FINAL REPORT

"AN INVESTIGATION OF THE LOAD CARRYING CAPACITY OF DRILLED CAST-IN-PLACE CONCRETE PILES BEARING ON COARSE GRANULAR SOILS AND CEMENTED ALLUVIAL FAN DEPOSITS"

BY

GEORGE H. BECKWITH, P. E.
AND
DALE V. BEDENKOP, P. E.

SUBMITTED TO

THE ARIZONA HIGHWAY DEPARTMENT
PHOENIX, ARIZONA 85007

FOR

RESEARCH PROJECT - ARIZONA HPR-1-10(122)
DRILLED CAST-IN-PLACE CONCRETE PILES

SPONSORED BY

THE ARIZONA HIGHWAY DEPARTMENT
IN COOPERATION WITH
DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION

SERGENT, HAUSKINS & BECKWITH
CONSULTING SOIL & FOUNDATION ENGINEERS
3940 WEST CLARENDON AVENUE
PHOENIX, ARIZONA

MAY 1973

TECHNICAL REPORT STANDARD TITLE PAGE

1. Report No.

2. Government Accession No.

3. Recipient's Catalog No.

4. Title and Subtitle "AN INVESTIGATION OF THE LOAD-CARRYING CAPACITY OF DRILLED CAST-IN-PLACE CONCRETE PILES BEARING ON COARSE GRANULAR SOILS AND CEMENTED ALLUVIAL FAN DEPOSITS"

5. Report Date

6. Performing Organization Code

7. Author(s)
GEORGE H. BECKWITH
DALE V. BEDENKOP

8. Performing Organization Report No.
SHB JOB NO. E71-272

9. Performing Organization Name and Address
SERGENT, HAUSKINS & BECKWITH
CONSULTING SOIL & FOUNDATION ENGINEERS
3940 WEST CLARENDON
PHOENIX, ARIZONA 85019

10. Work Unit No.

11. Contract or Grant No.
HPR-1-9(122) AFE 60013

12. Sponsoring Agency Name and Address
ARIZONA HIGHWAY DEPARTMENT
1739 WEST JACKSON
PHOENIX, ARIZONA 85007

13. Type of Report and Period Covered
FINAL REPORT
SEPTEMBER 1970 TO FEBRUARY 1973

14. Sponsoring Agency Code

15. Supplementary Notes
STUDY WAS CONDUCTED IN COOPERATION WITH U. S. DEPARTMENT OF TRANSPORTATION, FEDERAL HIGHWAY ADMINISTRATION

16. Abstract

SEVEN LOAD TESTS OF DRILLED PILES WERE PERFORMED ON COARSE GRANULAR SOILS AND 20 TESTS ON CEMENTED ALLUVIAL FAN DEPOSITS. MAXIMUM TEST LOAD WAS 1000 TONS. DETAILED SITE SELECTION AND SITE SOIL INVESTIGATION STUDIES WERE PERFORMED. A SPECIAL TRAILER-MOUNTED PORTABLE LOAD FRAME WAS DESIGNED AND FABRICATED FOR THE PROJECT. BELLED, STRAIGHT, SMALL MULTIPLE BELL, END-BEARING ONLY AND SIDE SHEAR ONLY TESTS WERE PERFORMED. A TELL-TALE INSTRUMENTATION WAS USED.

A COMPARISON OF VARIOUS SETTLEMENT ANALYSIS AND BEARING CAPACITY CALCULATION METHODS WAS MADE. CALCULATIONS FOR THE FINER-GRAINED CEMENTED ALLUVIAL FAN SOILS WERE BASED UPON CONSOLIDATION, DIRECT SHEAR AND FIELD PRESSUREMETER AND PENETRATION TESTS. DESIGN RECOMMENDATIONS AND CONSTRUCTION PROCEDURES ARE PRESENTED.

17. Key Words CAISSON, PILE, BEARING CAPACITY, LOAD TEST, BORED PILE, INSTRUMENTATION, PLATE BEARING TEST, PRESSUREMETER TEST, SETTLEMENT ANALYSIS, PENETRATION TEST, DIRECT SHEAR TEST, BECKER HAMMER DRILL

18. Distribution Statement

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
UNCLASSIFIED	UNCLASSIFIED		

Form DOT F 1700.7 (8-69)

SERGENT, HAUSKINS & BECKWITH
CONSULTING SOIL AND FOUNDATION ENGINEERS
APPLIED SOIL MECHANICS • ENGINEERING GEOLOGY • MATERIALS ENGINEERING
B. DWAINE SERGENT, P.E. JOHN B. HAUSKINS, P.E. GEORGE H. BECKWITH, P.E.
DALE V. BEDENKOP, P.E. ROBERT D. BOOTH, P.E. BRUCE J. LEISER, P.E.

February 16, 1973

Mr. William N. Price
State Highway Engineer
Arizona Highway Department
1739 West Jackson
Phoenix, Arizona 85007

SHB Job No. E71-272

Attention: Mr. Gene Morris
Research Engineer

Re: Contract No. 71-30 (Phase II)
Drilled Cast-In-Place Concrete Piles
Project No. HPR-1-9(172) AFE 60013

Gentlemen,

Submitted herewith is a final draft of our report "An Investigation of the Load-Carrying Capacity of Drilled Cast-In-Place Concrete Piles Bearing on Coarse Granular Soils and Cemented Alluvial Fan Deposits" prepared under the referenced contract. The report includes a description of site selection process, soil and geologic conditions at test sites, soil investigation of test sites, testing equipment and testing procedures along with our evaluation of test results, conclusions and recommendations.

The opinions, findings and conclusions contained in this report are those of the authors and are not necessarily those of the Arizona Highway Department and the Federal Highway Administration.

REPLY TO: 3940 W. CLARENDON, PHOENIX, ARIZONA 85019
PHOENIX (602) 272-6848
FLAGSTAFF (602) 774-4433
EL PASO (915) 772-3088
ALBUQUERQUE (505) 344-9940

Contract No. 71-30 (Phase II)
Drilled Cast-In-Place Concrete Piles
Project No. HPR-1-9(172) AFE 60013
SHB Job No. E71-272

Upon your review, we are looking forward to discussing the report with you.

Respectfully submitted,

Sergent, Hauskins & Beckwith Engineers

By______________________________
George H. Beckwith, P. E.
Principal in Charge

And______________________________
Dale V. Bedenkop, P. E.
Project Engineer

Copies: Addressee (5)

ACKNOWLEDGMENTS

The authors wish to thank a number of individuals and organizations whose contributions helped complete this research project. Special thanks are extended to Messrs. Martin Toney and Gene R. Morris, Chiefs of the Bridge and Research Divisions, respectively, of the Arizona Highway Department, whose support made the project possible. Messrs. Lyman W. Reese, Michael W. O'Neill and H. H. Dalrymple of the Center for Highway Research, The University of Texas at Austin, reviewed their studies, in detail, with the authors and were extremely helpful in answering numerous questions. Mr. B. Dwaine Sergent of this firm reviewed various aspects of the project as it proceeded and contributed many ideas which were incorporated into the study. Messrs. John B. Hauskins and Robert D. Booth also were very helpful in reviewing the project at various times and providing suggestions. Staff engineer, Lyle M. Tweet, performed many of the calculations involved. Mr. Robin E. Parke of Caruso, Parke & Associates, Inc., Consulting Structural Engineers, Phoenix, Arizona, performed the structural design of the load frame and contributed a number of ideas on the connection system. Mr. Bill Becker of Becker Drills, Inc., drilled test holes at test Site A for no charge when funds were not available for exploration with the Becker Hammer Drill which was not anticipated at the beginning of the program. Without this information, much less valuable results would have been obtained at that site. Evans Steel & Manufacturing Company of Mesa, Arizona, fabricated the load frame. Mr. James P. Watts of Bell Bottom Foundation Company of Arizona, whose firm performed most of the construction drilling for the project, contributed a number of ideas on the practical application of drilled piling to Arizona conditions. Bayou Industries of Channelview, Texas, furnished the jacking system.

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JOB NO. E71-272

NOTATION

THE FOLLOWING SYMBOLS ARE USED IN THIS REPORT:

A = COHESION REDUCTION COEFFICIENT
A_B = END AREA OF PILE
A_S = SIDE AREA OF PILE SHAFT
c = SOIL COHESION
D = BASE DIAMETER OF PILE
D = DEPTH FROM GROUND SURFACE TO BASE OF PILE
E = MODULUS OF DEFORMATION OF SOIL
I_W = STRESS INFLUENCE COEFFICIENT
K = LATERAL EARTH PRESSURE COEFFICIENT
N = STANDARD PENETRATION RESISTANCE, BLOWS/FOOT
N_B = BECKER HAMMER DRILL PENETRATION RESISTANCE, BLOWS/FOOT
N_C = BEARING CAPACITY FACTOR
N_Q = BEARING CAPACITY FACTOR
P_F = CREEP PRESSURE DETERMINED BY THE PRESSUREMETER TEST
P_L = LIMIT PRESSURE DETERMINED BY THE PRESSUREMETER TEST
P_O = INITIAL PRESSURE DETERMINED BY THE PRESSUREMETER TEST
P_Z = INITIAL VERTICAL CONFINING STRESS ON SOIL
S = SETTLEMENT OF PILE
s = SOIL SHEAR STRENGTH DEFINED BY DIRECT SHEAR TEST
s_O = SOIL SHEAR STRENGTH DEFINED BY PRESSUREMETER TEST
Q = TOTAL LOAD ON PILE
Q_B = TOTAL ULTIMATE END BEARING RESISTANCE OF PILE
Q_S = TOTAL ULTIMATE SIDE RESISTANCE OF PILE
Q_T = TOTAL ULTIMATE RESISTANCE OF PILE
q = UNIT BEARING PRESSURE ON BASE OF PILE, BEARING PLATE OR EQUIVALENT PIER

NOTATION

Q_B = ULTIMATE END BEARING PRESSURE ON BASE OF PILE
Q_S = ULTIMATE UNIT SIDE SHEAR ON PILE SHAFT
Q_U = UNCONFINED COMPRESSIVE STRENGTH OF SOIL
U = POISSON'S RATIO
Z = DEPTH FROM GROUND SURFACE TO POINT BEING CONSIDERED
γ = DENSITY OF SOIL
δ = ANGLE OF FRICTION BETWEEN SOIL AND SHAFT OF PILE
ϕ = ANGLE OF INTERNAL FRICTION OF SOIL

CHAPTER I - INTRODUCTION

Scope & Objectives

This report presents the results of an investigation of the load-carrying capacity of drilled cast-in-place concrete piles deriving support from very coarse granular deposits and cemented finer-grained alluvial fan deposits. These types of soils predominate in the heavily populated areas of central and southern Arizona. The major objective of the study was to develop rational and/or empirical methods of predetermining bearing capacities of drilled piling which could be used in routine design. The study is confined to downward axial loading. A secondary objective of the study was to evaluate design details and inspection procedures for local conditions and provide recommendations for routine use.

Summary of Program

Prior to the start of the actual load testing program, a search was made of available, pertinent engineering literature. The reports of previous studies of drilled piling were reviewed in detail. Following this review, a study of existing soil data was made to select criteria for load test sites, which lead to the selection of several available sites for preliminary test borings. As a result of the above studies, three sites were selected which are believed to be as representative as possible of the range of typical soils for the predominance of actual construction sites. One site was selected for "end-bearing" only studies on a coarse granular deposit and two for "end-bearing" and "side shear" studies in finer alluvial fan deposits.

A detailed investigation was then made of each of the three sites to determine the physical properties of the subsurface materials.

Drilled piling were constructed at each site along with the necessary anchor piling for uplift resistance for the load frame. The piling constructed for load testing were as follows:

1. In the coarse granular materials, 7 end-bearing piles. All but one of these were belled to varying diameters.

2. In the finer alluvial fan deposits where both end-bearing and side shear were evaluated, piling were constructed for end-bearing only, side shear only, with multiple small bells (shear collars) and without cleaning loose materials from base. The last series were used for evaluation of the effectiveness of piling which are "machine cleaned" only.

For the performance of the load tests, a self-contained, hydraulically operated load frame was designed and assembled. The frame capacity is 1000 tons. A "telltale" instrumentation system was developed for use in side shear load transfer measurements.

The tests were evaluated relative to various methods for the predetermination of ultimate bearing capacity and estimation of settlements. Recommended design procedures, construction and design details and inspection procedures were developed.

Soil Conditions Involved in Study

Much of Phoenix and the Salt River Valley are underlain by very coarse granular deposits consisting of mixtures of sand, gravel and cobbles. Similar deposits are present at many

OTHER AREAS IN THE LOWER VALLEYS OF CENTRAL AND SOUTHERN ARIZONA. THESE SOILS WERE DEPOSITED BY HIGH GRADIENT DISCHARGES OF THE SALT RIVER AND OTHER DRAINAGES. THIS SOIL TYPE IS HEREAFTER TERMED SGC IN THIS REPORT. THE SGC SOILS ARE FAR TOO COARSE TO ENABLE EVALUATION OF RELATIVE DENSITY OR COMPRESSIBILITY BY CONVENTIONAL PENETRATION AND LABORATORY TESTS COMMONLY USED FOR SANDS. THEIR COARSE NATURE ALSO MAKES IT EXTREMELY DIFFICULT AND COSTLY TO PERFORM IN-PLACE DENSITIES TO COMPARE WITH LABORATORY MAXIMUM AND MINIMUM DENSITIES. ONE OF THE LOAD TEST SITES IN THIS INVESTIGATION INVOLVED THE SGC DEPOSIT.

ABOUT 30 PERCENT OF THE LAND AREA IN THE ARID PORTION OF THE SOUTHWEST IS UNDERLAIN BY ALLUVIAL FAN DEPOSITS. THESE SOILS TYPICALLY CONSIST OF HIGHLY STRATIFIED SAND-SILT-CLAY MIXTURES. THEY ARE DEPOSITED BY SHEET FLOODS AND DISCHARGES OF SMALL INTERMITTENTLY FLOWING DRAINAGES. IN THIS SEDIMENTATION PROCESS, A LAYER OF SOIL DRIES OUT PRIOR TO DEPOSITION OF AN OVERLYING LAYER. IN WELL DRAINED AREAS, SOIL MOISTURE CONTENTS STABILIZE AT VERY LOW VALUES (WELL BELOW THE PLASTIC LIMIT) IN THE ARID ENVIRONMENT WHERE ANNUAL EVAPORATION EXCEEDS RAINFALL BY 60 INCHES OR MORE. IN SOME INSTANCES, DEPENDING ON THE NATURE OF THE PARENT MATERIAL AND EXACT MANNER OF DEPOSITION, ALLUVIAL FANS PRODUCE LOOSE MOISTURE SENSITIVE "COLLAPSING" SOIL DEPOSITS. IN MANY OTHER CASES, ALLUVIAL FANS RESULT IN LIME CEMENTED DEPOSITS WHICH ARE FIRM TO HARD IN CONSISTENCY AND NOT GREATLY WEAKENED BY MOISTURE INCREASES. THIS WIDESPREAD GENERAL CATEGORY OF DEPOSIT IS THE SECOND TYPE OF SOIL INVESTIGATED IN THIS STUDY AND IS HEREAFTER TERMED CAF (CEMENTED ALLUVIAL FAN) SOILS IN THIS REPORT.

THESE RELATIVELY DRY ALLUVIAL FAN SOILS ARE USUALLY FISSURED AND FRACTURED TO SOME DEGREE, POSSESS A FRIABLE TEXTURE AND

OFTEN CONTAIN GRAVEL PARTICLES AND CALCAREOUS CONCRETIONS AND NODULES. BECAUSE OF THESE CHARACTERISTICS, SAMPLING NORMALLY CANNOT BE ACCOMPLISHED WITH THIN WALLED OR DOUBLE BARRELED TUBE SAMPLERS AND OFTEN, BLOCK SAMPLES CANNOT BE EFFICIENTLY CUT AND TRIMMED. THE USE OF THICK WALLED OPEN-END DRIVE SAMPLERS LINED WITH METAL OR PLASTIC RINGS IS NECESSARY FOR EFFICIENT SAMPLING OF CAF SOILS AND USUALLY THE SAMPLES CANNOT BE EXTRUDED FOR TRIAXIAL OR UNCONFINED COMPRESSION TESTING. THUS, THE DIRECT SHEAR TEST HAS BEEN WIDELY ADOPTED FOR ARIZONA SOIL CONDITIONS BECAUSE TESTING APPARATUS WILL RECEIVE THE LINER RINGS WITH SOIL SPECIMENS AS SECURED IN THE FIELD. BECAUSE OF THE SAMPLE DISTURBANCE INDUCED BY THE HIGH DRIVING ENERGY INVOLVED IN OPEN-END DRIVE SAMPLING, THE FREQUENT PRESENCE OF SCATTERED COARSER SOIL PARTICLES OR CONCRETIONS AND THE THEORETICAL LIMITATIONS OF THE TEST, THE VALIDITY OF DIRECT SHEAR TEST DATA OBTAINED ON CAF SOILS AS IT APPLIES TO ANALYSIS OF PILE CAPACITIES HAS BEEN CONSIDERED HIGHLY QUESTIONABLE IN MANY CASES.

THE USE OF DRILLED CAST-IN-PLACE CONCRETE PILES IN ARIZONA

WITH THE DEVELOPMENT OF MODERN FOUNDATION DRILLING EQUIPMENT SINCE WORLD WAR II, THE USE OF HIGH CAPACITY DRILLED CAST-IN-PLACE CONCRETE PILES HAS GAINED WIDE AND STEADILY INCREASING USE IN MANY AREAS OF THE WORLD. DEVELOPMENT OF THE USE OF LARGE DIAMETER PILES IN THE UNITED STATES HAS BEEN REVIEWED BY WHITE (1)*.

*NUMBERS IN PARENTHESIS CORRESPOND TO REFERENCES LISTED IN APPENDIX A.

The soil conditions prevalent in Arizona lend themselves to the efficient use of drilled piling. In many areas, a surface layer of looser moisture sensitive alluvial fan materials, typically 8 to 25 feet in depth, overlie SGC or CAF soils. Belled piles bearing near the surface of the SGC or CAF soils have usually been employed for these conditions. In cases where CAF soils are near the surface, straight piles penetrating these deposits have often been used.

For the typical conditions previously discussed, excavations for drilled piles can be made without the use of casing and without significant caving and sloughing occurring. Because of these very favorable conditions for installation and the efficiency of available drilling equipment, drilled piles have proven considerably more economical than driven piling for typical Arizona conditions. Thus, the majority of heavily loaded building structures in Phoenix and Tucson constructed in the past 12 years are supported on drilled piles.

Drilled piles have proven to be particularly advantageous because the high concentrated loads involved for many projects can be supported on single piles. Building projects in Arizona have involved loads up to about 2000 tons imposed on single piles. Future elevated freeway structures in Arizona may involve long-term concentrated loads of 5000 tons or more. Drilled piles have also proven efficient where high upward or lateral loads are imposed and have been widely used in Arizona for electrical transmission towers and similar structures.

The Problem of Predetermining Pile Capacities

No dynamic driving record is developed during construction of drilled piles and load tests are very costly for most projects due to the high capacities involved. Thus, it is necessary,

IN MOST CASES, TO PREDETERMINE CAPACITIES BY RATIONAL OR EMPIRICAL METHODS FOR EVALUATION OF SETTLEMENT UNDER WORKING LOADS AND ULTIMATE CAPACITY.

THE PROBLEM OF BEARING CAPACITY PREDETERMINATION IS PARTICULARLY DIFFICULT FOR BOTH SGC AND CAF SOILS BECAUSE OF PREVIOUSLY DISCUSSED PROBLEMS IN EVALUATING ENGINEERING PROPERTIES BY FIELD AND LABORATORY TESTS. WIDE HORIZONTAL AND VERTICAL VARIATIONS OF THE PROPERTIES OF THESE SOILS, EVEN AT THE MOST UNIFORM SITES, ALSO COMPOUNDS THE PROBLEM OF PILE CAPACITY PREDETERMINATION. BECAUSE OF THIS FACTOR, IT WAS JUDGED NECESSARY TO TEST AS MANY PILES IN THESE MATERIALS AS POSSIBLE. FOR THIS REASON, A RELATIVELY SIMPLE AND INEXPENSIVE INSTRUMENTATION SYSTEM WAS SELECTED.

AS PREVIOUSLY STATED, THE COARSE NATURE OF THE SGC DOES NOT ALLOW THE PERFORMANCE OF MEANINGFUL PENETRATION OR SOIL MECHANICS TESTS. DESIGN PRACTICE FOR BELLED PILES BEARING ON THE SGC HAS BEEN TO ASSIGN SAFE SOIL BEARING PRESSURES BASED ON CLASSIFICATION AND PRECEDENT WITH PROJECTS BOTH IN ARIZONA AND ELSEWHERE. SAFE SOIL BEARING PRESSURES RANGING FROM ABOUT 8000 TO 20,000 PSF HAVE BEEN USED FOR VARIOUS PROJECTS DEPENDING UPON TOTAL CONCENTRATED LOADS AND EXTENT OF SUBSURFACE EXPLORATION PERFORMED. FOR THE BEARING PRESSURES EMPLOYED ON THE BASIS OF PRECEDENT AND THE DEPTH AND WIDTH OF TYPICAL BELLED PILES, EXTREMELY HIGH FACTORS OF SAFETY ARE COMPUTED BY THE SEMIEMPIRICAL TERZAGHI EQUATION FOR END-BEARING (2) USING SHEAR STRENGTHS FOR SGC ASSIGNED FROM VISUAL CLASSIFICATION. THUS, THE PRACTICAL PROBLEM OF DESIGN OF FOUNDATIONS BEARING ON THE SGC IS ONE OF ASSESSING SETTLEMENTS UNDER WORKING LOADS. SEVEN 1000 TON LOAD TESTS WERE PERFORMED IN THIS STUDY ON END-BEARING PILES OF VARIOUS WIDTHS BEARING ON SGC. THE MAJOR OBJECT OF THESE TESTS WAS TO INVESTIGATE THE RELATIONSHIP BETWEEN WIDTH OF LOADED AREA AND SETTLEMENT FOR VARIOUS BEARING PRESSURES.

The recently developed Becker Hammer Drill has provided an efficient method of exploration of the SGC. This method, involving a dynamically advanced double walled drive pipe and removal of cuttings with compressed air by a reverse circulation process, enables relatively accurate classification of coarse granular deposits. Classification in conjunction with the dynamic driving record developed provided a tool for comparison of the SGC load test site and other sites, and assessing the overall uniformity of the deposit. Prior to the advent of the Becker Drill, the cable-tool percussion drill rig was the only means of obtaining this data. From a practical standpoint, the rate of drilling in the SGC with a churn drill is too slow for major projects, even though the cost in some cases does not greatly exceed that for Becker drilling.

In local practice, various rational methods for computing the ultimate capacity of drilled piling based upon the summation of side shear for various layers and end resistance have been used for the analysis of both belled and straight piles bearing on CAF soils. A major difficulty in this procedure is in the sampling and performance of shear tests as previously discussed. Various rational settlement analysis procedures also have been employed in design analysis. The effect of sample disturbance on consolidation tests of CAF soils which are heavily overconsolidated by desiccation, introduces large uncertainties as to the validity of settlement computations.

Previous Studies

Although, to our knowledge, no comprehensive studies of drilled pile capacities in the types of soils studied in this investigation have been performed to date, a large

BODY OF DATA ON DRILLED PILES HAS BEEN REPORTED IN ENGINEERING LITERATURE. MUCH OF THIS DATA WAS NOT ONLY PERTINENT TO THE DESIGN AND EVALUATION OF TESTS PERFORMED IN THIS STUDY, BUT IS APPLICABLE TO THE DESIGN OF DRILLED PILING IN OTHER SOIL TYPES WHICH OCCUR IN ARIZONA.

THE CENTER FOR HIGHWAY RESEARCH AT THE UNIVERSITY OF TEXAS AT AUSTIN HAS PERFORMED DETAILED STUDIES OF VARIOUS ASPECTS OF DRILLED PILE DESIGN WHICH HAVE BEEN PRESENTED IN A SERIES OF NINE RESEARCH REPORTS PUBLISHED BETWEEN APRIL, 1968 AND DECEMBER, 1970. IN ONE OF THESE REPORTS, O'NEILL AND REESE (3) PRESENT A COMPREHENSIVE REVIEW OF BOTH THE STATE OF THE ART OF EMPIRICAL AND RATIONAL METHODS FOR THE ANALYSIS OF PILE CAPACITIES AND SETTLEMENTS AND OF PREVIOUS STUDIES ON DRILLED PILES.

A LARGE NUMBER OF STUDIES HAVE BEEN MADE ON STIFF, FISSURED CLAYS WHICH WERE GENERALLY SATURATED OR AT HIGH MOISTURE CONTENTS. A NUMBER OF THESE SOILS ARE SIMILAR TO THOSE WHICH OCCUR IN SOME AREAS OF NORTHERN ARIZONA. AMONG THE LOCALLY SIGNIFICANT STUDIES IN THIS CATEGORY ARE THOSE OF SKEMPTON (4); WHITAKER AND COOKE (5); AND BURLAND, BUTLER AND DUNICAN (6) ON LONDON CLAY; REESE AND HUDSON (7) AND O'NEILL AND REESE (3) ON TEXAS SOILS; MOHAN AND CHANDRA (8) ON INDIAN BLACK COTTON SOILS; WOODWARD, LUNDGREN AND BOITANO (9) IN CALIFORNIA; AND WATT, KURFURST AND ZEMAN (10) IN CANADA.

SEVERAL LOAD TEST PROGRAMS HAVE BEEN PERFORMED ON SHALES SIMILAR TO ONE OF THE TYPES THAT OCCUR IN THE COLORADO PLATEAU AREA OF NORTHEAST ARIZONA. AMONG THESE ARE DATA PRESENTED BY REESE, HUDSON AND VIJAYVERGIYA (11) AND U. S. ARMY ENGINEERS (12) ON TEXAS SITES; VAN DOREN, ET AL (13) ON A KANSAS SITE; AND MATICH AND KOZICKI (14) ON A CANADIAN SITE. A SOUTH DAKOTA DEPARTMENT OF HIGHWAYS STUDY BY BUMP, ET AL (15) INVOLVED THE

Pierre shale which is very similar to the widespread Chinle shale in northeastern Arizona. In most of these studies, however, moisture contents were considerably higher than at the typical Arizona site.

Factors involved in the design of drilled piles in sand are presented by Vesic (16). Martins (17) presents tests on drilled piles below the water table in predominantly sandy soils. Barker and Reese (18) have reviewed various research on load test data on drilled piling installed below the water table with bentonite slurry. This method may be economically competitive in Arizona at certain sites on the Colorado River and major drainages in northern Arizona.

CHAPTER II - SELECTION OF TEST SITES

General Site Selection Study

At the beginning of this study, the two basic types of soils to be investigated were recognized. It was concluded that a site for end-bearing load tests on the SGC and two load test sites on typical CAF soils would reasonably cover the majority of soils in Arizona not already investigated in previous research.

A detailed study of soil conditions prevailing in the Salt River Valley, the Tucson area and other valleys in central, southern and western Arizona was made to enable selection of as representative a group of sites as possible. About 2000 previous soils investigations made by this firm, including the preliminary investigation for the Papago East Freeway in Phoenix (19) and the I-710 extension in Tucson (20), were reviewed. Previous surface soils mapping of the Salt River Valley by the U.S. Bureau of Soils and Chemistry (21) and studies of the valley by the U. S. Geological Survey (22, 23, 24) also were reviewed.

General Criteria for Site Selection

Beyond the basic criteria of selection of sites that as well as possible represented the range of soils involved, several other general factors were considered in site selection. It was desired to locate the sites in the Phoenix area to minimize travel time. It was also considered preferable that the sites be on acquired right-of-way owned by the Arizona Highway Department to minimize administrative problems and to enable fencing for security in that long-term storage of equipment would be involved in the program.

It was further considered desirable that the sites be located within the area of freeway alignments where construction of elevated viaducts involving a large number of high capacity foundations is programmed for the relatively near future. It was thought that this might enable the development of supplementary information by measuring settlements of actual foundations during construction and operation of the freeways.

Special Criteria - SGC Site

Several special criteria for the selection of the SGC site were established as follows:

1. The site should be located in a uniform area of the SGC where the deposit is uncemented and thus represents the weakest general condition. A minority of the upper soils in the SGC deposit possess varying degrees of cementation. The deposit should be free of clay and sand layers beneath the test site.

2. Settlement of foundations on granular soils tend to decrease somewhat with increasing depth, all other factors being equal. Thus, the site should be in a relatively shallow area in order to test the most critical condition of depth commonly involved in drilled foundation design for local conditions.

3. The surface soils over the SGC should be strong enough to efficiently develop the necessary uplift capacity of belled reaction piles founded on the surface of the SGC.

Special Criteria - CAF Sites

The CAF soils under study embrace a wide variety of soil classifications with silty clays, sandy clays and clayey sands predominating (Unified Soil Classification CL and SC). Lesser amounts of highly plastic clay and sandy clays silty

SANDS, CLAYEY SILTS AND SANDY SILTS (CH, SM AND ML) OCCUR. WIDE VARIATIONS IN THE DEGREE OF CEMENTATION ALSO OCCUR WITH CALCAREOUS CONCRETIONS AND MINOR FRACTIONS OF GRAVEL BEING PRESENT IN MANY CASES. VARIATIONS IN THE GRANULAR FRACTION OF SOILS AND CHARACTER OF THE CEMENTATION PRODUCE WIDE DIFFERENCES IN THE SURFACE TEXTURE OF THE FACE OF DRILLED PILE EXCAVATIONS. IN VIEW OF THE FACT THAT SURFACE TEXTURE HAS BEEN SHOWN TO BE AN IMPORTANT FACTOR INFLUENCING THE DEGREE TO WHICH SOIL SHEAR STRENGTH IS MOBILIZED IN SIDE SHEAR, THIS WAS CONSIDERED IN SITE SELECTION. THE DEGREE AND CHARACTER OF CEMENTATION SUBSTANTIALLY INFLUENCES STRENGTH AND WAS THUS CONSIDERED.

THE INTENSITY, ORIENTATION AND SURFACE TEXTURE OF JOINTING AND FISSURING ARE ALSO IMPORTANT FACTORS IN SOIL SHEAR STRENGTH RELATIVE TO PILE CAPACITIES AND WERE CONSIDERED. CAF SOILS ARE TYPICALLY OVERCONSOLIDATED BY DESICCATION TO A CONSIDERABLE DEGREE AND PROBABLY POSSESS A RELATIVELY HIGH STATE OF HORIZONTAL STRESS IN SITU. IT WAS RECOGNIZED IN SITE SELECTION, THAT OVERCONSOLIDATION IS PROBABLY INTERRELATED WITH FISSURING AND JOINTING AND THAT STRESS RELIEF UPON EXCAVATION OF FOUNDATIONS MIGHT SIGNIFICANTLY AFFECT CAPACITY.

FINAL SITE SELECTION

GENERAL STUDIES REVEALED AN SGC AREA ALONG THE PROPOSED PAPAGO EAST FREEWAY IN PHOENIX WHICH APPEARED TO MEET CRITERIA AND AREAS ALONG PAPAGO WEST AND SUPERSTITION FREEWAY ALIGNMENTS WHICH APPEARED SUITABLE FOR THE TWO CAF SITES. THE AREAS WERE ON ACQUIRED RIGHT-OF-WAY AND AT CONVENIENT LOCATIONS. EXTENSIVE EXPLORATORY DRILLING WAS PERFORMED IN THESE AREAS TO VERIFY CONDITIONS INDICATED BY THE GENERAL STUDY. AS A RESULT, 3 LOAD TEST SITES WERE SELECTED

IN THESE AREAS DESIGNATED A THROUGH C. A VICINITY MAP, FIGURE 9*, SHOWS TEST SITE LOCATIONS. FIGURES 10 AND 11 SHOW EXACT TEST SITE LOCATIONS ON AERIAL PHOTOGRAPHS.

*TABLES AND FIGURES ARE GIVEN IN APPENDIX B.

CHAPTER III - SOIL INVESTIGATIONS OF TEST SITES

General

Detailed auger drilling, penetration testing and open-end drive sampling were performed at each site in the CAF soils by the procedures normally used in subsurface exploration in local practice. The purpose of this type of drilling at Site A was to provide data on the surface CAF soils for the design of reaction piles and comparative information on the various field and laboratory tests. In addition, both large diameter test holes and Becker Hammer Drill holes were made into the SGC soils at Site A. Detailed laboratory testing was performed on the recovered samples.

Several in situ testing methods were considered for use in the hope that higher quality data on strength and compressibility of the CAF soils could be obtained than normally achieved in the laboratory. The Pressuremeter, a balloon-like load cell that is employed to perform load tests in small diameter drill holes, was selected for use. Several studies using this device have shown promising results in stiff soils and soft rock (25, 26, 27, 28, 29).

Several other in situ testing methods were considered for use for investigating the shear strength of CAF soils. Mechanical methods such as the Handy Iowa State device (30) which involve expansion of plates against the borehole walls do not appear applicable because of point loading on the rough sides of the holes created in most CAF soils. A large diameter device of this type studied by Campbell and Hudson (31) did not perform well. A relatively high capacity vane shear apparatus was developed for investigations of the 1964 Alaska earthquake (32). However, its capacity is only 5000 psf (not enough to fail many CAF soils) and point loading of vanes on gravel particles and calcareous concretions would prevent its use in many cases. It was also thought that point loading on coarse particles would create serious problems with static cone penetration tests.

Subsurface Exploration

Five large diameter borings were drilled around the perimeter of Site A to about 15 feet into the SGC. The purpose of these borings was to investigate the deposit for layers of sand or clay. These borings were drilled with a Texoma 500-35 drill rig and 30 inch diameter noncontinuous flight auger.

Test borings were drilled at each corner of each test site with 6 5/8 inch O.D., $3\frac{1}{4}$ inch I.D. hollow stem auger and a CME-55 drill rig. The borings refused on the SGC at between 13 and 16 feet at Site A. The auger borings were drilled to about 40 feet at Site B and 25 to 30 feet at Site C.

At each boring location, a standard penetration test was performed at $2\frac{1}{2}$ foot intervals in an auger boring by the ASTM D1586-67 procedure (33). About 4 feet away, a second auger boring was drilled and open-end drive sampling performed at $2\frac{1}{2}$ foot intervals. Three inch O.D. samplers were used lined with 2.42 inch I.D. brass rings. The sampler drive shoe had the same I.D. as the brass liner rings (zero relief). The open-end drive samplers were driven with a 140 pound, 30 inch free fall drop hammer in the manner outlined in ASTM D1586-67. Blows required to advance the 2 inch O.D. standard penetration samplers and the 3 inch O.D. open-end drive samplers are recorded on the boring logs in 2 inch increments in order to provide information on the degree of stratification and the presence of thin, strongly cemented layers and scattered gravel and calcareous concretions.

At a location about 4 feet from the auger borings, continuous penetration tests were performed by driving a 2 inch O.D. blunt nosed penetrometer. The penetrometer consists of a 2 inch O.D. heavy cut washer slipped over a nipple which is attached to 1 5/8 inch O.D. drill rods (A-rod). Penetration values are recorded as the number of blows of a 140 pound, 30 inch free fall hammer required to advance the penetrometer in one foot increments or less. This "bullnose" penetration test has been widely used in local practice.

In order to provide detailed information on the nature of the SGC beneath Site A, borings were advanced with a Becker Hammer Drill immediately adjacent to each test pile after the load tests were complete. The principal features of the rig are shown on Figure 12. The Becker Hammer Drill advances a double walled drive pipe with a Link Belt pile driving hammer rated at 8000 foot-pounds energy per blow. Cuttings are removed by compressed air by a reverse circulation process. The drive pipe used in the test drilling was $5\frac{1}{2}$ inch O.D. x $3\frac{1}{4}$ inch I.D. and employed an expendable drill bit of slightly larger diameter than the O.D. of the drive pipe. Hammer blows required to advance the drive pipe were recorded in 6 inch increments and representative samples of cuttings were obtained.

Logs of test borings are presented in Appendix C. Figures 13, 14 and 15 show the location of borings at the test sites.

Laboratory Tests

Moisture content determinations were made on all samples recovered in standard penetration testing and open-end drive sampling while dry densities were determined for the open-end drive samples. Results of these tests are given on the boring logs.

Grain-size analysis and Atterberg Limits tests were performed on selected samples of the various soils involved.

