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Miscellaneous Paper GL-95-2
August 1995

US Army Corps
of Engineers
Waterways Experiment
Station

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U.S. Army Corps of Engineers
Waterways Experiment Station
3909 Halls Ferry Road
Vicksburg, MS 39180-6199

Final report

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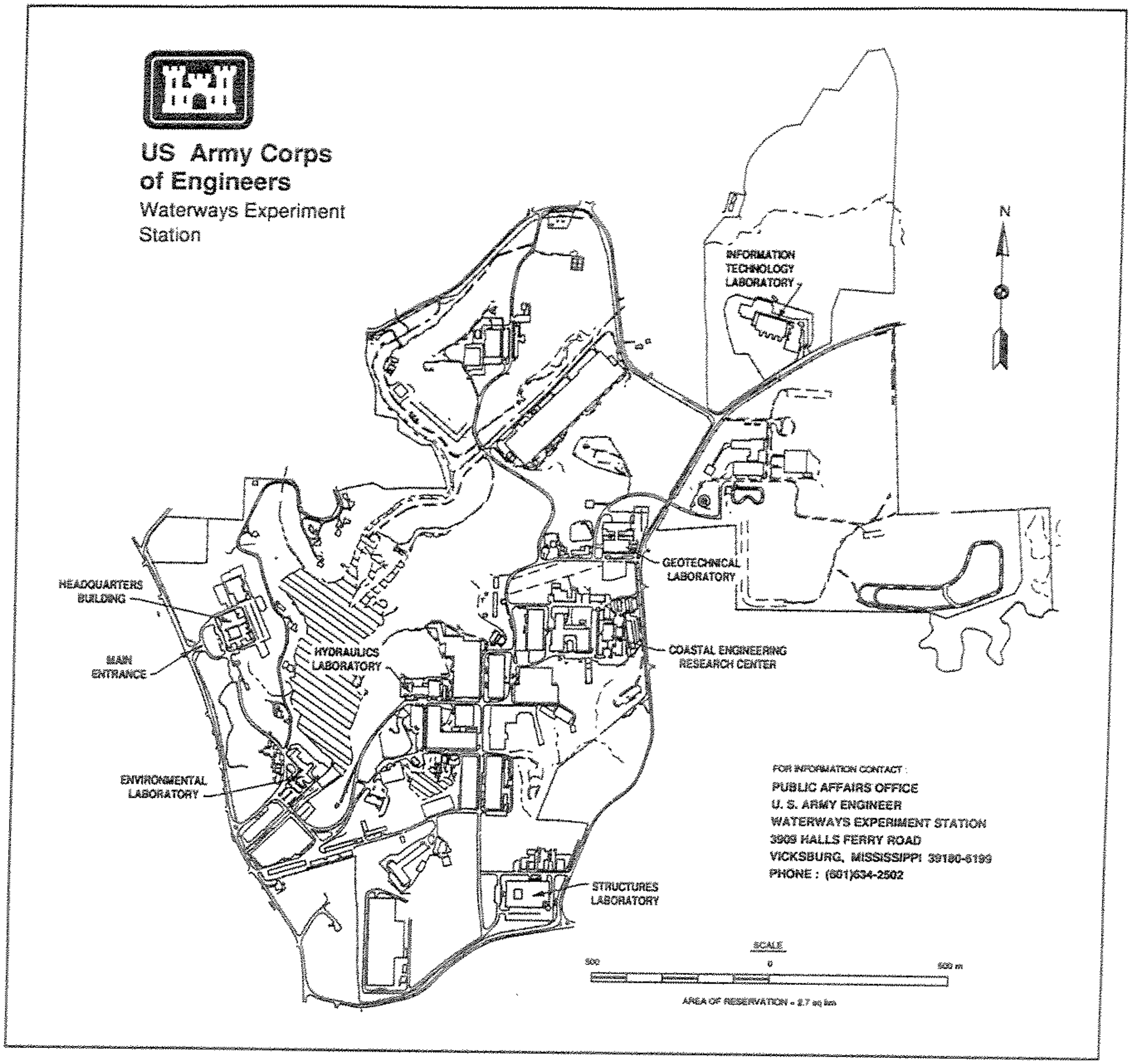

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Contents

Preface

While pile foundations have been used for centuries, design procedures remained empirical until relatively recently. The development of offshore resources and large-scale transportation facilities in difficult soils initiated a new era in deep foundations requiring design methodology based on solid theoretical knowledge.

The Geotechnical Laboratory (GL), U.S. Army Engineer Waterways Experiment Station (WES), has for years maintained an active role in deep foundation research in cooperation with various universities, industry, Minerals Management Service, Department of Interior, and other interested agencies.

To continue this worthwhile effort, as well as consolidate the knowledge accumulated by universities, Government agencies, and private industry, this workshop was conducted. A major aim of the workshop was defining of directions for further research in deep foundations. The workshop took place during 13 to 15 July 1993 at WES, under the leadership of a steering committee consisting of: Drs. Charles Smith, M.W. O'Neill, and L. Reese.

The workshop consisted of presented papers, contributed papers, and discussions on development of research topics. The presented papers, visual aids, and contributed papers are published with the permission of the authors.

This report was prepared by Messrs. John M. Andersen and W. Milton Myers, Soil Mechanics Branch, Soil and Rock Mechanics Division, GL, WES, under the general supervision of Dr. Don Banks, Chief, Soil and Rock Mechanics Division; and Dr. Marcuson, Director, GL, WES.

The sponsors of the workshop were jointly U.S. Department of the Interior, Minerals Management Service, Technology Assessment and Research Branch, and WES.

At the time of publication of this report, Director of WES was Dr. Robert W. Whalin. Commander was COL Bruce K. Howard, EN.

Conversion Factors, SI Units to Non-SI Units of Measurement

SI units of measurement used in the contributed papers in this report can be converted to non-SI units as follows:

Multiply	By	To Obtain
kilonewton meters	0.1449	pound-feet
kilopascals	0.145	pounds per square inch
megapascals	145	pounds per square inch
megapascals	10	kilograms per square centimeter
meters	3.2808	1 foot
millimeters	0.3937	1 inch

CORPS OF ENGINEERS
MATERIALS MANAGEMENT SERVICE WORKSHOP

EFFECTS OF PILES ON SOIL PROPERTIES

13 - 15 July 1993

Hosted by

Geotechnical Laboratory
U.S. Army Engineer Waterways Experiment Station
Vicksburg, Mississippi

Workshop Location:
Casagrande Building (Building 3396)
1st Floor Conference Room

Time	Subject	Speaker
Tuesday, 13 July 1993		
0815 - 0900	Registration and Get Acquainted	Lobby, 1st Floor Conference Room, Casagrande Bldg.
0900 - 0905	Welcome to WES	Dr. Robert W. Whalin, Director of WES
0905 - 0910	Welcome and Introduction to Workshop	Dr. W. F. Marcuson III, Director, GL
0910 - 0920	Workshop Theme	Mr. Charles Smith, Minerals Management Service
0920 - 0930	Orientation, Activities, and Introductions	Mr. William M. Myers, WES
SESSION ONE: The Nature of the Pile-Soil Property Problem, Mr. Charles Smith, Moderator		
0930 - 1015	Piles in Granular Soils	Dr. Michael W. O'Neill, University of Houston
1015 - 1045	Break*	
1045 - 1130	Piles in Granular Soils - Case Study	Dr. Reed Mosher, WES
1130 - 1215	Piles in Cohesive Soils	Dr. Lymon Reese, University of Texas at Austin

Time	Subject	Speaker
1215 - 1330	Lunch	Businessman's Lunch - Conference Room Via Goldie's Trail Barbeque
1330 - 1415	Piles Subjected to Downdrag	Dr. Jean-Louis Briaud, Texas A&M University
1415 - 1445	Break*	
1445 - 1530	Laboratory and Model Tests on Piles and Soil Properties	Dr. Andrew J. Whittle, Massachusetts Institute of Technology
1530 - 1615	Kwajalein Dry Dock Case History	Dr. Mike Holloway, Insitu Tech, Inc.
1615 - 1700	Questions and Discussion	All
1800 - 1900	No Host Social Hour	Park Inn International

Wednesday, 14 July 1993

SESSION TWO: Research Status and Case History Experiences Concerning Piles and Soil Properties, Dr. Lymon Reese, Moderator

0800 - 0830	Coffee and Conversation	
0830 - 0850	Current Research on Instrumented Piles Including Programs at Norwegian Geotechnical Institute	Dr. Andrew Whittle
0850 - 0920	Pile Load Test Data Base	Dr. Jean-Louis Briaud
0920 - 0930	Break*	
0930 - 1000	Equipment Experiences and Research Needs	Dr. Don Warrington, Vulcan Iron Works; Deep Foundations Institute
1000 - 1030	WES Centrifuge - Uses and Capabilities	Mr. Richard Ledbetter, WES
1030 - 1045	Break*	
1045 - 1115	Highway Experience and Research Needs	Mr. Carl Ealy, Federal Highway Administration
1115 - 1200	WES Large Scale Stress Chamber - Uses and Capabilities	Dr. Richard Peterson, WES
1200 - 1330	Lunch	Monsour's Restaurant* Via WES bus

Time	Subject	Speaker
1330 - 1400	Current/Projected Research at University of Arizona	Dr. William M. Isenhower, University of Arizona
1400 - 1430	Current/Projected Research at University of Florida	Dr. Frank Townsend, University of Florida
1430 - 1445	Break*	
1445 - 1515	Current/Projected Research at University of Texas	Dr. Roy E. Olson, University of Texas at Austin

SESSION THREE: Breakout Working Sessions and Research Needs Development, Mr. William M. Myers, Moderator

1515 - 1700 Working Sessions and Research Needs Development

Group One - Research Needs for Piles in Specific or Special Types Soils
Dr. Lymon Reese, Chairman
Geotechnical Laboratory
4th Floor Conference Room, Casagrande Building

Group Two - Laboratory and Small-Scale Test Research Needs
Dr. A. J. Whittle, Chairman
Rock Mechanics Work Room
1st Floor, Casagrande Building

Group Three - Instrumentation and Full-Scale Test Research Needs (Including equipment concerns)
Dr. Jean-Louis Briaud, Chairman
Geotechnical Laboratory
Main Conference Room, Casagrande Building

Group Four - Research Needs for Specialized Types of Piles or Other Deep Foundation Elements
Dr. Mike Holloway, Chairman
Pavement Systems Division Conference Room
1st Floor, Casagrande Building

1800 - 1900	Social Hour	WES Cafeteria #2
1900 - 2030	Deep Foundations Catfish Bash*	WES Cafeteria #2

Thursday, 15 July 1993

SESSION FOUR: Breakout Session Reports and Research Topic Recommendations, Dr. Paul F. Hadala, Asst. Director, GL, Moderator

0800 - 0830	Coffee and Conversation	
0830 - 0900	Group One - Research Needs for Piles in Specific or Special Types of Soils	Dr. Reese

Time	Subject	Speaker
0900 - 0930	Group Two - Laboratory and Small-Scale Test Research Needs	Dr. Whittle
0930 - 1000	Break*	
1000 - 1030	Group Three - Instrumentation and Full-Scale Research Needs	Dr. Briaud
1030 - 1100	Group Four - Research Needs for Specialized Types of Piles or Other Deep Foundation Elements	Dr. Holloway
1100 - 1200	Research Plan Formulation - Open Discussion, Suggestions; Future Planning	Dr. Hadala
1200	Adjourn	

*Please note: No registration fee will be collected for the workshop; however, a refreshment and meal fee of $25 per attendee will be collected to cover the cost of break refreshments, lunch on Wednesday (beef-kabob, rice, vegetable, salad, dessert, etc.), and the DEEP FOUNDATIONS CATFISH BASH on Wednesday night (genuine Mississippi Delta Farm-raised Catfish with all the trimmings, plus social hour).

1 Introduction

The Workshop on the Effects of Piles on Soil Properties was held 13 - 15 July 1993 at the U.S. Army Engineer Waterways Experiment Station (WES). The Workshop was sponsored by the U.S. Department of the Interior, Minerals Management Service (MMS), and hosted by the WES Geotechnical Laboratory (GL). Cooperation for the conduct of the Workshop stemmed from the continuing interest of both MMS and WES in design and construction of deep foundations including past joint research on certain aspects of pile design. The Workshop was held for the purpose of ascertaining whether a need existed for further research on the effects of piles on soil properties. A determination was made as to potential research topics that might best be explored to determine how soil properties are influenced by the emplacement of piles or other deep foundation elements and how such altered soil properties can best be taken into account during design of deep foundations. A selection of potential research needs or topics concerning the effects of piles (or other deep foundation elements) on soil properties was the goal of this workshop. Such needs might be further defined and possibly later funded by MMS, WES, or other interested agencies.

Conduct of Workshop

The Workshop was conducted in four sessions. Session One, moderated by Dr. Charles Smith, MMS, consisted of a series of presentations that described and illustrated aspects of the pile-soil property problem. Session Two, moderated by Dr. Lymon Reese, University of Texas at Austin, consisted of presentations that detailed present research status of the pile-soil property problem and introduced several case histories involving interaction of piles and soil properties. During Session Three, moderated by Mr. W. Milton Myers, WES, the Workshop participants divided into four working groups to develop research needs in a particular area of the pile-soil property problem. Workshop attendees participated in one or more working groups depending upon their interest in the theme being addressed by the group. Several hours were spent by each working group in developing potential research needs and refined research topics in each area of interest. Subjects addressed by the four working groups were as follows:

Group One	Research Needs for Piles in Specific or Special Types of Soils Dr. Lymon Reese, Chairman
Group Two	Laboratory and Small-Scale Test Research Needs for Piles Dr. Andrew J. Whittle, Chairman
Group Three	Instrumentation and Full-Scale Research Needs for Piles Including Equipment Concerns Dr. Jean-Louis Briaud, Chairman
Group Four	Specialized Types of Piles and Other Deep Foundation Elements Dr. Michael Holloway, Chairman

Session Four, moderated by Dr. Paul F. Hadala, WES, consisted of reconvening the full workshop to hear reports from the four working groups. During Session Four, discussion of the recommendations of each working group took place, tentative recommendations of the Workshop were formulated, and suggestions were accepted for the conduce and thrust of future Workshops. The agenda followed during the Workshop is included on page vii.

Workshop Attendees

Announcements of the Pile Workshop were provided to agencies, firms, academic departments, and consultants chosen jointly by MMS and WES that had interest in the Workshop theme. The Workshop was open to all interested parties with only those making presentations specifically invited. Attendees at the Workshop represented Government research and design-construct agencies, academia, consulting firms, and industry. A list of workshop attendees is given in Appendix A.

Workshop Proceedings

This report presents the results of the Workshop. Available written copies of presentations made at the Workshop (principally during Sessions One and Two) are included in this paper. A summary of the recommendations of each of the four working groups is provided. Conclusions and recommendations of the Workshop, together with suggestions for future workshops, are given. Appendix B contains three papers contributed to the Workshop by Dr. Andrew J. Whittle, Department of Civil and Environmental Engineering, Massachusetts Institute of Technology, and Dr. W.F. Van Impe, Director, Soil Mechanics Department, Ghent University, Belgium. Other materials distributed during the Workshop are included in Appendix C.

2 Presented Papers

Research Needs in Deep Foundations

Michael W. O'Neill[1]
Chair, Subcommittee on Research Needs for Pile and Pier Foundations, TRB Committee A2K03: Foundations for Bridges and Other Structures

Introduction

Deep foundations constitute the foundation system of choice for most highway structures. While they may be used needlessly in some instances only because of tradition, deep foundations, usually in the form of piles or drilled shafts, are essential from an economic perspective in cases where soft, compressible or expansive soils are found near the surface, for scour-susceptible river crossings, where large lateral loads are applied and where they are inherently less expensive than alternative foundations (e.g., overwater construction and limited workspaces). Despite recent advances in understanding the fundamental mechanics of pile-soil interaction, much of which has been advanced through the research program of the Federal Highway Administration, and the application of this understanding to design, the design process for deep foundations for highway structures remains largely empirical and fragmented—and probably unnecessarily conservative. Further advances in deep foundation design and construction practices are vitally needed in the coming period of infrastructure rehabilitation if public funds are to be used wisely.

Advances in deep foundation design must be accompanied by appropriate, scientific research, paralleled by a vigorous program of technology transfer. "Research" is a term that is widely used but that is understood differently by various components of the research and operations community. Two types of research are generally understood: basic and applied, which the author would prefer to call (a) visionary research and (b) research to address immediate needs. Visionary research is not cost-effective in the short term, and many projects conducted as a visionary research are not cost-effective in the long

[1] Professor and Chairman, Department of Civil and Environmental Engineering, University of Houston, Houston, TX 77204-4791.

term because promising ideas do not always bear fruit. Visionary research, nevertheless, is absolutely necessary to permit the continuing progress of immediate-needs research. Unfortunately, such research is clearly lacking in the subject area of deep foundations because the problems of deep foundations are often considered to be "mature" or "solved" because solutions, however imperfect, do exist and therefore do not receive the attention of companies and agencies that fund cutting-edge research in the engineering sciences. For example, in a recent workshop held under the auspices of the National Science Foundation (*1*) the principal needs for research were judged by a team of deep foundation researchers from the U.S. and Canada to be related to the "simple" computation of axial capacity of single piles. While many empirical and semi-empirical methods have been proposed, as well as advanced methods based upon complex constitutive models for soils and detailed models for computation of stress changes in the ground caused by installation of piles and drilled shafts, all methods have important limitations in relation to assessment of static capacity. Visionary research is needed to address this problem in a general manner that can eventually become useful to practitioners.

The mission of the Federal Highway Administration generally precludes the performance of "high-risk" research, so the research needs described in this report are not generally of the visionary type, although visionary components are necessarily embedded in some of them, and some of the statements build on the meager base of visionary research in deep foundations that has been conducted in the past. Instead, the focus is mainly on immediate-needs research. The research needs that are reported were articulated and ranked largely by practicing highway foundation engineers and applied research engineers who have encountered specific problems or classes of problems and who desire sound and, to the extent possible, general solutions to these problems. Although such research is essential to the national interest, it is also vital that the foundation engineering profession persist with visionary research so that the groundwork will exist for the performance and implementation of immediate-needs research in the future.

From the perspective of the researcher, immediate-needs research problems can be addressed through one of two methodologies.

a. Behavior-oriented methodologies.

b. Process-oriented methodologies.

Behavior-oriented research is research in which a result or an output can be defined uniquely in terms of a single mathematical or statistical model. Usually, this "output" can be expressed in terms of formulae, graphs or tables, often with considerable effort, which will lead the user of the research directly to the answer he or she seeks. This is the classic research that universities, companies and public agencies have conducted in foundation engineering for decades. For example, determination of influence factors among piles in a group or the correlation of static cone penetrometer values to pile capacity has followed a behavior-oriented methodology.

Process-oriented research, on the other hand, is fuzzy, in the sense that the results of such research must be couched in terms of a set of interrelated decisions (a process), some or all of which are not deterministic, that lead to a "best," but not necessarily unique, solution. The final output of such research might be represented in the form of a suggested method of practice, a decision tree or, on a more sophisticated level, expert system/decision support system software. An example of such research would be the development of acceptance criteria for drilled shafts that are suspected of having structural defects. There is usually no one definitive method of proving that the foundation is suitable to function as it was designed, but evidence can be weighted by combining technology and experience, through a scientifically-developed decision-making process, to arrive at a decision.

Both categories of intermediate-needs research methodologies are vital to the advancement of the state of the art and the state of the practice in deep foundations, and both can be directed at problems involving the research problem statements that are the subject of this report, which generally fall into one of several categories, outlined below:

a. Mechanics of deep foundation behavior.

b. Management of knowledge bases for piles and drilled shafts.

c. New design methodologies for deep foundations.

d. Site characterization for deep foundations.

e. Innovative technologies for constructing deep foundations.

f. Quality assurance for the construction of deep foundations.

The former class of problems tends to be more amenable to behavior-oriented research, while the latter classes involve to some degree the application of process-oriented researched methodologies.

Major Research Accomplishments of the Recent Past

Although deep foundations research has never been well funded, either in the public sector or in the private sector, considerable research adaptable to highway practice has been performed over the past three decades. A partial listing of successful research whose results have found or are finding their way into practice is given below:

a. Unit load transfer functions and simple numerical models for predicting rationally the axial and lateral performance of trial designs for piles and drilled shafts.

b. Wave equation methods, dynamic instrumentation and related computational models for assessing performance of pile driving systems and, in many cases, verifying pile capacities.

c. Methods for testing drilled shafts and piles for structural integrity.

d. Load and resistance factors for deep foundations.

e. Rational methods for assessing pile capacity from soil and rock properties, especially correlations with *in situ* soil testing devices.

f. Drilled shaft construction technology.

g. Degradation of pile capacity due to cyclic axial or lateral loading.

h. Small-strain dynamic behavior of piles and pile groups.

i. Group action in axially loaded piles.

j. Behavior of piles in integral-abutment bridges.

It is significant to point out that research in most of these areas has been performed or sponsored by the Federal Highway Administration, National Cooperative Highway Research Program, and by various states under the Highway Planning and Research Program. It is also important to point out that none of this research, the results of which form the fabric of rational foundation engineering practice in the early 1990's was the result of a single project funded for a 24-month period. Rather, the research was evolutionary, progressing from theory to practice over a period of years, during which many funding agencies sponsored various facets of the work. In most cases the results of the research were not embraced by practitioners until after a long period of exposure, sometimes expedited by formal demonstration projects.

Recent Research Initiatives

A number of significant research initiatives are currently under way or recently completed with the highway engineering community that are aimed at addressing immediate needs in the area of deep foundations. Not all of those who proposed the problem statements documented here were aware of these initiatives, so some problems were proposed that are now at least being partially addressed. Among the more significant research initiatives are studies of:

a. Drag mitigation in piles and pile groups.

b. Criteria for predicting load transfer for drilled shafts in geomaterials intermediate between soil and rock.

c. Decision trees for integrity evaluation of drilled shafts.

d. Computer modeling of installation of piles by vibration.

e. Performance monitoring systems for SPT tests and sounding rods for improved quantification of soil properties for assessing pile driveability and capacity.

f. Compilation of a comprehensive data base for deep foundation tests.

g. Scientifically-based specifications for drilling slurries for drilled shafts.

h. Expedient pile and drilled shaft testing methods.

(1) Flat jack testing.

(2) Rapid mass reaction testing.

(3) Bootstrap testing (separation of toe from shaft with jacking from head).

(4) Plug testing.

i. Assessment of unknown foundations using non-destructive testing techniques.

j. Methodologies for using existing analytical methods for evaluating seismic response of existing bridge foundations (in California).

These studies promise to enhance the practice of deep foundation engineering in the coming years. However, both complementary and totally new projects are also necessary in the transportation field.

Research Needs

Research needs identified by others

CERF. In a series of workshops conducted nationwide by the Civil Engineering Research Foundation in the fall of 1990, 22 topics were identified by some 75 geotechnical practitioners as having a high priority for research in the geotechnical engineering area (*2*). Three of these topics directly apply to deep foundations:

a. Integrity testing of piles and piers.

b. Drilled pier design.

c. Dynamic response of single piles under cyclic loads and pile capacity.

Eight other topics relate to deep foundations, including:

a. Documentation of case histories.

b. Improved *in-situ* testing methods.

c. State-of-the-art and state-of-the-practice for earthquake engineering.

d. Development of database of papers and reports.

e. Innovative construction materials and methods.

f. Classification systems for design reliability.

g. Improved contractual framework.

h. Screening criteria for seismic retrofit of bridge foundations.

Clearly, many of the projects in progress or recently completed within the highway engineering community, described in the preceding section, impact these topical areas; however, others remain unaddressed.

NSF. In the fall of 1992, the National Science Foundation sponsored a workshop on recent accomplishments and future trends in geomechanics in the 21st century (*1*). One of the ten target areas covered in that workshop was deep foundations. In that workshop a group of researchers from Canada and the United States identified four principal areas for fruitful research, the first of which has already been mentioned:

a. Prediction of pile capacity, perhaps incorporating.

(1) High-strain constitutive models.

(2) Interface modeling.

(3) Pore water pressure generation/dissipation models.

(4) Other time-dependent and load-dependent effects.

(5) Concurrent long-term testing.

- Clay, sand, silt, rock, frozen soils.
- Laboratory and field (National Geotechnical Experimentation Sites).

b. New methods of pile installation and new pile materials and shapes, including.

(1) Drilled-and-grouted piles.

(2) Auger-jet piles.

(3) Deep soil mixing columns.

(4) Inside-tapered pipe piles.

c. Improvements in site exploration and characterization for deep foundations, including.

(1) Extension of use of *in-situ* tools and their interpretation relative to piles.

(2) Development of rapid and effective means for deep site characterization for both regions and specific sites (for example, to accommodate moving of structure locations without either ignoring the differences in ground conditions between the new site and the old site or requiring a complete, duplicate subsurface investigation for the new site).

d. Probabilistic Methods in Pile Design and Analysis, to address.

(1) True probabilistic bases for LRFD design codes.

(2) Quantification of uncertainties in the design methodology itself.

Several other topics for research were suggested, although they were judged to be of slightly lower importance:

a. Monitoring of the behavior of deep foundations under service loads.

b. Methodologies for requalifying deep foundations supporting existing structures.

c. Improved predictions of pile driveability.

d. Incorporation of artificial intelligence in the design and construction of deep foundations.

e. Quality control and quality assurance methods in drilled shaft construction.

f. Influence of thermal regime in the ground on the behavior of deep foundations (primarily oriented towards deep foundations in frozen soils).

Again, there is some overlap among these perceived needs and research presently underway; however, the unaddressed issues from both the CERF and NSF lists should serve to reinforce the needs articulated by members of the transportation community in the following section.

Research needs identified by the transportation engineering community

Procedures. Committee A2K03 of the Transportation Research Board developed a set of research problem statements for deep foundations in 1988, based on a procedure similar to that followed for this report. Several problems identified in that set of problem statements that did not reach the funding stage within the FHWA in the ensuing four years were "carried over" and submitted for perusal to the membership of Committee A2K03 (including private sector and university members), a representative of each state DOT geotechnical office or bridge division, interested individuals who had declared themselves as friends of Committee A2K03 and all regional geotechnical engineers of the Federal Highway Administration. In addition, members of the A2K03 Subcommittee on Pile and Pier Research Needs proposed new statements, which were submitted to the above groups along with the carry-over statements. No attempt was made to edit the statements in light of any current or proposed research known by the author. The membership of this subcommittee consisted of

Roy Borden, North Carolina State University
Jean-Louis Briaud, Texas A and M University
Joseph Caliendo, Florida DOT
Richard Engel, Ohio DOT
George Goble, University of Colorado
John Ledbetter, North Carolina DOT
James Long, University of Illinois
William Lytle, Midlantic Piling, Inc.
Gary Norris, University of Nevada - Reno
Michael O'Neill (chair), University of Houston

In this manner, a total of 19 research problem statements (5 carry-over and 14 new) were developed and mailed, in the summer of 1992, to 141 individuals in every state in the United States and several Canadian provinces. Each respondent was asked to study each statement and then to rank all of the statements according to four categories, based on the urgency of the research perceived by the respondent. Since solutions for some problems may have existed outside of the transportation geotechnical engineering community but may not have been incorporated into highway practice, each respondent was also asked whether he/she considered the topic more appropriate for formal research or for a project involving the synthesis of existing information.

A copy of the instructions that were sent to the 141 respondents, which contains the 19 detailed problem statements, is given in the Appendix. A total of 56 responses were received by 15 September 1992. Those responses are summarized later in this report. In addition to ranking the problem statements according to urgency and research category (research or synthesis), each respondent was asked to provide any new problem statements that he/she felt were important but that were not included with the set of 19 that they received. Eleven such new statements were returned. Those are reproduced in the Appendix as Statements 20-30. Obviously, since the remaining respondents did not have the opportunity to rank these statements, no comparison of rankings with the other statements is shown; however, these statements are discussed. A list of all respondents, along with their employment category, is given in the Appendix, in Table A1.

Problem names. The research problem statements are summarized according to problem name in Table 1.

The carry-over statements were Statement Nos. 1, 4, 10, 11, and 12.

Results of urgency rankings. For each problem statement the ranking numbers (1-4) were summed and divided by the number of respondents expressing an opinion on that particular problem, giving a mean ranking ranging between 1 and 4. With the system used (most urgent = 1, research not needed = 4), the lowest mean ranking represented the project considered most urgent by the respondents. This number was then subtracted from 4 and the remaining differences divided by the value of the difference for the most urgent problem, resulting in normalized priorities ranging from 0 (lowest) to 1 (highest). These priorities are presented in Figure 1 for the entire population of respondents and in Figure 2 for only the state DOT geotechnical and materials engineer respondents. The results in the two figures are similar, but some differences exist. Table 2 summarizes the rank order of the problems statements from Figures 1 and 2 that have a normalized priority of 0.80 or above, which is interpreted to be the value associated with a general consensus of high priority by the respondents. These ranking have been used in the establishment of priorities for research in deep foundations by the FHWA.

Results of synthesis versus research evaluations. Those problems that were considered by the respondents to be most amendable to research, rather than synthesis, are indicated in Figure 3. It is clear that Problems 1, 3, 11 and 17 were considered the most amendable to formal research, while Problems 3, 6, 10, 13, and 19 were considered most amenable to solution by synthesis of existing information.

Proposed Research Program in Deep Foundations

The results of the comments by the respondents to the survey indicated that problems exist in the following broad areas and that those problems should be addressed through the performance of certain of the research projects described in more detail in the Appendix (Problem Statements 1-30). The broad problem areas of most significant concern were:

a. Improvements in integrity evaluation of piles and drilled shafts, including methods of quickly and inexpensively conducting quasi-static loading tests on piles or drilled shafts and long-term deterioration of pile materials (Statements 1, 9, and 12).

Table 1
Summary of Problem Statements by Name

Statement No.	Problem Name
	Original Statements
1	Improved use of nondestructive testing methods to evaluate structural integrity and behavior characteristics of piles and drilled shafts
2	Unit lateral load-deflection relations for drilled shafts and driven piles in rock
3	Loading test database for deep foundations
4	Long-term observations of settlement of piles groups
5	Determination of residual loads in driven piles
6	Artificial intelligence in the design and construction of driven piles and drilled shafts
7	Feasibility of and design methods for load-bearing sheet pile abutments
8	Evaluation of LRFD factors proposed by NCHRP for deep foundations
9	Expedient static loading tests for piles and drilled shafts
10	Increased use of timber piling in bridge foundations
11	Group action in axially loaded drilled shafts
12	Corrosion of driven steel piles
13	Universal definition of failure for piles in tension and compression from loading test results
14	Evaluation of augured piles for load bearing in highway structures
15	Fundamental behavior of driven piles in sand
16	Improved use of nondestructive evaluation methods to evaluate loading behavior of deep foundations
17	Capacity of bearing piles during earthquakes
18	Design methods for pile groups to resist dynamic lateral loads
19	Optimization of layouts for pile foundations
	Statements Proposed by Respondents
20	Penetration and driveability of steel H and pipe piles
21	End bearing resistance of steel H and open ended pipe piles
22	Effect of jetting on lateral stability of driven piles
23	Axially loaded performance of soil-mixing columns as deep foundation elements
24	Axial unit load transfer characteristics of downdrag loads
25	Unit lateral load transfer behavior of augured piles and pile groups
26	Evaluation of augured piles for load bearing in highway structures (additional topics)
27	Accurate estimate of pile tip elevations at existing bridges
28	Scourability of rock, decomposed rock and residual soils
29	Determination of critical scour depths at hydraulic crossings
30	Drilled shafts or piles in highly variable subsurface consisting of hard rock intermingled with soft claystone or stiff clay

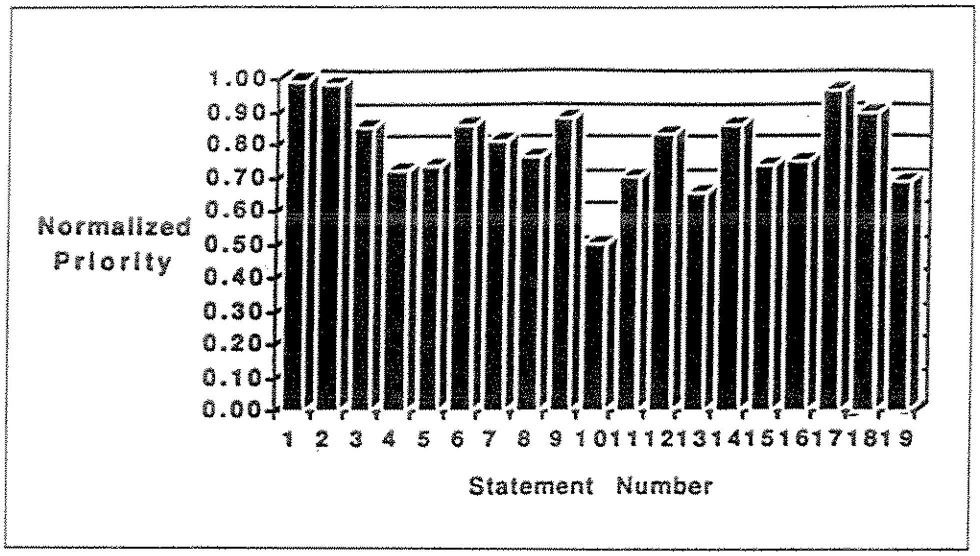


Figure 1. Normalized priorities - all respondents

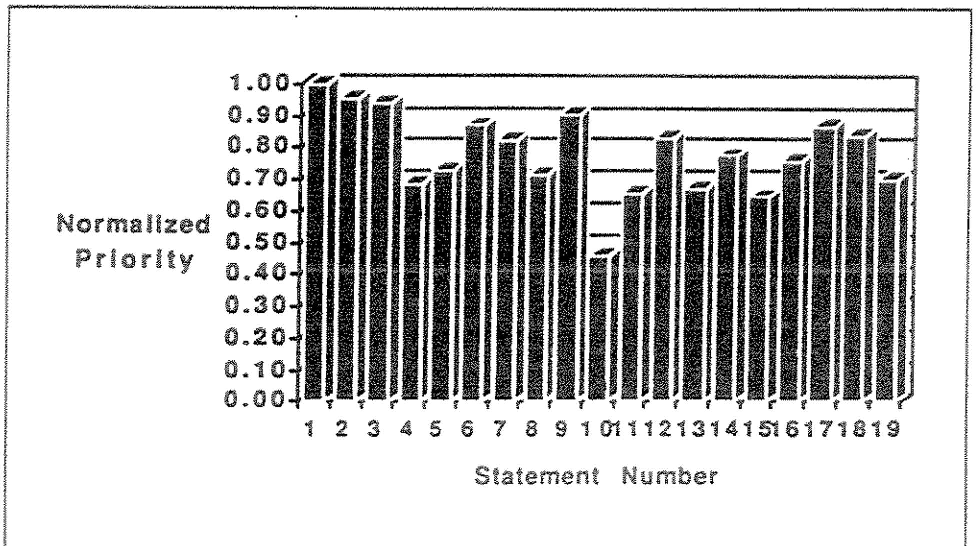


Figure 2. Normalized priorities - state DOT geotechnical and materials engineers

Table 2
Summary of Rank Order of Research Priorities

Rank Order	Statement Number	
	General Population	State Geotechs
1	1	1
2	2	2
3	17	3
4	18	9
5	9	6
6	6	17
7	3	18
8	14	12
9	12	7
10	7	-

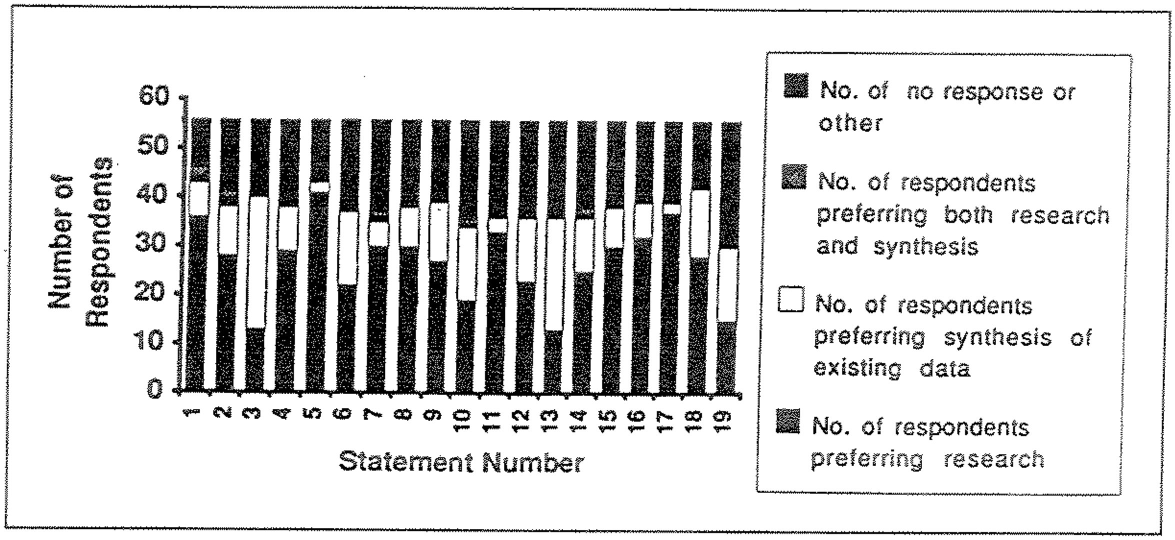


Figure 3. Category distribution by research or synthesis - all respondents

b. Development of an accessible knowledge base incorporating deep foundation construction methods, problems and problem solutions and behavior of piles and drilled shafts under load (Statements 3 and 6).

c. Prediction of the behavior of deep foundations and design methods for earthquakes and other dynamic loading events (Statements 2, 11, 17 and 18).

d. Incorporation of innovative techniques for deep foundation construction (Statements 7 and 14).

Area 1 correlates well with a similar concern expressed by the CERF study participants (2) Topic 1 - CERF. Area 2 was addressed by CERF (Topic 4 of related topics) and was one of the topic identified as a research need by

NSF (*1*) (Topic 4 in the list of topics of secondary importance). Area 3 was also identified by both CERF and NSF, the latter by implication through requalification of foundations for existing structures, and Area 4 was considered an important focus for research by NSF. The problems identified by the participants in this study, therefore, are consistent with research topics of critical concern expressed recently by other groups outside of the transportation engineering community. It is the author's opinion that, based on this evidence, the above four areas should be established as areas for *strategic research initiatives* by the Federal Highway Administration. Specific comments are offered below regarding the purpose and objectives of the problem statements that were the most highly ranked by the respondents and are therefore listed in connection with these strategic initiatives.

Strategic initiative 1 - evaluation of the structural integrity of deep foundations

Problem Statement 1. This statement was motivated by the results of a recently completed FHWA-sponsored study on methods of integrity testing of drilled shafts, which indicated that definitive procedures for detecting defects that are of the order of 25 percent of the cross-sectional area of the shaft or smaller are not presently available. An NCHRP initiative has begun that uses existing non-destructive evaluation (NDE) methods to determine lengths of piles and drilled shafts in bridges with unknown foundations, an application for which existing technology is apparently well-suited. However, fundamentally new techniques of both testing and interpretation must be developed and verified before the profession can confidently and consistently detect relatively small but potentially critical defects. The FHWA should develop an ongoing program of evaluation of new private sector technologies for NDE of piles and drilled shafts, per Problem Statement 1, while simultaneously funding, perhaps in cooperation with other agencies, new, innovative research in this area. The author considers the former component of this research to be "immediate-needs" research, while the latter component is clearly "visionary." This combined research should have the highest priority for funding.

Problem Statement 9. The FHWA is presently encouraging organizations with expedient deep foundation load testing methods to demonstrate their equipment and procedures on highway projects. However, doubts about the ability of some of these procedures to predict correctly the shaft and toe capacities persist. Independent, behavior-oriented studies are necessary to evaluate the effects of method and velocity of loading on load transfer development and capacity. Since some expedient load testing methods can be considered integrity testing methods, every attempt should be made to include this project in the funding cycle, although its priority is clearly lower than the high priority attached to Statement 1.

Problem Statement 12. This statement is associated with integrity problems that develop not during construction but at a later date. It is logical to combine this problem statement with that of *Problem Statement No. 26* and with *Problem Statement No. 10*. The urgency of this research is, according to the respondents, lower than the urgency of the above two research problems. Such research may be pursued by some combination of behavior and process methodologies. Its objective should be the recommendation of a decision process for requalification of existing deep foundations.

Strategic initiative 2 - development of knowledge bases for assistance in design and construction

Problem Statement 3. The need for an accessible data base of loading tests received very strong support from state DOT geotechnical engineers. Such a data base has the potential to expand greatly a given designer's judgment of the likely behavior of a deep foundation system in a given soil or rock profile that may not otherwise be available. The FHWA is presently developing such a data base; however, a number of other data bases already exist in various forms. There is a need to scrutinize these outside data bases and to capture within a single data base those loading tests for which adequate geotechnical site characterization information, construction information and loading data exist. This process will eliminate most of the data within existing data bases, as severe gaps exist in the knowledge base for perhaps 90 percent of all loading tests, but the remaining entries should be invaluable to both researchers and designers. In addition to performing the consolidation and scrutinization function, investigators in this project should develop a clear statement of what data should be obtained in a pile or drilled shaft loading test, especially geotechnical data, so that future tests contain enough useful information to be entered into the data base. This "immediate-needs" project combines elements of behavior and process methodologies and should be performed by a research team that is very experienced in pile and drilled shaft load testing, site characterization and data base management.

Problem Statement 6. This problem statement is closely related to Problem Statement 3 in that its intent is to make knowledge (in this case judgmental or expert knowledge) directly available to the designer and the construction engineer. The average urgency rating placed this problem fifth among state DOT geotechnical and materials engineers and sixth among all respondents. However, the responses were highly bimodal. Most rating this problem very low in urgency indicated that "we need real intelligence, not artificial intelligence" or that artificial intelligence will take the geotechnical engineer completely "out of the loop." On the contrary, the intent of expert systems or decision support software in both design and construction is to ensure that the designer and construction personnel have a comprehensive checklist of factors to consider in the solution of problems and the advice of experts, who are impartial, on methodologies to address those problems. Often, this advice will consist of recommendations to obtain more appropriate geotechnical information before proceeding to a decision, which will reinforce the importance of geotechnical input, not undermine it. The use of artificial intelligence to support decision making in the foundation engineering process has significant economic potential. Jeffries, for example (*1*) indicated that perhaps a 10 percent cost savings could be realized on most projects if engineers had better models for soil-structure interaction, but that a 30 percent savings is possible if methods can be developed to mitigate construction problems arising from disagreements, unforeseen subsurface conditions and similar construction operations problems. Construction-oriented decision support systems for both piles and drilled shafts can be of assistance in this effort. It is recommended that this project, which is grounded in process methodology, and which has both immediate needs and visionary components, include the development first of an overall shell that encompasses both design and construction methodologies in addition to details relating to driven piles, drilled shafts, and innovative deep foundations. Modules can then be added as the need arises and as funds are available in each of the subjects. It is suggested that the first modules be construction modules, particularly construction diagnostics modules.

Both of the projects in this strategic initiative should receive support, but their urgency is lower than that for Problem Statement 1 (above) and Problem Statement 18 (combined with 2, 11, and 17) (below).

Strategic Initiative 3 - modeling the dynamic behavior of deep foundation systems and their interaction with the superstructure

Problem Statement 18. An area presently (November, 1992) being considered for research funding by the FHWA is the mathematical modeling of the lateral and rotational stiffness of bridge substructures, primarily to consider how bridge foundations will respond to seismic loads, ship impact loads and flooding events (scour and hydraulic loading). This is essentially identical to the intent of the research proposed in Statement 18, except that the planned FHWA research will also consider the role of the superstructure in resisting and distributing the loads. Such a research program must expand to incorporate the objectives of *Problem Statement 2*, which seeks to model the lateral loading behavior of piles socketed into rock, which are often components of foundation systems subjected to seismic, impact and flood loads. *Problem Statements 11 and 17* are also necessary predecessor studies in order to develop appropriate inputs for the mathematical modeling-implied in Problem Statement 18. All of these statements represent "immediate needs" research that are associated with behavior-oriented research methodologies and can be performed by organizations capable of pursing traditional research methodologies. *Problem Statement 19*, on the other hand, should be deferred until the mathematical model for Statement 18 is completed, since that model can be used effectively in this related project.

Strategic Initiative 4 - innovative techniques for deep foundation construction

Problem Statement 7. As indicated in the problem statement, the potential exists for considerable cost savings with this innovative construction technique. The research approach to this problem would need to be comprehensive, with both behavior and process components, as suggested in the problem statement, in order to provide convincing information to designers that this type of foundation system carries no extra risk.

Problem Statement 14. Augured piles are used widely in Europe and elsewhere as bearing piles for transportation structures. Concerns over their structural integrity, procedures for reinforcement against lateral loads and procedures for design exist in the United States and have generally inhibited their use. In appropriate subsurface conditions augured piles can produce considerable cost savings in foundations. Because of the nature of the problem, that is, considerable information and experience available overseas, it is probably more appropriate that the project be pursued as a synthesis of available knowledge than as a traditional research project.

Both of the above statements have about equal urgency in the view of the respondents. These research problems should be considered at a lower priority than Problem Statements 1, 18, 9, 3, and 6, but some funding should be applied to encouraging the development and/or adaptation of innovative technologies, including the technology indicated in *Problem Statement 23*, which could conceivably be combined with Problem Statement 14 into a preliminary synthesis study, leading perhaps to more definitive studies at a later date.

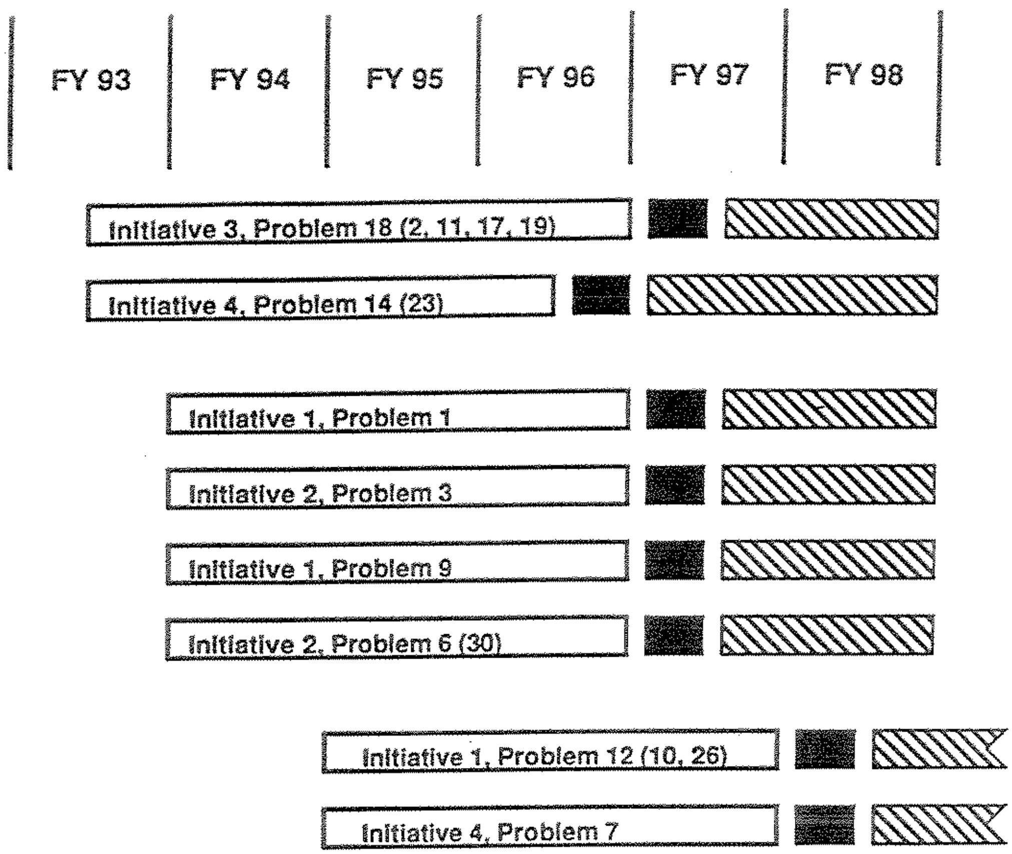
Problem statements not included in strategic research initiatives

Problem Statements 4, 5, 8, 13, 15, 16, 20, 21, 22, 24, 25, 27, 28, 29, and 30 were not included in the strategic initiatives for various reasons. In some cases, the problems addressed can be included in problem statements that are included (e.g., Statement 22 could be addressed in a research project ensuing from Statement 18, Statement 25 has the same purpose as Statement 14, and Statement 30 can logically be subsumed by Statement 6), others are very important but perhaps better belong in research areas other than foundations (e.g., Statements 28 and 29, which should be referred to soil mechanics and hydraulics researchers), and others have already been the subject of major research projects that are either ongoing or recently completed (e.g., Statements 20, 21, 24, and 27).

Potential deep foundations research plan

A summary of a possible FHWA research plan incorporating the various initiatives and problems (as subsets of the initiatives) is given in Table 3. Each research project that addresses a problem is assigned a period of 36 months (or more) for completion, which is more realistic than the 24-month periods that have been employed in the past. The first project (Problem 18) is assigned a 42-month period because it involves input from four other projects that should run in parallel with, or as part of, this project. An assessment period is indicated at the end of the each project, followed by a period in which additional research may be needed in order for the primary research to be of optimum use. The objective of the assessment should be to decide

Table 3
Potential Deep Foundations Research Plan



(Numbers in parentheses indicate problems to be combined with primary problem)

Period of review and evaluation

Technology transfer effort and/or follow-up research whether a project requires further investigation or development, in keeping with the observation that genuine advances in technology require long periods of continuous support to find their way into the design office. Programs to transfer the technology developed for the indicated project into highway practice should also be developed and executed during this period.

The exact order in which each of the research projects is performed will depend upon the progress of research conducted by other organizations, which should be monitored continually, and available funding. However, the order suggested is based on a balance of ranking by the respondents to the survey described in this report, anticipated research effort, and existence of preliminary efforts already underway within the FHWA.

For those projects requiring field testing, maximum use should be made of the newly-formed system of National Geotechnical Experimentation Sites funded jointly by the Federal Highway Administration and the National Science Foundation.

References

Anon. (1991). *Report No. 91-F1002.E*, Regional research needs in geotechnical engineering, Executive Summary on Regional Geotechnical Workshops for the National Science Foundation, Civil Engineering Research Foundation, Washington, DC.

Zaman, M. M. (1992). *Proceedings*, U.S. Canada workshop on recent accomplishments and future trends in geomechanics in the 21st Century, October 21 - 23, 1992, National Science Foundation, Washington, DC.

Research Needs in Deep Foundations (continued)

Appendix A:
Correspondence Accompanying Survey Problem Statements and Listing of Respondents

UH University of Houston

Department of Civil and Environmental Engineering
Houston, Texas 77204-4791
(713) 743-4250
FAX: (713) 743-4260

Cullen College of Engineering

June 16, 1992

Dear Highway Geotechnical Professional,

Approximately every three years, Committee A2K03 of the Transportation Research Board, on Foundations for Bridges and Other Structures, conducts a survey on research needs in the subject area of deep foundations. This survey is conducted by first collecting candidate research problem statements from members and friends of Committee A2K03, editing those statements into consistent formats, and then asking knowledgeable and influential highway engineering personnel, such as yourself, to rate the statements. A packet containing the statements that have been generated this year is enclosed in the attached packet. We ask you to review these statements, to complete the rating form that follows the statements, and return the rating form within two weeks after you receive this material. Your response will be collated with other responses and reported to the TRB. We expect that this report will be used to help determine the priorities for funding of FHWA and NCHRP research in deep foundations over the next three years.

You may be of the opinion that much information may already exist in the USA or abroad about a particular problem but that it is not in a form that can be readily used by you. If so, we request that you categorize the problem statement with the letter **S** in the last column of the rating form, to indicate that support of the problem should be considered for synthesis (literature review and manual development) rather than for research. Otherwise, please enter **R** in the last column, indicating that the statement should be considered to be a research statement (acquisition of new knowledge). If you don't have an opinion, leave blank.

We ask you to rate each problem statement on the rating form according to your perception of the urgency with which you feel the research needs to be performed. Ratings are 1 (very urgent) - 4 (research is not necessary). Ratings in Category 4 should include problem statements (a) for which you think adequate solutions already exist, (b) for which there is no practical need for a solution, and (c) for which you doubt that a solution can be found. Again, if you don't have an opinion on a particular problem statement, leave the rating for that statement blank.

To provide you with some perspective, in the front of the packet you will find a table that outlines the major research projects in the area of geotechnology that have been sponsored directly by the FHWA in the past several years, including projects in the area of deep foundations. In addition to the research projects listed in that table, two new projects, one related to improved predictions of soil properties for piling analysis using the SPT and one related to the behavior of drilled shafts in intermediate geomaterials (soft rock and hard soil) have begun. These two projects were funded as a direct result of a survey similar to this one that was conducted three years ago by Committee A2K03, so your input is extremely important.

While the committee members who proposed the problem statements are recognized as experts in the area of deep foundations, we are by no means collectively aware of all of the problems in deep foundation engineering for highway structures that require research, and we all have personal biases. Therefore, a problem statement for a very important research or synthesis problem may well have been left out. If such is true in your opinion, please complete and return the attached blank problem statement form so that such problem statements can be transmitted to the TRB and to the FHWA. Indicate whether you think the problem should be attacked as "research" or "synthesis." Please remember that research or synthesis problems should be able to be approached in an organized manner.

Thank you for your time. As a busy person myself, I realize that it is sometimes an annoyance to fill out survey forms. But this form is very important to the future of highway foundation engineering, so please let us hear from you.

For Committee A2K03

Michael W. O'Neill
Chairman, Subcommittee on
Research Needs for Piles and
Piers

enclosure (packet of problem statements, blank statement form
and rating form)

High Priority National Problem Area

HIGHWAY GEOTECHNOLOGY

FY 87 & Prior	FY 88	FY 89	FY 90	FY 91	FY 92	Remarks
A. Pile Foundation Design:						
Development of PILGPI /						
Allowable Stresses in Piles /						
Lateral Loads on Piles /						
Pile/Pile Group Load Testing /						
Stepped Bladed Vane Test /						
Pile Driving Inspection Manual /						
Pile Synthesis Report (NCHRP) /						
Pile Capacity Prediction (HPR) /						
Vibratory Pile Driving Capacity Prediction (NCHRP) /						
Load Factor/Resistance Factor Design (NCHRP) /						
Laboratory Testing of Model Piles (Staff) /						
Design Aids for Pile Foundations--Load Test Data Base/			/New Pile Predictive Techniques ($250K)/			
	/Negative Skin Friction (NCHRP) /					
	/Drilled Shafts ($500K) /					
		/Residual Stresses in Piles ($300K) /				
		/Influence of Pile Cap ($350K) /				
B. Spread Footings:						
Spread Footing Synthesis (NCHRP) /						
Tolerable Movement of Bridges /						
Spread Footings for Highway Bridges/						
Spread Footings on Compacted Fill /					/HIGHWAY GEOTECHNOLOGY MANUAL ($200K) /	COST EFFECTIVE FOUNDATIONS
	/Foundation Design of Abutments ($300K)/					
C. Alternate Retaining Wall Systems						
Earth Reinforcement (NCHRP) /						
Alternate Retaining Wall Systems Manual /						
Corrosion/Durability of Soil Reinforcement Elements /						
Development of Soil Nailing Guidelines Manual /						
D. Ground Improvement Methods						
Stone Columns Manual /						
Dynamic Compaction Manual /						
Wick Drains Manual /						
Geocomposites Manual /						
Geofabrics Manual /						
	/Permanent Ground Anchors Manual ($500K)/					
		/Rock Slope Manual ($300K) /				
			/Ground Improvement Manual ($150K) /			
IMPLEMENTATION DEMO NHI		/Spr. Ftg. Man. /	/Grnd. Impr. Man./ /Spread Footings /	/Pile Manual / /Ground Impr. / /Geotechnology Courses	/Piles / /	

TRB Committee A2K03 Survey of Research Needs in Deep Foundations, 1992

Statement Number	Urgency (1 - most urgent, 2 - intermediate, 3 - not urgent, 4 - not needed)	Category (Research or Synthesis)
1		
2		
3		
4		
5		
6		
7		
8		
9		
10		
11		
12		
13		
14		
15		
16		
17		
18		
19		

Your name (optional):

Your principal job function (check one):

- State DOT geotechnical or materials engr:__________
- Federal DOT geotechnical or materials engr:________
- State DOT bridge design engineer: ___________
- State DOT research engineer: __________
- Consulting engineer:_________
- University professor:________
- Other (please state):_______ ___________________

Please mail the completed form to:

Michael W. O'Neill
Professor and Chairman
Department of Civil and Environmental Engineering
University of Houston
Houston, TX 77204-4791

within two weeks of receipt. Thank you.

Proposed New Problem Statement

Your Name: ______________________________

I. **Name of Problem**:

II. **The Problem**:

III. **Objectives**:

IV. **Current Activities**:

V. **Urgency**:

Problem Statement No. 1

I. Name of Problem. Improved Use of Nondestructive Testing Methods to Evaluate Structural Integrity and Behavior Characteristics of Piles and Drilled Shafts.

II. The Problem. There are a number of situations in which the structural integrity of driven piles and drilled shafts may be in question after construction. For example: (1) In drilled shafts intensive use of reinforcing steel may potentially impede the outflow of concrete into the annular space between the reinforcing cage and the side of the borehole, producing a potentially important defect. Conventional low-strain sonic, ultrasonic or gamma logging methods in use today in the USA cannot reliably detect such defects unless they are very large (perhaps 25 percent of the cross section of the drilled shaft). (2) In the slurry displacement method for installing drilled shafts, which is becoming quite popular due to cost factors, cuttings can potentially settle out of suspension in the slurry and become entrapped between the concrete and base of the shaft or between the concrete and the reinforcing steel, producing serious defects. (3) Piles driven to potentially high refusal by developing toe bearing in rock may have defective toes due to structural failure. (4) In river or marine environments scour may remove soil from around piles or drilled shafts that may be needed to assure stability.

III. Objectives. Conventional methods that have proved to be successful for large defects should be studied fundamentally and modified so that either smaller defects can be detected or the behavior or piles or drilled shafts can be directly inferred. New methods, not previously employed in the USA, should be sought out (e.g., x-ray holographic mapping techniques) and evaluated. Finally, the reliability of the new method(s) should be demonstrated to be high, so that it will be possible to use results from tests in the evaluation of acceptance of constructed drilled shafts and driven piles. Having such methods available will increase the designer's confidence in the integrity of deep foundations and should result in the use of lower factors of safety, leading to lower cost.

IV. Current Activities. FHWA has just recently completed a major research effort (DFTH61-88-Z-00040), in which conventional low-strain integrity testing methods, such as those enumerated in I, above, were applied to drilled shafts constructed with known defects. The results were encouraging but not conclusive. Additional work needs to be done to identify other methods, enhance the most appropriate conventional methods and develop systems simple enough for routine use.

V. Urgency. ______

Problem Statement No. 2

I. Name of Problem. Unit Lateral Load-Deflection Relations for Drilled Shafts and Driven Piles in Rock.

II. The Problem. Many deep foundations for bridges and walls are socketed into rock beneath strata of weak soil. Such foundations are often subjected to large lateral loads and must be designed for this loading condition. Present practice uses simplified beam models, such as the point-of-fixity model. However, such models can give misleading results and fail to take proper account of the nonlinear behavior of both the concrete and the rock. On the other hand, design models, such as COM 624P, that use the so-called p-y (unit lateral load deflection) functions are readily available for analyzing drilled shafts or driven piles in soil, but adequate means to define the p-y curves in rock do not yet exist. Therefore, practice continues to develop around inappropriate design models that, for the most part, result in extremely overconservative foundations.

III. Objectives. p-y relations for various types of rock need to be developed so that existing models for laterally loaded pile analysis that use p-y methods, such as COM 624P, can be employed confidently by designers. Important effects such as rock structure, stress relief, in-tact strength and stress-strain behavior should be inputs; however, approximate means for evaluating these inputs based on tests that can be conducted routinely by state DOT's also need to be developed in order to make the research usable.

IV. Current Activities. The Kansas Highway Commission has developed criteria for p-y relations in small drilled shafts in shale and sandstone through loading tests. Other work outside of the transportation engineering community (for example, contract work for EPRI by Cornell University) has been conducted recently to address the general problem theoretically.

V. Urgency. _______

Problem Statement No. 3

I. Name of Problem. Loading Test Database for Deep Foundations

II. The Problem. Before methods of analyses and design are accepted for specifying deep foundations, the ability of the method to produce a safe and economical design should be established. A common procedure for assessing these methods is to compare predicted behavior with measured behavior. The types of foundations could include driven piles, drilled shafts, auger-grout piles, mini-piles, and drilled-and-grouted piles.

Uncertainties associated with predictive methods are also illuminated by comparing predicted and measured behavior for a large number of tests. Ideally, a convenient and readily accessible collection of carefully documented axial loading tests on deep foundations would provide the researcher, design engineer, or licensing agency a convenient tool for evaluating new and existing methods.

The results from this collection of axial loading tests represents a unique opportunity to evaluate current methods for predicting axial behavior, to improve the design methods, and to provide for an accessible data base so that design methods can be evaluated rapidly. Use of a database is especially important in the evaluation of resistance factors for the development of new LRFD factors for foundations.

III. Objectives. The objectives of the research effort are to collect, interpret, and transcribe a large amount of loading test data into an electronic, relational database and use the database to evaluate methods for predicting behavior of deep foundations. The database will be used to evaluate the ability of current methods to predict axial behavior of deep foundations. An additional objective of this research is to make the database available publicly so that other researchers, design engineers and licensing agencies can use the database for their purposes. An easily accessible database can potentially enhance and encourage the development of technology and new methods of deep foundation analysis, design and construction by making available the detailed results of past loading tests.

IV. Current Activities. Several deep foundation data bases of varying detail have been developed or are currently being developed. These include databases by the FHWA at Turner-Fairbank, University of Illinois, University of Florida, University of Texas, Cornell University, Texas A and M University, University of Houston, and perhaps others. These databases were developed for differing purposes and are not consistent in detail or format.

V. Urgency. _______

Problem Statement No. 4

I. Name of Problem. Long-Term Observations of Settlement of Pile Groups.

II. The Problem. Most pile group designs are based on capacity considerations and include very little settlement analysis. However, capacity is often of minor concern, whereas settlements is of major concern. While many studies have been conducted to investigate and predict the short-term settlement of pile groups, very little design guidance exists concerning the long-term settlement. Furthermore, many pile groups driven through soft soils into dense soils or rock, while they may not settle appreciably, may develop considerable drag loads due to settlement of the soil. In such as case the stresses in the piles become a major concern, and settlement may also be a concern if the bearing capacity at and near the toe is not sufficient to resist the drag loads.

III. Objectives. The objectives of the research effort are (1) to perform long-term observations on groups of piles instrumented to measure the distribution of load along the piles and the patterns of stresses, deflections and pore pressures in the surrounding soil, and (2) to correlate the observations to good-quality, comprehensive *in situ* tests, such as SPT, CPT, CPTU, PMT, and laboratory tests on undisturbed soil samples for soil parameters, including compressibility parameters.

IV. Current Activities. Currently, research funded by the NCHRP is being completed at Texas A and M University relating to the use of bond-breakers on the mitigation of drag loads in piles and pile groups. Otherwise, no formal research is known to be in progress.

V. Urgency. ________

Problem Statement No. 5

I. Name of Problem. Determination of Residual Loads in Driven Piles.

II. The Problem. It is imperative to known the actual distribution of loads along a pile both after the pile is driven and at the time of failure. Interpretation of results from static loading tests and high-strain dynamic tests (e.g., using CAPWAP) are affected by the distribution of residual loads developed in the pile during and after installation. Neglecting residual loads can lead to errors in estimations of toe and shaft resistances by more than 100 percent, which has lead the profession to support improbable concepts such as "critical depth." These concepts, which have found their way into mainstream design rules, have an important effect on determination of pile lengths.

III. Objectives. The objectives of the research effort are (1) to perform static and dynamic full-scale tests on piles instrumented with gages to sense load and strain, so that the true load distribution can be determined. These tests should be performed on piles that have a strong likelihood of exhibiting residual stresses (e.g., long, relatively flexible piles). (2) to determine the true distribution of static resistance in test piles as well as the distribution of residual load. (3) Fine-tune means to evaluate theoretically the resistance of a pile when the true resistance distribution along the pile is known.

IV. Current Activities. No formal activities in this subject area are known to exist currently.

V. Urgency. ________

Problem Statement No. 6

I. Name of Problem. Artificial Intelligence in the Design and Construction of Driven Piles and Drilled Shafts.

II. The Problem. Perhaps the single most pressing problem affecting the design and construction of deep foundations is the failure of state DOT's to employ knowledge that has been acquired through formal research and through experiences acquired both within the agency and by others. As a result, foundation costs remain excessive, and occasionally designs are inadequate. The problem is especially prevalent in the construction of deep foundations and in the evaluation of existing piles and drilled shafts, in which resident engineers and inspectors are not knowledgeable of proper construction methods and cost-effective remediation of problems that arise during construction or rehabilitation. An effective way of making this knowledge known to practitioners in a way that they will use it is by means of the decision support system, a user-friendly microcomputer program containing a knowledge base and a set of rules developed by experts and verified independently by other experts. This method of mass technology transfer is very effective, especially for younger personnel, who have had considerable experience learning through computers, and should be able to break the barrier of "if it is not done in my state (or district), I don't want to hear about it."

III. Objectives. The objectives of this research would be to (1) review existing expert systems and neural network programs that address design, construction and rehabilitation, both in the USA and elsewhere, and to adapt those that are feasible for use in DOT technology transfer, (2) to adapt or develop a shell program that would maximize user friendliness while permitting the proper logic to be employed (probably fuzzy logic), and (3) to develop the composite software system and to have it verified by recognized experts in design and construction. [A related objective would be to have an arm of the FHWA, perhaps the National Highway Institute, develop a course that could be given in house in each state to familiarize state personnel with the software.]

IV. Current Activities. No formal activities in this subject area are known to exist currently, although some research on expert system in deep foundations is known to be underway at Clemson University, Auburn University and the University of Houston.

V. Urgency. _______

Problem Statement No. 7

Name of the Problem. Feasibility of and Design Methods for Load-Bearing Sheet Pile Abutments.

II. The Problem. According to FHWA statistics, 41 percent of the nation's 578,000 bridges are either structurally deficient, thereby requiring replacement, or functionally obsolete, requiring either widening or structural replacement. Many of these bridges have short spans, and rehabilitation can benefit from the application of technology that produces cost-effective abutments. Steel sheet piling can carry both lateral and vertical loads, thus affecting considerable economy over separate wall and bearing-pile systems. Further economy can be realized by the fact that shoring and cofferdams can be eliminated during construction, thus accelerating the process. Cost savings nationally could potentially exceed $1,000,000,000.

III. Objectives. The objectives of this research would be to (1) acquire information on the design and performance of load-bearing steel sheet pile abutments in both the USA and elsewhere, (2) conduct either analytical or experimental studies (or a combination) to investigate the effects of sheet pile-soil interaction, use of tiebacks, application of combined loading and similar effects, so as to provide a rational basis for design methods, and (3) to develop a straightforward design method that can be confidently applied by design engineers.

IV. Current Activities. None, although some load-bearing steel sheet pile abutments are known to be in service in New York state.

V. Urgency. _______

Problem Statement No. 8

I. Name of Problem. Evaluation of LRFD Factors Proposed by NCHRP for Deep Foundations

II. The Problem. NCHRP Study 24-4 has recently proposed new resistance factors for the design of pile and drilled shaft foundations. If LRFD for foundation design is adopted by AASHTO, a major new philosophical approach to the design of bridge foundations will undoubtedly be put into practice by many states in diverse geological settings. While the resistance factors proposed were based on a rational analysis of the best available data, and they were calibrated against safety factors for some typical conditions, more detailed evaluation remains to be done. For example, site specific calibration of resistance factors against presently employed safety factors at many diverse sites, assessment of the effect of specific design methods on the choice of resistance factors and the testing of resistance factors at sites where both statistical variation of soil and foundation behavior are known important issues that need to be addressed before the new factors are accepted and placed into use. Additionally, the hypothesis that, with the use of LRFD, overall costs of bridge construction will be reduced should be investigated fully.

III. Objectives. The objective of the research is to verify or modify appropriately the resistance factors developed in NCHRP Study 24-4 so that AASHTO can confidently adopt the factors and publish an LRFD method of deep foundation design that will be accepted by state DOT's.

IV. Current Activities. None known.

V. Urgency. _______

Problem Statement No. 9

I. Name of Problem. Expedient Static Loading Tests for Piles and Drilled Shafts.

II. The Problem. While static methods of pile design have advanced considerably during the past years, and while dramatic advances have been made in the high-strain dynamic testing of piles and drilled shafts, there remains a need to conduct static loading tests. However, the costs for such testing are presently in the range of $50 - $100 per ton of maximum load, which is often too high to justify static loading tests in the case of smaller construction projects and for conducting fundamental research on pile-soil interaction, especially for prototype-sized drilled shafts, whose capacities can exceed 1,000 tons. There is a need to develop or adapt expedient testing methods that are capable of applying high static loads to piles and drilled shafts, that give reliable results and that are inexpensive.

III. Objectives. The objectives of this research are (1) to identify and evaluate existing low-cost static testing expedients, (2) to evaluate the reliability of the results obtained from such expedients and develop modifications, if necessary, and (3) to propose new expedient testing methods if existing expedient testing methods, as modified, are not considered feasible and reliable.

IV. Current Activities. None known by government agencies, but private industry in the USA and elsewhere is known to be developing expedient static testing methods.

V. Urgency. _______

Problem Statement No. 10

I. Name of Problem. Increased Use of Timber Piling in Bridge Foundations.

II. The Problem. Infrastructure rehabilitation will require that repair and expansion of existing bridges be performed as cost-effectively as possible. In this regard timber piles have potential economic advantages over other types of piles in certain applications. Although state highway engineers currently seldom specify timber piles, there is ample evidence that such piles perform very well structurally for long periods of time. Furthermore, economic pressure is increasing to use products produced in the USA. Unlike steel piles, timber piles are still a domestic product.

Past research has been conducted into the behavior of timber piles, but some results, especially regarding allowable stresses, has been contradictory.

III. Objectives. The research should have as its objectives assessment of the structural integrity of timber piles under varied conditions of installation and loading and the assessment of long-term behavior of timber piles under varied environmental conditions. The principal outcome should be a clear and concise document that outlines proper installation procedures, allowable stresses and addresses environmental effects on the design of timber piles that could be incorporated into design manuals or decision support systems to allow rapid dissemination of the acquired knowledge to state DOT designers.

IV. Current Activities. The American Wood Preservers Institute has previously sponsored small research efforts at the University of Colorado to determine allowable stresses in timber piles.

V. Urgency. _______

Problem Statement No. 11

I. Name of Problem. Group Action in Axially Loaded Drilled Shafts.

II. The Problem. A major advantage of drilled shafts is that one drilled shaft can be employed to replace a group of driven piles. However, in some large structures, the size of a single drilled shaft becomes infeasible, so that groups of drilled shafts must be used. As drilled shafts become more popular, the use of drilled shafts in groups will likely increase. However, there is no sound basis on which to account for group action in axially loaded drilled shafts. Research from driven piles is largely unusable in this respect due to the differences in soil stresses produced by driving piles versus boring piles. Without fundamental guidance on the subject, extreme conservatism is likely to be practiced by designers, but in some cases apparent overconservatism may actually result in inadequate designs.

III. Objectives. Rules for the effects of group size, depth, spacing, construction method and soil properties on group settlement, distribution of load among and along the drilled shafts and efficiency should be developed. This objective could be accomplished entirely by using sophisticated analytical tools, but it is essential that appropriate experimental modelling be carried out to verify the effects that are produced by the analytical model(s).

IV. Current Activities. None known.

V. Urgency. ________

Problem Statement No. 12

I. Name of Problem. Corrosion of Driven Steel Piles.

II. The Problem. Many older bridges are supported on driven steel piles. Whether foundation rehabilitation is needed may depend largely on whether such piles are substantially corroded. Some research studies have concluded that corrosion in driven steel piles is not a problem as long as a substantial portion of the pile is below the water table. Yet, there have been a number of instances in which corrosion has been observed when apparently "safe" piles have been exposed. There are no applicable guidelines for investigating a site to determine whether corrosion is likely to have occurred or will occur with new construction. As a result, protection is often recommended by corrosion engineers who are unfamiliar with the corrosive behavior of steel piles, which can result in solutions that are overly conservative and expensive.

III. Objectives. The primary objective is to develop reliable criteria which will allow the design engineer to determine the likelihood of corrosion in existing piles or in piles to be driven and to quantify the probability that the amount of corrosion will be detrimental to the foundation now or at some time in the future. This objective should include recommendations for *in situ* and/or laboratory tests that will be appropriate for assessing probable corrosion of steel piles on a specific site.

IV. Current Activities. None known.

V. Urgency. ________

Problem Statement No. 13

I. Name of Problem. Universal Definition of Failure for Piles in Tension and Compression from Loading Test Results.

II. The Problem. Information transfer regarding the capacity of tested piles suffers from inconsistent reporting of pile "capacity," or failure load. Inappropriate definitions of failure may also impact the design of piles based on the results of loading tests. Without consistent and universal definitions of failure, it becomes difficult to advance the state of the art in the determination of static capacity from both static and dynamic loading tests, since capacity is a nebulous term. The lack of agreement on the definition of failure is predicated on a lack of understanding of the fundamental mechanisms of failure in the pile-soil system and how they are reflected in load-movement relationships. These mechanisms vary among soil and rock types, and among pile types, as well. There is also disagreement on whether the definition of failure load should be based upon development of yield conditions in the soil or rock or upon a settlement that is dependent on the expected performance of the superstructure that the pile supports.

III. Objectives. The objectives of this study are (1) to determine first if it is feasible to develop a universal definition of failure and (2) to proceed to develop that definition if feasible.

IV. Current Activities. None known.

V. Urgency. _______

Problem Statement No. 14

I. Name of Problem. Evaluation of Augured Piles for Load Bearing in Highway Structures.

II. The Problem. Augured piles (piles that are constructed by drilling a borehole with a continuous flight auger and grouting or concreting the hole as the auger is withdrawn) have been used for decades as bearing piles in the private sector but are almost never used in that manner in highway construction and other segments of the public sector. Lack of usage in the public sector stems from a perceived lack of quality control and lack of appropriate methods for computing static capacity. There are many instances in which augured piles could be economically superior to either driven piles or conventional drilled shafts, so that further study of augured piles is warranted.

III. Objectives. The objectives of the research are (1) to evaluate methods of augured pile construction around the world and to determine which quality controls and quality assurance techniques will be needed to make augured piles acceptable in the bridge engineering community, and (2) to evaluate or develop design models for predicting the capacity of augured piles in the major soil types.

IV. Current Activities. None known.

V. Urgency. _______

Problem Statement No. 15

I. Name of Problem. Fundamental Behavior of Driven Piles in Sand.

II. The Problem. Predicting the static capacity of driven piles in sand is an imprecise science, even if accurate soil parameters are available. This is because fundamental effects of pile installation on pile-soil interaction are still not well understood and therefore design models are at best highly approximate. Many important issues have not been addressed from a fundamental perspective. For example, in H piles, under what conditions do partial plugs develop between the flanges and in pile piles and how do these plugs affect shaft and toe resistance? How do the *in situ* stresses and the presence of fines in the soil matrix affect plugging in open pipe piles? Under what conditions do these plugs develop under static loading conditions when they do not develop during driving? How do the shape of the pile, the displaced volume of the soil and the stress and compressibility conditions of the soil affect lateral effective stresses in the soil and thereby the final unit shaft and toe resistances? How does pile installation affect the angle of pile-soil friction, particularly in layered deposits and as a function of the penetration of the pile? Can relatively simple models such as the expanding cavity model be used to predict shaft and toe resistance? None of these questions has satisfactory answers, so that design methods remain approximate and largely empirical.

III. Objectives. The objectives of the research are (1) to identify the most important fundamental effects of pile-soil interaction in sand for those types of piles most frequently employed in highway bridge construction, (2) to study these effects both analytically and experimentally, and (3) to offer improved candidate methods for computing static capacity of this limited class of piles based on the results of this research.

IV. Current Activities. None known; however, a research project on improved assessment of the SPT to obtain soil properties for the design of piles is underway at the University of Colorado.

V. Urgency. ______

Problem Statement No. 16

I. Name of Problem. Improved Use of Nondestructive Evaluation Methods to Evaluate Loading Behavior of Deep Foundations.

II. The Problem. At present, the only effective means of evaluating the loading behavior of deep foundations is through high-strain dynamic testing or through static loading tests. These tests are expensive and are time consuming to set up for purposes of evaluating the capacities of piles or drilled shafts that are suspected of having inadequate capacity. Recently develop non-destructive evaluation methods, whose principal purpose is to detect defects, may also be capable of being used to infer stiffness and, either directly or indirectly, capacity. If appropriate enhancements of these techniques and/or modifications to methods for their interpretation can be developed, capacities of piles or drilled shafts suspected of having defects can be assessed, and the importance of the defect on bottom line performance evaluated quickly and economically.

III. Objectives. The objectives of the research will be (1) to determine which NDE methods can be adapted to capacity evaluation, (2) to adapt one or more such methods, and (3) to compare the results of static loading tests with results obtained using the adapted method, using piles and drilled shafts that have previously been tested at National Geotechnical Experimentation Sites and elsewhere.

IV. Current Activities. None known.

V. Urgency. _______

Problem Statement No. 17

I. Name of Problem. Capacity of Bearing Piles During Earthquakes.

II. The Problem. The transportation engineering community is now becoming aware of the need to ensure the stability of bridges and their foundations in all parts of the country against earthquake loading. Experience from the Mexico City Earthquake of 1985 and the Loma Prieta Earthquake of 1989, among others, have suggested that loss of pile capacity may occur during seismic events in both clay and sand. Such loss of capacity may be the result of buildup of pore water pressures to the point where they influence pile capacity but do not produce liquefaction, to the redistribution of effective stresses in the soil mass and/or to the degradation of soil fabric. Most existing information related to loss of capacity due to vibratory loading is based on experiments that considered loading of the pile through the structure or the pile head, not through the soil, as occurs during earthquakes. Since the degradation characteristics of soils near piles are likely to be very different where loading is applied through the soil, special attention needs to be given to this form of loading before design guidelines can be developed.

III. Objectives. The objectives of the research are to adapt or develop analytical procedures to forecast capacity changes in driven piles and/or drilled shafts in sands and clays during seismic events, to conduct experiments or to access existing data to use in verifying the analytical procedure(s) and to use the analytical procedures to develop recommendations for engineers who are producing new designs or evaluating the safety of foundations for existing structures.

IV. Current Activities. None known.

V. Urgency. _______

Problem Statement No. 18

I. Name of Problem. Design Methods for Pile Groups to Resist Dynamic Lateral Loads.

II. The Problem. Heavily loaded bridge foundations often consist of groups of driven piles or drilled shafts. Approximate methods have been developed in the past by several academic researchers to analyze groups of piles subjected to lateral shears and overturning moments due to static loads or to slow cyclic loading. However, design methods are not generally available to analyze pile groups subjected to dynamic loads, such as impact loads from ships, ice loads and seismic events. In particular, there is a need to develop methods that are as simple as possible for forecasting deflections and rotations at the interface between the piles and superstructure and distributions of shears, moments and axial thrusts among the piles in the group.

III. Objectives. The objectives of the research are to investigate existing methods for analysis of laterally loaded pile groups and to adapt such methods to the analysis of dynamic loading (both loading through the pile cap and loading through the soil). Using these analytical methods parametrically, and other information, design methodologies are to be developed and reported.

IV. Current Activities. None known.

V. Urgency. _______

Problem Statement No. 19

I. Name of Problem. Optimization of Layouts for Pile Foundations.

II. The Problem. In modern bridge design, many, perhaps several hundred, loading cases may need to be checked for each structure. Each of these cases results in different patterns of loading on the foundation. The local foundation loads (reactions) themselves, however, depend in turn on the lateral, vertical and rotational (and perhaps torsional) stiffness of the foundation if foundation-structure interaction is considered in the analysis of the bridge superstructure. If the foundation consists of a group of piles, economy of design can be realized if the layout of the piles is optimized for each loading condition, such that pile-head reactions in the most heavily loaded piles are minimized. In some cases, only though developing different pile layouts for different loading conditions can it be assured that a particular loading condition is truly critical. While it is possible to develop trial layouts and stiffness values for each loading condition by hand, it is very time consuming and expensive to do so, and optimum pile layouts may not be assured.

III. Objectives. The objectives of this research are to develop methods to optimize pile group layouts, in terms of indexes that relate to minimum cost of pile installation, for any given pattern of loading. These methods should consider state-of-the-art methods for assessing stiffness characteristics of the individual piles and of the group as a whole. The developed method, which might be based on principles of neural networks or some other analytical procedure, should be programmed so that it can be used by knowledgeable geotechnical and structural engineers with the aid of a microcomputer.

IV. Current Activities. None known.

V. Urgency. _______

Problem Statement No. 20 (proposed by V. A. Diyaljee, Alberta Transportation)

I. Name of Problem. Penetration and Driveability of Steel H and Pipe Piles.

II. The Problem. In designing driven pile foundations the geotechnical engineer has to determine the depth at which the piles have to be terminated as well as to determine whether the piles can be driven to the predetermined toe elevation. Without test pile installations the ability of piles to attain a predetermined depth is generally judged from the physical soil characteristics - type, strength, compressibility, moisture content, etc., location of deposits, whether on land or in a river environment, absence or presence of large-sized material - gravel, cobbles, boulders, energy to be applied by pile hammer, type of hammer, etc.

In the absence of dynamic measurements, which are prohibitive on small-sized projects, SPT data is normally used to judge the penetrability and driveability of such piles. However, this approach is not universal and correlations that may exist are site specific. Judgement and experience along with SPT blow counts and soil stratigraphy review are perhaps the factors that play a significant role in this important aspect of foundation design/construction. The use of SPT blow counts cannot be universal for obvious reasons. In sandy soils high blow counts may not deter piles from penetrating to great depths if such driving is in soils that are water lain, e.g., a river environment. In such instances penetration may accrue from "liquefaction" of the soil around the pile. However, if the same deposit is not in a water lain condition the depth of penetration may be much shallower.

The question of pile capacity also becomes a concern. Many construction personnel regard easy driving as sign of very low capacity since "set up" etc. is not assessed during driving. The designer on the other hand has to make these judgements that the desired toe elevation will provide the required capacity. Despite the fact that such capacities can be determined from dynamic measurements (PDA) for example, such measurements do not always provide definitive answers.

III. Objectives. The objectives of the research would be to try to determine from energy relationships and soil characteristic some way in which depth of penetration can be achieved, thereby allowing the right recommendations for hammer sizes and energy output. This would allow the geotechnical engineer to have a much more active role in pile foundation design and construction. At the time it is felt that while the geotechnical engineer is relied on to provide capacities from a design aspect, his role is very passive in relation to contractibility and related problems which should be directly linked to the behavior of the foundation soil.

IV. Current Activities. Not known.

V. Urgency. 1

Problem Statement No. 21 (proposed by V. A. Diyaljee, Alberta Transportation)

I. Name of Problem. End Bearing Resistance of Steel H and Open Ended Pipe Piles

II. The Problem. One of the concepts frequently debated by geotechnical engineers engaged in pile design is the assessment of end bearing resistance provided by steel H and pipe piles. The use of full section area, area enclosed by the flanges in the case of H piles, partial area, etc., have all been identified in the literature from time to time. However, a designer has to make judgements for his site in relation to how he conceives the behavior of the soil during and after driving. Lack of proper knowledge or understanding of what proportion of base area to utilize for such piles has lead to the use of extremely long piles in some instance since toe resistance valves used may have been conservative.

III. Objectives. There is a need to conduct research into assessing the toe resistance of steel H and pipe piles in a variety of soil types (with) both model tests and full-scale tests. This information can be assimilated with theoretical studies to perhaps develop guidelines of the practicing engineer that can be used with much more confidence than presently exists.

IV. Current Activities. Not known.

V. Urgency. 2

Problem Statement No. 22 (proposed by North Carolina DOT)

I. Name of Problem. Effect of Jetting on Lateral Stability of Driven Piles

II. The Problem. Many piles for bridges are driven with the help of jetting. Such foundations are always subjected to large material loads and must be designed for this loading situation.

III. Objectives. p-y relations for silty or sandy soils need to be developed such that existing models can be modified for jetting conditions.

IV. Current Activities. Not known.

V. Urgency. 1

Problem Statement No. 23 (proposed by Daniel O. Wong, McBride-Ratcliff and Associates)

I. Name of Problem. Axially Loaded Performance of Soil-Mixing Columns as Deep Foundation Elements.

II. The Problem. The soil mixing technique is gaining popularity in the U.S. The method is particularly attractive under contaminated soil conditions. Soil mixing columns have been used as deep foundation elements to support large loads. However, there is no sound design procedure for prediction of load capacity of such columns that results in a conservative design approach.

III. Objectives. Develop design methods for evaluation of compressive and uplift load capacities of soil-mixing columns as deep foundation elements. Also, develop an understanding of unit load transfer characteristics of such foundation elements.

IV. Current Activities. Not known

V. Urgency. 2

Problem Statement No. 24 (proposed by Daniel O. Wong, McBride-Ratcliff and Associates)

I. Name of Problem. Axial Unit Load Transfer Characteristic of Downdrag Loads.

II. The Problem. Downdrag loads are a concern for deep foundations supporting (abutments in) embankments or for (piles) embedded in soft materials. More understanding of the axial load transfer characteristics can lead to design with more confidence and certainty.

III. Objectives. Provide design unit load transfer values for downdrag considerations under various types of soil.

IV. Current Activities. Not known.

V. Urgency. 1

Problem Statement No. 25 (proposed by Daniel O. Wong, McBride-Ratcliff and Associates)

I. Name of Problem. Unit Lateral Load Transfer Behavior of (Augured) Piles and Pile Groups.

II. The Problem. Limited by their size and construction control, (augured) piles are not used as often as they (could) be, but the method is certainly gaining attention as more understanding is obtained. While Problem Statement No. 14

concerns load bearing characteristics of (augured) piles, the lateral load (behavior) of this type of deep foundation also needs more understanding.

III. Objectives. (Obtain) p-y relationship for (augured) piles in major soil types and assess group action under lateral loads.

IV. Current Activities. Not known.

V. Urgency. 2

Problem Statement No. 26 (proposed by James J. Brennan, Kansas DOT)

I. Name of Problem. Evaluation of Augured Piles for Load Bearing in Highway Structures (additional topics for Statement No. 14).

II. The Problem. Augured piles are difficult to reinforce with traditional reinforcing cages to resist lateral loads. In addition, the use of fly ash as a grouting agent in the (augured) pile mix design raises the possibility of alkali-silica reactions with opaline aggregates.

III. Objectives. Evaluate the potential for alkali-silica reactivity in grout mixes for (augured) piles that contain opaline aggregates and determine construction methods that allow the use of traditional reinforcing cages in (augured) piles.

IV. Current Activities. The Kansas DOT is presently considering undertaking research to evaluate the potential for alkali-silica reactivity in (augured) piles utilizing opaline aggregates.

V. Urgency. Not specified

Problem Statement No. 27 (proposed by Randy Ray Cannon, South Carolina DHPT)

I. Name of Problem. Accurate Estimate of Pile Tip Elevations at Existing Bridges.

II. The Problem. The FHWA requires a scour analysis for every bridge over water, even existing bridges. Many bridges in South Carolina are small, cheap bridges built by Maintenance forces on rural routes. Detailed plans were not made and detailed recorded were not kept for piles tip elevations, bearing, etc. Even with an accurate scour analysis, we do not know the depth of pile penetration and therefore cannot determine scour susceptibility.

III. Objectives. Develop an economical and accurate method of determining existing pile penetration at any bridge site. One method that has been suggested is using a device similar to the dynamic pile analyzer and inducing a shock wave into the pile to estimate pile lengths.

IV. Current Activities. According to our information the North Carolina DOT has done some work on using PDA's to estimate pile lengths; however, we have not contacted them about their research as of this time.

V. Urgency. ___1___

Problem Statement No. 28 (proposed by Randy Ray Cannon, South Carolina DHPT)

I. Name of Problem. Scourability of Rock, Decomposed Rock, and Residual Soils.

II. The Problem. The FHWA requires a scour analysis for every bridge over water. The only geotechnical property used in the current design methodology is the grain size of material that is subject to scour. In many locations scourable material overlays residual soils, decomposed rock or rock. There have been bridge foundation failures due to the scour of very dense materials or rock. The most conservative method would be to assume that the dense soil will scour as looser materials; however, this idea does not appear to be realistic, and very deep and expensive foundations can result from this assumption. There are no reliable estimates of the rate of scour of rock with very low (approaching 0 percent) RQD's and very low recovery rates.

III. Objectives. Recommend a scour rate based on RQD or a similar easily obtainable parameter for rock, and scour rate based on SPT for dense residual soils and decomposed rock. A new in-situ tool many need to be developed for very dense residual soils and decomposed rock.

IV. Current Activities. According to our information the FHWA has initiated a study of scour in rock.

V. Urgency. ___1___

Problem Statement No. 29 (proposed by Richard Sheffield)

I. Name of Problem. Determination of Critical Scour Depths at Hydraulic Crossings.

II. The Problem. Current FHWA and USGS guidelines determine scour depths around bridge foundations with regard to hydraulic properties of the stream and grain size of the soils. No allowance is made for the consistency (hardness) of the soils or geological characteristics of the formational soils. As a result, excessive scour depths have been determined for many bridge sites within a geological formation that has not eroded for years.

III. Objectives. Develop a workable guideline that can be used for scour-critical bridge sites that not only takes into account bridge geometry and hydraulic characteristics, but also geology and drainage area criteria.

IV. Current Activities. None indicated.

V. Urgency. ___1___

Problem Statement No. 30 (proposed by Bill Russman, Arizona DOT)

I. Name of Problem. Drilled Shafts or Piles in Highly Variable Subsurface Consisting of Hard Rock (Limestone or Sandstone) Intermingled With Soft Claystone or Stiff Clay.

II. The Problem. How should the shaft (or pile) be analyzed for axial and lateral resistance to determine the toe location? Also, since exploratory borings cannot be drilled everywhere, what assumptions can be made about the "rock" (i.e., assume everything is soft claystone?). What special provisions should be developed for construction - alterations in shaft length, contractor drilling in one type of rock when another was expected, claims (changed conditions), etc.

III. Objectives.

a. Review procedures used by various agencies.

b. Perform lateral load tests.

c. Develop specialized exploration techniques.

IV. Current Activities. None known.

V. Urgency. ___1___

Table A1
List of Respondents to Survey

Name of Respondent	Employment Category
Alampalli, S	4
Alexander, Douglas	1
Arcari, Donald J.	1
Benda, Christopher C.	1
Beuvie, Donald	5
Brennan, James J.	1
Briaud, J-L	6
Cannon, Randy Ray	7
Cheney, Richard	2
Cochran, David	1
Diyaljee, V. A.	1
Dunn, Philip	1
Elias, Victor	5
Fellenius, Bengt	6
Fennessy, Thomas	2
Fontaine, Leo	1
Fults, Kenneth W.	1
Geary, Georgene	1
Graham, James S.	7
Greer, Daryl	1
Hasan, Shafi	3
Ho, A.	1
Holder, Samuel T.	2
Houram, Nabil M.	1
Khan, Wasi	1
Lens, John	5
Mansfield, Jack	1
Mikkelson, Jorg	1
Munoz, Andy, Jr.	2
Nevels, James B., Jr.	1
O'Neill, Michael	6
O'Rourke, Patrick	1
Oberc, Edumund J.	1
Odom, George H.	1
Passe, Paul D.	1
Peters, Art	2
Prysock, Rod	1
Russman, Bill	1
Santo, Dennis D.	1
Sheffield, Richard H.	1

(Continued)

Table A1 (Concluded)

Name of Respondent	Employment Category
Thommen, G. R.	1
Walkinshaw, John L.	2
Wargo, Richard	5
Weaver, Monte	1
Wong, Daniel O.	5
Zandi, Firooz	1
Anon.	1
Anon.	3
Anon.	1
Anon.	1
Anon.	1
Anon.	1
Anon.	1
Anon.	1
Anon.	1
Anon.	1

Employment Categories:

1. State DOT geotechnical or materials engineer.
2. Federal DOT geotechnical or materials engineer.
3. State DOT bridge design engineer.
4. State DOT research engineer.
5. Consulting engineer (private sector).
6. University professor.
7. Other.

Topics Concerning Pile Driving Equipment and Pile-Soil Interaction

Don C. Warrington, P.E.[1]
Vulcan Iron Works, Inc.
Deep Foundations Institute

Introduction

This report summarizes the present situation of the current state of pile driving equipment in use as it relates to pile-soil interaction. The report reviews the current situation and offers suggestions for further action to advance the productivity of pile driving in general and the equipment in particular.

Background

Although the interaction of pile and soil is something not always considered by pile driving equipment manufacturers, its reality and the understanding of the reality are important factors in the application of the equipment and its efficacy in driving piles. The interaction of pile and soil is indeed the central question with driven piles. The entire purpose of driving piles is to increase the load capacity of the foundation by mobilizing more of the soil for load bearing purpose than spread footings or ring foundations. Since pile driving equipment is used in the installation of piles, we can concentrate on pile-soil interaction during installation.

The central role of pile driving equipment is to get the piles in the ground to the desired depth. However, from the middle of the last century onwards the role of the equipment has taken a dimension that is missing from that of, say an excavator in a spread footing. It is the role of a measuring device of the pile's bearing capacity. In its most elementary form, the hammer's energy output and other driving system factors combined with a measurement of the pile's reaction to driving are used to determine the bearing capacity of the pile. Historically the most widespread application of this has been the Engineering News Formula. This simple equation, with the more complicated Hiley and other formulae, established the impact pile driver as a simple measuring device for bearing capacity.

The central defect in this approach was the theory of dynamic formulae themselves. Especially with the development of longer concrete and steel piles, the pile could not be accurately modeled as a rigid mass. The soil modeling implicit in these models was rudimentary to say the least. It took the development of the electronic digital computer and the skillful application

[1] Vice Chairman, Vulcan Iron Works, Inc., 2403 Lennox Court, Chattanooga, TN 37421-1855.

of existing theory by E. A. L. Smith of Raymond to produce the tool that would overcome the main shortcomings of dynamic formulae -- the wave equation.

The later development of the wave equation, from a theoretical program into an accepted method of pile driving behavior, through its TTI and GRL developments and its transformation into a field instrument with CAPWAP and by TNO, is well documented. The wave equation has given the pile driving universe a reasonably accurate tool to determine both the driveability and capacity of piles, depending upon how the various methods are applied.

While these developments were taking place with impact hammers, the Russians were developing the vibratory and impact-vibration hammers. Vibratory hammers operate on a different principle from impact hammers. Instead of developing high peak forces and this pushing the pile past the soil, the vibratory hammers actually modify the soil properties below the weight of the system and thus allow the system to penetrate into the ground. The interaction of pile and soil is thus different from impact to vibratory driving and is in reality more complex. This has made establishing a method to relate the performance of the driving system with the bearing capacity of the pile a more difficult task with vibratory hammers than with impact ones. Analytical methods have been developed that take a variety of approaches to solving the problem, but none of these has progressed as far as those used now with impact hammers.

One more item that deserves mention is the existence of methods to determine pile capacity that do not rely to the method by which they are driven. Such methods are numerous and include those of Meyerhof, Nordlund, and Tomlinson. These methods are used widely in foundation design. Equipment independent methods are especially favored with offshore platform piling. The severe uplift experienced by these piles and the conditions under which they are driven puts both the analysis and the installation of these piles in classes by themselves.

Present Situation

Generally speaking, impact pile driving equipment for larger projects is first selected using a wave equation analysis, followed by a pile load test program with a pile driving analyzer. Once this phase of the project is complete, the hammer is certified. It is virtually impossible to substitute another piece of equipment. Often it is difficult to change any other part of the driving system, such as the cushion material. This has the effect of making a job "captive" to a certain piece of equipment, which can present problems if the equipment breaks down or more is needed to keep the job progress acceptable. A more serious problem arises if the equipment chosen is acceptable during the pile load test program but then encounters soil conditions that make it inadequate for the remaining piles.

With vibratory hammers, the most common way of employing equipment to assist in the determining bearing capacity is to drive the piles to nearly their desired depth and then finish the job with an impact hammer. Although this method is satisfactory, it requires an additional piece of equipment in the field. Another factor that complicates this situation is the strong evidence that impact driven piling and vibrated piling do not have the same bearing capacity (at least in the short term), even if the soil is identical. These effects, however, may be reduced or eliminated over time.

Conclusions and Recommendations

Although the role of the equipment in determining the bearing capacity of piles and thus their interaction with the soils has come a long way in the last century or so, there is still much improvement that is possible and indeed desirable, if pile foundations are to continue to be a productive and economical type of deep foundations. Since the issues for impact and vibratory equipment are different, they will be dealt with separately.

