., and I. M. Idriss, "Evaluation of Liquefaction Potential of Sand deposits Based on Observations of Performance in Previous Earthquakes," Proceedings of ASCE National Convention, St. Louis, Missouri, October 1981.

[4] Poulos, S. J., G. Castro, and J. W. France, "Liquefaction Evaluation Procedure," Journal of Geotechnical Engineering, ASCE 111(b), pp. 772-791, 1985.

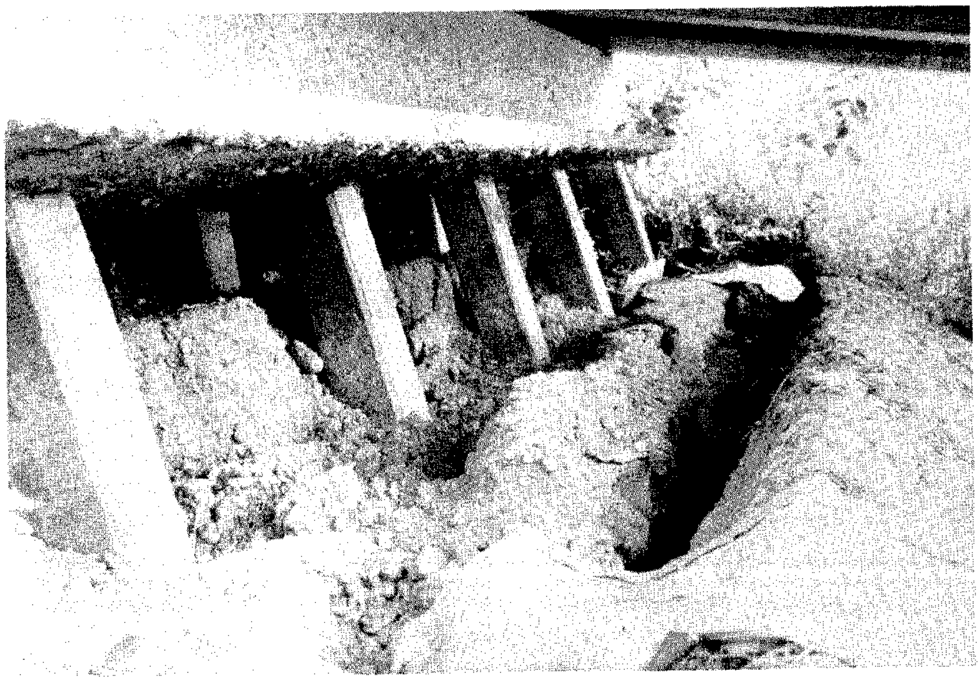
[5] Finn, W. D. Liam, M. Yogendrakumar, N. Yoshida, and H. Yoshida. (1986.) TARA-3: A Program for Nonlinear Static and Dynamic Effective Stress Analysis, Soil Dynamics Group, University of British Columbia, Vancouver, B. C., Canada.

[6] Byrne, Peter M. - A Model for Predicting Liquefaction Induced Displacement - Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, St. Louis, Missouri, pp. 1027-1035, Gnosch, 1991.

[7] Seed, H. B., and F. I. Makdisi, "A Simplified Procedure for Estimating Earthquake-Induced Deformation in Dams and Embankments," Report No. UCB/EERC-77-19, College of Engineering, University of California, Berkeley, 1977.

## 2.2 SOIL-STRUCTURE INTERACTION: THE STATE-OF-PRACTICE AND RECOMMENDED RESEARCH NEEDS

Stuart D. Werner
Dames & Moore
Oakland, California



Rio Buffalo Bridge: Abutment Rotation and Foundation Material Slumping in 1991 Costa Rica Earthquake (Ref. 7, fig. 6-12, pg. 68, reprinted by permission of Earthquake Engineering Research Institute).

# SOIL-STRUCTURE INTERACTION: THE STATE-OF-PRACTICE AND RECOMMENDED RESEARCH NEEDS

by

**Stuart D. Werner**
**Dames & Moore**
**Oakland, California**

## 1.0 INTRODUCTION

Within the general area of geotechnical earthquake engineering being addressed in this workshop, one of the more complex processes is that of dynamic soil-structure interaction (SSI). This process is initiated when incident seismic waves propagating away from the causative fault and through the geologic media encounter a structure and foundation whose inertial and stiffness characteristics differ substantially from those of the surrounding soils. As these incident waves strike the foundation, they are reflected and refracted. The resulting motions of the foundation generate inertia forces and motions throughout the overlying structure, which further alter the motions of the foundation and the surrounding soil.

This dynamic SSI process has several effects. First, it alters the motions, stresses, and deformations in the soil relative to their free field state. Second, a structure founded on soil will respond as a softer dynamic system with a longer natural period than that of the same structure founded on rock. Finally, the dissipation of part of the structure's vibrational energy by hysteretic action of the soil and by the radiation of waves away from the structure will increase the structure's effective damping. The potential significance of each effect, and whether the combined effects actually increase or decrease the seismic response of the structure, will depend on many factors related to: (a) the amplitudes, wave lengths, directions of approach, and relative phasing of the incident seismic waves; (b) the stratigraphy and material properties of the site soil materials; and (c) the configuration, stiffness, mass, and damping characteristics of the foundation and the overlying structure.

The objective of this paper is to summarize the current state of the practice for evaluating these complex SSI effects, and to use this summary as a framework for identifying SSI research needs in accordance with the overall goals of this workshop. To accomplish this objective, the remainder of this paper is organized into two main sections. The first of these sections (Section 2.0) summarizes how analysis procedures, experimental methods, and strong motion records are currently used to evaluate SSI effects, and how SSI is considered in current seismic design standards for conventional buildings and bridges. The final section of the paper (Section 3.0) contains recommended research needs that focus on (a) improving the current understanding of dynamic SSI effects, and when they may have an important effect on the seismic response of structures; (b) developing improved practical methods for incorporating SSI into the seismic design process; and (c) enhancing the ability of current methods of analysis to estimate SSI effects on the seismic performance of critical facilities. It is noted that all of these discussions and recommendations address SSI effects for a stable soil material only, for which the potential for significant pore water pressure effects or other possible modes of soil failure or instability can be considered to be negligible. This facilitates our focusing on the significant number of SSI issues to be addressed even for stable soils, and anticipates that potential soil instability effects will be covered at length in other papers presented at this workshop.

## 2.0 CURRENT STATE OF THE PRACTICE

### 2.1 Analysis Procedures

#### 2.1.1 Foundation-Soil Impedance Concepts

An initial milestone in the development of dynamic SSI analysis procedures was the formulation of analytical solutions for the harmonic response of a rigid circular and massless disk bonded to the surface of an elastic half space and subjected to a harmonic force (vertical or horizontal) or moment (rocking or torsional). Such theories, which were first developed by Lamb (1904), Reissner (1936), and Bycroft (1957) among others, have led to the determination of complex, frequency-dependent impedance functions that relate the applied harmonic forces and moments to the computed foundation displacements and rotations.

Since these initial development efforts, a multitude of new and significant analytical and numerical procedures have been developed to compute foundation-soil impedances: (a) for sites with uniform conditions or horizontally layered stratigraphy, and linear elastic, linear viscoelastic or equivalent linear material properties[1]; and (b) for surface or embedded foundations of arbitrary shape that may be rigid or deformable, and are bonded to the adjacent soil medium. The various procedures for obtaining these impedance functions include analytical solutions based on integral transform techniques, semi-analytical or boundary element methods, finite element or finite difference methods with "wave transmitting" boundary conditions along the edges of the soil grid, and hybrid techniques which combine finite element and analytical procedures. Reviews of many of these significant procedures are provided in several references including Werner (1976), Lysmer (1978), Luco (1982), Wolf (1985), Novak (1987), Roesset (1989), and Gazetas (1991a and b). Representative examples of these procedures are described in the subsections that follow.

#### 2.1.2 Substructure and Direct Analysis Procedures

The two principal methods of dynamic SSI analysis are substructure and direct methods. In substructure methods, the foundation-soil impedances (expressed as an impedance matrix) and a finite element model of the structure are developed separately and are then coupled along their common interface. Foundation input motions that incorporate scattering effects[2] are also computed separately and are applied to the coupled model. In current practice, substructure methods are most typically applied with linear viscoelastic or equivalent linear soil constitutive models; in such applications the dynamic analysis is typically carried out in the frequency domain (through the use of Fast Fourier Transform procedures) in order to incorporate the frequency-dependence of the impedance matrix. Substructure methods have occasionally been applied to fully nonlinear soil-structure

---

[1]Equivalent linear soil material models are intended to approximate nonlinear material behavior by using iterative procedures to adjust each soil element's secant shear modulus and damping after successive applications of the seismic input excitations, until these soil parameters are consistent with an effective strain level induced within the element by the seismic excitations.

[2]The practice for computing input motions for SSI analysis typically involves a process wherein the soil and assumed massless foundation are subjected to the free field motions. The resulting motion of the massless foundation differs from the free field motions because of scattering (i.e. reflection and refraction) of the incident waves that comprise the free field motions as these waves strike the foundation. The computed foundation motions that include these scattering effects are used as input to the soil-foundation-structure model (including mass) and the dynamic response of the model is computed.

models, for which the dynamic analysis is normally carried out in the time domain and the coupling between the substructures is implemented during each time step.

The direct method of dynamic SSI analysis involves construction of a finite element or finite difference model of the complete soil-structure system in a single step, rather than as a multi-step process. Free field input motions are then applied along the base and/or the sides of the soil portion of the model, in order to compute the soil-structure system response. Direct methods of analysis may be readily applied to linear, equivalent linear, or nonlinear soil-structure system models. Truly nonlinear SSI analyses including complete interaction effects are most commonly carried out using direct methods.

### 2.1.3 Computer Programs

Many examples of the current technology for substructure and direct methods of dynamic analysis are available as computer programs. To illustrate this technology, five of these programs are summarized in Table 1. These five programs encompass a range of methodologies, are established and well documented in the technical literature, and have typically been used to provide analytical correlations in major SSI test programs. Supplementary discussion of these programs is provided below.

FLUSH The FLUSH Code (Lysmer et al., 1975) uses a direct method of analysis of a two-dimensional finite element soil-structure system representation with an equivalent linear model of horizontally layered soils on a rigid base, and seismic input motions from vertically propagating shear waves. It approximates out-of-plane radiation damping effects through the use of in-plane dashpots attached to each soil node point. For many years, FLUSH has been among the most widely used SSI analyses procedures in engineering practice. An advanced version of FLUSH (named SUPERFLUSH) has been developed that has several new features, including accommodation of simultaneous horizontal and vertical excitations, traveling wave effects, and non-horizontal soil layers and topography (Udaka, et al., 1981).

CLASSI The CLASSI Program (Luco and Wong, 1982 and 1987; Luco et al., 1988a) employs a substructure approach in conjunction with a general three-dimensional structural model and a boundary element approach to compute foundation-soil impedances for a linear viscoelastic and horizontally layered soil medium. The superstructure properties used as input to CLASSI are its fixed base modes of vibration, which are computed externally by any arbitrary structural analysis program. Originally developed to accommodate rigid surface foundations of arbitrary shape, CLASSI can now also accommodate deformable surface foundations, embedded rigid foundations of arbitrary shape, and embedded deformable foundations of cylindrical shape, as well as spatially random input motions from arbitrarily incident seismic waves.

SASSI The SASSI Program (Lysmer et al., 1981; Ostadan, 1983; Tabatabaie et al., 1982; Bechtel, 1991) is a three-dimensional finite element program for SSI analysis of a structure located on or within horizontal soil layers with equivalent linear material properties, and subjected to input motions from arbitrarily incident seismic waves. It uses a substructuring method termed the flexible volume method, in which the excavated soil that is replaced by the foundation is modeled and subtracted from the structure, and the SSI is assumed to occur over a volume rather than at the boundaries of the foundation. This simplifies the computation of the foundation-soil impedance matrix, and eliminates the need for a separate analysis to obtain foundation input motions that include scattering effects. The equivalent linear model in SASSI cannot be automatically iterated after each dynamic analysis run; rather, this iteration must be accomplished manually. Also, SASSI uses octahedral strain as the strain ordinate in its equivalent linear model.

## TABLE 1
## SUMMARY OF SELECTED SEISMIC SSI COMPUTER PROGRAMS

| Code | | FLUSH | CLASSI | SASSI | HASSI-8 | TRANL |
|---|---|---|---|---|---|---|
| **Methodology** | | Direct | Substructure | Substructure | Substructure | Direct |
| **Type** | | Finite Element | Finite Element (structure) and Boundary Element (foundation-soil system) | Finite Element | Hybrid | Finite Element |
| **Soil-Structure System Configuration** | | 2-D | 3-D | 3-D | 3-D | 3-D |
| **Soil Model** | **Layering** | Horizontal | Horizontal | Horizontal | Horizontal | Arbitrary |
| | **Constitutive Model** | Equivalent Linear | Linear Viscoelastic | Equivalent Linear (not automatic iteration) | Equivalent Linear | Nonlinear Hysteretic Cap Model |
| | **Large Deformation Capability** | No | No | No | No | Yes |
| | **Nonlinear Foundation Soil Interface** | No | No | No | No | Yes |
| **Structural Model** | **Element Types** | Linear elastic beam and planar elements | Any number of linear elastic, elements from any outside structural analysis program | Wide range of linear elastic structural elements | Wide range of linear elastic structural elements | Nonlinear 3D brick, 3D plate, planar, and beam elements, and elastic beam substructure elements. |
| | **Foundation Characteristics** | Any number of deformable embedded or surface foundations | Surface Foundations:<br>- Any number<br>- Rigid or deformable<br>Arbitrary shape<br><br>Embedded Foundation:<br>- Single foundation<br>- Cylindrical deformable<br>- Arbitratily shaped rigid | Any number of deformable embedded or surface foundations of arbitrary shape | Single deformable embedded or surface foundation of arbitrary shape | Any number of deformable embedded or surface foundations of arbitrary shape |
| **Boundaries** | **Lateral** | Wave Transmitting | Semi-Infinite layered medium | Wave Transmitting | Semi-Infinite Medium | Soil Island |
| | **Base** | Rigid | Semi-Infinite layered medium | Wave Transmitting Half Space | Semi-Infinite layered medium | Soil Island |
| **Input Motions** | **Basis for Computing Free Field Motions** | Specified control motions<br><br>Vertically incident body waves | Specified control motions<br><br>Spatially random and arbitrarily incident body and surface waves | Specified control motions<br><br>Arbitrarily incident body and surface waves | Specified control motions<br><br>Arbitrarily incident body waves | Arbitrary wave field and criterion |
| | **Basis for Computing Input Motions for SSI Analysis** | Deconvolution of control motions | Free field motions modified to include scattering effects | Free field motions (scattering effects included through flexible volume substructure method) | Free field motions modified to include scattering effects | Free field motions used as input to SSI analysis |
| | **Application of Input Motions** | At rigid base of soil model | At foundation (along soil-foundation interface) | At foundation (throughout embedded foundation) | At foundation (along soil-foundation interface) | At sides and base of soil island |
| **Computational Technique** | | Implicit<br><br>Frequency Domain | Implicit<br><br>Frequency Domain | Implicit<br><br>Frequency Domain | Implicit<br><br>Frequency Domain | Explicit<br><br>Time Domain |

HASSI-8 The HASSI-8 Program (Katayama et al., 1991) is the latest in the HASSI family of programs which uses a hybrid model consisting of a three-dimensional finite element model of the structure and a near field segment of soil, and an analytical solution to represent the far field soil region (Gupta et al., 1982: Chen et al., 1990). This latest version accommodates free-field motions from arbitrarily incident body waves, considers the site to be comprised of horizontally layered soils with an equivalent linear material model, and incorporates scattering effects into the computation of the foundation input motions. The equivalent linear soil model in HASSI-8 is defined in terms of octahedral strain.

TRANL The TRANL Program (Baylor et al., 1974; Isenberg et al., 1978) is a fully nonlinear three-dimensional finite element program that incorporates nonlinear material models for the continuum and structural elements, debonding and rebonding along the soil-structure interface, and large deformations. It uses a "soil island" approach in which free field ground motions corresponding to any desired combination of incident seismic waves are computed along a ficticious boundary enclosing a volume of soil (termed the soil island) that surrounds the location of the structure. Then, the structure is inserted into this soil island, the above free field motions are applied along the boundaries of the island, and the seismic response of the soil-structure system is computed. TRANL does not have wave-transmitting boundaries; therefore, the size of the soil island must be sufficiently large to minimize the interference of back reflections from the boundaries.

### 2.1.4 Nonlinear SSI Analysis Procedures

In addition to the TRANL Program summarized in Section 2.1.3, other significant efforts have been directed toward the development of nonlinear SSI analysis procedures. Examples of such efforts are (a) the use of nonlinear constitutive models based on mathematical functions, mechanical models, or plasticity theory (eg. Pyke, 1979; Vaughn and Isenberg, 1983); (b) discrete element methods using finite difference equations applied to a network of material zones to approximate the differential equations of motion for a continuum (e.g., Cundall, 1976); (c) the use of nonlinear soil models coupled with pore water pressure models within an effective stress framework, in order to incorporate pore pressure effects during the seismic analysis of the soil medium (e.g., Finn, 1990; Prevost, 1981; NRC, 1985); and (d) the use of boundary elements for an elastic layered far field region together with a nonlinear model of the soil and structure near-field region (e.g., Wolf and Darbe, 1984 and 1986). It is noted that the above referenced work of Prevost, Cundall, and Finn has been used primarily to assess pore pressure effects and the stability of soil deposits, earth structures, and retaining-structure/soil systems (e.g., Roth et al., 1991), and have not yet been widely applied to SSI analyses for critical buildings, tanks, etc. The remaining nonlinear procedures identified above have occasionally been applied for SSI analysis of major structures subjected to earthquake shaking; however their principal role has been as a research tool for gaining insight into earthquake-induced SSI effects and as a resource for calibrating simpler SSI analysis procedures. Along these lines, it is noted that simplified procedures that focus on particular nonlinear response characteristics such as foundation uplift have been developed (e.g., Yim and Chopra, 1985) and have provided helpful insights for incorporating these effects in conventional engineering applications.

### 2.1.5 Spring-Dashpot Models

A widely used subset of the foundation-soil dynamic impedances discussed in Subsection 2.1.1 is frequency-independent springs and dashpots, in which the spring stiffness coefficients represent the stiffness of the foundation-soil system and the viscous damping coefficients associated with the dashpots simulate the energy losses due to radiation damping. This use of springs and dashpots was originally developed as a frequency-independent analog to the theory of a rigid circular disk on an elastic half space (e.g., Lysmer, 1965; Hall, 1967;

Richart et al., 1970); this analog has since been extended to more general conditions of non-circular surface or embedded footing foundations (e.g., Roesset, 1980a and 1980b; Novak, 1987; Gazetas, 1991).

For dynamic SSI analyses involving mat or footing foundations, the application of spring-dashpot methods has most typically consisted of: (a) use of current frequency-independent spring-dashpot analogs to estimate spring stiffness and viscous damping coefficients associated with each degree of freedom of an assumed rigid foundation; (b) attachment of these spring-dashpot elements to the base of a finite element model of the structure; and (c) application of free field motions (usually design spectra) to the base of the spring-dashpot elements, and computation of the foundation-structure system's dynamic response. In such applications, the effects of scattering on the input motions are typically not considered, even though such effects may be particularly important for embedded foundations. Also, it is always desirable to: (a) check the spring and dashpot coefficients against coefficients estimated from existing frequency-dependent impedance results over the estimated range of predominant frequencies of the soil-structure system being analyzed; and (b) carry out parametric SSI analyses to assess the sensitivity of the computed system response to uncertainties in estimating the spring and dashpot coefficients and the input motions.

In contrast to mats and footings, bridge abutments and pile elements are modeled as equivalent springs only; i.e., as noted in Subsection 2.4, radiation and material damping characteristics of these elements have not yet been incorporated into the SSI analysis procedures used in the typical engineering and design practice for these elements. The growing body of experimental and analytical data on general impedance characteristics of pile foundations and bridge abutments will hopefully provide a basis for rectifying this situation in the near future (eg., Novak, 1991; Crouse et al., 1987; Wolf and von Arx, 1982; Roesset et al., 1986; Dobry and Gazetas, 1988; Banerjee et al., 1987).

## 2.2 Experimental Procedures and Programs

A variety of experimental test programs and procedures have been developed to provide data for evaluating: (a) the effects of SSI on the response of foundations and structures; (b) frequency-dependent impedances of existing soil-structure systems; and (c) current SSI modeling and analytical procedures. Because other participants at this workshop will be addressing these test programs and procedures in great detail, the following paragraphs will provide only brief summaries of the procedures, together with selected examples of experimental programs specifically directed toward investigation of dynamic SSI effects.

### 2.2.1 Full Scale Testing of Structures and Foundation Elements

The most common type of dynamic field testing of full-scale structures involves the use of harmonic force-excitations or quick release of applied static forces. Harmonic force excitations are typically applied using eccentric mass shakers with counter-rotating weights; these forces are computed as the product of the magnitude of the weights, the eccentricity of the weights, and the square of the frequency of excitation. Different modes of vibration of the soil-structure system can be excited by changing the location of the shaker on the structure, the direction of the applied harmonic force, and the excitation frequency (i.e., the rate of rotation of the counter-rotating weights). The quick-release test method involves the application of a static force at an appropriate location on the structure (using various methods ranging from pulling on the structure with a cable to pushing on the structure using a pressurized hydraulic ram). Quick release of these forces generates damped free vibrations of the structure. For both the eccentric mass shaker or quick-release test methods, motions of the structure, foundation, and often the soil medium as well are measured by a suitable array of accelerometers.

Both methods have been used to assess dynamic SSI effects for numerous structure types including simple foundations (e.g., Lin and Jennings, 1984; Novak, 1985; Stokoe and Erden, 1985), buildings (e.g., Luco et al., 1988b), and bridges (e.g., Douglas and Buckle, 1985; Douglas et al., 1991).

Several different data analysis methods are available for obtaining soil-structure system response characteristics from the motions measured during these dynamic tests. For example, Crouse et al. (1984) used data from eccentric mass shaker-induced excitations to develop transfer functions between recorded accelerograph station motions and free field motions and to obtain foundation-soil impedance matrices, in order to assess the importance of SSI at a "free-field" accelerograph station. Other example data analysis methods include: (a) use of a variational form of Raleigh's Principle to estimate the foundation-stiffnesses of a single span bridge from eccentric mass shaker test data (Crouse and Hushmand, 1987); and (b) formal system identification methods to identify the modes of vibration excited from quick-release testing of a two-span bridge and the surrounding soil medium (Werner et al., 1990).

When used with sound data analysis and interpretations, full scale dynamic tests as described above have provided valuable insight into the dynamic SSI process. For example, tests of a nine-story reinforced concrete building by Foutch and Jennings (1978) and later by Luco et al. (1987 and 1988b) have demonstrated that the compliance of the building's foundation-soil system in lateral translation and rocking had an important effect on the building's response during the tests. Similar insights into the importance of SSI at short bridge structures has been provided by forced vibration test data (Werner et al., 1990; Crouse et al., 1987). Such data can serve to assess the adequacy of SSI provisions in current seismic design standards (as discussed in Section 2.4) and can provide an important basis for assessing procedures for dynamic SSI analysis (e.g., Wong et al., 1988; Crouse et al., 1990).

### 2.2.2 Explosive Testing

A second experimental approach for investigating dynamic SSI effects is through the use of explosive testing. This approach has used two-dimensional sequentially-fired explosive arrays, barriers, and specially designed source devices to develop ground motions that simulate earthquake-induced ground shaking. The most significant program using explosive testing, named SIMQUAKE, was sponsored by the Electrical Power Research Institute. Four tests under this program (mini-SIMQUAKE, SIMQUAKE IA, SIMQUAKE IB, and SIMQUAKE II) were conducted in mid-to-late 1970's by the Civil Engineering Research Facility of the University of New Mexico, at an alluvial soil site near Albuquerque, New Mexico. A later test, SIMQUAKE III, was conducted at a rock site in upstate New York (Higgins, 1991).

The SIMQUAKE tests deployed small (1/24-to-1/8) scale, partially embedded, nuclear plant containment structure models in the geologic medium. Arrays of buried explosive charges were set off, generating shaking throughout the medium and in the model structures. Accelerometers located in the model structures and throughout the geologic medium measured the motions of the soil-structure system.

A key objective of the SIMQUAKE tests in New Mexico was to provide data for evaluation and correlation with results from nonlinear SSI analysis methods. Two methods that were used were TRANL (in SIMQUAKE IA and IB), which is summarized in Section 2.1 of this paper, and STEALTH (in SIMQUAKE II) which is a two-dimensional explicit finite difference procedure with nonlinear constitutive modeling capability and debonding-rebonding elements along the soil-structure interface. Both methods used a soil-island modeling approach, and free-field accelerometers were deployed in the SIMQUAKE tests to record the motions around the periphery of the soil-structure system for use as input motions around the soil island.

The above analyses of the SIMQUAKE soil-structure system configuration in New Mexico consisted of pre-test predictions and post-test analyses. Both sets of analyses showed that nonlinear rocking response of the structures was primarily a result of debonding-rebonding and compaction of the soil along the soil-structure interface -- a trend that was confirmed by dynamic interface stress measurements and post-test inspection of the interface region (Isenberg, et al., 1978; Vaughn and Isenberg, 1983).

### 2.2.3 Laboratory Testing

Because of the expense and difficulty associated with the dynamic testing of full-scale soil-structure systems, laboratory testing of scale models has been used extensively. Such tests may have any one of a number of different objectives. For example, they may be needed to provide qualitative information on general deformation characteristics or potential failure modes. Alternatively, such tests may be required to provide quantitative measurements of soil-structure system response characteristics that can be used to verify system designs or to evaluate analytical procedures for performing SSI calculations. In either case, if the model tests are to represent prototype soil-structure system seismic response characteristics, it is necessary for the laboratory tests to be designed to: (a) satisfy similitude relationships that would enable the test results to be extrapolated to prototype conditions; (b) apply earthquake-like excitations to the model system; and (c) minimize wave reflections from the sides of the model container.[3]

For many years, dynamic scale model testing of soil-structure systems has been carried out under standard gravitational ("one-g") conditions. However, because the response characteristics of the soil depends on ambient conditions (eg., confining pressures for sand deposits) it is particularly important to account for these conditions if the model tests results are to be used to represent prototype seismic behavior. This can be accomplished by centrifuge testing, in which small-scale soil-structure system models are tested in an increased gravity field in order to properly incorporate ambient conditions and maintain similitude with prototype conditions. In such methods, a scale model soil-structure system is constructed that is p times smaller than its prototype, and contains the same soil materials as the prototype soils. The model is then placed in a centrifuge acceleration field that is p times greater than that of normal gravity conditions. When subjected to the assumed seismic excitations during the dynamic centrifuge tests, the model's measured stresses and strains will be the same as for the prototype conditions. Dynamic seismic excitations for the centrifuge tests most typically use external sources to shake the entire container such as servo-hydraulic shakers (e.g., Roth et al., 1986), shake tables (e.g., Schofield and Steedman, 1988) or a hammer impact (e.g., Weissman and Prevost, 1989). Methods for minimizing reflected waves at the side boundaries of the container are all based on the premise that the motions at the sides will be horizontal, due to vertically incident shear waves caused by applying horizontal motions to the container. Various methods for accomplishing this include: (a) use of a rectangular or circular container of the soil-structure system model that consists of thin aluminum plates or rings with a low-friction material or steel bearings between adjacent plates to absorb friction (e.g., Hushmand et al., 1987; Schofield and Steedman, 1988); and (b) use of an absorptive material along the sides of the container (e.g., Weissman and Prevost, 1989). An excellent summary of the current state of practice in dynamic centrifuge testing is provided by Ko (1991).

In his recent summary of modeling considerations for earth structures, Scott (1990) points out that, although centrifuge tests closely represent the actual stress-strain behavior of full scale soils, model tests under one-g conditions may also be used if they are carefully carried out using prepared materials with appropriate

[3]These requirements are desirable but not necessarily essential if it is desired to use SSI analysis procedures to represent an exact model test setup and to use the test measurements solely to check the analysis results.

scaling. This may be an economical and desirable supplement to centrifuge testing, although further evaluation and development of similitude requirements is still needed.

The centrifuge device has been used to assess dynamic SSI effects for a variety of structure and foundation types, including bridge-abutment-backfill systems (e.g., Hushmand et al., 1986), retaining walls (e.g., Ortiz et al., 1983), and pile elements (e.g., Finn and Gohl, 1987). Shake table tests of dynamic SSI effects under one-g conditions are described by Tamori and Kitagawa (1988) and by Fukutake et al. (1990). In these latter papers, no specific mention is made of similitude or scaling considerations; instead, the test results are used directly to verify analytical procedures.

## 2.3 Strong Motion Instrumentation

### 2.3.1 Current Instrumentation of Structures

A potentially invaluable source of information for dynamic SSI effects on the seismic response of structures would be the analysis of recorded earthquake motions from properly planned arrays of strong motion instruments in structures and the adjacent soil medium. To date, however, even with the substantially expanded instrumentation of buildings and other structures by the California Division of Mines and Geology (CDMG) and the U.S. Geological Survey (USGS), there are only limited arrays of recorded motions that are adequate for assessment of dynamic SSI effects during actual earthquakes. For example, current instrumentation arrays at many existing buildings are often not adequate to measure the full translational and rotational response of the foundations, and virtually never have accelerometers in the adjacent soil medium. One reason for this is that the strong motion instrumentation in many buildings has often been based on the minimum requirements specified in the Uniform Building Code (UBC). These code requirements for buildings in Seismic Zones 3 and 4 call for a minimum of only three triaxial accelerometers in: (a) buildings with 6 or more stories whose floor area exceeds 60,000 square feet; and (b) buildings with 10 or more stories, regardless of floor area (eg., ICBO, 1991). Experience from past earthquakes has shown that such limited instrument arrays are insufficient to evaluate SSI effects as well as many aspects of structural response. The expanded instrumentation of buildings by CDMG and USGS over this UBC minimum level has led to several sets of recorded building motions during recent earthquakes that can be used to assess certain SSI characteristics (Huang et al., 1989; Celebi et al., 1989; Werner et al, 1992); however, the current number of adequately instrumented buildings is still limited. Strong motion instrument arrays in bridges and other structures are similarly lacking.

One program where progress has been made along these lines has been at the Meloland Road Overcrossing (MRO) -- a two-span reinforced concrete bridge near El Centro, California. An array of 26 strong motion accelerometers was deployed at the MRO in 1978, and strong motions (with peak accelerations as high as 0.51 g at the midlength of the bridge deck) were subsequently recorded by all of the instruments during the 1979 Imperial Valley Earthquake ($M_s$ = 6.8). Formal system identification methods were applied to these data by Werner et al. (1987), and led to significant insights into the MRO's seismic response. However, because of insufficient instrumentation at the bridge abutments and central pier footing, the recorded motions were insufficient to fully assess the potentially important dynamic SSI effects at the MRO. For this reason, the California Department of Transportation funded a follow-on project involving joint efforts of the author, Bruce Douglas of the University of Nevada-Reno, and C.B. Crouse of Dames & Moore, that has featured full scale dynamic testing of the MRO to gain further insight into its dynamic SSI characteristics. An important result of this current study has been the development of a plan for expanding the instrumentation at the bridge's abutments, embankments, and pier foundation that will facilitate evaluation of its dynamic SSI effects during future earthquakes. This expanded instrumentation has now been deployed by CDMG.

### 2.3.2 Lotung Taiwan Program

A significant program directed toward evaluation of dynamic SSI analysis procedures commonly used in the U.S. nuclear industry was carried out in the mid-to-late 1980s by the Electric Power Research Institute and the Taiwan Power Company, with additional support from the U.S. Nuclear Regulatory Commission. This program is described in detail in EPRI (1989) and is briefly summarized below.

Under this program, two large-scale (1/4- and 1/2-scale) reinforced concrete models of nuclear plant containment structures were constructed, each with extensive instrumentation on the models themselves and in the adjacent soil. The models are located on a soft alluvium soil site in Lotung, Taiwan, where frequent earthquakes occur and where SMART-1, the strong-motion array sponsored by the National Science Foundation, is in operation. Over the months following completion of the Lotung facility in late 1985, several earthquakes with Richter magnitudes ranging from 4.0 to 7.5 were recorded at the site. This strong motion data base, together with measurements from forced vibration tests of the model structures, formed the basis for a cooperative program to evaluate existing SSI analysis procedures. The evaluation process consisted of blind predictions by the various analysis procedures, and comparisons of the predicted results with the measured results from the forced vibration tests and earthquake excitations. A total of 13 participants from the U.S., Taiwan, Japan, and Switzerland carried out these predictions using a variety of SSI analysis methods that ranged from simple soil-spring representations to more complex finite element methods and substructure impedance approaches. The recorded soil-structure system responses from the earthquakes were made available to the participants only after their predictions had been documented.

The Program concluded with a 2-1/2 day international workshop, attended by over 100 engineers and researchers, at which the blind prediction analysis results were presented and compared. Assessments of current SSI practice and recommendations for future research were developed from the workshop. Such assessments by Hadjian et al (1991) have indicated that differences in the various analysis results presented at the workshop were due more to the variations in the modeling of the soil-structure system and the characterization of the input motions than to the different computational methods used. According to Hadjian et al, the results from the Lotung SSI analysis results demonstrated the importance of scattering effects due to foundation embedment, as well as backfill stiffness effects on foundation impedances and possibly on input motions. In addition, the assumptions of vertical wave propagation and the equivalent linear soil model were judged to be acceptable when applied to the Lotung data, although the development of soil shear modulus and damping curves as a function of strain from geophysical and laboratory tests (as required for the equivalent linear soil model) were shown to exhibit significant variability.

It is noted that another program of this type is currently being developed at a stiff soil site in Hualien, a highly seismic area on the east coast of Taiwan. A circular array of strong motion instruments, named SMART-2, will be installed around Hualien by scientists from Taiwan. The primary objectives of the Hualien program will be to obtain earthquake-induced SSI data at a stiff soil site, and to use these data to further evaluate and develop SSI analysis methods and criteria (Tang et al, 1991; Youd, 1991).

## 2.4 Consideration of SSI in Current Design Standards

### 2.4.1 Building Design Standards

Current national design standards for buildings are contained in: (a) the Uniform Building Code (UBC) (ICBO, 1991); (b) the recommended provisions for buildings developed under the National Earthquake Hazards Reduction program (NEHRP) (FEMA, 1988); and (c) the seismic design provisions for conventional and essential Armed Services buildings that were developed jointly by the Departments of the Army, Navy and Air Force (DOANAF, 1982 and 1988). These latter documents are commonly termed the Tri-Services Manuals.

Neither the UBC nor the Tri-Services Manuals contain any specific provisions for considering SSI effects during the seismic design of structures and foundations. Such provisions are included in the NEHRP standards. However, they are relegated to an appendix rather than to the main body of the NEHRP standards because, as indicated in the commentary for this appendix, "use of (the SSI) procedures in the design of most buildings is considered to be unnecessary; therefore, it was decided that they are too specialized to be considered in the Provisions proper." This conclusion is contradicted by studies of several sets of building strong motion records that are sufficient to evaluate certain SSI effects; these studies have demonstrated the importance of SSI effects on the seismic response of many classes of buildings (e.g., Crouse and Jennings, 1975; Rojahn and Mork, 1982; Luco et al., 1987; Bard, 1988; Tajimi, 1988; Werner et al., 1992).

The SSI evaluation procedures contained in NEHRP (1988) include: (a) a method for estimating damping ratios for the structure-foundation system that, in turn, are incorporated into an equation for reducing the base shear force because of SSI; and (b) a procedure for estimating the lengthening of the building's period due to SSI that is dependent on the horizontal and rocking spring stiffness of the foundations. These spring stiffnesses are to be determined from "principals of foundation mechanics," with suggested equations in the commentary that are derived from analytical solutions for the response of a simple structure on a rigid circular disk foundation that is bonded to the surface of an elastic half space or is partially embedded within the half space. Therefore, these suggested procedures provide guidance for base mat or footing foundations only; no comparable guidance is provided for pile foundations.

### 2.4.2 Highway Bridge Design Standards

Seismic design guidelines developed under the auspices of the Applied Technology Council (ATC, 1981) have been approved as a national guide specification for highway bridges by the American Association of State Highway and Transportation Officials (AASHTO, 1983). A subsequent three-volume report published by the Federal Highway Administration supplements the AASHTO guide specifications by providing more complete information on the seismic design of highway bridge foundations and abutments (FHWA, 1986). This information, together with current foundation/abutment seismic design practice by the California Department of Transportation (Caltrans) has been summarized by Lam et al. (1991). The seismic design procedures outlined in FHWA (1986) and Lam et al. (1991) include the incorporation of SSI into the seismic design of pile foundations, drilled shafts, footings and abutments, as summarized below.

Piles and Drilled Shafts The FHWA (1986) provisions for estimating pile stiffnesses for SSI analysis range from hand calculation/graphical methods for a single pile in a uniform elastic soil medium to computerized methods for obtaining nonlinear vertical and lateral load deflection (t-z and p-y) curves for a single pile in a layered soil medium. No procedures are provided for incorporating pile group effects (i.e., through-soil coupling of adjacent piles). Provisions are outlined for assessing effects of the axial stiffness of individual piles on the overall rocking stiffness of the foundation, and for assessing the contributions of the pile cap to the foundation's overall stiffness. The SSI provisions given in FHWA (1986) for drilled shafts are similar to those for individual piles.

Footings Procedures incorporated into FHWA (1986) for representing SSI effects at highway bridge footings are based on simple spring analogs to the theory of a rigid circular disk on an elastic half-space, with corrections to incorporate rectangular footing shapes and foundation embedment.

Abutments The FHWA (1986) provisions consider abutment behavior only in the longitudinal direction of the bridge; no guidance for assessing abutment performance in the transverse or vertical direction is provided. For design of seat-type abutments, which do not depend on the bridge superstructure for stability, FHWA (1986) recommends checking to assure that pseudostatic displacements caused by loads computed using the Mononobe-Okabe Equation are within acceptable limits. Integral abutments are analyzed by using retaining wall solutions to estimate longitudinal translational and rotational spring stiffnesses for the end-wall/backfill system. These are combined with the stiffness contributions of the footing or pile foundation system to obtain an estimate of the total abutment stiffness. Then, an iterative approach is used to modify the abutment stiffness and design to (a) obtain equivalent abutment stiffnesses that are consistent with abutment displacements computed from a seismic analysis of the bridge; and (b) check that abutment force capacities and acceptable displacement limits are not exceeded. In their summary of the FHWA provisions, Lam et al. suggest the possible use of an abutment stiffness estimation procedure by Wilson (1988). Wilson's procedure can be used to obtain six discrete spring stiffnesses that correspond to three translational and three rotational degrees of freedom of the abutment, and include the contributions of the abutment walls, pile foundations, and soil.

Damping FHWA (1986) does not contain provisions for considering the radiation and material damping contributions of the various foundation elements; instead, dynamic seismic analysis of the bridge are carried out using a response spectrum approach with a modal damping ratio of 0.05. It is noted that data from dynamic tests of pile foundations (e.g., Crouse and Cheang, 1987; Han and Novak, 1988), bridge abutments (Crouse et al., 1987; Werner et al., 1990) and footings (see Section 2.3 of this paper) all suggest higher levels of damping; however, this information has not yet been synthesized for incorporation into bridge design practice.

## 3.0 RESEARCH NEEDS

The preceding summary of the current state-of-the-practice is intended to provide a framework for identifying recommended research needs directed toward: (a) providing insights into dynamic SSI effects during earthquakes; and (b) improving the current state of practice for SSI evaluation and analysis. These recommendations are provided in the following paragraphs.

### 3.1 Education

The overall engineering community has not had adequate exposure to SSI concepts, procedures, and evaluation results that would enhance the incorporation of SSI provisions in current seismic design practice. Therefore, provisions for workshops, conferences, and publications that present SSI to practicing engineers are encouraged.

### 3.2 Seismic Design Standards

Simplified and practical SSI evaluation procedures that are currently available (e.g., Veletsos et al., 1988) should be reviewed to identify (a) those elements of the current procedures that may be appropriate for inclusion into current seismic design standards; and (b) directions for future research to further develop and improve simplified SSI procedures. In particular, programs are recommended for calibrating simplified procedures against results from: (a) full scale and model tests of soil-structure systems; (b) SSI evaluations from strong motion records; and (c) detailed SSI analytical methods.

### 3.3 Expanded Strong Motion Instrumentation

There is a need for expanded strong motion instrumentation of existing structures analogous to the expanded program for the Meloland Road Overcrossing that is summarized in Subsection 2.3.1. Ideally, such instrumentation should be deployed to measure the significant translational and rotational degrees-of-freedom of the foundation response, lateral pressures applied to the embedded foundation elements, and horizontal and vertical variations of motion in the surrounding soil. Because the deployment of such arrays will invariably be limited by cost constraints, initial planning and prioritization of expanded instrumentation programs should focus on areas of high seismicity and major structures where SSI effects are likely to be important or are not well understood.

### 3.4 Full Scale Test Programs

Full scale tests of structures subjected to harmonic or quick-release excitations represent a potentially invaluable source of information for studying dynamic SSI effects and for calibrating SSI analysis procedures. Such tests are particularly encouraged at structures where strong motion records have been obtained during past earthquakes. Correlation of test-induced and earthquake-induced measurements and response characteristics for the same structure will be valuable for enhancing the future usefulness of full scale testing for evaluating dynamic SSI effects in an earthquake environment.

### 3.5 Data Evaluation Methods

Further development of methods that optimize the reliability of the information that can be extracted from earthquake or test excitations of soil-structure systems should be encouraged. For example, research should be directed toward enhancing formal system identification methods (that can now identify classical modes of vibration of a soil-structure system under dynamic excitation) to provide a capability for identifying non-classical modes, stiffness (impedance) matrices, and nonlinear system parameters.

### 3.6 Pile Foundations

The performance of pile foundations in a seismic environment is not well understood, and their current seismic design and analysis methods require further development. Research should be directed toward synthesizing existing experimental and analytical results to develop seismic design procedures for pile foundations that incorporate their dynamic response characteristics. Gaps in the technology for meeting this objective should be addressed by further research. For example, research should be directed toward the evaluation of soil-pile interface behavior, the response characteristics of batter piles, nonlinear pile-soil-pile interaction (pile group effects), and the contributions of the pile-cap and pile-head connection details to the overall foundation response. Along these lines, the implementation of well-planned pile load tests for developing additional data to calibrate nonlinear pile foundation analysis procedures is particularly encouraged. It is noted that recommendations comparable to these have also been made in a soon-to-be-published paper on highway bridge foundations by Lam and Martin (1992).

## 3.7 Bridge Abutments

The dynamic response characteristics of bridge abutments is not well understood. Further analytical and experimental research should be directed toward improving our ability to characterize the stiffness and damping characteristics of skewed and non-skewed abutments.

## 3.8 Underground Structures

Underground structures for subways and highway systems are typically designed to conform to the seismic deformations of the adjacent soil in a free field environment (Monsees and Merritt, 1991). Although this assumption is reasonable for long flexible tunnels, it is questionable at locations of "hard-points" in underground structural systems, such as end walls at an underground subway station-tunnel interface or at entrance structures for underground stations. Also, the effects of through-soil coupling between adjacent underground structures or between near-surface underground structures and above-ground buildings is not well understood. Research into these various aspects of the seismic response of underground structures is recommended.

## 3.9 Stochastic Analysis Procedures

Because of the complexities inherent in the specification of seismic input motions, the soil-structure system modeling process, and the SSI analyses procedures, results of any SSI analysis will contain inherent uncertainties. Several past efforts have been directed toward incorporating these uncertainties into the analysis of dynamic SSI (e.g., the Seismic Safety Margin Research Program at Lawrence Livermore Laboratories). However, these efforts have not found their way into the current practice for dynamic SSI analysis. Research directed toward building on these past efforts to develop practical stochastic SSI analysis procedures and models of input motions and soil material properties should be encouraged.

## 3.10 Equivalent Linear Model

The iterative equivalent liner model was originally developed for conditions of vertically incident shear waves in a horizontally layered medium. The extension of these concepts to the more complex soil stress-deformation states that will be induced under non-vertically incident wave conditions is not fully understood. Since major SSI analysis procedures now incorporate the equivalent linear model for such complex stress-deformation states (e.g., SASSI and HASSI), an experimental basis for extending the model to apply to these conditions should be developed.

## 3.11 Seismic Input Motions

There are insufficient ground motion data for assessing the current procedures for computing spatially varying free field motions (including wave passage and incoherence effects) for use as input to SSI analyses. Research directed toward deploying additional arrays for this purpose and toward using available data from existing arrays to assess current procedures is recommended.

## 3.12 Foundation-Soil Damping

One of the most important and least understood parameters for the seismic design of foundations is the foundation-soil damping ratio. A first step toward gaining insight into the characterization of this parameter for seismic design purposes should consist of a detailed review and synthesis of the existing experimental and analytical data base that could be processed to estimate damping characteristics for foundation-soil systems. This review would provide a basis for (a) evaluating whether the existing data base is sufficient to develop rational procedures for estimating foundation-soil damping for seismic design purposes; (b) if so, proceeding with the development of these procedures; and (c) defining additional research that would build on this initial information in order to enhance our ability to provide improved damping estimates in the future.

## 3.13 Soil-Structure Interaction Experiment

The Lotung Taiwan program summarized in Subsection 2.3.2 has provided an excellent basis for gaining insight into SSI phenomena and for assessing SSI analysis procedure. Programs of this type should be implemented at other highly seismic regions and should address other than the soft soil conditions and nuclear plant foundation conditions that were the focus of the Lotung Program. For example, other programs of this type could be deployed at firm sites and could include pile foundations, bridge abutments and/or building footing and mat foundations, and could be a focus for addressing many of the SSI research needs identified above. In additions, such programs (as well as more recent data from Lotung) could provide a basis for calibrating and assessing various extensions of several of the SSI analysis procedures that have been implemented since the completion of the Lotung program.

# REFERENCES


American Association for State Highway Transportation Officials (AASHTO) (1983). Guide Specifications for Seismic Design of Highway Bridges, Highway Subcommittee on Bridges and Structures, Washington D.C.

Applied Technology Council (ATC( (1981). Seismic Design Guidelines for Highway Bridges, Report No. FHWA/RD-81/081, Federal Highway Administration, Washington D.C., Oct.

Banerjee, P.K. et al (1987). "Dynamic Behavior of Axial and Laterally Loaded Piles and Pile Groups", Chap. 3 in Dynamic Behavior of Foundations and Buried Structures (Developments in Soil Mech. Found. Eng., Vol. 3) Ed. P.K. Banerjee and R. Butterfield, Elsevier App. Sc., London pp. 97-133.

Bard, P-Y (1988). "The Importance of Rocking in Building Motion: An Experimental Evidence", Proc. of Ninth World Conf. on Earthq. Eng., Vol. 8, Tokyo-Kyoto, Japan, pp. 333-338, Aug. 2-9.

Baylor, J.L. et al (1974). TRANL: A 3-D Finite Element Code for Transient Nonlinear Analysis, DNA-3501F, Weidlinger Associates, New York NY, June.

Bechtel (1991). SASSI Theoretical Manual, Bechtel Corporation, San Francisco CA.

Bycroft, G.N. (1956). "Forced Vibrations of a Rigid Circular Plate on a Semi-Infinite Elastic Space and on an Elastic Stratum" Philosophical Trans., Royal Society, London, Ser. A, Vol. 248, pp. 327-368.

Celibi, M. et al (1989). "Some Significant Records from Instrumented Structures in California - USGS Program", Seismic Engineering - Research and Practice, ASCE, New York NY, pp. 247-256, May.

Chen, C-H, et al (1990). "Correlation of Predicted Seismic Response using Hybrid Modeling with EPRI/TPC Lotung Experimental Data", J. of Earthq. Eng. and Struct. Dyn., Vol. 19, pp. 993-1024, Oct.

Crouse, C.B. and Cheang, L. (1987). "Dynamic Testing and Analysis of Pile-Group Foundations", Proc. of the Symposium on Dynamic Response of Pile Foundations, ASCE.

Crouse, C.B. and Hushmand, B. (1987). "Estimation of Bridge Foundation Stiffnesses from Forced Vibration Data", Proc. of Third International Conf. on Soil Dyn. and Earthq. Eng., June.

Crouse, C.B. and Jennings, P.C (1975). "Soil-Structure Interaction during the San Fernando Earthquake", Bull. of Seismol. Soc. of Amer., Vol. 65, No. 1, Feb.

Crouse, C.B., et al (1984). "Experimental Study of Soil-Structure Interaction at an Accelerograph Station", Bull. of Seismol. Soc. of Amer., Vol. 74, No. 5, pp. 1995-2014, Nov.

Crouse, C.B. et al (1987). "Dynamic Soil-Structure Interaction of a Single-Span Bridge", J. of Earthq. Eng. and Struct. Dyn., Vol. 15, pp. 771-729.

Crouse, C.B., et al (1990). "Foundation Impedance Functions: Theory vs. Experiment" J. of Geotech Eng., ASCE, Vol. 116, No. 3, pp. 432-449, March.

Cundall, P.A. (1976). "Explicit Finite Difference Methods in Geomechanics", Second Conf. in Numerical Methods in Geomechanics, Blacksburg VA, June.

Departments of the Army, Navy, and Air Force (DOANAF) (1982). Seismic Design for Buildings, TM5-809-10, NAVFAC P-355, AFM88-3 (Ch. 13), Washington, D.C., Feb.

Departments of the Army, Navy, and Air Force (DOANAF) (1988). Seismic Design Guidelines for Essential Buildings, TM5-809-1, NAVFAC P-355.1, AFM88-3 (Ch. 13, Sec. A), Washington, D.C., Feb.

Dobry, R. and Gazetas, G. (1988). "Simple Method for Dynamic Stiffness and Damping of Floating Pile Groups", Geotechnique, Vol., 38, No. 4, pp. 557-574.

Douglas, B.M et al (1991). "Parameter Identification Studies for Meloland Road Overcrossing", Proc. of U.S. - New Zealand Pacific Conf. on Earthq. Eng., Univ. of Aukland, Aukland NZ, Nov.

Douglas, B.M. and Buckle, I.G. (1985). "Field Response Studies of Two Highway Bridges Subjected to Simulated Earthquake Loads", Proc. of Second Joint U.S. - Japan Workshop on Performance and Strengthening of Highway Bridge Structures and Research Needs, San Francisco CA, pp. 203-208, Aug. 19-20.

Electric Power Research Institute (EPRI) (1989). Proceedings: EPRI/NRC/TPC Workshop on Seismic Soil-Structure Interaction Analysis Techniques using Data from Lotung, Taiwan, EPRI NP-6154, Vols. 1 and 2, March.

Federal Emergency Management Agency (FEMA) (1988). NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings, FEMA 95, October.

Federal Highway Administration (FHWA) (1986). Seismic Design of Highway Bridge Foundations, FHWA/RD-86/101, FHWA Office of Eng. and Hwy. Operations, McLean VA, June.

Federal Highway Administration (FHWA) (1986). Seismic Design of Highway Bridge Foundations, (3 volumes), Report No. FHWA/RD-86/101, Washington D.C., June.

Finn, W.D.L. (1990). "Analysis of Deformations, Stability, and Porewater Pressures in Port Structures", Proc. of Workshop on Seismic Safety Planning for Port of Los Angeles, Vol., 3, Los Angeles CA, March.

Finn, W.D.L. and Gohl, W.B. (1987). "Centrifuge Model Studies of Piles under Simulated Earthquake Lateral Loading" Proc. of Special Session of Geotech. Eng. Div., Convention, ASCE, Geotech Spec. Pub. 11, pp. 21-39.

Foutch, D.A. and Jennings, P.C. (1978) "A Study of the Apparent Change in the Foundation Response of a Nine-Story Reinforced Concrete Building", Bull. of Seismol. Soc. of Amer., Vol. 68, pp. 219-229.

Fukutake, K. et al (1990). "Analysis of Saturated Dense Sand-Structure System and Comparison with Results from Shaking Table Test", J. of Earthq. Eng. and Struct. Dyn., Vol. 19, pp. 977-992, Oct.

Gazetas, G. (1991a). "Formulas and Charts for Impedances of Surface and Embedded Foundations", J. of Geotech. Engr., ASCE, Vol. 117, No. 9, pp. 1363-1381, Sept.

Gazetas, G. (1991b). "Foundation Vibrations", Foundation Engineering Handbook, 2nd Edition (Edited by H.V. Fang). Van Nostran Reinhold, New York.

Gupta, S. et al (1982). "Three Dimensional Hybrid Modeling of Soil-Structure Interaction", J. of Earthq. Eng. and Struct. Dyn., Vol. 10, pp. 69-87.

Hadjian, A.H. et al (1991). "Assessment of Soil-Structure Interaction Practice Based on Synthesized Results from Lotung Experiemnt - Earthquake Response", Trans. of Eleventh Int. Conf. on Struct. Mech. in Reactor Technology (SMIRT-11), Vol K, Tokyo, Japan, pp. 201-212, Aug.