One dimensional consolidation tests were performed on selected samples of the CAF soils from each site. "Floating ring" apparatus designed to receive one inch high 2.5 inch O.D. brass liner rings with soil specimens as secured in the field were used in the tests. Procedures were generally those outlined in ASTM D2435-70 (33). The specimens were consolidated to loads approximately equal to the preconsolidation pressure and rebounded with the tests then being completed in the normal sequence. In the partially saturated soils involved, each increment of load was maintained until the rate of deformation was equal or less than 0.0001 inch per hour.

Direct shear tests were run on all open-end drive samples using an apparatus of the strain-control type. Shearing forces were applied at a rate deformation of approximately 0.05 inches per minute. The machine is designed to receive one of the one inch high 2.42 inch diameter specimens obtained by sampling. Generally, each sample was sheared under a normal load equivalent to the effective overburden pressure at the point of sampling. In some instances, samples are sheared at several normal loads. The plates of the direct shear device were separated 0.04 inch during the tests.

Unconfined compression tests were performed on a few block samples obtained in an anchor pile excavation at Site B. Because of the fissured, friable nature of the CAF soils block samples could not be obtained at Sites A and C.

Results of the laboratory tests are presented in Appendix C.

Pressuremeter Tests

In situ tests on the walls of small diameter boreholes were made in the CAF soils at all 3 sites with a pressuremeter. The boring locations were about 4 feet from the auger boring and continuous penetration test locations. The pressuremeter equipment now in use, shown schematically in Figure 16, was developed in the 1950s by Menard (34) in France and gained wide use in Europe since. The device became available in the United States in the mid 1960s.

The pressuremeter apparatus consists of a steel probe with rubber membranes forming 3 independent pressure cells which is lowered into testing position in the borehole. The cells are connected by plastic lines to a combined volumeter-manometer control apparatus at the surface. Pressure-deformation relationships are determined by observing the volume of water injected into the central cell during incremental pressure increases. The outer guard cells expand under an equal pressure and reduce the end effects on the central measuring cell. Detailed descriptions of the pressuremeter and its application have been presented by a number of authors (25, 26, 27, 28, 29, 34, 35).

A typical volumetric strain versus pressure curve for the stiffer CAF soils encountered in the study are shown in Figure 16. Several basic quantities are obtained in the test. The "initial" pressure, Po, is the beginning of the relatively elastic portion of the curve. In the portion of the curve up to Po, the natural horizontal stresses in the soil are restored. At pressures beyond the relatively linear phase, the rate of volume change increases rapidly until the limit pressure, Pl, which is considered to define failure, is reached. Also plotted on Figure 16 is the creep curve which shows the tendency of the material to deform with time. The creep pressure, Pf, is the pressure at which the creep curve takes a sharp upward break. This point generally agrees closely with the upper limit of the linear portion of the pressure-volume curve. The modulus of deformation, E, is derived from the slope of the curve between Po and Pf.

Pressuremeter probes designed to test NX (2.98 inch diameter) holes were used in this investigation. The initial uninflated diameter of the probe is slightly smaller than NX hole size and inflates to about twice the initial diameter. In order to perform tests in the stiff CAF soils, it was necessary to drill holes with the diameter varying between about 2.75 to 3.00 inches. Where the hole diameter was larger, the ends of the membranes tended to rupture before PL could be defined.

Considerable difficulty was experienced in drilling holes within acceptable tolerance of diameter in the friable, fissured soils involved. This represents a basic limitation of application of pressuremeter testing to some strongly cemented, fissured CAF soils. Initially, it was attempted to drill pressuremeter holes with 2.50 inch diameter continuous flight auger. This auger proved insufficiently rigid to effectively drill holes except in about the upper 10 feet at Site B. Tricone and bicone gear bits of various diameters and a special fabricated drag bit, 2.25 inches in diameter, were used in initial attempts in drilling. Compressed air drilling methods were used with these drilling tools. The use of tricone gear bits, 2.94 inches in diameter, resulted in holes far larger than required for testing. After considerable experimentation, a system of drilling with a 2.37 inch diameter gear bit or 2.25 inch drag bit then reaming the holes with a specially fabricated 2.75 inch pipe with sawcat teeth was developed. This technique proved very successful in the CAF soils at Sites A and B and a large number of tests were efficiently performed after the initial development of the system. Extreme difficulty was experienced at Site C due to the degree of fissuring and only 4 satisfactory tests were accomplished with considerable effort.

Results of the pressuremeter tests are presented on Table 6 and shown graphically on Figure 76.

CHAPTER IV - GENERAL GEOLOGY & CHARACTER OF SOILS INVESTIGATED

GENERAL GEOLOGY & SOIL CHARACTERISTICS

THE GEOLOGY OF THE GREATER PHOENIX METROPOLITAN AREA IS TYPICAL OF THE LOWER VALLEYS OF ARIZONA WHERE SGC AND CAF SOILS ARE INVOLVED IN CONSTRUCTION.

THE SALT RIVER EMERGES FROM A NARROW CANYON EAST OF PHOENIX INTO THE BROAD SALT RIVER VALLEY. THE HIGH GRADIENT DISCHARGES OF THIS RIVER HAVE RESULTED IN MASSIVE DEPOSITS OF SGC. THE AGUA FRIA RIVER, GILA RIVER, NEW RIVER, CAVE CREEK, SKUNK CREEK AND QUEEN CREEK HAVE CREATED SIMILAR DEPOSITS OF LESSER EXTENT. THE GEOLOGY OF THE SALT RIVER VALLEY IS DESCRIBED BY LEE (22) AND MCDONALD, ET AL (23) WHILE THE GEOLOGY OF THE WESTERN PART OF THE VALLEY IS DISCUSSED BY STULIK AND TWENTER (24). THE SGC SOILS AND MUCH OF THE CAF ALLUVIUM IN THE PHOENIX AREA WERE DEPOSITED DURING THE TERTIARY PERIOD. THE DEPOSITION WAS CAUSED BY UPLIFT OF THE HIGH PLATEAU COUNTRY NORTH OF THE MOGOLLON RIM AND A CORRESPONDING WIDELY EXTENDED SUBSIDENCE OF THE AREA TO THE SOUTH AND WEST RESULTING IN DEEP EROSION OF THE HIGHLAND COUNTRY AND RAPID FILLING OF THE VALLEY AREAS. A MORE RECENT EROSIONAL PHASE OF THE SALT RIVER CHANNEL ASSOCIATED WITH A PERIOD OF DRIER CLIMATE IS DESCRIBED BY LEE (22) WHO MAPPED SGC TERRACES IN THE MESA AREA ABOUT 50 FEET HIGHER THAN THE PRESENT CHANNEL. SIMILAR TERRACE LEVELS PRESENT IN THE PHOENIX AREA ARE OBSCURED BY THE OVERLYING LAYER OF ALLUVIAL FAN DEPOSITS. WELL LOGS INDICATE THAT THE SGC DEPOSITS EXTEND TO SEVERAL HUNDREDS OF FEET IN MANY AREAS OF THE VALLEY.

THE SGC SOILS ARE EXPOSED AT THE SURFACE IN THE STREAM CHANNELS AND ALSO ARE EXPOSED IN NUMEROUS MATERIALS PITS and roadway cuts in the Phoenix area. They are overlain by a thin layer of loose silty sands and sandy silts over much of the floodplains of the various major drainages. Adjacent to the floodplains, the SGC are overlain by coalescing alluvial fan deposits sedimented by sheet floods and intermittent flows from small drainages out of the surrounding mountains. They consist predominantly of silty clays, sandy clays and clayey sands (Unified Soil Classification SC and CL) with lesser amounts of highly plastic sandy clays, silty sands, sandy silts and relatively clean sands (CH, SM, ML, SP, SW, SW-SM, SP-SM).

Because these soils are formed by a "flash flood" type of deposition, each layer dries out prior to the deposition of the overlying layer. When the overlying layer is deposited, only the surface of the existing soils are rewetted. Thus, the mass of these deposits have never been consolidated in a saturated state under overburden pressures. Where low density soils, particularly silty sands, sandy silts and clayey sands of relatively low plasticity are involved, this depositional process often results in extremely moisture sensitive "collapsing" soils. Dudley (36) presents a general review of this phenomenon. Parent materials in the area of the study are predominantly granitics, schists and volcanics. A very high incidence of severely collapsing soils is usually involved with granitic parent materials. These soils which are severely weakened by moisture increase and often relatively high in permeability are, of course, not considered suitable for drilled foundations supporting heavier loads.

Considerable thicknesses of "moderately" moisture sensitive soils also occur which, although firmer than the very low density deposits, are weakened by moisture increases to an extent that they are not generally thought to be safe for support of heavy foundation loads. Usually, at least 1 or

2 feet of moisture sensitive alluvial fan soils are present at the surface. These deposits seldom extend below about 25 feet.

A great proportion of alluvial fan soils are lime cemented to varying degrees and are not greatly affected by moisture increases. They usually contain high enough clay contents so they are relatively low in permeability. Thus, they are, in most cases, unlikely to become saturated to depths of more than a few feet in the environmental conditions involved. These soils occur at or near the surface in many areas and are nearly always shallow enough to allow efficient construction of drilled cast-in-place piles for most projects. These highly stratified and cross-bedded soils are generally fractured and fissured to some degree and contain scattered gravel and calcareous concretions in many cases. Except where heavy irrigation has taken place, moisture contents are at the very low values associated with well drained areas in the dry climate of the Sonoran Desert (average annual rainfall in Phoenix is about 7 inches). The infiltration of surface waters into these soils and subsequent drying and precipitation of salts, apparently has created the cementation present. These relatively stable cemented alluvial fan deposits have been termed CAF soils for purposes of this report. All of the soils at Sites B and C below 1 or 2 feet fall into this category.

Character of the SGC

The range of gradation of typical SGC samples are shown in Figure 19. Also tabulated in Figure 19 are particle shape and approximate percentage of rock types making up each size range for a typical sample of the SGC. The SGC consists predominantly of sandy gravel and cobbles with a small amount of silt and generally classifies GP in the Unified Soil Classification System. It generally contains some particles up to about 12 inches and sometimes contains scattered boulders up to about 24 inches. As indicated, the SGC contains a very high percentage of quartzite, chert and other very hard particles. This is reflected by very high wear on drilling tools in both foundation drilling and exploratory drilling into the deposit.

The approximate extent of the area within the Salt River Valley where, because of a relatively shallow contact and the character of the overlying alluvial deposits, it is economical for heavier structures to extend drilled foundations to the SGC is shown on Figure 9.

Local Discontinuities in the SGC

Both exposure on the walls of gravel pits and extensive test drilling indicate that the SGC is relatively uniform for a river channel deposit. However, local discontinuities occur which are significant from the engineering standpoint. Loose river deposited clean sand layers overlie the SGC at isolated locations. Because these layers sometimes contain scattered cobbles they are hard to distinguish from the SGC in some instances in exploratory auger drilling. Caving in these sand zones sometimes creates severe difficulties in advancing drilled foundations or other excavations to the contact of the SGC. In a small number of cases, loose sand or soft clay layers have been encountered within the SGC. Because of the erratic distribution of these features, careful subsurface exploration is necessary to insure uniformity of the SGC at sites of heavier structures.

In general, in the Phoenix area, about the upper 30 feet of SGC is uncemented or very weakly cemented. However, in some areas, the upper few feet of the SGC are strongly cemented;

apparently due to the leaching and precipitation of carbonates from overlying soils. These upper cemented zones are distributed at scattered locations around the Salt River Valley. Some of the more cemented zones in the SGC display the characteristics of lean concrete. The SGC below about 30 feet is generally moderately cemented with clay and other agents and contains some strongly cemented layers. The cementation in some cases may be associated in part with the older terrace levels discussed earlier.

Soil Profile - Site A

Uncemented or weakly cemented silty clays and sandy clays extend to about 7 feet. Moderately to strongly lime cemented clayey sands and sandy clays underlie these soils and extend to about 11 to 13½ feet and in turn rest on moderately cemented clayey gravels. SGC was encountered at between about 14½ and 16 feet. The SGC is uncemented and relatively uniform except for clean, fine to medium sand encountered at between 17½ and 19 feet and 22 to 23½ feet at TPA-7.

Soil moisture contents were very low throughout the extent of the borings.

Soil Profile - Site B

Moderately firm silty clays with a few stratifications of clayey sand and silty sand extended from the surface to about 19 feet. These soils are, in general, weakly cemented. They are underlain by hard, strongly cemented, highly plastic clays, silty clays and sandy silts which extended to about 27 feet. These soils in turn rest on firm to very firm, moderately to strongly cemented silty clays, sandy silts and clayey sands.

Soil moisture contents were variable, ranging from well below to near the plastic limit. Moisture contents appeared to be elevated somewhat above those normally found in well drained desert areas. The site had been under cultivation in the past and irrigation during that period probably created elevated moisture contents. The soils are friable and contain a moderate amount of fissuring or jointing, generally spaced at 6 inches or more. As the boring logs indicate, the soils are highly stratified with a considerable degree of lateral variation being present across the small test site. A relatively smooth surface texture was produced on the walls of drilled pile excavations; particularly in the upper 19 feet.

Soil Profile - Site C

A layer of moderately firm, weakly to moderately cemented silty clay 2 or 3 feet in thickness is present at the surface. This layer is underlain by highly stratified clayey sands and sandy clays of medium to high plasticity which extend to 30 feet; the full depth of investigation. These soils are generally moderately to strongly lime cemented and contain varying amounts of gravel. A thin, very strongly cemented zone about 1 to 3 feet in thickness was encountered in the range of 6 to 9 feet in depth in the various test holes.

Soil moisture contents were generally well below the plastic limit with moisture conditions being typical for the desert environment. The soils are intensely fissured with fissure spacing being about 1 inch. However, the mass of soil appeared to be weakened by fissuring much less in a vertical than a horizontal direction. Stronger, less weakened horizontal layers occurred at various intervals throughout the depth of piles. However, by actual digging in the wall of the drilled shaft with a prospector's pick, a general

CONDITION OF GREATER VERTICAL THAN HORIZONTAL STRENGTH WAS NOTED AT ALL DEPTHS. THE SURFACE TEXTURE OF THE FISSURING IS ROUGH. THIS WAS SPECIFICALLY NOTED AT THE TIME SHEAR COLLARS WERE CUT IN THE WALLS OF THE DRILLED SHAFTS BY HAND METHODS. OWING TO THE SAND AND GRAVEL FRACTION OF THE SOIL, AND THE PRESENCE OF FISSURING AND CALCAREOUS CONCRETIONS, A ROUGH SURFACE TEXTURE WAS PRODUCED ON THE WALLS OF DRILLED FOUNDATION EXCAVATIONS.

CHAPTER V - LOAD FRAME, INSTRUMENTATION SYSTEM & LOAD TEST PROCEDURES

Design of Load Frame & Hydraulic Jacking System

Previous studies by Whitaker and Cooke (5) and O'Neill and Reese (3) indicate that tests of about 1000 tons are the practical limit for a program of this type because of the cost and size of frames necessary to transfer loads from the reaction piles. Reaction beam lengths of about 20 to 22 feet are necessary to adequately minimize stress influence overlap between reaction and test piles for straight piles in relatively homogeneous soils. Reaction pile spacing was not considered critical for the SGC tests at Site A because the test piles are end-bearing and the belled reaction piles at approximately the same depth tend to relieve overburden pressure from the stressed zone and make the settlements somewhat conservative for the depth tested. However, reaction pile spacing is important for tests on the CAF soils at test Sites B and C so the load frame apparatus was designed accordingly.

Thousand ton tests were selected for the program to make the test loads as close as possible to full-scale foundations for heavier highway structures.

The mobile load frame designed and fabricated for the project consists of an all-welded A36 steel plate girder approximately 6 feet wide and 25 feet long. The frame is mounted on a heavy-duty dual axle trailer wheel assembly with a kingpin and plate assembly in front to receive the fifth wheel of a semi-tractor rig. The frame weighs approximately 26 tons and is supported by four hydraulic ram outriggers when not in tow by the semi-tractor unit. Provisions for slight adjustment of the cross beams with small hydraulic jacks have been incorporated into

THE DESIGN SO THAT THE UNIT CAN BE POSITIONED OVER THE TEST PILE AND REACTION PILES WITH CONSIDERABLE EASE.

ALSO INCORPORATED INTO THE DESIGN OF THE LOAD FRAME, IS THE CAPACITY TO UTILIZE A SINGLE REACTION PILE AT EACH END OF THE FRAME FOR APPLIED TEST LOADS UP TO 500 TONS. THIS DESIGN FEATURE PROVIDES VERSATILITY IN THE MOBILE LOAD FRAME SO IT CAN BE USED ECONOMICALLY AND EFFICIENTLY IN VIRTUALLY ANY LOAD TESTING PROGRAM.

THE STRUCTURAL DESIGN OF THE MOBILE LOAD FRAME, DEFINED BY THE REQUIREMENT TO IMPOSE A DOWNWARD LOAD OF 1000 TONS ON A TEST PILE, ALSO PERMITS AN UPLIFT LOAD CAPABILITY OF THE FRAME TO A MAGNITUDE OF 375 TONS. THE CAPACITY TO PERFORM UPLIFT TESTS ON A PILE OR ANCHOR INSTALLED IN A VERTICAL POSITION, IS ACCOMPLISHED BY ADEQUATELY BLOCKING EACH END OF THE LOAD FRAME, PLACING ANCHOR RODS THROUGH A PATTERN OF HOLES PROVIDED IN THE CENTER OF THE LOAD FRAME AND BY PLACING 1 OR 2 OF THE HYDRAULIC RAMS FROM THE JACKING MODULE ON TOP OF THE MOBILE FRAME.

THE LOAD FRAME IS MOBILIZED BY A 3 AXLE SEMI-TRACTOR RIG. WHEN THE LOAD FRAME IS IN POSITION, THE FRAME IS LEVELED WITH THE HYDRAULIC RAM OUTRIGGERS AND THE SEMI-TRACTOR RELEASED FOR THE REMAINDER OF THE TEST SET-UP AND LOAD TESTING PERIOD.

FOR THIS TESTING PROGRAM, THE LOAD FRAME WAS ANCHORED TO EACH REACTION PILE BY FOUR 1 3/8 INCH DIAMETER HIGH STRENGTH STRESSTEEL RODS (160,000 PSI). THE 4 ANCHOR RODS WERE THREADED INTO A 15 INCH SQUARE, 2 INCH THICK, HIGH STRENGTH STEEL PLATE HELD IN PLACE BY A TEMPLATE TO A POINT APPROXIMATELY 6 INCHES ABOVE THE BASE OF THE REACTION PILE. THE RODS WERE CAST IN THE CONCRETE ANCHOR PILES AND EXTENDED FROM THE

ANCHOR PLATE TO 3 INCHES BELOW GROUND ELEVATION. WHEN THE LOAD FRAME WAS IN POSITION, THE ANCHOR RODS WERE ATTACHED TO THE LOAD FRAME RODS BY MEANS OF A THREADED COUPLER. FIGURE 20 SHOWS THE LOAD FRAME IN THE LOAD TESTING POSITION WITH REACTION PILES ATTACHED. FIGURES 1 THROUGH 8 SHOW VARIOUS PHOTOGRAPHS OF THE LOAD FRAME AND AUXILIARY EQUIPMENT.

THE HYDRAULIC JACKING UNIT, CONSISTING OF FOUR 300 TON DOUBLE ACTING HYDRAULIC RAMS OPERATING IN SERIES, IS LOWERED FROM THE LOAD FRAME TO THE TOP OF THE PILE BY MEANS OF 2 HAND OPERATED RATCHET CABLE PULLERS. HYDRAULIC PRESSURE IS SUPPLIED TO THE HYDRAULIC RAM SYSTEM BY AN SC MODEL 600 AIR-HYDRAULIC PUMP. AIR PRESSURE TO ACTUATE THE HYDRAULIC PUMP IS SUPPLIED BY AN 85 CFM OR LARGER AIR COMPRESSOR AND FINELY CONTROLLED BY A DIAPHRAM VALVE.

THE FOUR HYDRAULIC RAMS WERE MANUFACTURED AND INDIVIDUALLY CALIBRATED FOR THIS PROJECT BY BAYOU INDUSTRIES OF CHANNELVIEW, TEXAS. A 20,000 PSI STAINLESS STEEL HEAVY-DUTY PRESSURE GAUGE MARKED IN INCREMENTS OF 200 PSI AND READABLE TO 100 PSI IS USED TO MEASURE PRESSURES OF THE CALIBRATED SYSTEM. A SYSTEM PRESSURE OF 13,100 PSI IS REQUIRED TO APPLY A 993 TON LOAD TO THE TOP OF THE TEST PILE.

DESIGN OF A TELLTALE INSTRUMENTATION SYSTEM FOR TEST SITES B & C

PREVIOUS STUDIES BY BARKER AND REESE (37) EVALUATED FIVE VARIOUS METHODS OF PILE INSTRUMENTATION FOR THE PURPOSE OF DEFINING THE DISTRIBUTION OF LOAD ALONG A DRILLED SHAFT BY MEASUREMENT OF STRAINS AT VARIOUS POINTS IN THE SHAFTS. THE NUMEROUS ADVANTAGES AND DISADVANTAGES OF EACH TYPE OF INSTRUMENTATION SYSTEM WERE EVALUATED WITH THE FOLLOWING CRITERIA NOTED.

1. An indirect mechanical telltale method of instrumentation would eliminate the high cost of sophisticated readout equipment required for electrical systems.

2. Strain gauge devices are highly sensitive to moisture penetration when buried in concrete and elaborate methods to keep moisture out of the devices must be employed.

Considering these factors, a telltale instrumentation system was selected for the CAF soils at Sites B and C where an evaluation of load transfer was necessary. Due to the inherent lateral and vertical variability of soils involved, it was desired to perform as large a number of tests as possible and to minimize instrumentation costs.

Design considerations for the mobile load frame placed limitations on a telltale method of pile instrumentation. A maximum distance of 8 inches from top of pile to ground level is necessary to permit adequate clearance between the jacking module and top of pile at the time the mobile frame is pulled into testing position by the semi-tractor rig. The four 300 ton hydraulic rams in the jacking module have a minimum base and top plate dimension of 24 inches square. Likewise, a total test load of 1000 tons applied to the pile surface required a plate thickness of 5 inches. Little or no area remains available on the pile surface and in the peripheral area above ground level to efficiently permit the protrusion of a telltale system.

A telltale type compressometer, shown on Figure 21, was developed for the study using standard pipe fittings and sections. Movement of the device is sensed by a linear voltage displacement transducer (L.V.D.T.) held securely in a housing by allen screws. The L.V.D.T. stylus rests on top

OF A STAINLESS STEEL WIRE HELD TAUT BY A 2 INCH LONG COMPRESSED SPRING. FIGURE 21 SHOWS THE ASSEMBLED UNIT IN POSITION IN THE TEST PILE. A FLUKE DIGITAL VOLTMETER OWNED BY THE ARIZONA HIGHWAY DEPARTMENT WAS USED FOR MICROVOLTAGE READOUT OF EACH L.V.D.T. COMPRESSOMETER. A TRANSFORMER-SWITCHING UNIT PROVIDED AN EXCITATION OF 24 VOLTS D.C. FROM A 110 VOLT A.C. POWER SOURCE AND A MULTIPLE POSITION SWITCH TO PERMIT A RAPID VOLTAGE READING FROM 1 TO 8 L.V.D.T. UNITS.

THE L.V.D.T. COMPRESSOMETERS WERE ASSEMBLED TO PREDETERMINED LENGTHS AND ATTACHED IN PAIRS (180 DEGREES APART) TO THE REINFORCING CAGE OF EACH TEST PILE. THE REINFORCING STEEL CAGE WAS FABRICATED FROM SPIRALLY ROLLED NO. 3 SMOOTH WIRE AND FOUR NO. 7 LONGITUDINAL REINFORCING STEEL RODS.

THE INSTRUMENTED CAGE WAS SUSPENDED FROM A TEMPLATE AT GROUND LEVEL AND SO CONSTRUCTED TO HOLD EACH L.V.D.T. COMPRESSOMETER TELLTALE IN POSITION DURING THE CONCRETE POUR. THE CONCRETE, WHEN POURED IN EACH TEST PILE, WAS TEMPORARILY STOPPED JUST BELOW THE 2 INCH P.V.C. PLASTIC CAP; AT WHICH TIME THE SPRING WAS COMPRESSED AND THE WIRE SECURED WITH THE SCREW CAP. THE 2 INCH P.V.C. PIPE WAS THEN INSERTED IN THE P.V.C. CAP, STUFFED FULL WITH RAGS AND THE REMAINDER OF THE CONCRETE POURED AND FINISHED. FOLLOWING THE INITIAL SET, AND WHILE THE CONCRETE WAS STILL GREEN, GROOVES WERE ETCHED IN THE CONCRETE TO PERMIT PASSAGE OF THE L.V.D.T. WIRE UNDER THE 5 INCH STEEL PLATE AT TIME OF LOAD TESTING. THIS PROCEDURE WORKED VERY WELL DURING CONSTRUCTION STAGES OF THE TEST PILES AND PROVIDED AN EFFICIENT L.V.D.T. ASSEMBLY AT THE TIME THE LOAD TESTS WERE PERFORMED. THE L.V.D.T. AND HOUSING UNITS WERE INSERTED AT THE TIME OF LOAD TESTING AND REMOVED FOLLOWING EACH LOAD TEST.

Test Piles & Reaction Piles

Seven load tests were performed at Site A. Test pile diameters were selected varying from 2.5 to 9.3 feet so the effect of width of loaded area could be investigated. All but the 2.5 foot diameter piles were belled. The shafts of the piles were separated from the soil by sonotube so that the tests were strictly a measure of end-bearing. A visible annular space was present between the sonotube and surrounding soil and no friction was encountered in placing the sonotube. The maximum diameter was selected so that the bearing pressure at 1000 tons (28.6 ksf) would be on the order of those which would be used in full-scale design. Test pile depths varied from 15.5 to 18.5 feet.

Ten test piles were performed at both Sites B and C. Straight piles, belled piles and piles with small multiple bells or "shear collars" were tested. Piles TPC-4 through TPC-6 were tested in end-bearing only by separating the shafts from the soil by means of sonotube. Voids were created beneath piles TPC-7 through TPC-10 by placing ice in the lower 2 feet of the excavations so that they could be tested in side shear only. The effectiveness of creating voids was verified by probing through small casings installed at time of test pile construction.

The configuration and details of the test piles for the three sites are shown schematically in Figures 22 through 26. The location of the test and reaction piles are shown in Figures 13 through 15.

Reaction pile capacity was analyzed by methods developed by Meyerhof and Adams (38). Belled piles, 7.0 feet bell diameter and 2.5 feet shaft diameter, were designed for each

SITE. REACTION PILE DEPTHS WERE ABOUT 16 FEET AT SITE A, 20 FEET AT SITE B AND 16 FEET AT SITE C.

EITHER 4 REACTION PILES IN A 7 X 20 FOOT DIAMETER OR 2 REACTION PILES SPACED 23'6" APART, DEPENDING UPON THE TOTAL TEST LOAD, WERE USED. TABLES 1, 2 AND 3 SHOW THE EXACT DIMENSIONS OF THE VARIOUS TEST AND REACTION PILES.

CONCRETE WAS A 5000 PSI 28 DAY STRENGTH DESIGN PLACED AND VIBRATED AT A 3 TO 5 INCH SLUMP (A 7.5 SACK MIX). RESULTS OF THE CONCRETE COMPRESSION AND SLUMP TESTS ARE REPORTED IN APPENDIX C (PAGES C-105 THROUGH C-108).

LOAD TESTING PROCEDURES

THE "MAINTAINED LOAD" TEST PROCEDURE WAS USED FOR ALL TESTS IN THIS STUDY WITH LOADING INCREMENTS VARYING FROM ABOUT 23 TONS TO 60 TONS. EACH LOAD INCREMENT WAS MAINTAINED FOR 30 MINUTES AND IN SOME TESTS, 60 MINUTES. HOWEVER, TEST PILES TPA-7 AT A LOAD INCREMENT OF 689 TONS (19.9 KSF END-BEARING PRESSURE), TPB-2 AT A LOAD INCREMENT OF 263 TONS AND TPC-1 AT A LOAD INCREMENT OF 567 TONS WERE HELD FOR EXTENDED TIME PERIODS OF 1060 MINUTES, 990 MINUTES AND 1320 MINUTES, RESPECTIVELY.

DIGITAL VOLTMETER READINGS FROM THE L.V.D.T. COMPRESSOMETER TELLTALES WERE TAKEN AT 4 MINUTES AND 19 MINUTES WHEN 30 MINUTE TIME INCREMENTS WERE USED AND AT 4, 19 AND 49 MINUTES WHEN 60 MINUTE TIME INCREMENTS WERE USED ON THE VARIOUS TEST PILES AT SITES B AND C. STRAIN READINGS OBTAINED ARE PRESENTED IN APPENDIX C (PAGES C-109 THROUGH C-129).

SETTLEMENTS, REFERENCED TO THE GROUND SURFACE, WERE MEASURED

WITH TWO 3 INCH DIAMETER FACE EXTENSOMETER DIALS WITH A MEASUREMENT RANGE OF 3 INCHES AND SENSITIVE TO THE NEAREST 0.001 INCH. THE DIALS WERE SUPPORTED ON 4 X 4 INCH WOOD REFERENCE BEAMS WITH STEEL PIN REACTIONS BEING 10.5 FEET FROM THE CENTER OF THE TEST PILES.

THE LOAD-SETTLEMENT CURVES FOR THE TEST PILES AT SITES A, B AND C ARE GIVEN IN FIGURES 27 THROUGH 34. TIME-SETTLEMENT CURVES FOR EACH LOAD INCREMENT ARE SHOWN IN FIGURES 35 THROUGH 65. LONG-TERM PORTIONS OF THE TIME-SETTLEMENT CURVES FOR THE EXTENDED TIME PERIODS OF TPA-7, TPB-2, TPB-5 AND TPC-1 ARE SHOWN IN FIGURES 42, 45, 49 AND 56.

THE LOAD AND TIME SETTLEMENT CURVES ARE CORRECTED FOR COMPRESSION IN THE CONCRETE AT TEST SITE A AND REPRESENT MOVEMENT AT THE BASE OF THE PILES. TOTAL CORRECTION AT MAXIMUM LOADS OF 993 TONS APPLIED WAS ABOUT 0.13 INCH. THE REPORTED SETTLEMENTS ARE BELIEVED TO ACCURATELY REFLECT SETTLEMENTS AT THE GROUND SURFACE OF PROTOTYPE PILES AS COMPRESSION IN THE CONCRETE WILL BE VERY SLIGHT FOR NORMAL WORKING STRESSES. SETTLEMENT READINGS AT TEST SITES B AND C WERE NOT CORRECTED FOR ELASTIC COMPRESSION IN THE CONCRETE DUE TO THE LOWER TOTAL LOADS APPLIED TO MOST OF THE TEST PILES.

CHAPTER VI - EVALUATION OF RESULTS - SITE A

General

The load tests at Site A were intended to investigate the performance of drilled piles under the range of bearing pressures likely to be used for actual foundations. The practical problem for the SGC soils was to evaluate the relationship between settlement, bearing pressure and width of foundation so that design curves could be developed to estimate settlements of full-scale foundations. It does not appear that providing an adequate factor of safety against shear failure will be a problem for drilled foundations bearing on the SGC soils for the range of bearing pressures, diameters and depths likely to be used in design.

Ultimate bearing pressures for the test piles were calculated by the semi-empirical Terzaghi method (2) which is intended for the analysis of shallow foundations. A range of angles of internal friction, Ø, of 38 to 44 degrees were selected for use in computations based on data presented by Leps (39) for coarse granular materials. Results of ultimate bearing capacity computations are given in Table 4. As can be seen, an ultimate bearing pressure of 294 kips/sq. ft. was calculated for TPA-1 using $Ø = 44^{\circ}$. An actual bearing pressure of 425 kips/sq. ft. was reached at maximum test load and it did not appear that failure had been defined. For the larger diameter piles, the maximum capability of the load frame did not permit the test to reach the levels of the computed values. However, the test for the 2'5" diameter pile (TPA-1) exceeded the computed ultimate value of the 9'5" diameter pile. It appears the Terzaghi method is well on the conservative side for the configuration of piles involved and can be safely used as a check of ultimate bearing capacity. More realistic evaluations of ultimate bearing capacity might be obtained by methods proposed by other investigators such as Meyerhof (40).

ARCHING PHENOMENON DISCUSSED BY VESIC (16) SHOULD BE CONSIDERED IN THE COMPUTATIONS FOR DEEPER DRILLED PILES.

EFFECT OF VARIATIONS IN SOIL CHARACTERISTICS

FOR PURPOSES OF ANALYSIS, SETTLEMENT INDICATED BY THE LOAD TESTS IN THIS STUDY FOR 20 KIPS PER SQUARE FOOT BEARING PRESSURE HAVE BEEN PLOTTED VERSUS BASE DIAMETER IN FIGURE 66. AS INDICATED, A DEGREE OF SCATTER OF DATA IS PRESENT. THE LARGER SETTLEMENT FOR TPA-7 PROBABLY IS DUE, IN PART, TO THE SAND LENSE PRESENT AT ABOUT 3.5 TO 5.0 FEET BELOW ITS BASE. HOWEVER, MUCH OF THE SCATTER IS UNDOUBTEDLY DUE TO VARIATIONS IN RELATIVE DENSITY. RESULTS OF SOME LARGE DIAMETER LABORATORY COMPRESSION TESTS (ONE-DIMENSIONAL CONSOLIDATION TESTS) PERFORMED BY KJAERNSLI AND SANDE (41) ON COARSE GRAVELS SIMILAR TO THE SGC ILLUSTRATE THE IMPORTANCE OF RELATIVE DENSITY. THESE TESTS INDICATED THAT LOOSE MATERIALS WERE ABOUT 50 PERCENT MORE COMPRESSIBLE THAN DENSE MATERIALS OF THE SAME GRADATION.

OWING TO THE LARGE DIAMETER OF THE PARTICLES INVOLVED, THE RELATIVE DENSITY OF THE SGC SOILS IS DIFFICULT TO DETERMINE. HOWEVER, THE SGC UNDOUBTEDLY HAS THE VARIATIONS IN RELATIVE DENSITY THAT ARE INEVITABLY PRESENT IN NATURALLY OCCURRING GRANULAR SOIL DEPOSITS.

RELATIVE DENSITY DETERMINATIONS IN THE SGC HAVE BEEN MADE AT A PLATE LOAD TEST SITE ABOUT 1.5 MILES EAST OF SITE A. THE TESTS WERE, IN PART, IN FINER INTERVALS OF THE DEPOSIT. LARGE VOLUME IN-PLACE DRY DENSITIES DETERMINED BY THE USE OF CALIBRATED SILICA SAND RANGED FROM 116 TO 140 POUNDS PER CUBIC FOOT. LABORATORY DENSITY DETERMINATIONS BY ASTM D2049-69 (33) INDICATED MAXIMUM DENSITIES BETWEEN 139 AND 144 AND MINIMUM DENSITIES BETWEEN 118 AND 122. RELATIVE DENSITIES RANGE FROM 50 TO 96 PERCENT.

Although these tests may not be indicative of the general characteristics of the deposit, they do illustrate variations in relative density. Although the SGC is relatively uniform across the site, small variations in gradation are, no doubt, partly responsible for variations in the settlement of individual test piles from an "average" curve. In general, the compressibility of granular soils increases with increasingly finer gradation. The data reported by Kjaernsli and Sande (41) involving hard, smooth, rounded particles up to about $2\frac{1}{2}$ inches in diameter, indicate that well graded materials are somewhat less compressible than poorly graded materials at the same relative density.

Evaluation of Becker Hammer Drill Data

As a part of the foundation investigations for 2 sections of elevated freeway, a total of 285 Becker Hammer Drill borings were drilled along a corridor beginning about $\frac{1}{2}$ mile west of test Site A and extending easterly for 3 miles. Borings penetrated the SGC about 30 to 75 feet. They provide a valuable body of information on the uniformity of the deposit.

Clay and clayey sand layers within the SGC extensive enough to require special consideration in design were encountered in 10 borings; looser sand layers extensive enough to require special consideration occurred in 56 cases. Some of these layers were near the contact of the SGC and could be avoided by extending foundations a few feet below the contact.

