TENSION PILE STUDY

VOLUME VII

PROPOSED METHODS FOR DESIGN OF AXIALLY LOADED PILES IN NORMALLY CONSOLIDATED CLAYS

Report Number 82-200-7

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Report

to

Conoco Norway, Inc.
through
A. S. Veritec

Oslo, Norway

* * * *

By

The Earth Technology Corporation
Houston, Texas

February 1986

The Earth Technology Corporation

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February 27, 1985

Conoco Inc.
600 N. Dairy Ashford Rd.
Dubai Bldg., Room 2080
Houston, TX 77079

Attention: Mr. Jack Chan

Re: Tension Pile Study Volume VII "Proposed Methods for Design of Axially Loaded Piles in Normally Consolidated Clays"
Project CNRD 13-3

Gentlemen:

We are pleased to transmit herewith twenty (20) copies of our final report entitled "Proposed Methods for Design of Axially Loaded Piles in Normally Consolidated Clays". This report constitutes the seventh and last report produced under the CNRD 13-3 contract. Please forward the necessary copies to Conoco's staff, A.S. Veritec and your project participants, as you see appropriate.

We have enjoyed working for Conoco in this most challenging project and look forward to continue to be of service to you and your staff.

Very truly yours,

Jean M. E. Audibert, P.E.
General Manager
Gulf States Division

Dewaine Bogard
Senior Engineer

JMEA/plm

cc: H. Matlock

PREFACE

This report is the final of a series of reports documenting the work performed under Conoco Norway Research and Development Company Research Projects 13-2 and 13-3. This report contains the axial pile design methods which were developed from the results of experiments performed using 1.72-inch (4.37 cm) and 3.0-inch-diameter (7.62 cm) pile segment models and a 30-inch-diameter (76.2 cm) instrumented pipe pile at a decommissioned production platform in the West Delta Area, Gulf of Mexico.

The report also contains suggested methods for the analysis of single tension piles subjected to static and cyclic tensile loading, with recommendations regarding the selections of factors of safety to be used in evaluating the design of tension pile foundations.

The work was sponsored by Conoco, Inc. through Conoco Norway Research and Development Company, with participation by Chevron Oil Field Research Company, the American Bureau of Shipping, and the Minerals Management Service of the United States Department of the Interior.

The work reported herein is the result of the last phase of a larger research project, which was funded for the purpose of developing a better understanding of the axial pile-soil interaction associated with foundation piles for Tension Leg Platforms. The experimental work was performed under CNRD Research Projects 13-2 and 13-3, with the work being performed under a plan formulated in an earlier research project, identified as CNRD 13-1.

The work was performed by The Earth Technology Corporation, acting as a designated subcontractor to A.S Veritec. The Earth Technology Corporation project team was composed of the following staff members:

Hudson Matlock, Vice President for Research and Development, provided overall technical direction for the two projects, and contributed greatly to the design

philosophy and concepts incorporated in this report. Dewaine Bogard was responsible for the interpretation of the results of the experiments, the formulation of the design methods, and the production of this report. Chairat Suddhiprakarn was responsible for performing the computer solutions for the design examples. Jean Audibert, manager of the Houston office, assisted in the interpretation of the data and was responsible for Chapters 2 and 3 of this report.

At A. S. Veritec, Olav Furnes was instrumental in initiating the project which was largely carried out by Tore Kvalstad and Kjell Hauge. Rune Dahlberg served as Project Manager during later phases of the project.

Jack H. C. Chan assisted by J. L. Mueller, served as Project Manager for Conoco Inc., under the general direction of N. D. Birrell, Manager of Marine Engineering Division of PES, Conoco Inc. R. L. Gratz, H. W. Wahl and T. C. Ma served as Project Administrators under the general direction of R. L. Mc Glasson and R. M. Vennet, General Managers of CNRD.

Finally, recognition should be given to the value of the sponsorship of the entire program summarized in this report. The program and its results are unique in the breadth and depth of advancement of understanding of pile-soil interaction and, when released, the information should provide an extremely valuable contribution to the engineering profession. The far-sighted vision of the sponsors is deeply appreciated and commended.

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EXECUTIVE SUMMARY

Conoco, Inc., through Conoco Norway Research and Development (CNRD) of Conoco Norway, Inc. has sponsored a comprehensive program of research aimed toward improving the understanding of the pile-soil interaction along driven piles subjected to static and cyclic tension loading. The final goal of the research is the development of guidelines and procedures for the design and analysis of foundation piles for Tension Leg Platforms. Also participating in the sponsorship of the research were Chevron Oil Company, the American Bureau of Shipping, and the Minerals Management Service of the United States Department of the Interior.

The research was performed by the Earth Technology Corporation, acting as a designated subcontractor to A.S Veritec (formerly Det Norske Veritas).

The research has been performed in three interrelated phases.

The first, under CNRD Research Project 13-1, was the planning study for the remainder of the work.

The second, under CNRD Research Project 13-2, consisted of a site characterization study and in situ 3-inch-diameter (7.62 cm) model pile tests at the West Delta 58A platform site by The Earth Technology Corporation and a program of laboratory experiments using 1-inch-diameter (2.54 cm) model piles by A.S Veritec.

The third, under CNRD 13-2 and 13-3, consisted of additional in situ experiments using 1.72-inch (4.37 cm) and 3-inch-diameter (7.62 cm) model piles and load tests on a 30-inch-diameter (76.2 cm) instrumented test pile at the West Delta 58A platform site by The Earth Technology Corporation. Based on the results of the experimental work done under the CNRD 13-2 and 13-3 projects, further development of analytical models of axial pile-soil interaction and the development of guidelines for the design of tension piles in soft clays were also performed by The Earth Technology Corporation.

This report, which is the seventh and final report under the CNRD 13-1, 13-2, and 13-3 Tension Pile Studies, contains the design procedures and guidelines developed from the results of the experimental work performed at the West Delta 58A platform site by The Earth Technology Corporation.

The amount and variety of data produced by the project will provide a valuable source of information on axial pile-soil interaction and, in particular, on tension-pile performance, for a long time to come.

A number of significant accomplishments resulted from this five year program of research . Eight of the most important ones are listed below:

(1) Design, instrumentation and testing of a 30-inch (76 cm) diameter test pile embedded 234-ft (71 m) in soft clay at West Delta 58A.

(2) Successful measurements of:

    (a) pile head load-displacement
    (b) strains along the pile length (by 2 independent methods)
    (c) total soil pressure at six levels
    (d) pore pressures at six levels

(3) These measurements were made and recorded, both digitally and in analog fashion, during various phases of the pile life.

    (a) Total and pore pressures along the length of the pile were observed while the pile was driven into place.

    (b) Immediately after driving, a static tension test was performed, followed by a compression test.

    (c) Three months after installation the pile was load tested. The loading included static tension to failure, progressively increasing cycling about an initial tension bias and large-displacement two-way cycling to equilibrium at a degraded strength.

(d) Sixteen months after installation tests similar to (c) were also performed but are not within the scope of this contract and thus will be reported separately.

(e) Continuous measurements of pore pressure (and total soil pressure) provided an improved understanding of soil consolidation (set-up) and development of shear transfer capacity as a function of time after driving.

(4) A parallel field testing program was conducted at the same site using 1.72 and 3.0-inch diameter tools representing models of short segments of a real pile. Details are reported in Ref. 2. The program involved three different mobilizations to the site; with both installation and testing of multiple parallel probes in the first two. Three probes were left in place for long-term testing on the third trip. A wide variety of testing sequences (including static and cyclic loading) was accomplished with the probes, that would not have been possible with the large pile alone. Tests were done at four selected depths, enabling direct comparison among several adjacent boreholes. A total of 18 probe installations was done, with testing up to four times at each, resulting in a total of 49 independent static or cyclic load tests being performed. Measurements included shear transfer versus probe displacement and total lateral pressure and pore water pressure on the face of each probe. The data were recorded both digitally and by analog plotters.

(5) A parallel program of in situ and laboratory soil testing was conducted and is reported in Ref. 3.

(6) An extensive analysis of the raw data has been accomplished, together with correlation studies and interpretations. From the outset, the goal has been to combine information from the small probes and the 30-inch pile to enable extrapolation to prototype design.

(7) Backfitting of examples of the test data has been undertaken with analytical models using the CASH program. The program is a very general and versatile tool for modelling the time and deformation-dependent behavior of the soil surrounding a segment of a pile and has provided significant insight and understanding of the mechanics involved.

(8) A total of 6 reports has been issued previously, with the present report being the seventh and final report of the program.

The information developed in this program applies specifically to the behavior of long, axially-loaded piles in the soft normally-consolidated (or underconsolidated) clay soils surrounding the Mississippi Delta. However, with reasonable care, much of it should allow extensions to soft clay soils elsewhere.

For such soils, it is well recognized that the capacity of a pile during and immediately after driving may be quite low. The test results indicate that the immediate shear transfer as the pile is driven past any depth is extremely low. With sufficient time (dependent on diameter) the maximum load transfer approaches the in-situ undrained shear strength of the soil.

For large piles, with diameters of four feet or more, the time-rate of gain in strength may be of considerable importance. The permeability of highly plastic clay soils is so low that full set-up may require one or more years. However, an unanticipated result of this test program is that a typical open-ended pipe pile will reach full capacity much faster than a solid (or plugged) pile of the same diameter.

The amount of soil forced to flow around and outside the tip as the pile is driven appears to be the key. The pore pressures created in a thin layer of such clay can be dissipated fairly rapidly. A thick-walled pile will produce a thicker layer of pressurized and remolded soil and will thus require a longer time for set-up. At the other limit, a pile with a driving shoe that acts exactly as an ideal cookie cutter would create no zone of excess pore pressure and no delay in reaching maximum capacity.

The correlation of the consolidation-time behavior among the three sizes of the test probes and test pile, represents a major accomplishment of the program.

The correlation includes a wide range of diameters and diameter-to-wall-thickness ratios and is the keystone upon which gain in pile capacity with time after driving may be predicted. This is particularly important for very large piles, that, in clay soils, may require years to reach full capacity.

Correlation of data from the 1.72, 3.0 and 30-inch tests has provided an empirical adjustment to theoretical set-up rates that has been described in Chapter 4 and applied in Chapter 5 to an example design.

If, at any time during the consolidation process, the pile, or a portion of the pile, is moved up and down enough to cause a slip surface to form in the soil around the pile, and if the direction of such slip is reversed, the shear capacity is reduced. With perhaps 5 to 20 full reversals, the shear resistance will be reduced progressively to a value approximately equal to the remolded shear strength. Without full reversals of shear slip direction, the test data indicate that cyclic-loading losses are minimal. The field data indicate that the losses in shear transfer are temporary. However, they must be considered in design for storm loading.

To perform rational analyses of the behavior of long axially-loaded piles, it is necessary to describe the interaction of such piles with the soil along their lengths, since the stretch of the piles is sufficient to cause the soil resistance to be mobilized progressively along the length. In addition to maximum resistance reached, it is necessary to know the initial stiffness and the shape of curves of soil resistance per unit length of pile versus pile displacement. Also the effects of various patterns of cyclic loading displacement must be understood.

The shear transfer displacement characteristics represent a composite form obtained from the research. It is particularly important to note that one consistent formulation has been obtained from all of the test cases. With the wide range of diameters, it should be possible to confidently use this formulation to extrapolate to large-diameter piles in design.

The demonstration of the use of the design procedures developed from the experiments has been accomplished by use of the computer programs AXCOL and DRIVE which model the response of an axially loaded column under static or dynamic loading, respectively, of any prescribed pattern, including monotonic or repeated cyclic loading.

The design examples given in the report clearly illustrate, in terms of load and displacement at the pile head, the time-dependency of the pile capacity, the conventional static-loading characteristics, and the effect of one-way cycling simulating the loading from a tension leg platform.

The design of a tension leg pile is in general not directly dependent on its conventional static-loading axial capacity. The time-history of loading must be represented in some simplified way and the progressive loss of soil resistance must be described by some approximate but reasonable model.

For many cases and conditions the degradation of soil resistance may not be critically important. In such cases, simplifications of the presently proposed methods may be appropriate. However, until considerable experience is obtained by application to real design problems, the approach recommended is to use the proposed methods and models to describe as completely as possible the full cyclic loading behavior of the pile through the full range of its capacity.

The actual capacity, and the safety factor with respect to design loads, would thus be determined by a series of solutions with either the applied load or an imposed pile-head displacement being increased progressively to solve for the complete range of behavior.

No attempt should be made to introduce conservative allowances throughout the elements of the solution. Instead, the most realistic possible values should be estimated, together with an estimate of the statistical uncertainty of each significant component. Upper and lower bound solutions are also helpful. Then a rational application of analysis of variations and risk can be used to establish an appropriate margin of safety in the design.

The specific methods proposed herein are qualified as being tentative. Confirmation and extension will depend on analysis of the retest program conducted outside of the present contract and upon other data that may become available in the future. However, the quality and amount of the data already analyzed and the consistency of the results gives considerable encouragement toward confident application of the present tentatively proposed design method.

The present report is intended to serve primarily as a guideline for application of the research results to design practice. For that reason, more attention has been given to obtaining good overall agreement with the observed pile behavior rather than in obtaining a fundamental understanding of the details of soil behavior which have been responsible for the observed pile-soil performance.

It may be worth noting that, no matter how seemingly sophisticated any present and future method may be in trying to rationalize soil behavior according to accepted soil mechanics principles, unless prediction from such methods match the observed behavior as well as our proposed method they will not be as fundamentally correct.

Clearly, the behavior of a pile under static and cyclic loading is most directly dependent on soil very near the pile wall. The form of the relation of shear transfer versus pile displacement is directly the result of soil consolidation (set-up) near the pile as modified by shear deformation under loading and the formation of a slip plane, modified further by degradation during cycling. The measurement of total pressure and pore pressure variations at the pile, in parallel with the determination of resistance versus displacement, gives further understanding. Nevertheless, a complete and rigorous explanation of the observed phenomena remains to be developed. Meanwhile, use of the information in design is on a more or less phenomenological basis.

Several activities need to be undertaken beyond the present program that would further enhance and extend the knowledge and capabilites already developed. They are discussed below:

(1) The data obtained during the final retesting of the pile 16 months after original installation need to be analyzed and reported. The results should be simulated by the analysis and design methods tentatively proposed in the present report. Appropriate modifications or extensions should then be made.

(2) Using the recent correlations and the improved understanding described in Chapter 4, additional analytical simulations of the local pile-soil interaction should be done with the CASH program. It now appears likely that substantially improved understanding of the shear plane development and the cyclic degradation phenomena can be obtained by further adjustment of the CASH modelling to fit the interpretations in this report. Although the interpretations and correlations given in this report will serve as a guide and a checkpoint for the CASH modelling, it may also be desirable to perform additional checking against the self boring pressuremeter test results obtained at the WD58A site in a separate research program and against research-type soil tests including static and cyclic DSS tests, as well as some cubical triaxial tests. The program should then be edited thoroughly and documented for general distribution.

(3) The DRIVE program will likely be the primary tool for analysis and design of complete pile-soil systems under a variety of complex loading conditions. For pseudo-static calculation procedures, the program should be greatly streamlined and simplified for use in foundation design. Both the dynamic and pseudo-static versions should be documented for distribution.

(4) A tentative semi-empirical explanation is given in Chapter 4 of this report regarding the influence of pile diameter and wall thickness on the effective amount of soil forced outward around the pile as it is driven. These effects include variations in the rates of consolidation or set-up which may be very important for large diameter piles. Further fundamental experimental and analytical study is needed of the plastic deformation in the vicinity of the pile tip. Experiments presently planned to be conducted at Sabine, Texas should be of considerable benefit.

(5) Further development of understanding of the results of the recent installation and testing of a drilled and grouted pile at the West Delta 58A platform site should include comparative analyses with driven piles of the present report, thus providing an additional checkpoint on the effects of consolidation and reconsolidation and recovery on the shear transfer capacity in soft clay soils.

1.0 INTRODUCTION

1.1 Background

This report contains the results of CNRD Research Project 13-3, Task 14, entitled, "Design Procedures and Example Problems for Tension Piles in Soft Clays". The design methods which are presented herein are based directly on the results of in situ small-diameter and large-diameter instrumented pile tests at the site of the WD58A decommissioned production platform in the West Delta Area, Gulf of Mexico. The results of the experiments were described in earlier reports by the Earth Technology Corporation (Refs 1 and 2), with the soil properties taken from the results of field and laboratory tests reported in Ref 3.

The experimental work reported in Refs 1 and 2 was performed under CNRD Research Projects 13-2 and 13-3, with the site characterization study reported in Ref 3, performed under CNRD 13-2.

For the experimental work, both the large-diameter pile and the small-diameter pile segment models were instrumented to simultaneously measure the shear transfer, the pore water pressure, and the total radial pressure during installation, consolidation and setup, static and cyclic loading, and periods of reconsolidation and recovery after the enforcement of cyclic degradation in shear transfer capacity. In addition, the local relative pile-soil displacement of the pile models and the displacements at two embedded depths along the large-diameter pile were measured.

It was intended that sufficient data be taken in order that existing or new effective-stress analyses of axial pile capacity could be evaluated or developed, to rationally explain the observed pile-soil behavior.

As reported in Refs 1 and 2, consistent linear relationships among the radial effective pressure and the shear transfer capacity of the soil were not observed, except for the values of peak static shear transfer at the end of periods of undisturbed consolidation following installation. Furthermore, no relationship could be derived to explain the process of cyclic degradation in terms of effective stresses. A simple mechanistic interpretation of the process, in terms of pore water migration, was described during an interpretation of the pressure fluctuations observed during the small-diameter pile model tests.

An observation of the dependence of shear transfer capacity on the degree of consolidation, and with time after driving, has led to the development of methods to estimate the increase with time in the shear transfer capacity of normally consolidated clay soils.

1.2 Project Summary

The work reported herein is the culmination of efforts begun in early 1981, starting with the planning of the work as reported in Ref 4.

Based on the results of the planning study reported in Ref 4, a number of laboratory and field experiments were performed by A. S. Veritec (formerly Det Norske Veritas) and the Earth Technology Corporation, with the results of the work being contained in Refs 1, 2, 3, 5, 6, and 7.

As given in Ref 4, the work reported herein was to include consideration of the laboratory tests performed on 1-inch-diameter (2.54 cm) pile segment models by A.S Veritec (Ref 6), in situ small-diameter pile model experiments by the Earth Technology Corporation (Refs 2 and 6), experiments using a 30-inch-diameter (76.2 cm) instrumented pipe pile by the Earth Technology Corporation (Ref 1), and a site investigation and characterization study by the Earth Technology Corporation (Ref 3).

As reported by Veritec (Ref 6), problems with the pressure transducers and the variations in the applied confining pressure during the laboratory tests hindered the interpretation of the results of the experiments. Although the problems with the pressure transducers were solved during the latter stages of the research, the intentional variation of the confining pressure during the consolidation phases of the experiments rendered the evaluation of the soil behavior very difficult. As noted in Ref 6, the variations in the applied confining pressure resulted in increases in the pore pressure during consolidation; such behavior is completely at variance with the behavior observed during the in situ tests, or during other laboratory consolidation tests. The effects of such variations in the soil pressures on the shear transfer capacity of the soil are difficult to predict. The results of the experiments have not been considered in this report in view of their uncertain nature.

The work reported herein has thus been based primarily on the results of the in situ pile segment tests and the tests using the large-diameter pile. In addition, the results of in situ experiments performed using the small-diameter tools at other normally consolidated clay sites have been brought to bear in this effort.

As noted in Refs 1 and 2, a large quantity of high-quality data has been obtained at the West Delta 58A site. Due to the volume of data which was acquired during the eighteen independent experiments with the small-diameter tool, and during three series of load tests on the large-diameter pile (only two of which are considered herein), it was not possible to explore every facet of the behavior which was observed, nor to include consideration of all the effects of load history in the design methods which were developed. The methods do include consideration of those aspects of the experiments which are of most benefit to practical pile design. Further research and development of theoretical concepts of soil behavior are felt to be needed to fully explain the results of the experiments in detail.

The design methods proposed herein were derived solely from the observed pile-soil behavior. No assumptions were made regarding the shear strength or consolidation properties of the soil; nor are any contrived theoretical concepts of soil behavior introduced. The methods were developed as a means by which the results of the large-diameter pile load tests could be extrapolated to sites having similar soils. Consideration of the effects of variations in the depositional history, or any effects of variations in stress history, such as overconsolidation, have not been included.

As will be shown in Chapter 5, the method enables the results of the static and cyclic load tests reported in Ref 1 to be reproduced quite well, using the AXCOL and DRIVE computer programs to duplicate the loading history. It is therefore felt that the use of the method to extrapolate the results of the tests to similar sites is justified.

In summary, this report presents a proposed method to predict axial load-deformation behavior of piles embedded in normally consolidated clays similar to those found in the Gulf of Mexico. Although simple in nature, the proposed method, being founded directly on the results of the experiments, allows the measured pile behavior to be duplicated by established pile design computer programs. The methods are in good general agreement with established concepts of soil mechanics, and also with present methods of design, which, however, do not include the effects of time on the development of the ultimate pile capacity during consolidation.

The wealth of data obtained during this five-year research program has not yet been fully explored. It is hoped that future phases will allow further advances to be made in the proposed pile design methods.