Hall, J. R., Jr. (1967). "Coupled Rocking and Sliding Oscillations of Rigid Circular Footings", International Symposium of Wave Propagation and Dynamic Properties of Earth Materials, Albuquerque NM, Aug.

Han, Y. and Novak, M. (1988). "Dynamic Behavior of Single Piles under Strong Harmonic Excitation", Canadian Geotech. J., Vol. 25, No. 2, pp. 523-534, Aug.

Higgins, C.J. (1991). Explosive Simulation of Earthquake-Like Ground Motion - A State-of-the-Art Summary, prepared for National Science Foundation Workshop on Experimental Needs in Geotechnical Earthquake Engineering, Albuquerque NM, Nov. 4-5.

Huang, M-J. et al (1989). "Strong Motion Records from Buildings", Seismic Engineering - Research and Practice, ACSE, New York NY, pp. 237-246.

Hushmand B. et al (1986). "Centrifuge Testing of a Bridge-Soil Model", Seismic Design of Monolithic Bridge Abutments, Earth Technology Corp., Long Beach CA, July.

Hushmand, B. et al (1987). "Site Response and Liquefaction Studies Involving Centrifuge", Proc. of Third Int. Conf. on Soil. Dyn. and Earthq. Eng., Princeton Univ., Princeton NJ, June.

International Conference of Building Officials (ICBO) (1991). Uniform Building Code, 1991 Edition, Whittier CA.

Isenberg, J., et al (1978). Nonlinear Soil-Structure Interaction, EPRI NP-941, Elec. Power Res. Inst., Palo Alto CA, Dec.

Katayama, I., et al (1991). "Wave Scattering Effect in Soil-Structure Interaction", Trans. of Eleventh Int. Conf. on Struct. Mech. in Reactor Technology (SMIRT-11), Vol. K, Tokyo, Japan, pp. 153-158, Aug.

Ko, H-Y (1991). Dynamic Tests on Centrifuges and Shake Tables, prepared for National Science Foundation Workshop on Experimental Needs in Geotechnical Earthquake Engineering, Albuquerque NM, Nov. 4-5.

Lam, I.P. et al (1991). "Modeling Bridge Foundations for Seismic Design and Retrofitting", Proc. of Third Bridge Eng. Conf., Transportation Research Board/Federal Highway Administration, March, pp. 113-125.

Lam, I.P. and Martin, G.R. (1992). Geotechnical Considerations for Seismic Design and Retrofitting of Highway Bridges, paper submitted for preseentation at annual Transportation Research Board Meeting, Washington, D.C.

Lamb, H. (1904). "On the Propagation of Tremors over the Surface of an Elastic Solid" Philosophical Trans., Royal Society, London, Ser. A, Vol. 203, pp. 1-42.

Lin, A.H. and Jennings, P.C (1984). "Effect of Embedment on Foundation-Soil Impedances", J. of Eng. Mech., ASCE, Vol. 110, No. 7, pp. 1060-1075, July.

Luco, J.E. (1982). "Linear Soil-Structure Interaction: A Review", Earthquake Ground Motion and its Effects of Structures, ASME, New York NY, pp. 41-57.

Luco, J.E. and Wong, H.L., (1982). "Response of Structures to Nonvertically Incident Seismic Waves", Bull. of Seismol. Soc. of Amer., Vol. 72, No. 1, pp. 265-302, Feb.

Luco, J.E. and Wong, H.L (1987). "Seismic Response of Structures Embedded in a Layered Half Space", J. of Earthq. Eng. and Struct. Dyn., Vol. 15, pp. 233-245, Feb.

Luco, J. E. et al (1987). "On the Apparent Change in Dynamic Behavior of a Nine Story Reinforced Concrete Building", Bull. of Seismol. Soc. of Amer. Vol. 77, No. 6, pp. 1961-1983, Dec.

Luco, J. E. et al (1988a). "Response of Foundations and Structures to Spatially Random Ground Motion", Proc. of Second Workshop on Strong Motion Arrays, SMART-1, Taipei, Taiwan, pp. 423-442.

Luco, J.E. et al (1988b). "Isolation of Soil-Structure Interaction by Full-Scale Forced Vibration Testing", J. of Earthq. Eng. and Struct. Dyn., Vol. 16, pp. 1-21.

Lysmer, J. (1965). Vertical Motion of Rigid Footings, Ph.D. Dissertation, Univ. of Michigan, Ann Arbor MI, Aug.

Lysmer, J. (1978). Analytical Procedures in Soil Dynamics, UCB/EERC-78/29, Univ. of Calif., Berkeley CA, Earthquake Eng. Res. Ctr., Dec.

Lysmer, J., et al (1975). FLUSH, A Computer Program for Approximate 3-D Analysis of Soil-Structure Interaction Problems, EERC 75-30, Univ. of Calif., Berkeley CA, Earthquake Eng. Res. Ctr., Nov.

Lysmer, J. et al (1981). SASSI, A System for Analysis of Soil-Structure Interaction, UCB/GT81-02, Univ. of Calif.

Monsees, J. and Merritt, J. (1991). "Earthquake Considerations in the Design of the Los Angeles Metro", Proc. of Third U.S. Conf. on Lifeline Earthq. Eng., Los Angeles CA, ASCE, pp. 75-88, Aug.

National Research Council (NRC) (1985). Liquefaction of Soils during Earthquakes, National Academy Press, Washington, D.C.

Novak, M. (1985). "Experiments with Shallow and Deep Foundations", Proc. ASCE Symp., Vibration Prob. in Geotech. Eng., ASCE, New York NY, pp. 1-26.

Novak, M. (1987). "State-of-the-Art in Analysis and Design of Machine Foundations" Soil-Structure Interaction, Elsevier and Computational Mechanics, Ltd., New York NY, pp. 171-192.

Novak, M. (1991). "Piles under Dynamic Loads" Proc. of Second Int. Conf. on Recent Advances in Geotech. Earthq. Eng. and Soil Dyn., Vol. 3, St. Louis MO, pp. 250-273

Ortiz, L.A. et al (1983). "Dynamic Centrifuge Testing of a Cantilever Retaining Wall", J. of Earthq. Eng. and Struct. Dyn., Vol. 11, pp. 251-268.

Ostadan, F. (1983). Dynamic Analysis of Soil-Pile-Structure Systems, Ph.D. Dissertation, University of California, Berkeley CA, March.

Prevost J.H. (1981). DYNA-FLOW: A Nonlinear Transient Finite Element Analysis Program, Report 81-SM-1, Princeton University, Dept. of Civil Engineering, Princeton NJ.

Pyke, R.M. (1979). "Nonlinear Soil Models for Irregular Cyclic Loading", J. of Geotech. Eng., ASCE, Vol 11, No. 6, pp. 715-726, June.

Reissner, E. (1936). "Stationare Axialsymmetrische durch eine Schuttelnde Masse erregte Scheingungen eines Homogenen Elastischen Halbraumes", Ingenieur-Archiv, Vol. 7, Pt. 6, Dec.

Richart, F.E., Jr. et al (1970). Vibrations of Soils and Foundations, Prentice-Hall, Inc., Englewood Cliffs NJ.

Roesset, J. M. (1980a). "Stiffness and Damping Coefficients in Foundations", Dynamic Response of Pile Foundations (M. O'Neil and R. Dobry eds.), ASCE, New York NY, pp. 1-30.

Roesset, J.M. (1980b). "The Use of Simple Models in Soil-Structure Interaction", Civ. Eng. and Nuc. Pwr., ASCE 10/3, ASCE, New York NY, pp.1-25.

Roesset, J.M. (1989). "Perspectives on Soil Structure Interaction Analysis", Proceedings: EPRI/NRC/TPC Workshop on Seismic Soil-Structure Interaction Analysis Techniques using Data from Lotun, Taiwan, Vol. 1, pp. 1-1 to 1-14, Mar.

Roesset, J.M. et al (1986). "Dynamic Response of Vertically Loaded Small-Scale Piles in Sand", Proc. of Eighth Europ. Conf. on Earthq. Eng., Vol. 2, pp. 5.6/65-72.

Rojahn, C. and Mork, P.N. (1988). "An Analysis of Strong Motion Data from a Severely Damaged Structure - The Imperial County Services Building, El Centro, California", The Imperial Valley Earthquake of October 15, 1979, Prof. Paper 1254, USGS, Menlo Park CA.

Roth, W.H., et al (1986). "Nonlinear Dynamic Analysis of a Centrifuge Model Embankment", Proc. of Third U.S. Nat. Conf. on Earthq. Eng., Vol. 1, Charleston, S.C., pp. 505-516, Aug. 24-26.

Roth, W.H. et al (1991). "Pleasant Valley Dam: An Approach to Quantifying the Effects of Foundation Interaction", Proc. of Seventeenth Int. Cong. on Large Dams, Vienna, Austria, pp. 1199-1228.

Schofield, A.N. and Steedman, R.S. (1988). "State-of-the-Art Report -- Recent Development on Dynamic Model Testing in Geotechnical Engineering", Proc. of Ninth World Conf. on Earthq. Eng., Vol. 8, Tokyo-Kyoto, Japan, pp. 813-824, Aug 2-9.

Scott, R.F. (1990). "Modeling of Earth Structures", Proc. of Workshop on Seismic Safety Planning for Port of Los Angeles, Vol. 3, Los Angeles CA, March.

Stokoe, K.H. and Erden, S.M (1985). Influence of Base Shape on Dynamic Response of Surface Foundations, Geotech. Eng. Report GP85-1, Civ. Eng. Dept., Univ. of Texas, Austin TX.

Tabatabaie, M. (1982). The Flexible Volume Method for Dynamic Soil-Structure Interaction, Ph.D. Dissertation, University of California, Berkeley CA, Feb.

Tajimi, H. (1980). "State-of-the-Art Report: Seismic Observations and Vibration Tests for Verification of SSI Analysis Methods", Proc. of Ninth World Conf. on Earthq. Eng., Vol. 8, pp. 297-308, Tokyo-Kyoto, Japan, Aug. 2-9.

Tamori, S. and Kitagawa, Y. (1988). "Shaking Table Tests of Elasto-Plastic Soil-Pile-Building Interaction System" Proc. of Ninth World Conf. on Earthq. Eng. Vol. 8, Tokyo-Kyoto, Japan pp. 8-843 to 8-848.

Tang, H.T. et al (1991). "The Hualien Large-Scale Seismic Test for Soil-Structure Interaction Research", Trans. of the 11th Int. Conf. on Struct. Mech. in Reactor Technology, Vol K1, pp. 69-74, Tokyo, Japan, Aug. 18-23.

Tatabaie, M. et al (1986). "Effects of Seismic Wave Inclination on Structural Response" Dynamics of Structures ASCE, New York NY, March-April.

Udaka, T. et al(1981). "Soil-Structure Analysis for Varying Seismic Environments and Boundary Conditions" Trans. of Sixth Int. Conf. on Struct. Mech. in Reactor Tech., Paper K3/1, Vol K(a), Paris, France, Aug. 17-21.

Vaughn, D.K. and Isenberg, J. (1983). "Non-Linear Rocking Response of Model Containment Structures", J. of Earthq. Eng. and Struct. Dyn., Vol. 11, pp. 275-296.

Veletsos, A.S. (1988). "State-of-the-Art Report: Design Approaches for Soil-Structure Interaction" Proc. of Ninth World Conf. on Earthq. Eng., Vol. 8, Tokyo-Kyoto, Japan, pp. 8-341 to 8-352, Aug.

Weissman, K. and Prevost, J.E. (1989). "Centrifugal Modeling of Dynamic Soil-Structure Interaction", J. of Earthq. Eng. and Struct. Dyn., Vol. 18, pp. 1145-1161, Nov.

Werner, S.D. (1976). Seismic Soil-Structure Interaction Guidelines, Vol. II, State of the Art Procedures, SAN/1011-111, Energy Research and Development Administration, Washington D.C., April.

Werner, S.D., et al (1987). "Seismic Response Evaluation of Meloland Road Overcrossing using 1979 Imperial Valley Earthquake Records", J. of Earthq. Eng. and Struct. Dyn., Vol. 15, pp. 249-274, Feb.

Werner, S.D., et al (1990). "Dynamic Tests and Seismic Excitation of a Bridge Structure", Proc. of Fourth U.S. Nat. Conf. on Earthq. Eng., Palm Springs CA, Vol 1, pp. 1037-1046, May.

Werner, S.D. et al (1992). Assessment of UBC Seismic Design Provisions Using System Identification of Recorded Building Earthquake Motions, report to National Science Foundation under Grant No. BCS-9011136, Dames & Moore, Oakland, CA, Jan.

Wilson, J.C. (1988). "Stiffness of Non-Skew Monolithic Bridge Abutments for Seismic Analysis", J. of Earthq. Eng. and Struct. Dyn., Vol. 16, pp. 867-883, Aug.

Wolf, J. P. (1985). Dynamic Soil-Structure Interaction, Prentice-Hall, Inc., Englewood Cliffs NJ.

Wolf, J.P and Darbe, G.R (1986). "Nonlinear Soil-Structure Interaction Analysis based on Boundary Element Method in Time Domain with Application to Embedded Foundation", J. of Earthq. Eng. and Struct. Dyn., Vol. 14, pp.83-101, Jan.-Feb.

Wolf J.P. and Darbe, G.R. (1984). "Dynamic Stiffness Matrix of Soil by the Boundary Element Method", J. of Earthq. Eng. and Struct. Dyn. Vol. 12, pp. 385-400.

Wolf, J.P. and von Arx, G.A. (1982). "Horizontally Traveling Waves in a Group of Piles Taking Pile-Soil-Pile Interaction into Account" J. of Earthq. Eng. and Strut. Dyn., Vol. 10, No. 2, pp. 225-237.

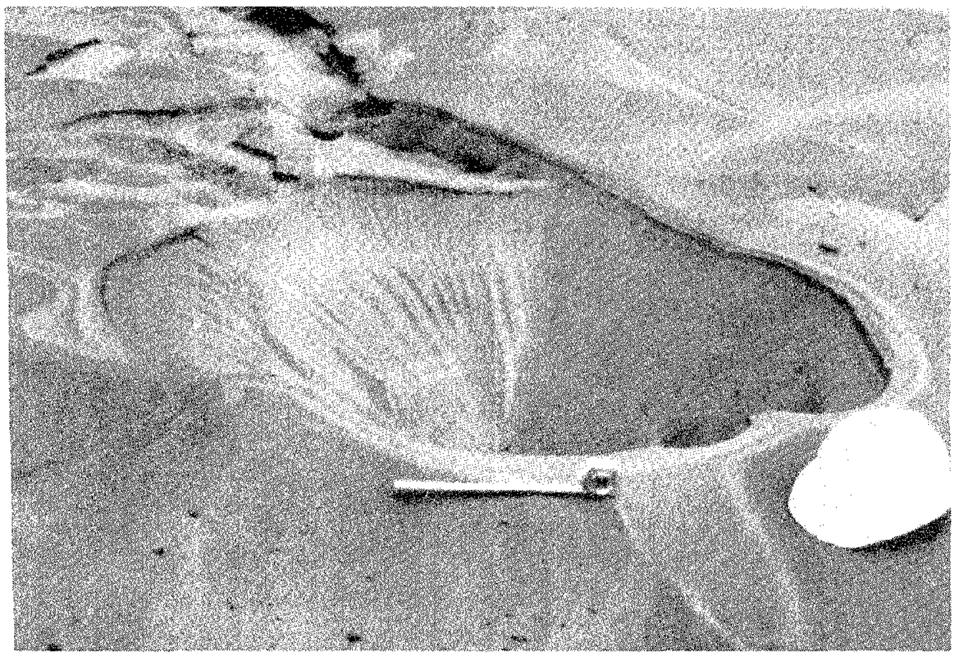
Wong, H.L. et al (1988). "A Comparison of Soil-Structure Interaction Calculations with Results of Full-Scale Forced Vibration Tests", J. of Soil Dyn. and Earthq. Eng., Vol. 18, No. 1, pp. 22-31.

Yim, S. and Chopra, A.K. (1985). "Simplified Earthquake Analysis of Multistory Structures with Foundation Uplift", J. of Struct. Eng. Div., ASCE, Vol. 111, No. 12, pp. 2708-2731, Dec.

Youd, T.L. (1991). Full-Scale Field Tests at Sites Subject to Earthquake Shaking, prepared for National Science Foundation Workshop on Experimental Needs in Geotechnical Earthquake Engineering, Albuquerque NM, Nov. 4-5.

## 2.3 GROUND MOTIONS: EXPERIMENTAL RESEARCH NEEDS SUMMARY

Geoffrey R. Martin
University of Southern California
Los Angeles, California



Massive Sand Boil at Oakland International Airport in Loma Prieta Earthquake (Photograph by University of California, Berkeley, Geotechnical Engineering Group) (Ref. 5, fig. 4.18, pg. 97, reprinted by permission of Earthquake Engineering Research Institute).

# GROUND MOTION: EXPERIMENTAL RESEARCH NEEDS SUMMARY

by Geoffrey R. Martin
University of Southern California

## INTRODUCTION

The determination of earthquake ground motion parameters for seismic design is now recognized by most engineers as a critical component of the seismic design process. The level of complexity of ground motion characterization depends on the importance or criticality of the particular facility being designed or evaluated. For example, for routine design of smaller building structures, ground motions are normally defined by code requirements. For more important structures, such as power plants, dams, major bridges, or port and harbor facilities, site specific earthquake ground motion parameters are normally selected to reflect levels of seismicity associated with the accepted risk and local site soil characteristics. For these cases, ground motion input would normally be defined by acceleration design spectra, together with representative time histories for dynamic response analyses. The effects of soil conditions on site response would normally be evaluated using one-dimensional wave propagation considerations. A third level of evaluation normally considered to be in the realm of research as opposed to engineering practice, would require consideration of two- or three-dimensional effects arising from the nature of geologic boundaries (for example, bedrock defining a valley and associated stratigraphy of alluvial soils), and input wave motion characteristics.

Clearly, experimental research needs would be seen differently by people practicing in the three modes defined above. Engineers using a building code approach would probably see the need for a better definition of the code soil factors used to determine design spectral shapes. On the other hand, engineers regularly using one-dimensional site response analyses on a site specific basis, probably see the need for improved *in situ* characterizations of shear modulus and damping ratios and a better definition of site conditions where one-dimensional analyses may not be appropriate. For those seismologists working in the area of two- and three-dimensional analyses of the effects of site conditions on ground motions, the greatest need may be related to a clearer understanding of the effects of nonlinearity and under what conditions nonlinear soil behavior becomes significant.

Clearly, the research needs of the three groups are strongly interrelated. An improved understanding of two-dimensional effects, for example, provides the means for identifying the applicability of one-dimensional approaches. Improved one-dimensional evaluations, in turn, provide the needed refinements to code approaches for characterizing ground motions.

The increasing recognition of the importance of good ground motion characterization for earthquake-resistant design is reflected by the many workshops and conferences in recent years where this subject has been highlighted. Of particular note are:

- The NSF/EPRI Workshop on Dynamic Soil Properties and Site Characterization held in 1989. This workshop was based on the premise that earthquake experience has shown that site geology and local soil properties exercise a decisive influence on seismic ground motions and structural damage potential. The objectives of the workshop were to discuss the current state of dynamic soil property measurement and site characterization, to explore ways to achieve necessary advances, and identify research priorities.

- A series of three workshops (1989, 1990, 1991) on ground motion parameters for seismic hazard mapping sponsored by the National Center for Earthquake Engineering Research. These workshops focused on the choice of ground motion parameters to be utilized for maps and codes related to earthquake-resistant design of buildings.

- The Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, St. Louis, March 1991.

- The Fourth International Conference on Seismic Zonation held at Stanford in 1991. As the definition of ground motion parameters plays a critical role in the concept of seismic zonation, many significant contributions aimed at developing an improved understanding of the nature of earthquake ground motions, were presented at this conference.

The encouraging feature of these workshops and conferences was the increased level of cooperation and understanding being developed between geotechnical engineers concerned with local site soil response, and seismologists interested in strong ground motion characteristics. A one day workshop between seismologists and geotechnical engineers to discuss nonlinear site response was recently sponsored by the Southern California Earthquake Center. A consensus is also gradually being reached between the two disciplines, as to experimental research needs to better define site specific earthquake ground motions for design. Such cooperation is critical to ultimate success.

Background comments on ground motion characteristics expressed in the following paragraphs are drawn largely from the above workshops, together with discussions with colleagues during meetings held at the National Center for Earthquake Engineering and the Southern California Earthquake Center. The NSF/EPRI Workshop was attended by about seventy specialists with expertise in the fields of engineering and the earth sciences. It resulted in the publication of a very well-documented Proceedings containing several major recommendations for research. Comments on research needs and priorities given in this summary draw heavily from this workshop. The consensus reached at the NSF/EPRI meeting forms a solid foundation for the development of recommendations at this Workshop in relation to earthquake ground motions.

## BACKGROUND: GROUND RESPONSE CHARACTERISTICS

### 1. CODES

The parameters defining ground motion in the seismic provisions of most codes are normally keyed to a particular parameter (for example, effective peak ground accelerations, or more recently, spectral ordinates at 0.3 and 1 second periods) appropriate for a reference site condition (for example, rock or deep alluvial soils). The effects of local soil conditions are normally taken into account by the introduction of a site coefficient S related to a number of broad categories of soil conditions. These site coefficients in turn modify spectral ordinates for design, as compared to those spectral ordinates for the reference site condition. The selection of these factors has been historically based on average ground motion spectra for recorded ground motions complemented by the results of one-dimensional analyses. Figure 1 shows site coefficients recommended by NEHRP (1988), while Figure 2 shows soil coefficients and categories adopted by the New York City building codes. The hard basement rock characterizing the Northeastern U.S. leads to higher site amplification factors because of the increased soil-rock impedance contrast.

Definitions of site soil factors and their use in revised building codes planned in 1994 (National Earthquake Hazard Reduction Program) are presently under review by several committees. Acceleration response spectra associated with recorded ground accelerations have clearly showed the need to incorporate the effects of local soil conditions on ground motions adopted for design. The experience during the Mexico City and Loma Prieta earthquakes, where significant amplifications of ground accelerations were recorded on soil sites (with associated amplification of spectral peaks at longer periods), has emphasized the need for site coefficients. However, the means of doing this within the framework of a code is still clearly a major issue. Guidance in this effort may be obtained through more detailed analysis of recorded earthquakes, coupled with improved analytical modelling, both one- and two-dimensional. Recent papers by Finn (1991) and Jacob (1991) address several of the issues related to this problem.

## 2. ONE-DIMENSIONAL SITE RESPONSE

The effects of site-specific soil conditions on earthquake waves propagating vertically from bedrock or underlying firm ground are often evaluated by 1-D shear beam analyses. These analyses assume that ground motions can be modelled by horizontal shear waves propagating vertically through horizontally soil stratigraphy. Methods of analysis are able to model nonlinear soil behavior and can identify the influence of site conditions in modifying the amplitude and period of maximum spectral ordinates and the effects of ground shaking intensity on the amplification or de-amplification of ground accelerations.

The predicted effects from nonlinear one-dimensional analyses have been identified in ground motions recorded during earthquakes. Evidence of a significant shift in site period arising from weak versus strong shaking is seen in data from Japanese earthquakes (Finn, 1991) reflecting the effects of nonlinear soil behavior. The reductions in spectral amplification factors for strong motions when compared to weak motions observed at many soil sites (for example, Treasure Island, Jarpe et. al, 1989) have also suggested the significance of nonlinear effects and the need for caution when identifying site periods associated with peak response deduced from low amplitude events, such as microtremors.

Following the introduction of the computer program SHAKE (Schnabel et. al., 1972), one-dimensional nonlinear site response analyses have been routinely performed by geotechnical engineers. Whereas the effects of nonlinearity on ground motion response has been the subject of considerable debate between seismologists and geotechnical engineers over the years, since the Loma Prieta earthquake, seismologists have taken an increased interest in both the potential influence of nonlinearity on recorded accelerograms and the value of one-dimensional analyses. Chin and Aki (1991), in a study of ground motions recorded during the Loma Prieta earthquake, report on a pervasive nonlinear site effect at sediment sites in the epicentral area, following analyses to remove the potential effects of earthquake source and propagation paths.

One major uncertainty in one-dimensional nonlinear analyses relates to the values of soil parameters required by the analysis methods. Changes in equivalent linear shear modulus with shearing strain amplitude are normally based on laboratory determined relationships between shear modulus and shear strain amplitude normalized by *in situ* geophysical measurements of low strain shear modulus. However, because of disturbance effects in soil sampling, considerable uncertainty exists as to actual large strain shear modulus values in the field, particularly for alluvial deposits. Similarly, damping values during earthquake-induced large shearing strains *in situ* remain uncertain in relation to values determined in the laboratory. Self boring pressure meter systems capable of large strain cyclic loading are seen as a promising means to obtain improved *in situ* data for both modulus and damping. However, in discussing this topic at the NSF/EPRI Workshop, considerable improvements in equipment design were recommended in order to perform this task with some degree of confidence.

Another major problem in determining site specific amplification factors or spectral ratios using 1-D analyses is the uncertainty as to input motions when comparing field measurements with calculations, as site response depends on both the intensity of ground motion input, and the frequency content. The evaluation of appropriate input motions is difficult. Whereas rock outcrop motions adjacent to a soil site are usually considered appropriate, Finn (1991) notes that studies of ground motions at the SCT strong motion site on the lake bed in Mexico City during the 1985 earthquake could not be simulated by using the rock outcrop motions in the University District as input motions. The rock motions had no preferred direction, whereas the observed motions at the SCT site on the lake bed had acquired a strong East-West orientation, possibly due to surface waves being propagated in the East-West direction by a local subsurface topography or lateral inhomogeneities. Finn makes similar observations in relation to computations of acceleration spectra for the Treasure Island site, using motions recorded at the Yerba Buena site from the Loma Prieta earthquake as input. Whereas one-dimensional analyses gave reasonable results in the case of the East-West direction, relatively poor correlations between observed and recorded spectra were obtained for the North-South direction, possibly reflecting the fact that topographic effects due to subsurface structure associated with the contact between rock outcrops and sediments can alter motions significantly.

The effects of site geometry and global characteristics on local site response was discussed extensively at the NSF/EPRI Workshop. Consideration of ground motions in alluvial valleys allows an assessment of the patterns in response from a vertically propagating plane shear wave assumption. The main effect of curvature of the sediment-basin interface is the generation of surface waves, as well as trapped body waves which propagate and superpose with the vertically propagating shear waves. This may result in an amplification of motions, as well as increased duration over 1-D soil effects alone.

A summary of 2-D effects presented at the Workshop is shown in the attached table. Heavily instrumented, experimental arrays at sites likely to experience a strong earthquake in the near future are seen as the best means of addressing many of the questions posed above. The effectiveness of this approach has been demonstrated by results obtained from the Lotung downhole ground motion array in Taiwan. Analysis of results obtained during a strong earthquake demonstrated that ground motion data obtained from closely spaced instruments can be effectively utilized to both infer *in situ* dynamic soil properties and variations of properties with levels of ground shaking (Chang et. al. 1991).

## 3. TWO- AND THREE-DIMENSIONAL SITE RESPONSE

With the increased power of small computers, there has been a recent upsurge in the number and complexity of two-dimensional site response analyses being performed. The seismological approach to this problem is to start with a model of the source at the fault break, propagate elastic waves between the source and the site with a two- or three-dimensional site (alluvial valley) characterized as an elastic medium. Aki and Irikura(1991) describe a number of techniques used for these types of analyses. However, they are normally restricted to computing displacement amplitudes at the site, where the interest is in the long period range. Such approaches have yet to be of value in the higher frequency ranges generating the peak ground accelerations of interest to engineers.

The engineering approach to two-dimensional modelling normally assumes plane waves (SH, SV or P) arriving at the interface between bedrock and an alluvial valley either vertically or at some angle of incidence. Valley stratigraphy may be modelled either as elastic and wave propagation solutions developed, or alternatively modelled by linear or nonlinear finite elements and numerical procedures used to compute site response. Results from such analyses have clearly indicated the importance of two-dimensional effects in many instances. Modaressi (1991) describes a finite element approach for prediction of two-dimensional site effects for the alluvial valley located at Turkey Flat in California. The model is capable of both elastic and nonlinear behavior and reasonable comparisons were obtained between computed and recorded accelerations at the site. Papageorgiou and Kim (1991) describe a method of analysis for the response of sediment-filled valleys arising from incident SV waves and SH waves. A study of the earthquake response of the Caracas Valley using a 2-D model and vertically propagating SV and SH waves showed important differences when compared to each other, while for the 1-D model there is no difference in response between SV and SH waves when the angle of incidence is vertical. A study of oblique incidence of SH and SV waves showed localized site amplification consistent with the localized damage observed during the Caracas earthquake. Recent studies by Bardet et. al (1991) on the two-dimensional dynamic response of the Marina District during the Loma Prieta earthquake, also demonstrated the significance of two-dimensional effects when compared to 1-D modelling. In this case, the two-dimensional effects resulted mainly from the irregular geometry of the bedrock and the hardpan layer separating the recent bay muds from the older bay mud deposits. Localized ground motion amplifications significantly greater than those predicted by 1-D analyses occurred when the site was subjected to vertical or inclined SV and P waves.

The results from these and similar studies clearly indicate the need for an improved understanding of the implications of two-dimensional response on earthquake ground motions. As suggested above, closely spaced arrays at selected sites appear the best means to obtain and validate this understanding. A number of strong motion arrays are presently operating in the United States, Japan, Taiwan, Mexico, and People's Republic of China. They typically involve accelerometers or seismometers located on or below the ground surface in a seismically active area. Some are oriented towards seismological research and others to more practical engineering issues. Several are instrumented with pore pressure transducers to monitor pore pressure increases in liquefiable soils. A good summary of these arrays and their characteristics was provided in the NSF/EPRI Workshop.

## 4. LIQUEFACTION

For the case of saturated, cohesionless soils, the effects of pore pressure buildup during ground shaking leading to potential liquefaction are of particular interest, both with respect to potential site stability problems and with respect to the effects of pore pressure buildup on recorded ground motions. Whereas there is a continuing need to improve methods for determining *in situ* soil parameters necessary for site response analyses in terms of effective stress, there is perhaps an even greater need for experimental studies leading to a better understanding of the post-liquefaction deformation behavior of cohesionless soil sites when subjected to continued ground shaking, particularly in the case of slightly sloping sites where potential large ground deformations or flow failures may occur. Pore pressure and deformation data obtained from instrumented sites can provide valuable data to enhance our understanding of post-liquefaction ground motions. Data recorded at the Wildlife Site in the Imperial Valley, California during a recent earthquake provided good insight as to pore pressure buildup effects and associated permanent ground deformations. Piezometer arrays with associated accelerometers have also been installed at the Parkfield Site in California and a site in San Bernadino.

Due to piezometer installation difficulties and time delays in waiting for earthquakes, increased attention has been given to centrifuge testing as a viable option to study the effects of liquefaction on ground response. With the development of centrifuge shakers, it is now possible to study the effects of simulated earthquakes on saturated cohesionless soils under controlled conditions over a very short period of time. The VELACS (Verification of Liquefaction by Centrifuge Studies) Project is an NSF funded coordinated geotechnical centrifuge study of simulated earthquake loading of a variety of different models in order to study the mechanisms of liquefaction induced failure and to acquire data of the verification of the various analysis procedures for liquefaction problems. The collaborating universities are University of California, Davis; University of California, Berkeley; California Institute of Technology, University of Colorado, Boulder; Massachusetts Institute of Technology, Rensselaer Polytechnic Institute, Princeton University, and Cambridge University. Numerous tests have already been performed and a program of analytical studies is about to commence on a variety of models, with predictions being made prior to the specific tests being performed. Whereas this program will provide greater insight into the mechanisms of post-liquefaction earthquake behavior and validate and improve numerical procedures, the challenge still remains to characterize *in situ* soil properties related to the analytical parameters needed for design related seismic response studies.

The centrifuge may also be effectively used to study the effects of ground remediation (such as vibro compaction or dynamic compaction) as a means of minimizing excess pore pressure buildup during earthquake ground shaking. The potential for remediated ground to attract higher acceleration levels than adjacent unremediated ground is of interest, and is usually not addressed when evaluating compaction criteria for ground remediation. Research needs associated with liquefaction problems and associated ground remediation techniques were recently discussed at an NSF sponsored workshop at the University of Washington, Seattle, and recommendations for future research in this area will shortly be published.

## RESEARCH NEEDS

A good starting point in relation to the development of experimental research needs for earthquake ground motion characterization may be found in the recommendations arising from the NSF/EPRI Workshop. The organizing committee for this workshop summarized research needs in relation to site characterization as follows:

- Development and operation of field test sites in seismically active areas

- Technology developments in the areas of *in situ* testing, laboratory testing, and ground response monitoring

- Fundamental studies of physical-chemical processes

- Sensitivity studies to evaluate the importance of dynamic soil property variation

- Analytical studies to develop improved data processing methods, laboratory and field data interpretation techniques, and ground-response modeling procedures

The first two items on this list relate more closely to the objectives of this workshop. In particular, the development and operation of test sites involving instrumental arrays located in areas of high earthquake probability generally receive very strong support. Recommendations at the NSF/EPRI Workshop, in relation to test sites, were summarized as follows:

- They should be tied in with new or existing strong motion seismic arrays to optimize planning, management, and maintenance.

- The sites should be located on loose, saturated, granular soil and soft clay to obtain data for a range of geologic conditions.

- The geometry should be well defined, and the test site area should be large enough to allow numerous experiments to be carried out.

- Soil sites should include sloping ground and have nearby rock outcrops.

- If possible, information on site characterization should be already available.

- The sites either should be available for non-earthquake-related studies or may be part of other projects involving earthquake or non-earthquake activities.

In relation to ground response monitoring at such sites, the recommendations focused heavily on new technology developments as follows:

- Cyclic and permanent strain and deformation measurements

- Systems to monitor cyclic stress-strain response

- Six-component accelerometers

- Continuous monitoring of $G_{max}$ during seismic events

- Pore pressure devices

- User friendly means to store, document and retrieve strong motion data

*In situ* testing techniques to determine soil properties, for both evaluation of collected array data and for earthquake resistant design of new projects, is clearly a high priority. Technology developments in this area recommended at the NSF/EPRI Workshop were as follows:

- Nondestructive, nonintrusive (geophysical, electrical, seismic, radar, etc.) procedures for delineating subsurface stratigraphy in a rapid and accurate manner

- Nonlinear cyclic deformation and degradation characterization (stress-strain, volumetric change)

- Material damping and its variation with level of shearing strain

- *In situ* density variation and/or *in situ* measurement of steady state strength ($S_{us}$) in saturated, loose sand

- Standardization of *in situ* testing methods

The use of the centrifuge for earthquake simulation experiments on the response of soil sites should also be emphasized as a developing valuable experimental tool, particularly for verification of analytical models and one-dimensional experiments for pre-earthquake simulation of the response of array sites.

Refining and prioritizing the above recommendations, in terms of the objectives of this workshop, would seem a worthy goal.

## REFERENCES


Aki, K. and Irikura, K., "Characterization and Mapping of Earthquake Shaking for Seismic Zonation," Proceedings, Fourth International Conference on Seismic Zonation, Stanford, 1991, Vol. 1, pp. 61-110.

Bardet, J.P., Kapuskar, M., Martin, G.R., and Proubet, J., "An Assessment of the Dynamic Response of the Marina District of San Francisco During the Loma Prieta Earthquake," Report to the National Science Foundation, Department of Civil Engineering, University of Southern California, September 1991.

Chang, C.Y., Mok, C.M., Power, M.S., Tang, Y.K., Tang, H.T., and Stepp, J.C., "Development of Shear Modulus Reduction Curves Based on Lotung Downhole Ground Motion Data," Proceedings, Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, St. Louis, March 1991, Vol. 1, pp. 111-118.

Chin, Byau-Heng and Aki, K., "Simultaneous Determination of Source, Path and Recording Site Effects on Strong Ground Motion During the Loma Prieta Earthquake - A Preliminary Result on Pervasive Non-Linear Site Effect," submitted to Bull. Seis. Soc. Am., 1991.

Finn, W.D.L., "Geotechnical Engineering Aspects of Microzonation," Proceedings, Fourth International Conference on Seismic Zonation, Stanford, 1991, Vol. 1, pp. 199-260.

Jacob, Klaus H., "Seismic Zonation and Site Response: Are Building-Code Soil-Factors Adequate to Account for Variability of Site Conditions Across the U.S.?," Proceedings, Fourth International Conference on Seismic Zonation, Stanford, 1991, Vol. 2, pp. 695-702.

Jarpe, S., Hutchings, L., Hank. T., and Shakal, A., "Selected Strong and Weak Motion Data from the Loma Prieta Earthquake Sequence," Seismol. Res. Letters, 60, pp. 167-176, 1989.

Modaressi, H., Bour, M. and Aubry, D., "A Finite Element Approach for Prediction of Site Effects," Proceedings, Fourth International Conference on Seismic Zonation, Stanford, 1951, Vol. 2, pp. 261-276.

Papageorgiou, A.S. and Kim, J., "Oblique Incidence of SV-Waves on Sediment-Filled Valleys: Implications for Seismic Zonation," Proceedings, Fourth International Conference on Seismic Zonation, Stanford, 1991, Vol. 2, pp. 581-596.

Proceedings: NSF/EPRI Workshop on Dynamic Soil Properties and Site Characterization, June, 1991, Vol. 1, Electric Power Research Institute, Report NP-7337, Project 810-4.

Proceedings: NSF Workshop on Soil Improvement and Foundation Remediation with Emphasis on Seismic Hazards, August, 1991, University of Washington, Report to be Published 1992.

Schnabel, P.B., Lysmer, J., and Seed, H.B., "SHAKE: A Computer Program for Earthquake Response Analysis of Horizontally Layered Sites," Report No. EERC 72-12, University of California, Berkeley, December 1972.

- Soil Profile Type $S_1$--Rock of any characteristic, either shale-like or crystalline in nature (such material may be characterized by a shear wave velocity greater than 2,500 feet per second), or stiff soil conditions where the soil depth is less than 200 feet and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.

- Soil Profile Type $S_2$--Deep cohesionless or stiff clay soil conditions, including sites where the soil depth exceeds 200 feet and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.

- Soil Profile Type $S_3$--Soft-to-medium stiff clays and sands characterized by 30 feet or more of soft- to medium-stiff clay with or without intervening layers of sand or other cohesionless soils.

- Soil Profile Type $S_4$--Soft clays or silts greater than 70 feet in depth and characterized by a shear wave velocity of less than 400 feet per second.

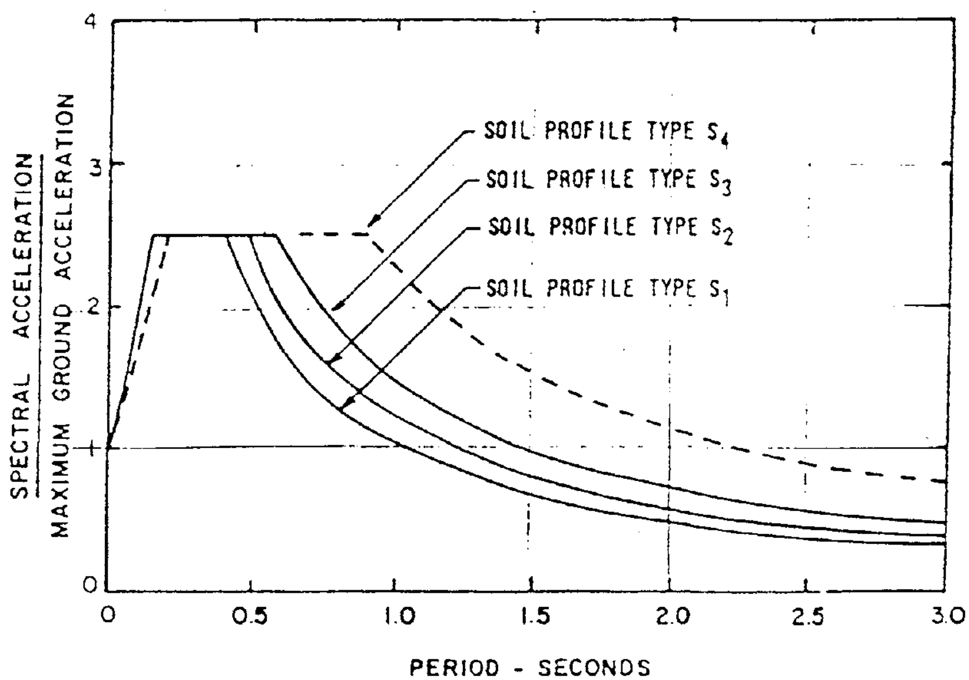



**FIGURE C1-9**
**Normalized response spectra recommended for use in building codes.**

FIGURE 1 Site Coefficients, NEHRP 1988

Table A-1: Site Coefficients (Currently Proposed Version for the NYCBC)

| Type | Description | S-Factor |
|---|---|---|
| So: | A profile of Rock materials of class 1-65 to 3-65 | **2/3** |
| S1: | A soil profile with either:<br><br>(a) Soft Rock (4-65) or Hardpan (5-65) or similar material characterized by shear wave velocities greater than 2500 fps, or<br><br>(b) Medium Compact to Compact Sands (7-65) and Gravels (6-65) or Hard Clays (9-65), where the soil depth is less than 100 feet. | **1.0** |
| S2: | A soil profile with Medium Compact to Compact Sands (7-65) and Gravels (6-65) or Hard Clays (9-65), where the soil depth exceeds 100 feet. | **1.2** |
| S3 | A total depth of overburden of 75 feet or more and containing:<br><br>more than 20 feet of Soft to Medium Clays (9-65) or Loose Sands (7-65, 8-65) and Silts (10-65),<br><br>but not more than 40 feet of Soft Clay or Loose Sands and Silts. | **1.5** |
| S4: | A soil profile containing more than 40 feet of Soft Clays (9-65) or Loose Sands (7-65, 8-65), Silts (10-65) or Uncontrolled Fills (11-65), where the shear-wave velocity is less than 500 feet per second. | **2.5** |

**Notes to Table A-1**

1. The site S Type and corresponding S Factor shall be established from properly substantiated geotechnical data, with the classes of materials being defined in accordance with the appropriate sections of the Administrative Code of the City of New York.

2. The soil profile considered in determining the S Type shall be the soil on which the structure foundations bear or in which pile caps are embedded and all underlying soil materials.

3. Soil density / consistency referred to in the table should be based on standard penetration test blow counts (N-values) and taken as:

| | |
|---|---|
| (a) for sands, | loose - where N is less than 10 blows per foot,<br>medium compact - where N is between 10 and 30, and<br>compact - where N is greater than 30 blows per foot, and |
| (b) for clays, | soft - where N is less than 4 blows per foot<br>medium - where N is between 4 and 8,<br>stiff to very stiff - where N is between 8 and 30, and<br>hard - where N is greater than 30 blows per foot. |

4. When determining the type of soil profile for profile descritpions that fall somewhere in between those categories that are provided in the above table, the S Type with the larger S Factor shall be used.

5. For Loose Sands, Silts or Uncontrolled Fills below the ground water table the potential for liquefaction shall be evaluated by the pertinent provisions of the code.

FIGURE 2 Site Coefficients, Proposed for the New York City Building Code

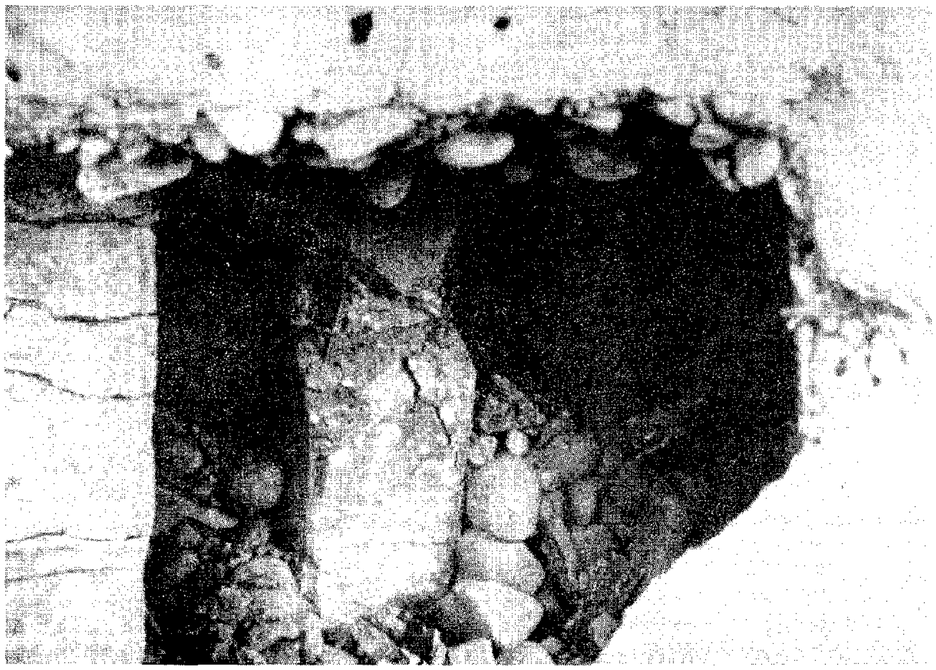
**Table**
**2-DIMENSIONAL GEOLOGIC STRUCTURAL EFFECTS INFLUENCE MATRIX**

| Structure | Conditions | Type | Size | Quantitative Predictability[a] |
|---|---|---|---|---|
| Surface Topography | Sensitive to shape ratio, largest for ratio between 0.2 to 0.6. Most pronounced when wavelength mountain width | Amplification at top of structure amplification and deamplification at base, rapid changes in amplitude phase along slopes | Ranges up to a factor of 30 but generally from about two to 10 | Poor: generally underpredict size. May be because of ridge interaction and 3-D effects |
| Sediment-Filled Valleys | | | | |
| 1) Shallow and wide (shape ratio <0.25) | Effects most pronounced near edges. Largely vertically propagating shear waves from edges. | Broad band amplification across valley because of whole valley modes | 1-D models may underpredict at higher frequencies by about two near edges | Good: away from edges 1-D works well, near edges extend 1-D amplifications to higher frequencies |
| 2) Deep and narrow (shape ratio $\geq$0.25) | Effects throughout valley width | Broad band amplification across valley because of whole valley modes | 1-D models may underpredict for a wide bandwidth by about two to four away from edges. Resonant frequencies shifted from 1-D. | Fair: given detailed description of vertical and lateral changes in material properties |
| 3) General | Local changes in shallow sediment thickness | Increased duration | Duration of significant motions can be doubled | Fair |
| 4) General | Generation of long period surface waves from body waves at shallow incidence angles | Increased amplification and duration because of trapped surface waves | Duration and amplification of significant motions may be increased over 1-D predictions | Good at periods exceeding 1 second |

[a] Good: generally within a factor of two.
Fair: generally within a factor of two to four.
Poor: qualitative only, can easily be off by an order of magnitude.

## 2.4 FULL-SCALE TESTS AT SITES SUBJECT TO EARTHQUAKE SHAKING

T. Leslie Youd
Brigham Young University
Provo, Utah



Rio Banano Bridge: Pile Cap and Piles of the South Abutment Showing Void Created by Settlement and Slumping of Soils in 1991 Costa Rica Earthquake (Ref. 7, fig. 6-20, pg. 73, reprinted by permission of Earthquake Engineering Research Institute).



Preceding page blank

## INTRODUCTION

Observed behavior at well-documented sites has been a major factor in the development of geotechnical earthquake engineering over the past few decades. For example, the development of criteria for assessing liquefaction hazard has developed primarily from correlations between measured soil properties and site conditions and observed effects of liquefaction (or lack of liquefaction) in the field. Likewise, field evidence and records of amplified ground response at soft soil sites during many past earthquakes have shown the nature of ground motion amplification and have led to development of both empirical and analytical procedures for estimating site response. Future developments in geotechnical earthquake engineering will require additional and more sophisticated measurement of field and prototype behavior during future earthquakes.

This paper reviews the general need for field observation in geotechnical earthquake engineering, notes recent progress in instrumenting sites and collecting case histories of field performance, and discusses specific research needs and issues raised in previous research evaluations and by members of this workshop. The final conclusion is that more and more sophisticated observations and experiments are required to develop engineering criteria for safe and economical design of new structures and for accurately analyzing and strengthening existing structures.

## REVIEW OF PAST RECOMMENDATIONS

The need for specific field observations and experiments to expedite the development of geotechnical earthquake engineering has been considered by several panels of experts at workshops convened over the past 15 years. Pertinent excerpts from the recommendations of several past workshops are reproduced below to provide members of this workshop a brief synopsis of past thought and to provide continuity in the development research initiatives for field experiments.

### 1977 Workshop

One of the early workshops on research needs in geotechnical earthquake engineering convened in 1977 in Austin, Texas (Lee and others, 1978). That workshop was divided into seven panels to consider a wide variety of topics. With respect to full-scale field testing, all panels of this workshop stressed "the importance of field studies as the logical beginning, middle and end of a worthwhile program in geotechnical earthquake engineering research," and that "detailed field work can provide needed specific data for use in developing certain analysis and design models and techniques. Well-documented field data provide the ultimate reference basis for comparing the results which may be predicted by any proposed analysis or design procedure." This workshop further stressed the importance of (1) compiling case histories of field performance and (2) initiation of a program of instrumenting sites and structures in highly seismic areas to specifically monitor prototype behavior during earthquake shaking.

### 1983 Workshop

In 1983, a workshop was convened in Blacksburg, Virginia to consider "Research Needs in Experimental Soil Engineering" (Clough and Silver, 1983). One of the several panels of this workshop considered cumulative deformation under cyclic loading conditions and concluded that, "the most valuable information for the prediction of cumulative deformation from cyclic loads comes from full scale tests. This type of information can be obtained by investigating field performance and by instrumenting sites where cyclic loading is expected to occur." Specifically this group recommended that priority be given to (a) "investigations of prototype structures which deform significantly," and (b) "soil investigations at instrumented sites in localities of expected seismicity or storms, and encourage the development of more instrumented sites."

### 1985 Workshop

In 1985, the National Research Council convened a workshop in Dedham, Massachusetts to assess the state-of-the-knowledge and the state-of-the-art for engineering analysis and practice with respect to liquefaction hazard (NRC, 1985). As part of that workshop, research needs were assessed. The first-listed research need is as follows: "Instrumentation of a limited number of selected sites is needed in highly seismic regions, where there is a high probability that liquefaction will soon occur, and at saturated cohesionless sites where pore pressure is expected to increase without liquefaction occurring. The installation of field instrumentation (e.g., pore pressure transducers and recorders strong-motion accelerometers) at both types of sites should proceed as expeditiously as possible."

Another noted research item states: "Validation of the improved behavior of foundations and soil structures that have been treated to increase dynamic stability has become a major need. The number of case studies concerning the stability of natural deposits far exceeds field evidence of improved behavior of deposits that have been altered by drainage or in-situ soil improvement. Almost completely lacking are case histories involving sites or earth structures that have been improved and then subjected to earthquake shaking."

### 1986 Workshop

The research recommendations with respect to liquefaction noted at the 1985 National Research Council workshop were reiterated at a 1986 NSF-sponsored workshop entitled, "Siting and Geotechnical Program Focus and Direction (Saxena, 1986)." In addition to the restated recommendations on liquefaction, the 1986 workshop made four specific recommendations for field studies for both ground response and liquefaction:

(1) For verification of nonlinear soil dynamic response models, "a team of engineers and earth scientists should be formed to plan and carry out a detailed three-dimensional soil dynamic response experiment in the Parkfield region or some other appropriate location. All of the site data necessary for making ground motion predictions should be supplied to interested geotechnical engineers, and the results of analytical predictions compared with observations."

(2) For verification of non-linear soil dynamic response models, "a small number of dense three-dimensional arrays should be deployed in soft soil deposits in regions of expected very strong shaking. These arrays could be in the U.S. or some other country."

(3) For verification of liquefaction models, "a team of engineers and earth scientists should be formed to plan and carry out a detailed liquefaction experiment in the Parkfield region or some other appropriate location."

(4) For verification of other analytical/numerical models, "a team of engineers and earth scientists should be formed to plan and carry out local topography experiments using aftershocks of strong earthquakes."

**1988 Workshop**

In 1988, a workshop was convened in Durham, New Hampshire to consider the topic: "Designated Sites for Geotechnical Experimentation in the United States." The workshop made the following recommendation with respect to liquefaction (Benoit and de Alba, 1988): "To improve our understanding and ability to analyze the deformation process, research using both field and laboratory experimentation is required. Well-instrumented sites in areas likely to be shaken by earthquakes are essential to this research for providing real data for both process evaluation and analytical or empirical model verification. Specifically, it would be most desirable to monitor pore pressure and deformation development induced by actual earthquake shaking within a saturated medium-dense sand deposit subjected to a significant driving shear stress, i.e. in a slope."