In order to evaluate how Becker Hammer Drill blow count, N_B, correlated with performance of the test piles, weighted N_B was determined for the test holes drilled immediately adjacent to the piles. Weighting was made on the basis of relative Boussinesq stress influence at the midpoint of each 6 inch layer to a depth of about 2 diameters below the base of piles. Calculation procedures were similar to those described by Coon and Merritt (42) for the correlation of diamond core recovery of rock to modulus of deformation. The following results were obtained.

Test Pile	Blows Per Foot: Weighted N_B	Blows Per Foot: Mean N_B	Difference
TPA-1	50.10	47.12	2.98
TPA-2	37.86	38.00	0.14
TPA-3	49.56	47.74	1.82
TPA-4	28.32	29.14	0.82
TPA-5	50.96	49.58	1.38
TPA-6	45.56	44.76	0.80
TPA-7	37.02	41.16	4.14

The weighted N_B values varied by no more than 4.14 blows per foot from mean values for the same range of depth. Modulus of deformation of the SGC was computed for each test pile at various bearing pressures by the theory of elasticity. Procedures outlined by Bowles (43) were used as follows:

$$E = \frac{qD\,(1-u^2)\,I_w}{S} \quad \ldots\ldots\ldots\ldots \quad \text{(Equation 1)}$$

Where:

E = modulus of deformation

q = bearing pressure

D = diameter of pile

S = settlement of pile

u = Poisson's ratio

I_w = influence factor (for end-bearing only)

An influence coefficient, I_w, of 0.88 was used for a rigid circular foundation. Void ratios for the SGC appear to range from about 0.2 to 0.4 and it is believed that the average

VALUE IS NEAR THE LOWER FIGURE. THUS, $u = 0.15$ WAS ASSUMED IN CALCULATIONS.

BECAUSE GRANULAR SOILS DO NOT BEHAVE ELASTICALLY, AND IN NATURE ARE NOT COMPLETELY HOMOGENEOUS OR ISOTROPIC, THE CALCULATED MODULUS E IS AN APPROXIMATION OF THE AVERAGE MODULUS OF DEFORMATION OF A ZONE OF SOIL WHICH IS SIGNIFICANTLY STRESSED.

THE RELATIONSHIP BETWEEN E AND Q FOR EACH TEST PILE IS SHOWN ON FIGURE 67. FIGURE 17 SHOWS THE RELATIONSHIP BETWEEN E AND WEIGHTED N_B FOR VARIOUS BEARING PRESSURES. WEIGHTED N_B VERSUS E AND Q VERSUS E FOR TEN 30 INCH PLATE LOAD TESTS PERFORMED AT VARIOUS DEPTHS IN THE BOTTOM OF TWO LARGE DIAMETER BORINGS ABOUT 1.5 MILES EAST OF SITE A ARE ALSO SHOWN ON FIGURES 67 AND 17. AS CAN BE SEEN, E INCREASES IN A GENERAL WAY WITH INCREASING N_B.

DURING THE COURSE OF EXPLORATORY DRILLING, STANDARD PENETRATION TESTS WERE TAKEN THROUGH THE BECKER DRIVE PIPE IN A NUMBER OF CASES WHERE FINE ENOUGH GRANULAR INTERVALS WERE PRESENT SO THE SPLIT BARREL SAMPLER COULD BE DRIVEN AT LEAST A FOOT. BECKER BLOW COUNT, N_B, IS PLOTTED AGAINST STANDARD PENETRATION RESISTANCE, N, FOR THESE TESTS IN FIGURE 18. THE STANDARD PENETRATION TESTS WERE UNDOUBTEDLY AFFECTED TO SOME DEGREE BY THE VIBRATIONS OF THE BECKER DRILLING. HOWEVER, THEY INDICATE A VERY GENERAL RELATIONSHIP BETWEEN N_B AND N. SIMILAR DATA DEVELOPED BY WALKER (44) ALSO IS SHOWN ON FIGURE 18. THIS DATA SUGGESTS THAT CAREFULLY PERFORMED AND ANALYZED BECKER DRILLING PRODUCES A DYNAMIC PENETRATION RESISTANCE AS MEANINGFUL AS THE STANDARD PENETRATION TEST. ALSO INCLUDED IN FIGURE 18 IS THE "BEST FITTED" CURVE CONSTRUCTED BY REGRESSION ANALYSIS.

FROM THE DATA DISCUSSED ABOVE, IT IS BELIEVED THAT N_B INDICATES GENERALLY THE DEGREE OF VARIABILITY OF COMPRESSIBILITY OF THE SGC. NORMAL DISTRIBUTION OF MEAN N_B IN THE UPPER 20

FEET OF THE SGC FOR THE TWO FOUNDATION INVESTIGATIONS INVOLVING 155 AND 130 BORINGS AND THE MEAN N_B AT SITE A WAS ASSUMED. GRAPHIC REPRESENTATION OF FREQUENCY OF BLOW COUNT OCCURRENCE FOR THE ABOVE MENTIONED THREE SETS OF BECKER BLOW COUNT DATA IS SHOWN ON FIGURE 68. THE WIDE RANGE OF DISTRIBUTION FOR UNIT II IS BELIEVED TO BE DUE TO THE FACT THAT ONE OF THE TWO BECKER HAMMER DRILLS USED WAS FOUND TO BE DELIVERING MUCH LESS THAN 8000 FOOT POUNDS PER BLOW HAMMER ENERGY. THIS WAS CORRECTED ABOUT HALFWAY THROUGH THE DRILLING PROGRAM. HOWEVER, AS FIGURE 68 INDICATES, THE MEAN N_B FOR SITE A IS CONSIDERABLY LOWER THAN INDICATED FOR THE DEPOSIT AS A WHOLE AND ITS RANGE OF DISTRIBUTION COVERS THE LOWER RANGE TO BE EXPECTED IN THE DEPOSIT.

THE VARIATION OF MEAN N_B FROM 29 TO 49 OVER TEST SITE A, 30 x 80 FEET IN PLAN DIMENSION, ILLUSTRATES THE EXTREME DEGREE OF LATERAL VARIATION WITHIN THE DEPOSIT.

EFFECT OF WIDTH OF LOADED AREA

THE MODULUS OF DEFORMATION OF GRANULAR SOILS INCREASES WITH DEPTH. THUS, FOR A GIVEN BEARING PRESSURE, SETTLEMENTS OF FOUNDATIONS ON RELATIVELY HOMOGENEOUS, CLEAN, COHESIONLESS, GRANULAR SOILS INCREASE WITH INCREASING WIDTH OF LOADED AREA AT A DECREASING RATE. THE FACTORS INFLUENCING THE COMPRESSIBILITY OF GRANULAR SOILS AND THEIR VARIATIONS ARE DISCUSSED IN DETAIL BY BURMISTER (45, 46).

FOR PURPOSES OF ANALYSIS OF THE EFFECT OF FOUNDATION WIDTHS ON SETTLEMENTS, A POINT OF S = 0.2 INCH AND D = 6.0 FEET WAS ASSUMED ON FIGURE 66 FOR Q = 20 KIPS/SQ. FT. THIS POINT IS CONSISTENT WITH THE PERFORMANCE OF TPA-3, TPA-4, TPA-5 AND TPA-6 AND WAS THOUGHT TO BE NEAR AN "AVERAGE" CURVE FOR THE deposit. A modulus of deformation, E, of 29,700 psi was computed for this point. Settlements were then calculated for other widths on a curve by Equation 1; using the "average point" method suggested by Lambe and Whitman (47). In this method, the "average point" of stress within the bulb of significant stress influence is considered to be 0.75 D below the base of footing. E can then be varied as a function of the initial vertical confining pressure P_z at the "average point" as D changes. Curves are plotted on Figure 66 computed on the assumptions of E varying directly with P_z and E varying with the square root of P_z. A curve is also shown constructed on the assumption that E is constant with depth.

A number of investigators (46, 48, 49, 50, 51, 52) have reported case histories where the relationship between foundation width and settlement for a given bearing pressure was developed by field measurements. Several curves constructed from this data are plotted on Figure 66 to compare with the calculated curves.

Both Terzaghi and Peck (48) and Bjerrum and Eggestad (49) have presented data on the relationship between the settlement of small square plates and foundations of various widths. The Bjerrum and Eggestad (49) curves are based upon the evaluation of 14 case histories where detailed information on the soils, plate bearing tests and measurements of settlements of full-scale foundations were available.

Burmister (46) reports on a case of a 65 foot wide reactor foundation bearing on well graded sand with some gravel where small diameter plate bearing tests were performed in preliminary design studies.

Moorhouse and Sheehan (50) have summarized information on

THE SETTLEMENT-WIDTH RELATIONSHIP FOR PILE GROUPS INCLUDING STUDIES BY SKEMPTON, ET AL (51) AND MEYERHOF (52). BECAUSE MUCH OF THE DATA USED IN DEVELOPING THESE CURVES INVOLVED DRIVEN PILES, THEY ARE BELIEVED TO BE INFLUENCED BY DENSIFICATION OF THE SANDS BELOW THE PILE TIPS DURING DRIVING; PARTICULARLY FOR SMALLER WIDTHS.

DVORAK (53) REPORTS THE RESULTS OF BEARING TESTS WITH 0.8 AND 1.2 FOOT DIAMETER CIRCULAR PLATES AND A 2.3 FOOT SQUARE PLATE ON WELL GRADED SAND AND GRAVEL UP TO ABOUT 2.5 TO 4.0 INCHES IN DIAMETER WHICH APPEAR SIMILAR TO THE SGC. THE SETTLEMENTS INDICATED BY THESE TESTS AT 20 KSF ALSO ARE PLOTTED ON FIGURE 66 FOR COMPARISON ALONG WITH THOSE FROM THE 30 INCH PLATE BEARING TESTS PERFORMED ON THE SGC.

IN FURTHER EVALUATION OF THE BJERRUM AND EGGESTAD (49) DATA, MEIGH (54) STATES THAT SETTLEMENTS FOR COARSE, WELL GRADED MATERIALS FELL IN THE LOWER SECTOR OF THEIR LIMITS WHILE SETTLEMENTS FOR FINE, POORLY GRADED MATERIALS FELL WITHIN THE UPPER SECTOR. THE SGC, OF COURSE, FALLS IN THE CATEGORY OF COARSE, WELL GRADED MATERIALS.

FROM EVALUATION OF THE VARIOUS CASE HISTORY DATA, IT APPEARS THE MOST LIKELY "AVERAGE CURVE" FOR THE SGC IS NEAR THE CURVE BASED UPON E VARYING DIRECTLY WITH THE SQUARE ROOT OF P_z. THE CURVE, BASED UPON E VARYING DIRECTLY WITH P_z, SEEMS TO BE THE UPPER LIMIT OF WHERE AN "AVERAGE CURVE" COULD REASONABLY BE EXPECTED TO FALL. "AVERAGE CURVES" CONSTRUCTED ON THIS BASIS ARE PRESENTED IN FIGURE 77 FOR Q = 10, 15, 20, 25 AND 30 KIPS/SQ. FT.

EFFECT OF TIME

THE LONG-TERM MAINTAINED LOAD ON TPA-7 SHOWED THAT SETTLEMENT

WAS COMPLETE AT ABOUT 11 HOURS. THIS LOAD WAS MAINTAINED AT 19.9 KIPS PER SQUARE FOOT WHICH IS EXPECTED TO BE IN THE GENERAL RANGE OF BEARING PRESSURES WHICH WILL BE USED IN THE DESIGN OF FOUNDATIONS FOR VERY HIGH TOTAL LOADS ON THE SGC DEPOSIT. ULTIMATE SETTLEMENT WAS 19 PERCENT HIGHER THAN THE SETTLEMENT AT 30 MINUTES. SIMILAR TIME-SETTLEMENT RELATIONSHIPS WERE INDICATED BY THE 30 INCH DIAMETER PLATE BEARING TESTS. THIS RELATIVELY RAPID TIME-RATE OF SETTLEMENT RELATIONSHIP IS TYPICAL OF MUCH PREVIOUS DATA REPORTED FOR FOUNDATIONS ON GRANULAR SOILS.

CHAPTER VII - EVALUATION OF RESULTS - SITES B & C

Evaluation of Load Transfer

The telltale instrumentation system was calibrated in the shafts of TPA-3, TPB-3, TPC-4, TPC-5 and TPC-6, which are a free-standing column without side friction. Measured strains in TPA-3 closely corresponded with those calculated using the modulus of elasticity of the concrete determined in the laboratory. Telltale data, however, indicated considerably lower strains in the free-standing calibration piles at Sites B and C. The reason for this performance is uncertain.

Load transfer was calculated from the relative strains indicated by the telltales in the test piles by use of a calibration constant developed from free-standing calibration piles. In evaluating the tests, failure was defined as "plunging failure" where little or no additional resistance was developed with increasing settlement. Defined failure loads are listed in Table 8 along with the portion of ultimate loads estimated to have been transferred in end-bearing and side shear. Shaft compression indicated by the instrumentation is shown in Figures 79 through 95.

Basic Approach for the Calculation of Ultimate Capacity

The ultimate capacity of the test piles was calculated by several rational and empirical procedures in order to compare available methods for predetermining capacities with various field and laboratory test data.

The basic rational procedure is usually expressed as follows:

$$Q_T = Q_S + Q_B = A_S q_S + A_B q_B \quad \text{. . . . . . (Equation 2)}$$

In which: Q_T = total ultimate capacity
Q_S = ultimate side load
q_S = average ultimate unit side shear
Q_B = ultimate base load
q_B = ultimate unit base resistance
A_S = side area of pile
A_B = base area of pile

The side shear expression is as follows:

$$Q_S = A_S q_S = A_S \,(ac + K\gamma z \tan\delta) \quad \ldots \text{(Equation 3)}$$

Where:
a = cohesion reduction coefficient
c = unit cohesion
K = earth pressure coefficient
γ = average effective density of soil above depth z
z = depth from ground surface to center of section considered
δ = angle of friction between the soil and surface of the pile

Because most CAF soils are stratified, side shear is usually analyzed in several separate layers.

The Terzaghi expression for end-bearing of shallow footings can be modified for deep foundations as follows:

$$Q_B = A_B q_B = A_B \,(c\,N_c + D\gamma N_q) \quad \ldots\ldots \text{(Equation 4)}$$

D = depth of pile base
γ = average effective density of soil above depth D

N_C AND N_Q ARE DIMENSIONLESS FACTORS DEPENDING UPON THE SHAPE OF THE PILE, THE ANGLE OF INTERNAL FRICTION, $\emptyset$, AND THE SHAPE OF THE FAILURE PLANE AND ROUGHNESS OF BASE ASSUMED BY VARIOUS INVESTIGATORS.

O'NEILL AND REESE (3) PRESENT A DETAILED DISCUSSION OF VARIOUS FACTORS AFFECTING CAPACITY COMPUTATIONS BY THE RATIONAL APPROACHES.

CALCULATION METHODS USED IN ANALYSIS

THE FOLLOWING CALCULATION PROCEDURES WERE USED IN ANALYSIS OF ULTIMATE BEARING CAPACITIES.

METHOD 1. THE RATIONAL APPROACH OUTLINED ABOVE WITH SHEAR STRENGTH BEING ESTIMATED FROM DIRECT SHEAR TEST DATA. SIDE SHEAR WAS COMPUTED FROM EQUATION 3 WITH THE FOLLOWING ASSUMPTIONS:

$$Q_s = A_s\,(ac + K\gamma z \tan\delta) \quad \ldots\ldots\ldots \quad \text{(Equation 3)}$$

$$a = 1.0$$

$$\delta = \emptyset$$

$$K = 1.0$$

THERE IS CONSIDERABLE QUESTION AS TO THE STATE OF HORIZONTAL STRESS PRODUCED AT THE PERIMETER OF A DRILLED PILE, PARTICULARLY IN HEAVILY OVERCONSOLIDATED DEPOSITS LIKE THE CAF SOILS INVOLVED IN THIS STUDY.

AS CONCRETE IS PLACED AND VIBRATED FOR DRILLED PILING, IT IS BELIEVED THAT LATERAL PRESSURES SLIGHTLY LESS THAN THE FLUID PRESSURES OF THE CONCRETE ARE ESTABLISHED AGAINST THE WALLS OF THE EXCAVATION. FOR THIS REASON, AN EARTH PRESSURE COEFFICIENT, K, OF 1.0 WAS CONSIDERED A REASONABLE MINIMUM VALUE. WITH THIS ASSUMPTION, WHEN A DIRECT SHEAR TEST IS PERFORMED

WITH THE NORMAL STRESS BEING EQUAL TO THE EFFECTIVE OVERBURDEN PRESSURE AT THE POINT OF SAMPLING, THE SHEARING STRESS DEFINES Q_S AT THAT DEPTH. AVERAGE Q_S INDICATED BY DIRECT SHEAR TESTS WAS USED. SHEAR TEST DATA FOR THE TWO SITES ARE SHOWN IN FIGURES 74 AND 75.

END-BEARING WAS DETERMINED BY EQUATION 4 WITH THE FOLLOWING ASSUMPTIONS:

N_Q = TERZAGHI VALUES OF N_Q* PRESENTED BY VESIC (16)
N_C = ORIGINAL TERZAGHI (2) GENERAL SHEAR VALUES FOR SHALLOW FOOTINGS

THE FOLLOWING SHEAR STRENGTH PARAMETERS WERE ESTIMATED FOR END-BEARING COMPUTATIONS. BEARING CAPACITY FACTORS USED IN THE COMPUTATIONS ARE ALSO LISTED.

SITE	DEPTH	c	Ø	N_C	N_Q
B	0-19'	1.9 KSF	33.0^{o}	48	41
B	19'+	2.0 KSF	31.5^{o}	42	33
C	0-30'	2.0 KSF	29.2^{o}	34	28

METHOD 2. SAME AS METHOD 1 EXCEPT THE UPPER RANGE OF DIRECT SHEAR TESTS INSTEAD OF AVERAGE VALUES WERE USED IN DEFINING Q_S FOR SIDE SHEAR COMPUTATIONS.

METHOD 3. SIDE SHEAR DETERMINED AS IN METHOD 1. END-BEARING DETERMINED APPLYING AN EMPIRICAL FACTOR TO DIRECT SHEAR TESTS PERFORMED AT THE EFFECTIVE OVERBURDEN PRESSURE AT THE POINT OF SAMPLING IN A MANNER SIMILAR TO THAT PRESENTED BY MOORE (55). THE FOLLOWING EXPRESSION WAS USED BASED UPON ANALYSIS OF THE LOAD TEST DATA.

Q_B = 10s . . . . (EQUATION 5)

WHERE: $s = c + z\gamma \tan Ø$ (AS DEFINED BY DIRECT SHEAR TEST DATA)

Average of direct shear test data was used in calculations.

Method 4. Same as Method 3 except that the upper limit of direct shear test data was used in calculations of Q_B.

Method 5. Bearing capacity computed by Equation 2 with Q_S and Q_B determined empirically from pressuremeter data. The procedure is a modification of that presented by Menard (34). The following expressions were used:

$$Q_S = S_O$$

Where: S_O = shear strength computed from pressuremeter data

$$Q_B = 1.4\ P_L$$

Where: P_L = limit pressure determined by pressuremeter tests

Average of pressuremeter data was used. Pressuremeter data for Sites B and C is shown on Figure 76 and Table 6.

Method 6. Modification of Method 5 as follows:

$$Q_B = P_L$$

$$Q_S = S_O$$

except maximum Q_S = 7.0 ksf for piles with rough surface texture (Site C condition)

maximum Q_S = 5.0 ksf for piles with smooth surface texture (Site B condition)

Method 7. Strength values were correlated empirically with standard penetration test resistance, N, resulting in the following assumptions for determining capacities:

$q_s = \frac{N}{10}$ kips/square foot maximum q_B = 75.0 ksf

$q_B = N$ kips/square foot

maximum q_s = 7.0 ksf for piles with rough surface texture (Site C condition)

maximum q_s = 5.0 ksf for piles with smooth surface texture (Site B condition)

Load test data, the relationship between N and the shear strength data developed in this investigation, given on Figure 71, and various correlations summarized by Campbell and Hudson (31) were considered in developing these expressions. Average values of N were used, ignoring isolated refusal.

Method 8. For the shallow piles at Site B where data is available, capacities were computed on the basis of unconfined compression, q_u, as follows:

$$q_s = \frac{q_u}{4} \qquad\qquad q_B = 4.5\ q_u$$

The average of all unconfined compressive strength tests performed was used. Results of ultimate capacities calculated by the eight methods outlined above are summarized in Table 8.

Estimates of Settlements

A nearly elastic response of piling was indicated by the load tests for the range of likely working loads and it is thought that long-term settlements will not be substantial. Thus, settlements were analyzed by elastic methods at loads of one-third ultimate capacity. Moduli of deformation, E, determined from pressuremeter tests were used in computations. For Site B, average and upper values were selected based on all tests at the site for the test piles at 16 feet ±. Values based on

EVALUATION OF ALL TESTS BELOW 18 FEET WERE USED FOR THE DEEPER TESTS. THE LOWER PRESSUREMETER TEST WAS DISCARDED FOR SITE C WITH THE REMAINING TESTS BEING CONSIDERED IN DETERMINING AVERAGE AND HIGH VALUES.

AVERAGE VALUES OF E COMPUTED FROM THE FIRST AND SECOND LOADINGS OF CONSOLIDATION TESTS BY METHODS OUTLINED BY LAMBE AND WHITMAN (47) ALSO WERE USED. A POISSON'S RATIO, U, OF 0.4 WAS USED IN ALL COMPUTATIONS FOR THE OVERCONSOLIDATED PARTIALLY SATURATED CLAYEY SOILS INVOLVED.

A SET OF SETTLEMENT COMPUTATIONS WERE MADE ON THE BASIS OF AN "EQUIVALENT PIER" ASSUMPTION. FOR ALL PILES EXCEPT TPB-1, TPC-4, TPC-5 AND TPC-6, THE EQUIVALENT PIER WAS ASSUMED TO INVOLVE A 1:4 SPREAD OF LOAD TO A DEPTH OF 2/3D AS RECOMMENDED BY TOMLINSON (56). FOR TPB-1, THE EQUIVALENT PIER WAS EXTENDED TO THE BASE OF THE PILE BECAUSE THE PILE EXTENDED THROUGH A SOFTER LAYER WITH THE TIP BEARING ON A VERY STIFF LAYER. TPC-4, TPC-5 AND TPC-6 WERE END-BEARING PILES ONLY, SO WERE ANALYZED AS RIGID PILES AT THEIR ACTUAL DEPTH AND DIAMETER. THE EQUIVALENT PIER ASSUMPTIONS ARE ILLUSTRATED ON FIGURE 72. EQUATION 1 WAS USED WITH SETTLEMENTS AT THE CENTER OF A FLEXIBLE CIRCULAR LOADED AREA BEING COMPUTED FOR ALL CASES EXCEPT TPC-4, TPC-5 AND TPC-6. A HOMOGENEOUS CONDITION WAS ASSUMED FOR ALL PILES; ALTHOUGH IT WAS RECOGNIZED THAT THE PILES AT SITE B DID NOT MEET THIS ASSUMPTION, PARTICULARLY IN THE CASE OF TPB-1.

INFLUENCE COEFFICIENTS GIVEN BY JANBU, ET AL (57) WERE USED FOR THE FLEXIBLE CASE. THIS METHOD CAN CONSIDER AN INCOMPRESSIBLE BOUNDARY AT DEPTH.

SETTLEMENT COMPUTATIONS ALSO WERE MADE USING THE METHOD GIVEN BY POULOS AND DAVIS (58) FOR A STRAIGHT INCOMPRESSIBLE PILE IN

A homogeneous elastic media. Values of E used were the same as above. The Poulos and Davis method also can consider an incompressible boundary at depth. Their expression for settlement is as follows:

$$S = \frac{Q}{DE} I_W \quad \ldots\ldots\ldots\ldots\ldots \quad \text{(Equation 6)}$$

Where:
- Q = total load on pile
- D = depth to pile base
- E = modulus of deformation
- I_W = influence coefficient varying with Poisson's ratio, u, depth to an incompressible boundary, h, and D/d ratio of pile as shown in Figure 10 of the Poulos and Davis paper

Results of settlement computations are summarized in Table 7.

Discussion of Ultimate Bearing Capacity Calculations

Method 1 indicates that the use of the Terzaghi equation for computation of Q_B yields highly erratic results with the use of direct shear test data. The method is highly sensitive to the angle of internal friction, Ø, and this quantity cannot be determined with sufficient accuracy with direct shear tests. Q_B was, in general, greatly overestimated with Methods 1 and 2. Q_B can be estimated from direct shear test data by the empirical procedures used in Methods 3 and 4, and underestimates capacities in most cases.

Direct shear test methods greatly underestimated side shear for the Site C soils and the firmer Site B soils below 19 feet where samples undoubtedly were greatly disturbed. The average of direct shear test data provided a relatively accurate estimate of side shear in the softer soils above 19 feet at Site B. Method 3, based on average test data, appears

TO BE THE BEST METHOD OF APPLYING DIRECT SHEAR TEST DATA TO SOFTER CAF SOILS WHILE METHOD 4, BASED ON THE UPPER RANGE OF DIRECT SHEAR TESTS, APPEARS TO BE THE BEST METHOD FOR HARD, CEMENTED CAF SOILS.

CONSIDERING VARIATIONS IN SOIL PROPERTIES ACROSS THE TEST SITES AND THE THEORETICAL PROBLEMS INHERENT IN BEARING CAPACITY PREDETERMINATION, PRESSUREMETER DATA CORRELATED VERY WELL WITH ULTIMATE CAPACITIES. METHOD 5 CONSIDERABLY UNDERESTIMATED THE CAPACITY FOR TPB-2 AND CONSIDERABLY OVERESTIMATED THE CAPACITY FOR TPB-6, BUT WOULD HAVE PROVIDED A SAFE PREDETERMINATION FOR THAT PILE WITH A FACTOR OF SAFETY OF 3. METHOD 6 PROVIDES A SOMEWHAT MORE CONSERVATIVE APPLICATION OF PRESSUREMETER TESTS.

METHOD 7, BASED ON STANDARD PENETRATION TESTS, CONSERVATIVELY PREDICTS CAPACITIES.

FOR THE SOFTER SOILS AT SITE B, WHERE BLOCK SAMPLES COULD BE CUT AND UNCONFINED COMPRESSIVE STRENGTH TESTS PERFORMED, DATA WAS AVAILABLE FOR COMPUTATIONS BY METHOD 8. COMPUTED CAPACITIES WERE GENERALLY SOMEWHAT LOWER THAN INDICATED BY TESTS. THE EXPRESSION FOR SIDE SHEAR IN METHOD 8 CONTAINS A REDUCTION FACTOR OF 0.5 APPLIED SHEAR STRENGTH. THIS VALUE MAY BE TOO HIGH FOR MANY CAF SOILS, BUT WAS USED BECAUSE OF THE LIMITED AMOUNT OF UNCONFINED COMPRESSION TESTING THAT COULD BE ACCOMPLISHED IN THE SOILS INVOLVED. METHOD 8 PROBABLY WILL GIVE AT LEAST A SOMEWHAT CONSERVATIVE ESTIMATE OF CAPACITY FOR THE FEW CAF SOILS UPON WHICH UNCONFINED COMPRESSION TESTS CAN BE EFFICIENTLY PERFORMED.

IT APPEARS THAT THE VERY LOW END-BEARING INDICATED FOR TPB-7 AND TPB-8 IS DUE TO A VERY SOFT LAYER INDICATED BY PRESSUREMETER TESTS AT 15 FEET AND SOME STANDARD PENETRATION TESTS AT THAT DEPTH.

Discussion of Settlement Analysis Computations

Except possibly for wide, shallow, belled piles with little load transfer in side shear, it appears that settlements will be well below tolerable limits for most structures at a factor of safety of 3. Settlements of piles deriving a substantial proportion of their ultimate capacity in side shear were generally less than $\frac{1}{4}$ inch, as side shear is mobilized at very low strains. Considerably higher strains are required to mobilize end-bearing resistance, as is indicated by TPC-4 and TPC-6 (end-bearing only). TPC-5 (also end-bearing only) apparently was underlain by a very strongly cemented lense as it had considerably less settlement in the range of working loads and higher ultimate capacity than TPC-4 and TPC-6.

In general, computed settlements, based on E from pressuremeter data, were in the range of actual settlements for both the equivalent pier and Poulos and Davis (58) methods. Results, based on average E from pressuremeter data, generally conformed more closely to actual values for Site B where a large number of tests were available. At Site C, where great difficulty was experienced in performing tests, the maximum value of E produced better results. Because the other tests at that site may have been affected by disturbance, the maximum E is thought to better represent soil conditions.

Exceptions to the general conformance of computed and actual settlements based upon pressuremeter E are, TPC-5 (thought to be underlain by a very strongly cemented lense) and TPB-2. TPB-2 varies the most from the assumption of a homogeneous condition extending through the softer surface layer to hard cemented soils.

The equivalent pier method resulted in slightly lower calculated settlements than the Poulos and Davis (58) method, but can be applied to stratified soil conditions.

Results of settlement computations are summarized in Table 7.

Estimates based on E derived from the second cycle of consolidation tests were in the general range of actual values for Site C and the harder, cemented soils below 19 feet at Site B. However, estimates for the softer, weakly cemented soils above 19 feet at Site B were considerably lower than actual values, even when E, based on the first cycle of consolidation tests, is used. Calculation of E is extremely sensitive to Poisson's ratio, so consolidation tests appear to be of somewhat limited value for determining E.

Based on load test data and other case histories, it is believed that elastic settlement analysis procedures are applicable to CAF soils and that little longer term settlement in addition to that indicated by the load tests will occur.

The Effect of Cleaning Base

Cleaning of loose, disturbed material from the base of piles is a significant economic factor in construction, particularly with new federal safety regulations which require casing all excavations entered by workmen. Thus, about 3 inches of loose material was left in the base of TPB-8 to obtain information on the effect of cleaning. This is about the amount of loose material that could be reasonably expected with many CAF soils with careful machine cleaning. The companion cleaned pile, TPB-6, developed a somewhat higher ultimate capacity which may be due to variations in soil conditions across the test site. However, about twice as much settlement in the range of probable working loads occurred for the uncleaned pile than for the cleaned pile. This limited data indicates that a considerable reduction of capacity should be applied to "machine cleaned" piles.

The Effect of Shear Collars

Other investigators (59, 60) have reported substantially

INCREASING CAPACITIES WITH THE USE OF LARGE DIAMETER MULTIPLE BELLS. A NUMBER OF DIFFERENT CONFIGURATIONS OF SMALL MULTIPLE BELLS OR SHEAR COLLARS WERE TESTED. IT WAS THOUGHT THAT THEY MIGHT PROVE TO BE ECONOMICAL; POSSIBLY IN CONJUNCTION WITH MACHINE CLEANING. RESULTS WERE MIXED. TPB-4 DEVELOPED 18 PERCENT HIGHER ULTIMATE CAPACITY WITH 3 INCH SHEAR COLLARS ON 5 FOOT CENTERS THAN THE COMPANION TPB-5, A STRAIGHT PILE OF ABOUT THE SAME DIAMETER AS THE SHEAR COLLARS, WITH MOST OF THE INCREASE DUE TO HIGHER SIDE SHEAR. TPC-2 WITH 3 INCH SHEAR COLLARS ON 5 FOOT CENTERS DEVELOPED THE SAME CAPACITY AS TPC-3, A COMPANION STRAIGHT PILE OF THE SAME DIAMETER AS THE SHEAR COLLARS. TPC-7 AND TPC-8, TPC-9 AND TPC-10 WERE COMPANION PILES OF THE SAME DIAMETER WITH AND WITHOUT SHEAR COLLARS. THE PILES WITH SHEAR COLLARS DEVELOPED ONLY 58 AND 81 PERCENT OF THE CAPACITY OF THE COMPANION STRAIGHT PILES. TPB-7, TPB-9 AND TPB-10, ALL 16 FEET OR SLIGHTLY GREATER IN DEPTH, HAD 3 INCH SHEAR COLLARS ON 5 FOOT CENTERS, 6 INCH SHEAR COLLARS ON 5 FOOT CENTERS AND 3 INCH SHEAR COLLARS ON 2.5 FOOT CENTERS, RESPECTIVELY. THEY DEVELOPED 70 TO 80 PERCENT OF THE ULTIMATE CAPACITY OF TPB-6, A STRAIGHT COMPANION PILE OF THE SAME DIAMETER OF THE SHEAR COLLARS, BUT SHOWED SOMEWHAT HIGHER SIDE SHEAR BASED UPON THE SHAFT DIAMETER.

THUS, RESULTS OF SHEAR COLLAR EXPERIMENTS WERE INCONCLUSIVE WITH NO CLEAR TREND TOWARD IMPROVED CAPACITIES. IT DOES NOT APPEAR THAT THE USE OF SHEAR COLLARS IS ECONOMICALLY JUSTIFIED FOR CAF SOILS CONSIDERING THE MACHINE TIME INVOLVED IN BELLING THE COLLARS AND THE NECESSITY OF MOBILIZING ADDITIONAL EQUIPMENT TO THE CONSTRUCTION SITE.

CHAPTER VIII - RECOMMENDATIONS

IMPLEMENTATION

RECOMMENDATIONS ARE PRESENTED IN THE FOLLOWING SECTIONS OF THIS REPORT WHICH THE AUTHORS BELIEVE CAN BE APPLIED DIRECTLY TO THE DESIGN OF DRILLED PILING FOR BOTH CAF AND SGC SOILS ON A ROUTINE BASIS. BECAUSE OF THE TYPES OF SOILS INVOLVED, THE RECOMMENDATIONS ARE LARGELY EMPIRICAL. FOR EXAMPLE, FOR CAF SOILS, FACTORS SUCH AS THE EFFECT OF LOAD TRANSFER IN END-BEARING ON SIDE SHEAR OF THE LOWER PORTION OF THE SHAFT OR THE REDUCTION OF SHEAR STRENGTH DUE TO MOISTURE FROM CONCRETE WERE NOT DIRECTLY CONSIDERED. HOWEVER, SUFFICIENT DATA WAS DEVELOPED SO IT IS BELIEVED THE DESIGN PROCEDURES CAN BE APPLIED WITH CONFIDENCE, PROVIDED ENGINEERING JUDGMENT IS USED IN APPLYING TEST DATA FROM A GIVEN SITE TO THE METHODS PROPOSED AND APPROPRIATE FACTORS OF SAFETY ARE USED.

AS A PART OF IMPLEMENTATION, IT IS RECOMMENDED THAT SETTLEMENT OBSERVATIONS ON FULL-SCALE, HEAVILY LOADED FOUNDATIONS BE OBTAINED AT FIRST OPPORTUNITY. THIS IS PARTICULARLY IMPORTANT FOR THE SGC SOILS AS IT WOULD ENABLE REFINEMENT OF DESIGN CURVES FOR LARGE WIDTHS.

RECOMMENDATIONS - SGC SOILS

IN MOST, IF NOT ALL, CASES, THE DESIGN OF DRILLED PILING ON SGC SOILS, BEARING PRESSURES WILL BE CONTROLLED BY SETTLEMENT UNDER WORKING LOADS. THE FACTOR OF SAFETY AGAINST SHEAR FAILURE SHOULD BE CHECKED BY THE CONSERVATIVE TERZAGHI (2) METHOD FOR SHALLOW FOOTINGS.

FIGURE 78 PRESENTS RECOMMENDED DESIGN CURVES FOR USE IN EVALUATION OF SETTLEMENTS FOR VARIOUS WIDTHS OF FOUNDATIONS and bearing pressures. Settlements shown on the curves are 50 percent higher than the "average" curves shown on Figure 77. This is intended to account for variations in the compressibility of the SGC soils due to variations in relative density and small differences in gradation and slightly longer term settlements than indicated by the load-settlement curves. The configuration of the curve is based upon E varying directly with P_z which seems to be conservative.

The exact characteristics and uniformity of SGC soils beneath a given site should be carefully established by a comprehensive soil investigation prior to design. Soils should be investigated to the depth below the base of foundations where stresses due to the foundations are about 10 percent of the effective overburden pressure.

The Becker Hammer Drill is recommended as the primary method of subsurface exploration. The same diameter drive pipe and the same type of drill bits used in the previous investigations mentioned in this report should be used. Care should be exercised that the full hammer energy of 8000 ft./lbs. is delivered and that sufficient compressed air is provided to rapidly clean cuttings. Blow count should be kept in 6 inch increments to fully define the degree of stratification and the presence of boulders. A 2-man field engineering crew should be used with one man directing operations and keeping blow count with a tally pace and the other continuously observing cuttings recovery and taking samples. Reference samples of cuttings should be taken at 5 foot intervals or each soil change, whichever is less. Where sand layers are encountered, standard penetration tests should be performed. Shelby tube or open-end drive samples should be taken of any clay or clayey sand layers encountered.