1.3 Report Organization

Chapter 2 of this report gives recommended procedures for site investigations, especially as pertains to deep water sites.

Chapter 3 outlines the associated program of laboratory testing to be performed on soil samples retrieved from deep water sites, so as to furnish valid quantitative soil properties.

These soil properties will in turn be used as input to the proposed Total Stress Design Methods described in Chapter 4. Hindcasting of the load test performed on the 30-inch (76 cm) diameter test pile is also presented in Chapter 4.

Chapter 5 summarizes the proposed method and demonstrates its use with example calculations for a 60 inch (1.5 m) diameter pile which could be part of the foundation for a TLP at a possible deep water site in the Gulf of Mexico.

2.0 RECOMMENDED SITE INVESTIGATION PROCEDURES

2.1 General

Before discussing and developing the characteristics needed for a site investigation program it is important to review the objectives to be met by a site investigation project. The objectives are as follows:

(1) Define geological processes and understand depositional environment which have led to the formation of the particular foundation soils at the site.

(2) Define the areal extent and the degree of variability of major geological units forming the foundation soils (e.g., use a combination of geophysical methods and geotechnical soundings).

(3) Define the variations with depth of key geotechnical parameters to be used in the foundation design analyses.

(4) Identify any peculiar conditions which could possibly lead to difficulties in either installation of the foundation members or which could affect the choice of the type of foundation members and their geometry e.g., the choice between driven and drilled and grouted piles, using more small diameter piles than fewer large ones or using more shorter piles than fewer longer ones.

(5) Provide the foundation designer with valid quantitative soil properties to effectively design the platform foundation.

2.2 Background

Many offshore platforms have been successfully sited around the world in relatively shallow waters (e.g., less than 1,000 ft). In the Gulf of Mexico the approach generally used has been as follows:

(1) Little, or at least not enough, use has been made of geological and geophysical studies to guide in the development of an optimum soil investigation program.

(2) Generally only one soil boring has been taken at any given offshore platform site.

(3) Reliance has been almost exclusively on laboratory tests on soil samples with little use of in situ tests.

Only for frontier and hostile areas has the conventional approach been more sophisticated than discussed above. In particular, in situ tests and multiple soil probings have been used in the North Sea and the Arctic. For the deep water environment it will be necessary to improve upon the present Gulf of Mexico practices. The proposed method is addressed in the next paragraph.

2.3 Proposed Site Investigation Procedures

The proposed approach to site investigation programs for the deep waters of the Gulf of Mexico should follow these general guidelines:

(1) Recognize the usefulness and make use of geological and geophysical surveys.

(2) Rely on more than one soil boring.

(3) Recognize and solve sampling problems presented by deep waters.

(4) Further introduce in situ tests and attempt to use more directly their results.

(5) Make best use of in situ capabilities by matching their test characteristics and capabilities to particular problems i.e.,

(a) Electronic Cone (preferably the piezocone) for definition of stratigraphy

(b) Remote Vane for definition of in situ shear strength

(c) Pressuremeter for definition of in situ shear strength and modulus, both static and cyclicly degraded.

A proper site investigation program should also yield an adequate number of high quality (pushed) samples to be used in a comprehensive laboratory soil testing program aimed at further defining the necessary foundation design parameters. This subject is addressed in the following chapter.

3.0 LABORATORY TEST PROGRAM

3.1 General

Soil samples retrieved from beneath the seafloor are only imperfect representations of the soil existing in situ. Conventionally, the objective of laboratory tests has been to subject the soil samples to a suite of loadings under stresses which are supposed to represent the expected changes in stress or strain to be experienced by the soil in situ. Strength and deformation parameters obtained from such tests have been empirically correlated with observed behavior in an attempt to allow prediction of, and extrapolation to, other situations, geometries and conditions.

3.2 Background

Sampling disturbance generally yields strength parameters which are lower than really exist in situ. The problem is further compounded by the high stress relief experienced when soil samples are brought to the surface at deep water sites. Samples have been noticed to grow out of sampling tubes and the structure of the soil samples to be highly disturbed by the expansion of dissolved gases in the pore fluid. Under special conditions such as experienced at Green Canyon, hydrates may also suddenly sublimate thus further compounding the problem.

Correction of sampling disturbance has been attempted and proven at times to be successful through the recompression of samples to confining pressures beyond those estimated to exist in situ. While such SHANSEP-type laboratory test programs may indeed correct reasonably small sample disturbance, they require relatively high quality undisturbed soil samples to be successful. Such samples from deep-water sites are not normally obtained and the SHANSEP approach may not suffice to correct the high degree of sample disturbance experienced by the stress relief described above.

Correlations with less susceptible index property tests should be used to guide judgment of the validity of test results. In fact, the reliability of soil parameters such as $c/\bar{p}$ ratios obtained from such correlations may be better than that of results from laboratory tests on disturbed samples.

Correlations with in situ tests will further help in making a choice for design parameters. The reliability of in situ test methods for the deep waters and for conditions where such things as hydrates exist may need to be addressed. In particular such items as N-values for interpreting CPT results, or reduction parameters for remote vane tests, or direct use of in situ tests results to formulate axial pile capacity may need to be carefully scrutinized and calibrated.

The conclusion is that, unless reliable high quality samples of the seabed soils are obtained, a combination of engineering judgment and empirical correlations will need to be used to arrive at foundation design parameters. The measured shear transfer on the Conoco test pile indicated that values of shear strength from present sampling and testing procedures adequately represented the shear transfer for the instrumented test pile.

For deep-water sites, with samples being obtained from a floating platform, and being subjected to a higher degree of pressure relief, more sophisticated sampling and laboratory testing procedures may be required, in order to obtain values of engineering properties for the soils which are equal in quality and reliability to those now obtained at shallow-water locations.

3.3 Proposed Laboratory Testing Program

The above discussions have indicated that testing programs should be aimed at producing reliable, conventional (well known and tried out) parameters. Rather than relying on exotic, sophisticated and costly tests, a well planned laboratory program should be complete and such as to allow cross checking of results through a suite of complementary tests. Such a program should thus include the following:

(1) Identification Tests: a series of Atterberg limits and water content determinations throughout the profile. These tests are very useful indicators which can allow checks to be made of other test results through well established empirical correlations.

(2) Static, Total-Stress Strength Tests: a usual series of unconsolidated undrained (UU) and miniature vane (MV) throughout the profile. These are again standard reference tests which can be useful in determining shear strength profile trends at low costs.

(3) Remolded, Static, Total-Stress Strength Tests: a series of remolded UU and MV tests to define remolded strength profile and thus a sensitivity profile throughout the deposit. Note that sample disturbance (due to sampling and mostly due to gas expansion) may severely invalidate such a determination and yield sensitivity values which are too low (i.e., close to 1). The best fit for the degraded shear transfer seems to be the remolded shear strength for UU-type triaxial tests; the residual vane strengths appear to be too low; however, such tests may be valuable indicators of the maximum degree of reduction in resistance, and are thus very useful.

(4) Static, Effective-Stress Strength Tests: a series of $\overline{\text{CIU}}$Compression and $\overline{\text{CIU}}$Extention triaxial tests to define normalized strength parameters. The confining pressure should at least cover the range one-half, one and two times the effective overburden pressure. A series of DSS tests would also be recommended to further define the behavior of soil under simple shear. The latter tests would be used as check points for the triaxial tests and thus do not need to be as numerous.

(5) Stress History and Consolidation Behavior: an adequate series of consolidation tests throughout the profile to accurately define the maximum past pressure and C_v profiles. Note that correlations with the liquidity index and liquid limit, respectively, should help complete the picture. Regarding $\bar{p}_c$, it may be useful to complement the Casagrande method with the strain energy method. The latter is helpful in the case of

disturbed samples (i.e., flattened consolidation curves). In regard to the coefficient of consolidation, it will be important to make available the raw data, more particularly plots of settlement versus time (both square root and log methods), to define t_{90} and t_{50}, respectively. These parameters are most important when attempting to define evolution of capacity with time. Also it is hoped to match the laboratory consolidation versus time curves to the normalized consolidation curves developed with the pile segment and the 30-inch pile experiments.

3.4 Summary

In summary, the primary goal of the laboratory testing program should be to obtain soil shear strengths and consolidation properties which are equal in quality to those presently obtained at shallow water sites using present procedures. Due to the extreme pressure relief to which the soil samples are subjected in the process of retrieval, consideration of a parallel program of more-sophisticated laboratory test program, and, perhaps, more reliance on empirical correlations between the shear strength and the consolidation characteristics of the soil with the plasticity and other basic soil parameters, may be required to ensure that equality.

4.0 TOTAL STRESS METHOD OF AXIAL PILE DESIGN

In this chapter, methods for evaluating the development of shear transfer capacity along friction piles in normally consolidated clays will be presented. The methods were developed for use in extrapolating the results of the load tests on the 30-inch-diameter (76.2 cm) pile to locations with similar soils.

The method consists of two parts. The first is concerned with the determination of the time-dependent increase in shear transfer capacity of the soil after pile installation. The second part of the method is concerned with the manner in which nonlinear curves describing the development of shear transfer as a function of relative pile-soil displacement are constructed.

The methods result in the formulation of depth-dependent nonlinear support curves to be used in conjunction with computer programs designed for the analysis of axially-loaded piles. Although the ultimate static-loading pile-head capacities which are predicted by the use of the method can be compared with those predicted by other methods, the actual pile-head load-displacement behavior which results from the proper computer modelling of the pile-soil system will result in calculated pile capacities which are less than the sum of the predicted ultimate resistances along the pile. This is the result of progressive yield occurring along the pile length.

4.1 Method for the Prediction of Axial Capacity with Time

The method developed for the calculation of the ultimate shear transfer capacity of the soil is based on the observation that, during the experiments with both the small-diameter pile models and the large-diameter pile, the magnitude of the peak static shear transfer was, to a large extent, linearly related to the degree of consolidation. Thus, the development of the method was primarily concerned with developing a reliable method of predicting the progress of consolidation (and thereby the peak shear transfer) with time after driving.

Although cyclic loading temporarily disrupted the process of consolidation, the return of the pore pressure dissipation curve to a previously-established pattern (and the simultaneous recovery of shear transfer capacity) suggested such an approach.

The first step in evaluating the time-dependent increase in shear transfer capacity consisted of developing a method for evaluating the progress of consolidation as a function of the time after driving. To accomplish this purpose, a time factor was calculated for each of nine small-diameter pile model experiments in the soil in Stratum III (below the 160-ft (48.8 m) depth), and for each level of pressure instrumentation along the large pile.

In accordance with principles of soil mechanics regarding one-dimensional consolidation, a time factor, T, can be defined in relation to the coefficient of consolidation, C_V, and the square of the length of the drainage path. For radial consolidation around a pile, the drainage path length is indefinite, and the pile diameter is substituted as a characteristic dimension. The time factor was determined by the equation

$$T = t\, C_V / D^2$$

where, t is the time after driving, expressed in minutes

C_V is the coefficient of consolidation, in cm^2/min, and

D is the diameter of the pile (or model), in cm.

For various times after the insertion of the pile and the models, the degree of consolidation was determined by dividing the difference between the initial maximum excess pore pressure and the instantaneous values by the difference between the maximum value and the ambient pore pressure, which was determined experimentally by the instruments along the large pile, the pressure measurements made using the 3-inch-diameter tools, and the free-field piezometers installed by a representative of the Norwegian Geotechnical Institute. The coefficient of consolidation (taken as 0.06 cm2 / min) represents an average value taken from laboratory consolidation tests of the soil from the site which are given in Ref 3.

The relationship between the degree of consolidation and the time factor for the 1.72-inch-diameter (4.37 cm) X-probe is shown in Plate 4.1. As seen in the plate, the relationship is similar to one expected for a clay in one-dimensional consolidation in which the degree of consolidation can be determined from either the measured changes in the height of the sample or the relative reduction in the excess pore pressures.

The relationships which were derived from eight of the nine experiments in the Stratum III soil using the open-ended 3-inch-diameter pile models are shown in Plate 4.2. As expected from the results reported in Ref 2, a degree of scatter is evident in the results; however, a very clear trend is shown.

In order to obtain a representative curve from the data shown in Plate 4.2, the values at various degrees of consolidation were averaged, resulting in the curve shown in Plate 4.3, with the maximum and minimum values from all the experiments also shown.

In a similar manner, the consolidation data from the pressure transducers at all six depths along the large-diameter test pile were averaged, yielding the curve shown in Plate 4.4. Again, a very clear relationship between the degree of consolidation and the time after driving is evident. Much less scatter is shown for the instruments along the large-diameter test pile than was seen in Plate 4.3. Although some degree of scatter is evident in the early part of consolidation, the data points converged to near-equal values after 12 months of consolidation, in December, 1984.

Plate 4.5 contains a composite plot of the curves shown in Plates 4.1, 4.3, and 4.4. As seen in the plate, a time factor based on the consolidation characteristics of the soil and the diameter alone is not sufficient to develop a unique relationship for the time required for the pore pressures to dissipate. The time factor must also be a function of the relative wall thickness of the pile (or the degree of cavity expansion), as expressed by the diameter to wall thickness ratios shown on the right hand side of Plate 4.5.

The time factors for the curves shown in Plate 4.5 were divided by the empirical term ($100 - 2D/w_t$), where D is the diameter of the pile (or model) and w_t is the wall thickness. The empirical term was developed from the curves shown in Plate 4.5 by assuming that the relationships between the time factors at equal degrees of consolidation remained consistent throughout the full range of consolidation, enabling the time for 100 percent of consolidation for the 1.72-inch-diameter model to be extrapolated. This produced the relationship shown in Plate 4.6, from which the effects of wall thickness on the consolidation time can be estimated.

As seen in the plate, the additional term ($100 - 2D/w_t$) has resulted in an effective normalization of the curves, within experimental accuracy.

The newly defined, normalized time factor, T_f, can be interpreted as follows.

As the ratio of the diameter to wall thickness increases (that is, for a given diameter, the wall thickness decreases), the time required for consolidation also decreases, eventually approaching zero. For extremely thin-walled piles, the amount of cavity expansion (the radial thickness of the layer of clay which is forced outward), will be extremely small; the excess pressures which are developed will extend into the adjacent soil for a smaller radial distance, and will thus require a shorter time to dissipate.

From the relationships shown, the limit value of D/w_t of 50 will yield zero consolidation time. While the relationship shown in Plate 4.6 is obviously not valid for extremely thin-walled piles (i.e., with diameter to wall thickness ratios larger than 50), this relationship remains valid over a large range (2 to 40 plus) of practical D/w_t ratios. For thinner-wall piles than those which were tested, a similar trend would be expected, with the consolidation time continuing to decrease with the increase in the D/w_t ratio, and approaching near-zero for extremely thin walls.

For fully-plugged piles, the time for full consolidation will be equal to 24 D^2/C_v. The exact value of the constant (100) cannot be established with certainty from the experimental data taken at the West Delta site, in which the consolidation during the experiments with the full-displacement X-probes were allowed to proceed to only approximately 80 percent of completion. For practical purposes, however, the value can be shown to fit the data taken in all the experiments, again well within experimental accuracy.

For the sake of simplicity, the term expressing the effect of the diameter to wall thickness ratio has been expressed as $2D/w_t$. The relationship should be more properly expressed as $D/(w_t/2)$, that is, the diameter divided by half the wall thickness. The expression can now be related to the physical phenomenon of approximately half the volume of soil displaced by the pile wall flowing outside the pile, with the remaining half-wall thickness of displaced soil flowing into the pile.

The variation in the consolidation times (and the variation in the magnitudes of the excess soil pressures) during the experiments with the 3-inch-diameter (7.62 cm) models with open-end cutting shoes can now be clearly seen as being due to the relative volume of the displaced soil (nominally equal in volume to half of the wall thickness, 0.0625 inch (1.6 mm)) which flowed outside the cutting shoe, as opposed to that which flowed into the cutting shoe. For the differences noted in the consolidation times during the experiments, a variation from 0.040 to 0.080 inch (1.0 to 2.0 mm) in the radial extent of soil expelled (as compared with the 0.125-inch (3.2 mm) wall thickness) would result in a doubling of the consolidation time between two experiments. Thus, the apparent scatter is in fact minimal, as it corresponds to a variation of no more than $\pm$0.02 inch (0.5 mm) in cavity expansion.

In order to obtain a normalized consolidation curve which would approximate the behavior shown in Plate 4.6 reasonably well, the data were replotted in the format shown in Plate 4.7. The data are seen to fall very nicely on a straight line, thus indicating that the experimental relationship between T_f and degree of consolidation, U, is hyperbolic. This hyperbolic relationship has been plotted in Plate 4.8 as:

$$T_f = \frac{0.012\ U}{1 - 0.94\ U}$$

The normalized consolidation curve which has been derived from the experimental data thus appears to be reasonable. The time required for consolidation is a function of both the pile diameter and the amount of cavity expansion, expressed here as the relative volume of soil which is displaced outward. Within the limitations of the experiments, in terms of D/w_t ratios, the curve has been shown to fit the experimental data quite well, allowing the relationship to be used for extrapolating the results of the experiments to other soft clay sites, and to piles having wall thicknesses within the ranges used in the experiments. As noted earlier, extrapolation to extremely thin-walled piles would lead to erroneous results. However, the interpolation between the D/w_t ratios used in the experiments (2 to 40+) should be valid.

It now remains to relate the shear transfer capacity to the degree of consolidation, and to thereby develop a relationship for the time-rate of development of shear transfer, and thereby axial pile capacity, at any time after installation.

Since we had only two degrees of consolidation for the test pile (immediate and 3 month tests), the data from the pile segments were used to develop a relationship between shear transfer and time. Additional experience with the pile segment models at other normally consolidated sites were also considered to broaden the data base.

The time-rate of development of shear transfer during the experiments with the open-ended 3-inch-diameter (7.62 cm) pile models is shown in Plate 4.9. In the plate, the values of shear transfer at each depth have been divided by the corresponding value of the interpreted shear strength reported in Ref 3, with the time factor derived from the consolidation behavior in each individual experiment.

The peak shear transfer recorded in eighteen load tests during the nine experiments at the depths of 178 and 208 ft (54.3 and 63.7 m) are included in Plate 4.9. Except for the experiment in which the pressure transducers failed, the results of all initial static load tests to failure and the results of the first loading to failure during the first retest after reconsolidation are shown.

With the exception of two load tests, excellent agreement is noted among the peak values of shear transfer measured during the load tests performed at low degrees of consolidation and the dimensionless time factor. Although there is some scatter for the values recorded after 3 months of consolidation, the values from Experiment 5 and 6 are respectively above and below the curve by only 12 percent. The average value of these 3-month experiments matches the trend quite well.

As indicated by Plate 4.9, the peak shear transfer rapidly increases, approaching the in situ shear strength of the clay near the end of the consolidation process.

Based on the trends shown in Plate 4.9, a relationship was developed to estimate the increase with time in the shear transfer capacity of the soil along the large-diameter pile, based on the normalized consolidation curve in Plate 4.8. The curve was developed on the basis that the increase in the ratio of shear transfer to shear strength (Alpha) was a linear function of the degree of consolidation (with a lower limiting minimum Alpha value of 0.33), and then calculating values of Alpha versus the time factor at equal degrees of consolidation. The resulting curve is shown in Plate 4.10, with the average values of peak shear transfer measured along the pile during the immediate load tests and the load test after three months of consolidation shown for comparison.

As seen in Plate 4.10, excellent agreement was obtained between the measured and the calculated values of average peak shear transfer.

Again, as noted for the small-diameter model tests in the soil layer beyond the 160-ft (49 m) depth (Stratum III), the minimum resistance after cyclic degradation was half the undrained shear strength.

Based on the excellent agreement shown in Plates 4.9 and 4.10, it can be concluded that the development of shear transfer with time in normally consolidated clays can be predicted with reasonable certainty, provided that the in situ shear strength and consolidation characteristics of the clay are known.

4.2 Nonlinear Shear Transfer Versus Displacement Relationships

In this section of the report, a method for approximating the nonlinear shear transfer-displacement relationships which were derived from the results of the large-diameter pile load test will be presented. Because of the depositional history of the soil at the West Delta site, recommendations for extrapolating the curves to normally-consolidated clay soils will also be presented.

The pre-yield portions of the shear transfer versus displacement relationships which were derived from the results of the static load test of the large-diameter pile in April, 1984 are shown in Plate 4.11. As shown in the plate, the value of peak shear transfer generally increases with depth, with the magnitude of the displacement required to mobilize the peak shear transfer also generally increasing with depth.

The curves shown in Plate 4.11 are again shown in Plates 4.12 and 4.13. Plate 4.12 contains the curves for the depths above 92 ft (28 m), with Plate 4.13 containing all remaining curves from the depths below 92 ft (28 m). In the curves shown in Plates 4.12 and 4.13, the values of shear transfer have been normalized by dividing the shear transfer values at all displacements by the peak value.

The curves shown in each of Plates 4.12 and 4.13 exhibit a remarkable degree of similarity.

The values of displacement, u_{50} and u_{100}, corresponding respectively to values of shear transfer of 50 and 100 percent of the peak shear transfer in Plates 4.12 and 4.13, are shown in Plate 4.14 plotted against the depth to the midpoint of each load measurement interval. Also plotted with depth are the values of 5 times the displacement at the 50-percent load point, 5 u_{50}.