With respect to ground motion response, the 1988 workshop report states (Benoit and de Alba, 1988): "How these [soft] soils will respond during strong earthquake shaking, however, is not fully understood, and there are disagreements among knowledgeable professionals concerning the level of ground motion amplification that should be expected. Unfortunately, we do not have strong motion records from soft soil sites in this country to resolve this issue, which probably can not be resolved without such measurements. Therefore, the instrumentation of a well defined site or sites is a high priority need to answer this critical question. To meet this need, development of instrumented, well documented experimental sites containing soft soils are required. These sites should be part of the national set of geotechnical experimentation sites, so that the many interested parties may access the compiled data and parameters, and may conduct additional tests to develop specialized data for individual needs."

**1989 Workshop**

In 1989, the National Science Foundation and the Electrical Power Research Institute jointly convened a workshop in Palo Alto, California, to consider the topic of "Dynamic Soil Properties and Site Characterization." With respect to test sites, the organizing committee and workshop panels unanimously adopted the following general conclusion (Anderson and Tang, 1991):

The topic 'Development and Operation of Test Sites' was identified by the organizing committee as being of special importance and requiring a more detailed discussion. It was selected for more detailed discussion because:

- The topic was identified by all panels as one that will contribute to significant advances in the state of the practice for site characterization and soil property measurement.

- Information obtained from the development and operation of test sites will be of use to multiple disciplines, including geologists, geophysicists, seismologists, and engineers.

- The program will complement efforts already underway to identify sites for geotechnical experimentation (Benoit and de Alba, 1988).

- Finally, development and operation of test sites would likely require industry wide support.

The last reason cited above is thought to be critical. Development and operation of tests sites will require significant planning, capital investment, and annual maintenance costs. It is unlikely that any single private organization or government agency presently has either the budget or staff to successfully operate the proposed test sites without significant contributions from other organizations. Consequently, development and operation of the test sites is expected to require an industry-wide cooperative effort."

The following excerpts are extracted from the "highest priority" recommendations listed by the various assembled panels:

Panel on Low- and High-Strain Cyclic Properties:

> "The use of arrays to back-calculate and verify the cyclic properties of the soil from actual earthquake records is strongly recommended. Priority should be given to soft cohesive and loose saturated granular sites, as they have been shown to be most hazardous during earthquakes."

Panel on Energy Dissipation

> "A test site should be established that could be used for experimental studies and that could provide an opportunity for recording seismic motions. This site would be used for:
> - Study of ground motions in order to back-calculate damping
> - In-situ measurements of soil damping
> - Laboratory tests on soil samples to determine damping
> - Development and calibration of new testing techniques (using forced vibrations, explosions, etc.) to determine values of damping under large strains"

Panel on Site Geometry and Global Characteristics

> "The effects of vertical gradients of shear wave velocity and material damping in the upper 1 to 2 km beneath the site have been shown to exert a significant influence on the spectral content of ground motions recorded at rock sites for frequencies exceeding about 1 Hz. Presently, disagreement exists as to the cause of such effects as related to source processes or site effects or perhaps to both. In order to resolve this issue as well as improve the prediction of short period ground motion properties, observations of ground motions at deep borehole sites are needed. Ideal experiments would include boreholes in both soft and hard rock, drilled to depths of 2 to 3 km (some of which already exist), measurement of in situ shear wave velocity and damping values, and placement of several three-component instruments within the borehole and at the surface. Analyses would consist of determining how the spectral content varies with depth, coupled with appropriate 1-D and 2-D modeling of the site effects."

> "When feasible, a basin should be instrumented with hundreds of instruments to collect aftershock data to capture the complex response of the basin. This experiment would provide much needed data to study the effects of basin geometry upon strong ground motion. In lieu

of the completion of such an experiment, new low-strain microtremor data should be collected at existing strong motion sites and others sites that may be of interest to the basin geometry problem. The site characteristics of the recording sites should be determined in detail; i.e., shear-wave velocities, boundary depths, damping, and density should be obtained to depths of at least 100 to 200 m. At some sites, uphole/downhole installations should be analyzed. These data would provide the bases for a variety of studies."

"A common engineering practice is to assume that a seismic recording from a rock outcrop nearby to a soil site can be deconvolved to generate the input motion at the base of the soil column. Two problems arise: (1) the spatial incoherence of waves contributes variability in the ground motion, even for relatively homogeneous site conditions and (2) the deconvolution process is often ill-constrained because of insufficient knowledge of the soil properties (seismic velocities) and assumes simple, vertical plane-wave propagation. Special 3-D arrays should be designed to place uncertainties on this approach. At the surface, arrays should be designed to capture both soil and rock motions. At depth, seismometers should be placed at each soil and/or rock horizon in order to directly measure the variability in the input motion at each interface."

Panel on Seismic Arrays

"Additional strong-motion arrays should be established. Joint industry or university/industry experiments should be supported to leverage manpower and funding. ... Different soils should be instrumented to provide a database covering a range of site conditions. The highest priority should be given to sites with soft deposits, such as those in the San Francisco Bay area, which suffered heavy damage during the October 17, 1989, Loma Prieta earthquake. ... To address the problem of dynamic soil response and site characterization, the minimum array configuration should consist of a layered 2-D array (horizontal array on the surface and at depth) combined with several vertical arrays with multiple receivers."

Panel on Sloping Ground Sites

"The documentation and collection of case history data should continue and be augmented to provide detailed three-dimensional delineation of sediment stratigraphy, measured soil properties, topography, and distribution of ground displacements. ... Well-characterized and -instrumented sites, which are potentially susceptible to major deformations, are needed to provide ground-motion and pore-pressure records, not generally available at case history sites. A special effort should be made to measure displacements within the failed materials."

Summary

Clearly, the consensus of opinion of the various panels of experts that have convened over the past 15 years is that results of field observations, including measurements from instrumented field sites during strong earthquake shaking, are essential to the advancement of geotechnical earthquake engineering. Lessons learned from the 1985 earthquake in Mexico, the 1989 earthquake in northern California, and several other recent events are that (1) amplification of ground motions and (2) liquefaction of loose granular soils greatly intensify damage locally in areas that otherwise were not severely affected by earthquake shaking. Consequently, geotechnical engineers are being called upon to develop predictive and design criteria to better cope with these severe local hazards.

## STATUS OF INSTRUMENTED GEOTECHNICAL TEST SITES

In response to the need for instrumented sites and the great interest shown by the several workshops that have considered this topic as listed above, several sites have been instrumented and several more instrumented sites are planned in the near future. Because of the required long-term maintenance, most of these experiments are being conducted by organizations with other instrumentation programs and in-house personnel to operate and maintain those arrays. These organizations include the California Division of Mines and Geology Strong Motion Instrument Program, the US Geological Survey, the US Army Corps of Engineers, the US Bureau of Reclamation, and the California Division of Water Resources. US engineers and scientists have also cooperated in foreign instrumentation experiments including the Lotung and Hualien sites in Taiwan. In addition, engineers and scientists in several foreign countries have developed many instrumented sites for geotechnical studies.

### California Strong Motion Instrumentation Program (CSMIP)

For many years CSMIP has installed and maintained an extensive array of free-field instruments in California and have instrumented numerous tall buildings, particularly in the Los Angeles area. These instruments have been placed for many purposes, including seismologic studies and investigation and verification of structural behavior. Only a few of the sites instrumented prior to the Loma Prieta earthquake (1989) were designed to measure effects of specific importance to geotechnical earthquake engineering.

APEEL Array.--Several years ago, an array called the APEEL array was placed to monitor the response of sites underlain by sediments with varying stiffnesses near San Francisco Bay. That array consists of several strong-motion instruments placed at ground surface along a line extending from the margins of San Francisco Bay westward to the Santa Cruz Mountains. The array recorded ground response on the various sediments during the 1989 Loma Prieta earthquake. Part of the stations in that array are operated by CSMIP and part by USGS.

Meloland Overcrossing.--The Meloland Road-Interstate Highway 8 overcrossing in the Imperial Valley of California was instrumented prior to the 1979 Imperial Valley earthquake. With to the response of this instrumented structure, Rojahn and others (1982) report the following:

> At the time or the October 15 main shock, the Meloland Road-Interstate Highway 8 overcrossing, a contiguous two-span reinforced concrete bridge 0.5 km southwest of the Imperial fault [surface rupture], was instrumented with two 13-channel remote-accelerometer central recording accelerograph systems. Although the film transport in one of the two recorders malfunctioned during the earthquake, these instruments provided an important and usable data set. Peak accelerations in the north-south, vertical and east-west directions at the base of the bridges central support column were 0.28, 0.17, and 0.33 g, respectively, whereas those at an free-field site were 0.32, 0.23, and 0.30, respectively. Peak accelerations recorded on embankment sites adjacent to each abutment were substantially higher than those recorded at the base of the bridge's central support column; these data suggest that the structure itself altered the motion at the embankment sites. Other important features of the records include (1) an acceleration pulse 1 s long occurring in the east-west components of the free-field, column-base, and embankment sites; and (2) strong evidence of modal

**Electrical Power Research Institute (EPRI)**

EPRI has cooperated in the installation of instrumentation at several sites operated by other groups, such as the Lotung and Hualien sites in Taiwan, and the Cholame Valley USGS site. In addition, EPRI has sponsored some in-house experiments such as the one at Stone Corral noted next.

Stone Corral.--EPRI has installed instrumentation at a site called Stone Corral at Scobie Ranch, near Parkfield, California. That site consists of 13 surface and 8 downhole triaxial FBA accelerometers. The surface array has four instruments placed in each of three arms that are 120 m in length and laid out at 120 degree angles to each other. The 13th instrument is place at the apex of the three arms. The downhole instruments are clustered near the center of the array at depths of 7.5 m to 90 m. The main purpose of the surface array is to measure three-dimensional wave coherence.

**National Center for Earthquake Engineering Research (NCEER)**

Owens' Pasture Site.--NCEER has funded Jeremy Isenberg and his associates to conduct the following experiment at a site called Owens' Pasture as described by Isenberg and others (1991): "A field experiment designed to investigate the performance of buried pipelines subjected to lateral offset and to ground strains from seismic wave propagation has been constructed at Owens' Pasture, near Parkfield, California. The site was chosen to capitalize on the predicted recurrence of the 1966 Parkfield-Cholame earthquake sequence on the San Andreas Fault."

"As noted on the site plan shown on Figure 3, two segments of welded steel pipeline have been constructed and buried in trenches that across the San Andreas fault. These two segments are oriented to optimally generate tension and compression, respectively in the two pipeline segments as the fault shifts. In addition to the welded steel pipes, a segment of ductile iron pipes with push-on joints has also been buried across the fault at the site. "Each welded steel pipe segment is instrumented with 18 strain gages; two gages are placed on opposite sides of a springline at nine stations. For the ductile iron pipes, each of six push-on joints is instrumented with one transducer to measure relative extension (or compression) and another to measure relative rotation. To measure permanent offset across the pipeline segments, survey monuments were placed approximately on a straight line about perpendicular to the assumed fault strike. There are 12 such monuments with 5-m spacing near the fault and 10-m spacing at greater distances. ... To measure ground motion due to transient wave propagation, three three-axis SAMTAC 17E seismographs are installed at the corners of a roughly equilateral triangle 50 m on a side."

**US Geological Survey (USGS) - Pre Loma-Prieta Sites**

The USGS has conducted a program in strong-motion seismology for many years, including installation and maintenance of strong-motion instruments. Most of those instruments have been placed at free-field sites, including sites on soft and stiff sediments. In addition, a few buildings have also been instrumented. Most of these sites were planned for seismologic studies. Prior to the 1989 Loma Prieta earthquake, however, two major sites were instrumented specifically to monitor ground response and generated pore pressures. Those two sites are the Wildlife Array in the Imperial Valley of southern California north of Brawley, which recorded ground motions and large pore pressures during the 1987 Superstition Hills earthquake, and a similar array in the Cholame Valley near Parkfield, California.

Wildlife Site.--The Wildlife Array was developed in 1982 at a site where liquefaction occurred during the 1981 Westmorland, California earthquake. That site is in the northern part of the Imperial Valley, California, an area within which earthquakes have generated an occurrence of liquefaction within a few km of the site on an average of once in every seven to eight years over the past 50 years. The Wildlife array consists of a downhole 3-component accelerometer placed at a depth of 7.5 m and a second 3-component accelerometer at ground surface (Figure 4). Five pore-pressure transducers were placed in a liquefiable layer at depths ranging from 3 to 6 m (Youd and Wieczorek, 1984). Several field and laboratory investigations have been conducted to measure pertinent soil properties at the site including penetration resistances, seismic velocities, stress-strain-pore pressure behavior, etc. (Bennett and others, 1984). These measurements were made by investigators from several different universities and governmental agencies with funding from several sources.

The Wildlife array was shaken by the 1987 Elmore Ranch and Superstition Hills earthquakes and two aftershocks. The system responded to each of these earthquakes with all components of acceleration being recorded and four of the five pore-pressure transducers responding and giving recorded pore-pressure responses. No detectable pore pressures were generated by the Elmore Ranch event or by either of the two after shocks. The pore pressures rose to equal the overburden pressure following the Superstition hills event, bringing the sediment into a state of liquefaction (Holzer and others, 1989). These records, reproduced in Figure 5, are the first to show the development of such high pore pressures in a field setting. These data have provided information to better define the liquefaction process and to verify predictive criteria and analyses. Even with this success, the recorded pore pressure response raised additional questions. Unexpectedly, much of the pore pressure increase occurred after strong ground shaking had diminished to a low level. The reasons for this delayed response are not entirely clear and additional records are needed to further verify this aspect of the liquefaction process. Hushmand, Scott, and Crouse (1991) also discuss questionable pore pressure measurements.

Lateral ground displacements as large as several tenths of a meter were also recorded at the site, but no transducers were in place to record a time history of these displacements (Youd and Bartlett, 1989). Thus, we cannot fully examine the important interactions between ground motion, liquefaction and ground displacement. Additional recordings at this and other sites with additional instrumentation will be required to answer these and other important questions.

Cholame Valley Site.--To take advantage of the prediction of a moderate magnitude earthquake on a segment of the San Andreas fault near Parkfield, California, several instrumental experiments are in place, such as the Turkey Flat array and the Owens' Pasture pipeline experiment mentioned previously. Another experiment is a ground motion-liquefaction array placed near Cholame Creek at a locality where ground water levels are relatively high and loose to dense granular sediments lie at shallow depth. This site was instrumented as a joint project between EPRI and USGS with four triaxial downhole accelerometers at depths ranging from 3 to 30 m and a fifth accelerometer at ground surface (Figure 6). Twenty-four pore pressure transducers are in place at depths ranging from 5 to 15 m (Holzer and others, 1988; Anderson and Tang, 1991). Some pore pressure transducers have failed and others have been installed to replace them. As of October, 1991, the predicted earthquake had not occurred; the instrumentation had been triggered, however, by at least one small earthquake. This site has been thoroughly investigated with penetration tests, seismic velocity tests and some laboratory analyses. This work has been performed by several different universities and geotechnical consulting firms.

**USGS - Post-Loma Prieta Sites**

Since the 1989 Loma Prieta earthquake and new funding and initiatives that followed that event, USGS has planned, and is in the process of implementing, several field experiments requiring instrumentation. Those experiments are briefly described below:

Marina District, San Francisco.--During the 1989 Loma Prieta earthquake, liquefaction developed in a hydraulic fill beneath a large part of the Marina District of San Francisco. To study ground motion amplification and liquefaction, USGS is developing an instrumented site at a school yard in that area. To date, a downhole accelerometer has been placed beneath the fill and soft mud, and work is underway to install additional accelerometers and several electrically transduced pore-pressure piezometers (Tom Holzer, USGS, oral communication, October 1991).

San Bernardino Site.--Instrumentation of a liquefaction site in the San Bernardino area of southern California is also being planned with instrumentation similar to that being installed in the Marina District. Instruments are being purchased and site selection and permitting is in progress (Tom Holzer, USGS, oral communication, October 1991).

Foot of Market Array, San Francisco.--A project is underway to install an array of sensors in the Foot of Market section of San Francisco. That instrumentation will include surface and downhole accelerometers and pore-pressure transducers. The project is designed to provide data for a combination of seismologic and geotechnical engineering studies (Roger Borcherdt, USGS, oral communication, March 1991).

Pacific Park Plaza Building, Emeryville, California.--Prior to the 1989 earthquake, USGS scientists placed instruments in the Park Plaza building founded on soft San Francisco Bay sediments in Emeryville, California. The building is a thirty-story, three winged, ductile moment-resisting reinforced-concrete fame structure constructed in 1983. In 1985, accelerometers producing 21 channels of output were deployed throughout the structure. A 3-component free-field accelerometer was located just north of the building. All of these instruments are connected to a central recording system. An additional independent free-field triaxial accelerometer is deployed south of the building. Those instruments provided a suite of records during the Loma Prieta earthquake that is sufficient for analyzing the building response, but the data are insufficient for analyzing soil-structure interaction effects which clearly occurred (Celebi and Safak, in press). Two downhole instruments are to be placed on bedrock beneath the site in the next year. These instruments will provide additional information for better assessment of soil-structure interaction as well as structural response during future earthquakes.

Landslide Arrays.--Plans are underway at USGS to instrument two landslides with accelerometers, electrical piezometers and electrically transduced extensometers. One landslide is located near LaHonda in the Santa Cruz Mountains south of San Francisco, and the other site is planned for Santa Anita Canyon near Pasadena in southern California. In the initial phase, one triaxial accelerometer will be placed on each landslide with a second accelerometer nearby on unfailed ground. Four piezometers are planned for placement within or near the failure zone of each slide, and 2 extensometers will be stretched from unfailed to failed ground at each site. These installations should be complete by fall of 1992 (Ed Harp, USGS, oral communication, October, 1991).

Los Angeles Basin Arrays.--Two parallel experiments are being implemented in the Los Angeles basin to study ground response and the influence of the basin topography on ground motion. In both experiments 24 bit recorders are being used with high-quality 3-component accelerometers to generate wide-band records with high dynamic range. In one experiment, several instruments are being installed along a line stretching from a bedrock outcrop near the Whittier Narrows southward across a sediment-filled basin. In the other experiment, pairs of accelerometers are being placed with one at ground surface and the other in the bottom of a borehole drilled into bedrock. One pair of instruments will be located south of the Whittier Narrows as part of the above-mentioned linear array. The other pair will be located near San Bernardino where there is a high probability of a nearby large earthquake in the next few decades. Both of the pairs are being placed at sites underlain by stiff sediments "typical" of the Los Angeles basin (Al Rogers, USGS, oral communication, October, 1991).

**Instrumentation of Dams**

Instrumentation of major dams has become a routine practice in seismic areas of the United States. Three primary agencies, the US Army Corps of Engineers (USAE), the US Bureau of Reclamation (USBR), and the California Department of Water Resources (CDWR), have instrumented major dams under their jurisdiction. More than 150 dams have been instrumented to date. Typically, instrumentation consists of one 3-component accelerometer on the crest of the dam and a similar instrument on natural ground immediately below the dam. A third accelerometer is commonly, but not always, placed on one of the abutments. Several large and important dams, such as Isabella, New Melones and Oroville, all in California, have been instrumented more extensively with multiple instruments (John Wilson, USBR, written communication; A.G. Franklin, USAE, written communication; Les Harder, CDWR, oral commun.).

**Instrumented Sites in Canada**

In his summary for this workshop, Peter K. Robertson makes note of the following experiment to be conducted in Alberta. The site will eventually be shaken by artificially induced rather than earthquake generated excitation.

> In Canada a national test is about to be initiated to study the characterization of sands for dynamic and static liquefaction. This study has been initiated by Professors P.K. Robertson and N.R. Morgenstern. The objective of the study is to develop a site consisting of loose saturated sands extending to a depth of 40 m. Extensive in-situ testing and sampling will be preformed, including in-situ freezing to obtain undisturbed frozen samples. One objective will be to calibrate the less expensive in-situ test techniques. ... The study will conclude with a predictive event, where movement will be induced by shakers or blasting. ... The main site has already been selected and will be a large tailings dam in Alberta. A second site has been located for a smaller scale site investigation (including frozen samples). This second site will consist of natural sands with all the natural variability in grain size, density, etc.

**Instrument Arrays in Taiwan**

The following information on instrumented sites and arrays are summarized from a report by Tang and others (1991).

Lotung, Taiwan Site.--In 1985, the Electrical Power Research Institute (EPRI) in cooperation with the Taiwan Power Company designed, constructed and instrumented 1/4 and 1/2 scale reinforced concrete models of nuclear containment vessels at a field site in Taiwan to study soil-structure interaction (SSI) under strong earthquake shaking. The model structures are located in Lotung, an area of frequent moderate to large earthquakes. A circular array of Accelerometers (SMART-1 Array) had been previously installed in the area surrounding the site by the Institute of Earth Sciences in Taiwan in cooperation with the University of California at Berkeley. The Lotung site and scaled structures were instrumented to record free-field and structural motions. Pore-pressure transducers were also placed in granular sediments beneath the site. Since the site became operational, more than 30 earthquakes with magnitudes greater than 4 have shaken the site. Some earthquakes have generated peak accelerations greater than 0.2 g. The records produced by these earthquakes have been analyzed to assess the validity of SSI models and to quantify uncertainties.

The Lotung site is classed as a soft site and is underlain by sediments characterized by shear-wave velocities that range from 90 m/sec to 300 m/sec to a depth as great as 60 m. The results of studies conducted to date have provided useful verification and insight for response heavy vessels on a soft site. These results, however, leave many unanswered questions concerning the response of stiff sites, which are in general more typical of localities where nuclear power plants have been constructed. Hence, the Lotung site is being demobilized and a new site on stiffer soils near Hualien is being developed.

Hualien Site.--To further study SSI, a second and stiffer site is being developed in Hualien. Hualien is located on the east coast of Taiwan in an area of equal or greater historical seismicity to that at Lotung. A circular array of strong-motion instruments, the SMART-2 Array, will be installed around Hualien by the Institute of Earth Sciences of Taiwan in cooperation with scientists at the University of California at Berkeley. The primary objectives of the Hualien experiment are to obtain earthquake induced SSI data at a stiff soil site and to further develop and verify SSI criteria.

**Instrumented Sites in Japan**

Japanese geotechnical engineers have been instrumenting sites since 1975. In that year, a site on Ohgishima Island was instrumented with accelerometers and piezometers. That site was decommissioned in 1979. A site on Owi Island was similarly instrumented and operated from 1977 to 1986. The Owi Island site produced some of the first recordings of both ground motion and pore pressure response during actual earthquake shaking Ishihara and others, 1981). After the site on Owi Island was abandoned, the Tokyo Metropolitan Government provided another site near Sunamachi for installation of accelerometers and piezometers. That site was shaken by the December 17, 1987 Chiba-Toho-Oki earthquake (M = 6.7) which produced 122 gal peak accelerations at the site and generated pore pressures as great as 14 percent of the mean effective confining stress (Ishihara and others, 1987; 1989).

In a report to the 1988 Workshop on Designated Sites for Geotechnical Experimentation, T. Kokusho (in Benoit and de Alba, 1988) summarized progress in instrumentation of geotechnical sites in Japan. He noted that several organizations have developed their own earthquake measurement sites, many with multi-level instrumentation systems. Figure 7 shows the number, types and agencies that had established sites in 1988. Some of the more important geotechnical test sites are described below:

Seaward Extension of Haneda Airport.--A linear array containing many instruments has been installed parallel to a new runway for Haneda Airport that is being extended onto land reclaimed from Tokyo Bay. That array consists of 8 instrument localities, each with accelerometers at ground surface and at 20, 40, and 80 m depths. Each locality also has three piezometers placed in medium dense sand at depths of about 10 m (Figure 8). The array has been in operation since 1988, but no records have yet been obtained that show transient buildup of excess pore pressures.

Sunamachi Project.--At this site, several piezometers have been installed at depths ranging from 5 to 10 m, and a vertical array of accelerometers has been installed with one at ground surface and others at depths of 10, 20, 40, and 89 m, respectively. These instruments are maintained by The Tokyo Metropolitan Government, Port and Harbor Section, in collaboration with Prof. Kenji Ishihara, University of Tokyo.

Chiba Dense Array.--A dense array of seismometers and instrumented buried pipes has been operating at Chiba Experiment Station of the Institute of Industrial Science, University of Tokyo, since April, 1982. In this array, a total of 155 components of ground motion, comprising 123 components of ground acceleration and 32 components of strains in buried pipes, are simultaneously recorded. The topography and geological setting is simple with the ground surface being almost flat. Figure 9 shows the typical profiles obtained from three bore holes in which seismometers were installed. The top 4-5 m of the site consists of loam with standard penetration resistances less than 10. Underlying the loam is a 3- to 4-m thick clayey layer with N values less than 10. A dense sand layer underlies the clay with N-values greater than 20 to 30. Figure 10 shows the layout of the surface seismometers (within 1 m of ground surface). As plotted on the figure, the network consists of an outer array of several widely separated seismometers surrounding a denser inner array. Stacks of downhole seismometers are installed beneath each of the sites in the inner array and beneath several sites in the outer array. As many as 5 seismometers are placed in a stack with instrument depths ranging from 1 m to 40 m. The seismometers consist of three piezoelectric accelerometers oriented along orthoganal axes installed in a 65 mm diameter steel casing.

**Instrumented Sites in Mexico**

Some of the most significant strong-motion seismograms ever recorded on soft soil sites were registered on lake-bed sediments in Mexico City during the 1985 Michoacan earthquake (M = 8.1). Since 1985, the number of strong-motion accelerometer stations in Mexico City has been augmented with several additional surface accelerometers and at least 3 downhole accelerometers. The latter instruments will allow better definition of ground motions in stiff sediment and rock strata beneath the soft lake-bed sediments.

**Arrays in China**

Figure 11 lists several arrays operating in China as reported in 1988 (Anderson and Tang, 1991). The cryptic notes on the figure give the number of instruments and general purpose for each installation.

**Arrays in Italy**

Two strong motion arrays have been developed in Italy as reported by Anderson and Tang (1991).

Southern Lazio Array.--The Southern Lazio Array, 100 km south of Rome, was established in 1984. That array is designed to study boundaries between two areas, one of which does not show any seismicity. The array consists of 10 triaxial accelerometers placed at ground surface and spaced over a distance of 20 km. While this array was being installed, a magnitude 4.7 earthquake occurred 30 km from the site, but with only six records obtained, the data were insufficient for an analysis of the response of the different geologic units.

Cerreto di Spoleto Array.--The Cerreto di Spoleto array (Umbria, Central Appennine) is designed to study topographical effects, mainly for the seismic behavior of an historical town on a 200-m high ridge. The array consists of 4 triaxial accelerometers: one at the top of the ridge, one at mid height, one at the base. The fourth instrument is located on the alluvial floodplain of a nearby river. The instruments on the hill are 300 m apart and founded on calcareous bedrock; the instrument on alluvium is 500 m from the accelerometer at the base of the hill, and is founded on a pile driven into the underlying alluvial sediment.

### Instrumented Sites in Russia

During a joint US-USSR workshop on joint research opportunities, held in Moscow in September 1991, Prof. A. Nikolaev, Deputy Director and Head of Experimental Geophysics of the IFZ, briefly described two experimental pieces of equipment being used in Russia to study nonlinear properties of soils. The descriptions below are developed from my memory of information orally presented at the workshop. I was unable to obtain written reports or diagrams of this equipment to provide a more detailed and accurate description of the equipment.

The first piece of equipment is a 30-ton seismic vibrator with an 8-ton inertial mass that is used to generate large amplitude seismic waves (Figure 12). Monitoring instruments have been placed at varying distances from the vibrator to measure the attenuation of seismic waves with distance. The results are used to evaluate the influence of nonlinear soil properties on wave propagation.

The second piece of equipment is a set of large diameter tubes laid out in a parallel configuration as diagrammatically illustrated in Figure 13. These tubes have been constructed near an oil field in Siberia. With the aid of an elaborate pumping system, water is sloshed back and forth in the tubes in a controlled and synchronized manner to generate large amplitude, low frequency seismic waves for study of several properties of waves including the influence of nonlinear soil properties on wave attenuation.

## FUTURE NEEDS FOR INSTRUMENTED GEOTECHNICAL TEST SITES

The need for instrumented geotechnical test sites, as noted in the many excerpts from past workshops, has only partially been met by presently instrumented or planned sites. There are several reasons why additional instrumented sites are needed: (1) Instrumented sites are required to answer several important questions that have not yet been addressed. (2) Because earthquakes are relatively rare events, even in highly seismic areas, instrumented sites in different areas are required to optimize the opportunity for collecting needed data. (3) Because site conditions and soil properties are fixed at a given site, several sites may be required to test a reasonable range of soil and site conditions.

Some of the more important unmet needs for instrumented sites are tabulated in the following paragraphs. This listing is based on recommendations from past workshops, suggestions from member's summaries submitted to this workshop, and the writer's personal experience.

**Ground Motion**

More instrumented sites are needed:

- For verification of nonlinear 1-D and 3-D soil dynamic response models for both soft and stiff soil sites.
- For back calculation of nonlinear soil properties to verify field and laboratory test data.
- To evaluate procedures and models for deconvolving measured ground motions.
- To further analyze and verify the influence of basin structure and topography on ground response.
- To provide sites for class-A predictions which are needed for verification of design procedures.

**Ground Instability and Improvement**

More detailed studies of ground failures and instrumentation of sites are needed:

- To provide case history information for better assessment of mechanisms controlling ground failure and to obtain quantitative data for resolving the importance of various factors.
- To verify analytical and empirical models for prediction of pore pressure response, ground instability and ground displacement for both liquefiable and non-liquefiable granular soils.
- For back calculation of nonlinear soil properties controlling pore-pressure and ground deformation and to verify field and laboratory test data.
- For analysis and verification of the performance of modified or stabilized sites.

**Natural Slopes**

More instrumented sites are needed:

- To further evaluate and analyze factors controlling brittle fracture of rocks and stiff soils and yield of ductile soils.
- To further develop and verify analytical models and to develop empirical criteria and models for predicting ground deformation and displacement.
- For back calculation of rock and soil properties controlling deformation or failure and to verify field and laboratory test data.

**Earth Dams and Embankments**

More instrumented sites are needed:

- To evaluate and analyze pore pressure and deformation induced by actual earthquake shaking. Particularly needed are measurements within saturated loose to medium-dense sand deposits subjected to a significant driving shear stress, such as beneath sloping ground or embankments.

- To provide case histories of embankment response and deformation to aid model development and to provide real data for verification of analytical procedures.

**Retaining Walls and Underground Structures**

More instrumented sites are needed:

- To evaluate and analyze pore pressure and deformation response of bulkheads, sea walls and other shoreline retaining structures.

- To provide data for analysis and verification of response and deformation of various types of walls including reinforced earth, soil nailing, tied-back walls, conventional gravity walls, bridge abutments, etc.

- To provide data for analysis and verification of response and performance of underground structures including tunnels, underground chambers, pipelines, etc.

- To develop and verify methods for analyzing soil-structure interaction at "hard points" and points of intersection with ground-failure or fault displacement in underground structures and pipelines.

**Foundations**

More instrumented sites and structures are needed:

- To aid development and verification of analytical procedures for evaluating soil structure interaction for various types of structures ranging from high-rise buildings to heavy containment vessels, to bridges.

- To aid development and verification of procedures for evaluating the stiffness of pile foundations.

- To provide data to evaluate the stiffness, deformation, and performance of various foundation types including foundations with batter piles.

- To evaluate pile and caisson performance in areas subjected to various amounts of transient and permanent ground displacement.

## FINAL COMMENTS

Clear Need.--Based on the concentration and severity of damage that has occurred in areas with unfavorable ground conditions during recent earthquakes, such as the concentration of severely damaged and collapsed buildings in an area of soft lake deposits in Mexico City in 1985, and the concentration of damage on soft or liquefiable sediments in San Francisco and Oakland in 1989, better understanding and predictive analyses are required in geotechnical earthquake engineering to provide improved criteria for engineering design. A major component required to advance the state of the art in geotechnical earthquake engineering are data from carefully instrumented sites. Those data are needed to provide insight into ground response and failure processes, to provide data for development of analytical and empirical design procedures, and to provide case histories for verification of those procedures. The state of the art cannot advance very rapidly without these essential field data. Conversely, unsafe structures may continue to be built without verification of seemingly correct but untested design criteria.

Various Levels of Instrumentation.--Various levels of investigation and instrumentation may be used to develop field data for the advancement of geotechnical earthquake engineering. Many of the needs listed above require a high level of electronic instrumentation, and work should proceed in this area as expeditiously as possible to develop sites with that level of effort. Many other needs can be met, however, with a much less expensive, lower level of instrumentation. For example, non-electronic instrumentation and reference points, such as slope inclinometers, survey monuments, and even large scale aerial photographs, provide extremely useful reference points for ground failure studies. Development of sites or areas with this lower level of instrumentation should be pursued to provide a more quantitative base of non-electronic field data.

Cooperative Effort.--I repeat here the important note made at the 1989 workshop, that the cost and manpower required to develop and operate an adequate program of test sites would likely require cooperation and wide support between government agencies, universities and private industry. Such a program would likely require some super projects, with manpower and financial requirements far beyond the individual-investigator projects that have been the primary mode of operation in the past.

Improved Technology.--In their written summaries, several members of the workshop pointed out the need for improved instrumentation, and in some instances the development of new instruments. For example, strong motion accelerometers have been in service for many years, and reliable instruments are rather readily available from several suppliers. On the other hand, displacement transducers have not been widely needed in the past, but are a key component for many geotechnical test requirements. Transducers to measure stresses and strains in-situ are another instrumental need for which present technology is inadequate. Thus, a companion research program in instrument development also may be required to meet the needs of a concerted field instrumentation program.

Must Be Supported By Laboratory and Analytical Research.--Although full scale field studies and experiments are a necessary part of the research required to advance the state of the art in geotechnical engineering, laboratory and analytical research are equally necessary components. Thus, any major instrumentation projects should be closely coordinated with analytical and laboratory analyses. Such coordination is needed to assure that field projects are instrumented to measure pertinent soil and structural responses. An integrated program will aid the development and verification of analytical procedures, and will require proper field and laboratory measurement of material properties to assure valid analyses.

## REFERENCES CITED


Anderson, D.G., and Tang, Y.K., eds., 1991, Proceedings: NSF/EPRI Workshop on Dynamic Soil Properties and Site Characterization: Electric Power Research Institute, EPRI NP-7337, Project 810-14, 2 vols.

Bennett, M. J., McLaughlin, P. V., Sarmiento, John, and Youd, T. L., 1984, Geotechnical investigation of liquefaction sites, Imperial Valley, California: U. S. Geological Survey Open File Report, 84-252, 103 p.

Benoit, J., and de Alba, P., eds., 1988, Designated Sites for Geotechnical Experimentation in the United States: unpublished workshop report to the National Science Foundation, University of New Hampshire, Durham, NH, 165 p.

Celebi, M., and Safak, E., in press, Seismic Response of Pacific Park Plaza I: Data and Preliminary Analysis: Journal of Structural Engineering, ASCE.

Clough, G.W., and Silver, M.L., eds., 1983, A Report of the Workshop on Research Needs in Experimental Soil Engineering: unpublished report to the National Science Foundation, Virginia Polytechnic Institute and State University, Blacksburg, VA, 177 p.

Holzer, T.L., Bennett, M.J., Youd, T.L., and Chen, A.T.F., 1988, Parkfield, California, Liquefaction Prediction: Bulletin of the Seismological Society of America, v. 78, no. 1, p. 385-389.

Holzer, T.L., Youd, T.L., and Hanks, T.C., 1989, Dynamics of Liquefaction During the Superstition Hills Earthquake (M = 6.5) of November 24, 1987: Science, April 7, 1989, v. 244, p. 56-59.

Hushmand, B., R.F. Scott, and C.B. Crouse, "In-Situ Calibration of USGS Piezometer Installations," in "Recent Advances in Instrumentation, Data Acquisition and Testing in Soil Dynamics," Ed. Bhatia, S.K. and G.W. Blaney, ASCE Geotechnical Special Publication No. 29, pp. 49-69, 1991.

Isenberg, J., Richardson, E., Kameda, H., and Masata, S., 1991, Pipeline Response to Loma Prieta Earthquake: Journal of Structural Engineering, ASCE, Vol. 117, No. 7, p. 2135-2145.

Ishihara, K., Shimizu, K., and Yamada, Y., 1981, Pore Water Pressure Measured in Sand Deposits During an Earthquake: Soils and Foundations, Vol. 21, No. 4, p. 85-100.

Ishihara, K., Anazawa, Y., and Kuwano, J., 1987, Pore water pressures and ground motions monitored during the 1985 Chiba-Ibaragi Earthquake: Soils and Foundations, Vol.27, No. 3, p. 13-30.

Ishihara, K., Muroi, T., and Towhata, I., 1989, In-Situ Pore Water Pressures and Ground Motions During the 1987 Chiba-Toho-Oki Earthquake: Soils and Foundations, Vol. 29, No. 4, p. 75-90.

Jackson, G.O., 1987, Instrumentation of a Site Near Parkfield, California for the Measurement of Earthquake-Induced Ground Motion and Pore Pressures: unpublished MS thesis, Brigham Young University, 102 p.

Lee, K.L., Marcuson, W.F., Stokoe, K.H., and Yokel, F.Y., eds., 1978, Research Needs and Priorities for Geotechnical Earthquake Engineering Applications: Workshop report to the National Science Foundation, University of Texas, Austin, TX, 134 p.

NRC, 1985, Liquefaction of Soils During Earthquakes: National Research Council, National Academy Press, Washington, DC., 240 p.

Rojahn, C., Ragsdale, J.T., and Ragett. J.D., 1982, Main-shock strong-motion records from the Meloland Road-Interstate Highway 8 overcrossing in The Imperial Valley, California, Earthquake of october 15, 1979: US Geological Survey Professional Paper 1254, p. 377-384.

Saxena, S.K., ed., 1986, Siting and Geotechnical Program Focus and Directions, unpublished workshop report to the National Science Foundation, Illinois Institute of Technology, Chicago,IL., 194 p.

Tang, H.T., Stepp, J.C., Cheng, Y.H., Yeh, Y.S., Nishi, K., Iwatate, T., Kokusho, T., Morishita, H., Shirasaka, Y., Touret, J.P., Sollogoub, P., Graves, H., and Costello, J., 1991, The Hualien Large-Scale Seismic Test for Soil-Structure Interaction Research: Transactions of the 11th International Conference on Structural Mechanics in Reactor Technology, Vol. K1, pp. 69-74.

Youd, T.L., and Bartlett, S.F., 1989, US case histories of liquefaction-induced ground displacement: Proceedings, First U.S.-Japan Workshop on Liquefaction, Large Ground Deformation and Their Effects on Lifeline Facilities, US National Center for Earthquake Engineering Research, p. 22-31.

Youd, T. L., and Wieczorek, G. F. 1984, Liquefaction During the 1981 and Previous Earthquakes Near Westmorland, California: U. S. Geological Survey Open-File Report 84-680.

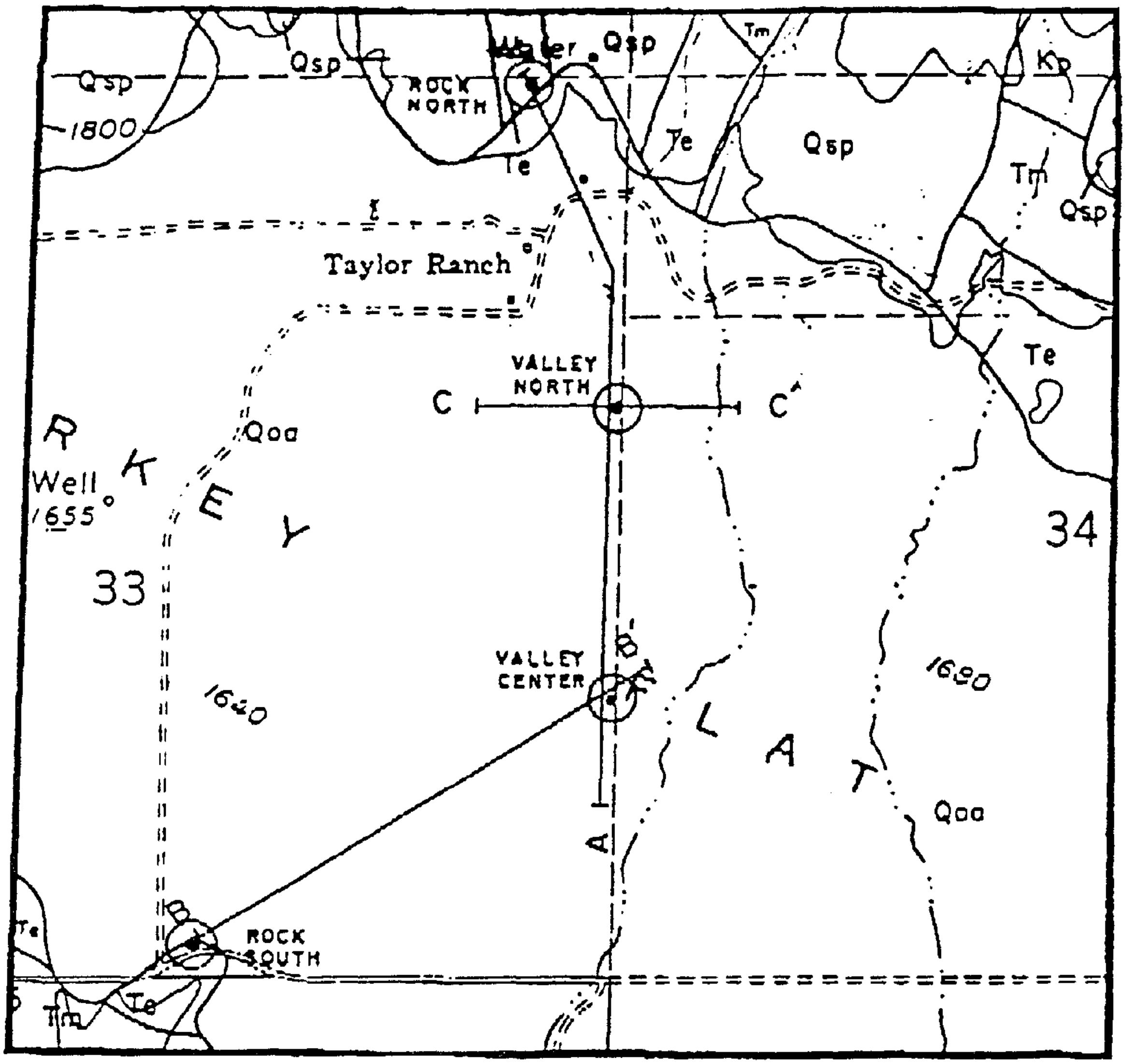



Figure 1. Map of Turkey Flats test area showing locations of the four ground motion recording sites, and three reference lines for the profiles shown in Figure 2. Numerous geophysical surveys and laboratory test on rock and soil samples have been conducted to character-ize the test area for ground response analyses. (After Real, in Anderson and Tang, 1991)

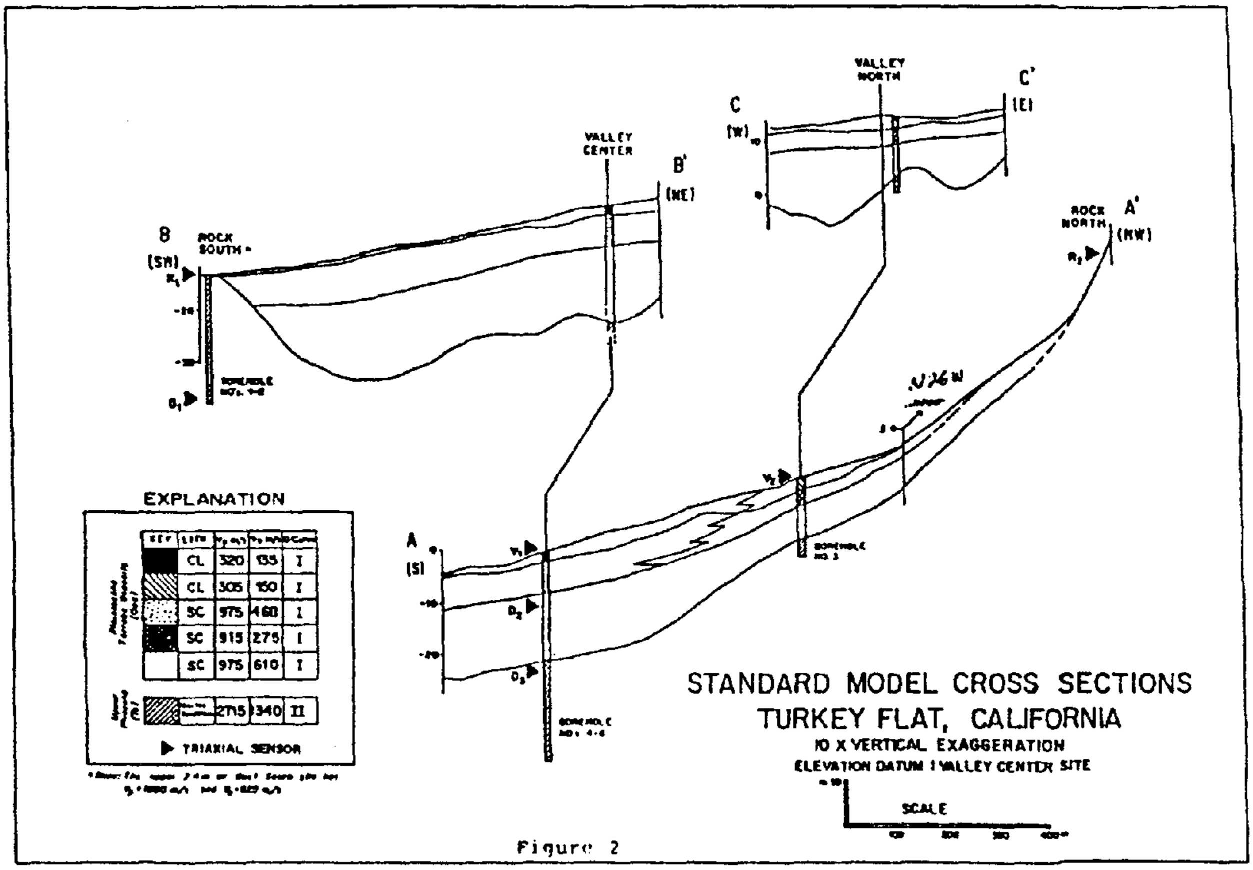



Figure 2. Cross sections showing sediment lithology beneath Turkey Flat for reference lines shown on Figure 1. (After Real, in Anderson and Tang, 1991)

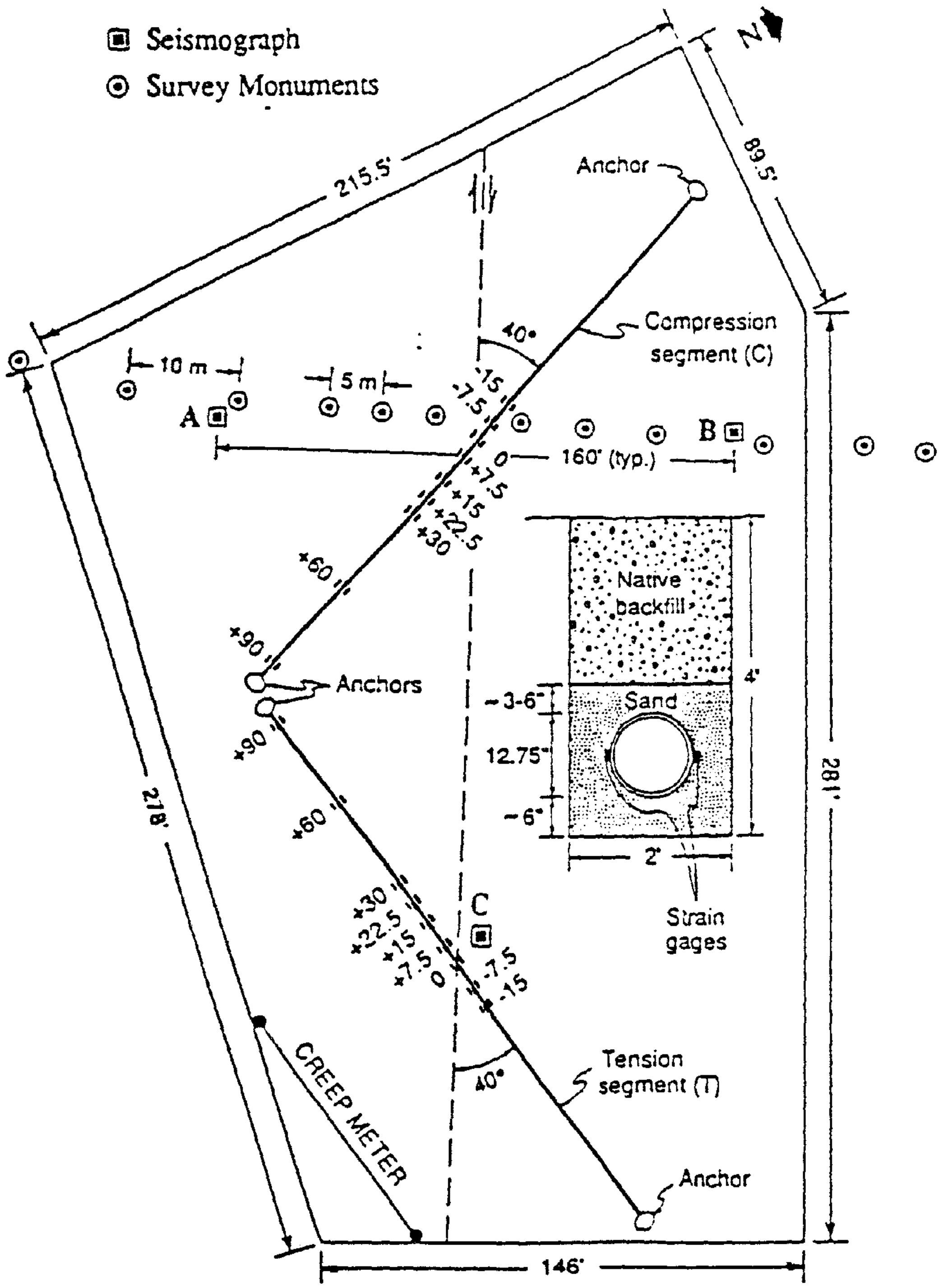



Figure 3. Site plan for Owens' Pasture site, near Parkfield, California, showing locations of welded-steel pipeline segments and attached strain gages, plus other installed instrumentation including accelerometers, survey monuments and a creep meter. The approximate location of the San Andreas fault is shown by the dashed line. (After Isenberg and others, 1991)

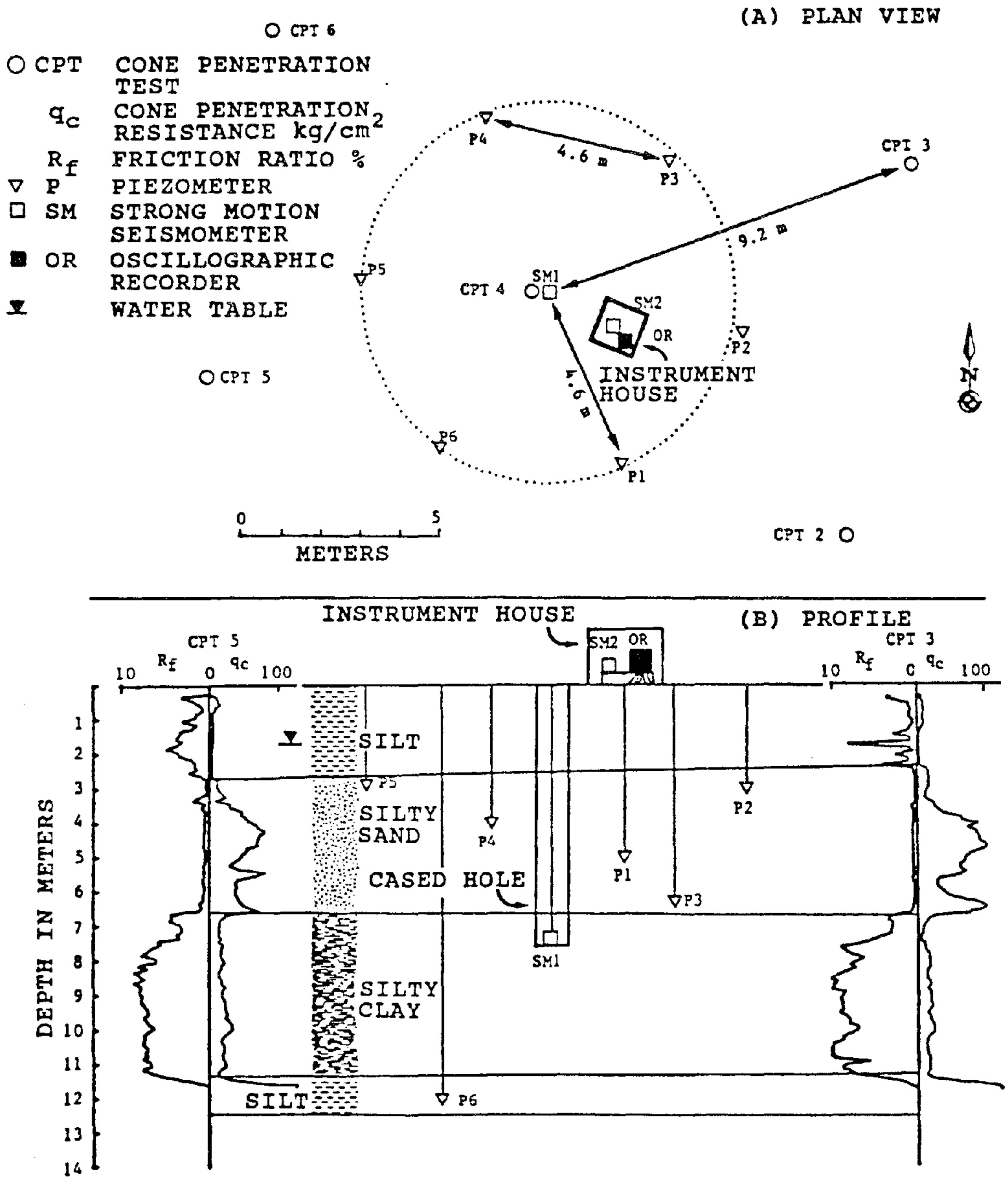



Figure 4. Plan and cross section of Wildlife site showing locations of accelerometers and piezometers. (After Youd and Wieczorek, 1984)