THE FOLLOWING LIMITATIONS AND ASSUMPTIONS SHOULD BE CONSIDERED IN APPLYING FIGURE 78 TO DESIGN.

1. THE CURVES APPLY TO THE COARSE SGC SOILS OF THE SALT RIVER VALLEY AND OTHER SOILS WITH VERY SIMILAR GRADATION, PARTICLE SHAPE AND HARDNESS OF MINERAL GRAINS TO THOSE INDICATED BY FIGURE 19. EXTENSIVE DATA SUMMARIZED BY LEPS (39) ILLUSTRATES THAT SOFTER MATERIALS OF THE SAME GRADATIONS ARE OFTEN MUCH MORE COMPRESSIBLE THAN HARD MATERIALS. LABORATORY TESTS CONDUCTED BY KJAERNSLI AND SANDE (41) SHOW THAT GRADATION, HARDNESS AND PARTICLE SHAPE CAN ALL HAVE A SUBSTANTIAL INFLUENCE ON COMPRESSIBILITY, ALL OTHER FACTORS BEING EQUAL. POORLY GRADED, ANGULAR, SOFT MATERIALS ARE, IN GENERAL, CONSIDERABLY MORE COMPRESSIBLE THAN WELL GRADED, HARD, ROUNDED MATERIALS. COMPARATIVELY SMALL FRACTIONS OF CLAY AT HIGHER MOISTURE CONTENTS WOULD, NO DOUBT, GREATLY INCREASE COMPRESSIBILITY.

2. THE CURVES APPLY TO SGC FREE OF SAND OR CLAY LAYERS WITH A WEIGHTED BECKER BLOW COUNT, N_B, OF ABOUT 28 OR MORE. WHERE SAND OR CLAY LAYERS ARE PRESENT, THEIR EFFECT ON BEARING CAPACITY AND SETTLEMENT SHOULD BE EVALUATED BY NORMAL PROCEDURES BASED ON STANDARD PENETRATION, SHEAR AND CONSOLIDATION TESTS.

3. FIGURE 78 APPLIES TO CLEAN, UNCEMENTED SGC SOILS AND IS CONSERVATIVE IN PREDICTING SETTLEMENTS WHERE THE SGC SOILS POSSESS AN APPRECIABLE DEGREE OF CEMENTATION.

4. FIGURE 78 SHOULD BE DIRECTLY APPLIED ONLY TO FOUNDATIONS 10 FEET OR MORE IN DEPTH. ALTHOUGH THE DEPTH OF TEST PILES WAS 15.5 TO 18.5 FEET, IT IS BELIEVED THAT RELIEF OF OVERBURDEN PRESSURES BY THE ACTION OF THE ANCHOR PILES CREATED AN EFFECTIVE DEPTH SOMEWHERE NEAR 10 FEET. DATA REPORTED BY D'APPOLONIA, ET AL (61) SHOWED THAT FOR FOUNDATIONS ON DENSE SANDS, SETTLEMENTS FOR A DEPTH TO WIDTH RATIO OF 0.5 WERE

about 15 to 20 percent greater than for a ratio of 1.0. This factor should be carefully evaluated in applying Figure 78 to depths less than 10 feet.

5. Figure 78 applies to square or circular foundations. Corrections should be made on the basis of relative stress influence when considering rectangular foundations.

6. The present overburden pressure at the site is believed to be the maximum which has existed. Thus, where the stress history involves the removal of overburden due to erosion or man-made excavations, settlements can be expected to be somewhat less than those observed in this study. Preloading has been shown to reduce the compressibility of granular soils.

Pressures based on dead plus relatively long-term live loads should be used in settlement analysis. Pressures can be increased by 50 percent or more for very short-term live loads such as seismic forces. The load deformation curves illustrate the degree that settlements are time dependent for very short-term load applications.

The time-rate of settlement and sequence of construction and live load application should be considered in establishing allowable settlements. For instance, settlements of elevated freeway structures due to the weight of the columns and pedestals will occur rapidly and not affect differential settlements of the deck. For the SGC deposit, only dead loads of the deck plus realistic live loads will apply to evaluation of differential settlements of the deck. Both this study and previous studies of granular soils indicate that differential settlements can be expected to be 75 percent or more of maximum total settlements.

Recommendations - CAF Soils

It appears that the design of nearly all drilled piles in CAF soils will be controlled by the factor of safety against shear failure. A factor of safety of 3.00 is recommended for design. Many designers have used factors of safety of about 2.50 or lower in relatively homogeneous deposits such as London clay. However, a factor of safety of 3.00 is recommended because of the extensive and erratic horizontal and vertical variability in engineering properties inherent in most CAF soil deposits. Few actual projects will have as intensive soils investigations as performed at the test sites in this study. With this factor of safety, and the methods proposed, working stresses should be in the relatively elastic portion of load-deformation curves and settlements in most cases will be less than $\frac{1}{4}$ inch for total loads covered in this study. Settlements can be expected to increase generally with increasing total load for a given soil condition. Settlements should be evaluated for each project by either the equivalent pier or Poulos and Davis method. Preferably, pressuremeter test data should be developed for settlement analysis. Where E is determined by consolidation tests, a conservative approach should be used in interpretation of data. Modulus of deformation, E, calculated from the first cycle of consolidation tests, based on a Poisson's ratio of 0.45, should be considered the likely lower limit.

A program of auger borings with standard penetration tests at 5 foot intervals should be a minimum subsurface exploration. Generally, all borings should extend to 10 feet below the anticipated maximum tip elevation of drilled piling. In general, some borings should extend to a depth where new stresses due to the foundations are about 10 percent of the effective overburden pressure. Blow count on standard penetration tests for CAF soils should be kept in 2 or 3 inch increments in order to enable the evaluation of thin, strongly cemented laminations and the presence of scattered gravel and calcareous concretions. Minimum laboratory tests should consist of selected grain-size analysis, Atterberg Limits, moisture content and dry density tests on representative samples to enable thorough classification of materials. The pressuremeter appears to produce the best information on the compressibility and shear strength of more strongly cemented CAF soils of available methods. Accordingly, it is recommended that extensive pressuremeter tests be performed for major projects. Previously described techniques are available for efficiently performing pressuremeter tests in many CAF soils and it is believed that if tests are performed on a routine basis, methods can be developed to efficiently perform tests on more difficult soils such as those at Site C.

Where pressuremeter tests are not performed, direct shear and consolidation tests should be run on open-end drive samples for significant projects.

On major projects, large diameter test holes should be drilled in order to more accurately determine the soil structure, intensity and nature of fissuring and laminations and the surface texture produced by drilling on the walls of holes and potential construction problems. Where possible, block samples should be cut from these holes and unconfined compression tests performed.

Methods 5 and 6, based on pressuremeter tests, are recommended as the prime method of calculating ultimate capacities. Method 5 is recommended where a large number of tests are available, while the more conservative Method 6 is recommended for cases where limited tests are available, such as Site C. Where pressuremeter tests are not available or, as a comparative method on large projects, Methods 3 and 4, based on direct shear tests are recommended. Method 4 should be used for softer (standard penetration resistance, N, less than about 20), weakly cemented soils while Method 3 should be used for firmer, more strongly cemented soils. Where unconfined compression tests are available, ultimate bearing capacity can be evaluated by Method 8.

On small jobs that do not justify a detailed testing program, ultimate capacity can be evaluated by Method 7 based on average standard penetration resistance. Higher or refusal blow counts due to thin, cemented laminations or scattered gravel should be discarded in computing average blow counts. Caution should be exercised in using this method and it is recommended that it be limited to total pile loads of 100 kips. The condition at a given site should be carefully compared to the load tests and other data developed in this investigation and performance records of existing structures in applying this highly empirical approach.

Practical Design Considerations

From data developed in this study, it appears that straight piles will be more economical than belled piles in many cases for the CAF soils. In the relatively homogeneous case of Site C, belling a 2.5 foot diameter straight pile to 5.0 feet at 15 feet in depth would increase the capacity about 52 percent (effective side area would be lost throughout the height of the bell). The same capacity could be achieved by extending the 2.5 foot diameter straight pile to about 25 feet. Because belling is extremely difficult and time consuming in strongly cemented soils of the type involved at Site C, the latter approach probably would be less costly.

However, for the case of Site B where belling in the upper soils is relatively easy and a hard layer is present at

19 feet, belling at that depth would almost certainly be more economical. At Site B, belling a 2.5 foot diameter pile to 5.0 feet at 19 feet would increase the capacity about 234 percent while in order to achieve the same capacity, a straight 2.5 foot diameter pile would require deepening to about 39 feet.

Additional equipment and often, different types of drill rigs must be mobilized to the site for belling, thereby increasing costs. Thus, the hardness of the materials that would be involved in belling should be carefully considered in selection of pile configuration and depth. Occasionally, very strongly cemented layers in CAF soils are so hard to penetrate, it is more economical to bell above them even though belling is extremely difficult.

If possible, drilled piles should not be extended through cleaner granular layers which are subject to caving; belling, particularly, should be avoided in these layers. Concrete overruns due to caving can be a large cost factor. Often, the amount of caving that occurs is very sensitive to the equipment and techniques used by the contractor in excavation. Thus, specifications should be written in such a way that payment is made on a lineal foot basis and the contractor is responsible for the costs of overruns of concrete in excess of neat volumes indicated by the plans.

Where the extent of construction problems such as difficult belling in strongly cemented layers, caving or scattered cobbles or boulders which obstruct drilling are uncertain, large diameter test borings should be excavated with foundation drilling equipment. The configuration of any caving which occurs should be sketched on the boring logs to provide information to the contractor. Time required to drill through each interval as well as a detailed description of the rig and type of drill bit used should be noted on the logs.

New Federal Safety Regulations require casing of all drilled pile excavations entered by workmen, irrespective of the stability of the soils involved. Mobilization of casing to the site and cranes or additional drill rigs to handle it involves large costs. Thus, designing straight piles on a "machine cleaned" basis has economic advantages. Where "machine cleaning" is involved, no more than 2 inches of loose material should be permitted in the bottom of excavations. Calculated Q_B should, generally, be reduced by 50 percent in design prior to applying the factor of safety to obtain working capacities. Reduction factors for end-bearing should be applied very cautiously; particularly for high capacity piling (over 200 kips $\pm$). The percentage of total ultimate capacity Q_T transferred in end-bearing varies from 6 to 46 percent for the various straight piles tested. Thus, reducing Q_B by 50 percent for "machine cleaned" piles is unlikely to reduce capacities over about 20 percent for all but very shallow piles.

The quantity of concrete per unit capacity of the pile should be considered in selecting pile sizes. Side area effective in side shear increases directly in proportion to the diameter, while volume of concrete increases in proportion to the square of the diameter. Thus, the use of deeper straight piles of smaller diameter is sometimes the more economical design.

Two foot diameter piles can be entered and manually cleaned with a hinged shovel. Normally, however, 2.5 feet is about the minimum practical diameter where manual cleaning is required.

As can be seen from load-deformation curves for TPC-4 through TPC-10, considerably higher strains are required to mobilize end-bearing than side shear. Thus, straight piles generally experience much less settlement under working loads at the

SAME FACTOR OF SAFETY THAN SHALLOW, BELLED PILES WITH LITTLE LOAD TRANSFER IN SIDE SHEAR. SETTLEMENT OF SHALLOW, BELLED PILES CAN BE GREATLY AFFECTED BY THIN, SOFT LAYERS OR LENSES IMMEDIATELY BENEATH THE BASE. STRESSES FOR STRAIGHT PILES ARE TRANSMITTED INTO THE SOIL IN SUCH A WAY THAT SETTLEMENTS ARE MUCH LESS AFFECTED BY THIN, SOFT ZONES. THUS, THE USE OF STRAIGHT PILES GENERALLY HAS THE ADVANTAGE OF DECREASING SETTLEMENTS AT WORKING LOADS FOR CAF SOILS.

ALTHOUGH SHALLOW, BELLED PILES BEARING NEAR THE SURFACE OF THE SGC ARE NORMALLY THE MOST ECONOMICAL APPROACH, STRAIGHT PILES PENETRATING THE SGC MERIT CONSIDERATION IN SOME CASES. IN PARTICULAR, WHERE A CAVING, LOOSE, SAND LAYER IS PRESENT ABOVE THE SGC, OR SOFT LAYERS ARE PRESENT WITHIN THE DEPOSIT, OR WHERE VERY CRITICAL SETTLEMENT CRITERIA ARE INVOLVED, STRAIGHT PILES PENETRATING THE SGC MAY BE APPLICABLE. THIS TYPE OF PILING HAS BEEN SUCCESSFULLY CONSTRUCTED IN THE SGC WITH CASING AND HEAVY DRILL RIGS. SETTLEMENTS OF SHALLOW, BELLED PILES BEARING ON THE SGC ARE LARGELY A FUNCTION OF TOTAL LOAD, SO CAN BE REDUCED TO A LIMITED DEGREE BY THE USE OF LOWER BEARING PRESSURES. BECAUSE SIDE SHEAR IS MOBILIZED AT LOWER STRAINS, THE MODULUS OF DEFORMATION OF THE SGC INCREASES WITH DEPTH AND SETTLEMENTS OF STRAIGHT PILES IS LESS AFFECTED BY THIN, SOFTER ZONES; SETTLEMENTS FOR A GIVEN TOTAL LOAD CAN BE REDUCED BY THE USE OF DEEP, STRAIGHT PILES. LOAD TRANSFER IN THE OVERLYING CAF SOILS WOULD CONTRIBUTE SIGNIFICANTLY TO THE CAPACITY AT SOME SITES.

SIDE SHEAR FOR STRAIGHT PILES PENETRATING THE SGC CAN BE COMPUTED BY EQUATION 3 WITH SHEAR STRENGTHS ESTIMATED FROM CLASSIFICATION AND ARCHING EFFECTS BEING CAREFULLY CONSIDERED. END-BEARING CAN BE EVALUATED BY EQUATION 4 AND ANALYSIS OF LOAD TESTS AT SITE A.

Concrete in drilled piling should normally be placed at 4 to 6 inches slump and should be thoroughly vibrated in order to prevent the creation of voids at the pile interface. This is important for piles deriving support from side shear to develop as high as possible horizontal stresses at the pile perimeter. The water-cement ratio should be no more than 0.6 to minimize weakening of the soil by moisture migration from the concrete (this is particularly important for CAF soils).

Concrete should be placed as soon as possible after excavation to minimize weakening of the soil due to stress relief, weathering, etc. Piles at Sites B and C were cast about 24 hours after excavation, a time period that was believed could reasonably be achieved as a maximum in routine construction.

Inspection

Construction of drilled piling should be continuously inspected by a specially trained technician or engineer. Where full end-bearing is used in design, complete cleaning of all loose, disturbed material from the base of piles should be verified by physically entering and inspecting each excavation. In granular soils where the depth of disturbance is difficult to visually determine, maximum depth of advance of drill bits should be noted. Where "machine cleaning" is involved, adequate cleaning should be verified by measurements of the maximum advance of drill bits and the surface elevation of the bottom of the excavation plus visual observation.

Placement and vibration of concrete should be continuously observed by the inspector to insure that no caving or sloughing occurs that would have a detrimental effect on the piling.

The inspector should verify proper dimensions and depths of each pile. Where piles are designed to bear on a specific stratum, proper contact and penetration of the bearing stratum should be verified for each pile. Detailed inspection records should be kept for each pile recording depths, dimensions, verification of cleaning, bearing strata, proper concrete placement, and dates of acceptance and construction.

Summary of Recommendations

Detailed soil investigations are recommended to provide information for foundation design for both SGC and CAF soils. The use of the Becker Hammer Drill is recommended for evaluation of the uniformity of SGC soils. This method provides dynamic penetration resistance for a rough approximation of relative density and allows classification of the materials. Considering the quality of information obtained and the drilling time required, it is believed to be much more economical than alternate methods. Pressuremeter testing is considered the preferred method of investigating the shear strength and compressibility of CAF soils.

Figure 78 is recommended for estimating settlements of foundations 10 feet or more in depth bearing on SGC with weighted Becker blow count, N_B, of 28 or more. Adjustments of settlements shown on Figure 78 are necessary for shallower foundations or cases where clay or sand layers are present in the SGC. Adequate factor of safety against shear failure for foundations on the SGC should be checked by the Terzaghi method using estimated shear strengths.

Elastic settlement analysis methods using moduli of deformation based on pressuremeter or one-dimension consolidation

TESTS ARE RECOMMENDED FOR ENGINEERING ANALYSIS OF DRILLED PILES IN CAF SOILS. ADEQUATE FACTOR OF SAFETY AGAINST SHEAR FAILURE USUALLY WILL CONTROL DESIGN FOR CAF SOILS. EMPIRICAL METHODS OF DESIGN, BASED ON PRESSUREMETER TEST DATA, ARE RECOMMENDED FOR ESTIMATING ULTIMATE BEARING CAPACITY FOR MAJOR PROJECTS. METHODS BASED ON DIRECT SHEAR TESTS ARE RECOMMENDED AS A PARALLEL PROCEDURE AND FOR USE ON SMALLER PROJECTS. METHODS BASED ON STANDARD PENETRATION TESTS AND UNCONFINED COMPRESSION TESTS ALSO ARE PRESENTED FOR USE ON SMALL PROJECTS AND THE LIMITED NUMBER OF CASES WHERE SAMPLES CAN BE OBTAINED FOR UNCONFINED COMPRESSION TESTING.

CONSTRUCTION FACTORS SUCH AS DIFFICULTY IN DRILLING-AND-BELLING, CAVING AND POTENTIAL CONCRETE OVERRUN, AND THE NECESSITY OF CASING SHOULD BE CONSIDERED IN DESIGN. CONTINUOUS INSPECTION OF CONSTRUCTION BY QUALIFIED PERSONNEL IS RECOMMENDED AS BEING VITAL FOR THE SUCCESSFUL CONSTRUCTION OF DRILLED PILING.

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TABLE 1

DATA ON TEST PILES & REACTION PILES - TEST SITE A

Test Pile No.	Depth Below Grade	Average Tip Diameter	Average Height of Bell	Pile Length
TPA-1	17'10"	29"		18'4"
TPA-2	15'6"	56.5"	3'6"	16'0"
TPA-3	16'2"	55"	3'6"	16'6"
TPA-4	17'1"	74.5"	6'1"	17'7"
TPA-5	18'2"	71.5"	4'1"	18'8"
TPA-6	18'3"	95.5"	7'1"	18'7"
TPA-7	18'6"	113"	6'6"	18'10"

Anchor Pile No.	Depth Below Grade	Average Tip Diameter	Average Height of Bell
1A	18'6"	80"	6'3"
2A	16'9"	89"	6'3"
3A	16'8"	89"	6'2"
4A	15'9"	86"	9'0"
1B	18'0"	85"	6'0"
2B	18'0"	89"	5'7"
3B	16'0"	88"	7'6"
4B	16'3"	88"	6'9"
1C	16'0"	91"	5'10"
2C	16'2"	91"	5'3"
3C	15'6"	85"	7'0"
4C	16'2"	87"	7'3"
1D	16'2"	88"	4'3"
2D	15'8"	89"	4'4"
3D	15'10"	90"	7'1"
4D	15'10"	92"	7'0"
3E	16'1"	84"	6'11"
4E	16'5"	86"	7'4"

TABLE 2

DATA ON TEST PILES & REACTION PILES - TEST SITE B

TEST PILE NO.	DEPTH BELOW GRADE	AVERAGE TIP DIAMETER	PILE LENGTH
TPB-1	21' 5½"	32.06"	22' 1"
TPB-2	16' 3½"	40.20"	16' 11"
TPB-3	35' 10"	29.25"	36' 6"
TPB-4	31' 2½"	24.96"	31' 8½"
TPB-5	31' 9"	29.67"	32' 3"
TPB-6	16' 2"	36.25"	16' 10"
TPB-7	15' 11"	31.10"	16' 5"
TPB-8	15' 7"	36.08"	16' 3"
TPB-9	16' 2"	25.00"	16' 8"
TPB-10	16' 1"	24.83"	16' 7"

ANCHOR PILE NO.	DEPTH BELOW GRADE	AVERAGE TIP DIAMETER	AVERAGE HEIGHT OF BELL
1A	20' 8"	85.17"	5' 1"
2A	21' 7"	84.67"	5' 0"
3A	20' 4"	84.20"	4' 10"
4A	20' 2"	84.33"	4' 7"
1B	20' 5"	84.25"	4' 11"
2B	21' 0"	87.83"	4' 9"
3B	20' 6"	91.50"	5' 3"
4B	20' 4"	84.00"	5' 1"
1C	20' 6"	84.34"	3' 8"
2C	20' 8"	86.50"	4' 9"
3C	21' 3"	85.50"	4' 8"
4C	20' 4"	85.17"	3' 10"
1D	20' 6"	85.00"	4' 11"
2D	20' 5"	86.34"	5' 0"
3D	20' 8"	86.00"	5' 0"
4D	20' 4"	85.17"	4' 7"

TABLE 3

Data on Test Piles & Reaction Piles - TEST SITE C

Test Pile No.	Depth Below Grade	Average Tip Diameter	Average Height of Bell	Pile Length
TPC-1	20'4"	30.08"		20'10"
TPC-2	15'7"	30.25"		16'1"
TPC-3	15'5"	36.00"		16'1"
TPC-4	16'2"	49.83"	3'2"	16'8"
TPC-5	21'1"	42.42"	3'2"	21'7"
TPC-6	17'0"	59.00"	3'9"	17'6"
TPC-7	22'5"	28.67"		22'11"
TPC-8	16'8"	29.56"		17'2"
TPC-9	16'10"	37.00"		17'6"
TPC-10	22'0"	23.46"		22'6"

Anchor Pile No.	Depth Below Grade	Average Tip Diameter	Average Height of Bell
1A	18'6"	6'8"	6'3"
2A	16'9"	7'5"	6'3"
3A	16'8"	7'5"	6'2"
4A	15'9"	7'2"	9'0"
1B	18'0"	7'1"	6'0"
2B	18'0"	7'5"	5'7"
3B	16'0"	7'4"	7'6"
4B	16'3"	7'4"	6'9"
1C	16'0"	7'7"	5'10"
2C	16'2"	7'7"	5'3"
3C	15'6"	7'1"	7'0"
4C	16'2"	7'3"	7'3"
1D	16'2"	7'4"	4'3"
2D	15'8"	7'5"	4'4"
3D	15'10"	7'6"	7'1"
4D	15'10"	7'8"	7'0"

TABLE 4

TEST SITE A

CALCULATED ULTIMATE BEARING PRESSURE

TPA Test Pile	Pile D*	Pile d*	Max. Test Q* (kips)	Max. Test q* (ksf)	Computed Ultimate Bearing Pressure (ksf) *$\phi=38^o$	$\phi=40^o$	$\phi=42^o$	$\phi=44^o$
1	2'5"	17'10"	1956	425	230	245	269	294
2	4'8½"	15'6"	1986	114	212	229	259	296
3	4'7"	16'2"	1986	121	219	237	267	303
4	6'2½"	17'1"	1986	66	238	259	296	341
5	5'11½"	18'2"	1986	71	250	271	308	353
6	7'11½"	18'3"	1986	40	261	285	329	385
7	9'5"	18'6"	1986	28	271	297	347	411

*LEGEND:

D = Average pile bell diameter

d = Depth of pile below grade

Q = Maximum applied load reached in test (kips)

q = Maximum bearing pressure reached in test (kips/sq. ft.)

ϕ = Angle of internal friction

TABLE 5 Job No. E71-272

TEST SITE A

SUMMARY OF BECKER HAMMER DRILL RESULTS

DEPTH (Feet)	Becker Hammer Drill Blow Count (6" Increments)						
	TPA-1	TPA-2	TPA-3	TPA-4	TPA-5	TPA-6	TPA-7
15.0-15.5	18	19	11	12	16	22	16
15.5-16.0	21	17	15	13	20	25	13
16.0-16.5	26	18	22	15	18	28	12
16.5-17.0	26	16	23	17	16	29	12
17.0-17.5	28	17	28	12	16	57	12
17.5-18.0	25	23	23	12	15	33	10
18.0-18.5	26	24	23	16	16	23	12
18.5-19.0	38	21	24	15	23	30	14
19.0-19.5	35	18	24	13	31	27	11
19.5-20.0	34	14	19	13	41	28	12
20.0-20.5	25	16	21	10	38	24	14
20.5-21.0	21	19	22	10	43	28	14
21.0-21.5	20	19	23	12	42	24	13
21.5-22.0	23	17	25	13	40	29	11
22.0-22.5	31	17	33	18	31	27	12
22.5-23.0	20	20	33	14	26	21	13
23.0-23.5	13	27	37	12	22	21	19
23.5-24.0	14	23	38	12	22	16	18
24.0-24.5	19	18	28	10	18	32	11
24.5-25.0	17	21	25	11	17	22	12
25.0-25.5	20	19	20	11	17	19	13
25.5-26.0	26	18	16	11	13	20	14
26.0-26.5	20	21	11	19	10	20	14
26.5-27.0	17	14	12	22	11	22	14
27.0-27.5				19	11	25	16
27.5-28.0				18	15	18	17
28.0-28.5				17	18	16	19
28.5-29.0				21	24	17	17
29.0-29.5				16	29	19	19
29.5-30.0				16	31	23	21
30.0-30.5					35	24	18
30.5-31.0					29	21	19
31.0-31.5						20	17
31.5-32.0						17	16
32.0-32.5						22	16
32.5-33.0						27	15
33.0-33.5						22	19
33.5-34.0						18	19
34.0-34.5						20	16
34.5-35.0						19	16
35.0-35.5							28
35.5-36.0							38
36.0-36.5							44
36.5-37.0							34
37.0-37.5							39
37.5-38.0							40
38.0-38.5							42
38.5-39.0							58
39.0-39.5							35
39.5-40.0							28
Avg. Blows	21	19	23	14	23	24	20

TABLE 6

SUMMARY PRESSUREMETER TEST RESULTS

TEST SITE A

Boring Location	Depth to Center (feet)	P_o Kg/cm^2	P_f Kg/cm^2	P_l ksf	E ksf	S_o ksf
3	3	1.89	6.21	16.79	2054	1.72
3	6	1.78	8.53	36.41	1182	5.67
4		3.26	6.58	13.68	306	1.17
1	9	3.35	14.20	57.92	1059	10.01
2		1.71	11.65	43.91	1265	7.21
4		2.83	7.29	23.86	334	3.65
3	12	1.64	6.20	24.51	1020	3.46

TEST SITE B

Boring Location	Depth to Center (feet)	P_o Kg/cm^2	P_f Kg/cm^2	P_l ksf	E ksf	S_o ksf
1	3	3.84	5.46	12.37	327	0.68
4		1.19	6.76	18.19	520	2.79
1	6	3.43	8.20	21.46	351	2.72
2		3.26	7.28	20.95	1269	2.09
4		2.20	6.47	16.57	284	2.29
1	9	3.70	7.17	18.29	277	1.99
2		3.02	5.67	10.65	513	0.63
1	12	2.82	5.97	13.13	189	1.37
4		0.42	4.34	12.78	260	2.29
2	15	2.34	4.12	8.50	250	0.57
3		2.23	4.03	9.32	319	2.31
1	18	3.20	9.03	25.09	1441	2.76
2		2.65	8.76	33.05	923	4.85
3		1.77	4.22	20.36	1358	2.48
4		0.77	4.83	18.84	508	3.11
1	21	4.44	14.57	56.20	1397	8.07
3		1.72	13.90	62.53	1616	10.22
4		2.86	15.97	72.99	2124	11.02
1	24	5.53	22.26	81.31	3209	11.53
2		2.86	7.46	35.70	1170	4.96
3		5.64	21.42	91.42	3369	13.39
4	27	2.20	5.48	32.60	1168	4.73
1	30	1.58	16.41	54.03	601	10.40
2		3.30	7.44	31.99	1372	4.03
4		1.01	4.39	14.77	393	2.27
1	33	2.71	13.80	56.93	881	10.57
2		3.27	5.97	28.67	1075	3.58
4		2.33	8.80	31.33	516	5.30
1	36	2.55	12.80	60.62	818	11.86

TABLE 6

SUMMARY PRESSUREMETER TEST RESULTS

TEST SITE C

Boring Location	Depth to Center (feet)	P_o Kg/cm^2	P_f Kg/cm^2	P_l ksf	E ksf	S_o ksf
4	3	3.24	8.77	35.84	653	5.63
4	6	2.10	4.66	19.09	264	3.01
4	12	6.95	15.93	53.45	535	8.58
4	15	6.71	14.46	59.23	2382	7.31

LEGEND:

P_o = "Initial" Pressure - the beginning of the elastic stress range (seating pressure)

P_f = "Creep" Pressure - the end of the elastic stress range

P_l = "Limit" Pressure - the failure pressure

E = Compression Modulus - derived from the slope of the compression curve between P_o and P_f

S_o = Shear Strength

TABLE 7 - SUMMARY OF SETTLEMENT ANALYSIS - SITES B & C

					CALCULATED SETTLEMENTS																
					Poulos & Davis Method (Ref. 58)								Equivalent Pier Method								
					E From Pressuremeter Tests				E From Consolidation Tests				E From Pressuremeter Tests				E From Consolidation Tests				
Test Pile	D	D*	Q	Act. S	Avg. E	Calc. S	Max. E	Calc. S	Avg. E 2nd Cyc.	Calc. S	Avg. E 1st Cyc.	Calc. S	Avg. E	Calc. S	Max. E	Calc. S	Avg. E 2nd Cyc.	Calc. S	Avg. E 1st Cyc.	Calc. S	
TPB-1	21.45	2.67	418	0.35	9.8	0.22	15.1	0.14	34.2	0.06	15.6	0.14	9.8	0.12	15.1	0.07	34.2	0.03	15.6	0.07	
TPB-2	16.27	3.35*	327	0.25	6.0	0.31	9.4	0.20	21.6	0.09	13.6	0.14	6.0	0.26	9.4	0.17	21.6	0.07	13.6	0.12	
TPB-3	35.89	2.44	397	0.15	9.8	0.14	15.1	0.09	34.2	0.04	15.6	0.09	9.8	0.14	15.1	0.09	34.2	0.04	15.6	0.09	
TPB-4	31.20	2.50*	485	0.10	9.8	0.20	15.1	0.13	34.2	0.06	15.6	0.12	9.8	0.14	15.1	0.09	34.2	0.04	15.6	0.09	
TPB-5	31.75	2.47	410	0.12	9.8	0.15	15.1	0.10	34.2	0.04	15.6	0.10	9.8	0.11	15.1	0.07	34.2	0.03	15.6	0.07	
TPB-6	16.16	3.02	303	0.34	6.0	0.31	9.4	0.20	21.6	0.09	13.6	0.14	6.0	0.23	9.4	0.15	21.6	0.06	13.6	0.10	
TPB-7	15.92	3.05*	153	0.07	6.0	0.16	9.4	0.10	21.6	0.04	13.6	0.07	6.0	0.11	9.4	0.07	21.6	0.03	13.6	0.05	
TPB-8	15.56	3.00	143	0.13	6.0	0.15	9.4	0.10	21.6	0.04	13.6	0.07	6.0	0.11	9.4	0.07	21.6	0.03	13.6	0.05	
TPB-9	16.16	2.88*	153	0.18	6.0	0.16	9.4	0.10	21.6	0.04	13.6	0.07	6.0	0.12	9.4	0.08	21.6	0.03	13.6	0.05	
TPB-10	16.08	3.02*	163	0.09	6.0	0.17	9.4	0.11	21.6	0.05	13.6	0.07	6.0	0.13	9.4	0.09	21.6	0.04	13.6	0.06	
TPC-1	20.33	2.50	660	0.14	8.3	0.42	16.3	0.22	18.2	0.19	--	--	8.3	0.31	16.3	0.16	18.2	0.14	--	--	
TPC-2	15.56	3.18*	500	0.20	8.3	0.32	16.3	0.16	18.2	0.14	--	--	8.3	0.12	16.3	0.06	18.2	0.06	--	--	
TPC-3	15.42	3.00	500	0.17	8.3	0.36	16.3	0.19	18.2	0.16	--	--	8.3	0.28	16.3	0.14	18.2	0.13	--	--	
TPC-4**	16.16	4.15	267	0.25	8.3	0.61	16.3	0.31	18.2	0.28	--	--	8.3	0.09	16.3	0.05	18.2	0.04	--	--	
TPC-5**	21.08	3.54	267	0.08	8.3	0.71	16.3	0.36	18.2	0.32	--	--	8.3	0.07	16.3	0.04	18.2	0.03	--	--	
TPC-6**	17.00	4.92	327	0.45	8.3	0.59	16.3	0.32	18.2	0.29	--	--	8.3	0.07	16.3	0.04	18.2	0.03	--	--	
TPC-7	22.42	2.50	457	0.07	8.3	0.27	16.3	0.14	18.2	0.12	--	--	8.3	0.20	16.3	0.10	18.2	0.09	--	--	
TPC-8	16.50	3.10*	267	0.12	8.3	0.19	16.3	0.10	18.2	0.09	--	--	8.3	0.15	16.3	0.08	18.2	0.07	--	--	
TPC-9	16.83	3.00	327	0.09	8.3	0.21	16.3	0.11	18.2	0.10	--	--	8.3	0.17	16.3	0.09	18.2	0.08	--	--	
TPC-10	22.00	2.39*	267	0.04	8.3	0.16	16.3	0.08	18.2	0.07	--	--	8.3	0.13	16.3	0.07	18.2	0.06	--	--	

LEGEND:

Q = 1/3 of ultimate test load in kips

d = depth in feet

D = diameter in feet

E = modulus of deformation in psi x 1000

S = settlement in inches

*D = diameter of shear collars

**Treated as rigid foundation at actual diameter and depth

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TABLE 8 - SUMMARY OF ULTIMATE BEARING CAPACITY COMPUTATIONS - SITE B