As seen in the plate, there are two distinct trends in the increase in the displacements with depth; one for the soil above a depth of approximately 80 ft

(24 m), and another for the soil below this depth. Similar relationships were also plotted for intermediate load levels, with very similar results.

It is of interest to note that lines drawn through the values of u_{50} and through displacements corresponding to other load levels (not shown) also converged at a value of zero at a depth of 80 ft (24 m) for the deeper soils, and somewhat above the mudline for the upper clay layer.

For a layered soil system, such as that described in Ref 2, the soils below a depth of 160 feet (49 m) would be expected to display a certain pattern of behavior; those above the 80-ft (24 m) depth a second pattern, and those in the middle layer perhaps a third pattern. As noted in Ref 1, a strain module at each of the 122 and 152-ft (37 and 46 m) depths was inoperable; it is therefore not possible to discriminate the behavior in Stratum II as well as would have been possible with complete instrumentation. The substitution of a pair of extensometers for the strain modules at the 122-ft (37 m) depth has been of considerable benefit, however.

Based on the trends shown in Plates 4.12, 4.13, and 4.14, methods for calculating a normalized curve shape for the development of shear transfer with displacement were developed.

As shown in Plate 4.14, the magnitude of pile displacement required to mobilize the peak shear transfer (u_{100}) was approximately 5 times larger than the value at half the peak resistance. Since the value of displacement at the 50-percent level can be defined far more consistently and accurately than can the displacement at the peak, the curves were normalized using values of displacement equal to 5 times the value at one-half the peak shear transfer.

The pre-yield portions of the curves were approximated by a hyperbolic relationship of the form:

$$y = \frac{x}{Ax + B}$$

where, $x = (u / u_{ref})$, u being the pile displacement,
$u_{ref} = u_{100} \approx 5u_{50}$ and

$y = (f / f_{max})$, f being the shear transfer, and f_{max} being the maximum value of shear transfer.

The coefficients A and B are the slope and intercept, respectively, of the straight line shown in Plate 4.15, in which the ordinates correspond in form to

$$(x / x_{max}) / (y / y_{max}),$$

and the abscissae correspond to

$$(x / x_{max}).$$

As shown in the plate, a hyperbolic approximation of the experimental curve is a reasonable choice since a straight line can be drawn through the data points. The slope A is 0.76, and the intercept B is 0.24.

For each value of f / f_{max} corresponding values of displacement u were then determined from the equation

$$(u / u_{ref}) = (0.24 \; f / f_{max}) / (1 - 0.76 \; f / f_{max})$$

The value of u_{ref} (see Plate 4.14) may be calculated from the equation

$$u_{ref} = 0.003 \; X$$

where X is the depth in feet below the zero-shear strength intercept.

The nonlinear curves which were calculated for each mid-point of load measurement interval are shown in Plates 4.16, 4.17 and 4.18. The curves in Plate 4.16 correspond to the soil in Stratum I, between the depths of 0 and 80 ft (0 and 24 m) below the mudline; those in Plate 4.17 correspond to the soil layer in Stratum

II, between the depths of 80 and 160 ft (24 and 49 m) below the mudline, and those in Plate 4.18 correspond to the soil below a depth of 160 ft (49 m) in Stratum III.

As shown in the three plates, excellent agreement has been obtained between the measured and the calculated shapes of the nonlinear relationships, for the pre-yield portion of the curves.

In order to complete the construction of the static curve, the post-peak reduction in shear transfer must be considered.

The ratio of the post-peak residual shear transfer to the peak shear transfer at the depths of 152, 197, and 223 ft below the mudline (46.3, 60.1, and 68.0 m) was 0.90, 0.84, and 0.81, respectively. A line passing through the three points intersects a value of 1.0 at a depth near 80 ft (24 m) below the mudline. Thus, the increase in the degree of loss in peak resistance in the normally-consolidated clay layer seems to be directly related to the depth below 80 ft (24 m), much in the same way as the increase in the peak displacement shown in Plate 4.14.

For the soil at the depths above 152 ft, no such trend was evident, with the ratio of the residual value to the peak value of shear transfer decreasing with depth. The average value, over the embedded length of the pile, was 82 percent.

An examination of the curves of shear transfer and displacement derived from the test data at all depths showed that the displacement at which the value of shear transfer reduced to the minimum value was approximately 0.3 inch (7.62 mm), or 0.01D, larger than that at the peak resistance.

Thus, for the construction of static shear transfer curves, the displacement at which the peak shear transfer reduces to the residual value may thus be assumed to be ($u_{peak} + 0.01\,D$), where D is the pile diameter and u_{peak} is the same as u_{ref} mentioned previously.

For purposes of simplification of the construction of the support curves (since the post-peak response of the pile is only of limited practicality), the value of residual shear transfer may be estimated to be the average of the values measured, or approximately 80 percent of the peak resistance.

4.3 Demonstration of the Use of the Procedures at the West Delta Site

In order to test the validity and consistency of the proposed procedures, the pile design methods will be applied to the soils at the West Delta site, and the load test results predicted using the AXCOL computer program (Ref 8).

First, a plot (similar to Plate 4.9) was prepared, depicting the estimated increase with time in the ratio of peak shear transfer to shear strength, based on the normalized curve shown in Plate 4.8, with the shear transfer increasing linearly with the degree of consolidation.

For the load test performed immediately after driving, the soil was divided into three layers, at depths which correspond to the successive depths of penetration of the pile tip during the installation of the test pile. The depths were 0 to 60 ft (0 to 18 m), which correspond to the initial installation of pile section TP1; 60 to 145 ft (18 to 44 m), which correspond to the installation of pile section TP2; and the remaining depths, from 145 to 234 ft (44 to 71 m), which correspond to the final driving of the test pile.

With a time of 118 hours after driving for the soils between the depths of 0 to 60 ft (0 to 18 m), a time factor T_f of 0.0015 was calculated. At a degree of consolidation corresponding to this time factor, the ratio of f_{max} / S_u was estimated to be 0.43. For the soil between the depths of 60 and 145 ft (18 to 44 m), the elapsed time after driving was 56 hours, yielding a value for the time factor of 0.0007, and a corresponding value of f_{max} / S_u of 0.38. For the soils below a depth of 145 ft (44 m), the time factor was very nearly zero; a value of f_{max} / S_u of 0.33 was therefore used.

In a similar manner, a time factor of 0.035 was derived for the load test which was performed in April, 1984. For this time factor, the degree of consolidation was calculated to be approximately 78 percent, yielding a value of peak shear transfer equal to 86 percent of the undrained shear strength.

The nonlinear shear transfer versus displacement curves were then created, using the equations defined earlier in Section 4.2. Values of displacement u / u_{peak} were determined for values of f / f_{max} of 0.50, 0.70, 0.85, and 1.0, where u_{peak} is the displacement at the peak value of shear transfer and is the same as u_{ref} mentioned previously. Values of u_{peak} were determined from the equation

$$u_{peak} = 0.0001 \, D \, X$$

where: D is the pile diameter, in inches, and

X is the depth, in ft.

The values of depth X were calculated from the two-layered soil system shown in Plate 4.14. For the depths above 90 ft (27.5 m), the displacement values were calculated with respect to a fictitious mudline at 20 ft (6 m) above the actual mudline; for the depths below 90 ft (27.5 m), a fictitious mudline at a depth of 80 ft (24 m) was used.

The nonlinear support curves for the static load tests were then constructed, with the values of the peak shear transfer being either those for the time immediately after driving, or at 115 days after driving. For the immediate load test, the shear transfer was assumed to be constant after yield. For the test performed in April, 1984, the values of the residual shear transfer were taken as 0.80 times the peak shear transfer, with the peak values reducing to the residual values at a displacement 0.3 inch (7.6 mm) (0.01 D) greater than the displacement at the static peak.

The two sets of nonlinear support curves were then used as input for the AXCOL program to hindcast the results of the immediate and the 3-month load tests.

The results of the axial pile solutions for the static load test performed immediately after driving the pile are compared in Plate 4.19 with the observed behavior. As seen in the plate, excellent agreement was obtained between the predicted and the measured load-displacement behavior of the pile, throughout the range of loading. The predicted maximum pile capacity was also near that measured, being 440 kips (1958 kN) during plastic slip, as compared with the measured value of 410 kips (1824 kN).

The results of similar solutions for the static load test performed after 3 months of consolidation are shown in Plate 4.20. Again, excellent agreement is shown throughout the range of loading.

It was noted in Ref 1 that, during the load test, the maximum pile head load was somewhat less than that which would be calculated from the integration of the values of peak shear transfer which were developed along the pile. Similar behavior is demonstrated in Plate 4.20. For the instrumented pile test, a pile capacity of 1070 kips (4761 kN) would be calculated from the reported values of shear transfer, as compared with the measured value of 963 kips (4285 kN). The comparable values from the use of the proposed design methods yield a value of 1040 kips (4628 kN) for the integrated values of peak shear transfer, and a peak pile head load of 921 kips (4099 kN) from the computer simulation of the load test.

Thus, the recommended methods of predicting the peak shear transfer with time, and the construction of the nonlinear support curves, used in a computer program which properly models the pile-soil interaction, have been demonstrated to result in a very close approximation of the real behavior of the instrumented test pile under static, monotonic tension loading. This demonstrates that the proposed method, based on a combination of pile segment and full scale test pile results, is consistent and thus can be reasonably applied to other piles installed at other sites.

5.0 DESIGN EXAMPLE OF A TENSION PILE IN NORMALLY CONSOLIDATED CLAY

In this chapter, the design procedures outlined in Chapter 4 will be summarized, and then applied to the design of a single tension pile in a normally consolidated clay.

5.1 Summary of the Recommended Design Procedures

The procedures to be used for estimating the time-dependent increase in axial pile capacity in normally-consolidated clays consist of four steps:

1. Establish a relationship between the degree of consolidation and the empirical time factor, T_f, using the following relationship:

$$T_f = \frac{0.012\ U}{1.0 - 0.94\ U}$$

   where U is the degree of consolidation, expressed as a decimal fraction.

2. Establish a relationship between the time factor, T_f, and the peak shear transfer, f_{max}, on the basis of a linear relationship between a value (f_{max} / S_u) of 0.33 immediately after driving, at U = 0.0, and a value of 1.0 at U = 1.00.

   It should be noted that, except during the very early stages of consolidation, the calculated value of (f_{max}/ S_u) is not extremely sensitive to the value selected for the minimum value, immediately after driving. It is not intended that the value of 0.33 be used to predict the dynamic soil resistance during driving, but to be used to reasonably predict the static capacity later in the life of the pile. Actual values, measured with the 1.72-inch-diameter model, the 3-inch-diameter models, and the large pile, were temporarily much lower, with individual values being as low as 0.1.

For clays with greater sensitivity, the value of 0.33 should probably be reduced, to reflect the higher degree of loss in shear strength due to the severe remolding of the near-pile clay.

3. Using a representative value of the consolidation coefficient, C_v, from laboratory consolidation tests, calculate a value for the time factor, T_f, at the desired time after driving, using the relationship

$$T_f = \frac{C_v t}{D^2 (100 - 2D / w_t)}$$

where: t is the time after driving,

T_f is the empirical time factor,

D is the pile diameter,

w_t is the wall thickness of the pile, and

C_v is the coefficient of consolidation.

4. Using the relationships derived in Step 2, apply the factor (f_{max}/ S_u) to the shear strength to determine a calculated ultimate pile capacity.

For a particular site, a chart or table may be constructed, using the particular values of C_v for the site, and the pile diameter and wall thickness varied to aid in selecting a pile (or piles) having the required capacity at the desired time in the schedule of platform construction and operation.

By varying the time t, the axial capacity of particular piles (with constant wall thickness) may be estimated at various times, to aid in planning and scheduling construction operations.

Once the distribution with depth of the peak shear transfer, f_{max}, has been determined, and a pile diameter and length selected, the nonlinear curves relating the development of shear transfer and pile displacement can be constructed.

5. The nondimensional curve shape may be constructed from the following equation:

$$(u / u_{peak}) = \frac{0.24 (f / f_{max})}{1.0 - 0.76 (f / f_{max})}$$

where (f / f_{max}) is the percentage of peak shear transfer, and

(u / u_{peak}) is the corresponding displacement ratio,

in which $u_{peak} = 0.0001$ D X, where X is the depth to the curve being considered.

Only a few points along the curve are required for the proper modelling of axial pile-soil interaction. It is suggested that the points be limited to those at values of (f / f_{max}) of 0.50, 0.70, 0.85 and 1.00. The corresponding values of (u / u_{peak}) are 0.19, 0.36, 0.58, and 1.00. As may be noted in Plates 4.19 and 4.20, these points will allow the axial pile response to be predicted with sufficient accuracy.

For cyclic pile solutions using the DRIVE computer program, the construction of the curve is complete.

For static pile solutions, the post-yield portion of the curve requires two additional points: a value of 0.80 f_{max} at a displacement of (u_{peak} + 0.01 D), and the same value at a large displacement, to provide a flat portion of the support curve.

5.2 Application of the Design Methods to a Typical Tension Pile

In practice, the design of a tension pile subjected to cyclic loads cannot be adequately performed without assessing the performance of the pile under loading conditions which, although fictitious, simulate the actual service conditions under which the foundation will be expected to perform. The assessment of the performance of such a pile requires that the effects of cyclic loading on the axial capacity of the pile be estimated, and that either the design loads, or the numbers or lengths of piles, be revised in order to provide confidence in the design of the foundation.

The recommended steps in the design of a single tension pile are given in the following section, in which the design procedures are applied to the design and analysis of a typical tension pile.

The soil profile at the site is shown in Plate 5.1. The soil profile indicates a normally-consolidated clay of high plasticity, with an undrained shear strength of 0.1 ksf (4.79 kPa) at the mudline, which increases with depth at the rate of 0.01 ksf per foot (1.57 kPa/m).

The results of laboratory tests indicate a sensitivity of approximately 2, and an average value of the consolidation coefficient C_v of $7.8 \times 10^{-5} in^2/sec$ ($5.0 \times 10^{-4} cm^2/sec$).

Using the equation given in Step 1 of Section 5.1, the chart in Plate 5.2 was constructed. At the bottom of the chart is the real-time relationship between the estimated pile capacity (from f / f_{max}) and the time after driving for two 60-inch (1.52 m) diameter piles with two wall thicknesses, 1.5 and 3.0 inches (38 and 76 mm). As seen in the Plate, the thicker-walled pile requires almost 18 years to approach the ultimate capacity, while the thinner-wall pile approaches the ultimate capacity in approximately 6 years.

A pile diameter of 60 inches (1.52 m) has been selected for the foundation, with the wall thickness of 1.5 inches (30 mm) being required for the axial stress to remain within allowable limits with the pile capacity being that at the ultimate shear transfer capacity.

The construction schedule has required that the platform be put into service approximately 12 months after driving. Using this time period to calculate a time factor of 0.034, and entering the chart on Plate 5.2, the ratio of f / S_u is determined to be 0.85.

Based on the value of 0.85, the distribution of the peak shear transfer with depth was next determined, as shown in Plate 5.3.

The variations of the peak displacement, up, with depth were then calculated, as given in Step 5 and are shown in Plate 5.3.

For the computer model of the pile, also shown in Plate 5.3, a set of nonlinear support curves describing the development of shear transfer with pile displacement are constructed, with a typical curve shape shown in Plate 5.4. For the degradation model in the DRIVE program, the fully-degraded shear transfer was taken to be equal to the remolded shear strength, or 59 percent of the 12-month capacity.

In order to establish the static axial capacity of the pile, two series of solutions were performed using the AXCOL program. One series of solutions included the post-peak residual shear transfer, to establish the peak static axial capacity, and an additional solution performed, without the post-peak residual shear transfer.

As shown in Plate 5.5, the 12-month ultimate capacity of the pile was determined to be 6700 kips (25,364 kN), including the pile weight of 272 kips (1210 kN). The actual peak static capacity, with the effects of the post-peak reductions in resistance and the progressive development of shear transfer along the pile length being considered, was calculated to be 5950 kips.

Based on the results of the static solutions, the peak shear transfer for the DRIVE analyses were reduced proportionately, to yield a maximum static capacity of 5950 kips. The results of a series of static solutions using the DRIVE computer program and the reduced values of peak shear transfer are shown in Plate 5.6.

In order to estimate the cyclic pullout capacity of the pile, a series of DRIVE solutions were performed. A bias load of approximately 1/2 the static capacity of 5950 kips (3000 kips) was placed on the pile head. The cyclic component of the load (above and below the bias) was increased in the following increments; 10, 20, 30, 35, 40, 41, 42, and 43 percent of the static bias. Ten cycles of loading were performed at each load level.

The results of the DRIVE solutions are also shown in Plate 5.6, with the pile-head displacement from the first and tenth cycles of loading being compared with those calculated from the static solutions. As seen in the Plate, the calculated pullout capacity of the pile was 5580 kips or 94 percent of the peak static capacity.

Five load levels are shown in Plate 5.6. The topmost level of load is the ultimate axial capacity of the pile, as computed using the accepted API design method. The next lower level of load is the 12-month ultimate capacity, based on the peak shear transfer acting simultaneously along the full length of the pile. Both of these load levels are fictitious values, one which a real pile in a real soil will never attain.

The next two load levels, the peak static capacity and the cyclic pullout capacity, are of most concern for design. The peak static capacity is that load level which would be attained under monotonic tension loading. The cyclic pullout capacity is that load level which would be attained under cyclic tensile loading. Thus, any consideration of safety factors and allowable load levels should be addressed toward these capacities, and not toward the unattainable ultimate static capacity.

Plate 5.7 shows the relationships among the load levels shown in Plate 5.6 and the available shear transfer in the soil. In Plate 5.7, the only constants are the undrained shear strength, on the right, and the fully degraded or remolded shear strength, on the left. The pile capacity line, shown here at a time of 12 months after driving, increases with time, according to the chart shown in Plate 5.2. The shaded area, which is the distribution of degraded shear transfer with depth, comes from the DRIVE program, and can be determined for any level of cyclic load. The distribution of shear transfer at the pullout load is shown here merely for purposes of comparison.

It should be noted that current design practice for piles in normally consolidated clays (as typified by the API RP2A (Ref 9)) uses the full shear strength of the soil, and does not consider the effects of time, the reductions in available capacity due to the progressive development of shear transfer along the pile, nor the effects of degradation on the capacity of the soil to resist cyclic loads.

A comparison of the pile capacities shown in Plate 5.6 with the ultimate capacity calculated using methods recommended in the API RP2A (as an example of current design practice) shows that the 12-month ultimate capacity (which considers the effects of remolding and shear during driving, and the subsequent time-dependent increases in axial capacity) is 85 percent of that allowed in the code. The peak static capacity (which considers the progressive development of shear transfer along the pile during static axial loading) is only 78 percent of that allowed, and the cyclic pullout capacity (which considers the effects of cyclic degradation in resistance) is even smaller at 73 percent.

5.3 Summary and Recommendations for Design

As shown in the example problem given in the preceding section, neither the static nor the cyclic axial capacities of tension piles are constant, unchanging values, such as now done in present practice. An examination of the time-dependent relationships between axial capacity and time, shown in Plate 5.2, shows that, for piles with low D / w.t. ratios (or piles driven with thick-walled driving shoes) present pile foundation designs (and associated safety factors), may be unconservative for foundation piles subjected to cyclic loads. The allowable limits recommended by API (Ref 9) include a maximum design load not to exceed 2/3 of the ultimate static capacity (i.e., a safety factor of 1.5 against pullout). The pile of the design example barely falls within the recommended limits and its actual reserve is quite small i.e., the pullout capacity of 73 percent of the ultimate long-term capacity is only slightly greater than 2/3 of the design load. Stated another way the 1.5 factor of safety has been almost fully used up. A pile having only a slightly thicker wall would not have had an actual pull out capacity equal to the design load.

It was shown in Chapter 4 that the evolution of static axial capacity with time can now be estimated reasonably well. The cyclic axial capacity is not so easily established, requiring that both the static and the cyclic components of the load be estimated reasonably well, and the computer model exercised throughout the expected range of loading to estimate the effects on the soil.

The soil model in the DRIVE program, as presently formulated, does not properly represent the post-peak reduction in resistance, as measured during the instrumented pile test. Minor modifications, which were beyond our present scope and schedule will, however, allow the soil model to properly represent the real behavior of clay soils.

REFERENCES

REFERENCES

1. Tension Pile Study, Volume IV, Testing of a 30-inch Diameter Instrumented Pile in Soft Clay, Report No. 82-200-4, June 1985.

2. Tension Pile Study, Volume V, In Site Model Pile Experiments in Soft Clay, Report No. 82-200-6, November 1985.

3. Volume I: Site Investigation and Soil Characterization Study at Block 58A, West Delta Area, Gulf of Mexico, CNRD Project 13-2 Tension Pile Study, Report No. 82-200-1, April 1982.

4. Final Technical Report, CNRD Subproject 13-1 Tension Pile Planning Study, Report No. 81-204, 28 August 1981.

5. Volume III: Final Report on Small-Diameter Pile Segment Tests, CNRD Project 13-2 Tension Pile Study Interim Technical Report No. 82-200-01, August 1983.