Impact hammers

a. Although the spring/dashpot model for soils has done well over the years, it needs to be reexamined in view of further developments in the modeling of soils, either towards refinement or replacement. This is especially true with the development of finite element methods, and it is also true for pile analyzers as well. The values for these parameters need to be tied more effectively to the soils being penetrated rather than the blanket values that have been employed since the beginning of the wave equation analysis.

b. The entire process of soil properties entry and use in the wave equation analysis needs to be automated as much as possible to reduce errors at the analysis level.

c. The methods by which hammers are certified need to be made more flexible to allow equipment substitutions during the job. It is ironic that the advance in technology and in the behavior of piles during driving has in some ways added to the rigidity of hammer selection and use. This could be effected by minimum energy or enthru requirements, or by more advanced pile analysis methods and equipment that would allow more confident substitution of hammers during the job.

d. Methods that would take equipment out of use as a measurement tool should be developed whenever possible. This would add to the flexibility of hammer selection and substitution. Although equipment has been used as a measuring tool for a long time, the variations in hammer performance due to design changes, lubrication, cushion material wear and other operating variations do not make pile drivers the optimum measuring device.

Vibratory hammers

a. A comprehensive model of soil response to piles vibrated by vibratory hammers needs to be developed at the first opportunity. This cannot be over-emphasized; the development of such understanding and its quantification is *absolutely essential* to using vibratory hammers to drive bearing piles, and for driveability studies. Once this has been done, then the development of analytical methods and their field correlation is possible. Too much research has concentrated on empirical or semi-empirical methods. While these have added to our understanding of vibratory pile driving, without a more general model the long term development of methods to predict pile driving performance during vibration will be limited. One possible source of information on this would be seismic research. The phenomena witnessed during vibratory pile driving are similar and in some cases the same as those seen during seismic activity.

b. Methods that are independent of the equipment itself for pile capacity are as desirable with vibratory equipment as they are with impact equipment. The variations between resistance or capacity for impact driven piles, for vibrated piles, or even for those which are jacked in the ground need to be understood thoroughly before this is carried out. If the capacity or resistance differences between vibrated and impact driven piles are reduced or eliminated over time, this needs to be established.

The Action of Soft Clay Along Friction Piles Bay Mud Revisited

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Introduction

Research at Berkeley more than three decades ago gave new insight into the behavior or axially-loaded piles in clay. Numerous experiments and analytical studies performed since, many of which have been supported by the oil industry with respect to offshore platforms, have added significantly to a better understanding of the problem. Yet, the prediction of the "real" behavior of such piles with effective-stress methods remains far beyond the present capabilities of geotechnical engineers.

A brief discussion is presented to elucidate the factors involved in the interaction between a pile and the clay with a view of establishing fundamental concepts. Models are described that serve as guidance to further research. Some results of studies at Berkeley and elsewhere are presented that are relevant to an improved understanding.

The thrust of the paper is to lay out the kinds of experiments that must be performed if the problem of an axially-loaded pile in clay is to be solved rationally. Further development of methods to predict the load versus settlement of a pile in soft clay must await the collection of a body of reliable data from field measurements.

The discussion that is presented is built around the behavior of a single pile for simplicity but the concepts presented apply to a group of closely-spaced piles. Also, for simplicity only the load transfer in side resistance is considered. End bearing is important, of course, but for a pile in soft clay the load carried in end bearing is frequently a small fraction of the load carried in side resistance.

Effects on Soil of Installing a Pile

Bored piles are not well adapted for use in soft clay so it can be reasonably assumed that the pile will be driven by an impact hammer. The interaction of an elastic pile and the inelastic soil under the last blow of the hammer will likely result in the distribution of a residual load nonuniformly along the length of the pile.

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Immediate effects of pile driving

The first effect is the displacement of a volume of soil nearly equal to the volume of the material that is inserted. Soil displacement around a driven pile has been discussed by a number of authors, for example by Zeevaert 1950 [1] and Hagerty and Peck 1971 [2]. The displacement of the soil during pile driving can cause uplift and lateral movement of piles previously driven [3]. Baligh 1985 [4] has proposed a method for obtaining the deformations around an inserted pile by assuming that the insertion was at a constant velocity, that the soil was incompressible, and that the results were independent of the constitutive relations of the soil. The ability to predict the deformations of the soil even for such a special case is useful; however, the solution to the general problem remains unsolved. For example, if a tubular pile is driven, it is not now possible to predict if, or at what depth, the pile will plug.

There will inevitably be some lateral vibration of the pile during driving because of the eccentricity of the application of the impact loading. The lateral deflection could be enough to cause the soil near the groundline to be pushed away from the pile and the axial capacity would be adversely affected.

The driving of a pile into soft clay causes the total pressure at the pile wall to increase above the at-rest pressure and the pore pressure to increase above the equilibrium pressure. Increases in the pressures around piles in clay have been observed at the pile wall and at several diameters from the pile wall [5] [6] [7] [8] [9] [10]. The increase in the pore pressure around a driven pile in clay is analogous to the increase in temperature in a slab due to the imposition of a line source of heat.

As a pile is driven past a particular point in the soil, shearing deformation of large magnitude will occur. Information is unavailable as to the conditions under which the sliding will occur at the pile wall or at some distance into the soil. However, Tomlinson reported that the excavation of driven piles revealed that soil from upper strata were moved downward into completely different strata [11] [12].

Effects of time with no axial loading

A considerable amount of time must elapse before excess pore pressure is substantially dissipated around a pile driven into a saturated, homogeneous clay. Along with the decrease in excess pore pressure is an increase in axial capacity. Figure 1 [13] [14] presents curves for eight experiments that show the rate of increase in axial capacity as a function of time. The curve for test B-1 in Figure 1 was developed from data on the decay of excess pore pressure and by making the assumption that the axial capacity of the small rod went up as the pore pressure went down [14]. As may be seen in the figure, some of the piles had not reached the full capacity for more than a month after installation.

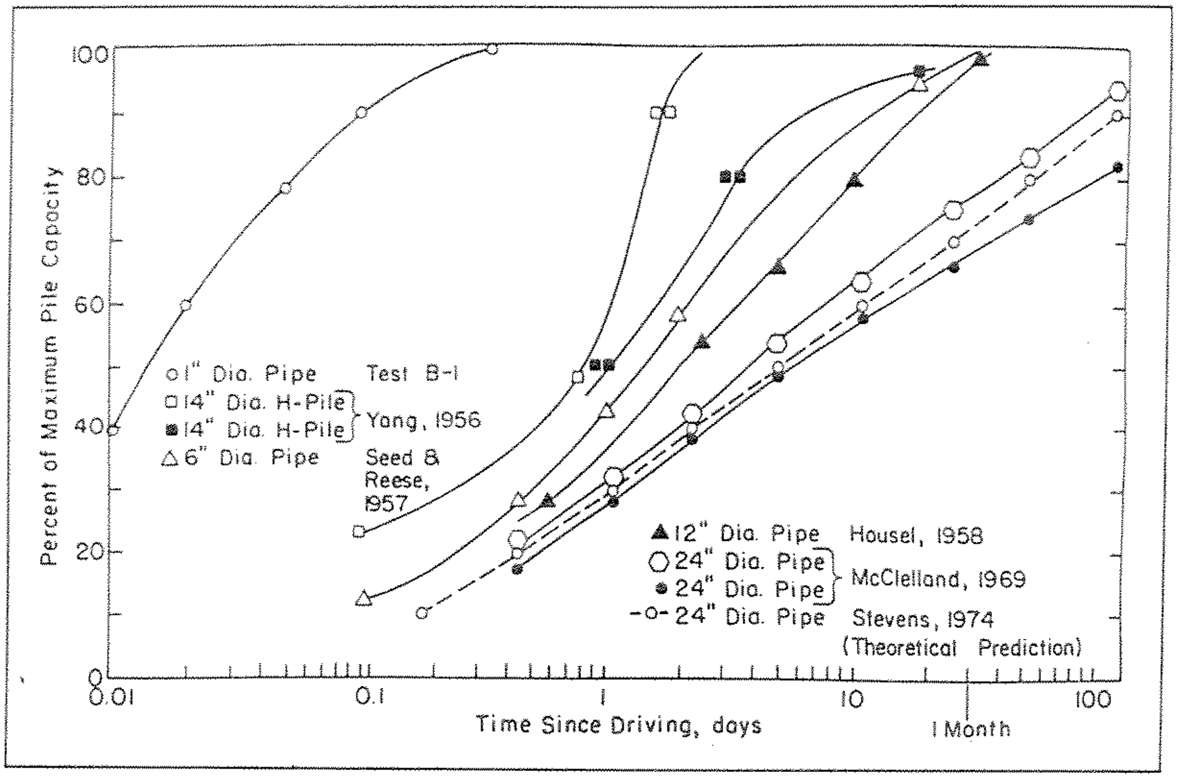

Figure 1. Pile diameter versus time to 50 percent consolidation (log-log plot) (after Grosch and Reese 1980) 1 in. = 25.4 mm

The data from Figure 1 were used to construct Figure 2 that shows a log-log plot of the time for 50 percent of final capacity as a function of pile diameter. Only the data from full-displacement piles were used in the plot. The results shown in Figure 2 are consistent with the concepts that the magnitude of the "bulb" of increased pore pressure is a function of the amount of soil that is displaced and that the capacity of a pile under axial loading is inversely related to the decay of excess pore pressure.

The state of stress in a homogeneous, saturated clay immediately after pile driving is complex. The excess pore pressure at a particular point on the pile wall probably reaches its maximum value just after the pile is driven. Then, there is an exponential decrease in pore pressure and a corresponding decrease in water content. The change in water content is significant [15] [6] and the shear strength of the clay at the pile becomes markedly higher than the strength just after the pile was driven. The physical process of the outward flow of water and the inward packing of soil grains is not well understood, and no theory has been proposed for predicting the state of stress and the properties of clay as a function of distance from the pile wall.

If a driven pile in saturated clay remains unloaded, there is some evidence to show that compressive stress develops in the pile. The remolding of the clay due to pile driving causes its consolidation characteristics to change so

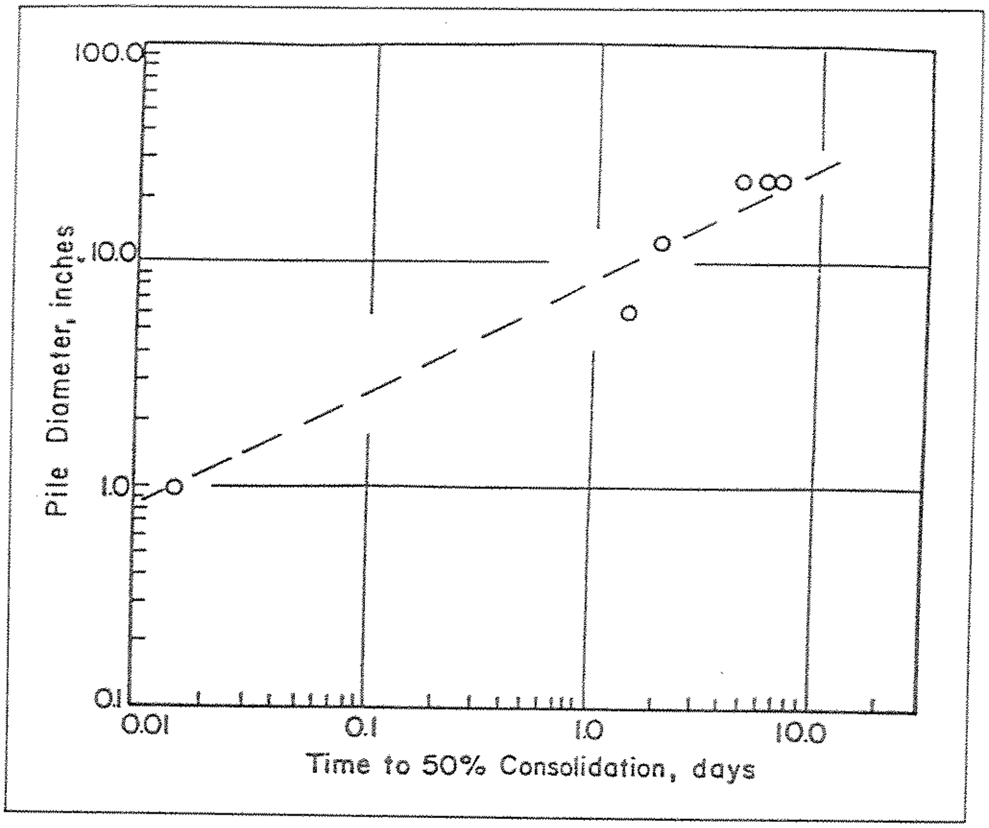

Figure 2. Pile diameter versus time to 50 percent consolidation (log-log plot) (after Grosch and Reese 1980) 1 in. = 25.4 cm

that it will settle under self weight. Thus, some downdrag will result with a consequent change in the residual stresses that existed immediately after pile driving [1] [16].

Effects of loading

The axial load on a pile will cause an increase in the stress in the pore water that leads to consolidation and settlement. For most offshore structures, the settlement due to consolidation is of little or no consequence. The increase in shear strength of the clay as a result of consolidation may or may not be significant in terms of the additional capacity of a pile.

The cyclic, axial loading of a pile could result in an increase in the pore pressures so that the axial capacity would be affected during a storm [17] [14].

Summary

The soft clay around a driven pile experiences drastic time-related changes that are dependent on the geometry of the pile, the roughness of the pile wall, the method of installation, the distance from the pile wall, the stratigraphy at the site, and the properties of the clay. The result is that the clay that controls the axial capacity of the pile in side resistance will have properties that are quite different from the in situ properties of the soil.

Models for Computing Behavior in Side Resistance Pile

A wide variety of types of piles can be driven into soft clay where the variables are the geometry of the pile and the material of which it is made. In all cases the pile will deform under axial load and the deformation will usually be elastic or nearly so.

For the purposes of this discussion, the assumption will be made that load transfer from pile to soil will occur at or near a pile-soil interface that is flat or curved, and it is convenient in the presentation to consider a cylindrical pile. Tapered piles and H-piles are eliminated from consideration, although many of the concepts that are developed will apply generally.

Soft clay

A section from a driven pile is shown in Figure 3. Elements are shown at the pile wall at Points (a) and (b) and elements are shown radially away from the pile wall. If it can be assumed that the properties of the clay were perfectly uniform prior to pile driving, the properties of each of these elements will be different. Furthermore, the state of stress will differ from the in situ state. An element of the clay at some radial distance from the pile wall will retain its in situ properties and state of stress.

An element of the clay at the interface of the pile is shown in Figures 4; a generalized state of stress is shown on the face next to the pile. The three stresses that are indicated are all related to time. The normal stress is a function of the in situ soil properties, the in situ state of stress, and the effects of driving the pile. The normal stress could be zero or nearly so near the ground surface, especially if the pile had experienced lateral deflection during placement.

The shearing stress in the vertical direction is due to the residual stresses, if any; the possible effects of downdrag; and the loads that are applied. The shearing stress in the horizontal direction is probably very small and may be due to variation in soil properties and the bending of the pile during driving.

A failure surface, not at the interface of the pile and the soil but in the soil at some distance from the wall of the pile, is shown in Figures 3 and 4. A conceptual approach to the location of the failure surface at some point along the length of a pile is shown in Figure 5. The shearing resistance of the clay, determined either by the undrained-strength approach or by the effective-stress approach, is shown as a curved line extending from the wall of the pile. Field

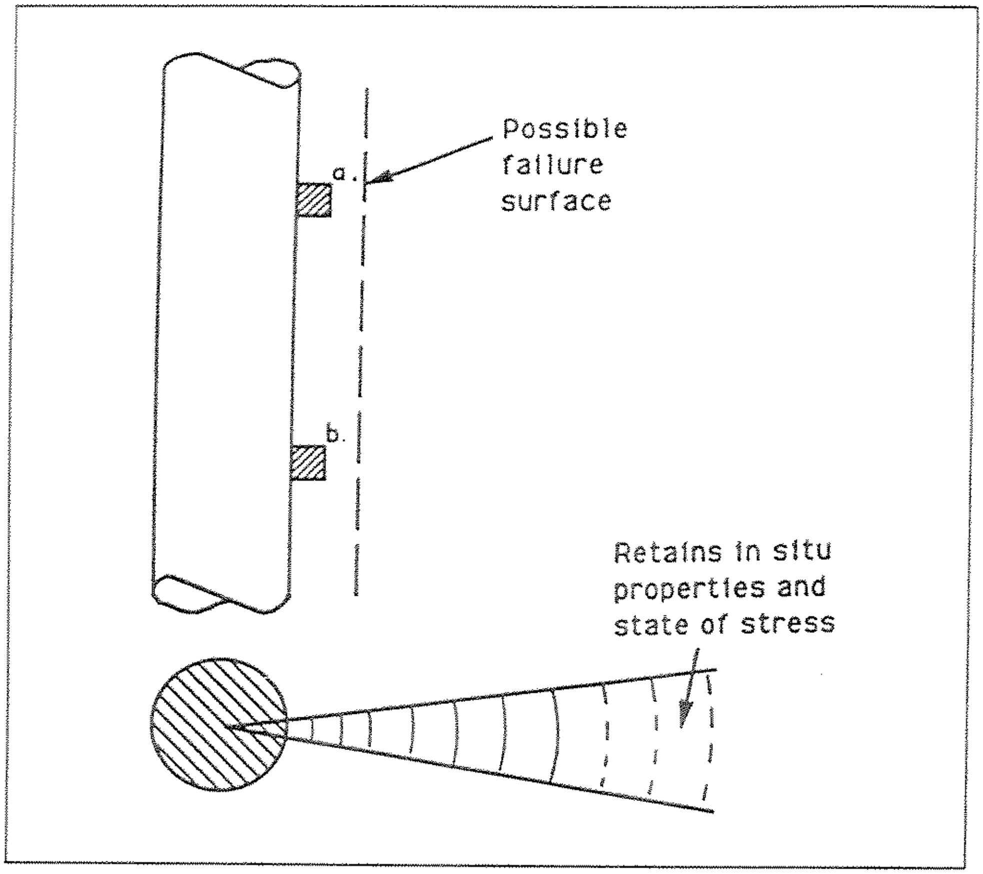

Figure 3. Variation of state of stress and soil properties around a driven pile

measurements, some of which are mentioned later, show that the water content of the clay is less at the pile wall than the in situ water content. Furthermore, piles in clay that are recovered have a layer of clay clinging to the wall of the pile. If soil particles move toward the pile while excess pore pressures are dissipated, it follows that water molecules must move outward; thus, at some distance from the interface, the water content could be greater than the in situ value and the strength would be less than the in situ value.

A point at the wall of the pile is shown in Figure 5 to represent the shearing resistance at the interface. This point is plotted below the strength of the soil at that point but it could possibly be above the soil strength if the wall of the pile is rough. Also, there could be a chemical reaction between the clay and the material of the pile that could cause a strong resistance at the interface.

The dashed line in Figure 5 shows the applied stress, and that stress will decrease with distance from the wall of the pile. At some distance from the pile the applied stress will equal the shearing resistance of the clay and sliding will tend to occur when the pile is fully loaded. The thickness of the clay layer that will move with the pile is indicated in the figure. That layer of clay in effect enlarges the cross-sectional area of the pile. However, it is unlikely

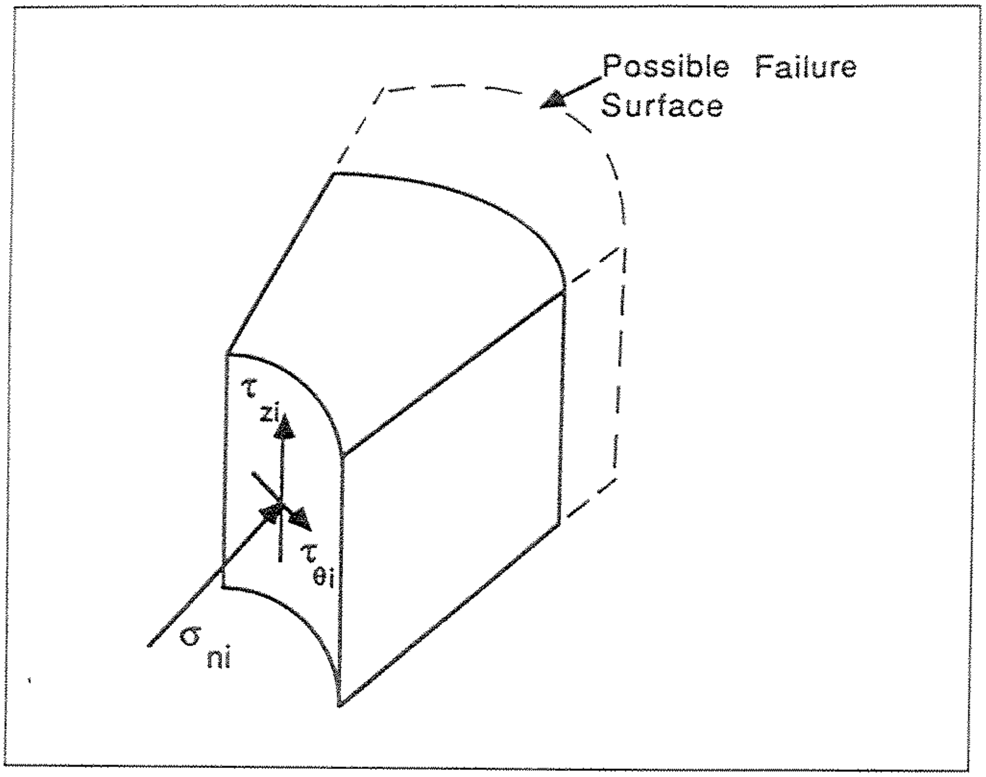

Figure 4. Generalized state of stress at interface of pile and soil

that the distance to the potential sliding surface will be the same at all points along the pile so that the actual thickness of the clinging layer would involve consideration of bearing stresses and kinematics.

The curve given in Figure 5 conceptually could represent the undrained strength of the clay; however, the use of effective stresses is more satisfactory and obligatory if the true behavior of the pile is found. Thus, the parameters that define the strength of the clay must be ascertained on surfaces, millimeter by millimeter, from the pile wall. These parameters will be related to the in situ parameters as influenced by the driving of the pile. Also, the effective stresses must be found at the pile wall and out into the continuum.

Interaction between pile and soil

Many procedures for piles in clay are directed only at the computation of the ultimate capacity under short-term loading at the time when pore pressures are fully dissipated. However, it is desirable to be able to predict the load versus settlement and the distribution of load along a pile for any time after a pile has been driven. Such predictions can be made by utilizing the model shown in Figure 6.

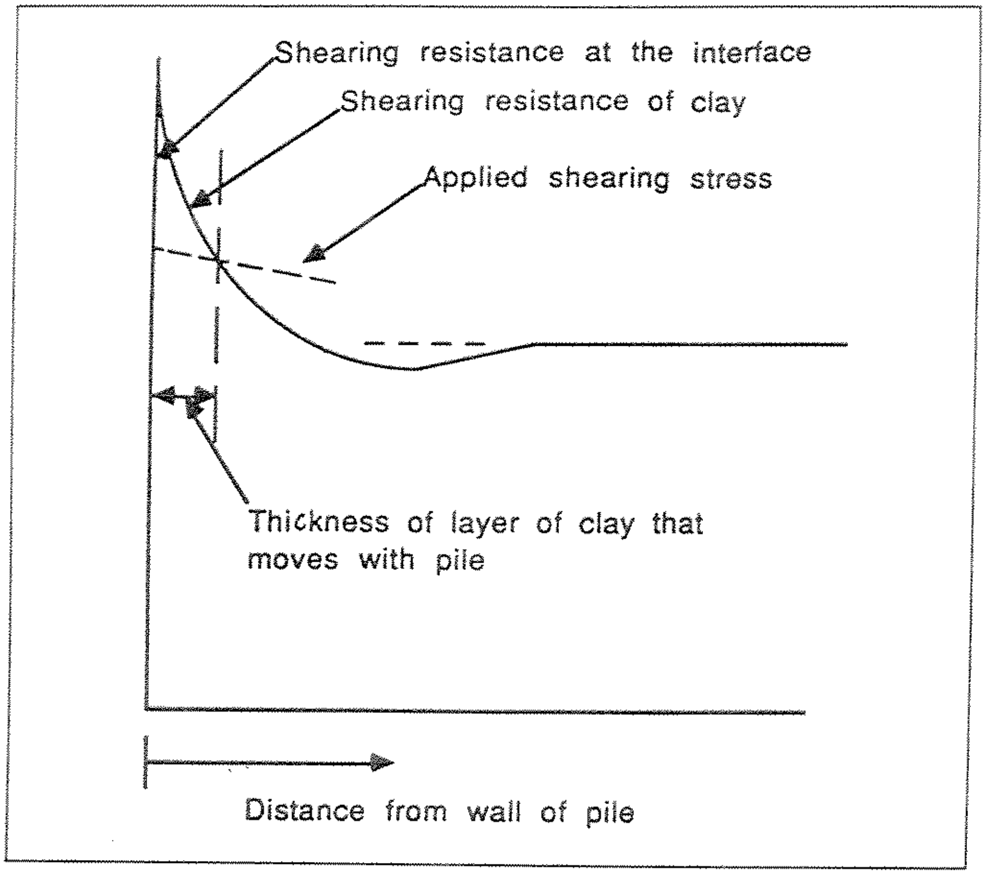

Figure 5. Conceptual curves for locating position of failure surface

The sketch in Figure 6 shows the pile as a deformable body; the soil has been replaced by mechanisms that merely indicate that the unit load transfer in skin friction is a nonlinear function of the movement of the pile. The unit load transfer is characterized by a family of s-z curves (sometimes called t-z curves), where s is the load transfer in side resistance at a point along the pile and z is the movement of a point on the pile relative to the position of the point prior to loading. The sketch indicates that the movement of the pile head under a given load requires the solution of a nonlinear differential equation because the movement of the pile and the load transfer are mutually dependent. The concept of load-transfer curves as a means of explaining the interaction was presented by Seed and Reese 1957 [6] and developed further by Reese 1964 [18] and Coyle and Reese 1966 [19].

The model can be criticized because single-valued curves, as shown in the sketch, cannot be used to characterize the behavior of a continuum. However, the curves that are reported in technical literature have been obtained from experiments with instrumented piles where the continuum effect was satisfied. Furthermore, the solution for the behavior of a pile could proceed equally well if multi-valued curves, including the influence of the continuum, could be predicted for various points along a pile.

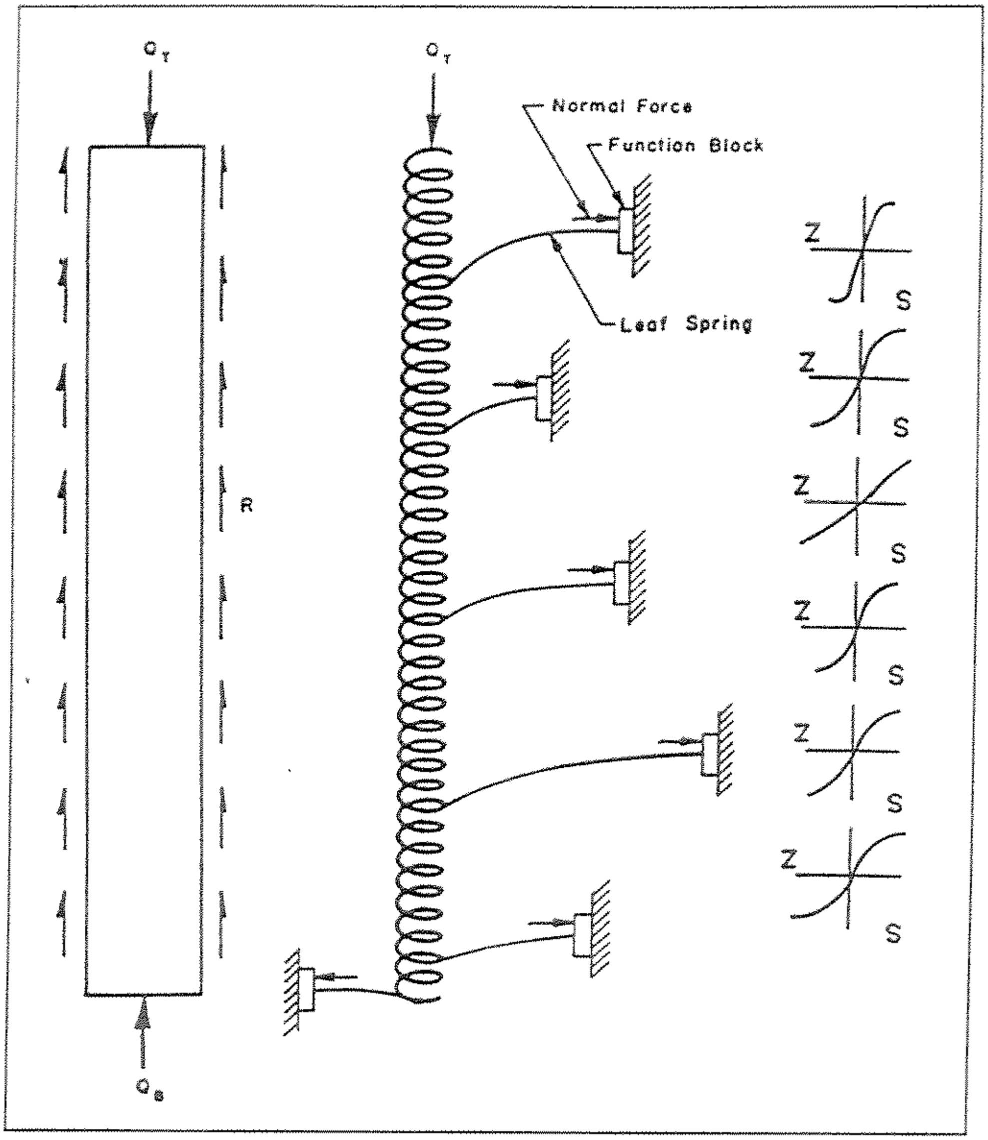

Figure 6. Model of axially loaded pile (after Reese 1964)

Other investigators have proposed formulations for load-transfer curves for piles in clay since the earlier studies that were cited; for example, Kraft et al. 1981 [20] and Vijayvergiya 1977 [21]. The method is being used in practice, but the number of tests with instrumented, full-scale piles is insufficient to allow such predictions to be developed with confidence.

The load transfer at some time after the installation of a pile could be represented by a family of curves that are movement-softening such as indicated by the Points A, B, and C in Figure 7. Then, the ultimate capacity, and the load-settlement curve, must be computed in consideration of the axial stiffness of the pile. Because of the deformation of the pile under load, the pile movement and load transfer near the bottom could be represented by Point A, while Point B could represent the soil response at the midheight, and

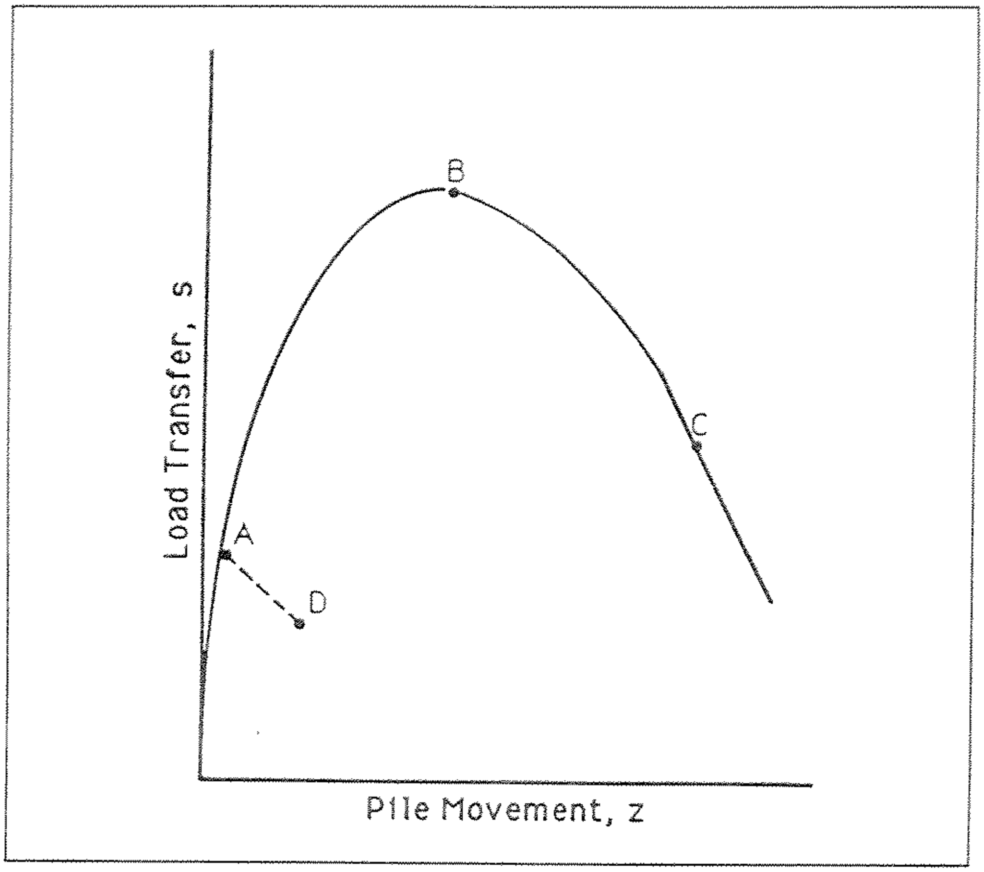

Figure 7. Hypothetical movement-softening load-transfer curve

Point C could represent the soil response near the top. Thus, the solution of a differential equation would be required if soil response is represented by curves such in Figure 7.

If a pile in soft clay is subjected to sustained loading such that Point A in Figure 7 gives the load transfer and pile movement, consolidation and creep could cause a displacement to Point D. Research on the behavior of piles in soft clay can lead to the ability to predict load-transfer curves as a function of time and loading on a pile. Thus, the complete response of a pile to axial loading at any time after installation could be found analytically.

Summary

The models describe a system that can explain the action of soft clay along an axially-loaded pile. While the system is complex in terms of many that are in current use, researchers currently have the capability to gain data to implement the models. Such research is necessary if significant advances are to be made in predicting the response of piles in soft clay.

Some Pertinent Field Measurements

The influences of the driving of piles on soft clay are so dominant that laboratory studies are of limited benefit. Furthermore, the amount of field research in the last several decades is so large that it is impossible to review all of it. Therefore, starting with the work of Seed and Reese 1957 [6] and Reese and Seed 1955 [5] (called the Berkeley studies for convenience), a few studies are described briefly. Useful studies have been made with probes that are inserted into clay [14] [22] but only those results from driven piles are included below. The nature of the research is indicated that will further the understanding of the subject problem.

The Norwegian Geotechnical Institute carried out a noteworthy set of experiments at Haga with a fully instrumented, cylindrical pile that was 5 m in length and with a diameter of 153 mm [23]. The tests were carried out over a period of 4 yrs and data were acquired concerning the behavior of both pile and supporting soil.

Total and pore pressures

The pipe pile (152 mm in diameter) installed at Berkeley contained gauges that measured pressures at six positions along its length. Significant differences were shown between the results of measurements of pore pressures and total pressures, indicating that there were effective stresses at the pile wall during the driving. There was an exponential decrease in the pressures with time and data, taken at the end of the test period of 800 hr, showing that good agreement was obtained between the measured pore pressures and the computed hydrostatic pressures. The pore pressures were subtracted from the residual total pressures and the result was divided by the computed effective overburden pressure; the values ranged from 0.24 to 0.54. Pressures were measured at nearby piles when other piles were driven in the vicinity with interesting results.

The Haga experiments revealed that the maximum values of total pressure and pore pressure were almost equal and that these values varied almost linearly with depth. The residual values of the total pressure varied almost linearly with depth and was about one-fourth of the values measured immediately after installation. The residual pore pressure was very close to the hydrostatic over the length of the pile. Data on the decay of the excess total and pore pressures were not presented in the 1986 paper.

Water content at wall of piles

Samples were taken close to the walls of piles that were driven in the Berkeley experiments and the orientation was carefully marked so that the portion of the sample next to the wall of a pile could be identified. The results showed that the average values of water content were 48.1 percent prior to pile driving, 43.6 percent to wall of pile one day after pile driving, and 41.1 percent next to wall of pile 30 days after pile driving.

The researchers at Haga made an excavation near the wall of the 153 mm pile and obtained block samples for study. The water content within a zone of about 16 mm from the pile wall was about 23.6 percent while the water content of the undisturbed soil ranged from about 48 percent to 59 percent. These remarkable studies showed that the change in water content and soil distortion occurred principally in a zone that was about 200 mm from the pile wall.

Studies with a timber pile with a length of 13.1 m and an average diameter of 250 mm were described by Eide et al. 1961 [24]. At the conclusion of loading tests, samples were taken to a depth of 9 to 10 m that skimmed the pile wall. Distortion in the layers of soil was noticed to a distance of 60 mm from the pile wall and the average moisture content in the "reconsolidated" zone was found to be 24 percent. The average moisture content of the natural soil at the same depth was found to be 35 percent.

Axial capacity as a function of time

The test pile in the Berkeley studies was loaded the first time at 3 hrs after pile driving and then at 1, 3, 7, 14, 23, and 33 days after installation. At a particular time after driving, one of the piles at the site was loaded to failure three times in quick succession with almost identical load-settlement curves, indicating that the previous loading had little if any effect on subsequent loadings. The plotted results from the tests showed a rapid increase in capacity with time during the early hours with the full capacity being achieved at about 23 days after installation.

The results from the Haga experiments showed that preloading had a significant effect on subsequent loading; thus, different piles had to be tested in order to obtain information on increase in capacity with time. The researchers found that capacity increased almost linearly with time from about 60 kN at 1 week after installation to about 74 kN at 5 weeks after installation. The authors postulated that the increase occurred due to the increase in the effective angle of internal friction near the pile wall as well as to the increase in effective stress.

The timber pile tested by Eide et al. 1961 [24] was subjected to both short-term loading and sustained loading. The first test was 3 days after installation and the final test was 799 days after installation. As observed by others, the pile showed a sharp increase in capacity during the early days but some increase was shown after the pile had been in place for three months.

Four steel pipe piles (356 mm in diameter) were tested by Cox et al. 1979 [25] in under-to-normally consolidated clays. The piles were driven through casings so that strata of soil at different depth below the ground surface could be investigated and tested both in tension and compression. The first tests were a few days after driving and other tests were done after more than 300 days after driving. Some significant increases in pile capacity with time were noted.

A pipe pile (762 mm in diameter) was driven into silty clay to a depth of 80.2 m below the ground surface; the top 57.9 m of the pile was isolated from the soil by a casing [26]. The authors report a four-fold increase in pile capacity with time. Increases of 9 percent to 32 percent in capacity within a few hours after a previous test were attributed to the dissipation of shear-induced pore pressures.

Load-distribution curves

Data from the Berkeley tests showed that little of the load was carried in end bearing and that the load transfer in the top few diameters was much less than that over the lower portion of the pile.

Cox et al. 1979 [25] obtained data that showed load transfer increasing slightly with depth. For one of the piles with 15.2 m of length in contact with the soil, only 11 percent of the load was carried in end bearing.

Load-transfer curves

Results from vane-shear tests at the Berkeley site were used to predict curves showing the transfer of load in skin friction as a function of pile movement. The distribution of load with depth, considering the elastic shortening of the pile, was predicted reasonably well.

Cox et al. 1979 [25] obtained a number of such curves and most showed that the maximum load transfer occurred at a pile movement of 10 mm or less. The curves remained approximately constant with additional pile movement or showed some slight decrease.

Comments on requirements of methods of investigation

Investigation of the behavior of a pile driven into clay presents some formidable challenges. Instrumentation must have appropriate sensitivity and reliability not only during the installation but with time after installation.

With regard to the measurement of the axial loading with depth, instrumentation installed prior to driving must survive the dynamic loading and stresses. Measurements must be made at more than one point around the circumference in order to eliminate the effects of bending moment from eccentric loading. The instrumentation should not experience a zero-shift during driving so that the residual loading in the pile can be found.

With regard to the measurement of total or pore pressures, the rapid rise of these pressures during the installation of pile presents challenges. In the case of total pressures, a system is required that allows no deflection; otherwise, arching would occur around the gauge. A similar nulling method is required for pore pressures because the drainage of a few cubic millimeters of water could cause undesirable inaccuracies. A fundamental difficulty in measuring the pressures along a driven pile is that the maximum pressures are undoubtedly generated when the pile first penetrates the soil at a particular depth; then, as the pile is driven farther, the pressures at that depth would change with time. Thus, not only is instrumentation required that will eliminate the inward movement of soil or water, but pressures are needed at several points along a pile during the driving.

The measurement of water content and other properties of the soil around the pile is of considerable importance. The technique of opening an excavation along the pile so that samples can be taken perpendicular to the pile wall is an ideal solution but is not always feasible. Arrangements can be made to sample from the interior of a closed-end pipe pile and in other cases special arrangements can be made to allow vertical sampling next to the wall of the pile.

The angle of internal friction (f) of the clay probably changes millimeter by millimeter from the wall of the pile outward. Assuming that specimens of the clay of acceptable quality can be obtained, a technique is required that can obtain these values of ϕ. The development of such a technique would appear to be a formidable challenge.

Concluding Comments

The understanding of the response of a pile in soft clay to loading awaits the performance of a series of field tests that provide data on a number of relevant parameters. With such data at hand, analytical tools are available that allow the development of methods of prediction. In addition to the Berkeley researchers who employed the theory of the diffusion of heat [27] [28], a number of investigators have proposed methods of predicting the interaction of a pile and the supporting clay [29] [30] [31] [4] [32] [33] [34] [35] [36]. Each of the methods makes a contribution to the eventual solution to the behavior of piles in soft clay under axial loading but each of them suffers from a lack of high-quality data.

The relatively recent high-quality experiments at Haga and the earlier ones at Berkeley, while somewhat crude by today's standards, are examples of those that are necessary before predictions can be much improved. But because the response of clay to the driving and loading of a pile is plainly related to the kinds of soil profiles and pile geometry, those tests only provide "points of light" in a large matrix.

The scope of the problem of gaining the amount of data that will be required to allow rational predictions is large and may seem overwhelming.

However, major construction projects could easily support the cost of the necessary instrumentation and the collection of data that allow progress. The writer closes the paper, as done in the past, with a quote from the late Karl Terzaghi [37] that remains fresh and pertinent: "Our theories will be superseded by better ones, but the results of conscientious observations in the field will remain as a permanent asset of inestimable values to our profession."

Acknowledgment

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3 Reports of Working Groups

Introduction

Session Four of the workshop allowed members to report on topics and ideas generated during the working sessions. A summary of the report of each working group is given below. Where known, the originator or principal developer of suggested research topics is shown in parentheses after the recommendation.

Group 1 - Research Needs for Piles in Specific or Special Types of Soils

Overview: Discussions of this working group centered around the effects of piles on specific types of soils (e.g., cemented sands, stiff clays) and the possible effects of driving piles into or through special types of soils (e.g., contaminated soils). Both field and laboratory investigations were proposed as well as modeling and sampling techniques to investigate the effects of piles on specific types of soils. Research suggested to address this problem is below.

Participants: L. Reese, Chairman
D. Bogard
D. Lourie
R. Olsen
J. Pestana
F. Townsend

Subject: Contaminated soils.

Problem: Punching through soils in the process of installation promotes deeper contamination by mixing layers of material. A conduit for fluid transport of dissolved contaminants is also created.

Research Needs: The type of pile installed has a significant effect on the magnitude of the problem. Simulation techniques to determine how a contaminant spreads in specific punched soil profiles could be an answer. Centrifuge testing to determine how contaminants are transported in punched soil would be another valuable tool.

Recommendation: Research guidelines and coordination of research are needed. An agency to provide a review mechanism and dissemination of results/information should be designated. (L. Reese)

Subject: Modeling pile-soil interaction by the finite element method (FEM).

Problem: Effects of pile driving hammer impact on clay must be studied.

Research Needs: An FEM code has been developed at UT, Austin. This code should be completed and adjusted to study the effects of the state of stress around the pile on the surrounding soil. Furthermore, the code results need to be compared with instrumented tests and adjusted. Such comparison would serve to establish guidance for in situ testing techniques.

Recommendation: Review existing FEM code. Investigate effects of pile driving on the properties of clay and incorporate in the code. Apply to a case where the site was extensively tested and compare experimental and FEM simulated data. (L. Reese)

Subject: Cemented, calcareous sand, loess, silts, soft layered soils, and limerock.

Problem: Heterogeneity of soil is not reflected in design. Soils are not "clay" or "sand" but exist in a spectrum. The properties of soils change in the process of driving, but mechanism of the change is not understood. End bearing capacity I soft rock cannot be estimated. Mixing of soils in layered systems has not been studied to determine the contribution that this phenomenon makes to changing soil properties.

Research Needs: Develop sampling methods for soils prior, during, and after driving of piles. Study a possibility of soil liquefaction resulting from pile driving. Study breakdown of cemented sands. Investigate process of mixing affecting t/z curves.

Recommendation: Carry out numerical simulation for changes in stress/strain in various phases of driving to determine its potential effect of soil properties. Continue with special soil laboratory tests on models, with field tests following these tests. The field tests must be accompanied by in situ sampling. Results should help new design procedures for specific and special soils. (F. Townsend)

Subject: Special sampling techniques.

Problem: Soil properties near the pile shaft are not verified. Reconsolidation and shear in axial loading in soils near the pile are not fully interpreted.

Research Needs: Study effects of installation on the properties of soil I zones adjacent to pile skin. Investigate reconsolidation effects and setup process.

Recommendation: Develop special sampling tools for horizontal radial samples. (D. Bogard)

Subject: Clay properties near the pile wall.

Problem: Soil disturbed during pile driving is not investigated by in situ sampling.

Research Needs: Obtain horizontal samples.

Recommendation: Develop a latched-in access door to obtain samples after the pile is positioned in soil. Employ scanning electron microscope (SEM) and X-ray techniques to analyze radial variation in clay. (D. Bogard)

Subject: Sand properties near pile wall.

Problem: Soil is disturbed by pile driving. Radial variation in strain effect resulting in redistribution of grain size and density is not known.

Research Needs: Obtain undisturbed in situ samples after pile driving in sand.

Recommendation: Study application of epoxy solidifiers and freezing technique for undisturbed sampling. (D. Bogard).

Subject: In situ testing of soil properties.

Problem: There is no program for evaluating in situ testing techniques for deep foundations.

Research Needs: Develop a program to evaluate in situ techniques and their usefulness in obtaining relevant information about soil properties adjacent to piles. Stress changes due to device installation (cone penetration, plate contact) and the relationship between idealized and actual conditions should be taken into account. Modeling the stress-strain-strength relationships of soils, especially sand, would be helpful. Comparison between analytical models and large-scale prototype experiment results should be undertaken. Assessment of the stress history of soil element during pile installation based on measurement by piezocone and pressuremeter, by spectral analysis surface wave (SASW) characterization, nuclear technology, or resistivity measurement should be considered in such a program. Assessment of stress history of soil based on in situ instruments, including pore pressure, stiffness, and stress (stress cells) is central for the solution of this problem.

Recommendation: Determine optimal location for built-in instruments and driven instruments. Develop high-resolution SASW for density changes in macrostructure. Develop and improve constitutive models accounting for changes in mechanical properties of materials as a function of stress level and density (for sands) and drainage condition (for clays). Implement and/or validate device-specific measurement of soil characteristics compatible with computational model(s) and numerical techniques. Well documented case histories are necessary for feed-back. (J. Pestana)

Group 2 - Laboratory and Small-Scale Test Research Needs for Piles

Overview: Research involving full-scale driving and testing can be slow and expensive, but laboratory and small-scale tests offer the prospect of economical research results obtained under controlled conditions. This working group endeavored to formulate research topics which would address the pile-soil property problem from the laboratory perspective. Topics developed are outlined below.

Participants: A. Whittle, Chairman
C. Ealy
W. Isenhower
R. Olson
W.M. O'Neill
R. Mosher
R. Peterson

Subject: Suction piles.

Problem: There is no database or consistent records for full-scale suction piles. Only a small number of scaled-down tests to offer design confidence.

Research Needs: Full-scale tests are too expensive due to the type and application of this type of pile. Extensive small-scale tests are necessary. Needed are tests on sand where piping may occur; and tests on clays where consolidation and setup may occur. The effect of cyclic and tensile loading unclear and should also be investigated.

Recommendation: Develop an instrumented model of suction piles with possibility of controlling and monitoring stresses at the wall interface with soil on both sand and clay. Derive a theoretical model for further design application. (R. Olsen)

Subject: Open-ended piles.

Problem: Plugging of open-ended piles.

Research Needs: An understanding of the mechanism of plugging is needed. Soil properties outside and inside a plugged pile are not known and are possibly not same. Pile capacity under the plugged condition as a function of time should be investigated.

Recommendations: Develop instrumentation for measurement of total, radial, vertical, and tangential stresses and pore pressures in soil adjacent to a plugged pile with emphasis on stability and miniaturization. Model tests in a stress chamber should assess effects of the installation mode (jacking, impact, vibration) on soil properties and pile behavior. Test in a centrifuge the efficiencies of pile and pile groups in each installation mode. Evaluate experimental models analytically to provide design procedures. (M. O'Neill)

Subject: Drilled shafts.

Problem: Group efficiency of drilled shafts is not well understood. Drilled shafts in stiff clays and tills present special problems. Off-center striking of piles can cause variation in pile behavior and probably in soil properties. Structural integrity of the drilled shafts is difficult to verify.

Research Needs: Field testing with emphasis on high-quality site characterization is needed. Information is scattered and no consistent database exists.

Recommendation: Accumulate detailed and consistent data integrating test results, high-quality site characterization and construction characteristics. (M. O'Neill)

Subject: H-Piles.

Problem: Bearing capacity of H-piles in sand.

Research Needs: Perform model tests in a stress cell and/or a centrifuge. Model pile must be instrumented as well as the soil. The tests should include pull-out capacity. The soil should be excavated and the deformations assessed. Alternatively, X-ray technique should be applied for examination of deformations. (C. Ealy)

Subject: Performance of piles in intermediate soils.

Problem: Soil properties change in the process of pile driving and are not known. Wall friction and end bearing capacity design is based on undisturbed soil properties, which are not properties of the soils supporting the piles.

Research Needs: Monotonic and cyclic load tests on piles in instrumented soils in full scale would be desirable. Determination and selection of engineering properties of soil/pile controlling the actual design parameters could be accomplished. Determine the relationship (distribution and its functional parameters) of wall friction and end bearing capacity based on soil properties determined.

Recommendation: Model in laboratory so as to determine theoretical pile behavior in silty clays and clayey silts. Conduct small-scale field tests on a national test site. Conduct full-scale test and validate the numerical model from the tests results. (R. Petersen)

Subject: Fundamental behavior of piles in sand under controlled conditions.

Problem: Sands change properties during pile driving and the effects of the changes on the bearing capacity of the pile are not known.

Research Needs: Identify change of sol properties in the process of pile driving. Quantify the interaction response near the tip of the pile in soils. Measure stress within near field around the pile to calibrate further models.

Recommendation: Conduct a simple stress test in a stress cell to explain process of densification and stress changes around the pile. Develop new instrumentation schemes to measure soil properties. (R. Mosher)

Subject: Cyclic loading.

Problem: Pile capacity under combined axial and lateral loading in cyclic conditions is not known. Little data are available on the subject.

Research Needs: Investigate the rate effect and material properties under cyclic loading through laboratory testing.

Recommendation: Conduct model tests both in stress cell and centrifuge with combined axial and lateral loading.

Group 3 - Full-Scale Research Needs for Piles Including Instrumentation and Equipment Concerns

Overview: Direct determination of the effects of piles on soil properties would necessitate full-scale instrumentation of a pile-soil system. Discussionss of this working group addressed the possibilities such full-scale research. Recommendations are summarized below.

Participants: Jean-Louis Briaud, Chairman
D. Warrington
R. Peterson

R. Boggese
R. Mosher
M.W. O'Neill

Subject: Pile instrumentation.

Problem: Instrumentation on piles and in soil is not scientifically reliable.

Research Needs: Identify the best existing instruments. Develop new instruments if needed.

Recommendation: Write recommended practice for instrumentation of piles (dynamic, static loading for pile and soil). Design and build new instruments.

Subject: Instrumented single pile.

Problem: The fundamental mechanism of soil structure interaction is not fully understood.

Research Needs: Collection of detailed quality data on full-scale mechanics of a single pile.

Recommendation: Install (driven, bored, vibrated) heavily instrumented piles in an instrumented soil. Investigate possibility of reuse of the piles and instrumentation. Investigate effect of lateral load and variability of soil condition.

Subject: Pile groups.

Problem: Piles are mostly driven in groups, but there are little data on full-scale pile group action.

Research Needs: Gather quality and detailed data on small- and full-scale pile groups.

Recommendation: Install heavily instrumented pile groups (driven, bored, vibrated) in an instrumented soil mass and test; interpret test data. Investigate possibility of reusing piles and instrumentation. Study conditions of variability.

Subject: Large structures on deep foundations.

Problem: Actual behavior of large pile supported structures is not well known.

Research Needs: Collect quality data on the deep foundations of locks, dams, platforms, and bridges.

Recommendation: Instrument the foundations of large structures for settlement and minimum load and moment at the top of piles. Develop and train intervention team in charge of instrumentation. Create another independent team for data analysis and data base formation.

Group 4 - Specialized Types of Piles and Other Deep Foundation Elements

Overview: This working group adopted as its purview auger cast piles, jet grouted piles, suction piles, helical anchors, pin piles, bottom-loaded piles, Franki piles, and stone columns. It was generally agreed that installation effects, integrity, structural properties, group effects, and loading effects were concerns common to almost all specialized types of piles.

Participants: M. Holloway, Chairman
W. Isenhower
D. Lourie
J. Andersen

Problems: Because these pile types are not very common, they generally represent unproven methods and applications. Guidelines for design and construction are often not universally available. There are (often) no national, private industry, or Government standards for deep foundations of this kind. Verification of the structural integrity of auger cast, jet grouted, and pin piles is difficult, often inconclusive, and often not done. Installation effects of any of the piles in this group are not entirely researched. Group effects have been studied on Franki piles but have not been studied on other specialized types of piles. Effects of loading and unloading, rate of loading, as well as possible lateral loading have not been thoroughly investigated. The instrumentation of special types of piles is difficult or not always possible. Small-scale modeling does not offer a primary value. Numerical modeling could have some value if all factors were included and if the modeling was followed by a prototype-scale, single-element test.

Research Needs: A coherent plan and research strategy with prioritized objectives are needed. A load transfer mechanism must be established through model studies. A national pile test site or a specific project should be selected for such studies. The effects of load/stress/strain path on bearing capacity and deformation should be studied at specific projects. Test data should be synthesized to assess methodology and to develop model design specifications.

Recommendation: A testing program aimed at evaluation of prototype foundations at an appropriate test site should be devised. A methodology for project-specific field testing is needed. Demonstration projects should be considered at other sites. General guidance for application of particular types of foundations should be provided beyond simplified versions. Technology transfer should be accomplished by publications and seminars or other forms of dissemination.

4 Conclusions and Recommendations

Conclusions

During Session Four of the workshop, verbal reports of the deliberations of the working groups were presented by the chairman of each group. Discussions of the reports of the working groups by the assembled workshop participants was encouraged and, in some cases, ideas were contributed which supplemented the initial report of the working group. For this reason, the conclusions given below represent the group consensus as to the most general and widely needed avenues of investigation in the topic of effects of piles on soul properties. Suggestions for specific research topics or for research on specific types of piles may be found in Section 4, Reports of Working Groups. Evaluations of the workshop, which were requested from each workshop participant, are summarized as recommendations for future workshops.

Reports of the Working Groups and the group discussions during Session Four of the workshop left no doubt that the topic of the effects of piles on soil properties is a viable and necessary area of geotechnical research and that present knowledge in this area is inadequate. Workshop participants were unanimous in the belief that emplacement of piles or other deep foundation elements can alter the properties of the adjacent soils and that knowledge of the nature of these changes could lead to improved (and more economical) methods of design for piles and other deep foundation elements. There was also general agreement among the workshop participants that a need exists for an overall clearinghouse or coordinating entity which could maintain awareness of, integrate, and disseminate results of pile research in all sectors. Included would be, in addition to research results, databases of pile test data from tests accomplished for both research and design purposes as well as records of case histories relevant to the pile-soil property problem. There was also general agreement that an overall research plan, incorporating the efforts of all sectors and with review and input from all interested parties, would be desirable. While not necessarily able to financially sponsor or assign research topics to other research agencies, the coordinating entity could remain cognizant of research results and determine if and when results of such research had fulfilled the objectives of the overall research plan. No conclusion was reached as to what entity might undertake the responsibilities described above.

Reports of the working groups and the discussions which followed revealed a consensus that research is needed which addresses the following topics to contribute to the solution of the pile-soil property problem.

a. Undisturbed sampling of soils both before and after pile driving at a given site—with ensuing laboratory tests to determine possible changes in soil properties.

b. In situ testing of soils both before and after pile driving at a given site. Especially desirable would be in situ testing of soils through or around specially designed or instrumented piles. Measurement of radial, vertical, and tangential stresses and pore pressures adjacent to piles would be facilitated by such a study. Important time effects could be studied with this arrangement as well.

c. Design special sampling tools and/or specially designed piles which would permit undisturbed sampling or in situ testing of soils radially outward from a pile. In conjunction with undisturbed sampling before pile driving, such special devices could do much to determine how soil properties change with pile driving.

d. Development and testing of more reliable instrumentation for installation with/on driven piles and other deep foundation elements should be made a priority. Emphasis on dependability and survivability of instruments should be paramount.

e. High quality site characterization is essential to the success of almost any full-scale or reduced-scale research involving effects of piles on soil properties. This is especially true of research of the type described in *a* through *c*, above. For this reason, National Geotechnical Experimentation Sites should be developed and supported and research conducted or confirmed at these sites insofar as possible.

f. High quality data from instrumented piles is desirable to develop the fundamental mechanisms of pile-soil interaction and to refine numerical models. Special attention should be paid to pile-driving projects where data might be captured for relatively low cost, which could assist in this purpose.

g. The centrifuge and large-capacity stress cell are two devices which can be used in the laboratory to effectively address pile-soil property problems. Neither device has been widely utilized to address the pile-soil property problems to which they are amenable. These devices offer the advantages of a controlled environment, relative speed and low-cost, and direct observation of results and could be used to address both basic concerns or to verify or duplicate numerical models or field behavior. Consideration should be given to research which includes these devices.

h. Special topics pertaining to the pile-soil property problem such as contaminated soils, mixed or layered soils, specialized types of piles, pile groups and lateral loadings all represent research needs in the soil-property problem and could each be addressed by research efforts modeled after the needs listed above. Sensitivity studies into method of pile emplacement, time effects, type or diameter of pile also are necessary to determine which parameters play the largest role in the alteration of soil properties. It was suggested during the workshop that assembly and study of case histories might contribute substantially to the advancement of the pile-soil property problem in these special topic areas. If so, collection of case history results by the coordinating entity mentioned above would be very useful.

Recommendations

At the conclusion of the workshop, participants were encouraged to fill out an evaluation sheet to provide suggestions for future workshops and to evaluate the usefulness of workshops of this type. Prominent comments and those comments common to a number of evaluations are given below.

a. The majority of the workshop participants agreed that the format, length, and scope of the workshop had been appropriate. Participants also agreed that the size and composition of the group facilitated production and interchange of ideas on the workshop topic. Several suggestions were received that additional workshops be held to explore the topic further and possibly develop the overall coordinated research plan described above.

b. A need for more international representation and participation by industry practitioners was expressed. Suggestions were that representatives of the petroleum industry could contribute significantly.

c. Some suggestions were received indicating that workshops on other geotechnical topics following the format used for this workshop would be profitable.

d. It was recommended that copies of the workshop proceedings be furnished to organizations interested in undertaking or sponsoring some of the research suggested during the workshop.

Appendix A
Workshop Attendees

Name	Organization and Mailing Address	Phone Number Fax Number
Don Banks	Chief, Soil & Rock Mechanics Division USAE Waterways Experiment Station 3909 Halls Ferry Road Vicksburg, MS 39180-6199	601-634-2630 601-634-4219
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Appendix B
Contributed Papers

DMT Measurements Around PCS Piles in Belgium: Evaluation

H. Peiffer
Research Assistant N.F.S.R., Ghent University, Belgium

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Professor-Director of Soil Mechanics Laboratory,
Ghent University, Belgium
Professor, Leuven Catholic University, Belgium

G.Cortvrindt
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Synopsis

This paper evaluates the soil stress changes around a Socofonda PCS auger pile using the dilatometer test. Results of three test sites are reported. The evaluation of the execution parameters and other external influences are gathered through DMT-measurements during installation of the pile. The paper further describes a comparison of the results of soil investigation at different stages before, during, and after pile installation.

Introduction

The bearing capacity, especially the shaft friction, of auger piles is strongly dependent on the execution parameters of pile. PCS-piles (Pressurized Concrete Screw-piles) installed with a continuous auger are brought to depth causing no or a very limited soil displacement. During casting the concrete, an additional pressure is applied on the fresh concrete. For this type of pile, the execution parameters are the downward force during penetration N_i, the torque M_i, rotation speed (downwards) n_i, downward velocity of the auger v_i, upward velocity v_u, upward force N_u, concrete pressure σ_b, the ratio diameter auger to diameter stem, the pitch (for all the discussed piles, p was 45 cm) of screwing down, and the quality of the concrete and the way of casting. This is an important factor governing the arching effect and determining the real fresh concrete pressure in equilibrium with the total horizontal soil stresses. By the use of hyperplastifiers, the W/C ratio can be limited to 0.45 and the cubic strength nowadays reaches 4-5N/mm^2 and higher. All of these parameters are continuously measured (each 80 mm) during pile installation.

Type of Pile

Generally the PCS-auger piles are using an inner stem diameter of 100 mm. During casting of the concrete an overpressure of 2 to 4 bar is applied on the fresh concrete, while the auger is regained slowly. This procedure doesn't cause vibrations. After casting the concrete, the reinforcement is brought into the pile using a vibrator. Eventual difficulties can be avoided using a greater inner stem diameter. So the reinforcement can be placed inside before casting the concrete. The outer diameter for such piles ranges between 35 and 45 cm. The high torque (100 kNm) that can be applied avoids excavating too much of soil and allows for penetration in resistant bearing layers. The degree of soil displacement can be deduced out of the overcomsumption (occ) of concrete, defined as:

$$occ = \frac{V_b - V_p}{V_p} \times 100 \text{ (in percent)} \tag{1}$$

in which:

V_b = concrete volume consumption
V_p = theoretical volume of the pile with known nominal dimensions

This parameter (occ) gives a mean value and hides local effects such as lense or layers where a higher excavation and/or soil displacement resulting in varying concrete consumption exists. For the discussed piles, the overconsumption was 16 percent for the test site at Oudenaarde, 28 percent for Dendermonde and 95 percent for Doel-test sites. The high overconsumption at Doel is mainly due to the fact that the upper layer (0 to -8.00) is composed of dredged material and to the presence of another very soft layer on the levels from -8.70 to -10.30.

Soil Test Program

For each pile, the following test soil results are gathered: (CPT) electrical cone penetration test before execution of the pile; (DMT1-A) DMT-test before installation of the pile at 1.5 times pile diameter out of the center of the pile. In addition, during installation of the pile, a DMT1-B test is performed with the

DMT-blade installed at a fixed depth. The A-reading of the DMT curve was considered equivalent to an oedometer time-deformation curve. Using Casagrande's log t/fitting method: $t_{100\ percent\ cone}$ was determined before the start of the piles installation. Pile installation started when the decreasing ratio of A-readings became less than 5 kPa/hour. By this, consolidation and relaxation, due to the installation of the DMT blade did only have a negligible influence on the measurements later on during pile installation. Finally a (DMT-1-C) test after installation of the pile at 1.5 times pile diameter out of the center of the pile was performed. The membrane is oriented towards the pile shaft.

For the orientation of the blade, one has two possibilities.

a. Radial position (membrane towards the pile). The advantage here is the direct measurement of horizontal stress variation. For large soil displacements, arching effect can occur around the blade.

b. Tangential position: the advantage is a smaller disturbance of the initial stress field around the pile. On the other hand as long as no clear relationship exists between the principle stress changes, the interpretation of such DMT readings with tangential blade orientation mainly stays difficult.

The aim of this research program was to evaluate the effect of PCS pile installation, in sandy layers, on the surrounding soil stress field.

Discussion of the Test Sites

Doel-test site

The results of this test site, with all of the difficulties linked normally to each new research experience have been discussed in an earlier paper Peiffer et al (1991).

Dendermonde-test site

The results of the field tests are given in Figure 1a, b, c and Figure 2.

From the linearly increasing CPT in the sandlayers where the DMT (1-B) is installed, it becomes evident the sandlayer was normally consolidated. The starting DMT A-reading is slightly higher than expected probably due to the local increased stress field around the blade. The DMT-membrane is directed towards the pile center. The screwing-in energy resulting in very high downward penetration in the cohesive top layers, induces excess pore water pressures and "heavy liquid pressures" which compensate a normally expected soil relaxation. When the auger passes by at DMT-level, the whole of the remolded soil column along the auger being more or less in suspension, induces for the rest of the screwing down movement a remarkable total stress increase (water pressure increase). So one can easily explain an input of total stress

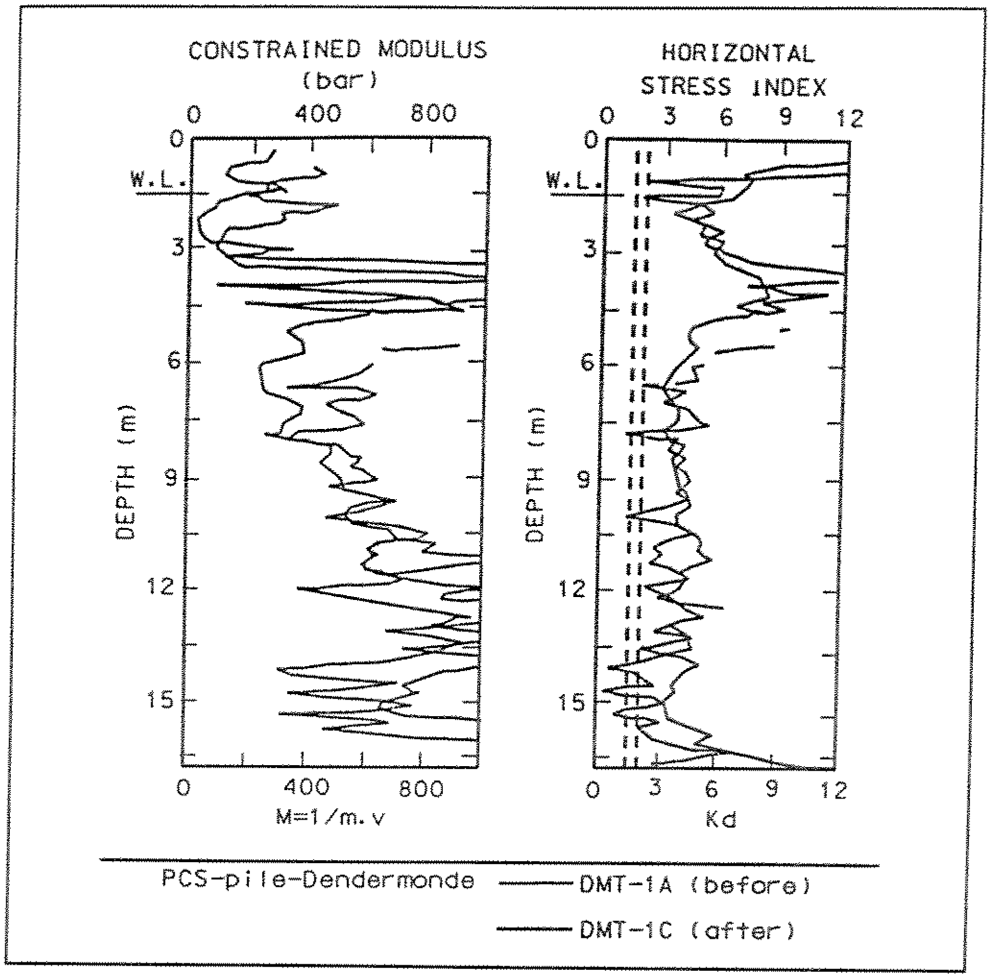


Figure 1a. DMT results at Dendermonde

increase of order of ≤ 30 kPa. Obviously, the water overpressure fades out with time. During the casting process the total stresses, induced by the fresh concrete are detected, especially again starting at the level of the blade. The final DMT A-reading is apparantly flattening out at about 160 kPa, being almost 50 kPa higher than the starting value. From this point of view this pile system would be somewhat beneficial to the soil-condition. One however must be careful since only DMT A-readings, performed some days after pile installation would indicate more reliable results. In Figure 2 DMT1-A/1-C one sees such difference between the DMT-test before and after full pile installation indicating that there is almost no change in horizontal stress index and constrained modulus.

Oudenaarde-test site

Two piles were examined at this site; piles n° 205 and n° 207. The results are gathered in Figures 3, 4, and 5.

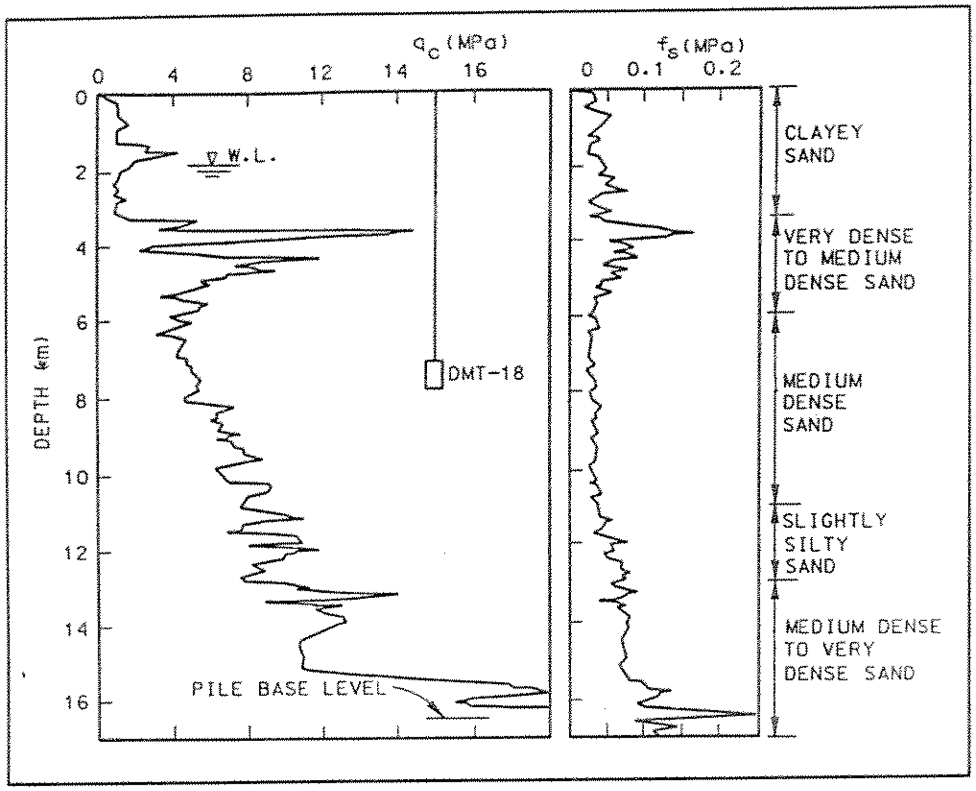


Figure 1b. Initial CPT at test site

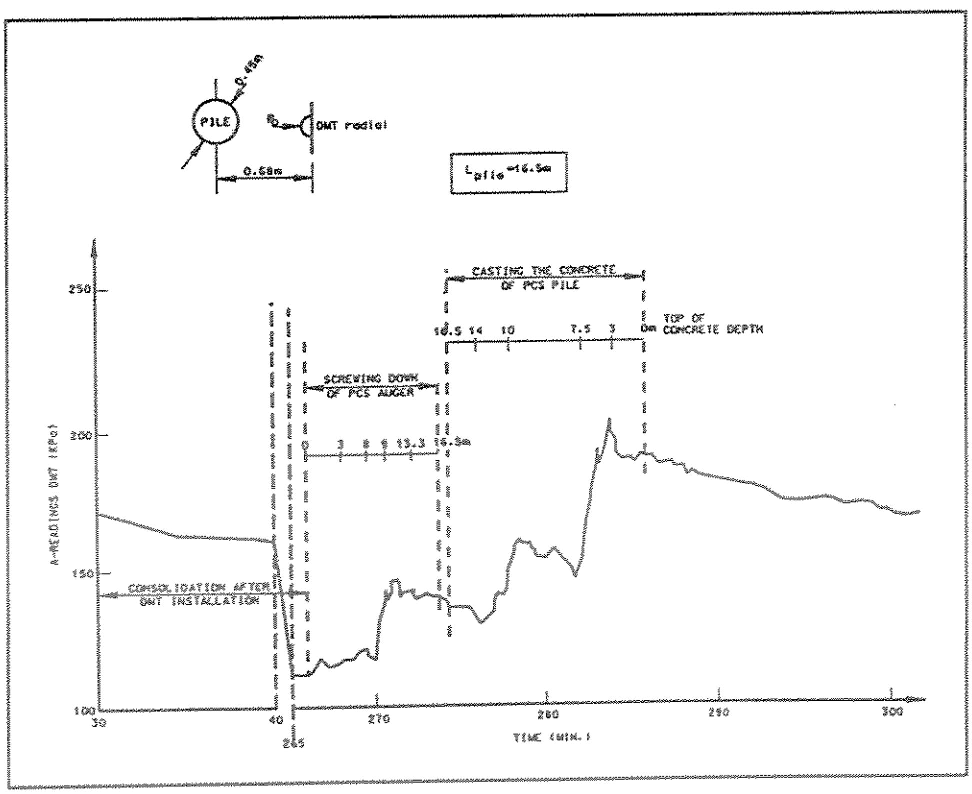


Figure 1c. DMT1B result at Dendermonde

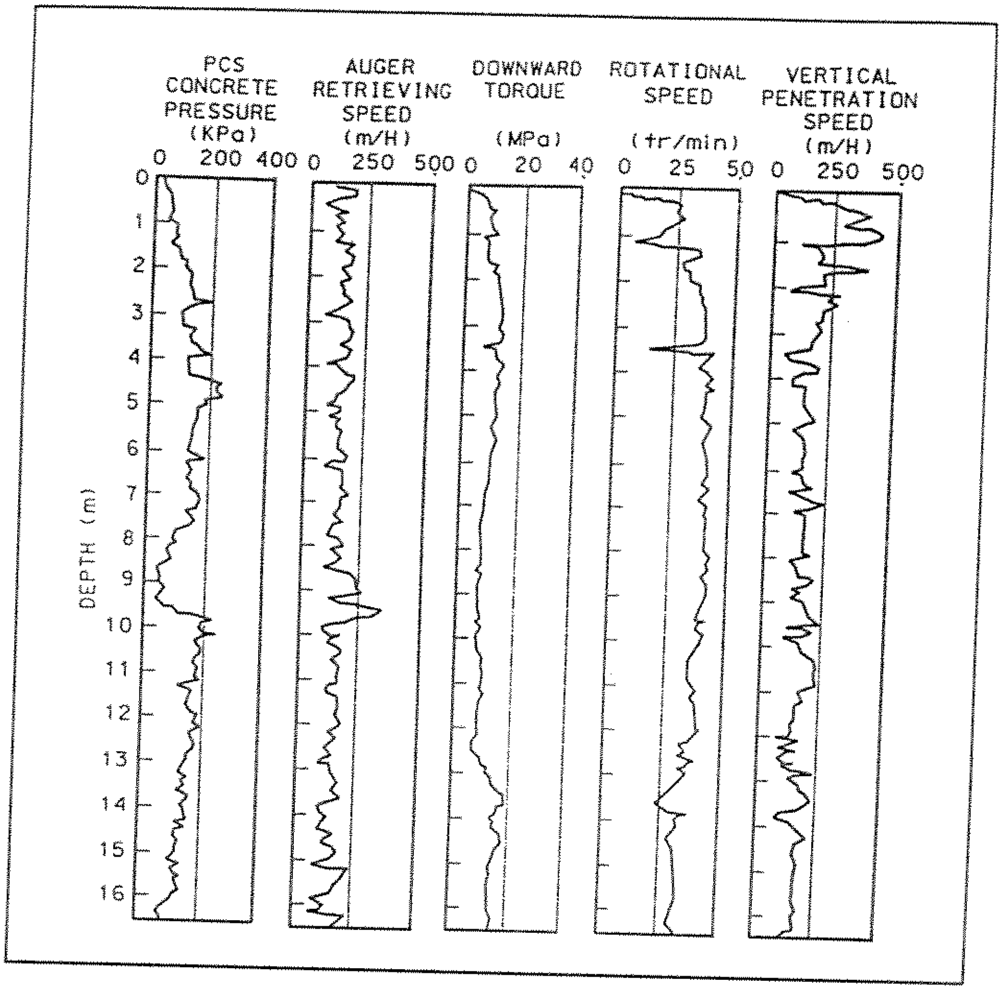
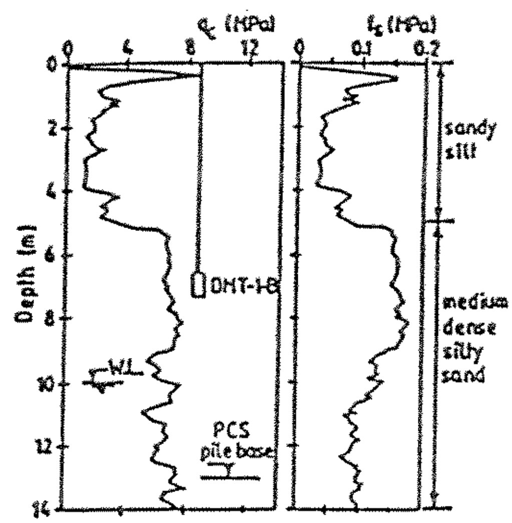


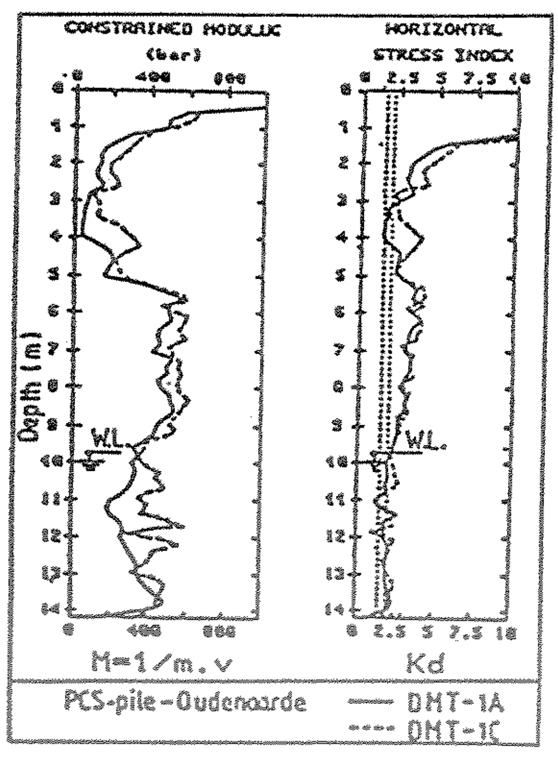
Figure 2. PCS-pile installation parameters at Dendermonde test site

From the Figure 4b, the (tangential) DMT-1B analysis on pile n° 205 indicates that soil arching is built up gradually during screwing down of the auger, resulting in a general overall (gradual) decrease of the A-value with some peaks in between, probably due to collapses of former arches while the auger continues to penetrate. The further general decrease after DMT A-reading during casting the fresh concrete, only shows the gradual outfading influence of the soil arching with increasing fresh concrete weight.

From the readings for pile n° 207, with radial DMT-1B readings, a lot of interesting differences with the previous analysis are shown. Over the first five meters, the auger penetrates with a rather small torque, but with a high downward velocity through the loose silt/silty sand. This results this type of dry and only very slightly cohesive material in a dramatic decrease of the total stress combination felt by the DMT blade. The fresh concrete passing the level of the DMT, the concrete pressure influences tremendously the A-reading of the DMT-total stress in radial direction. Such overpressures are again fading out in time, partly compensated by the increasing fresh concrete weight above the DMT-measurement level.

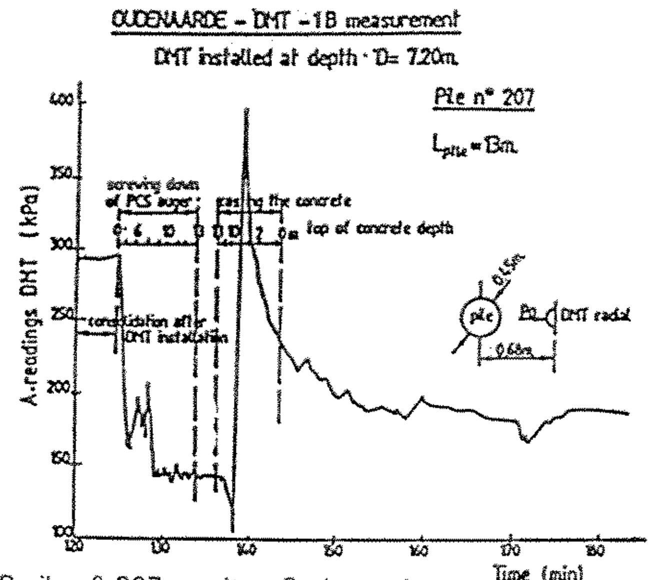


a. DMT-results at Oudenaarde

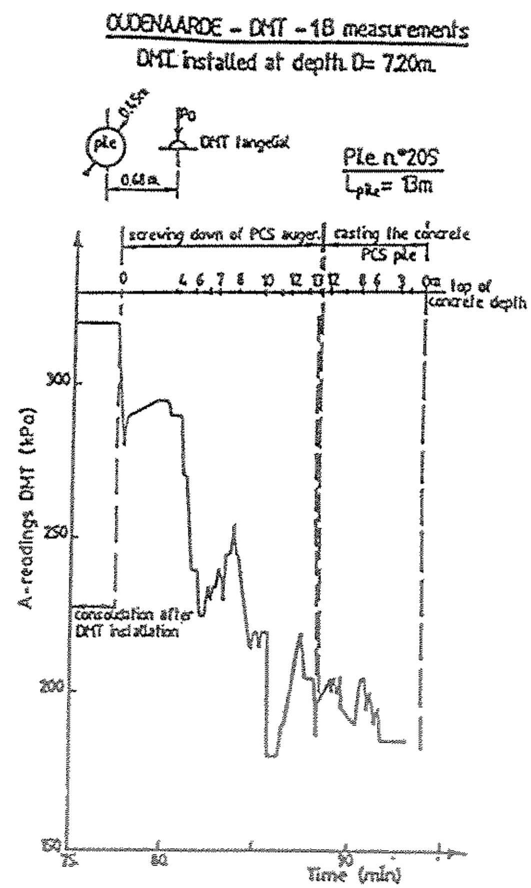


b. Initial CPT results at Oudenaarde

Figure 3.



a. DMT 1B pile n° 207 result at Oudenaarde



b. DMT 1B pile n° 205 result at Oudenaarde

Figure 4.

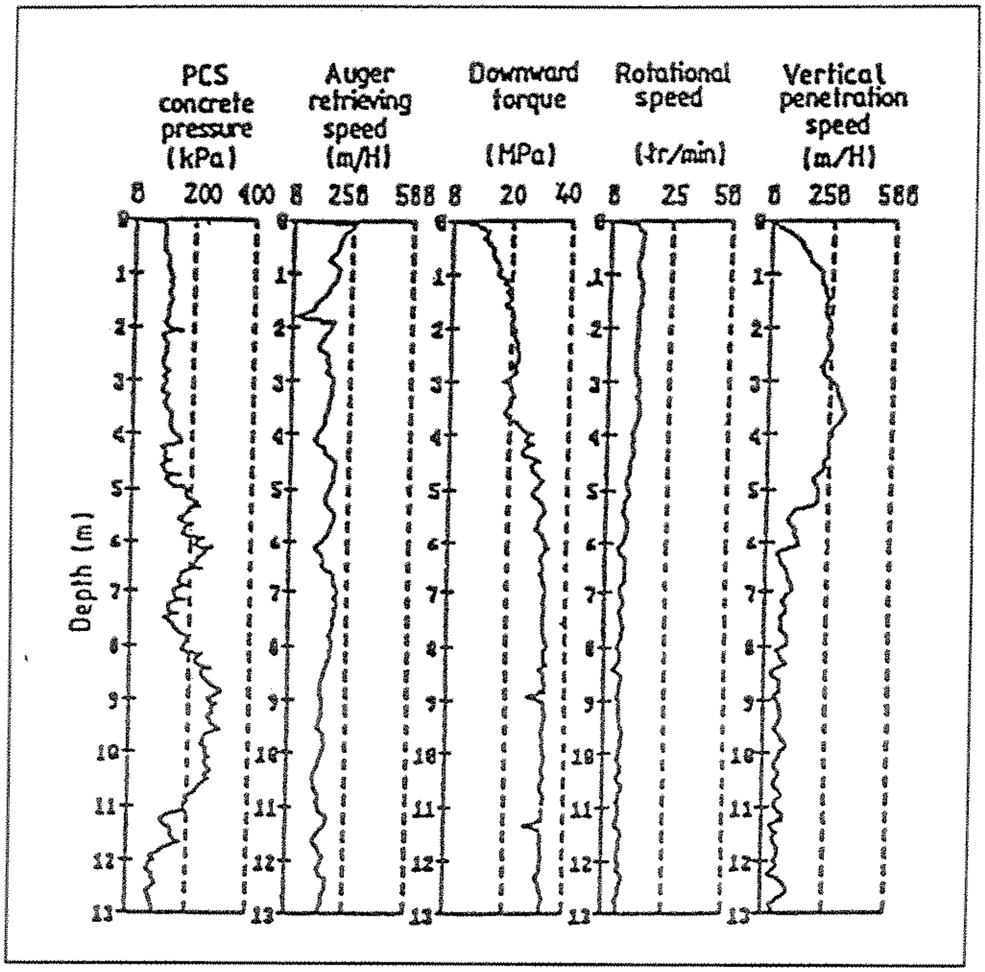


Figure 5. PCS-pile n° 205 installation parameters recorded at Oudenaarde

Considerations on Pile Installation Effects

Our main interest in this kind of research is to finally understand much better the shaft capacity of screw piles. For this purpose, the pile installation effects have to be evaluated more carefully with respect to the effective soil stress changes around the shaft. Comparison between DMT-1-A and DMT1-C readings gives an indication of the degree of such stress changes in terms of total stress. Referring to the horizontal stress index (Table 1), the results of such comparison are presented for the three test sites.

Moreover, the increase of DMT-horizontal total stress in the top layer and the decrease near the pile tip are remarkable. This is discussed in an earlier paper Peiffer, Van Impe, Cortvrindt, Van Den Broeck (1991). Also because the pile installation parameters are available, a proposal could be to calculate the idealized specific screwing-in energy Esc. The use of this specific energy value in relation to the pile installation parameters is suggested in Van Impe (1988).

Table 1
Comparison of DMT-Stress Index

Site	Measured Concrete Overconsumption	$\frac{K_{D,\ after\ inst.}}{K_{D,\ before\ inst.}}$	Soil Type
Doel	95 percent	1.30	Medium dense sand
Oudenaarde	28 percent	1.20	Medium dense sand (NC)
Dendermonde	16 percent	1.00	Medium dense sand-slightly (OC)

The results of such Esc-data can either be presented in a time related diagram (Figure 6 - Doel) or an auger penetration depth diagram (Figure 7 - Dendermonde). In these figures we can see a quite good agreement between DMT-A-readings and installation energy curves. For Doel, the input energy curve is even quite similar to the A-reading curve. Peaks of energy-input can also be found on the A-reading curve. For Dendermonde one sees clearly the effect of the energy-input on the horizontal stress state when the auger passes. One also can see the influence of execution time and time delays on the final stress-state. A similar analysis for Oudenaarde and also the evaluation of screwing out energy is discussed in Van De Velde, Van Hoye (1992).

The today's analysis in this research program is also going out from continuous pressures measurements at the DMT1-B blade during pile installation. Together with pile installation parameter, a more complete stress field analysis so became available.

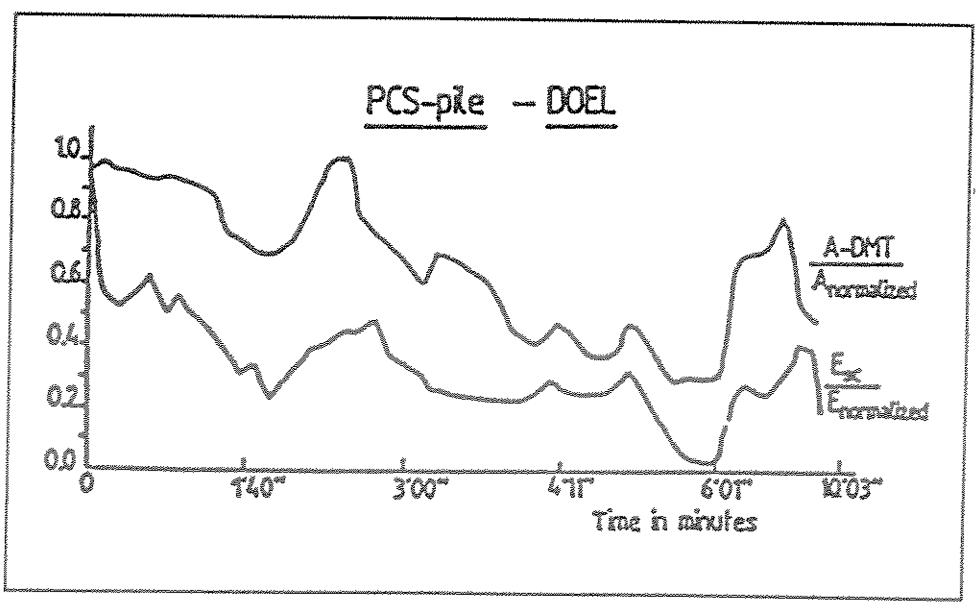


Figure 6. PCS-installation energy function as compared to DMT A-readings

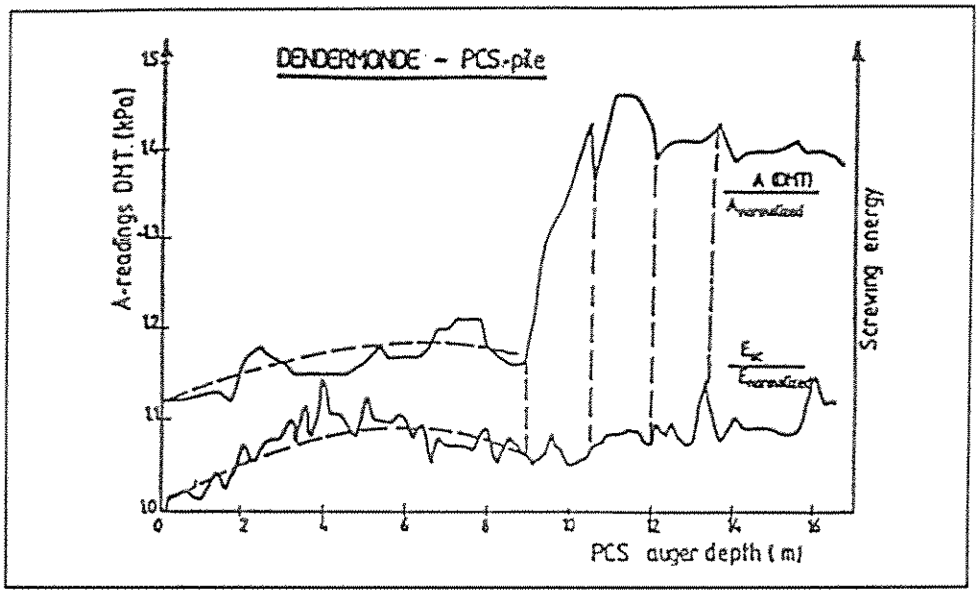


Figure 7. PCS-pile installation energy and DMT A-readings as a function of time

Conclusions

In this paper the dilatometer is described as a tool that could help to evaluate the influence of pile execution parameters on the soil condition around the pile shaft. For PCS-piles some increase of horizontal stress is mainly dependent on the OCR, soil type, dilatant character of the soil, and predominantly on the installation details. It can be deduced that a too low auger penetration velocity and losses of time during installation can affect largely the final stress state around the pile.

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Recent Experiences and Developments of the Resonant Vibrocompaction Technique

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Synopsis

In many cases of improvement of cohesionless soils, the improvement requirements vary over the soil profile, especially when some layered variable densities are distinguished. In the case of solid densification by resonant vibrocompaction, a specially designed low dynamic stiffness compaction probe is used, achieving a more efficient transfer of vibration energy at the interaction with the surrounding soil. In the paper, data will be given and analyzed on recent experiences with the resonant vibrocompaction technique for densifying study layered soil behind a large quay wall; moreover, the sensitivity of the compaction efficiency to the frequency change, as compared to other compaction techniques will be discussed.

Introduction

The densification of granular soils using dynamic methods has been discussed largely in literature. An overview of the various possibilities of heavy temping, soil blasting, vibroflotation, vibrocompaction, and resonant compaction methods has recently been presented in a report to the Seminar on "Soil Dynamics and Geotechnical Earthquake Engineering - 26-29 July 1992, Lisbon, Portugal."

Resonant compaction soil improvement techniques have been applied for same years in problems related to liquefaction potential reduction, foundation of structures, pile length reduction, slope stability improvement, etc.

Vibratory Compaction

Loose granular deposits can be most suitably densified, up to great depths, by vibratory compaction techniques. Among those methods, vibroflotation and casing driving with soil replacement, and certainly vibratory probes (vibro-wings, Y- or double Y- shaped probes) at constant or varying frequencies (resonant vibratory compaction) and applied shear strains larger than about 0.1 percent up to 10 percent, are the most well-known procedures.

Vibratory probes in contrast with vibroflotation techniques use heavy vibrators clamped at the upper end of long steel probes, which can be either suspended at a crane or guided by a mast. The probe is excited in the vertical direction and the vibration energy is transmitted to the surrounding soil along the whole length of the probe. The soil is compacted mainly as a result of vertically polarized waves. Water jetting is normally not required, which makes the method simple to execute. Different types of compaction probes were developed in Japan, North America and Europe, Massarsch (1991). The geometric shapes of simple probes such as steel tubes or H-beams is not very efficient for soil compaction. Therefore, special probe shapes were developed for soil compaction. In loose to medium dense saturated sands the strong ground vibrations result in a sudden increase of pore water pressure in a soil column surrounding the vibrating probe which can be considered leading to a state of cyclic mobility of the soil mass. Whenever the sand in its original density was loosely enough packed, so real liquefaction can even occur.

Commonly used vibrocompaction probes are the Swedish Vibrowing Massarsch (1982), the stiff Franki (Tristar) Y-profile and the more flexible double-Y shaped probe (MRC-profile), Figures 1 and 2.

The Franki Y-probe (Figure 1) has three long steel plates 500 mm wide and 20 mm thick which are attached to a long steel rod 15 m - 20 m long, at 120° to each other. Additional steel ribs 300 mm x 50 mm x 10 mm are welded on to the two sides of each plate at 2 m intervals in order to improve further the efficiency of the probe. A motor driven vibrator mounted on top of the probe delivers vertical vibrations with frequencies in the range 10 to 30 Hz.

The degree of soil improvement that can be achieved for a given soil deposit depends mainly on the power of the vibrator, the duration of compaction, the vibration amplitude and frequency, the rate and sequence of insertion and withdrawal of the probe and the spacing between the points of insertion. Also the geotechnical conditions are of importance such as the fines content (permeability) of the soil deposit, the depth of compaction, the initial density of the soil and the location of the ground water level.