Test Pile No.	d Feet	D Feet	q_s KSF	Q_s Kips	q_B KSF	Q_B Kips	Q_T Kips	Calc. Data Desc. *	Method 1 *	Method 1 Ratio	Method 2 *	Method 2 Ratio	Method 3 *	Method 3 Ratio	Method 4 *	Method 4 Ratio	Method 5 *	Method 5 Ratio	Method 6 *	Method 6 Ratio	Method 7 *	Method 7 Ratio	Method 8 *	Method 8 Ratio
TPB-1			4.3					q_s	2.7	1.59	3.3	1.30	2.7	1.59	3.3	1.30	3.0	1.43	2.5	1.72	1.9	2.27		
				776				Q_s	483	1.61	592	1.31	483	1.61	592	1.31	539	1.45	456	1.69	339	2.27		
	21.4	2.67			86			q_B	172	0.50	172	0.50	34	2.50	42	2.04	87	0.99	62	1.39	81	1.06		
						480		Q_B	965	0.50	965	0.50	190	2.50	235	2.04	485	0.99	346	1.39	454	1.05		
							1256	Q_T	1447	0.87	1557	0.80	673	1.85	827	1.51	1024	1.22	802	1.56	793	1.59		
TPB-2			4.4					q_s	2.5	1.75	3.2	1.37	2.5	1.75	3.2	1.37	2.2	2.00	2.2	2.00	1.5	2.94	2.0	2.72
				560				Q_s	320	1.75	410	1.37	320	1.75	410	1.37	285	1.96	285	1.96	192	2.94	256	2.17
	16.3	3.35			48			q_B	158	0.30	158	0.30	31	1.54	38	1.26	21	2.27	15	3.22	15	3.22	18	2.63
		2.50				420		Q_B	1396	0.30	1396	0.30	273	1.54	335	1.25	182	2.32	130	3.22	132	3.22	159	2.63
							980	Q_T	1716	0.57	1806	0.54	593	1.64	745	1.31	467	2.08	415	2.38	324	3.03	415	2.38
TPB-3			6.4					q_s	3.9	1.64	4.7	1.37	3.9	1.64	4.7	1.37	7.8	0.82	5.0	1.28	4.0	1.59		
				710				Q_s	436	1.64	526	1.35	436	1.64	526	1.35	867	0.82	559	1.26	451	1.56		
	35.8	2.44			103			q_B	233	0.44	233	0.44	44	2.32	51	2.00	56	1.85	21	5.00	34	3.03		
						480		Q_B	1086	0.44	1086	0.44	205	2.32	238	2.00	260	1.85	186	2.56	159	3.03		
							1190	Q_T	1522	0.78	1612	0.74	641	1.85	764	1.56	1127	1.05	745	1.59	610	1.96		
TPB-4			5.5					q_s	3.0	1.82	3.7	1.49	3.0	1.82	3.7	1.49	3.7	1.49	3.3	1.67	2.7	2.04		
				1116				Q_s	620	1.78	758	1.47	620	1.78	758	1.47	753	1.49	675	1.67	540	2.08		
	31.2	2.50			100			q_B	213	0.47	213	0.47	32	3.12	49	2.04	56	1.78	40	2.50	34	2.94		
		2.08				340		Q_B	723	0.47	723	0.47	108	3.12	167	2.04	189	1.78	135	2.50	116	2.94		
							1456	Q_T	1343	1.09	1481	0.98	728	2.00	925	1.56	942	1.54	810	1.78	656	2.22		
TPB-5			3.3					q_s	3.1	1.06	3.7	0.89	3.1	1.06	3.7	0.89	4.7	0.70	3.3	1.00	2.7	1.22		
				800				Q_s	753	1.06	920	0.87	753	1.06	920	0.87	1146	0.70	822	0.97	656	1.22		
					90			q_B	110	0.82	110	0.82	32	2.78	49	1.85	56	1.61	40	2.27	34	2.63		
						430		Q_B	530	0.81	530	0.81	154	2.78	235	1.82	267	1.61	191	2.27	163	2.63		
							1230	Q_T	1283	0.96	1450	0.85	907	1.35	1155	1.06	1413	0.87	1013	1.22	819	1.49		
TPB-6			3.2					q_s	2.5	1.28	3.2	1.00	2.5	1.28	3.2	1.00	2.2	1.45	2.2	1.45	1.5	2.13	2.0	1.59
				490				Q_s	384	1.28	492	1.00	384	1.28	492	1.00	343	1.43	343	1.43	230	2.13	307	1.59
	16.2	3.02			17			q_B	174	0.10	174	0.10	31	0.55	38	0.45	21	0.81	15	1.14	15	1.14	18	0.93
						120		Q_B	1247	0.10	1247	0.10	222	0.54	272	0.44	148	0.81	106	1.14	107	1.14	129	0.93
							610	Q_T	1631	0.37	1739	0.35	606	1.01	764	0.80	491	1.25	449	1.35	337	1.82	436	1.40
TPB-7			3.4					q_s	2.5	1.35	3.2	1.06	2.5	1.35	3.2	1.06	2.2	1.54	2.2	1.54	1.5	2.27	2.0	1.69
				434				Q_s	322	1.35	412	1.05	322	1.35	412	1.05	287	1.51	287	1.51	193	2.27	258	1.69
	15.9	3.05			5			q_B	173	0.03	173	0.03	31	0.16	38	0.13	21	0.24	15	0.33	15	0.33	18	0.28
		2.58				26		Q_B	1260	0.02	1260	0.02	226	0.11	277	0.09	150	0.17	108	0.24	110	0.24	131	0.20
							460	Q_T	1582	0.29	1672	0.27	548	0.84	689	0.67	437	1.05	395	1.16	303	1.52	389	1.18
TPB-8			2.6					q_s	2.5	1.04	3.2	0.81	2.5	1.04	3.2	0.81	2.2	1.18	2.2	1.18	1.5	1.72	2.0	1.30
				382				Q_s	368	1.04	470	0.81	368	1.04	470	0.81	328	1.16	328	1.16	221	1.72	294	1.30
	15.6	3.00			7			q_B	171	0.04	171	0.04	31	0.22	38	0.18	21	0.33	15	0.47	15	0.47	18	0.39
						48		Q_B	1210	0.04	1210	0.04	219	0.22	269	0.18	146	0.33	104	0.46	106	0.45	127	0.38
							430	Q_T	1578	0.27	1680	0.25	587	0.73	739	0.58	474	0.91	432	1.00	327	1.32	421	1.02
TPB-9			3.9					q_s	2.5	1.56	3.2	1.23	2.5	1.56	3.2	1.22	2.3	1.69	2.3	1.69	1.5	2.63	2.0	1.96
				410				Q_s	265	1.54	339	1.20	265	1.54	339	1.20	236	1.72	236	1.72	159	2.56	212	1.92
	16.2	2.88			15			q_B	174	0.09	174	0.09	31	0.48	38	0.39	21	0.71	15	1.00	15	1.00	18	0.56
		2.08				50		Q_B	1134	0.04	1134	0.04	202	0.25	247	0.20	134	0.37	96	0.52	98	0.51	117	0.43
							460	Q_T	1399	0.33	1473	0.31	467	0.98	586	0.79	370	1.25	332	1.39	257	1.79	329	1.39
TPB-10			4.0					q_s	2.5	1.59	3.2	1.25	2.5	1.59	3.2	1.25	2.2	1.82	2.2	1.82	1.5	2.63	2.0	2.00
				422				Q_s	262	1.61	335	1.26	262	1.61	335	1.26	233	1.82	233	1.82	157	2.70	209	2.00
	16.1	3.02			20			q_B	174	0.11	174	0.11	31	0.64	38	0.53	21	0.95	15	1.33	15	1.33	15	1.33
		2.07				68		Q_B	1243	0.05	1243	0.05	222	0.31	272	0.25	147	0.46	106	0.64	107	0.64	129	0.53
							490	Q_T	1505	0.32	1518	0.31	484	1.01	607	0.81	380	1.28	339	1.45	264	1.85	338	1.45

NOTE: TPB-1, TPB-3, TPB-4 AND TPB-5 - q_s ARE AVERAGE VALUES FOR THE ENTIRE LENGTH OF SHAFT. Q_s WAS COMPUTED BY ANALYZING TWO OR MORE SEPARATE LAYERS WITH DIFFERENT q_s.

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TABLE 8 - SUMMARY OF ULTIMATE BEARING CAPACITY COMPUTATIONS - SITE C

Test Pile No.	L Feet	D Feet	Ultimate Values From Load Test Data q_S KSF	Q_S Kips	q_B KSF	Q_B Kips	Q_T Kips	Calc. Data Desc. *	Method 1 *	Method 1 Ratio	Method 2 *	Method 2 Ratio	Method 3 *	Method 3 Ratio	Method 4 *	Method 4 Ratio	Method 5 *	Method 5 Ratio	Method 6 *	Method 6 Ratio	Method 7 *	Method 7 Ratio
TPC-1			9.9					q_S	2.0	5.00	3.1	3.22	2.0	5.00	3.1	3.22	5.0	1.96	7.0	1.41	3.8	2.63
				1580				Q_S	319	5.00	494	3.22	319	5.00	494	3.22	792	2.00	1116	1.41	606	2.63
	20.3	2.50			81			q_B	139	0.58	139	0.58	30	2.70	41	1.96	69	1.18	49	1.67	38	2.13
						400		Q_B	682	0.58	682	0.58	147	2.70	201	2.00	340	1.18	243	1.64	187	2.13
							1980	Q_T	1001	1.96	1176	1.70	466	4.17	695	2.86	1132	1.75	1359	1.45	792	2.50
TPC-2			7.9					q_S	1.8	4.35	2.8	2.86	1.8	4.35	2.8	2.86	7.2	1.10	7.0	1.12	3.8	2.08
				980				Q_S	222	4.35	346	2.86	222	4.35	346	2.86	885	1.11	864	1.14	469	2.08
	15.6	3.18 2.52			104			q_B	195	0.53	195	0.53	40	2.63	57	1.82	110	0.94	79	1.31	61	1.69
						520		Q_B	973	0.53	973	0.53	199	2.63	286	1.82	550	0.94	393	1.31	302	1.69
							1500	Q_T	1195	1.25	1319	1.14	421	3.57	632	2.38	1435	1.04	1257	1.20	771	1.96
TPC-3			7.5					q_S	1.8	4.17	2.8	2.70	1.8	4.17	2.8	2.70	7.2	1.04	7.0	1.07	3.8	1.96
				1092				Q_S	261	4.17	406	2.70	261	4.17	406	2.70	1040	1.05	1015	1.07	551	2.00
	15.4	3.00			58			q_B	122	0.48	122	0.48	25	2.32	36	1.61	69	0.84	50	1.16	38	1.52
						408		Q_B	861	0.47	861	0.47	177	2.32	255	1.59	489	0.83	350	1.16	269	1.52
							1500	Q_T	1123	1.33	1267	1.19	438	3.45	661	2.27	1530	0.98	1365	1.10	820	1.82
TPC-4								q_S														
								Q_S														
	16.2	4.15			59			q_B	125	0.47	125	0.47	25	2.32	36	1.64	69	0.85	49	1.20	38	1.56
						800		Q_B	1685	0.47	1685	0.47	338	2.38	487	1.64	937	0.85	669	1.19	514	1.56
							800	Q_T	1685	0.47	1685	0.47	338	2.38	487	1.64	937	0.85	669	1.19	514	1.56
TPC-5								q_S														
								Q_S														
	21.1	3.54			81			q_B	142	0.57	142	0.57	31	2.63	42	1.92	69	1.18	49	1.67	38	2.13
						800		Q_B	1396	0.57	1396	0.57	305	2.63	413	1.92	682	1.18	487	1.67	374	2.13
							800	Q_T	1396	0.57	1396	0.57	305	2.63	413	1.92	682	1.18	487	1.67	374	2.13
TPC-6								q_S														
								Q_S														
	17.0	4.92			52			q_B	127	0.41	127	0.41	26	2.00	37	1.41	69	0.75	50	1.04	38	1.37
						980		Q_B	2422	0.40	2422	0.40	494	2.00	703	1.39	1316	0.75	941	1.04	722	1.35
							980	Q_T	2422	0.40	2422	0.40	494	2.00	703	1.39	1316	0.75	941	1.04	722	1.35
TPC-7			7.8					q_S	1.8	4.35	3.2	2.44	1.8	4.35	3.2	2.44	7.2	1.09	7.0	1.11	3.8	2.04
				1370				Q_S	316	4.35	563	2.44	316	4.35	563	2.44	1260	1.09	1230	1.11	668	2.04
	22.4	2.50						q_B														
								Q_B														
							1370	Q_T	316	4.35	563	2.44	316	4.35	563	2.44	1260	1.09	1230	1.11	668	2.04
TPC-8			6.2					q_S	1.8	3.45	2.8	2.22	1.8	3.45	2.8	2.22	7.2	0.86	7.0	0.88	3.8	1.64
				800				Q_S	232	3.45	361	2.22	232	3.45	361	2.22	924	0.86	903	0.88	490	1.64
	16.7	3.10 2.46						q_B														
								Q_B														
							800	Q_T	232	3.45	361	2.22	232	3.45	361	2.22	924	0.86	903	0.88	490	1.64
TPC-9			6.2					q_S	1.8	3.45	2.8	2.22	1.8	3.45	2.8	2.22	7.2	0.86	7.0	0.88	3.8	1.64
				980				Q_S	285	3.45	443	2.22	285	3.45	443	2.22	1135	0.86	1108	0.88	601	1.64
	16.8	3.00						q_B														
								Q_B														
							980	Q_T	285	3.45	443	2.22	285	3.45	443	2.22	1135	0.86	1108	0.88	601	1.64
TPC-10			5.8					q_S	2.1	2.78	3.2	1.82	2.1	2.78	3.2	1.82	7.2	0.81	7.0	0.83	3.8	1.52
				800				Q_S	290	2.78	442	1.82	290	2.78	442	1.82	991	0.81	967	0.83	525	1.52
	22.0	2.30 2.00						q_B														
								Q_B														
							800	Q_T	290	2.78	442	1.82	290	2.78	442	1.82	991	0.81	967	0.83	525	1.52

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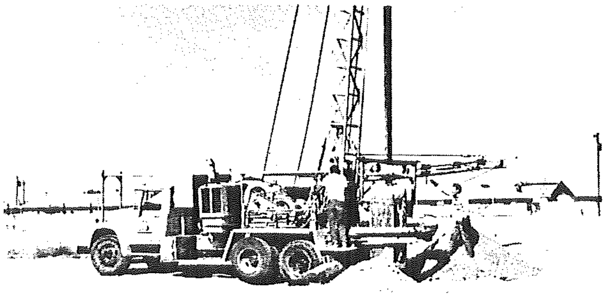
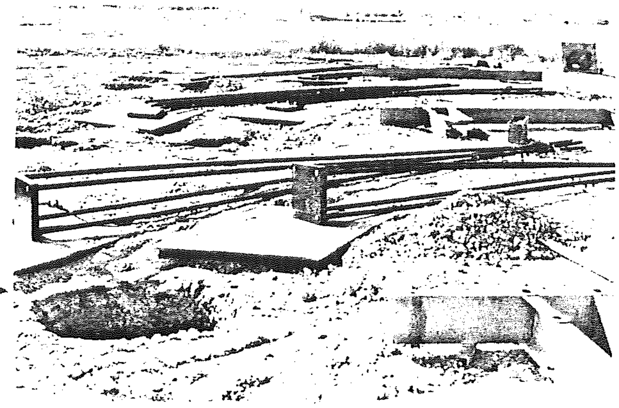


FIG. 1. A CALWELD BUCKET AUGER DRILL RIG FORMING THE BELLED PORTION OF THE ANCHOR PILES.

FIG. 2. FOUR 1 3/8 INCH DIAMETER HIGH STRENGTH STEEL RODS AND A 2 INCH THICK HIGH STRENGTH STEEL PLATE EMBEDDED IN THE BELLED PORTION OF THE CONCRETE ANCHOR PILE AFFORD ANCHORAGE TO THE LOAD FRAME.



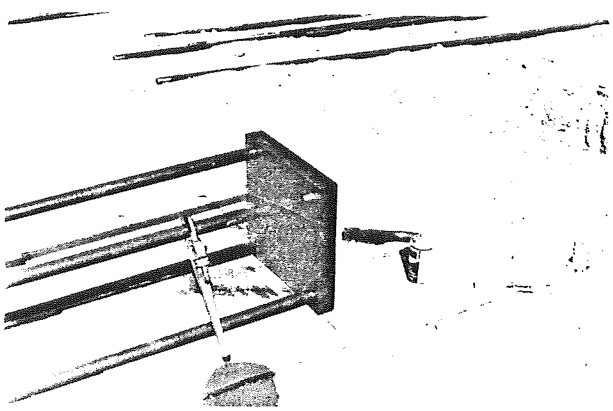
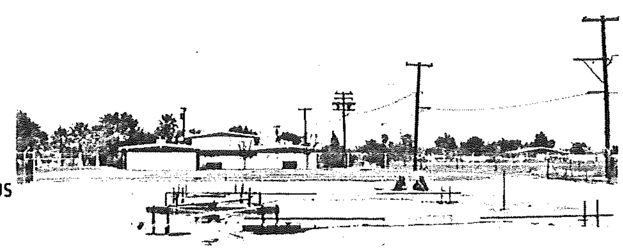


FIG. 3. SPECIAL THREADS ON THE HIGH STRENGTH 160 KSI RODS AND IN THE HIGH STRENGTH STEEL PLATE PERMIT ACCURATE POSITIONING AND RAPID ASSEMBLY.

FIG. 4. A STEEL CHANNEL TEMPLATE SPECIFICALLY DESIGNED AND FABRICATED TO POSITION AND SUPPORT THE ANCHOR RODS AND PLATE ASSEMBLY PERMITTED A CONTINUOUS POUR OF EIGHT CONCRETE ANCHOR PILES.



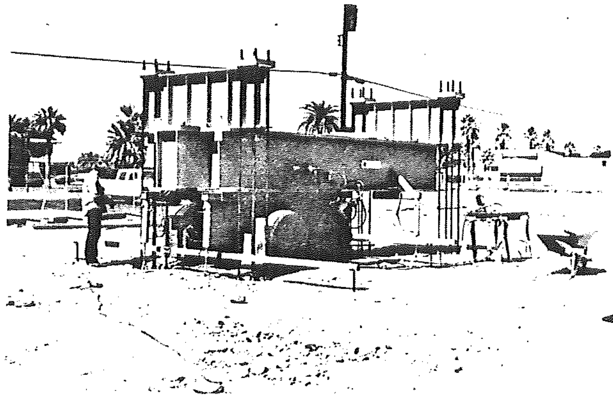
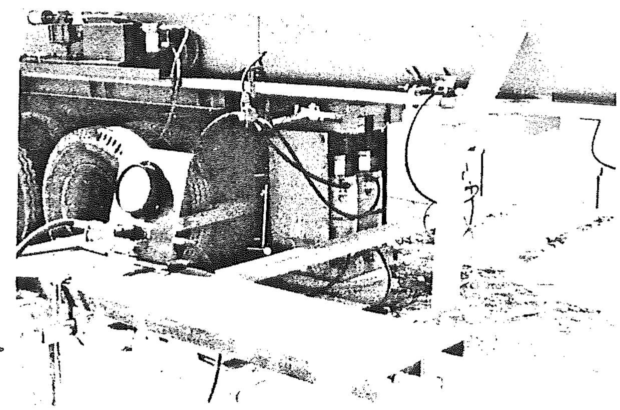


FIG. 5. MOBILE LOAD FRAME ANCHORED INTO LOAD TESTING POSITION FOR APPLIED LOADING INCREMENTS UP TO 1000 TONS AND SHOWING ANCHOR ROD CONNECTION WITH HIGH STRENGTH THREADED COUPLERS AT GROUND LEVEL.

FIG. 6. THE FOUR 300 TON HYDRAULIC DOUBLE ACTING RAMS ARE IN POSITION ON THE TEST PILE. THE HYDRAULIC SYSTEM IS CALIBRATED IN SERIES WITH PRESSURE READOUT BY A 20,000 PSI STAINLESS STEEL GUAGE GRADUATED IN 200 PSI INCREMENTS.



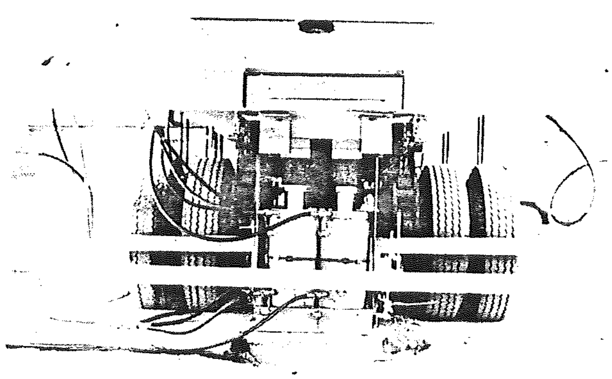
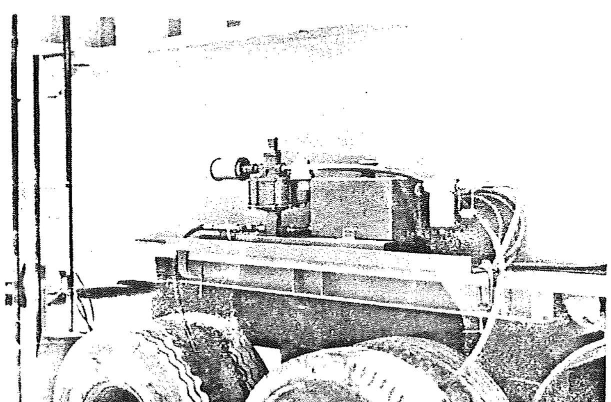
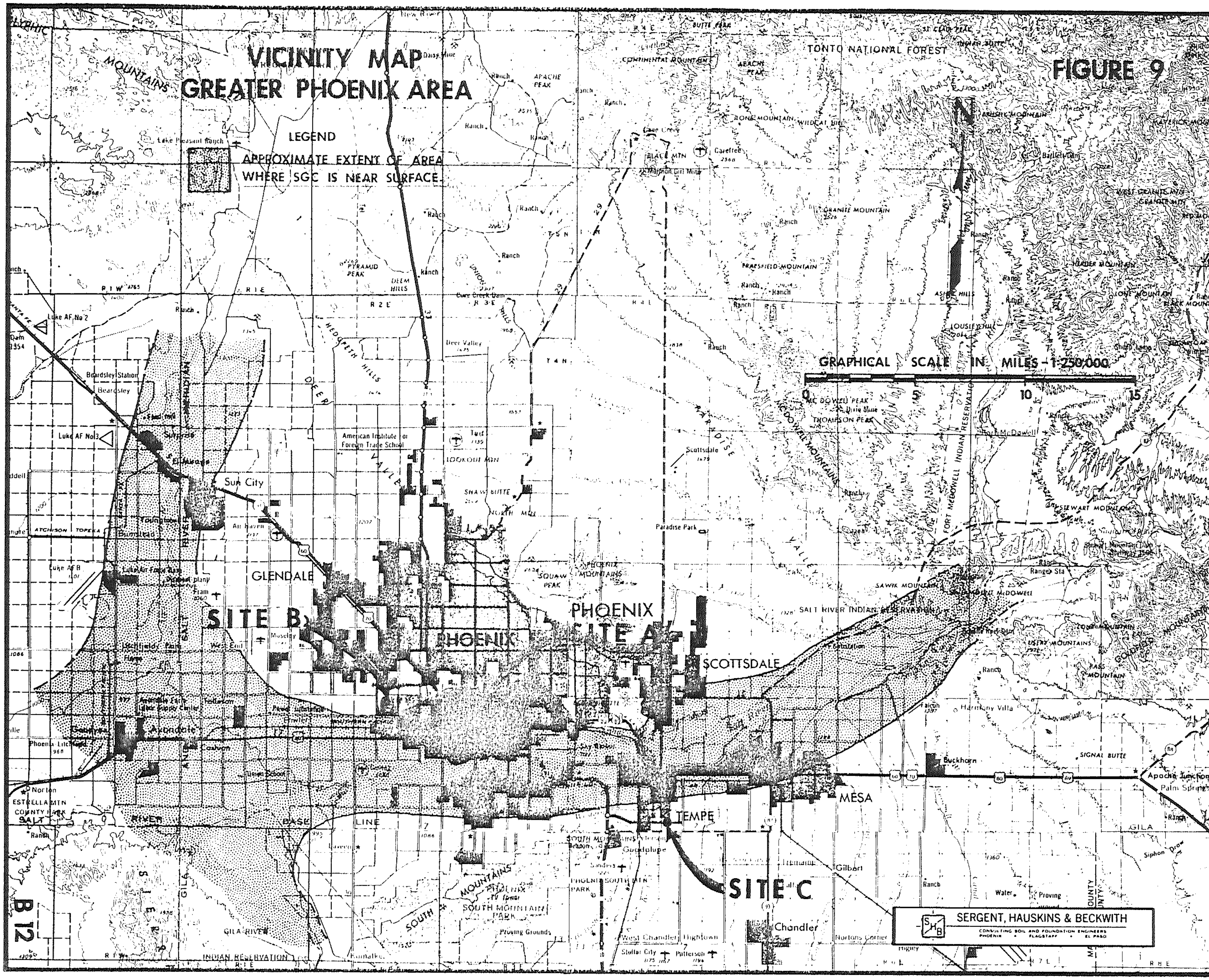


FIG. 7. THE FOUR 300 TON HYDRAULIC DOUBLE ACTING RAMS ARE CONTAINED BY TWO 24 INCH SQUARE BY 5 INCH THICK STEEL PLATES AND CAN BE RAISED FROM THE IN-TEST POSITION BY TWO CABLE RATCHET PULLERS AND BOLTED IN PLACE; THEN THE HYDRAULIC RAM MODULE IS PORTABLE AS A PART OF THE MOBILE LOAD FRAME.

FIG. 8. HYDRAULIC PRESSURE TO THE FOUR 300 TON HYDRAULIC RAM MODULE IS MAINTAINED BY USE OF AN AIR-HYDRAULIC PUMP CAPABLE OF APPLYING AND MAINTAINING A FINE CONTROL OF PRESSURE TO THE SYSTEM UP TO 13,100 PSI FOR A TOTAL APPLIED LOAD OF 1,000 TONS.





VICINITY MAP
GREATER PHOENIX AREA
FIGURE 9
LEGEND
APPROXIMATE EXTENT OF AREA WHERE SGC IS NEAR SURFACE
GRAPHICAL SCALE IN MILES 1:250,000
0
5
10
15
N
SITE A
SITE B
SITE C
PHOENIX
GLENDALE
SCOTTSDALE
TEMPE
MESA
Chandler
Sun City
Avondale
TONTO NATIONAL FOREST
SALT RIVER INDIAN RESERVATION
FORT McDOWELL INDIAN RESERVATION
McDOWELL MOUNTAINS
SOUTH MOUNTAIN PARK
Apache Junction
Gilbert
Guadalupe
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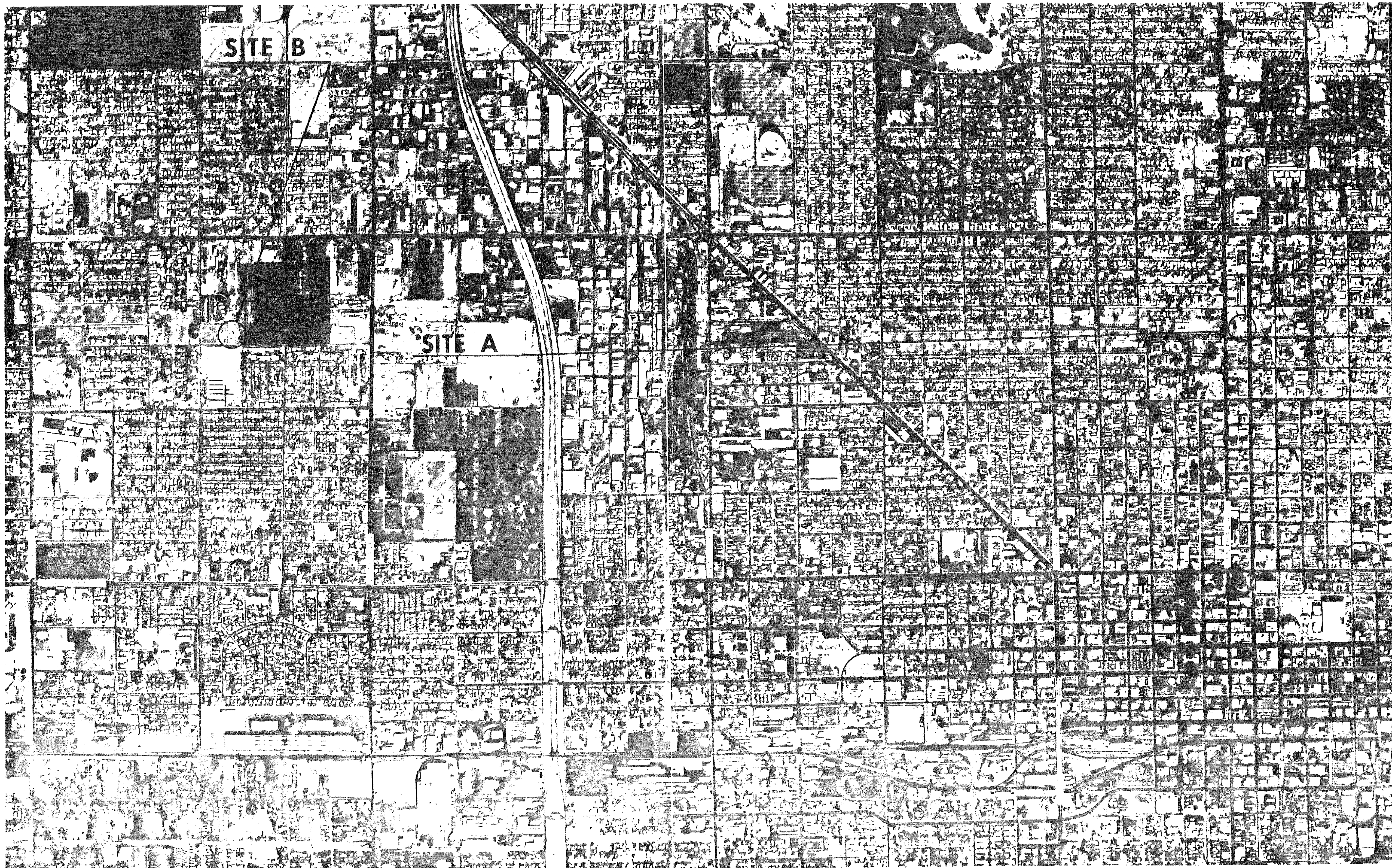
B 12



VICINITY MAP
PHOENIX ARIZONA

SERGENT, HAUSKINS & BECKWITH
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VICINITY MAP
SOUTH TEMPE ARIZONA

SERGENT, HAUSKINS & BECKWITH
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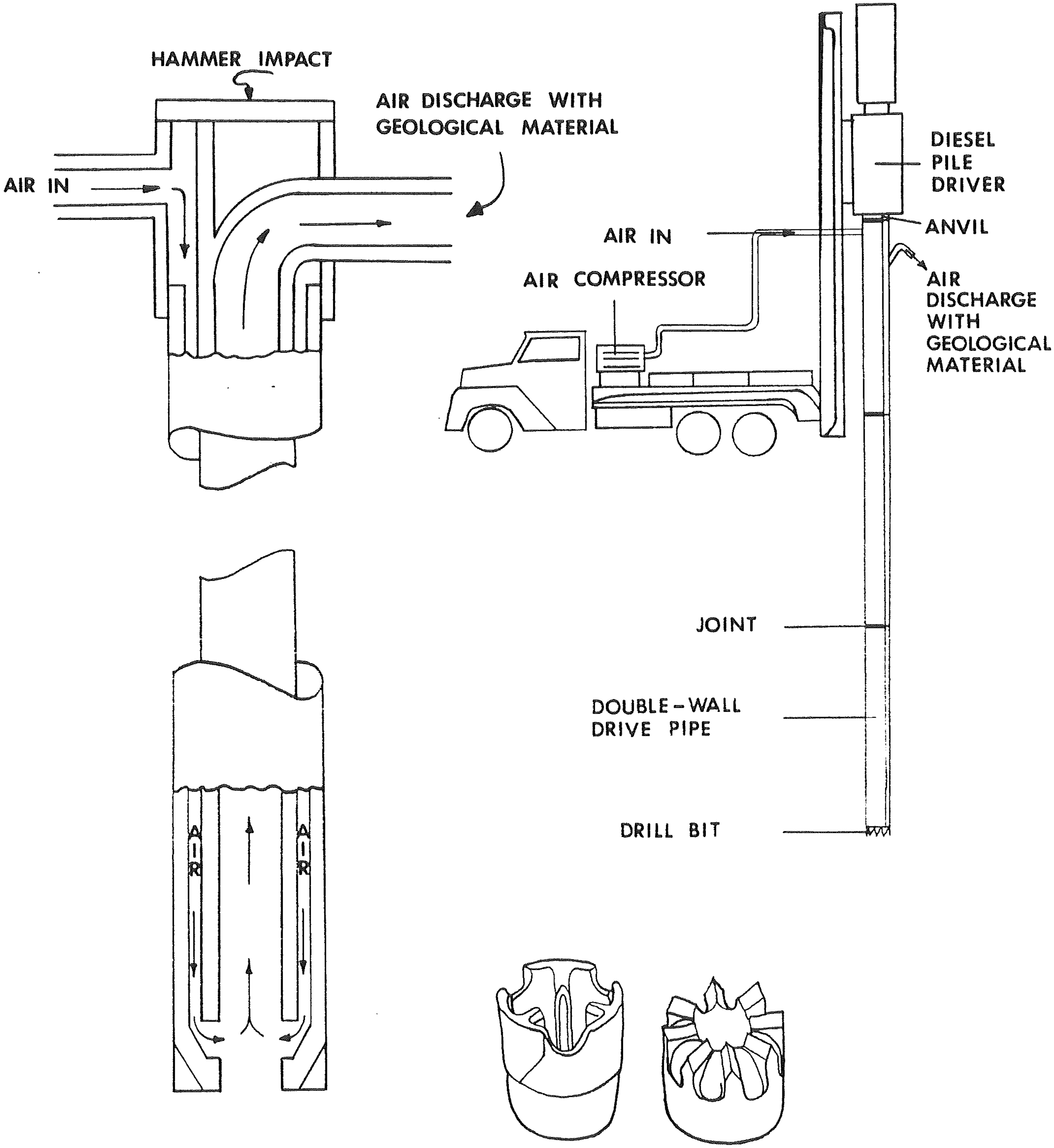
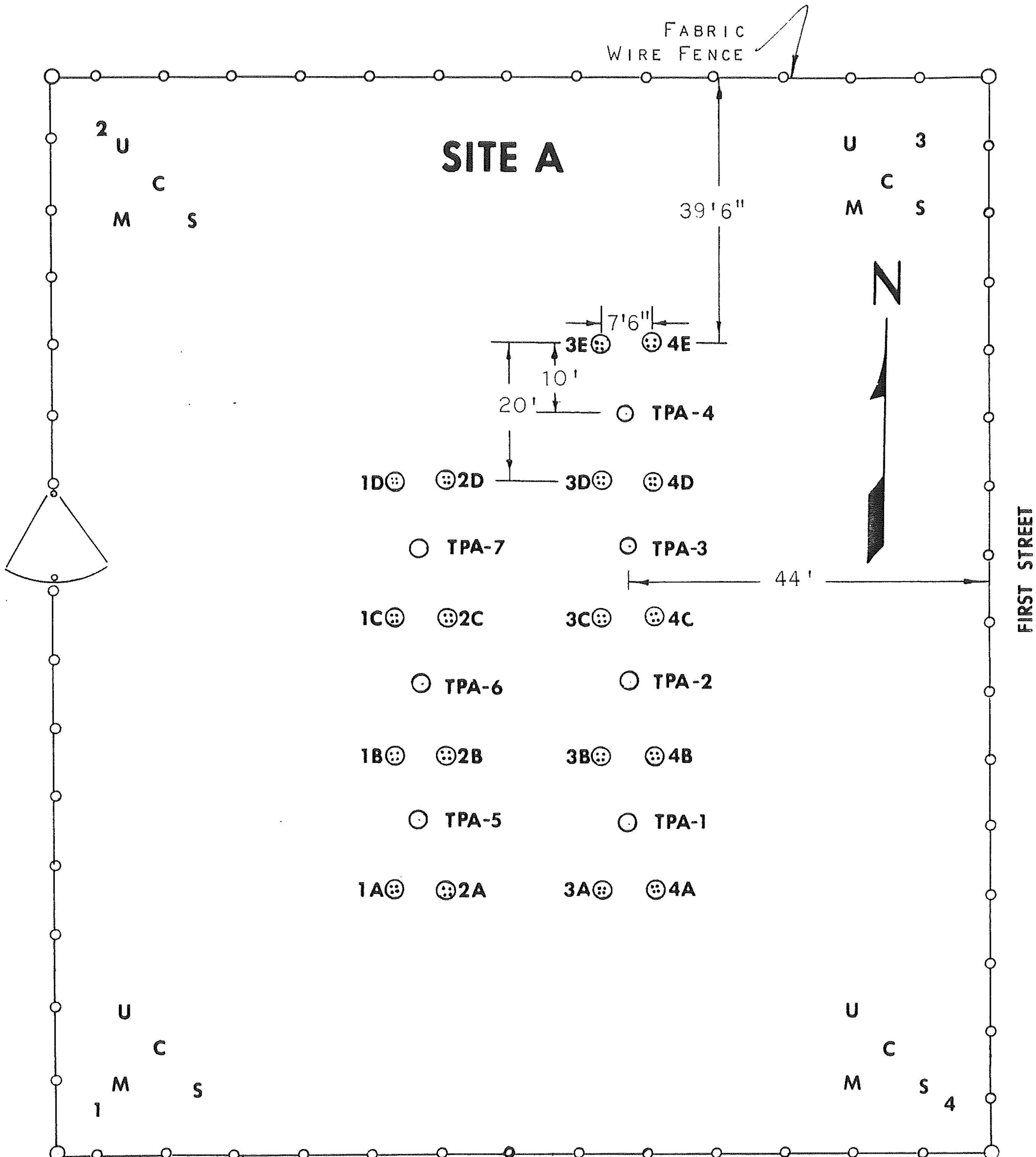
FIGURE 12
PRINCIPAL FEATURES OF BECKER HAMMER DRILL



BECKER DRIVE BITS

FIGURE 13
SITE PLAN – SITE A



C – Continuous Penetration
S – Standard Penetration
U – Undisturbed Ring Samples
M – Menard Pressuremeter