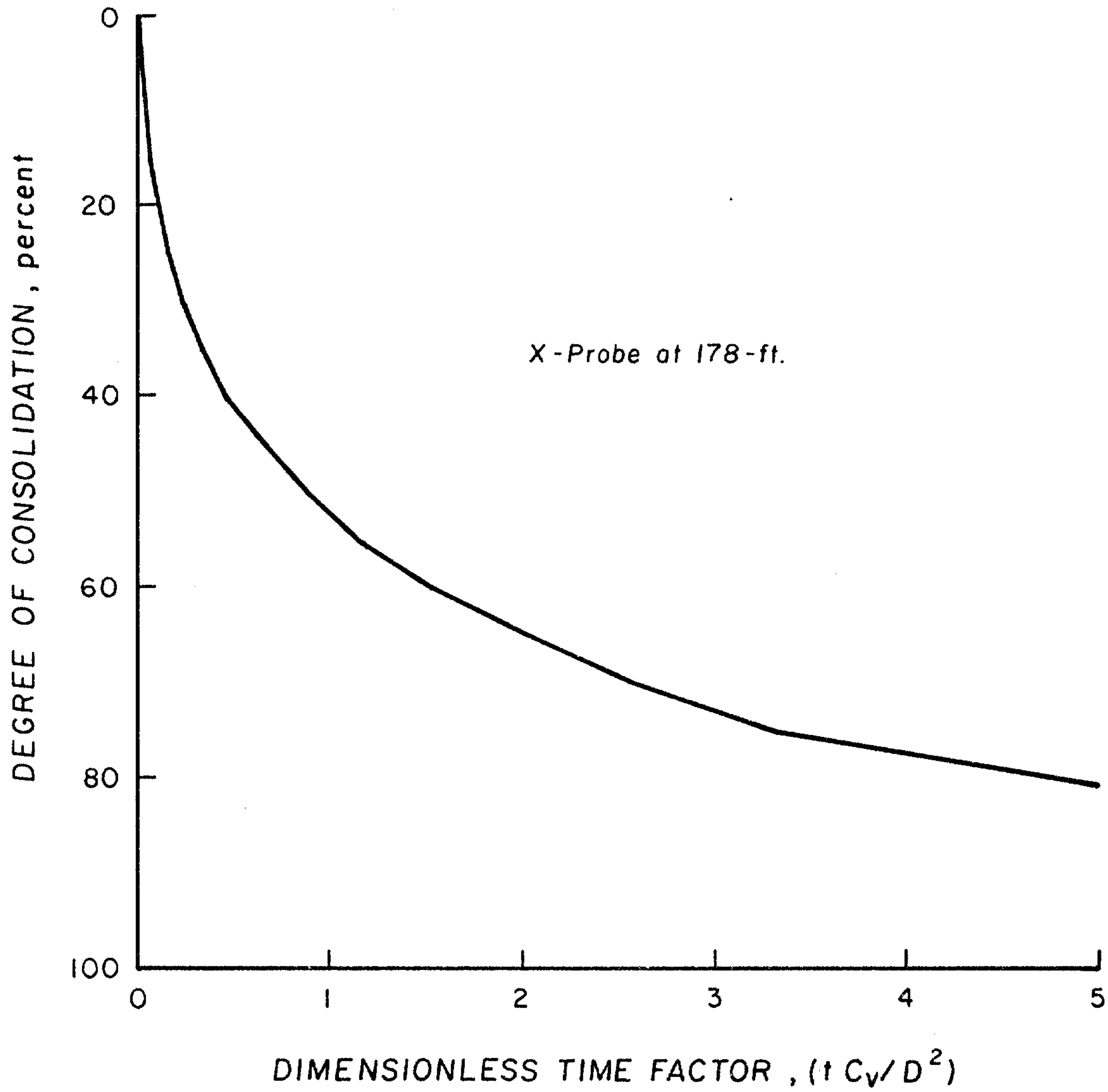
6. Final Report, CNRD 13-2: Tension Pile Study, Veritec Report No. 84-3361, June 1984.

7. Norwegian Geotechnical Institute, "Piezometer Readings up to 3.10.83", CNRD 13-2, West Delta, Block 58A, Gulf of Mexico, 10 April 1984.

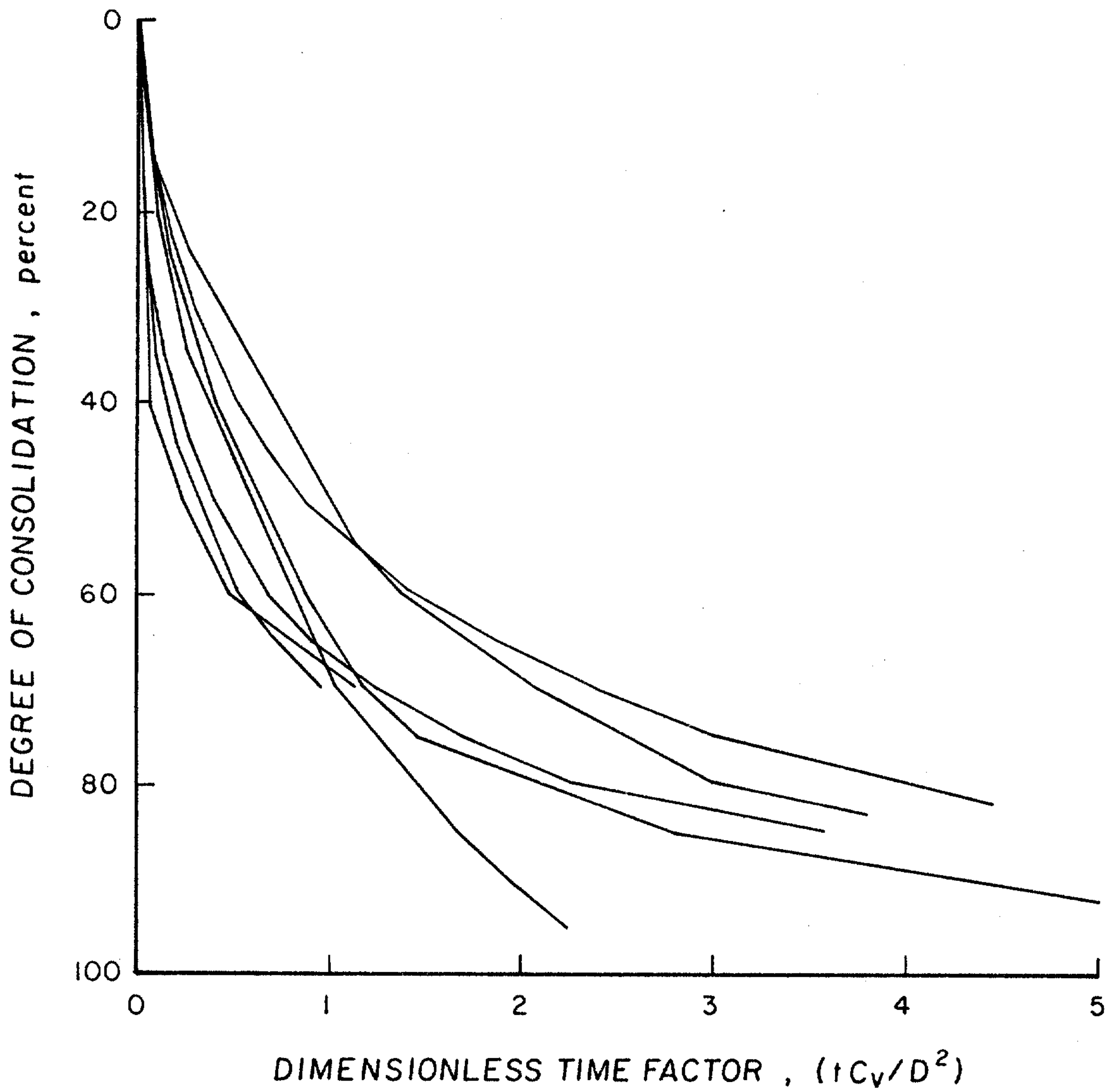
8. Matlock, Hudson, Patrick L. Meyer, and Darrel V. Holmquist, "AXCOL 3: A Program for Discrete-Element Solution of Axially Loaded Members on Nonlinear Supports", Report to the American Petroleum Institute, The University of Texas at Austin, 5 March 1976.

9. American Petroleum Institute, "APIRP2A, Recommend Practice for Planning, Designing and Constructing Fixed Offshore Platforms", Twelfth Edition, October 1984.

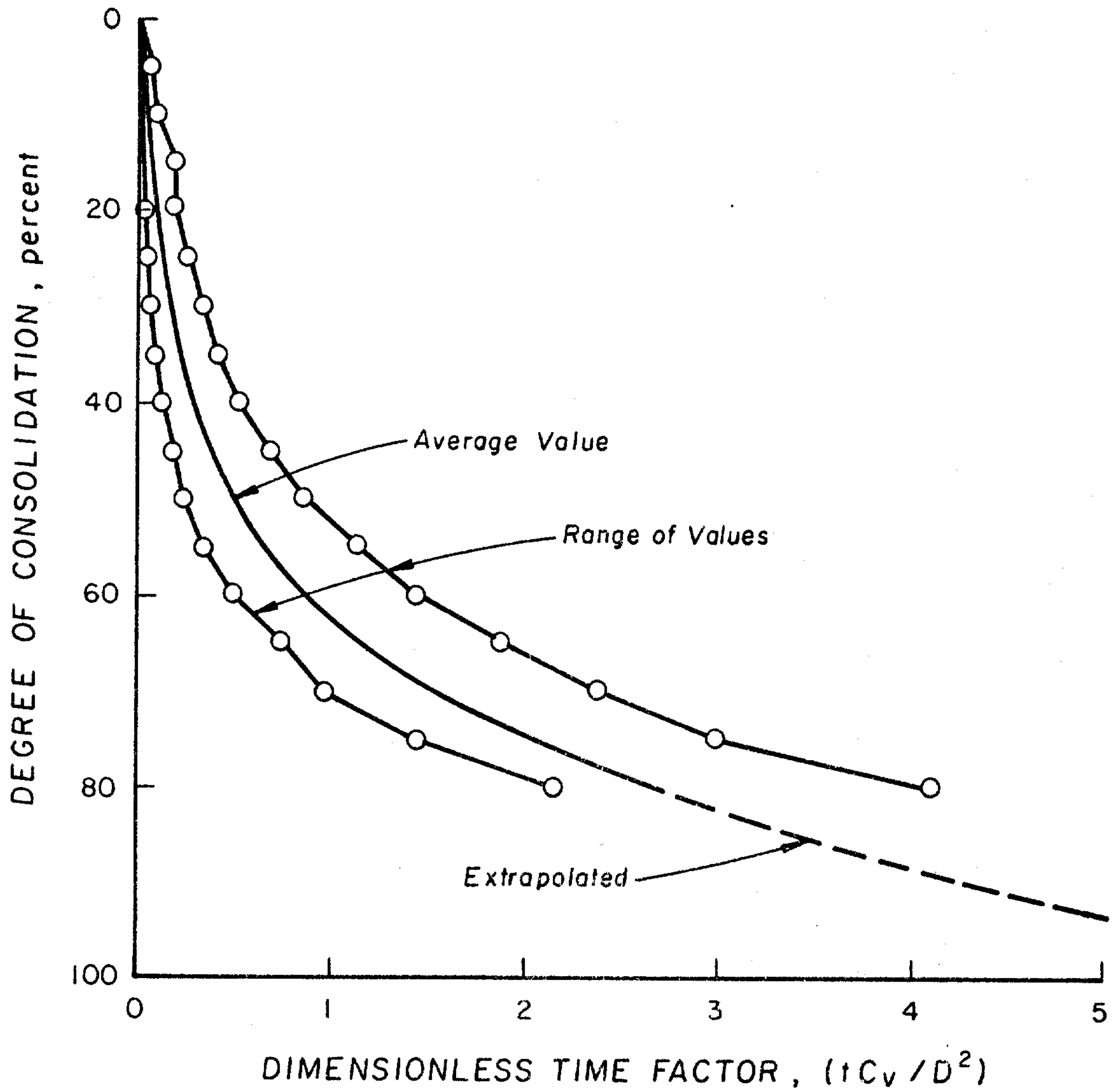
ILLUSTRATIONS



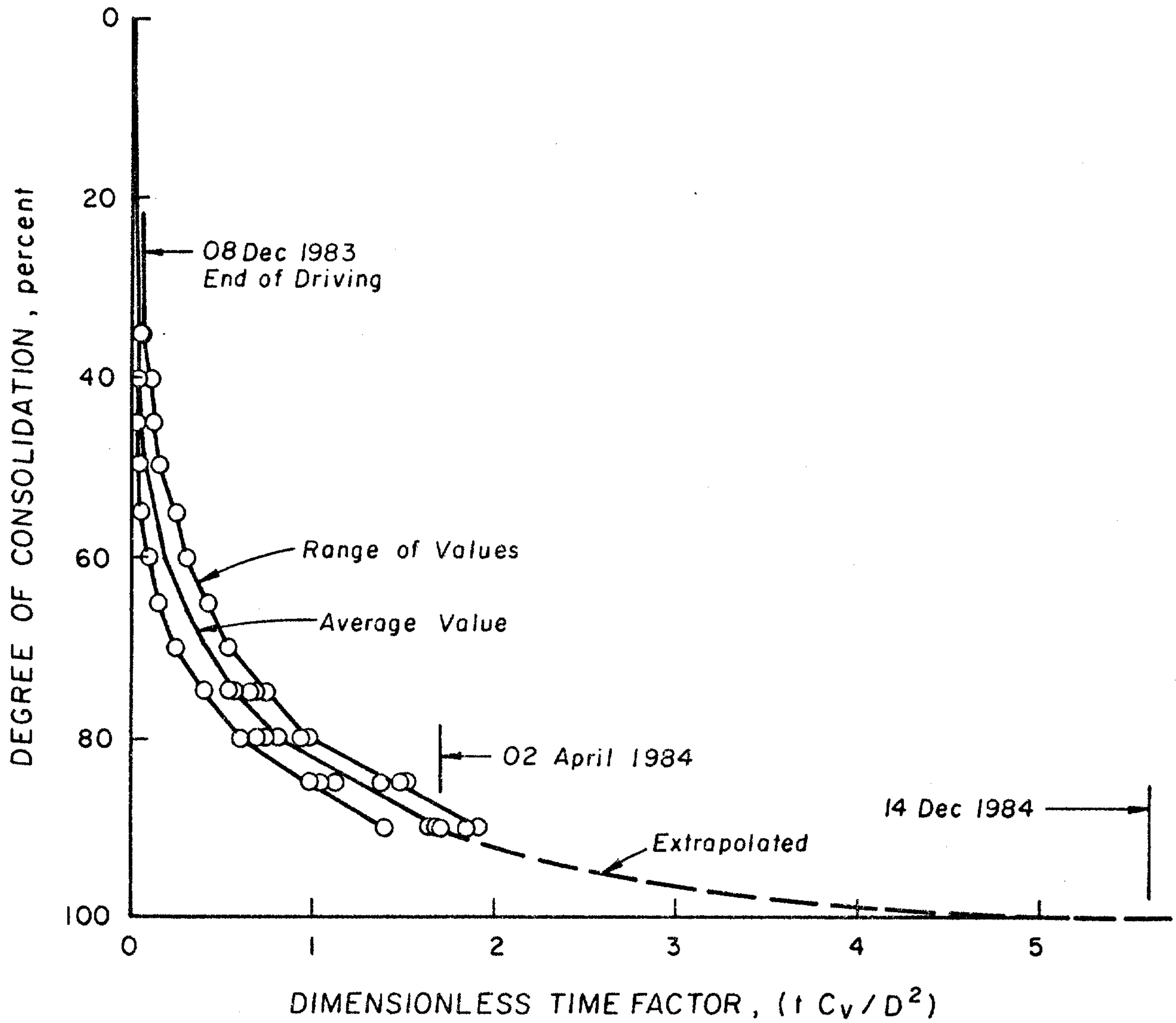
NORMALIZED PORE PRESSURE DISSIPATION DURING THE 1.72-INCH-DIAMETER X-PROBE EXPERIMENT IN STRATUM III



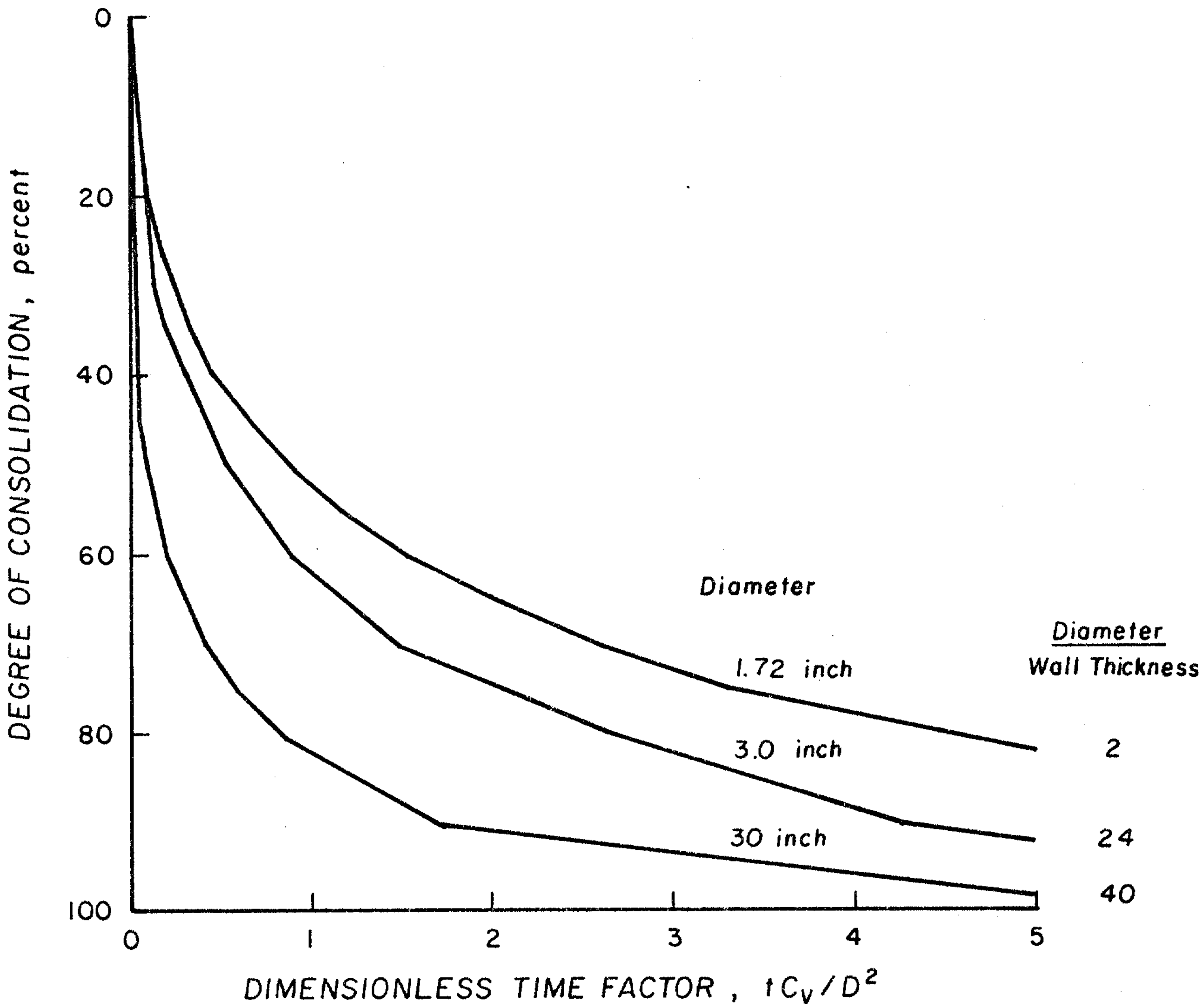
NORMALIZED PORE PRESSURE DISSIPATION DURING ALL 3-INCH DIAMETER MODEL EXPERIMENTS IN STRATUM III