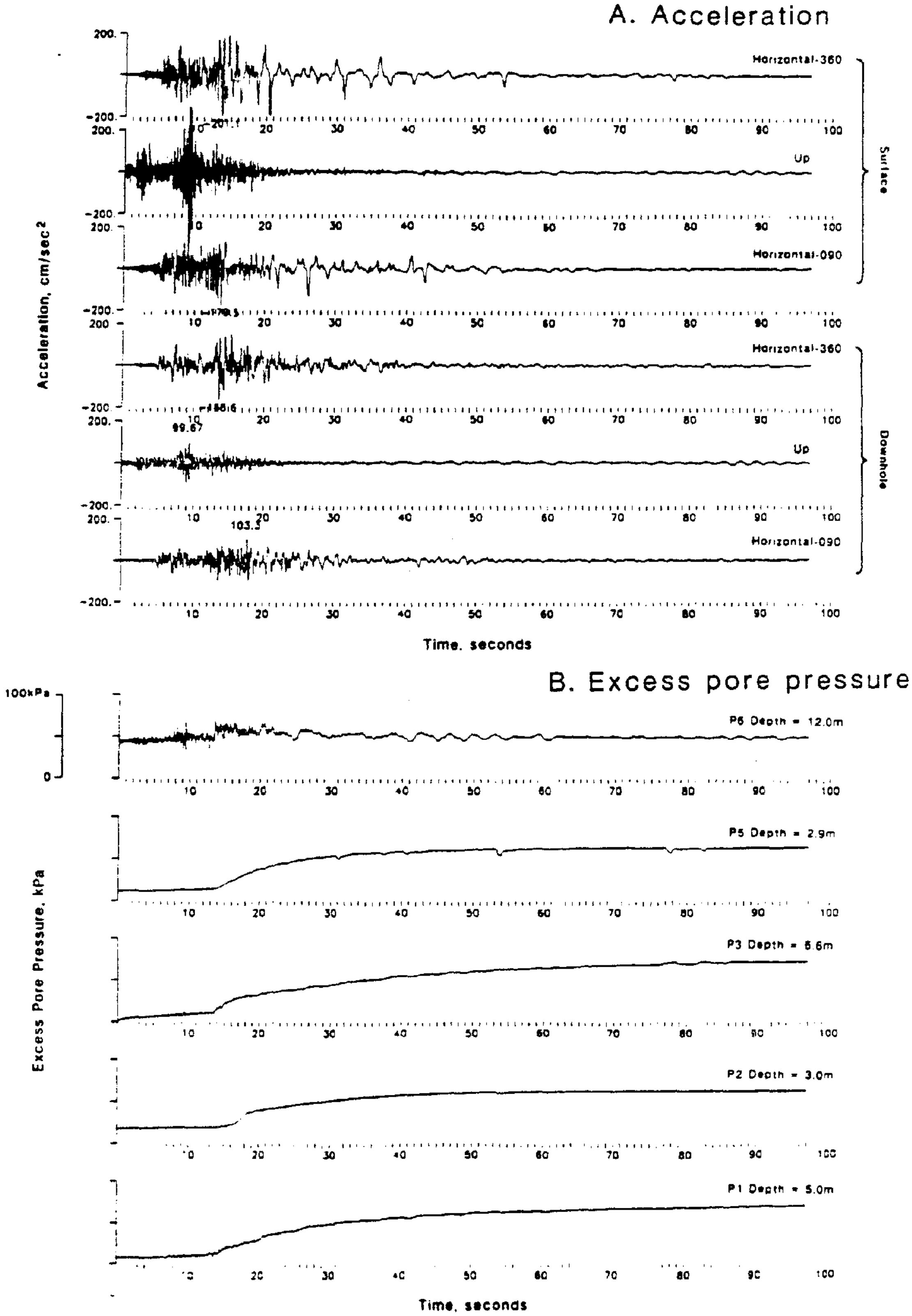



Figure 5. Accelerations and pore pressures recorded at the Wildlife site during the 1987 Superstition Hills Earthquake (After Holzer and others, 1989)

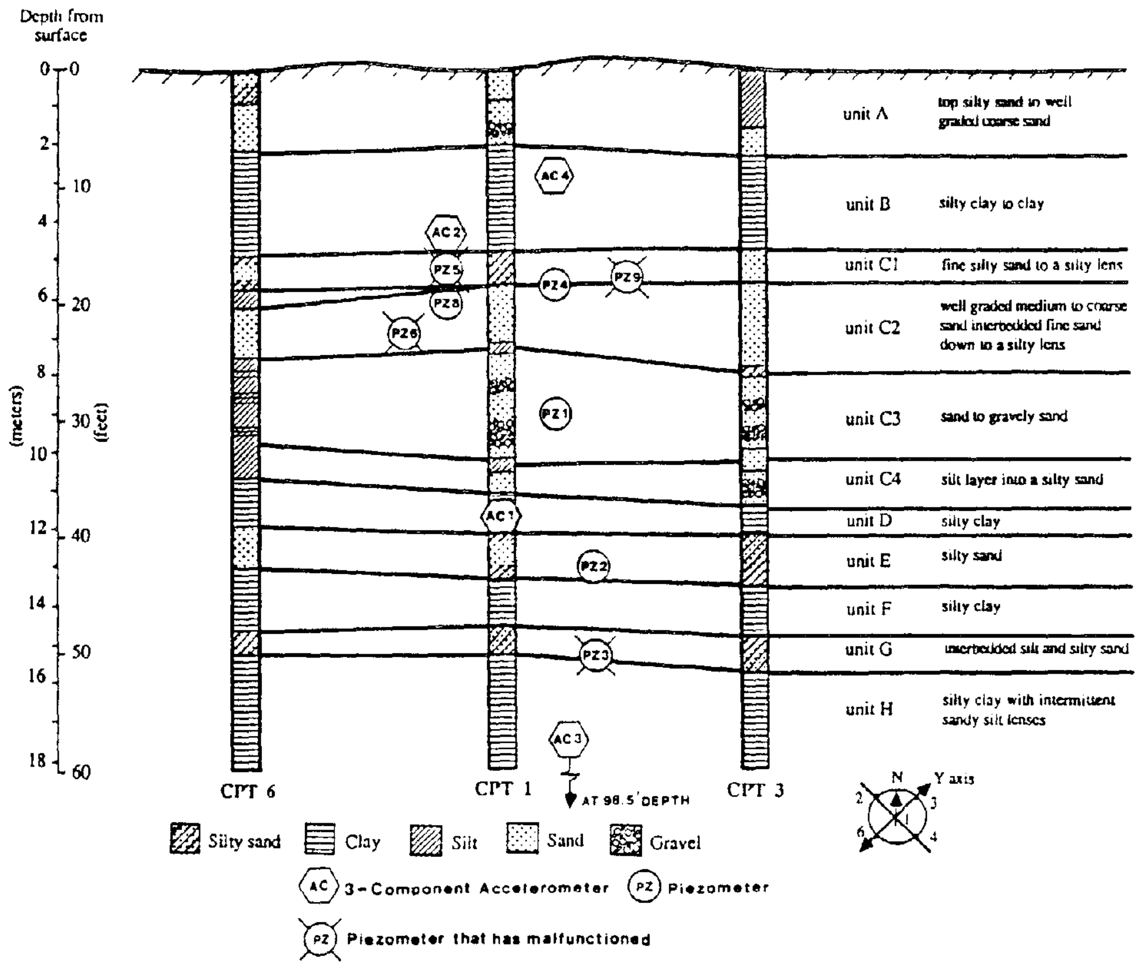



Figure 6. Cross section of Cholame Valley liquefaction and ground response site showing sediment lithology and locations of instruments. All of the piezometers shown on this cross section malfunctioned and have been replaced by additional piezometers at a nearby location. (After Jackson, 1987)

**Table 1**

**Purposes of Earthquake Recording for Earthquake and Geotechnical Engineering Research in Japan (excluding Seismological Research)**

| | Number of Earthquake Recording Sites |
|---|---|
| Seismic Amplification in Subsurface Soils: | 28 |
| Relative Seismic Motion for Buried Structure Design: | 15 |
| Ground Motion for Soil-Structure Interaction: | 13 |
| Liquefaction by Seismic Pore Pressure Measurements: | 9 |
| Stability of Fill and Slope (excluding dams) plus Improved Soil | 4 |

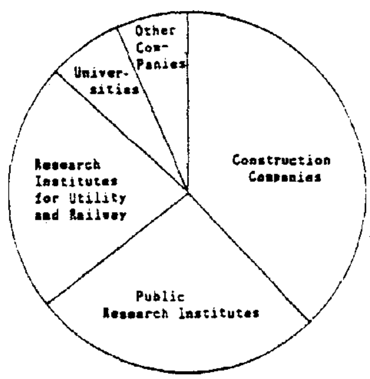



Figure 7. Table listing the number and purpose of instrumented sites in Japan as listed by T. Kokusho (in Benoit and de Alba, 1988).

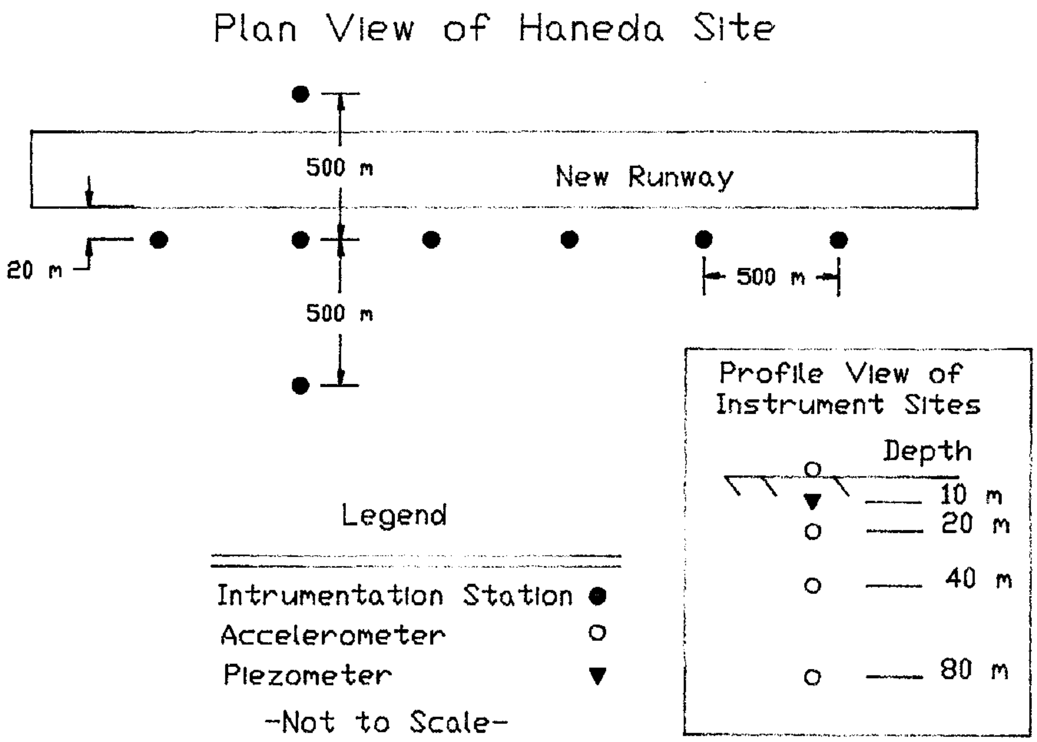



Figure 8. Map and profile showing approximate locations of instruments at Haneda Airport, near Tokyo, Japan. The instruments were installed in 1988 adjacent to a new runway that is constructed on land reclaimed from Tokyo Bay. (From K. Ishihara, University of Tokyo, written communication, August 1991)

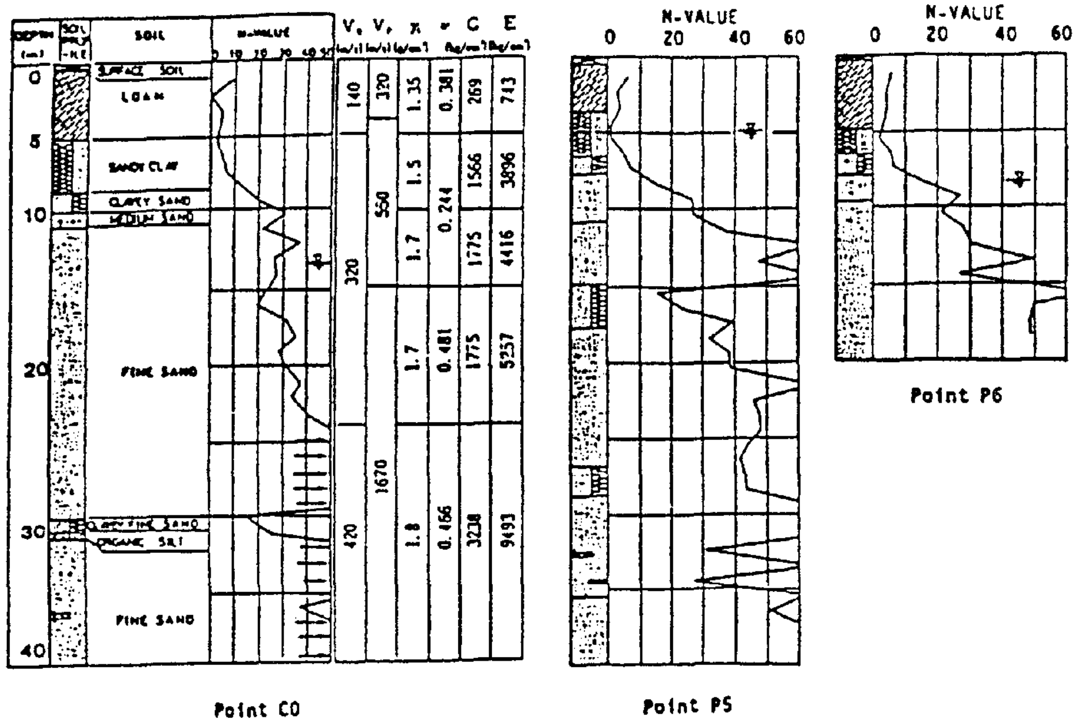



Figure 9. Soil profiles from three drill holes at the Chiba test site, Japan site showing sediment lithology beneath that locality. (After T. Katayama in Anderson and Tang, 1991)

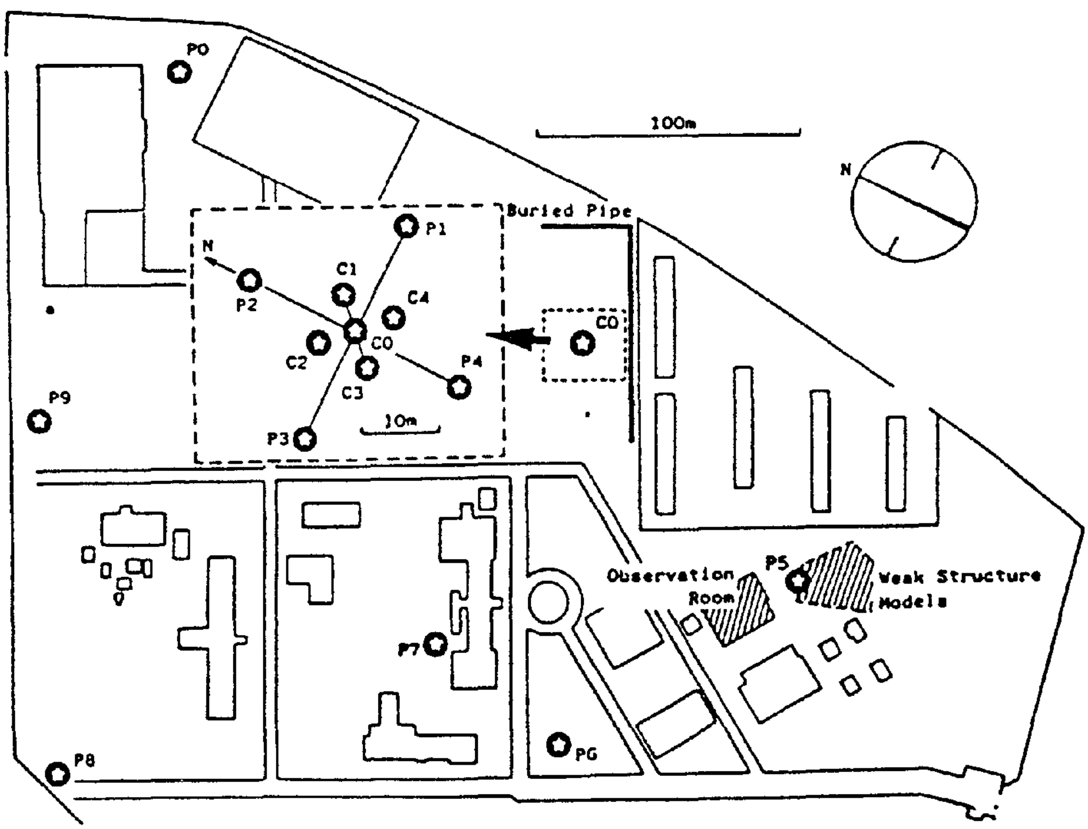



Figure 10. Map showing pipeline and accelerometer locations at the Chiba test site, Japan. (After T. Katayama, in Anderson and Tang, 1991)

BP — Beijing Parking Array
TS — Tangshan Experimental Array
TDSA — Three Dimensional (Space) Array
KD — Kangding Array
DX — Yunnan (West) Array or Dianxi Array

~~Peng, Kezhong~~ ~~TABLE 1~~

| Array Code | Array Types | Observ. Objects | Period | No. of Stations | No. of Deploying | No. of Maintenance (BSMOC/ Local) | No. of Events | No. of Records | Fig. of Arrays |
|---|---|---|---|---|---|---|---|---|---|
| BP Beijing Parking Array | TE, SR DA, PA | 2,3,5, 6,7,8 | 82.7– now | 28 | 28 | 462/0 | 0 | 0 | Fig. 1 Fig. 2 |
| TS Tangshan | SE, SM DA | 2,3,4, 5,7 | 82.7– 87.12 | 24 | 65 | 491/0 | 707 | 1622 | Fig. 3 |
| TDSA* | TD, SM | 2,3,4, 5 | 83.3– 87.12 | (8) | (33) | (60)/0 | (36) | (236) | Fig. 4 |
| KD | SE | 1,2,3, 4 | 82.12– now | 5 | 5 | 29/220 | 0 | 0 | Fig. 5 |
| DX | SE, LS SM | 1,2,3, 4,5,7 | 83.5– now | 20 | 29 | 192/627 | 15 | 24 | Fig. 6 |
| HR | MO | 1 | 82.12 | 3 | 3 | 12/0 | 1 | 2 | |
| LQ | MO | 1 | 85.4 | 1 | 1 | 10/0 | 1 | 1 | |
| Total | | | | 81 | 131 | 1196/847 | 724 | 1649 | |

NOTES:

Array Types:

TE – Topography Effect
SR – Structure Response
TD – Three Dimension Space
LS – Local Site Effect
SM – Source Mechanism and Wave Propagation
PA – Parking Array
DA – Dense Array
SE – Simple Extended
MO – Mobile Observation

Observ. Objects

1. For catching large earthquakes.
2. For measuring the source parameters and studying source mechanism.
3. For studying the nature of the rupture propagation.
4. For deriving the law of wave propagation and attenuation.
5. For the study of site effects.
6. For the study of soil-structure interaction.
7. For the study of variations of strong ground motions in a small area.
8. Parking array.

No. of deploying – No. of station-times deployed.
No. of maintenance – No. of station-times maintained.

* All the data with bracket of TDSA is included in TS.

Figure 11. Tabulation of seismic arrays in China listing array types, objectives, number of stations, etc. (After L-L Xie, in Anderson and Tang, 1991)

Figure 12. 30-ton seismic vibrator with 8-ton inertial mass used in Russia for generation of seismic waves for study of nonlinear soil properties on attenuation of waves.

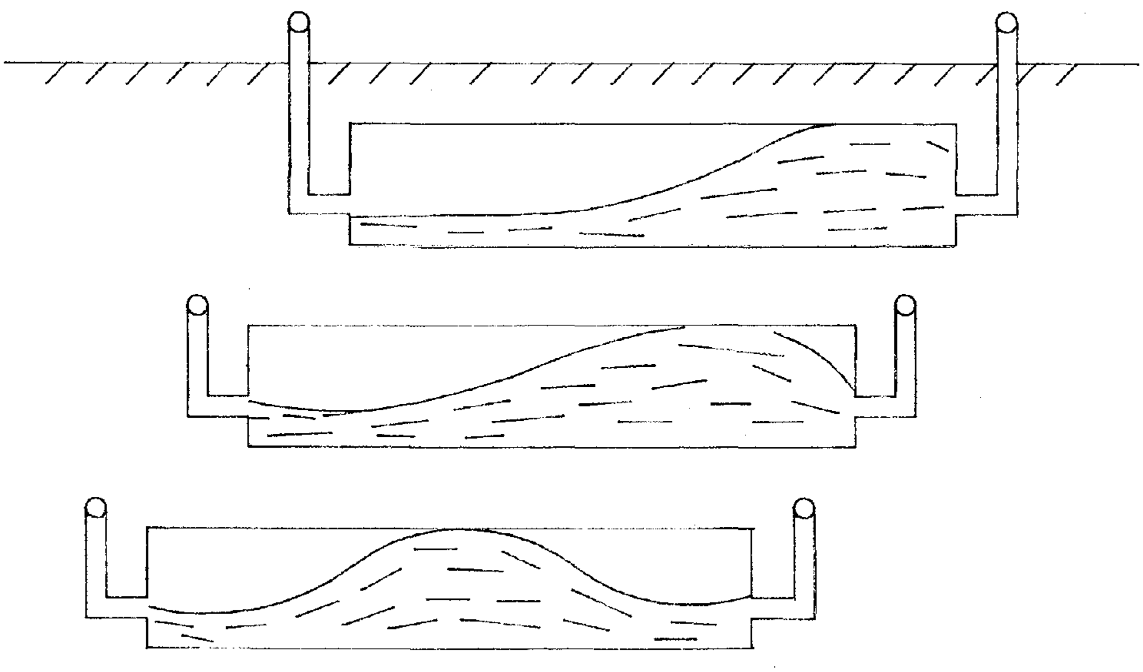
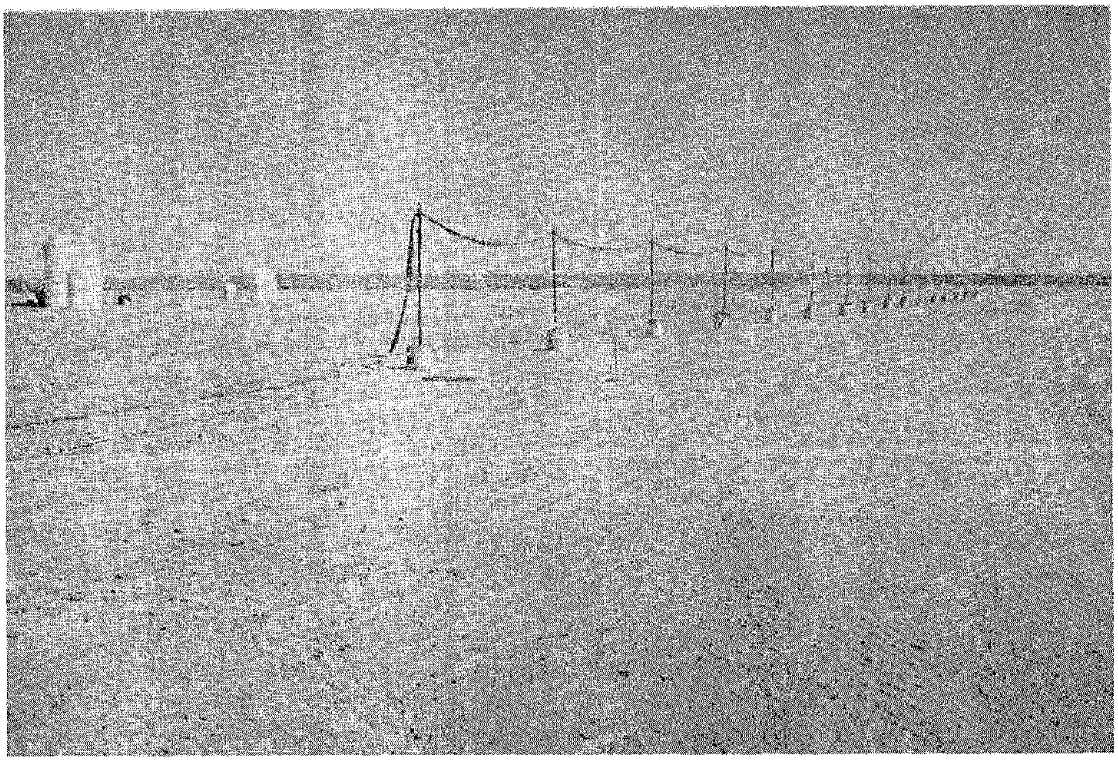


Figure 13. Diagrammatic sketch of Russian large (approximately 1-2 m) diameter tubes used to generate high amplitude, low frequency seismic waves for study of nonlinear soil properties on wave attenuation. The facility consists of several parallel tubes with an elaborate pumping facility to slosh water back and forth through the tubes to generate the waves. The facility is located in Siberia.

## 2.5 EXPLOSIVE SIMULATION OF EARTHQUAKE-LIKE GROUND MOTION

Cornelius J. Higgins
Applied Research Associates, Inc.
Albuquerque, New Mexico



Close-up View of Front Array (SIMQUAKE IB).
Cable Protection System with Detonation in Place.

## 1. HISTORICAL BACKGROUND

High explosive simulation is the use of conventional high explosives or propellants in various arrays and in combination with enhancement techniques to produce a wave propagation environment with earthquake-like ground motion amplitudes and frequencies. Although the use of nuclear explosions is feasible, they have not seen use probably because of restrictive nature.

Explosives have been investigated and utilized in the defense and blasting industries for their direct shock effects over a long period of time. Applications to earthquake environment simulation have come more recently, motivated mainly by the sparseness in space and time of actual large earthquakes. Rarely, if ever, have the right combination of measured input, damaging amplitude, and measured structural response occurred for a variety of interesting structures in a large earthquake. This situation has been partially remedied by the recent Lotung and planned Hualien experiments in Taiwan.

The first use of explosives to investigate seismic level responses of structures seems to be in Russia in the 1950's, where they used small high explosive blasts to evaluate the liquefaction potential of industrial and dam sites. America's scientists and engineers visiting the Soviet Union report a continuous experimental program at a test site near Dushanbe, Tadzhik S.S.R. In the late 1970's, the USGS participated in a cooperative test program with the Soviets which used single charges and sequentially fired explosions on a prototype multi-storey building (Ref. 1). In a recent visit, Professor T.L. Youd of Brigham Young University found that similar work is continuing and some new innovative source concepts are being developed.

U.S. work using explosives to evaluate the dynamic response of structures for vibration and seismic effects began in the 1950's and 1960's (e.g., Refs. 2, 3). In the early 1970's, a group at UCLA (Ref. 4) began to use explosives to simulate strong motion earthquake effects on nuclear power plants. In the early studies, the investigators used the output from explosions mainly to generate a dynamic excitation without special attention to control of the motion. Beginning in the mid 1970's, the NSF funded two programs designed to enable control of the ground motions to more closely resemble earthquakes. One program, at the University of New Mexico, addressed arrays of conventional buried explosions with potential enhancement techniques (Ref. 5) and improved prediction methods (Ref. 6). This prediction work was combined with DOD studies to develop a unified scaling approach for buried explosive arrays (Ref. 7).

The other NSF funded program, at SRI International, focused on the development of controlled explosive sources which could be used to create seismic excitation (Refs. 8, 9, 10). The sources were designed to be contained and reusable so that they could be used close to existing structures. In the course of the SRI program, they fabricated and evaluated a planar array of sources. SRI's most recent work (Ref. 11) has been concerned with achieving higher levels of displacement from their method by improving source design and modifying the test site to achieve a cantilever condition.

The only major experimental application of high explosive simulation in the U.S. was the EPRI sponsored SIMQUAKE series (Refs. 12, 13, 14). This series consisted of 4 events in alluvium: Mini-SIMQUAKE, SIMQUAKE IA, SIMQUAKE IB, and SIMQUAKE II; and one event in rock, SIMQUAKE III. In SIMQUAKE II, two arrays of explosive, one with about 40 tons and one with about 30 tons of ANFO, were detonated 1.2 seconds apart to load several structures including a nominal 1/8 size of a generic nuclear power plant. A later experiment, SIMQUAKE III, was conducted in rock at a Niagara Mohawk Power Company site in Northern New York State (Ref. 14).

## 2. METHOD DESCRIPTION

The basic dynamic ground motion input is obtained by the explosion of chemical (high explosive or propellant) or nuclear devices usually buried deep enough to achieve maximum coupling. Single point charges usually do not have the required combination of acceleration, velocity and displacement, overall frequency content, and shaking duration to provide a direct simulation of earthquake ground motions. As a result, several methods, separately or in combination, have been used to enhance the environment. Some of these methods include:

(1) the use of two-dimensional explosive arrays to reduce the attenuation rate associated with single point explosions and to provide some frequency and duration control,

(2) the use of sequentially-fired arrays to extend the time duration of motion,

(3) the use of barriers (relief trenches or shock shields) to obtain advantageous reflections and/or diffractions which can tailor motion amplitudes and/or durations, and

(4) the use of specially designed source devices, sometimes reusable, which increase energy coupling into the ground and control the motion amplitude and duration at the source.

The first two approaches are illustrated in Figures 1 and 2 which show a plan and cross section, respectively, of the SIMQUAKE II experiment. The drawings show the layout of the two explosive arrays and their position with respect to the structures. The back array (with respect to the structures) was fired first and the front array 1.2 seconds later to achieve a longer excitation duration.

Attempts to enhance motions by modifying the test area were first made in the defense community. Figure 3 shows the layout for an experiment which used relief trenches and relief holes around the test areas to increase the displacements (Ref. 15). Figure 4 shows the configurations for an SRI experiment on a small clay soil cantilever designed to evaluate displacement enhancement. The University of New Mexico (Ref. 5) investigated the use of massive barriers and screening trenches in front of test areas to reduce accelerations from explosive sources. It was found that the mass required to achieve significant acceleration reduction was impractical but that barrier trenches had potential. A barrier trench forces the incident waves to diffract around the trench and, as a result, the waveform diffuses, leading to reduced accelerations. Figure 5a illustrates the diffraction phenomena, while Figure 5b shows the results of finite difference calculations of the effect of different trench depths.

SRI's research focused on the design and fabrication of controlled, contained, repeatable sources for creating ground motion without major disruptions of the surrounding area. Figure 6 shows a single source used to produce three pulses of ground motion. The single sources can be arranged in arrays. Reference 11 describes the process as follows:

"Each source is 12 m (40 ft) long and 0.31 m (1 ft) in diameter. The source contains three vented steel canisters bundled inside a rubber bladder. For each pulse, a small amount of propellant (rifle powder) is burned in one canister to produce a source of high-pressure gas. The gas vents into the rubber bladder through specially designed vent plugs that redirect the flow of the pressurized gas away from the rubber bladder and adjacent canisters. The bladder expands against the soil, pushing it out. A valve at the top of the bladder opens to release the pressure after a specified time interval. The cycle of pressurizing and releasing pressure in the bladder produces a pulse of ground motion. Eight sources spaced at 2.4 m (8 ft) are arranged in a linear array to produce planar motion, ...."

Recent work at SRI has involved the redesign of the source to help achieve higher ground displacement. The source was modified from cylindrical to rectangular to increase the soil area directly pressurized. The new source is essentially an expandable box concept with a rubber bladder around a rectangular mandrel. The source is then surrounded by a steel frame to provide soil confinement at the free surface. Figure 7 shows a drawing, and Figure 8 a photograph, of a 3-m source.

With both conventional and special explosive sources, the explosive inputs are applied to the soil or rock at some distance from the structure or geotechnical system of interest. Waves then propagate through the media and subsequently excite the system of interest.

Substantial instrumentation is required to adequately characterize any major experiments. An explosive simulation test is a major experiment. Instruments must measure both soil and structure response. Free-field response must be defined with range, depth and transverse distance. Near-field response must be defined in the immediate vicinity (~ 2 diameters) of the structure. Soil response parameters of interest include triaxial kinematic response (a, v, d) which can be measured with some combination of accelerometers or velocity gages. The stress and strain tensor in the soil is also of interest for evaluating the wave types. Structural responses include rigid body translation, rocking, and torsion, plus deformational response of major structural elements. In addition, interface normal and shear stress measurements are desirable to define the input loads. These responses are measured by arrays of motion sensors, strain gages, interface stress gages, and special deformation sensors.

## 3. SIMULATION CRITERIA

Experiment design requires careful definition of simulation criteria. Simulation criteria, here, is defined to mean the level of fidelity that the simulation must achieve in matching prototype earthquake conditions. The UNM study (Ref. 5) considered various levels of simulation and concluded that the criteria would be system dependent.

The word simulation generally implies that the prototype environment cannot be generated at will and that some characteristics of the prototype environment may not be reproduced exactly in a simulation. It is necessary to determine those characteristics of the full-scale environment which are essential to adequately evaluate the system of interest. Certain features of earthquakes may be important for one structure but not for another. For example, above ground structures founded on soil may be adequately tested by simulating certain kinematic features of earth motion (acceleration, frequency content, duration) while below ground structures may require both kinematic and dynamic simulation (i.e., both motions and stresses).

If the dynamic characteristics of the structure are such that maximum response will be achieved at, say, one-fourth the duration of the earthquake, then it may not be necessary to simulate the complete duration. If the structure is not acceleration sensitive but velocity of displacement sensitive, then certain acceleration amplitude features of the prototype earthquake may be compromised while still achieving an adequate simulation. The major point here is that the adequacy of a simulation should be judged by the degree to which the response of the system of interest matches or yields insight into prototype response.

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. Both are incompletely understood. 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 insure precise duplication of structure response for a full-size prototype structure. Such a severe criteria specification would be economically impractical, technically difficult to achieve, and is probably inconsistent with the state-of-knowledge of the prototype environment.

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 insure 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, and

(4) some level and type of response in the structure.

For structures in which the incident stress system is not of major importance (i.e., where the structural strength is sufficient to withstand the incident stresses regardless of their distribution and type), then the specification of a motion time history at a point in the ground may be sufficient criteria. This would be the case where the ground motion excites structure base motion which, in turn, excites motion and stresses in other parts of the structure not directly loaded by the incident waves (internal or above ground components). This is a common problem in seismic design and is the one which most design codes treat for above ground structures. If only motion time history at a point is of interest, there is a wide range of wave types or combinations which can be used to produce it.

The least restrictive simulation criteria in terms of defining the waves, stresses, or ground motions is the use of one or more structural response parameters as measures of simulation. The response spectrum is a convenient tool for relating the characteristics of an input excitation to the response of a system. The SIMQUAKE series of experiments was designed using the shock spectrum approach.

## 4. CHARACTERISTICS OF DYNAMIC INPUT

Single explosions cause stresses and motions in the surrounding medium which are a function of the explosive type, size and depth of burst, and the nature of the medium, including layering. The amplitudes reduce exponentially with range from the source. The frequency content is a function of the source, especially size, and the medium.

Arrays of explosives can be used to alter the attenuation rates with range, change the frequency content of the motion, and produce a more uniform motion field over a larger region.

Sequential detonations with time can be used to reinforce certain frequencies and lengthen the duration of the pulse. Data and analysis methods from DOD research provide a good basis for predicting the ground motions. Figure 9, for example, shows the behavior of particle velocity versus range on a series of planar events. The data show a flat attenuation rate followed by a break to a steeper attenuation rate at a range which is dependent on the array dimensions. The break location is different for acceleration, velocity, and displacement. It is this behavior which can be used to design arrays suited to earthquake simulation. Procedures for these predictions have been documented in References 5, 6, and 7.

Example time histories have been selected from SIMQUAKE II to show strong ground motions. In this experiment, the arrays were designed to achieve amplitudes and frequency content on the order of those expected in a large 1/2 to 1 g earthquake, scaled to 1/8 to 1/12 size.

Figure 10a shows the free-field horizontal acceleration and its integrations at the 200-ft (61-m) range and 5-ft (1.52-m) depth. This position corresponds closely with the location of the largest (1/8-scale) structure. It can be seen that the double-array explosion caused about four cycles of excitation in both the acceleration and the velocity time histories. The ground motion 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 Hz to 2 Hz. These ground motions achieved a good simulation on the structure under a NRC 1/2 g to 1 g earthquake.

The vertical acceleration and its integrations at the same location are shown in Figure 10b. 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 one-half 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. The peak downward acceleration at the 200-ft (61-m) range is about 1 g. There does not appear to be a significant dwell time at this acceleration level and, hence, tensile failure in the soil has probably not occurred.

Figures 11a and 11b show the horizontal and vertical accelerations and their integrations at the 350-ft (106.7-m) range and 5-ft (1.52-m) depth. The motions become increasingly oscillatory with increasing range. The motion duration at this range is on the order of 3.5 s. The peak horizontal motions are 0.78 g, 0.54 m/s (21 in/s), and 0.072 m (2.8 in). The peak vertical acceleration is about equal to the peak horizontal acceleration, the peak vertical velocity is about 65% of the peak horizontal, and the peak vertical displacement is about 40% of the peak horizontal. The vertical record appears to contain higher frequency components than the horizontal record. The motions at the 350-ft range are probably more relevant for traditional civil design applications than the very strong motions used for nuclear power plants.

Figure 12 compares the SIMQUAKE II horizontal spectra at the 200-ft (61-m) range and 5-ft (1.52-m) depth with 1/8-scaled prototype spectra based on the Newmark, Blume and Kapur approach (Ref. 16). The motion spectrum shows strong frequency content in the 0.8 to 3 Hz range. The comparison with the prototype spectra suggests that the 1/8-scale structure was excited at levels above those of a 1/2-g earthquake for frequencies below about 4 Hz.

An example of ground motions produced by the original SRI technique is shown in Figure 13. These records were obtained on the ground surface 6.1 m (20 ft) from the center of an array of 8 sources of the type shown in Figure 6. Three pulses of ground motion were produced with a delay of 0.25 s between the pulses. The peak ground motions were about 0.1 g for acceleration, 2.0 cm/s (0.80 in/s) for velocity, and 0.10 cm (0.04 in) for displacement. Reference 11 states that the maximum ground motion produced (in other experiments) with the sources shown caused peaks of 0.3 g acceleration, 5 cm/s (2 in/s) velocity, and 0.2 cm (0.08 in) displacement.

A major difference between actual earthquakes and explosive simulations is in the wavefields causing the ground motions. The horizontal motion in earthquakes has traditionally been thought to be caused mainly by upwardly propagating horizontally polarized shear (SH) waves. Until the San Fernando earthquake, where bridge decks were lifted off their supports, very little attention was paid to vertical components. In the Imperial Valley earthquake of 1979, measurements made near the source showed very high vertical components, in one case about 1.5 g upward. It appears that earthquake fields are quite complicated, especially in the strong near-in region. We still do not have adequate seismic arrays which can carefully define the motion fields with depth, range, and transverse locations.

The motion fields from explosions are better understood. Figure 14 shows the wavefronts which would theoretically result from a single planar array if the site were a perfect elastic medium. These wavefronts were determined by considering the geometry of the source and the presence of the free surface. The figure shows three primary types of body waves: compressional (P), shear (S), and von Schmidt-like (SP) waves, which provide continuity between the P and S waves and have features similar to both. The particle motions associated with the waves are shown on the wavefronts. The actual wavefronts will be influenced to some degree by material nonlinearity, inelastic behavior, and layering. Figure 15 shows typical velocity waveforms near the surface and at depth measured during a planar experiment in dry alluvium. Horizontal and vertical velocities are shown with directional orientation as noted on each ordinate. The waveforms are divided by vertical lines into temporal regions where a particular wave or waves are thought to be dominating motion. These regions have been identified by observing direction and timing of particle motion.

The dissection of the planar wavefield demonstrates two important points. The first is that the initial motion everywhere is due to the initial P-wave. At the surface it causes initial upward motion. Even at distant ranges, where the P-wave travels almost parallel to the ground surface, there is initial upward motion due to a Poisson expansion necessary to meet the boundary conditions. The second major point is that a significant amount of the horizontal motion in the near-surface region is caused by the Von Schmidt wave (SP1) which has characteristics similar to those of an SH wave. That is, the SP1 wave is traveling upward while causing mainly horizontal motion.

Although we have some insight into earthquake and explosive wavefields, it will take some time before we can discern the importance of the differences to system response. Cursory analyses performed in Reference 12 suggest that a relatively rigid body in a soil media will tend to filter specific wave type effects. Particle motion into a rigid surface will cause loads and reflect as if it were P-wave induced regardless of whether the incident wave is a P or S wave. Likewise, particle motion parallel to a rigid boundary will cause loads and reflect as if it were an S-wave regardless of the incident wave causing the particle motion.

## 5. APPLICABILITY TO GEOTECHNICAL SYSTEMS

Explosive simulation is especially applicable and was developed for geotechnical systems where the soil or rock either makes up or surrounds the structure and is the medium through which the load travels and is applied. A large part of the geologic medium must be included in the experiment. In addition, the large source size possible with explosive simulation can load large structures of any type. The method has no inherent size restrictions. Because some systems have a strong gravity dependence, the larger the scale, the better the fidelity of the simulation.

## 6. EXPERIENCE

Explosive simulation using conventional explosives in planar arrays has been applied to nuclear power plant contaminant models in the SIMQUAKE series of experiments. These experiments were mentioned in paragraph 1 and are reported in References 12, 13, and 14. Figures 1 and 2 showed the plan and elevation of SIMQUAKE II. The main objective of the SIMQUAKE experiments was investigation of soil-structure interaction, especially rocking behavior, for massive, relatively rigid, partially embedded structures. The conduct of the experiments encompassed all of the major tasks involved in a major experiment including:

- Development of Simulation Criteria
- Explosive Array Design
- Structural Design
- Empirical and First Principal Pre-Test Predictions
- Instrumentation Layout and Selection
- Data Acquisition and Reduction
- Analysis

In SIMQUAKE II, six structures with variations in size, embedment, backfill type, and ground motion amplitude were tested. One hundred forty-five (145) active measurements were made.

Figure 16 shows a general view of the site. Figure 17 shows the near-field and structure instrumentation for the largest structure (1/8 scale). Typical response data are illustrated by Figures 18 and 19 which show measurements which quantify the amount of rocking experienced by this structure. Typical spectra comparisons are shown in Figure 20.

The SRI contained explosion research has been mainly devoted to the design, fabrication, and evaluation of alternative devices and planar arrays. A one-storey model structure having a first mode frequency of 5.7 Hz was tested using an array of eight 12-m (40-ft)-long sources, each capable of producing three pulses (Ref. 10). At a standoff of 20 ft, the free-field response was 0.25 g acceleration, 1.4 in/s velocity, and 0.071 in displacement. The corresponding structure motions in the base were 0.21 g, 2.4 in/s and 0.085 in. Reference 10 concluded that very little soil-structure interaction occurred at the level of excitation.

## 7. STRENGTHS AND WEAKNESSES OF THE METHOD

The main strengths and weaknesses of the conventional and contained explosion simulation approaches are listed below.

a. Conventional Buried Explosions

Strengths

(1) Large motion (strain) amplitudes are possible.

(2) Large structures can be tested.

(3) Environment is a wave propagation environment containing wave interactions as complex as in earthquakes.

(4) Full interaction without boundary interference can occur.

(5) Environment (amplitude and frequency) control is possible through sequenced firing and enhancement techniques such as gas venting control, array shape variations, and relief trenches and barriers.

(6) Can be fully instrumented.

Weaknesses

(1) Wave types differ from earthquake wave types.

(2) Single detonations have limited duration.

(3) Access to large test areas containing soils of interest is required.

(4) Large tests create craters and ejecta which must be evaluated and require safety plans.

b. Contained Explosive Sources (RESCUE)

Strengths

(1) Can be used near existing structures.

(2) Devices can be reused.

(3) Smaller quantity of explosive/propellant used.

(4) Minimal disruption of site.

(5) Environment is a wave propagation environment containing wave interactions as complex as in earthquakes.

(6) Full interaction without boundary interference can occur.

(7) Environment (amplitude and frequency) control is possible through sequenced firing and enhancement techniques such as gas venting control, array shape variations, and relief trenches and barriers.

(8) Can be fully instrumented.

Weaknesses

(1) Restricted to small displacements and concomitantly high frequencies due to small amount of explosive/propellant.

(2) Relatively high expense of device.

(3) Wave types differ from earthquake wave types.

(4) Single detonations have limited duration.

8. REQUIRED DEVELOPMENT

A partial list of research required to improve the effectiveness of explosive simulations follows:

(1) Investigation of the use of high explosives for simulation purposes should be continued, especially with regard to the control of frequency content and duration.

(2) Experimental investigations should be expanded to include more fundamental investigation of such enhancement techniques as sequential firing, relative array location (on one side of test area or on opposite sides), and barrier trenches.

(3) Theoretical and experimental investigations should be initiated to examine potential methods for generating shear wave excitations (e.g., excite a near-surface hard layer to generate an SV head-wave in the upper soil layer).

(4) Evaluate alternative sources, e.g., Soviet idea using water in pipes to generate shear waves (See T.L. Youd's paper in these proceedings).

(5) Experimental needs and simulation criteria for various generic geotechnical systems should be established.

(6) A steering committee should be formed to plan and direct needed large-scale experiments. Since a major cost of an experiment is related to the environment and free-field instrumentation, it appears reasonable to plan projects which provide a basic environment and free-field instrumentation. To this basic program, more specific projects may be added (and funded) by federal and private agencies.

9. UNCERTAINTIES

Major uncertainties in the use of high explosive simulation revolve around the difference in the wave fields between actual earthquake and explosive simulations. Further analysis is needed to define these differences and their importance for various geotechnical systems. Scale also introduces uncertainty because of the importance of gravity forces in many geotechnical applications. This can be evaluated with carefully designed experiments and analytical studies.

There are other uncertainties associated with predicting the ground motion environments from explosive simulations. There is an inherent uncertainty of $\pm$ a factor of 2 in ground motion data in repeated experiments due to site inhomogeneities, source variability, and instrumentation error. This uncertainty can grow substantially at new sites due to material property uncertainties. The use of laboratory properties can lead to large errors in initial predictions. It is prudent to perform in-situ calibration tests before major experiments at a site.

## REFERENCES


1. Rojahn, C. and Negmatullaev, S.H., "Forced-Vibration Tests of a Three-Story Reinforced Concrete Frame and Shear-Wall Building in Tadzhik, S.S.R.," Proceedings, ASCE, Conference on Dynamic Response of Structures: Instrumentation, Testing Methods, and System Identification, University of California at Los Angeles, March 1976.

2. Hudson, D.E., Alford, J.L., and Iwang, W.D. , "Ground Accelerations Caused by Large Quarry Blasts," Bull. Seism. Soc. Am., 51 (2), April 1961.

3. Hudson, D.E., Alford, J.L., and Housner, G.W., "Measured Response of a Structure to an Explosive-Generated Ground Shock," Bull. Seism. Soc. Am., 44 (3), July 1954.

4. Chrostowski, J., et al, Simulating Strong Motion Earthquake Effects on Nuclear Power Plants Using Explosive Blasts, UCLA-34P193-10, UCLA-ENG-7119, Nuclear Energy Laboratory and Earthquake Engineering and Structures Laboratory, School of Engineering and Applied Science, University of California at Los Angeles, February 1972.

5. Higgins, C.J., Johnson, R.B., and Triandafilidis, G.E., The Simulation of Earthquake-Like Ground Motions with High Explosives, University of New Mexico report on Grant ENG 75-21580 to the National Science Foundation, August 1978.

6. Steedman, D.S. and Higgins, C.J., Prediction of Free Field Motions in Large-Scale Earthquake Simulations Using High Explosives, Applied Research Associates, Inc. report on Grant PFR-7923500 for the National Science Foundation, October 1982.

7. Drake, J.L. and Higgins, C.J., "A Method for Scaling Explosion-Produced Ground Motion from Various Buried Source Configurations," Proceedings of the ASCE/ASME Conference on Response of Geologic Materials to Blast Loadings and Impact, Albuquerque, New Mexico, June 1985.

8. Abrahamson, G.R., Lindberg, H.E., and Bruce, J.R., Simulation of Strong Earthquake Motion with Explosive Line Source Array, Final Report prepared for the National Science Foundation, SRI Project 6004 (1977).

9. Bruce, J.R., Lindberg, H.E., and Abrahamson, G.R., Simulation of Strong Earthquake Motion with Explosive Line Source Arrays, Final Report prepared for the National Science Foundation, SRI Project 7556 (1979).

10. Simons, J.W., Lin, A.N., and Lindberg, H.E., Dynamic Testing of a Soil-Structure System Using the Technique of Repeatable Earth Shaking by Controlled Underground Expansion (RESCUE), Final Report prepared for the National Science Foundation, SRI Project 4644 (1985).

11. Simons, J.W. and Gefken, P.R., Improvements to the RESCUE Technique for Dynamic Testing of Soil-Structure Systems, Final Report prepared for the National Science Foundation, SRI Project PYU-2134 (1990).

12. Higgins, C.J., Simmons, K.B., and Pickett, S.F., SIMQUAKE I - - An Explosive Test Series Designed to Simulate the Effects of Earthquake-Like Ground Motions on Nuclear Power Plant Models, Volume I: Summary, EPRI NP-1728, Electric Power Research Institute, Palo Alto, California, February 1981.

13. Higgins, C.J., Simmons, K.B., and Pickett, S.F., SIMQUAKE II - - A Multiple Detonation Explosive Test to Simulate the Effects of Earthquake-Like Ground Motions on Nuclear Power Plant Models, EPRI NP-2916, Electric Power Research Institute, Palo Alto, California, October 1983.

14. Dass, W.C., Higgins, C.J., and Labreche, D.A., Support of the NMPC/EPRI Nine Mile Point Explosive Structure-Media Interaction Experiments, Applied Research Associates, Inc. report to the Electric Power Research Institute, June 1984.

15. Schlater, D.R., DIHEST Improvement Program Test DIP IIIA, Data Report, AFWL-TR-74-16, Air Force Weapons Laboratory, Kirtland AFB, New Mexico, April 1974.

16. Newmark, N.M., Blume, J.A., and Kapur, K.K., "Seismic Design Spectra for Nuclear Power Plants," Journal of the Power Division, ASCE, Vol. 99, No. P02, American Society of Civil Engineers, November 1973.