Anchor (Reaction) Piles

Test Piles

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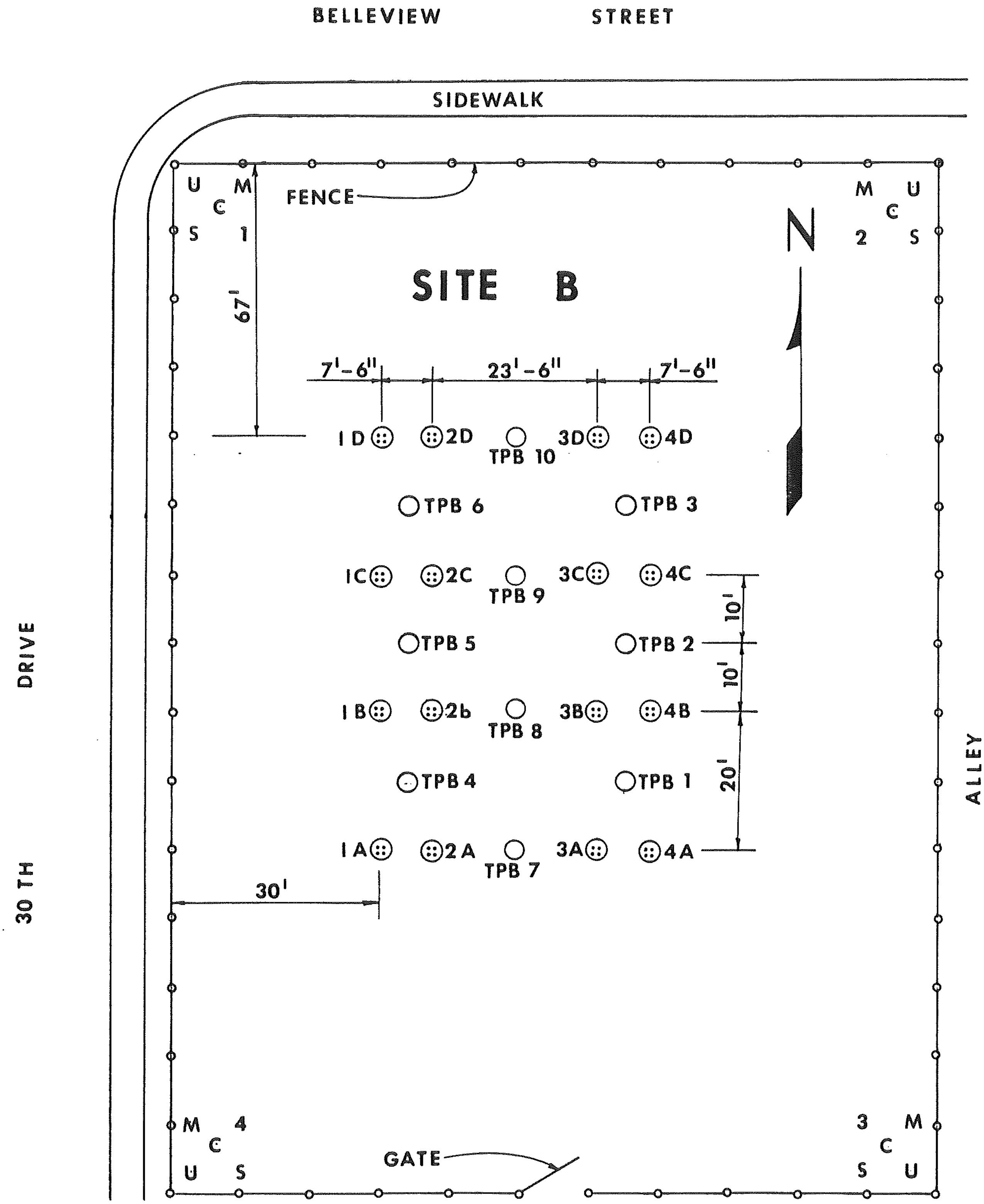
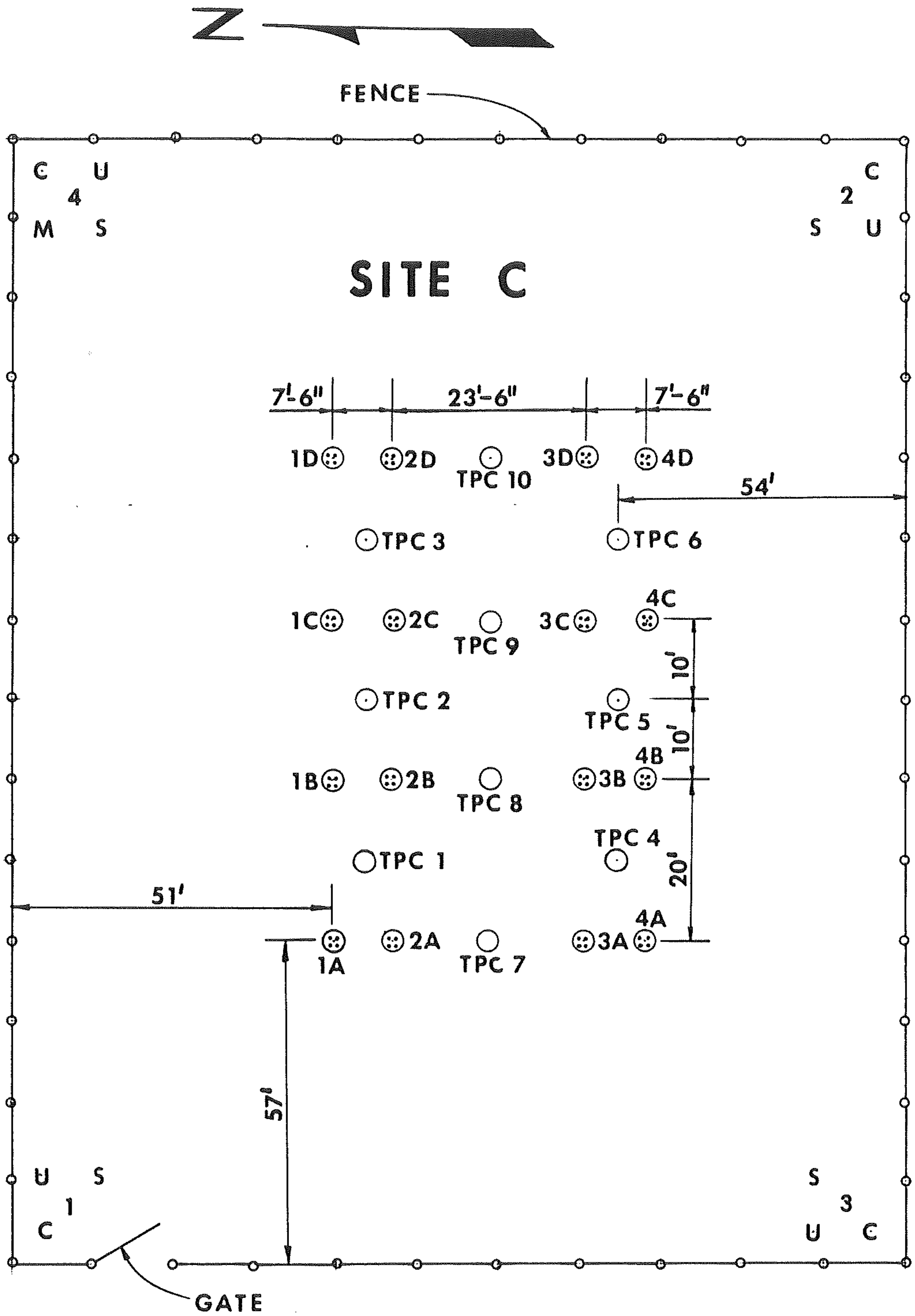
FIGURE 14
SITE PLAN – SITE B



C – Continuous Penetration
S – Standard Penetration
U – Undisturbed Ring Samples
M – Menard Pressuremeter

⊚ Anchor (Reaction) Piles
○ Test Piles

FIGURE 15
SITE PLAN - SITE C



C - Continuous Penetration
S - Standard Penetration
U - Undisturbed Ring Samples
M - Menard Pressuremeter

⊚ Anchor (Reaction) Piles
○ Test Piles

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FIGURE 16

SCHEMATIC OF PRESSUREMETER AND TYPICAL LOAD DEFORMATION CURVE

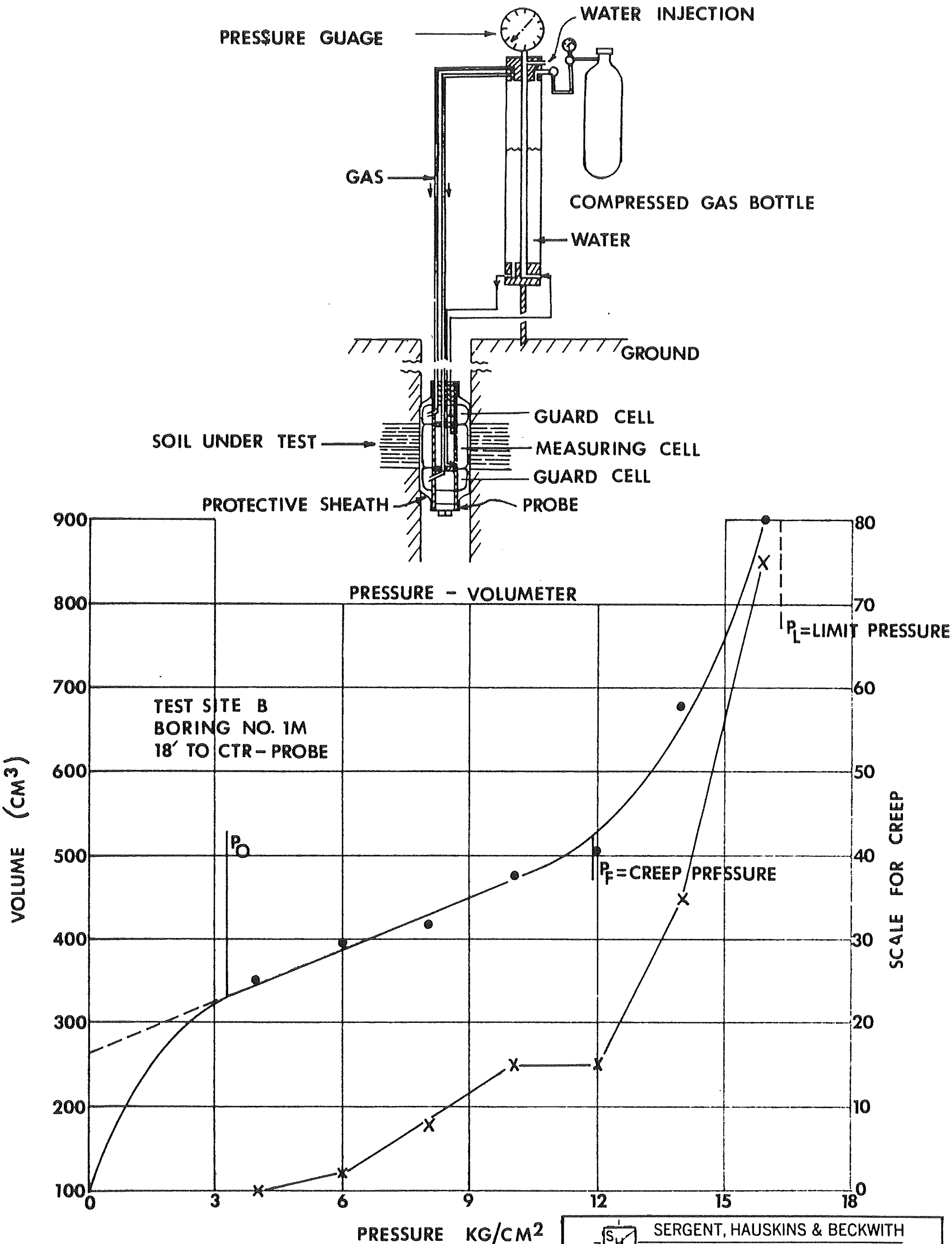
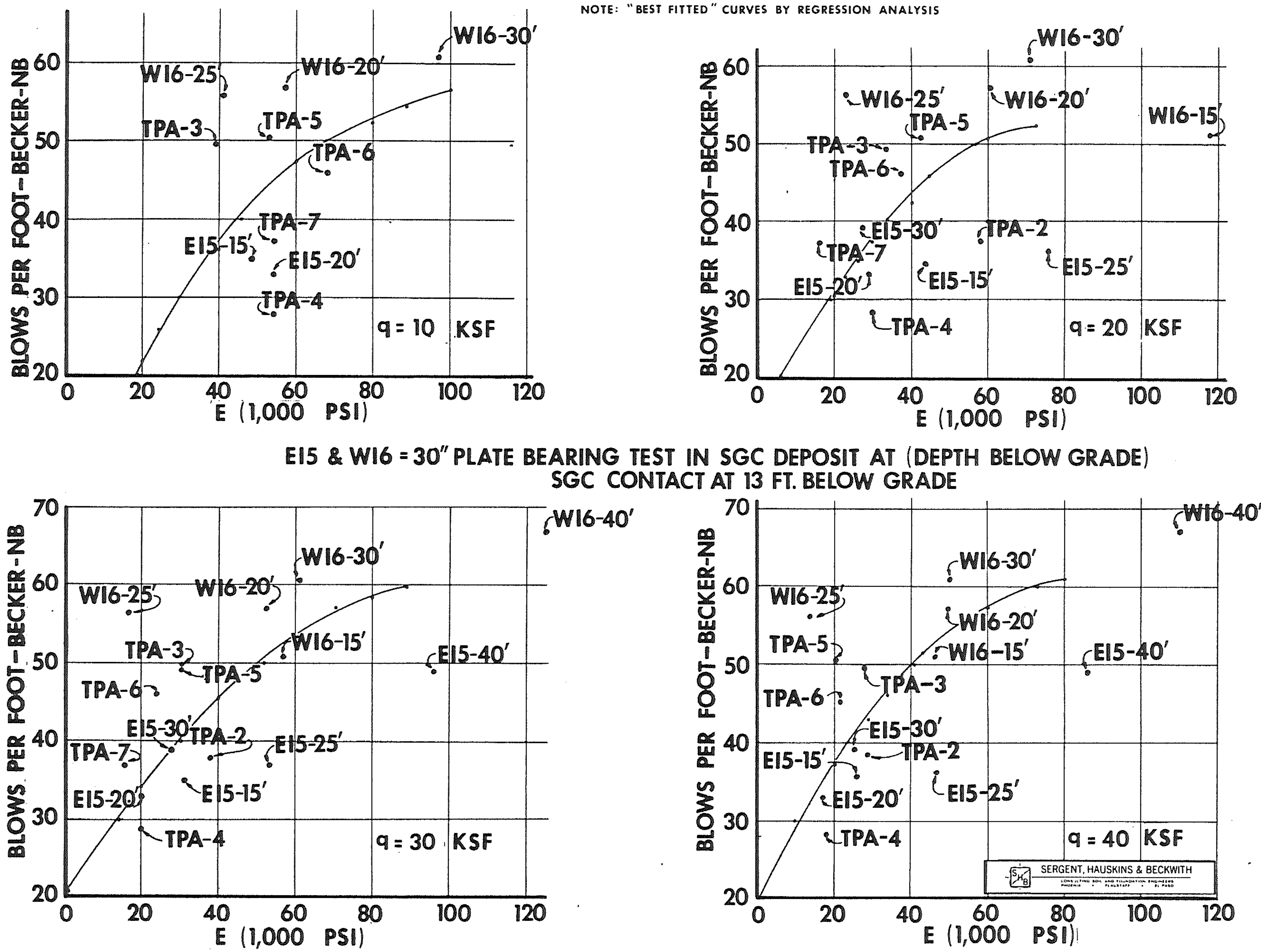


FIGURE 17

SOIL MODULUS VS. WEIGHTED BECKER BLOWS

NOTE: "BEST FITTED" CURVES BY REGRESSION ANALYSIS



E15 & W16 = 30" PLATE BEARING TEST IN SGC DEPOSIT AT (DEPTH BELOW GRADE)
SGC CONTACT AT 13 FT. BELOW GRADE

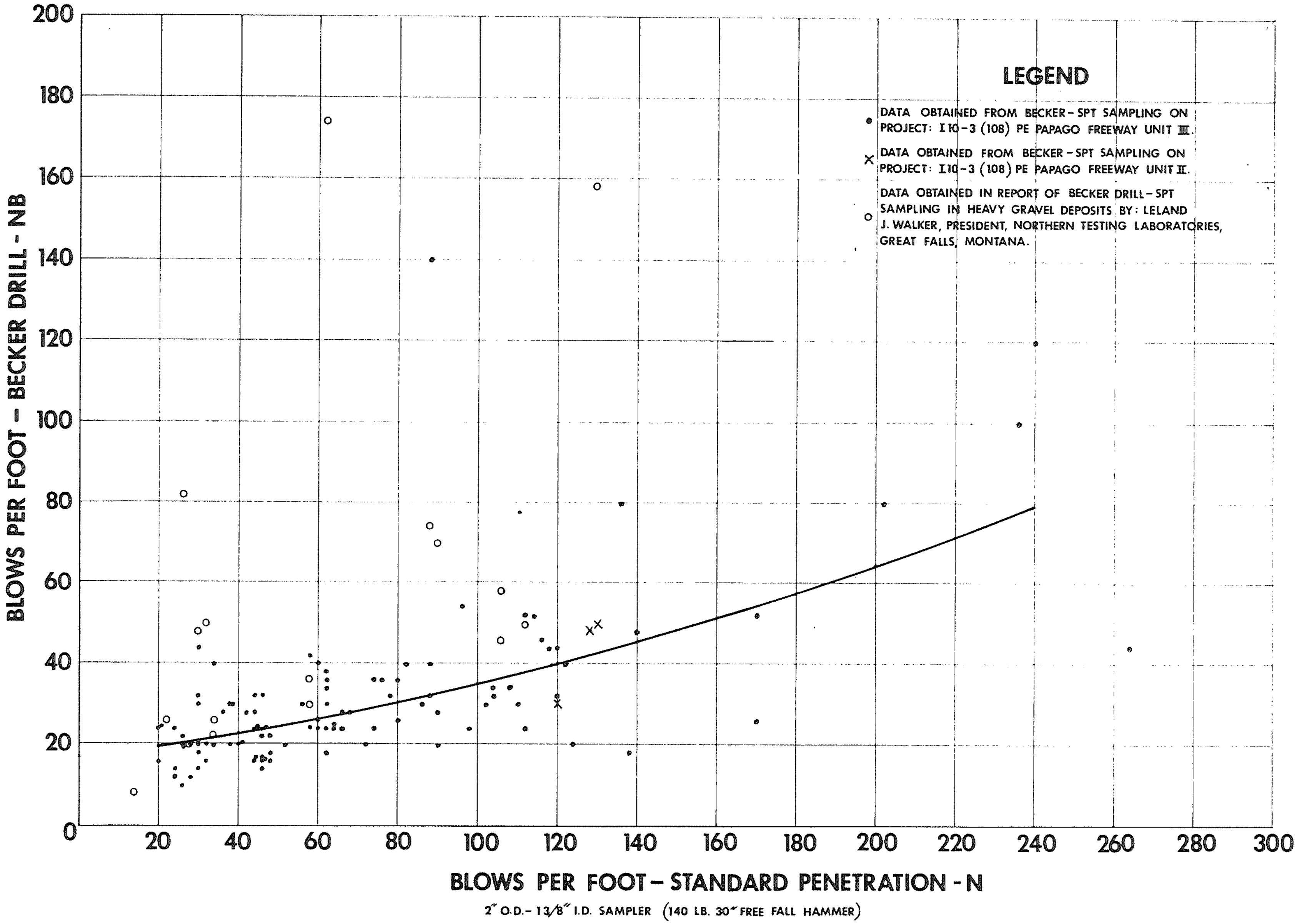


FIGURE 18 RELATION BETWEEN BECKER BLOW COUNT AND STANDARD PENETRATION RESISTANCE

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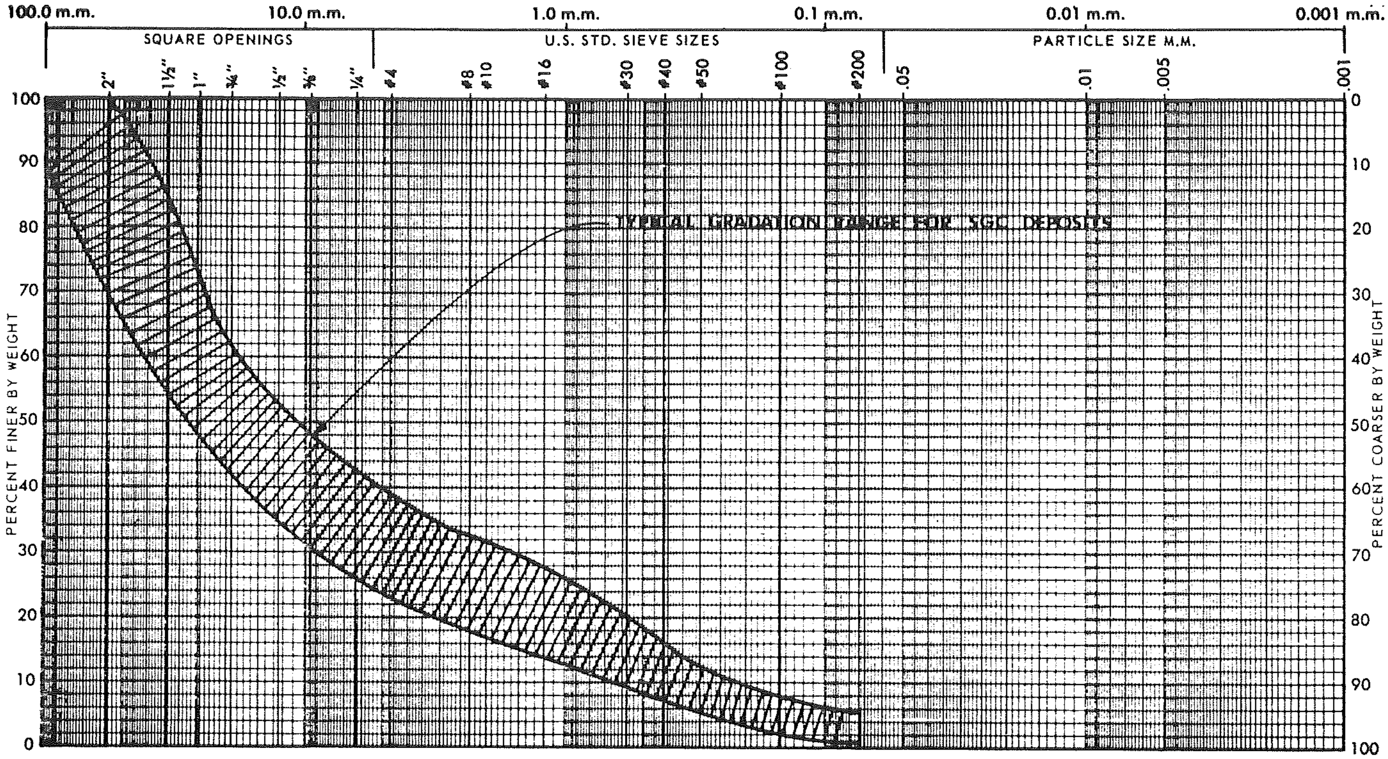
TYPICAL CHARACTERISTICS OF SGC — FIGURE 19

PARTICALE SIZE DISTRIBUTION CURVE



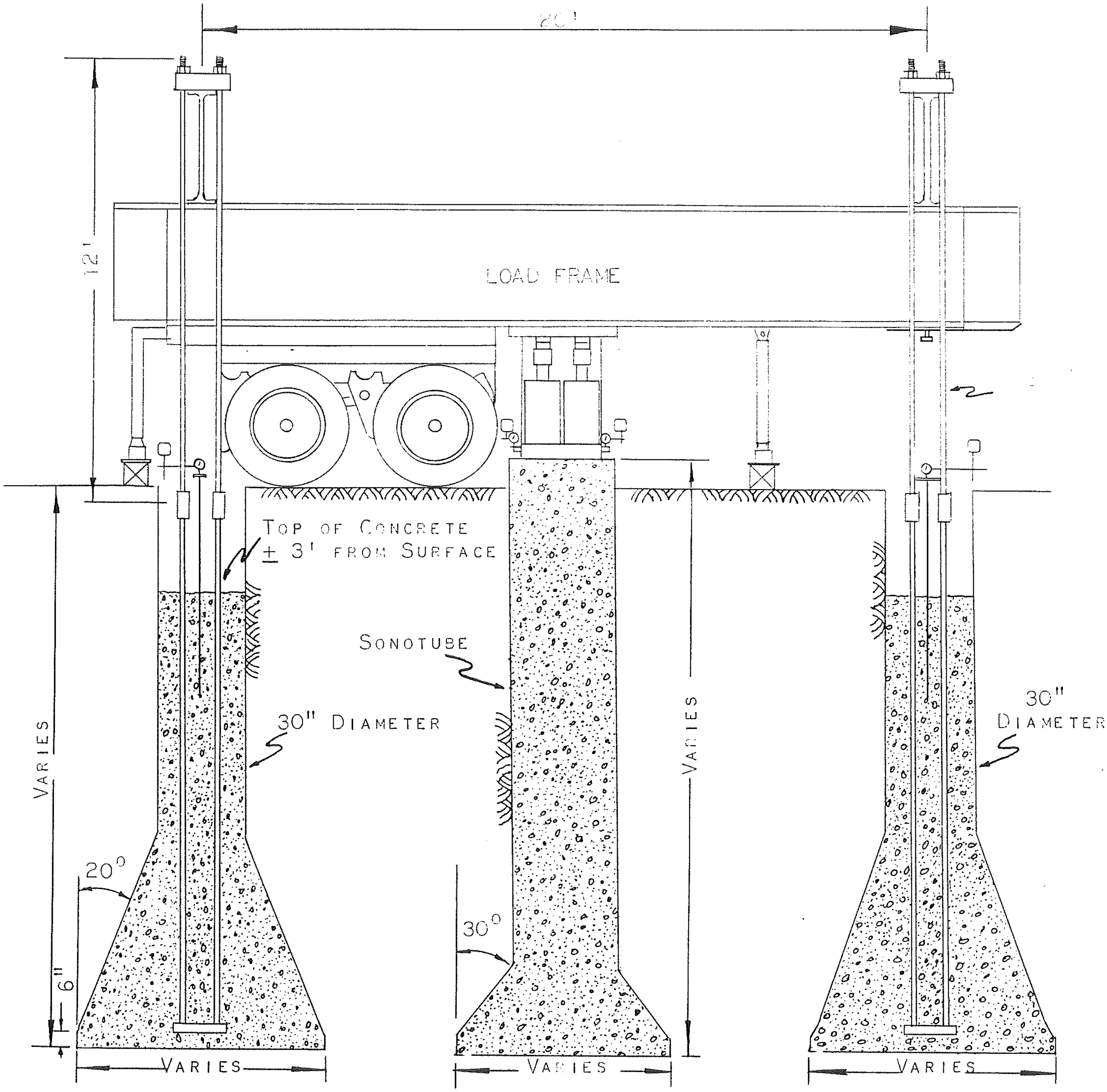
Site A
Sand-Gravel-Cobble Mixture - Sampled from Bottom of Test Pile
Petrographic Analysis by Count

Particle Size	Particle Shape	% Quartzite & Chert	% Sandstone & Siltstone	% Granitics	% *Volcanics	% Other - Metamorphics
$3\frac{1}{2}$"	Subrounded	100	-	-	-	-
3"	Subrounded	100	-	-	-	-
2"	Subrounded to Subangular	73	-	18	9	-
$1\frac{1}{2}$"	Subrounded to Subangular	41	18	23	-	18
1"	Subrounded to Subangular	51	-	22	27	-
$\frac{3}{4}$"	Subrounded to Subangular	57	11	13	18	1
$\frac{1}{2}$"	Subrounded to Subangular	66	8	11	10	5
3/8"	Angular to Subrounded	46	23	11	13	7
$\frac{1}{4}$"	Angular to Subrounded	67	13	8	9	3
#4	Angular to Subrounded	61	12	12	10	5
#8 - #200	Angular	50	15	10	15	10

*Basalt, Andesite, Latites
Plasticity Index - Nonplastic

FIGURE 20

LOAD FRAME, TEST PILE & REACTION PILES



SEE DATA TABULATION FOR EACH TEST SITE

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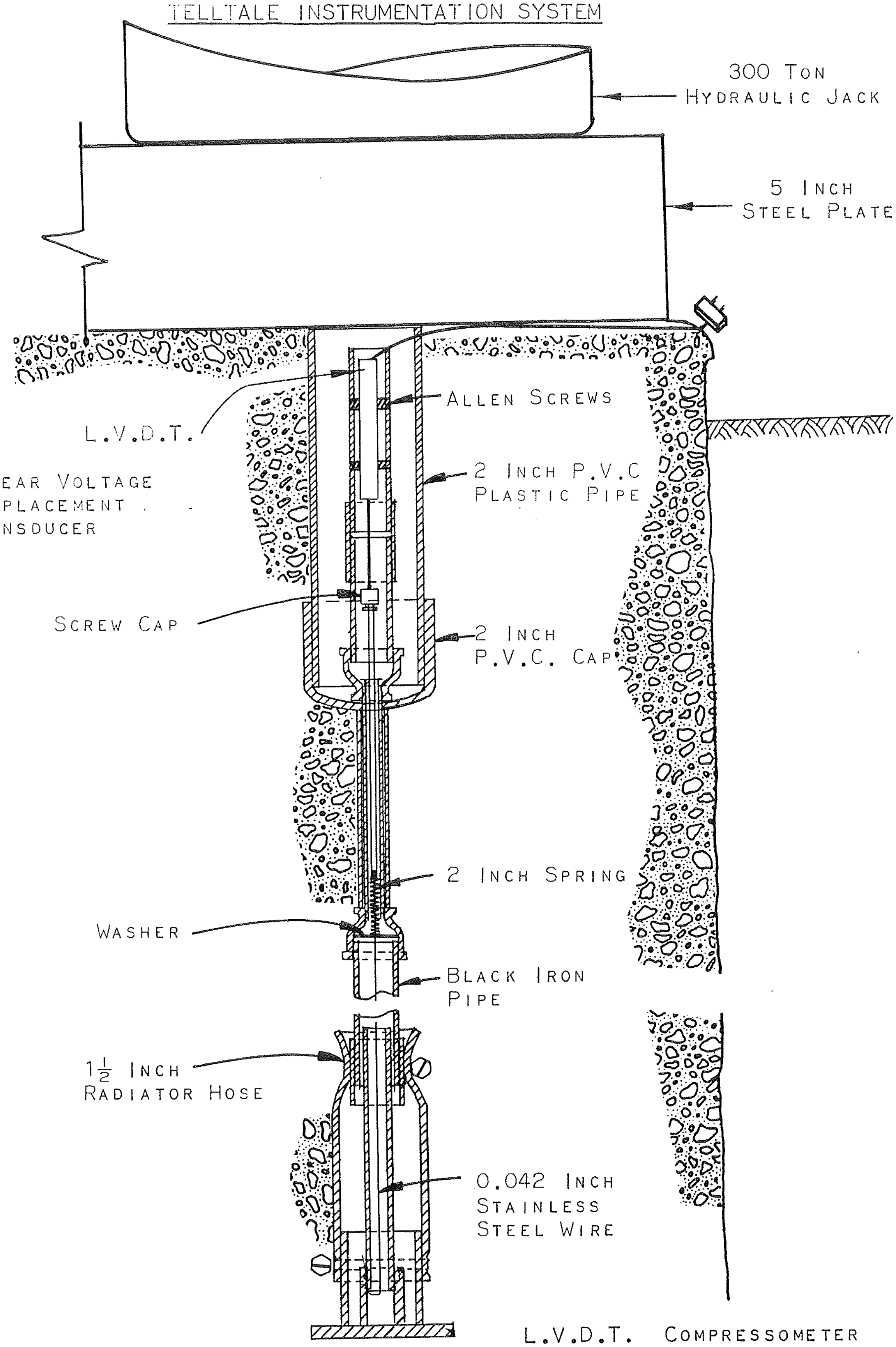
FIGURE 21
TELLTALE INSTRUMENTATION SYSTEM



L.V.D.T. COMPRESSOMETER

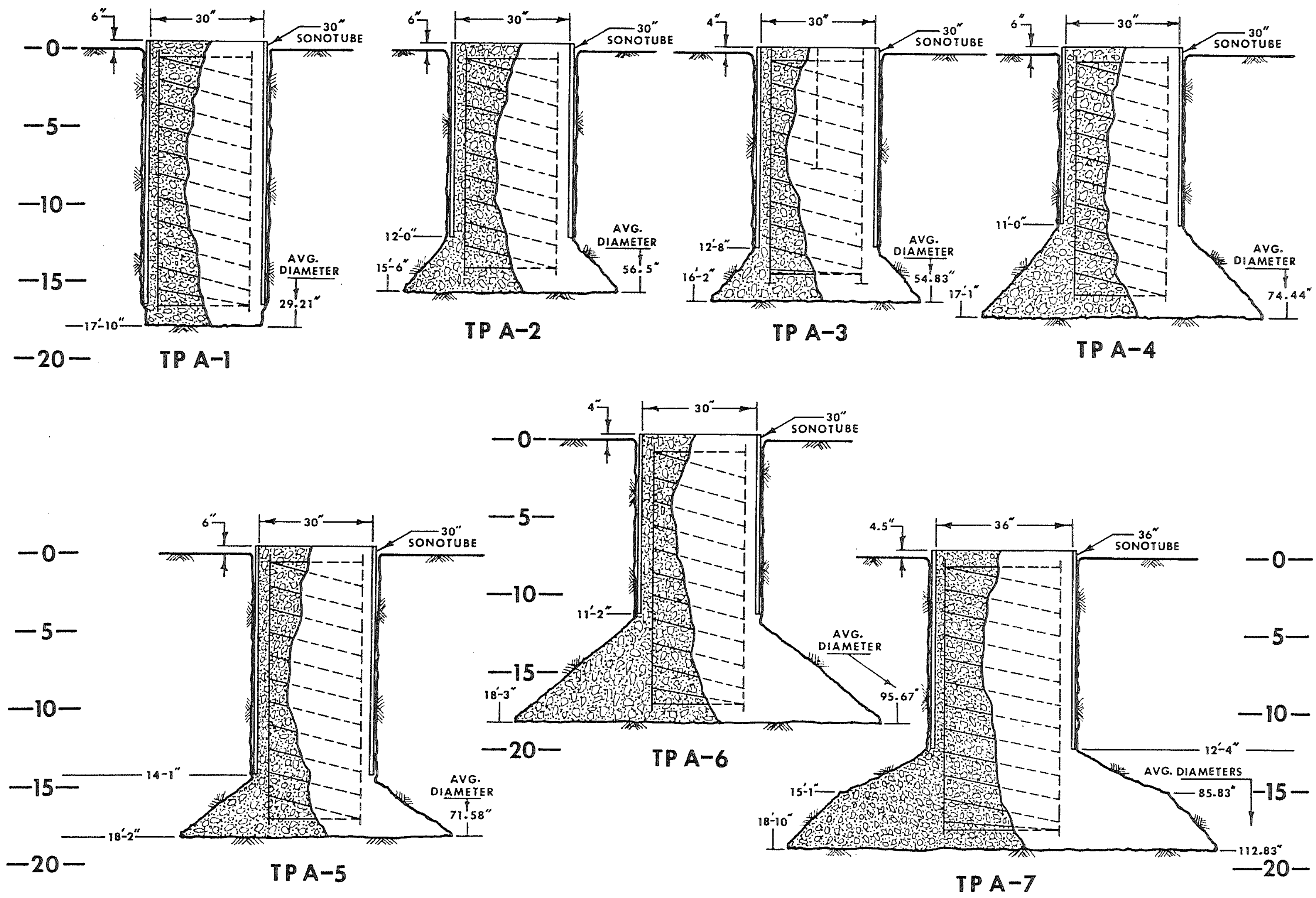


FIGURE 22 SCHEMATIC OF TEST PILES – SITE A

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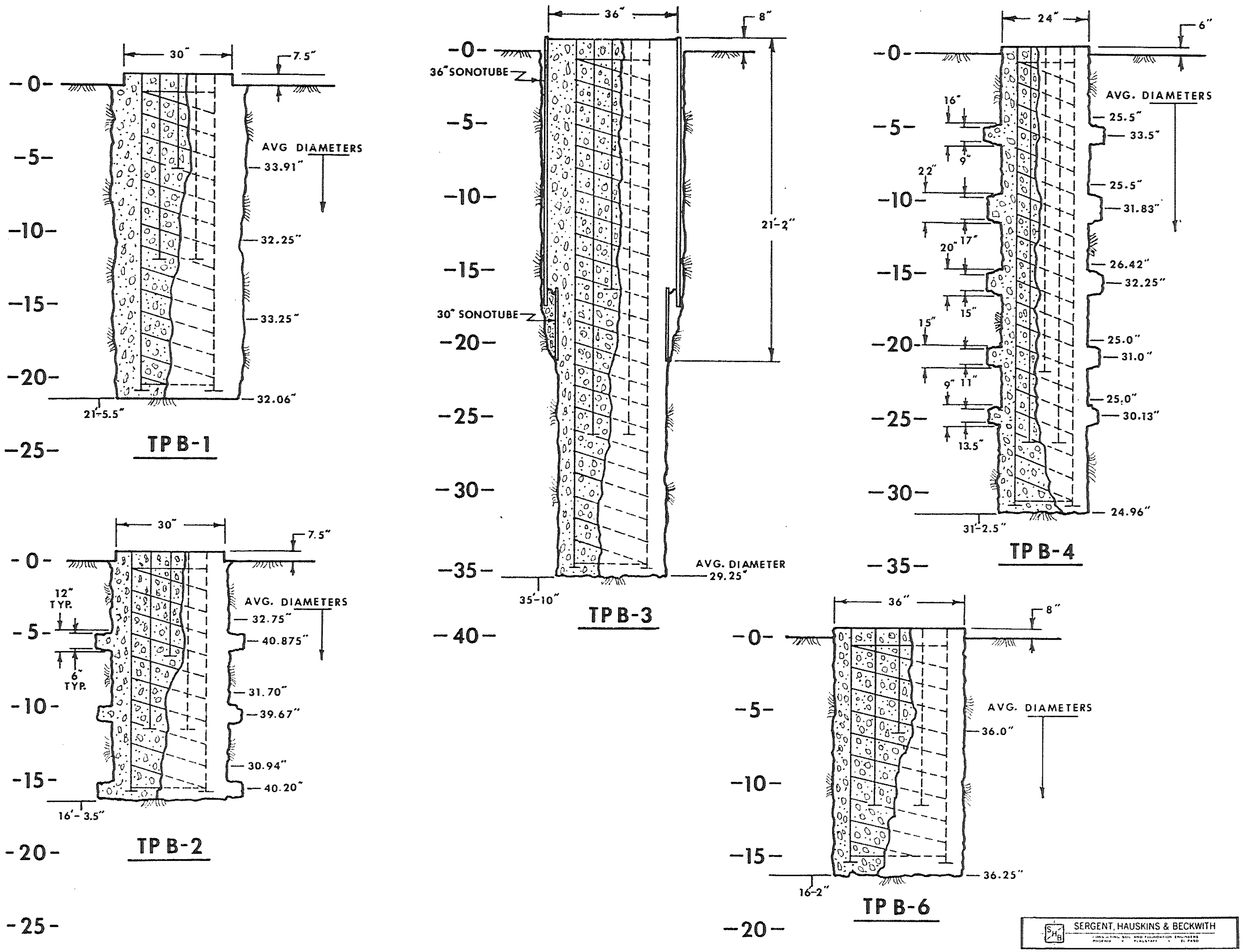