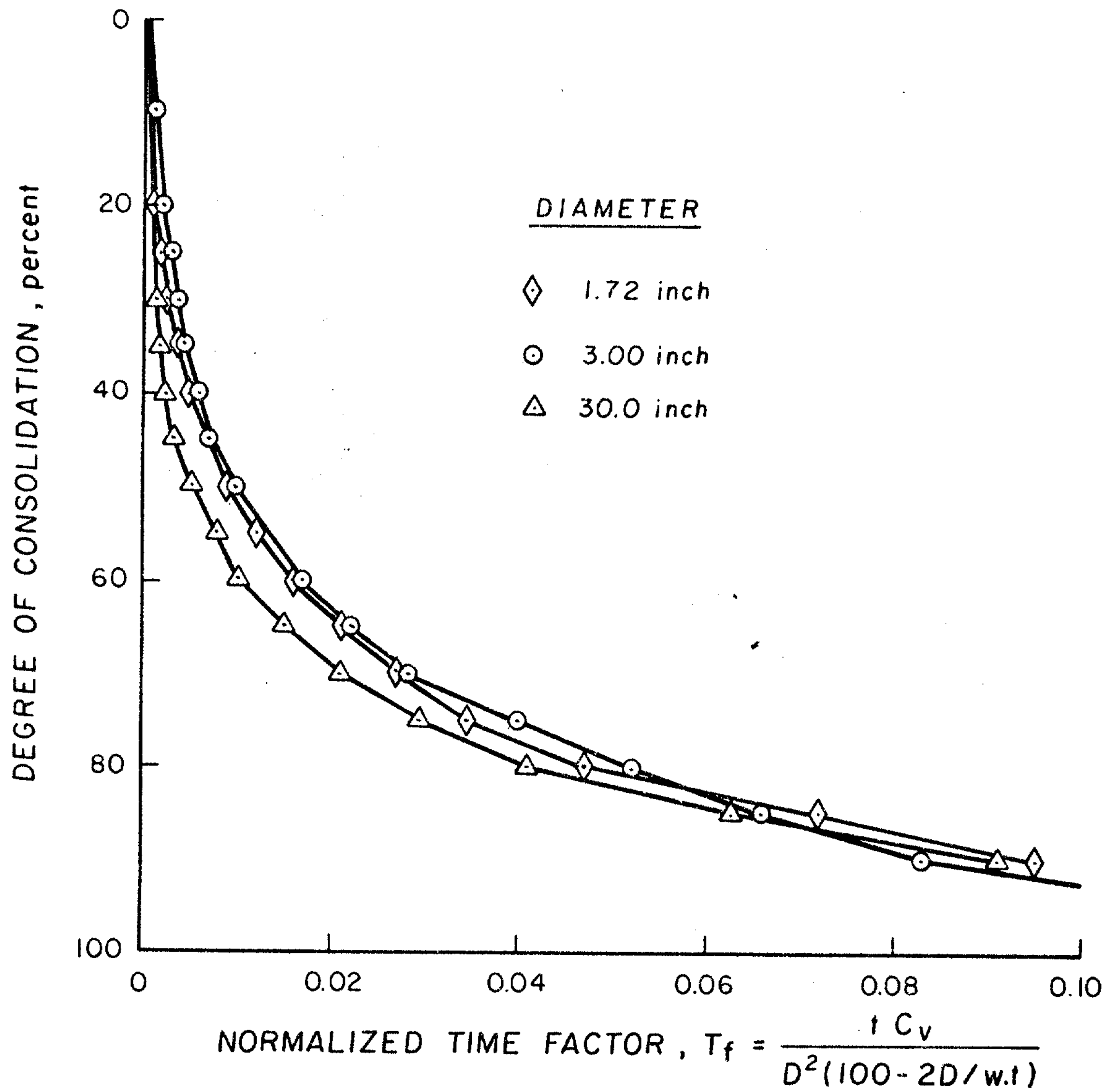
AVERAGE AND RANGE IN NORMALIZED PORE PRESSURE DISSIPATION FOR THE 3" DIAMETER MODEL EXPERIMENTS IN STRATUM III



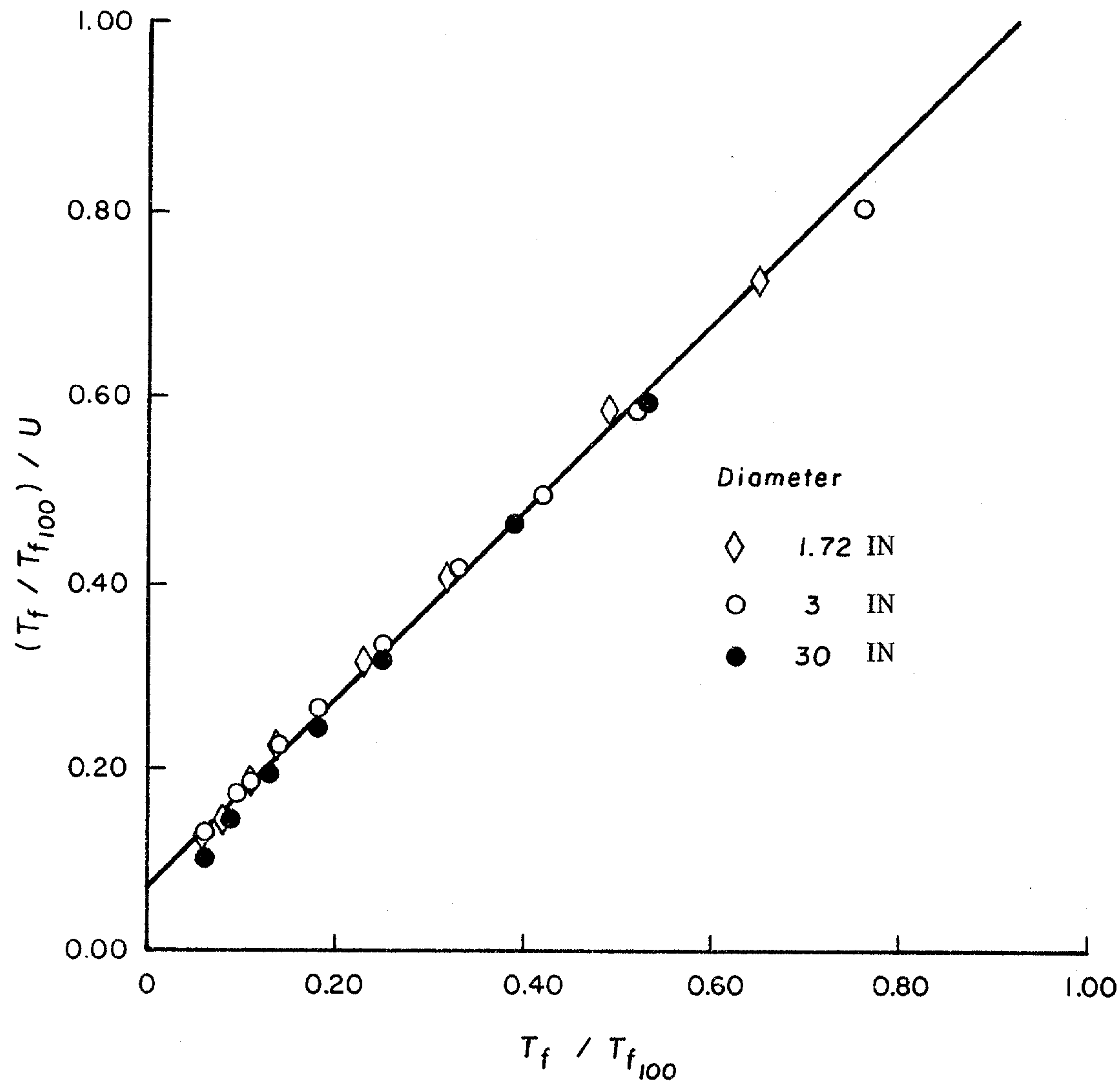
NORMALIZED PORE PRESSURE DISSIPATION DURING THE 30-INCH DIAMETER PILE TEST, ALL DEPTHS



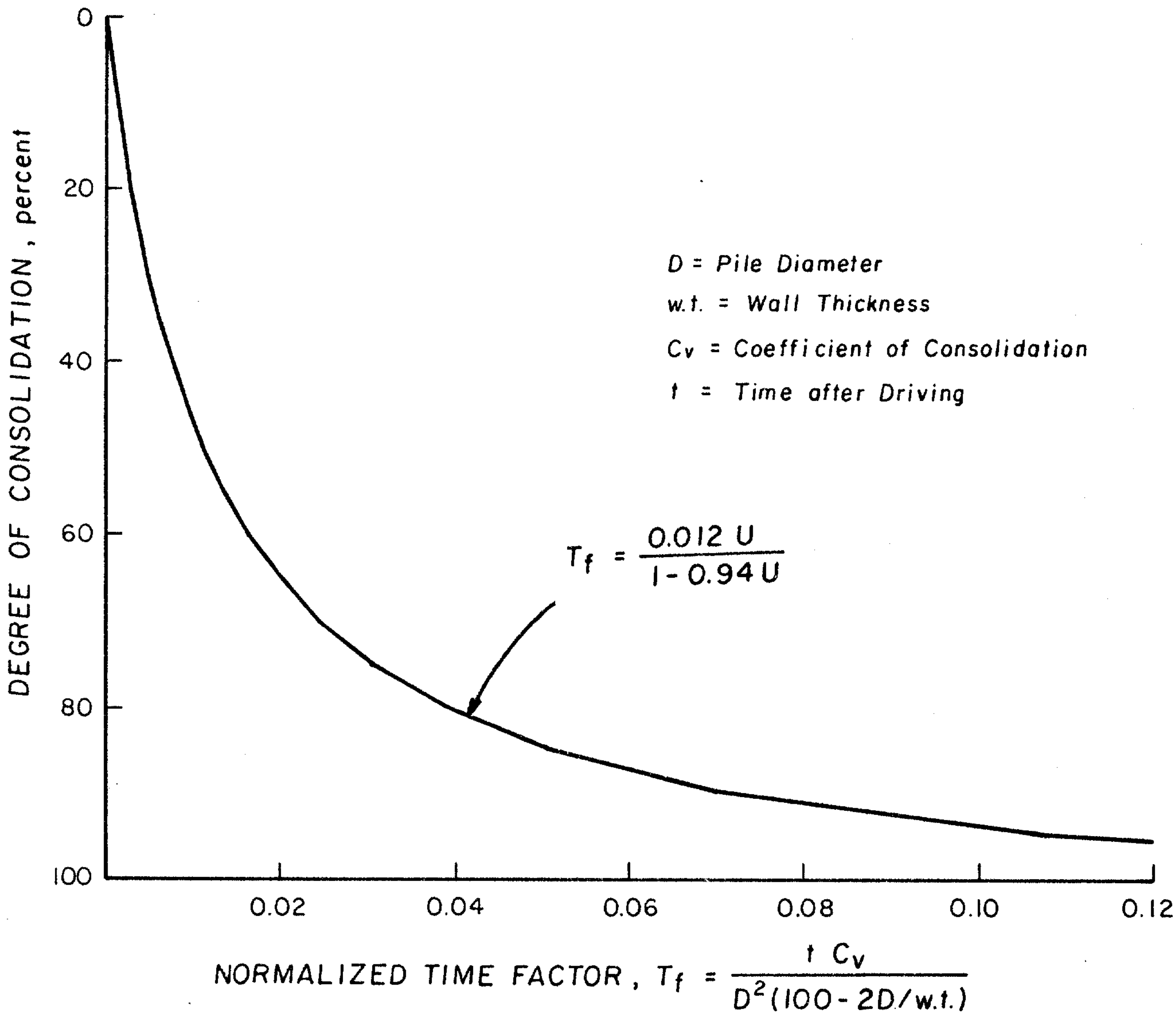
EFFECT OF $D/_{wt}$ RATIO ON NORMALIZED PORE PRESSURE DISSIPATION CURVES FROM STRATUM III



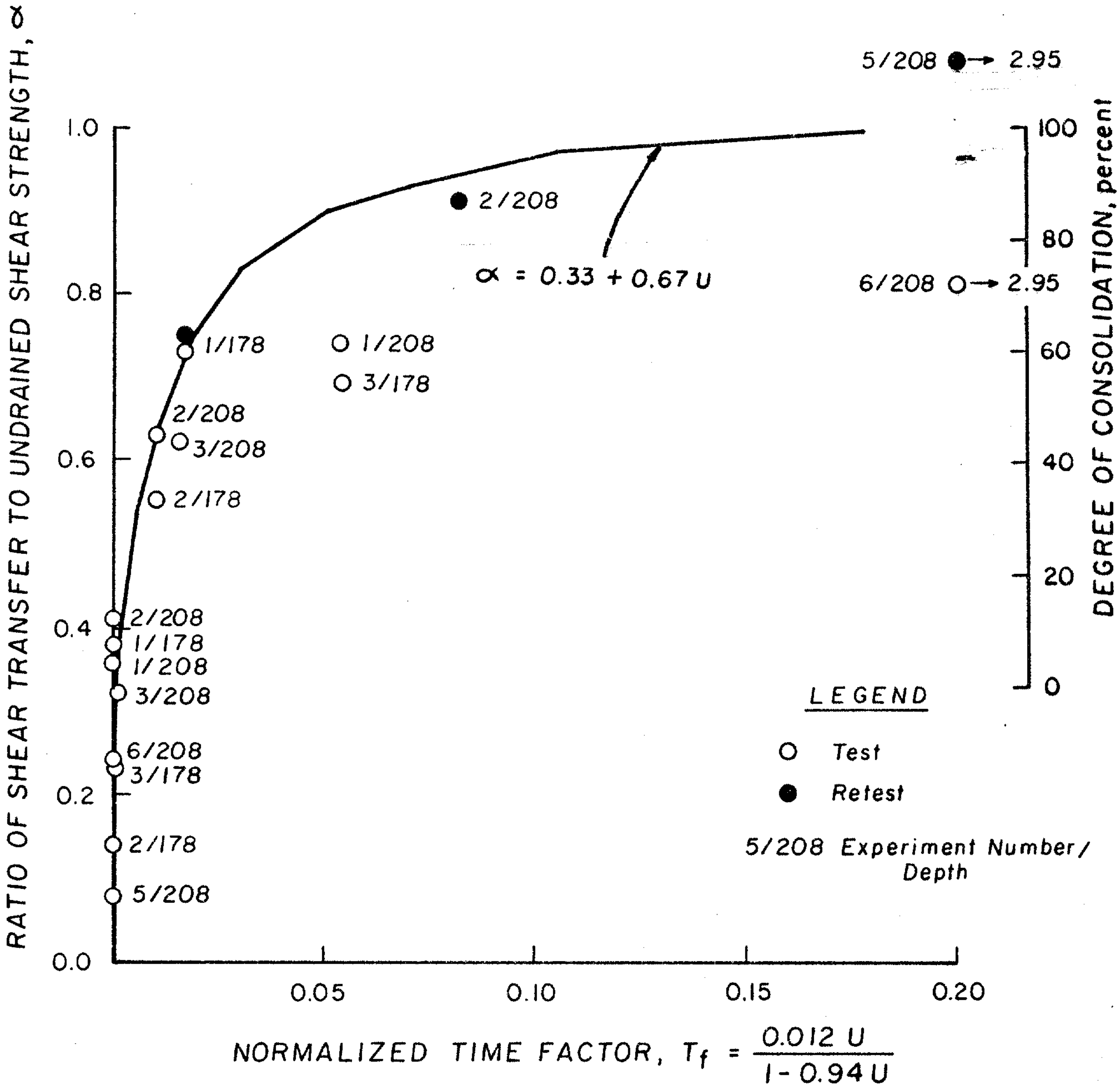
FULLY NORMALIZED PORE PRESSURE DISSIPATION CURVES FROM STRATUM III



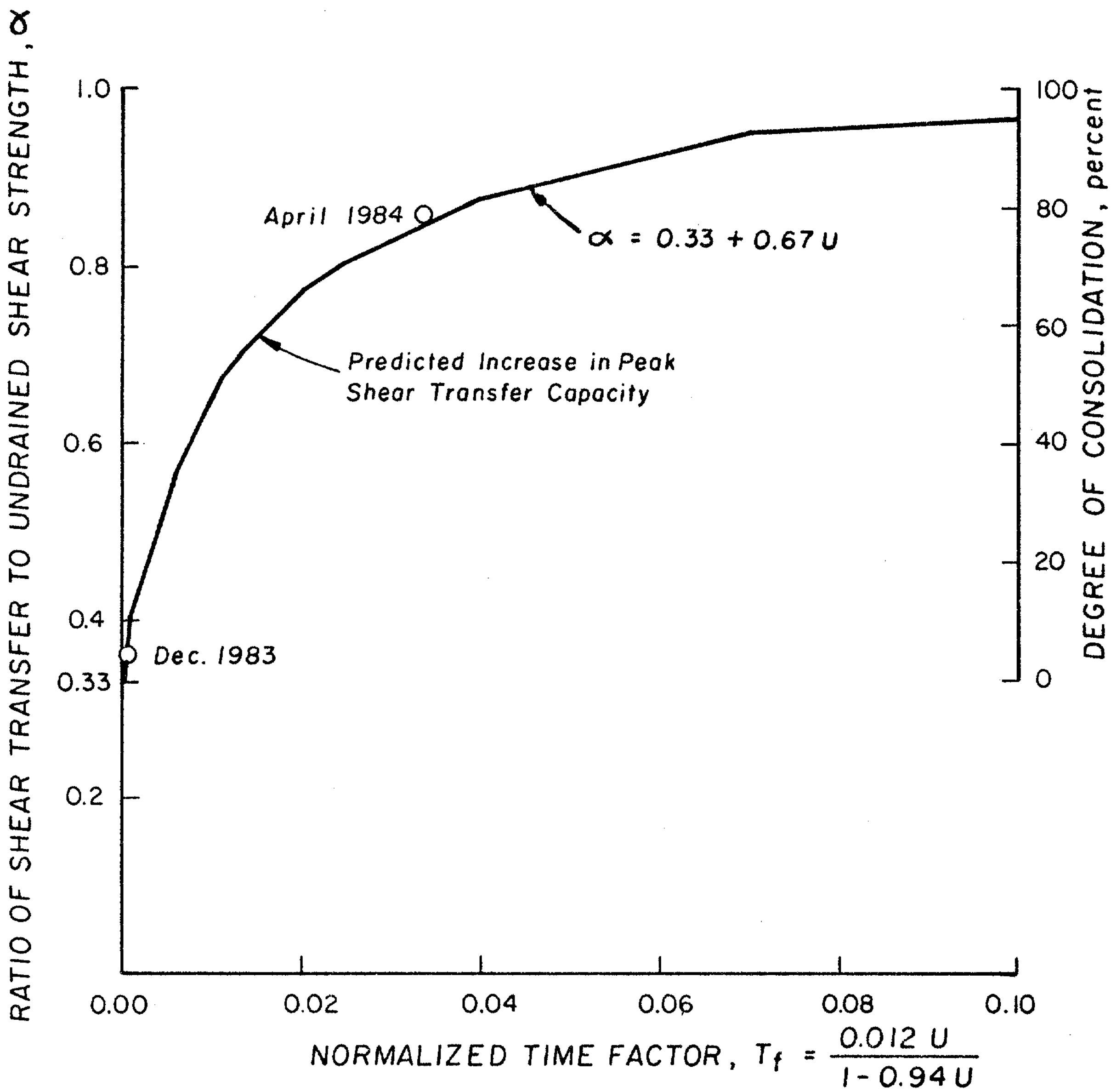
HYPERBOLIC PLOT OF FULLY NORMALIZED RELATIONSHIP SHOWN IN PLATE 4.6



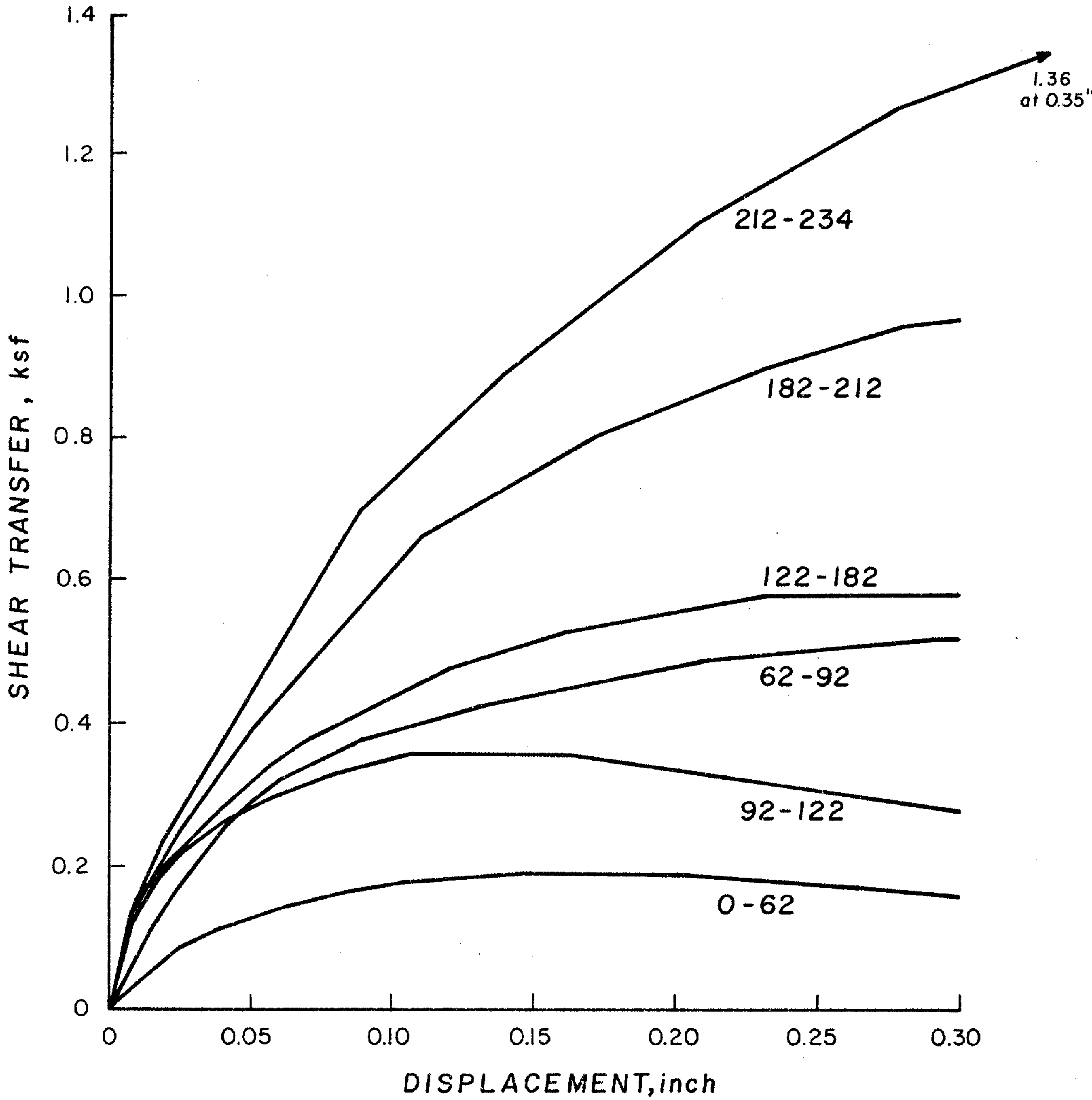
HYPERBOLIC RELATIONSHIP FOR NORMALIZED PORE PRESSURE DISSIPATION AS A FUNCTION OF THE NORMALIZED TIME FACTOR