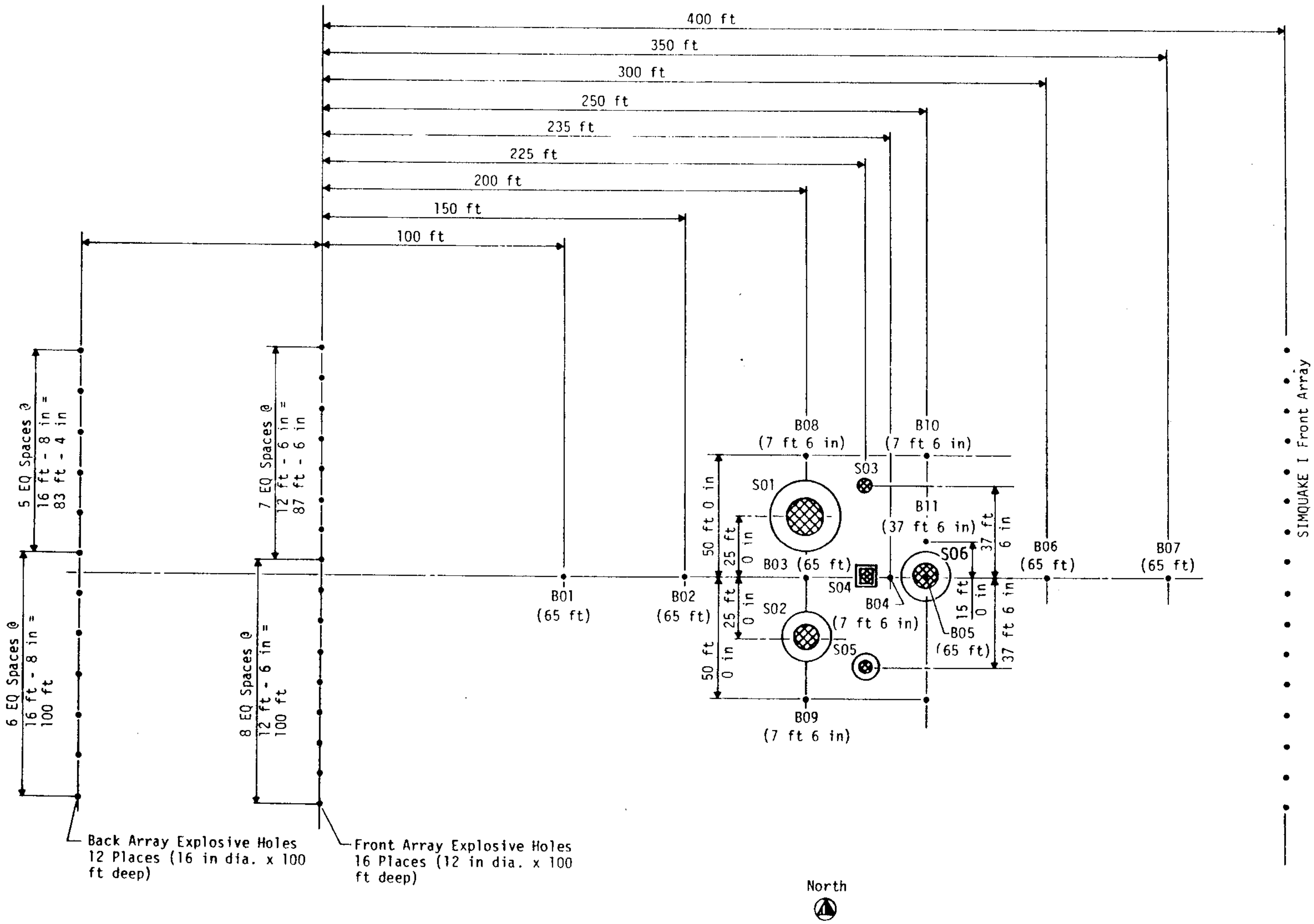



Figure 1. SIMQUAKE II Test Bed Layout (Ref. 13).

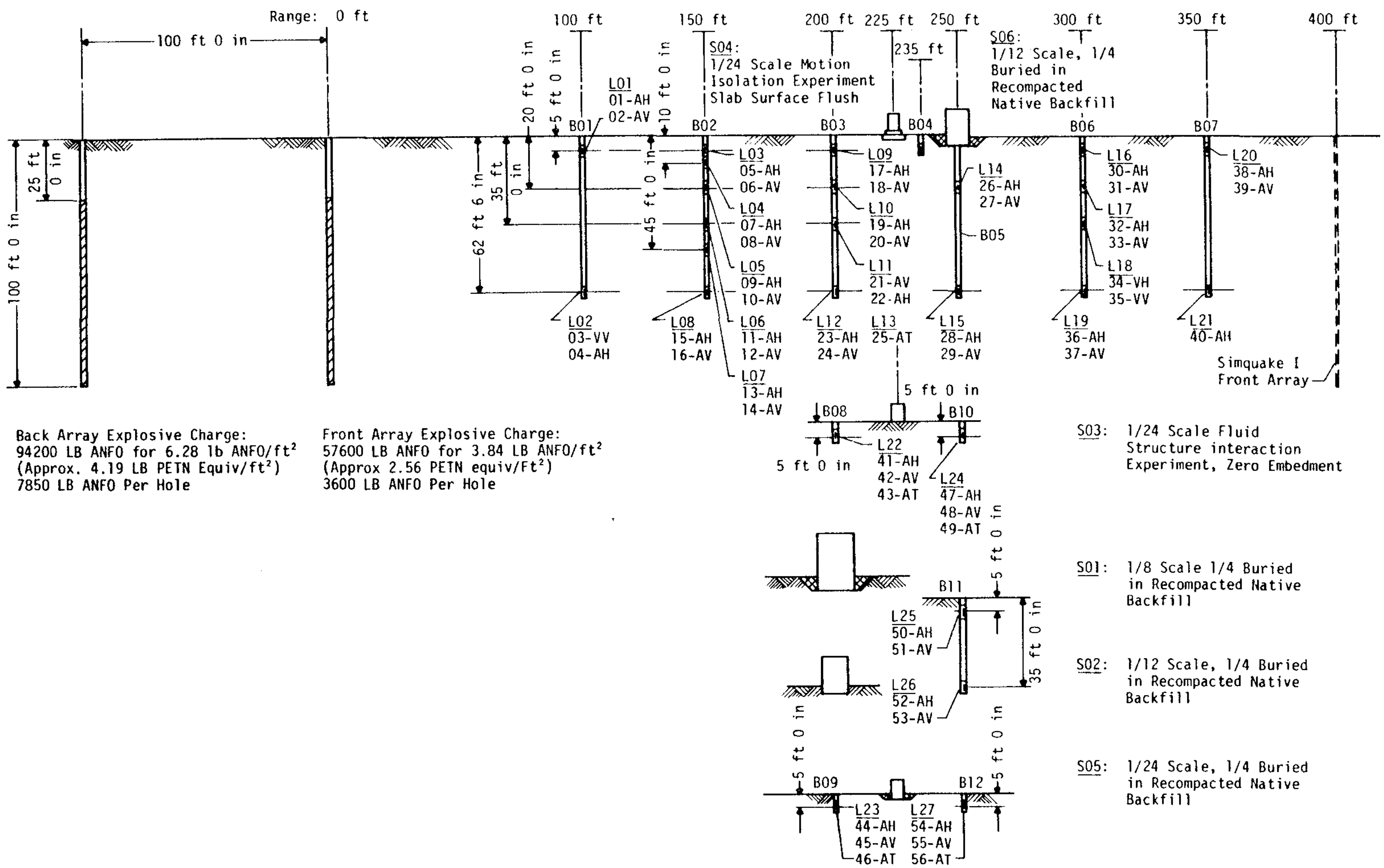



Figure 2. SIMQUAKE II Elevation (Ref. 13).

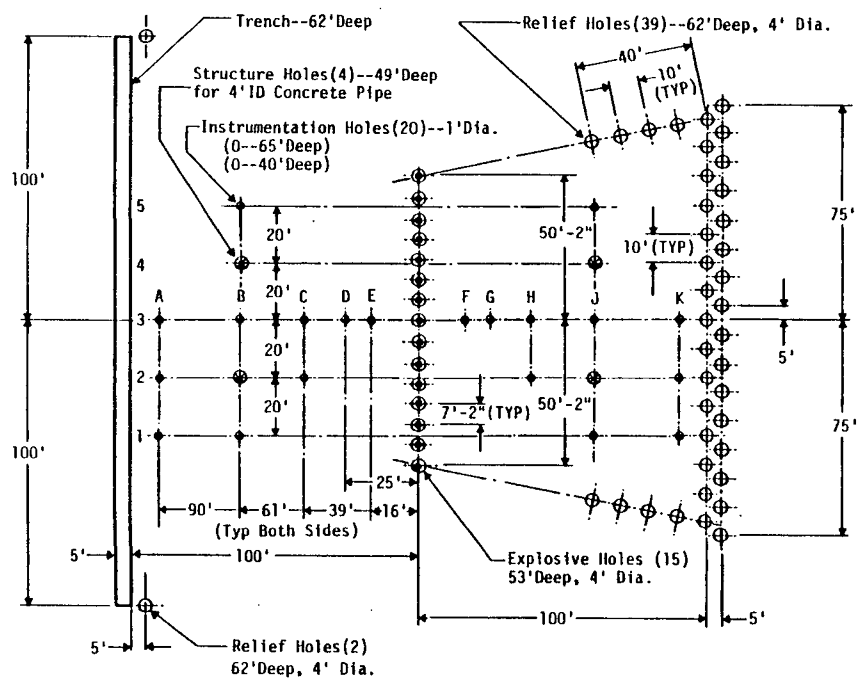



Figure 3. DIP IIIA Layout Showing Relief Trench and Relief Holes (Ref. 15).

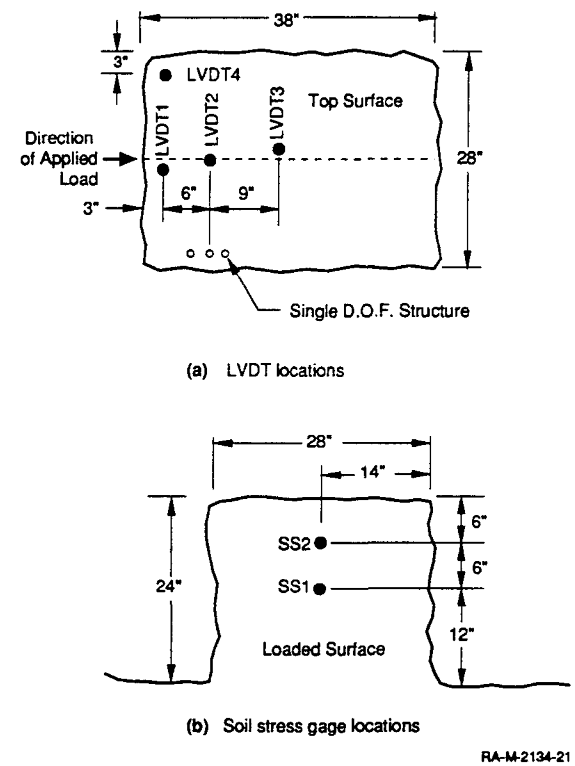



Figure 4. SRI Soil Cantilever Experiment (Ref. 11).

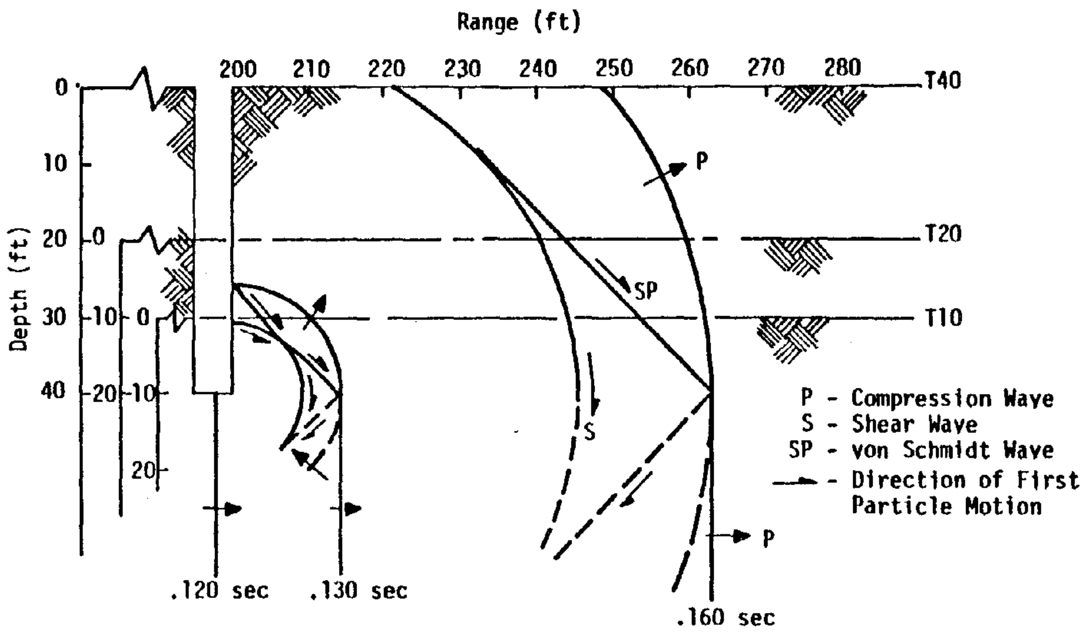



Figure 5a. Idealized Primary Wavefronts Initiated at the Bottom of a Trench Assuming Planar Incident Wave (Ref. 5).

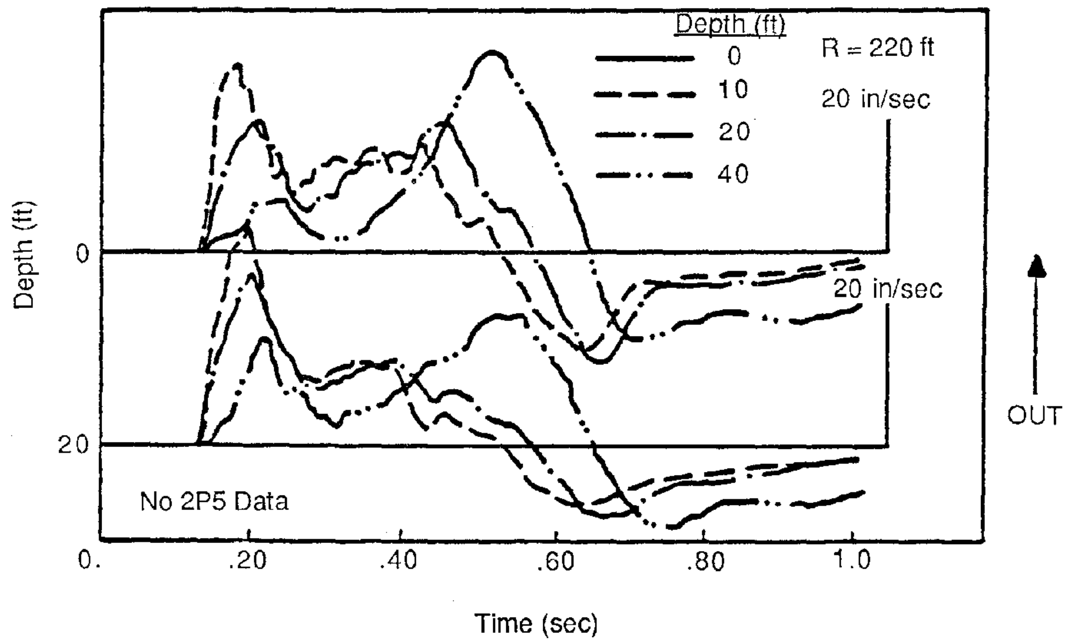



Figure 5b. Horizontal Velocity History Comparisons Trench Screening Calculations (Ref. 5).

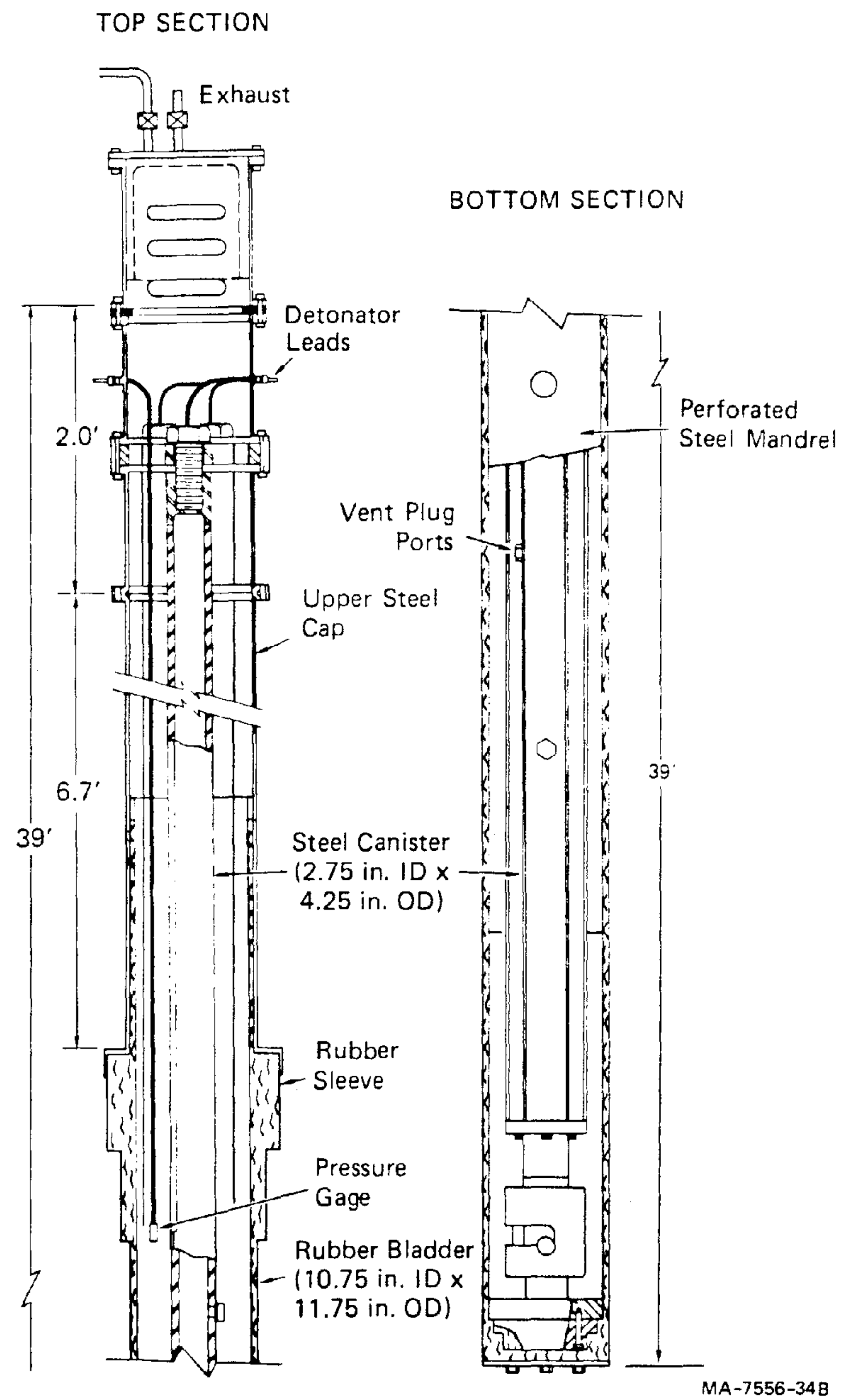
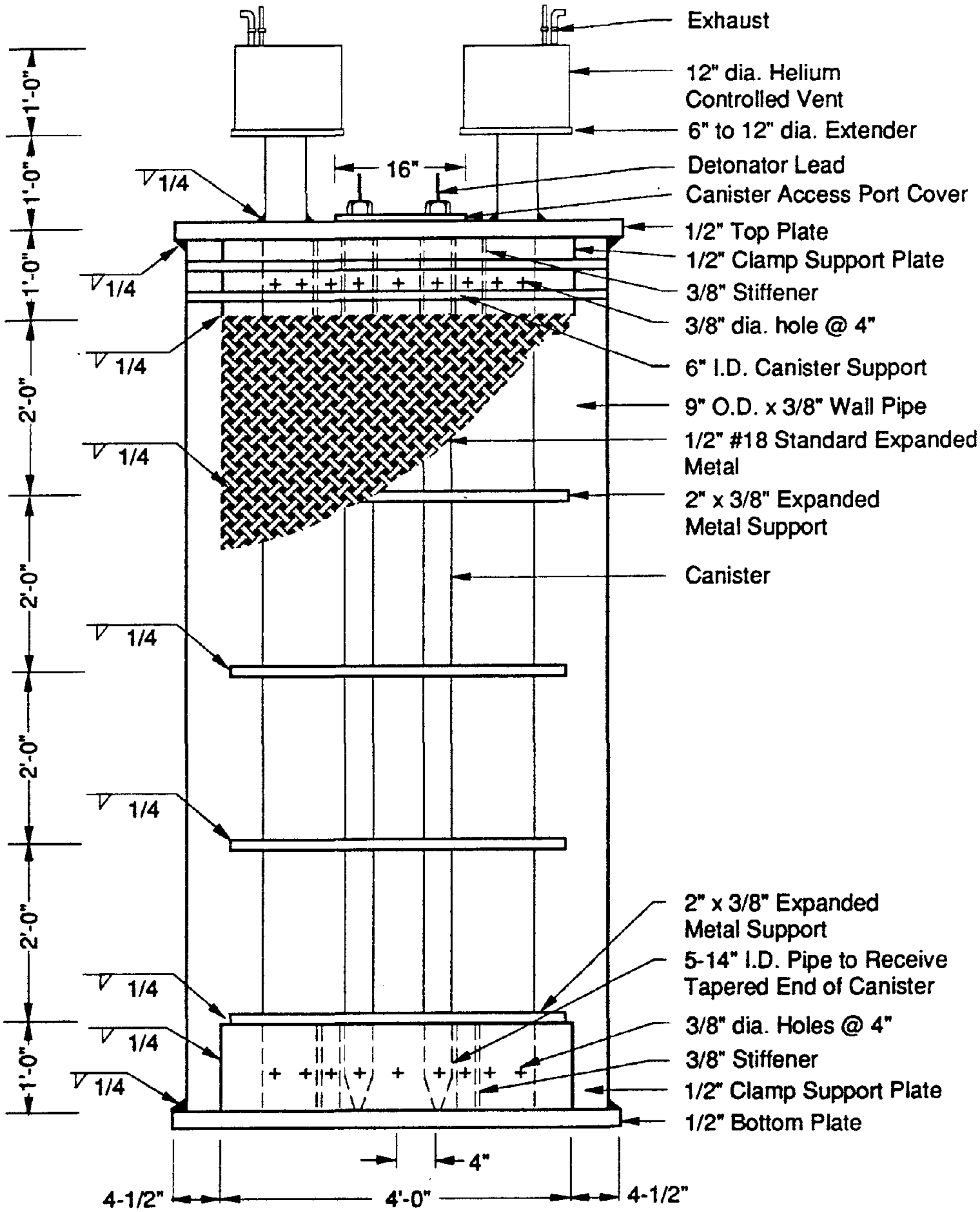



Figure 6. Line Source (Ref. 11).




(a) Source details

RM-2134-39

Figure 7. Plans for 3-m Source (Ref. 11).

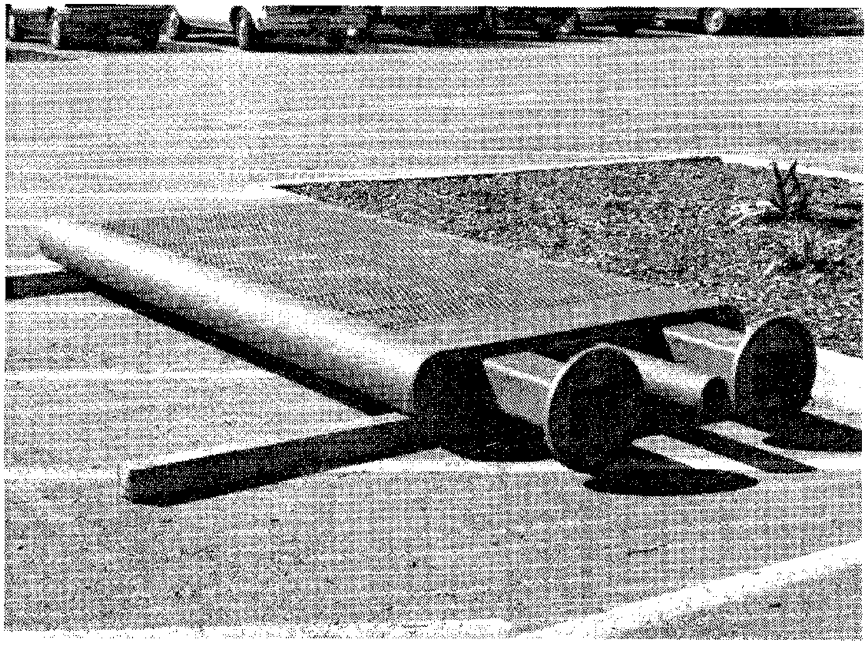


Figure 8. Photograph of 3-m Source During Fabrication (Ref. 11).

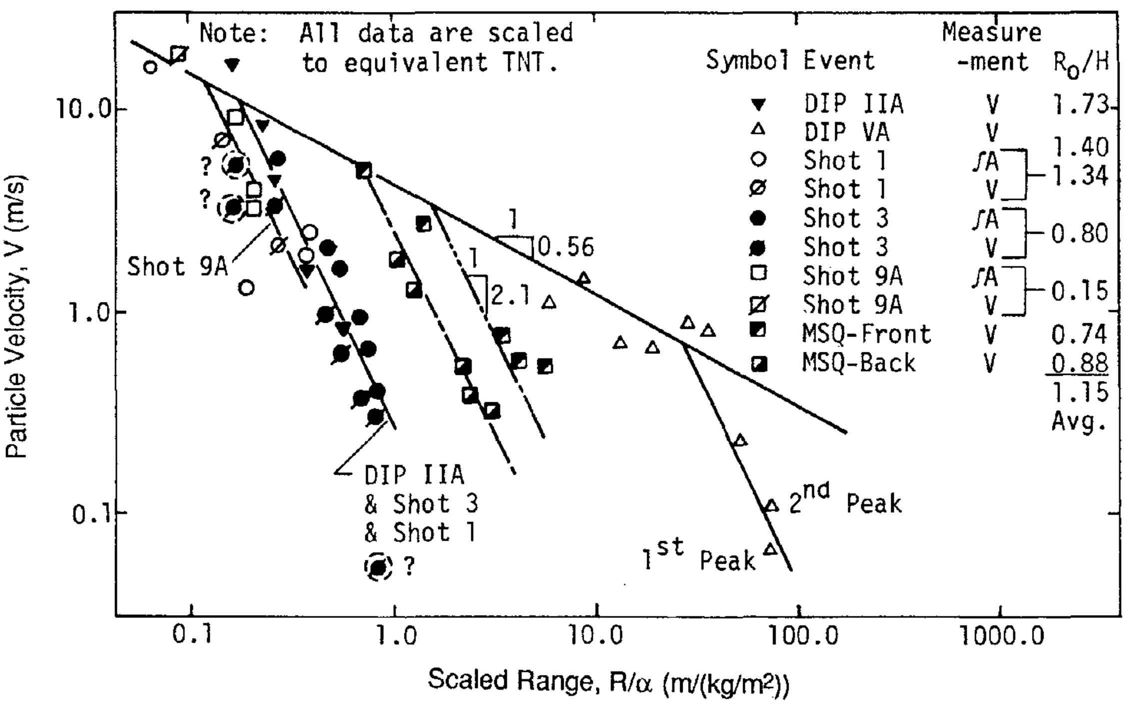



Figure 9. Peak Horizontal Velocity Versus Range on the Centerline of Planar Events (Ref. 5).

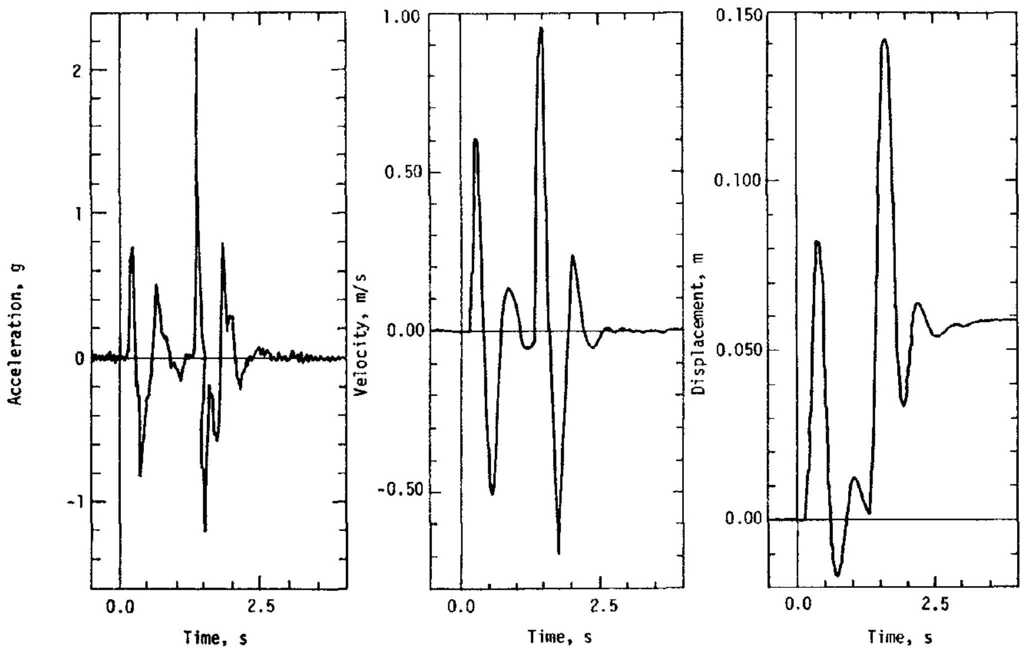



Figure 10a. Horizontal Motions at 61-m (200-ft) Range and 1.53-m (5-ft) Depth on SIMQUAKE II (Ref. 13).

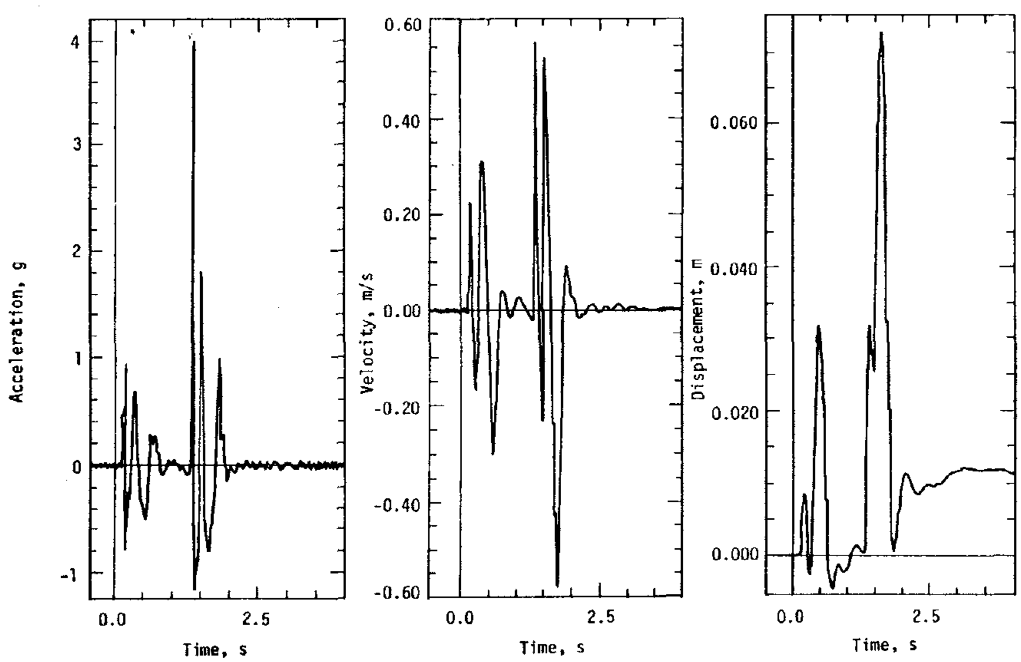



Figure 10b. Vertical Motions at 61-m (200-ft) Range and 1.52-m (5-ft) Depth on SIMQUAKE II (Ref. 13).

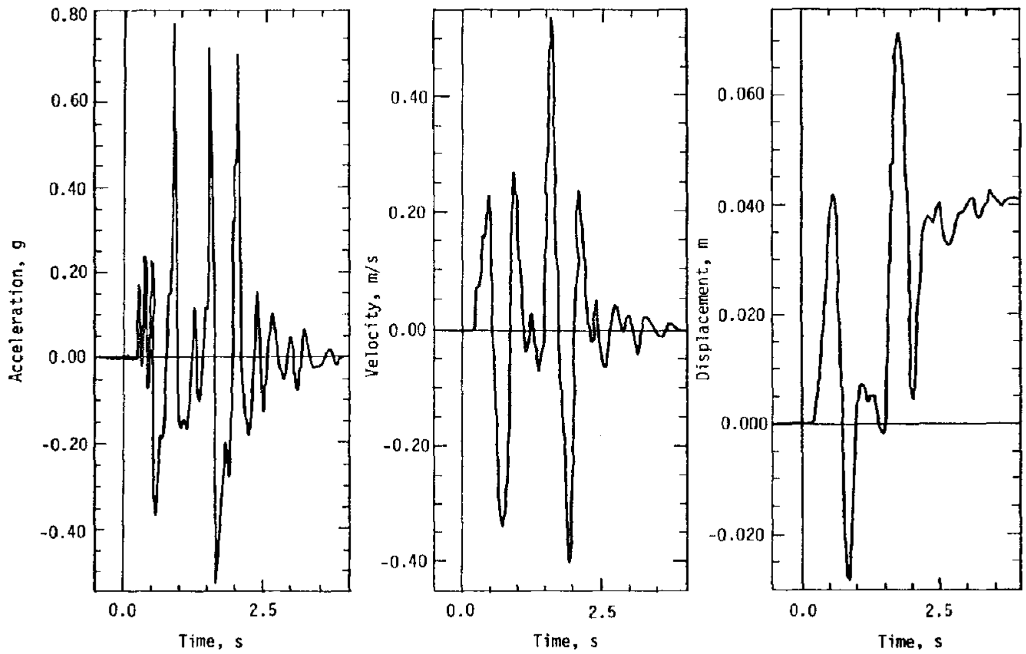



Figure 11a. Horizontal Motions at 106.7-m (350-ft) Range and 1.52-m (5-ft) Depth on SIMQUAKE II (Ref. 13).

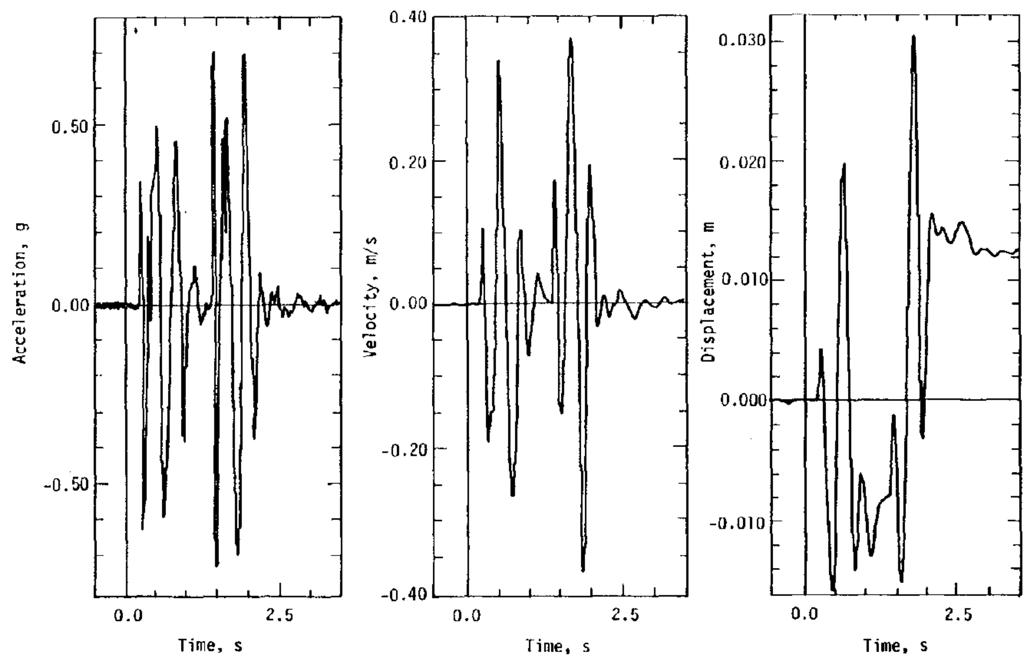



Figure 11b. Vertical Motions at 106.7-m (350-ft) Range and 1.52-m (5-ft) Depth (Ref. 13).

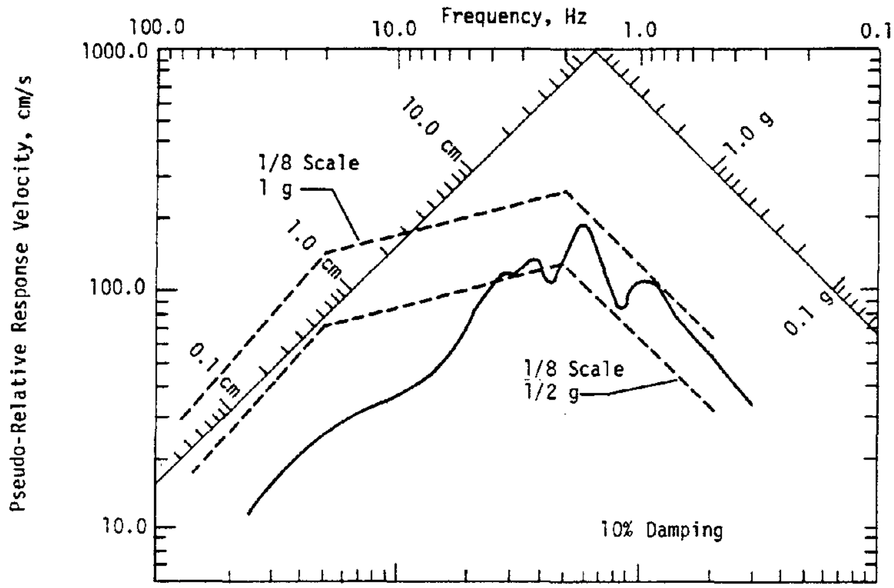



Figure 12. Comparison of SIMQUAKE II Response Spectra at 61-m (200-ft) Range and 1.52-m (5-ft) Depth With Appropriately Scaled Prototype Spectra Based on Reference 16 (Ref. 13).

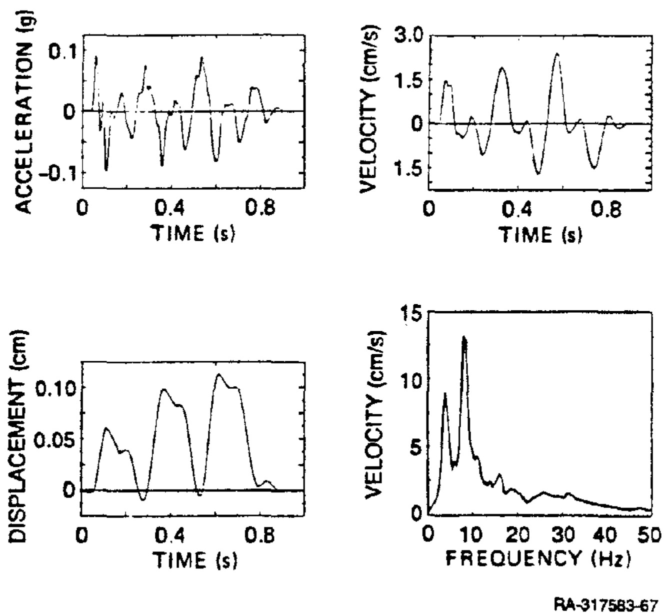



Figure 13. Ground Motion From Test FF-1 (Ref. 11).

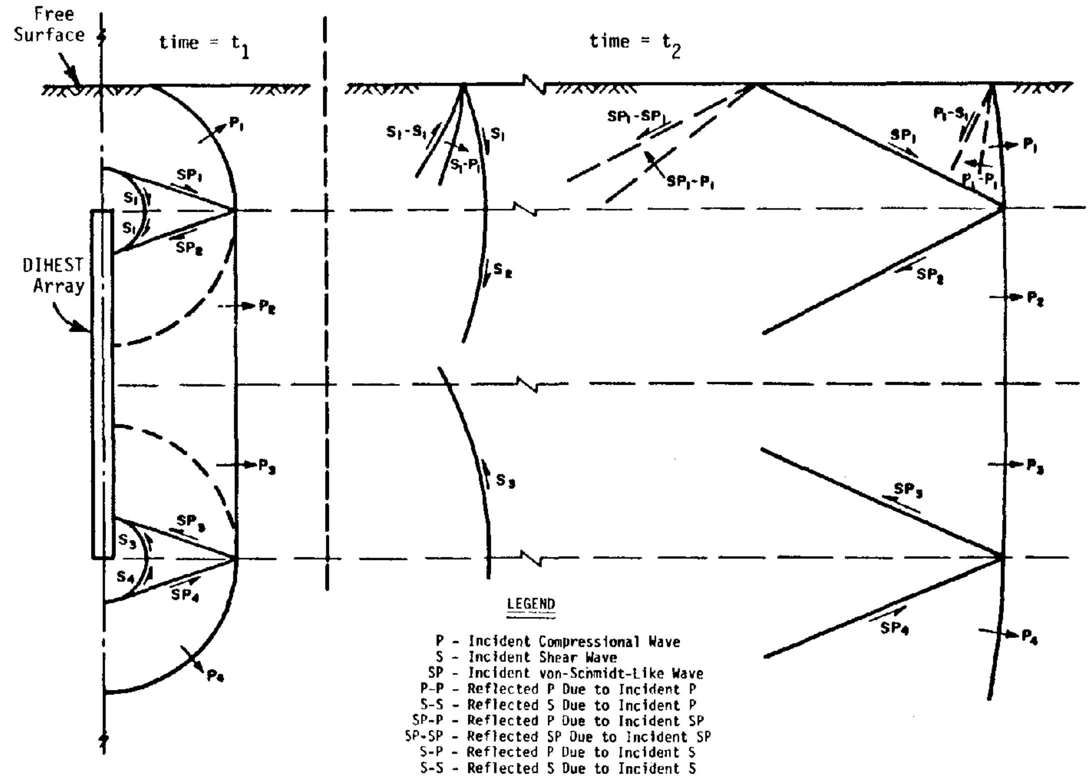



Figure 14. Incident and Reflected Wavefronts from a Planar Array.

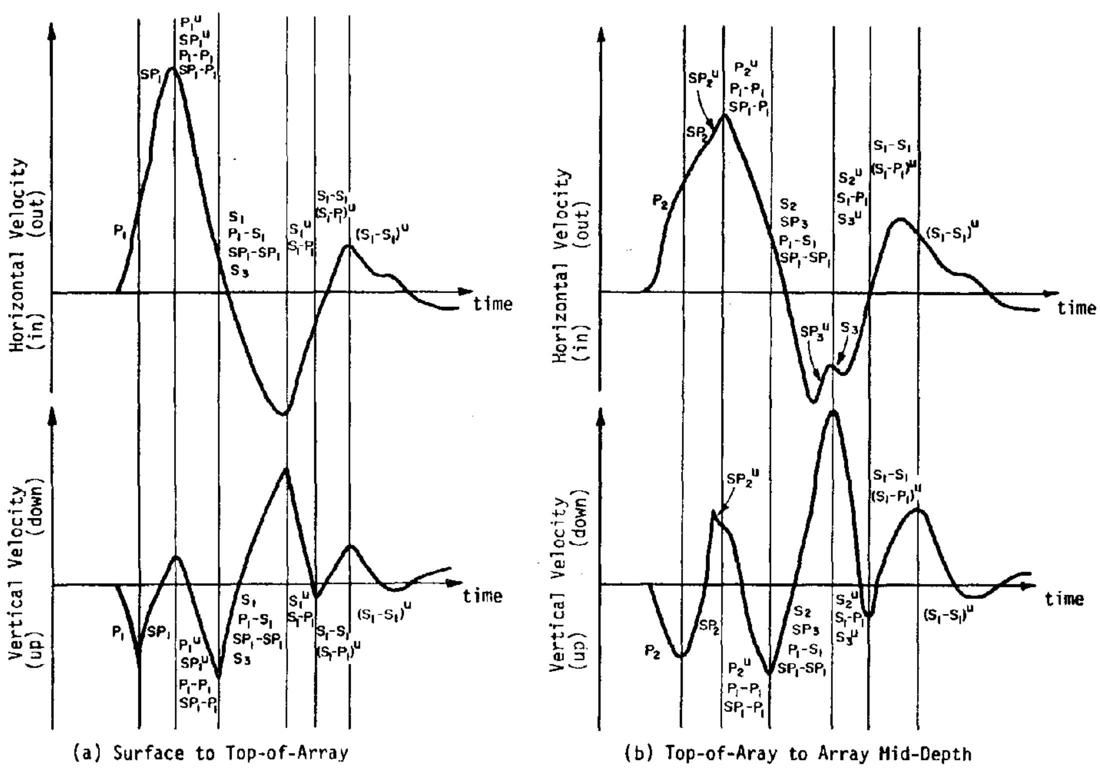



Figure 15. Composition of Typical Waveforms from a Single Planar Array.

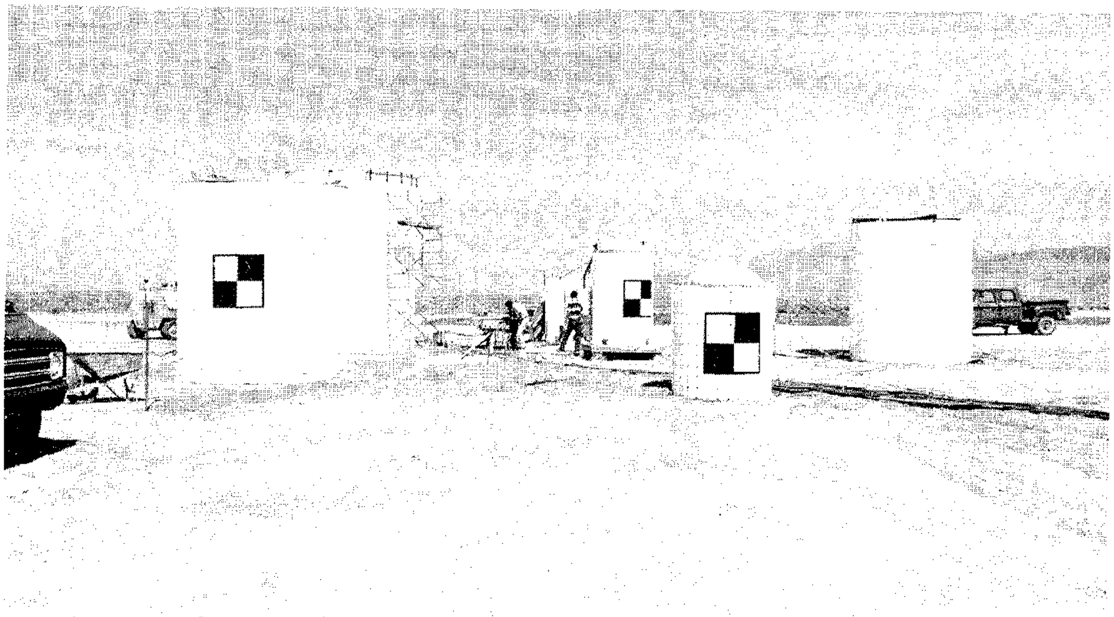


Figure 16. General Site View. Explosive Arrays are to the Left. Markings on Structures are Camera Targets.

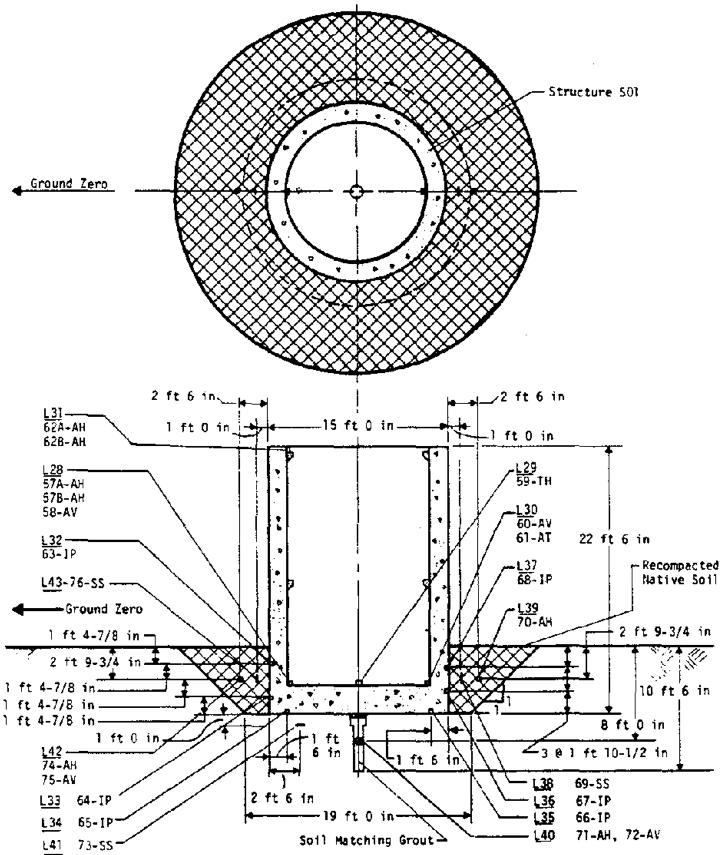
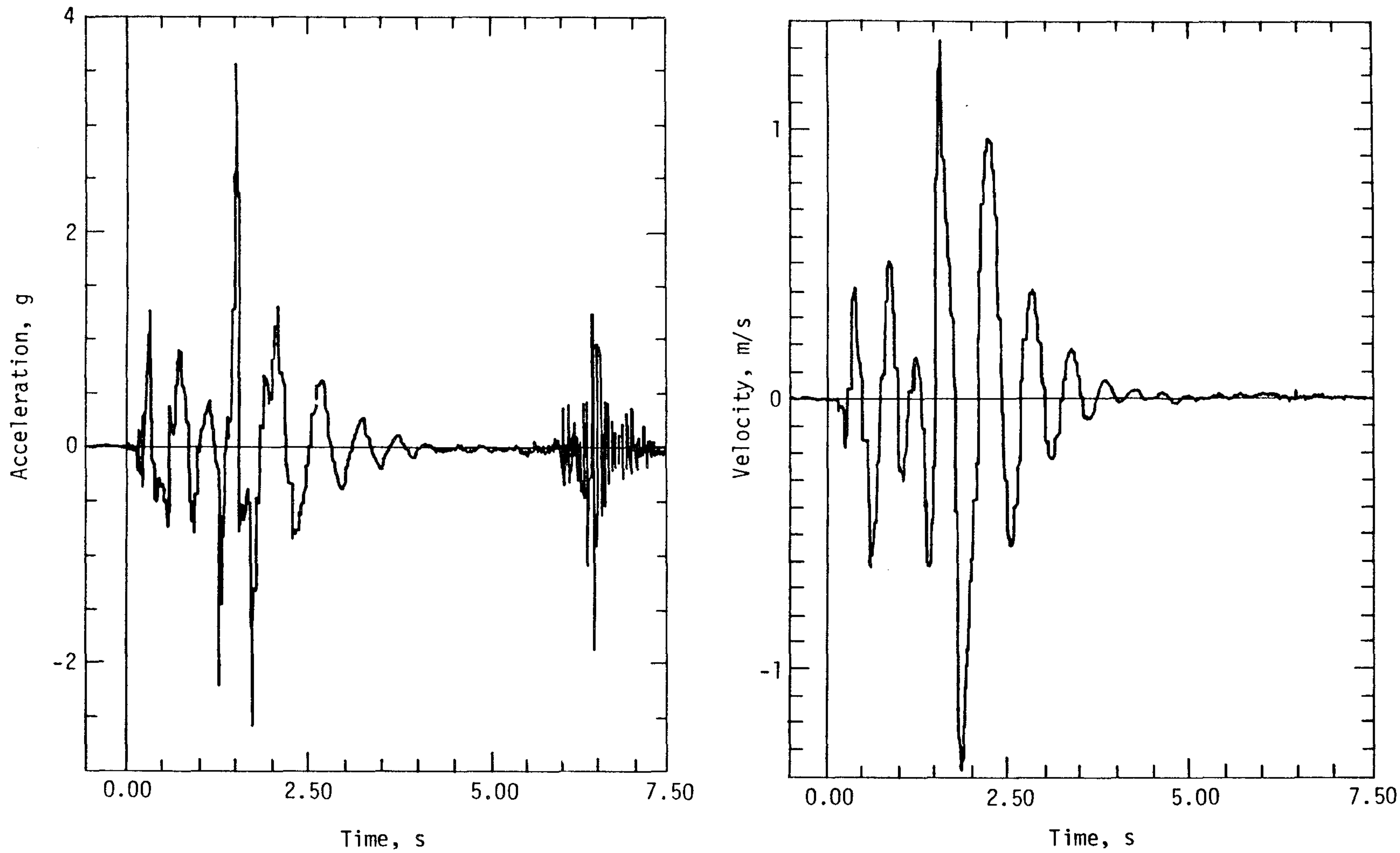



Figure 17. Structure S01 Near-Field and Structure Instrumentation.




(a) Differential Acceleration

(b) Differential Velocity

Figure 18. Differential Acceleration and Velocity Between the Top and Base of the 1/8-Scale Structure on SIMQUAKE II.