The efficiency of the densification is usually monitored by electric CPT carried out before and after densification, measurement of porewater pressures set up, measurements of vibration velocities next to the probes, settlements and the overall ground subsidence evaluation, etc. Field measurements provide valuable information on the increase of soil stiffness, and reduced compressibility. In terms of soil parameters, this is generally reflected as

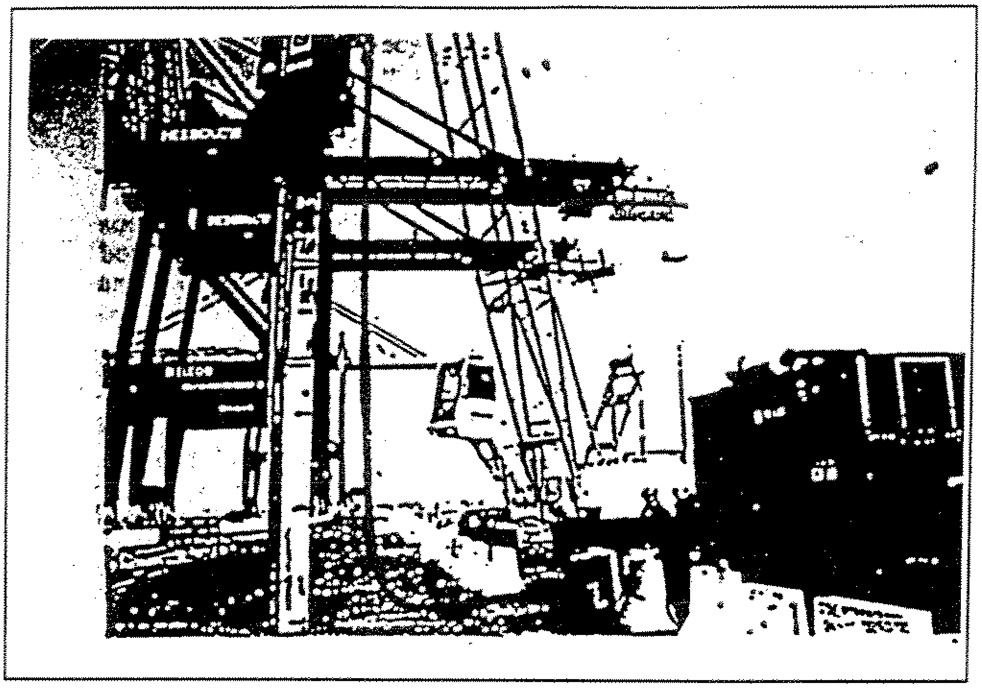


Figure 1. The Y-probe on the test site

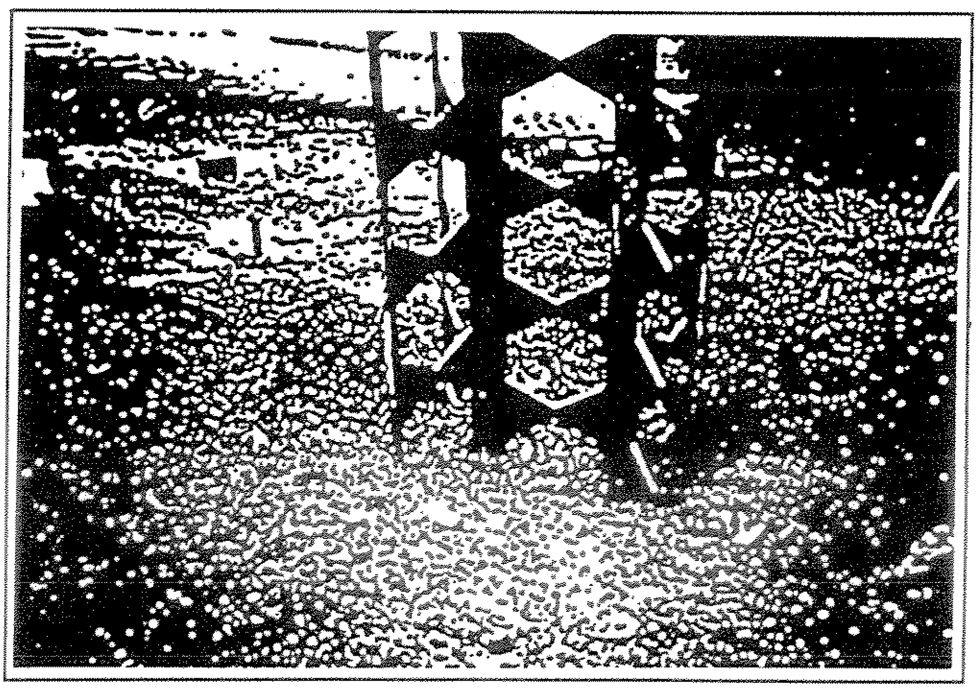


Figure 2. The double Y-probe on the test site

increases in the angle of internal friction ϕ; shear modulus G; elastic modulus, E; and decreases in compression index C_c. Furthermore, soils improved by vibrocompaction and vibroreplacement have shown increased resistance to liquefaction.

The tendency of a soil to generate excess pore pressure during undrained loading is correspondent to the volume changes that occur during drained loading. Loose soils tend to contract upon shearing and, if loading is too quick for drainage to occur, generate excess pore pressures. For soils that derive all strength from confinement, this generation of excess pore pressures can lead to a condition of zero effective stress (when $\sigma = u + \delta_u$), resulting in loss of strength and fluid-like behavior with only residual resistance to deformation.

Resonant Compaction

Over the years, the deep vibratory compaction technique has been largely improved by the introduction of the resonance vibro-compaction concept, the basic principles of which have been discussed in several papers. In these references, case studies are reported which document the practical experience gained from a variety of projects, where resonance compaction successfully has been applied to solve the foundation problems, such as the problem of loose saturated sands (liquefaction), the reduction of pile length for bridge foundations, foundations for tanks and other heavy structures, quay walls harbour projects, the improvement of slope stability etc..., Franki (1986), Massarsch and Vanneste (1988), Wiesner (1983), Massarsch (1991), Neely and Leroy (1991).

The key features of the resonant compaction technique are the use of a specially designed compaction probe and of a heavy vibrator with variable operating frequency on top of the probe. Vibration excitation of the probe is in the vertical direction only. After probe insertion, the frequency of the vibrator is adjusted to the resonant frequency on the soil layer, thereby amplifying the ground response. An important advantage of resonant compaction, compared to other vibratory methods, is that the whole soil layer oscillates simultaneously during compaction. Moreover, because of the special design of the probe and the possibility to adjust the compaction frequency to the resonant frequency of the soil layer, an optimal transfer of vibration energy to the surrounding soil can be achieved, resulting in a more efficient compaction process.

Recently very promising improvements of the resonant compaction method have been introduced within the MRC concept. This concept overcomes a number of limitations of the former used equipment by the development of a harmonized unit, comprising a modern vibrator, a flexible double-Y-profile (FLEXI-probe) and an electronic process control unit, Figure 2.

The vibrator can vary the frequency continuously during operation without reduction of operating speed of the diesel engine (centrifugal force of up to 4,000 kN) and is attached to the top end of the flexible probe. Usually, everything is guided by a mast to assure vertical insertion of the probe. Vibration excitation of the probe is in the vertical direction only. The patented FLEXI probe has been specially designed to achieve optimal transfer of compaction energy from the vibrator to the soil. This is obtained by the reduction of the dynamic stiffness of the probe, together with the increase of the interaction area of the profile. Finally, the electronic monitoring system, which continuously records all essential compaction parameters, such as vibrator frequency, probe penetration depth, ground vibration velocity, power supply etc., allows the recording of the actually performed compaction process, displays relevant information of the process as well as instructions to the operator of the compaction machine, and delivers a complete report of the work performed to the site inspector and the engineer.

The capacity of the vibrator must be chosen with respect to the specific project requirements, such as soil type, initial soil density, required degree of compaction and penetration depth. The vibration amplitude required to compact the soil can be determined from a semi-empirical relationship between initial cone penetration resistance, vertical ground acceleration and soil layer depth, Figure 3.

The resonance frequency of a soil layer is rather difficult to predict reliably from theory, but is relatively simple to measure directly on site through seismic measuring techniques. The ground response during the on- or off-switching of the probe vibrator is measured by velocity transducers at a distance from the compaction probe. The equivalent frequency spectrum (Figure 4) indicates that the resonance of the relevant soil layer to be densified occurred in this case study around 15 Hz for both probes. It also can be seen that at resonance the vertical vibration amplitude is strongly amplified up to much higher values than at the other operating frequencies.

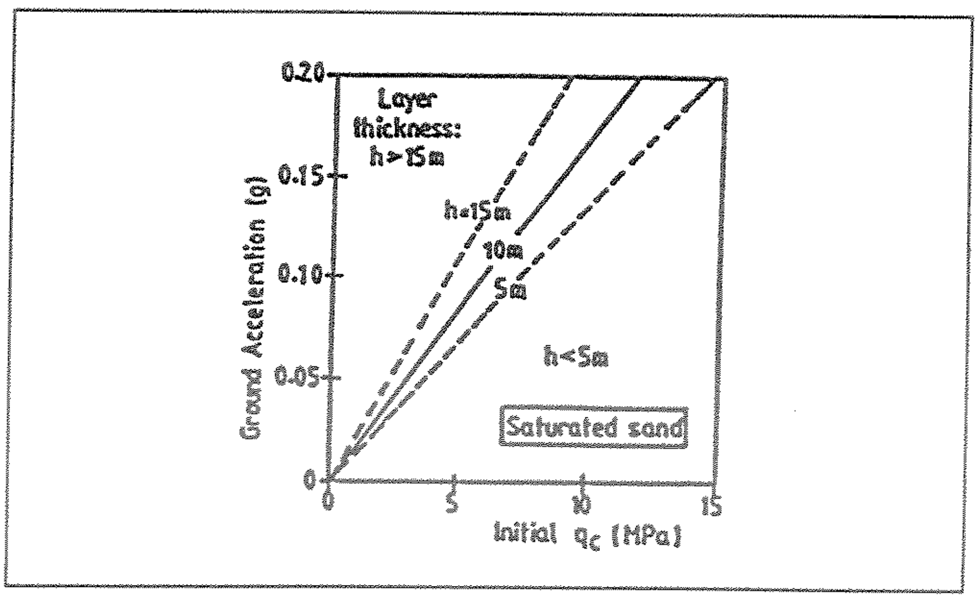


Figure 3. Required ground acceleration for vibratory densification of saturated sand as a function of initial soil density (CPT) and soil layer depth, Massarsch (1991)

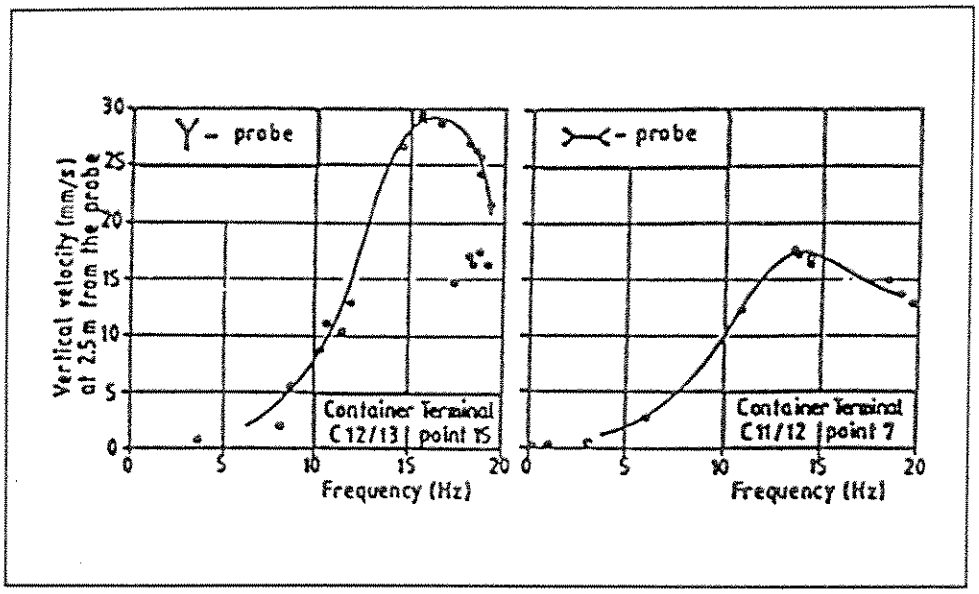
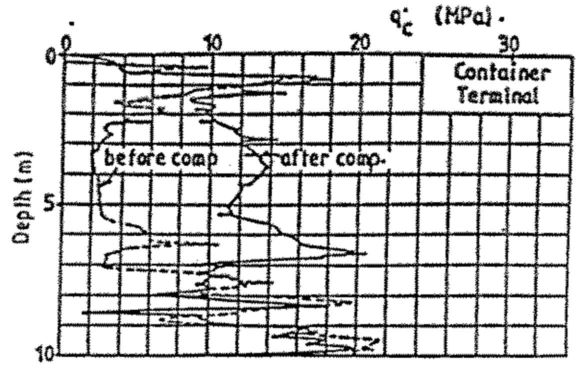


Figure 4. Frequency spectrum-vertical particle velocity peaks, for the two probes

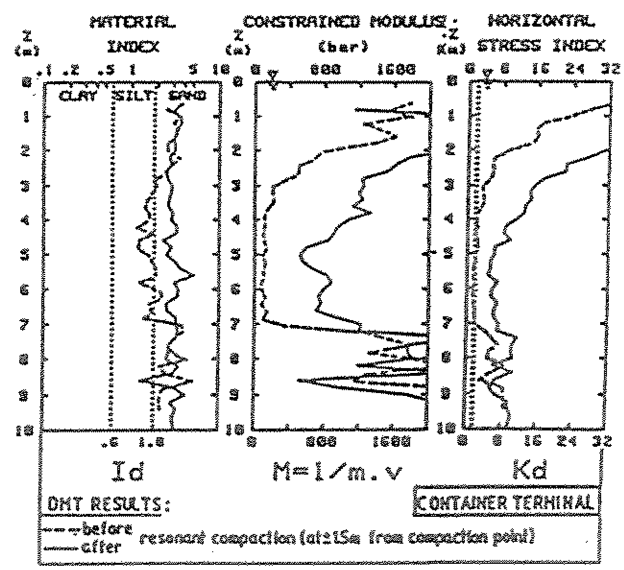
Frequency analyses of ground response at the start of a densification often show in addition to the fundamental vibration mode also several of the higher vibration modes, suggesting that soil layers of varying stiffness exist. With progressing compaction, the resonance frequency increases and higher vibration modes tend to disappear, indicating more homogeneous soil conditions. The resonance frequency can be readily determined at any stage of soil compaction, and makes it possible to adapt the vibrator frequency to the optimal operating conditions.

Geotechnical Requirements for Vibratory Compaction

Under present practice, the efficiency of the soil improvement and even the liquefaction potential of mechanically improved site is typically evaluated through the use of in situ tests particularly the Standard Penetration Test and Cone Penetration Test, and recently even dilatometertests DMT. When CPT is used, the data are either converted to equivalent SPT values, or used directly to assess liquefaction using CPT-based evaluation methods, eg, Robertson and Campanella (1985). It must be noted however, that these correlations were developed from natural sites where no ground improvement has been performed. When soils are subjected to mechanical modification, variables associated with pre-straining, such as horizontal stresses and time effects (aging), may not be adequately represented by SPT and CPT results. An example of a CPT and DMT evaluation at the test site discussed in this paper, is shown in Figure 5a, b. Measurements of pore pressure ratios with a piezocone (CPTU) with the porous element behind the tip show loose untreated soils generating high excess pore pressure during penetration. Well densified soils, on the other hand, exhibit



a. Typical CPT before and after resonant compaction



b. Typical DMT rest before and after resonant compaction

Figure 5. In situ test results before and after resonant compaction

pressures below hydrostatic and even negative, indicating a tendency for the soil to dilate during shear.

Vibratory compaction should be used only in granular, free-draining soils with an "effective" particle diameter d_{10} (corresponding to 10 percent of the grain size curve) larger than approximately 0.03 mm. Grain size curves are useful but can usually only be obtained from disturbed samples at certain depth intervals. Thus, they may not be representative for the entire soil deposit, especially if the soil is stratified. Even relatively thin silt and clay layers in a sand deposit can significantly affect the drainage conditions and reduce the densification effect. Therefore, it is advisable to also use in situ testing methods such as the static cone penetration test (CPT) with sleeve friction measurements. Good compaction results can generally be expected, when the friction ratio (local sleeve friction as percentage of point resistance, F_R in percent) from electric cone penetration tests is lower than about 1 percent. When the friction ratio exceeds 1.5 percent, then vibratory compaction is usually not efficient.

Pore pressure measurements performed in connection with cone penetration tests - CPTU - can provide additional information concerning soil stratification and the existence of even thin, fine-grained layers. Soil with excess pore water pressures higher than about 10 percent are often not suitable for vibratory compaction. It is also important to establish the level and variation of the ground water in connection with a soil compaction project. Usually, dry soils or soil layers with negative pore water pressures are more difficult to densify than saturated soils and need to be identified carefully. The effect of thin impermeable seams in a soil deposit can be evaluated by measuring the permeability in situ. Soils suitable for vibratory compaction should have a permeability higher than approximately 10^{-6} m/s.

Another factor of great practical importance for the design of vibrocompaction is the aging effect which can occur even several weeks after soil compaction. A large number of well-documented case histories suggests that also granular soils show a marked aging behavior, i.e. that the engineering properties can improve by 50 to 100 percent over a period of few weeks, Schmertmann (1991). The soil strengthening and stiffening effects can be explained by increased soil friction and should be considered when the time of penetration testing after compaction is chosen. Mitchell (1986) has complied several case histories from soil improvement projects in different parts of the world where significant soil stiffening with time has been observed. Massarsch (1991) reported a case where after vibratory compaction of a silty sand, the cone penetration resistance increased within one week by almost 50 percent. Based on experience from compaction projects in different soil conditions, at least 5 days should be allowed for recovery of soils after compaction. In the case of larger projects it is recommended to establish the time effect by penetration tests at different time intervals after compaction.

Additional Practical Considerations

For resonant compaction, as in the case of heavy tamping and blasting of cohesionless material, the final higher degree of relative density guarantees a more dilative deformation behavior. This implies a much higher resistance to liquefaction, since one mostly has to deal in such dense cohesionless soils with the phenomenon of cyclic mobility, Castro (1976), Van Impe (1982). As reported by R. A. López et al. (1992), Mitchell et al. (1976) studies the effects of pre-straining and soil fabric on liquefaction potential. This research determined that soils prepared to the same relative density do not necessarily exhibit similar undrained cyclic behavior. For example, soils densified by vibratory procedures produced no preferred axis of particle orientation, and exhibited better static and cyclic performance than soils prepared to the same density.

Four important parameters must be determined before the start of the compaction work: the interdistance of compaction points, the compaction time at each point, the mode of probe movement (insertion and extraction) and the compaction points' sequence. The grid spacing ranges typically between 1.5 and 4.5 m depending on the shape, size and stiffness of the probe. The duration of each compaction energy at a given point depends on the layer thickness, grid spacing, degree of soil improvement required and varies typically between a few minutes up to half an hour.

Two examples of compaction procedures at our test site of the container terminal are shown in Figure 6.

For the mode of probe insertion and extraction, in the layer to be compacted, plays an important role, the optimal procedure should be monitored with the aid of vibration sensors on the ground surface. They indicate the ground response during all probe movements, Figure 6 show partly, the vertical and horizontal vibration velocities (RMS-values) during penetration, change of frequency, penetration and extraction of the probes in an easily drained partially saturated sand layer. Initially, during the probe penetration from the surface, the vibration amplitudes increase. Lowering the frequency involves a larger soil volume in the densification process and increases the vibration velocities. During the stepwise extraction of the probe, the vibration amplitudes also show peaks, but generally of a lower level than during insertion.

The vibration attenuation can be approximately estimated from the equation:

$$A_2 = A_1 \sqrt{\frac{R_1}{R_2}} \times e^{-\alpha(R_2 - R_1)} \qquad (1)$$

where A_1 and A_2 are the vibration amplitudes at the respective distances R_1 and R_2 from the probe. The coefficient of wave attenuation α is a measure of the soil damping, depending on the vibration frequency and the dynamic soil properties. Figure 7 shows the results at the test site of the relationship between the vertical vibration acceleration (velocity) and the distance from the vibrating probe to the measuring devices on the soil surface. The attenuation coefficient α for

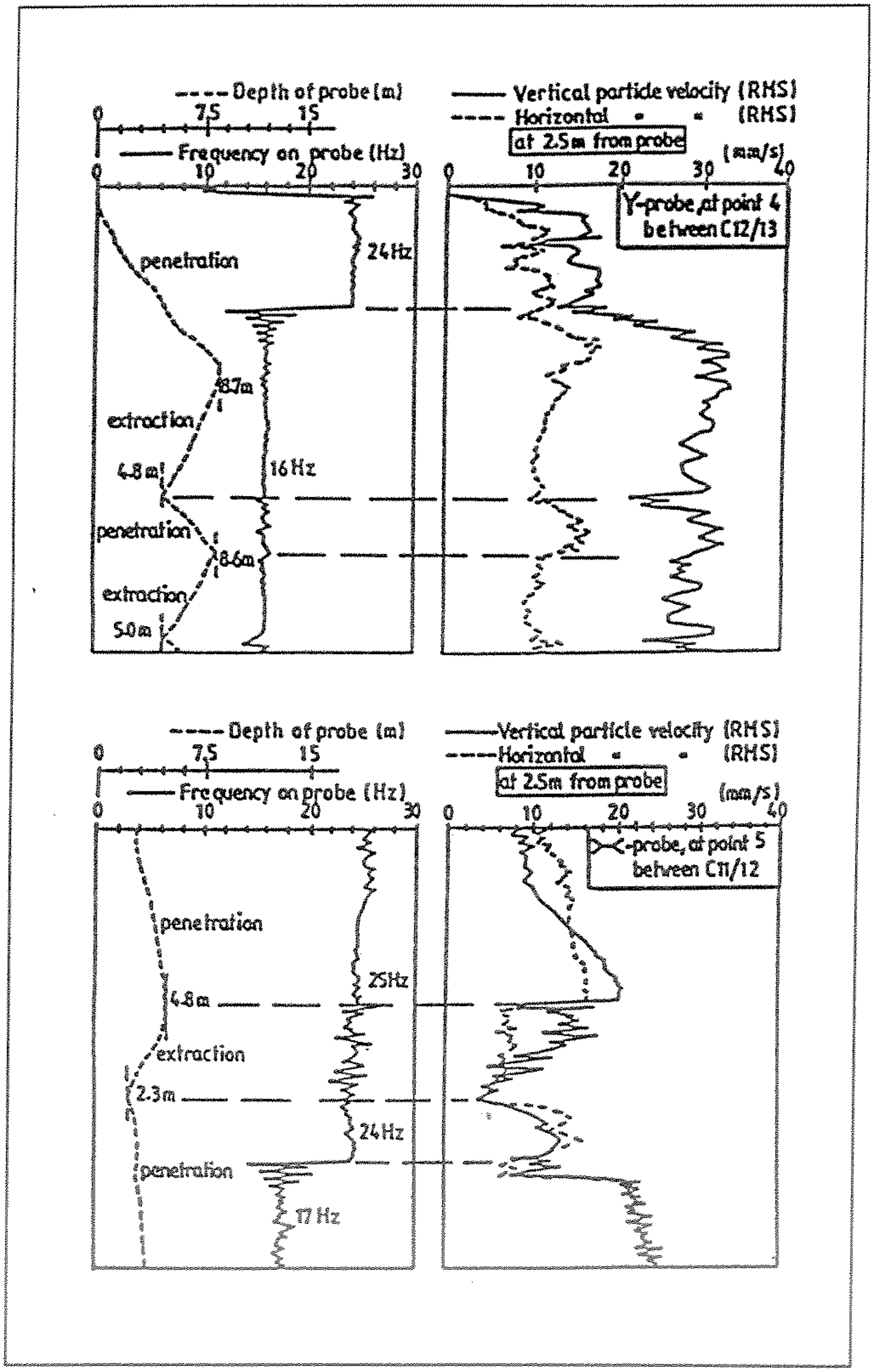


Figure 6. Part of resonant compaction procedure at container terminal test site

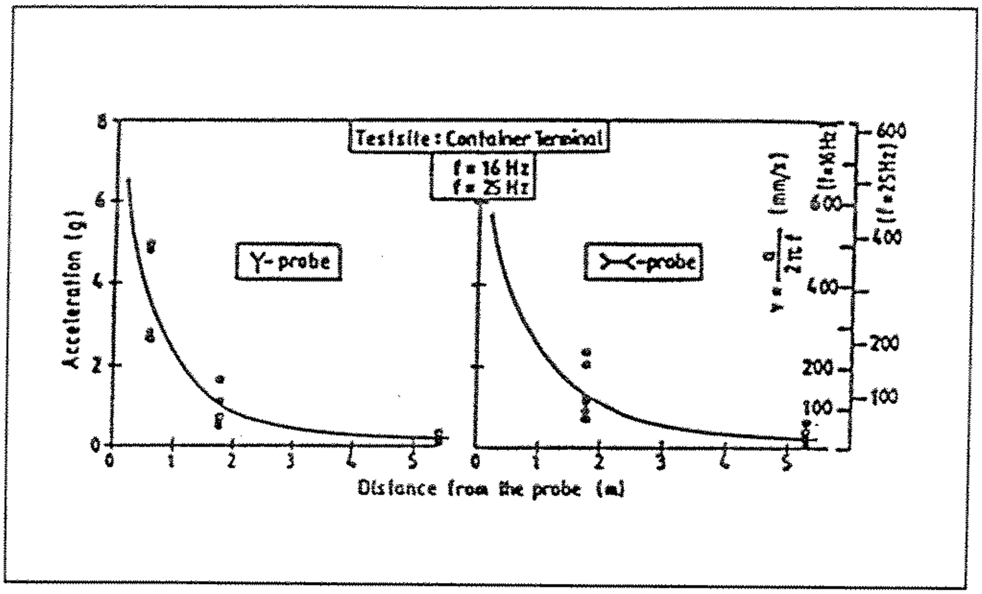


Figure 7. Attenuation of ground acceleration with distance from the probes

vibratory compaction varies in saturated clean sands typically between 0.05 and 0.10 m^{-1}. In partially saturated to dry sands α can rise to 0.5 m^{-1}.

Resonant Compaction Results on the Test Site

A container terminal in Belgium, with large diameter caissons constituting the quay wall, was designed for a working load on top of the quay floor behind the caissons of 60 kN/m^2. The backfilling of the caisson quay wall with granular material had, for some reasons such as earth pressure reduction and settlement minimizing of the quay floor, to be densified. The aim of the densification work was to reach an overall mean cone resistance, after densification, of q_c = 6 MPa.

The layout of the compaction points with the mentioned techniques and the controlling CPT/DMT are indicated on Figure 8. On the soil surface, vibration accelerations and settlements in many points of the testing area at 0.59 m, 1.76 m and 5.3 m of the densifying profile, were measured. Before and after the compaction work, a series of CPT and DMT results performed at the same locations, was gathered in order to analyze the efficiency and the fulfilling of the requirements of the densification.

For this particular test site in between caissons 11 and 13, soil compaction was carried out by Franki Foundations with a variable frequency vibrator (type Muller MS 50). The operating frequency of the vibrator can be varied between 10 and 30 Hz. The maximum oscillating amplitude is 22 mm at a centrifugal force of 1,500 kN. For soil densification, 12 m long compaction probes (stiff Y-probe and double Y- or MCR flexible probe) were used.

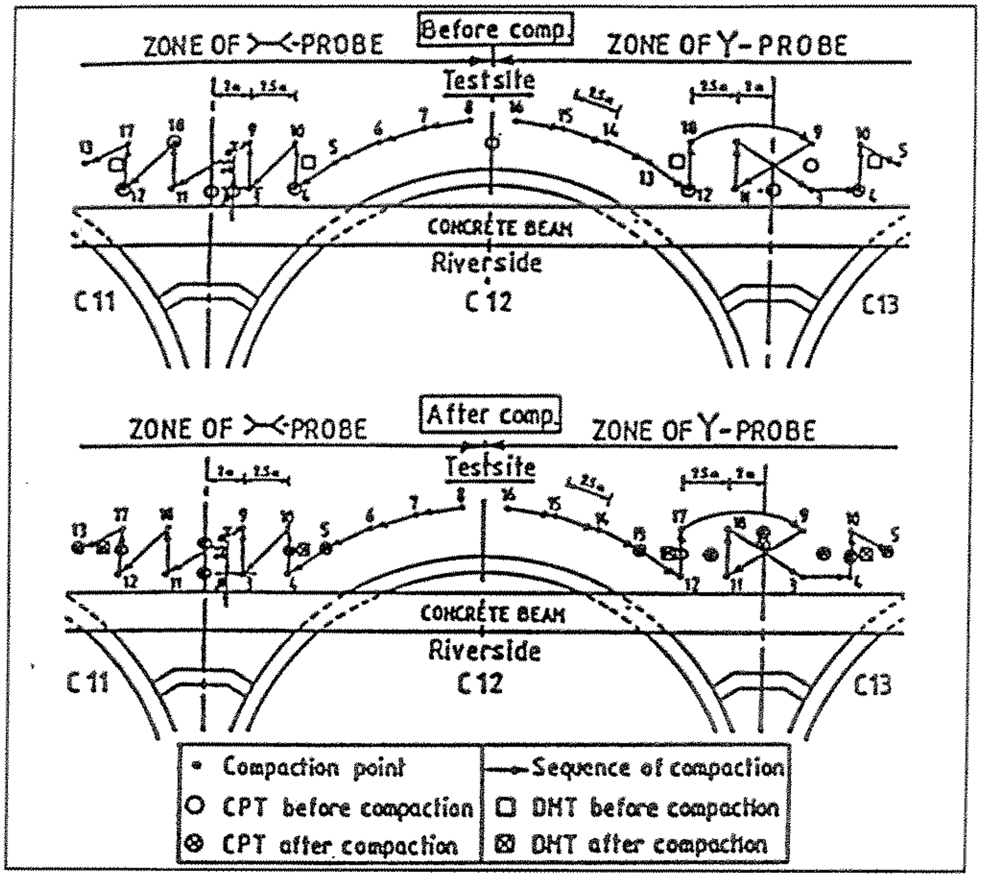
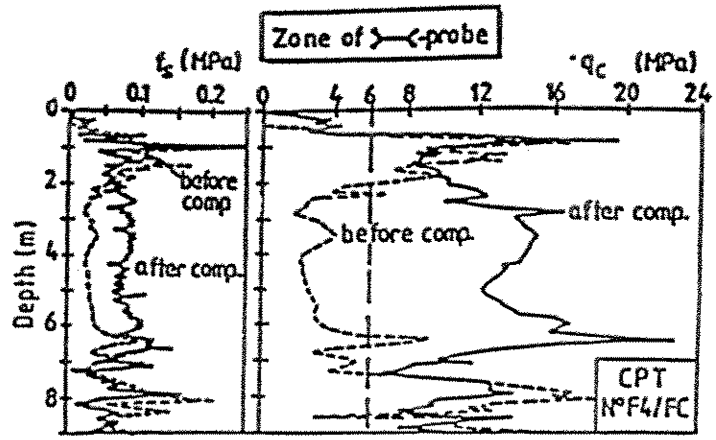


Figure 8. Layout of the compaction points and the controlling CPT/DMT at the test site

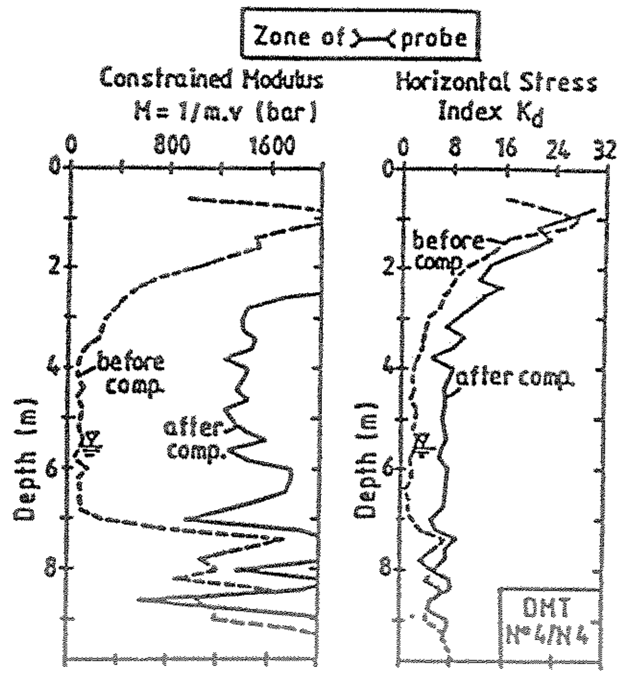
Simultaneous with the densification work, a small research project was implemented in order to investigate more closely the efficiency of those different applied compaction methods (vibro-compaction using the Franki stiff Y-probe and flexible double Y-probe (MRC).

Examples of the monitoring and quality evaluation of the compaction work at the test site, mainly done by CPT and dilatometer (DMT) before and after the compaction, are given in Figure 9a, b. It can be seen that for both types of compaction probes, the requirement of reaching a mean cone resistance q_c-value of 6 MPa after the densification work, could be fulfilled rather easily. Gathering all CPT results in the densification area, one could also compare the average CPT cone diagrams before and after the densification (Figure 10), reaching the same conclusions with respect to the densification quality. The DMT-results show the expected increase of the stiffness modulus and the horizontal stress index, indicating from the deformation point of view its usefulness in the control of the quality of compaction work.

Because of the individual scatter of CPT-results, which often occurs in sandy layers with thin seams of variable penetration resistances, it is sometimes advantageous, instead of going out from individual cone resistance comparison



a. CPT cone and friction resistance comparison at container terminal before and after resonant compaction (C11/12)



b. DMT-comparison before and after resonant compaction at container terminal (C11/12)

Figure 9. In situ test results before and after resonant compaction (Site C11/12)

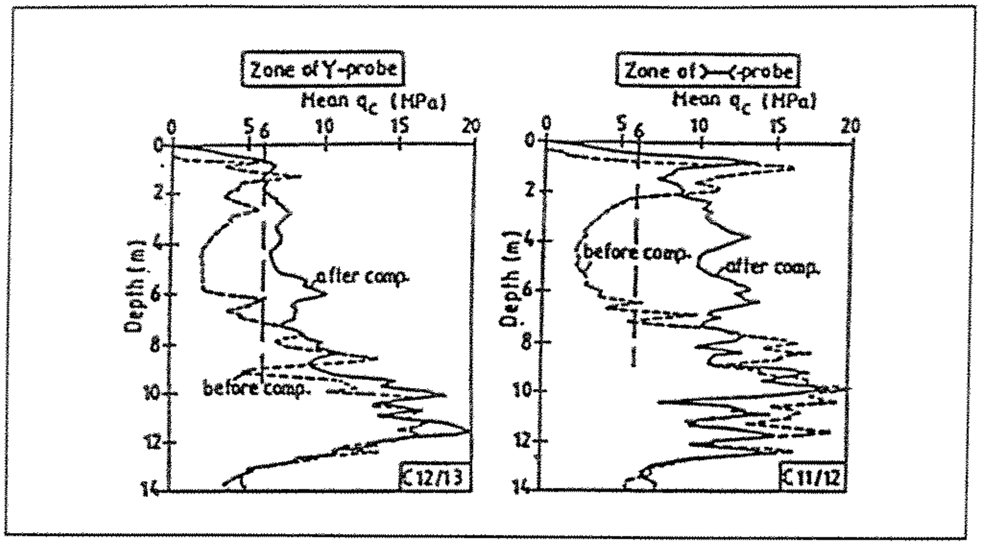


Figure 10. Average densifying efficiency from cone reistances

or from average cone resistance diagrams, to link the quality control to the comparison of the overall cone penetration energy before and after the compaction work.

For this purpose, one calculates the penetration energy E_c as:

$$E_c = \int_{z=0}^{h} q_{c,z} \times \omega_{cone} \, dz \qquad (2)$$

where

$\omega_{cone} = 10^{-3}$ m^2
$q_{c,z}$ = electrical cone resistance at depth z, (in MPa)

Such an analysis would allow to finally predict more reliably the expected improvement level Ec(after)/Ec(before) to the interdistance of the compaction (tributary area) for a given probe, compaction input parameters and soil condition, Van Impe (1989).

Examples of such an energy comparison for this test site are shown in the Figure 11.

Another simple and very useful compaction control method goes out from the measurement of the soil surface settlements during compaction, Figure 12. This figure shows the final settlements and horizontal displacement vectors for various locations of the settlement measuring devices in between the compaction points. Figure 13 shows how those settlement data match in an empirical group of curves correlating average settlements at the ground surface to the vibratory compaction specification and the initial CPT cone resistance of the compacted layer. The settlements indeed mainly depend on the initial soil density and the soil particle acceleration. The data of this test site, suggest that using the

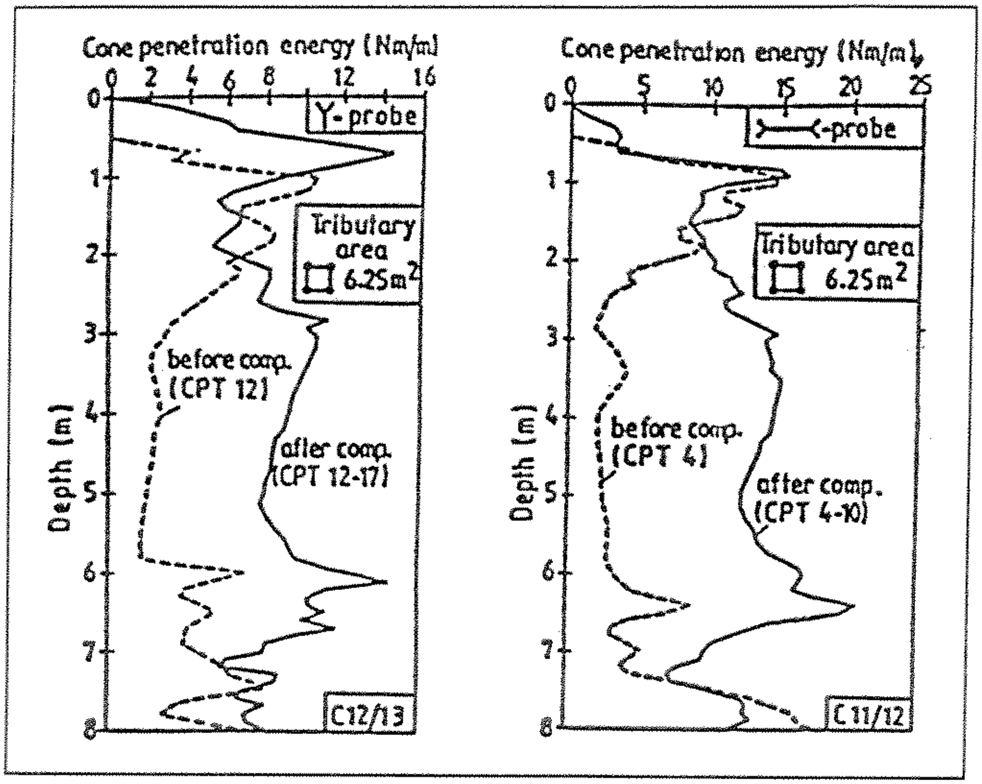


Figure 11. Efficiency of the resonant compaction from the cone penetration energy at a given tribuary area for the compaction points

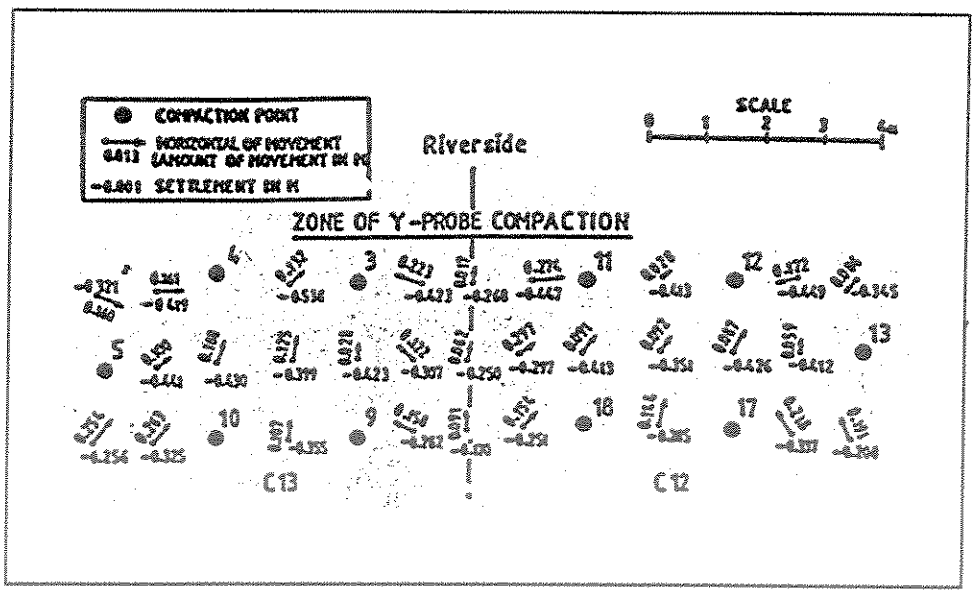


Figure 12. Final settlements and horizontal displacement vectors for various locations of the settlement measuring devices in between the compaction points

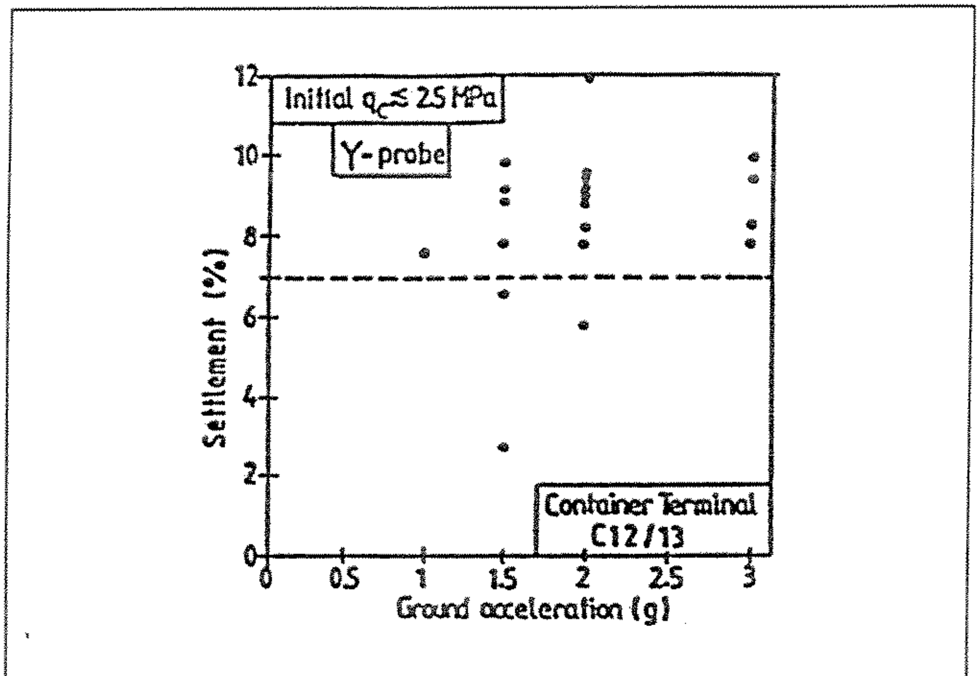


Figure 13. Ground surface settlements due to resonant compaction

resonant compaction technique in sandy layers with initial cone resistances lower than or around 2.5 MPa, leads easily to settlements of 7 percent and more of the initial thickness varying from 3.5 m to 4.5 m of the layer to be compacted.

Conclusions

From what has been discussed in this paper, it is obvious the resonant compaction technique has great potential in cohesionless soil improvement cases. The double Y-probe concept (MRC probe) including the advances for continuous monitoring of the compaction work through the electronic process control for the probe.

Acknowledgment

We want to acknowledge the DOLSO-department of the Flemish Government, the Antwerp city, the main contractors and the owner of the Container Terminal for allowing us to set up this small research work.

Müller Geo-System provided the MRC process control system. The support and assistance of the personnel of Müller Bautechnik and of Loster MCT is gratefully acknowledged.

We will thank also Mss. Minna Leppannen - Soil Mechanics Laboratory of Tampere University, Finland for helping, during her stage at the Ghent University Laboratory of Professor Van Impe - analyzing the data.

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Assessment of An Effective Stress Analysis for Predicting the Performance of Driven Piles in Clays[1]

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Abstract

Researchers have advocated systematic analyses, which model changes in effective stresses and soil properties through successive phases I the life of a pile, as a rational method for understanding the factors which control pile performance. Work at MIT has included the development of analytical models which simulate soil disturbance effects associated with pile installation (Strain Path Method), and constitutive models (e.g., MIT-E3) which describe the effective stress-strain behavior of normally and lightly overconsolidated clays (OCR≤4) through successive phases in the life of the pile. This paper summarizes the role of these analyses in predictions of pile shaft behavior. The results illustrate the effects of soil properties, mode of pile installation and other factors affecting the limiting skin friction which can be mobilized at the pile shaft. Predictive capabilities and limitations of the proposed 'objective analysis' are reviewed based on comparisons with high quality field data measured by the piezo-lateral stress (PLS) cell and by instrumented model pile tests.

Introduction

The geotechnical group at MIT has been involved in a sustained research effort to develop more reliable methods for predicting the capacity and performance of friction piles driven in clays. Originally, these efforts were motivated by the uncertainties involved in extrapolating empirical correlations from onshore pile load tests to offshore applications where much larger piles are used and where soil conditions often include deep layers of weak, normally and lightly overconsolidated clays. More recently, the work has focused on the performance of piles supporting Tension Leg Platforms (TLP; Whittle 1987; Whittle et al. 1988; Malek et al. 1989).

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The research makes the fundamental assumption that pile performance should be evaluated using a rational framework in which the changes in stresses and soil properties are described through successive phases I the life of a pile (Esrig et al. 1977; Randolph et al. 1979; Baligh & Kavvadas 1980). For TLP piles this includes: a) the initial, in situ conditions in the ground; b) pile installation; c) soil consolidation or 'set-up'; d) monotonic shearing due to quasi-static tensile forces imposed by mooring of the TLP superstructure; and e) cyclic shearing caused by storm waves. Due to the complexities of these pile-soil interactions, extensive research efforts have been required in four complementary lines of activity:

1. The formulation of analytical models which are capable of making realistic predictions of pile performance. This work has included the development of : a) the Strain Path Method (Baligh 1985, 1986a,b) to describe the mechanic of the pile installation process; and b) effective stress soil models (MIT-E1, Kavvadas 1982; MIT-E3, Whittle 1987) which can describe realistically the constitutive behavior of K_0 consolidated clays which are normally to moderately overconsolidated ($OCR \leq 4$).

2. In situ measurements on a closed-ended model pile shaft referred to as the Piezo-Lateral Stress cell (PLS; Morrison 1984; Azzouz & Lutz 1986; Azzouz & Morrison 1988). The PLS cell has the capability of providing simultaneous measurements of the total lateral stress, pore pressures and average skin friction acting on the shaft of a small diameter pile (D=3.83cm) during installation, consolidation and axial loading.

3. Extensive laboratory testing to support the analytical and field studies, and to develop more comprehensive understanding of complex aspects of clay behavior. This work has included: a) test programs to characterize in situ soil properties (e.g., Azzouz & Lutz 1986); b) undrained cyclic direct simple shear testing to simulate pile-soil interaction during TLP storm loading conditions (e.g., Malek et al. 1989); c) measurement of anisotropic properties in the Directional Shear Cell (DSC) which are used to evaluate the constitutive models (e.g., Whittle et al. 1992).

4. Evaluation of pile shaft predictions was initially accomplished using PLS cell measurements (Whittle & Baligh 1988; Azzouz et al. 1990). Subsequently, analytical predictions have been compared with field data from instrumented pile tests at a number of sites (e.g., Whittle 1991b).

This paper describes typical predictions of pile shaft performance using Strain Path analyses in conjunction with the MIT-E3 soil model. The analyses provide objective predictions of soil stresses and pore pressures during installation, consolidation and axial loading, based on specified in situ soil properties and stress conditions. The results illustrate the effects of stress history, mode of pile installation and other factors on the limiting skin friction which can be mobilized at the pile shaft. Predictive capabilities and limitations of the analyses are assessed from comparisons with high quality field data at a number of soft clay sites.

The Strain Path Method

Pile driving causes severe disturbances and leads to significant changes in the stresses, pore pressures and properties of the surrounding soils. The analysis of these installation effects represents a highly complex problem due to: a) high gradients of the field variables (displacements, stresses, strains, and pore pressures) around the pile; b) large deformations and strains which develop in the soil; c) the complexity of the constitutive behavior of soils, including non-linear, inelastic, anisotropic, and frictional response; and d) non-linear pile-soil interface characteristics. The Strain Path Method (SPM; Baligh 1985) assumes that, due to the severe kinematic constraints in deep penetration, the deformations and strains in the surrounding soil are essentially independent of its shearing resistance and can be estimated with reasonable accuracy based only on kinematic considerations and boundary conditions. The application of the Strain Path method for analyzing piles driven in low permeability clays assumes:
a) there is no migration of pore water during penetration and hence, the soil is sheared in an undrained mode; b) pile driving can be modeled as a quasi-static (steady), deep penetration problem (i.e., there is no inherent difference due to pile installation by jacking or driving); and c) the deformations and strains can be estimated from the steady, irrotational flow of an incompressible, inviscid fluid around the pile (Baligh & Levadoux 1980; Baligh 1986a; Whittle et al. 1991). By considering two-dimensional deformations of soil elements, the Strain Path analyses provide a more realistic framework for describing the mechanics of deep penetration than one-dimensional, cylindrical cavity expansion methods (CEM; e.g., Kraft 1982; Randolph et al. 1979). On the other hand, the assumptions of strain controlled behavior used in the Strain Path Method greatly simplify the penetration problem and avoid the complexity of large deformation finite element analyses (e.g., DeBorst & Vermeer 1984; Kiousis et al. 1988).

Figure 1 compares SPM solutions of strain paths experienced by individual soil elements for two pile geometries: 1) a closed-ended (or fully plugged) pile of radius, R, with a rounded tip geometry (the 'simple pile'; Baligh 1985); and 2) an open-ended pile of with aspect ratio, B/t = 40 (where B = 2R is the outside diameter, and t the wall thickness), which penetrates the soil in an unplugged mode (Chin 1986). These solutions correspond to the two extreme modes of penetration for large diameter, open-ended pipe piles used in offshore foundations. The strain history at a point is fully described by three independent components of shear strain; E_1, E_2, and E_3 which correspond to triaxial, pressure meter (cylindrical cavity expansion) and direct simple shear modes, respectively.

For the simple pile geometry, there is monotonic increase in the E_2 shear component as the pile tip passes the soil element while the components E_1 and E_3 exhibit reversals in direction (which are not included in CEM analyses). The overall magnitude of shear strain in the soil is described by the second invariant of deviatoric strains, $E = 1/\sqrt{2}\left\{E_1^2 + E_2^2 + E_3^2\right\}^{1/2}$ as shown in Figure 2. At locations around the pile shaft (Figure 2), there is an inner zone of soil which experiences much larger shear strain levels than can be imposed in conventional laboratory shear tests (E>10% at r/R<2) and is characterized also by large net changes in all three strain components (e.g., elements $r_0/R = 0.5, 0.2$; Figure 1a).

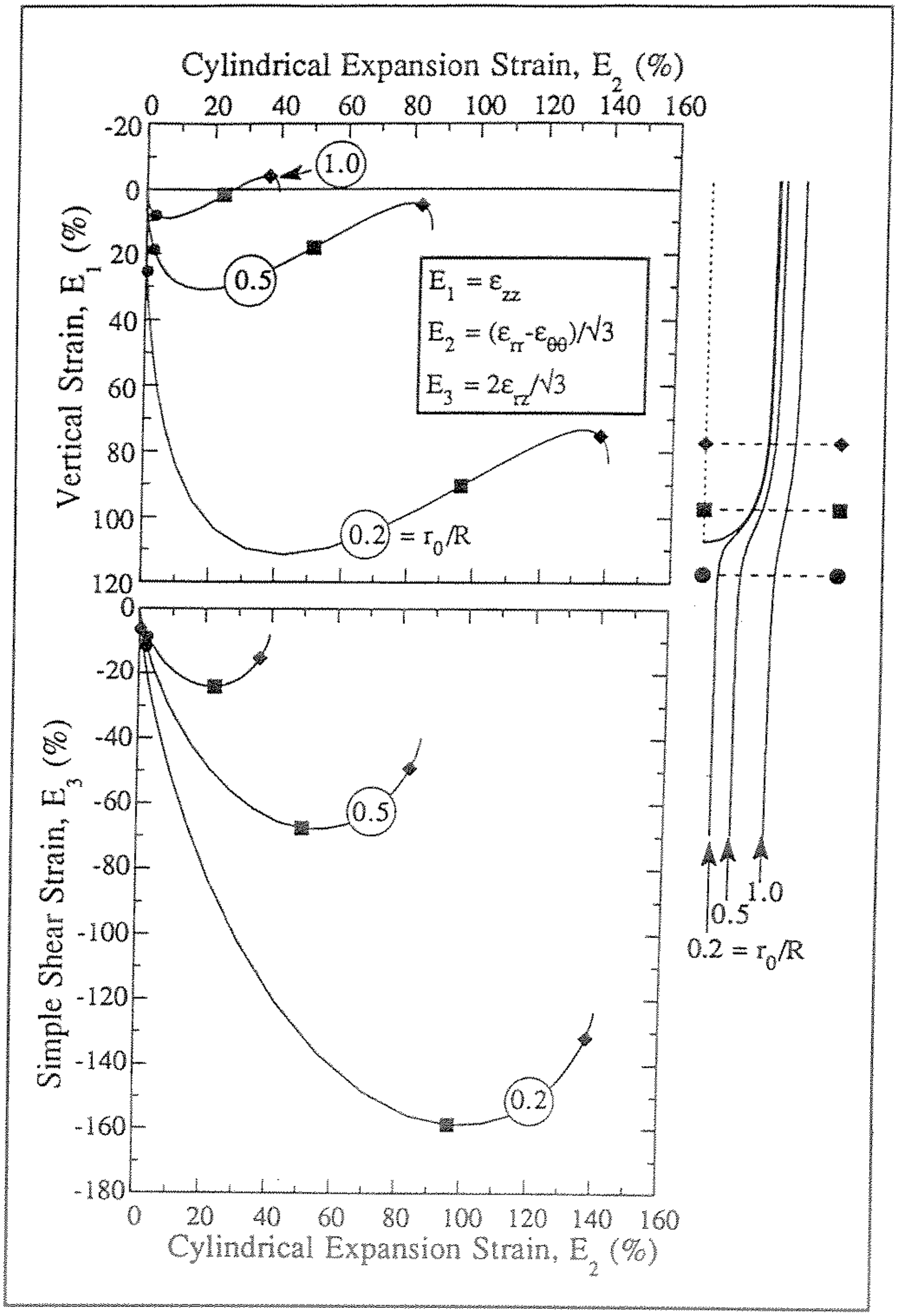


Figure 1a. Strain paths during simple pile penetration (after Baligh, 1985)

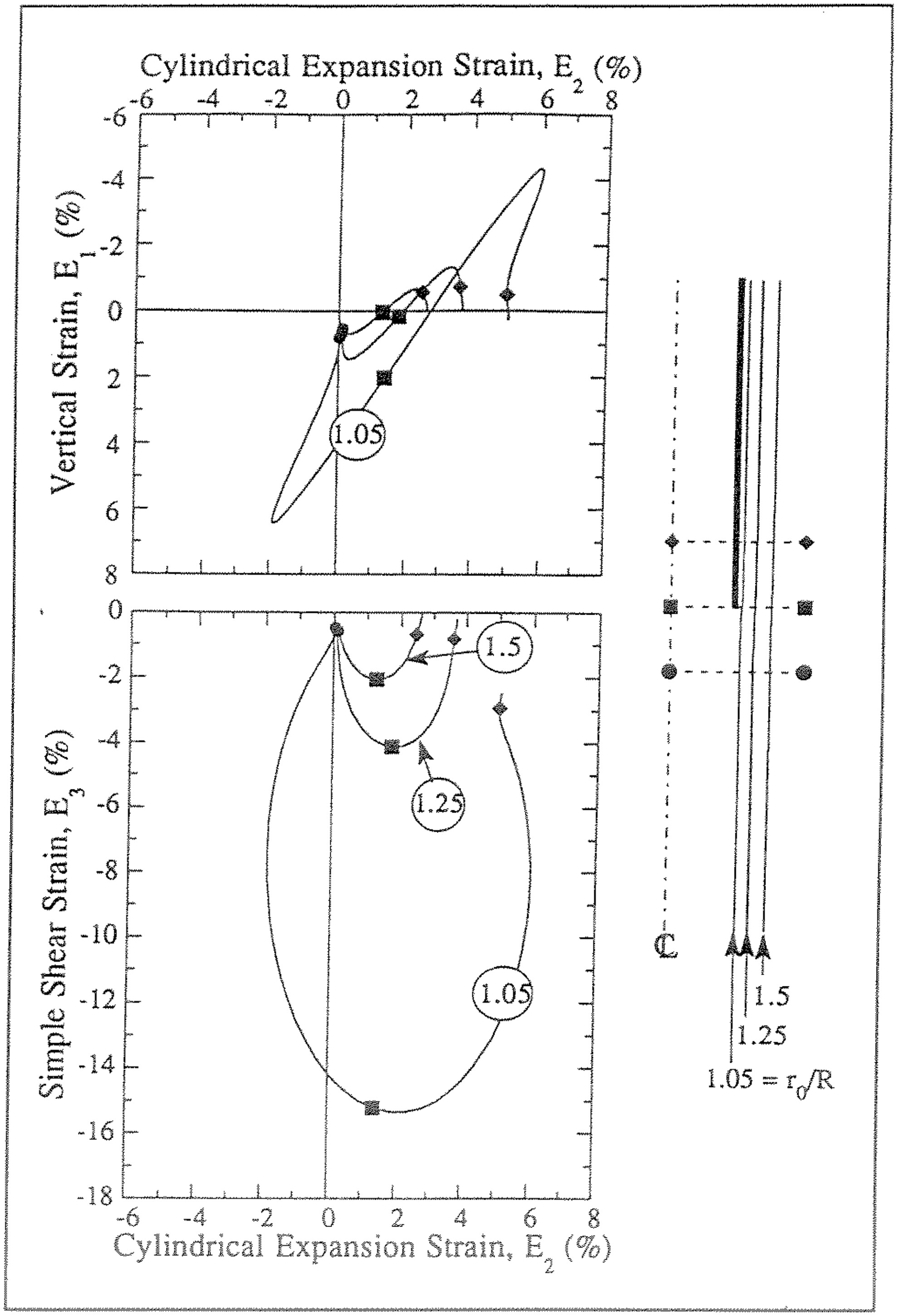


Figure 1b. Strain paths during unplugged penetration of open-ended pile, B/t=40 (after Chin, 1986)

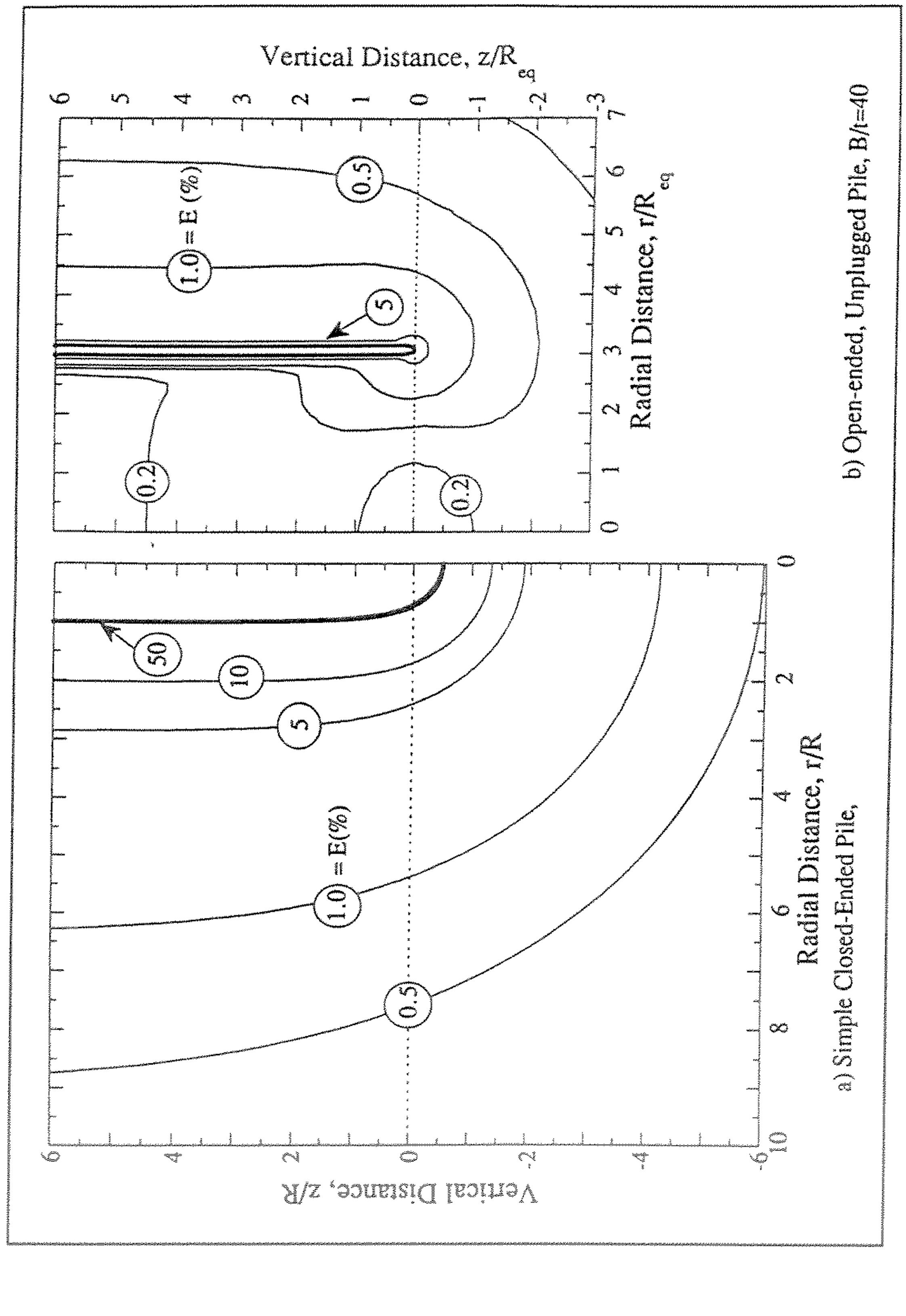


Figure 2. Shear strains during pile installation

At radial locations further from the pile shaft, E_1, E_3->) (e.g., $r_0/R = 1.0$; Figure 1a) and the final strain state is controlled by the volume of soil displaced by the pile.

The unplugged, open-ended pile causes much less disturbance of the surrounding soil. The zone of high shear strains ($E \geq 10\%$; Figure 2b) is confined to a thin annulus (comparable to the thickness of the pile wall) around the shaft, while a far field strain levels are controlled by the volume of soil displaced by the pile. In order to compare the strain levels for the open- and closed-ended piles, it is convenient to normalize the radial dimensions by the equivalent radius of a solid section pile, $R_{eq} \approx \sqrt{Bt}$ (i.e., for $B/t = 40$, $R_{eq} = 0.316R$) as shown in Figure 2b. The mode of penetration (closed vs. open-ended) also causes important differences in strain paths close to the pile wall, especially in the pressuremeter shear component, E_2.

In this presentation of the Strain Path Method, effective stresses, σ'_{ij}, are computed directly from the strain paths of individual soil elements using a generalized effective stress-strain soil model (see next section). This approach can be contrasted with previous total stress analyses (Levadoux & Baligh 1980; Baligh 1986a; The & Houlsby 1991) which compute shear stresses through a deviatoric stress-strain model, and introduce a separate constitutive relationship for shear induced pore pressures. The main advantage of the effective stress analysis is that the same soil model can be used to study stress changes during consolidation and pile loading.

The installation excess pore pressures around the pile shaft are computed from the effective stresses by satisfying conditions of radial equilibrium (Baligh 1986b). Further predictions of excess pore pressure distributions around the tip of the pile are difficult to achieve due to approximations used in the Strain Path Method. The most reliable estimates of pore pressure distributions are obtained by solving equilibrium conditions in the form of a Poisson equation using finite element methods (Aubeny 1992; Whittle & Aubeny 1992).

The MIT-E3 Effective Stress Soil Model

The MIT-E3 model (Whittle 1987, 1990, 1991a) is a generalized effective stress soil model for describing the rate independent behavior of normally to moderately overconsolidated clays ($OCR \leq 8$) which exhibit normalized engineering properties. The model describes a number of important aspects of soil behavior which have been observed in laboratory tests on K_0-consolidated clays but are not well described by most existing soil models:

1. There is no well defined linear region of soil behavior, even at small strain levels or immediately following a reversal of loading.

2. The unload-reload behavior of clays is characterized by a hysteretic response, but also involves small irrecoverable strains.

3. Clays exhibit anisotropic properties due to their consolidation history and subsequent straining.

4. In some modes of deformation, normally and lightly overconsolidated clays exhibit undrained brittleness.

5. Uniform, undrained cyclic loading of overconsolidated clays causes an accumulation of shear induced pore pressures. Thus, coupling of volumetric and shear behavior is essential to accurate modeling of overconsolidated clays under cyclic loading.

The model formulation comprises three components: 1) an elasto-plastic model for normally consolidated clays, which describes anisotropic properties and strain softening behavior; 2) equations for the small strain non-linearity and hysteretic stress-strain response in unload-reload cycles; and 3) bounding surface plasticity of irrecoverable, anisotropic and path dependent behavior of overconsolidated clays. Other observations of clay behavior, collectively referred to as 'rate effects' (e.g., variation in undrained shear strength with strain rate, undrained creep and secondary compression) are not described by the MIT-E3 model. The model used 15 input parameters which are determine from standard types of laboratory tests:

1. One dimensional consolidation tests (either incremental oedometer or constant rate of strain consolidation) using a load sequence that includes at least one cycle of unloading-reloading and measurements of lateral effective stresses.

2. Undrained shear test on K_0-consolidated clay including in triaxial compression (CK_0UC) at OCR = 1, 2 and triaxial extension (CK_0UE) at OCR = 1. These tests should be performed using SHANSEP consolidation procedures in order to ameliorate the effects of sample disturbance on the measured soil behavior (Ladd & Foott 1974).

3. Measurements of elastic shear wave velocity using either resonant column apparatus or field cross-hole tests. Alternatively, reliable measurements of the small strain stiffness can now be obtained from local strain measurements in triaxial tests (e.g., Jardine et al. 1984; Dyvik & Olson 1989; Clayton et al. 1989; Goto et al. 1991).

The model input parameters have been selected for a number of clays using a standard procedure (Whittle 1990). Extensive comparisons with measured data in undrained shear tests performed in different modes of shearing and with overconsolidation ratios (OCR) up to eight have shown that the model a) gives excellent predictions of peak shear resistance and can describe accurately the non-linear stress-strain behavior, but becomes less reliable for OCR≥4. The most comprehensive evaluations have been presented for Boston Blue clay (BBC), a low plasticity (I_p = 19-23%), illitic, marine clay of moderate sensitivity (s_t = 3-7) whose engineering properties have been studied extensively at MIT (Whittle 1990; Whittle et al. 1992). Figure 3 compares the computed and measured shear stress strain behavior for K_0-normally consolidated BBC in the three modes of shearing which occur during pile installation (cf., Figure 1).

The measured data in undrained triaxial compression and extension tests (CK_0UC and CK_0UE) at OCR = 1 illustrate important aspects of the anisotropic behavior of soft clays. The undrained shear strength in the compression mode ($s_{uTC}/\sigma'_{vc} = 0.33$) is mobilized at very small shear strains ($\varepsilon_{ap} \approx 0.3 - 0.5\%$) and there is a significant post-peak reduction in shear resistance. In comparison, the undrained shear strength in triaxial extension is mobilized at relatively large strain levels ($s_{uTE}/\sigma'_{vc} \approx 0.14$ at $\varepsilon_{ap} > 5\%$). There is a large difference in the undrained shear strengths in the two modes of shearing, $s_{uTE}/s_{uTC} = 0.42$. The MIT-E3 model matches closely the measured undrained shear strengths in both modes of shearing, as well as the axial strain at peak resistance and post-peak strain softening. The measurements in monotonic compressions and extension tests are part of the data set used to select input parameters and hence, these comparisons do not constitute an evaluation of model predictions. Model predictions for an undrained triaxial test with a single unload-reload cycle (i.e., two reversals of strain direction) are also shown in Figure 3a. The model describes closely the non-linear and hysteretic nature of the unload-reload process, but tends to overpredict the stiffness during reloading.

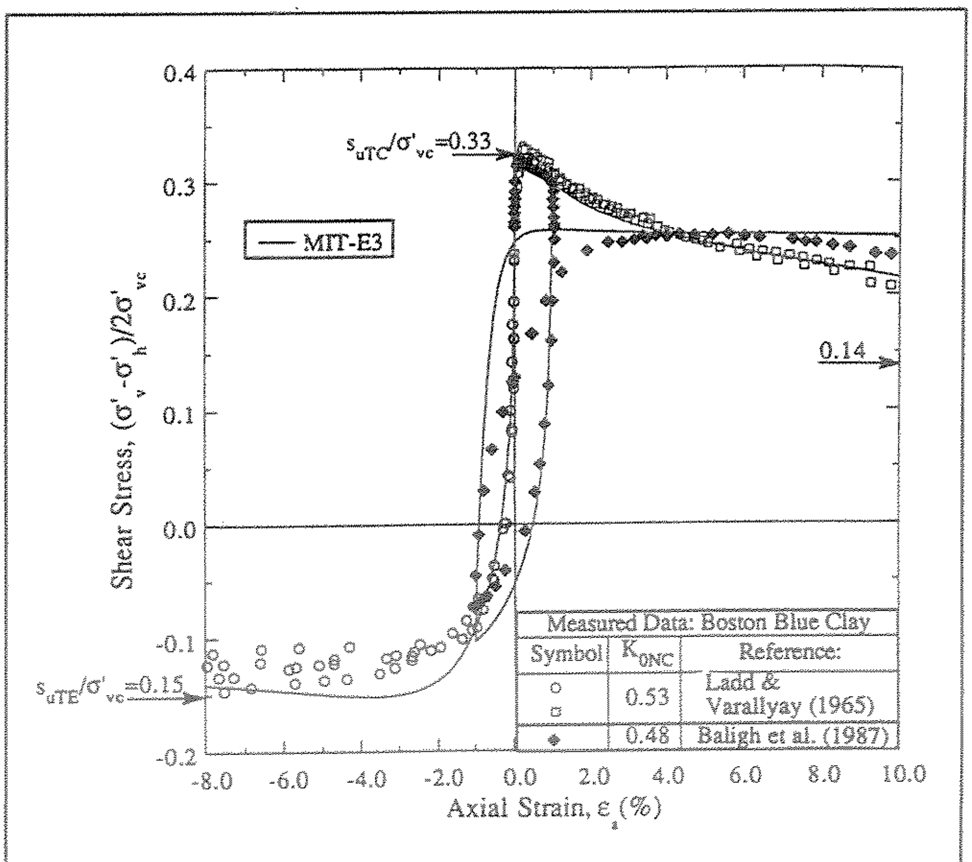


Figure 3a. Comparison of MIT-E3 predictions and measured data for undrained triaxial shearing of K_0-normally consolidated Boston Blue Clay

Figure 3b compares predictions with measurements in undrained Direct Simple Shear tests (CK_0UDSS) using a Geonor simple shear apparatus. This mode of shearing is also directly relevant to predictions of pile-soil behavior in axial loading (e.g., Randolph & Wroth 1981; Azzouz et al. 1990) and is discussed in more detail in the axial pile loading section. The MIT-E3 model gives very good predictions of the shear stress strain response in tests with monotonic shearing and with reversals of strain direction. The model is in excellent agreement with the measured peak shear resistance ($\tau_{max}/\sigma'_{vc} = 0.21$), but tends to overestimate the stiffness in reloading.

The pressure meter shear mode (E_2; Figure 1) is especially important for estimating the effects of pile installation and can be simulated in laboratory element tests using more sophisticated equipment such as the True Triaxial Apparatus (TTA) or Directional Shear Cell (DSC). Unfortunately, there is very little data of this type reported in the literature. Figure 3c compares model predictions with data reported by Wood (1981) using the Cambridge University TTA. The model predictions match the measured peak shear resistance, $s_{uPM}/\sigma'_{v0} = 0.21$, which is mobilized at a shear strain, $\gamma \approx 5\%$. Further comparisons with more comprehensive pressure meter tests in the DSC at OCR = 4 (O'Neill 1985; Figure 3c) show similar predictive accuracy for the peak shear resistance, but confirm that the model tends to overestimate the prepeak shear stiffness.

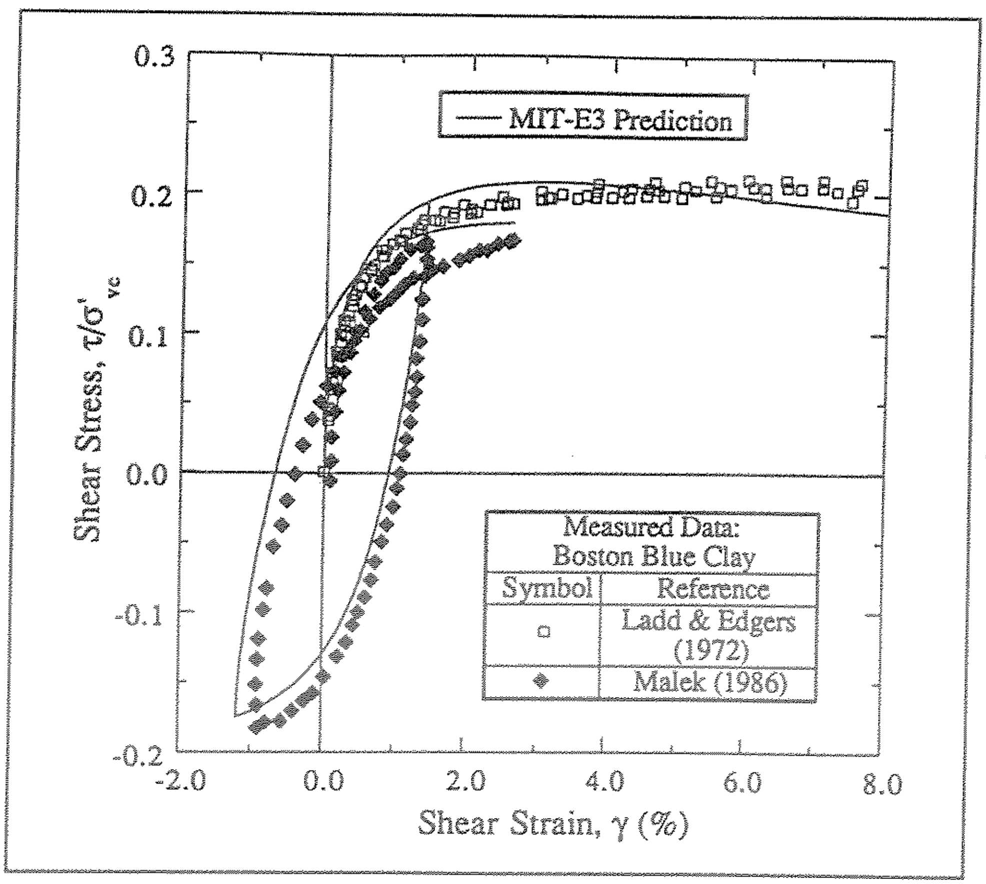


Figure 3b. Comparison of model predictions and measured data for direct simple shear of K_0-normally consolidated Boston Blue Clay

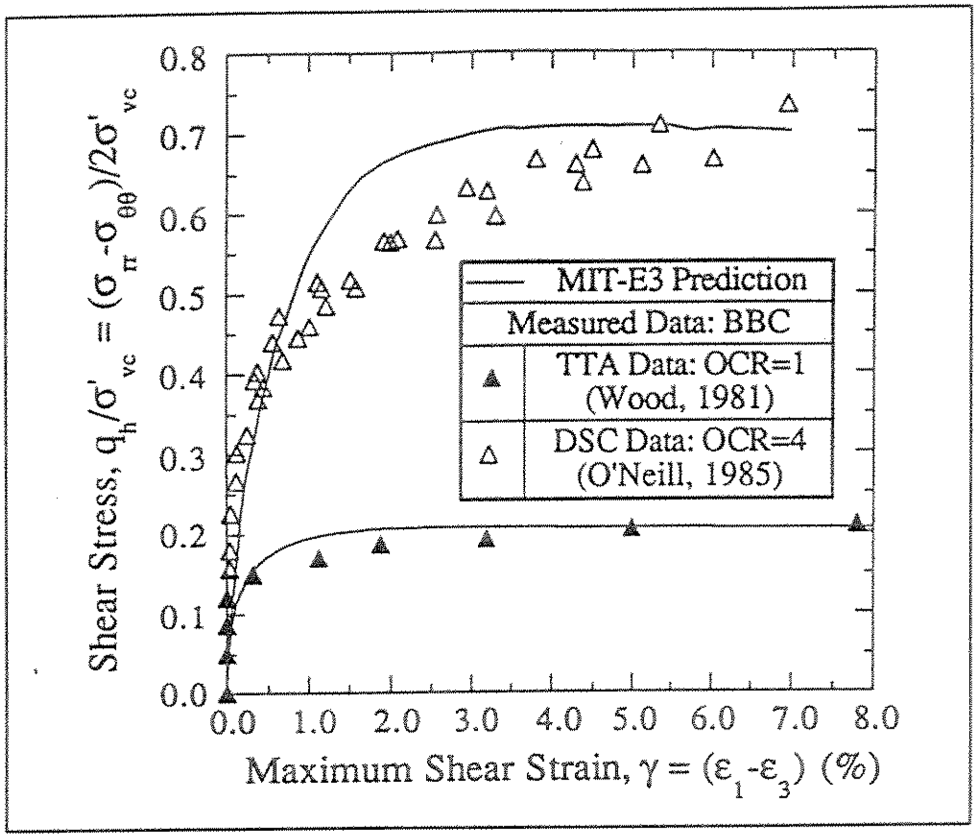


Figure 3c. Comparison of model predictions and measured data for a pressure meter mode of shearing of K_0-consolidated Boston Blue Clay

The shear resistance at large strain levels is important in predicting stress conditions close to the pile shaft during installation. However, it is difficult to obtain reliable large strain measurements in laboratory tests due to nonuniformities in stress conditions, strain localization etc. The predictions in Figure 3 show that there is a large post-peak reduction in the shear resistance of K_0-normally consolidated BBC in undrained triaxial compression ($s_{ur}/s_u \approx 0.4$), but negligible softening in the pressure meter shear mode. Strain softening measured in Direct Simple Shear tests may be partly attributed to non-uniform stress conditions in the Geonor apparatus (e.g., DeGroot et al. 1992).

Although it is not possible to duplicate the complex strain paths caused by pile installation using existing laboratory tests, the results in Figure 3 demonstrate the predictive capabilities of MIT-E3 and provide a sound basis for applying the model in conjunction with SPM analyses.

Pile Installation

Predictions of installation conditions

Figure 4 presents Strain Path (SPM) and Cavity Expansion (CEM) predictions of installation stresses and pore pressures around the shaft of a pile in

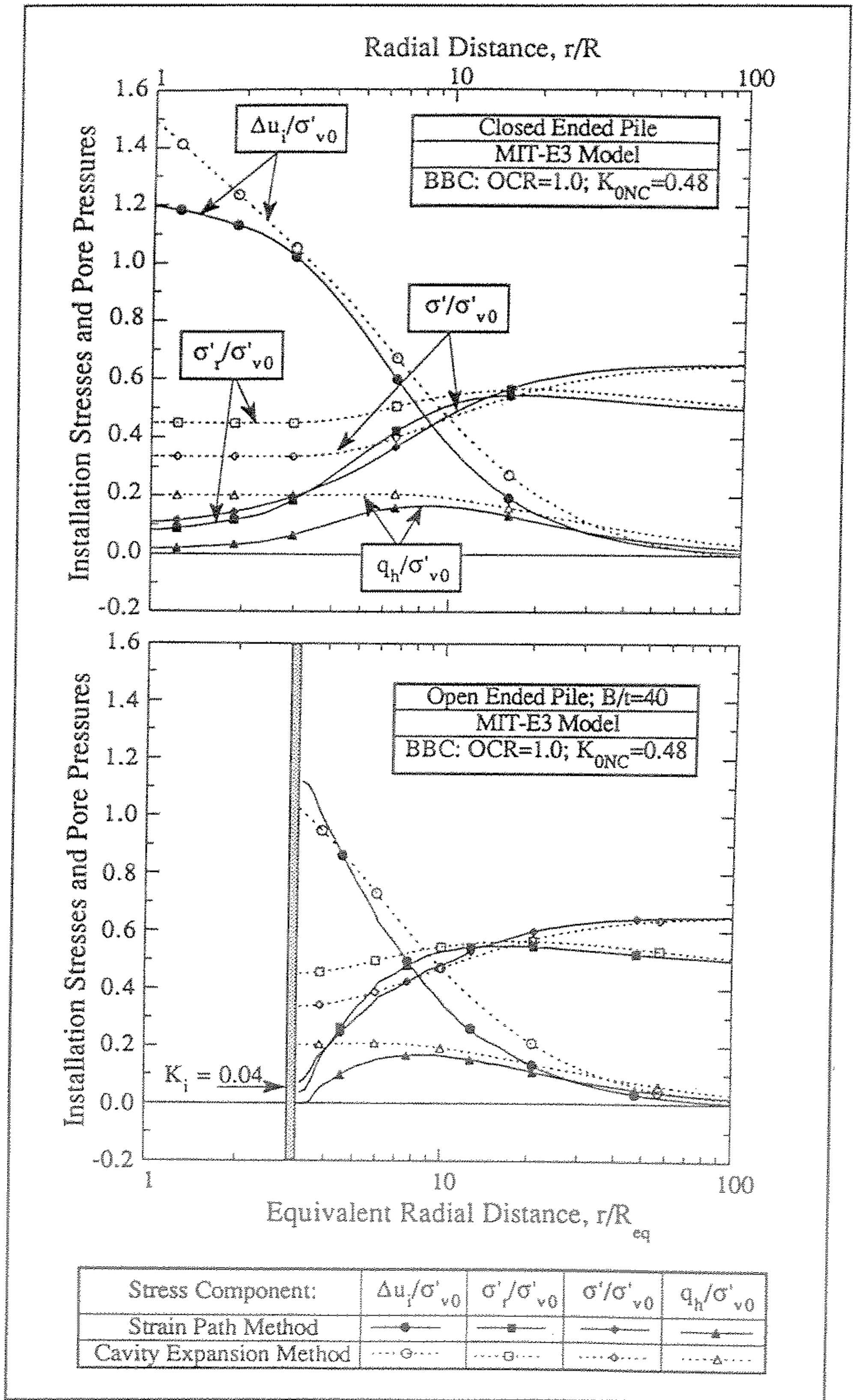


Figure 4. Strain path predictions of installation stresses in K_o-normally consolidated BBC

K_0-normally consolidated BBC using the MIT-E3 soil model. The individual stress components are normalized by the in situ vertical effective stress, σ'_{v0}, while radial dimensions are related to the equivalent radius of a solid section pile, R_{eq}, in order to unify results for the two limiting modes of pile penetration (plugged and unplugged). The two principal parameters of interest in these analyses are the excess pore pressures, $\Delta u_i/\sigma'_{v0}$, and radial effective stresses, $K_i = \sigma'_{ri}/\sigma'_{v0}$ which can be measured at the pile shaft. The results in Figure 4 show the following:

1. For a normally or lightly overconsolidated clay, undrained shearing generates positive shear induced pore pressures and a corresponding net reduction in the mean effective stress, σ'/σ'_{v0} close to the pile shaft. Differences in the magnitude of σ'/σ'_{v0} for SPM and CEM analyses ($r/R_{eq} \leq 6$; Figure 4a) reflect how the anisotropic and strain softening properties described by the MIT-E3 model are affected by differences in strain histories.

2. The effects of the analysis used to model installation can be seen most clearly in predictions of the radial effective stress, σ'_r/σ'_{v0} and cavity shear stress, q_h/σ'_{v0} (where $q_h = [\sigma'_r - \sigma'_\theta]/2$ is the maximum shear stress in the horizontal plane). The Strain Path Method predicts very low radial effective stresses ($K_i = 0.08 - 0.10$) and cavity shear stresses ($q_h/\sigma'_{v0} > 0$) acting at the pile shaft for both modes of penetration. This means that the radial effective stress is similar in magnitude to the mean effective stress (i.e., $\sigma'_r/\sigma'_{v0} \approx \sigma'/\sigma'_{v0}$) for $r/R_{eq} \leq 15$. In contrast, CEM analyses give higher values of radial effective stress σ'_r/σ'_{v0} and predict that $\sigma'_r/\sigma'_{v0} > \sigma'/\sigma'_{v0}$ over a wide radial zone ($r/R \leq 20$). The cavity shear stress, $q_h/\sigma'_{v0} \approx 0.20$, is approximately constant for $r/R < 7$ and can be deduced from the pressure meter shear behavior described in Figure 3c. Strain path predictions of σ'_r/σ'_{v0} are affected significantly by soil properties (including strain softening), while Baligh and Levadoux (1980) show that the geometry of the pile tip has an important effect on the cavity shear stress close to the shaft.

3. The excess pore pressures at the pile shaft are obtained from conditions of radial equilibrium and hence depend on the entire field of effective stresses in the soil. The results in Figure 4a show that undrained pile installation generates large excess pore pressures in the soil which extend to a radial distance $r/R = 20 - 30$. Although both CEM and SPM predict a similar accumulation of excess pore pressure in the far field ($3 \leq r/R \leq 30$), there are significant differences in the distribution close to the pile shaft ($r/R \leq 3$). The net result is that the cavity expansion method predicts excess pore pressures which are typically 20-25 percent larger than those obtained from corresponding strain path analyses. The characteristic shapes of the pore pressures distributions (CEM and SPM; Figure 4a) have been discussed in detail by Baligh (1986b) and are not affected significantly by the modeling of soil behavior.

4. The mode of penetration (Figures 4a, 4b) only affects the magnitude and distribution of stresses and pore pressures close to the pile wall, $r/R_{eq} \leq 3$.

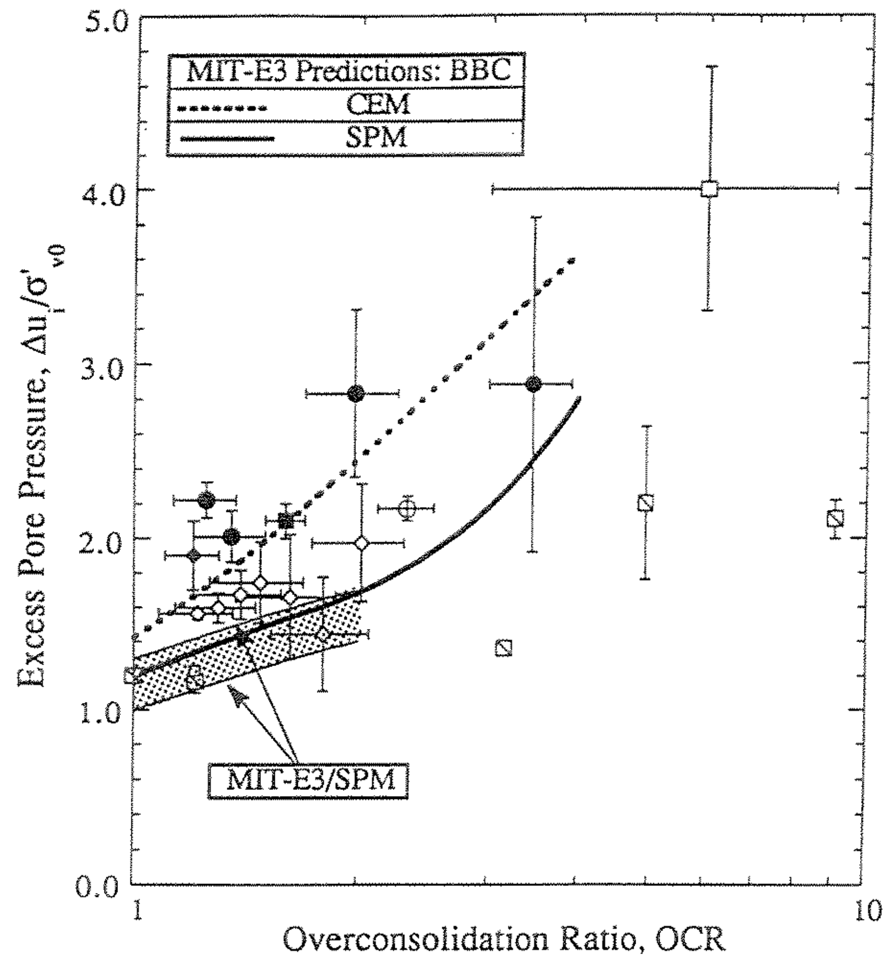
The strain path predictions of cavity shear stress, radial and mean effective stress components acting at the shaft are very similar for both modes of penetration, while the excess pore pressures are slightly smaller for the unplugged pile. It is interesting to note that the strain path method actually predicts slightly larger shaft pore pressures than the CEM analyses for the unplugged pile.

Evaluation of installation predictions

Simultaneous measurements of shaft pore pressures and lateral earth pressures (radial total stresses) during installation have been obtained at a number of sites using a) instrumented pile shaft elements or probes(PLS cell, Morrison 1984; t-z and x-probes, Bogard et al. 1985; IMP, Coop & Wroth 1989), and b) instrumented model piles (Karlsrud & Haugen 1985; Karlsrud et al. 1992; Bond et al. 1991). Further measurements of installation pore pressures are associated with the development of in situ testing devices such as the piezocone and include both field tests and laboratory experiments in large-scale calibration chambers. The reliability of these measurements depends, in large part, on the design of the instrumentation (response time, calibration for thermal changes, etc.) and quality of test procedures (de-airing of porous filters etc.) can also affect the measured parameters (e.g., Azzouz & Morrison 1988) due to factors such as partial drainage which are not considered in the analysis.

Figure 5 compares the predictions of excess pore pressures at the pile shaft with measurements obtained during steady penetration in BBC using the PLS cell (Morrison 1984; Azzouz & Morrison 1988) as functions of the stress history (OCR). The measured data are very consistent at low OCR, but exhibit large scatter in the more overconsolidated clay due to the presence of sand seams, etc. In general, the strain path predictions underestimate the measured excess pore pressures, particularly at low OCR, while there is better agreement with results from CEM analyses. The figure also includes field measurements from pile and penetrometer tests compiled from five other sites. The results tend to confirm the previous assessment of Baligh and Levadoux (1980) that there is no well define correlation between the shaft pore pressures and clay type (as described by plasticity index, I_p), stress history (OCR; Figure 5), undrained shear strength or sensitivity (s_t). At low OCR, the excess pore pressures measured at five sites are in the range $\Delta u_i/\sigma'_{v0} = 2.0 \pm 0.4$. Significantly lower installation pore pressures ($\Delta u_i/\sigma'_{v0} = 1.2 - 1.3$; Figure 5) have been reported recently from large-scale, laboratory calibration chamber tests in kaolin (May 1987; Nyirenda 1989).

In principle, measurements of the radial distribution of excess pore pressures (using piezometers in the surrounding soil) can provide a more comprehensive evaluation of the strain path predictions. In practice, these measurements are difficult to obtain (especially in the critical region close to the shaft, $1 \leq r/R \leq 5$) due to a) interference of the measuring device and the soil deformations, and b) uncertainties in the alignment and position of the piezometers. Figure 6 compares MIT-E3 predictions of the excess pore pressure distribution for BBC at OCR = 1.0, 2.0, and 4.0 with 1) field measurements (Roy et al. 1981) around an instrumented pile (R =11cm) installed in highly sensitive, structured St Alban



Measured Data		
Symbol	Clay	Reference
●	BBC	Morrison (1984)
○	St Alban	Roy et al. (1981)
◆	Onsøy	Karlsrud et al. (1992)
■	Empire	Azzouz & Lutz (1987
□	Haga	Karlsrud & Haugen 91985)
◇	Inchinnan	May (1987)
⊠	Kaolin	May (1987), Nyirenda (1989)

Figure 5. Evaluation of installation pore pressures at pile shaft

clay (liquidity index, $I_L \geq 2$) at OCR ≈ 2.3, and 2) measurements around the shaft of a cone pentrometer installed in K_0-normally consolidated kaolin (R = 1.26cm) within a large calibration chamber (of radius, R_c = 50cm). The SPM predictions at OCR = 1 are in very good agreement with penetration pore pressures measured in kaolin. The predictions at OCR = 2 underestimate both the magnitude and the radial extent of the zone of pore pressure accumulation in the

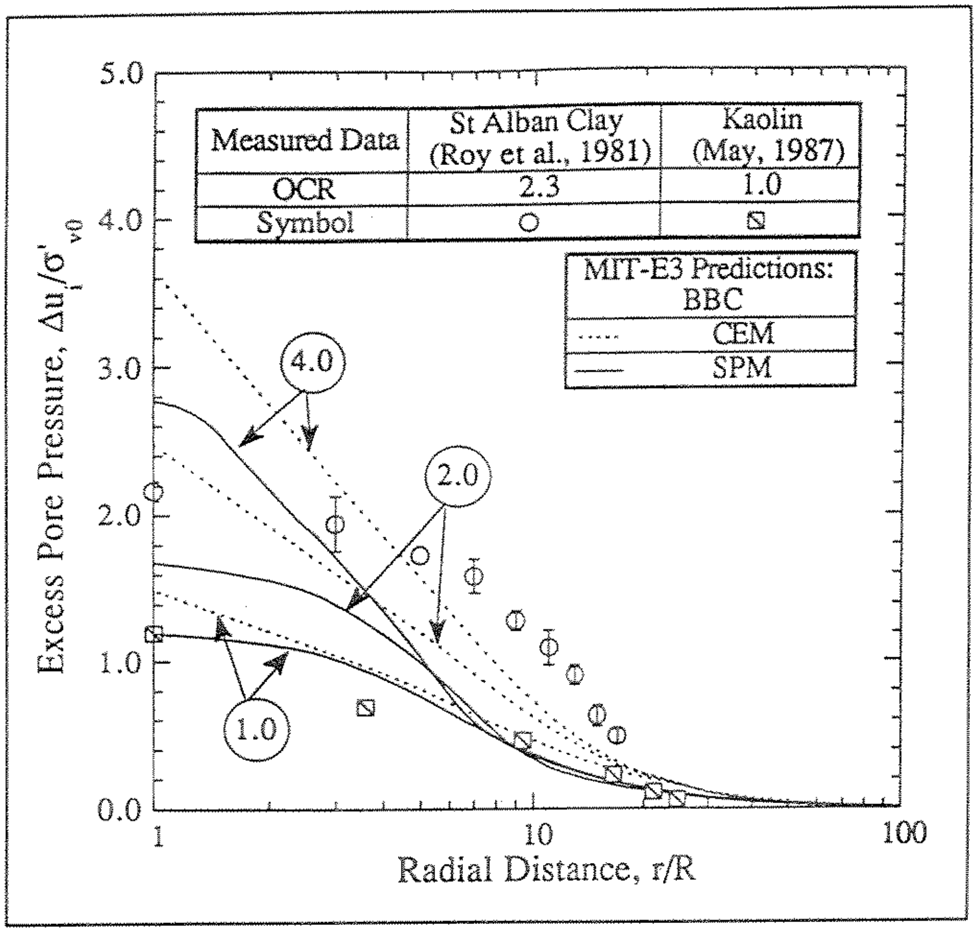


Figure 6. Distribution of excess pore pressures during installation

St Alban clay. These results indicate that the main source of discrepancy between strain path predictions and measured pore pressures are the effective stresses in the far field (r/R ≥ 5-6, where E ≤ 1%. Figure 2a), which are affected by small strain properties of the clay and can be addresses through further refinement of the constitutive model. In contrast, CEM analyses do not describe accurately the shape of the pressure distribution and hence, overestimate $\Delta u_i/\sigma'_{v0}$ at the shaft while underestimating pore pressures measured in the far field.

Overall, it is not possible to draw definitive conclusions from the results presented in Figures 5 and 6. Installation pore pressures around the pile shaft are very difficult to evaluate due to the complexity of the analysis and the sensitivity of the predictions to soil non-linearity and inelastic behavior. Predictions using the MIT-E3 model (with input parameters for BBC) suggest that, although the strain path method underestimates the installation excess pore pressures, it provides a more consistent description of the lateral distribution of effective stresses than corresponding CEM analyses.

Radial effective stress, K_i, during pile installation are obtained by subtracting the installation pore pressures from measurements of total radial stress at the same location. Figure 5 shows that pile installation in normally and lightly overconsolidated clays generates large excess pore pressures in the surrounding soil. It is therefore apparent that small errors in the measured pore pressures (and/or total stress) can affect significantly the compute radial effective stress. Azzouz and Morrison (1988) report $K_i = 0.05\text{-}0.20$ from PLS measurements in the lower Boston Blue Clay (OCR =1.2 ± 0.1) which are in excellent agreement with strain path predictions (cf., Figure 4). Very small values of K_i are also reported from instrumented pile tests in other sensitive, low plasticity clays (Haga; Karlsrud & Haugen 1985; Onsøy, Karlsrud et al. 1992). Significantly larger radial effective stresses, $K_i = 0.38\text{-}0.54$, were measured in the more plastic, less sensitive Empire clay (Azzouz & Lutz 1986) and are also in good agreement with strain path predictions using the MIT-E3 model ($K_i = 0.37\text{-}0.45$; Whittle & Baligh 1988).