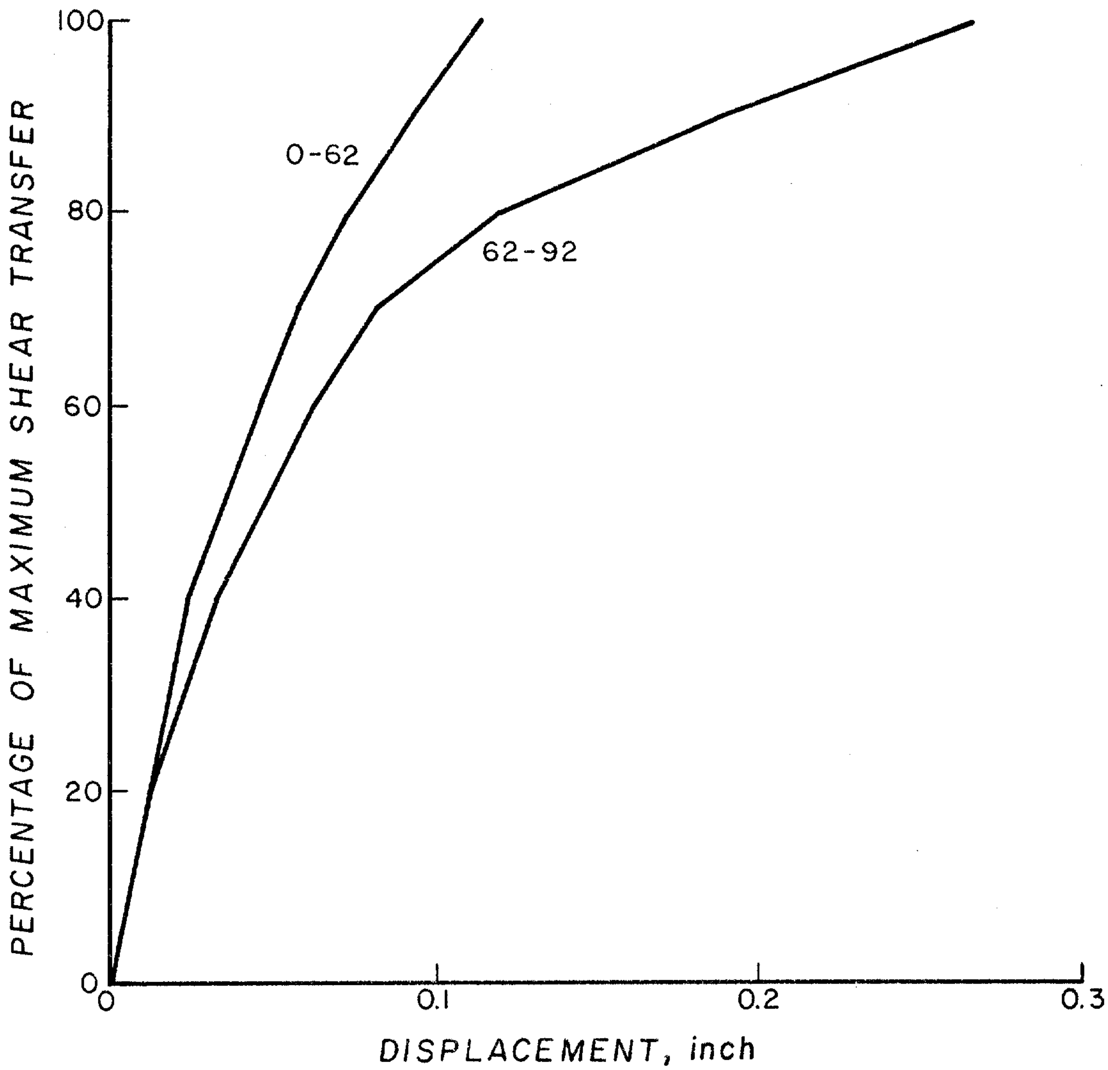
INCREASE WITH TIME IN THE SHEAR TRANSFER DURING MODEL PILE EXPERIMENTS IN STRATUM III



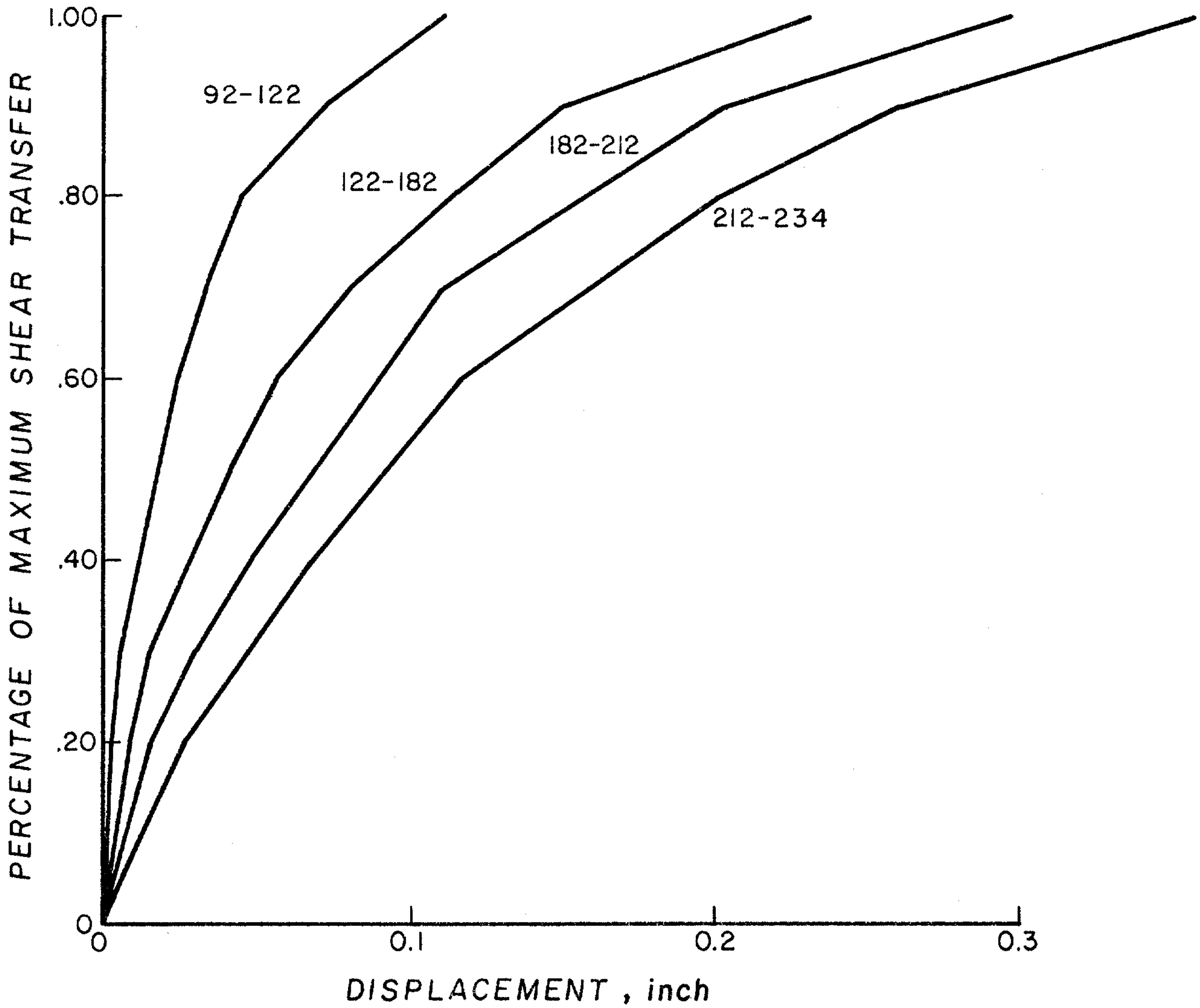
INCREASE WITH TIME IN THE AVERAGE SHEAR TRANSFER ALONG THE INSTRUMENTED TEST PILE



DEVELOPMENT OF PEAK SHEAR TRANSER WITH DISPLACEMENT ALONG THE TEST PILE



DEVELOPMENT OF SHEAR TRANSFER WITH DISPLACEMENT IN STRATUM I



DEVELOPMENT OF SHEAR TRANSFER WITH DISPLACEMENT IN STRATA II AND III

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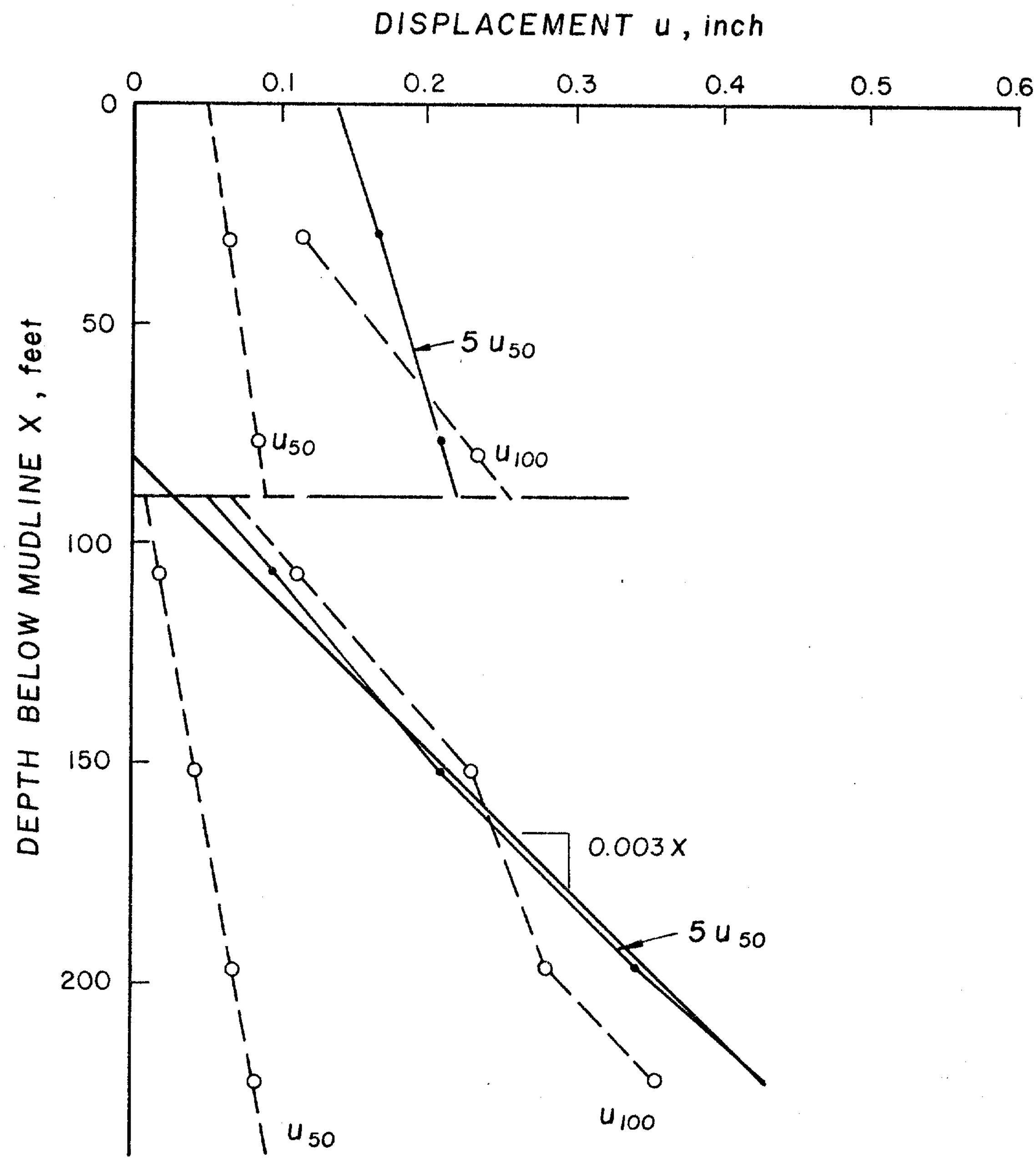
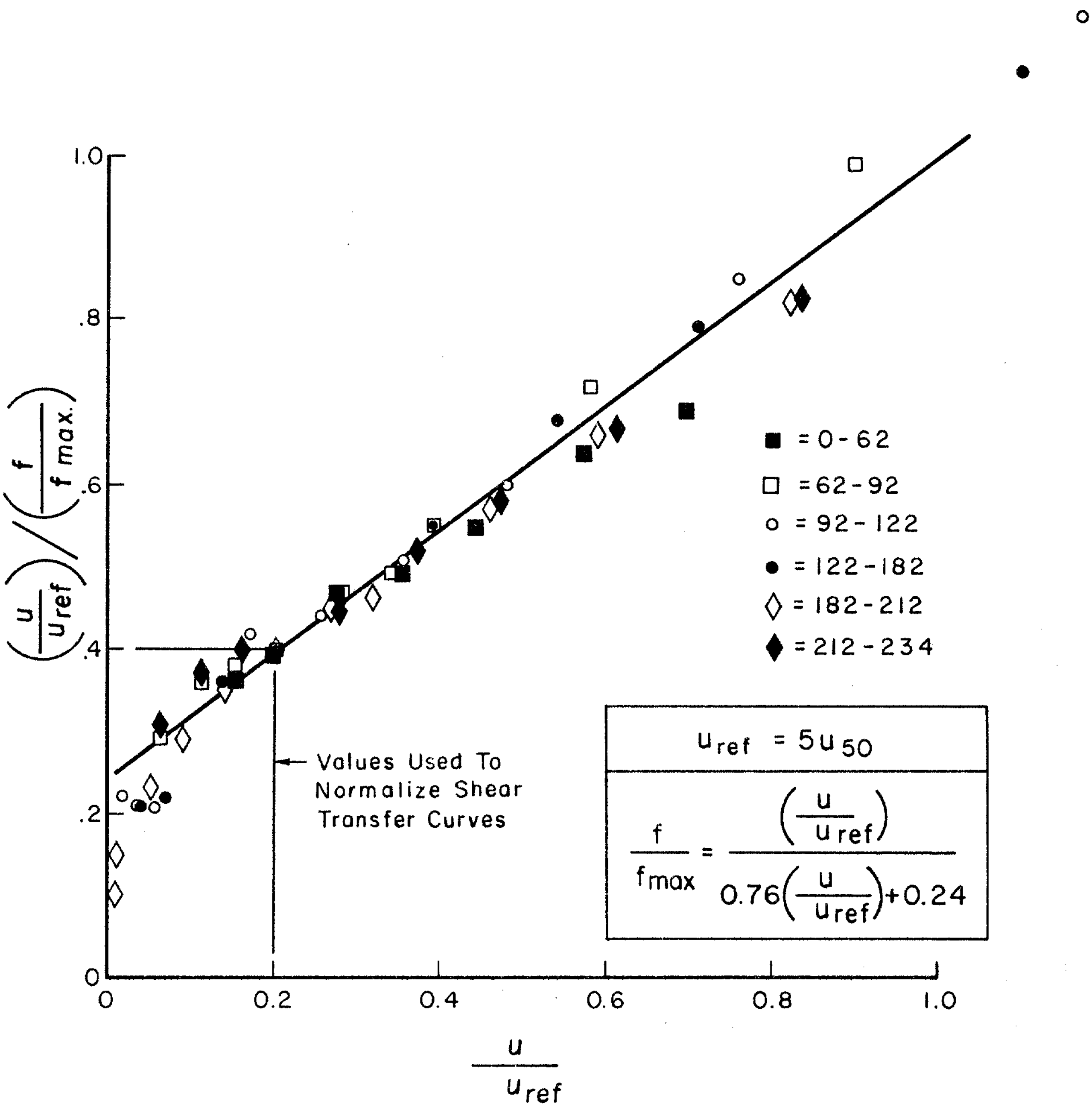
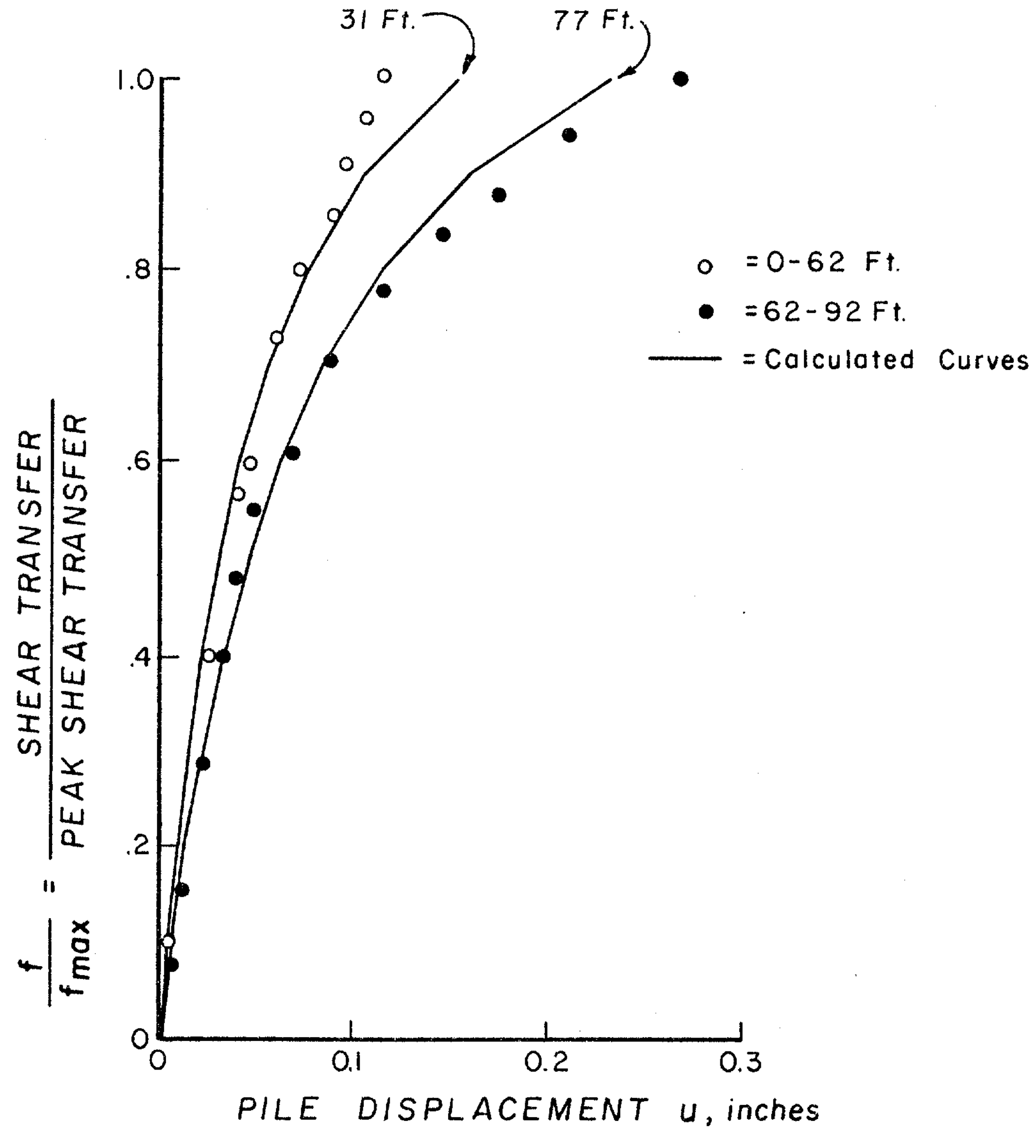
PLATE 4.13