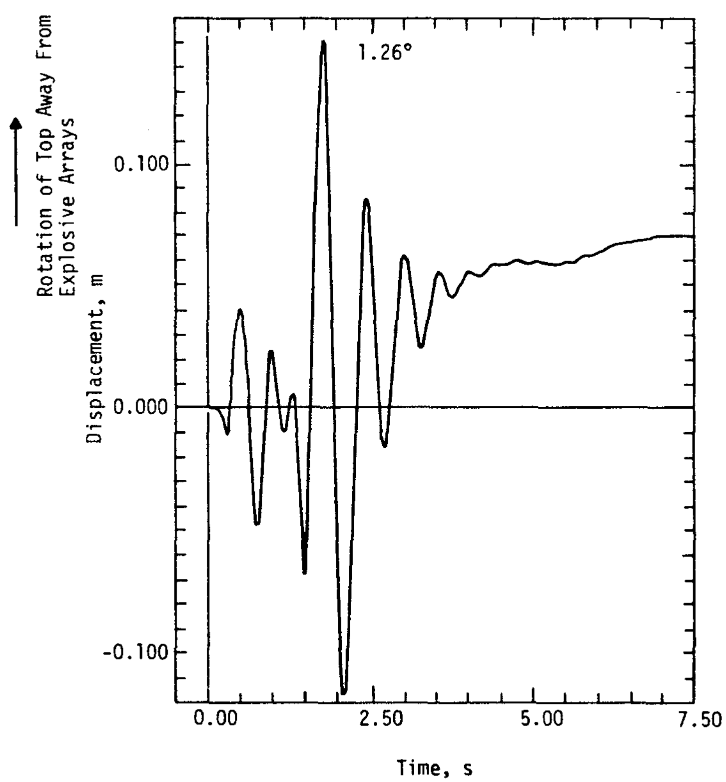



Figure 19. Differential Displacement Between the Top and Base of the 1/8-Scale Structure on SIMQUAKE II.

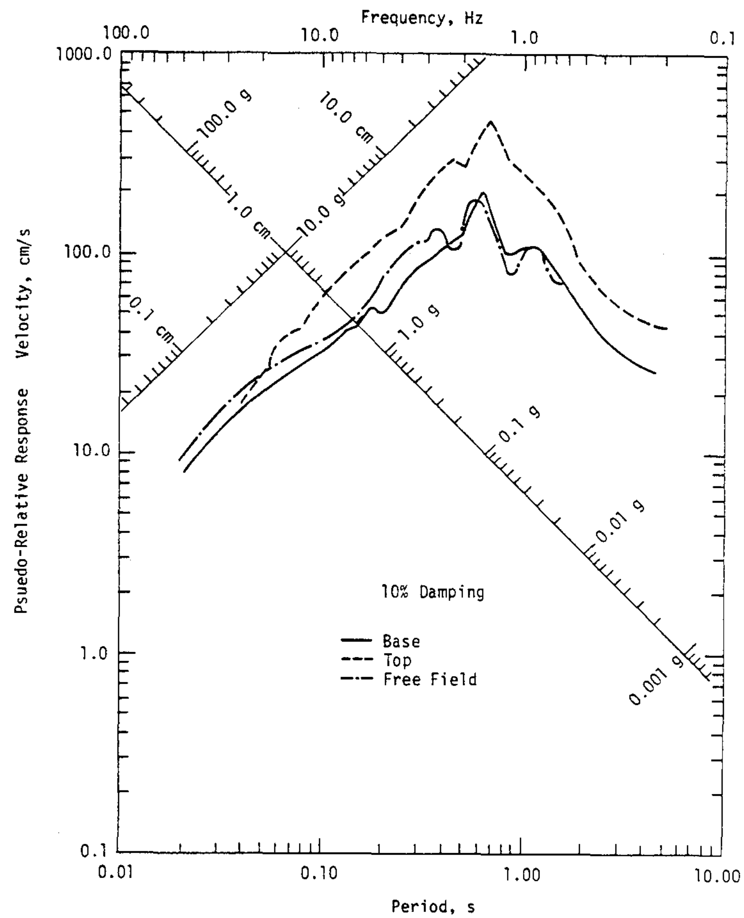
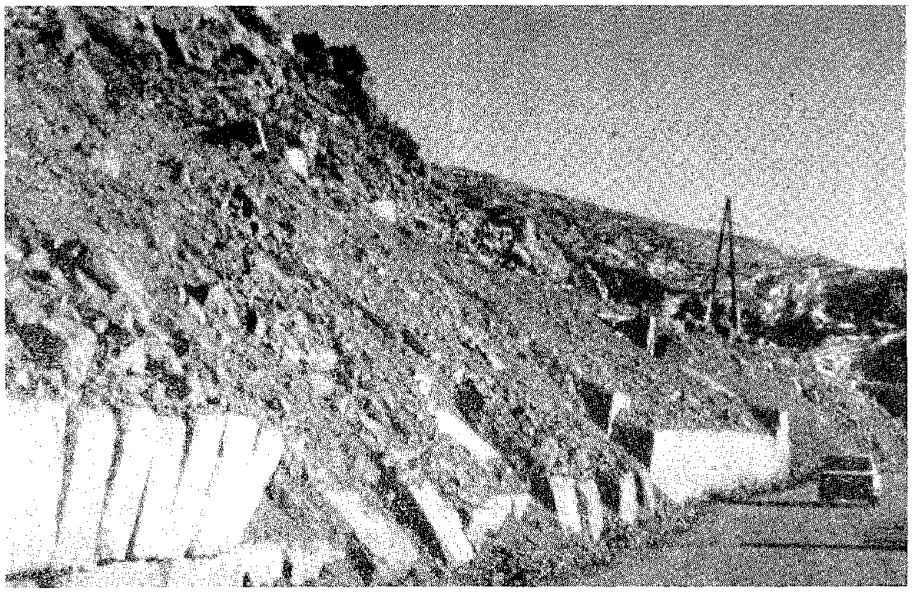



Figure 20. Comparison of 1/8-Scale Structure (S01) Horizontal Spectra in the Free-Field, Structure Base, and Structure Top.

## 2.6 DYNAMIC TESTS ON CENTRIFUGES AND SHAKE TABLES

Hon-Yim Ko
University of Colorado
Boulder, Colorado



Failed Concrete Retaining Walls Near Spitak in 1988 Armenia Earthquake (Ref. 4, fig. 5.13, pg. 65, reprinted by permission of Earthquake Engineering Research Institute).

# DYNAMIC TESTS ON CENTRIFUGES AND SHAKE TABLES

by
Hon-Yim Ko
University of Colorado

## Introduction

The need to conduct dynamic tests to study the behavior of soil masses and structures founded in soil is obvious. From such experiments, we can make qualitative observations of the various phenomena that occur in soils during vibrations which simulate earthquake conditions, obtain quantitative data to check the accuracy of a numerical procedure used to analyze the performance of the structure, or validate the design of a particular project. Ideally, these tests should be carried out in the field on the full scale structure, but the costs and consequences of loading a structure to failure usually preclude an active experiment from being carried out to failure.

On the other hand, physical model experiments can be made on shake tables to simulate the effects of earthquakes. Here, we include experiments conducted under normal gravity (1 g) conditions as well as under elevated gravity conditions as obtained in a centrifuge. As in other fields of engineering where scale models are tested to obtain results for extrapolation to the full scale (prototype) conditions, the important factors influencing the response of the prototype structure must be faithfully simulated in test model. Then, through the appropriate scaling relations, the model test results can be interpreted as representing the prototype response.

Shake table tests under 1 g conditions have been used primarily for structural testing by employing input motions that represent the ground shaking at the foundation level. For these purposes, shake tables as large as 10 m by 10 m have been constructed in Japan, with up to 6 degrees of freedom to simulate the base motions experienced by building structures during earthquakes. However, it is much less common to use these test facilities for purely geotechnical or soil-structure interaction studies, mainly due to the recognition that similarity is much more difficult to achieve when body force effects, which are important as far as earth materials are concerned, are not simulated properly in 1-g experiments, and that the scaling relations are considerably more complex. It is safe to conclude that the trend in geotechnical earthquake engineering is in the direction of dynamic simulation in the centrifuge where the self-weight induced effects on an N-th scale model are simulated by testing in an N.g gravity field obtained in the centrifuge. Under such conditions, the scaling relations summarized by Ko (1988) are applicable, as shown in Table 1. The emphasis in this paper is tilted heavily in that direction.

Table 1
Scaling Relations for Centrifuge Modeling

| Quantity | Prototype | Model |
|---|---|---|
| Length | $N$ | 1 |
| Area | $N^2$ | 1 |
| Volume | $N^3$ | 1 |
| Velocity | 1 | 1 |
| Acceleration | $1/N$ | 1 |
| Mass | $N^3$ | 1 |
| Force | $N^2$ | 1 |
| Energy | $N^3$ | 1 |
| Stress | 1 | 1 |
| Strain | 1 | 1 |
| Time (Dynamic) | $N$ | 1 |
| Time (Diffusion) | $N^2$ | 1 |
| Time (Creep) | 1 | 1 |
| Frequency | $1/N$ | 1 |

Centrifuge testing of earth models was first attempted in the 1930's in the US for mining applications and in the USSR for civil engineering applications. The need for this type of testing is clearly indicated when it is necessary to properly simulate the gravity induced effects that arise through the self weight of the material which is acting to produce the loading as well as to control the material response (stiffness and strength) to other stimuli. However, in the next four decades following the first centrifuge experiments, little progress was made in advancing the state of the art in this field of geotechnical testing, largely because of a lack of suitable hardware and due to preference for numerical modeling as the path to take for solving complex problems. However, beginning in the mid-1970's, there began a strong revival of interest, particularly in the US, in using scale model testing in the centrifuge as a means of (1) observing simulated prototype response, (2) studying new physical phenomena, (3) calibrating numerical models, and (4) conducting parametric studies for formulating design charts (Ko, 1988). In the last 15 years, there are at least 8 small centrifuges and three medium to large centrifuges which have been brought into operation in the US for research and testing of different geotechnical problems.

The need to conduct physical model testing is even more acute for dynamics problems, such as for simulating earthquake loadings. This is because earthquakes do not strike at predetermined times and their locations have largely defied scientific predictions. Most of what has been learned about earthquake damages have been derived from post mortem investigations in which much of the evidence has been destroyed by the event itself. Thus, scale model experiments in the centrifuge remains the only viable means of obtaining an insight to the various phenomena occurring in soil masses when shaken by earthquakes.

In the last decade, major advances have been made in developing techniques of simulating earthquake-type vibratory motions in the elevated gravity environment existing in a rotating centrifuge. In the US alone, there are at least 5 centrifuge facilities where earthquake simulation capabilities exist, and a corresponding number of overseas facilities are similarly equipped. The rate of progress in this area is best judged by the number of papers appearing in the last two international centrifuge conferences (Corte, 1988; Ko and McLean, 1991) which were devoted to the subject of earthquake simulation. In 1988, there were 8 papers addressing this subject. The number increased to 13 in 1991. In the following section, the experimental techniques used in these facilities are reviewed.

## Earthquake Simulation in Geotechnical Centrifuges

The first attempt at simulation earthquakes in centrifuge testing was made by releasing a cocked spring to produce damped vibration of the model container carried on the payload platform (Morris, 1983, and Ortiz, et al., 1983). The frequency and amplitude of vibration were determined by the spring characteristics and were not easily varied from test to test. At about the same time, the "bumpy road" technique was developed at Cambridge University in which, through a lever arrangement, the payload container was allowed to experience the motion of a wheel riding on a track mounted on a portion of the wall of the centrifuge chamber (Schofield, 1981). Although the wave form machined on the track is fixed, the amplitude of the base motion experienced by the payload can be adjusted in flight and repeated earthquakes can be triggered. However, due to the time and expense associated with preparing and changing to a different track in order to produce a different type of input motion, only sinusoidal motion has been used in the bumpy road method. It appears that it would be costly to simulate a wide range of input motion as experienced in real earthquakes. The above methods excite both the soil model and the container.

Arulanandan, et al., (1982) described a system which uses the piezoelectric effects to produce motion by applying fluctuating voltages to a stack of piezoelectric ceramic elements buried in the soil. The main drawbacks of this method are that high voltages are required to produce sufficient amplitude of motion for earthquake simulations and that only high frequency motions are obtainable. Zelikson, et al., (1981) used the detonation of explosives at the boundary of the soil container to generate soil motion which is transmitted to the test model. A similar method of boundary excitation was described by Prevost and Scanlon (1983) using a hydraulic hammer to strike a plate buried at the bottom of the soil model. Although the frequency of the input motion can be adjusted by changing the dynamic characteristics of the plate, only high frequency motion has been obtained so far. All three methods are handicapped by the fact that only one event could be set off at a time, and that the resulting motion is not particularly representative of earthquakes.

A more versatile method is the servo-control, electro-hydraulic method first applied by Aboim, et al., in 1986 to earthquake simulation in centrifuge testing. The basic technique of closed-loop testing using hydraulic actuators has been applied to structural testing for many years. However, the key to its successful application to testing in the centrifuge environment lies in mounting hydraulic accumulators on the centrifuge to serve as the main power supply, thus bypassing the small, restrictive passages of the hydraulic rotary joints in the centrifuge. Over the relatively short duration of the earthquake (time scale is compressed by the factor N), the decompression of the hydraulic fluid in the charged accumulators serves to drive the double-acting actuator to deliver the preprogrammed input motion to the mass being shaken. The feedback signal is either the acceleration or the displacement of the mass. This method is superior to the others because of the large forces which can be generated by the hydraulic actuators and because of the versatility in producing pre-programmed motion to match the desired frequency and amplitude.

Whitman (1988) gave an excellent review of the above methods of dynamic excitation in earthquake simulation in centrifuge testing and produced a comparison of them as shown in Fig. 1. Since all recent developments in earthquake simulation testing in the centrifuge are adaptations of the versatile electro-hydraulic method, the next section gives a discussion of the pros and cons of this particular method.

Performance of Electro-hydraulic Shake Tables in Centrifuge

To date, only one-dimensional shaking in the prototype horizontal direction has been possible in the electro-hydraulic method of earthquake simulation. Whitman (1988) compared the response spectrum of the horizontal motion produced by Aboim, et al., (1986) in the Caltech centrifuge with that of the record of the 1971 San Fernando earthquake and found that in general the method is capable of capturing the essential features of the motion of the prototype earthquakes.

In the ground motion simulator in the Caltech centrifuge, the soil container is suspended from the payload platform by rods while the hydraulic actuator applies the driving force from below the container using the platform as the reaction mass. The base motion shakes the soil model in the circumferential direction in the centrifuge, which brings into question the effects of a divergent centrifuge acceleration field over the dimension of the soil model in the direction of shaking. To minimize these undesirable effects, researchers using ground motion simulators which shake the model in the centrifuge circumferential direction have prepared the soil model with a curved surface conforming to the radius to the center of the rotating centrifuge. (See, for example, Arulanandan, et al., 1988). On the other hand, a different approach is adopted for the University of Colorado centrifuge, in which the soil model container is supported on a slip table which is also shaken from below, Ketcham, et al. (1988, 1991), Fig. 2. However, to avoid the need to curve the soil model, the Colorado shake table produces motion in the vertical direction in Earth's reference frame. Thus, a parallel body force field prevails on the model equivalent to the prototype conditions.

However, prototype vertical motion is unavoidable in most apparatuses employing the electro-hydraulic method of excitation. This is due to the fact that when the driving force is applied by the hydraulic actuator mounted below the center of mass of the payload, as dictated by space limitations on the small centrifuges on which the technique has been implemented, a rocking motion is developed, which is superimposed on the horizontal motion desired by the experimenter. It has been argued that such coupled motion, although not purposely planned, produces a better simulation of actual earthquakes which usually have all three components of motion present. Irrespective of the desirability of this additional component of the shaking motion, it has to be measured and characterized along with the input horizontal motion, so that the effects of both can be quantified.

<u>Containers for Model Testing</u>

Whether model testing is to be conducted on a shake table in 1-g conditions or in the centrifuge, a container has to be used to hold the soil mass. Thus, special attention has to be given to the interaction of the soil mass with the container, particularly in view of the fact that the finite size of the centrifuge limits the size of the model container that can be used to carry the soil model.

When a container with rigid walls is used in the experiment, particular care is required to ensure that the stress waves reflected from the boundaries are either measured and quantified in regards to their effects on the model performance, or reduced by suitable absorption at the boundaries. Prevost (Coe, et al., 1985) pioneered the use of Duxseal as a liner material for absorbing the stress waves at the boundary in dynamics experiments in rigid containers. The effects of such attempts have been evaluated by Cheney, et al., (1988) and Lenke, et al., (1991). There are indeed beneficial effects in situations where an embedded foundation is excited from above, with relatively small soil deformations involved in the vibration problem. However, when the soil mass is subjected to earthquake-like shaking from the base with larger soil motion developed, there has been no systematic evaluation of the effectiveness of the Duxseal lining.

On the other hand, a totally different approach has been taken in dealing with the boundary effects when large soil motion is involved in the experiment. Whitman and Lambe (1986) described a circular stacked ring device used as the model container in earthquake simulation experiments. The lateral flexibility of the device allows the soil column to move as a shear beam. Under ideal conditions, the soil movement is unrestricted by the confinement from the container. Improvements were implemented by Hushmand, et al. (1988), who developed a rectangular laminar container as shown in Fig. 3. Similar devices have been constructed by others. For example, Law, et al. (1991), described a systematic evaluation of the performance of the laminar container by measuring the uniformity of soil motion inside the container and demonstrated the effectiveness of such efforts in dealing with the boundary reflection problem. It is now widely accepted that the laminar container is an indispensable tool in earthquake simulation experiments in the centrifuge where stress wave reflections from the end boundaries of the container are of concern when input motion is supplied at the base of the model.

## Experience with Earthquake Simulation in Centrifuges

As indicated earlier, much progress has been made in the last few years in developing the capability for simulating earthquake ground motion in centrifuge testing. The National Science Foundation is currently funding a research project on the Verification of Liquefaction Analysis by Centrifuge Studies (VELACS), involving a number of universities with dynamic simulation facilities on their centrifuges. The primary objective of the project is to produce a data base on the performance of earth structures shaken by simulated earthquakes in the centrifuge to the point of liquefaction for validating analytical procedures commonly used in practice. In order to establish confidence in the ability of this method of dynamic testing to produce reliable results, the first phase of the project was focused on conducting identical model tests of a horizontally layered soil deposit in the various centrifuges to determine any machine dependence of the test results. Although the results from the participating laboratories showed the type of scatter usually encountered in experimental research, similar trends of the behavior of the experiments in terms of the pore pressure generation and ground motion characteristics transmitted through the soil layers were clearly identified. In the second phase of the project, several experiments have been defined in which various geotechnical structures will be subjected to simulated earthquake ground motions in the centrifuge. Analytical predictions of the performance of these experiments have been invited from analysts around the world. When these truly Class A predictions are submitted next April, the experiments will then be performed to provide data for the refinement of the analytical procedures.

It has been widely recognized that one of the outstanding issues in earthquake simulation experiments in the centrifuge is the conflict in the time scaling relations for dynamic phenomena ($1/N$) and diffusion phenomena ($1/N^2$), as shown in Table 1. This factor becomes important when the soil mass is shaken long enough to the point where dissipation of the pore water pressure generated during shaking begins to be significant. This effect is probably not as important in fine grained soils with low permeability as for most sandy soils. Attempts have been made to slow the dissipation process by using a substitute pore fluid (such as silicone oil) with a viscosity N times greater than water. However, there is no documented record of the success of these attempts, while the question of the alteration of the properties of the soil by the substitute pore fluid has not been examined.

Very little work has been done in using the existing dynamic simulation capability to model structures that have been shaken by actual earthquakes. This is primarily due to the insufficient capacity of existing centrifuges which are equipped with the shaking capability. Thus, much of the testing has been limited to examining basic phenomena. The lack of larger shake tables also hampers the application of the "modeling of models" testing scheme, which could be beneficially utilized to delineate the limits of a particular dynamic centrifuge model testing program by showing when modeling at a certain scale fails to apply. Results of "modeling of model" testing can also point out when boundary constraints become unacceptable, and when it is necessary to use a substitute soil material by altering its grain size distribution.

## Research Needs

The VELACS project just described will go a long way in establishing centrifuge testing as a viable alternative to full scale testing in earthquake geotechnical engineering. However, much research is still needed to expand the capability and ensure the accuracy of this method. The following represents a listing of these needs:

1. The effects of boundary conditions on the performance of the test model require considerably more systematic investigation. The need to use an absorbing boundary versus a laminar container should be clarified for different problem types.

2. The conflict in the time scaling relations for dynamic and diffusion phenomena must be addressed. Fundamental research is needed to justify the need for the use of substitute fluid and to determine the changes in the dynamic constitutive properties of the soil containing such substitute pore fluid.

3. Further miniaturization of transducers, or development of new methods, for accurate measurement of dynamic soil pressures and pore water pressures is needed.

4. Methods for consistent soil model preparation, particularly for fine grained materials, as well as techniques for in-flight material characterization, have to be developed, particularly in those applications where prototype conditions are to be simulated.

In addition, shake tables need to be developed for the large centrifuges which have been put into operation in recent years. The availability of such testing capabilities will greatly enhance the power of the method by allowing for the full utilization of the modeling-of-models testing scheme and for better representation of the details in a structural model. While the technology for servo-control electro-hydraulic excitation has been proven for application in small centrifuges, the scaling up to larger centrifuges is not necessarily straightforward. The potential benefits certainly outweigh the anticipated costs for such development.

References

Aboim, C., R. F. Scott, J. R. Lee & W. H. Roth (1986). Centrifuge Earth Dam Studies: Earthquake Tests and Analyses. Report to National Science Foundation, Grant No. CEE-7926691, Dames and Moore, Los Angeles.

Arulanandan, K., C. Yogochandran, K. K. Muraleetharan, B. L. Kutter & G. S. Chang (1988). Seismically Induced Flow Slide on Centrifuge, *J. Geotechnical Eng. Div., ASCE*, 114, 1442-1449.

Arulanandan, K., J. Canclini & A. Anandarajah (1982). Simulation of Earthquake Motions in the Centrifuge. *J. Geotechnical Eng. Div., ASCE*, 108, 730-742.

Cheney, J. A., O. Y. Z. Hor, R. K. Brown & N. R. Dhat (1988). Foundation Vibration in Centrifuge Models. *Centrifuge 88*, J. F. Corte, ed., Balkema, Rotterdam, 481-486.

Coe, C. J., J. H. Prevost & R. H. Scanlon (1985). Dynamic Stress Wave Reflections/Attenuation: Earthquake Simulation in Centrifuge Soil Models. *Earthquake Engineering and Structural Dynamics*, 13, 109-128.

Hushmand, B., R. F. Scott & C. B. Crouse (1988). Centrifuge Liquefaction Tests in a Laminar Box. *Geotechnique*, 38, 253-262.

Ketcham, S. A., H. Y. Ko & S. Sture (1991). Performance of an Earthquake Motion Simulator for a Small Geotechnical Centrifuge. *Centrifuge 91*, H. Y. Ko and F. G. McLean, eds., Balkema, Rotterdam, 361-368.

Ko, H. Y. (1988). Summary of the State-of-the-Art in Centrifuge Model Testing. *Centrifuges in Soil Mechanics*, W. H. Craig, R. G. James and A. N. Schofield, eds., Balkema, Rotterdam, 11-18.

Law, H., H. Y. Ko & S. Sture (1991). Development and Performance of a Laminar Container for Earthquake Liquefaction Studies. *Centrifuge 91*, H. Y. Ko and F. G. McLean, eds., Balkema, Rotterdam, 369-376.

Lenke, L. R., R. Y. S. Pak & H. Y. Ko (1991). Boundary Effects in Modeling of Foundations Subjected to Vertical Excitation. *Centrifuge 91*, H. Y. Ko and F. G. McLean, eds., Balkema, Rotterdam, 473-480.

Morris, D. V. (1983). An Apparatus for Investigating Earthquake-Induced Liquefaction Experimentally. *Canadian Geotechnical Journal*, 20, 840-845.

Prevost, J. H. & R. H. Scanlon (1983). Dynamic Soil Structure Interaction: Centrifugal Modelling. *Soil Dynamics and Earthquake Engineering*, 2, 212-221.

Ortiz, L. A., R. F. Scott & J. Lee (1983). Dynamic Centrifuge Testing of a Cantilever Retaining Wall. *Earthquake Engineering and Structural Dynamics*, 11, 251-268.

Schofield, A. N. (1981). Dynamic and Earthquake Geotechnical Centrifuge Modelling. *Proceedings of the International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics*, 3, 1081-1100.

Whitman, R. V. (1988). Experiments with Earthquake Motion Simulation. *Centrifuges in Soil Mechanics*, W. H. Craig, r. G. James and A. N. Schofield, eds., Balkema, Rotterdam, 203-216.

Whitman, R. V. & P. C. Lambe (1986). Effect of Boundary Conditions Upon Centrifuge Experiments Using Ground Motion Simulation. *Geotechnical Testing Journal*, 9, 61-71.

Zelikson, A., B. Devaure & D. Badel (1981). Scale Modeling of Soil Structure Interaction During Earthquakes Using a Programmed Series of Explosions During Centrifugation. *Proceedings of the International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics*, 1, 361-366.

| | COST | SIMPLI-CITY | ADJUST-ABILITY | FREQUENCY RANGE (Low – High) |
|---|---|---|---|---|
| COCKED SPRINGS | Very Low | Very Simple | Poor | |
| PIEZO-ELECTRIC | Low | Simple | Good | |
| EXPLOSIVE | Low | Simple | Moderate | |
| BUMPY ROAD | High | Complex | Moderate | |
| HYDRAULIC | Very High | Very Complex | Very Good | |

Fig. 1. Comparison of Various Methods for Simulating Earthquake Ground Motions on Centrifuge (After Whitman, 1988)

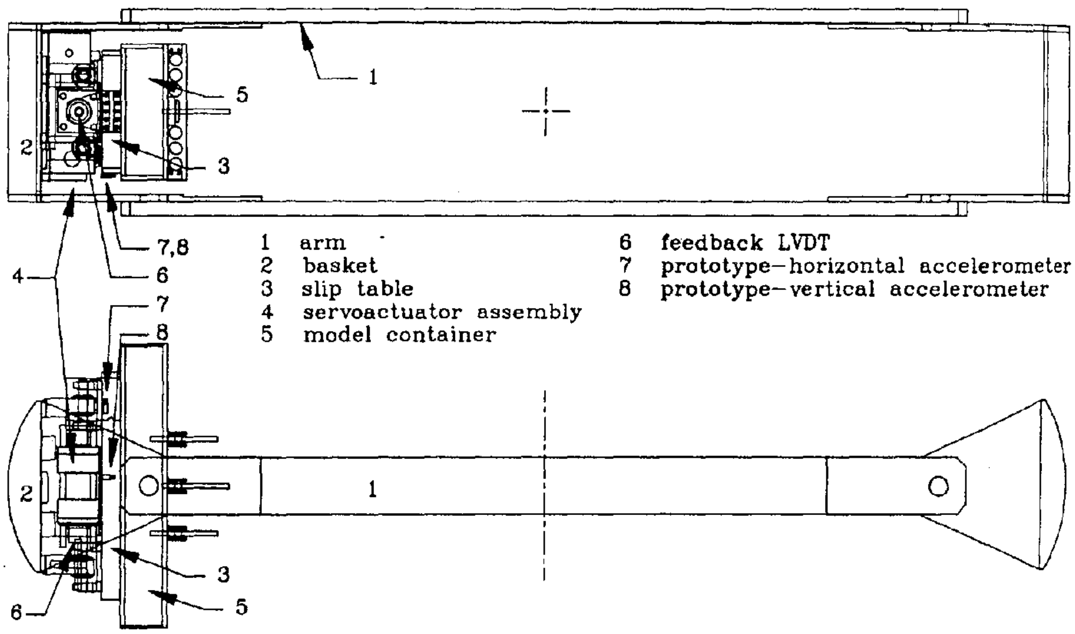



Fig. 2. Shake Table on 15 g-ton Centrifuge at University of Colorado (After Ketcham, et al., 1991)

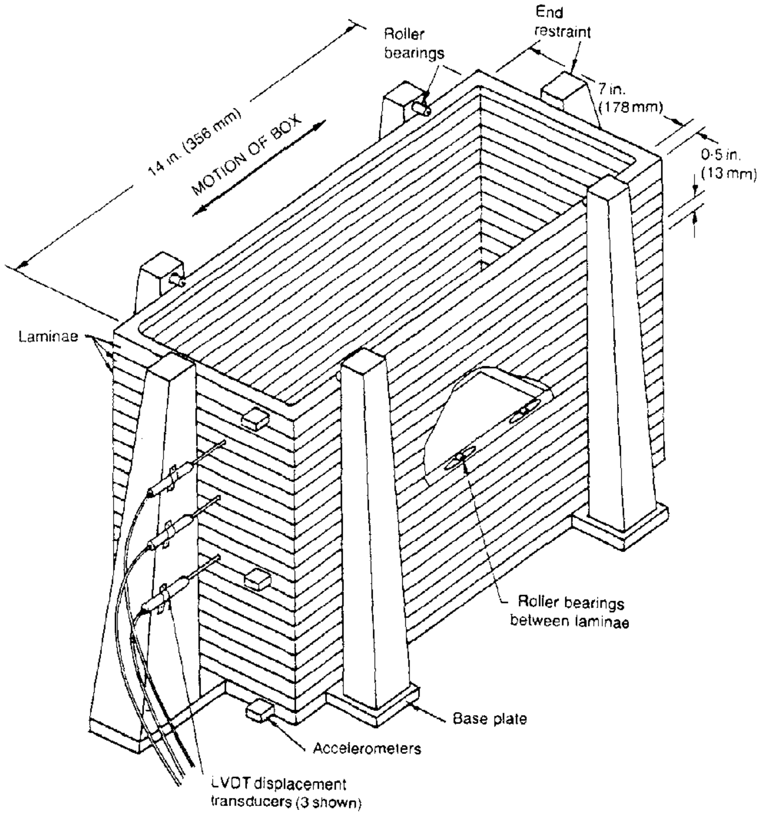



Fig. 3. Laminar Container for Earthquake Simulation in Centrifuge (After Hushmand, et al., 1988)

# CHAPTER 3
# COMMON RECOMMENDATIONS

## 3.1 INTRODUCTION

Each of the panels was asked to consider experimental needs using the following format:

(1) Knowledge Gaps - What are the major knowledge gaps in your panel's area that force conservative decisions or avoidance of economically desirable alternatives in new construction or remediation?

(2) Suggested Experiments - What experiments or groups of experiments would close or narrow these gaps? In answering this question, we are not looking for detailed experimental design or planning. We hope that the panel would state an objective, a descriptive name, some indication of the experimental approach recommended (there may be more than one, i.e., laboratory bench, field test, long-term performance observation on existing structures, centrifuge, shake table, etc.), and whether the resources required are small (3 staff years of effort), median (3 to 10) or large (>10). We hope you will not constrain yourselves to existing test capabilities or sites in answering this question. We do not want the answer to this question to be an exhaustive "laundry list". Perhaps the most difficult challenge for the chair will be to lead the group to a consensus as to which experiments should be on the short list (certainly no more than 10).

(3) Experimental Methods - Which experimental methods appear to be the most effective ways of carrying out these experiments?

(4) Technical Barriers - What are the technical barriers relating to the short list experimental programs? Here we would like you to focus on such things as inadequacies in presently available instrumentation, test apparatus, test site, and our state of understanding of scaling methods and model construction techniques. Another way of asking this question is "what research or development must be successfully completed before you are ready to do the experiment?"

(5) Non-Technical Barriers - What are the non-technical barriers and resource limitations making it difficult to carry out the short list experiments? The obvious one is funding. There may also be issues of inter-university, university-government, university-private firm, or government-government cooperation or support. Environmental or public safety constraints may also represent real or perceived barriers.

(6) Targets of Opportunity - Can the panel identify any "targets of opportunity" where a "short-list" experiment can be piggy-backed on an already planned or funded activity or event?

Each panel followed this format to varying degrees. Detailed panel results are presented in Chapter 4. In this section, we have tried to identify items, issues, and areas which seem to be common to each of the panels.

## 3.2 KNOWLEDGE GAPS

Knowledge gaps identified by the panels can be placed in one of six categories:

(1) Input Ground Motion
(2) Site Characterization
(3) Fundamental Behavior and Properties of Soil and Rock
(4) Behavior/Response/Failure Modes of the Geotechnical System
(5) Analytical Models
- Finite Element and Finite Difference Models
- Simple Models for Practice
- Model Validation
- Model and Parameter Selection

(6) Mitigation/Soil Improvement

### 3.2.1 Input Ground Motion

Ground motion was the main subject of one of the panels. In addition, it was raised by the Earthdams, Natural Slopes, and Retaining and Underground Structures panels. Specific concerns included coherency and spatial variation of motions, the effects of wave type (P, S, R, etc.), vertical motions, local site and topographic amplification, fault rupture, especially beneath dams, and overall prediction uncertainties. The recommendations of the ground motion panel, if pursued, will address most of these issues.

### 3.2.2 Site Characterization

Detailed site characterization in three dimensions was especially important for large geotechnical systems where the site can be variable over the dimensions of the system and site details can dominate response. The Earthdams, Natural Slopes, and Ground Instability panels, especially, raised this issue.

An important subissue in site characterization was the use of insitu measures of site characteristics and material properties, and correlation of those measures with field behavior. Replacement of crude measures, such as the Standard Penetration Test (SPT) and cumbersome methods, such as cross-hole seismic, should be pursued. The Cone Penetration Test (CPT) with associated measures such as down-hole seismic, dilatometers, etc. should be emphasized. Insitu index tests of dynamic properties, such as small explosive tests, should also be considered.

Although site characterization is clearly important for earthquakes and most other geotechnical problems, dynamic and otherwise, it was not the focus of this workshop. The importance of this issue, however, demands attention by the NSF and other research agencies.

### 3.2.3 Fundamental Behavior and Properties of Soil and Rock

Soil and rock behavior is a basic input into understanding and predicting the overall behavior of geotechnical systems. The constitutive models and parameters which define fundamental element behavior are important for analytical modeling. Major uncertainties exist on issues such as insitu versus laboratory properties; damping; saturated soil behavior; the effect of plasticity and cementation on liquefaction; and strength degradation and residual strength, especially of clays. As with site characterization, the behavior of geotechnical materials was not a direct topic in this workshop. Fundamental behavior is of paramount importance, however, and must be pursued by NSF and other research agencies. A major NSF/EPRI workshop entitled "Dynamic Soil Property and Site Characterization" was held in November 1989. The recommendations of this workshop, if pursued, will greatly improve our site characterization capability and understanding of basic properties.

### 3.2.4 Behavior/Response/Failure Modes of the Geotechnical Systems

Major knowledge gaps which require experimental evaluation for their resolution/illumination begin to emerge in the consideration of the response modes of geotechnical systems. Response and failure modes are major uncertainties because either earthquakes have not occurred in the vicinity of major geotechnical systems and/or post-earthquake observation and information are inadequate. The Natural Slopes panel noted the lack of well documented (quantitatively) slope failures. The Foundations and Retaining and Underground Structures panels noted the wide diversity of foundations and structure types for which little or no response/failure mode observations are available. Both the Earthdams and Ground Instability panels noted response uncertainties in heterogeneous material. The documentation of transient and permanent deformations were also of interest to both of these groups.

It appears that response/failure mode definition, including illumination, documentation, and quantification, for real geotechnical systems is the major knowledge gap requiring experimental evaluation.

### 3.2.5 Analytical Models

Finite element or finite difference models can provide some insight into response and failure modes. However, they are limited in their ability to handle large strain, nonlinear and inelastic behavior, three dimensional input, site conditions and system geometry, and details of boundary and interface conditions. Once response and failure modes have been identified from observations or experiments, numerical models can be focused to reproduce those modes and then used to evaluate the effect of various parameters. Several panels pointed out the lack of validated models, as well as concerns about three dimensional effects, large strain modeling, and constitutive parameters. Two panels, earthdams and foundations, commented on the need for simple models for use in practical design and analysis. The development and validation of models is a knowledge gap closely related to that associated with identifying modes of response and failure. Experimental programs can integrate the evaluation of both of these gaps.

### 3.2.6 Mitigation/Soil Improvement

Three panels, Natural Slopes, Foundations, and Ground Instability, noted a lack of information on the effectiveness of mitigation and soil improvement measures, including the use of geotextiles and density improvements in earthquake engineering applications. The Ground Instability panel observed that there is significant uncertainty on the spatial extent and depth of improvement required.

## 3.3 EXPERIMENTS AND EXPERIMENTAL METHODS

The suggestion of relevant experiments and the evaluation of experimental methods were generally treated together by the panels. The experimental methods recommended by the panels can be placed in six major categories:

(1) Post-Earthquake Observations
(2) Instrumented Sites in Seismically Active Regions (and Mobile Instrumentation for Aftershock Monitoring)
(3) Artificially-Induced Ground Shaking
(4) Centrifuge Tests
(5) Shake Table Tests
(6) Field Shaker and Snapback Tests

There was also mention of laboratory single-element (e.g., cyclic triaxial, cyclic simple shear) and specialized tests (e.g., foam rubber models) by some panels, but these did not receive much emphasis.

### 3.3.1 Post-Earthquake Observations

All panels supported the continued conduct of post-earthquake observation of geotechnical systems. However, some panels emphasized more quantitative measurements of deformations and analysis of case histories. The Earthdams panel pointed out the need for a large database of analyzed case histories. The Natural Slope panel recommended timely quantitative measurement of slope failures. The Ground Instability panel recommended measurement of deformations.

### 3.3.2 Instrumented Sites in Seismically Active Regions

This approach was recognized by all panels as the ideal experimental condition, without controversy regarding such factors as wavefields, boundary conditions, and scaling. Instrumented sites are essential for resolving the knowledge gaps on earthquake ground motion per se. All panels recommended permanent instrumented geotechnical projects at active sites. The obvious locations for such installations are sites which already have adequate strong motion arrays installed or planned. Very little attention was given to the location and type of instrumentation, although the Ground Motion panel recommended very dense surface and vertical arrays, including accelerometer and piezometer measurements, and settlement gages. The organizing committee concurs with this recommendation and also recommends stress measurements in the ground to help identify the type of waves causing the motion. A few accelerometers with more than 3-axis measurements would also be useful. Instrumentation on the individual experiments would be experiment dependent but might include motion, interface stress, total stress and pore pressure, rotation, and transient and permanent displacements.

Recognizing that aftershocks can themselves be substantial earthquakes, the Natural Slopes panel recommended the establishment of a mobile instrumentation capability for extensive monitoring of slope response to aftershocks. The Earthdams panel expressed interest in the measurement of aftershock effects on damaged dams.

### 3.3.3 Artificially-Induced Ground Shaking

Instrumented sites in seismically active regions provide ideal experimental conditions, but the expected results from such experiments are limited by the lack of knowledge of precise time and location of the event, as well as restrictions on numbers of measurements and limited geotechnical site information. The data are not provided on a known schedule and therefore cannot be counted on in a planned, organized program.

The Earthdams panel suggested construction of a controlled full scale embankment at a seismically active site which could also be tested with artificial shaking. The Retaining and Underground Structures panel noted that controlled artificially-induced ground shaking to simulate earthquake motion would permit concentration of instrumentation in spatially optimized locations. Some other panels suggested artificial shaking as well. It would permit better organized and more efficient use of manpower and other resources. Furthermore, knowledge of the time, location, and source characteristics would permit pretest predictions, better experimental planning, and higher instrumentation reliability. Candidates for artificially-induced ground shaking include explosives and vibroseis. Youd reported to the workshop on Russian work using water filled pipes (perhaps water pulse devices) as a seismic source. All artificial methods have shortcomings associated with the type, amplitude, frequency, duration, and spatial extent of ground shaking. Explosives and, perhaps, water pipes approaches have the greatest potential for producing ground motions characteristics of interest for earthquake applications.

### 3.3.4 Centrifuge Tests

Centrifuges have the advantage of permitting small scale experiments in a properly scaled gravity field. Most panels noted the potential usefulness of centrifuge studies for revealing failure mechanisms and system responses. However, the panels also noted several shortcomings which are discussed under Technical Barriers.

### 3.3.5 Shake Table Tests

Passing interest was expressed by only 2 panels in shake tables for geotechnical studies. This was due mainly to concerns about scaling and boundary influence because of the limited size of shake tables in the U.S. Japanese participants at the workshop stated that geotechnical studies are conducted in Japan on their large shake tables. Such studies are more common because of the availability of tables. They seem to be limited to basic studies of level soil layer response in dry and saturated sands.

### 3.3.6 Field Shaker and Snapback Tests

Field shaker and snapback tests were mentioned by the Foundations panel as methods for obtaining information on existing foundations. Such tests are limited to small to intermediate level strains but they may have value in determining insitu characteristics of complex installations. These tests could be performed on systems installed in seismically active areas or to be tested in artificially-induced earthquake conditions and then correlated with response.

## 3.4 TECHNICAL BARRIERS

Technical barriers were defined as those issues which must be resolved before adequate experimental programs can be conducted. Barriers might include inadequacies in test methods or apparatus, instrumentation, test site conditions, or lack of understanding of technical factors such as scaling methods, model construction and characterization, etc. Technical barriers are best organized by experimental method.

### 3.4.1 Post-Earthquake Observations

No major technical barriers were identified for this method. However, unknowns associated with site geotechnical conditions (see 3.4.2 below), and earthquake input at the site limit the analyses of post-earthquake observations.

### 3.4.2 Instrumented Sites in Seismically Active Regions

Four significant technical issues were raised here. The first and most important was the issue of instrument reliability/longevity especially for downhole instruments and instruments in inaccessible areas of structures. The second was site aging effects. Soils and backfills are known to settle and change in properties over long periods of time. Consideration must be given to monitoring these changes so site conditions are reasonably well known at the time of the earthquake. The third technical barrier is the difficulty of measuring stresses in the soil. Stress conditions associated with the earthquake waves help with interpreting the earthquake wave field which, in turn, affects how certain geotechnical systems respond. The fourth technical barrier is the uncertainty associated with time and location of the earthquake. This is really the origin of the instrument reliability and site aging issues. It also leads to some inefficiency in instrumentation allocation and some non-technical barriers.

### 3.4.3 Artificially-Induced Ground Shaking

Explosive simulation is the main artificially-induced ground shaking method that has been used to date. Since the primary motion from explosions in the near-field is from P-waves, there is concern that the response will be different than S-wave induced response, a main component in earthquakes. Concerns were also expressed about non-uniformity of motion, higher frequency content, shorter duration, and fewer cycles of motion in explosions.

The main experience with explosive simulation was in the SIMQUAKE series supported by EPRI. The results are not widely known in the earthquake engineering community. A state of the art paper in this workshop discussed those experiments. The experiments utilized several enhancement techniques to overcome many of the objections. Planar arrays of explosives were used to provide more uniform motions over a larger test area. Two arrays of explosives were used to extend the duration and increase the number of motion cycles. The array was designed to produce frequencies and motion amplitudes that were relevant to earthquake conditions. Other methods for tailoring waveforms, including the use of relief trenches or mitigating materials, require evaluation.

The P-wave versus S-wave issue is important. There is evidence that the response of a relatively rigid inclusion in soil, e.g. a foundation or wall, is relatively insensitive to the wave details but responds mostly to the kinematic characteristics of the ground motion. There are also concepts for producing shear-waves with explosives. One such approach is the generation of near vertically propagating S-waves in a near surface layer by driving a horizontal P-wave in a hard underlying layer.

As mentioned earlier, the Russians seem to be producing ground motions using arrays of 1m diameter x 50m long water filled pipes. Youd reported that these are buried in the ground in three-dimensional arrays. Although it is not clear how they are being used, it is speculated that they may be exciting waves in the pipes which cause back and forth longitudinal pipe motion, thereby simulating S-waves. This concept should be investigated as a potential simulator for the U.S.

### 3.4.4 Centrifuge Tests

Several technical barrier issues were raised regarding the use of centrifuges for geotechnical earthquake engineering studies. One problem is the size of even miniature transducers and instrumentation cables at the scales used in centrifuges. A 0.5 inch transducer becomes about a 4 foot block at 1/100 scale. There is a need for even smaller transducers or alternative methods of measurement at small scale.

Another significant barrier for studies of saturated soils, especially liquefaction, is the fact that time for water flow (diffusion) scales as $N^2$ while time for dynamic response scales as N. This certainly affects studies of pore pressure dissipation after liquefaction. It may also affect the onset of liquefaction since pore pressure increases and the tendency for pore water to flow occurs well in advance of actual loss of total effective stress. Substitute fluids and their influence on other behavior need to be evaluated to help overcome this barrier.

Other technical barriers include size limits and resulting boundary reflections and in-flight property determination. Approximations to non-reflecting boundaries have been investigated to diminish size limitations. Hon Yim Ko's paper in this workshop refers to a laminar container for this purpose. This needs further work.

Perhaps the major technical barrier to progress with centrifuges is the lack of high capacity shaker systems for the larger centrifuges in the U.S. One-dimensional systems are an immediate requirement. The need for two or three dimensional shaking requires analysis of the trade-offs between payload capacity and shaker capability.

### 3.4.5 Shake Table Tests

The major technical barrier to meaningful shake table studies is the limited size and capacity of U.S. shake tables. The largest table for earthquake testing in the U.S. is the 6.1m x 6.1m table at the University of California Earthquake Engineering Research Center. The relatively small size limits the model size and causes boundary reflections in a short time. The fact that tests are conducted at 1g causes scaling conflicts at less than full scale. The importance of this issue depends upon the relative amplitude of transient stresses versus body stresses, and their relative influence on response.

Larger shake tables, similar to the large tables in Japan (15m x 15m at Tadotsu Engineering Laboratory) would enable some geotechnical studies to be conducted. For example, the interface stresses and total forces on retaining walls might be investigated on such a table. Boundary reflections might be reduced with artificial non-reflecting boundaries as mentioned for centrifuges. However, even at large size, there is a limit to the height of the column of soil included in the model, caused by payload considerations.

### 3.4.6 Field Shaker and Snapback Tests

There are no fundamental technical barriers to these methods because they are not direct earthquake simulators. However, because of this, they provide only limited insight into earthquake response. They can, however, be part of a larger program.

## 3.5 NON-TECHNICAL BARRIERS

The lack of sufficient funding was noted at the workshop as an overriding non-technical barrier. This leads immediately to the non-technical barriers associated with a lack of equipment such as the lack of large shake tables in the U.S. and the lack of shakers on large centrifuges. It also leads to a hesitation on the part of the community to pursue aggressive large size projects, such as large explosive tests, because they are expensive.

Other non-technical barriers included the lack of central organizations tasked with the deployment and long term maintenance of permanent instrumentation at seismically active sites and mobile instrumentation systems for monitoring aftershocks. Access to sites and data after earthquakes was cited as another non-technical problem.

All of these non-technical barriers could be readily resolved with adequate funding for geotechnical earthquake engineering research. This funding could lead to well organized programs for providing expertise, equipment, planning, and implementation of significant research.

## 3.6 TARGETS OF OPPORTUNITY

It was noted that there are in place or planned strong motion arrays at potentially active seismic sites at Palmdale, CA, Treasure Island, CA, and several overseas locations. Geotechnical experiments could be fielded at those sites. The most promising foreign site for adding experiments is the Hualien, Taiwan site. Here, EPRI is teamed with the USNRC, Taiwan Power Company, and several other French, Korean, and Japanese organizations to evaluate the response of nuclear power plant models at a seismically active site. The site is well instrumented with surface and downhole strong motion arrays. It would seem to provide a good opportunity for piggy back experiments. Parallel experiments using artificial ground shaking and other approaches could also be considered.

# CHAPTER 4
# PANEL RECOMMENDATIONS

## 4.1 REPORT OF THE PANEL ON EARTHDAMS


| | |
|---|---|
| Chair: | Mary Ellen Hynes, U.S. Army Engineer Waterways Experiment Station |
| Recorder: | Pedro de Alba, University of New Hampshire |
| Panel: | K. Arulanandan, University of California, Davis<br>Jean Prevost, Princeton University<br>Larry Von Thun, U.S. Bureau of Reclamation |

Organizing Committee Representative: Paul Hadala, U.S. Army Engineer Waterways Experiment Station

NATIONAL SCIENCE FOUNDATION

Earth Dams Panel Report

January 14, 1992

1. Background

a. User Requirements: The State of the Art papers presented by Von Thun reviewed the evaluation of the seismic safety of existing earth dams. This, rather than design of new dams in seismic regions, is the principal geotechnical seismic investigation focus of the U.S. dam owners today.

b. Dams are built to provide some combination of flood control, water supply, power generation, navigation and recreation. Seismic safety policies require dam performance to ensure (1) life safety, and (2) mitigation of economic damage. Generally, the requirements are (1) that the pool is retained during and after the earthquake, (2) that the outlet works are operational during and after the earthquake, and (3) that reservoir functions such as navigation, power generation and water supply can be restored within a short period of time after an earthquake.

Earth dam seismic stability evaluation field problems have typically led to field and laboratory research projects because of shortcomings in the current state of the art and the state of practice in geotechnical earthquake engineering. With the greater number of alternatives available in design of a new project, the designer can often "design-around" uncertainties in the state of the art. In analyzing existing dams this cannot be done. Such projects involve extensive studies of seismic hazard as well as extensive experimentation in the field, in the laboratory, and on the computer. The results of such studies suffer from large, residual uncertainties. These uncertainties combined with the potential for disastrous downstream consequences if failure occurs, force the adoption of conservative, expensive, defensive design measures for both new construction as well as remediation of existing dams. Advancement of the state of the art can get us out of the expensive cycle of undertaking research to solve each site specific problem.

c. The process of seismic safety evaluation of a dam can be viewed as six steps or stages.

(1) Estimation of ground motions for use in the evaluation.

(2) Site characterization.

(3) Defining the possible failure modes.

(4) Measuring the materials behaviors and conducting numerical analysis.
(5) Assessing stability.
(6) Planning and executing remediation if needed.

d. Another panel in this workshop will deal with Ground Motion Response so there will be no discussion of experimental needs in that area (item (1)) in this section of the report. It should also be noted that the panel on Ground Instability and Site Improvement shares interest in items (2), (3), (4), (5), and (6) albeit for level ground rather than for a dam and its foundation.

e. State of the Art Report: Von Thun listed eight items as the greatest uncertainties in seismic safety evaluations of earth dams.

(1) "The nature of the alluvium deposits at the site" (para 1.c(2))
Fluvial deposits found at many dam sites are very heterogeneous and there are many examples where understanding this heterogeneity is key to the analyses.
(2) "The nature of dynamic deformation and its effects" (para 1.c(5))
The author notes four areas requiring additional study: secondary effects, criteria for evaluation, generalized settlements and slip along predefined surfaces.
(3) "The characterization of deposits actually susceptible to flow slides" (para 1.c(4))
(4) "The effect of an embankment (overburden) on the liquefaction resistance of a foundation deposit" (para 1.c(4))
(5) "The amount of deformation that will occur post liquefaction--can we predict (it)?"
(6) "SPT Correction Factors" (para 1.c(4))
(7) "Residual shear strength estimation" (para 1.c(4))
(8) "Becker Hammer results" (para 1.c(4))

This list suggests that most of our uncertainty in seismic safety analysis of dams (and thus our need for experimental research) lies in measuring material properties or indexes of material properties and in site characterization.

f. Selected Issue Papers: Before the conference, participants were asked to submit issue papers on research needs in geotechnical earthquake engineering. A review of those papers was conducted. Sentences for those relevant to earth dams are extracted below:

"The current methods...(of dynamic analysis)...need to be extended to include nonlinear effective stress based methods" (K. Arulanandan)

"The biggest problem...is determining how much deformation will result during and immediately after earthquake shaking. Site characterization is a key step here" (M. E. Hynes)

"Development/improvement of testing methods..." (1) for damping in situ at large strains and for separating damping associated with compression and shear, (2) for undisturbed sampling of cohesionless soils, (3) field measurement of void water and fabric description and (4) measuring ground displacement and pore water pressure during earthquakes (J. L. Chameau)

"There is...a need for (large) experiments that involve shaking of saturated (cohesionless) slopes to verify trends observed in cyclic triaxial testing...where $\alpha = \tau_h/\bar{\sigma}_o \neq 0$ " (P. Hadala)

"What is the post-earthquake deformed shape (of a dam)? There is a need for large or properly scaled experimentation...to gain insight as to the potential for analytical methods to address this question" (P. Hadala)

...need for improved methods of characterization of soil and foundation conditions" (S. Kramer)

"full scale facilities to evaluate soil structures under cyclic loading" (i.e. Canadian National Test on characterization of sands for static and dynamic liquefaction (P. K. Robertson)

"experimental investigators using 2D simple shear or large centrifuge with 2D shaking capability should be considered to study liquefaction resistance of soils under the $K_o$ condition" (C. K. Shen)

"more experimental tests such as the centrifuge and shake table are needed to show how...soil improvement techniques can...prevent liquefaction." (J. P. Welsh)

g. Other workshops: Other recent workshops and studies at least peripherally touched on research needs in the subject area of the workshop. Reference 1 validated the need for research on "Protecting Dams Against Earthquakes and Floods." The focus of the proposed research thrust was on the safety of existing dams but the report gave no specific details regarding what research should be conducted, or on the need for large experiments.