Consolidation

Non-linear analysis of radial consolidation

After pile installation, soil consolidation occurs duet to the dissipation of excess pore pressures around the pile. For shaft locations far from the pile tip and the mudline, it is assumed that excess pre pressures dissipate radially away from the shaft and there are concomitant changes in the effective stresses due to radial displacements of the soil. Thus the underlying mechanism of pile 'set-up' (changes in shaft capacity with time after installation) is attributed to changes in effective stresses in the soil at (or close to) the pile-soil interface due to radial consolidation.

Previous studies (Randolph et al. 1979; Baligh & Levadoux 1980; Baligh & Kavvadas 1980) have shown that predictions of the change in radial effective stress acting on the pile shaft during consolidation are strongly affected by non-linearity of the soil. For the special case of a linear, isotropic (and elastic) soil, the decrease in excess pore pressure is exactly balanced by an increase in radial effective stress, such that at the end of consolidation, $\sigma'_{rc} = \sigma'_{ri} + \Delta u_i$. Comparison with measured data shows that this leads to a vast overprediction of the set-up around the pile shaft in soft clay deposits. Thus, comprehensive analyses of the couple non-linear consolidation (i.e., coupling of total stresses and pore pressures; non-linear effective stress-strain and permeability properties of the soil) are required in order to achieve reliable predictions of the set-up process. The analyses described in this section solve the radial consolidation by non-linear finite element methods using with the following assumptions:

1. Initial conditions are described by the radial distributions of effective stresses and pore pressures predicted around the shaft during pile installation using the Strain Path Method.

2. The non-linear effective stress-strain response of soil elements around the pile shaft is represented consistently by the MIT-E3 model (i.e., with the same input parameters used during pile installation). The compressibility

of the soil is a function of the radial location of the soil element (due to installation disturbances) and varies with effective stresses during consolidation.

3. Non-linearities associated with changes in soil permeability are not considered in the analysis. This assumption implies that the radial permeability is spatially constant after pile installation and that changes in permeability during consolidation can be neglected. Recent experimental data from model tests on resedimented BBC suggest that permeability can decrease by up to a factor of 3 at locations close to the pile shaft following complete set-up (Ting et al. 1990). However, these changes in permeability are small compared with inherent uncertainties in the measurement of permeability of natural deposits and with non-linearities associated with soil stiffness predicted during consolidation.

Figure 7 summarizes predictions of the excess pore pressures, $\Delta u/\sigma'_{v0}$, radial total and effective stresses ($H = (\sigma_r - u_0)/\sigma'_{v0}$ and σ'_r/σ'_{v0}, respectively) acting at the shaft of a pile installed in K_0-normally consolidated BBC. The results are presented using a dimensionless time factor T, which is defined as:

$$T = \frac{\sigma'_0 k t}{\gamma_w R_{eq}^2} \qquad (1)$$

where

$\sigma'_0 = 1/3(1+2K_0)\sigma'_{v0}$ = in situ mean effective stress in the ground
t = time
R_{eq} = equivalent radius
k = (horizontal) coefficient of permeability
γ_w = unit weight of water.

The principal parameter of interest in the analysis is the magnitude of the radial effective stress acting on the pile shaft after full dissipation of excess pore pressures $K_c = \sigma'_{rc}/\sigma'_{v0}$. For the closed-ended pile example shown in Figure 7, the analyses using the Strain Path Method and MIT-E3 model predict a final set-up stress ratio, $K_c = 0.37$ (Figure 7a) which is significantly smaller than the initial, in situ earth pressure coefficient ($K_0 = 0.48$), while comparable linear, consolidation analyses (i.e., based on the same installation conditions) would give $K_c = 1.28$ ($= K_i + \Delta u_i/\sigma'_{v0}$). Thus, the predictions of set-up stresses vary by a factor of 3 to 4 depending on the modeling of non-linear soil behavior. The set-up stresses predicted for the open-ended (unplugged) pile are approximately 25 percent lower than for the closed-ended pile.

Predictions using cavity expansion analysis of pile installation (CEM) show significantly higher set-up stresses ($K_c = 0.72$; Figure 7a). This result is primarily due to the predicted initial conditions ($K_i = 0.45$; Figure 4) since the net change in radial effective stress during consolidation is relatively small.

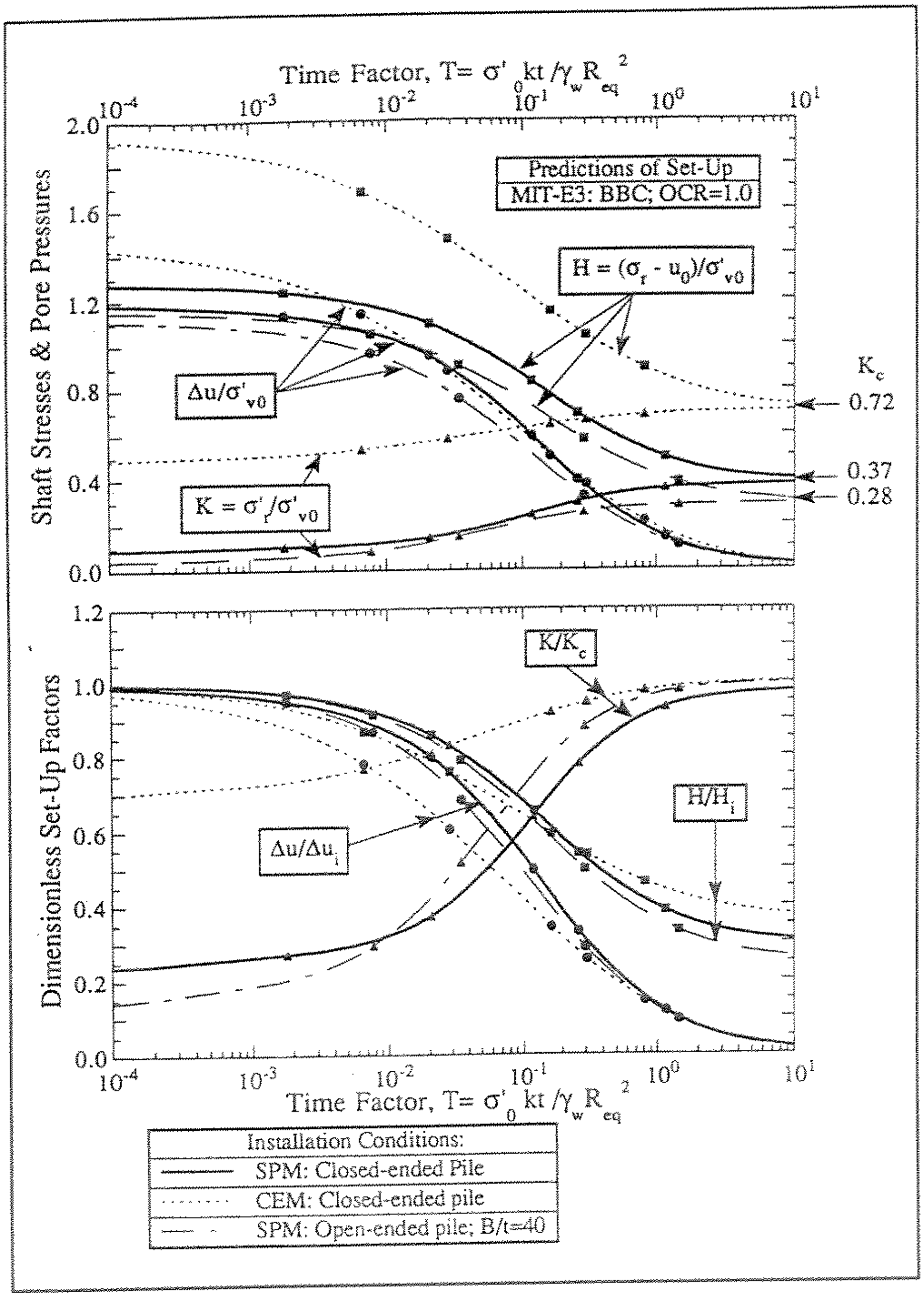


Figure 7. Typical predictions of consolidation at the pile shaft in K_0-normally consolidated BBC

Further insights into factors affecting the set-up predictions can be achieved by introducing the dimensionless stress ratios shown in Figure 7b. The excess pore pressure ratio (degree of consolidation), $U = \Delta u/\Delta u_i$, depends primarily on the normalized radial distribution of installation excess pore pressures ($\Delta u/\Delta u_{sh}$ where Δu_{sh} is the excess pore pressure at the pile shaft; Baligh & Levadoux 1980). Strain path analyses for both closed and open-ended piles give very similar rates of consolidation (U vs T; Figure 7b), while CEM predictions show more rapid dissipation of pore pressures for $U \geq 0.3$. The set-up effective stress ration, K/K_c , illustrates most clearly the importance of the installation analysis (SPM vs CEM). The strain path results in Figure 7b show that the largest change in set-up stresses occurs over the time period $0.01 \leq T \leq 1.0$. In contrast, the total stress release ratio, H/H_i include the shear behavior at large shear strains (i.e., during installation) and the radial compressibility during consolidation. The MIT-E3 predictions for normally consolidated BBC show $H_c/H_i = 0.25\text{-}0.3$ (Figure 7b), while results obtained for other soils and stress histories range from $H_c/H_i = 0.2$ to 0.6 (Whittle & Baligh 1988).

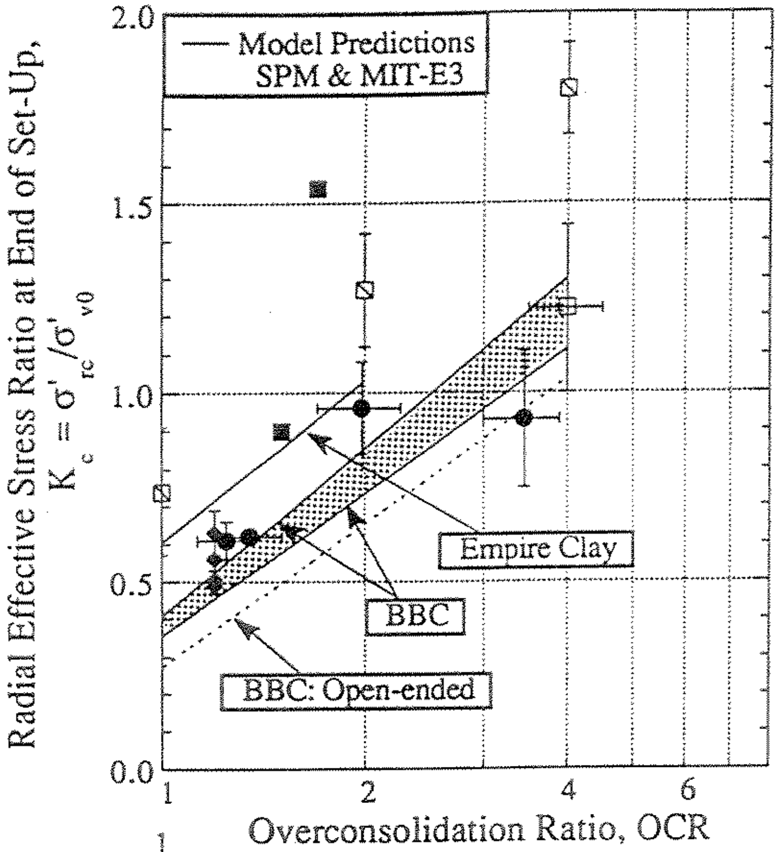
Evaluation of shaft stresses during set-up

Effective stresses at the pile shaft are currently calculated from measurements of total stresses and pore pressures during consolidation (e.g., Azzouz & Morrison 1988; Karlsrud & Haugen 1985), although direct measurements of σ'_r have been reported recently in a highly overconsolidated clay (using effective pressure transducer; Karlsrud et al. 1992). In addition to obvious sources of experimental errors, there are two main difficulties in determining accurately the set-up stress ratio, $K_c = \sigma'_{rc}/\sigma'_{v0}$ (Azzouz et al. 1990): 1) incomplete consolidation, and 2) limitations of total lateral stress measurements.

The time delay required to achieve complete consolidation depends on the (equivalent) radius of the pile and the soil properties (permeability and compressibility). For example, Azzouz and Morrison (1988) report consolidation times of 4-8 days using the PLS cell ($R \approx 1.9$cm), while 1-2 months of set-up are typical of instrumented model piles with $R = 7\text{-}10$cm (e.g., Karlsrud et al. 1992). In practice, K_c values are frequently quoted from measurements obtained with incomplete consolidation. In these situations, calculations using measurement of σ_r and u tend to underestimate K_c; while those based on σ_r and u_0 (in situ pore pressures) overestimate the effective set-up stress.

Measurements of pore pressures at later stages of consolidation can easily be checked through comparison with known values of u_0. Thus, most of the uncertainties in σ'_{rc} are due to possible errors in total stress measurements which are commonly caused by: a) significant seating errors (zero reading) of the σ_r cell due to soil arching; b) the cross sensitivity of σ_r measurements to changes of the axial load in the pile; and c) the changes of total stress cell calibration with time after installation (zero shift). Significant improvements in the design of instrumentation (e.g., Bond et al. 1991; Karlsrud et al. 1992) represent and important contribution in reducing these errors.

Figure 8 compares the predictions of the effective stresses at full set-up with measurements obtained by the PLS cell in BBC (Morrison 1984; Azzouz & Morrison 1988) as functions of the stress history. The predicted K_c increases significantly with the OCR and range from $K_c = 0.37\text{-}0.42$ at OCR = 1 (for $K_{0NC} = 0.48\text{-}0.53$) to $K_c = 1.1\text{-}1.30$ at OCR = 4 (where $K_0 = 0.75 \text{-}1.0$). Although the set-up stress is similar in magnitude to the in situ earth pressure, the average ratio K_c/K_0 ranges from 0.8 at OCR =1, to 1.4 at OCR = 4. The PLS



Measured Data		
Symbol	Clay	Reference
●	BBC	Morrison (1984)
◆	Onsøy	Karlsrud et al. (1992)
■	Empire	Azzouz & Lutz (1986)
□	Haga	Karlsrud & Haugen (1985)
⧅	Kaolin	Francescon (1983)

Figure 8. Evaluation of radial stress at end of set-up

measurements are generally in very good agreement with the predictions and support the result that K_c increases with OCR, although there is a large scatter in the data for OCR≥3. The figure also includes field measurements of K_c at three other sites (Empire, Onsøy and Haga) together with laboratory tests on a miniature pile in kaolin. There is consistent agreement in the data obtained for three clays of moderate sensitivity (s_t = 3-7; BBC, Onsøy and Haga); while higher set-up stresses are measured in the Empire clay and kaolin. Although he predictions of K_c for Empire clay (Figure 8) are 50 - 60 percent higher than those described for BBC, they still underpredict the PLS measurement in zone I (cf., Azzouz & Lutz 1986). No analyses (using SPM and MIT-E3) have yet been performed for the kaolin, however, Azzouz et al. (1990) suggest that these data are affected significantly by boundary conditions.

The results in Figure 8 indicate that the combination of strain path installation analyses and non-linear radial consolidation with MIT-E3 are capable of providing good predictions of the effective stresses at the end of set-up. However, more comprehensive predictions are necessary to establish how predictions of K_c are related to engineering properties of the soil.

Figures 9 and 10 present a detailed evaluation of the predictions of set-up behavior at the pile shaft based on PLS measurements (Morrison 1984) in the lower deposit of BBC (OCR = 1.3±0.1). The consolidation predictions are presented at OCR = 1.0 and 1.5 using a modified time factor, $T = \sigma'_p kt/\gamma_w R^2$, where σ'_p is the vertical preconsolidation stress.

This time factor unifies predictions of the consolidation rate (U vs T; Figure 9b) for normally and lightly overconsolidated BBC (Aubeny 1992). Measurements of the total radial stress and excess pore pressure ratios (H/H_i and U, respectively) are then compared with the predictions (Figure 9). An average coefficient of horizontal permeability, $k_h = 8 \times 10^{-7}$ cm/sec is selected from laboratory CRS and constant head measurements which range form $3.0 \times 10^{-8} \leq k_h \leq 1.5 \times 10^{-7}$ cm/sec in the lower BBC (no significant variation with depth in the deposit. The predictions are in excellent agreement with the measured excess pore pressure ratio throughout consolidation, but tend to underestimate slightly the reduction in total radial stress. Effective stress changes at the pile shaft during consolidation, K(T), are calculated from the average measurements of total radial stress and excess pore pressure (Figure 10). These data agree with predictions of radial effective stresses for OCR = 1.5 and confirm the capabilities of the analysis for describing pile shaft performance throughout the set-up process.

Stress conditions in the soil after consolidation

Predictions of the stress state in the soil after consolidation have an important influence on the subsequent prediction and interpretation of pile shaft capacity. Figure 11 presents predictions of the stress distribution around the pile shaft in BBC at OCR = 1 and 2. The results show a net reduction in the mean effective stress (σ'_c/σ'_{v0}) in the soil around the pile (i.e., compared to the in situ conditions). Radial consolidation produces cavity shear stresses (q_{hc}/σ'_{v0}) in the soil extending to a distance $r/R \leq 3$ (cf. Figure 4). At the pile shaft, σ'_{rc} is the major principal effective stress, while $\sigma'_{\theta c}/\sigma'_{rc} \approx K_{0NC}$. Figure 12 compares the volumetric behavior (i.e., mean effective stress and volumetric strain) of soil elements adjacent to the pile shaft with the K_0-Virgin Consolidation Line (K_0-VCL) predicted for the undisturbed clay (using MIT-E3). Pile installation (paths A-B) generates large shear induced pore pressures (reductions in σ') associated with severe shearing of the soil at constant water content. During set-up (paths B-C) the soil elements do not return towards the K_0-VCL, but instead exhibit a more compressible behavior such that the final states of stress (C_1, C_2) coalesce on a new re-consolidation line which lies parallel to the compression of the

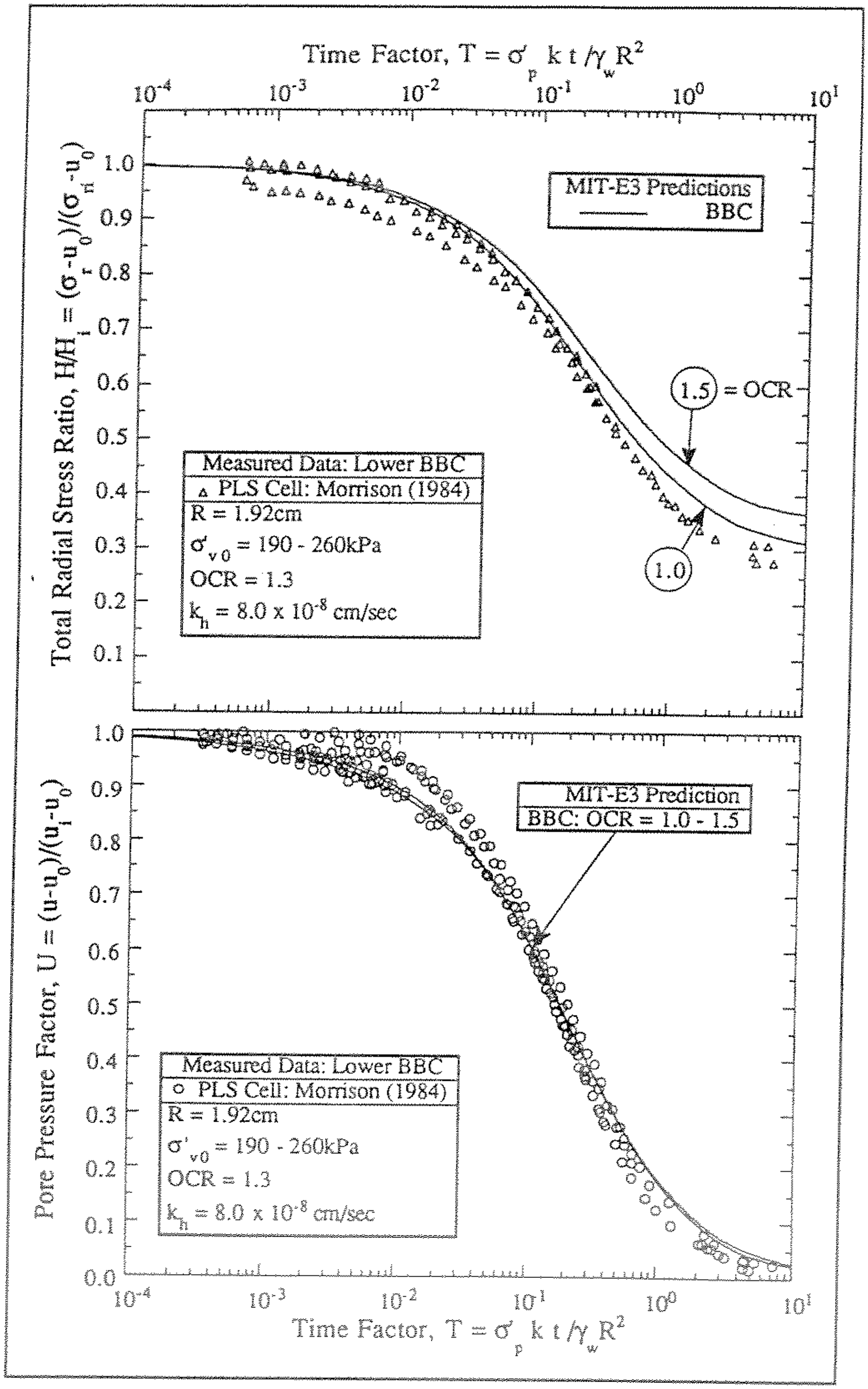


Figure 9. Comparison of predictions and measurements during consolidation in Boston Blue Clay

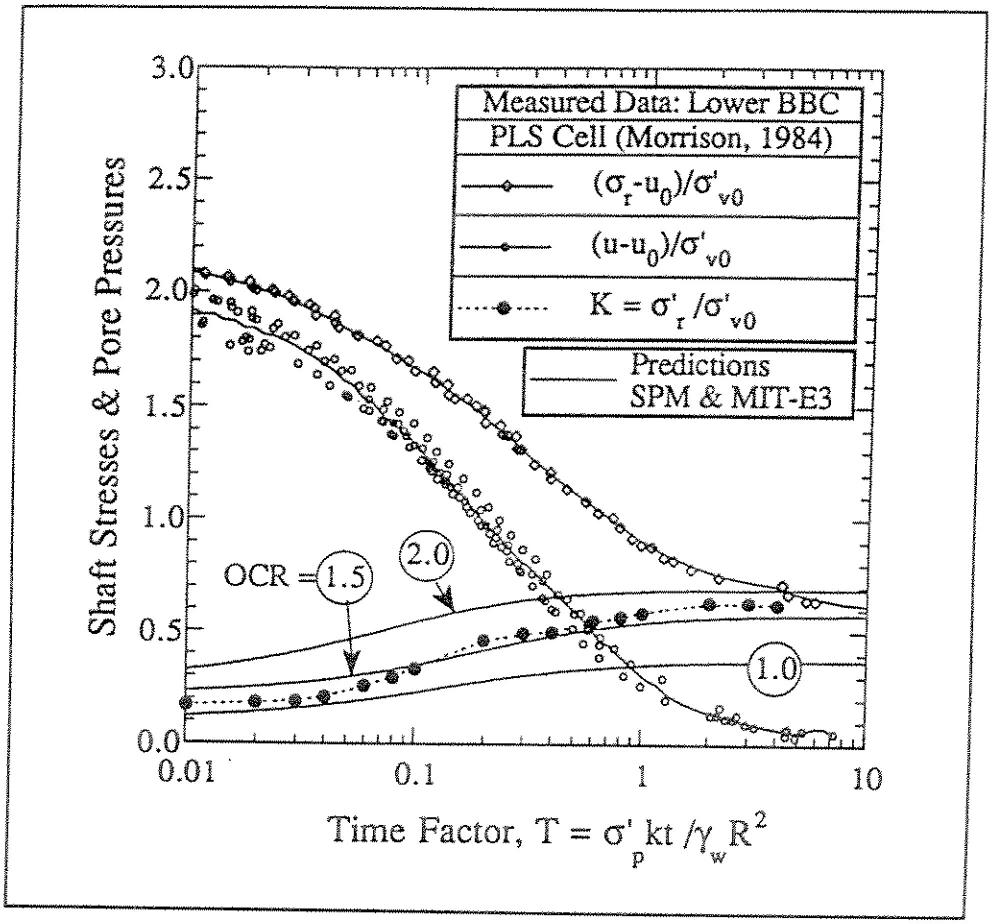


Figure 10. Evaluation of effective stress set-up in lower BBC

undisturbed clay. This results is qualitatively similar to the observations of the consolidation behavior of remolded Haga clay reported by Karlsrud and Haugen (1985).

Axial Pile Loading

Analysis of shaft capacity

This section focuses on predictions of the limiting skin friction, f_s, which is mobilized at the pile shaft after full dissipation of the installation excess pore pressures. The analyses are restricted to the case of a long, rigid pile for which the pile tip and the mudline have negligible effects on the shaft resistance. Even after introducing this simplification, there are three main factors which complicate significantly the prediction and interpretation of pile-soil interaction during axial loading:

1. Although slip surfaces are constrained to form parallel to the pile shaft, the actual slippage may occur either at the pile-soil interface or within the surrounding soil mass. Slippage at the interface is of practical importance

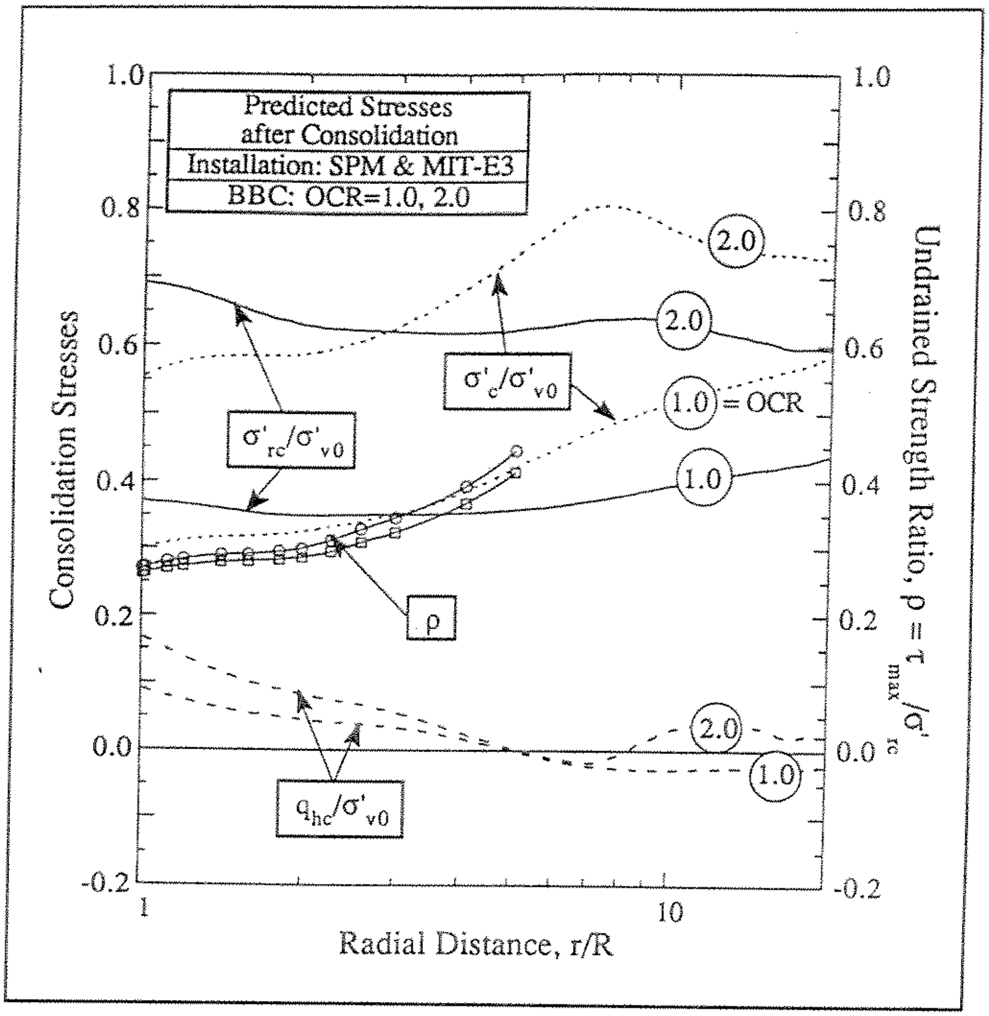


Figure 11. Predicted stress conditions at the end of consolidation

when the limiting angle of interface friction δ', is less than the effective stress obliquity α'_p, mobilized along vertical planes in the soil at peak shear resistance (i.e., $\alpha'_p = \tau_f/\sigma'_{rf}$).

2. The shear resistance of the soil is controlled by effective stresses and properties around the shaft prior to loading. The previous sections have shown that relatively sophisticated analyses can predict many aspects of the set-up behavior measured at the shaft for piles installed in normally and lightly overconsolidated clays. However, many aspects of the predictions cannot be evaluated from field measurements and hence, represent a source of uncertainty in the subsequent comparison with measured capacity.

3. Drainage conditions during pile loading are not well defined. For most conventional test procedures, the pile is loaded to failure over a period of several hours, such that the surrounding soil is sheared under conditions of partial (or complete) drainage which must be interpreted using effective stress methods.

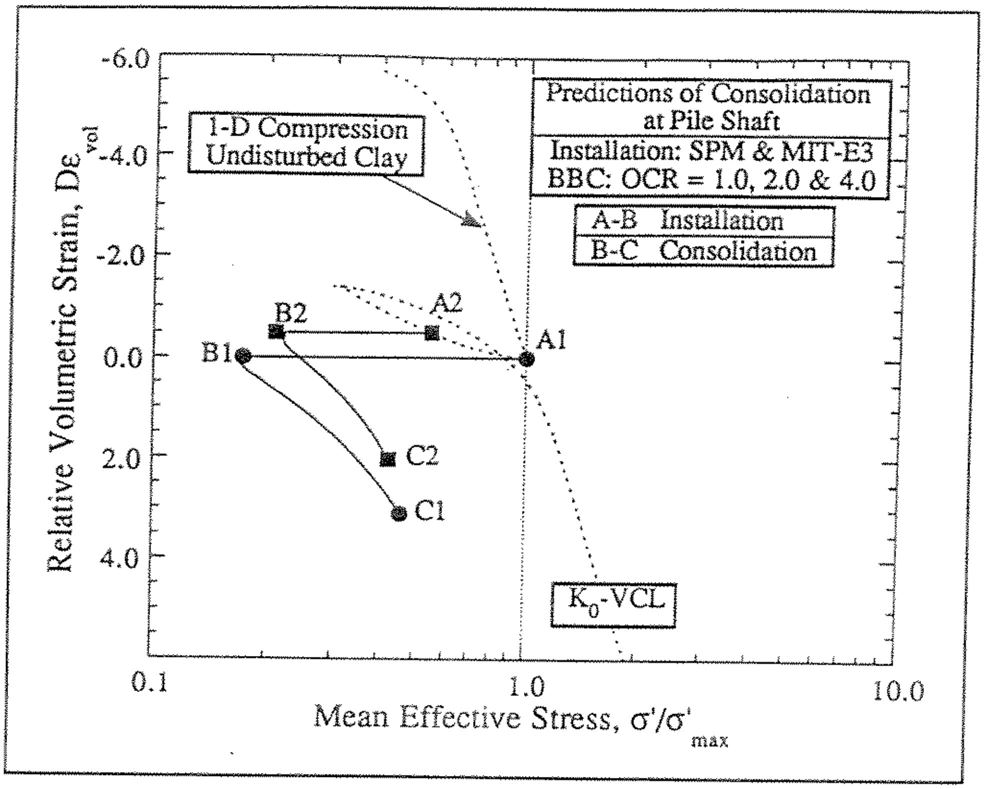


Figure 12. Predicted volumetric behaviour of soil elements adjacent to the pile shaft

For piles installed in lightly overconsolidated clays, the critical conditions (i.e., minimum shaft capacity) are likely to occur when the pile is loaded rapidly with no radial migration of pore water, and hence undrained shearing of the clay. In this situation, the mode of deformation corresponds to the shearing of concentric cylinders around the pile shaft (i.e., γ_{rz} is the only non-zero strain component), while vertical equilibrium controls the radial distribution of shear stresses ($\tau_0 R = \tau r$, where τ_0 is the interface shear stress). Figure 13a shows MIT-E3 predictions of the normalized effective (σ'_r/σ'_{rc}, τ/σ'_{rc}) and total stress paths ($[\sigma_r - u_0]/\sigma'_{rc}$, τ/σ'_{rc}) acting in the vertical plan for undrained shearing of soil elements adjacent to the pile shaft in Boston Blue Clay. The results show the following:

1. The initial stress history of the soil does not affect the normalized effective stress paths predicted at the pile shaft after full re-equilibration of the installation excess pore pressures for OCR≤4.

2. The peak shear resistance of the soil elements can be described by an undrained strength ratio, $\rho = \tau_f/\sigma'_{rc} \approx 0.26$ (Whittle et al. 1988; Azzouz et al. 1990) which is mobilized at an effective stress obliquity, $\alpha'_p \approx 24^0$ (which is significantly lower than the reference critical state friction angle measured in triaxial compression tests, $\phi'_{TC} = 33.4^0$).

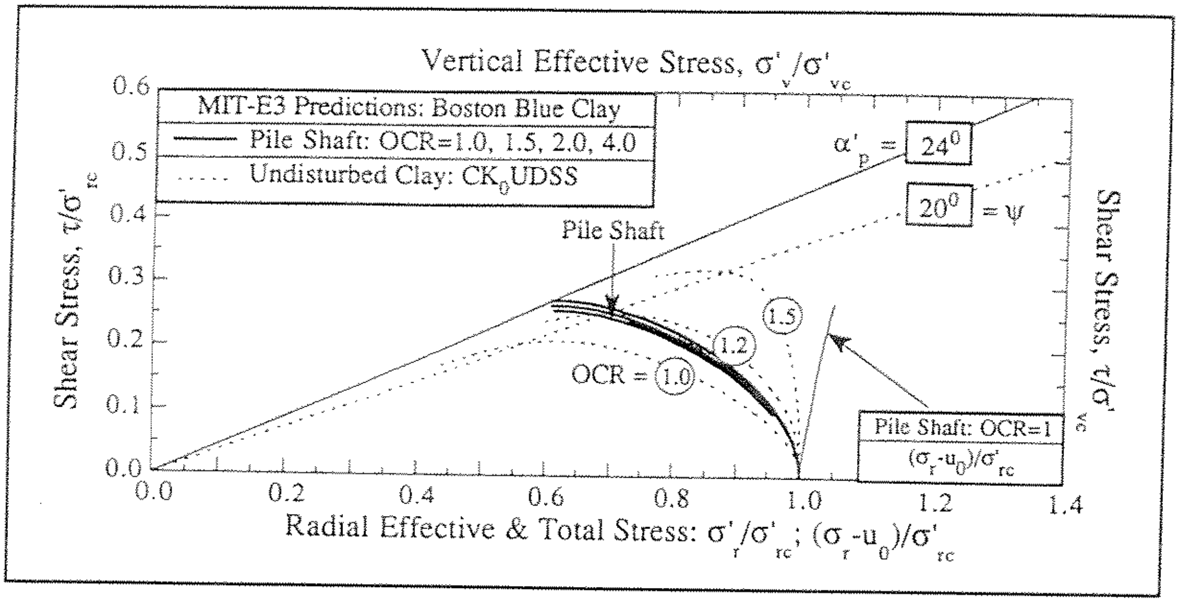


Figure 13. MIT-E3 predictions of soil behaviour for elements at the pile shaft during axial loading. a) Undrained shear behaviour

3. There is a reduction in the radial effective stress, $\Delta\sigma'_{rf}/\sigma'_{rc} \approx 0.4$, together with positive excess pore pressures, $\Delta u_f/\sigma'_{rc} \approx 0.4$-$0.5$, and a small net increase in the total stress ($[\sigma_r - u_0]_f/\sigma'_{rc} \approx 1.05$, Figure 13a) for shearing up to peak shaft resistance. The zone of excess pore pressures extends laterally to a distance, $r/R \leq 5$ (Figure 13b).

The analyses can be extended to consider the radial distribution of undrained shear resistance $\rho(r/R)$ by computing the response of individual soil elements to the same mode of shearing. The predictions in Figure 11 show that a) the shaft capacity is limited by the undrained shear strength of soil elements adjacent to the pile, and b) the shear resistance, $\rho(r/R)$, is proportional to the mean effective stress predicted at the end of consolidation (σ'_c/σ'_{rc}) for $r/R \leq 2$.

The mode of shearing assumed for soil elements adjacent to the pile shaft is identical to that imposed in laboratory constant volume (undrained) Direct Simple Shear (DSS) tests. Thus, it is useful to compare model predictions of soil behavior for the remolded soil around the pile shaft with that of the undisturbed K_0-consolidated clay in undrained Direct Simple Shear tests. Figure 13a shows MIT-E3 predictions of the normalized effective stress paths (σ'_v/σ'_{vc}, τ/σ'_{vc}) for K_0-consolidated BBC at OCR = 1.0, 1.2, and 1.5. The undrained strength ratio ($s_{uDSS}/\sigma'_{vc} = 0.25$) and effective stress path of the undisturbed clay at OCR =1.2 are in close agreement with the normalized behavior predicted at the pile shaft, although there is a difference ($\psi = 20^0$ versus $\alpha'_p = 24^0$) in the stress obliquity mobilized at peak shear resistance. Differences in the normalized response at the pile shaft and the K_0-normally consolidated clay (OCR = 1.0) can be attributed mainly to the predicted ratios of effective stress components in the soil at the end of consolidation (i.e., $\sigma'_{rc}:\sigma'_{zc}:\sigma'_{\theta c}$ versus $\sigma'_h = K_{0NC}\sigma'_v$) (Whittle 1987).

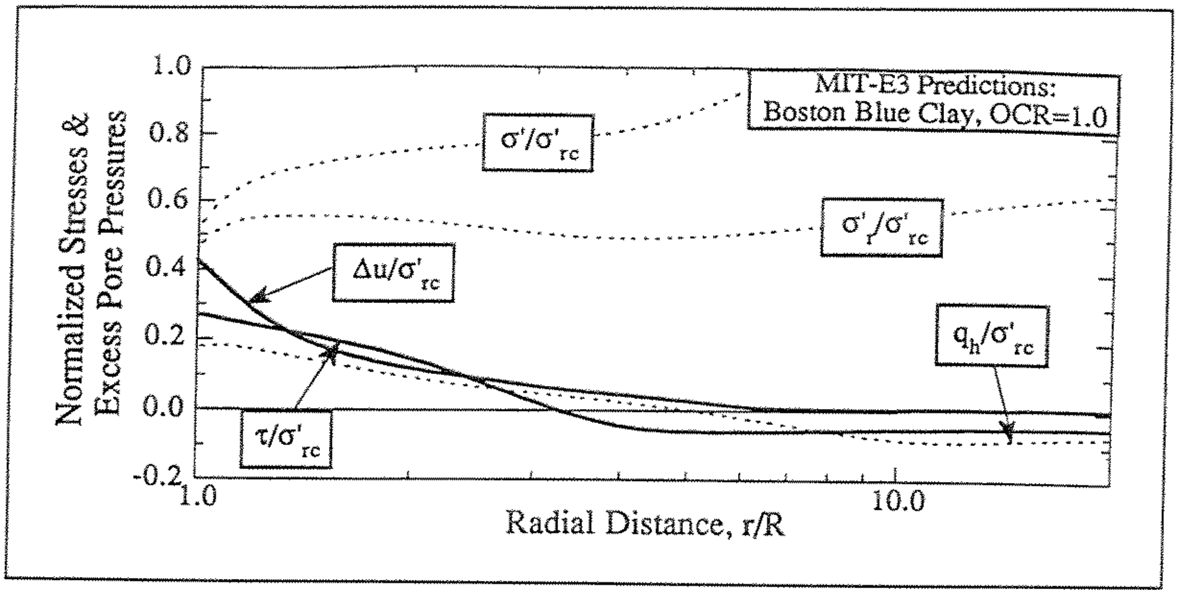


Figure 13. b) Radial distribution of stresses and excess pore pressures at peak shear resistance

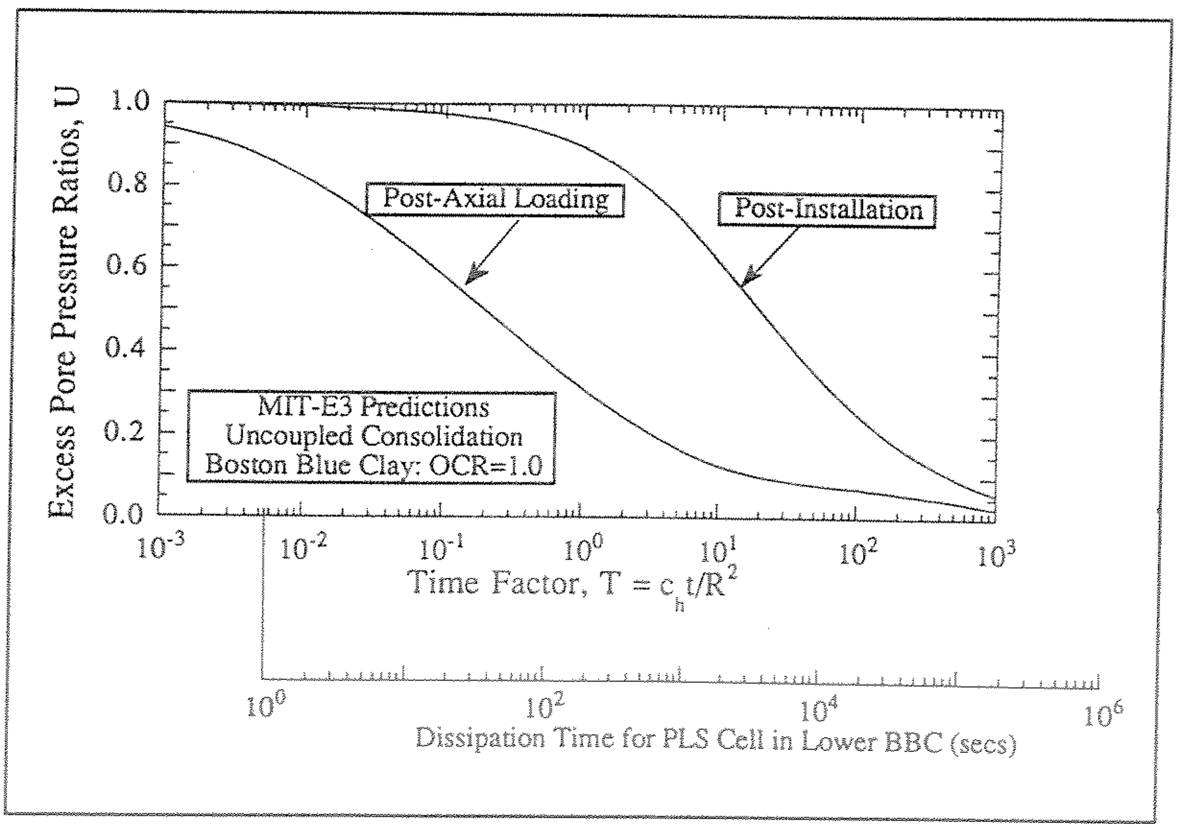


Figure 13. c) Dissipation of excess pore pressures

Azzouz et al. (1990) have proposed that the undrained strength ratio, ρ, can be used to provide realistic estimates of the limiting value of f_s for the design of friction piles in lightly overconsolidated clays (OCR≤4). The limiting skin friction is written as:

$$f_s = K_c \rho \sigma'_{v0} \tag{2}$$

where σ'_{v0} is the in situ effective overburden stress.

This method re-expresses the well known β parameter ($\beta = f_s/\sigma'_{v0}$; Chandler 1968; Burland 1973) as the product of the lateral earth pressure coefficient, K_c, and the undrained shear strength ratio, ρ, of the soil adjacent to the pile shaft. Predictions for normally and lightly overconsolidated clays show that K_c is affected by the stress history of the clay (OCR) and by its sensitivity, while ρ can be estimated from the strength ratio measured in undrained direct simple shear tests on the undisturbed clay at OCR ≈ 1.2.

In principle, lower values of the limiting skin friction can occur if the interface friction angle, δ', is smaller than the effective stress obliquity, α'_p, mobilized at peak shear resistance in the soil. Measurements in the ring shear apparatus show that δ' depends on numerous factors including soil mineralogy and fabric, consolidation and shear history, rate of shearing, interface roughness and hardness (Lemos 1986). Comparisons between MIT-E3 predictions of α'_p for a number of clays (in the range, $\alpha'_p = 17^0\text{-}24^0$) and recent correlations for δ' (Jardine & Christoulas 1991) show that $[\alpha'_p - \delta'] \leq 5^0$, and suggest that interface slippage can reduce the calculated undrained shear resistance, ρ, by up to 25 percent.

The model predictions also provide a basis for evaluation drainage conditions during pile loading. Figure 13b shows the radial distribution of stresses and excess pore pressures at conditions corresponding to peak undrained shear resistance in the soil at the pile shaft (for BBC at OCR = 1). Subsequent dissipation of the excess pore pressures provides some guidance on the characteristic time required to achieve undrained loading of the pile. Figure 13c compares predictions of the uncoupled dissipation of excess pore pressures generated during installation and axial loading. For the PLS cell (R = 1.92cm) installed in the lower deposit of BBC ($c_h \approx 0.02\text{cm}^2/\text{sec}$; Baligh & Levadoux 1980), the predictions show $t_{50} \approx 40$ secs, while undrained conditions are only achieved when the shaft is loaded to failure within 1 sec. In this situation, the soil around the pile is sheared at an average strain rate which is significantly higher than that imposed in conventional laboratory CK_0UDSS tests ($\dot{\gamma}$ = 5 percent/hr). Data from CK_0UDSS tests with $t_f \approx$ 1-10 secs (e.g., Lacasse 1979) show a 15-25 percent increase in the undrained strength ratio and develop much smaller shear induced pore pressures compared to tests performed at conventional strain rates. These measurements suggest that predictions using the rate independent MIT-E3 model (Figure 13a) will tend to underestimate the shaft resistance and overestimate the excess pore pressures for undrained loading.

The one-dimensional (radial) predictions described in this section implicitly assume that the pile shaft performance is similar for loading in either axial compressions or tension. In practice, the local shaft resistance can be affected by factors such as proximity to the pile tip (e.g., Lehane & Jardine 1992) or residual axial loads in the pile (e.g., Whittle 1991b). More comprehensive two-dimensional analyses of set-up and loading are required in order to evaluate these effects. However, some insight can be obtained from simple model predictions of material behavior in laboratory element tests. For example, residual axial loads in the pile generate shear stresses during set-up at the pile-soil interface (i.e., $\rho_c = \tau_c/\sigma'_{rc}$). These effects can be simulated in laboratory CK_0UDSS tests by consolidating the soil under an applied shear stress, τ_c/σ'_{vc}. Figure 14 compares MIT-E3 predictions with the measured effective stress paths and shear stress-strain response for CK_0UDSS tests on normally consolidated BBC for consolidation shear stress ratios $-0.2 \leq \tau_c/\sigma'_{vc} \leq 0.2$. The results show the following:

1. Specimens consolidated with shear stresses applied in the same direction as the subsequent undrained loading ($\tau_c/\sigma'_{vc} > 0$) exhibit higher undrained shear strengths, smaller shear induced pore pressures, lower effective stress obliquity and stiffer stress-strain response (at $\tau_c/\sigma'_{vc} = 0.2$: $s_{uDSS}/\sigma'_{vc} = 0.29$, $\Delta u_s/\sigma'_{vc} = 0.07$, $\alpha'_p = 17^0$, and $\gamma_p = 0.7$ percent) than standard tests performed at $\tau_c/\sigma'_{vc} = 0$. In contrast, the consolidation shear stress has little effect on the undrained strength for $\tau_c/\sigma'_{vc} < 0$, but affects significantly the stress-strain response and the development of shear induced pore pressures.

2. The MIT-E3 model is very good agreement with the measurements of prepeak stress-strain behavior and effective stress paths in these tests. However, the model tends to overpredict the undrained shear strength for $\tau_c/\sigma'_{vc} > 0$ and does not describe accurately the post-peak strain softening measure in these tests.

Overall, the results in Figure 14 show important aspects of soil behavior which can be related directly to the effects of residual axial loads on pile shaft performance.

The ρ-method of estimating capacity (Azzouz et al. 1990) assumes that there is full dissipation of installation excess pore pressures prior to pile loading. However, for large diameter offshore piles, loading is often carried out under conditions of incomplete set-up. These situations can also be readily analyzed using the proposed framework. Figure 15 illustrates predictions of the normalized shaft capacity β/β_∞ (where $\beta_\infty = K_c\rho$) and pore pressure ration, U, as functions of the dimensionless time factor, T. The results show that the shaft capacity increases by a factor of 2-3 during the set-up process, with most of the strength gain occurring over the time frame, $0.01 \leq T \leq 1.0$.

Evaluation of shaft capacity

The predictions of the undrained strength ratio, ρ, can be evaluated by dividing the measured values of β from rapid axial load tests (which ensure undrained shearing of the clay) and the lateral effective stress ratio measured at

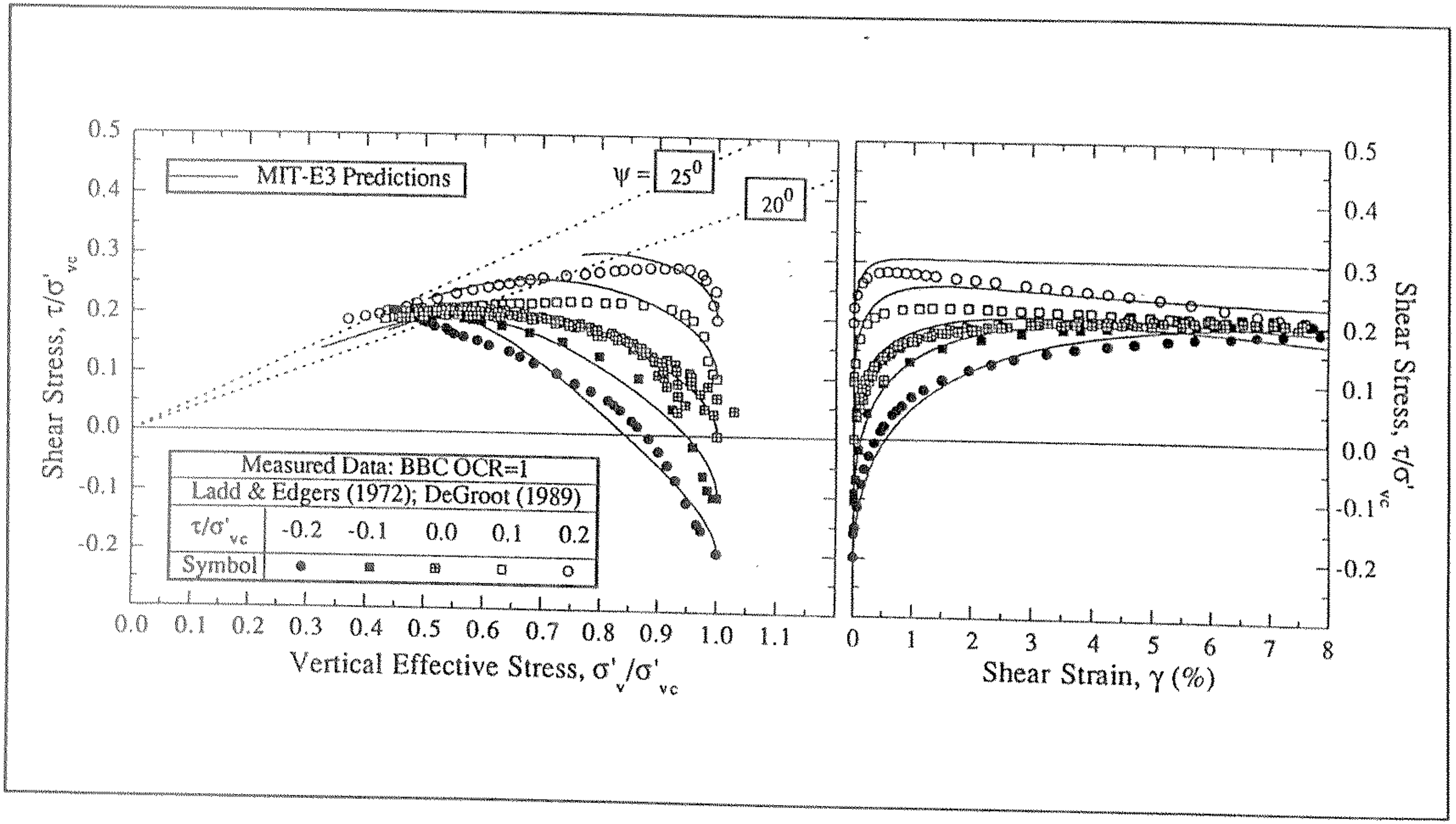


Figure 14. Effect of consolidation shear stress in undrained direct simple shear tests on BBC

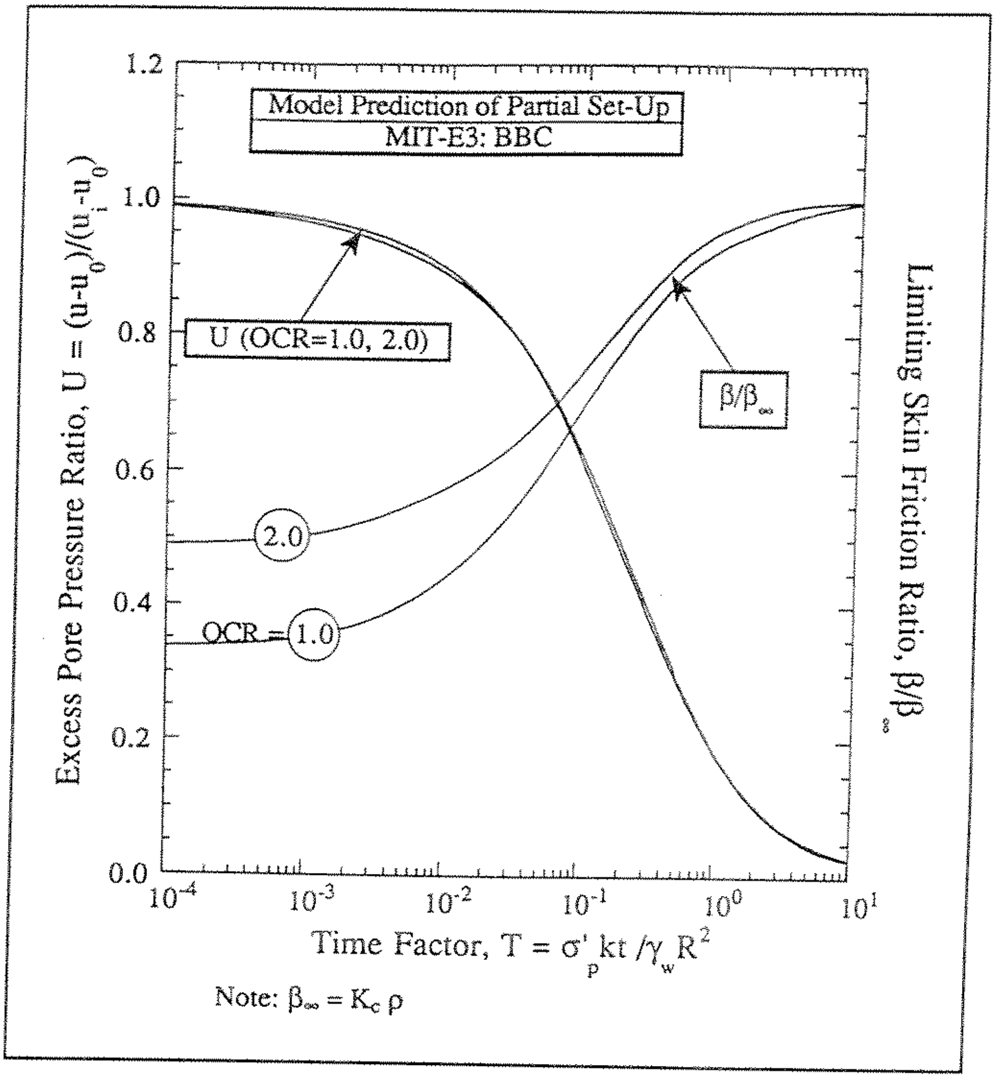


Figure 15. Prediction of shaft capacity for partial set-up in BBC

the end of set-up, K_c . Measurements of β can be obtained with a high degree of reliability from measurements of the distribution of axial load along the axis of the pile. In contrast, concurrent measurements of the local pore pressures, shear and normal lateral stresses are required in order to interpret the effective stress paths of soil elements at the pile shaft. Reliable data of this type have only recently been reported from high quality instrumented pile tests in a soft clay (Lehane & Jardine 1992).

Figure 16a compare the MIT-E3 predictions and measurements of ρ from axial load tests obtained using the PLS cell and instrumented model piles. The PLS data in BBC and Empire clay ($R = 1.92$ cm, $t_f = 10$-40 sec) (Morrison 1984; Azzouz & Lutz 1986) are obtained from axial load cell measurements and provide the average shaft resistance acting between the pile tip and the elevation of the PLS cell. In contrast, six levels of strain gauges are used to estimate the distribution of shaft friction along the length of the Haga piles ($L = 6$ m, $R = 7.6$ cm, $t_f = 20$ min) (Karlsrud & Haugen 1985). The results show very good

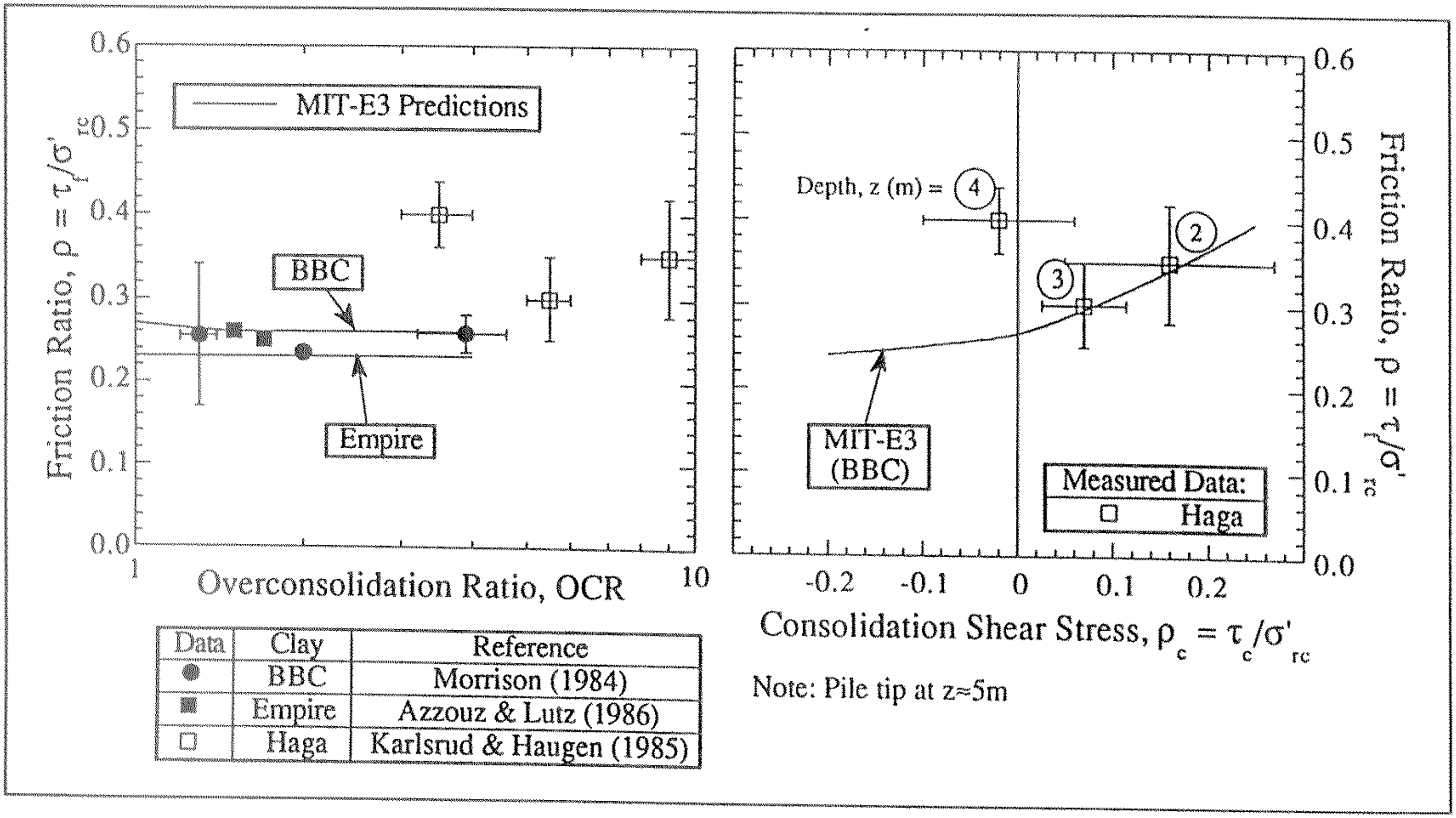


Figure 16. Evaluation of the undrained strength ratio, ρ, of soil adjacent to the pile shaft

agreement between predictions and measurements of ρ form PLS tests, but underestimate significantly the skin friction ration reported in the Haga tests. This result can be explained, in large part, by residual loads in the piles which generate significant consolidation shear stresses in the upper half of the pile. Figure 16b compares model predictions of the undrained shear resistance, ρ, as a function of the consolidation shear stress, ρ_c. The results show good agreement with data at depths, z = 2 and 3m, but still underpredict the behavior at z = 4m probably due to factors such as the proximity of the pile tip (Whittle 1991b).

Summary and Conclusions

This paper has described the application of a systematic analysis for predicting the performance of driven piles in lightly overconsolidated clays (OCR≤4). The predictions are evaluated through comparisons with field measurements from the Piezo-Lateral Stress (PLS) cell and instrumented model piles. The key components of the analysis are: a) the Strain Path Method, which models the effects of the severe soil disturbances caused by pile installation; and b) MIT-E3 , a generalized effective stress soil model with well documented capabilities for describing the non-linear and anisotropic behavior of K_0-consolidated clays with normalized, rate independent properties. The results focus on conditions during pile installation, stress and pore pressure changes during subsequent consolidation, and the shaft capacity during axial loading.

The Strain Path Method (SPM) provides a realistic model of the mechanics of undrained deep penetration in clays. Predictions using the SPM with the MIT-E3 model give good agreement with the radial effective stresses measured at the pile shaft during steady penetration, but tend to underestimate the installation excess pore pressures. This latter result probably reflects limitations of the constitutive model for describing the behavior of structured natural clays.

Radial consolidation around the pile shaft is solved by a coupled, non-linear finite element method with initial conditions from the installation phase and using the MIT-E3 model to describe the effective stress-strain properties of the soil. The consolidation process can be characterized by the pore pressure ratio, $U = \Delta u/\Delta u_i$, the total stress release, H/H_i (where $H = \sigma_r - u_0$) , and the set-up effective stress, $K = \sigma'_r/\sigma'_{v0}$, as functions of a dimensionless time factor, T (Figure 7). The analyses show that the radial distribution of installation excess pore pressures control U, while H/H_i is primarily a function of the non-linear radial compressions behavior of the soil. The paper presents detailed comparisons between predictions and PLS measurements in Boston Blue Clay using average values for the horizontal coefficient of permeability for laboratory tests. The predictions consistently describe the changes in the effective stresses, the total stress release, H/H_i and the pore pressure ratio, U, throughout consolidation. The predictions are in good agreement with measurements of the radial effective stress at the end of consolidation ($K_c = \sigma'_{rc}/\sigma'_{v0}$) as reported from a number of field sites (Figure 8). The parameter K_c is affected by the in situ overconsolidation ratio and sensitivity of the clay.

Predictions of pile shaft performance are presented for rapid axial loading of a long, rigid pile after full dissipation of installation excess pore pressures. The analyses assume that minimum values of the limiting skin friction, f_s, can be estimated from the undrained shear resistance in the soil adjacent to the pile shaft. Results using the MIT-E3 model show that the normalized shear resistance of the soil, ρ ($= f_s/\sigma'_{rc}$, where σ'_{rc} is the radial effective stress at full set-up) is independent of the initial OCR, and can be equated approximately with the undrained strength ratio (s_{uDSS}/σ'_{vc}) of the undisturbed clay measured in an undrained direct simple shear test at OCR $\approx$ 1.2 (Figure 13a). The paper discusses other factors affecting the shaft resistance of the pile including partial set-up, drainage conditions in the soil, loading rate and residual loads in the pile. Predictions of the undrained strength ratio, ρ, are in good agreement with field measurements obtained using the PLS cell measured at two sites (Figure 16).

Acknowledgments

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Appendix C
Selected Workshop Handouts

Some (Random?) Thoughts on the Axial Behavior of Piles in Granular Soils

by

Michael W. O'Neill

Professor and Chairman
Department of Civil and Environmental Engineering

University of Houston

ISSUES COVERED HERE

- *Traditional Approach to Static Capacity Assessment*
- *Unit Shaft and Toe Resistance vs. Penetration*
- *Why are Unit Shaft and Toe Resistances not Proportional to Depth?*
- *Effect of Pile Geometry on Load Transfer*
- *A Look at Plugging*
- *Effect of Installation Method*
- *Group Behavior*
- *Seismic Behavior*
- *Effect of Installation Aids (Drilling Slurry)*
- *Some General Ideas about Research Needs*

Piles in Granular Soils

Principal Factors Affecting Axial Capacity

- **Method of Installation:**
  - Bored
  - Augered
  - Impact-driven
  - Vibrated

- **Pile Shape:**
  - Open cylinder
  - Open cylinder with shoe
  - Closed cylinder
  - Closed cylinder with bell
  - H
  - Nodular
  - Tapered

- **Pile Length and Flexibility:**
  - Level-to-level interaction
  - Toe-shaft interaction
  - Installation interaction (residual stresses)

- **Initial Effective Stresses:**
  - σ'_v
  - K'_o
  - Orientation of principal stresses

- **Soil Properties:**
  - Nonlinear failure envelope
  - Crushability (change in ϕ with stress change)
  - Compressibility
  - Propensity for dilation/contraction
  - Hydraulic conductivity

- **<u>Details of Pile-Soil Interface</u>:**
  - **Prefabricated or cast-*in situ***
  - **Change in propensity of soil for dilation/contraction**
    - **(installation method, shape, initial soil properties)**
  - **Interface roughness pattern**
  - **Aging**
  - **Chemical reactions between pile and soil**
  - **Residual effects of installation aids (e.g., drilling slurry)**

- **<u>Geologic Details</u>:**
  - **Silt and clay interbedding**

- **<u>Loading</u>:**
  - **Seismic**
  - **Non-seismic cycle**
  - **Non-seismic static (short-term)**
  - **Non-seismic static (long-term)**

- **<u>Grouping</u>:**
  - **Geometric pattern of group**
  - **Order/method of installation**
  - **Cap-soil-pile interaction**

Traditional Approach to Assessment of Pile Capacity in Granular Soil

$$\boxed{f_{max} = D_\tau D_r K\,(\tan\delta)\,\sigma'_v \le f_{max}\,(\text{limit})}$$

$0.7 \le K \le 2.0$ (Compression loading)

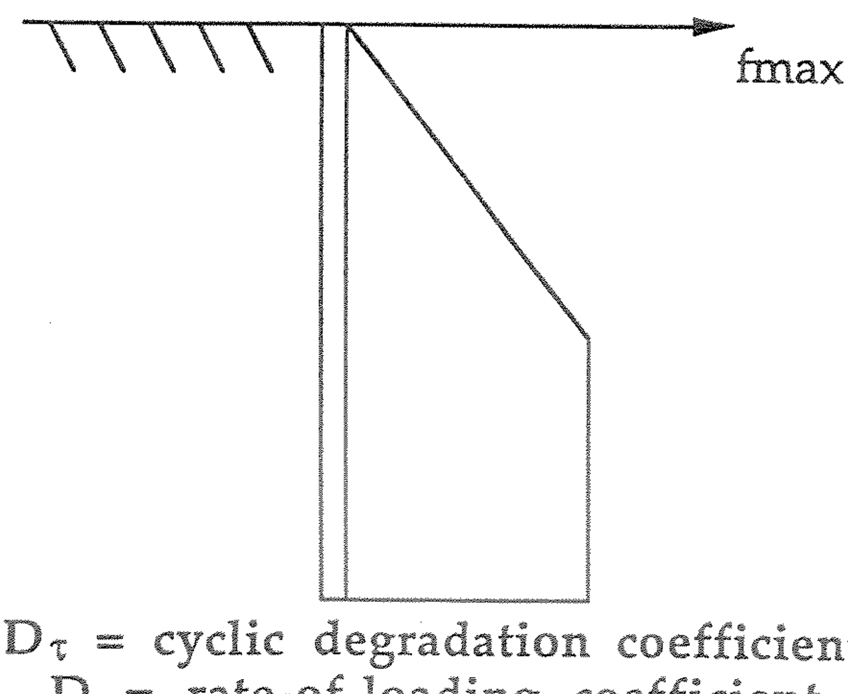
$0.7 \le K \le 1.0$ (Uplift loading)

$0.67\,\phi \le \delta < \phi$ (depending on grain size of soil and texture of interface)

f_{max} (limit) about 100 kPa for clean sand

or

f_{max} value at 10 - 20 d based on above equation and relative density of sand



D_τ = cyclic degradation coefficient
D_r = rate-of-loading coefficient

$$\boxed{q_b = N_q \, \sigma'_v \text{ (toe)} \le q_b \text{ (limit)}}$$

N_q is a function of ϕ' or relative density at the toe

. q_b (limit) varies according to silt and clay content -- about 10 MPa for clean, fine sand

or

$$q_b = N_\sigma \, \sigma'_m \text{ (toe)}$$

where N_σ is a function of ϕ' and rigidity index (G/τ_{max}) at the toe, and

σ'_m = mean effective stress at toe before driving

<u>Factor of Safety</u> = function of use of the supported structure (seldom the uncertainty in the design method or in assessment of soil properties)

Stress Wave Methods can be used to verify (approximately) static capacity upon restriking the pile

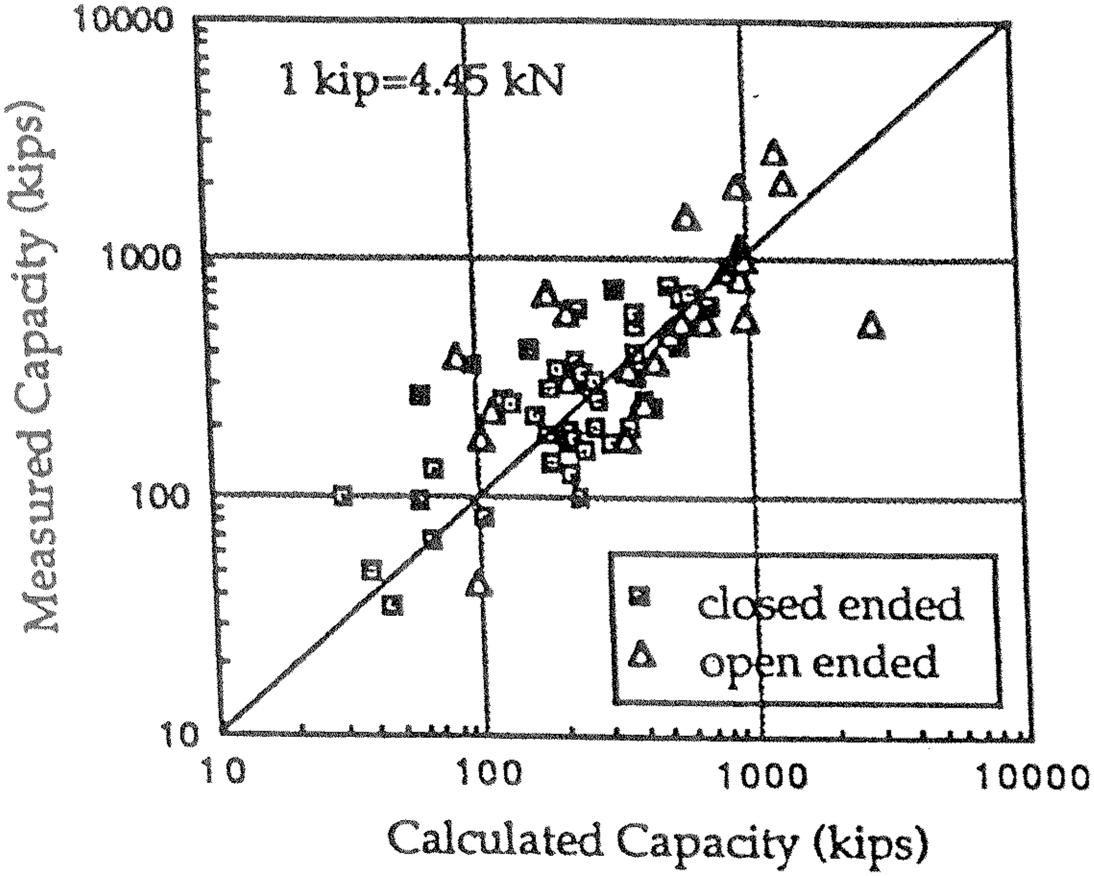
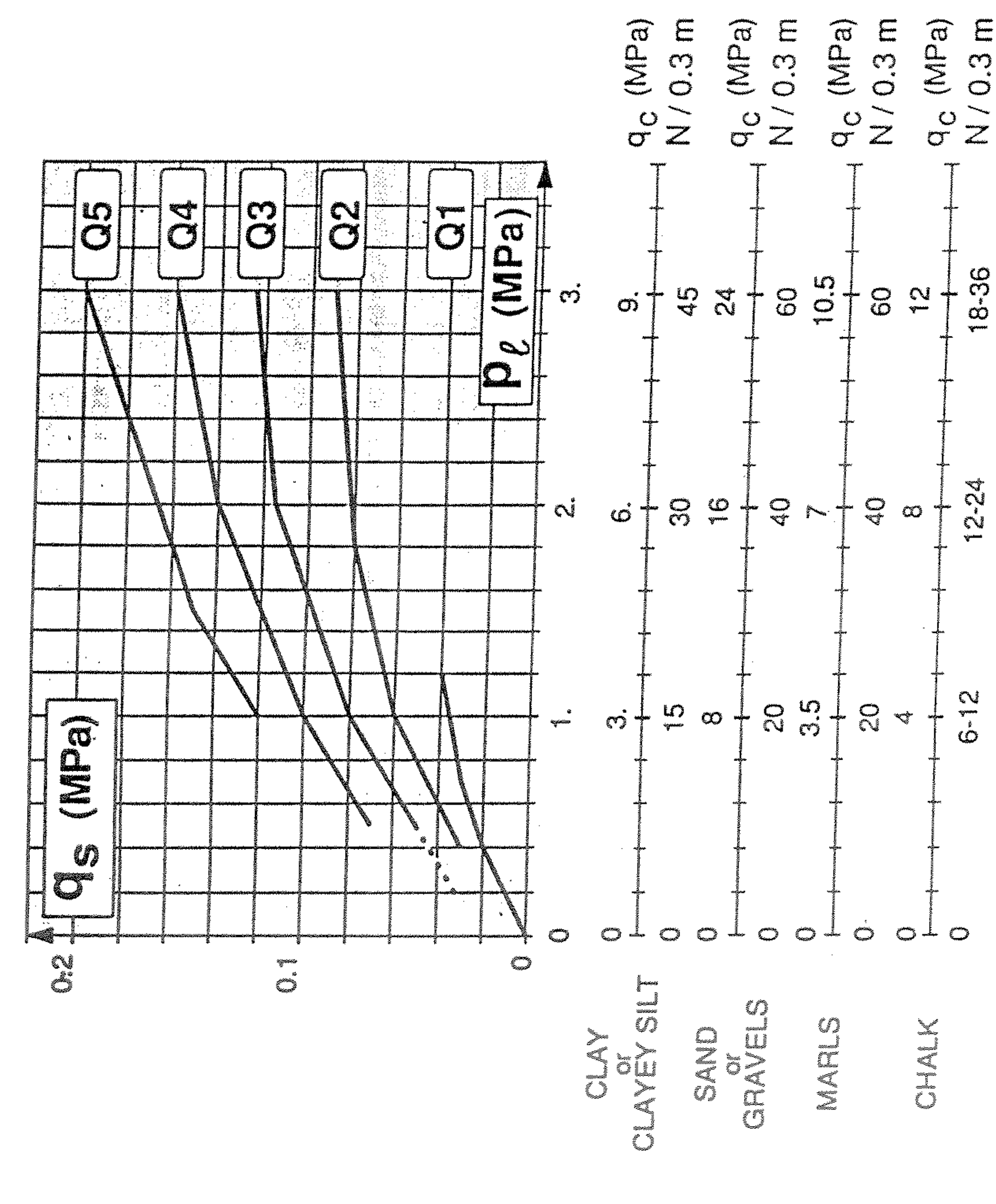
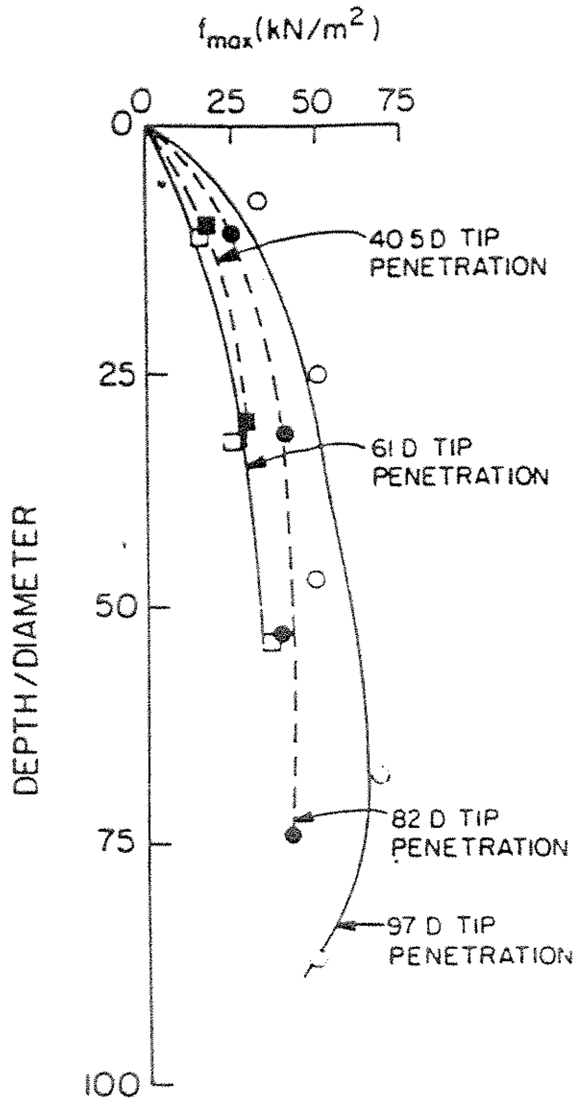


Fig. 1 Comparison of Measured (Defined Failure) and Computed (API, 1992) Capacities of Open-Ended Steel Pipe Piles in Cohesionless Soils

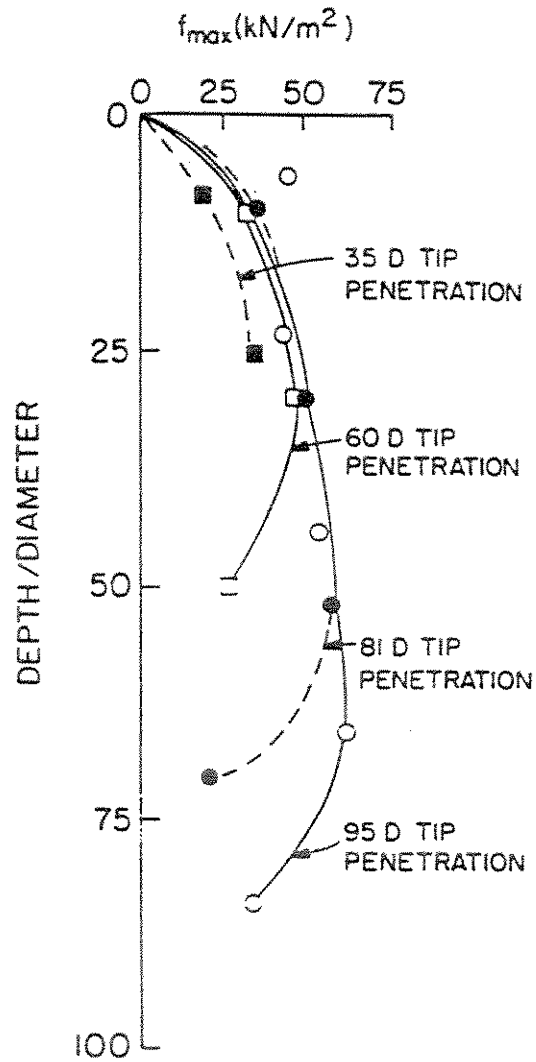


q_s (MPa)
0.2
0.1
0
Q5
Q4
Q3
Q2
Q1
p_ℓ (MPa)
0
1.
2.
3.
CLAY or CLAYEY SILT
0 3. 6. 9. q_c (MPa)
0 15 30 45 N / 0.3 m
SAND or GRAVELS
0 8 16 24 q_c (MPa)
0 20 40 60 N / 0.3 m
MARLS
0 3.5 7 10.5 q_c (MPa)
0 20 40 60 N / 0.3 m
CHALK
0 4 8 12 q_c (MPa)
0 6-12 12-24 18-36 N / 0.3 m

Unit Shaft and Toe Resistance vs. Penetration



Medium Dense Sand

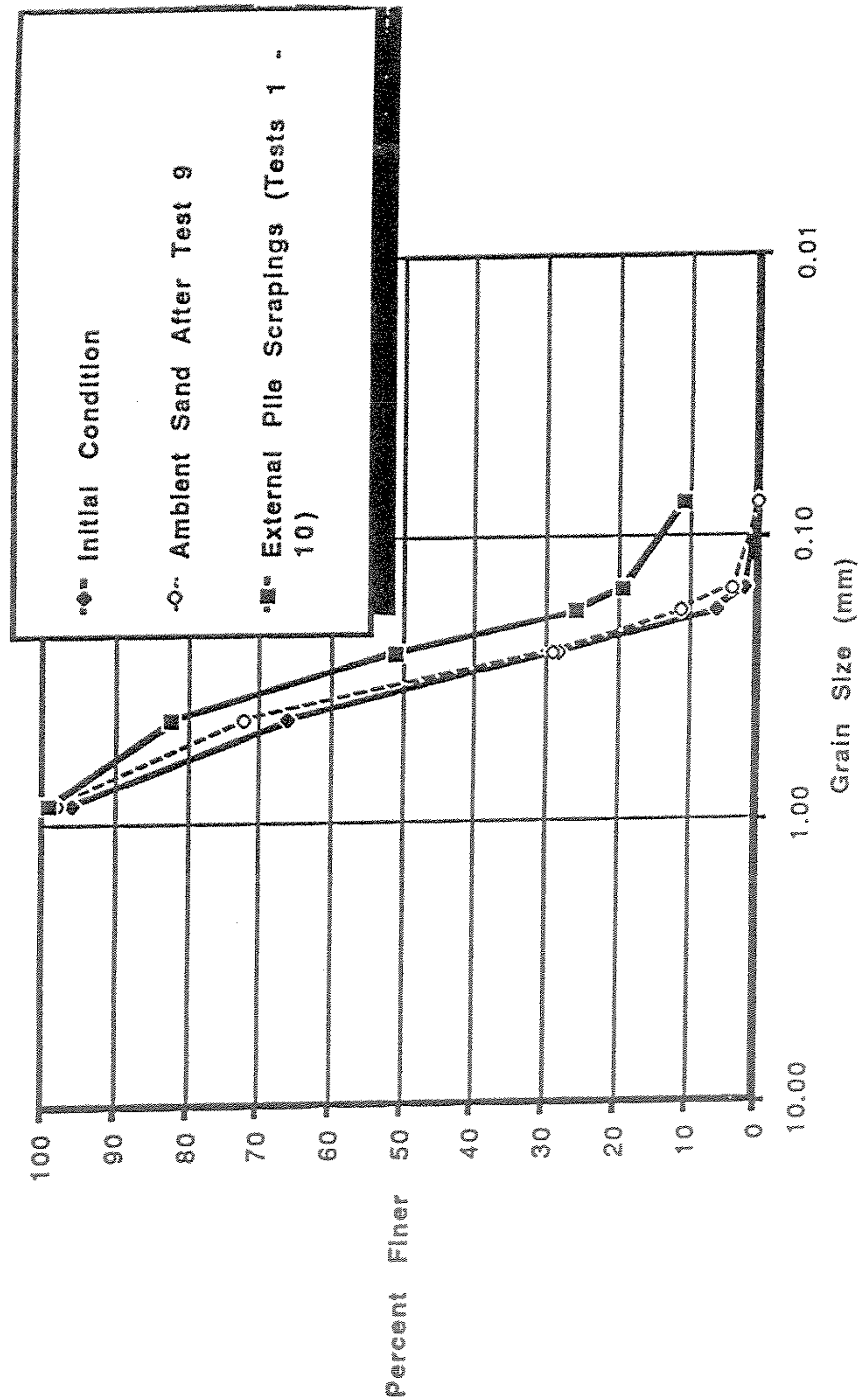


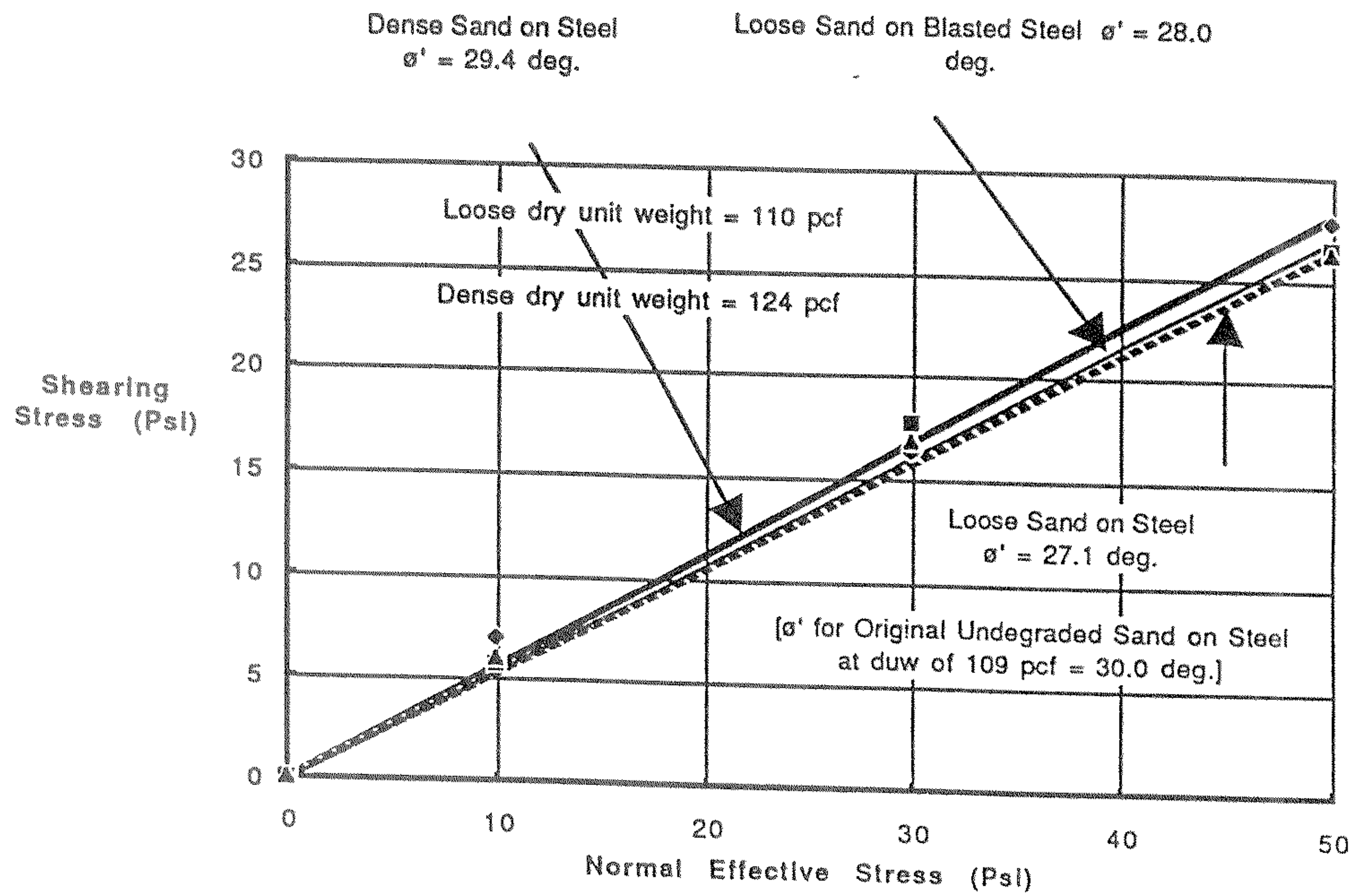
Dense Sand

Yazdanbod et al. (1984)

Why are f_{max} (and q_b) Not Proportional to Depth?