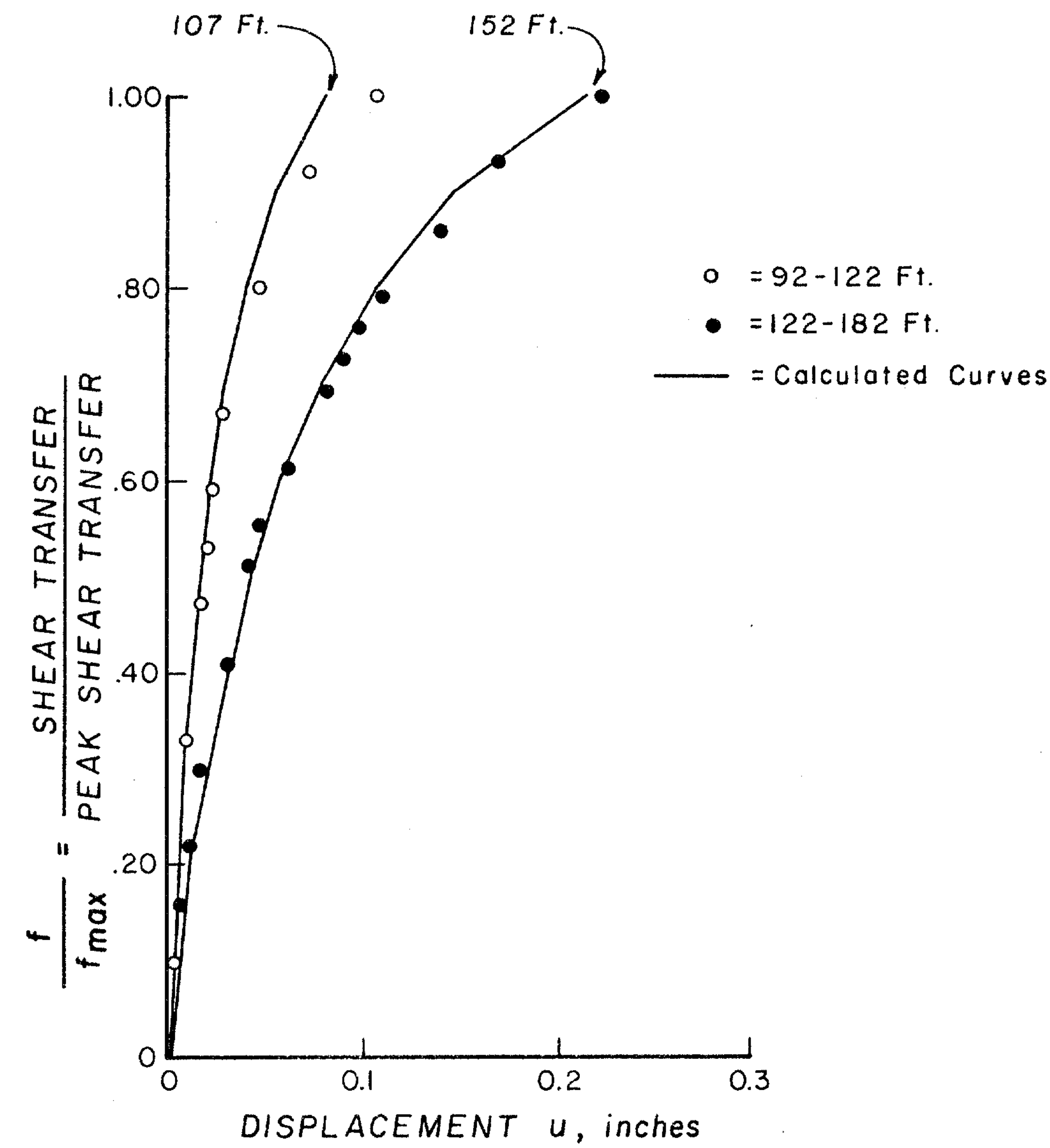
VARIATION WITH DEPTH IN THE DISPLACEMENT REQUIRED TO DEVELOP THE PEAK SHEAR TRANSFER



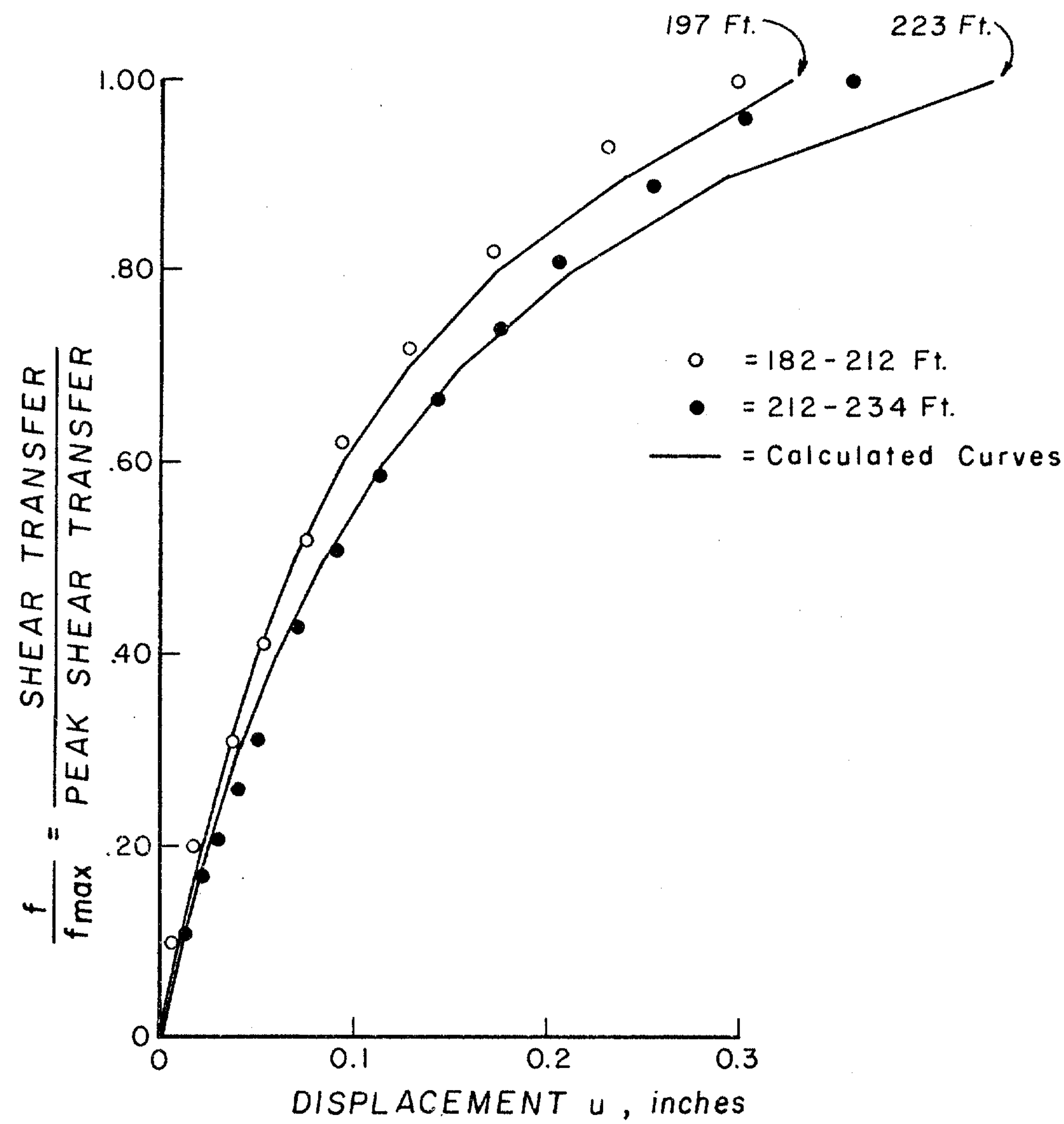
METHOD USED TO DEVELOP NORMALIZED SHEAR TRANSFER CURVES



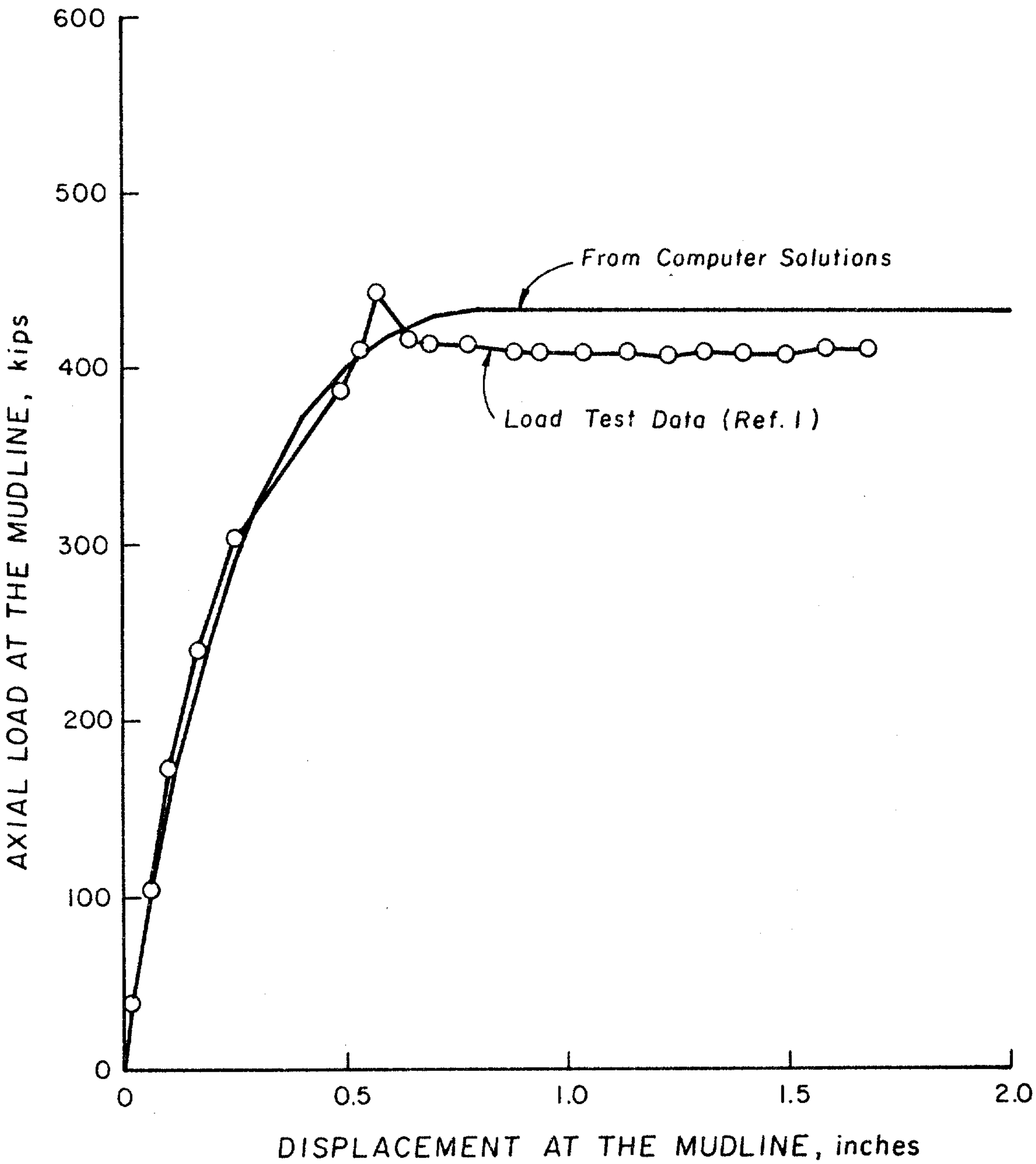
MEASURED AND CALCULATED SHEAR TRANFER CURVES IN STRATUM I



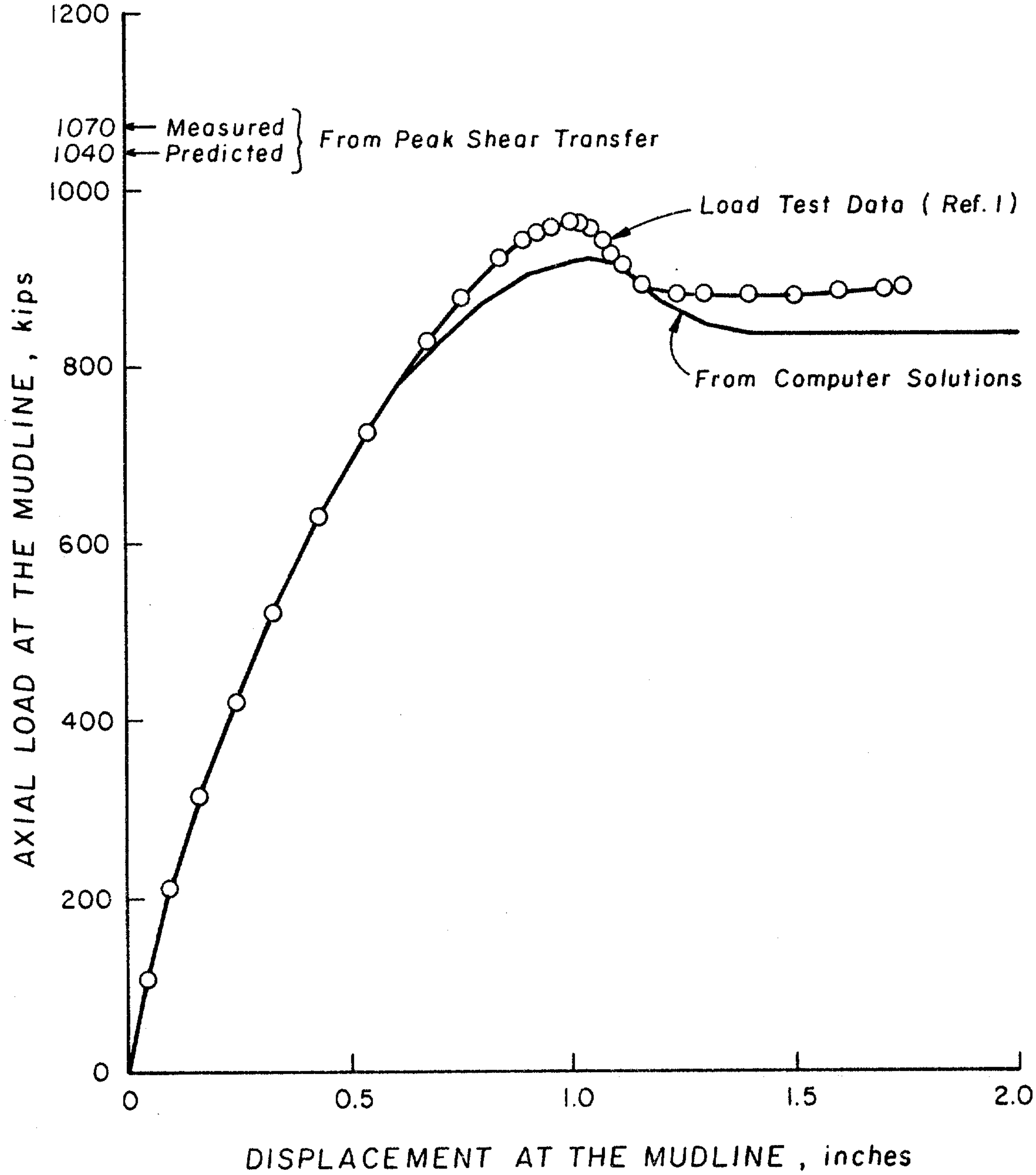
MEASURED AND CALCULATED SHEAR TRANSFER CURVES IN STRATUM II



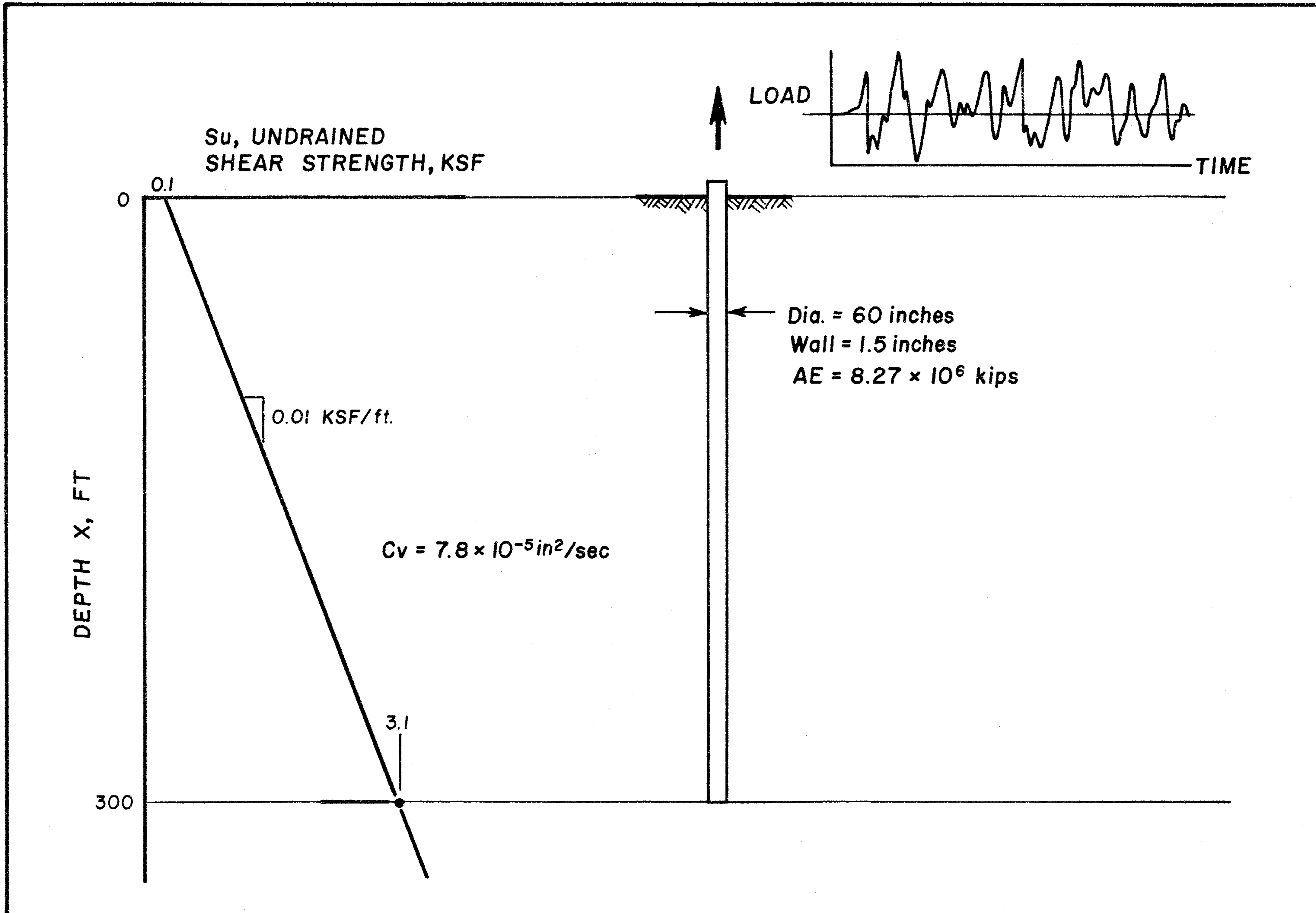
MEASURED AND CALCULATED SHEAR TRANSFER CURVES IN STRATUM III



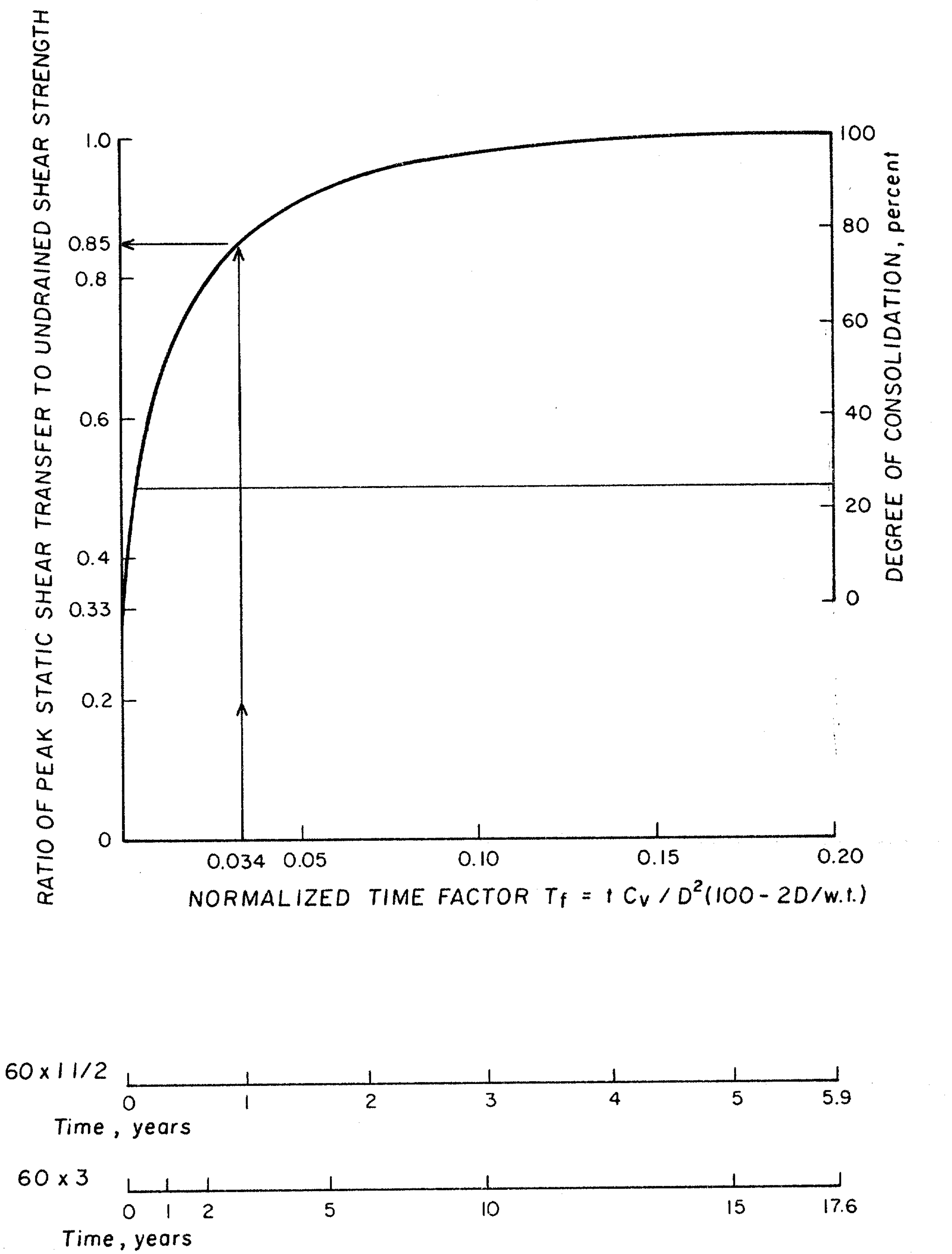
RESULTS OF THE APPLICATION OF THE TENTATIVE CRITERIA TO THE LOAD TEST PERFORMED IMMEDIATELY AFTER DRIVING