Reference 2 summarized four regional workshops on research needs in geotechnical engineering. One of the four (held at San Francisco) focused attention on geotechnical earthquake engineering. They noted "estimating seismic-induced ground displacement" high on their prioritized list of recommended research topics.

Reference 3 recommended the development and operation of two large size natural test sites in zones of high seismic activity in the U.S. to complement efforts described in Reference 4. These two sites would be used for field research on site characterization technology and in-situ dynamic property measurement. Reference 3 also recommended research in use of surface geophysics in site characterization, in-situ cyclic deformation tests, strength degradation testing and material damping tests, instrumentation to support field testing, undisturbed sampling of gravels, easier laboratory material damping tests, relating dynamic properties of soils to physical and chemical processes improved analytical and numerical methods of dynamic analysis. Of interest to the current workshop was the recommendation to use centrifuge, shake table and full scale embankment tests to calibrate or validate numerical models.

h. Summary: The panel found, as the foregoing material shows, similar themes being voiced by many people. Perhaps the strongest of these are the need for better site characterization tools for use at the sites of existing dams being evaluated for their seismic response and the appreciation of the complexity of the subsurface conditions at these sites.

2. Major Deficiencies Identified by the Panel

a. The panel accepted the ideas presented in the previous section as valid. Recognizing that a purpose of the workshop was to link needed geotechnical earthquake research with large scale experimental capability needs, the panel concentrated on research topics which lent themselves to field test sites, centrifuge or shake table experiments. The deficiencies listed here only deal with that subset (i.e., laboratory bench scale material property research, research on numerical and analytical methods of dynamic analysis, and other valid and needed areas of geotechnical earthquake engineering research have been deliberately avoided).

b. Site Characterization: The panel members were collectively aware of a large number of cases where the understanding of the nature of foundation soils under existing dams changed significantly during the course of a seismic safety evaluation from that which existed at the end of construction or in the early stage of the evaluation. Fluvial deposits, especially those in glaciated regions, are extremely heterogenous and three-dimensional so it should be no surprise that this was the case. The panel felt that site characterization technology needed significant advancement so that the profession can:

(1) Accurately map, and visualize, in three dimensions, materials, initial conditions (pore pressures, stresses) and property fields (permeability, void ratio, compressibility, velocity) existing at the site with improved and new in-situ testing techniques, including surface (non-destructive) and subsurface techniques, with a high degree of precision and resolution.
(2) Develop empirical correlations to directly relate in-situ measurements such as shear wave velocity ($V_s$) and cone penetration tests to observed seismic performance such as residual strength, liquefaction, pore pressure development and cracking.
(3) Improve in-situ geophysical techniques so that we can measure $V_s$ with a spatial resolution of a few inches.
(4) Develop a Standard Penetration Test (SPT) device that will accomplish full sample recovery.
(5) Develop an in-situ testing technique that will allow us to assess contractive/ dilative material behavior in-situ.
(6) Develop simplified criteria for screening out soils such as a corrected blow count ($N_{1,60}$) threshold, gradation characteristics or site configurations (drainage conditions) that are not susceptible to liquefaction.
(7) Develop a rational way of characterizing and analytically dealing with a complex site (i.e., alluvial deposit, irregular boundary conditions).
(8) Use stochastic fields to represent material parameter spatial variation.

c. Anticipating Failure Modes: The panel considered possible failure modes and felt that the least understood were:

(1) Seismically induced movements in the vicinity of and damage to filters and drains near stiff inclusions and complex boundaries that might weaken a dam's defenses against piping.
(2) Response and failure under three dimensional loading.
(3) Hydrodynamic loading on the upstream face.
(4) Effects of aftershocks on already damaged dams or with seismically elevated pore pressures remaining in the foundation.

In opening remarks at the beginning of the workshop, Dr. Ralph Peck, reminded us that a case history is only one realization of a complex problem with many parameters, many degrees of freedom and many possible outcomes. The panel recognized that our understanding of the best understood failure modes involving liquefaction and residual strength is really tied to one detailed quantitative case history of an embankment dam (Lower San Fernando) and a very few qualitative (or poorly defined quantitative) case histories of other embankment failures.

d. Material Behavior and Numerical Models: In the materials area the panel re-iterated basic need for accurate 3-dimensional characterization of soils. A field in-situ test device that can clearly distinguish contractive verses dilative behavior in granular soils is needed as are better (cheaper and less complex than freezing) sampling methods for obtaining nearly undisturbed samples of granular and low plasticity materials. Today, no currently available laboratory test can impose fully satisfactory cyclic loading boundary conditions even for the simple horizontally polarized SH wave case. Our understanding of $K_\alpha$ which is used to evaluate whether or not liquefaction will occur under the complex state of stress imposed on the embankment and the foundation under the embankment and to extrapolate level ground field empirical liquefaction data to the sloping ground case is based on such laboratory tests and deserves evaluation both in the laboratory under better boundary conditions and in large size experiments. Finally more research is needed in the laboratory in the area of microstructure studies.

In the numerical modeling arena, major issues include:

(1) Large deformation problems are currently numerically intractable.
(2) Numerical procedures that incorporate or accommodate stochastic representation of soil properties and input motion are not generally available.
(3) To validate numerical models (present and future) there is a need for several densely instrumented, well characterized embankments on relatively simple foundations in highly seismic areas.

(4) More emphasis and study should be directed to input motion at the boundaries between embankment dams and narrow canyon walls. This is often ignored and may be very important. Centrifuge shaker tests might help explore the question.

(5) A model for computing permanent deformations of embankment-foundation systems in the case where there are substantial pore pressures mobilized is not available. There is a lack of case histories to assist in model development and to validate analysis procedures for this case.

(6) Numerical models for composite, remediated or reinforced materials in dynamic analysis are not available. There is a lack of case histories to assist in model development and to validate analysis procedures for this case.

e. Assessing Stability: Criteria for static stability of earth dams is largely based on a long history of many dams built to similar factors of safety against sliding that have performed well. We need more full-scale case histories to test our seismic design procedures against before we have the desired level of confidence.

We need to assess the trade off between conservatism in the selection of ground motions and uncertainties in the site characterization, material properties, identified failure modes and methods of analysis.

Centrifuge models may help us better understand how much displacement/disruption constitutes a threat of internal erosion (filter disruption) or overtopping although the modeling of the process of the erosion of soil by water in the centrifuge environment has not been proven.

f. Summary of Most Important Needs Requiring Major Experiments

(1) Site characterization, define 3D variability.

(2) Empirical correlation of in-situ tests to seismic performance.

(3) Case histories involving large seismically induced deformations for use in numerical model validation.

(4) Re-evaluation of $K_\alpha$ in large and/or better boundary condition tests.

(5) In-situ test to define contractive or dilative behavior.

(6) Experiments to provide insight into uncommon seismic failure modes or loading conditions such as disruption of filters and drains and response of dams in narrow valleys.

3. <u>Recommended Experimental Research</u>

a. <u>General</u>: Not all of the research needs identified could be addressed in the time available. Four experimental programs are outlined below. That was all that there was time available at the workshop to address. The other areas listed in paragraph 2f above, not treated in this section, are also important areas for experimental research.

b. <u>Full Scale Embankment Test</u>: This experiment is described in detail in Appendix 4.1A. It would be one of the largest if not the largest geotechnical field test ever carried out. It is a large controlled, layered, full-scale foundation test bed in an area with a high ground water table which is also a seismic "hot spot." (The site is envisioned as a part of the National Test Site Program. Reference 4, p 4.)

(1) Phase (1) involves constructing the foundation (Figure 1), using the constructed foundation to evaluate all available in-situ test methods, and installing of pore pressure and strong motion instrumentation to record site response in future earthquakes.

(2) Phase (1A) involves constructing an embankment on the foundation and additional insitu testing to show how the presence of the embankment changes the insitu test results in the foundation under the embankment.

(3) Phase (2) involves the addition of a retaining dike and a reservoir (See Figure 2) in preparation for future earthquake loadings of the potentially liquefiable foundation under the dam. Instrumentation installation and long-term maintenance is also included.

(4) Phase (2A) involves pre- and post-earthquake centrifuge modeling of the Phase 2 system. A large size model would be required to replicate the layers in the foundation. A shaker capable of operating on a large geotechnical centrifuge would of course be needed.

(5) Phase (3) involves a buried explosive test on a companion test site constructed to the same specifications so that response (probably short of failure) could be obtained without waiting for an earthquake.

The major technical barriers to this experiment are the (1) placement of certain subsurface layers in the desired state of looseness while placing denser layers above them, (2) definition of size sufficient to remove significant boundary effects and (3) achieving saturation in the foundation layers. This would be a several million dollar, multiagency, multi-year experiment and would require development of a long term funding base, extensive coordination and the location of an appropriate remotely located seismic hot spot. The experimental program would contribute to items 2f(1), 2f(2), and 2f(3).

c. Saturated Slope Test Under Earthquake Loading Conditions: This is aimed at item 2f(4). It is desired to duplicate conditions in which applied shear stresses act on a soil element before the seismic loading and examine $K_{\alpha}$ under different boundary conditions and at larger scale. There are two possible ways of conducting this type of experiment: (1) large laboratory shaking table and (2) constructing an instrumented field experiment in a seismic hot-spot and waiting for an earthquake to occur.

(1) Shaking table test concepts: There are two ways of looking at the problem in the shaking table environment. The first involves testing an element of loose soil placed under controlled conditions, confined with a membrane, backpressured and subjected to a downslope gradient as shown in Figure 3. The second simpler shake table experiment, shown in Figure 4 is a series of shaking tests of small sand slopes of different slope angles saturated as much as possible under atmospheric conditions. This alternative has the disadvantages of being unable to fully saturate the specimens and very low confining stresses (i.e. the sand may be dilative rather than contractive at low effective stress loads). Both of these shake table experiments are feasible on existing U.S. shake tables. Only one directional shaking is required. Because of scaling issues, these tests are, necessarily, phenomenological in nature.

(2) Field Test Concepts: In this case, loose sand slopes of different slope angles would be built against the upstream shell of an existing earth dam, or the upstream face of a gravity dam when the reservoir was empty or low (or simply as embankments in the reservoir during a dry period) at a site in a seismically active region. The sand would be placed under known controlled conditions. Instrumentation to measure induced pore pressure, flow velocity, and extent of run-out would be installed and maintained until the desired full scale shaking event occurred. Given that a site could be found, there are no serious technical barriers present.

d. Case-History Database of Dam Response: A cooperative government agency-university effort is recommended by the panel to develop an international database of case histories of dams that have suffered displacement and/or cracking during earthquakes. This should include both modern engineered large dams as well as smaller structures. Perhaps post-earthquake investigations to evaluate material properties may be required to flesh out an otherwise nearly complete case history. This would, if accomplished, allow the profession to deduce criteria and evaluate new procedures for assessing deformation potential, a need identified in 2f(3) above.

e. Centrifuge Case Histories of Dam Response: This was recognized as an ongoing effort related to d above. The thrust of acquiring new case histories using n·g models subjected to base shaking should be validation of effective stress, constitutive model based, dynamic numerical analysis procedures for predicting seismically induced liquefaction. Of course such experiments should be coupled with laboratory determination of the state and soil parameters required by the constitutive models. The reader should be cautioned that achieving similitude for simultaneous modeling of the elastic wave propagation equation and the diffusion equation is not a simple or fully solved matter and that boundary condition effects in such models are not fully resolved either. The second thrust involves n·g centrifuge model experiments involving unusual failure modes (See 2f(6) above). In addition to the above similitude and boundary issues, the problem of valid experiments is further complicated by a lack of knowledge of how to simulate internal erosion and by the fact that such processes are controlled by elements of the dam whose prototype dimensions are so small that they are difficult to simulate geometrically except on the largest centrifuges. Experimentation to attempt to overcome these difficulties appears to be worthwhile.

f. Benefits of Proposed Experiments: The remediation of an existing large embankment dam (or its foundation) judged to be unsafe in a large earthquake is a project in the $10M+ range. If the proposed experiments were to provide information causing an owning agency to change its conclusion from "unsafe" to "safe" only once, the research would pay for itself. There are many U.S. dams currently being studied to evaluate the seismic safety in the light of improved knowledge of earthquake ground motion threats and the proposed experiments will likely pay for themselves several times over. The converse case, i.e. that new experimental research results will cause us to change our perception from "safe" to "unsafe" and thus require more money to be spent or remediation must be recognized but in that case the avoidance of a failure as the result of new knowledge will have even larger cost savings (property damage prevention) and will also save lives. The proposed experimental research is definitely in the interest of greater public safety.

## 4. Concluding Remarks

a. Impacts on Facilities: To carry out experiments described in paragraphs 3b and 3c(2), we need acceptable full scale test sites in seismically active areas which do not presently exist. As an initial step there should be a search for test sites of the type described. Obviously, if they cannot be found or do not have a likelihood of seeing significant shaking in the next decade, the experiments would not be feasible and without a specific site in mind it is difficult to evaluate technical feasibility, time or cost. No needs for additional shake tables, upgraded shake tables or further development of explosive ground motion generation technology were identified as barriers. What we have now appears to be good enough for the applications proposed here. A need for a centrifuge with a shaker capable of shaking fairly large models of earth dams in a high-g environment was identified in this workshop. Regrettably, the panel did not explore the question of whether this shaker capability should be more than one-dimensional as this is an important cost driver.

b. Barriers: The technical barriers have been discussed with each experiment but can be summarized as follows:

- (1) Full scale experiments
  - (a) Finding a suitable site
  - (b) Difficulty in construction of uniform low density layers and saturating them
  - (c) Uncertainty in how large the experiment must be to avoid boundary effects
- (2) Centrifuge experiments
  - (a) Simultaneous scaling of diffusion and wave propagation
  - (b) Lack of knowledge of how to simulate piping
  - (c) Boundary conditions

One of the research needs expressed in section 2f—an insitu test to define contractive or dilative behavior in soils is itself a technical barrier. There is no generally agreed approach to follow in pursuing this goal and there will have to be quite a bit of trial and error before (hopefully) someone comes upon a test technique with real promise.

The major nontechnical barriers are the coordination planning, multi-organizational effort, justification and funding required to carry out and maintain major long term field experiments. The profession is not used to forming the long term umbrella technical management organizations (government and academia) that would be required to successfully carry out the experiment and maintain its instrumentation until the earthquake eventually occurs. However, recent work in organizing a U.S. system of geotechnical experimentation sites (ref. 4, p. 4) is currently coming to fruition, and it is foreseen that a basic overall management system for this experimentation site system will be in place by 1993. A major charge to this umbrella management group is to facilitate joint efforts by government, academia and private industry at these designated test sites. The proposed large scale experiment would fit very well in this context.

c. Prioritization of Experiments: The panel prioritized the experimental research described in Section 3 in the order listed.

The exercise conducted in this workshop panel was worthwhile in that in it focused attention on the experimental needs related to the seismic safety evaluation of existing earth dams and allowed us to look at whether there were needs for more research or development on dynamic loading apparatus or technologies. The only requirement in this area identified was the need for specimen shaking capabilities on the larger of the U.S. geotechnical centrifuges.

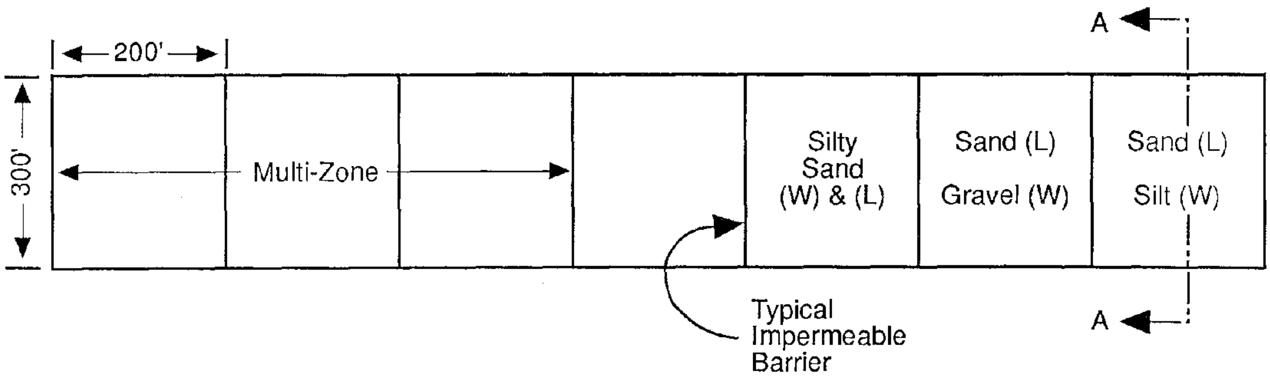
References


1. Civil Engineering Research Foundation, "Setting A National Research Agenda for the Civil Engineering Profession", Report 91-F1003.E, September 1991.

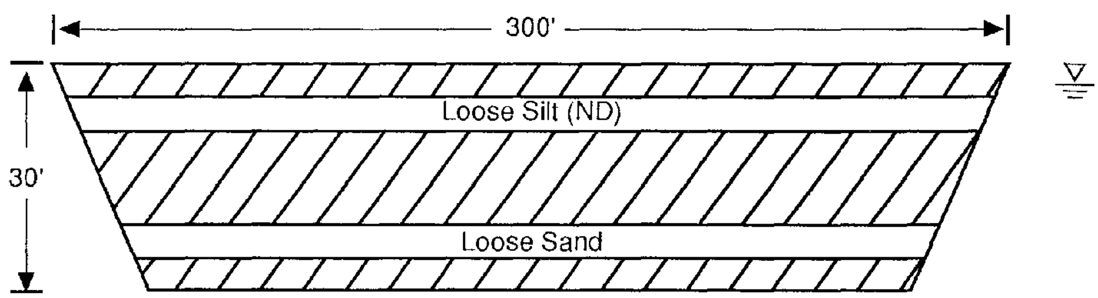
2. Civil Engineering Research Foundation, "Regional Research Needs in Geotechnical Engineering", Report 91-F1002, June 1991.

3. CH2M Hill, "Proceedings NSF/EPRI Workshop", Dynamic Soil Properties of Site Characterization, February 1991.

4. Benoit, J. and de Alba, P. ed, "Designated Sites for Geotechnical Experimentation in the United States", Proceedings of the Workshop at the University of New Hampshire, September 1988.

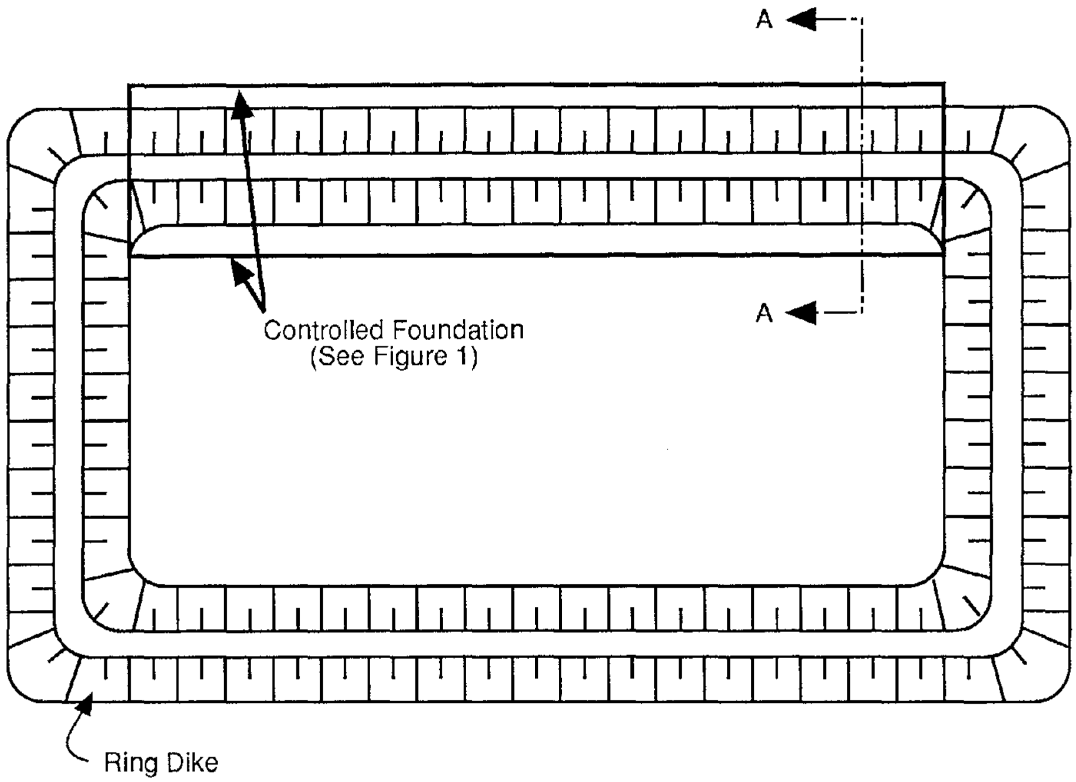


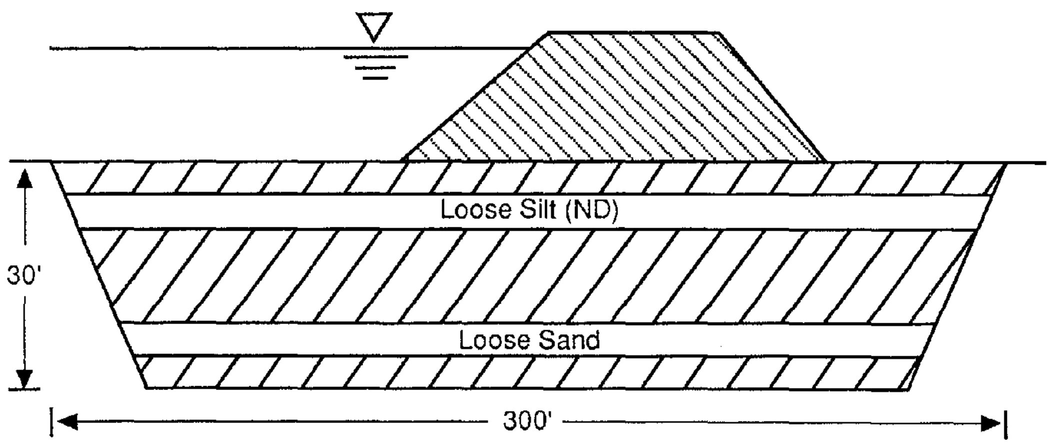

PLAN VIEW




SECTION A-A

Figure 1. Controlled Foundation Test-Bed Construction, Phase 1.




SECTION A-A

Figure 2. Embankment (Phase 1A) and Reservoir (Phase 2) Over Controlled Foundation.

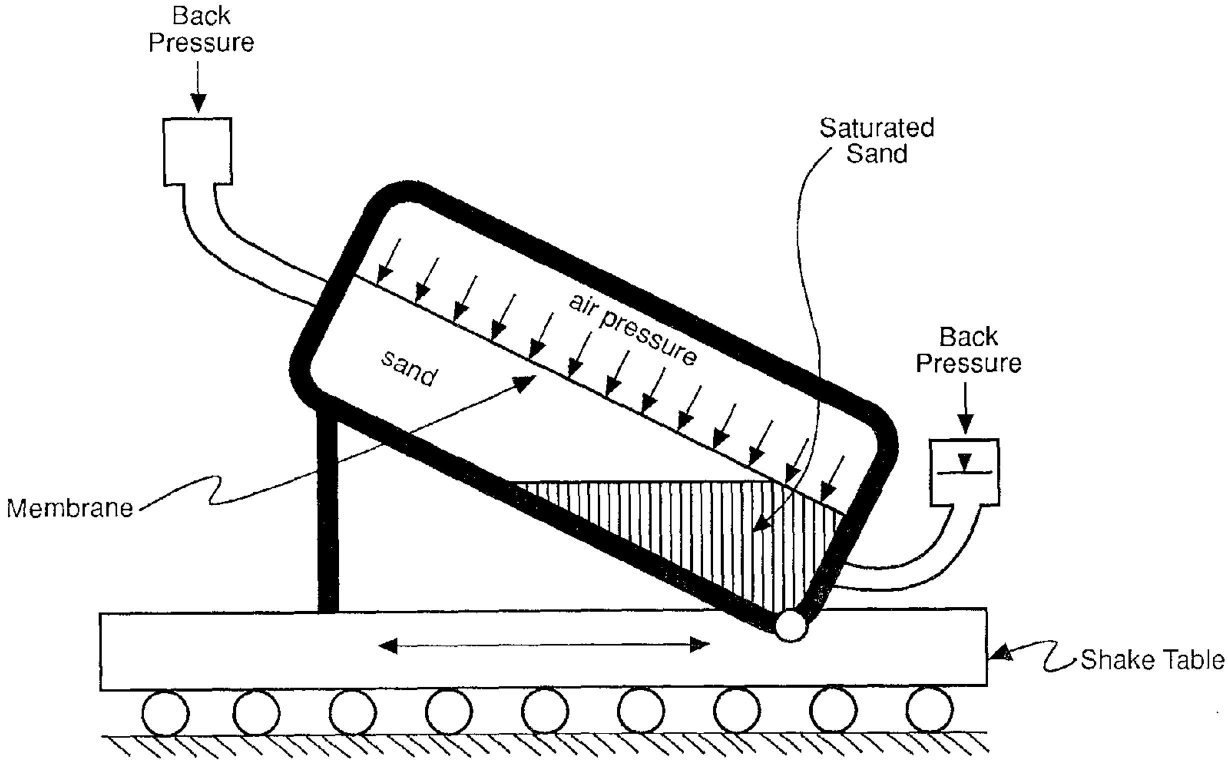



Figure 3. Saturated Slope Under Cyclic Loading, Alternative A.

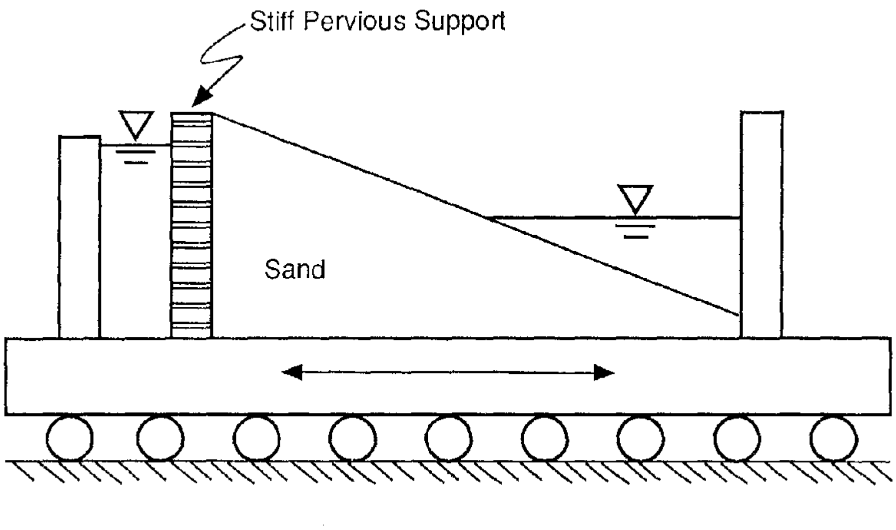



Figure 4. "Saturated" Slope Under Cyclic Loading, Alternative B.

## APPENDIX 4.1A

### Controlled Constructed Foundation Proposal

Proposal—to construct a foundation with known properties for a variety of materials including—clean sands, silty sands, gravels, gravely sands, non plastic silts, sandy silts and silts and sand with some plasticity. These materials would be placed in various potentially liquefiable states and in various configurations (e.g. gravel overlying sand, clay over sand). The deposits would be separated by clay barriers. Approximate dimensions are 1500 ft by 300 ft by 30 ft.

Purpose—Phase 1 - In-situ testing for level ground

- To provide a site with known properties of placed materials to allow testing by various in-situ methods
- This site would in fact become and be operated in a manner similar to a national test site
- The site would be instrumented with pore pressure devices and strong motion instruments to determine the response in the event of an earthquake

Rationale

The use of verified procedures to analyze a field situation requires verification of the effectiveness of various in-situ testing techniques to evaluate properties such as void ratio, $K_o$, OCR, compression index $\lambda$, swell index K, slope M of the critical state line on p-q space, permeability, etc., in situ. Research into the assessment and development of methods to determine the above properties in-situ is needed.

Purpose—Phase 1A - Embankment construction - In-situ testing through the embankment

- To determine the influence of the embankment construction on the materials and allow comparison of the various methods of in-situ testing through the embankment

Purpose—Phase 2 - Addition of a reservoir and retaining dike in preparation for a failure test

- To determine the influence, if any, of the presence of a reservoir on material properties.
- To provide a dam on a potentially liquefiable foundation, "earthquake test bed" that would be well instrumented and would allow verification of analysis methods given an earthquake (with or without failure).

- The deformation response could also be modeled pre- and post-earthquake by centrifuge to determine the liquefiable layer of a soil at a particular depth. Dynamic centrifuge tests on large models are necessary so that small layers of soils can be incorporated in the models. Large capacity shakers are necessary for this purpose.

Purpose—Phase 3 - Explosive test on a companion site constructed to the same specification

- Obtain results on relative liquefiability of various deposits immediately, without waiting for an earthquake

Technical barriers

- Placement of the materials in the desired state of liquefiability.
- Determination of adequate size to remove significant boundary conditions.
- Problems in centrifuge scaling and interpretation of results (Phase 2).
- Dis-similar wave fields from earthquakes and explosions (Phase 3).

<u>Implementation</u>

<u>Phase I</u>

Obtain multiagency, multischool, multiorganization funding support for the controlled foundation construction. Put the fill under control of the National Test Site organization for management purposes. Specifications preparation and site inspection could be accomplished by a multiagency technical team on a "donation basis." Agencies anticipated that could be participating include USCE, (CADWR & other states), USBR, EPRI, EERI, USCOLD, DOE, PGE (and other utilities). Phase I completed by 1994.

Problem with Proposal

- Difficult to ensure saturation of loose deposits and placement of overlying materials.

## 4.2 REPORT OF THE PANEL ON FOUNDATIONS

Chair: Gary Norris, University of Nevada
Recorder: Carl Constantino, City College of New York

Panel: Jean Benoit, University of New Hampshire
George Gazetas, State University of New York
Jeffrey Simons, SRI International
Kenneth Stokoe, University of Texas
Stuart Werner, Dames and Moore

Organizing Committee Representative: Ronald Scott, California Institute of Technology



Preceding page blank

## Knowledge Gaps

Foundation response under seismic excitation can be a very complex problem with which to deal. There are a number of foundation types to consider. They are, in order of complexity:

- Footings/rigid mats
- Flexible mats
- Caissons/large diameter shafts
- Pile groups
- Hybrid, e.g., abutment wall
- Special, e.g., containment structures

Likewise, there are a number of analytical techniques available ranging from the simple springs model (springs for vertical and lateral rotational response requiring only material stiffness and damping), all the way to complex numerical models with detailed constitutive relations. Figure 1 presents a matrix of analytical techniques versus foundation type.

Figure 1. Matrix of Methods of Analysis vs. Foundation Type.

| Analysis Methods | Foundation Type | | | | | |
|---|---|---|---|---|---|---|
| | Footings | Flexible Mats | Caissons | Piles | Hybrid (Abutments) | Special |
| Simple Springs | | | | | | |
| Complicated Springs (p - y) | | | | | | |
| Hybrid Finite Element Springs to Far Field | | | | | | |
| Finite Element Boundary Solutions | | | | | | |
| Complete Numerical Models with Constitutive Relations | | | | | | |

In Figure 1, general foundations are presented varying from individual footings to hybrid systems. These foundations may be further subdivided into the following types:

- Individual footings
- Rigid mats
- Flexible mats
- Multiple footings
- Caissons
- Single piles
- Pile groups
- Hybrid systems (for example: bridge abutments on piles, etc.)

Our knowledge of the performance of different foundation types during earthquake shaking decreases as system complexity increases. Further, site characterization increases in importance and extent as system complexity increases.

Special foundations refer to foundations of special and unique facilities (e.g., nuclear plants, LNG facilities, etc.) or to unique foundation conditions for which detailed foundation design and more complex SSI analysis procedures would be required.

It may be observed that the axis of the matrix representing foundation types also refers to the soil conditions on the one hand and complexity and cost of the structure on the other. Simple, lightly loaded structures in good soils are supported by individual footings. Heavier structures in poorer soils require foundations which range from mats in relatively better surface soils to caissons and piles in bad soil conditions, where, for example, soil improvement may be considered as an alternate technique. Figure 1 is thought to represent a logical way of thinking about the relationships, but the panel did not have adequate time to explore the relationships in detail.

The knowledge gaps relative to the use of such analytical models include:

1. Choice of soil modulus for intermediate to large strain response in the simple model, to assessing the multiple soil property value input to the constitutive relationship for the complex model.

2. Considerations of damping and its characterization, i.e., material, geometric, and other energy dissipation mechanisms.

3. Effect of foundation embedment.

4. Pile group or multiple footing interference effects.

5. Consideration of soil softening profile (cyclic degradation of clay or pore water pressure buildup in sand).

6. Limitations in terms of associated response.

    a. Near surface liquefaction and flow-around loading of the sides of the foundation (from the residual strength of the liquefied soil).

    b. Consideration of foundation behavior in an improved foundation soil (geotextiles, straps, root piles, etc.).

It should be noted that in going down the list of analytical methods in Figure 1, issues of embedment, group interference effects, cyclic degradation, etc. are more directly taken into account. However, the more complex the model, the more soil property input values are required.

With regard to the analytical models, there is a need to discuss the full range of models because it may be more effective to use the simpler ones with a better delineation of standard soil properties for applications where there is classical resistance in the profession to the consideration of soil-structure interaction response. Simple models will go a long way to demonstrating, economically, the importance of such considerations.

Simple SSI models and procedures may be appropriate in seismic design standards and codes for conventional buildings, bridges, and other structures. The conditions of applicability of such models and procedures should be assessed through correlation with data from suitable strong motion instrumentation arrays, field test programs, and centrifuge test programs.

Once complex nonlinear models have been validated, an important use will be as an additional method for defining the conditions under which simple spring-dashpot models can be applied to the conventional design applications noted above.

Experimental studies are required to provide a basis for validation of each method of analysis and its range of applicability. The more complex analytic methods which may be feasible require detailed specification of appropriate constitutive models.

While it was decided not to prioritize investigation of all the different foundation types, the panel did feel that seismic characterization of the pile group is a very high priority. However, toward such an end, single pile behavior also needs to be investigated because it is part of the full (pile group) behavior.

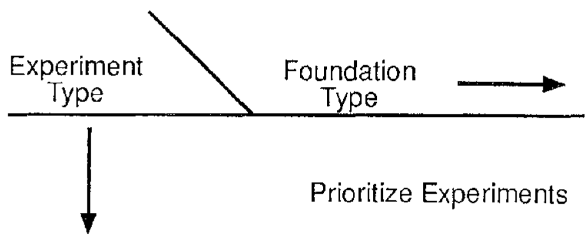
Highest Priority Research Item

Both kinematic and inertial interaction effects need to be considered. The effects of nonlinear soil behavior are presently accounted for only in ad-hoc empirical/intuitive ways; research is needed towards developing more systematic methods of handling nonlinearities.

Experiments

Similar to the matrix characterization of the method of analysis vs. the foundation type, a similar matrix of experiment type vs. type of foundation might be proposed (See Figure 2). As with Figure 1, Figure 2 presents a logical way of thinking about the relationships, but the panel did not have adequate time to explore these relationships in detail. Nevertheless, we present them as a basis from which other researchers can develop research programs to complete the matrices. The types of experiments that would constitute the vertical axis of that matrix are listed in Table 1. Related parameters and conditions are listed in Table 2.

Figure 2. Matrix of Experiment Type Vs. Foundation Type.




With regard to the different experimental techniques, there is the question of what will be experimentally simulated or tested. Some of these parameters and conditions are listed in Table 2.

Table 1

Types of Experiments[1]

1. Centrifuge - Laboratory experiments to evaluate applicability of equivalent linear model to 2D and 3D computations

2. Model lab/field test (hydraulic loading)

3. Scaled field test dynamically loaded (blast, bladder, water hammer, vibroseis)

4. Piggy-backed test (Parkfield, Lotung, Hualien, DOE/DOD programs)

5. Full-scale tests on foundations
   a. Existing/abandoned sites (Cypress Viaduct in Oakland (I-880), Naval Facility at Oakland outer harbor)
   b. Newly designed/constructed with planned instrumentation (Treasure Island)

6. Full-scale bridge/structural tests[2]
   a. Meloland Overcrossing
   b. Milliken Library
   c. San Bernadino Bridge

7. Instrumented bridge/structural response in EQ[2]

8. 6 & 7 compared

9. Shake table test using appropriately scaled material strength

10. Foam rubber modeling

---

1. Need high quality lab/field tests for soil property characterization for all of the above.
2. Need movable instrumentation package for aftershocks for structure/foundation response characterization.

Table 2.
Some Experimental Parameters or Conditions

1. Mode of Inertial Excitation
   - Lateral
   - Rotational
   - Combined

2. Free-Field Strain (simulated or not)

3. Soil
   - Sand
   - Clay
     - Virgin cyclic
     - Degraded
   - Layered (impedance contrast)

4. Elevated Pore Water Pressure (sands)

5. Loading Rates

6. Footing or Pile Caps
   - Free standing or at ground surface
   - Embedded

7. Pile Number and Spacing
   - Fixed
   - Variable

Most Effective Expenditures

The following statements outline the main considerations for, in the panels' judgement, the most effective expenditures of money on an experiment.

1. Earthquake response experiment - 1 and 4 pile clusters - full scale

   a. Parkfield
   b. Taiwan - Lotung and Hualien - piles, caissons, footings at different depths
   c. San Bernardino
   d. Meloland (depending on ground water level)
   e. Other US and foreign arrays or targets of opportunity

2. DOD blast simulation programs

3. Movable instrumentation package for aftershock
   Structure/foundation response

4. Centrifuge

5. Full-scale testing on foundations/structures - no free field excitation

   - Millikan Library
   - Cypress Viaduct in Oakland (I-880), Naval Facility at Oakland outer harbor)

Technical Barriers

Some technical barriers to experiments are listed below:

1. Earthquake response experiments

   - longevity of buried instrumentation
   - installation of transducers

2. Instrumentation Package (see footnote to Table 1)

   - Possible site descriptions and structural response evaluation
   - Pore pressure transducers
   - Accelerometer strings (need development)

3. Centrifuge

    - Miniaturization of instrumentation
    - Pore pressure simulation (dynamic/dissipation scaling)
    - Inflight installation of piles difficult
      (See H.Y. Ko) - Inflight determination of soil properties

4. Full scale tests

    - Virgin tests to high loads last

<u>Non Technical Barriers</u>

Some non-technical barriers are listed:

1. Earthquake

    - Maintenance and demobilization/clean up, access, both site and data

2. Coordinating with DOD, mob/demob, special equipment

3. Availability of maintenance

    - Liability - Preplanning for instrumentation setup

4. Coordinating with host facility

5. Coordinating with owner -access

    - Liability - Mob/demob, special equipment

## 4.3 REPORT OF THE PANEL ON GROUND INSTABILITY AND SITE IMPROVEMENT

Chair: Arley Franklin, U.S. Army Engineer Waterways Experiment Station
Recorder: Maurice Power, Geomatrix Consultants

Panel: Scott Blouin, Applied Research Associates, Inc.
Wayne Charlie, Colorado State University
Thomas Holzer, U.S. Geological Survey
Kenji Ishihara, University of Tokyo, Hongo
Bruce Kutter, University of California
Chih-Kang Shen, The Hong Kong University of Science & Technology
Joseph Welsh, Hayward Baker, Inc.

Organizing Committee Representative: Koon Meng Chua, University of New Mexico



Preceding page blank

## INTRODUCTORY SUMMARY

There are fourteen knowledge gaps identified related to the consideration of ground instability and site improvement in earthquake geotechnical engineering. The major headings are liquefaction, deformation, site improvement and site characterization.

The specific knowledge gaps considered in subsequent sections are presented in the following manner: (a) a description of the specific area in which the lack of knowledge is impeding accurate predictions of earthquake effects, (b) a discussion of the various experimental approaches which may close the knowledge gap, and (c) targets of opportunity which may be afforded in performing these experiments.

The attached table summarizes the issues considered and prioritized in the descending order of importance. The different experiments which may be considered are ranked with three (3) being the most promising.

The knowledge gaps identified were:

Liquefaction

1. Influence of stress state on pore pressure generation.
2. Mechanisms of pore pressure buildup, dissipation, and redistribution in heterogeneous materials.
3. Effect of grain size, plasticity, and cementation on liquefaction susceptibility.
4. Correlation of in situ test data with liquefaction susceptibility.

Deformation

5. Factors affecting residual strength.
6. Estimation of liquefaction-induced settlements and lateral displacement.
7. Effects of 3-D geometry on earthquake-induced deformations.
8. Permanent deformations in cohesive soils under cyclic loading.

Site Improvement

9. Dynamic behavior of complex heterogeneous materials.

| KNOWLEDGE GAPS* | EXPERIMENTS** | | | | | |
|---|---|---|---|---|---|---|
| | POST EARTHQUAKE FIELD OBSERVATIONS | INSTRUMENTED FIELD SITE | | CENTRIFUGE MODEL | SHAKING TABLE | LARGE LABORATORY ELEMENT TEST |
| | | EARTHQUAKE | ARTIFICIALLY INDUCED SHAKING | | | |
| LIQUEFACTION | | | | | | |
| 1. Influence of stress state on pore pressure generation | 0 | 3 | 2 | 1 | 1 | 3 |
| 2. Mechanisms of pore pressure buildup, dissipation and redistribution in heterogeneous materials | 1 | 3 | 3 | 2 | 1 | 2 |
| 3. Effect of grain size, plasticity, and cementation on liquefaction susceptibility | 2 | 3 | 1 | 3 | 1 | 3 |
| 4. Correlation of in situ test data with liquefaction susceptibility | 3 | 3 | 2 | 2 | 1 | 3 |
| DEFORMATION | | | | | | |
| 5. Factors affecting residual strength | 3 | 3 | 1 | 3 | 1 | 2 |
| 6. Estimation of liquefaction-induced settlements and lateral displacement | 3 | 3 | 1 | 3 | 1 | 2 |
| 7. Effects of 3-D geometry on earthquake-induced deformations | 3 | 3 | 1 | 2 | 1 | 0 |
| 8. Permanent deformations in cohesive soils under cyclic loading | 2 | 3 | 1 | 2 | 2 | 3 |
| SITE IMPROVEMENT | | | | | | |
| 9. Dynamic behavior of complex heterogeneous materials | 1 | 3 | 3 | 2 | 1 | 3 |
| 10. Assessment of effectiveness of site improvement techniques | 1 | 3 | 3 | 3 | 1 | 1 |
| 11. Determination of spatial extent and degree of improvement required | 0 | 1 | 3 | 2 | 1 | 0 |
| 12. New methods for site improvement | | | | | | |
| 13. Design and control of underwater fill | 1 | 2 | 2 | 3 | 1 | 2 |
| SITE CHARACTERIZATION | | | | | | |
| 14. Determination and description of site conditions for input to analysis | 1 | 1 | 1 | 1 | 1 | 0 |

* ranked in the order of importance/urgency
1=most important/urgent

**ranked in terms of applicabiliy
3=highly applicable

10. Assessment of effectiveness of site improvement techniques.

11. Determination of spatial extent and degree of improvement required.

12. New methods for site improvement.

13. Design and control of underwater fill.

Site Characterization

14. Determination and description of site conditions for input to analysis.

**LIQUEFACTION**

1. Effect of initial shear stress including $K_o$-condition can influence the pore water pressure generation under earthquake loading

The effects of the at-rest stress condition in the soil on the pore-water pressure generation mechanism during an earthquake are ill-defined.

Experiment:

- Laboratory triaxial, simple shear or torsional shear with different $K_o$-conditions in the field using pressuremeter or dilatometer to determine $K_o$-stress in situ stress state.

2. Mechanisms of pore pressure buildup, dissipation and redistribution

Current state-of-the-art does not consider influence of site inhomogeneities on pore pressure buildup, dissipation and redistribution. Therefore occurrence and duration of pore pressure build-ups is uncertain.

Experiments:

- Improved site characterization to determine location and extent of inhomogeneities.
- In situ tests to measure,
  -- permeability (packer tests, active cone penetration tests,

etc.),

-- pore pressure dissipation (packer, cone, vane shear with pore water pressure measurements), and

-- stress-strain properties (pressuremeter, dilatometer, step blade).

- Laboratory tests to correlate and calibrate in situ techniques,

  -- calibration vessel tests, and

  -- lab single-element tests.

- In situ calibration tests at sites of past field studies and future experiments such as national test sites.

3. <u>Determine the effect of gravel, plasticity and amount of fines, and cementation on the cyclic resistance of sand, also the cyclic resistance of wetted collapsible soils</u>

The effect of gravel, plasticity and amount of fines, and cementation on the cyclic resistance of sand is not well-defined. Additionally, the cyclic resistance of wetted collapsible soils, which may be naturally deposited, or placed as land-fills, is becoming an important issue. Industrial developments are beginning in these less preferred areas.

Experiments:

- Laboratory tests.
- Centrifuge tests.

4. <u>Correlation of in situ test data with liquefaction susceptibility</u>

The most widely used correlation for predicting liquefaction is Seed's correlation between occurrence of liquefaction and site properties determined by the Standard Penetration Test [SPT]. The geotechnical community should develop new correlations for liquefaction based on more modern site characterization techniques. Reasons for this recommendation are:

(a) The SPT is beset with numerous shortcomings, (e.g., lack of standardization, operator bias, difficulty obtaining uniform energy at the sampler, general lack of continuous sampling, insensitivity to thin layers, etc.)

(b) Availability of more accurate, relevant characterization techniques including

-- shear wave velocity profiles
-- piezo-electric cone
-- CPT
-- downhole density
-- pressuremeter
-- stepped blade
-- dilatometer
-- piezo vane
-- standardized contained explosive tests

Experiments:

- Develop correlations in controlled laboratory environment.
- Development of field correlations at sites of known occurrence or non-occurrence of liquefaction.
- Development of a data base at sites of previous correlation efforts and future characterization efforts, such as the national test sites.

**DEFORMATIONS**

5. <u>Factors Affecting Residual Strength</u>

The application of residual strength or steady state strength relies on the assumption that the soil is either completely drained or completely undrained. The excess pore pressure produced during shaking, however, causes flow of pore water from zones of high pressure to zones of low pressure. This can cause local drainage or changes in void ratio which result in a change in steady state strength. If there are layers of non-uniform permeability, layers of non-uniform permeability, the pore water may tend to accumulate at the interface causing large changes in void ratio at the interfaces. Thus residual strength is likely to be a function of more factors than just relative density or penetration resistance such as thickness and permeability of the granular, and permeability of the soil capping the liquefied zone.

Other soil properties that likely influence residual strength include grain size, fines content, and clay content.

Experiments:

- Post-earthquake field observations.
- Instrumenting earthquake-prone sites.
- Perform centrifuge experiments.

## 6. Estimation of liquefaction-induced settlement and lateral displacement

The prediction of displacement in soil that has experienced high pore water pressure buildup during earthquake remains one of the most difficult problems to solve in earthquake engineering.

(a) The lateral deformation caused by liquefaction is affected by the duration of time over which high pore pressures are present. This is affected by the permeability of the liquefying soil, the permeability of the overburden and the thickness of the various layers. In addition, the permeability of "liquefied" soil may be substantially higher than that for the intact soil. Those relationships must be clarified.

(b) The triggering of flow slides depends on whether the soil is softened to a steady state strength which is lower than that required for static stability. The amount of straining (and perhaps void ratio redistribution) required to trigger this softening needs further study.

(c) It has been proposed that dilatant soils will not suffer large deformation. It is possible that due to pore pressure redistribution, that initially dense soils may become loose enough to flow. It is also possible that medium dense soils may experience large deformation through accumulated effects of many cycles of strong shaking.

(d) Settlement. The magnitude of settlement of level ground is due primarily to its volumetric strains. It may be that in areas where boils are prominent, additional settlement can be caused by ejection of deep soils to the surface. For cases where sloping ground is present (even for slopes of a few degrees), small lateral movements can result in significant vertical movements which may be swelling of higher deposits, of lower deposits, and settlement.

Experiments:

- Field Observation, instrumented sites and centrifuge testing to investigate the effects of layers. The number, thickness, permeability, slope angle and end conditions (free face, or infinite slope) should be varied.
- Simple shear tests could also address interface (at top of sample) and sustained stresses.

*Technical problems.*

Field observation and instrumented site approach: (a) obtaining the time histories of deformation, and (b) ensuring adequate means of tracking displacements.

Centrifuge: (a) the limited size of model, (b) the large transducer dimensions, and (c) scale effects.

Explosives: may result in non-uniformity of motion over depth and area of the test domain.

*Non-Technical Problems.*

Field Observation: (a) getting data from restricted damage zones, (b) the lengthy return period, and (c) there are only a small number of events.

Centrifuge: complex geometry requires large centrifuge shake tables which can be costly.

Target of Opportunities:

- Surveying lateral movement from previous earthquakes.
- Low altitude aerial photography.
- Instrument Treasure Island site with inclinometer.

7. <u>Effects of 3-D Geometry</u>

Lateral displacement generated by liquefaction, usually in the form of lateral spreading, depends on several factors including intensity and duration of ground motion, density, permeability, thickness and extent, and 3-D boundaries to the liquefaction layer. Present analytical and empirical criteria for predicting displacement are not sufficiently developed to account for many of these factors, particularly the boundary effects. More research is required to evaluate the influences of these factors. With respect to boundary effects, more detailed investigations of failures are required where measured vectors and profiles of ground displacement can be compared. With carefully delineated site stratigraphy and measured soil properties.

Experiments:

- Post-earthquake field observations.
- Instrumenting earthquake-prone sites.

8. Permanent Deformations in Cohesive Soils Under Cyclic Loading

Although the main knowledge gap in predicting soil displacements (particularly lateral displacements) is with respect to liquefiable cohesionless soil, cohesive soils, particularly sensitive clays, also pose significant problems in geotechnical engineering practice. Large deformations in sensitive clay layers are thought to have been the cause for the Fourth Avenue landslide during 1964 Alaska earthquake, and soft, sensitive clays exist in other seismically active parts of the United States. The combined effects of frequency and loading rate, number of cycles, and amount of displacement on the strength available to resist deformation throughout the duration of strong shaking are not well understood at present. Higher rates of loading may increase soils strengths above static values, but if larger displacements start to develop, strength might be significantly reduced in sensitive soils, leading to still larger displacements.

Experiments:

- Laboratory cyclic tests on soil samples subjected to initial static shear stresses.
- Instrumentation of shoreline sites of cohesive soils in highly

seismic regions (survey measurements, aerial photography, inclinometers, accelerometers).

- Measure displacements at various depths by means of instrumentation emplaced at various depths), centrifuge (if modeling of slopes can be adequately accomplished), and shaking table (if confining pressures and soil strengths can be achieved).
- Post-earthquake observations of sites (even if not instrumented) that have deformed or not deformed. The most effective techniques appear to be instrumentation of field test sites and laboratory testing. Shoreline sites in the San Francisco Bay area underlain by San Francisco Bay Mud appear to be good candidate sites.

## SITE IMPROVEMENTS

### 9. Dynamic Behavior of Complex Heterogeneous Materials

Site improvement to solve potential earthquake problems is a relatively new and continually expanding field. Many of the site improvement techniques actually liquefy the soil in accomplishing their improvement (e.g., Dynamic Compaction, vibro-compaction, vibroplacement, compaction by explosives, etc.), but seldom is this monitored as it occurs. This is an important missed project opportunity.

Additionally, foreign materials, such as geosynthetics, stone columns, etc., are usually introduced into the ground as part of the site improvement process. The effect of the resulting heterogenuity (vertically and/or laterally) on the integrity of the improved site against dynamic loading is not well-defined. The problem of accurately modeling material interfaces under dynamic loading is a challenging one.

Experiments:

- Observations of the behavior of treated test sections under either natural earthquake shaking or by artificially induced shaking.

- A promising initial approach is to test large-diameter specimens of composite materials, such as sands with inclusions of grout, in a laboratory triaxial device.

10. Assessment of Effectiveness of Site Improvement Techniques

The techniques to modify the ground have been basically contractor-developed and this has left gaps in potential development of techniques with little basic research. There is a need to determine the effectiveness, as well as the comparative effectiveness of the different site improvement techniques. The research community should attempt to catch up with the state-of-practice by studying the fundamental mechanisms for site improvement.

Experiments:

- Observations of the behavior of treated test sections under either natural earthquake shaking or by artificially induced shaking.
- Centrifuge models also offer some promise.

11. Determination of Spatial Extent and Degree of Improvement Required

Designers of projects in liquefiable potential specify soil improvement techniques with increasing level of confidence in the end results depending on their experience with the techniques, and specify verification testing such as SPT, CPT, etc., also depending on their comfort level with that verification method and the size of the project. What is constantly missing is instrumentation to verify the effectiveness of the site improvement when an earthquake occurs. Other questions remaining unanswered are: (a) what areal extent of the site needs be improved to adequately afford stability during ground motion, and (b) what degree of improvement over the characteristics of the original is considered adequate.