- *δ nonlinear with z?*
- *Rate of increase in radial effective stress decrease with z?*
- *Residual stresses that vary with z?*
- *Toe-shaft interaction?*
- *Something else?*


Initial Condition
Ambient Sand After Test 9
External Pile Scrapings (Tests 1 - 10)
Percent Finer
100
90
80
70
60
50
40
30
20
10
0
10.00
1.00
0.10
0.01
Grain Size (mm)


Dense Sand on Steel
ø' = 29.4 deg.
Loose Sand on Blasted Steel ø' = 28.0 deg.
Loose dry unit weight = 110 pcf
Dense dry unit weight = 124 pcf
Loose Sand on Steel
ø' = 27.1 deg.
[ø' for Original Undegraded Sand on Steel
at duw of 109 pcf = 30.0 deg.]
Shearing Stress (Psi)
Normal Effective Stress (Psi)
0
5
10
15
20
25
30
0
10
20
30
40
50

Expanding Cylindrical Cavity Theory (Vesic version)

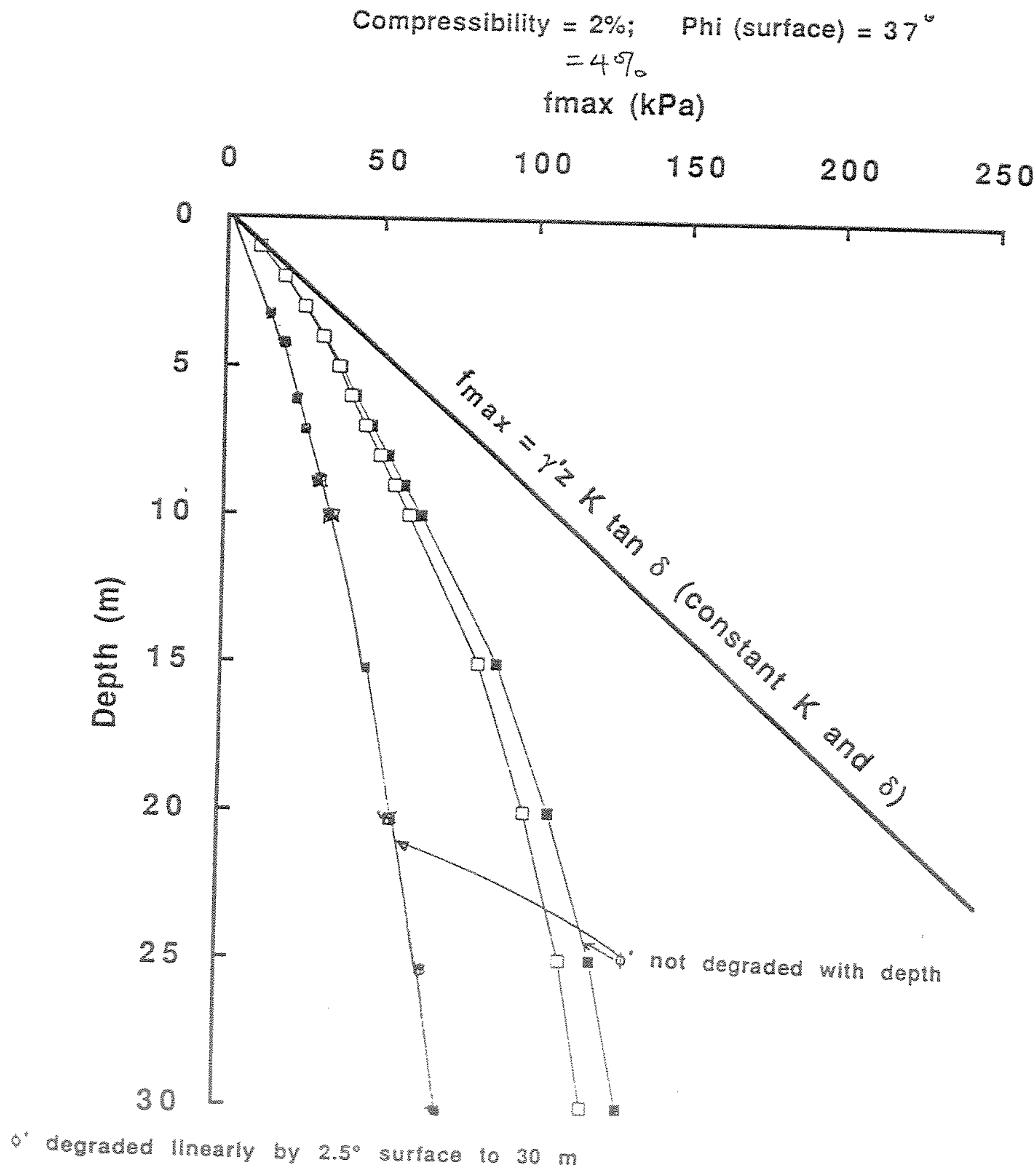
$$f_{max}(z) = \sigma'_m(z)\, F_q(z) \tan\delta(z)$$

$$F_q = (1 + \sin\phi')\,(I_{rr} \sec\phi')^{[\sin\phi' / (1 + \sin\phi')]}$$

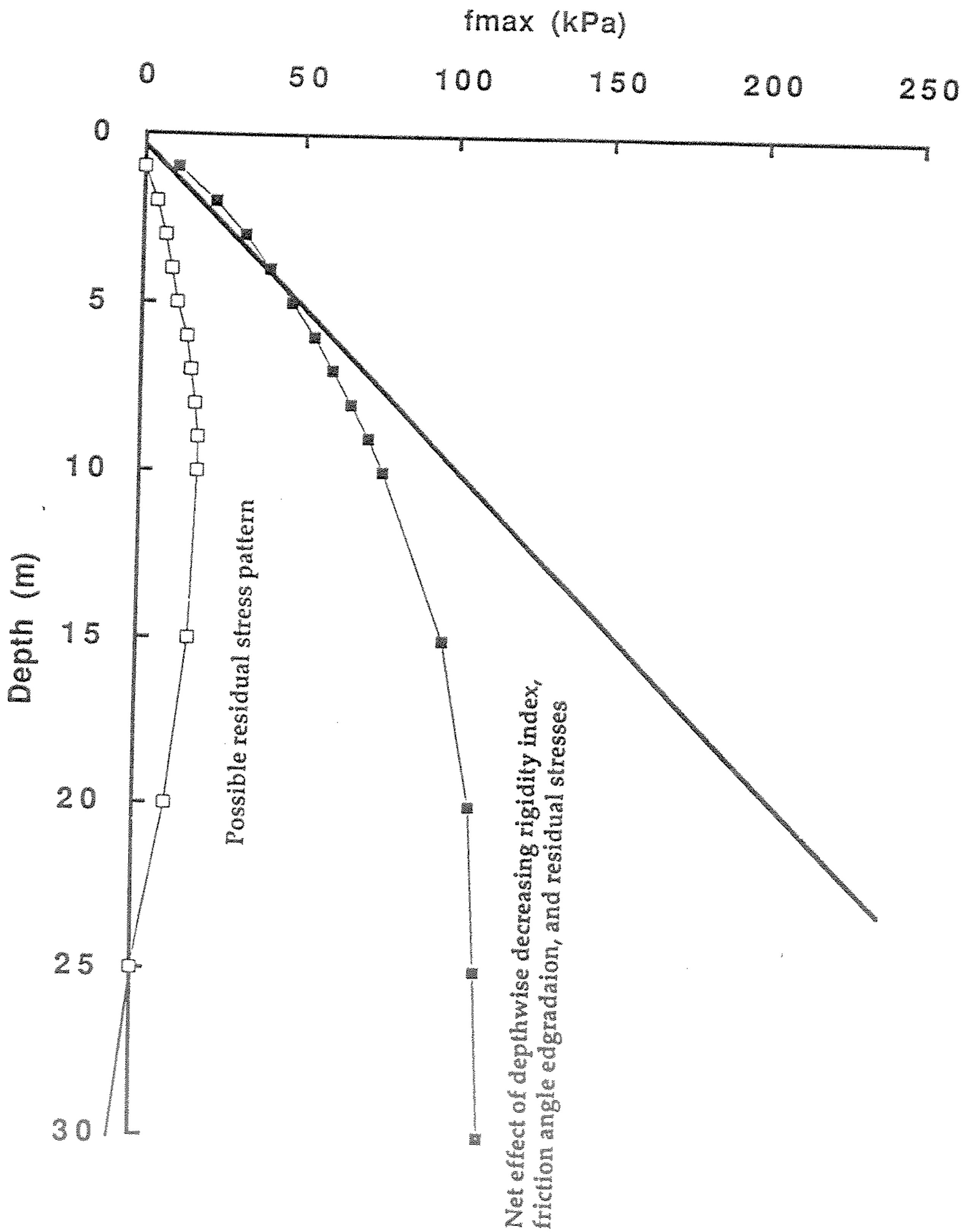
$$I_r = \frac{G}{s} \quad \alpha \quad z^{-0.5}$$

$$I_{rr} = \frac{I_r}{1 + I_r \Delta \sec\phi'}$$ and Δ is a compressibility factor

K' is constrained to be 1.0, and Δ cannot be negative (soil cannot dilate)



Compressibility = 2%; Phi (surface) = 37°
=4%
fmax (kPa)
0
50
100
150
200
250
Depth (m)
0
5
10
15
20
25
30
fmax = γ'z K tan δ (constant K and δ)
φ' not degraded with depth
φ' degraded linearly by 2.5° surface to 30 m


fmax (kPa)
0
50
100
150
200
250
Depth (m)
0
5
10
15
20
25
30
Possible residual stress pattern
Net effect of depthwise decreasing rigidity index, friction angle edgradaion, and residual stresses

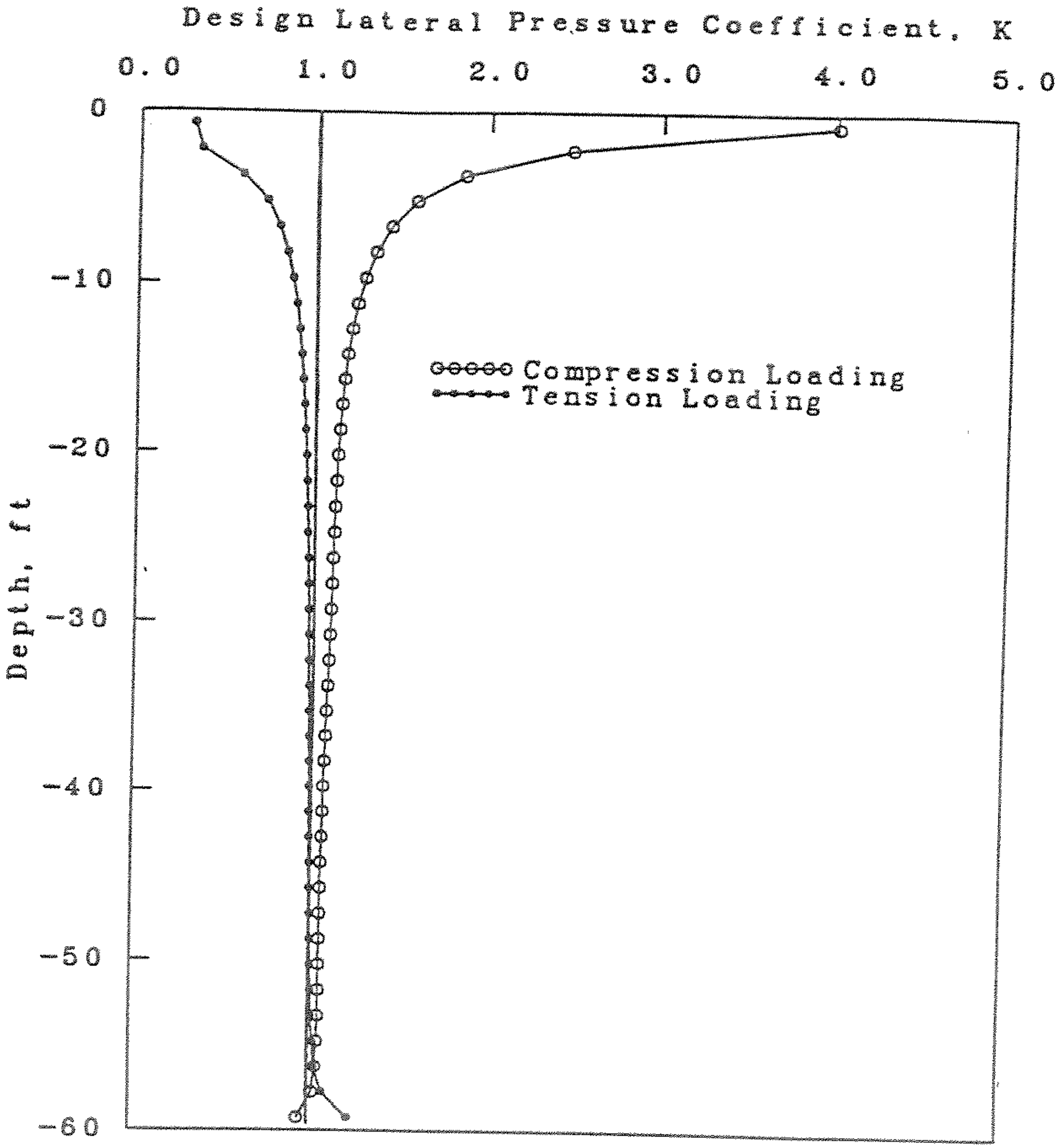


FIG. 65. K (Eq. 4) VS. DEPTH, 60-FT PILE, COMPRESSION VS. TENSION, DENSE SAND, NONDILATANT CASE, K_o = 1

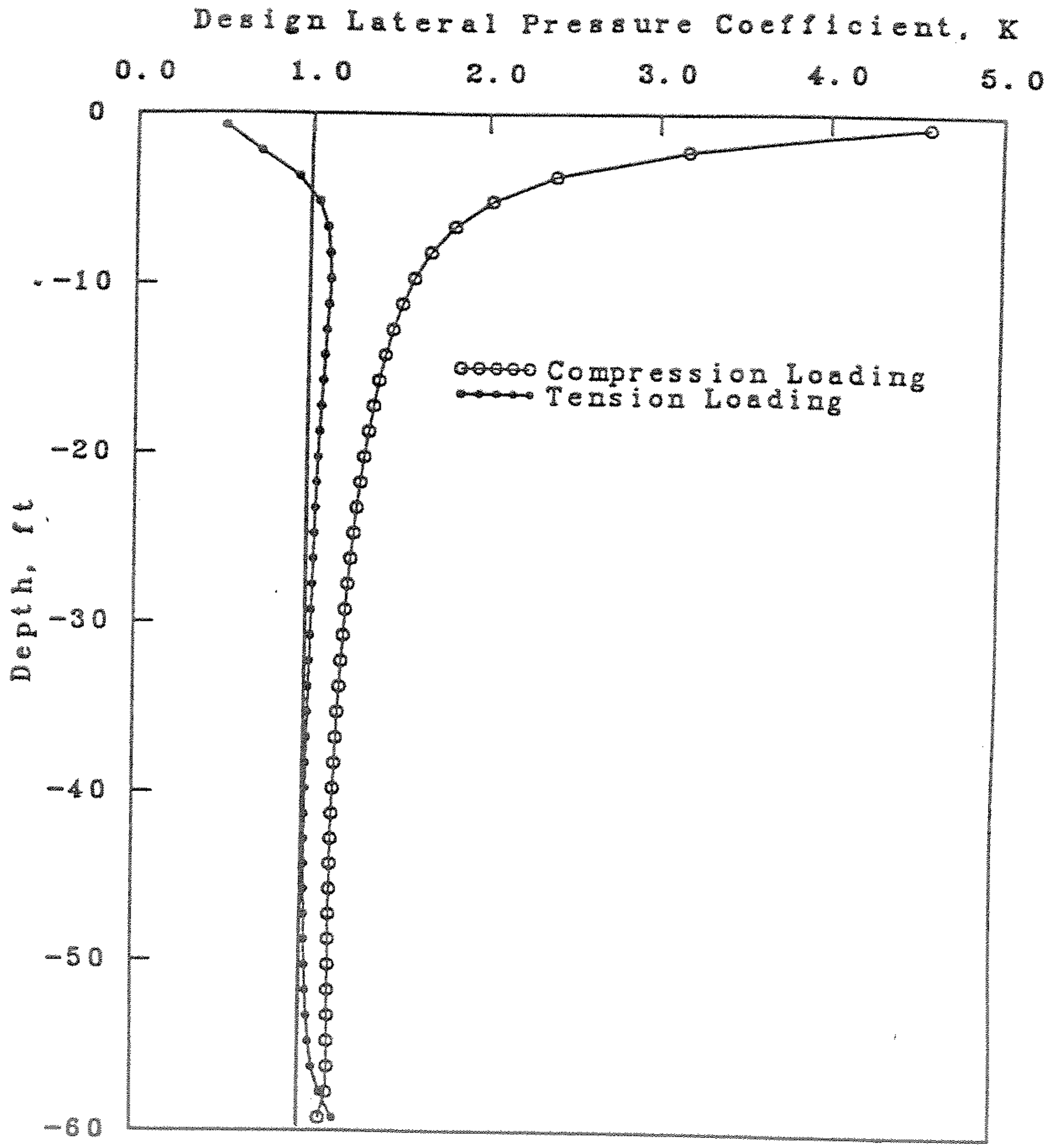


FIG. 66. K (Eq. 4) VS. DEPTH, 60-FT PILE, COMPRESSION VS. TENSION, DENSE SAND, DILATANT CASE, $K_o = 1$

Effect of Pile Geometry on Load Transfer

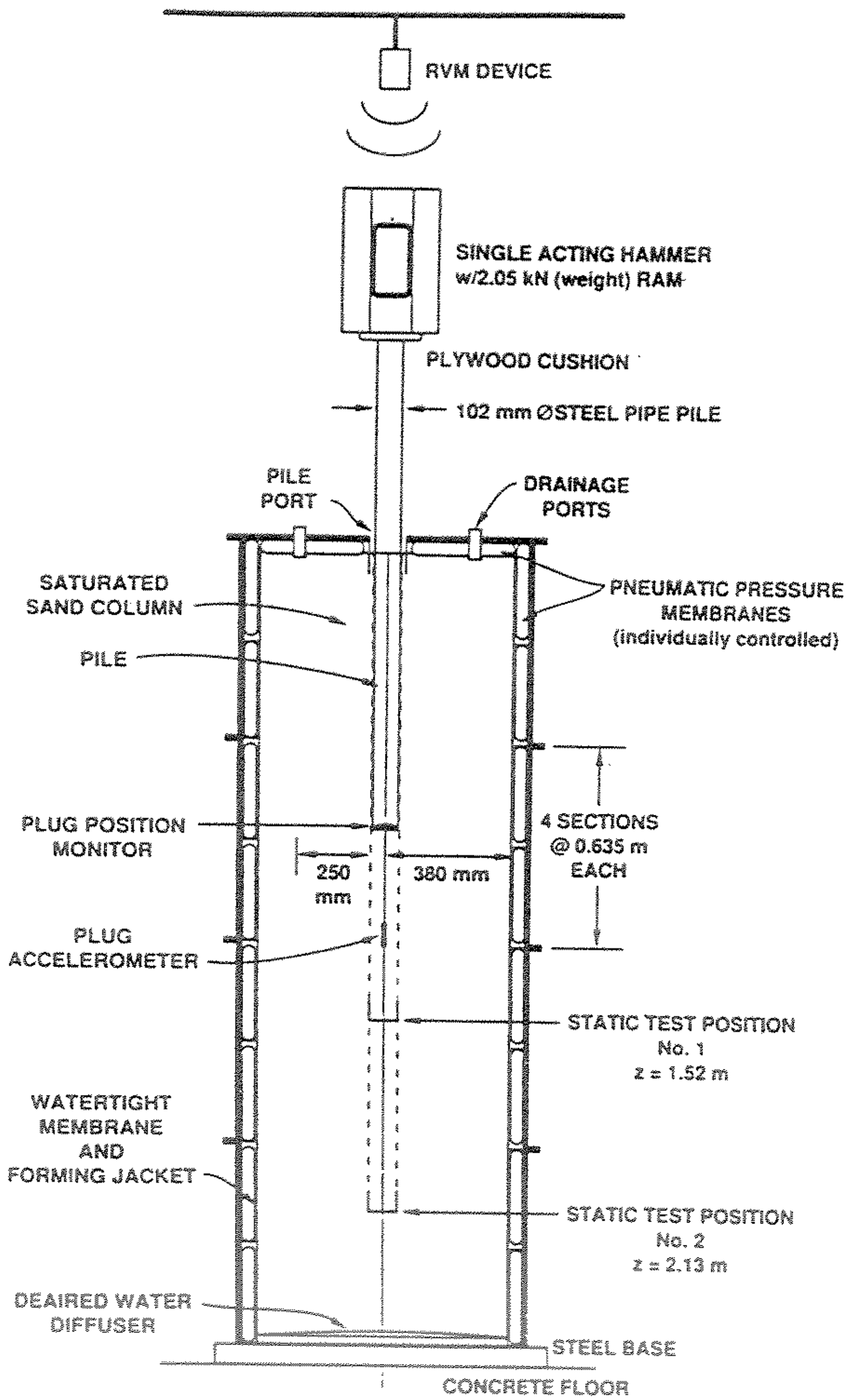
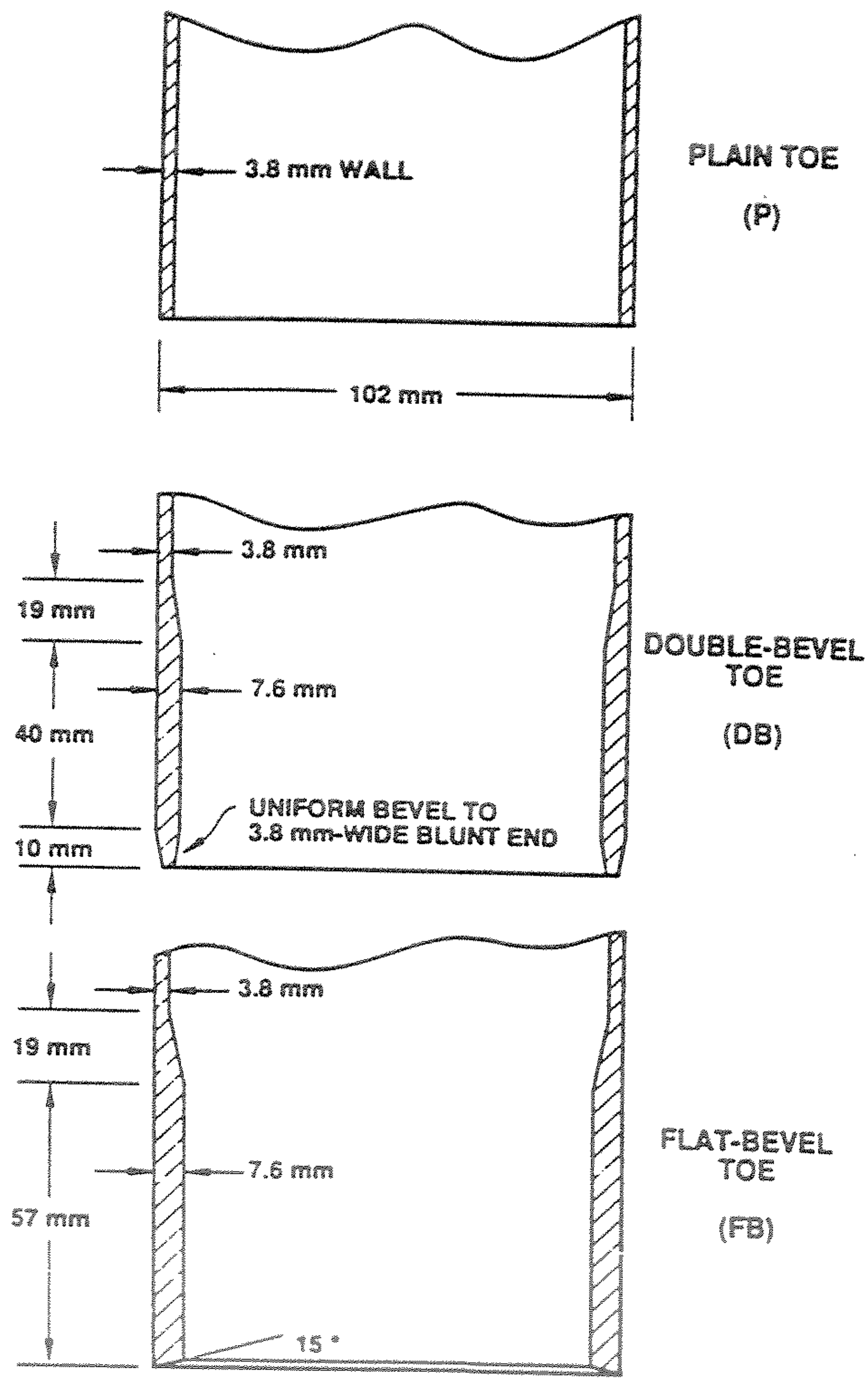


FIG. 1. Schematic Elevation View of Testing Chamber, Hammer and RVM Device


3.8 mm WALL
102 mm
PLAIN TOE
(P)
3.8 mm
19 mm
7.6 mm
40 mm
UNIFORM BEVEL TO
3.8 mm-WIDE BLUNT END
10 mm
DOUBLE-BEVEL
TOE
(DB)
3.8 mm
19 mm
7.6 mm
57 mm
15 °
FLAT-BEVEL
TOE
(FB)

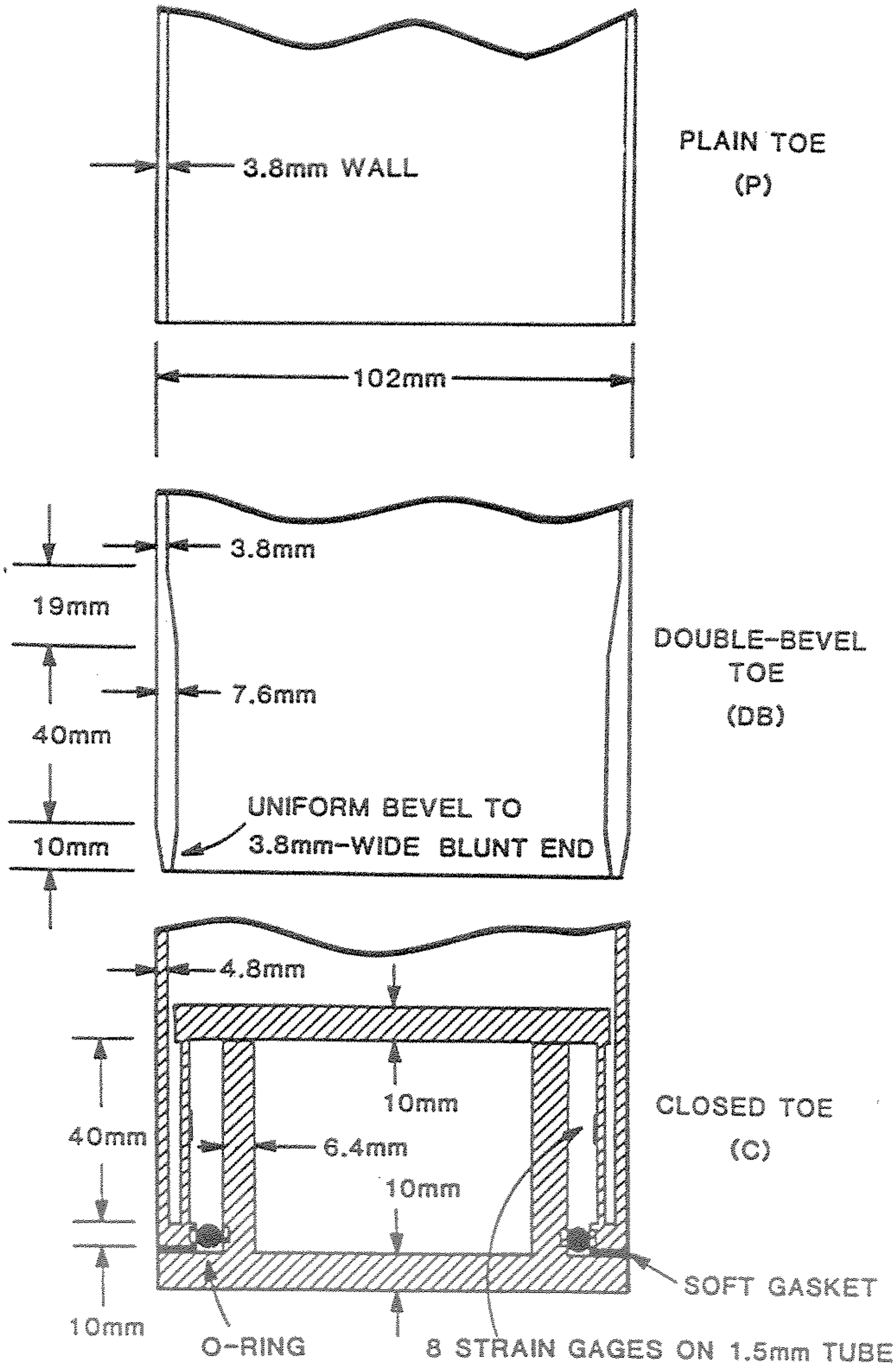


FIG. 3. **Schematic of Pile Toes**

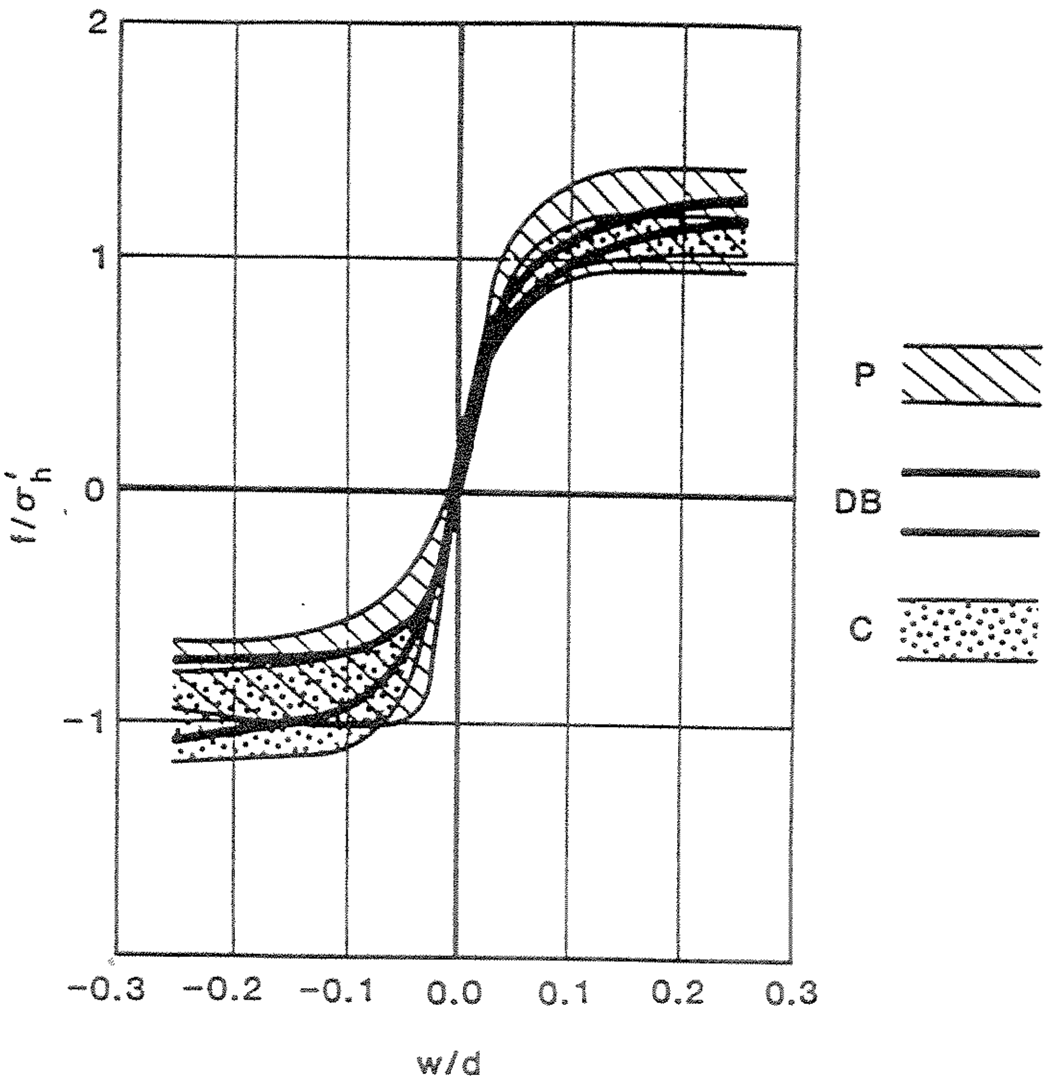


FIG. 5. Normalized *f-w* Relations; $z = 12\text{–}15d$

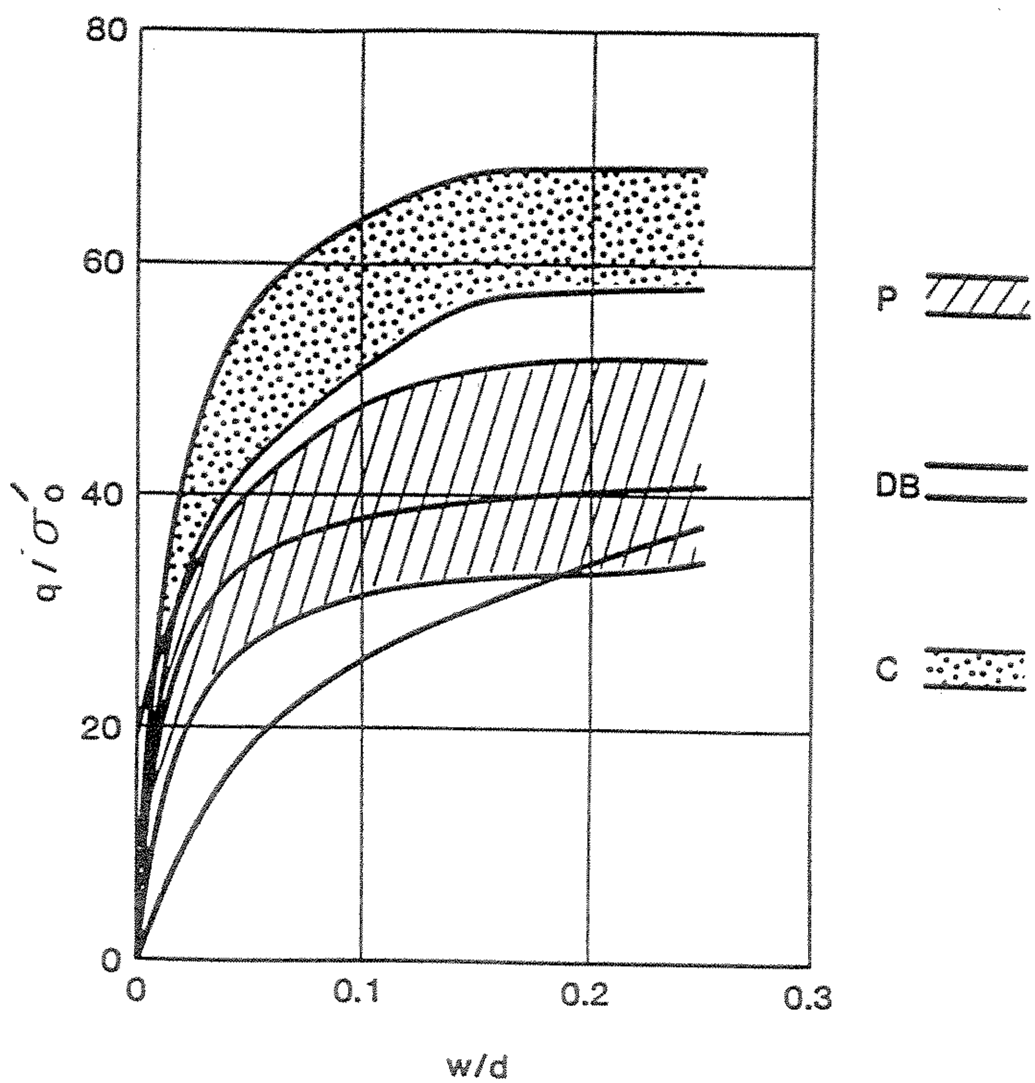


FIG. 7. Normalized q-w Relations

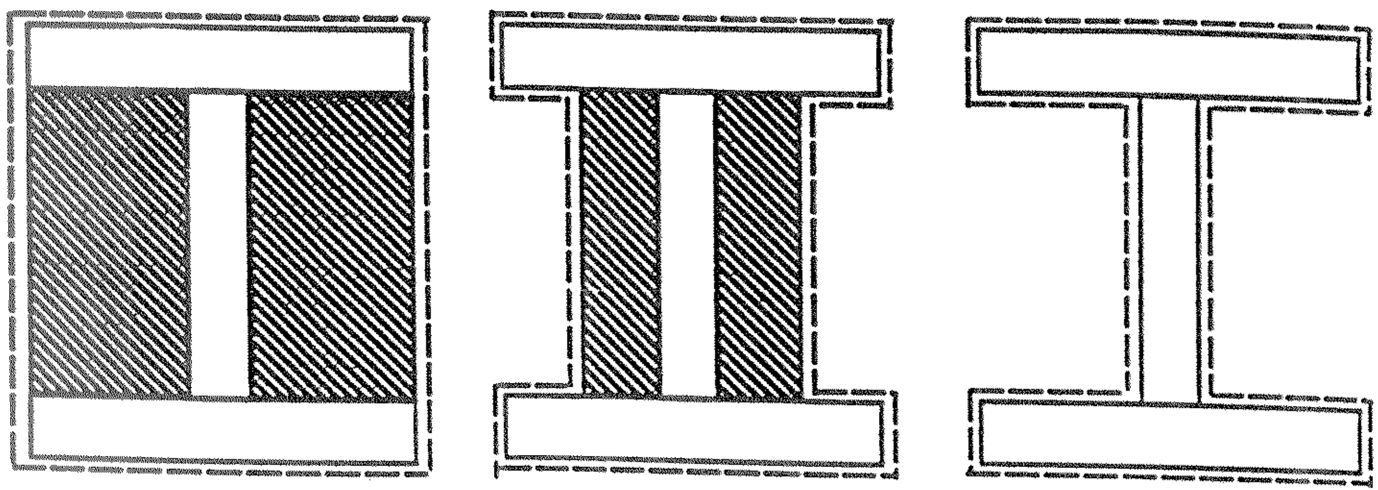
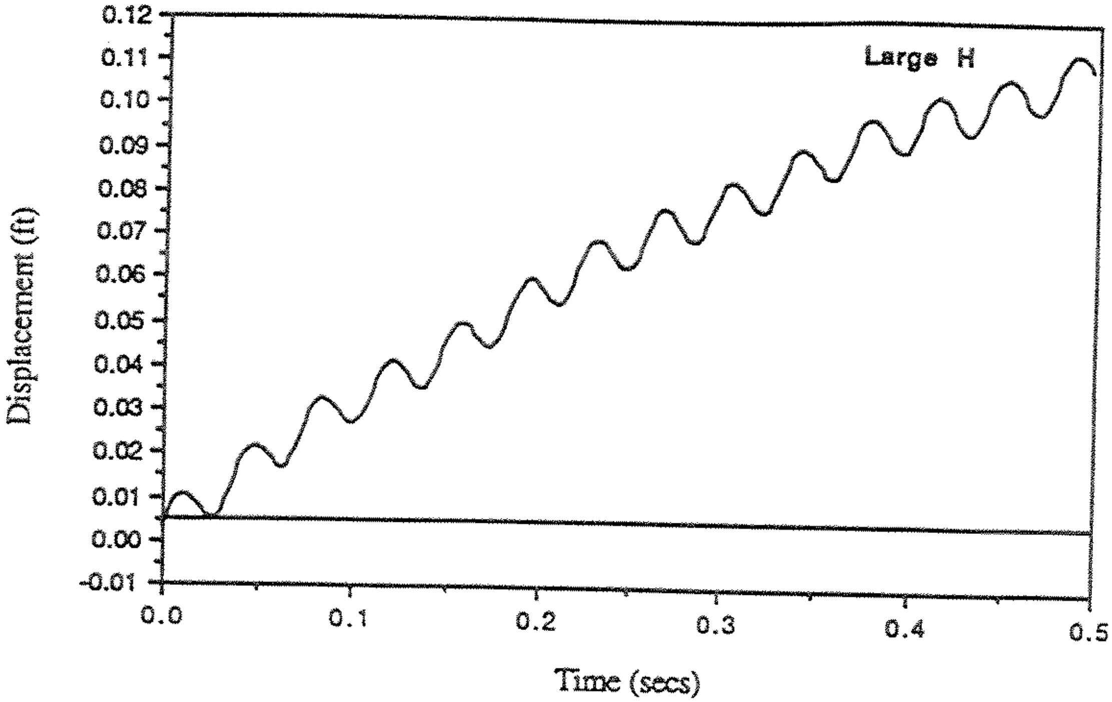
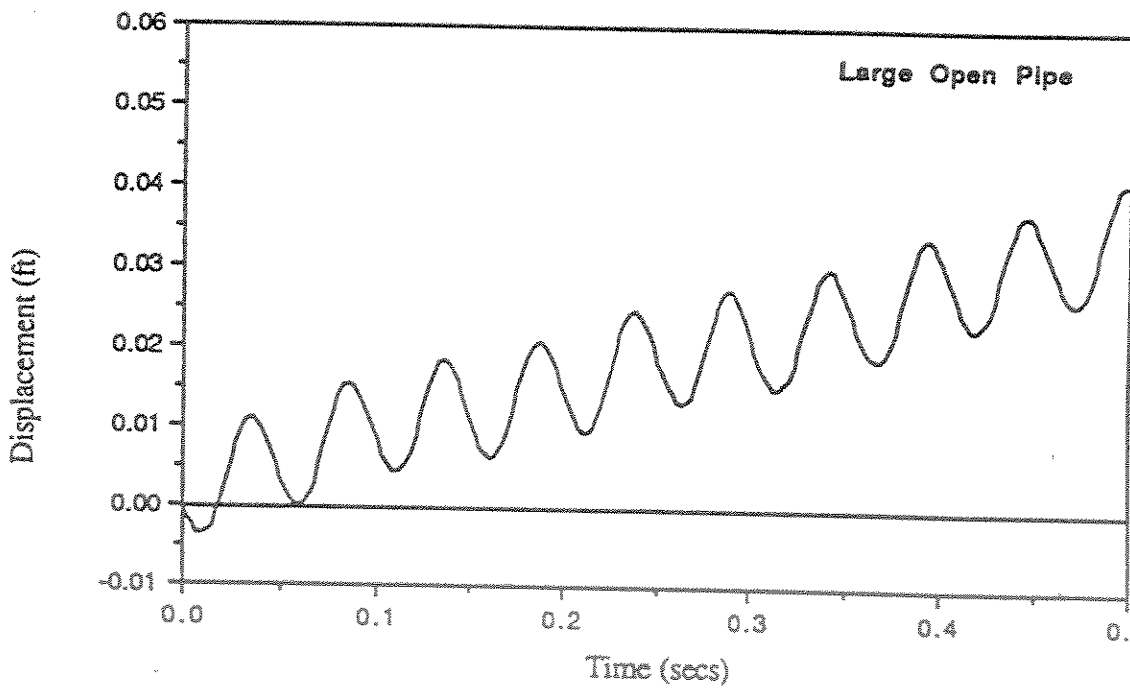


Fig. 5.22. Possible Plugging Conditions in the H Pile

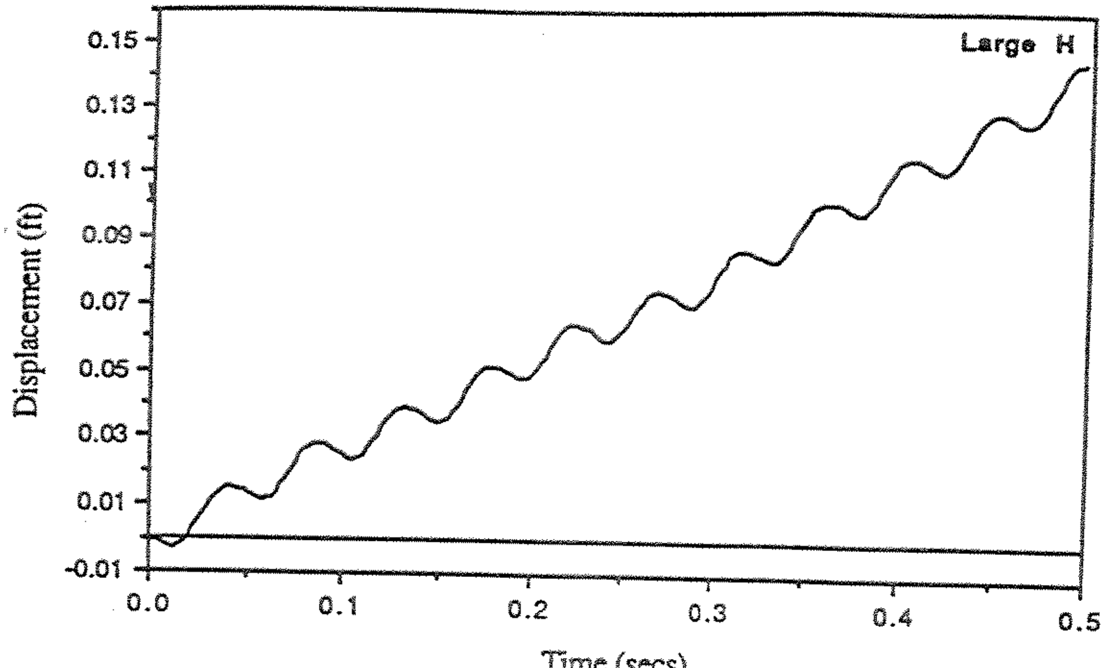


A) TEST NO : 6C (H/D/10) - At 40 in. Penetration Depth

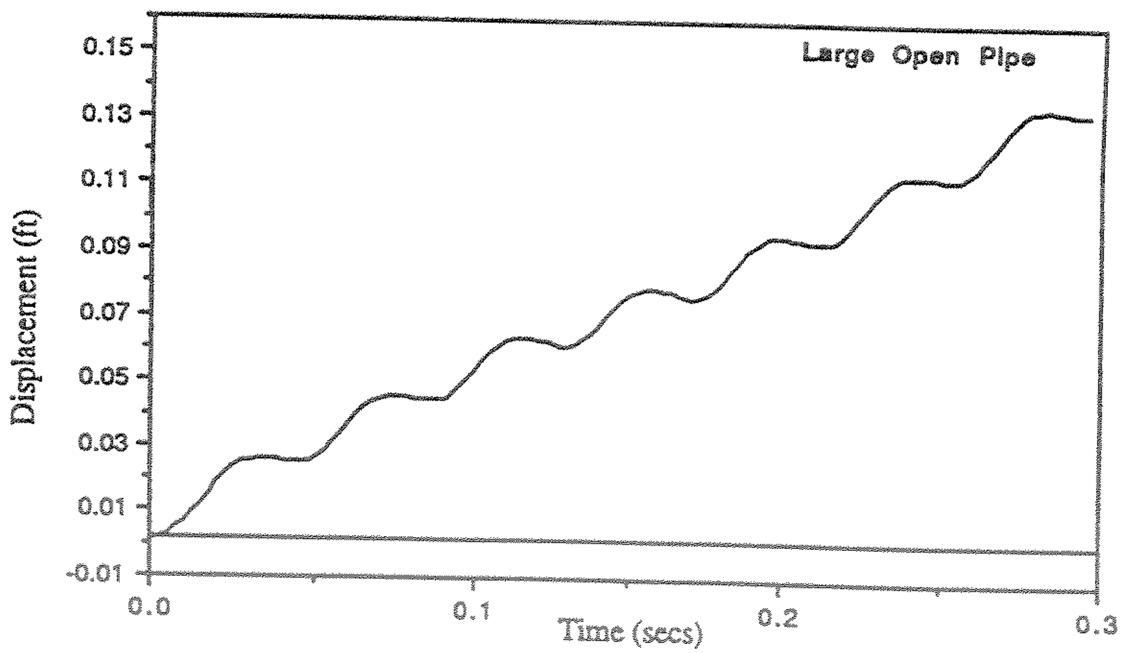


B) TEST NO : 5B (COE/D/10) - At 45 in. Penetration Depth

Fig. 4.75. Comparison of Displacement vs. Time at Mid-Depth Penetration in Dry Sand. A) H Pile B) Open Pipe Pile - Large-Size Piles



A) TEST NO : 16C (H/S/R/10) - At 55 in. Penetration Depth (Prior to Redrive)



B) TEST NO : 17B (COE/S/R/10) - At 55 in. Penetration Depth (Prior to Redrive)

Fig. 4.91. Comparison of Displacement vs. Time Prior to Redrive in Saturated Sand. A) H Pile B) Open Pipe Pile - Large-Size Piles

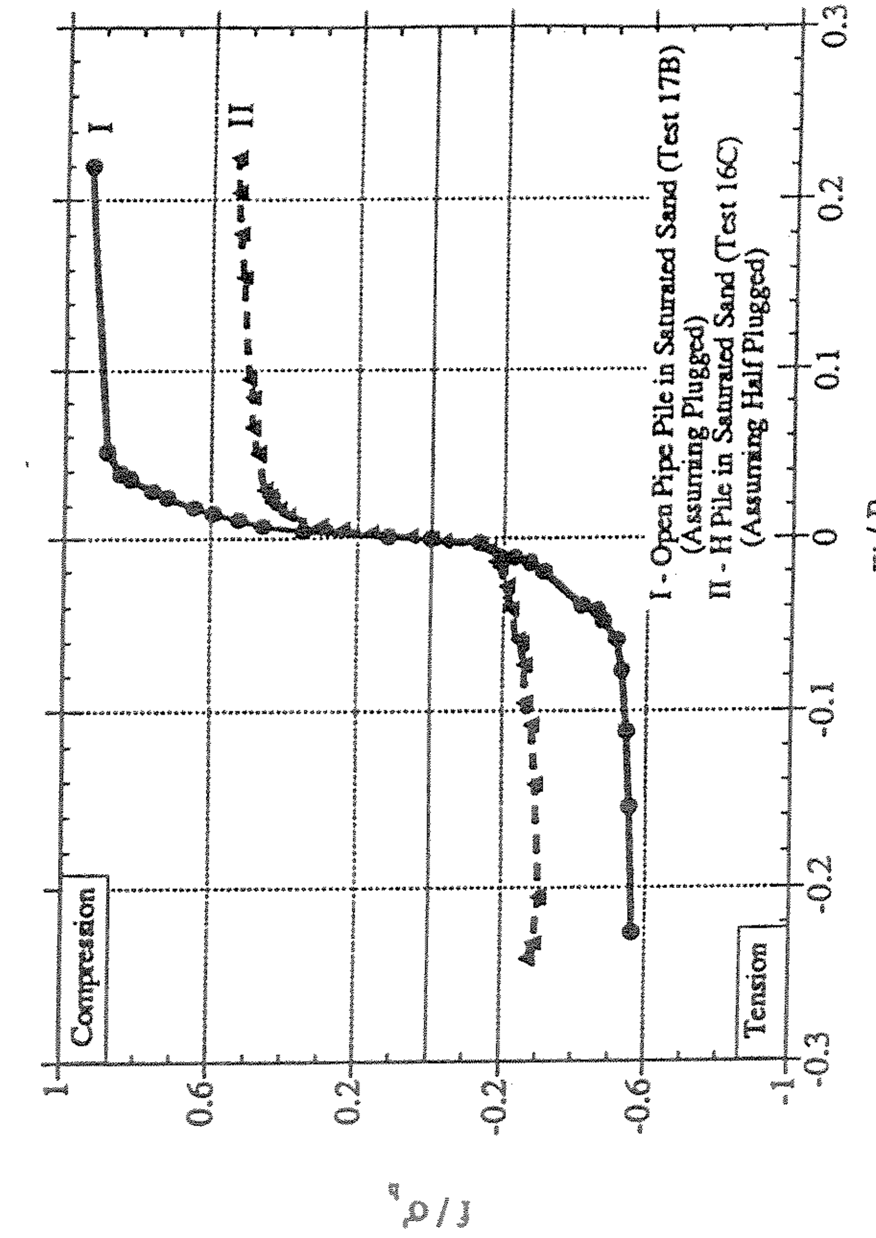


Fig. 5.33. Average Normalized f - w Relationships for Large Piles in Saturated Sand at Final Pile-Penetration Depth

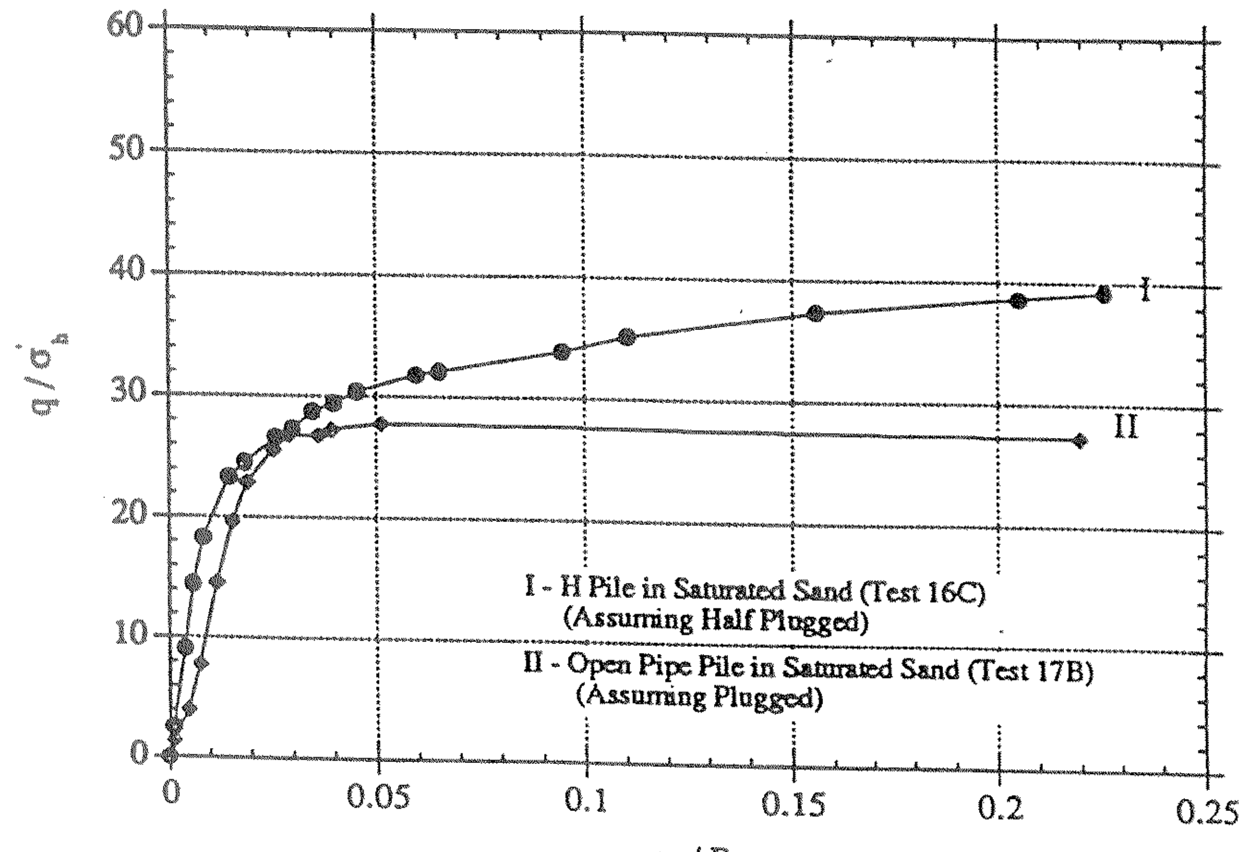


Fig. 5.37. Normalized q - w Relationships for Large Piles in Saturated Sand at Final Pile Penetration Depth

A Look at Plugging

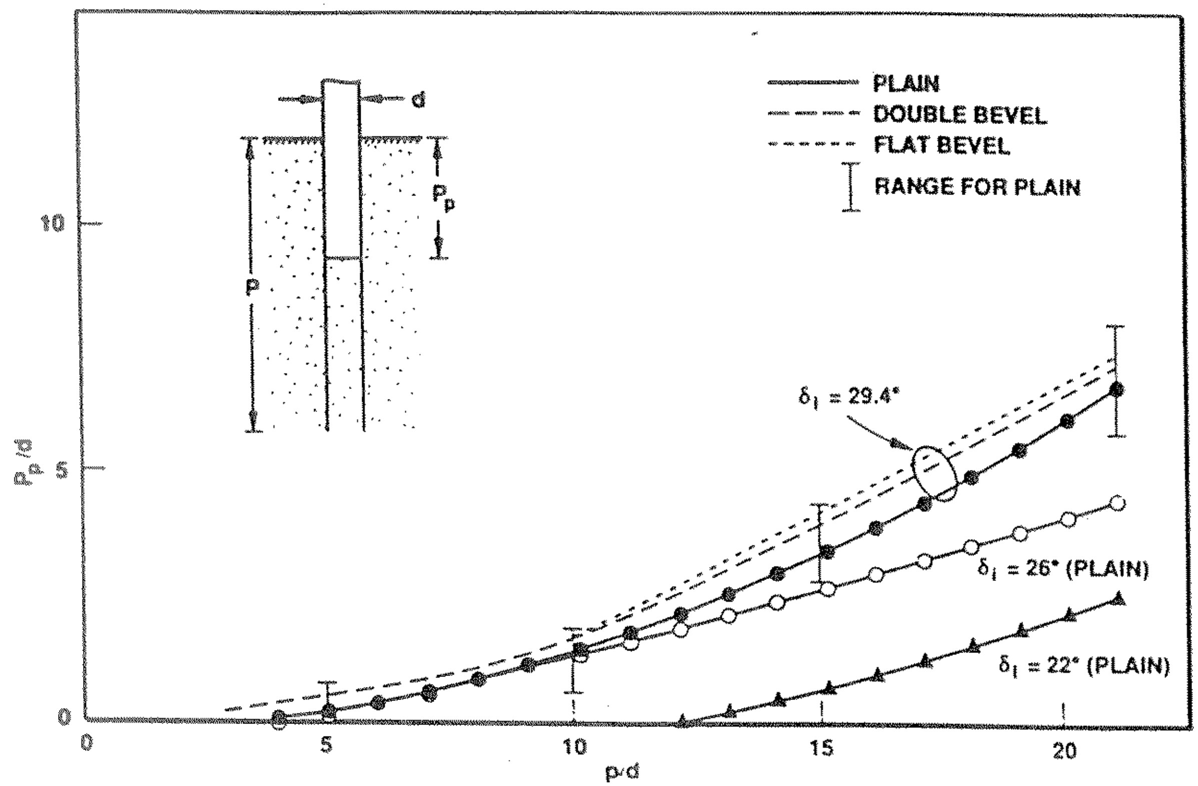


FIG. 5. Plug Penetration versus Pile-Toe Penetration for various Toe Geometries and Angles of Internal Wall Friction

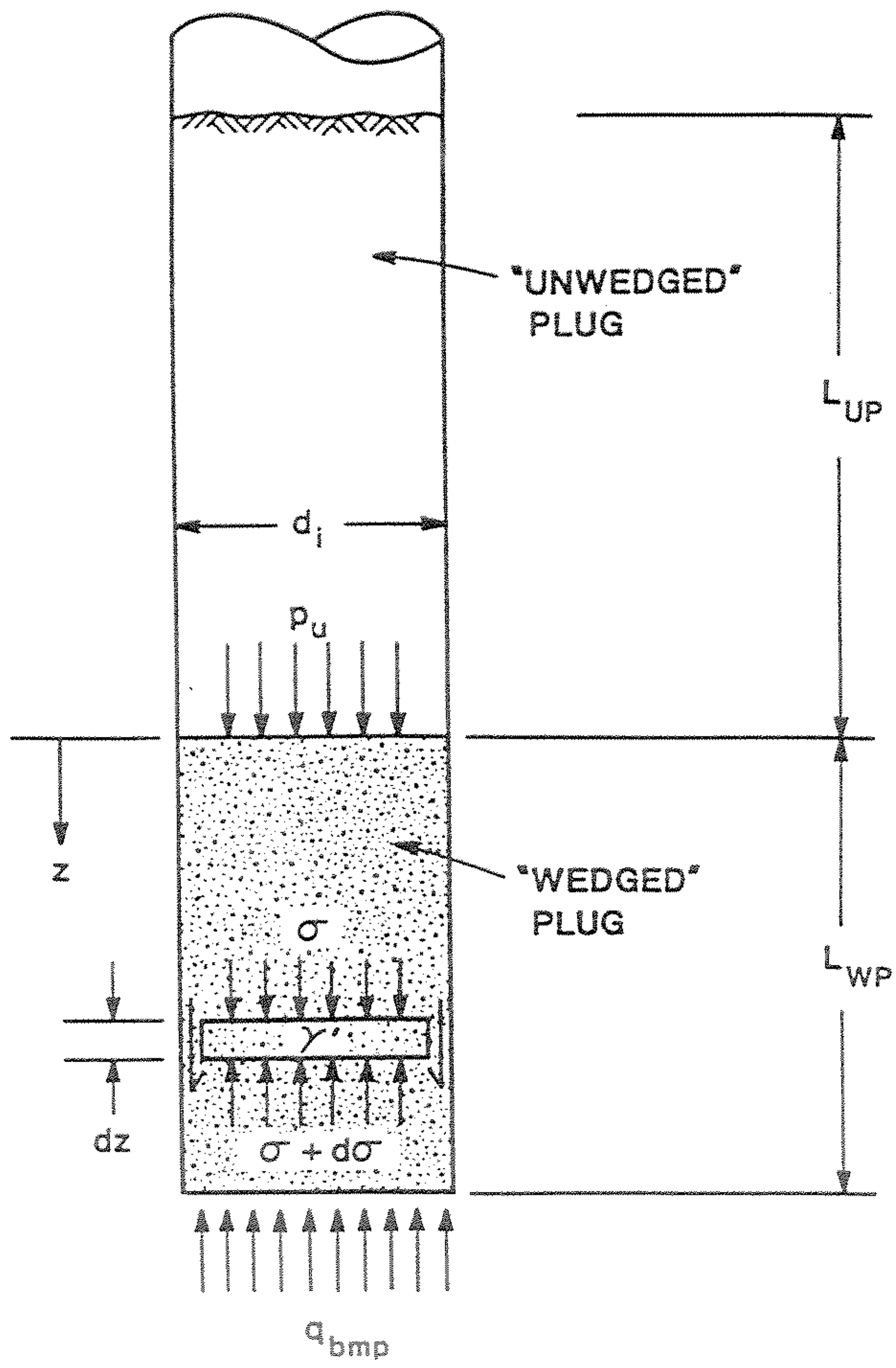


FIG. 12. Schematic Definition Diagram for Plug Equations

$$\frac{q_{mbp}}{\gamma' L_{wp}} = \left[\frac{L_{up}}{L_{wp}} + \left(\frac{d_i}{L_{wp}}\right)\frac{1}{4\beta}\right]e^{4(L_{wp}/d_i)\beta} - \left(\frac{d_i}{L_{wp}}\right)\frac{1}{4\beta}$$

$\beta = K \tan \delta$ within the plug
K taken as 1 for dense (incompressible) sand
δ is angle of inside wall friction

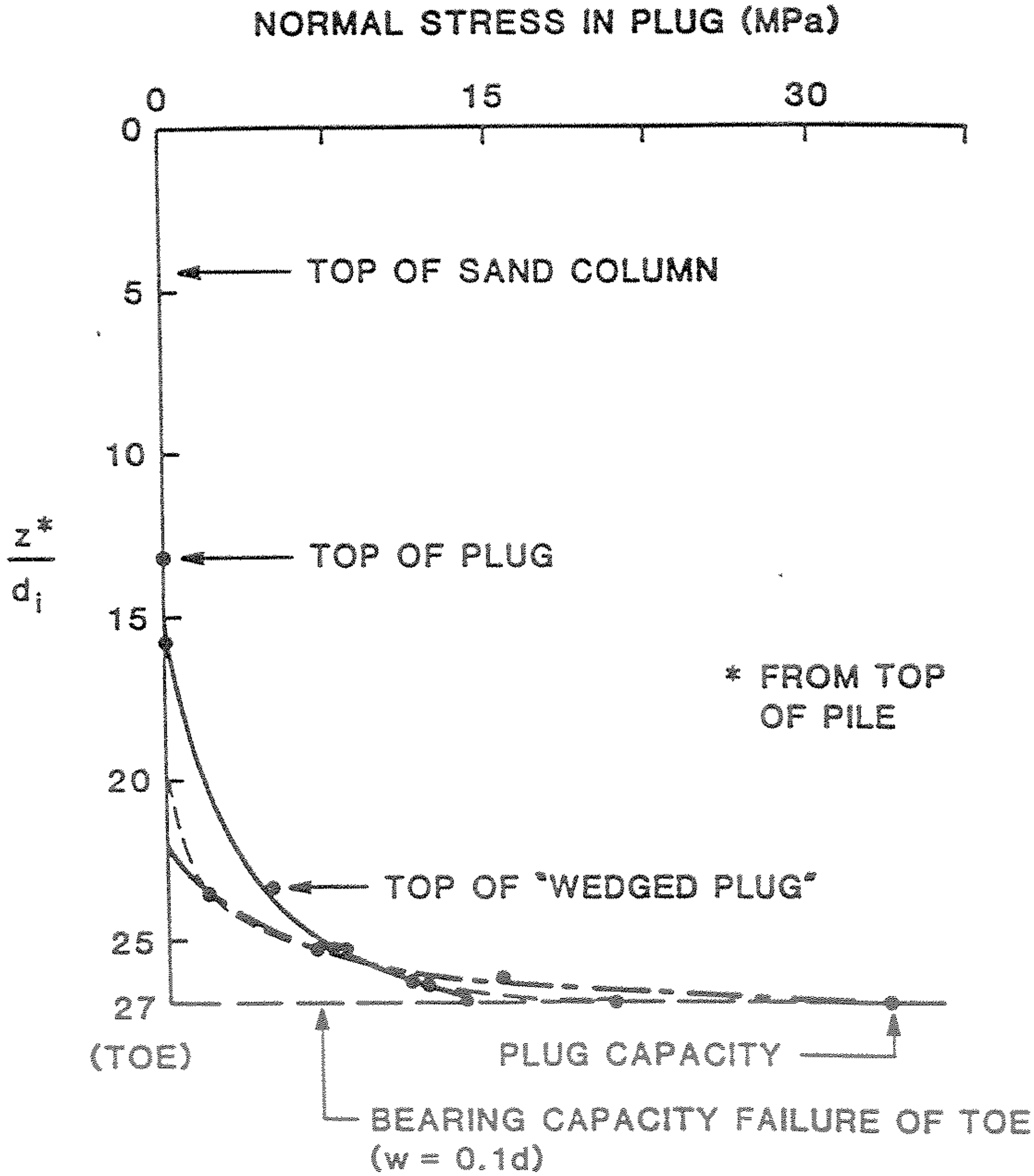


FIG. 11. Vertical Stress in Soil Plug versus Depth

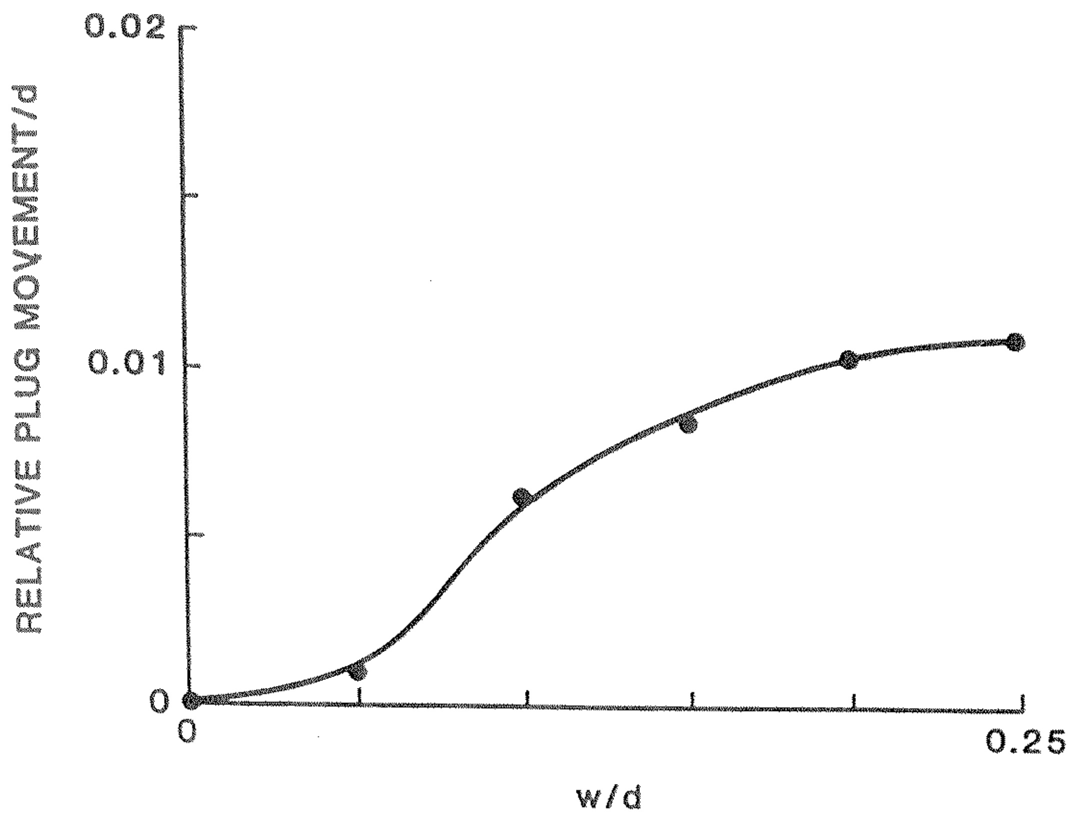
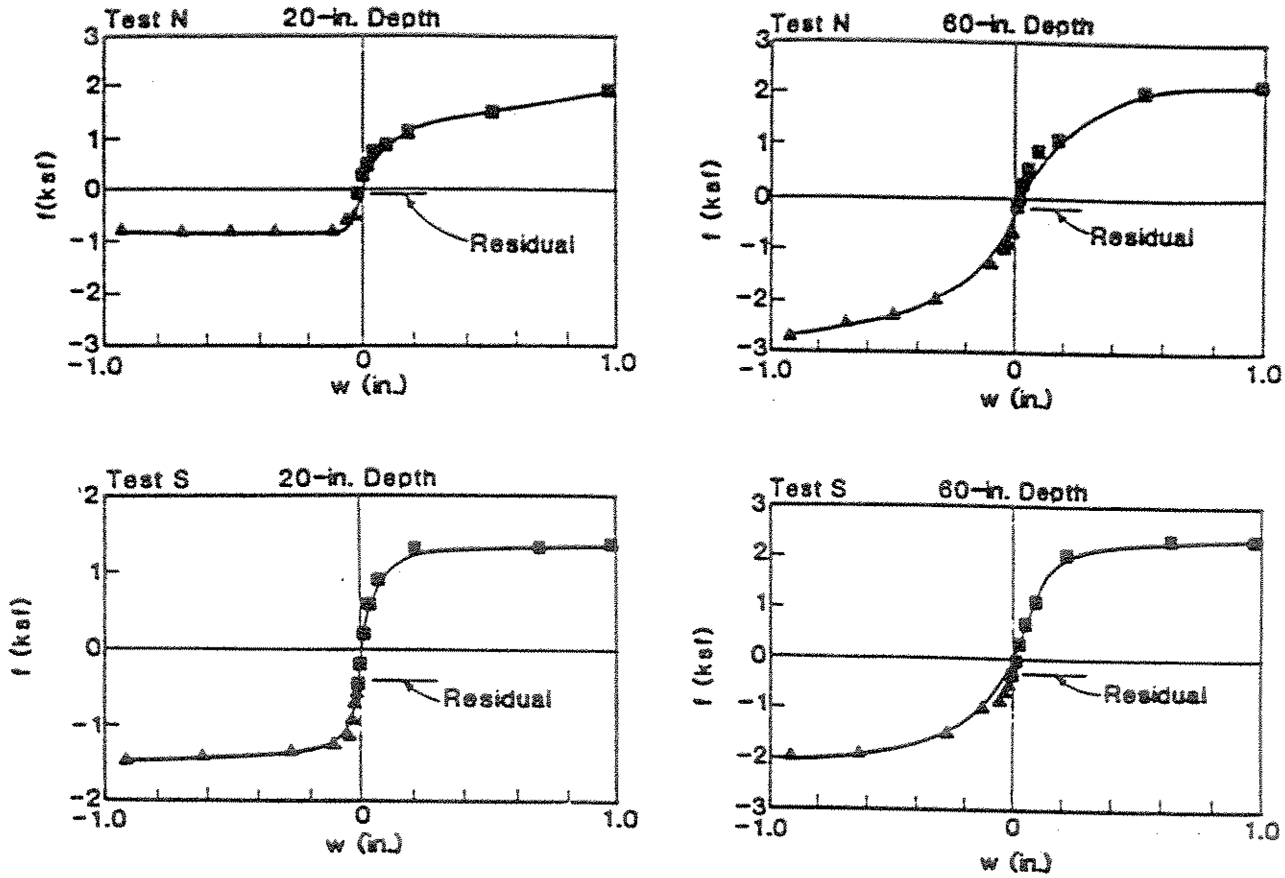


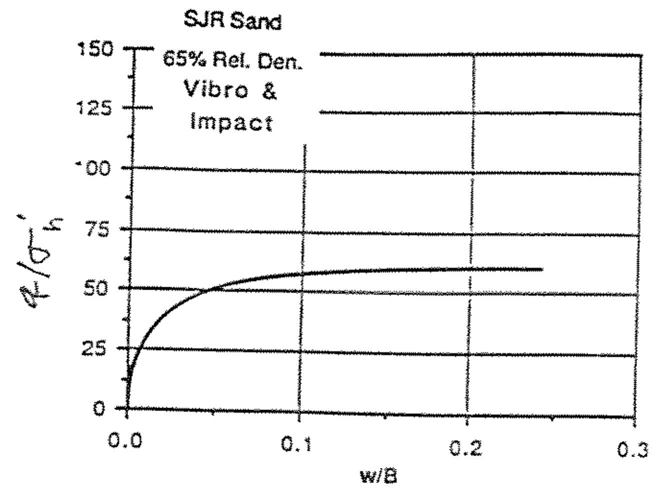
FIG. 8. Plug Movement versus Pile Movement; Compression Test on Plain-Toe Pile

Effect of Installation Method



Top: 4-in. ϕ closed toe model pile VIBRATED into coarse sand at 90% relative density / 10 psi effective confining pressure.

Bottom: 4-in. ϕ closed toe model pile driven by IMPACT into coarse sand at 90% relative density / 10 psi confining pressure.



...re 34. *Summary normalized* q-w *relation for pile driven by* ...act *and vibrated into SJR sand at 65% relative density.*

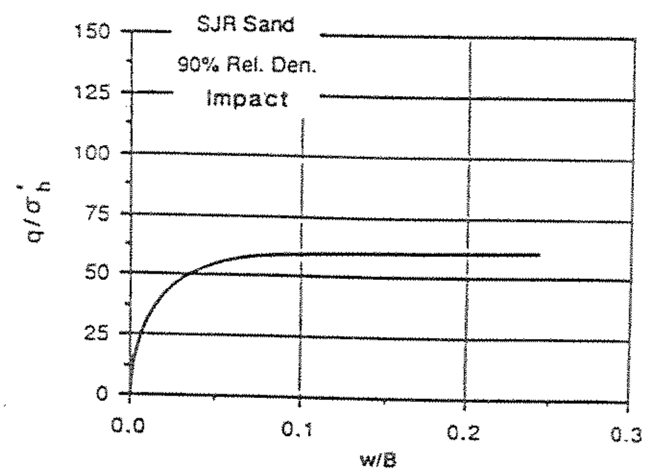


Figure 35. Summary normalized q-w *relation for pile driven by impact into SJR sand at 90% relative density.*

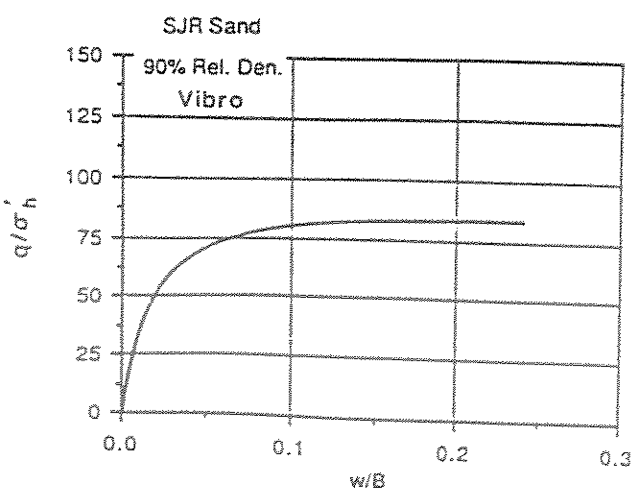
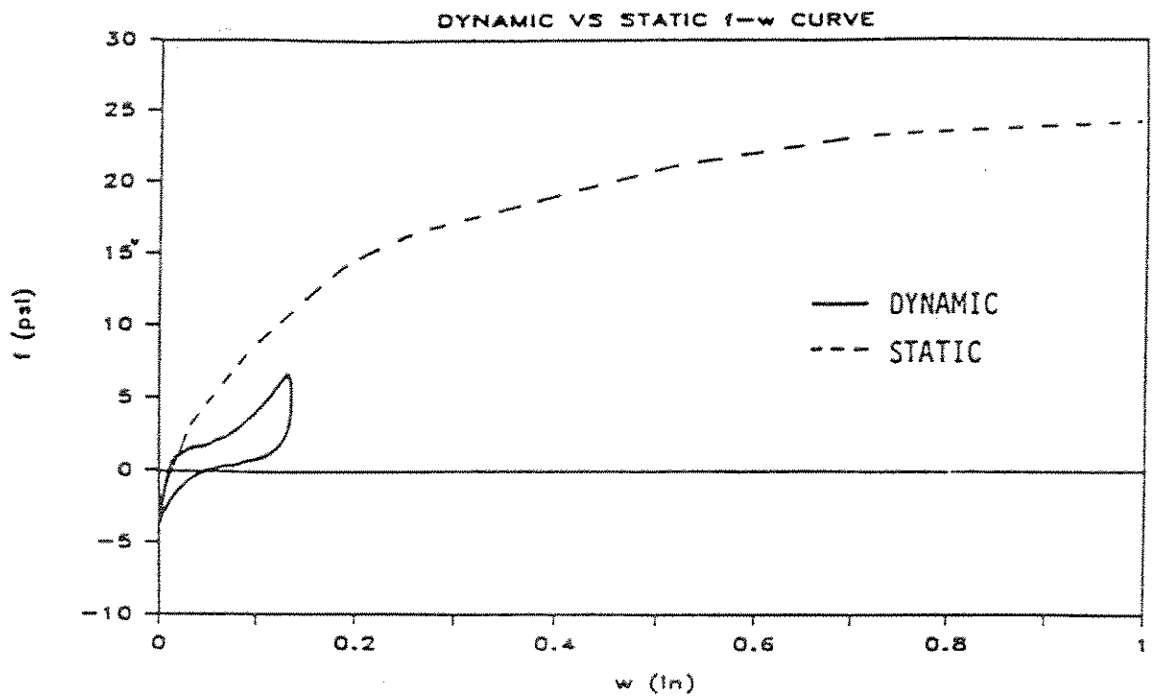


Figure 36. Summary normalized q-w *relation for pile vibrated into SJR sand at 90% relative density.*



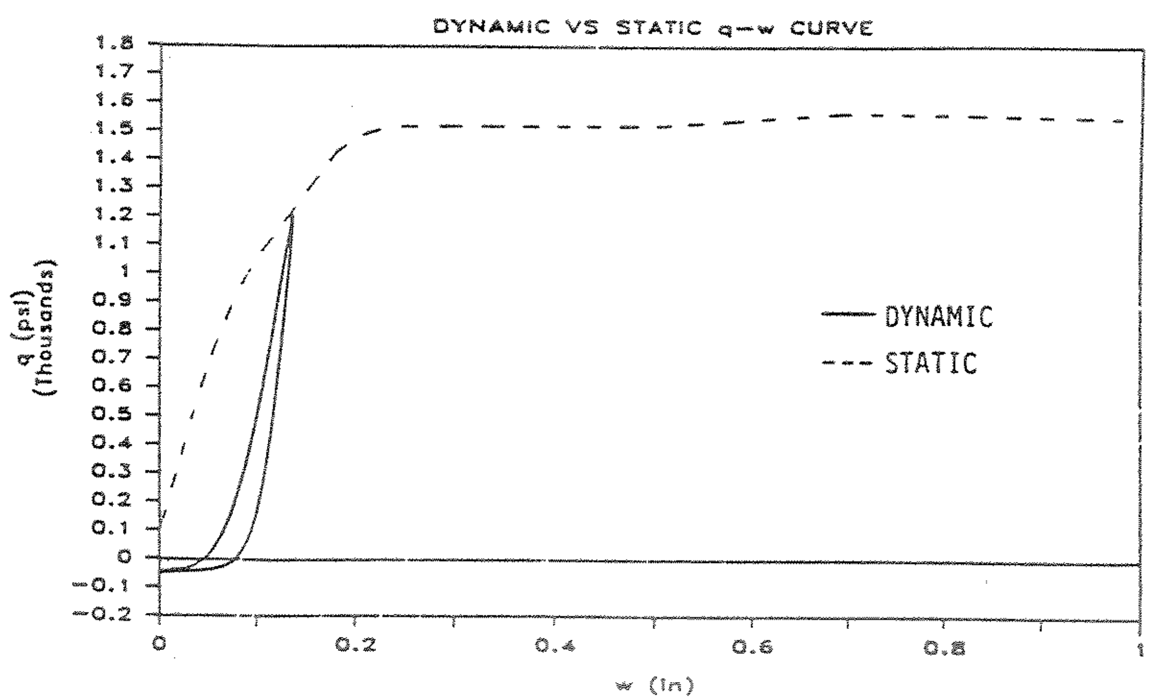
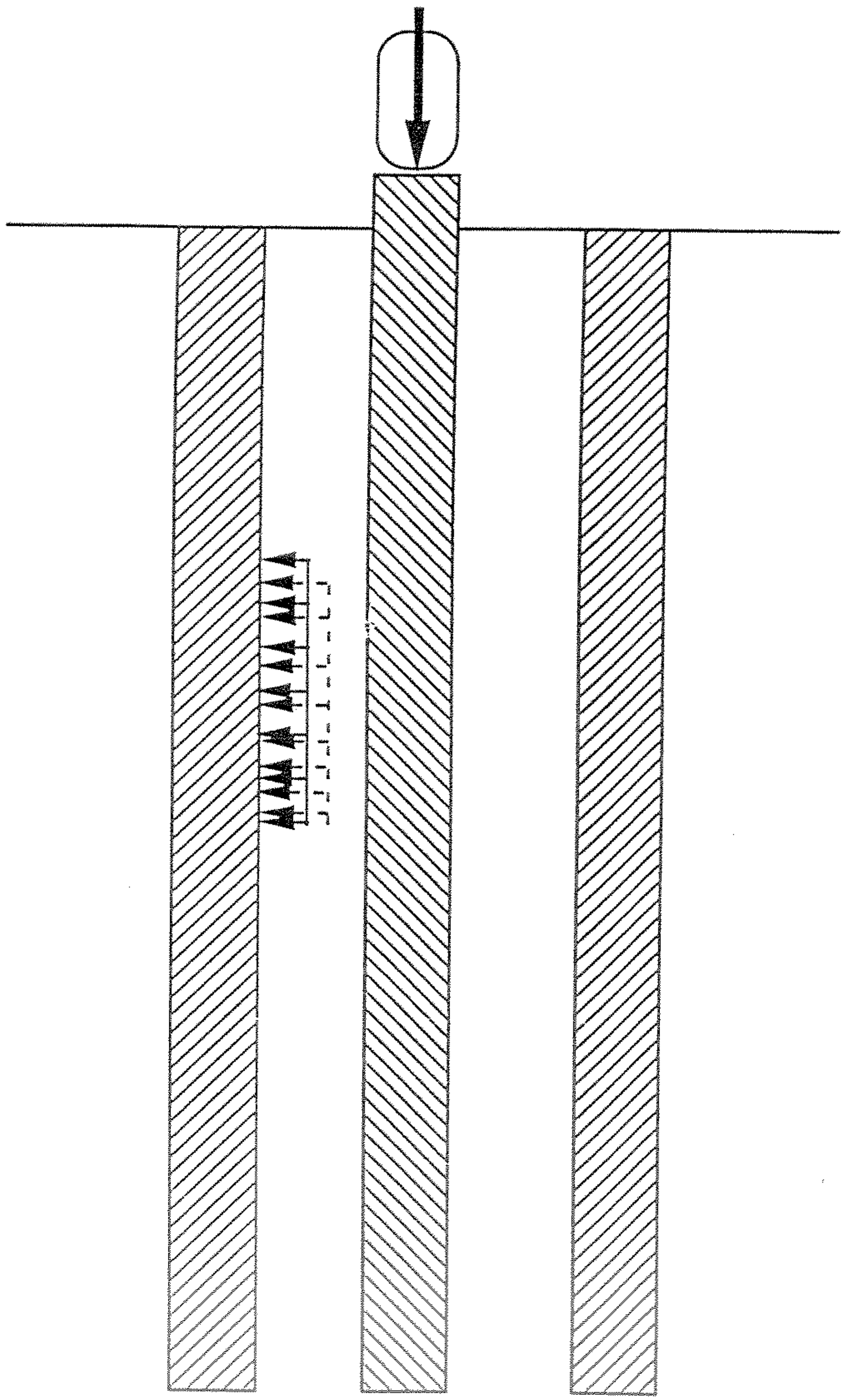
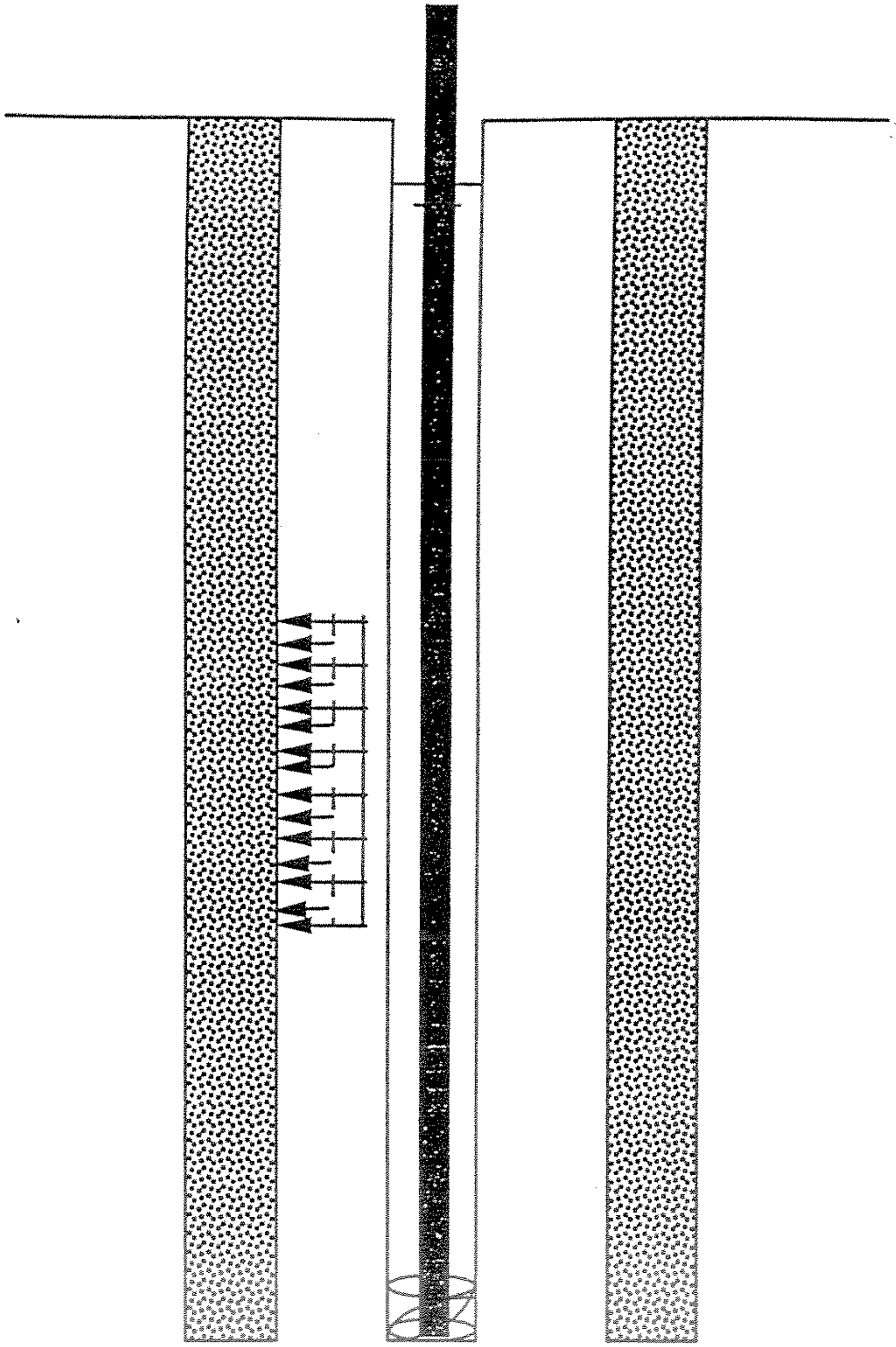


Figure 53. Comparison of unit load transfer curves for pile at refusal and for static loading; Test 17; BLS (coarse) sand; 90% relative density; 20-psi effective chamber pressure; pile-at-refusal curves for 74-inch penetration.

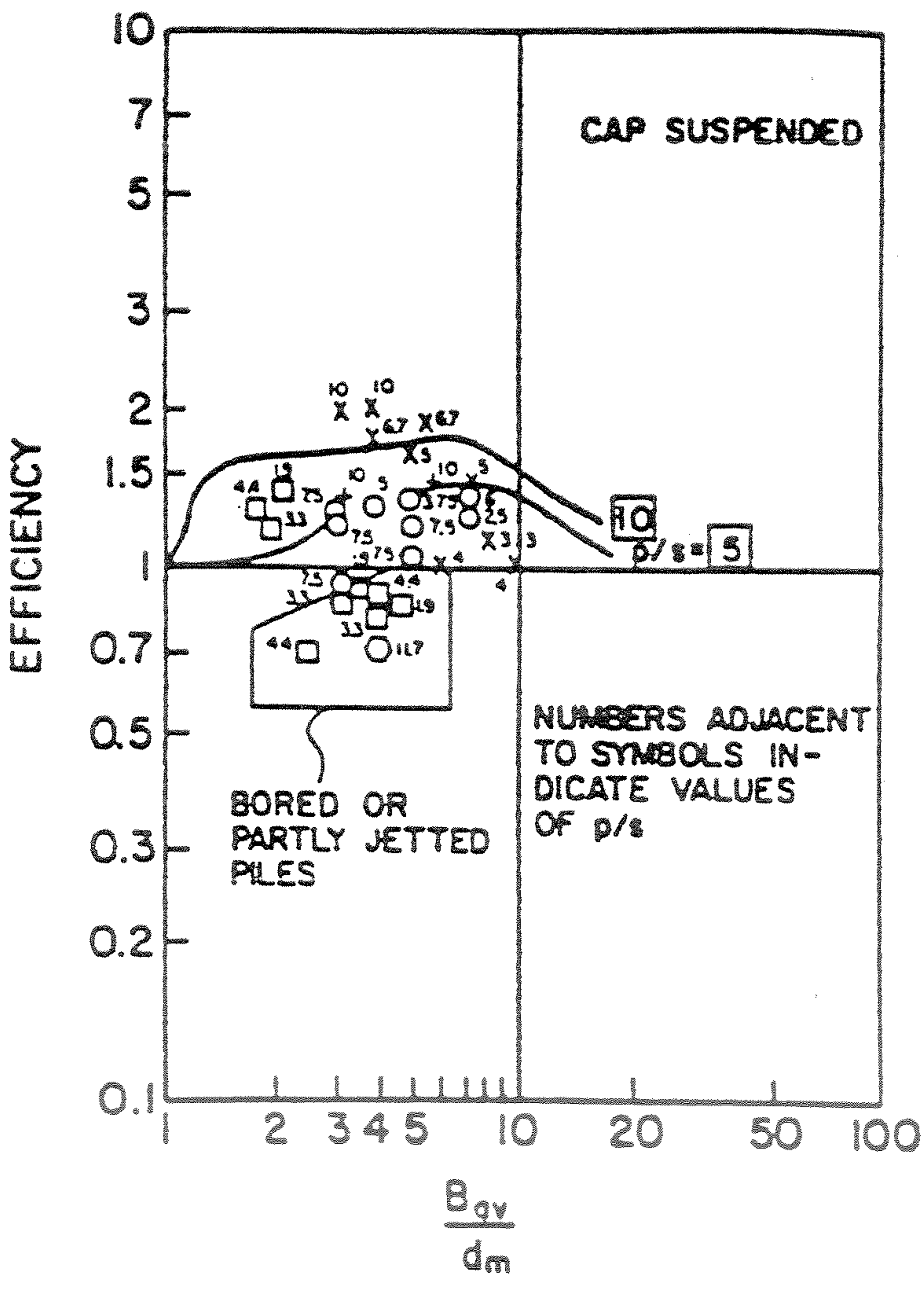
Group Behavior



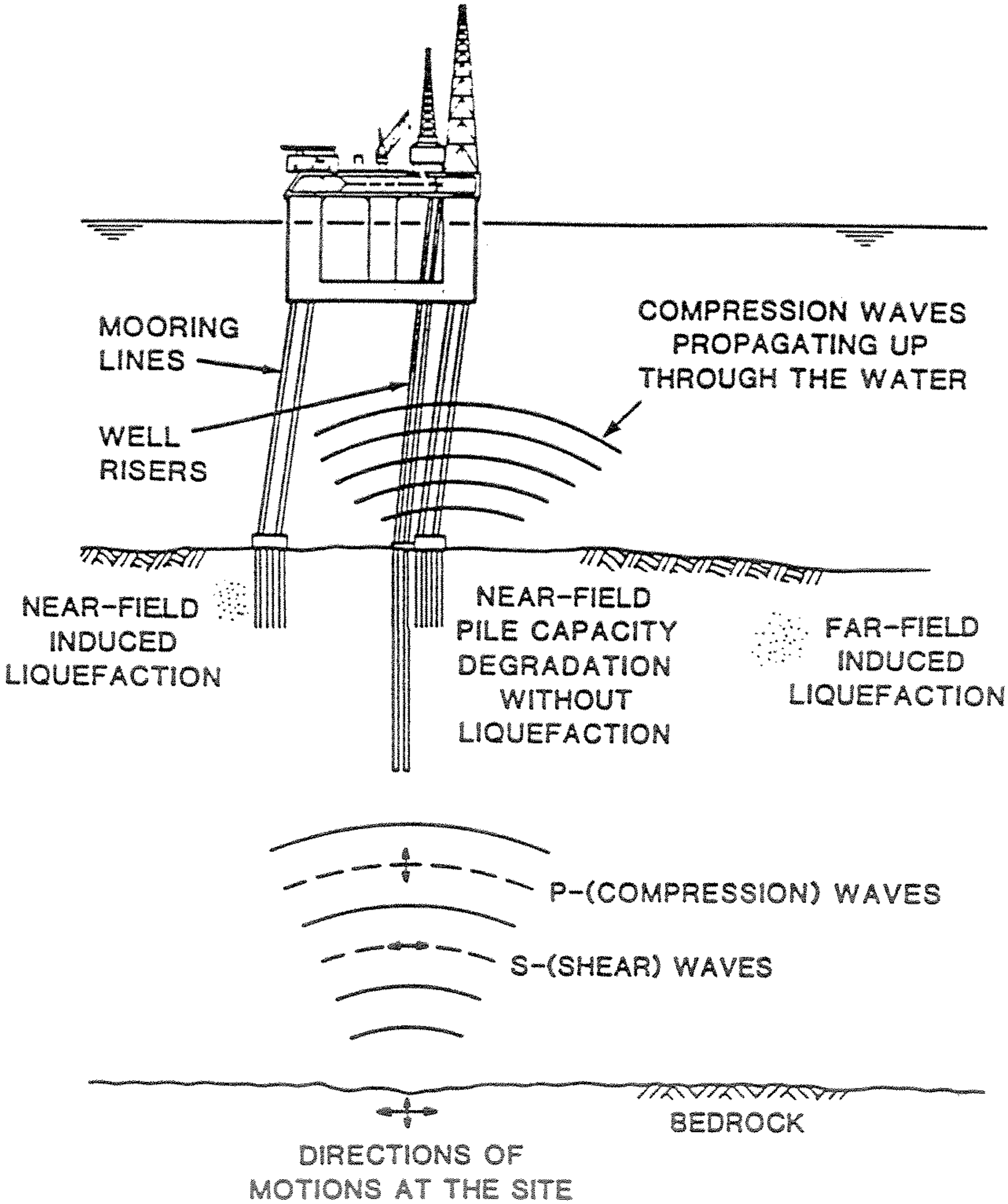
Driven Piles



Drilled Shafts

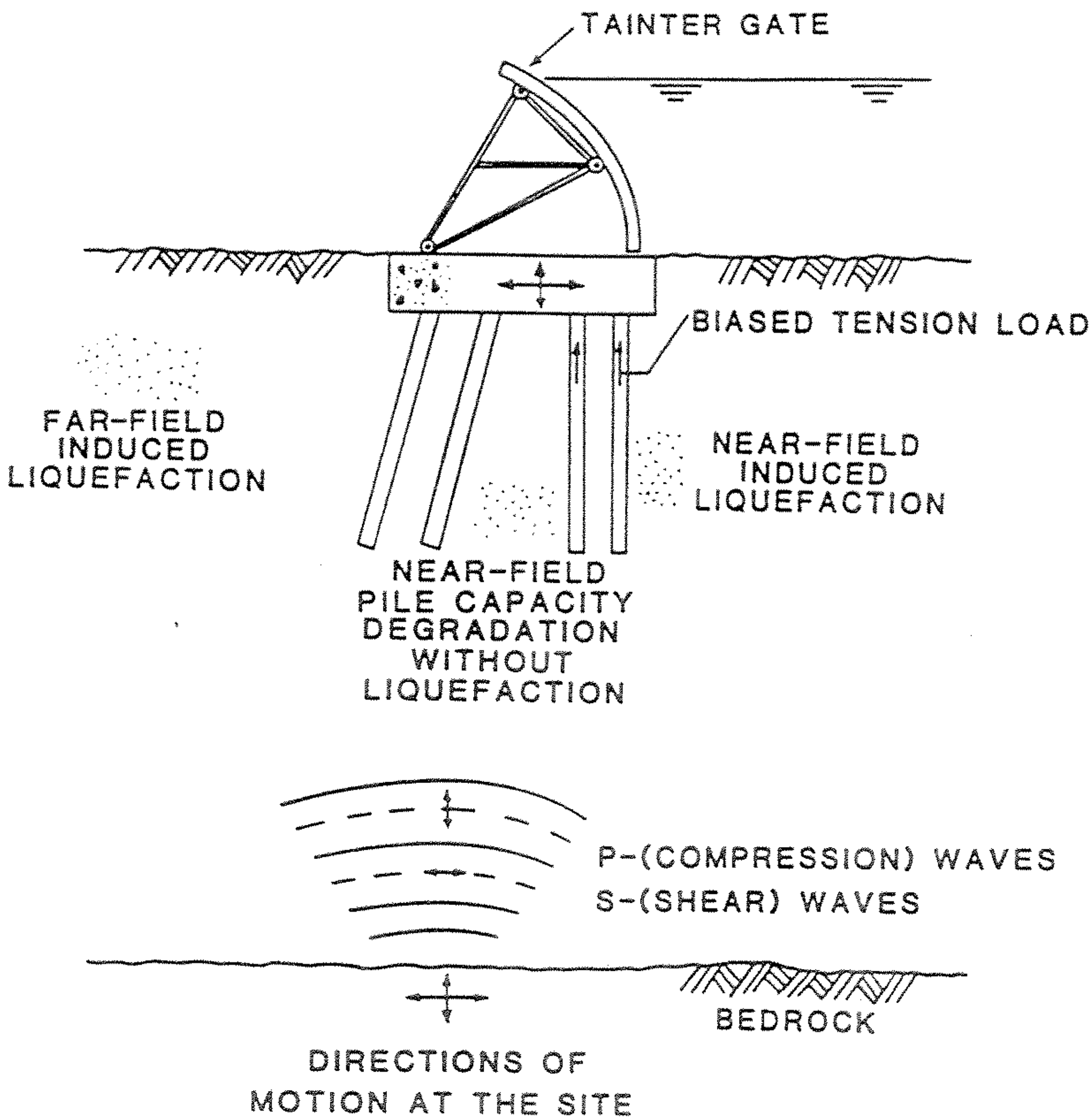


CAP SUSPENDED
EFFICIENCY
NUMBERS ADJACENT TO SYMBOLS INDICATE VALUES OF p/s
BORED OR PARTLY JETTED PILES
p/s = 5
10
$\frac{B_{av}}{d_m}$

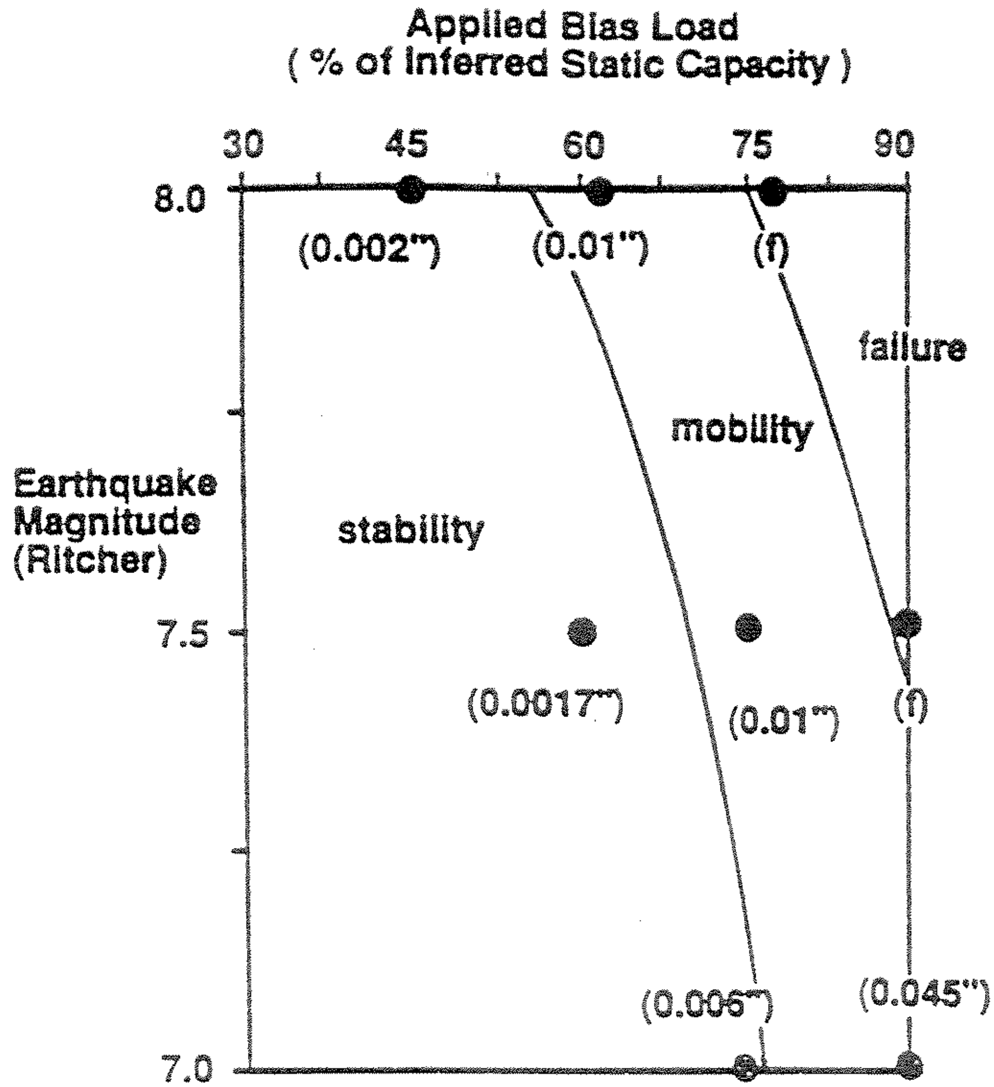


Idealization of Induced Stress Waves During a Seismic Event for an Offshore TLP.