FIGURE 23 SCHEMATIC OF TEST PILES – SITE B

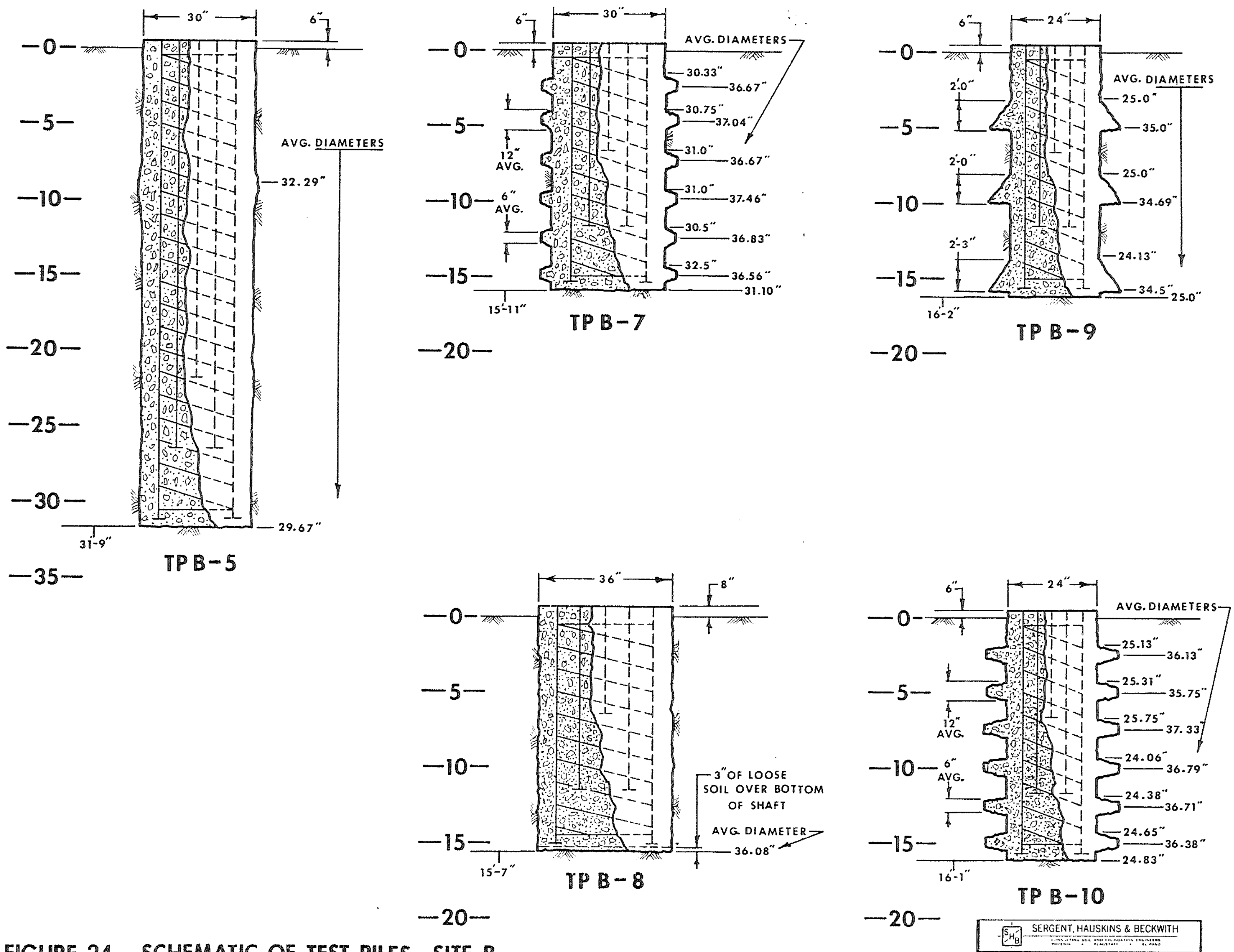


FIGURE 24 SCHEMATIC OF TEST PILES – SITE B

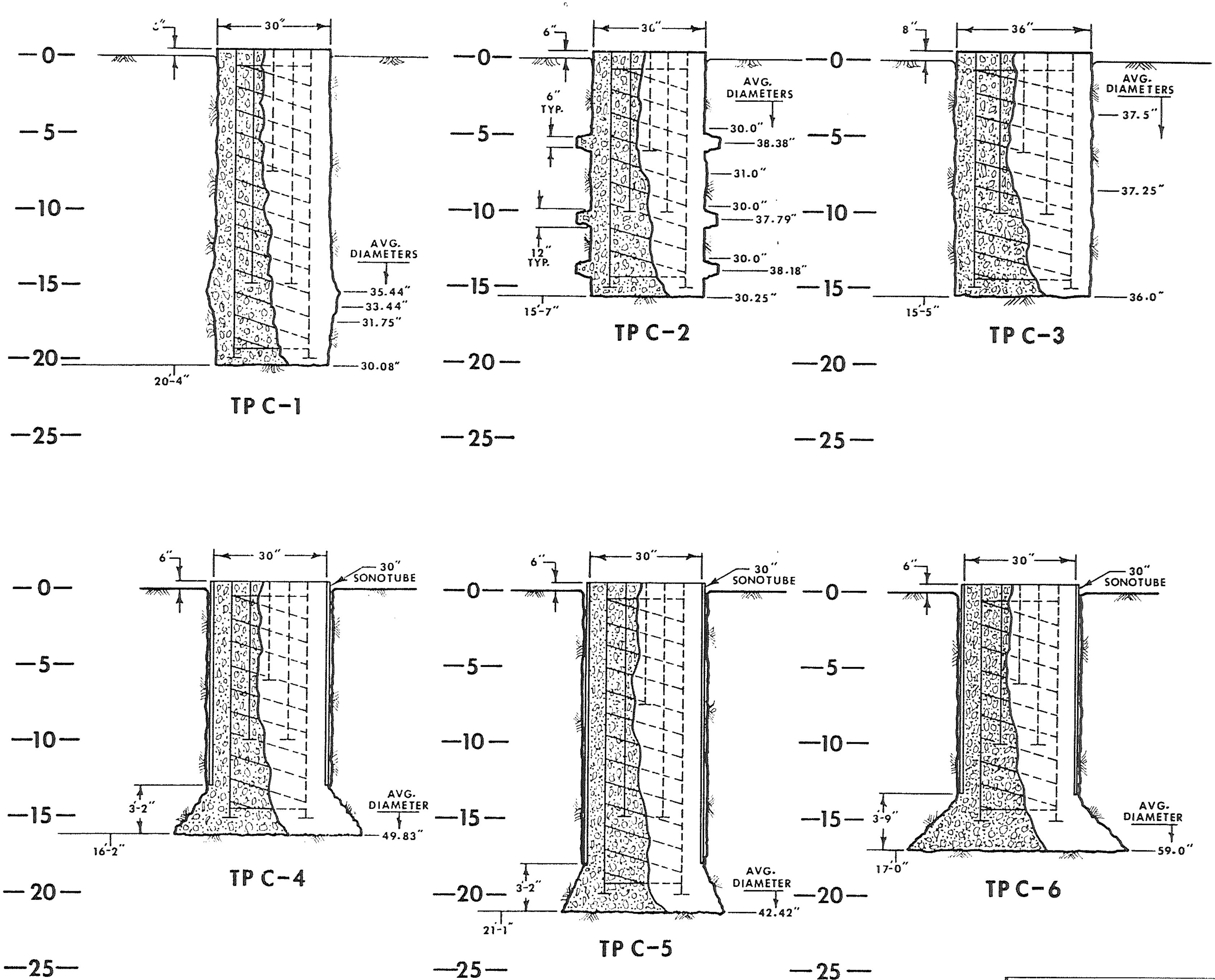
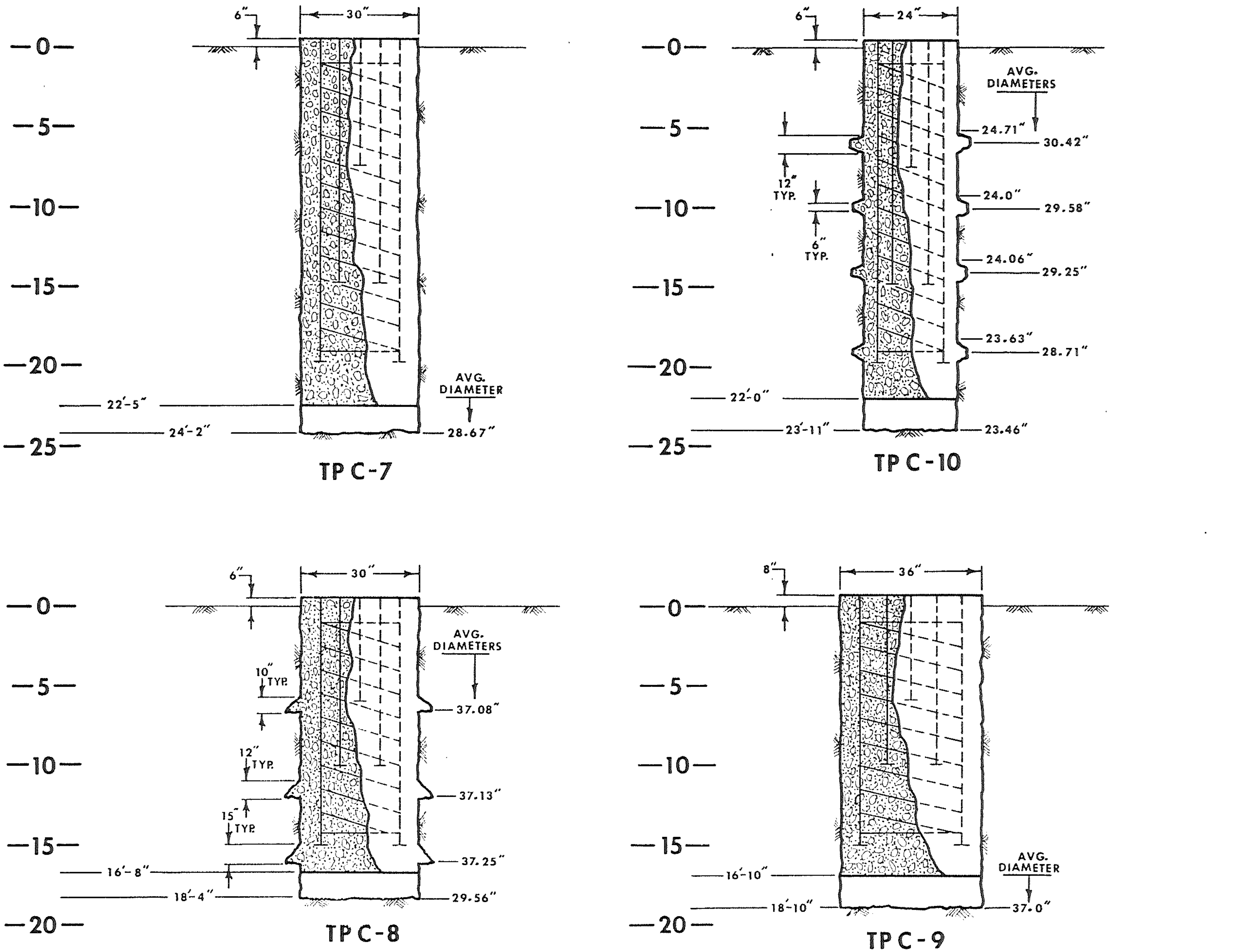


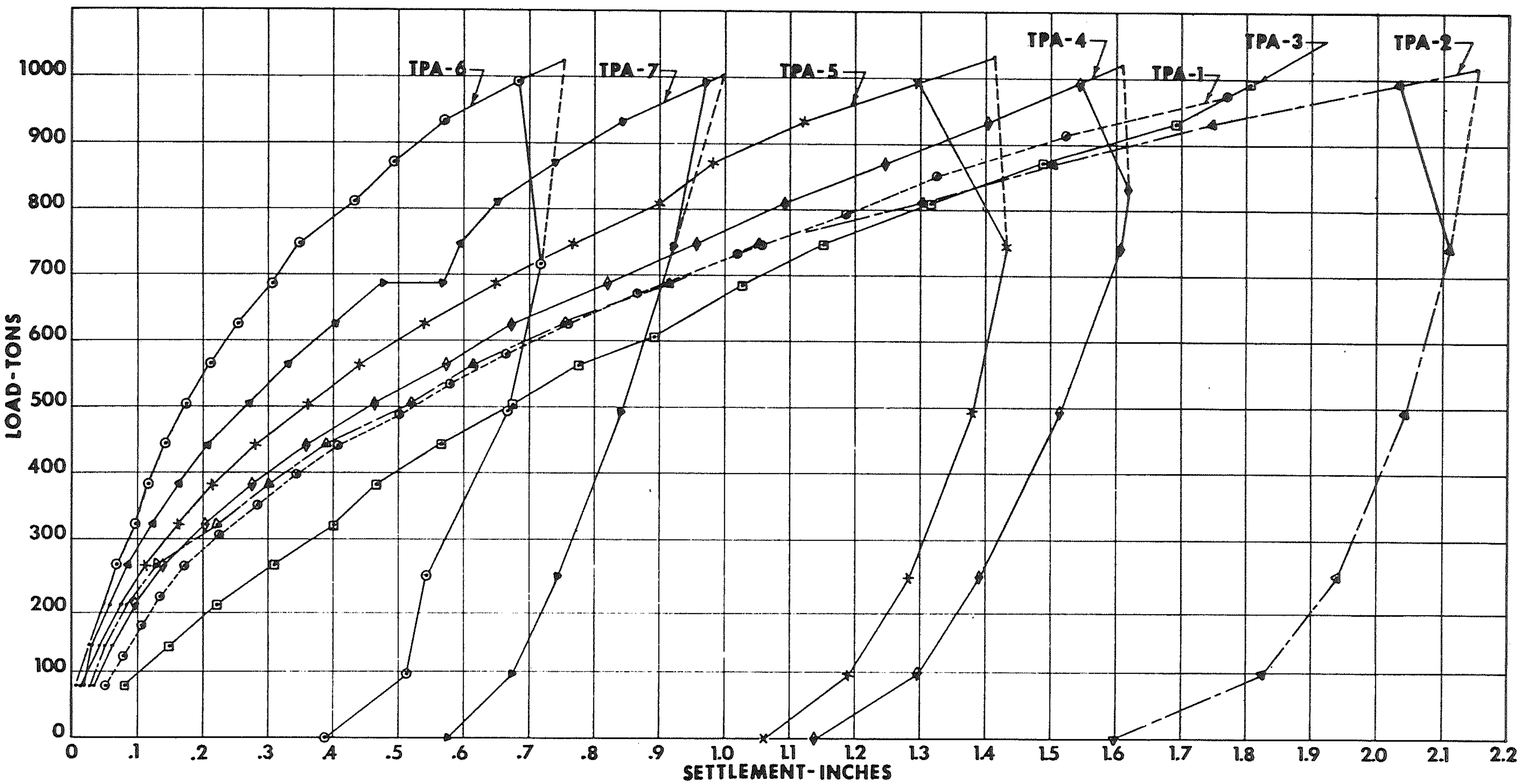
FIGURE 25 SCHEMATIC OF TEST PILES – SITE C