Experiments:

- Laboratory testing should also be the normal first approach to experimental work in support of the development of new methods of site improvement.
- Centrifuge models may offer some promise.

12. New Methods for Site Improvement

New methods of site improvement may be considered once the questions concerning the dynamic behavior of fills modified by inclusions (as in geosynthetics, etc.) and regarding the effectiveness of existing site improvement techniques are answered.

13. Design and Control if Underwater Fill

There are land reclamation projects around the world and in earthquake-prone areas. Little is known as to how the fill should be best placed in order to improve its resistance to earthquake loading, nor are there standard measures which describe the integrity of this type of fill.

Experiments:

- Laboratory testing may explain the behavior of underwater fills placed using different methods.
- Centrifuge models may offer some promise.

Target of Opportunity:

An attractive target of opportunity for investigation of underwater fill is seen in the proposed Los Angeles Harbor Project.

**Site Characterization**

14. Determination and Description of Site Conditions for Input in Analysis

Site characterization is vital to all types of geotechnical problems. Two issues are involved:

(1) Determination of soil properties with sufficient detail to

define all variations that are important to site response,

(2) Representation of the site conditions in a way that is suitable for input to analyses.

The geotechnical community should be using more sophisticated methods of in situ testing and evaluation, such as: CPT, shear wave velocity measurements, other geophysical techniques, and should be developing a data base which can be used to relate the exploration methods to each other and to information on site characteristics.

Experiments:

- Experimental methods in the matrix are only indirectly applicable to this problem.

## 4.4 REPORT OF THE PANEL ON GROUND MOTION RESPONSE

Chair: William Joyner, U.S. Geological Survey
Recorder: John Schneider, Electric Power Research Institute

Panel: Mehmet Celebi, U.S. Geological Survey
James Costello, U.S. Nuclear Regulatory Commission
Takeji Kokusho, Central Research Institute of Electric Power Industry
Abiko Research Laboratory, Japan
Geoffrey Martin, University of Southern California
Tang-Tat Ng, University of New Mexico
Anthony Shakal, California Division of Mines and Geology
Felix Yokel, National Institute of Standards and Technology

Organizing Committee Representative: H.T. Tang, Electric Power Research Institute



Preceding page blank

# Ground Motion Panel Report

## 1. Background

Ground motion defines earthquake loading input to a structure. It plays a fundamental role in seismic design considerations. To properly characterize earthquake-induced ground motion, one needs an understanding of earthquake source mechanisms, wave propagation path behavior and local site effect. Although the boundary of seismology and geotechnical engineering is not clear cut, the first two areas (involving such topics as rupture mechanics, wave attenuation, and others) are of greater interest to seismologists and the third area (concerned more with soil-structural interaction and soil amplification) is of greater interest to geotechnical engineers. With this in mind, the ground motion panel tried to narrow its discussions on geotechnical engineering topics; experimental needs to better define seismological parameters were not explicitly identified. The state-of-the-art paper on ground motions given in this proceedings points to the fact that the level of complexity of ground motion characterization depends on the importance or criticality of the facility being designed or evaluated. Experimental needs vary from those needed to verify simple code requirements to those needed to calibrate complex two- or three-dimensional soil-structure interaction evaluations.

Some research needs were identified and summarized in the state-of-the-art paper. Development and operation of field tests in seismically-active areas was identified to be one of the highest priority needs since laboratory testing has limitations in characterizing ground motion characteristics. Also identified to be important is characterization of in-situ soil properties, which demands continued research efforts even though significant progress has been made in recent years.

In the area of soil-structure interaction (SSI), a separate state-of-the-art paper on the subject offers an extensive review. Large-scale field test programs such as the one at Lotung, Taiwan have yielded quantitative information and findings. However, for pile foundation, earth dam and bridge structures, case histories and/or field test data are lacking.

Prior to the workshop, participants were requested to submit in writing their comments on experiment needs for geotechnical earthquake engineering. Pertinent to ground motion, most participants expressed needs in field experiment and soil-structure interaction effects quantification.

## 2. Major Deficiencies Identified by the Panel

Eleven topics were identified by the ground motion panel to require further experimental research. Five of these were judged as having high priority and the rest low priority. In the following, the major deficiencies in each topic are discussed and summarized. Subsections 2.1 to 2.5 describe high priority topics.

### 2.1 Characterization of Near-Source Ground Motion

There is insufficient near-source ground motion data, especially for strong motion from M>7 earthquakes, for quantifying variability of source rupture and thereby reducing uncertainties in estimating near-source ground motion. The scarcity and variability of the data makes the earthquake data gathered in active tectonic regions difficult to apply to stable regions.

### 2.2 Soil-Structure Interaction and Spatial Variation of Ground Motion

There have been major experiments conducted to address the needs. However, SSI effects for different site conditions, foundation configuration (pile, mat, ...) and structure types (bridge, dam, high-rise, massive structure, ...) lack quantification because of data insufficiency. Spatial coherency and amplitude variation effects on SSI is a major current research topic. However, experimental data that can be used for validation study are extremely limited, particularly if vertical spatial variation is also considered.

### 2.3 Characterization of In-Situ Soil Properties

To date, there has been significant research in this area, particularly for low-strain properties. Pressure meters, and other instruments have been and are being developed for in-situ high-strain applications. However, these are still at the research stage either because of uncertainty in interpreting data or due to lack of capability to invoke strains sufficiently high to produce nonlinear effects. As a result, in-situ characterization of damping and stiffness (shear modulus) from low to high strain as functions of frequency and depth are either unavailable or unreliable.

### 2.4 Site Response at Soft Soil Sites

There are large uncertainties in understanding soft site soil dynamic properties, especially for clay sites. The categorization of soil-site is ambiguous and lacks precise definitions.

### 2.5 Hidden Liquefaction Potential

Although liquefaction in the stability sense is not the focus of the ground motion panel, it nevertheless affects ground motion determination. In the case of a hidden liquefaction layer at depth, because of no expressions at surface, experimental data to assist ground motion prediction and evaluation is not available. The implication of hidden liquefaction to engineering application is not well understood.

### 2.6 Variability of Site Response at Rock Sites

Rock outcrop motion is often ill-constrained because of variability in near-surface velocity and damping. Moreover, because rock motions are often assumed to be the input to a soil column, for response analysis, greater discrimination and understanding of rock-site variability is greatly needed.

### 2.7 Long Period Motion

This issue has significant bearing on long-period or base-isolated structures where displacement motion characteristics dominate. The effect of shallow sedimentary basins on generating long-period surface waves is not well studied because of lack of data. For 2 to 5 second period motions, experimental data has large uncertainties.

### 2.8 Uncertainty of Ground Motion Estimation

The contribution of source, path and site to variability (randomness) of ground motion is well understood. However, the uncertainties of these contributions cannot be quantified without a large database.

### 2.9 Effect of Surface and Subsurface Topography

The effect of surface and subsurface topography on ground motion estimation is poorly understood. There is a need for better quantification of complex 3-D effects, including inhomogeneities in soil properties, through 3-D modeling/simulations and analysis of dense array data.

### 2.10 Damping at High Confining Pressure

For deep, stiff-soil sites, damping at high confining pressure is of particular importance for soil response determination. Experimental data in this regard is not available.

### 2.11 Vertical Motions

Most attention to ground motion study has been on horizontal motions. For high frequency response in structures, vertical motion is of importance. In recent years, research has been expanded to include vertical motions. Data may be available since earthquakes are almost always recorded with three-component sensors (two horizontal and one vertical). More analysis and modeling is needed.

## 3. Recommended Experimental Research

Associated with deficiencies identified for each topic given in the previous section, the ground motion panel recommended specific experimental research programs. In most cases, field earthquake monitoring stands out as the most needed experiment in dealing with ground motion deficiencies.

### 3.1 Characterization of Near Source Ground Motion, Especially for Earthquakes With M>7

Strong motion arrays close to faults with high probability of large earthquakes are desirable. For prudence, one needs economical and wide-band instrumentation for large quantity deployment. Some specific recommendations include piggyback on existing programs (e.g., Southern California Earthquake Center, USGS and CDMG in California) and assisting maintenance and deployment in foreign countries with high likelihood of large earthquakes (e.g., Turkey, Philippines, Mexico, Peru and Chile). It was also pointed out that industry support should be sought for special arrays or instrumentation sites in areas of high seismic risk.

### 3.2 Soil-Structure Interaction and Spatial Variation of Ground Motion

Dense arrays of strong-motion instruments are needed with the following elements:

- embedded (e.g., 20 ft.) and surface structure of simple design.
- Surface distribution of accelerometers (to 1 Km aperture), including perimeter of structure.
- Vertical array of sensors to define 3-D wave-field.

Specifically, the panel recommended developing arrays around existing structures in San Bernadino County and one of EPRI's four strong-motion structural arrays along the Mexican subduction zone. Also, exploring installation of array at stiff, shear-wall structure (85 ft.) at Mt. Umunhum in the Santa Cruz mountains of California is recommended, recognizing that there may be an extreme topographic effect.

The panel recognizes that there are barriers in accessing desired sites/structures and their general availability.

### 3.3 Characterization of In-Situ Dynamic Soil Properties

Vertical arrays of strong motion accelerometers to record both weak and strong motion are recommended to provide data to back calculate in-situ soil properties. This can also be achieved in conjunction with the SSI arrays described in 3.2. For longer term research, innovative technology development for high strain in-situ testing should continue including further development of a pressure meter for cyclic loading at depth, in-situ freezing for laboratory testing of undisturbed samples, vibroseis (vertical or shear source) to generate strong surface waves, in-situ strain measurements and tools for in-situ tests. Particularly for developing innovative devices, international cooperation is encouraged, although patent rights and other practical matters may be obstacles for such an effort. Further improvements of cross-hole and down-hole measurements for high-strain conditions should be explored together with investigating P-SV conversions from explosive sources.

The panel recommended using bore holes already drilled for characterization of strong motion sites such as in the San Francisco bay area for more extensive in-situ testing.

### 3.4 Site Response at Soft Soil Sites, Especially Clay Sites

It is desirable to have vertical arrays of accelerometers to bedrock with multi-level monitoring in the soil column to record weak and strong motions. Centrifuge tests of the soil column were recommended to characterize and validate soil constitutive properties. Some specific proposals include exploring and instrumenting a bay-mud site in the San Francisco bay area and Osaka Bay in Japan.

### 3.5 Hidden Liquefaction Potential

Existing strong motion data should be examined to identify potential sites for in-situ monitoring. In addition to vertical arrays of accelerometers, piezometers in a liquefiable layer supplemented by settlement gages are recommended to substantiate interpretation of ground motion data. The San Francisco bay area and Imperial Valley may offer some sites for the desired experiment.

## 3.6 Variability of Site Response at Rock Sites

Routine characterization of local geology and near-surface velocity and damping at strong motion sites is recommended. This information should be utilized to develop rock-site categories for input to ground motion databases.

## 3.7 Long-Period Motion

In addition to instrumenting large basins with strong motion accelerometers, it is recommended that surface waves from the world-wide database ($M>6$) be analyzed in depth.

## 3.8 Uncertainty of Ground Motion Estimation

Experimental needs identified are:

- Near-source ground motion for large events from strong motion arrays.
- Estimation of material property measurement uncertainty (e.g., Turkey Flat, Lotung).
- Estimation of ground motion variability due to wavefield scattering using dense arrays.

Coupled with data acquisition, numerical modeling of 3-D effects and scattering in inhomogeneous media should be conducted.

## 3.9 Effect of Surface and Subsurface Topography

Experimental needs identified include:

- Quantification of 3-D structure effects from small arrays by recording weak and strong motions.
- Topographic modeling from dense 2-D surface arrays with explosion, vibroseis or small-earthquake as source.
- Scaled laboratory modeling test using foam rubber.
- Application of radar, electrical resistivity and other geophysical methods to site characterization.
- Small-scale centrifuge simulation in laboratory.

### 3.10 Damping at High Confining Pressures

The deep vertical arrays recommended in previous topics should provide data for quantification of damping at high confining pressure. For longer-term research, one needs to look into development of improved laboratory techniques for high-pressure measurements.

### 3.11 Vertical Motions

No specific experiment needs are identified. All the field data recorded normally always include vertical motion components. The panel's recommendation was that existing data be analyzed and modeled.

## 4. Concluding Remarks

Field arrays (surface and vertical, around structure and in the free-field) were identified in almost all the topics discussed for both high priority topics and low priority ones. The ground motion panel also identified specific potential sites for such an effort. Although there may be difficulties in accessing certain desirable sites or structures, the major non-technical barriers appear to be financial support and administrative coordination. Some sites have ongoing programs already. One needs to get funding to piggyback (coordinate) with the existing program to expand the scope for various specific needs. Out of the potential sites identified, the San Francisco bay area was mentioned the most and therefore offers the best candidate sites for ground motion investigation. Cooperation with other programs such as the Lotung and Hualien ones sponsored by EPRI and Taiwan Power Company together with the U.S. NRC and companies in Japan, France and Korea, should be explored. The Lotung and Hualien programs and the Turkey Flat one are good models of pooling resources and ideas for large scale experiments at limited ideal sites. The state-of-the-art paper on Full-Scale Field Tests at Sites Subject to Earthquake Shaking in this proceedings describes in some detail the many ongoing cooperative programs and testing sites. Some of these have potential for expanded field experimental program development to meet the identified needs.

Equally important to field array installation, soil property characterization was identified as needing further improvement, in particular, in the area of in-situ test and measurement. Some advances have been made, e.g., the pressure meter technique and large penetration test. However, uncertainties remain in interpreting the data. Furthermore, in-situ high strain data is not attainable to correlate with laboratory findings. Innovative ideas and techniques need to be developed. The centrifuge test is theoretically feasible for scaled testing in all strain ranges in the laboratory to characterize soil properties and other geometric effects. However, the application of centrifuge data is a topic of research itself. For the case of combining centrifuge with earthquake motion, data modeling and interpretation pose an even greater challenge.

Explosive experiment was not considered as having much merit by the panel for ground motion study primarily because it lacks the ability to generate wave field of earthquake motion characteristics and thus makes data interpretation and application ambiguous. However, in a limited way, it can furnish strong motion data to investigate dynamic characteristics of a structure in a soil environment and validate in part analytical modeling capabilities for soil-structure systems.

## 4.5 REPORT OF THE PANEL ON NATURAL SLOPES

Chair: Nicholas Sitar, University of California at Berkeley
Recorder: Peter Robertson, University of Alberta, Canada

Panel: Edwin Harp, U.S. Geological Survey
Roman Hryciw, University of Michigan
David Keefer, U.S. Geological Survey

Organizing Committee Representative: T. Leslie Youd, Brigham Young University

## Natural Slopes

The group decided that the scope should be limited to all natural slopes subjected to earthquake loading excluding those that experience liquefaction. This would also exclude sensitive clay slopes that may liquefy. The group felt that liquefaction would be adequately covered by other panels.

The major perceived knowledge gaps related to earthquake engineering of natural slopes are as follows:

1. Documented quantitative studies of slope failures.
2. Prediction of site response and topographic amplification.
3. Dynamic behavior of complex heterogeneous materials.
4. Groundwater/pore pressure response in heterogeneous soil/rock slopes.
5. Runout and crest setback distances.
6. Methods of analysis for complex failures.
7. Effectiveness of mitigation measures.

In selecting the specific areas of experimental research, the panel has carefully considered the effectiveness of the various techniques, including field studies, laboratory tests, scale model tests (centrifuge and shaking table) and numerical or analytical methods. It is the panel's conclusion that there are large gaps in our understanding of the phenomena on the field scale (regional and site-specific), and therefore our recommendation is that field studies and experiments and observations of the phenomena on the field scale should be of highest priority. Scale model tests may play a useful role in follow-up studies and analytical model development, generally after sufficient understanding of the field problems is obtained.

The areas of greatest need and potential greatest impact are as follows:

1. Documentation of case histories and post-earthquake analysis of failed and unfailed slopes

    - Timely quantitative measurements of slope failure characteristics, such as:

        - failure geometry (shape and size)

        - lithology and stratigraphy

        - material properties, to include:
            - strength characteristics,
            - distribution of materials and discontinuities

- displacement patterns, to include:
    - runout distances
    - continuity of failure mass, and
    - deformation of adjacent ground

- hydrogeological conditions, especially post-earthquake changes

- Performance of mitigation measures.
  This should include observations of previously-stabilized and engineered slopes.

- Verification of analytical procedures
  This should be based on the observed case history data

2. Topographic amplification and slope response

    - Timely installation of mobile instrumentation to capture slope behavior during aftershocks

    - Research is needed to develop portable instrumentation packages

    - Installation of permanent instrumentation to measure slope response in highly active seismic regions

    - Calibration and evaluation of analytical models for site response

3. Behavior of heterogeneous and discontinuous soil/rock masses

    - Determination of representative properties of heterogeneous and discontinuous soil/rock masses for use in stability analyses
    - Evaluation and development of field techniques for characterization and identification zones of weakness and discontinuities
    - Studies of complex failure mechanisms

The primary technical barrier for effective rapid field response for instrumentation of slopes is development and improvement of compatible, portable instrumentation packages.

A major non-technical barrier is the lack of long-term management of maintenance and deployment of mobile instrument packages and the associated expert personnel.

## 4.6 REPORT OF THE PANEL ON RETAINING AND UNDERGROUND STRUCTURES


Chair: Asadour Hadjian, Bechtel Corporation
Recorder: Donald Anderson, CH2M Hill

Panel: Jean-Lou Chameau, Georgia Tech
Kenji Harada, University of Tokyo, Hongo
Michael Katona, HQ AFESC/RD, Tyndall AFB, FL
Hon Yim Ko, University of Colorado, Boulder

Organizing Committee Representative: Richard Woods, University of Michigan

NSF WORKSHOP ON EXPERIMENTAL
NEEDS IN GEOTECHNICAL EARTHQUAKE ENGINEERING
REPORT FROM THE PANEL ON RETAINING AND UNDERGROUND STRUCTURES

Types of Retaining Walls

- Gravity
- Cantilever
- Tied-Back
- Unyielding Basement
- Earth Nailing
- Mechanically Stabilized
  - Reinforced earth
  - Geosynthetic
  - Etc.

Knowledge Gaps

- Saturated Soil
  - Dynamic pore pressure distribution
  - Strength deterioration
  - Liquefaction

- Wall-Soil Interface
  - Reinforcing from inclusions
  - Contact pressures/shear
  - Gapping

- Failure Modes
  - Types of walls
  - Types and direction of waves
  - Types of soil

Experimental Methods

- Explosives*
- Centrifuge*
- Shake Table
- Passive Monitoring* (instrumentation of structures and post-earthquake investigations)
- Shaker Tests

  *Level of Effort: 3 - 10 staff years

Objectives

Develop better, more realistic predictive method

- Understand deformation and failure mechanisms in near-wall soil
- Understand interaction between structure and soil
- Develop appropriate instrumentation to monitor wall-soil response

Key Parameter Variations

- Stiff versus flexible walls
- Saturated versus dry
- Reinforced versus unreinforced soil
- Cohesive versus cohesionless soil
- Unrestrained versus top restraint

Technical Barriers

- General Barriers
  - Shear force instrumentation
  - Local soil effects (hard/soft spots)

- Explosive Tests
  - Wave field simulation
  - Saturation of backfill

- Centrifuge
  - Reflection of waves/3D effects
  - Pore fluid modeling
  - Inflight soil property profiles
  - Miniaturization of transducers

- Shake Table
  - Weight
  - Size/scaling/boundary conditions
  - Sample preparation
  - Amplitude/frequencies

- Passive Monitoring
  - Maintenance of instrumentation
  - Reliability
  - Time/aging effects (annual $G_{max}$ determination)

- Shaker and Plucking/Snapback Tests
  - Partial results

Non-Technical Barriers

- Explosives
  - Location
  - Expensive
  - Safety (1/2 mile)
  - Environmental restrictions
  - Acceptance by profession

- Centrifuge
  - No shakers in large centrifuge
  - Acceptance by profession

- Shake Table (Large Ones!)
  - Expensive

- Passive Monitoring
  - Owner cooperation
  - Long-term maintenance
  - Waiting time for experiment

- Shaker Tests
  -
  -

<u>Targets of Opportunity</u>

- EPRI Hualien SSI Experiment
  - Use model structure
  - Construct wall

- FHWA Walls
  - Shakers and snapback testing
  - Explosives

- Port/Drydock Construction
  - Navy Lab
  - Port of LA

- Canadian National Test Site
  - Construct wall

- U.S. National Geotechnical Experimentation Sites

Table 1. Technical Priority Selection Matrix (1 highest, 0 is not applicable).

| Gaps | Post EQ Studies | Plucking | Passive | Centrifuge | Shake Table | Explosives |
|---|---|---|---|---|---|---|
| Dynamic pore pressure and Strength Deterioration | 0 | 0 | 3 | 3 | 2 | 2 |
| Liquefaction | 1 | 0 | 2 | 3 | 2 | 2 |
| Wall-Soil Interface | 0 | 2 | 2 | 3 | 3 | 3 |
| Failure Mode Wave Field | 0 | 0 | 3 | 1 | 1 | 2 |
| Failure Mode Wall Type | 2 | 0 | 2 | 3 | 1 | 1 |

## EXPERIMENTAL METHODS

### 1. EXPLOSIVES

Here it is envisioned that explosives would be used to simulate earthquake motion in full scale, fully instrumented retaining wall tests. Explosive testing techniques have been demonstrated in the SIMQUAKE test series. The advantages of explosive testing, as opposed to waiting for natural earthquakes are evident in that the precise time and location of the event are known. This permits concentrating the instrumentation in spatially optimized locations as well as reducing the manpower and other resource requirements that are necessary for monitoring natural earthquakes. Further, by knowing the time, location, and source strength of the explosive event, pre-test predictions can be made to better plan the experiment, and pre-test calibration shots can be made to test the instrumentation equipment.

#### Technical Barriers

Research is needed to devise ways to tailor the wave forms from an explosive source to better simulate the ground motion of natural earthquakes. Wave forms from explosive events typically differ from earthquakes by: higher frequency content, shorter periods, fewer cycles, and are p-wave dominant vs. earthquakes which are often s-wave dominant. Several ideas are worth pursuing to help tailor the waveforms from explosive events: spatial layout and temporal firing sequence of explosives, trenches or mitigating materials to diffuse/refract waveforms, and generation of s-waves by driving a horizontal basement slab with explosives.

#### Non-Technical Barriers

A major issue is safety. Thus, remote locations are often required for explosive testing. A second issue is environmental restrictions, e.g., noise, dust, and ground shock effects on the environs.

### 2. CENTRIFUGE TESTING

Scale model testing in the centrifuge allows the body force effects on soil structures to be correctly represented. Several techniques have been developed in recent years for simulating earthquake ground motion in centrifuge testing, of which the electro-hydraulic technique is the most effective. It is assumed here that this technology will be successfully transferred to the recently completed large centrifuges in the U.S. to allow meaningful earthquake simulation experiments to be conducted.

Shake table experiments on small centrifuges have so far been focused on producing horizontal base motion on the test model. Two-and three-dimensional shaking is also conceptually possible, and needs to be implemented ultimately for full simulation of earthquake effects.

#### Technical Barriers

A significant shortcoming of centrifuge testing is the fact that the soil has to be confined in a container of limited dimensions. Stress wave reflections from the container boundaries produce contaminations, which could be reduced by lining the container with absorbent materials. On the other hand, a laminar construction for the container could be used to accommodate horizontal motion of the soil. The choice of the proper boundary conditions for a particular type of boundary value problem is crucial to achieving fidelity of the simulation.

In addition, the conflict in the time scaling for dynamic (pore pressure generation) and diffusion (dissipation) phenomena must be resolved. A suitable substitute pore fluid must be developed to slow the flow without changing the fundamental constitutive properties of the soil. Alternatively, the grain size distribution of the soil could be altered to achieve a lower permeability.

Because centrifuge testing is usually carried out at a linear scale of 100, conventional transducers of normal sizes would appear unduly large in relation to the soil mass. Thus, miniaturization of these transducers, or development of new measurement devices with telemetric signal transmission would be highly desirable to reduce, if not eliminate, transducer embedment effects.

Procedures must be developed for preparing the soil model to various strength and stiffness specifications. It is also necessary to be able to measure the property profile inflight so that the initial conditions can be completely defined.

3. SHAKE TABLES

Only full-scale tests are valid on shake tables (until or unless we learn that body forces are not important to this phenomena - unlikely). This means that large shake tables with very large excitation capabilities will be necessary. Up to now, the shake tables that are large enough for a minimum prototype wall with appropriate size backfill, say, 10 feet high and 20 x 20 in plan, do not have sufficient excitation force to excite the soil/wall system to significant accelerations (less than 1% g). However, if a sufficiently large and powerful shaker were available, shake table tests could be performed. Some appropriate account would have to be taken with respect to reflections off boundaries which are artifacts of the shake table prototype.

Technical Barriers

(1) No facilities available now which can provide required amplitude and frequency input. (2) Saturation of the backfill of a prototype on the shake table causes some problems.

Non-Technical Barriers

Cost for adequate shake table.

4. PASSIVE MONITORING

Facilities, at which earthquake monitoring instrumentation already exists and at which there also exists a retaining wall, offer the opportunity for collection of valuable earthquake response data. In some instances, additional instrumentation will be required to obtain meaningful data and funding should be available to install same. Adequate soil characterization must also exist or be obtained for each specific site. Instrumentation might include pore pressure transducers, tilt meters, ground displacement benches, interface effects transducers, and others.

Technical Barriers

(1) Any instrumentation installed to monitor response to earthquake excitation must be maintained on a regular basis. Frequency of monitoring depending on experience gained as time

goes on. (2) Instruments of proven reliability or instruments which can be removed, modified, and replaced should be selected or developed for this application. (3) Soil characterization must be repeated at appropriate intervals to assure that the properties have not changed, or measure how they have changed since installation.

Non-Technical Barriers

(1) The owner of the target retaining wall must agree to the passive monitoring program, and agree to the long-term maintenance requirements. (2) The waiting time to useful event is unknown. In many situations, money will be spent for no data, but with a sufficient number of installations, eventually valuable data will be gained.

5. SHAKER AND PLUCKING TESTS

Using steady state vibration exciters and plucking tests, existing retaining wall/backfill systems can be evaluated at any time for small strain behavior. Changes in condition of the wall or soil may be determined. These might be before and after earthquake events or period measurements before an earthquake event to monitor system changes.

A more important application of these procedures might be in the evaluation of soil/wall interface phenomena. Instrumentation to measure this effect could be tested, evaluated, and improved using these procedures. Methods of interpreting interface could be developed.

Technical Barriers

(1) All instrumentation to do the interface tests may not yet be available. (2) Only a small part of the total soil/retaining wall problem could be addressed.

Non-Technical Barriers

Permission of owner and liability for unexpected, unintentional vibratory damage.

GAPS AND OBJECTIVES

The design and performance of retaining structures under seismic loads do not correlate well. On one hand there are numerous documented cases of major failures, e.g., quay wall structures; on the other hand, there are indications that some structures may be over-conservatively designed. The design procedures are simplified extensions of methods developed for static conditions, without sound physical basis for a number of assumptions. Analysis techniques on computer codes do exist; however, they suffer from similar limitations and the lack of verification or calibration.

Fundamental knowledge gaps were identified in three areas: (1) water-soil-wall interaction effects, (2) behavior of interfaces, and (3) identification of failure modes.

Defining the dynamic pore water pressures and the resulting soil strength deterioration which takes place in a saturated backfill material are critical to understanding the response of the wall and predicting its response during the loading process. Evaluation of these characteristics should be made up to and subsequent to any condition of dissipation mechanisms that result from the propagating waves and relative wall/soil movements. It relates to other issues in liquefaction such as the existence of a steady state strength condition in some parts of the backfill.

The response of soil-wall and/or soil-inclusion (e.g., reinforcement) interfaces during dynamic loading is also poorly, if at all, understood. It is required for the development of any analytical and numerical technique applicable to retaining structures. The interface behavior should be known not only in terms of stress-strain characteristics, but also in terms of possible bonding-debonding effects and formation of gaps. Although this is important for all wall types, this knowledge gap may be most significant for reinforced soil structures.

The third major knowledge gap relates to the determination of the failure modes of the different wall types. The critical failure modes must be identified as a function of wall type and wave field. One must give consideration to the following characteristics: (1) rigid vs. flexible walls, (2) saturated vs. dry backfill, (3) reinforced vs. unreinforced soil, (4) soil type, and (5) geometric considerations (e.g., tiebacks, 3-D, etc.).

Identification of these knowledge gaps and issues lead to the selection of four fundamental and inter-related objectives that should guide the planning and implementation of experiments:

Goal 1: Develop more accurate prediction capabilities applicable to different wall types, soil, and geometric conditions. This goal is parallel to, and can only be achieved through, successful completion of goals 2 and 3 below.

Goal 2: Understand the deformation and failure mechanism in the near-wall soil. This includes assessment of dynamic water pressures and possible liquefaction conditions.

Goal 3: Understand the interaction mechanisms between the structure and the soil, and inclusion elements and the soil.

Goal 4: Develop/improve the appropriate instrumentation for monitoring the different aspects of the structure-soil-water interaction. This goal is critical to the completion of goals 2 and 3.

## CHAPTER 5
## ORGANIZING COMMITTEE SUMMARY

### 5.1 RELATIONSHIP TO OTHER WORKSHOPS

A substantial number of earthquake engineering workshops has been held since the early 1970's. Table 1 of this report summarized several. T. Leslie Youd, in his state of the art paper in these proceedings, provides an excellent summary of geotechnical workshops and their recommendations. The subjects of the geotechnical workshops were either the broad needs of geotechnical earthquake engineering, as with the 1977 workshop, or some specific geotechnical topic, e.g., liquefaction, soil improvement, soil properties, etc. Each workshop recommended field observations and field experiments as a major input required for progress in the specific subject area. The present workshop has concentrated on the experimental approaches and capabilities available to support the experimental needs of the various geotechnical topic areas. Because of this focus, the current workshop was able to spend more time in assessing strengths and weaknesses of the various experimental methods and developing suggested experimental approaches.

### 5.2 RECOMMENDED EXPERIMENTAL APPROACHES

The workshop panels recommended experiments of the following types:

Post-Earthquake Observations
Instrumented Sites in Seismically Active Regions
Artificially Induced Ground Shaking
Centrifuge Tests
Shake Table Tests
Field Shaker and Snapback Tests

The committee concurs that there is no question that post-earthquake observations and instrumentation of sites and facilities subjected to actual earthquakes are the two most important activities contributing to improved understanding. Youd's paper described a large number of sites in the U.S. and overseas which are instrumented for this purpose. It is strongly recommended that investigators interested in specific geotechnical studies consider the installation of experiments that can piggyback on the large instrumentation and site investigation investments already made at the existing sites.

Although actual facilities subjected to real earthquakes provide an excellent test environment, the committee concurs with several of the panels that uncertainties in place and time of actual earthquakes, as well as site and structure uncertainties and complexities, place limits on their usefulness. Accordingly, other test methods should be pursued. Artificially-induced earthquakes on geotechnical systems at relatively large scale provide the potential for evaluating response in detail. In addition, such tests provide excellent environments for evaluating and validating analytical methods. Explosive methods and alternatives, such as the water filled pipe approach, should be evaluated and developed to provide as close an approximation to earthquakes as possible. Large, multi-application experiments should be planned.

Centrifuge development should continue. Centrifuges provide the opportunity to perform multiple experiments in a scaled gravity field at modest cost. This capability should enable evaluation of the effects of changes in important parameters on some responses. A good deal of additional work is necessary on diffusion scaling, instrumentation, and shakers to achieve substantial progress in this area.

Shake tables, field shakers, and snapback tests are of lower fidelity in investigating earthquake response. Yet, they can play a valuable role as parts of overall programs. The committee encourages individuals with an interest in these techniques to pursue good applications of these methods using existing facilities. Very large shake tables exist in Japan and can be used for some geotechnical studies. Perhaps international cooperation is a possibility.

## 5.3 NON-TECHNICAL BARRIERS

Non-technical barriers consist mainly of funding and organizational issues. The panels were asked to give some consideration to costs in their deliberations. However, for lack of time, little attention was devoted to specific costs. Yet the committee is aware of the fact that serious experiments cost significant amounts of money. In fact, we believe that progress in the area of geotechnical earthquake engineering has been significantly retarded by lack of funding. Inadequate funding causes lack of experimental equipment (e.g., shakers for shake tables and large shake tables) and an inability to pursue aggressive large artificial shaking tests.

It appears to this committee that research funding to support the needs identified by this workshop is too low in proportion to the past and continuing investment in geotechnical infrastructure. Loma Prieta damage, mainly due to geotechnical causes, is estimated at $7 to $10 billion. A 1 percent investment in research of say $100 million might have reduced the damage by as much as 25 percent. Considering a ten year investment period (perhaps roughly on the order of the return period for a significant damage-causing earthquake), one could derive an investment of an added $10 million per year for geotechnical earthquake engineering research. There are many other ways to derive similar estimates. The conclusion is simply that the nation must find ways to invest additional resources into this research area.

This committee is not qualified to recommend ways of raising and funding these requirements. Nevertheless, we recognize that some combination of government and industry must find ways to fund these requirements. Regrettably, the construction industry in the U.S. has little incentive to innovate and support building research; steps should be taken to increase the incentive. The adversarial relationship that often exists between owner, engineer, and constructor, the high level of litigation in the U.S., and constraining government regulations lead to uncreative, traditional solutions only because they have been accepted in the past. As a result, the bulk of building research funding traditionally has been and must continue to be provided by the Federal Government. The earthquake engineering community must find ways to encourage both Government and Industry to provide adequate funding by presenting technical evidence of the need for research and cost-benefit analyses which show the potential returns. This information must be presented to the government agencies responsible for earthquake engineering research, to state and Federal legislators, to industry organizations, and to individual firms.

Organizational barriers relate to the fact that large experimental programs require cooperation between and among government, university, consulting, and industry organizations. The committee does not consider these barriers to be insurmountable. There are ample examples of successful past and current multi-organization programs. Nevertheless, investigators that propose to pursue large experimental programs must recognize the need, time, and costs associated with establishing cooperative agreements. Furthermore, they must cover the need for strong program management.

## REFERENCES


1. Lee, K.L., Marcuson, W.F. III, Stokoe, K.H., and Yokel, F.Y., Editors, Research Needs and Priorities for Geotechnical Earthquake Engineering Applications, National Science Foundation and National Bureau of Standards Report under NSF Grant No. AEN77-09861, University of Texas, Austin, TX, June 1978.

2. Brandon, G.E., Coordinator, and Leeds, D.J., Editor, Reconnaissance Report, Imperial County, California Earthquake, October 15, 1979, Earthquake Engineering Research Institute, Berkeley, CA, February 1980.

3. "The Mexico Earthquake of September 19, 1985," Earthquake Spectra, Earthquake Engineering Research Institute; Vol. 4, No. 3, August 1988; Vol. 4, No. 4, November 1988; and Vol. 5, No. 1, February 1989.

4. "Armenian Earthquake Reconnaissance Report," Special Supplement, Earthquake Spectra, Earthquake Engineering Research Institute, August 1989.

5. "Loma Prieta Earthquake Reconnaissance Report," Supplement to Vol. 6, Earthquake Spectra, Earthquake Engineering Research Institute, May 1990.

6. "Philippines Earthquake Reconnaissance Report," Supplement A to Vol. 7, Earthquake Spectra, Earthquake Engineering Research Institute, October 1991.

7. "Costa Rica Earthquake Reconnaissance Report," Supplement B to Vol. 7, Earthquake Spectra, Earthquake Engineering Research Institute, October 1991.

8. Esteva, Luis, "Consequences, Lessons, and Impact on Research and Practice, The Mexico Earthquake of September 19, 1985," Vol. 4, No. 3, Earthquake Spectra, Earthquake Engineering Research Institute, August 1988.

9. Competing Against Time, Report to Governor George Deukmejian from the Governor's Board of Inquiry on the 1989 Loma Prieta Earthquake, George W. Housner, Chairman, May 1990.

Rio Vizcaya Bridge: Fissures in the South Roadway Approach in 1991 Costa Rica Earthquake (Ref. 7, fig. 6-25, pg. 76, reprinted by permission of Earthquake Engineering Research Institute).

APPENDIX A
LIST OF ATTENDEES

Donald G. Anderson
CH2M HILL
777 - 108th Ave. NE
Bellevue, WA 98004
(206) 453-5000 - phone
(206) 462-5957 - fax

K. Arulanandan
University of California ID#1855
Dept. of Civil Engineering/Bainer Hall
Davis, CA 95616
(916) 752-0895 - phone
(916) 752-8924 - fax

Clifford J. Astill*
National Science Foundation
1800 G. Street, NW, Room 1132
Washington, D.C. 20550
(202) 357-9500 - phone
(202) 357-9803 - fax

Jean Benoit
University of New Hampshire
Dept. of Civil Engineering - Kingsbury Hall
Durham, NH 03824-3591
(603) 862-1428 - phone
(603) 862-2364 - fax

Scott Blouin
Applied Research Associates, Inc.
Box 120A Waterman Rd.
S. Royalton, VT 05068
(802) 763-8348 - phone
(802) 763-8283 - fax

Mehmet Celebi
USGS (MS 977)
345 Middlefield Rd.
Menlo Park, CA 94025
(415) 329-5623 - phone
(415) 329-5163 - fax

Jean-Lou Chameau
Georgia Tech
Civil Engineering Department
790 Atlantic Dr.
Atlanta, GA 30332
(404) 894-2201 - phone
(404) 894-2278 - fax

Wayne A. Charlie, PE
Colorado State University
Department of Civil Engr/ERC
Fort Collins, CO 80523
(303) 491-5048 - phone
(303) 491-8671 - fax

Koon Meng Chua*
University of New Mexico
Department of Civil Engineering
Albuquerque, NM 87131
(505) 277-6063 - phone
(505) 277-0813 - fax

Carl J. Constantino
City College of New York
Dept. of Civil Engineering
New York, NY 10031
(212) 650-8003 - phone
(212) 690-6965 - fax

James F. Costello (NLS-217A)
U.S. Nuclear Regulatory Commission
Washington, DC 20555
(301) 492-3818 - phone
(301) 492-3696 - fax

Pedro de Alba
Dept. of Civil Engineering - Kingsbury Hall
University of New Hampshire
Durham, NH 03824
(603) 862-1417 - phone
(603) 862-2364 - fax

Ahmed-W. Elgamal
Rensselaer Polytechnic Institute
Department of Civil Engineering
110 Eight St.
Troy, NY 12180
(518) 276-2836 - phone
(518) 276-4833 - fax

Arley G. Franklin
USAE WES
3909 Halls Ferry Rd.
Vicksburg, MS 39180-6199
(601) 634-2658 - phone
(601) 634-4134 - fax

George Gazetas
212 Ketter Hall - University at Buffalo
State University of New York
Buffalo, NY 14260
(716) 636-3634 - phone
(716) 636-3733 - fax

Paul F. Hadala*
USAE WES
3909 Halls Ferry Rd.
Vicksburg, MS 39180-6199
(601) 634-3475 - phone
(601) 634-3139 - fax

Asadour H. Hadjian
Bechtel Corporation
12440 E. Imperial Hwy.
Norwalk, CA 90650
(213) 807-2454 - phone
(213) 807-3456 - fax

Kenji Harada
Department of Civil Engineering
University of Tokyo, Hongo
Bunkyo-ku, Tokyo JAPAN
03-3812-2111 - phone
03-3818-5692 - fax

Edwin L. Harp
U.S. Geological Survey
Branch of Geologic Risk Assessment
1711 Illinois, Rm. 326
Golden, CO 80401
(303) 236-1628 - phone
(303) 236-0618 - fax

Cornelius J. Higgins
Applied Research Associates, Inc.
4300 San Mateo Blvd., NE, # A220
Albuquerque, NM 87110
(505) 883-3636 - phone
(505) 883-3673 - fax

Thomas L. Holzer
U.S. Geological Survey
345 Middlefield Rd. (MS 977)
Menlo Park, CA 94025
(415) 329-5613 - phone
(415) 329-5163 - fax

Roman D. Hryciw
University of Michigan
Department of Civil Engineering
2366 GG Brown Bldg., 2350 Hayward
Ann Arbor, MI 48109-2125
(313) 763-5491 - phone
(313) 764-4292 - fax

M.E. Hynes, CEWES-GG-H
USAE WES
3909 Halls Ferry Rd.
Vicksburg, MS 39180-6199
(601) 634-2280 - phone
(601) 634-3139 - fax

Kenji Ishihara
University of Tokyo, Hongo
Department of Civil Engineering
Bunkyo-ku, Tokyo JAPAN
011-81-03-3812-2111 - phone
011-81-03-3818-5692 - fax

William B. Joyner
U.S. Geological Survey
345 Middlefield Rd. (MS 977)
Menlo Park, CA 94025
(415) 329-5640 - phone
(415) 329-5163 - fax

Michael Katona
Headquarters AFESC/RD - Bldg. 1120
Tyndall AFB, FL 32403-6001
(904) 283-6272 - phone
(904) 283-6499 - fax

David K. Keefer
U.S. Geological Survey
345 Middlefield Rd. (MS 998)
Menlo Park, CA 94025
(415) 329-4893 - phone
(415) 329-5163 - fax

Hon Yim Ko
University of Colorado Boulder
Civil Engineering Dept./ECOT-421
Boulder, CO 80309-0428
(303) 492-6716 - phone
(303) 492-7317 - fax

Takeji Kokusho
Central Research Institute of
Electric Power Industry
Abiko Research Lab. 1646
Abiko, Abiko City JAPAN
011-81-0471-82-1181 - phone
011-81-0471-83-2962 - fax

Bruce L. Kutter
University of California
Department of Civil Engineering
Davis, CA 95616
(916) 752-8099 - phone
(916) 752-8924 - fax

Geoffrey R. Martin
University of Southern California
Department of Civil Engineering
3620 S. Vermont Ave. KAP-210
Los Angeles, CA 90089-2531
(213) 740-9124 - phone
(213) 744-1426 - fax

Tang-Tat Ng
University of New Mexico
Dept. of Civil Engineering/Wagner Hall 106
Albuquerque, NM 87131
(505) 277-4844 - phone
(505) 277-0813 - fax

Gary Norris
Civil Engineering Department/268
University of Nevada
Reno, NV 89557
(702) 784-6835 - phone
(702) 784-4466 - fax

Ronald Pak
University of Colorado
Department of Civil Engineering
Boulder, CO 80309-0428
(303) 492-8613 - phone
(303) 492-7317 - fax

Ralph B. Peck
1101 Warm Sands, SE
Albuquerque, NM 87123
(505) 293-2484 - phone

Maurice S. Power
Geomatrix Consultants
One Market Plaza
Spear St. Tower, Suite 717
San Francisco, CA 94105
(415) 957-9557 - phone
(415) 957-0965 - fax

Jean H. Prevost
Princeton University
Department of CEOR
Princeton, NJ 08544
(609) 258-5424 - phone
(609) 258-1270 - fax

Peter K. Robertson
University of Alberta
Civil Engineering Department
303 Civil-Electrical Eng. Bldg.
Edmonton, Alberta, CANADA T6G 2G7
(403) 492-5106 - phone
(403) 492-8198 - fax

John F. Schneider
Electric Power Research Institute
P.O. Box 10412
Palo Alto, CA 94303
(415) 855-7921 - phone
(415) 855-1026 - fax

Ronald F. Scott*
California Institute of Technology
Civil Engineering Department
391 S. Holliston Ave. (MC 104-44)
Pasadena, CA 91125
(818) 356-4233 - phone
(818) 568-2719 - fax

Anthony Shakal
California Division of Mines & Geology
Strong Motion Instrumentation Program
630 Bercut Drive
Sacramento, CA 95814
(916) 322-7481 - phone
(916) 323-7778 - fax

C.K. Shen
The Hong Kong University
of Science & Technology
12/F World Shipping Centre
7 Canton Road
Tsimshatsui, Kowloon, HONG KONG
011-852-358-7152 - phone
011-852-358-1534 - fax

Jeffrey Simons
SRI International
333 Ravenswood Ave./AF235
Menlo Park, CA 94025
(415) 859-4495 - phone
(415) 859-2260 - fax

Nicholas Sitar
University of California at Berkeley
Department of Civil Engineering
Berkeley, CA 94720
(415) 643-8623 - phone
(415) 643-5264 - fax

Kenneth H. Stokoe
The University of Texas
Geotechnical Eng. Center/ECJ 9.227
26th Street @ San Jacinto
Austin, TX 78712
(512) 471-4929 - phone
(512) 471-6548 - fax

H.T. Tang*
EPRI
3412 Hillview Ave.
Palo Alto, CA 94303
(415) 855-2473 - phone
(415) 855-1026 - fax

Larry Von Thun
U.S. Bureau of Reclamation
Denver Office/Code D-3620
Bldg. 67 Denver Federal Center
Denver, CO 80225-0007
(303) 236-3859 - phone
(303) 236-6763/6764 - fax

Joseph P. Welsh
Hayward Baker, Inc.
1875 Mayfield Road
Odenton, MD 21113
(301) 551-8200 - phone
(301) 551-1900 - fax

Stuart Werner
Dames & Moore
2101 Webster, Ste. 300
Oakland, CA 94612
(510) 839-3600 - phone
(510) 839-4461 - fax

Richard D. Woods*
University of Michigan
Dept. of Civil & Environmental Eng.
Rm. 2360 G.G. Brown Bldg.
2350 Hayward
Ann Arbor, MI 48109-2125
(313) 764-4303 - phone
(313) 764-4292 - fax

Felix Y. Yokel
NIST
Quince Orchard Rd. & Route 270
Bldg. 226, Room B158
Gaithersburg, MD 20899
(301) 975-6065 - phone
(301) 975-4032 - fax

T. Leslie Youd*
Department of Civil Engineering
368 Clyde Building
Brigham Young University
Provo, UT 84602
(801) 378-6327 - phone
(801) 378-2478 - fax

*Organizing Committee

APPENDIX B
WORKSHOP AGENDA

November 4 (Monday), 1991

| | |
|---|---|
| 7:30 - 8:00 am | Registration and Continental Breakfast |
| 8:00 - 8:15 | Plenary Session<br>(Objectives, format, expectations outlined) |
| 8:15 - 8:30 | Observational Methods<br>Ralph B. Peck |
| 8:30 - 9:15 | Earth Dams and Natural Slopes<br>Larry Von Thun |
| 9:15 - 10:00 | Soil Structure Interaction<br>Stuart Werner |
| 10:00 - 10:30 | Break |
| 10:30 - 11:15 | Ground Motion<br>Geoffrey Martin |
| 11:15 - 12:00 pm | Tests on Actual Full Scale Facilities<br>T. Leslie Youd |
| 12:00 - 1:00 | Lunch |
| 1:00 - 1:45 | Explosive Simulation<br>Cornelius J. Higgins |
| 1:45 - 2:30 | Dynamic Tests on Centrifuge and Shake Tables<br>Hon-Yim Ko |
| 2:30 - 3:00 | Instructions to Panels |
| 3:00 - 3:30 | Break |
| 3:30 - 5:30 | Panels Meet |
| 6:30 - 8:00 | Dinner Banquet |
| 8:00 - 10:00 pm | Panels Reconvene<br>(Prepare a handwritten outline of conclusions for distribution) |

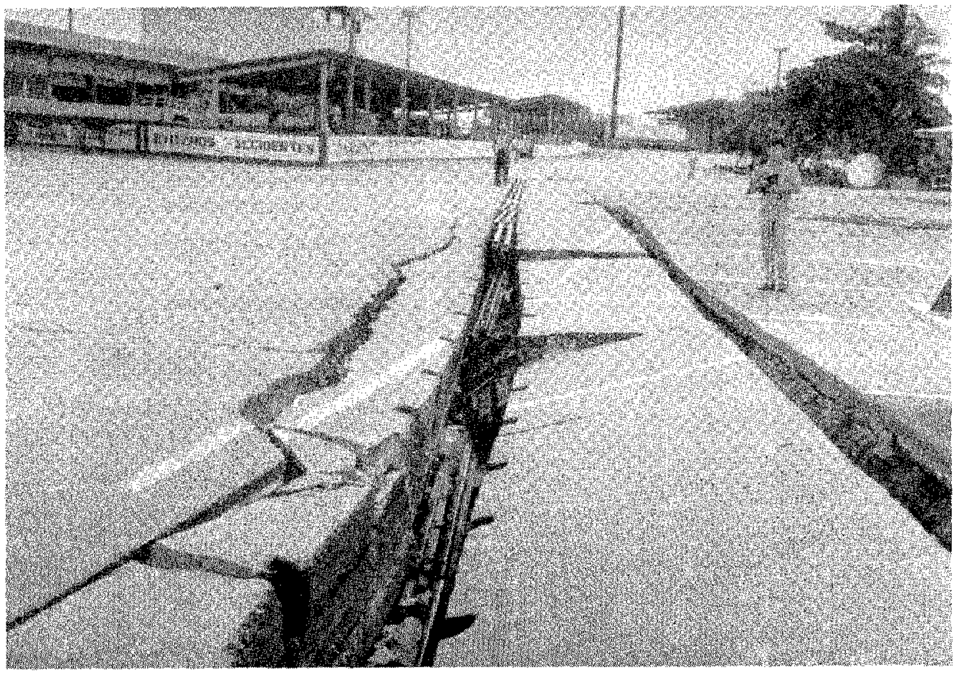
November 5 (Tuesday), 1991

| | |
|---|---|
| 7:30 - 8:00 am | Continental Breakfast |
| 8:00 - 9:00 | Plenary Session and Panel Presentations<br><br>Presentation: 5-7 minutes<br>Questions / Clarifications: 3 minutes |
| 9:00 - 9:30 | General Discussion & Consensus<br>(Instructions to Panels) |
| 9:30 - 10:00 | Break |
| 10:00 - 12:00pm | Panels Meet & Write |
| 12:00 Noon - 1:00 | Lunch |
| 1:00 - 3:00 | Write |
| 3:00 | Turn in Materials to Organizing Committee |
| 3:30 - 5:30 | Plenary Session<br>Final Presentations & Responses |

APPENDIX C
PANEL ASSIGNMENTS

| | | |
|---|---|---|
| Earthdams | Arulanandan, K. | University of California, Davis |
| | de Alba, Pedro (R) | University of New Hampshire |
| | Hynes, Mary Ellen (C) | USAE WES |
| | Prevost, Jean H. | Princeton University |
| | Von Thun, Larry | U.S. Bureau of Reclamation |
| Foundations | Benoit, Jean | University of New Hampshire |
| | Constantino, Carl J. (R) | City College of New York |
| | Gazetas, George | State University of New York |
| | Norris, Gary (C) | University of Nevada |
| | Simons, Jeffrey | SRI International |
| | Stokoe, Kenneth H. | University of Texas |
| | Werner, Stuart | Dames and Moore |
| Ground Instability and Site Improvement | Blouin, Scott | Applied Research Associates, Inc. |
| | Charlie, Wayne A. | Colorado State University |
| | Franklin, Arley G. (C) | USAE WES |
| | Holzer, Thomas L. | U.S. Geological Survey |
| | Ishihara, Kenji | University of Tokyo |
| | Kutter, Bruce L. | University of California |
| | Power, Maurice S. (R) | Geomatrix Consultants |
| | Shen, Chih-Kang | The Hong Kong University |
| | Welsh, Joseph P. | Hayward Baker, Inc. |
| Ground Motion Response | Celebi, Mehmet | U.S. Geological Survey |
| | Costello, James F. | U.S. NRC |
| | Joyner, William B. (C) | U.S. Geological Survey |
| | Kokusho, Takeji | Central Research Inst. Japan |
| | Martin, Geoffrey R. | University of Southern California |
| | Ng, Tang-Tat (Percy) | University of New Mexico |
| | Schneider, John F. (R) | EPRI |
| | Shakal, Anthony | CA Division of Mines & Geology |
| | Yokel, Felix Y. | NIST |
| Natural Slopes | Harp, Edwin L. | U.S. Geological Survey |
| | Hryciw, Roman D. | University of Michigan |
| | Keefer, David K. | U.S. Geological Survey |
| | Robertson, Peter (R) | University of Alberta |
| | Sitar, Nicholas (C) | Univ. of California at Berkeley |
| Retaining and Underground Structures | Anderson, Donald G. (R) | CH2M Hill |
| | Chameau, Jean-Lou | Georgia Tech |
| | Hadjian, Asadour H. (C) | Bechtel Corporation |
| | Harada, Kenji | University of Tokyo |
| | Katona, Michael | AFESC/RD |
| | Ko, Hon Yim | University of Colorado, Boulder |

(C) Chair (R) Recorder



Spreading of Fill Beneath the Port of Limon Opened Gaps of a Meter or More Adjacent to the Container Handling Docks in 1991 Costa Rica Earthquake (Ref. 7, fig. 5-10, pg. 57, reprinted by permission of Earthquake Engineering Research Institute).