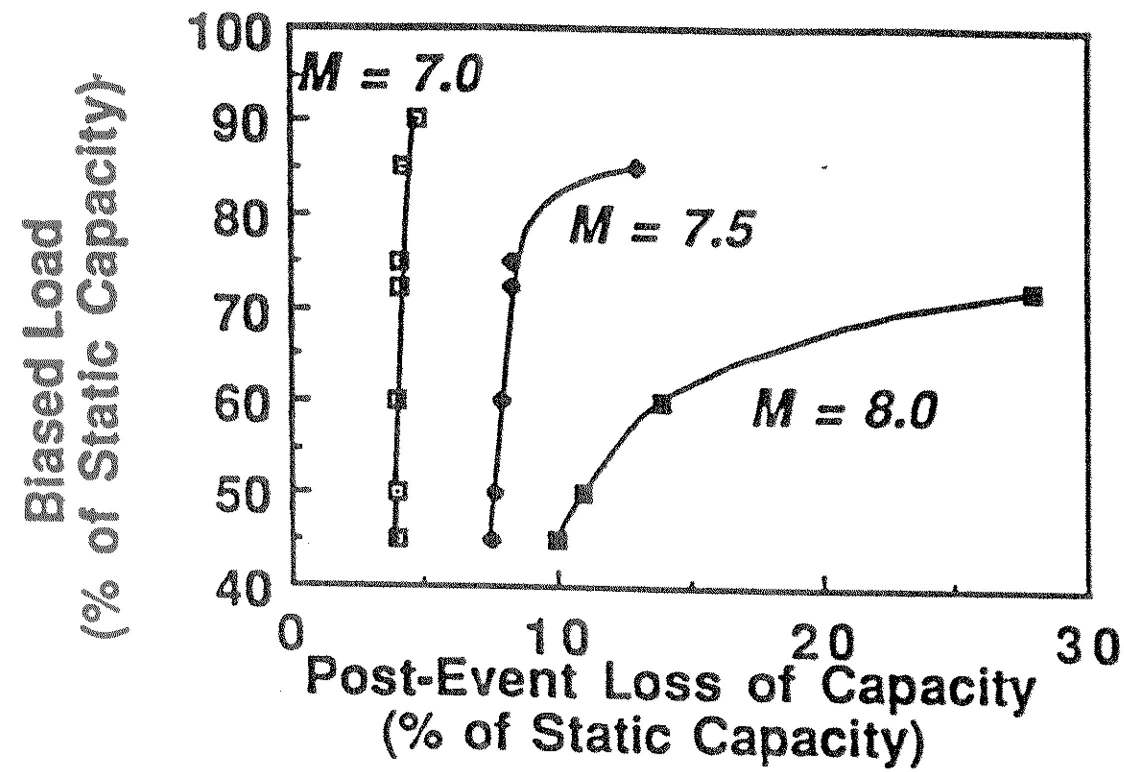
Seismic Behavior



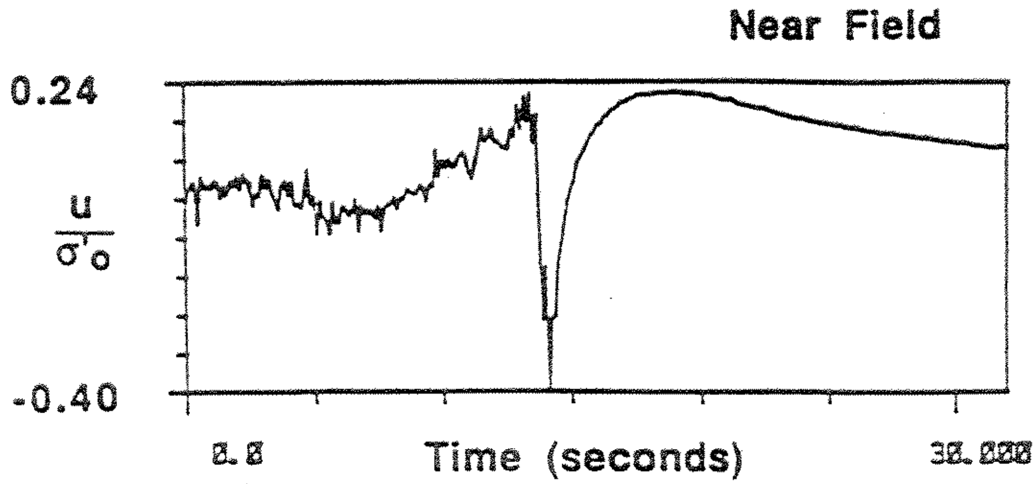
Idealization of Induced Stress Waves During a Seismic Event for a Hydraulic Structure.

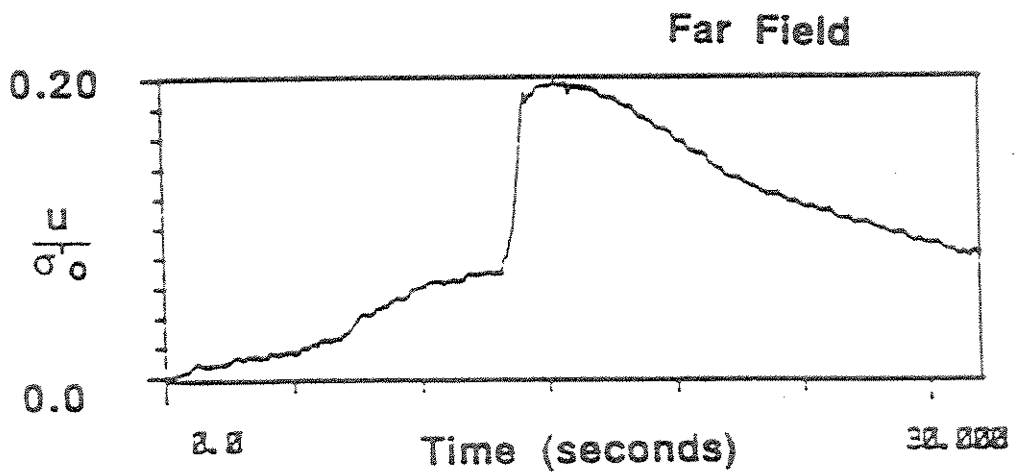


Stability, Mobility and Failure Conditions; Horizontal Soil Motion; σ'_o = 2.5 psi; Epicentral Distance = 74 km; D_r = 55%

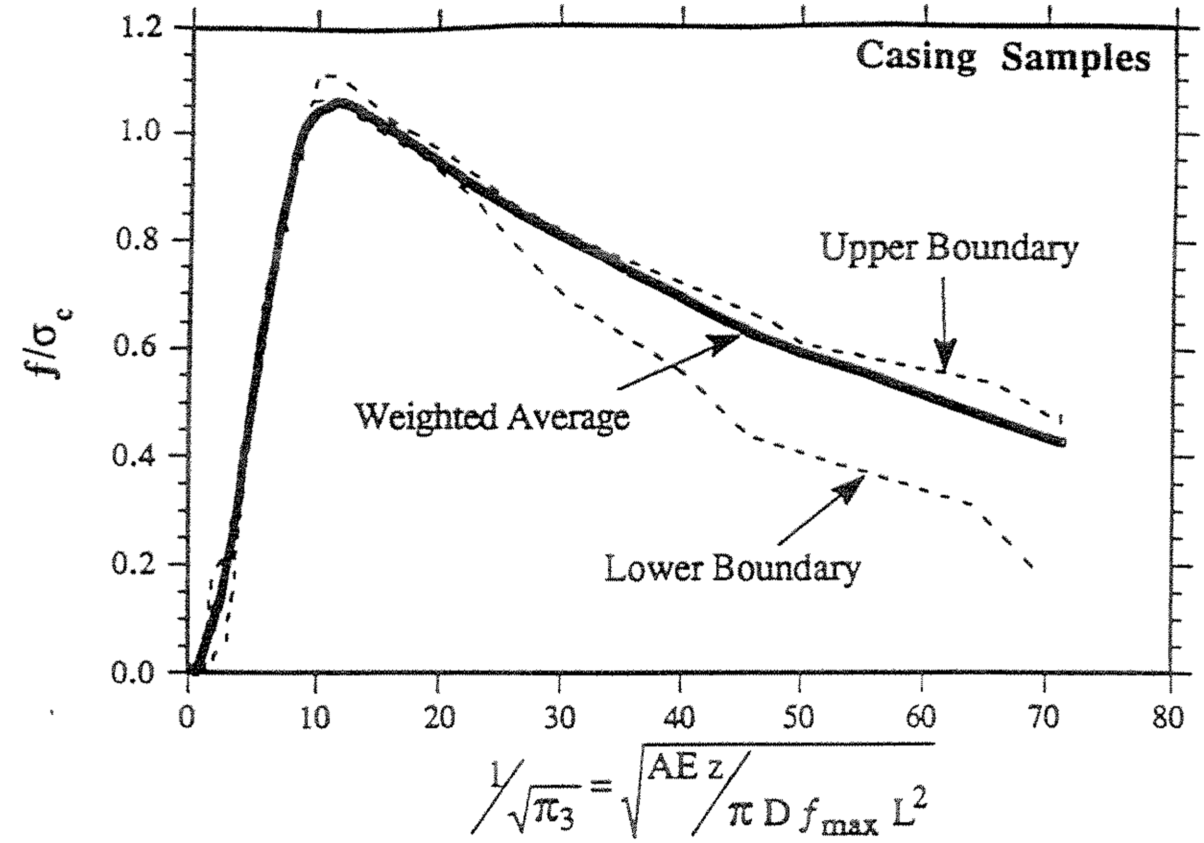


Loss of Static Capacity Following Strong Ground Motion (Relative Density = 55%)

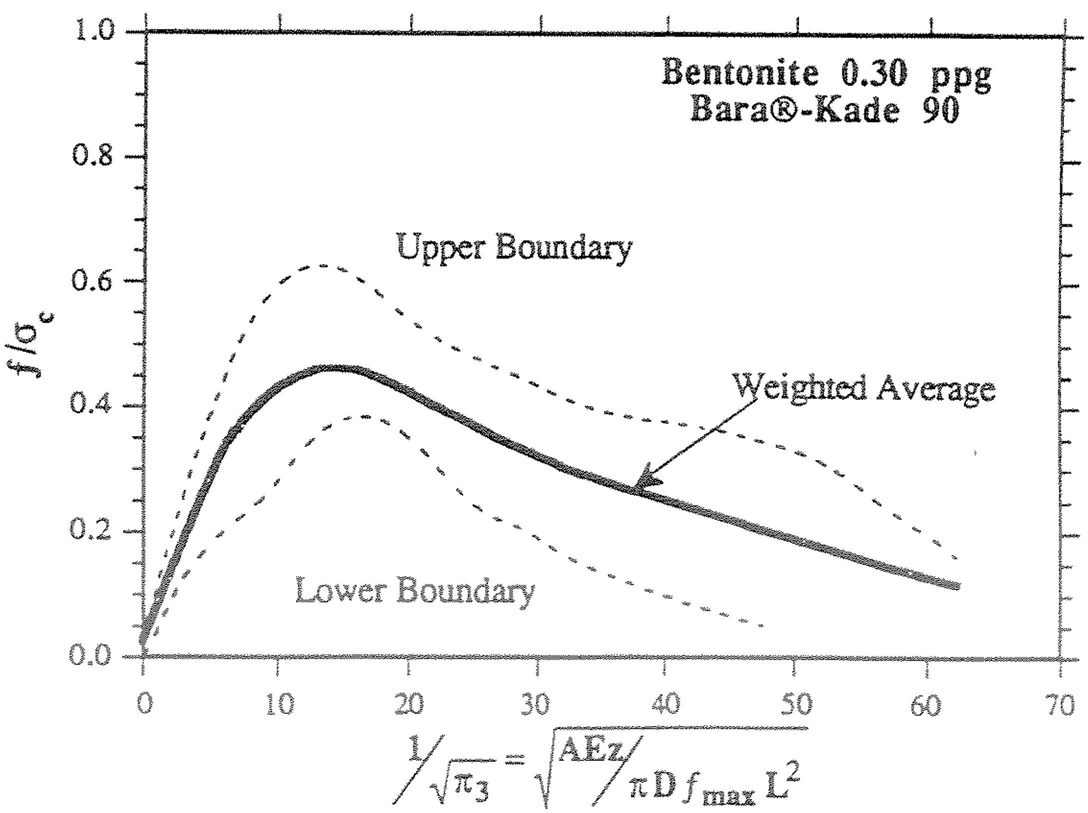

Near Field
0.24
u
σ'o
-0.40
0.0
Time (seconds)
30.000


Far Field
0.20
u
σ'o
0.0
0.0
Time (seconds)
30.000

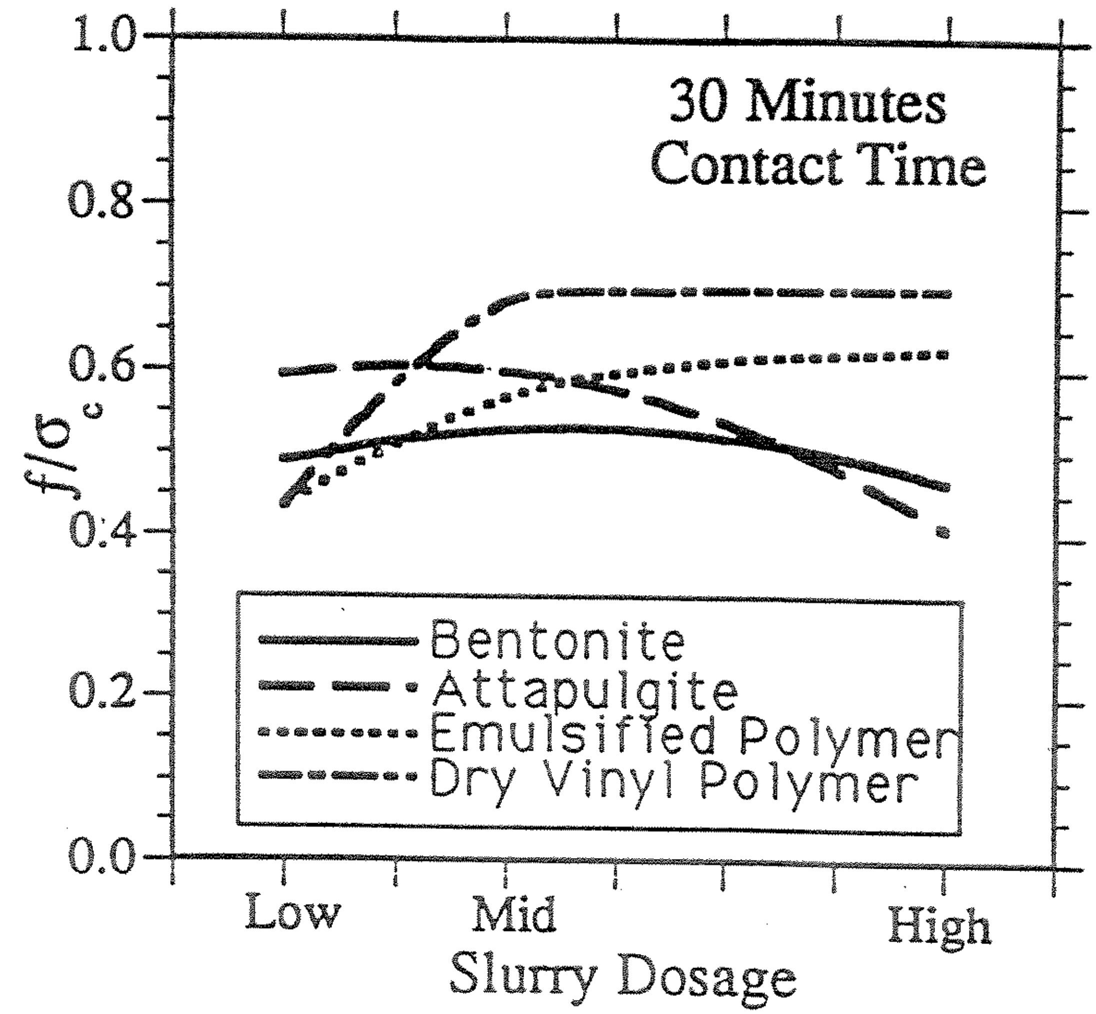
Effect of Installation Aids
(Drilling Slurry)



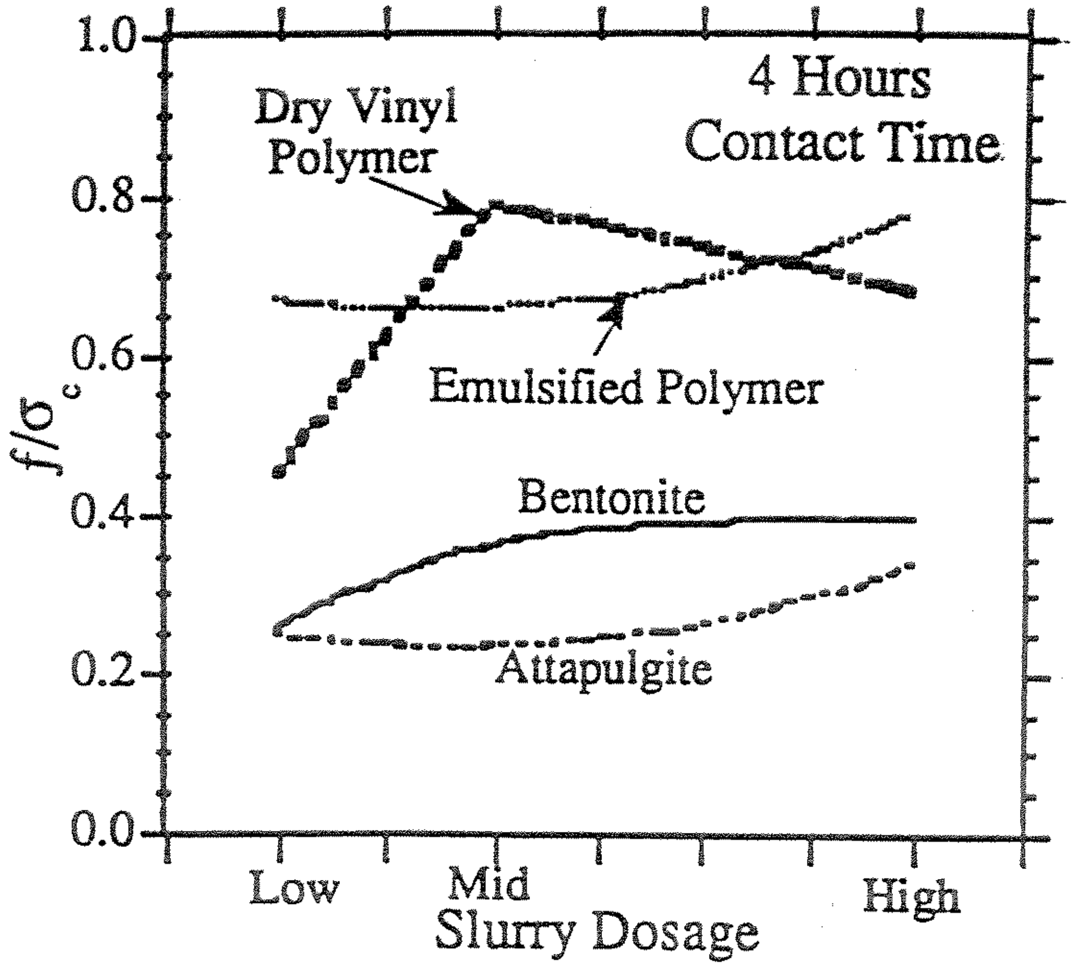
Casing Samples
Upper Boundary
Weighted Average
Lower Boundary
f/σ_c
1.2
1.0
0.8
0.6
0.4
0.2
0.0
0
10
20
30
40
50
60
70
80
$1/\sqrt{\pi_3} = \sqrt{AE\,z/\pi D f_{max} L^2}$



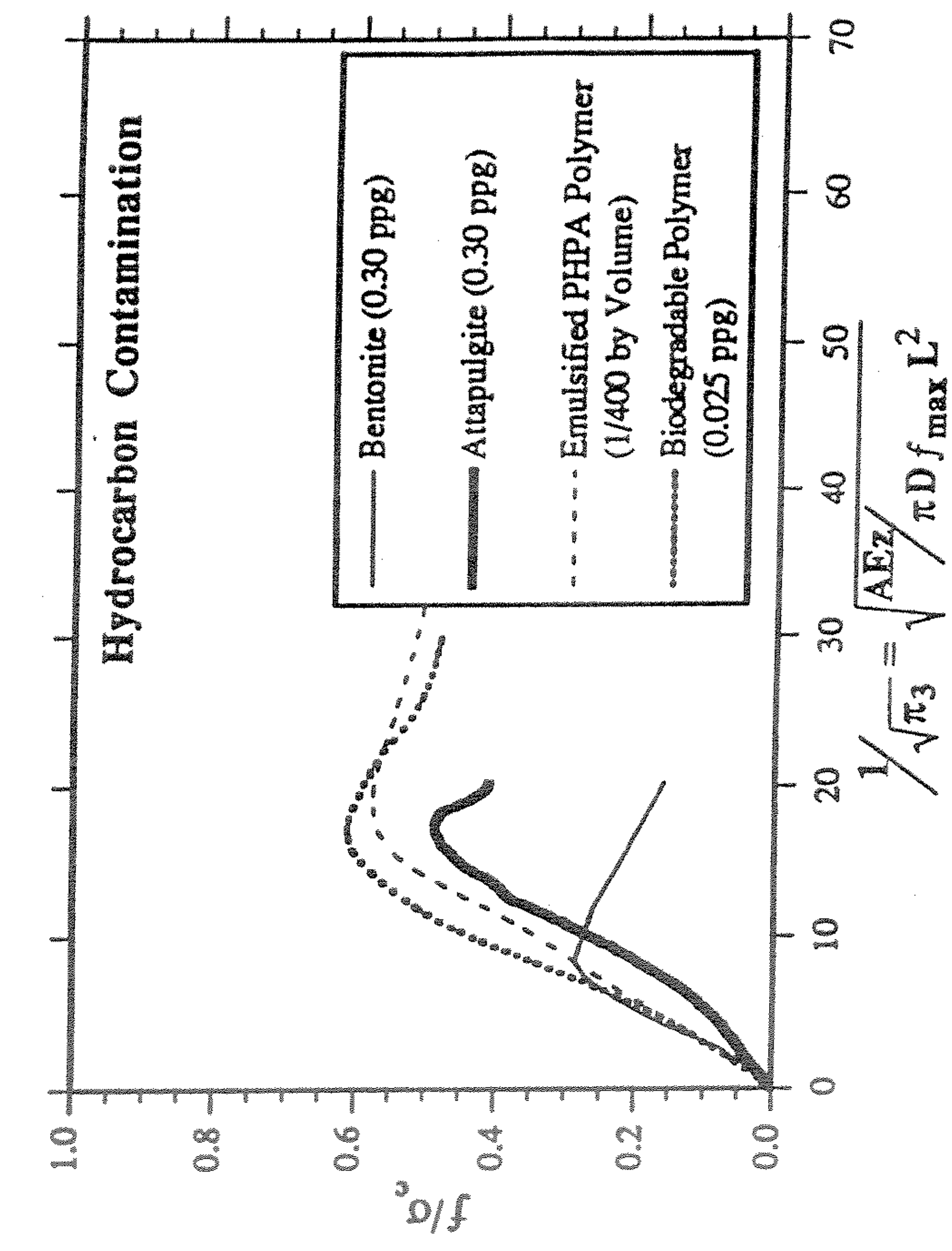
Bentonite 0.30 ppg
Bara®-Kade 90
Upper Boundary
Weighted Average
Lower Boundary
f/σ_c
1.0
0.8
0.6
0.4
0.2
0.0
0
10
20
30
40
50
60
70
$1/\sqrt{\pi_3} = \sqrt{AEz/\pi D f_{max} L^2}$



Comparison of mean normalized perimeter shear for 30 minutes contact time.



Comparison of mean normalized perimeter shear for 4 hours contact time.



Hydrocarbon Contamination
Bentonite (0.30 ppg)
Attapulgite (0.30 ppg)
Emulsified PHPA Polymer (1/400 by Volume)
Biodegradable Polymer (0.025 ppg)
f/σ_e
1.0
0.8
0.6
0.4
0.2
0.0
0
10
20
30
40
50
60
70
$1/\sqrt{\pi_3} = \sqrt{AEz/\pi D f_{max} L^2}$

Some Ideas for Research Needs -- Piles in Granular Soils

1. Prediction and Measurement of Radial, Vertical, Tangential and τ_{rz} Total Stresses Immediately Adjacent to the Pile-Soil Interface After Installation of Driven, Vibrated, Bored and Augered Piles in a Uniform Granular Soil Formation

• Taking account of soil compressibility, degradability, stress (or strain) path of soil (including shear stresses developed along pile wall and dynamic effects) during installation

2. Determination of the Effect of Soil Stratification on Single Pile Shaft and Toe Capacity

• Taking account of soil mixing during installation and of effects of stratifications on the alteration of load transfer patterns due to loading

3. Prediction of Group Efficiency

• Especially in bored and augered piles and taking account of cap-soil interaction and stress alterations due both to construction and to loading

4. **Predication of Pore Water Pressure Variation Immediately around Vibro-Driven Piles During Installation**

5. **Quantifying the Effect of Interface Roughness and Dilation in Bored and Augered Piles in Granular Soils**

• Ultimately, must be a probabilistic approach, perhaps following that at Monash University for bored piles in soft rock

6. **Quantifying Effect of Plugging of Both Pipe and H Piles**

• Particularly Accounting for Compressibility, Dilatancy and Interface Shear in Uniform and Nonuniform Soil Masses (per pioneering work at University of Western Australia)

7. **Evaluating or Developing New In Situ Testing Devices that will give Reliable Information on Pile Capacity**

• Describe the Controlling Parameters (Compressibility, Crushability, Interface Stresses)
• Provide Direct Analogous Design Data (e. g., Driven CPT, following up current work at Texas A and M)

and

Effects of Seismic Loading on Axial Capacity of Bored and Driven Piles

Effects of Slurries on Axial Capacity of Bored Piles

MIT RESEARCH ON PRÉDICTION AND INTERPRETATION OF DISPLACEMENT PILE PERFORMANCE

Andrew J. Whittle
Department of Civil & Environmental Engineering
Massachusetts Institute of Technology

- REVIEW OF PREVIOUS WORK
- CURRENT RESEARCH ACTIVITIES
- FUTURE DIRECTIONS

REVIEW OF PREVIOUS WORK

- ANALYSIS

  * Strain Path Method - Pile Installation Model
  * MIT-E3 - Effective Stress Soil Model

- EXPERIMENTS:

  * Instrumented Pile Shaft (PLS Cell)
  * two soft clay sites

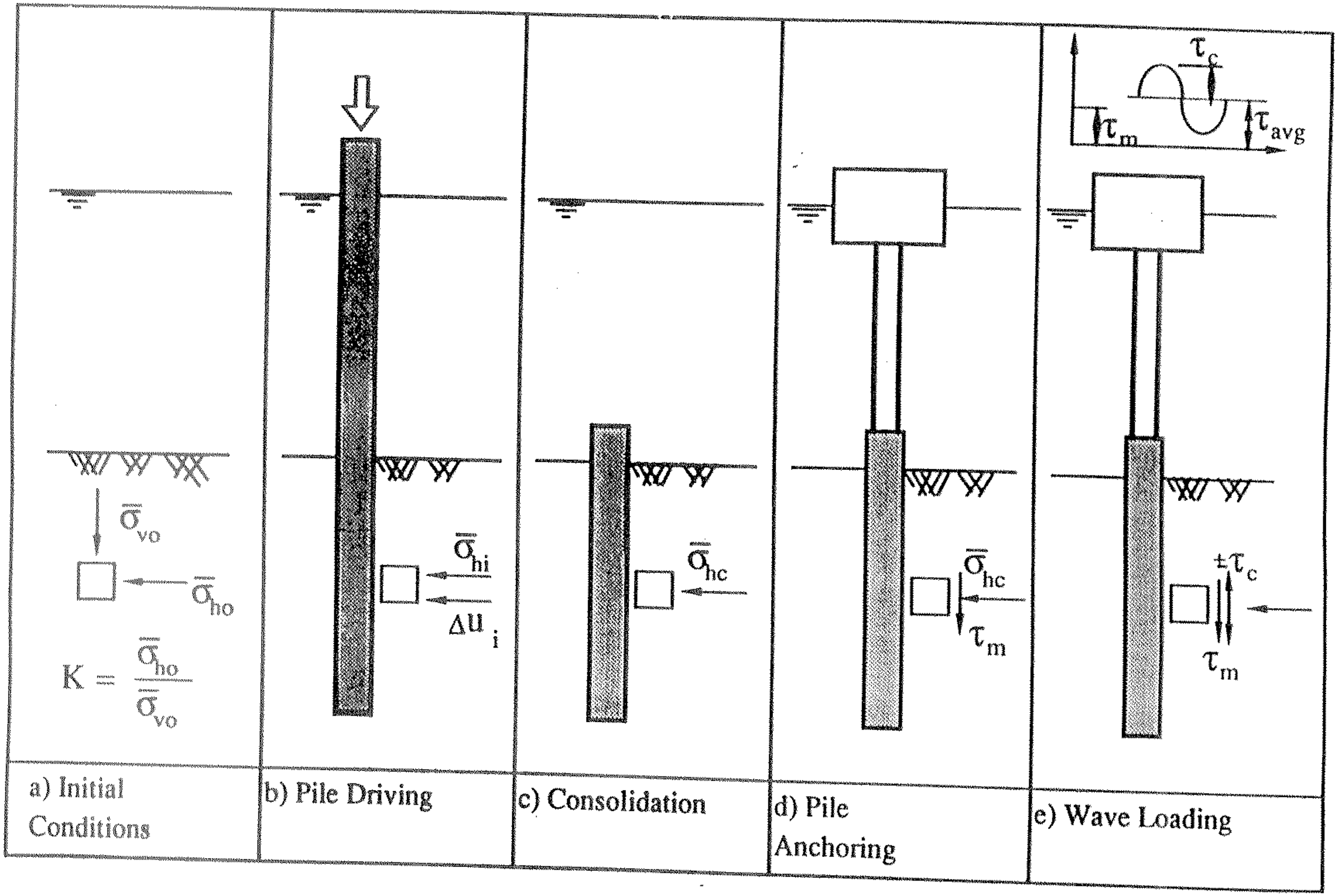
- PREDICTIVE CAPABILITIES & LIMITATIONS

  * installation/consolidation/axial load

RESEARCH RELATED TO DISPLACEMENT PILES AT MIT
- chronology since late 1970's

	DEVELOPMENTS		APPLICATIONS
THEORETICAL	LABORATORY	FIELD	
Strain Path Method	Anisotropic Properties of Clays (Clay Resedimentation)	Piezocone	In-situ Measurement of Clay Properties
Non-linear consolidation		PLS Cell (model pile shaft)	Friction Piles in Clay (open-ended piles)
	Clay Response under cyclic loading		
MIT-E3 Soil Model		Instrumented Model Piles*	TLP Piles
	Rate Effects in Clays (Flexible Automation Technology)		
General Non-Linear FE Analysis (SPM+MIT-E3)	Instrumented Model Caissons	Interpretation of Centrifuge & Field Tests*	SUCTION CAISSONS

* In cooperation with NGI, Imperial College



Phases in the Life of a TLP Pile

THE STRAIN PATH METHOD

(Baligh: 1985- ASCE JGED; 1986a,b - Géotechnique)

- **Model Mechanics of Deep Penetration**
  Alternatives: 1-D Cavity Expansion
  Large Strain Finite Element

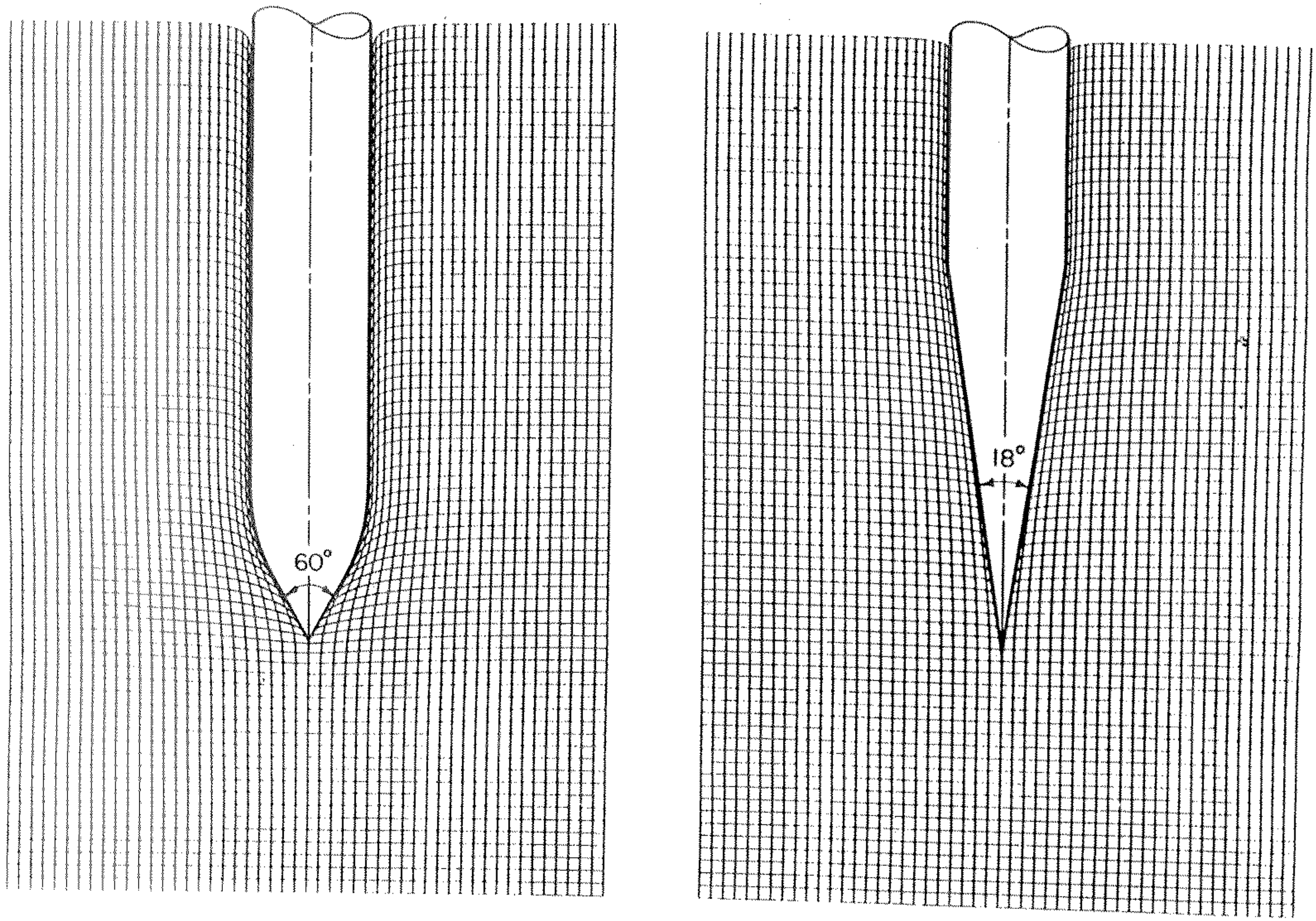
- **Assumption:**
  - Undrained Deep Penetration in Clays
  - Kinematically controlled
  - Estimate Deformations & Strains Independent of Soil Properties
  - Soil Modelling based on known Strain Histories

- **Limitation:**
  - Approximation of Equilibrium (Excess Pore Pressures)

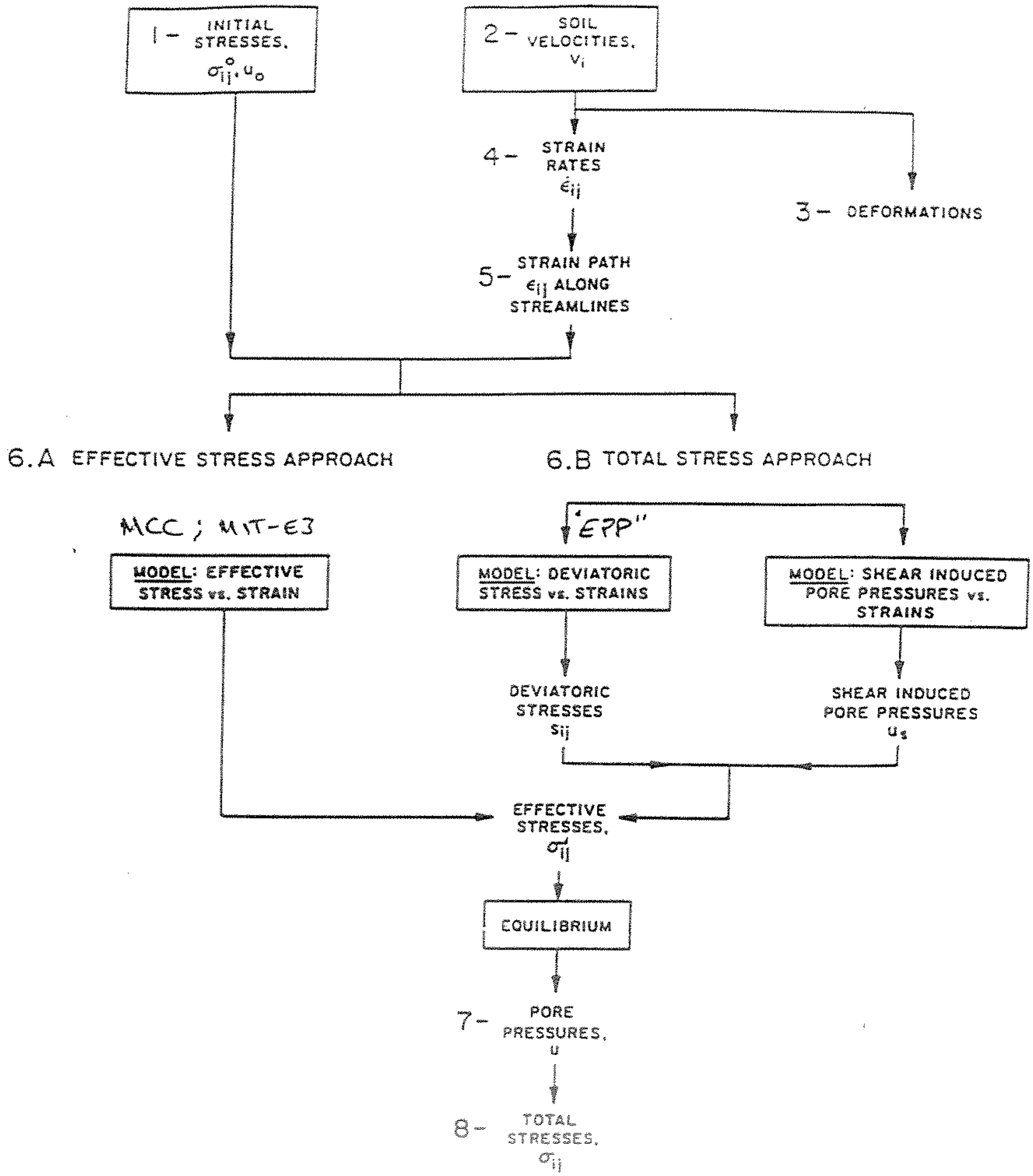
- **Recent Developments**
  - Robust Numerical Solution of Pore Pressure Distribution
  - Link to Finite Element Code ABAQUS

- **Requirements for Suction Caisson Project**
  - Complete Solutions INSIDE and Outside Caisson
  - Effect of Mudline/Free Surface

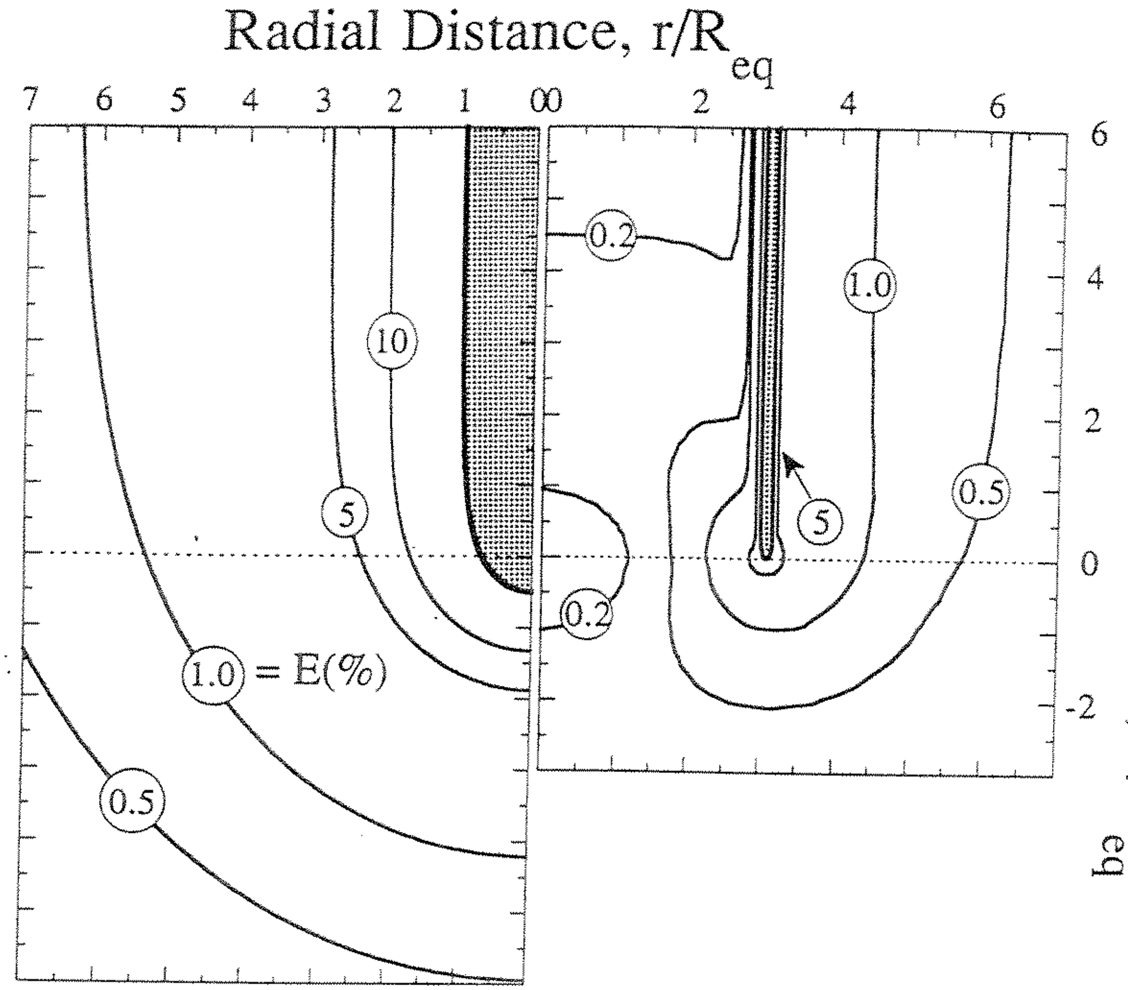
- STANDARD CONE TIP



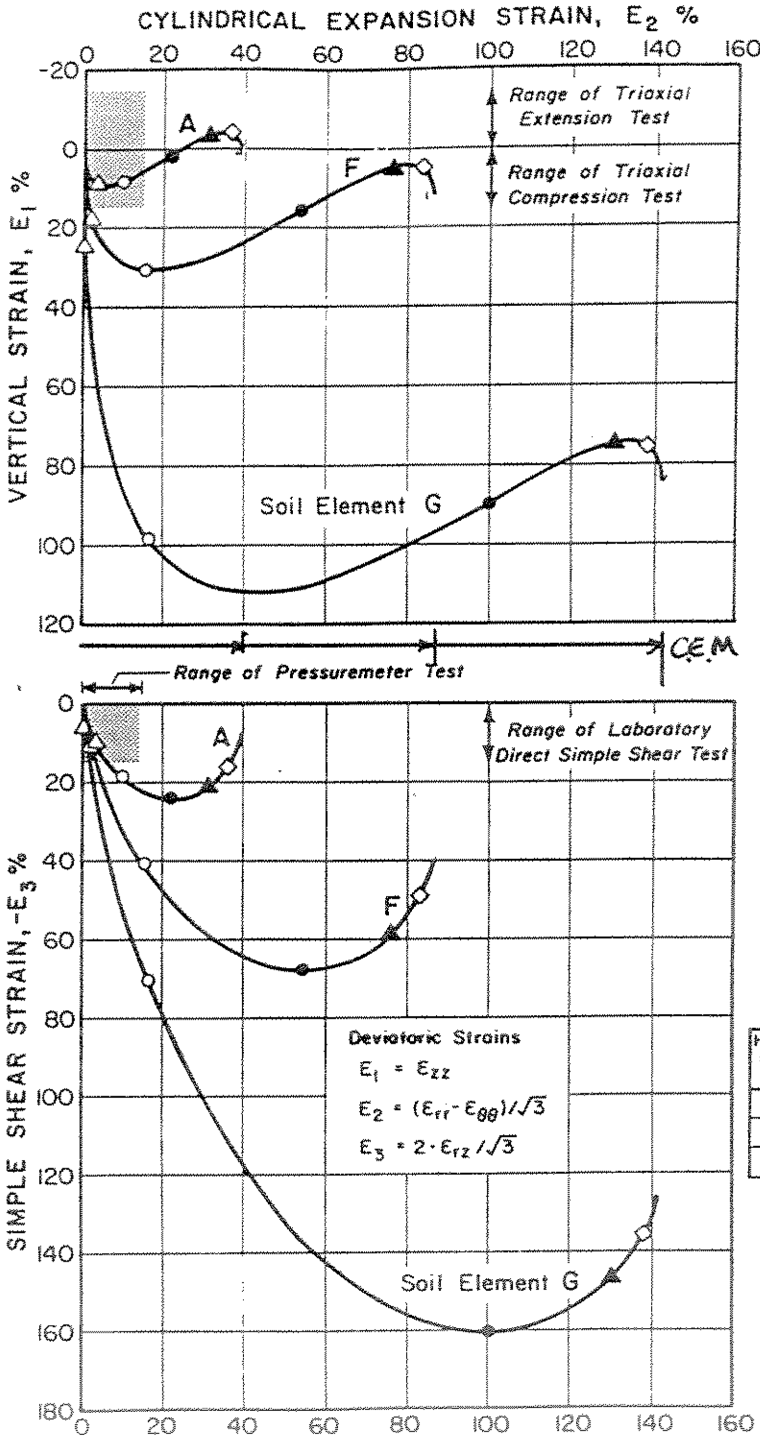
- DEFORMATION FIELDS FOR AXISYMMETRIC CONE PENETROMETERS (BALIGH + LEVADOUX, 1980)

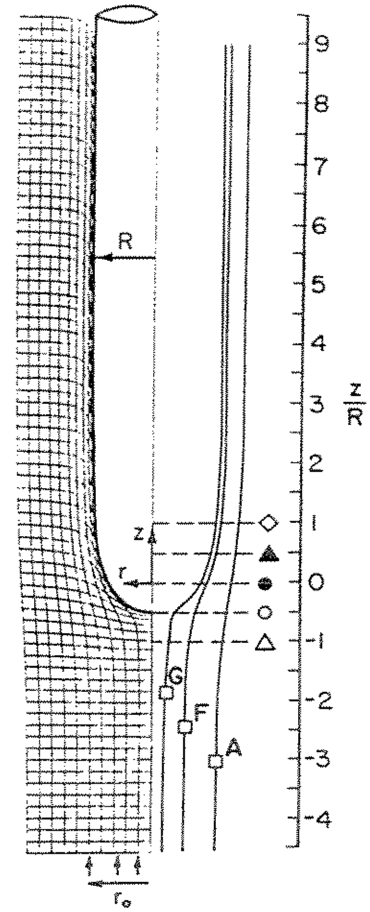


The Strain Path Method (Baligh, 1985)



Radial Distance, r/R eq
7 6 5 4 3 2 1 0 0 2 4 6
Vertical Distance, z/R eq
6 4 2 0 -2
0.2
1.0
10
0.5
5
5
0.2
1.0 = E(%)
0.5





Horizontal Location r_o/R	Soil Element
1.0	A
0.5	F
0.2	G

Vertical Location z/R	Symbol
1	◇
1/2	▲
0	●
-1/2 (tip)	○
-1	△

• STRAIN PATHS FOR INDIVIDUAL SOIL ELEMENTS

E_2 = CAVITY EXPANSION MODE

THE MIT-E3 MODEL

(Whittle: PhD 1987, MITSG90-15, ~~1992~~ 1993 Géotechnique)

• MODEL FORMULATION

- Simple Conceptual Model: unload-reload
- Three Components:
- • Elasto-Plastic Model for Normally Consolidated Clay
- • Equations for Perfect Hysteresis
- • Bounding Surface Plasticity

• INPUT PARAMETERS

- 15 Input Parameters
- Obtain from Standard Types of Laboratory Test

• EVALUATION AT THE ELEMENT LEVEL

- Standard Shear Modes; $1 \leq OCR \leq 8$; Several Types of Clays:
  Triaxial (C/E), Plane Strain (A/P), Direct Simple Shear
- Unique Data:
  - Directional Shear Cell
  - Multi-Directional Simple Shear Apparatus
- Cyclic Data: Direct Simple Shear

• LIMITATIONS

- Rate Independent Clay Behavior
- Normalized Clay Behavior
- Less Reliable at $OCR \geq 8$

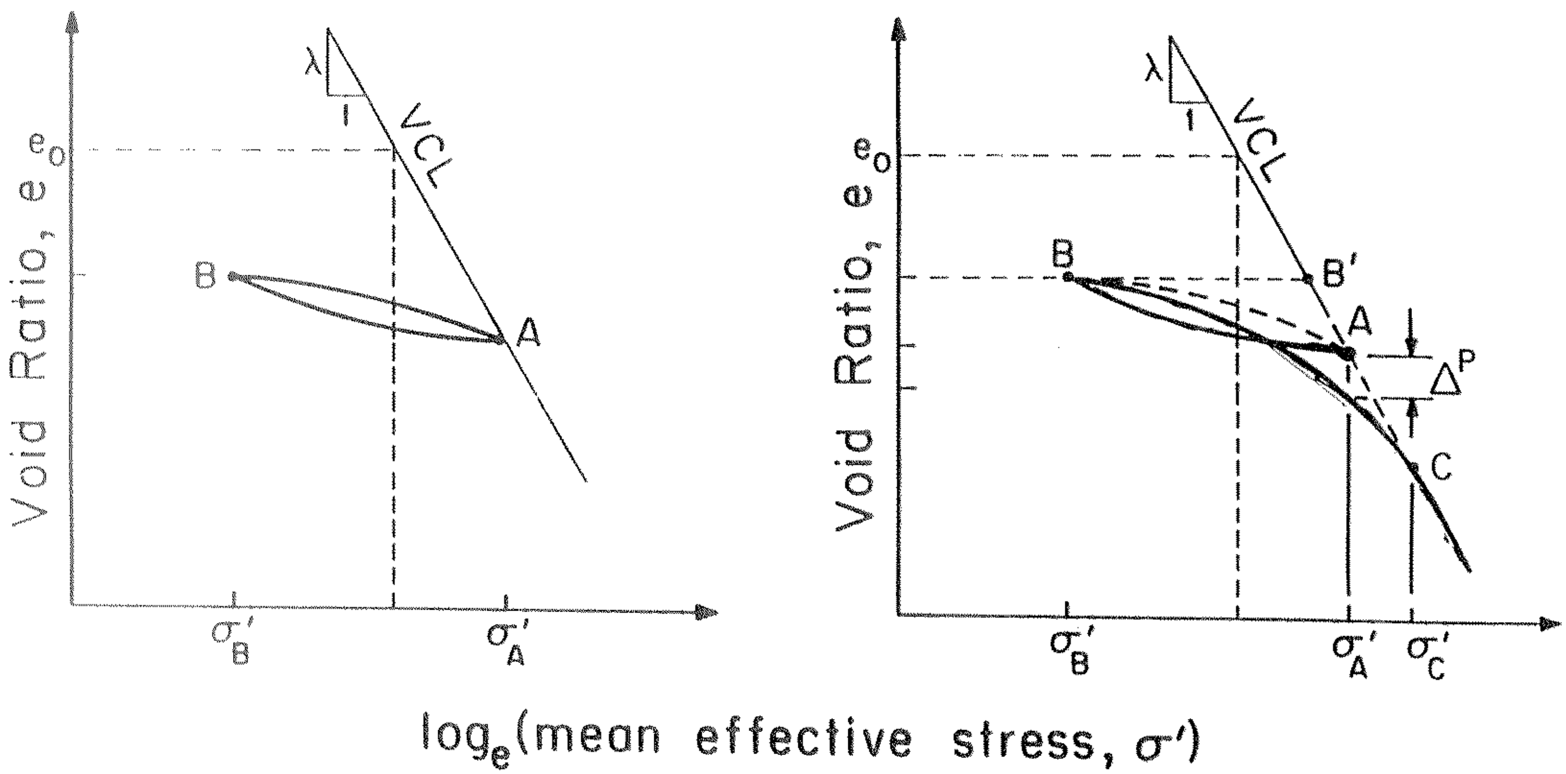
• EVALUATION: DISPLACEMENT PILES

- • Pile Installation (via SPM)
- • Non-Linear Radial Consolidation (Set-Up)
- • Axial Loading (Including Cyclic Loading)
- PLS Data 2 Sites (BBC, Empire)
- Instrumented Model Piles (Haga, Onsøy, Bothkennar)

• INCORPORATION IN ABAQUS CODE

- UMAT interface
- Application/Interpretation in Boundary Value Problem
  (Braced Excavations)

• CONCEPTUAL BASIS FOR MIT-E3

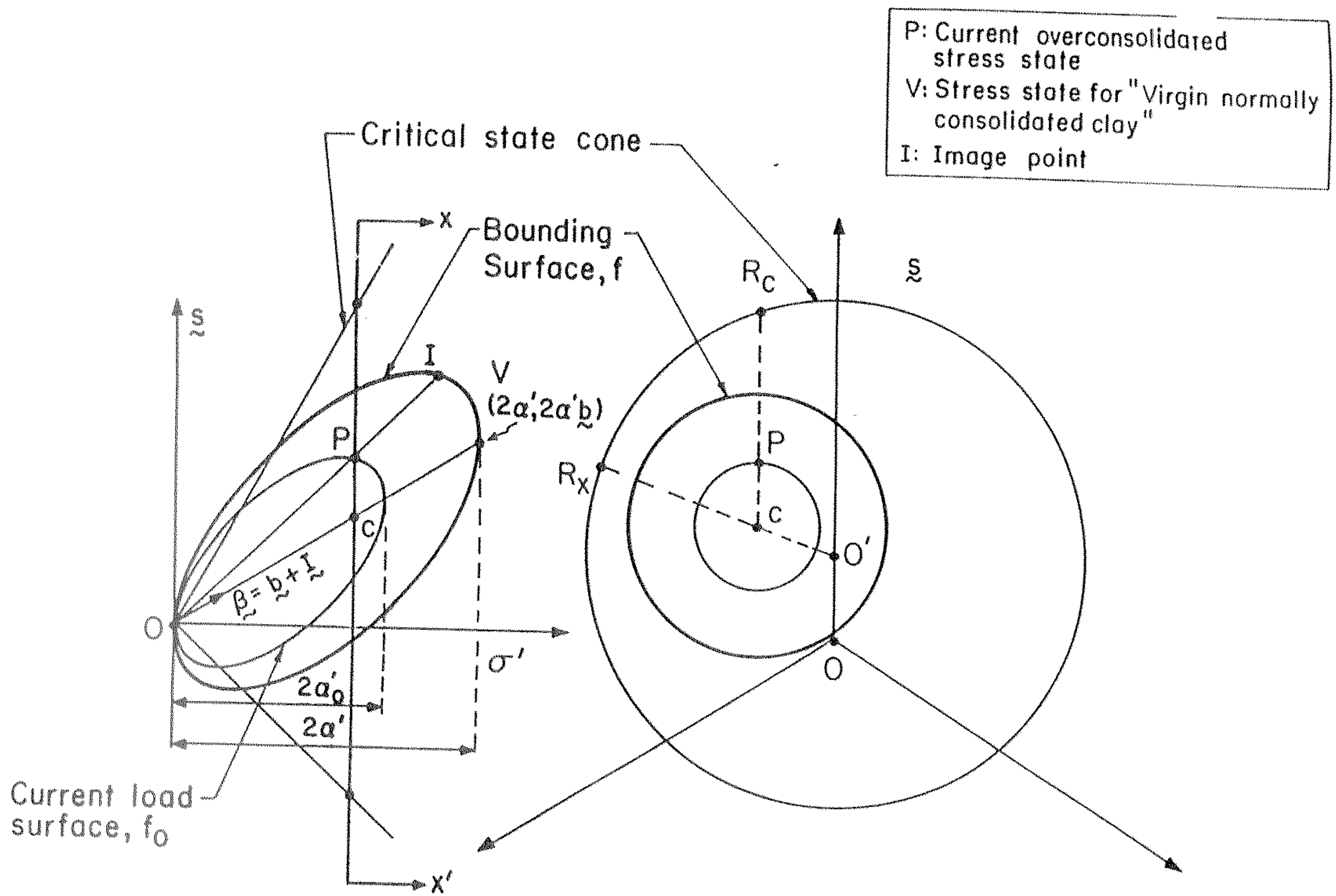


a) 'Perfect Hysteresis'

b) Hysteresis and Bounding Surface Plasticity

Figure 1. Conceptual Model of Unload-Reload used by MIT-E3 for Hydrostatic Compression

• CONCEPTUAL MODEL - VOLUMETRIC BEHAVIOUR



Yield and Failure Surfaces for Normally Consolidated Clay

→ ANISOTROPY
STRESS - STRAIN - STRENG
→ STRAIN SOFTENING

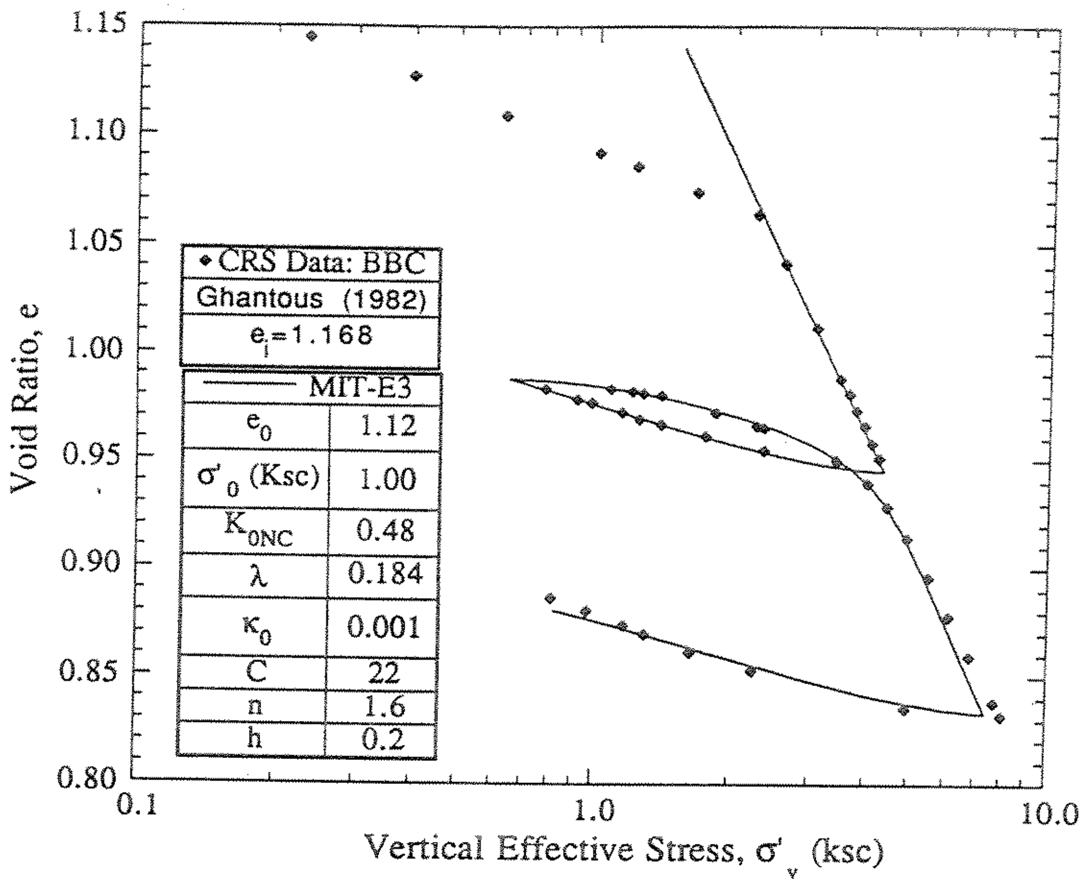
Test Type	Parameter/ Symbol	Physical contribution/ meaning	Boston Blue Clay	Empire Clay
Oedometer or CRS	e_0	Void ratio at reference stress on virgin consolidation line	1.12	1.26
	λ	Compressibility of virgin normally consolidated clay	0.184	0.274
	C	Non-linear volumetric swelling behaviour	22.0	24.0
	n		1.6	1.75
	h	Irrecoverable plastic strain	0.2	0.2
K_0-oedometer or K_0-triaxial	K_{0nc}	K_0 for virgin normally consolidated clay	0.48	0.62
	2G/K	Ratio of elastic shear to bulk modulus (Poisson's ratio for intial unload)	1.05	0.86
Undrained Triaxial Shear Tests: OCR=1; CK_0UC OCR=1; CK_0UE OCR=2; CK_0UC	ϕ'_{TC}	Critical state friction angles in triaxial compression and	33.4^0	23.6^0
	ϕ'_{TE}	extension (large strain failure criterion)	45.9^0*	21.6^0
	c	Undrained shear strength (geometry of bounding surface)	0.86	0.75
	s_t	Amount of post-peak strain softening in undrained triaxial compression	4.5	3.0
	ω	Non-linearity at small strains in undrained shear	0.07	0.2
	γ	Shear induced pore pressure for OC clay	0.5	0.5
Resonant Column	κ_0	Small strain compressibility at load reversal**	0.001	0.0035
Drained Triaxial	ψ_0	Rate of evolution of anisotropy (rotation of bounding surface)	100.0	100.0

Table 4.2 Input Material Properties used by the MIT-E3 Model

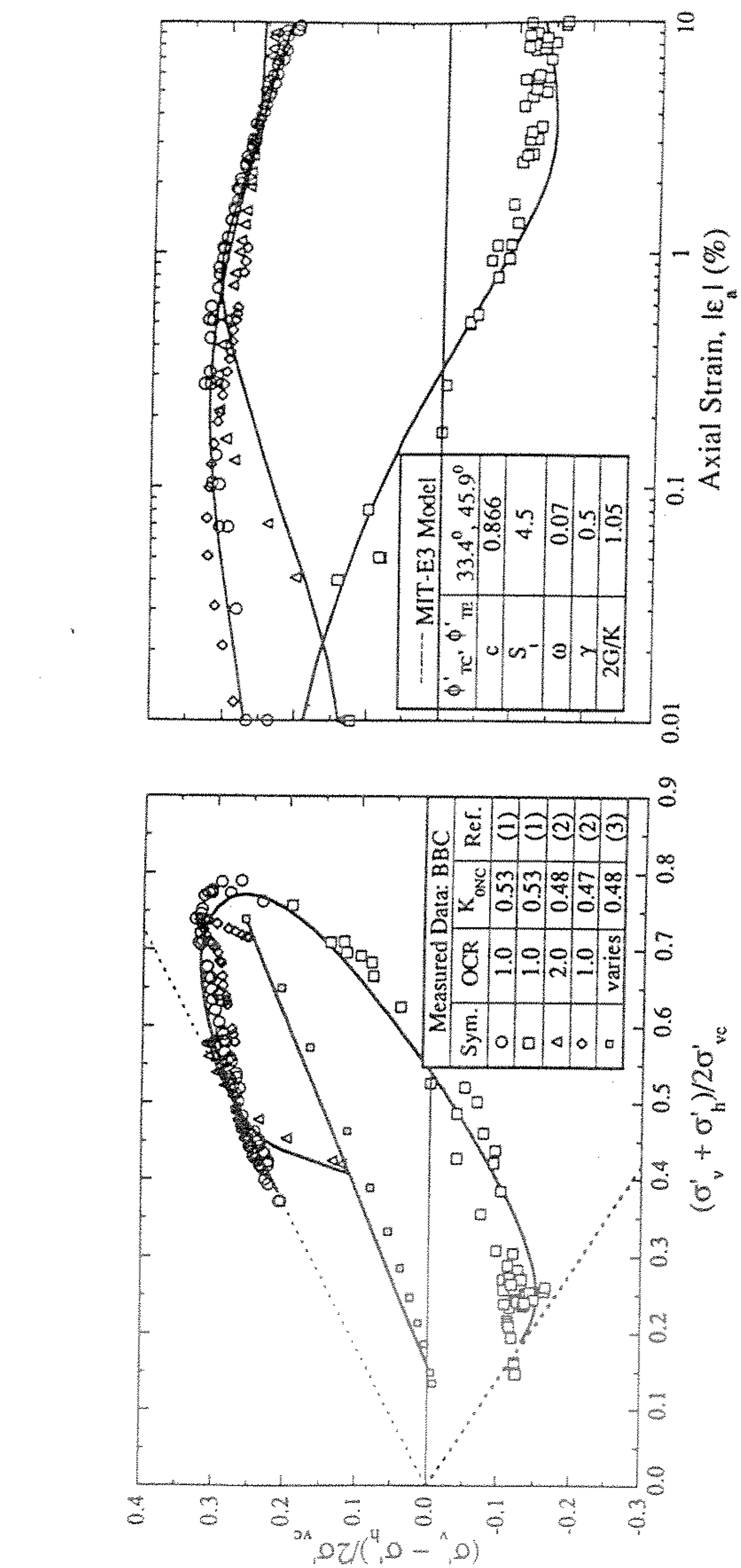
* Value is based on data from Ladd & Varallyay (1965). More recent data suggest ϕ'_{TE}=40^0

** Alternatively use field shear wave velocity test data

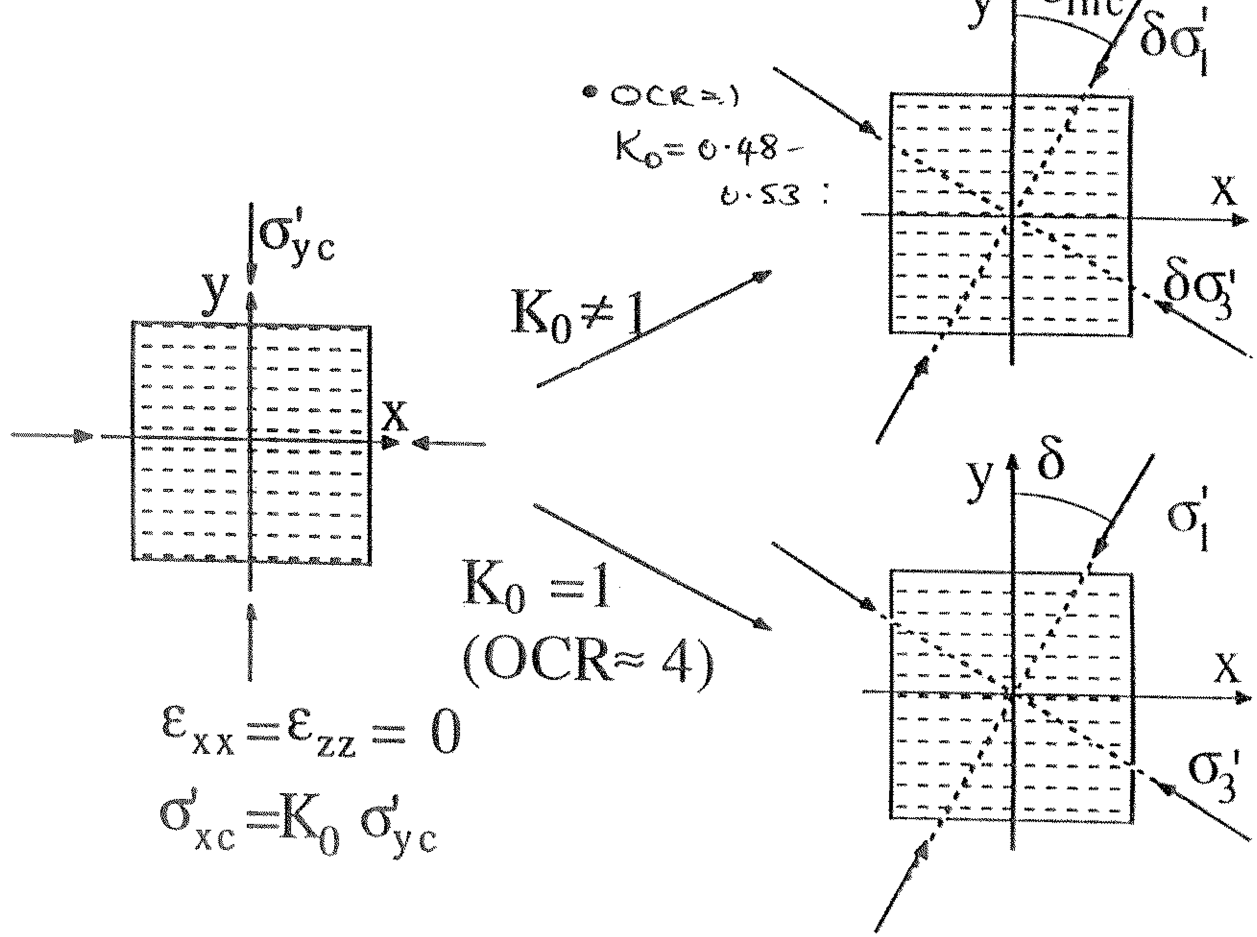
° MODEL PARAMETERS FROM



1.15
1.10
1.05
1.00
0.95
0.90
0.85
0.80
0.1
1.0
10.0
Void Ratio, e
Vertical Effective Stress, σ'v (ksc)
CRS Data: BBC
Ghantous (1982)
ei=1.168
MIT-E3
e0 1.12
σ'0 (Ksc) 1.00
K0NC 0.48
λ 0.184
κ0 0.001
C 22
n 1.6
h 0.2


(σ'_v − σ'_h)/2σ'_vc
(σ'_v + σ'_h)/2σ'_vc
Measured Data: BBC
Sym. | OCR | K_ONC | Ref.
○ | 1.0 | 0.53 | (1)
□ | 1.0 | 0.53 | (1)
△ | 2.0 | 0.48 | (2)
◇ | 1.0 | 0.47 | (2)
▫ | varies | 0.48 | (3)
Axial Strain, |ε_a| (%)
MIT-E3 Model
φ'_TC, φ'_TE | 33.4°, 45.9°
c | 0.866
S_t | 4.5
ω | 0.07
γ | 0.5
2G/K | 1.05

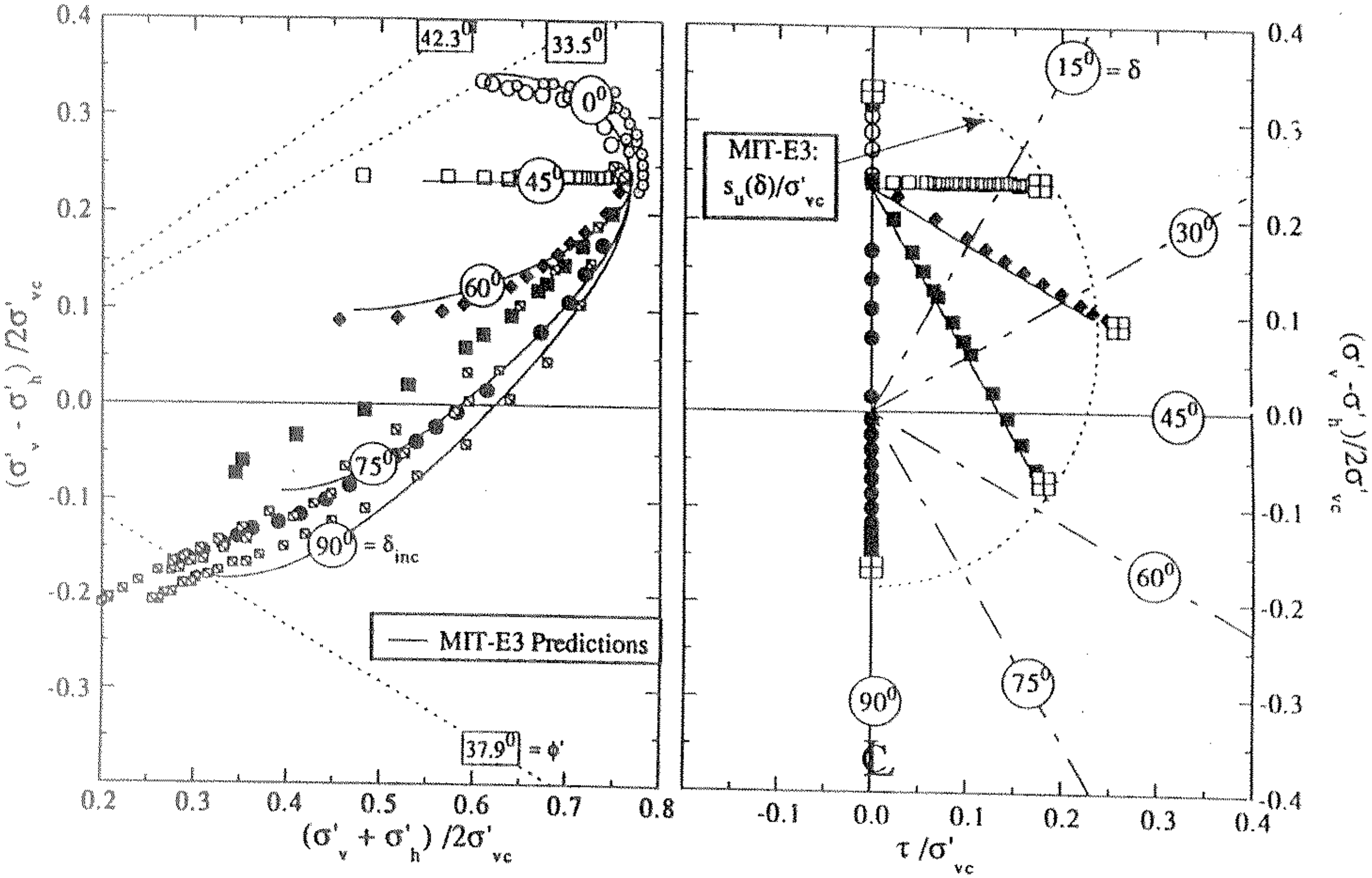
* UNDRAINED PLANE STRAIN SHEAR BEHAVIOUR



a) K_0-Consolidation

b) Undrained Shearing

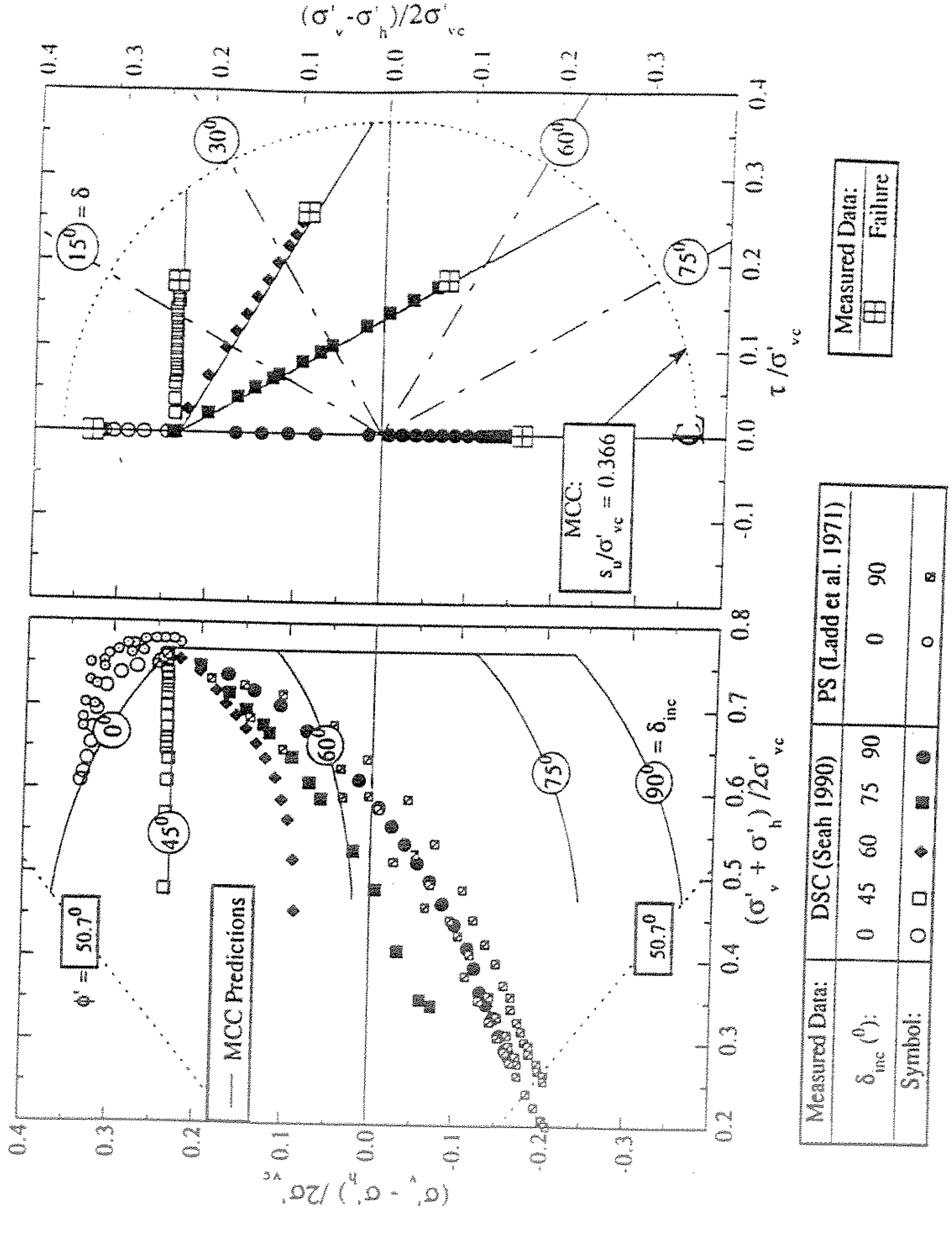
EFFECT OF PRINCIPAL STRESS ROTATIONS: DIRECTIONAL SHEAR CELL (DSC) TESTS



Measured Data:	DSC (Seah 1990)	PS (Ladd et al. 1971)
δ_{inc} (°):	0 45 60 75 90	0 90
Symbol:	○ □ ◆ ■ ●	○ ⊠

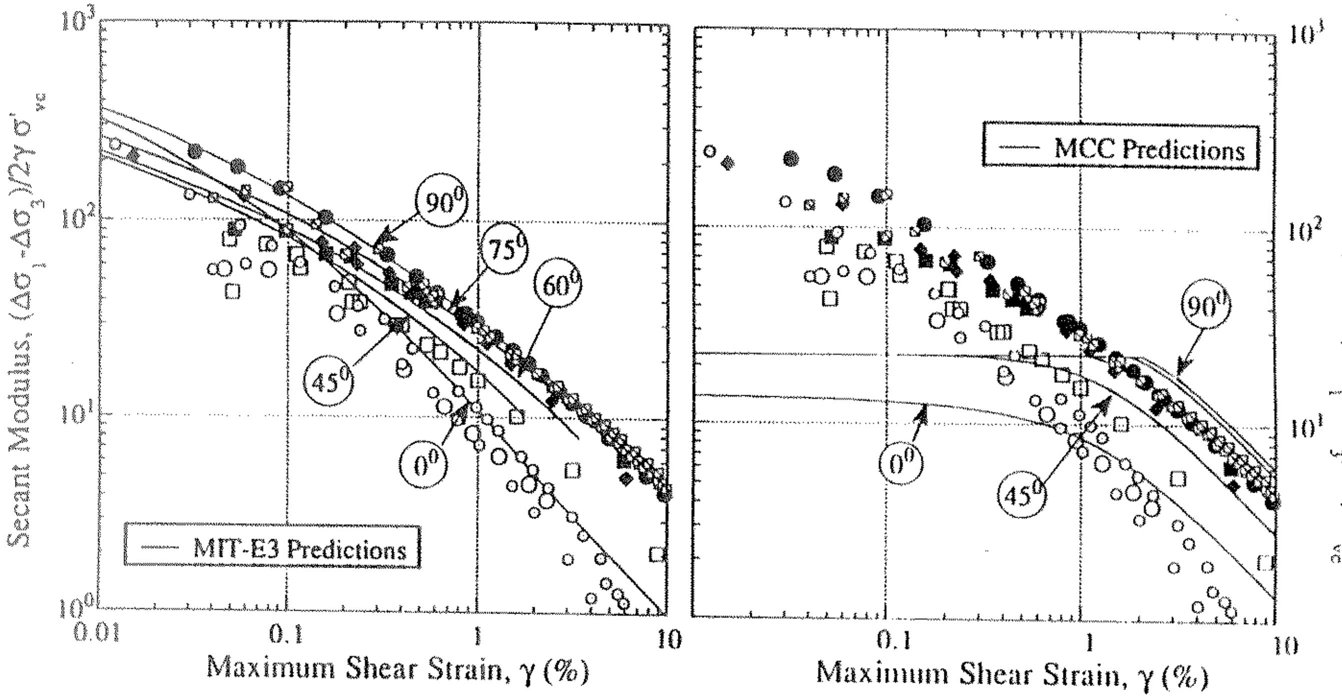
Measured Data:	
⊞	Failure

Figure 6a. Evaluation of Stress Paths Predicted by the MIT-E3 Model for Undrained Plane Strain Shear of K_0-Normally Consolidated BBC in the Directional Shear Cell



MCC Predictions
Measured Data:
δ_{inc} (°):
Symbol:
DSC (Seah 1990)
0 45 60 75 90
PS (Ladd et al. 1971)
0 90
$(\sigma'_v + \sigma'_h)/2\sigma'_{vc}$
$(\sigma'_v - \sigma'_h)/2\sigma'_{vc}$
$\phi' = 50.7^0$
50.7^0
$90^0 = \delta_{inc}$
MCC:
$s_u/\sigma'_{vc} = 0.366$
τ/σ'_{vc}
$15^0 = \delta$
Measured Data:
Failure

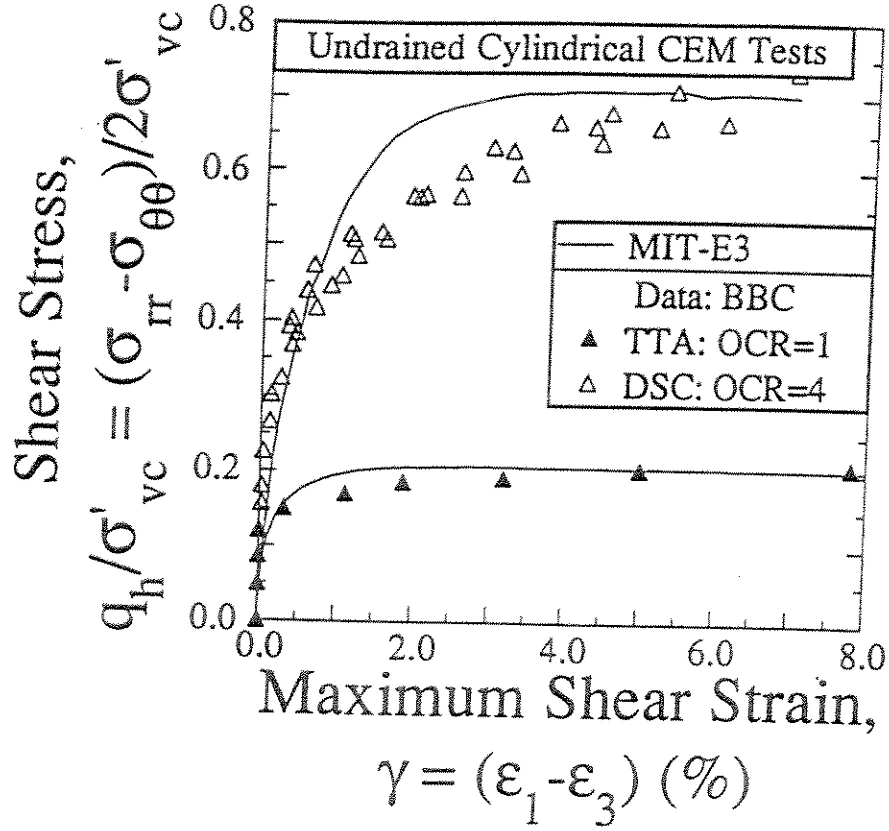
* MIT-E3

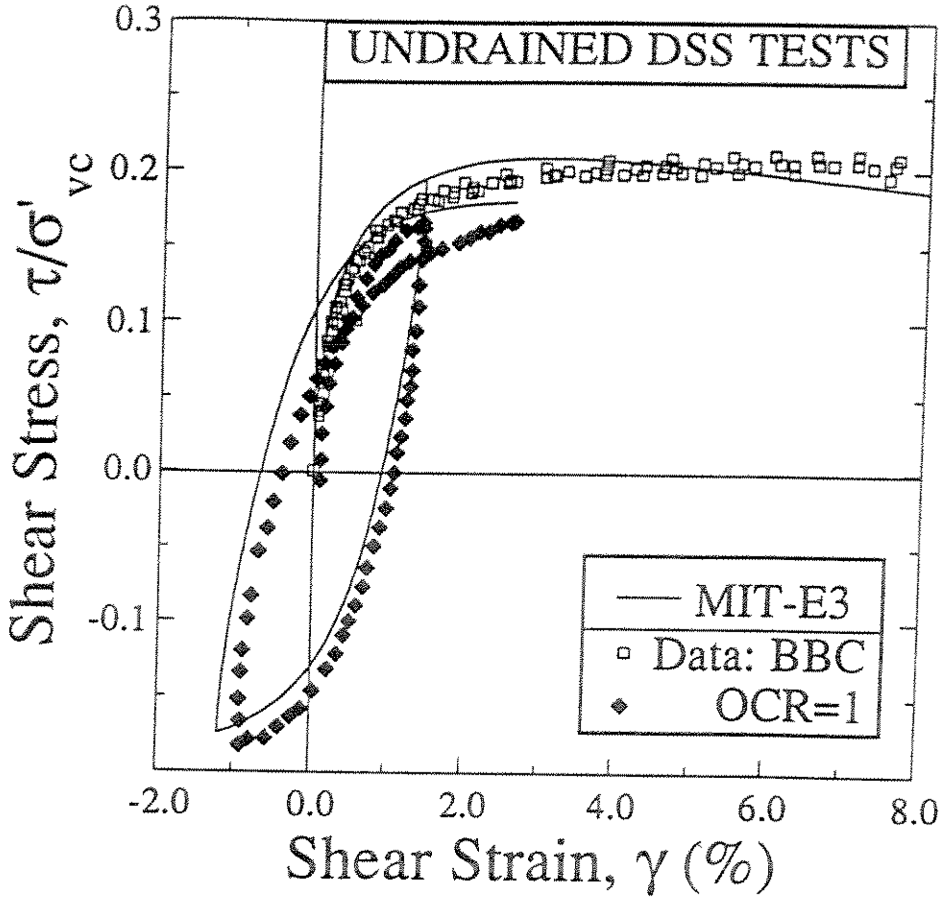


Measured Data:	DSC (Seah, 1990)	PS (Ladd et al., 1971)
δ_{inc} (^{0}):	0 45 60 75 90	0 90
Symbol:	○ □ ◆ ■ ●	○ ⊠

SMALL STRAIN STIFFNESS OF BBC AT OCR=1

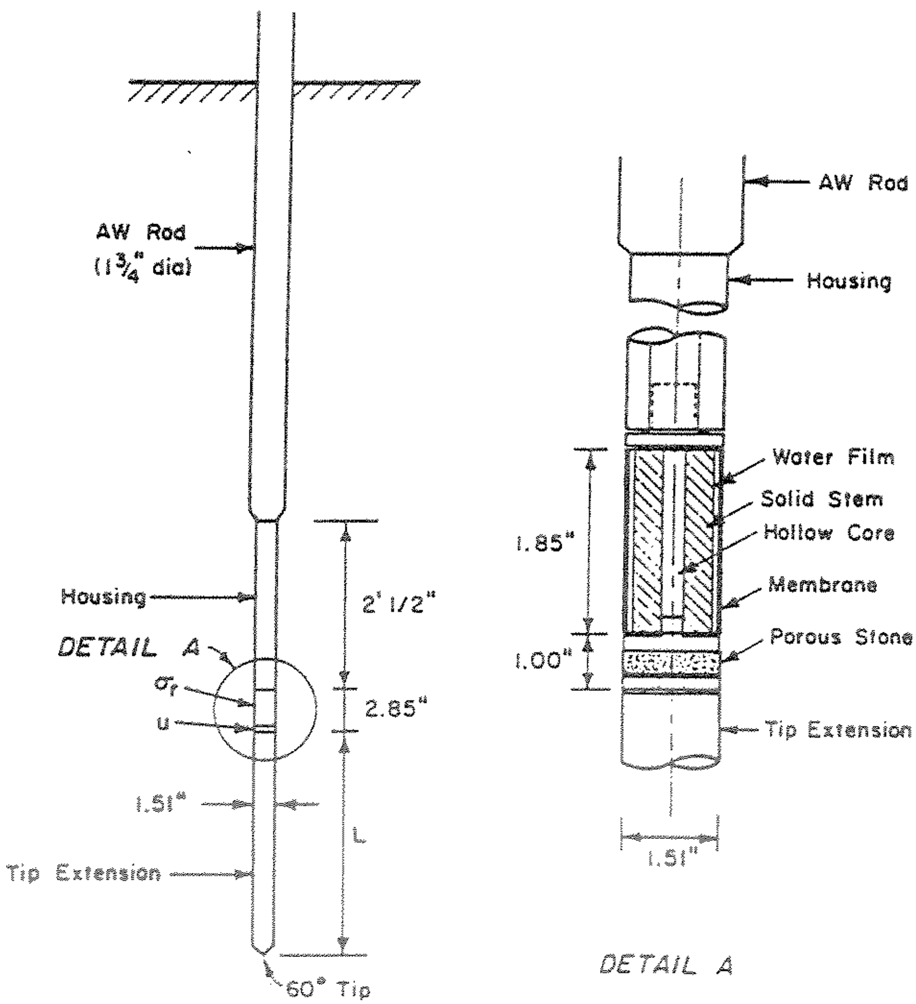
SECANT SHEAR MODULUS

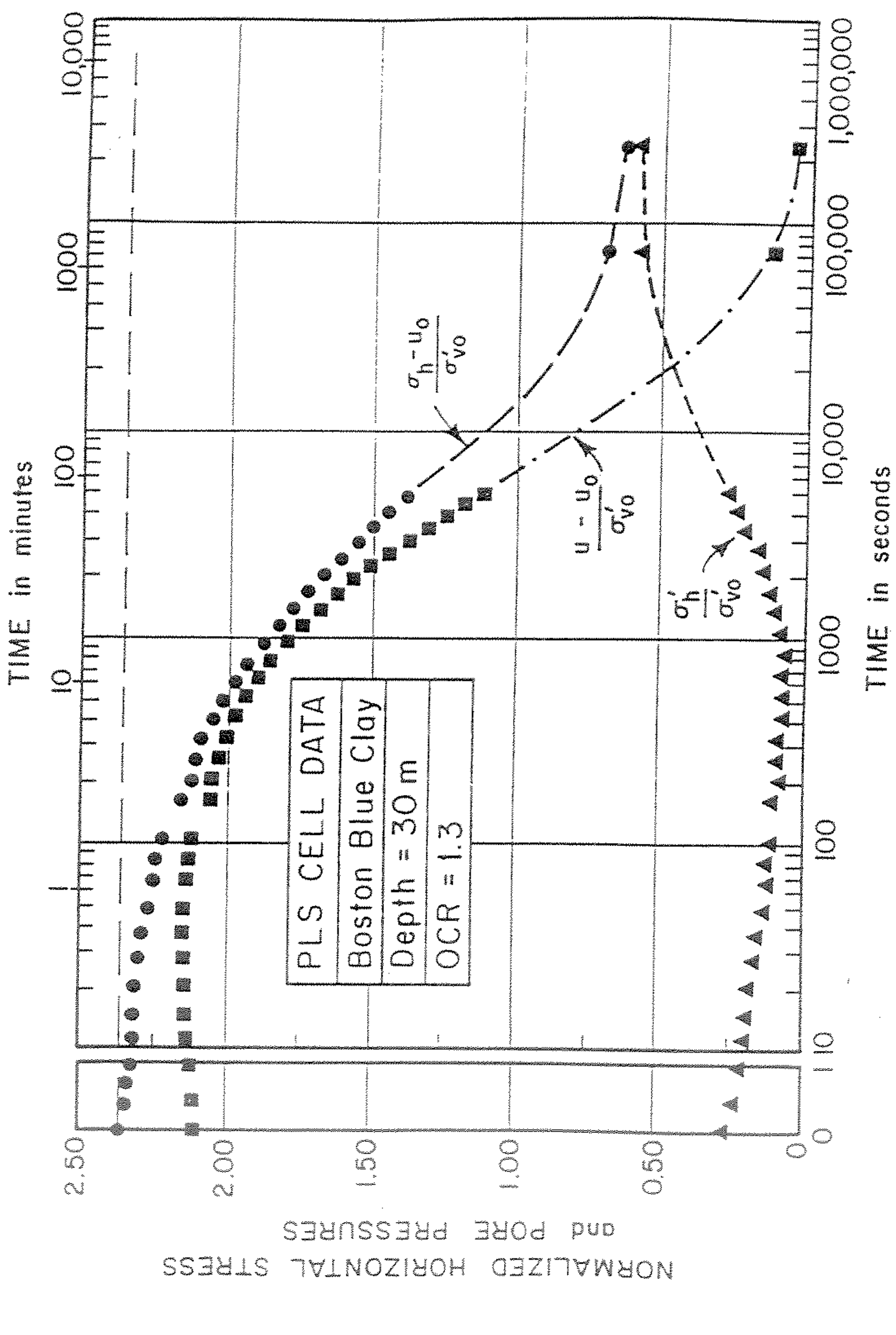

Undrained Cylindrical CEM Tests
MIT-E3
Data: BBC
TTA: OCR=1
DSC: OCR=4
Shear Stress, $q_h/\sigma'_{vc} = (\sigma_{rr}-\sigma_{\theta\theta})/2\sigma'_{vc}$
0.0
0.2
0.4
0.6
0.8
0.0
2.0
4.0
6.0
8.0
Maximum Shear Strain, $\gamma = (\varepsilon_1-\varepsilon_3)$ (%)


UNDRAINED DSS TESTS
Shear Stress, τ/σ'_{vc}
Shear Strain, γ (%)
MIT-E3
Data: BBC
OCR=1
0.3
0.2
0.1
0.0
-0.1
-2.0
0.0
2.0
4.0
6.0
8.0