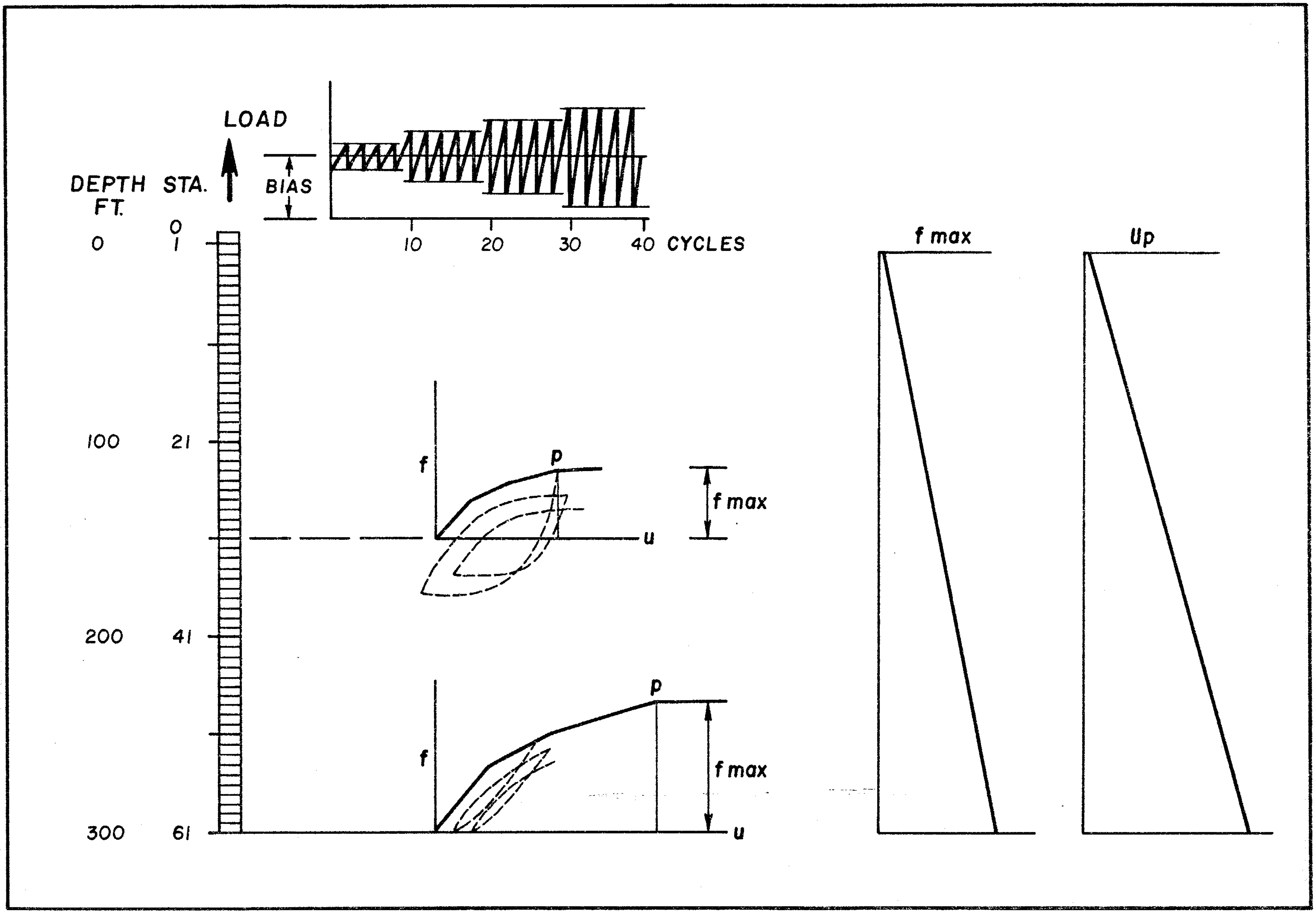
RESULTS OF THE APPLICATION OF THE TENTATIVE CRITERIA TO THE STATIC LOAD TEST PERFORMED AFTER 3 MONTHS OF CONSOLIDATION



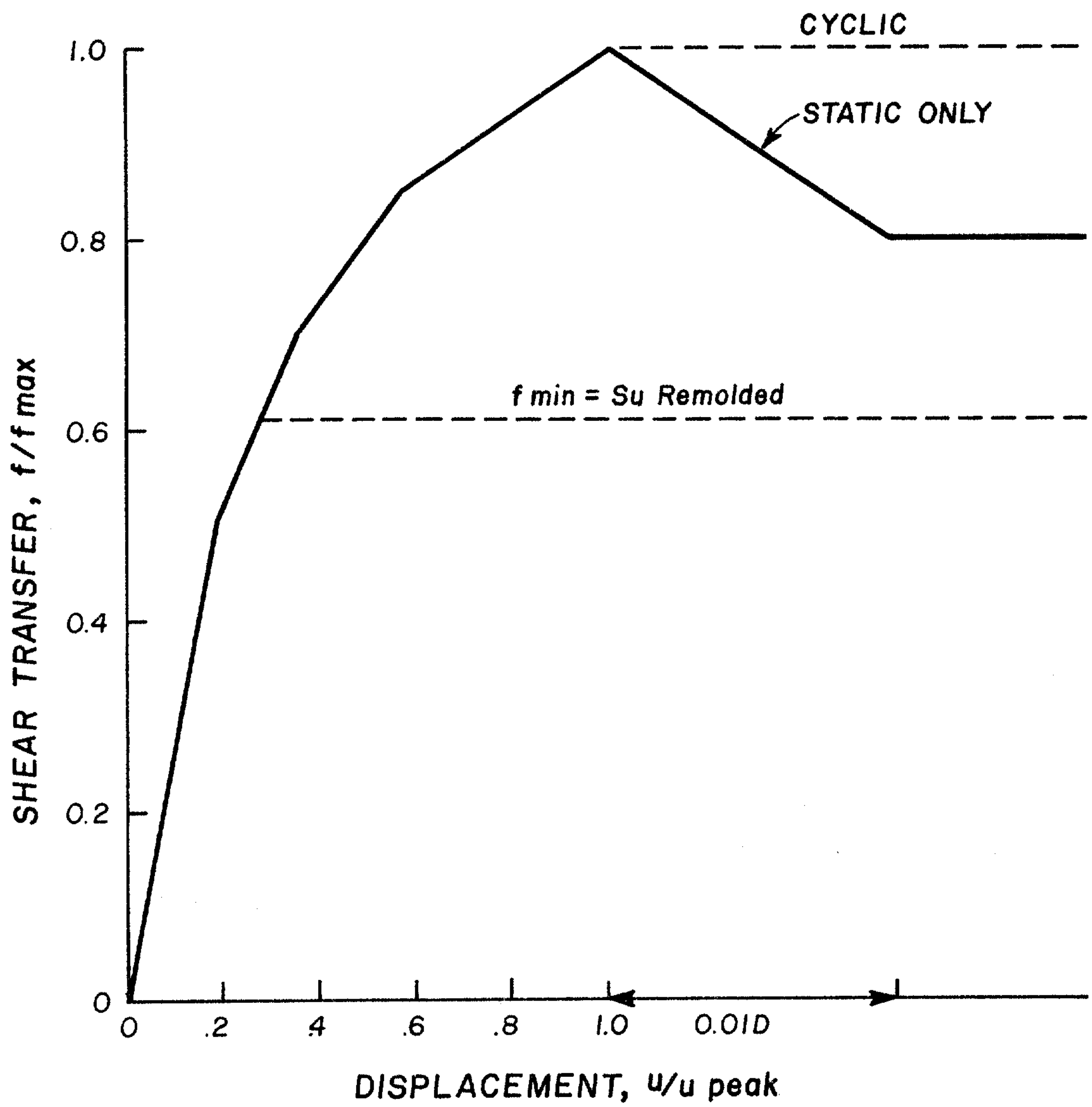
SOIL PROFILE AND TENSION PILE FOR EXAMPLE PROBLEM



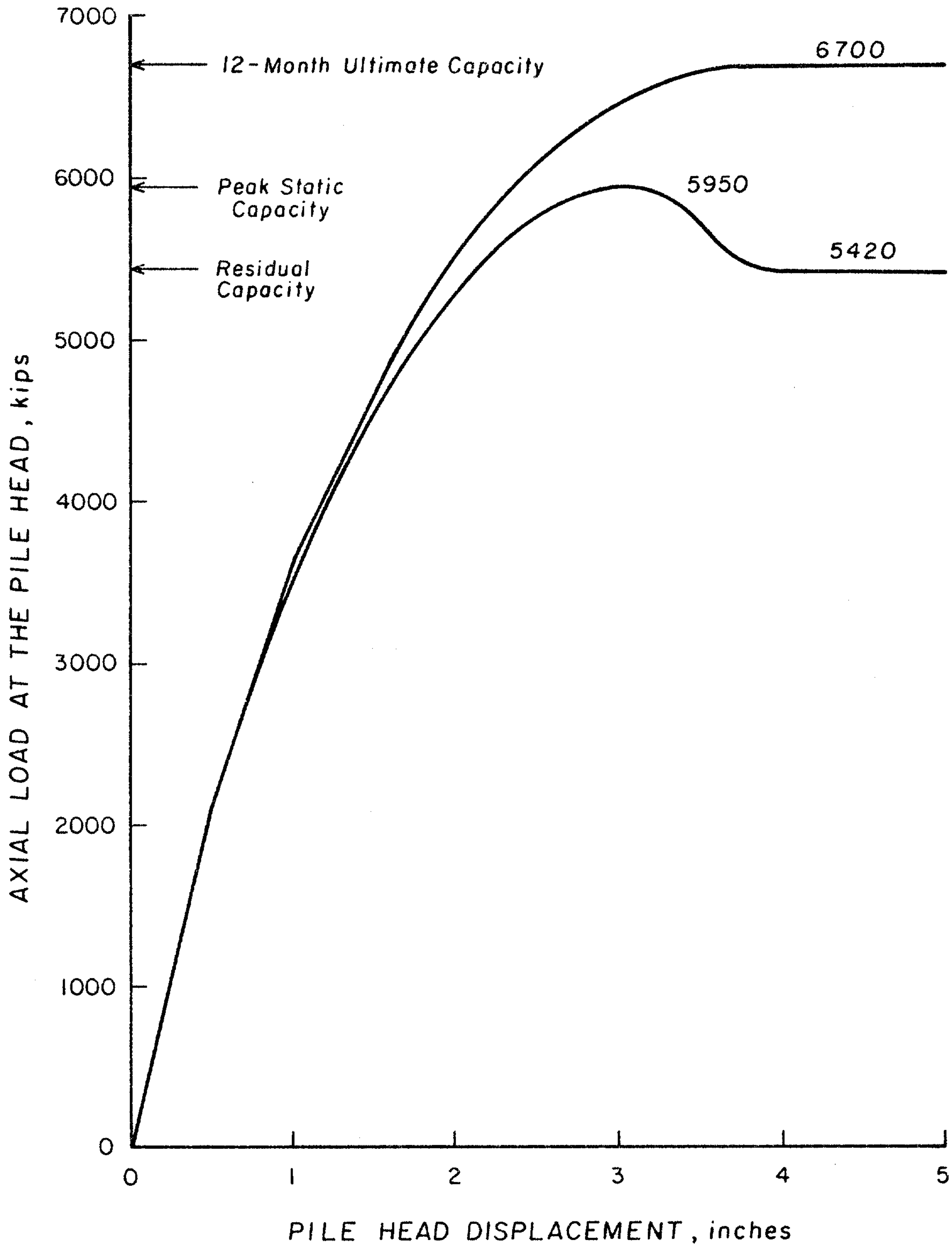
TENTATIVE DESIGN CRITERIA FOR AXIAL PILE CAPACITY, WITH TYPICAL RESULTS SHOWN FOR THE DESIGN EXAMPLE



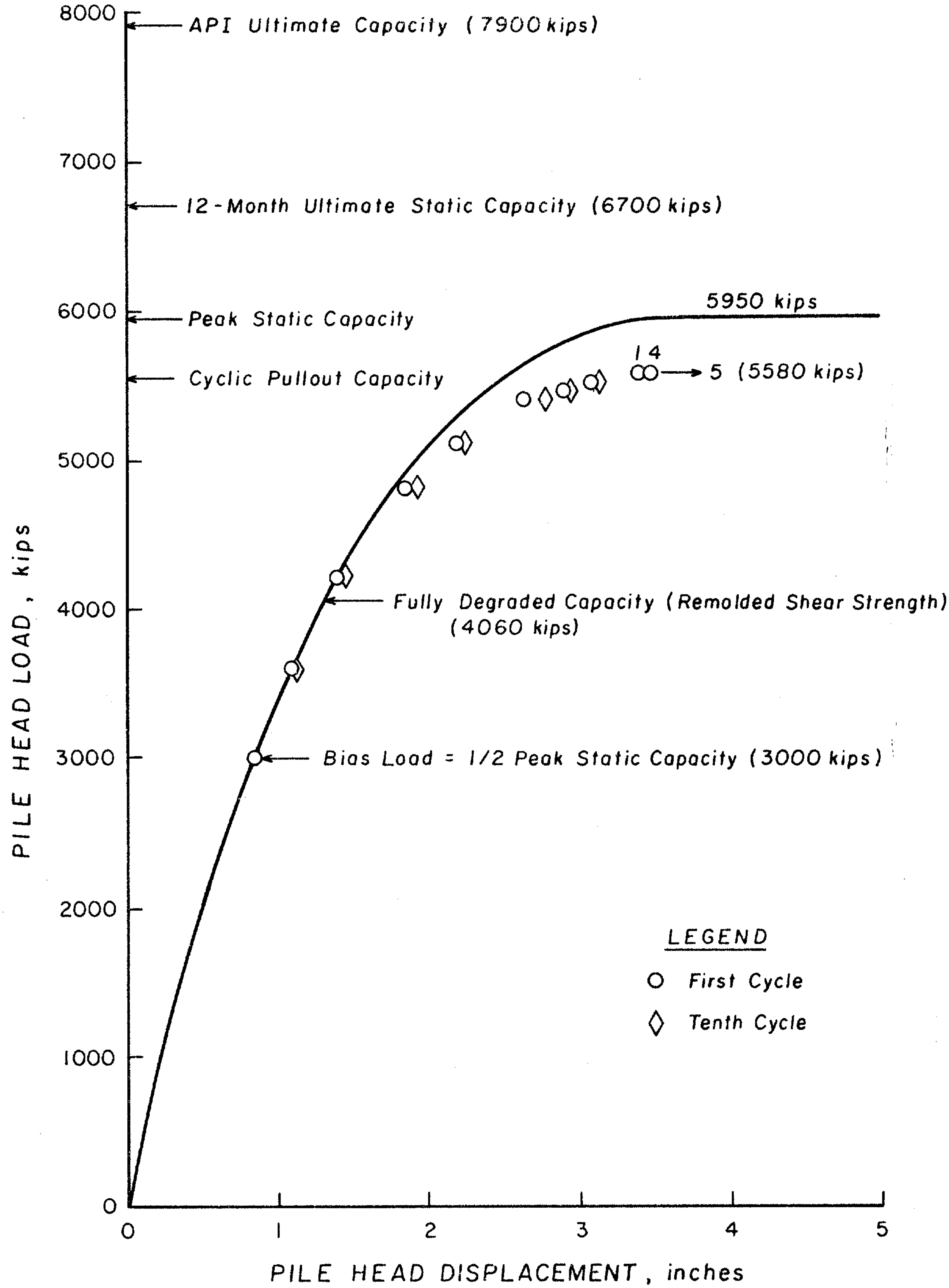
COMPUTER MODEL OF THE TENSION PILE



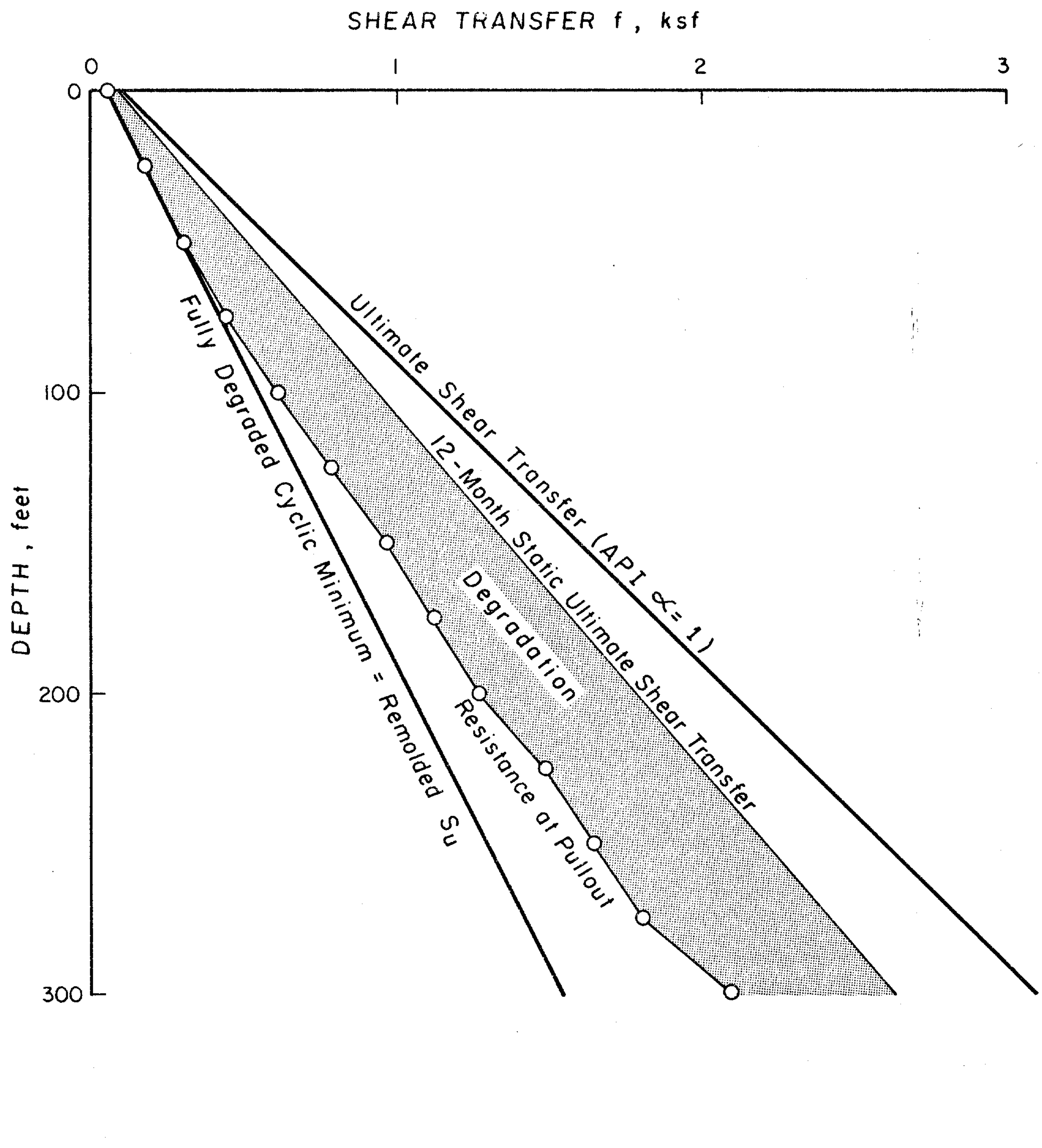
CONSTRUCTION OF SHEAR TRANSFER-DISPLACEMENT CURVES



EFFECT OF POST-PEAK REDUCTION IN SHEAR TRANSFER ON STATIC AXIAL CAPACITY



RESULTS OF STATIC AND CYCLIC SOLUTIONS, SHOWING DEFINITION OF TERMS



RESULTS OF DRIVE SOLUTION OF EXAMPLE PROBLEM

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