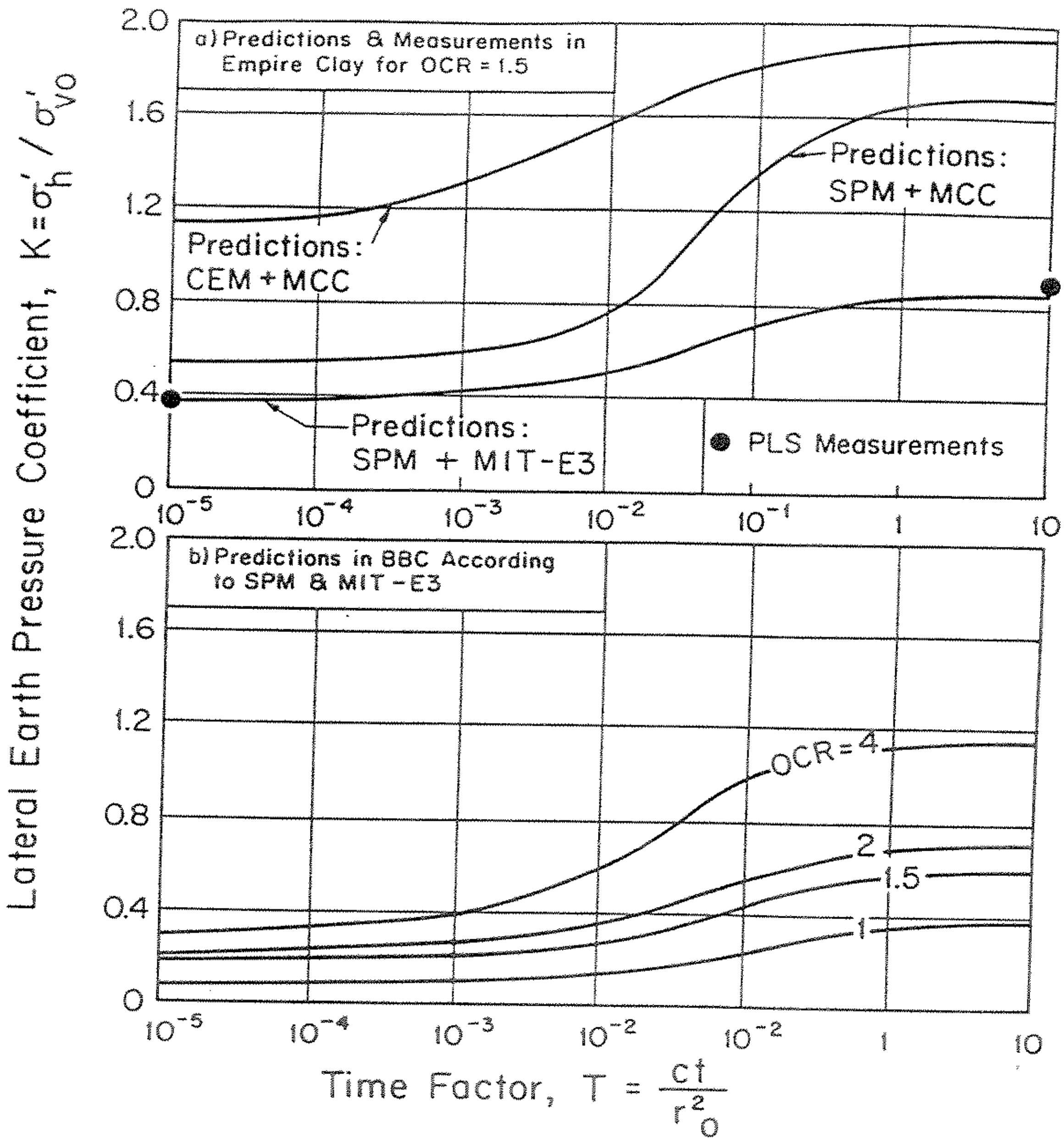
Field Data: The PLS Cell

- Developed at MIT
- Data available at 2 sites

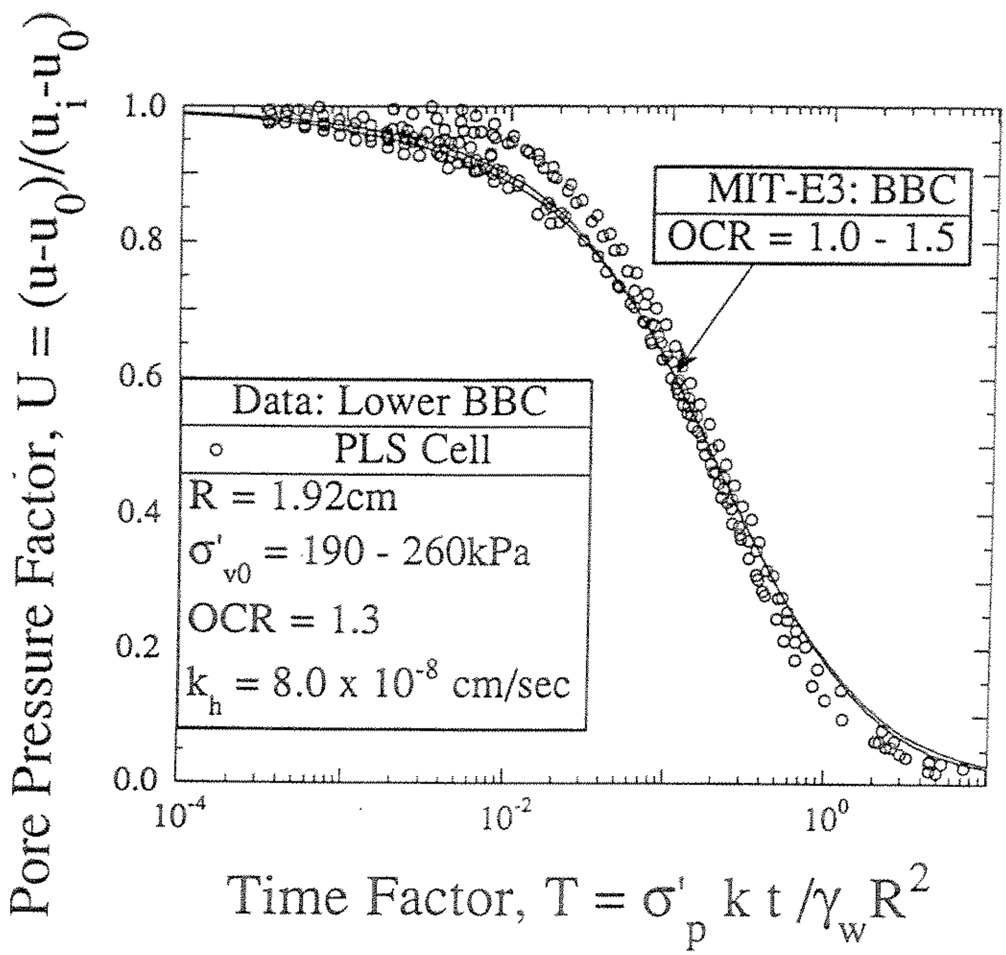




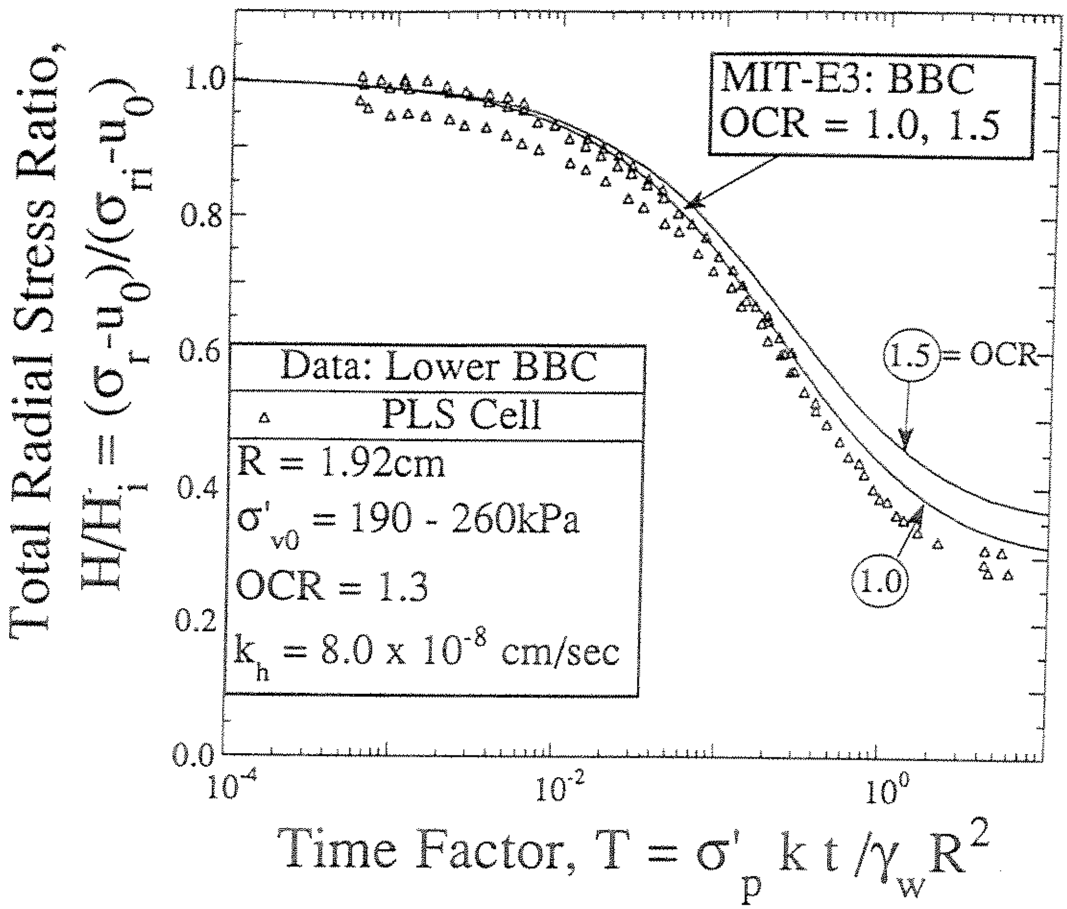
TIME in minutes
1
10
100
1000
10,000
TIME in seconds
0
10
100
1000
10,000
100,000
1,000,000
NORMALIZED HORIZONTAL STRESS and PORE PRESSURES
2.50
2.00
1.50
1.00
0.50
0
PLS CELL DATA
Boston Blue Clay
Depth = 30 m
OCR = 1.3
$\frac{\sigma_h - u_o}{\sigma'_{vo}}$
$\frac{u - u_o}{\sigma'_{vo}}$
$\frac{\sigma'_h}{\sigma'_{vo}}$

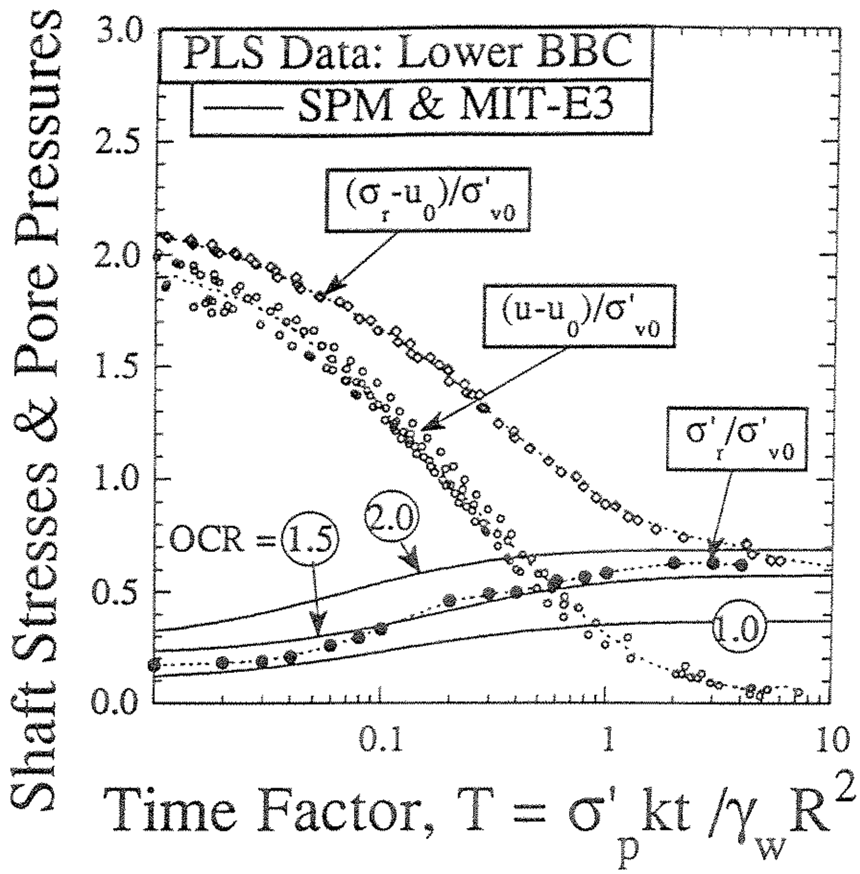


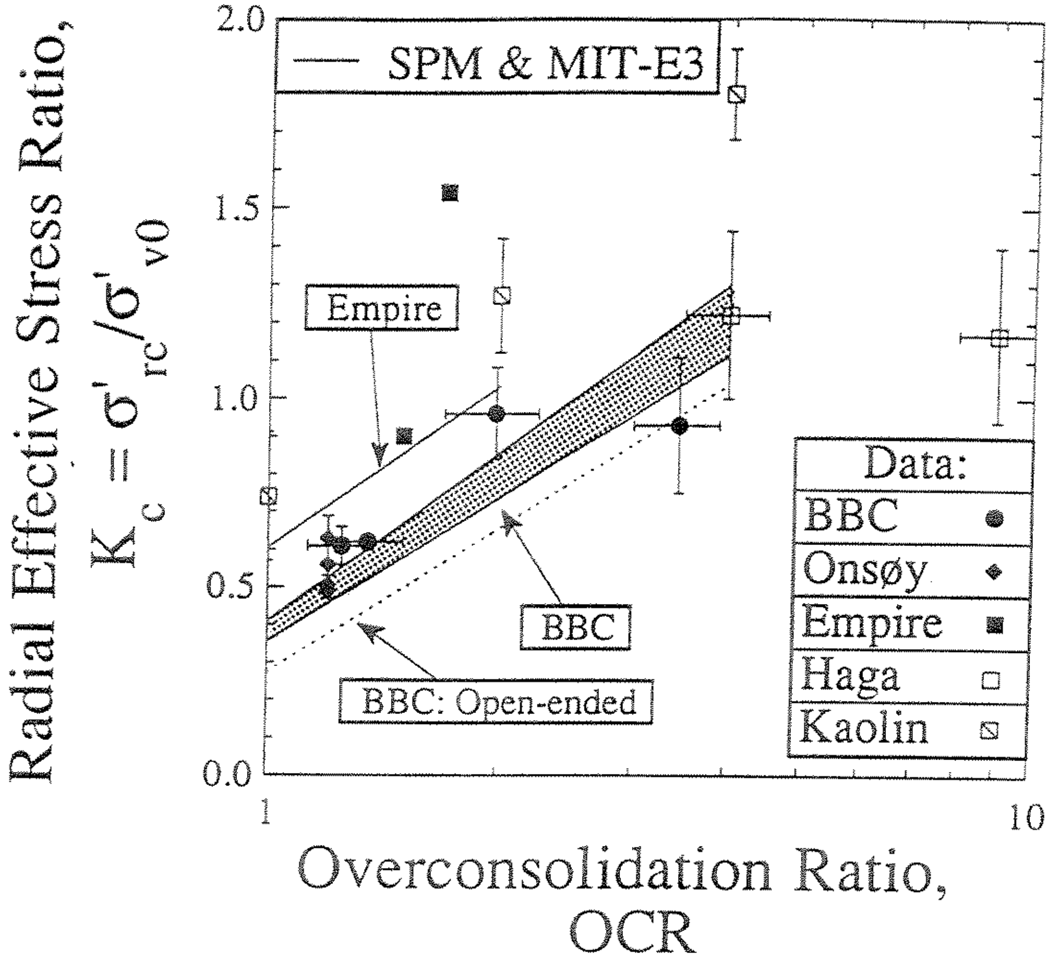
a) Predictions & Measurements in Empire Clay for OCR = 1.5
Predictions: SPM + MCC
Predictions: CEM + MCC
Predictions: SPM + MIT-E3
PLS Measurements
b) Predictions in BBC According to SPM & MIT-E3
OCR = 4
2
1.5
1
Lateral Earth Pressure Coefficient, K = σ'h / σ'vo
Time Factor, T = ct / r²o
2.0
1.6
1.2
0.8
0.4
0
10⁻⁵
10⁻⁴
10⁻³
10⁻²
10⁻¹
1
10

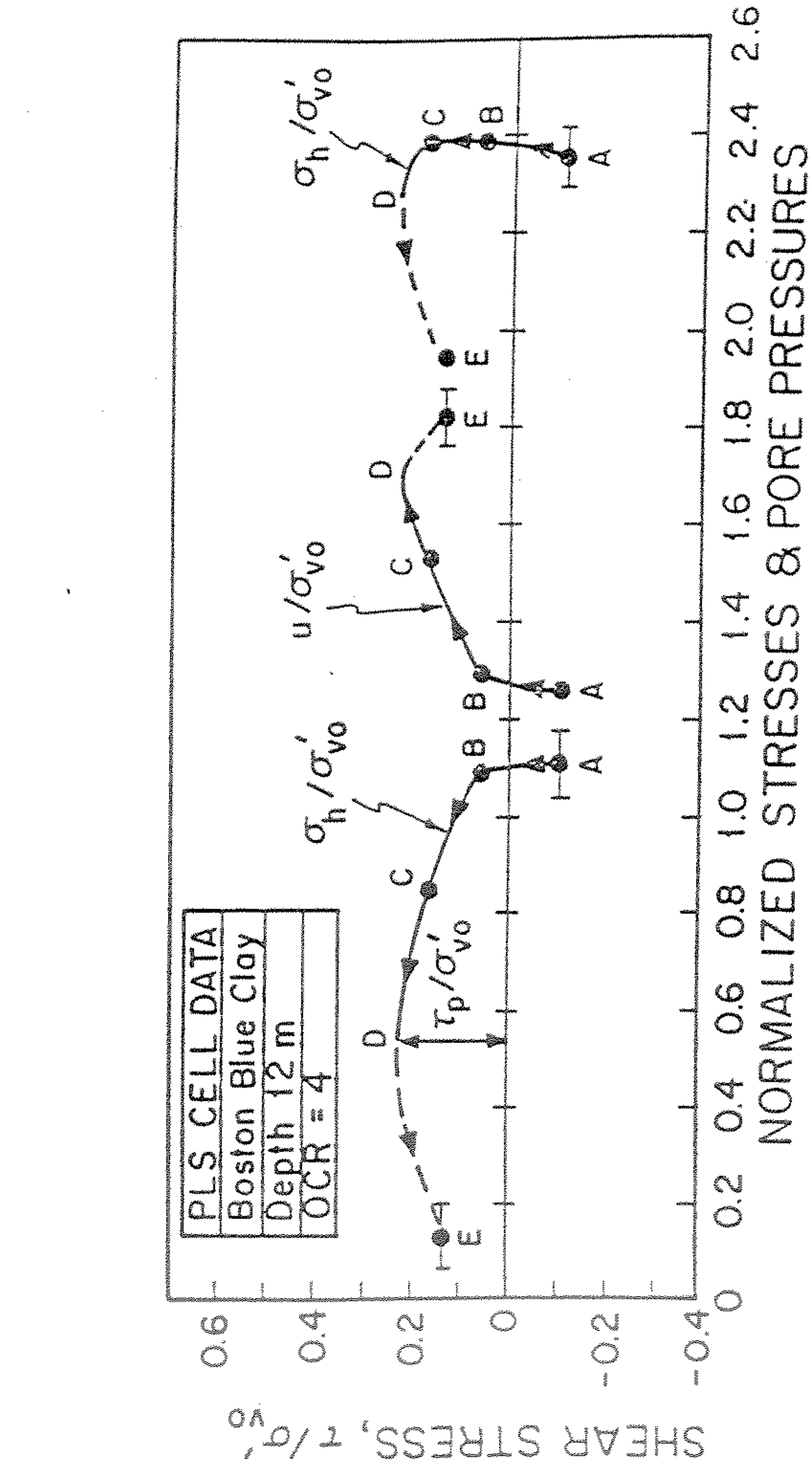


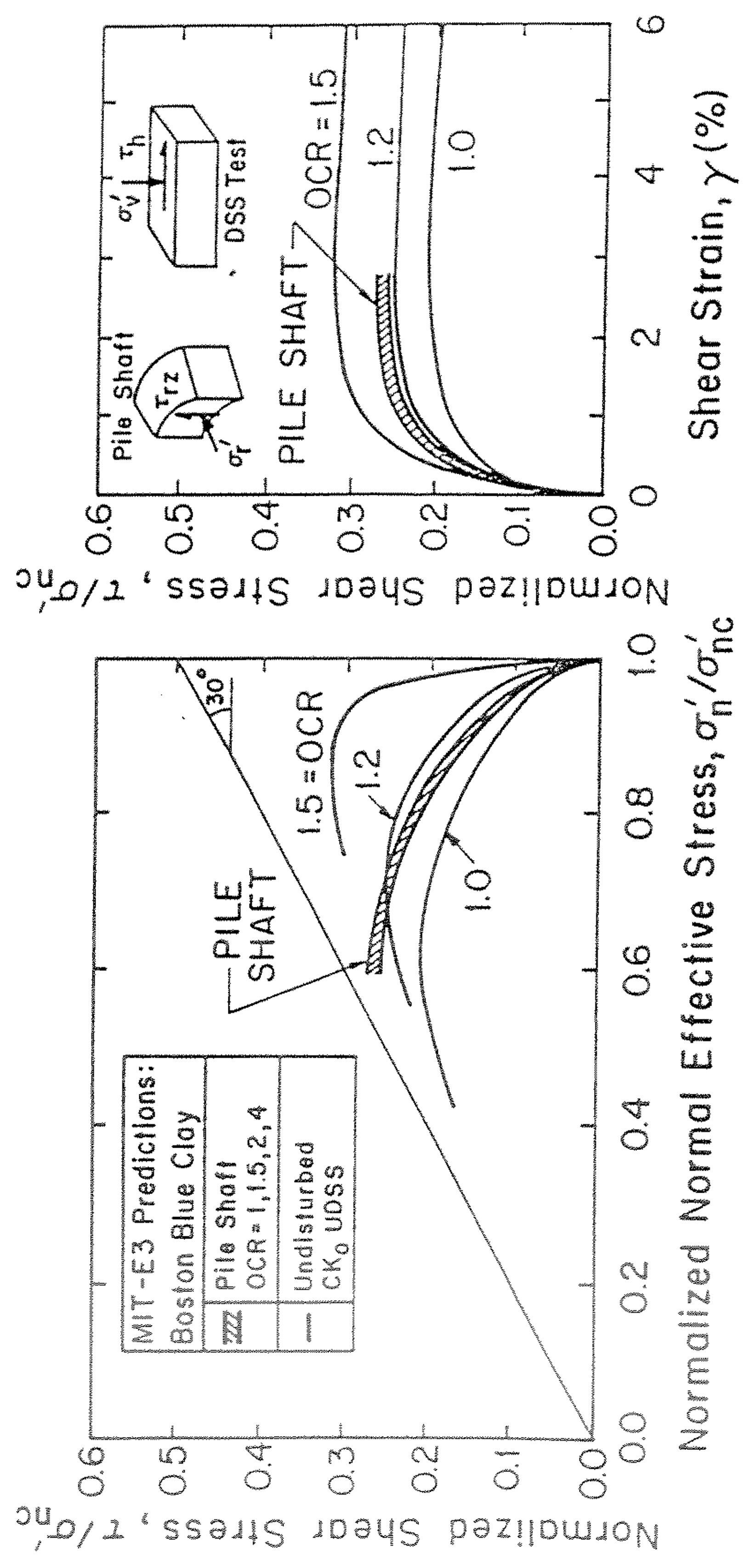
Pore Pressure Factor, $U = (u-u_0)/(u_i-u_0)$
1.0
0.8
0.6
0.4
0.2
0.0
10^{-4}
10^{-2}
10^{0}
Time Factor, $T = \sigma'_p k t / \gamma_w R^2$
MIT-E3: BBC
OCR = 1.0 - 1.5
Data: Lower BBC
PLS Cell
R = 1.92cm
σ'_{v0} = 190 - 260kPa
OCR = 1.3
$k_h = 8.0 \times 10^{-8}$ cm/sec

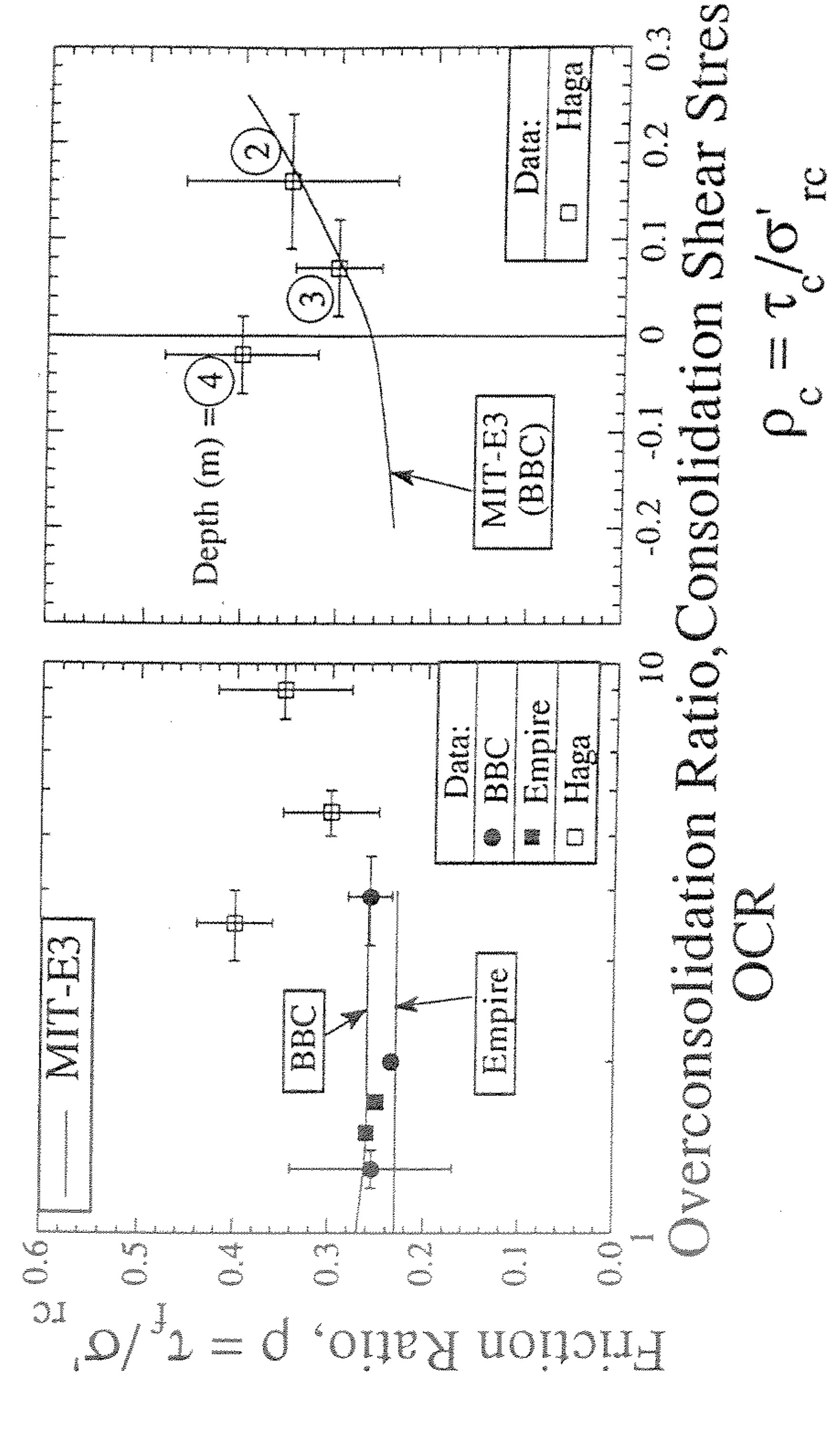

Total Radial Stress Ratio,
$H/H_i = (\sigma_r - u_0)/(\sigma_{ri} - u_0)$
MIT-E3: BBC
OCR = 1.0, 1.5
1.5 = OCR
1.0
Data: Lower BBC
PLS Cell
R = 1.92cm
σ'_{v0} = 190 - 260kPa
OCR = 1.3
$k_h = 8.0 \times 10^{-8}$ cm/sec
Time Factor, $T = \sigma'_p\, k\, t / \gamma_w R^2$
1.0
0.8
0.6
0.4
0.2
0.0
10^{-4}
10^{-2}
10^{0}


PLS Data: Lower BBC
SPM & MIT-E3
$(\sigma_r-u_0)/\sigma'_{v0}$
$(u-u_0)/\sigma'_{v0}$
σ'_r/σ'_{v0}
OCR = 1.5
2.0
1.0
Shaft Stresses & Pore Pressures
0.0
0.5
1.0
1.5
2.0
2.5
3.0
0.1
1
10
Time Factor, $T = \sigma'_p kt/\gamma_w R^2$


Radial Effective Stress Ratio,
$K_c = \sigma'_{rc}/\sigma'_{v0}$
SPM & MIT-E3
Empire
BBC
BBC: Open-ended
Data:
BBC
Onsøy
Empire
Haga
Kaolin
2.0
1.5
1.0
0.5
0.0
1
10
Overconsolidation Ratio,
OCR


PLS CELL DATA
Boston Blue Clay
Depth 12 m
OCR = 4
SHEAR STRESS, τ/σ'_{vo}
NORMALIZED STRESSES & PORE PRESSURES
σ_h/σ'_{vo}
u/σ'_{vo}
σ_h/σ'_{vo}
τ_p/σ'_{vo}
A
B
C
D
E
0.6
0.4
0.2
0
-0.2
-0.4
0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
2.2
2.4
2.6


MIT-E3 Predictions:
Boston Blue Clay
Pile Shaft
OCR = 1, 1.5, 2, 4
Undisturbed
CK$_0$ UDSS
PILE SHAFT
1.5 = OCR
1.2
1.0
30°
Normalized Normal Effective Stress, σ'_n/σ'_{nc}
Normalized Shear Stress, τ/σ'_{nc}
Pile Shaft
τ_{rz}
σ'_r
DSS Test
σ'_v
τ_h
PILE SHAFT
OCR = 1.5
1.2
1.0
Shear Strain, γ (%)
Normalized Shear Stress, τ/σ'_{nc}


Friction Ratio, $\rho = \tau_f/\sigma'_{rc}$
0.0
0.1
0.2
0.3
0.4
0.5
0.6
1
10
Overconsolidation Ratio, OCR
MIT-E3
BBC
Empire
Data:
BBC
Empire
Haga
-0.2
-0.1
0
0.1
0.2
0.3
Consolidation Shear Stress,
$\rho_c = \tau_c/\sigma'_{rc}$
Depth (m) = 4
3
2
MIT-E3 (BBC)
Data:
Haga

The ρ-Method

- **Assumptions:**
  - * Single, vertical, rigid, cylindrical pile
  - * Pile driven in deep deposit of clay
  - * Complete set-up
  - * Axial loading is "rapid" (i.e. undrained shearing of clay)
  - * Pile surface is rough
- **Method:**

$$f_s = \rho K_c \sigma'_{vo}$$

$$K_c = \sigma'_{hc}/\sigma'_{vo}$$

$$\beta = \rho K_c$$

- K_c, ρ **Predicted by analysis**

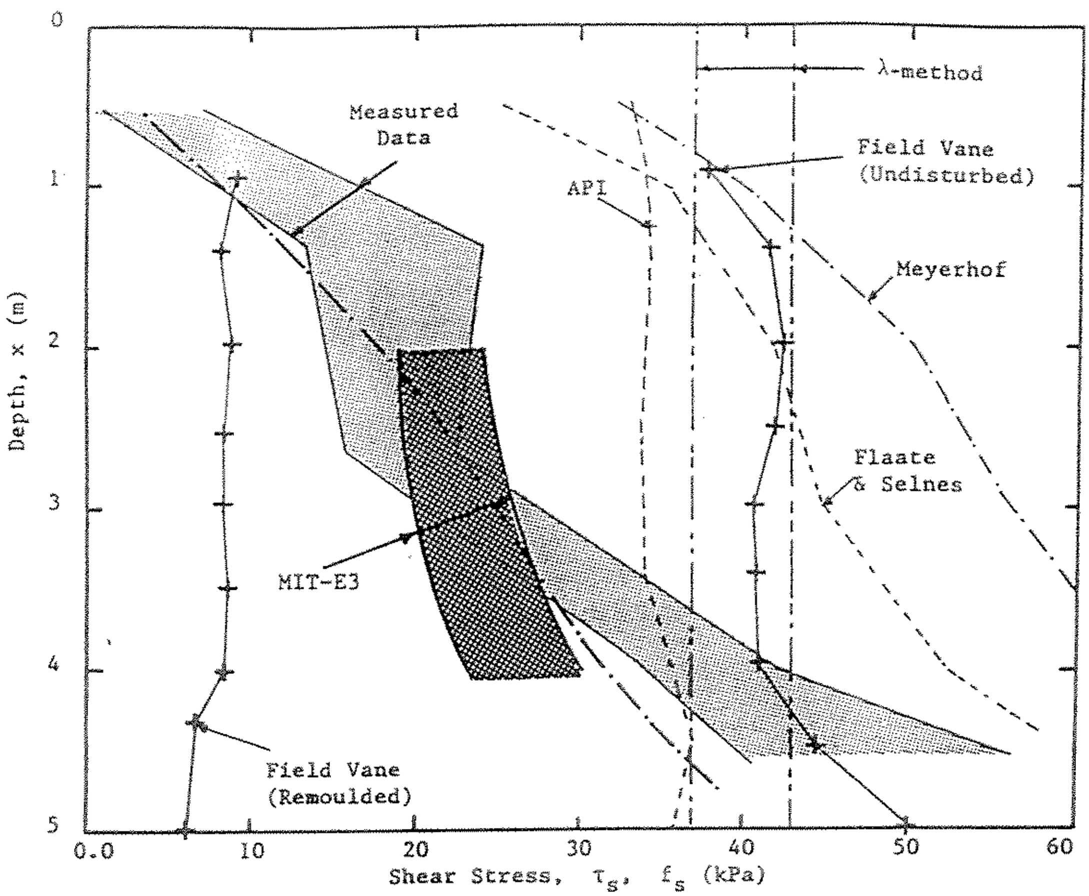
Field Data: Pile Load Tests

- **Haga Test Program.**
  NGI 1980-1984
  - * large scale model piles
    (L=5m, ϕ=0.15m)
  - * heavily instrumented
  - * laboratory test data on Haga clay
  - * data include
    - installation
    - consolidation
    - initial monotonic axial loading
    - cyclic axial loading
    - final axial loading

- **Most comprehensive data available in public domain (1987)**

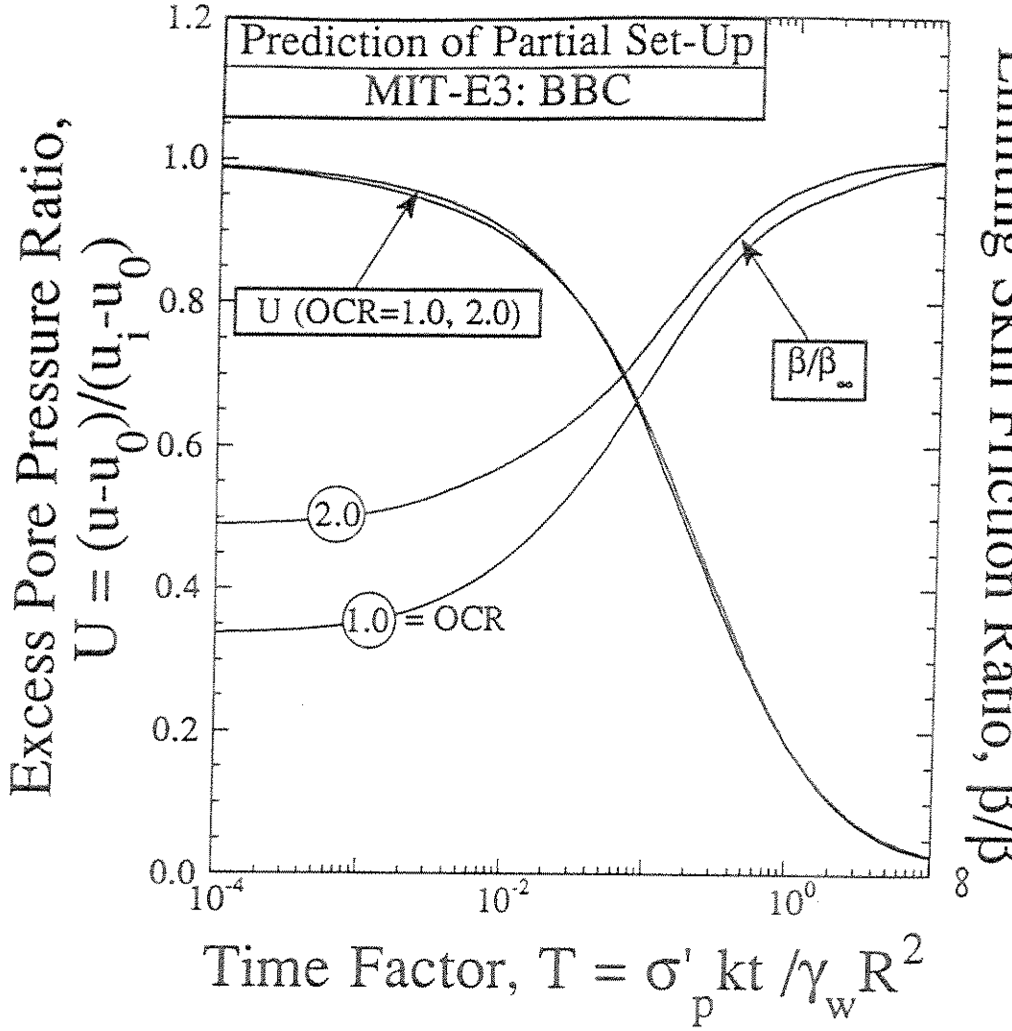
- WHITTLE (1991; OMAE CONFERENCE)

- MORE RECENTLY – PREDICTIONS FOR
  - IMPERIAL COLLEGE INSTRUMENTED PILE
  - BOTHKENNAR TEST SITE (U.K. NATIONAL TEST SITE)

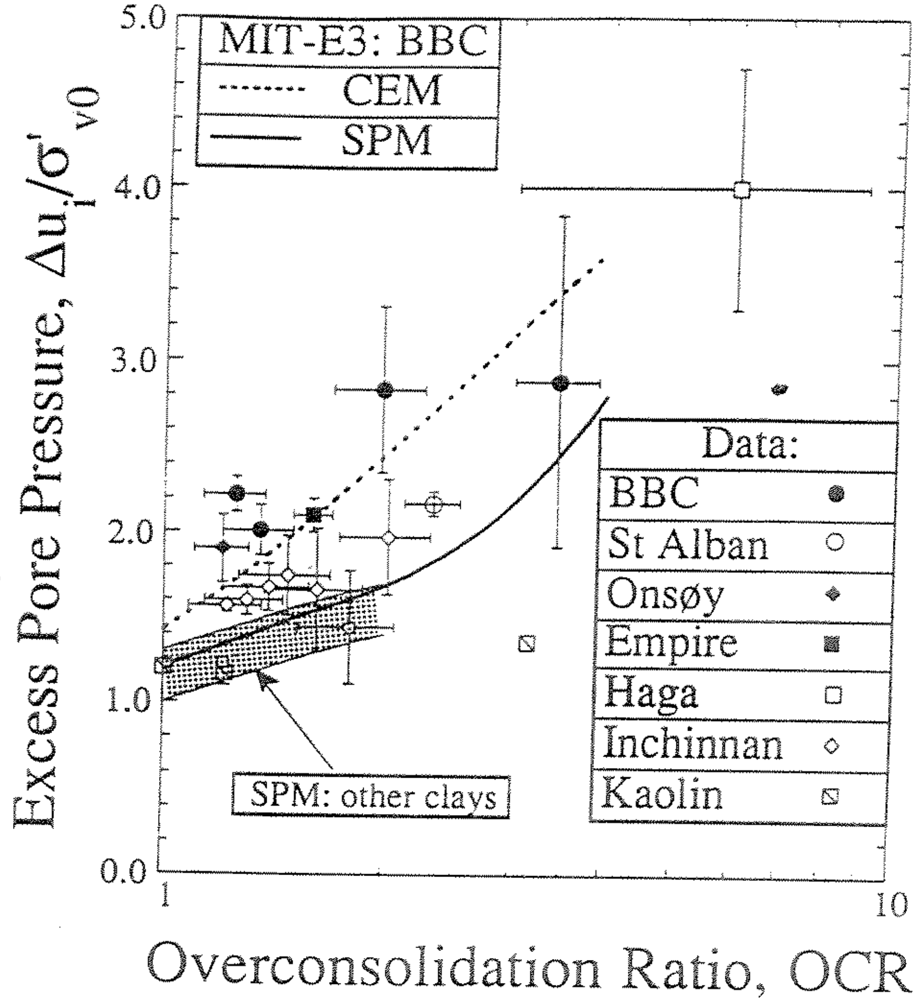


1. API-RP2A (1981):	$f_s = \alpha s_u$; $\alpha = f(s_u)$
2. λ-Method:	$f_s = \lambda(\bar{\sigma}_{vo} + 2 s_u)$; $\lambda = f(L)$ 1. Vijayvergiya and Focht (1972) 2. Kraft et al. (1981)
3. Flaate & Selnes (1977):	$f_s = 0.4\sqrt{OCR}\,\mu_L\,\bar{\sigma}_{vo}$; $\mu_L = (L + 20)/(2L + 20)$ (in m)
4. Meyerhof (1976):	$f_s = 1.5\,(1 - \sin\bar{\phi})\tan\bar{\phi}\sqrt{OCR}\,\bar{\sigma}_{vo}$

MIT-E3 Predictions of Limiting Skin Friction, f_s



Prediction of Partial Set-Up
MIT-E3: BBC
U (OCR=1.0, 2.0)
β/β∞
2.0
1.0 = OCR
Excess Pore Pressure Ratio, U = (u-u0)/(ui-u0)
Limiting Skin Friction Ratio, β/β∞
Time Factor, T = σ'p kt /γw R²


MIT-E3: BBC
CEM
SPM
Data:
BBC
St Alban
Onsøy
Empire
Haga
Inchinnan
Kaolin
SPM: other clays
Excess Pore Pressure, $\Delta u_i/\sigma'_{v0}$
Overconsolidation Ratio, OCR
5.0
4.0
3.0
2.0
1.0
0.0
1
10

CONCLUSIONS

- **REVIEW OF SYSTEMATIC ANALYSIS**
- **IMPORTANCE OF:**
  - Pile Installation Analysis
  - Soil Model
  - PLS Cell & Instrumented Piles
- **INSTALLATION PHASE**
  - Underpredict Pore Pressures
- **CONSOLIDATION**
  - Consistent Predictions
  - Resolve Factors $\Rightarrow K_C$
- **AXIAL PILE LOADING**
  - ρ-Method For Minimum f_s
  - Extend Analysis $\Rightarrow$ Pile Length
- **FLEXIBILITY OF FRAMEWORK**
  - Plugged vs. Unplugged Penetration
  - Partial Set-Up
  - Cyclic Axial Loading

CURRENT RESEARCH ACTIVITIES

* **In-situ Penetration Tests**
(piezocone, dilatometer etc)

- predict in-situ measurements in soft clays
- compare with high quality field data
- evaluate basis for empirical correlations

* **Suction Caisson Foundations**

- analysis
  SPM/MIT-E3 & ABAQUS
  complete foundation (mudline-tip)
  installation/consolidation/axial load

- instrumented laboratory tests
  very small scale model caissons
  properties of resedimented BBC
  very controlled measurements
  very small scale

- interpretation of centrifuge & field tests

DEVICE	MEASUREMENT: INSTALLATION	MEASUREMENT: SUBSEQUENT ACTIVITY
PIEZOCONE	• Tip resistance, q_T • Pore Pressure, u_i (Location: Tip, Base, Shaft) • *Sleeve Friction, f_s*	• Dissipation Test, u(t)
PRESSUREMETER	• Lift-Off Pressure, p_i • *Pore Pressure, u_i*	• Membrane Expansion; p(t), $\Delta V/V(t)$ (Membrane Contraction) • Holding Test, p(t)
DILATOMETER	• Contact Pressure, p_0	• Expansion Pressure, p_1 (Closure pressure, p_2) • Holding Test, p(t)
EARTH PRESSURE CELLS	--	• Equilibration Pressure, p(t) -> p_∞
FIELD VANE	--	• Torque, T

TYPES OF IN-SITU MEASUREMENT IN CLAYS

• EXTENSIVE STUDY 1988-1991

• 2 MIT RESEARCH REPORTS 1) [illegible]

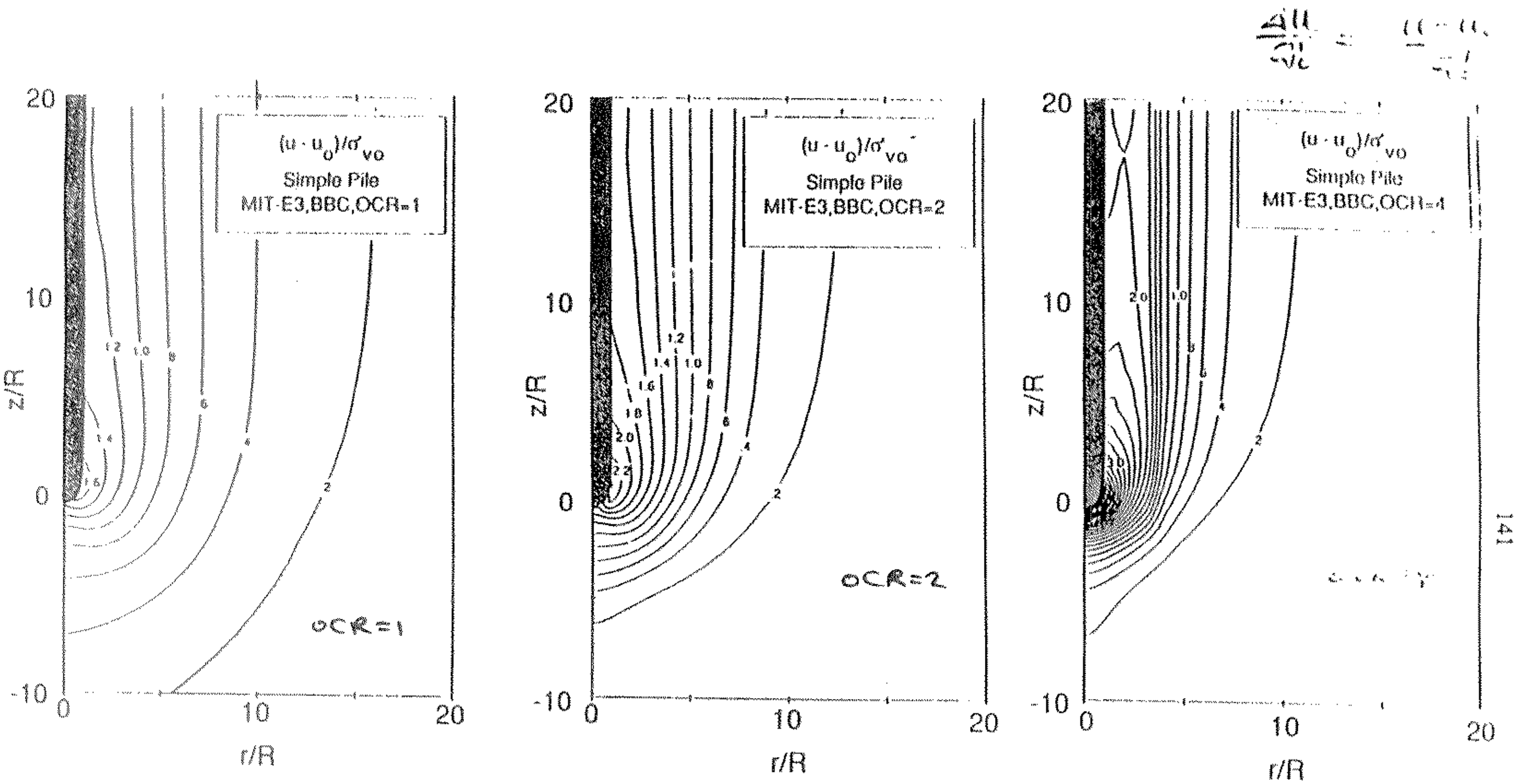
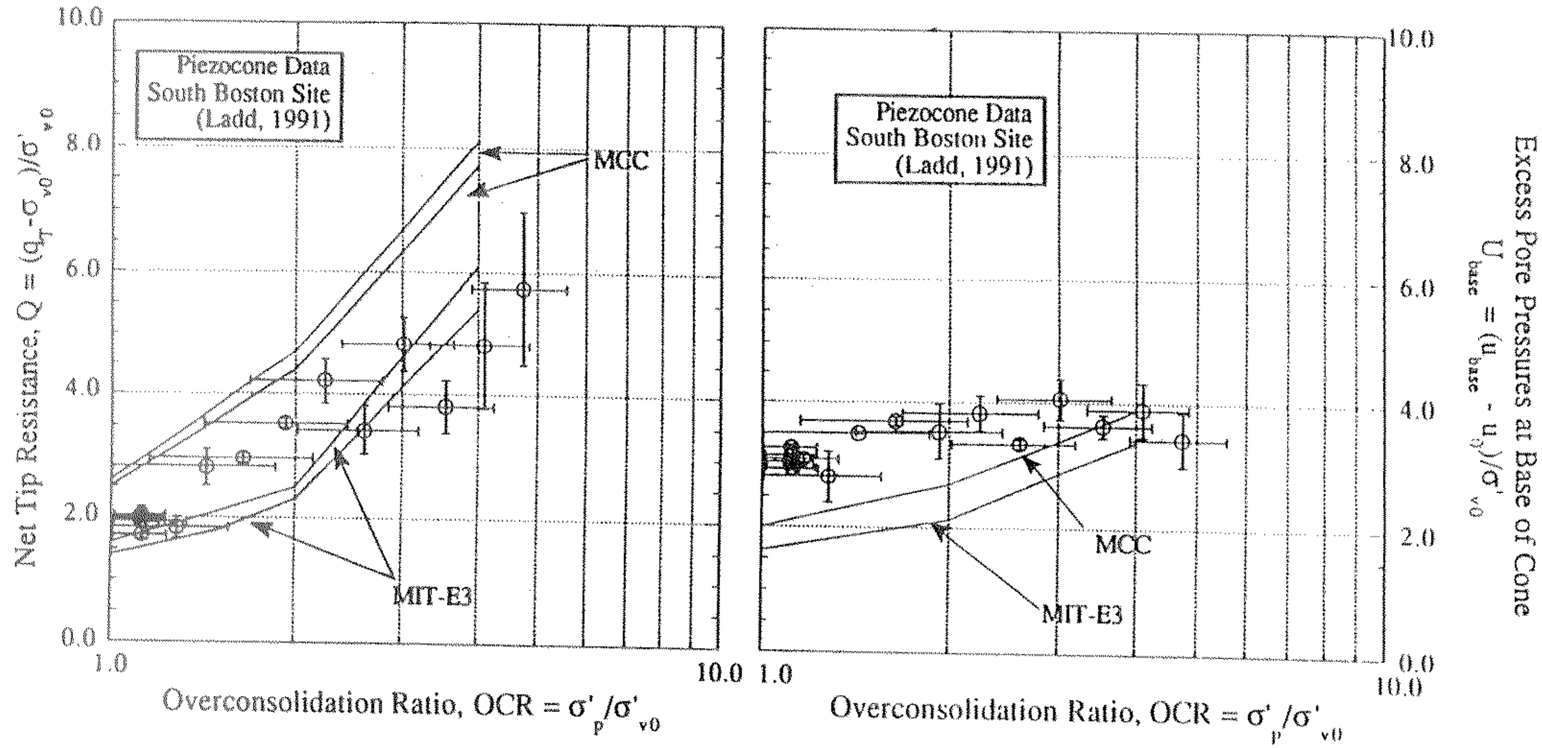


Figure 4.16 Predictions of effective stresses and pore pressures using the MIT-E3 model (Contd.) d) $(u-u_0)/\sigma'_{v0}$

• INSTALLATION PORE PRESSURES

• DEVELOPMENT OF NEW METHOD TO SOLVE EQUILIBRIUM

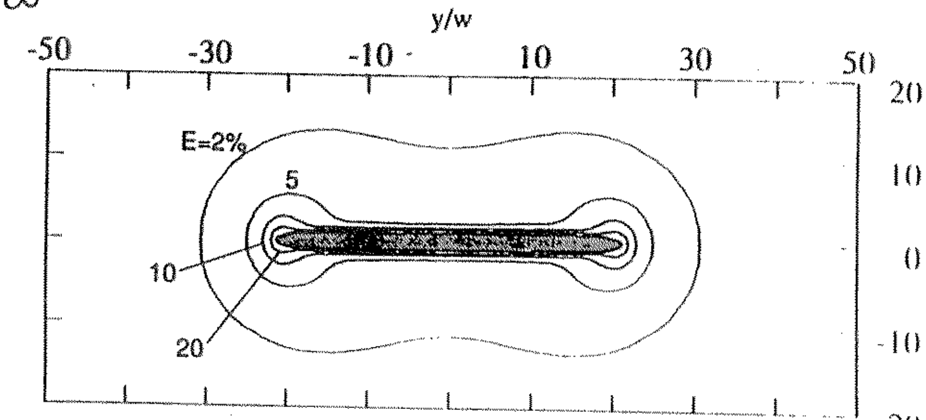


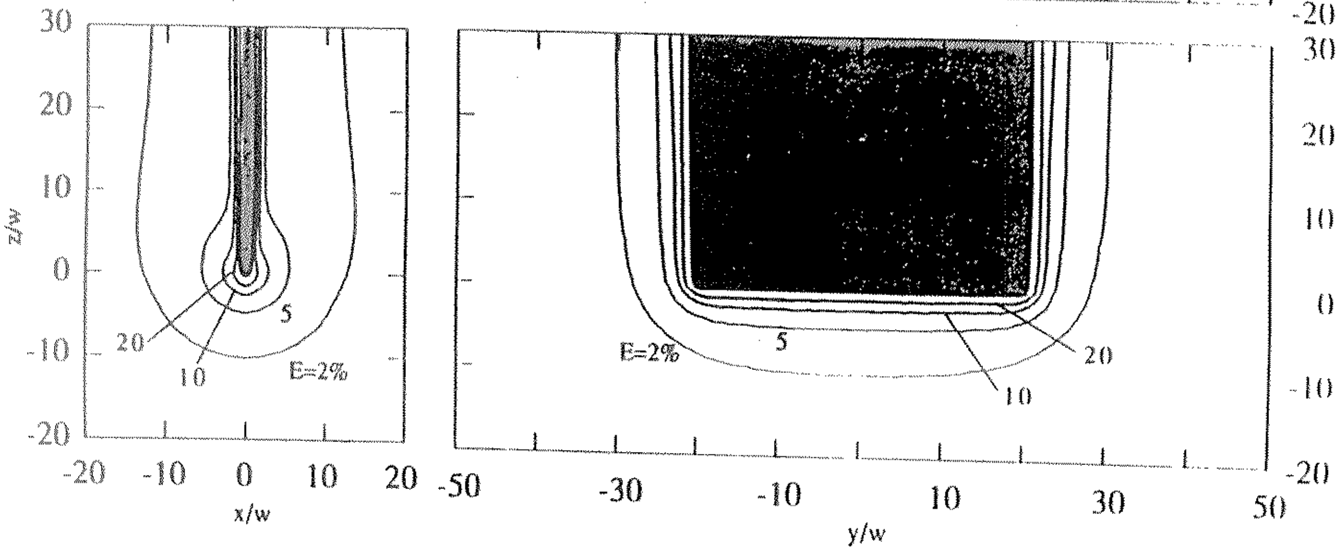
EVALUATION OF PIEZOCONE MEASUREMENTS IN BOSTON BLUE CLAY

- DIMENSIONS

$R_{eq} = \sqrt{\frac{4Bw}{\pi}}$; B/ω

Device	Aspect Ratio B/w	Equivalent Radius, R_{eq} (mm)
Piezocone/ Cone Pressuremeter	1.0	17.8
Dilatometer	6.8	20.6
Earth Pressure Cell	25	34.0
Field Vane	32.5	12.9
Push-in Pressuremeter	12	25.0
Self-Boring Pressuremeter	12	-





SIMPLE PLATE, B/w = 20

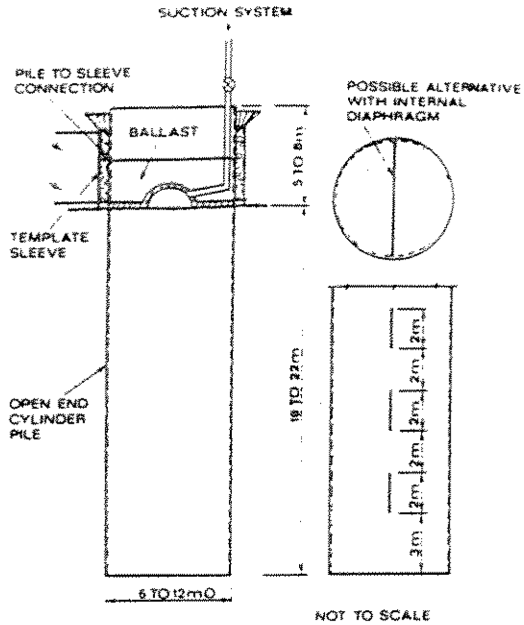


Fig. 2a Superpiles for Tension Leg Platform (Albert et al., 1987)

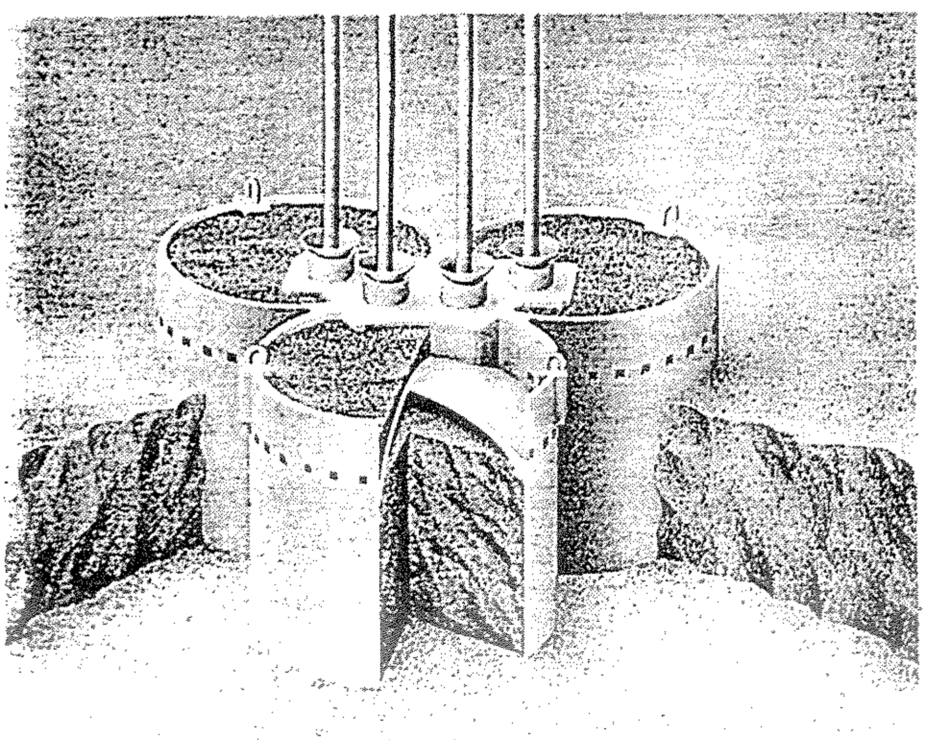
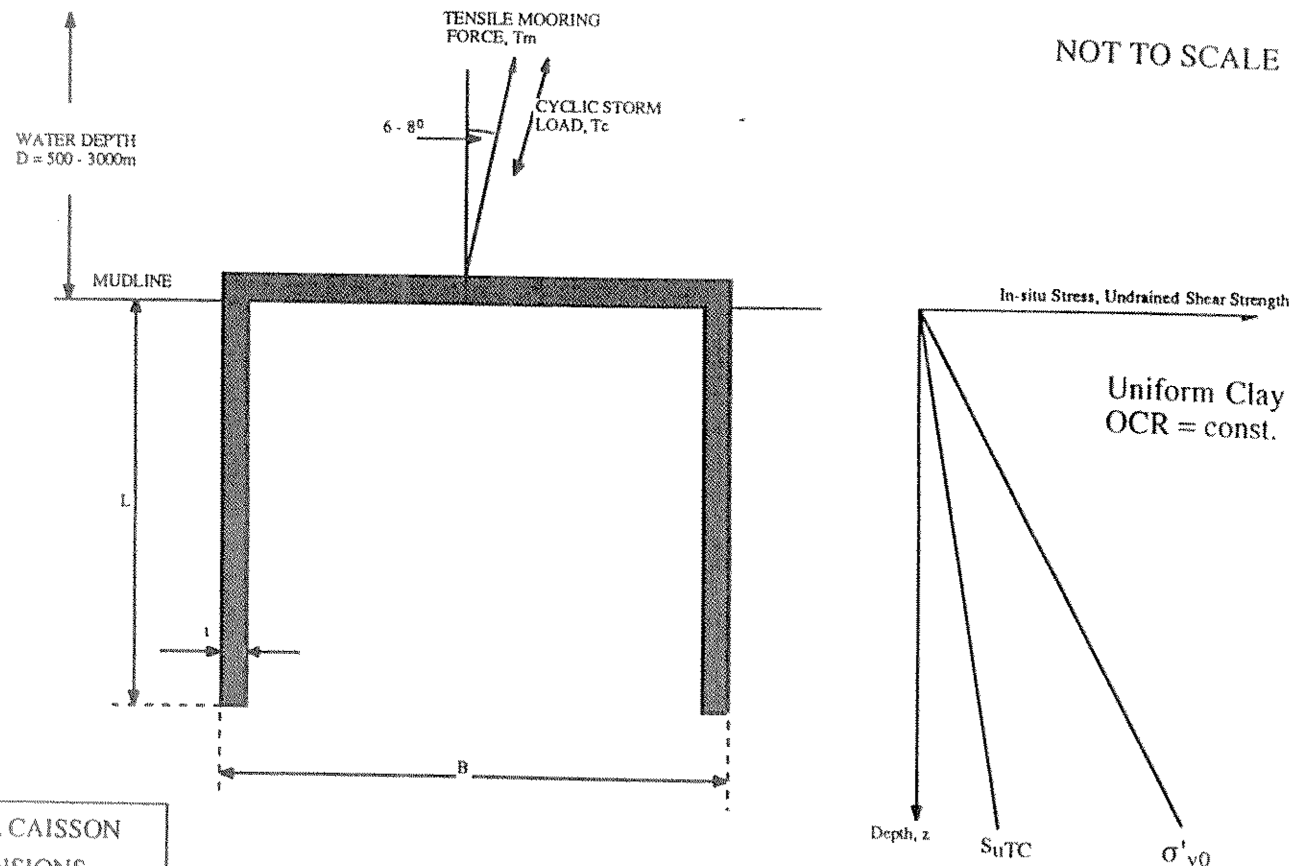


Fig. 2b Concrete Foundation Templates for Snorre TLP (Offshore Engineer, 1990)



TYPICAL CAISSON DIMENSIONS	
B (m)	10 - 30
L (m)	10 - 60 (?)
B/t	40 - 100
L/t	20 - 500

GEOMETRY OF SUCTION CAISSON

BEHAVIOR OF SUCTION CAISSON FOUNDATIONS

GEOTECHNICAL PROBLEMS

1. Suction required to Achieve Caissson Penetration

2. Prediction of Caisson Set-Up Performance

3. Predictions of Soil Reaction Forces (TLP Applications)
   - Mechanisms affecting generation and release of suction pressures
   - Prediction of frictional resistance (inner and outer caisson walls)

4. Effects of Caisson Geometry
   - Individual Caisson
   - Cell Configuration

5. Influence of Soil Properties (mainly soft clays)

BEHAVIOR OF SUCTION CAISSON FOUNDATIONS

RESEARCH TASKS

1. ANALYTICAL MODELLING

- Rational Predictive Framework:

  Installation - Consolidation - Loading

- Focus:

  Loading Conditions expected for TLP Applications

- Method:

  Integrate Analytical Tools

  STRAIN PATH METHOD - Model of Installation Disturbance

  MIT-E3 SOIL MODEL - Effective Stress Soil Model for Clays

  ABAQUS - General Non-Linear Finite Element Program

- Main Parameters:

  Individual Caisson Geometry

  Cell Configuration

  Soil Properties

2. EVALUATION OF ANALYSES

- Large Scale Model Caisson Tests (NGI)
- Centrifuge Model Tests (EPR)

3. LABORATORY MEASUREMENTS

- Small Scale
- Detailed Evaluation of Analyses

  Effects of Caisson Installation: Pore Pressures & Set-Up

  Release of Suction Pressures under Sustained Tensile Loads

FUTURE DIRECTIONS

- **Improvements in Soil Modelling**

  * High OC clays
    - explain complex pile response
    - role of fabric -> residual shear
    - experiments: artificial high OCR clay

  * Rate Effects
    - long term response
    - cyclic behavior
    - need much better lab. data

  * Sands
    - predictive capability over wide range of stress & density (σ', e)

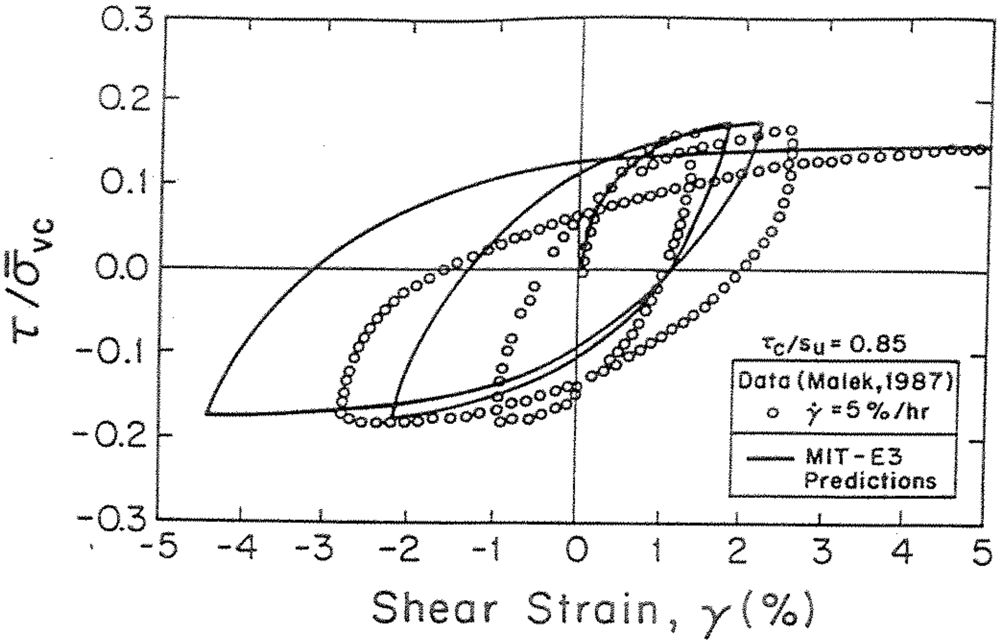
- **Model for (Partially) Drained Installation**

  * Options:
    - extend SPM
    - Eulerian FE
    - combination of SPM/FE

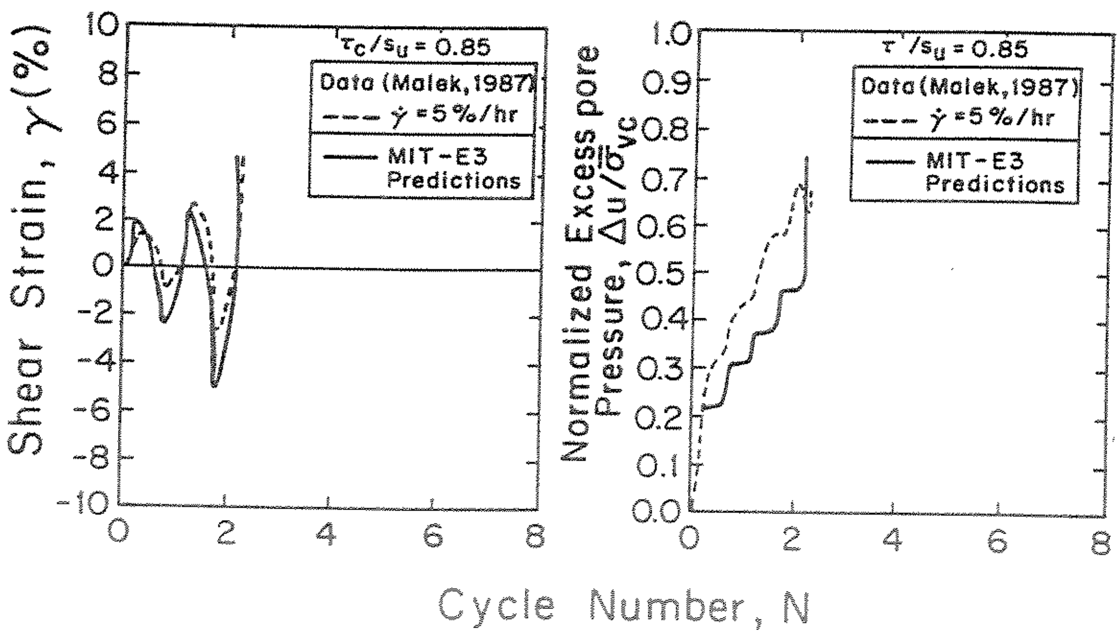
  * Need
    - better constitutive model
    - more experimental data
      * lab. element tests
      * lab. model tests (CC)
      * instrumented model piles

- **Scale Effects & Geometry**

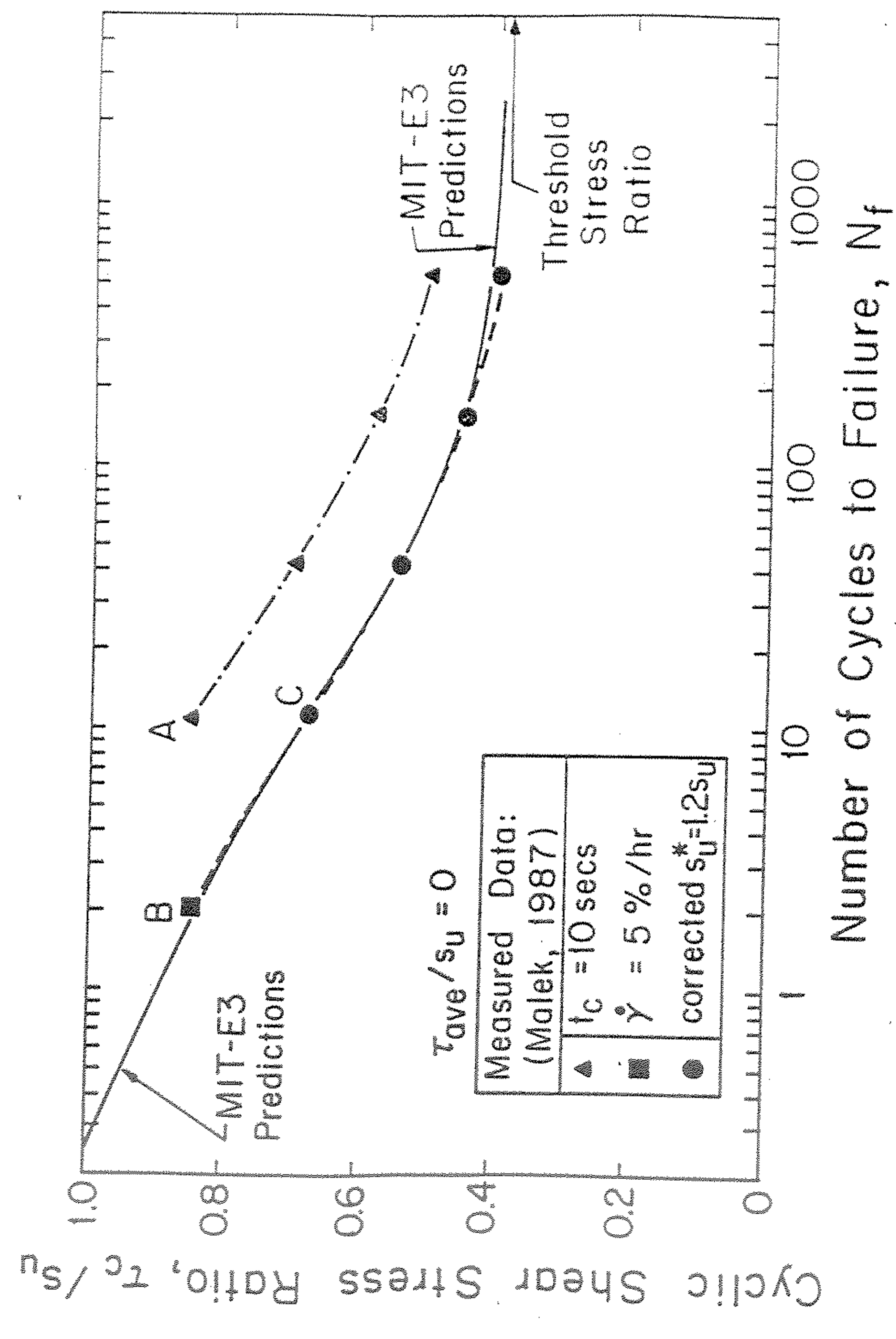
  * Complete analysis of pile length (mudline -> tip)
  * Calibrate/compare vs. simple load-transfer model
  * Effect of pile diameter - especially in sands
  * Open-ended piles - Need Experimental Data

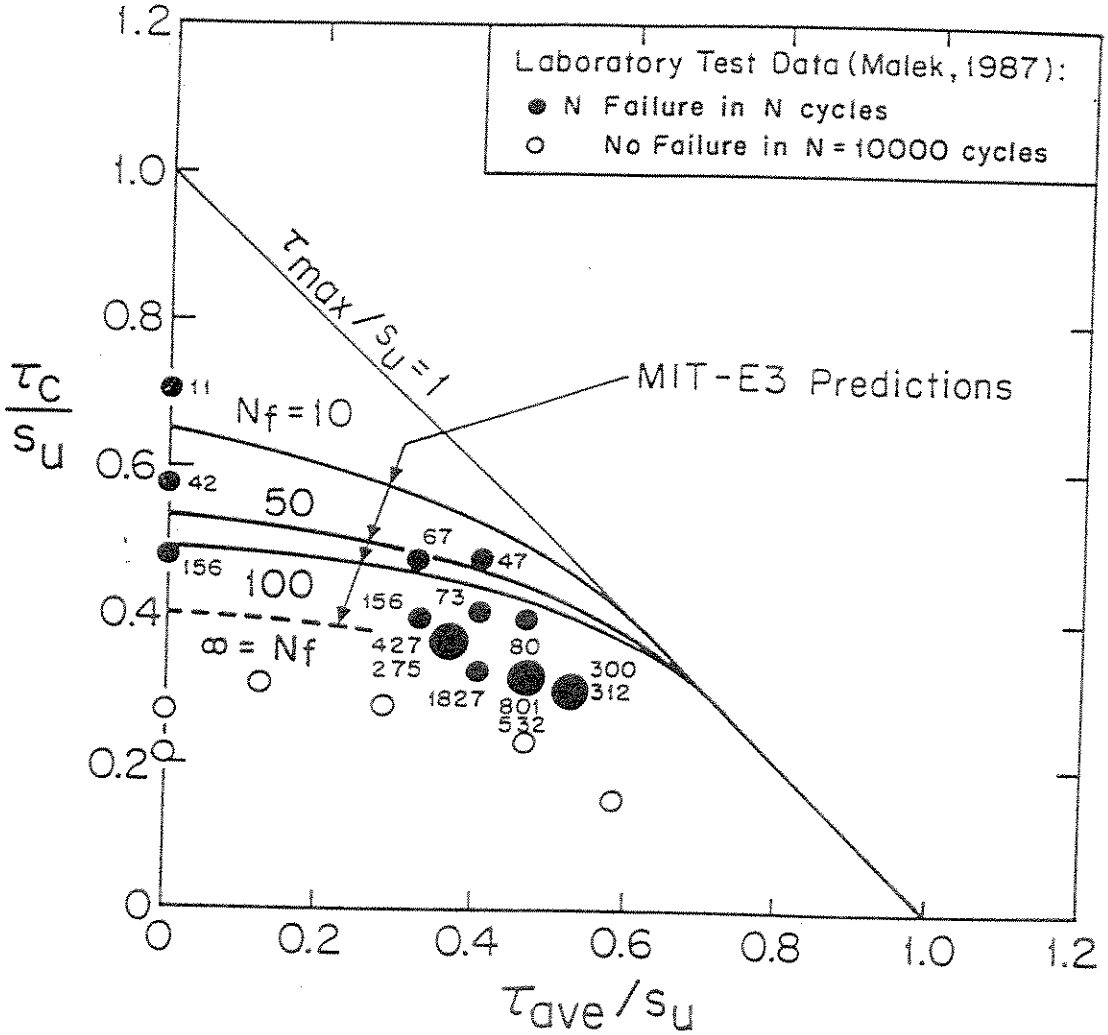


$\tau/\bar{\sigma}_{vc}$
Shear Strain, γ(%)
$\tau_c/s_u = 0.85$
Data (Malek, 1987)
$\dot{\gamma}$ = 5 %/hr
MIT-E3 Predictions

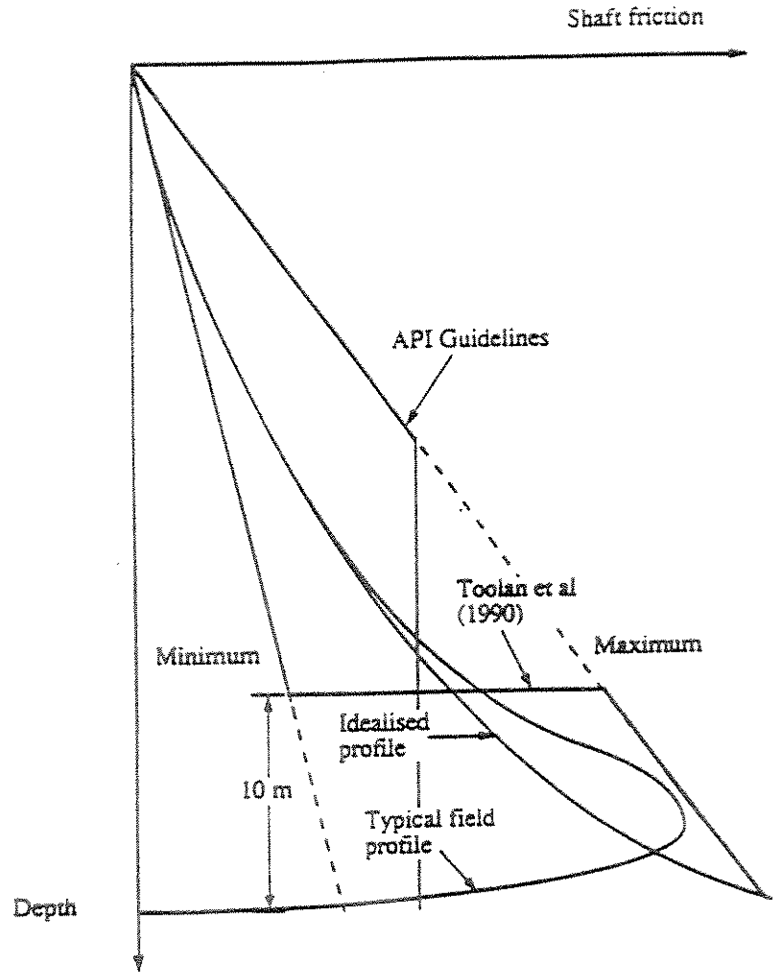


Shear Strain, γ(%)
$\tau_c/s_u = 0.85$
Data (Malek, 1987)
$\dot{\gamma}$ = 5 %/hr
MIT-E3 Predictions
Normalized Excess pore Pressure, $\Delta u/\bar{\sigma}_{vc}$
$\tau/s_u = 0.85$
Data (Malek, 1987)
$\dot{\gamma}$ = 5 %/hr
MIT-E3 Predictions
Cycle Number, N


Cyclic Shear Stress Ratio, τ_c / s_u
1.0
0.8
0.6
0.4
0.2
0
Number of Cycles to Failure, N_f
1
10
100
1000
MIT-E3 Predictions
MIT-E3 Predictions
Threshold Stress Ratio
A
B
C
$\tau_{ave}/s_u = 0$
Measured Data: (Malek, 1987)
t_c = 10 secs
$\dot{\gamma}$ = 5 %/hr
corrected $s_u^* = 1.2 s_u$

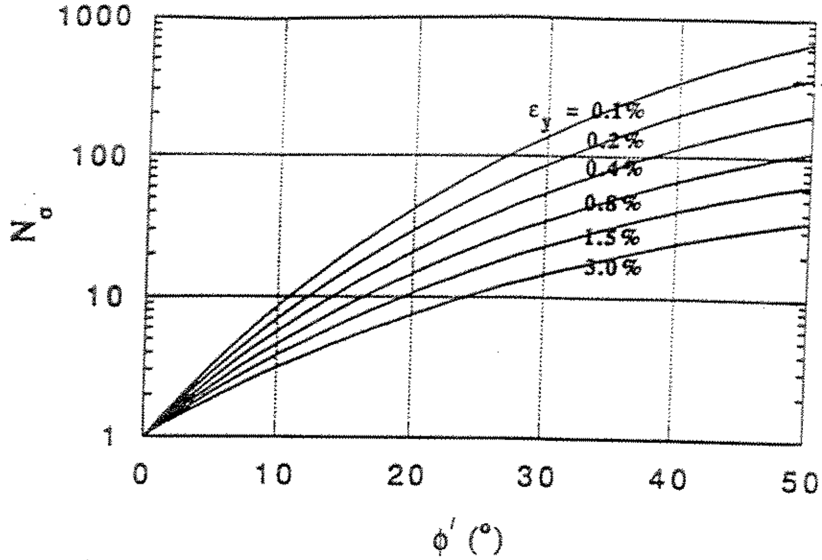


Laboratory Test Data (Malek, 1987):
N Failure in N cycles
No Failure in N = 10000 cycles
$\tau_{max}/s_u = 1$
MIT-E3 Predictions
$N_f = 10$
50
100
$\infty = N_f$
τ_c / s_u
τ_{ave} / s_u
11
42
156
67
47
156
73
80
427
275
1827
801
532
300
312
1.2
1.0
0.8
0.6
0.4
0.2
0
0.2
0.4
0.6
0.8
1.0
1.2



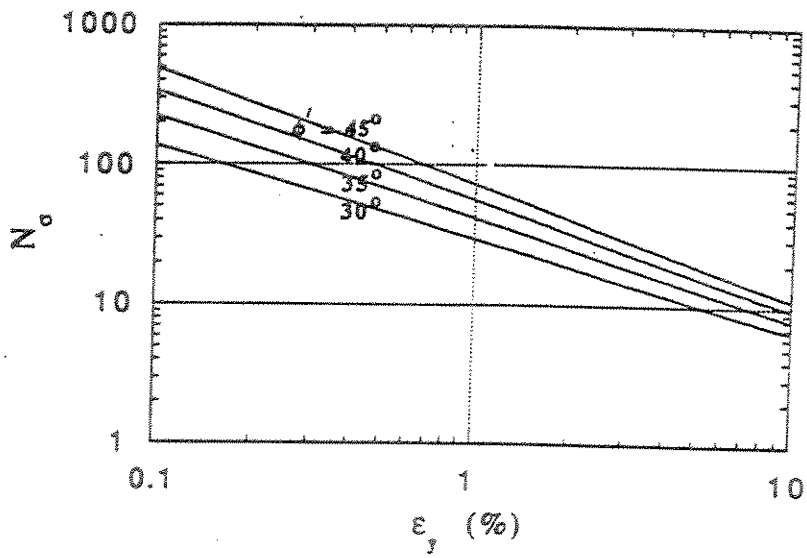
Major observation from report by Randolph: Skin friction along shaft is related to tip resistance (probably obtained by cone penetrometer). Proposed design methods show a large variation in skin friction close to the tip of the pile. Limited experimental data (e.g. Imperial College instrumented pile) show similar change in the normal stress acting on the pile during installation.

$q_c = N_\sigma \sigma_o'$



ε_y = YIEL STRA (TC

(a) Point Resistance Factor Versus Friction Angle



(b) Point Resistance Factor Versus Axial Yield Strain

Figure 4.8 (a) Point Resistance Factor Versus Friction Angle
(b) Point Resistance Factor Versus Axial Yield Strain

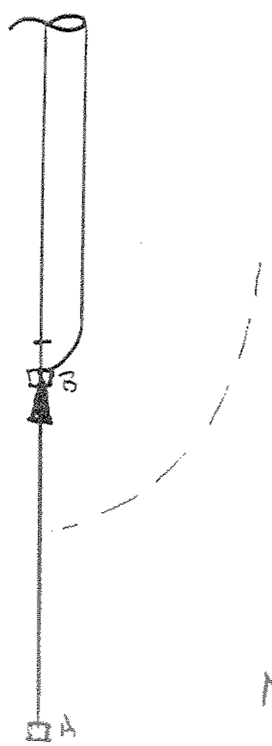
SPM INCOMPRESSIBLE MATERIAL
TIP RESISTANCE FACTORS

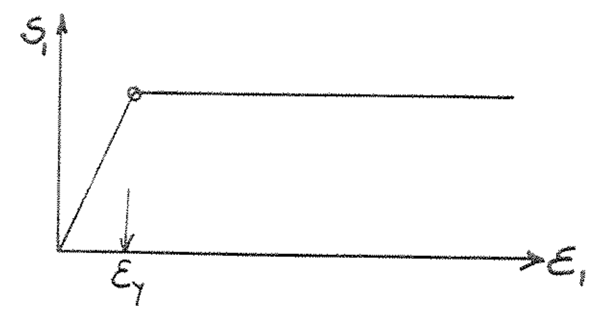
DRAINED PENETRATION

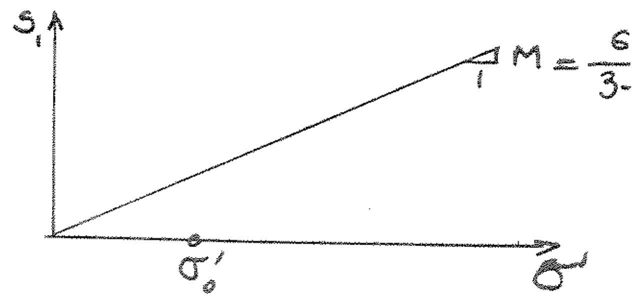
- $u_{tip} = 0$
- DEPENDENCY OF MATERIAL PROPERTIES ON MEAN EFFECTIVE STRESS,
- ESP FUNCTION OF PATH OF LOADING
- VOLUME CHANGE DURING SHEAR
- CENTERLINE SOLUTION

$$q_c' = \left(\sigma_{zz}'\right)_{tip} = \frac{3(1+\sin\phi')}{3-\sin\phi'}\,($$

- BILINEAR MODEL

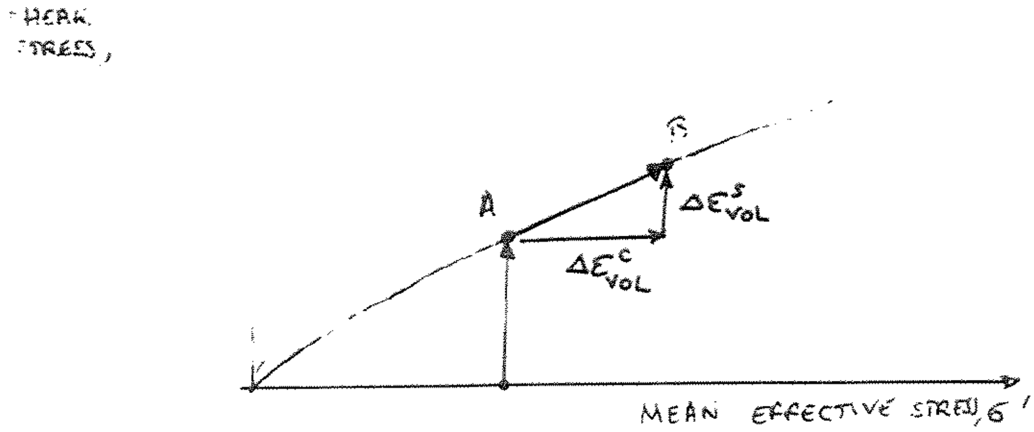






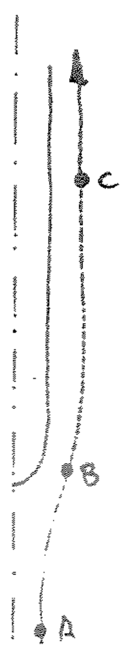
$$N_\sigma = \frac{q_c'}{\sigma_0'} = \frac{3(1+\sin\phi')}{(3-\sin\phi')}\left(\frac{1}{3}+\frac{8}{3}\left(\frac{z_p}{R}\right)^2\right)^{\frac{2\sin}{1+\sin}}$$

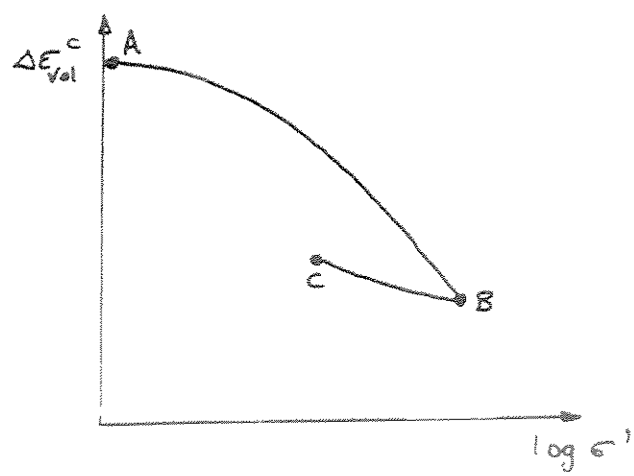
- CONCEPT OF VOLUME CHANGE MODEL

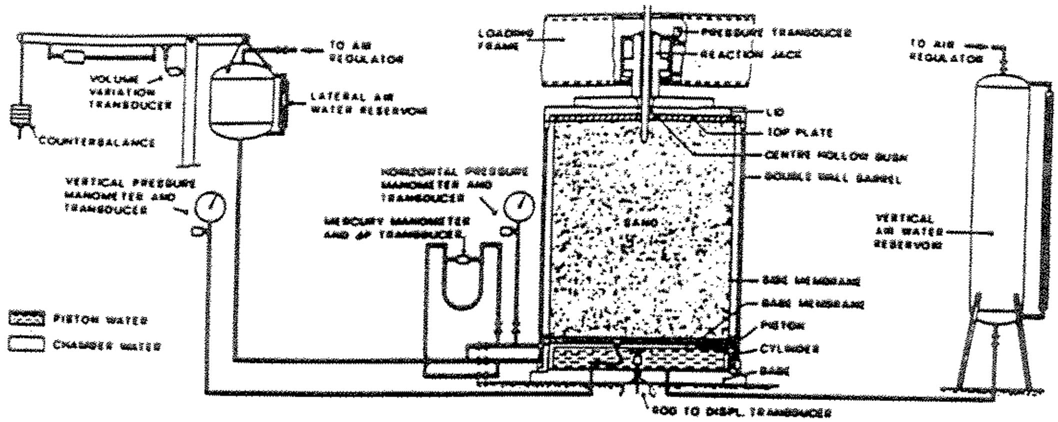


COUNTER ACTING

- $\Delta \varepsilon^{c}_{vol}$ – VOLUME CHANGE DUE TO HYDROSTATIC COMPRESSION
- $\Delta \varepsilon^{s}_{vol}$ – VOLUME CHANGE DUE TO SHEAR (DILATION RATE)





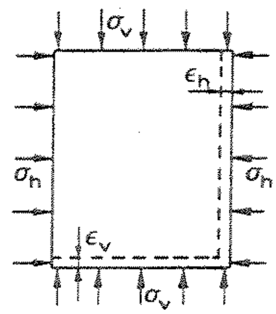


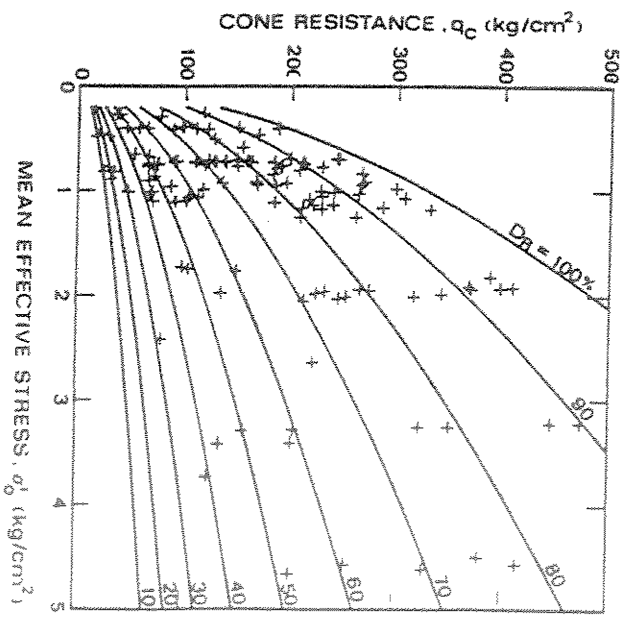
BC1 $\sigma_v = \text{const}$ $\quad \sigma_h = \text{const}$

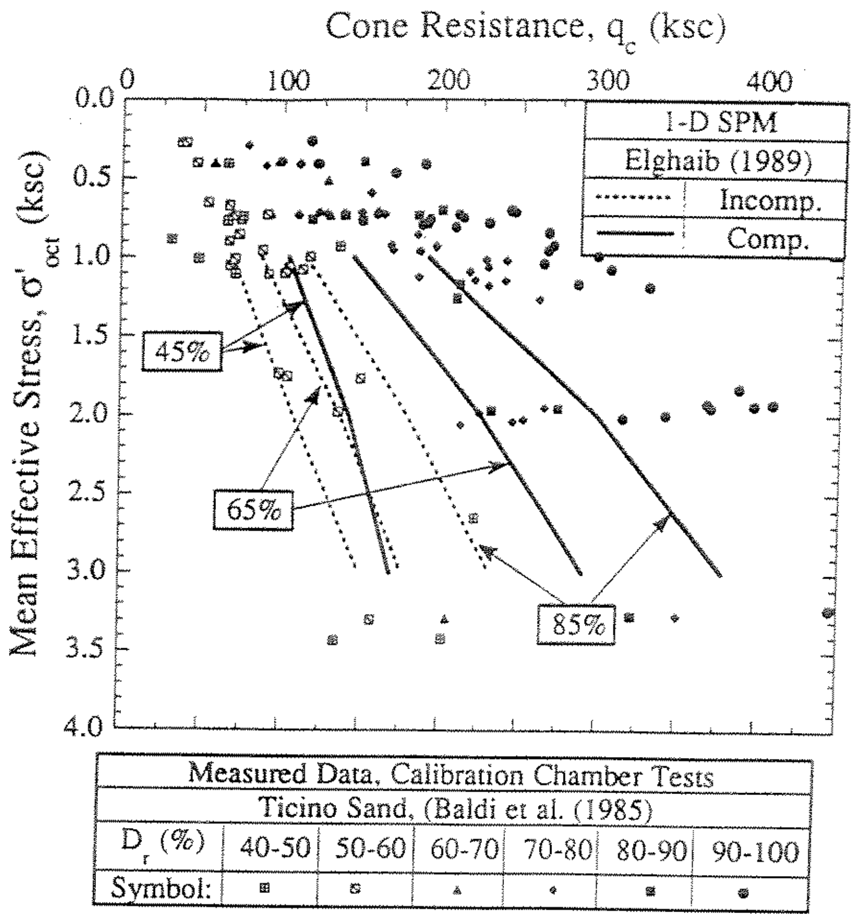
BC2 $\Delta\epsilon_v = \Delta\epsilon_h = 0$

BC3 $\sigma_v = \text{const}$ $\quad \Delta\epsilon_h = 0$

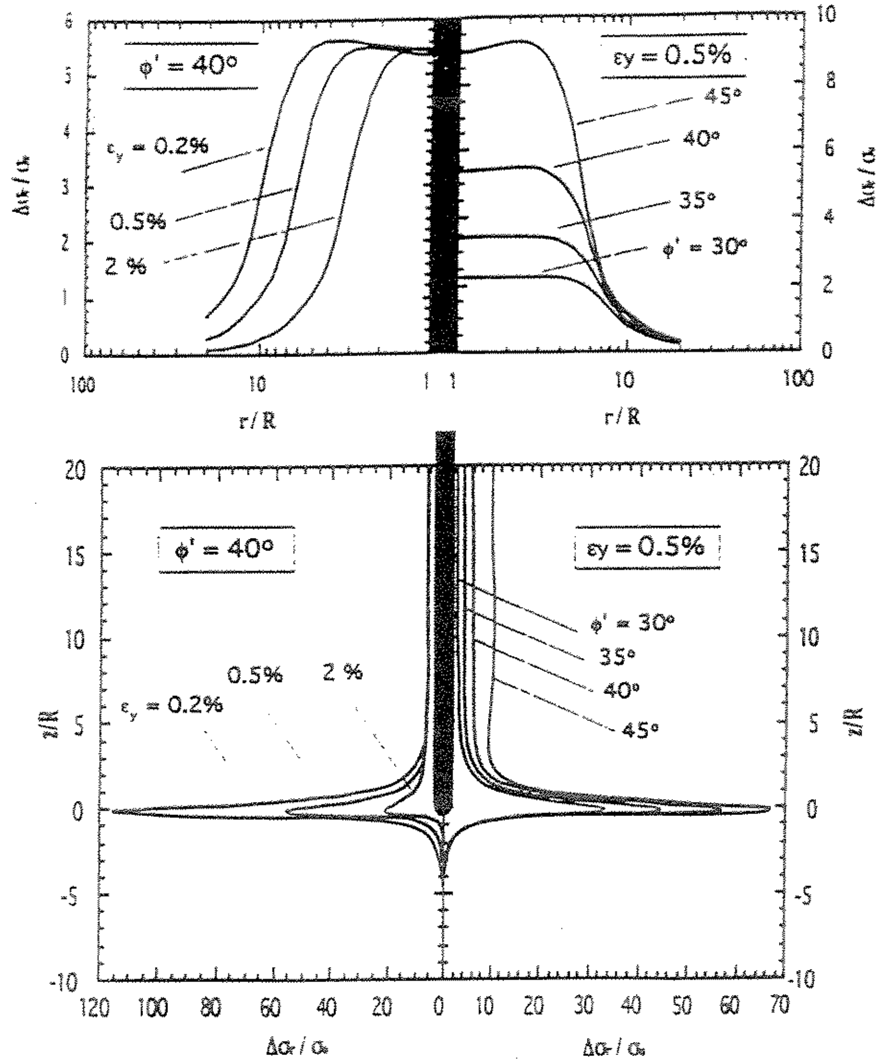
BC4 $\Delta\epsilon_v = 0$ $\quad \sigma_h = \text{const}$







EVALUATION OF STRAIN PATH PREDICTIONS OF CONE RESISTANCE IN SANDS



Effect of peak friction angle and yield strain on the changes in radial stress: a) radial distribution around the pile shaft, b) along the centerline and surface of the penetrometer. (Results not shown at meeting).
These results can help to understand the skin friction developed around pile in sands.

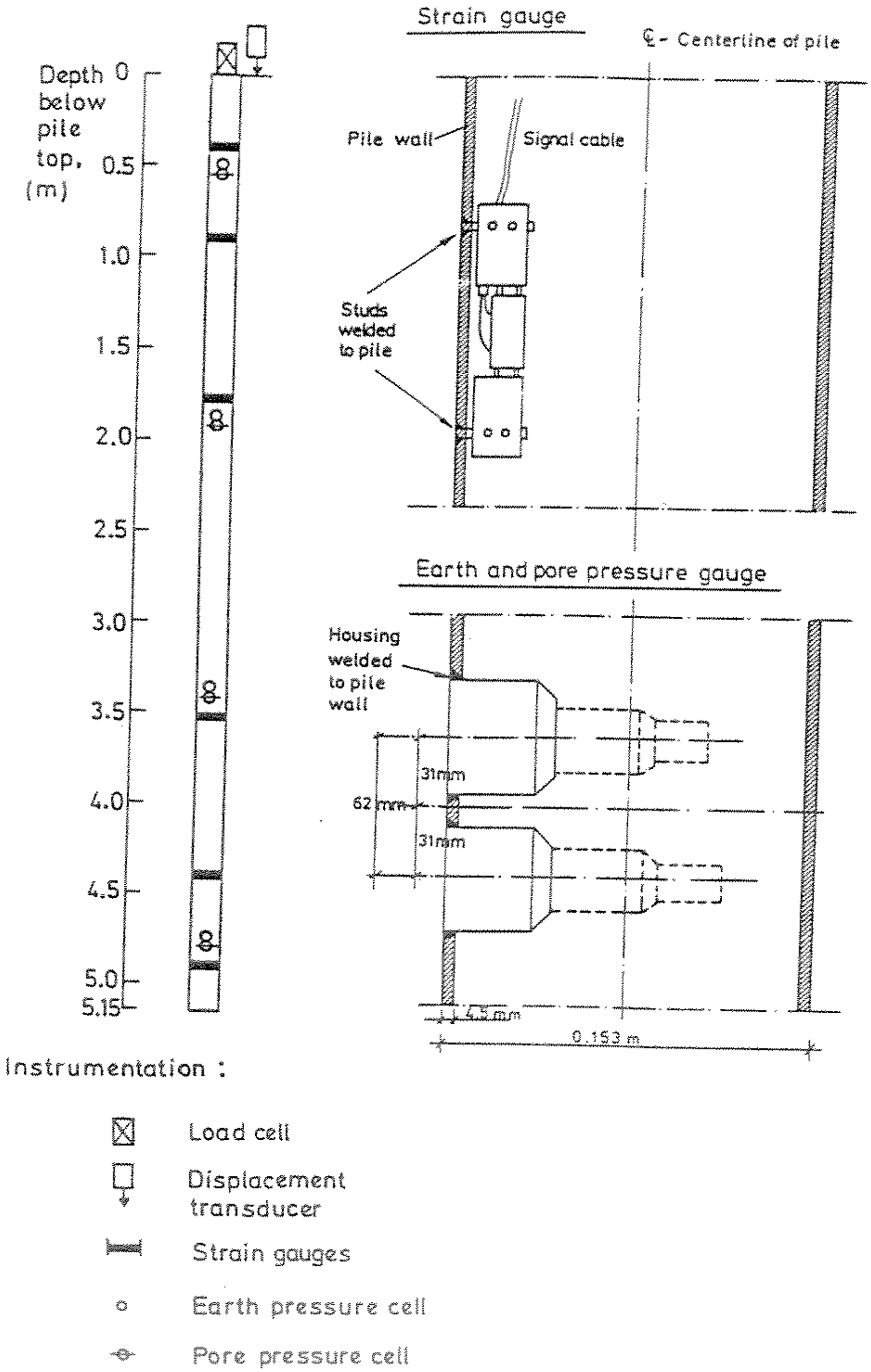
REVIEW OF RECENT RESEARCH EXPERIENCE WITH INSTRUMENTED MODEL DISPLACMENT PILES

Andrew J. Whittle
Department of Civil & Environmental Engineering
Massachusetts Institute of Technology

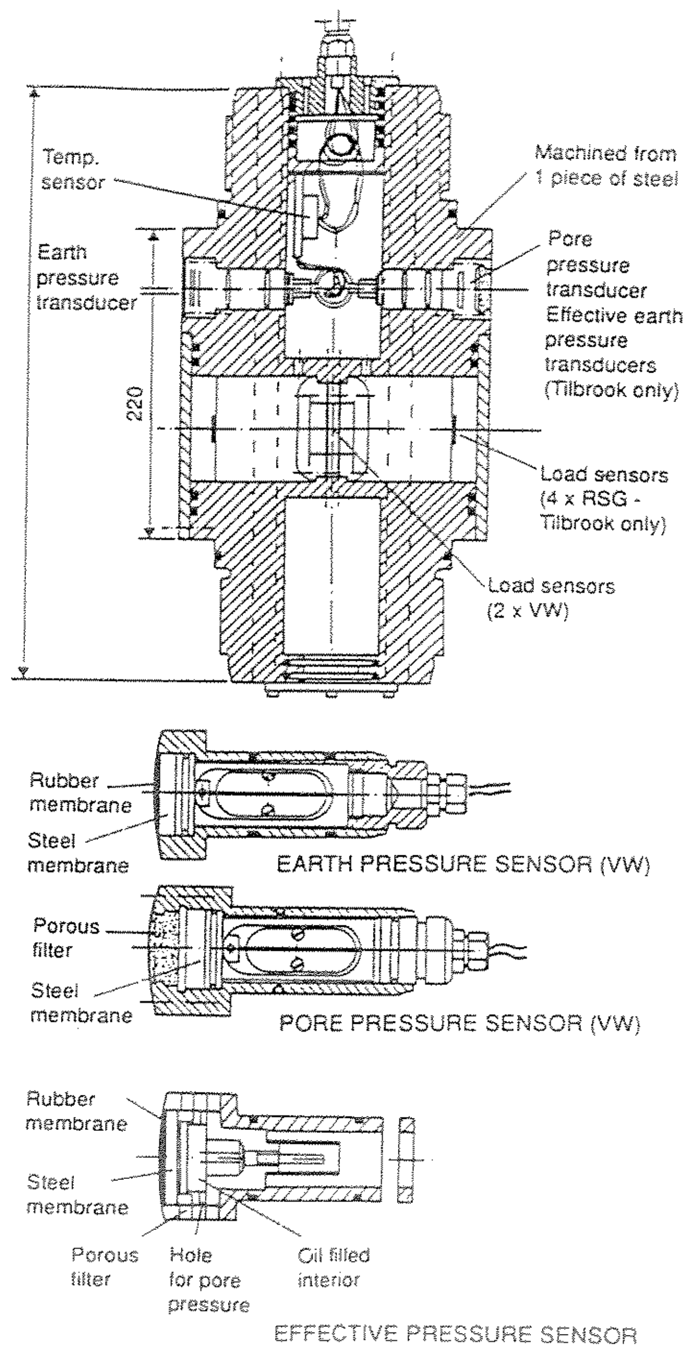
- INTRODUCTION
  - * field experiments:
    - - NGI, Imperial College 1982-1992
  - * disclaimer
- SCOPE OF STUDIES
  - * instrumentation of model piles
  - * classification of test sites
- MEASUREMENTS & INTERPRETATION
  - * Highly Overconsolidated Clays & Tills
  - * Normally & Lightly Overconsolidated Clays
  - * Silty Clays
  - * Sands

Soil Type	Site (soil)	Av. Index Properties		#	Geometry			Inst. Method	Consol. Time	Load History			Comments
		I_p (%)	I_L (%)	# Piles	# Instr. Levels	Length L (m)	Dia. d (m)	($\Delta z/R$)	Δt (days)	Static t_f (hrs)	Cyclic t_c (secs)	Q_u/Q_{us} (2-way)	
HIGH OC	Canons' Park (London Clay)	38	<0	4 (2)	3	6	0.1	Jack(4) (Driven)	>50	≈1	-	-	ICP 88-89 Negative installation pwp Residual shear surfaces
CLAYS	Tilbrook Grange Lowestoft Till & Oxford Clay	22 (37)	<0	4	3	10	0.22	Driven	60 (1)	≈1	10	85	NGI 89 @ LDP site comparison: pile diameter
& TILLS	Cowden Till	20	<0	4	3	6	0.1	Jack(5)	4-50	≈1	-	-	ICP 89-90 @BRE test site well defined clay matrix large variation in f_s
SOFT	Haga	15	0.9	31	3	6	0.1	Jack(3)	7-36	0.5	6	90-100 (0.45)	NGI 83-85 largest database (esp. cyc.) unreliable installation data
LOW OCR	Bothkennar	33	0.7	4	3	6	0.1	Jack(4)	≥4	≈1	-	-	ICP 90-91 @UK National Test Site excellent SI
CLAYS	Onsøy	40	0.85	4	3	10	0.22	Driven	30-50	<1	9	90 (35)	NGI 88 sensitive clay
SILTY	Lierstranda	15	0.8	4 (1)	3 (5)	10 (30)	0.22	Driven	30-50	<1	9	100 (35)	NGI 88 very low set-up stress
CLAYS	Pentre	15	50	2	3	6	0.22	Drop H.	31	≈1	10	90	NGI 87 @ LDP site part. drained installation? limited SI
SANDS (quartz)	LaBenne	D_r=50-100% d_{50}=0.15mm		2	3	6	0.1	Jack(5)	0.6 (<1min)	≈1	-	-	ICP 89 first study in sand effect of h/R on f_s

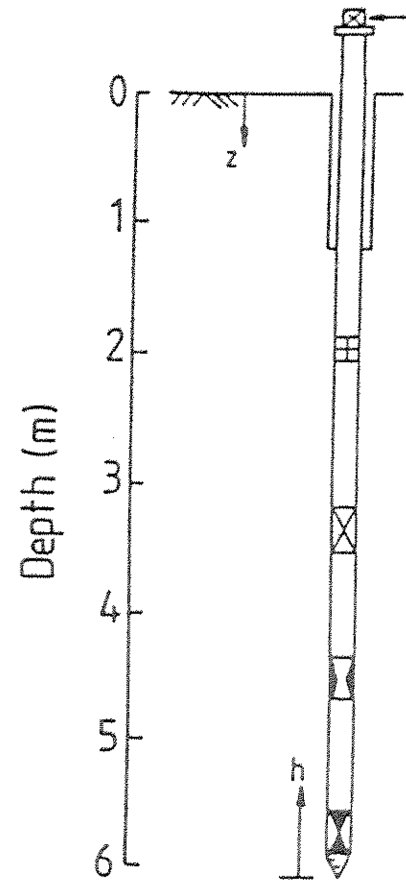
SUMMARY OF RECENT INSTRUMENTED PILE TESTS PERFORMED BY NGI & IMPERIAL COLLEGE



Instrumentation used in Haga Tests
(Karlsrud & Haugen, 1985)

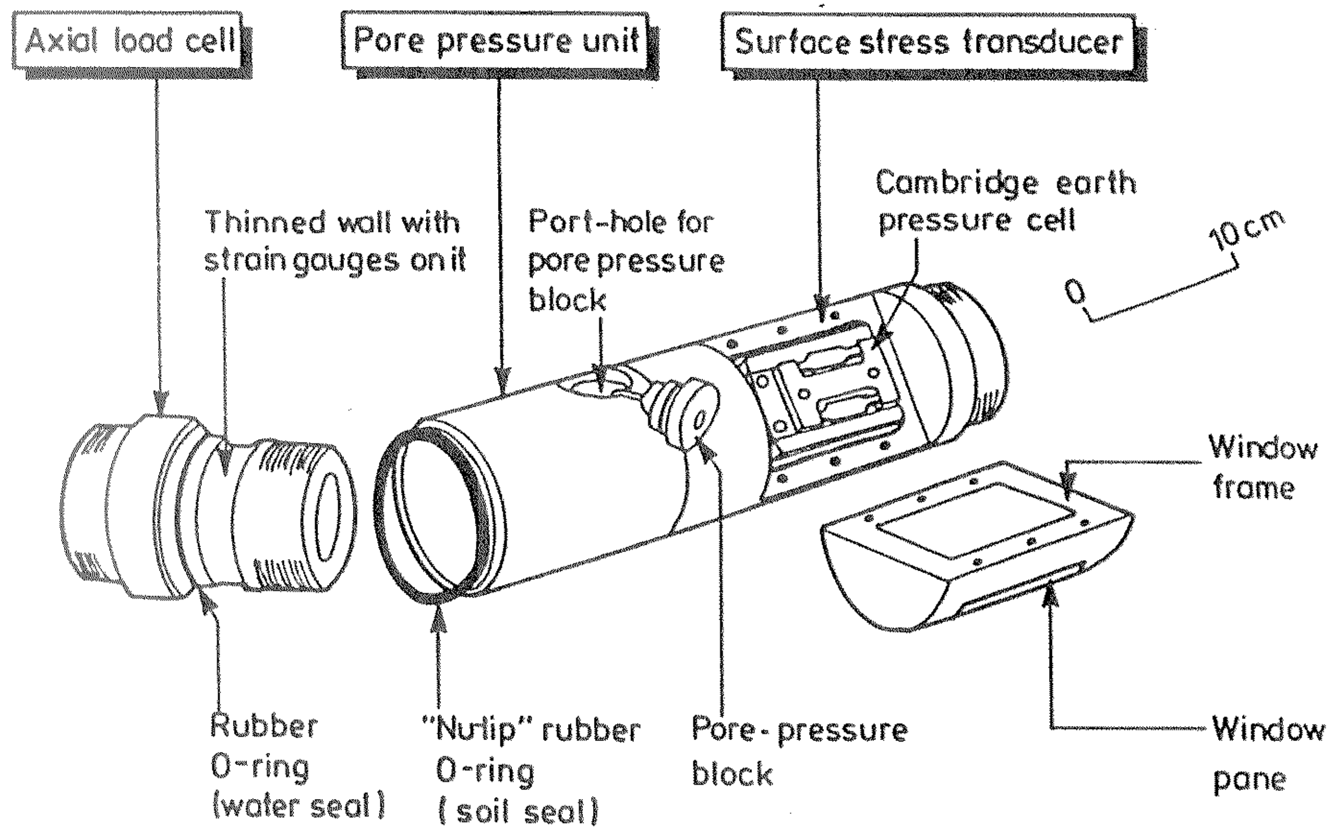


NGI Instrument Cluster
(circa. 1987)

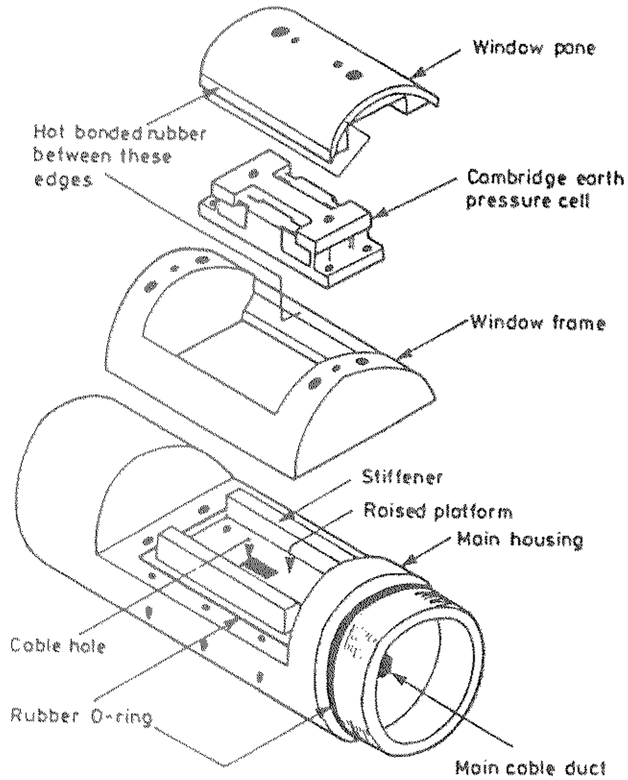


Instrument cluster	Sensor location (h/R)		
	σ_r, τ_{rz}	u	p
Leading	8	5	3
Following	28	30	32
Trailing	50	53	55
Lagging	-	-	77

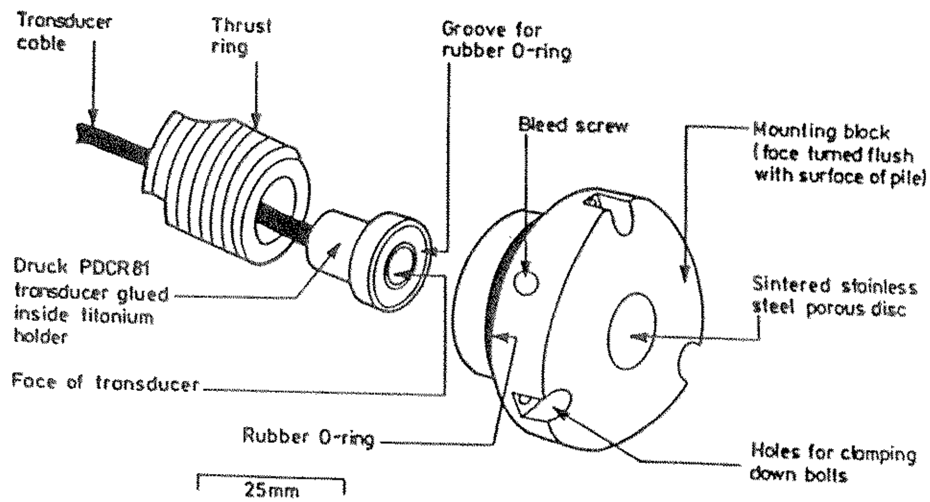
The Imperial College Pile (ICP)
(Bond, 1989)



Instrument Cluster for ICP
(Bond, 1989)



The surface stress transducer



Fast-acting pore pressure probe

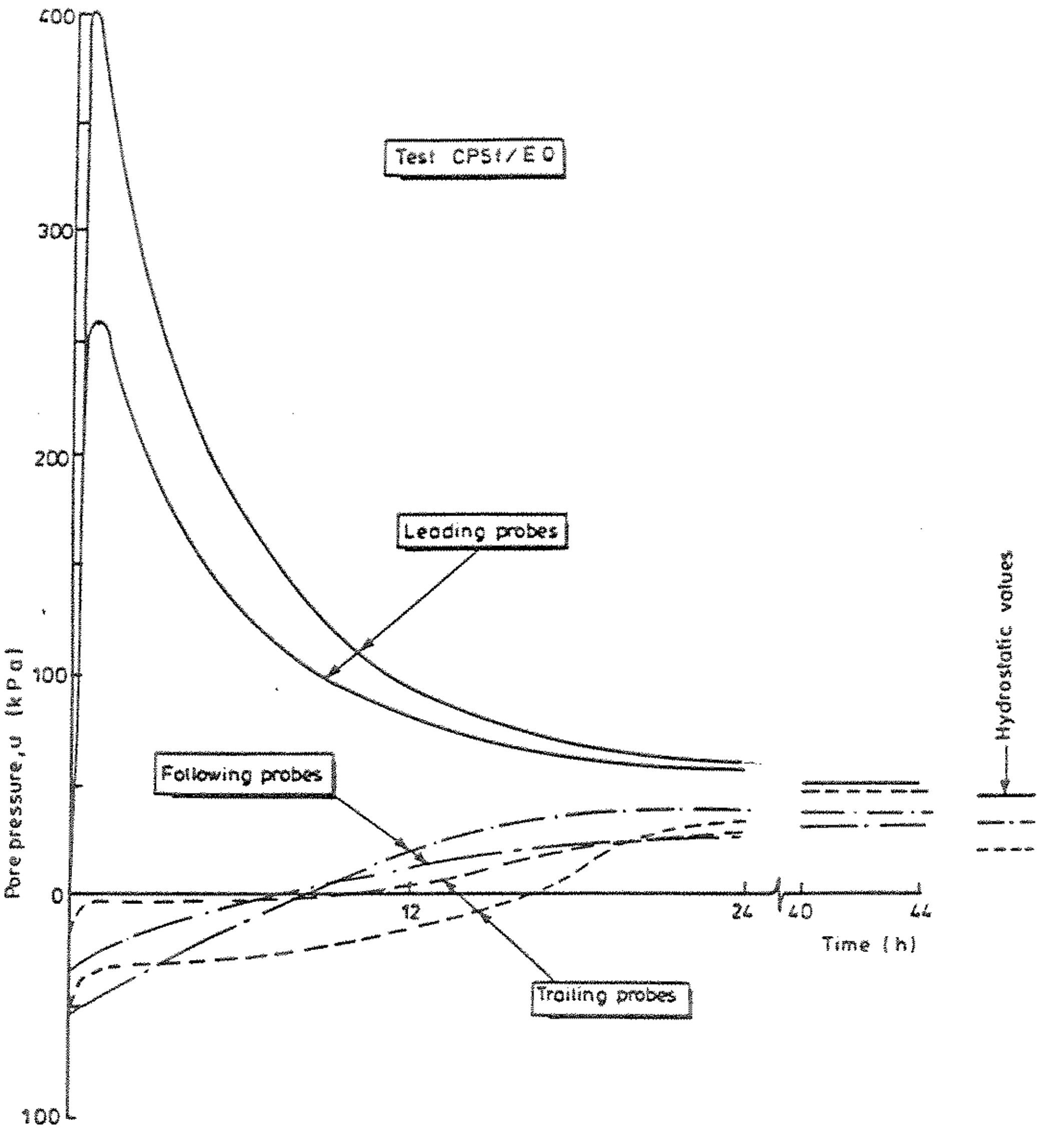
HIGHLY OVERCONSOLIDATED CLAYS & TILLS

• SUMMARY OF PRINCIPAL OBSERVATIONS

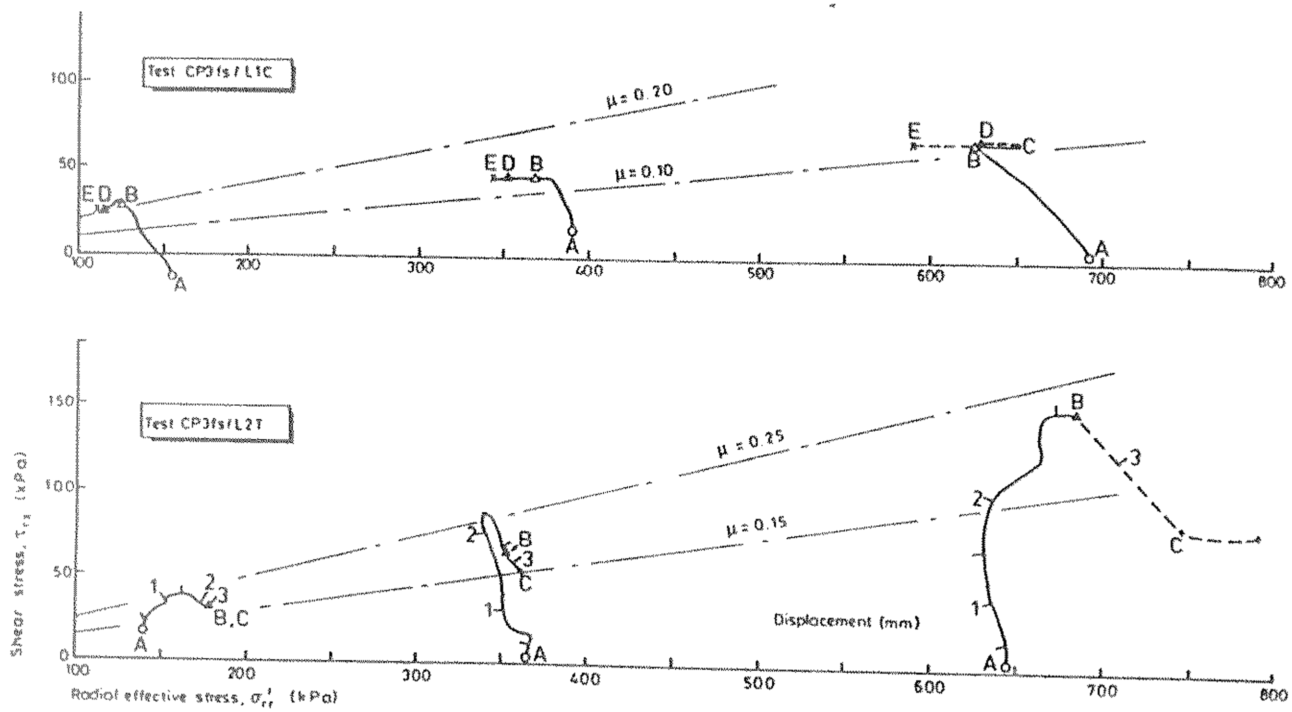
	Site		
Parameter:	Canons Park	Tilbrook Grange	Cowden
Av. s_u(UU) (kPa)	70 - 120	400 - 700	100 - 150
Av. s_u/σ'_{v0}	1.10	5.0 -> 1.3	5.0 -> 0.7
K_0	2.3 -> 1.6	3.0 -> 1.2	1.8 -> 0.8
Av. $\Delta u_i/\sigma'_{v0}$	< 0 along shaft >0 near tip	varies w/ position	complex pattern
Av. K_c	5.0 - 12.0	1.4 - 3.0	3.0 - 5.0
$\alpha = \bar{f}_s/s_u$(UU)	0.64 - 1.70	0.35 - 0.50	0.55 -0.60
$\beta = \bar{f}_s/\sigma'_{v0}$	1.0 - 3.0	1.0 - 1.2	1.4 - 2.6
$\rho = \bar{f}_s/\sigma'_{hc}$	0.20 - 0.26	0.35 - 0.70	0.25 - 0.33
$\delta = \tan^{-1}(\bar{f}_s/\sigma'_{hf})$	7^0 - 14^0	17^0 - 35^0	19^0 - 24^0
Comments	Very High K_c Very low δ (residual friction)	Large scatter in $\bar{f}_s$ damped out in simple correlations	Large scatter in $\bar{f}_s$

• ISSUES ARISING FROM TESTS
- Large scatter in soil properties (cf. soft clays)
- Important effects of soil fabric (e.g., residual shear surfaces)
- Measurement of large negative pore pressures
  (cavitation of instruments)
- Importance of residual stresses afer installation
- No existing predictive capability
- (practical importance?)

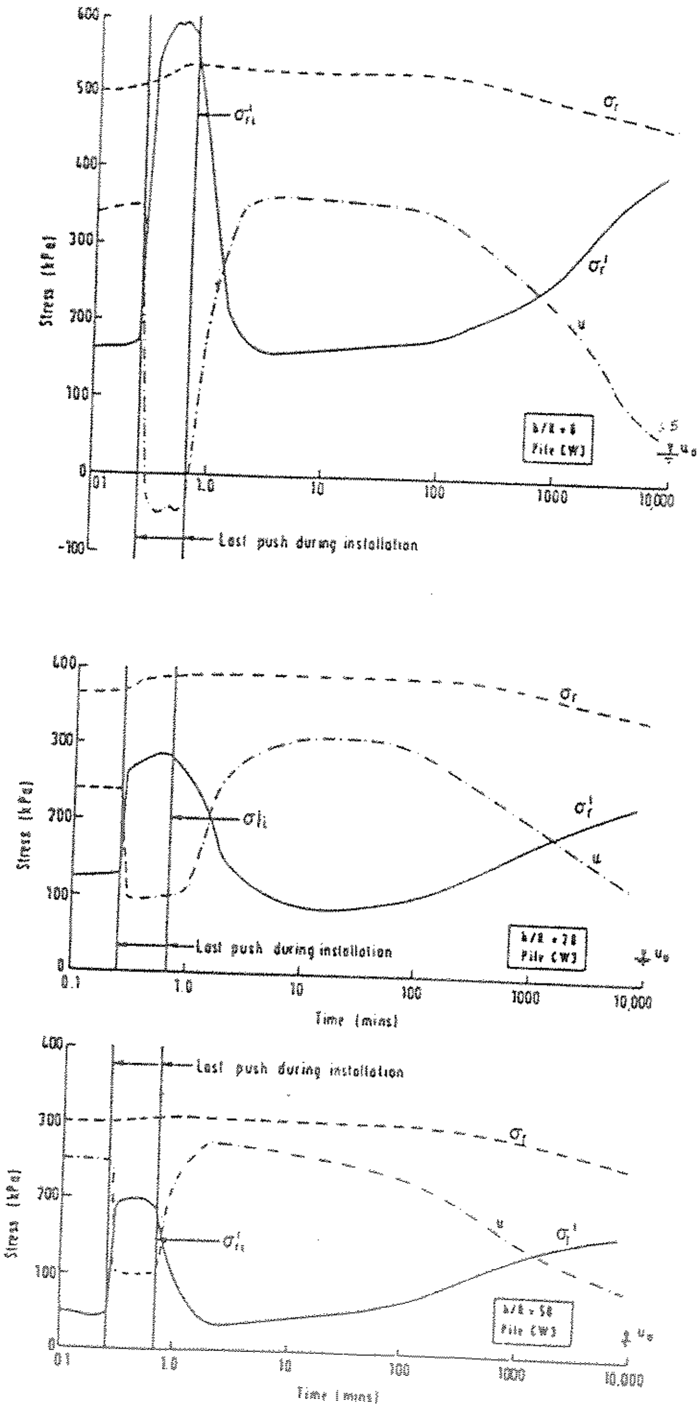
• POSSIBLE RESEARCH DIRECTIONS
- Artificial high OCR clays with $I_L<0$
  (Goal: Repeatable properties)
- Controlled laboratory element and reduced scale tests
- Development of modelling capability



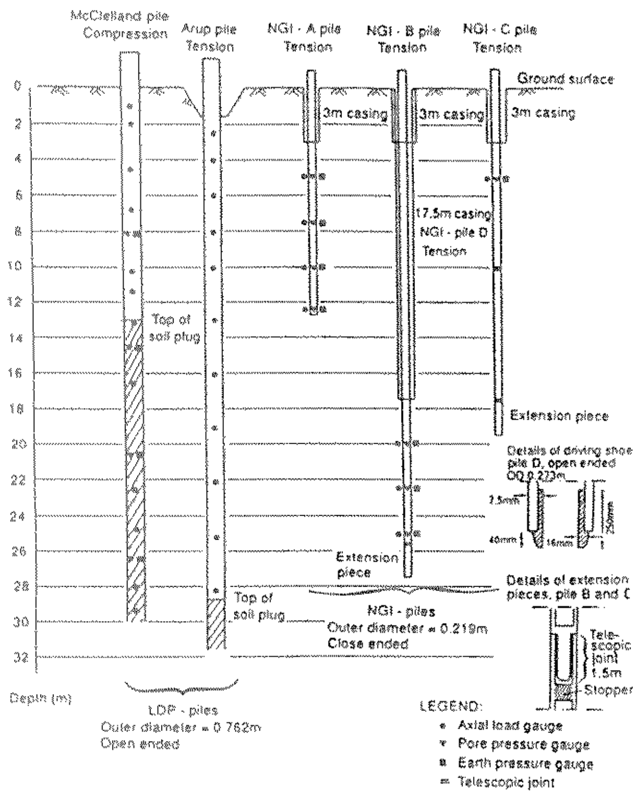
Pore Pressure Measurements during Equilibration
London Clay (Bond, 1989)

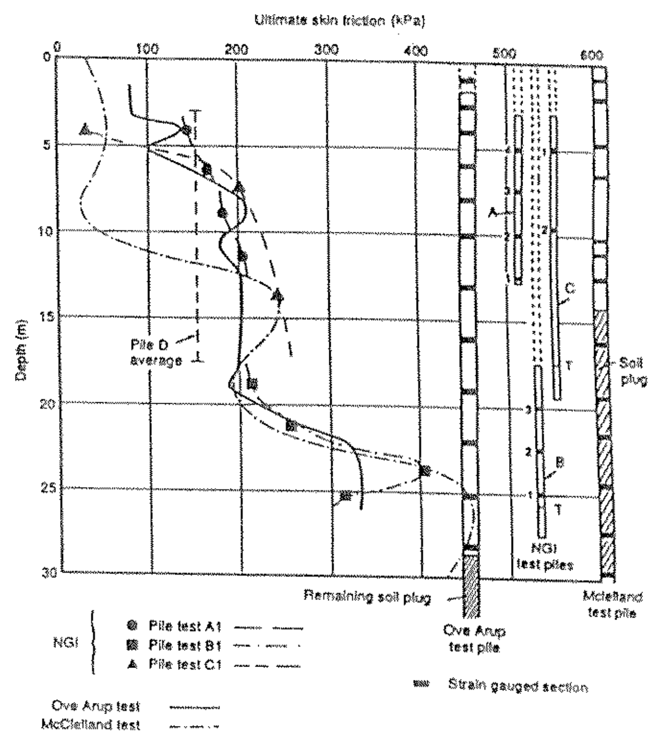


Effective Stress Paths in Pile Loading
London Clay (Bond, 1989)

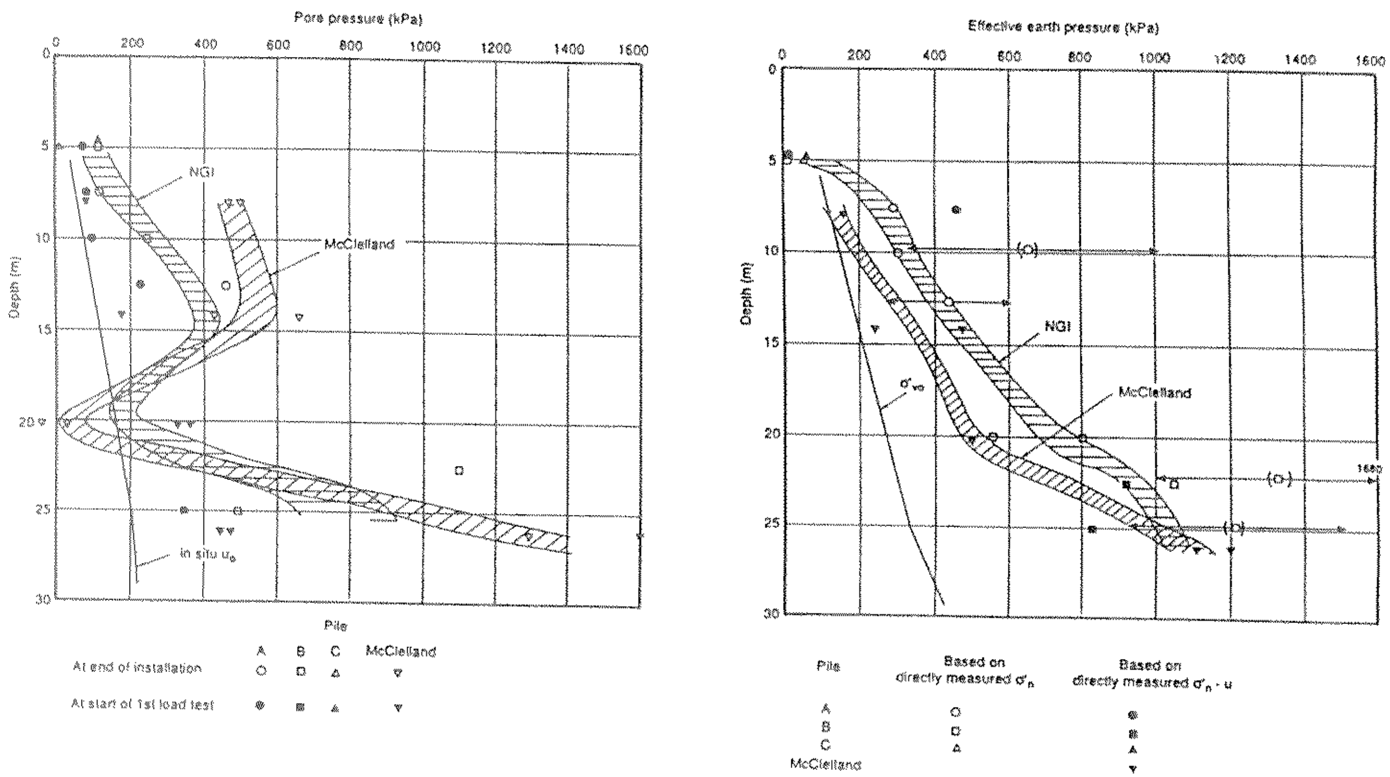


Set-Up Measurements in Cowden Till
(Lehane, 1992)





Comparison of Pile Tests: Tilbrook Grange
(Nowacki, Karlsrud, & Sparrevik, 1992)



Comparison of Pile Tests at Tilbrook Grange
Installation Pore Pressures & K_c
(Nowacki, Karlsrud, & Sparrevik, 1992)

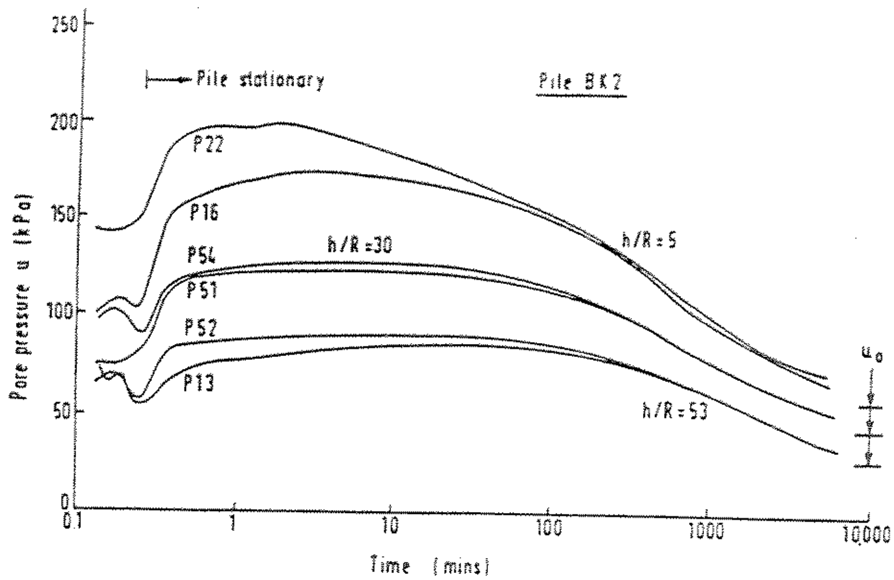
SOFT CLAYS
(Normally & Lightly Overconsolidated)

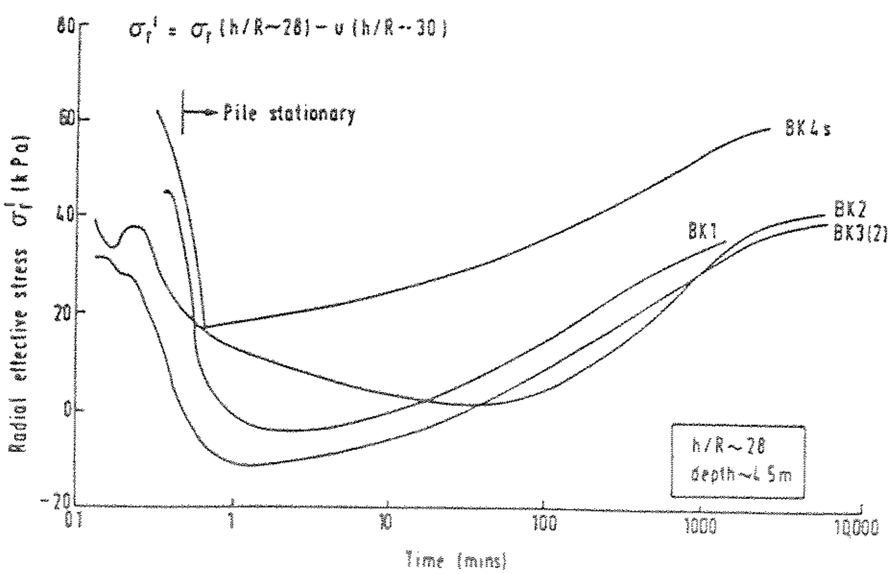
• SUMMARY OF PRINCIPAL OBSERVATIONS

	Site		
Parameter:	Haga	Bothkennar	Onsøy
OCR	7.0 -> 3.5	2.0 -> 1.3	1.3
Av. $s_u(TC)/\sigma'_{v0}$	0.7 - 1.2	0.65 - 0.70	0.35 - 0.50
Av. $s_u(DSS)/\sigma'_{v0}$	0.55 - 0.6	0.45 - 0.55	0.25 - 0.29
K_0	1.10 -> 0.85	0.65 -> 0.50	0.60
Av. $\Delta u_i/\sigma'_{v0}$	3.2 - 4.1	2.5 - 3.8	1.8 - 2.2
t_{90}(days)	4.0 (R=11cm)	4.5 (R=5cm)	20 (R=11cm)
Av. K_c	1.0 - 1.2	0.40 - 0.54	0.48 - 0.60
$\alpha = \bar{f}_s/s_u(DSS)$	0.64 - 1.70	0.80 - 1.15	0.55 -0.60
$\beta = \bar{f}_s/\sigma'_{v0}$	0.35 - 0.60	0.44 - 0.55	0.20 - 0.30
$\rho = \bar{f}_s/\sigma'_{hc}$	0.30 - 0.40	0.35 - 0.55	0.32 - 0.55
$\delta = \tan^{-1}(\bar{f}_s/\sigma'_{hf})$	$22^0 - 24^0$	$28^0 - 30^0$	-
Comments	inst. reliability in long term tests? residual stress -> f_s creep -> f_s increases	time delay in u_{max} effects of cementation?	OCR more variable than SI suggests

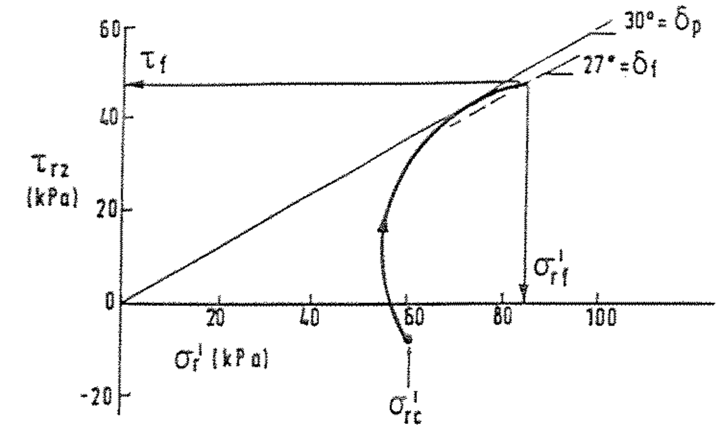
• ISSUES ARISING FROM TESTS
- Data conform to basic predictive framework (SPM/soil models)
- Importance of driving record on penetration pore pressures
- Long term performance & cyclic behavior: still very uncertain
- Mechanisms of shaft failure (3 working hypotheses)

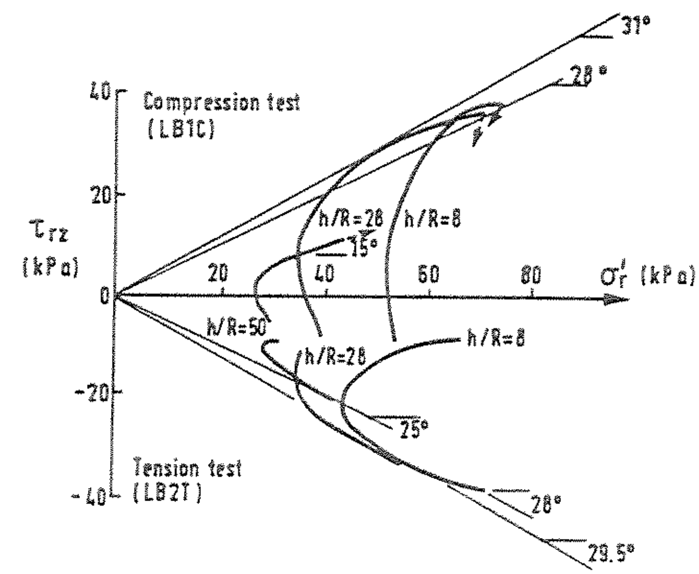
• POSSIBLE RESEARCH DIRECTIONS
- Generalization of factors affecting K_c
  (when is $K_c<K_0$)
- Improve predictive model for load transfer (length effects)
- Open-ended piles: - almost no available data
- Improvements for <u>Rate Effects</u> (cyclic and long term loads)





Set-Up Measurements at Bothkennar
(Lehane, 1992)





Effective Stress Paths during Axial Loading
(Labenne Sand; Lehane, 1992)

SILTY CLAYS

• SUMMARY OF PRINCIPAL OBSERVATIONS

	Site	
Parameter:	Lierstranda	Pentre
OCR	2.5 -> 1.0	2.0 -> 1.3
Clay Fraction (%)	40 -> 20	10 - 25
Av. $s_u(DSS)/\sigma'_{v0}$	0.21 - 0.30*	0.28 - 0.30*
K_0	0.7 -> 0.45	0.65 -> 0.50
Av. $\Delta u_i/\sigma'_{v0}$	2.4 -> 1.2	2.9 -> 0.7
t_{90}(days)	4.0	<0.25
Av. K_c	0.40 -> 0.12	0.40 -> 0.17
$\alpha = \bar{f}_s/s_u(DSS)$	0.20 - 0.50	0.45- 0.80
$\beta = \bar{f}_s/\sigma'_{v0}$	0.05 - 0.15	0.13 - 0.25
$\rho = \bar{f}_s/\sigma'_{hc}$	0.33 - 0.39	0.63 - 0.79
$\delta = \tan^{-1}(\bar{f}_s/\sigma'_{hf})$	-	37^0
Comments	inst. reliability in long term tests? residual stress -> f_s creep -> f_s increases	sand layers: high c_h ime delay in u_{max} effects of cementation?

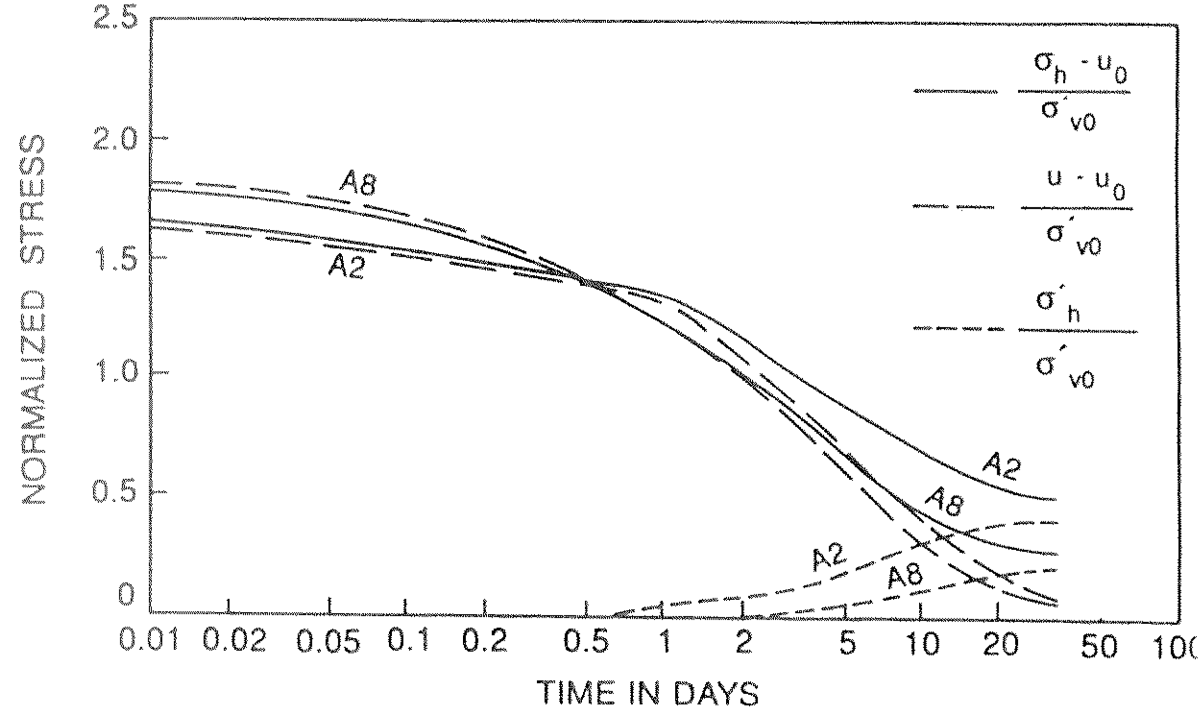
* undrained shear shows significant dilation

• ISSUES ARISING FROM TESTS

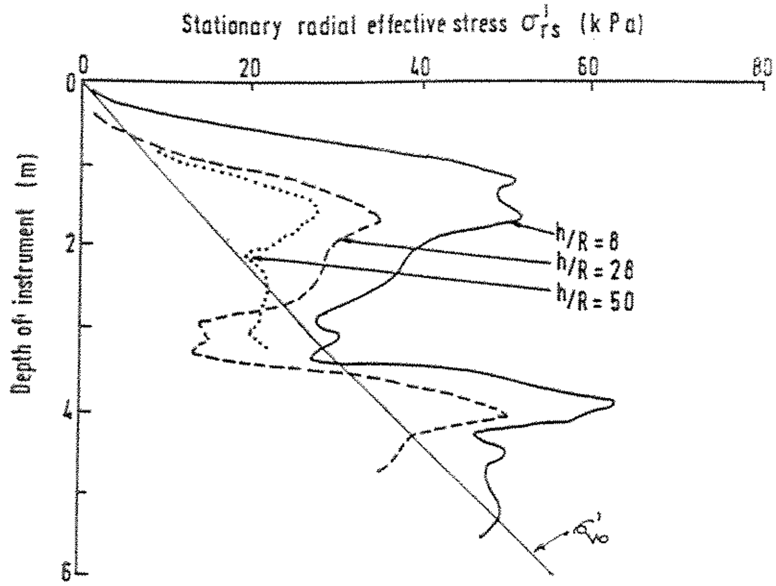
- Very low set-up stresses; major practical problem
- No a priori prediction of installation/set-up behavior
- No reliable characterization of compressibility, dilative behavior

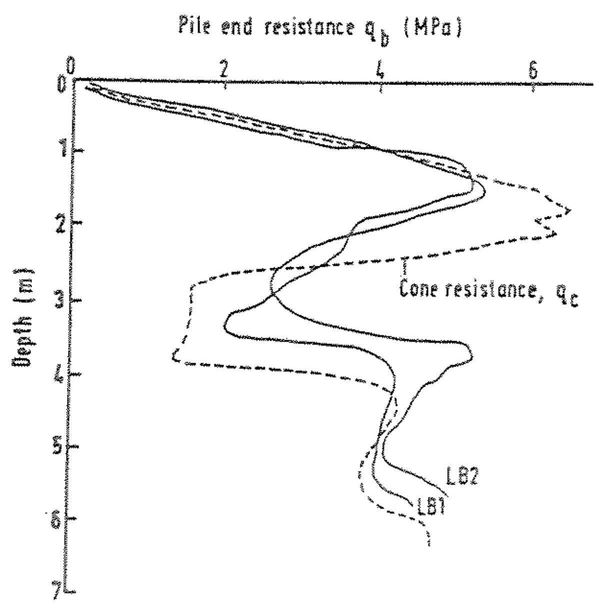
• POSSIBLE RESEARCH DIRECTIONS

- Identification of parameters controlling performance
- Model: Effects of partial drainage on penetration
- Model: Constitutive behavior of silts
- Further experiments needed at well documented sites

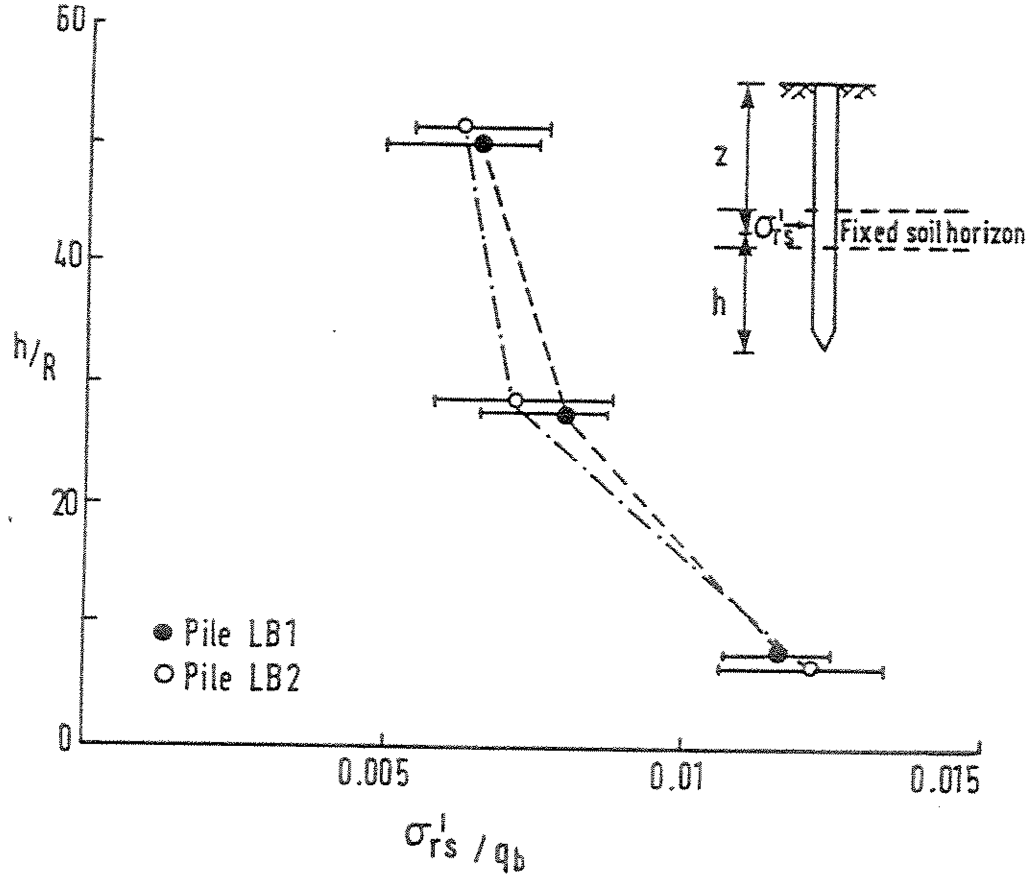


Set Up Measurements at Onsøy & Lierstranda
(Karlsrud, Kalsnes & Nowacki, 1992)

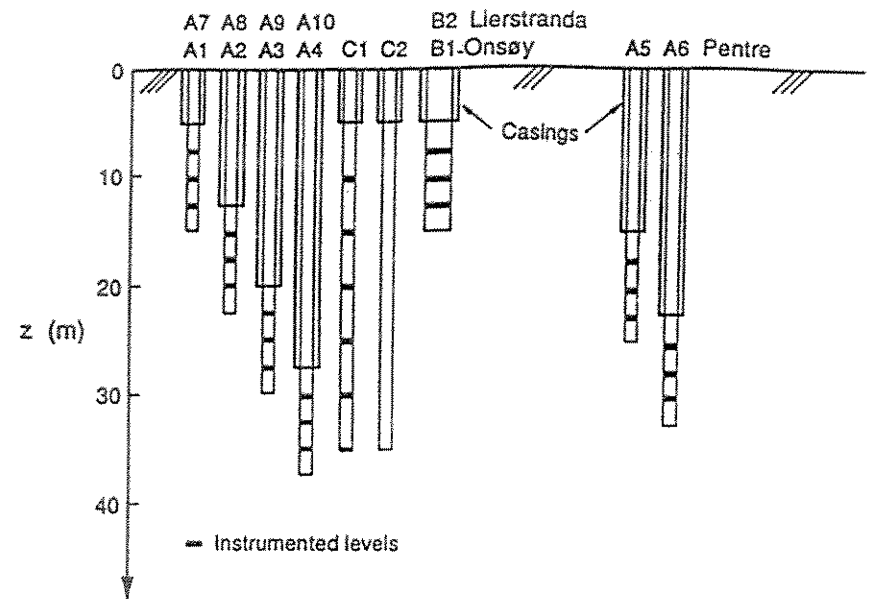


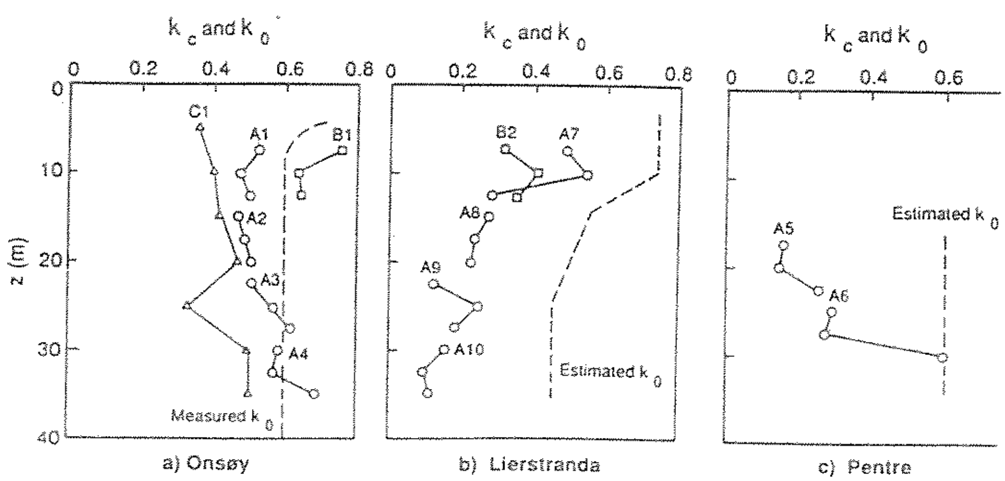


Radial Effective Stress and Tip Resistance
(ICP test in Labenne Sand, Lehane, 1992)

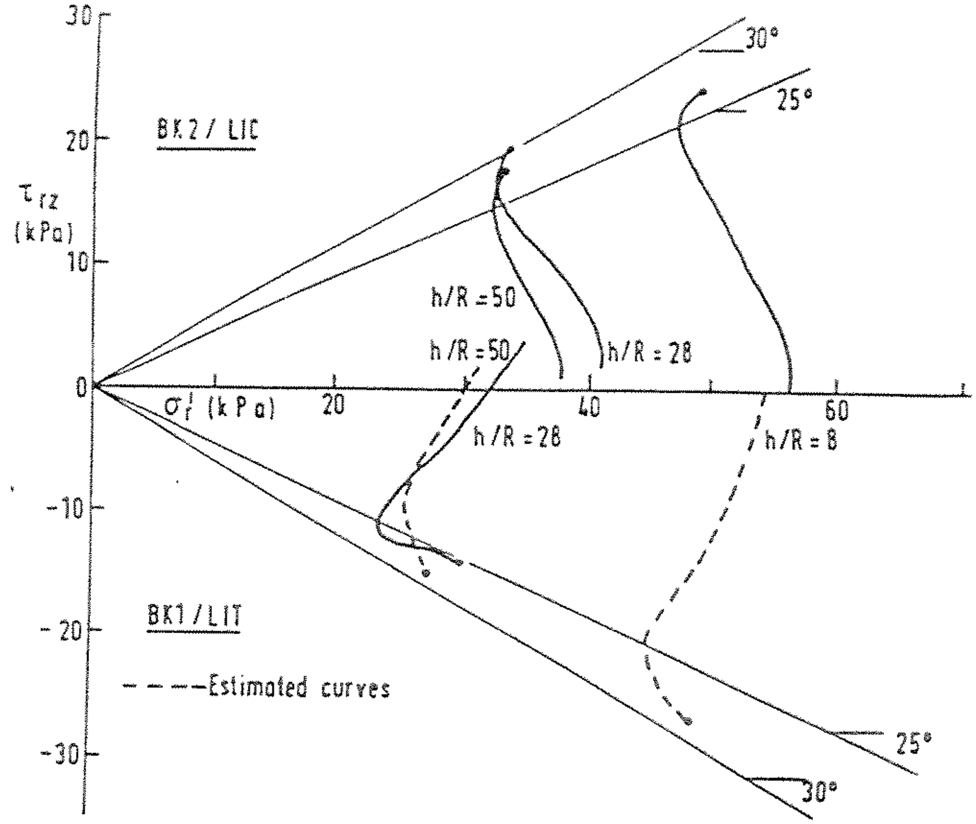


Effect of Pile Tip on Lateral Stress
(Labenne Sand; Lehane, 1992)





K_C at Three Sites
(Karlsrud, Kalsnes & Nowacki, 1992)



Effective Stress Paths during Axial Loading
(Bothkennar Clay: Lehane, 1992)

U.S. Army's Civil Engineering Centrifuge

R. H. Ledbetter (USAEWES)

The U.S. Army Engineer Waterways Experiment Station (WES) is developing a CE/Army unique centrifuge capable of application in most military and civil works problem areas. Scheduled to be operational in 1995 at WES, it will be the largest and most diverse in the world.

The centrifuge will keep the CE at the state of the art, particularly in physical model testing, and facilitate a very economical alternative to field tests. Large 1g models can be built and tested but are expensive; prototype predictions from 1g model results are very difficult if the materials are time, stress, and/or strain state dependent. The primary advantage of a centrifuge is that the model stresses and strains can be maintained equal to those in the prototype and translation of results to prototype conditions is much simpler and more reliable than in 1g models. Prototype, realistic, time dependent effects can also be achieved in a centrifuge. A centrifuge is an engineer's "time machine" to future events and behaviors. Events that take 10,000 days (27.5 years) can be completed in one day in a centrifuge at 100g's; 3,000 years can be completed in one week at 500g's. Force increases as the square (at 100g's, one pound of force on a model is equivalent to a load of 10,000 pounds on the full-sized object); energy increases as the cube (a gram of explosives at 100g's generates a force of a million grams, or a metric ton of explosives).

The centrifuge is applicable to problems in the engineering fields of geotechnical, structures, hydraulics, environmental, and coastal. Some of the important advantages/benefits of a centrifuge are: (1) prototype behaviors can be predicted for complicated problems for which calculation methods are not completely reliable or adequate; (2) numerical methods can be validated, improved, or developed based on realistic prototype behavior; and (3) centrifuge tests can be used for parameter studies to investigate particular properties and behaviors in relation to variations in loads and other boundary conditions. Centrifuge testing will allow: (1) the economical proof testing of designs, investigation of problem areas, and validation of numerical methods that have been prohibitively expensive to study with prototype testing; (2) evaluation and verification of analyses which have never been verified against significant truth data of behavior; and (3) the study of important behavior phenomena, including failure, that is impractical or not feasible in prototype situations or for complex problems.

Specific geotechnical problem areas involve all earth works which can be modeled at a fraction of the cost of large-scale or prototype structure tests and include such as earth and rockfill dams, levees, appurtenant structures, walls, foundations, tunnels and excavations subjected to a wide range of loadings (including earthquake and vibratory) and parameter variations.

Key Specifications:

Radius of centrifuge arm: 6.5 meters (21.3 ft)
g range: 10 to 350g
Maximum payload @ 143g: 8000kg (8.8 tons)
Maximum payload @ 350g: 2000kg (2.2 tons)
Maximum g force: 1,144,000 g-kg (1256 g-tons)

Large-Scale Laboratory Stress Chamber System for Research in Geotechnical Engineering

Potential Uses:

- Provide a cost-effective means for developing, assessing, and comparing equipment, interpretative guidelines, and procedures for geotechnical site investigations
- Conduct validation tests on in situ testing and soil sampling devices
- Investigate soil-structure interaction problems such as earth anchors or piles and pile groups
- Assess the behavior of earth-rock mixtures and expansive or collapsible soils
- Evaluate cementing and aging effects of soils
- Validate numerical models / equipment for geoenvironmental investigations
- Develop correlations for complex soil-structure interaction problems that can not be modeled by numerical syntheses due to the discontinuity of behavioral modes, such as from elastic behavior to consolidation behavior or from consolidation behavior to shear failure

Unique Features of the Large Stress Chamber System:

- 1.5 m (5 ft) diameter soil specimen
- Stacked cylinder modules (specimen height can be varied by 0.3 m [1 ft] increments to a maximum height of 1.8 m [6 ft])
- Two hydraulic systems are used for loading the soil specimen
- Maximum vertical stress is 1.5 MPa (225 psi)
- Maximum lateral stress is 1.0 MPa (150 psi)
- Vertical stress is applied through rigid end platens
- Chamber is designed for use with a cylindrical triaxial membrane or with torroidal bladders; torroidal bladders permit testing of layered soils and/or variation of lateral stress with depth
- Soil specimens can be saturated by back pressure
- Hydraulic (back pressure) gradients can be applied across the soil specimen in a vertical or horizontal mode
- Instrumentation and data acquisition systems permit computer feedback to control horizontal and vertical stresses and/or strains (deformation) within the soil specimen
- Instrumentation can be placed within the soil specimen to monitor the internal (soil-structure interaction) response of the soil specimen due to the applied loading conditions

Projects (Ongoing / Completed):

- New / improved methods for accessing the penetration resistance of silty soils (CWR&D)
- Validation tests of a new SCAPS penetrometer (AEC)

Design and Fabrication Costs:

- Cost plus fixed fee contract for \$538.4^{k}

Point-of-Contact:

Richard W. Peterson
Office: (601) 634-3737
FAX: (601) 634-4656

Topics Concerning Pile Driving Equipment and Pile-Soil Interaction

Don C. Warrington, P.E.

Vulcan Iron Works Inc.

Deep Foundations Institute

Impact Hammers

- Dynamic Formulae Gave Inadequate Soil and System Modeling, but established hammers as a measuring device
- Wave Equation Advanced System Modeling

Vibratory Hammers

- Interaction between pile and soil different between impact and vibratory hammers
- Changes in soil resistance during vibration make capacity prediction difficult
- Changes presently not well quantified

Hammer Independent Methods

- Methods exist to determine pile/soil capacity without reference to hammer
- Pile capacity may vary with method of installation

Recommendations for Impact Hammer Research

- Present wave equation soil models need reexamination
- Automate wave equation soil property input
- Make impact hammer certification more flexible
- Eliminate hammer as measuring device

Recommendations for Vibratory Hammer Research

- Quantify and qualify a comprehensive model or models of soil response to vibration, and the relationship of soil resistance during and after driving
- Encourage Development of pile capacity methods that are equipment independent

Current & Projected Research at the University of Arizona

William M. Isenhower

Projects at Arizona

- *Non-orthogonal Bending Stiffness of Deep Foundations Under Combined Axial and Lateral Loading* - USACE WES ITL
- *Reliability of Pile-supported Navigation Structures* - USACE WES ITL
- *Laboratory Study of Shaft Friction in Sand* - National Science Foundation

Non-orthogonal Bending Stiffness of Deep Foundations

Typical Conditions Affecting *EI*

- Off-axis bending
- Out-of-place reinforcement
- Non-symmetric cross-sections
- Confined concrete
- Prestressed concrete

Analytical Procedure

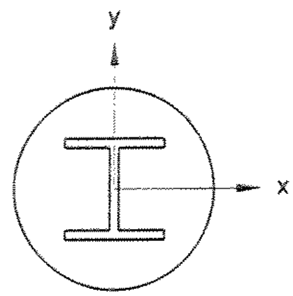
- Calculate geometric properties for angle of loading
- Specify curvature and axial thrust loading
- Assume a position of the neutral axis
- Calculate strain as function of curvature
- Calculate stresses as function of strains
- Integrate stresses to obtain axial thrust, adjust position of neutral axis until convergence is achieved

Analytical Procedure (cont.)

- Calculate bending moment by integrating product of stress times offset from centroid
- Calculate bending stiffness using

$$EI = \frac{M}{\kappa}$$

As Designed



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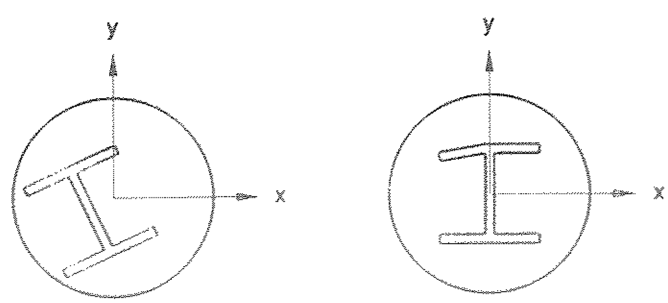
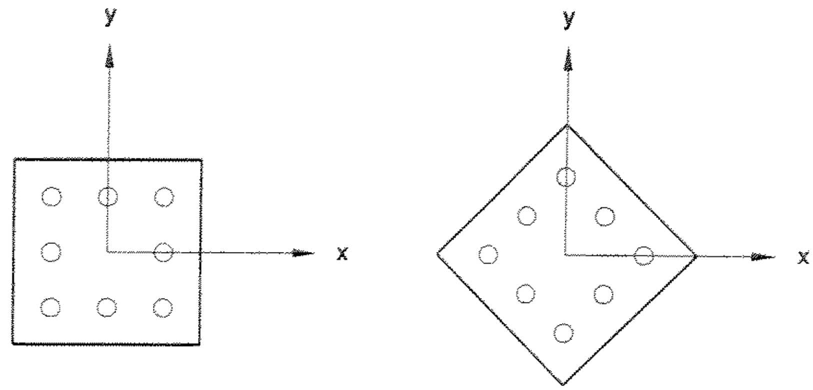


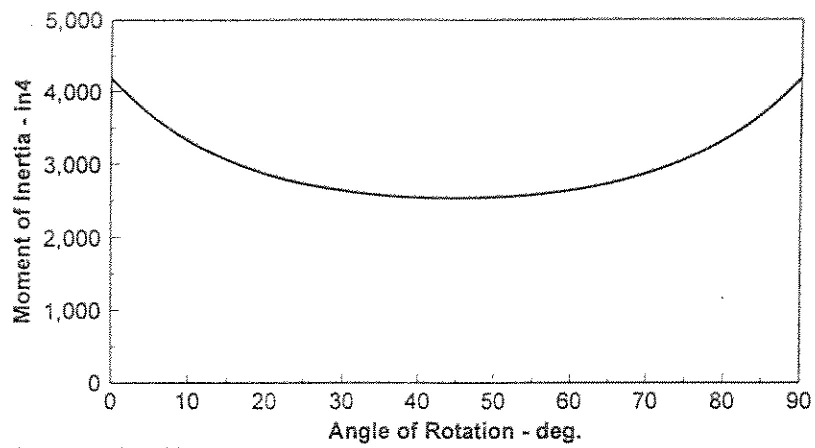
Off-axis Bending



Moment of Inertia Transformation for Elastic Sections

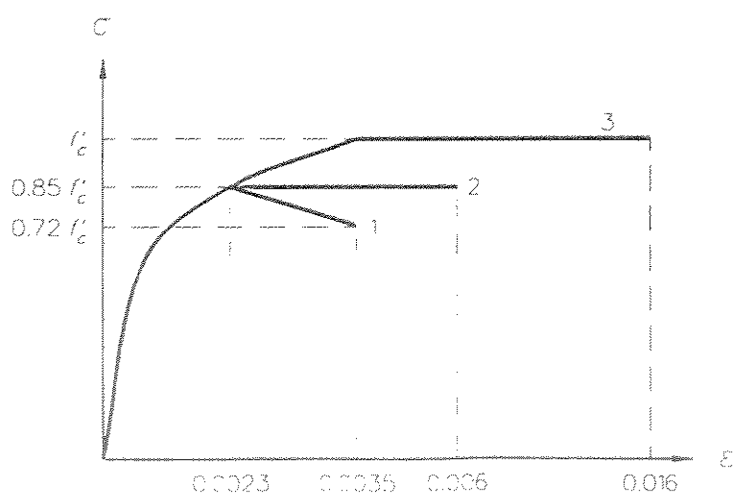
$$I = \frac{I_x + I_y}{2} + \frac{I_x - I_y}{2}\cos 2\theta - I_{xy}\sin 2\theta$$

Variation of *I* with Rotation of Square Concrete Pile

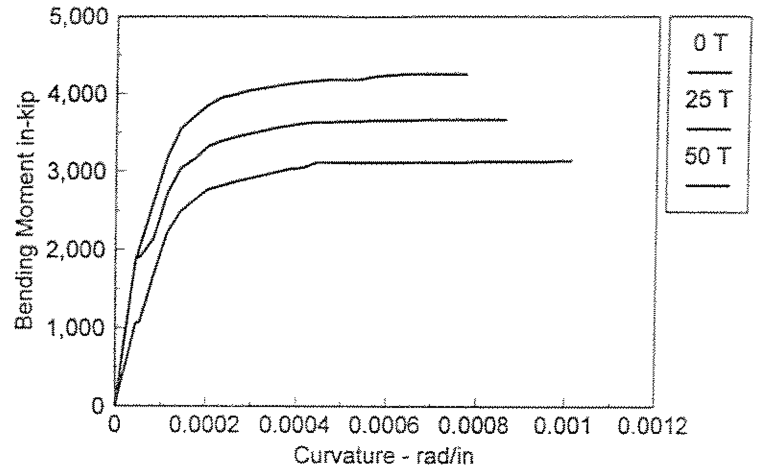


Curvature = 0.00005 rad/in

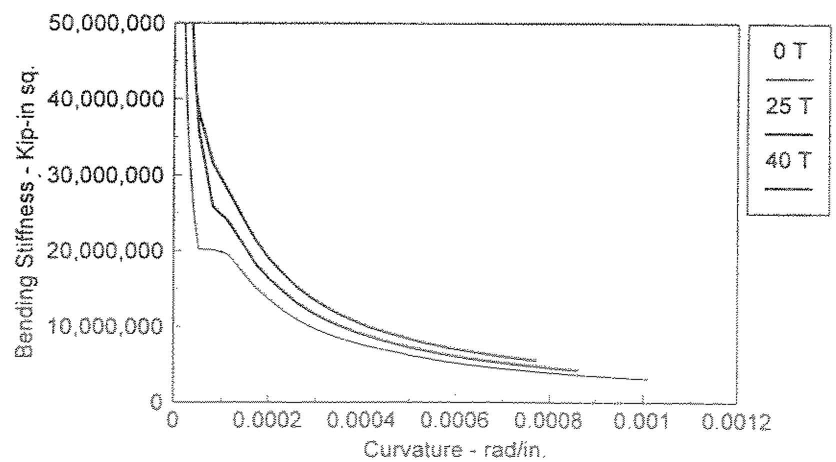
Effects of Confinement on Strength and Ductility of Concrete



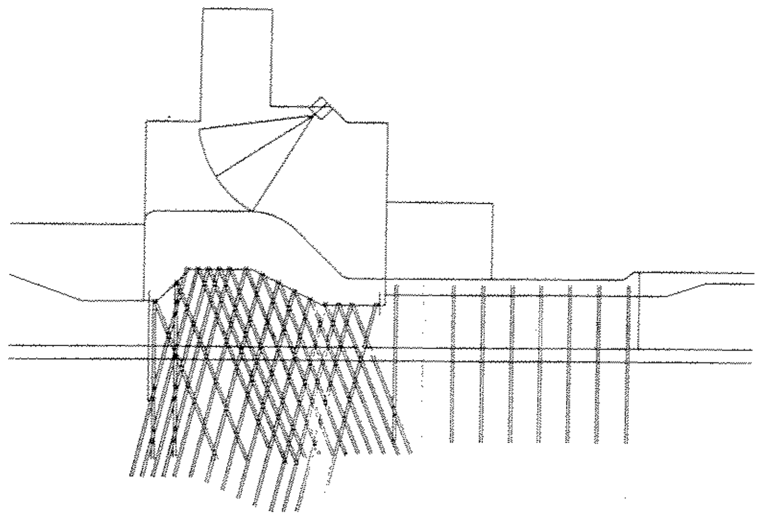
Curvature Effects on *EI*
Moment-Curvature



Effects of Curvature on *EI*
EI-Curvature



Reliability of Pile Supported Navigation Structures



Reliability analyses are used for:

- Assessing the reliability of structures based on current conditions
- Identifying sources of variance in structural performance
- Evaluating relative performance of dissimilar structural elements
- Providing a consistent method for prioritizing rehabilitation expenditures for dissimilar structures

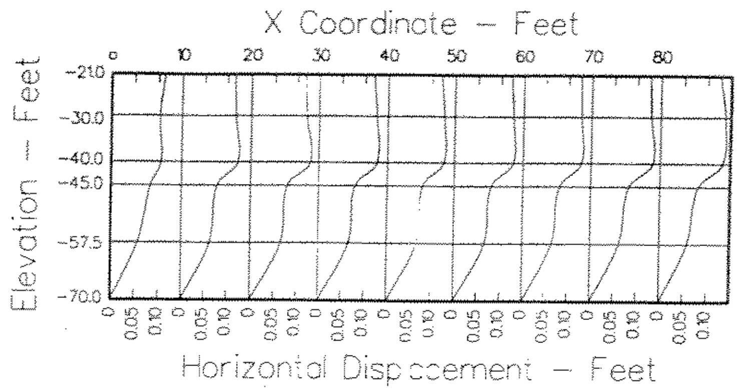
Procedure for Reliability Analysis

- Identify an initiating event
- Collect data for soil conditions and strength
- Evaluate means, standard deviations, and covariances of variables
- Perform seepage analysis to evaluate pore water pressures under structure
- Locate potential sliding surface

Reliability Analysis (cont.)

- Analyze soil displacements using FEM analysis including loading history and seepage

Soil Displacement under Structure from Finite Element Analyses



Reliability Analysis (cont.)

- Analyze soil movements using FEM analysis including loading history and seepage
- Perform soil structure interaction analysis

Reliability Analysis (cont.)

- Analyze soil movements using FEM analysis including loading history and seepage
- Perform soil structure interaction analysis
- Analyze variance of a performance factor i.e. bending moment or deformation of a particular pile

First Order Second Moment Analysis of Variance

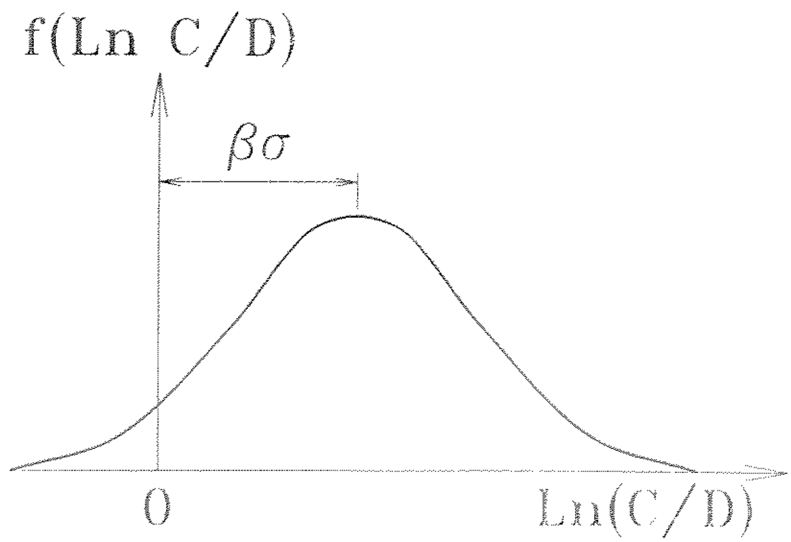
$$[F(x)] = \sum_{i=1}^{n} \left(\frac{\partial F}{\partial x_i} \right)^2 V(x_i)$$

$$+ 2 \sum_{i=1}^{n-1} \sum_{i+1}^{n} \left(\frac{\partial F}{\partial x_i} \right) \left(\frac{\partial F}{\partial x_j} \right) \rho_{ij}$$

Reliability Analysis (cont.)

- Analyze soil movements using FEM analysis including loading history and seepage
- Perform soil structure interaction analysis
- Analyze variance of a performance factor i.e. bending moment or deformation
- Compute the reliability index β

Definition of Reliability Index for Log-normal Distribution



Reliability Index for Log-normal Distribution

$$\beta = \frac{\ln\left|\dfrac{E[C/D]}{\sqrt{1+V_{C/D}^2}}\right|}{\sqrt{\ln\left|1+V_{C/D}^2\right|}}$$

Target Reliability Indices

Performance	β
High	5
Good	4
Above Average	3
Below Average	2.5
Poor	2.0
Unsatisfactory	1.5
Hazardous	1.0

Reliability Analysis (cont.)

- Analyze soil movements using FEM analysis including loading history and seepage
- Perform soil structure interaction analysis
- Analyze variance of a performance factor i.e. bending moment or deformation
- Compute β
- Repeat analysis for other performance factors or different initiating events

Laboratory Study of Shaft Friction in Sand

Objective

The objective of this study is to investigate changes in the three-dimensional state of stress in sand near a shaft as a function of loading.

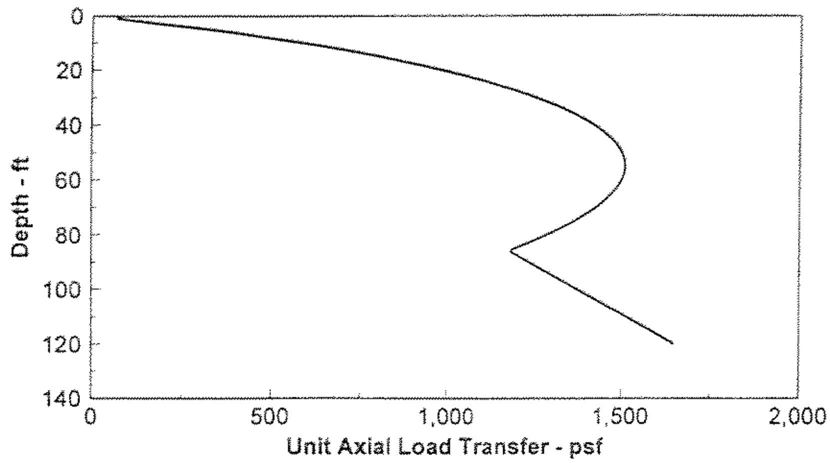
FHWA Design Equations for Drilled Shafts in Sand

$$Q_s = \pi B \sum_{i=1}^{N} \gamma_i' z_i \beta_i \Delta z_i$$

$$\beta_i = 1.5 - 0.135 \sqrt{z_i \text{ ft}}$$

$$\text{where} \quad 1.2 \geq \beta \geq 0.25$$

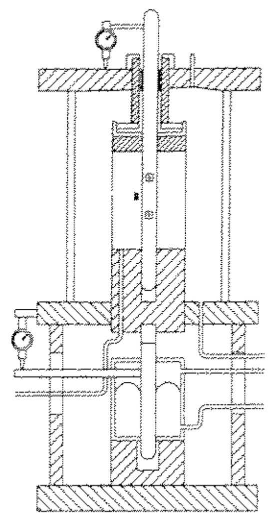
Unit Load Transfer from FHWA Design Equations



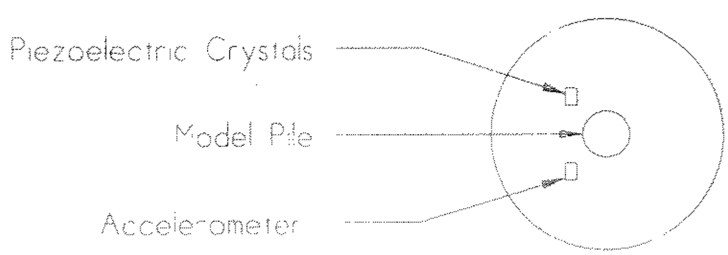
Experimental Apparatus

- Rod shear cell similar to that used by Chandler & Martins at Imperial College
- Measurements of
  - axial load transfer
  - radial stress at shaft wall
  - tangential stresses in sand near wall
- Testing of dry sand only

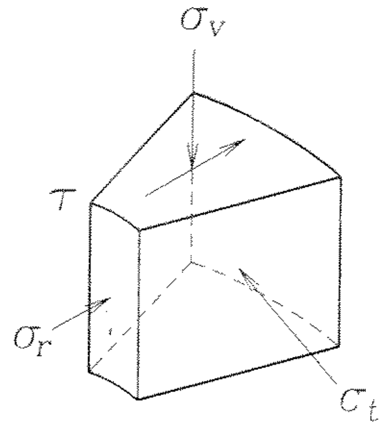
Rod Shear Apparatus



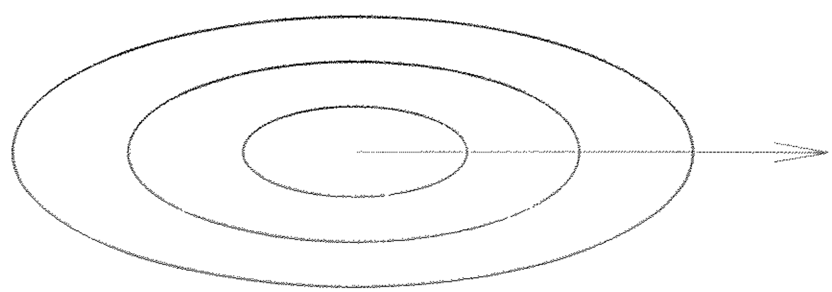
Plan View of Sensor Array



Tangential Stress Measurements



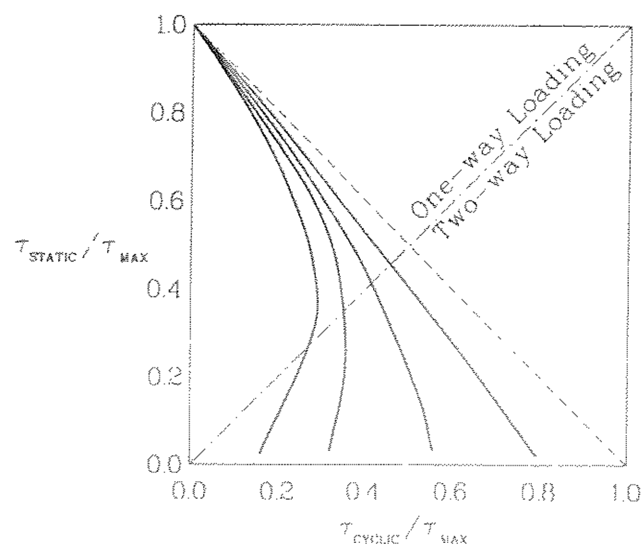
Propagation of Compression Waves



Factors to be studied:

- Ranges of confining stress
- Effects of earth pressure coefficient
- Ratio of cyclic and static loading components
- Number of cycles of loading

Effects of Loading Combinations



Current & Projected Research at the University of Arizona

William M. Isenhower

Projects at Arizona

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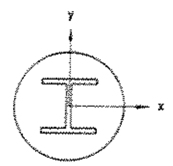
As Designed



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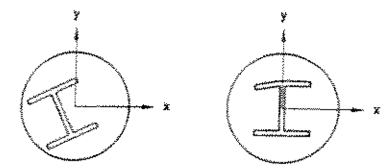
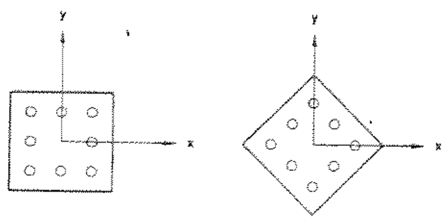


Off-axis Bending



Moment of Inertia Transformation for Elastic Sections

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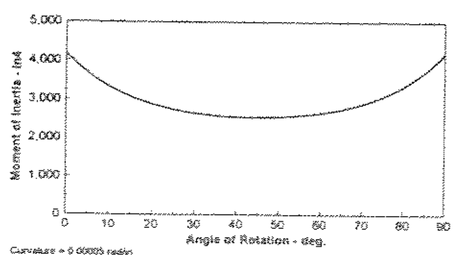
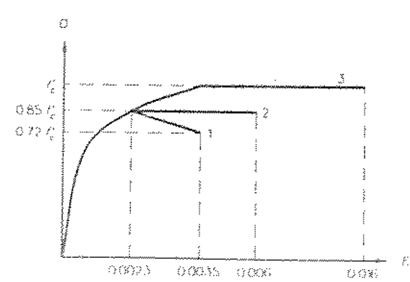
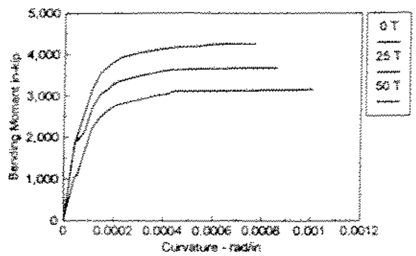
Variation of *I* with Rotation of Square Concrete Pile



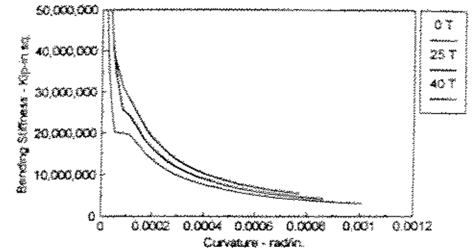
Effects of Confinement on Strength and Ductility of Concrete



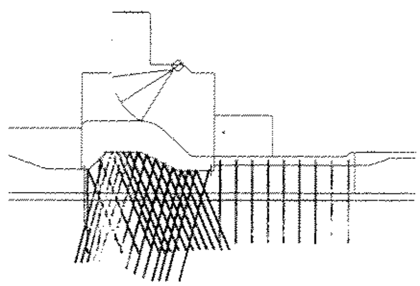
Curvature Effects on *EI*
Moment-Curvature



Effects of Curvature on *EI*
EI-Curvature



Reliability of Pile Supported Navigation Structures



Reliability analyses are used for:

- Assessing the reliability of structures based on current conditions
- Identifying sources of variance in structural performance
- Evaluating relative performance of dissimilar structural elements
- Providing a consistent method for prioritizing rehabilitation expenditures for dissimilar structures

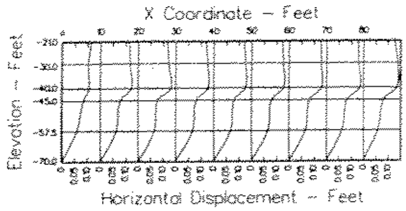
Procedure for Reliability Analysis

- Identify an initiating event
- Collect data for soil conditions and strength
- Evaluate means, standard deviations, and covariances of variables
- Perform seepage analysis to evaluate pore water pressures under structure
- Locate potential sliding surface

Reliability Analysis (cont.)

- Analyze soil displacements using FEM analysis including loading history and seepage

Soil Displacement under Structure from Finite Element Analyses



Reliability Analysis (cont.)

- Analyze soil movements using FEM analysis including loading history and seepage
- Perform soil structure interaction analysis

Reliability Analysis (cont.)

- Analyze soil movements using FEM analysis including loading history and seepage
- Perform soil structure interaction analysis
- Analyze variance of a performance factor i.e. bending moment or deformation of a particular pile

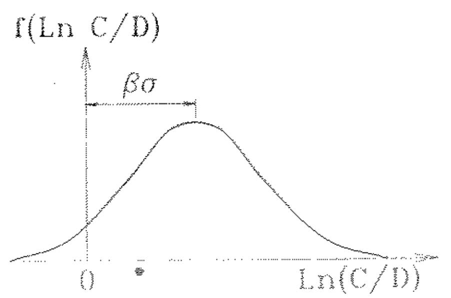
First Order Second Moment Analysis of Variance

$$[F(x)] = \sum_{i=1}^{n} \left(\frac{\partial F}{\partial x_i}\right)^2 V(x_i) + 2\sum_{i=1}^{n-1} \sum_{i+1}^{n} \left(\frac{\partial F}{\partial x_i}\right)\left(\frac{\partial F}{\partial x_j}\right)\rho_{ij}$$

Reliability Analysis (cont.)

- Analyze soil movements using FEM analysis including loading history and seepage
- Perform soil structure interaction analysis
- Analyze variance of a performance factor i.e. bending moment or deformation
- Compute the reliability index β

Definition of Reliability Index for Log-normal Distribution



Reliability Index for Log-normal Distribution

$$\beta = \frac{\ln\left|\dfrac{E[C/D]}{\sqrt{1+V_{C/D}^2}}\right|}{\sqrt{\ln\left|1+V_{C/D}^2\right|}}$$

Target Reliability Indices

Performance	β
High	5
Good	4
Above Average	3
Below Average	2.5
Poor	2.0
Unsatisfactory	1.5
Hazardous	1.0

Reliability Analysis (cont.)

- Analyze soil movements using FEM analysis including loading history and seepage
- Perform soil structure interaction analysis
- Analyze variance of a performance factor i.e. bending moment or deformation
- Compute β
- Repeat analysis for other performance factors or different initiating events

Laboratory Study of Shaft Friction in Sand

Objective

The objective of this study is to investigate changes in the three-dimensional state of stress in sand near a shaft as a function of loading.

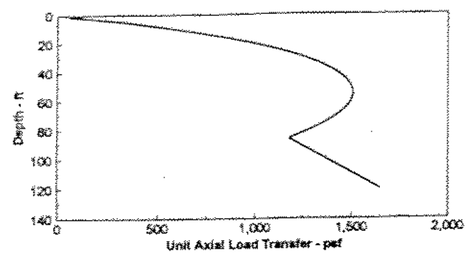
FHWA Design Equations for Drilled Shafts in Sand

$$Q_s = \pi B \sum_{i=1}^{N} \gamma_i' z_i \beta_i \Delta z_i$$

$$\beta_i = 1.5 - 0.135\sqrt{z_i \text{ ft}}$$

where $1.2 \geq \beta \geq 0.25$

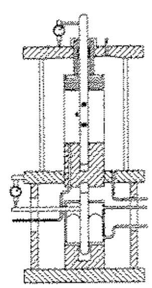
Unit Load Transfer from FHWA Design Equations



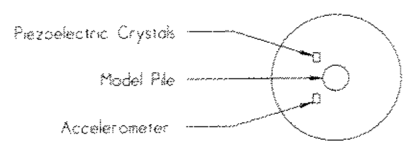
Experimental Apparatus

- Rod shear cell similar to that used by Chandler & Martins at Imperial College
- Measurements of
  - axial load transfer
  - radial stress at shaft wall
  - tangential stresses in sand near wall
- Testing of dry sand only

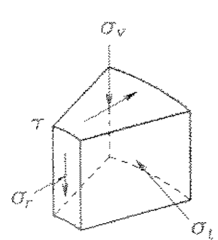
Rod Shear Apparatus



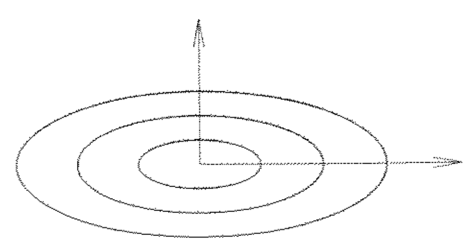
Plan View of Sensor Array



Tangential Stress Measurements



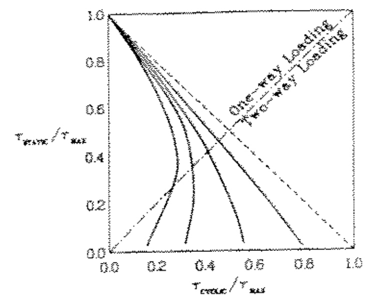
Propagation of Compression Waves



Factors to be studied:

- Ranges of confining stress
- Effects of earth pressure coefficient
- Ratio of cyclic and static loading components
- Number of cycles of loading

Effects of Loading Combinations



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13. ABSTRACT *(Maximum 200 words)*
The Workshop on Effects of Piles on Soil Properties was held at the U.S. Army Engineer Waterways Experiment Station on 13-15 July 1993, and this report presents the proceedings of that workshop. The Workshop was conducted to evaluate needs for research to be conducted by the private sector, institutions of higher learning, or Government agencies concerning the changes in soil properties caused by emplacement of piles or other deep foundation elements. Suggestions for research in soil structure interaction, effects of pile driving on soil properties, verification of soil property changes, and methods of testing and numerical modeling for investigating the problems which were developed at the Workshop are included in the proceedings. Papers presented and contributed to the Workshop are also included. Conclusions reached by the Workshop participants indicate that research into the changes in soil properties caused by pile driving is a valid area of inquiry which could lead to improved design techniques for piles and other deep foundation elements.

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