SERGENT, HAUSKINS & BECKWITH



SERGENT, HAUSKINS & BECKWITH

FIGURE 26 SCHEMATIC OF TEST PILES – SITE C



SERGENT, HAUSKINS & BECKWITH

FIGURE 27 LOAD SETTLEMENT CURVES – TEST PILES – SITE A

FIGURE 28 LOAD SETTLEMENT CURVES – TEST PILES – SITE B

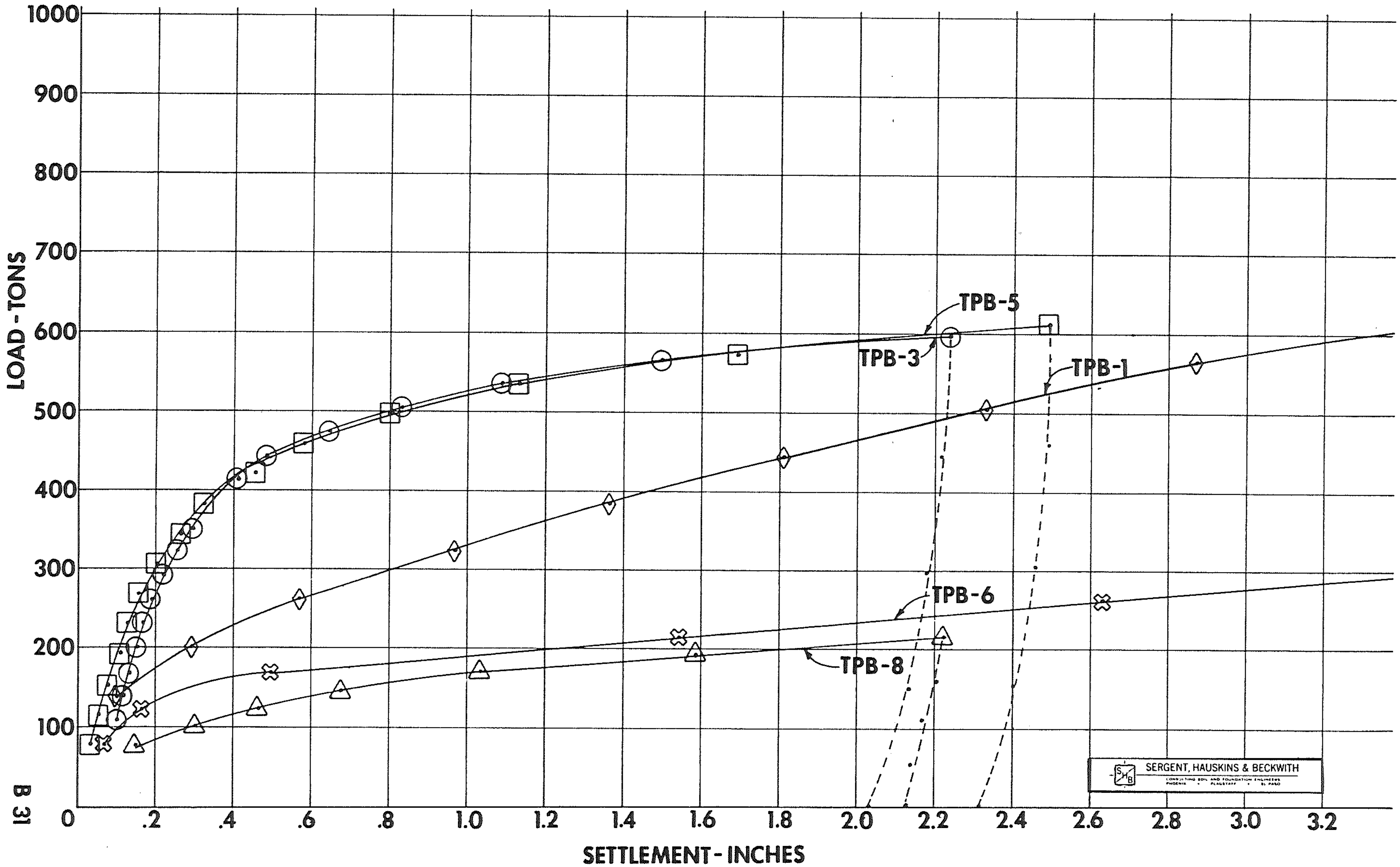


FIGURE 29 LOAD SETTLEMENT CURVE – TEST PILES – SITE B

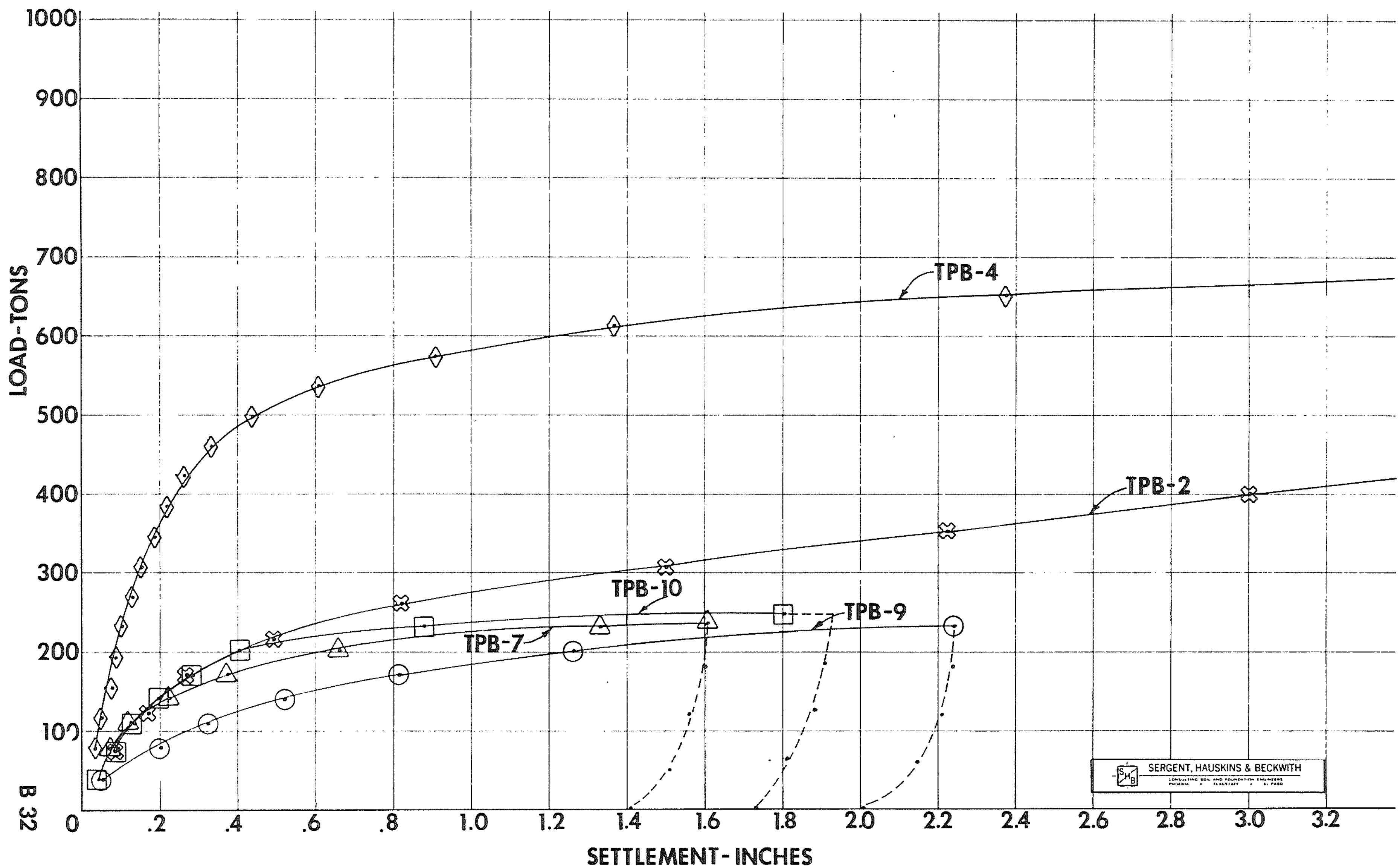


FIGURE 30 LOAD SETTLEMENT CURVES – LOAD TESTS – SITE B

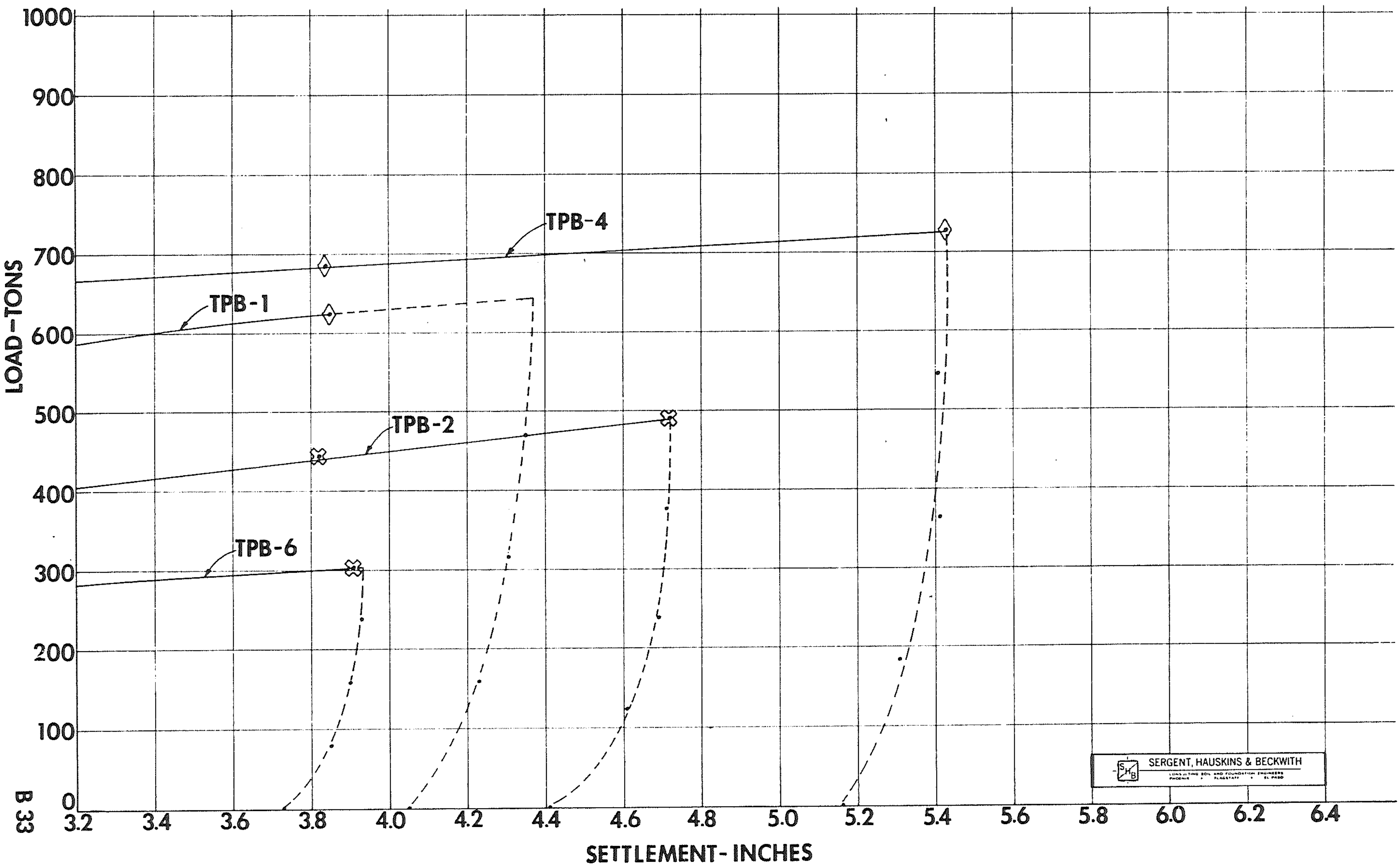


FIGURE 31 LOAD SETTLEMENT CURVES – LOAD TESTS – SITE C

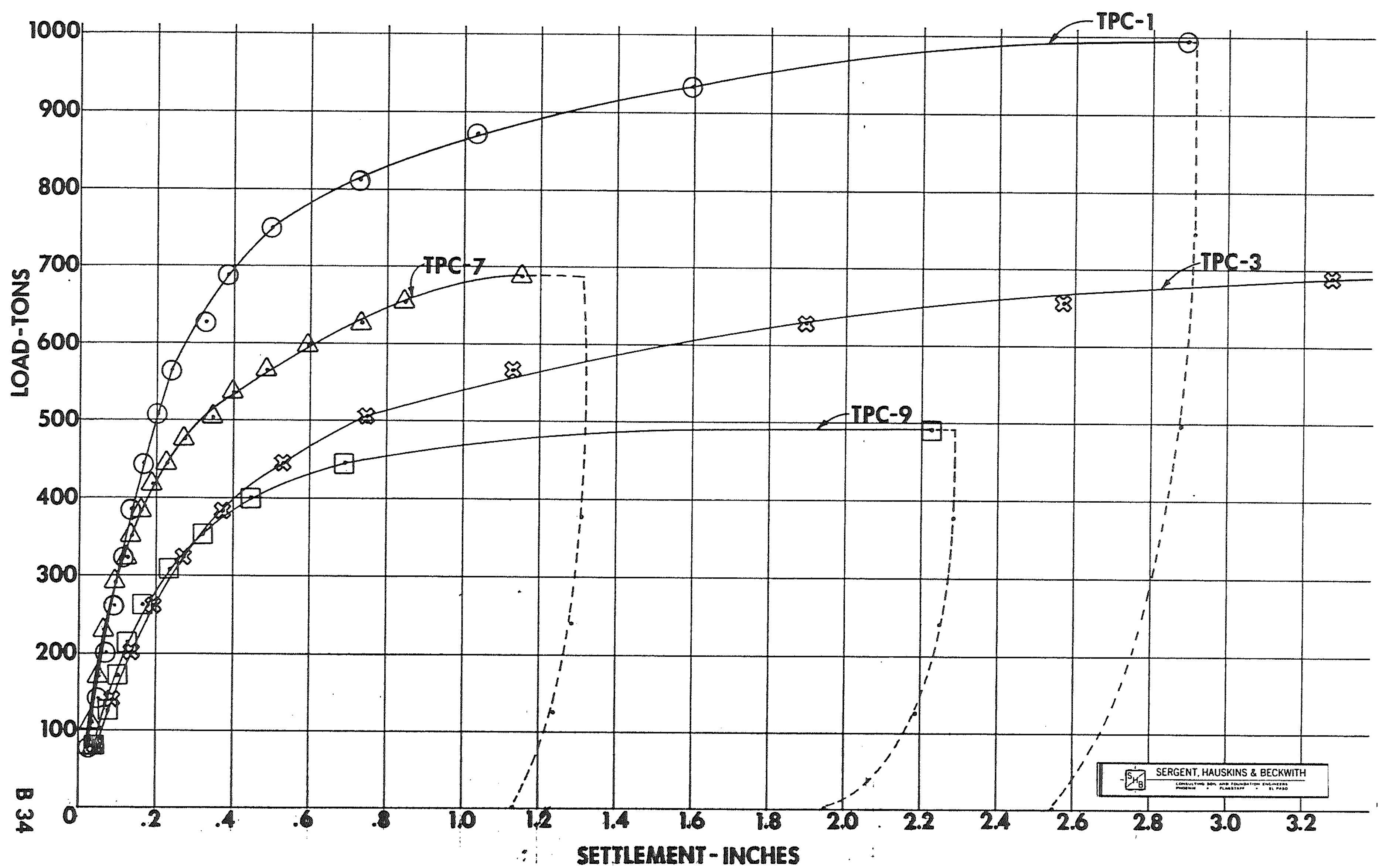


FIGURE 32 LOAD SETTLEMENT CURVES – LOAD TESTS – SITE C

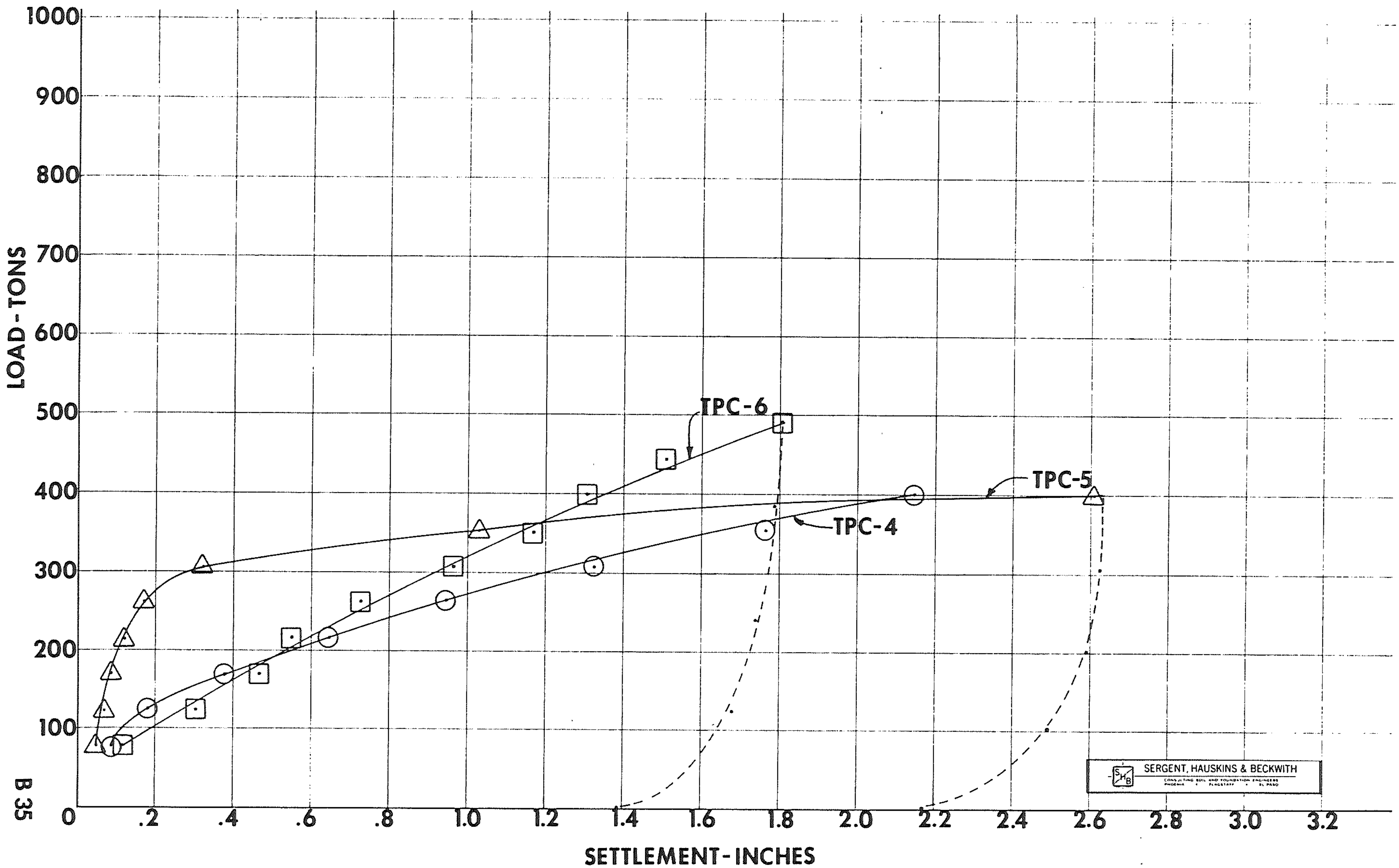


FIGURE 33 LOAD SETTLEMENT CURVES – LOAD TESTS – SITE C

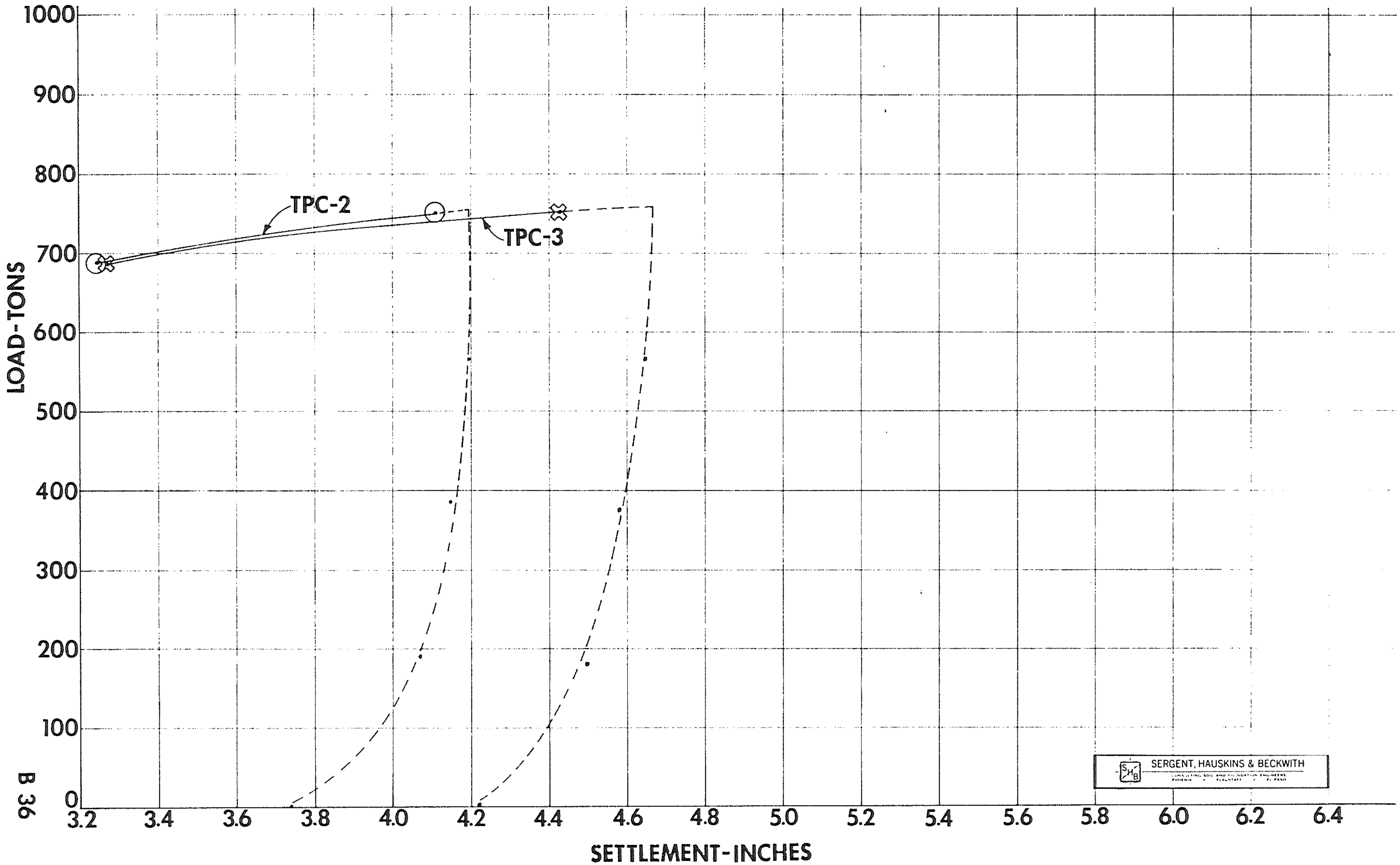
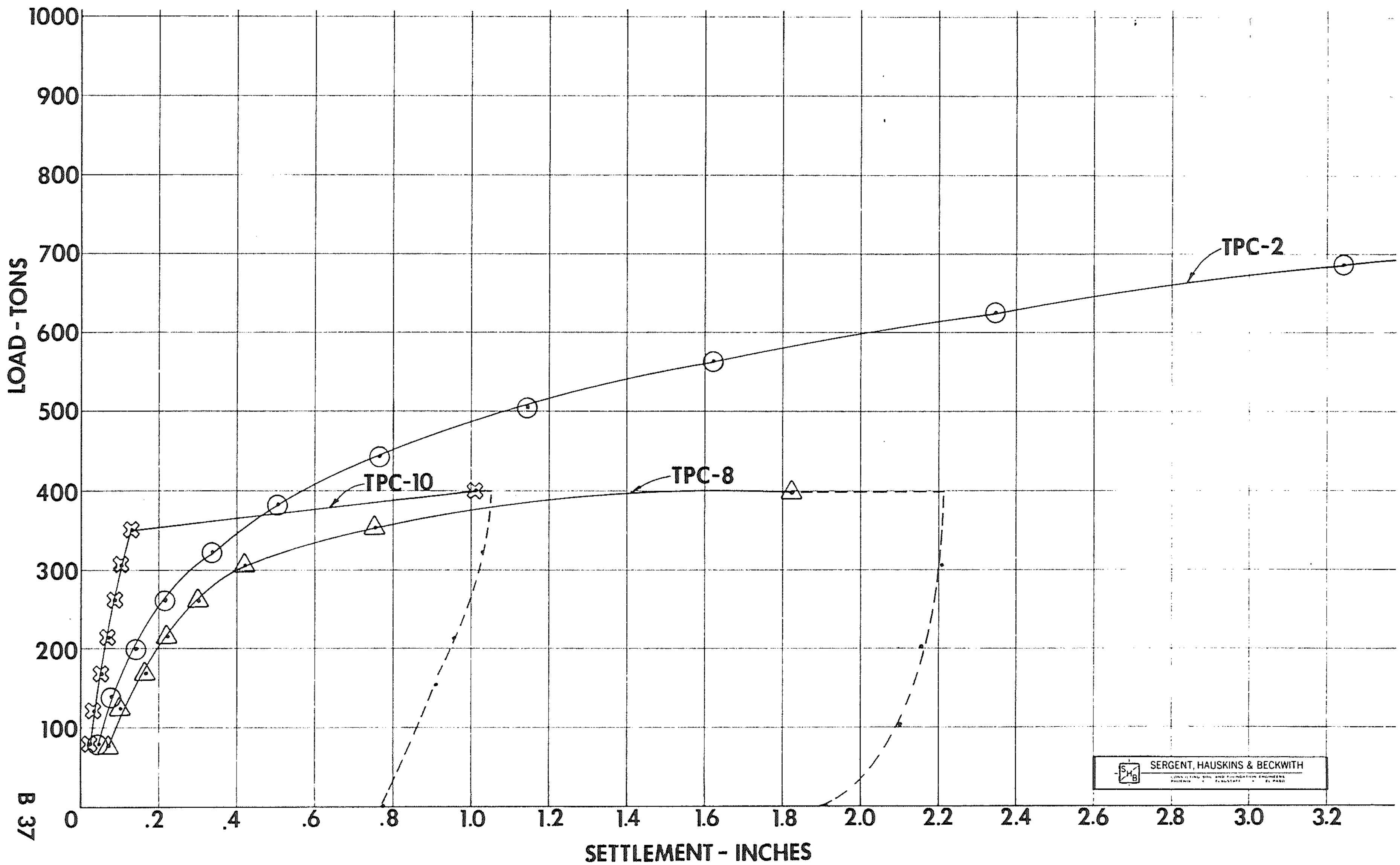


FIGURE 34 LOAD SETTLEMENT CURVES – LOAD TESTS – SITE C



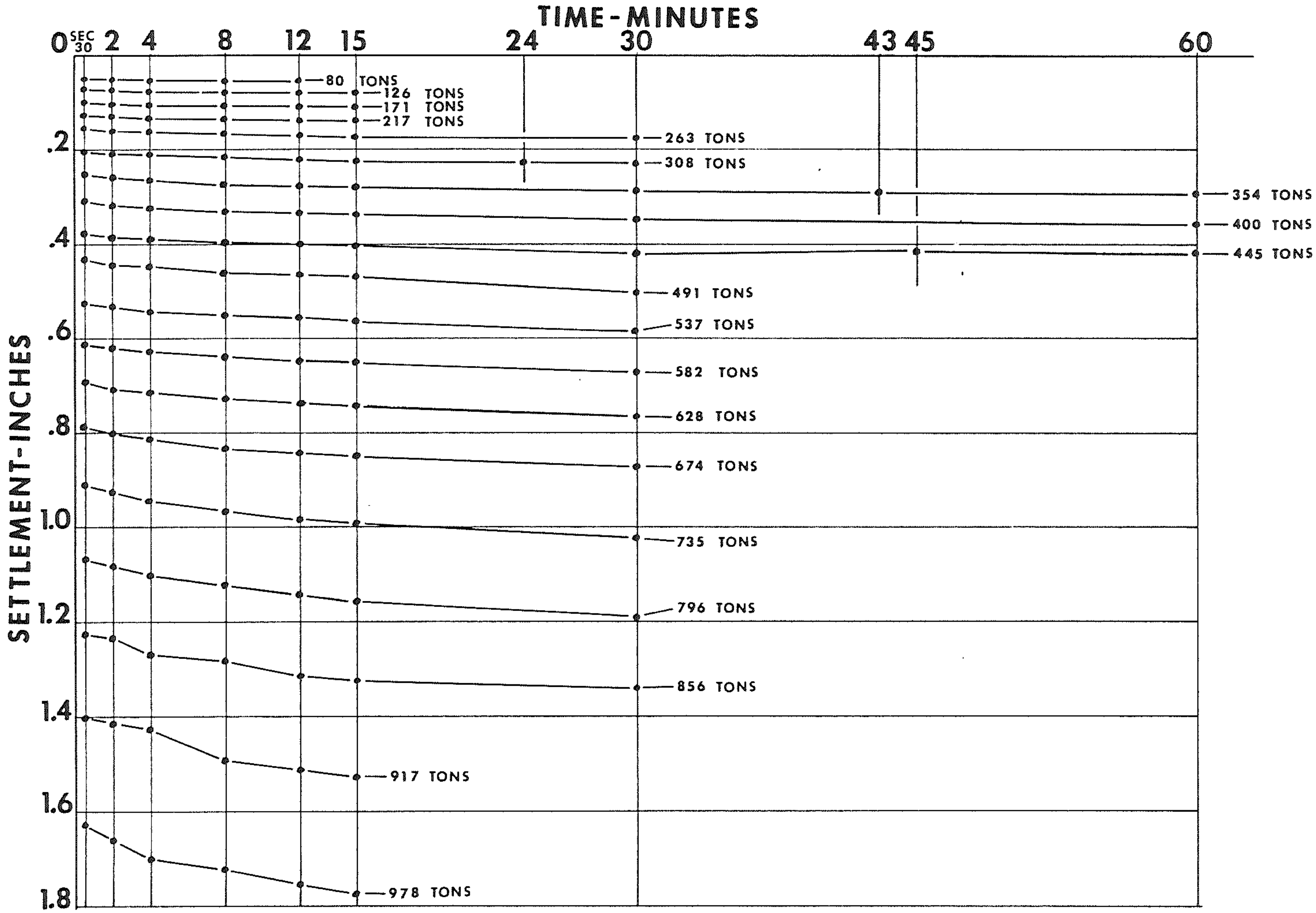
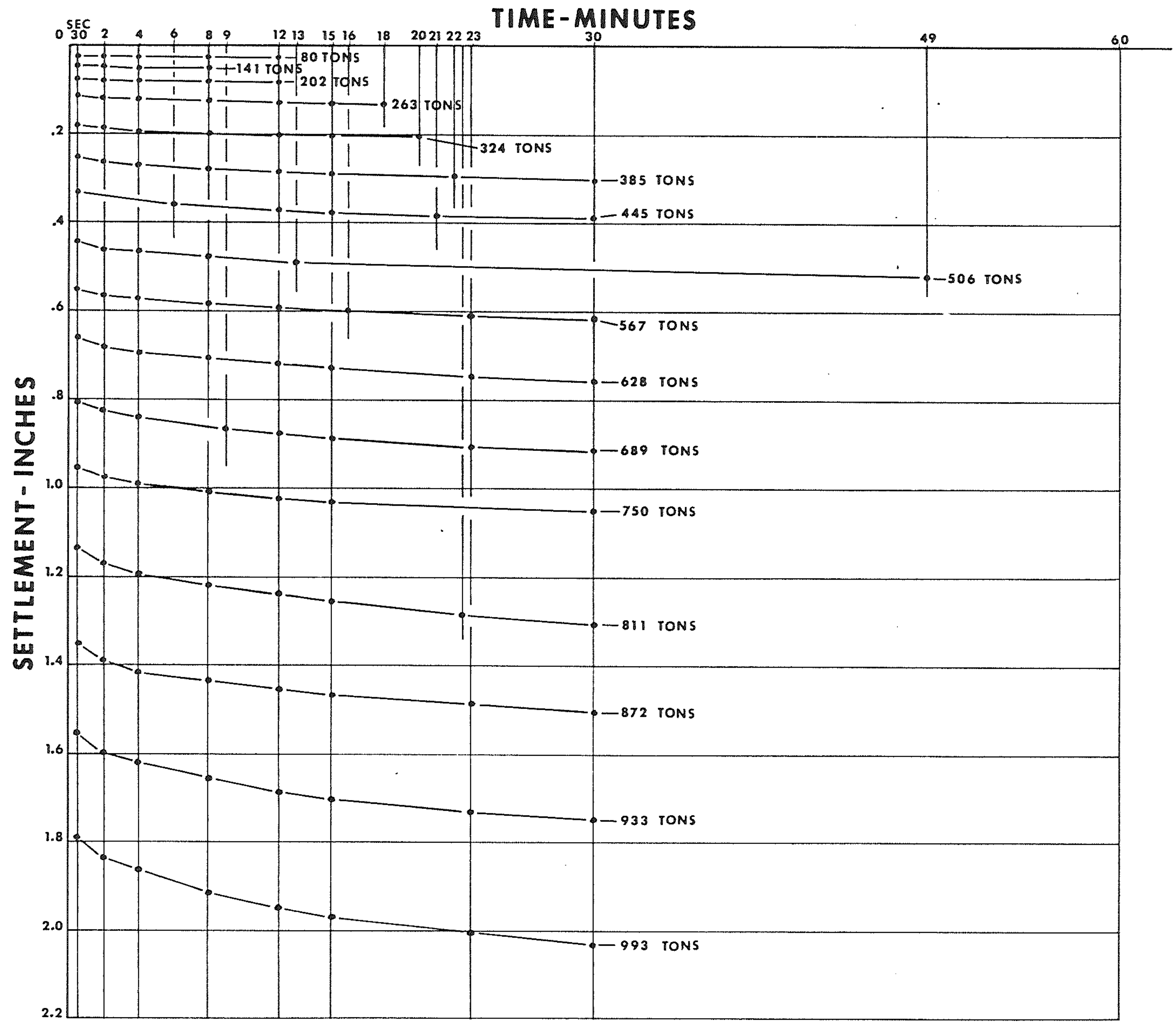


FIGURE 35 TIME SETTLEMENT CURVES

TPA—1

SERGENT, HAUSKINS & BECKWITH



TPA—2

FIGURE 36 TIME SETTLMENT CURVES

SERGENT, HAUSKINS & BECKWITH

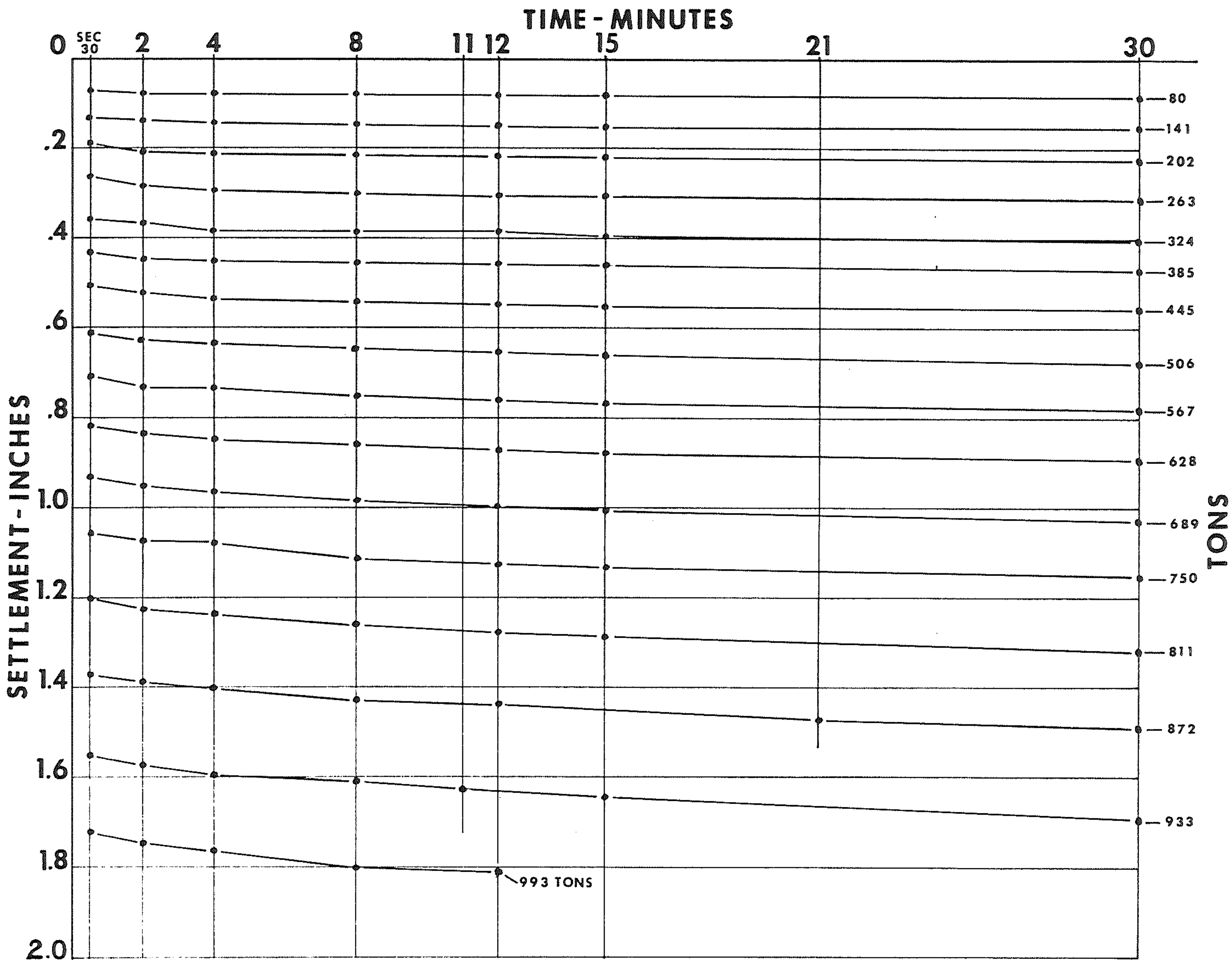


FIGURE 37 TIME SETTLEMENT CURVES

TPA—3

SERGENT, HAUSKINS & BECKWITH
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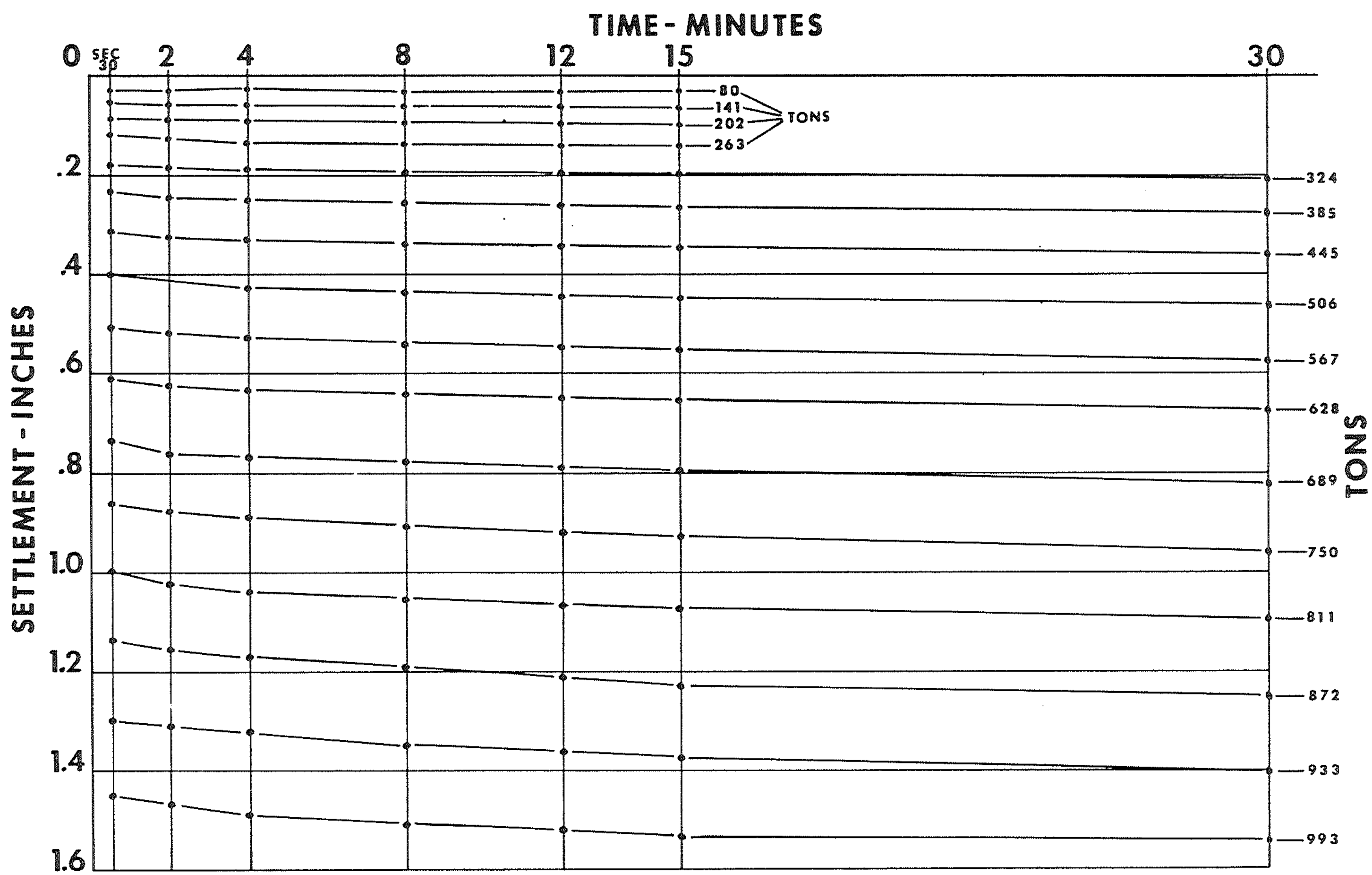

FIGURE 38 TIME SETTLEMENT CURVES

TPA—4

SERGENT, HAUSKINS & BECKWITH

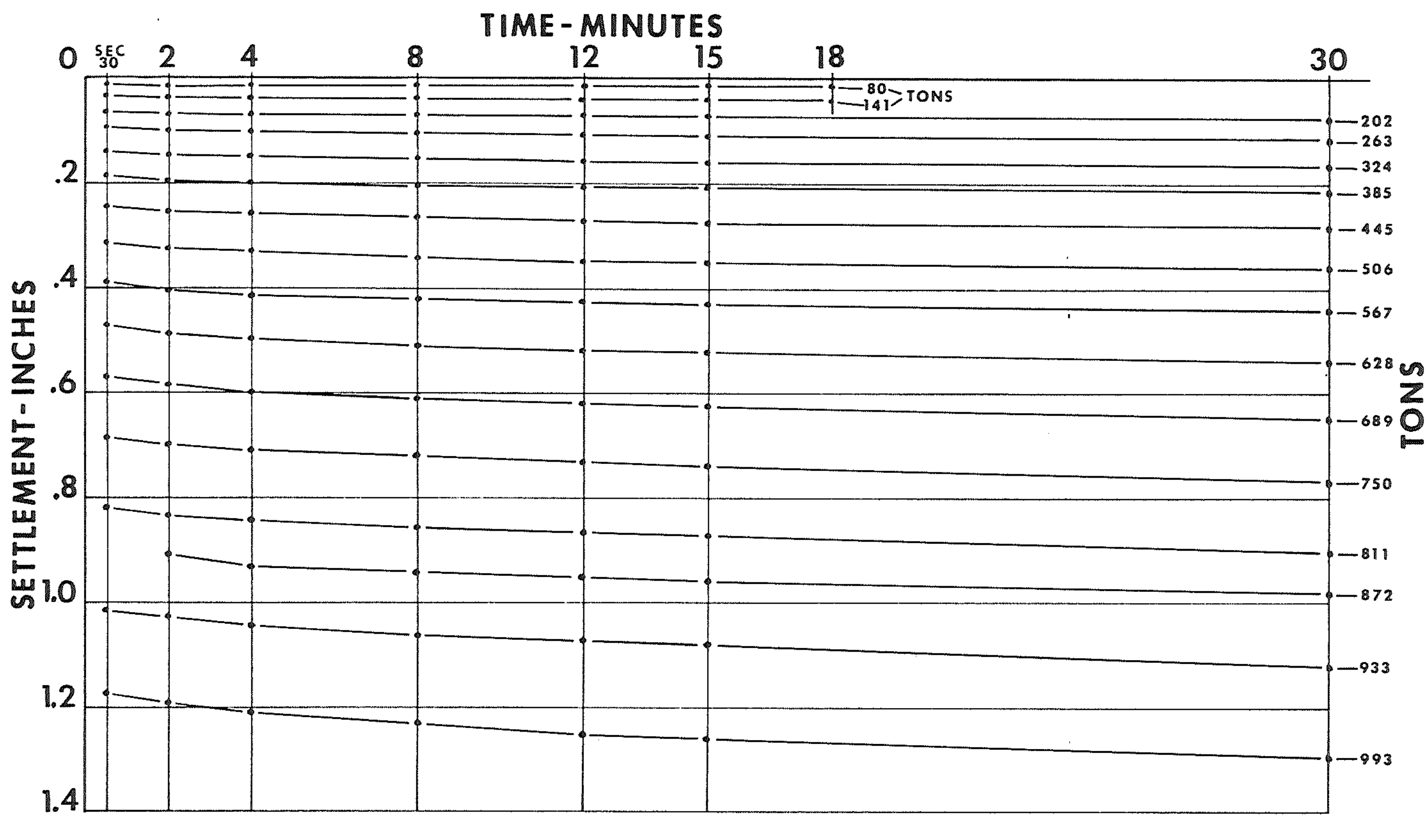

FIGURE 39 TIME SETTLEMENT CURVES

TPA—5

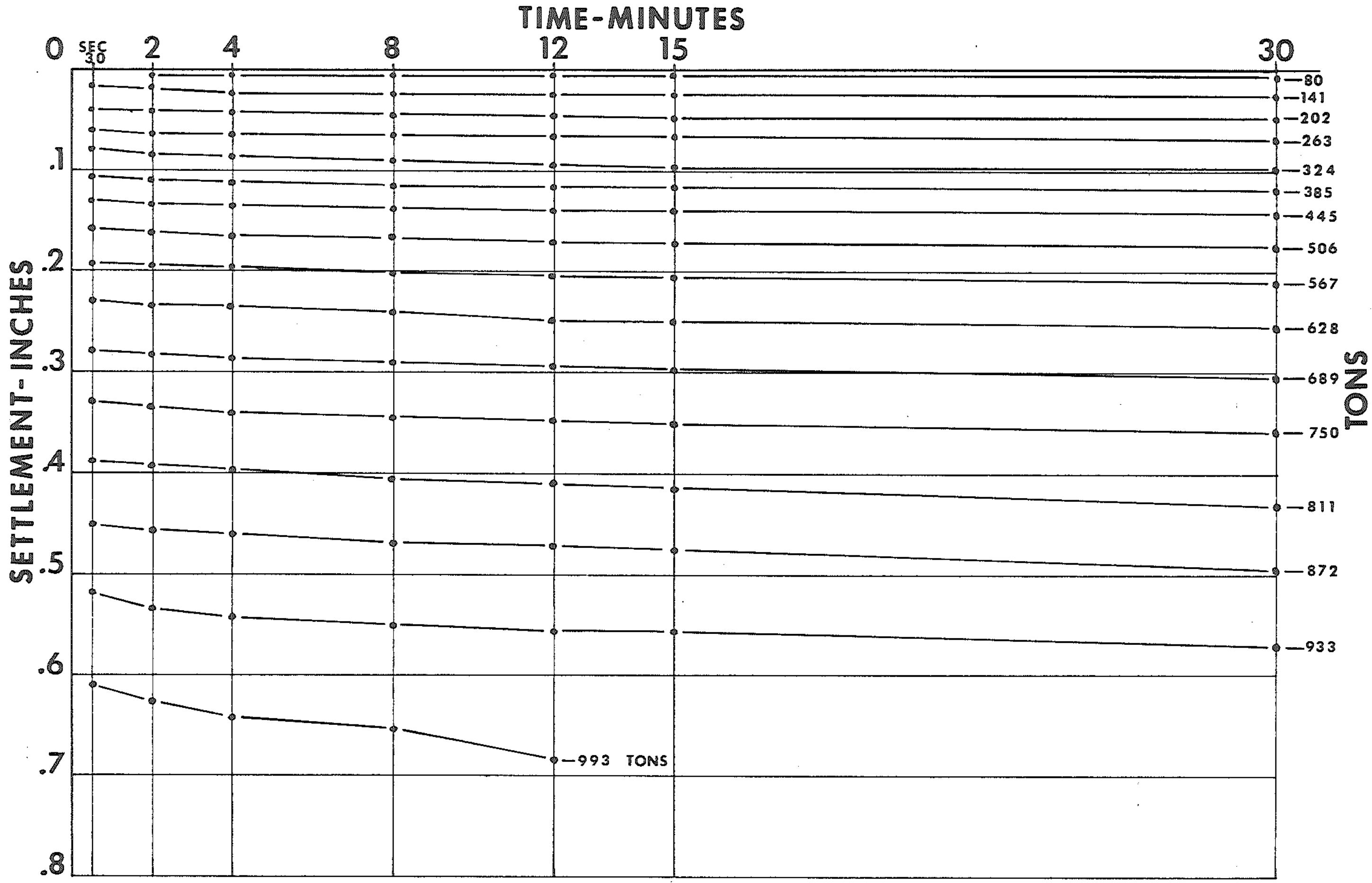

FIGURE 40 TIME SETTLEMENT CURVES

TPA—6

SERGENT, HAUSKINS & BECKWITH

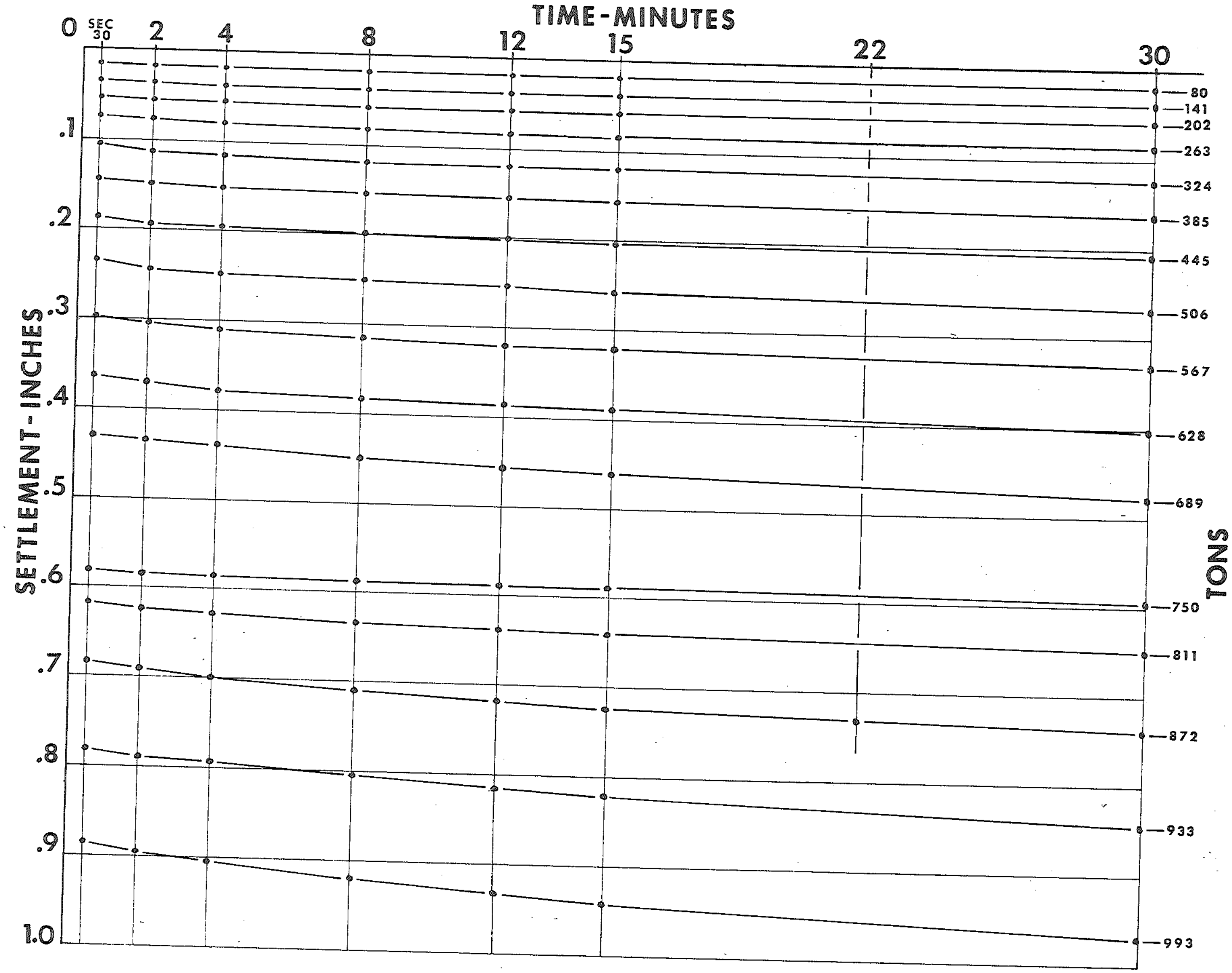

FIGURE 41 TIME SETTLEMENT CURVES

TPA—7

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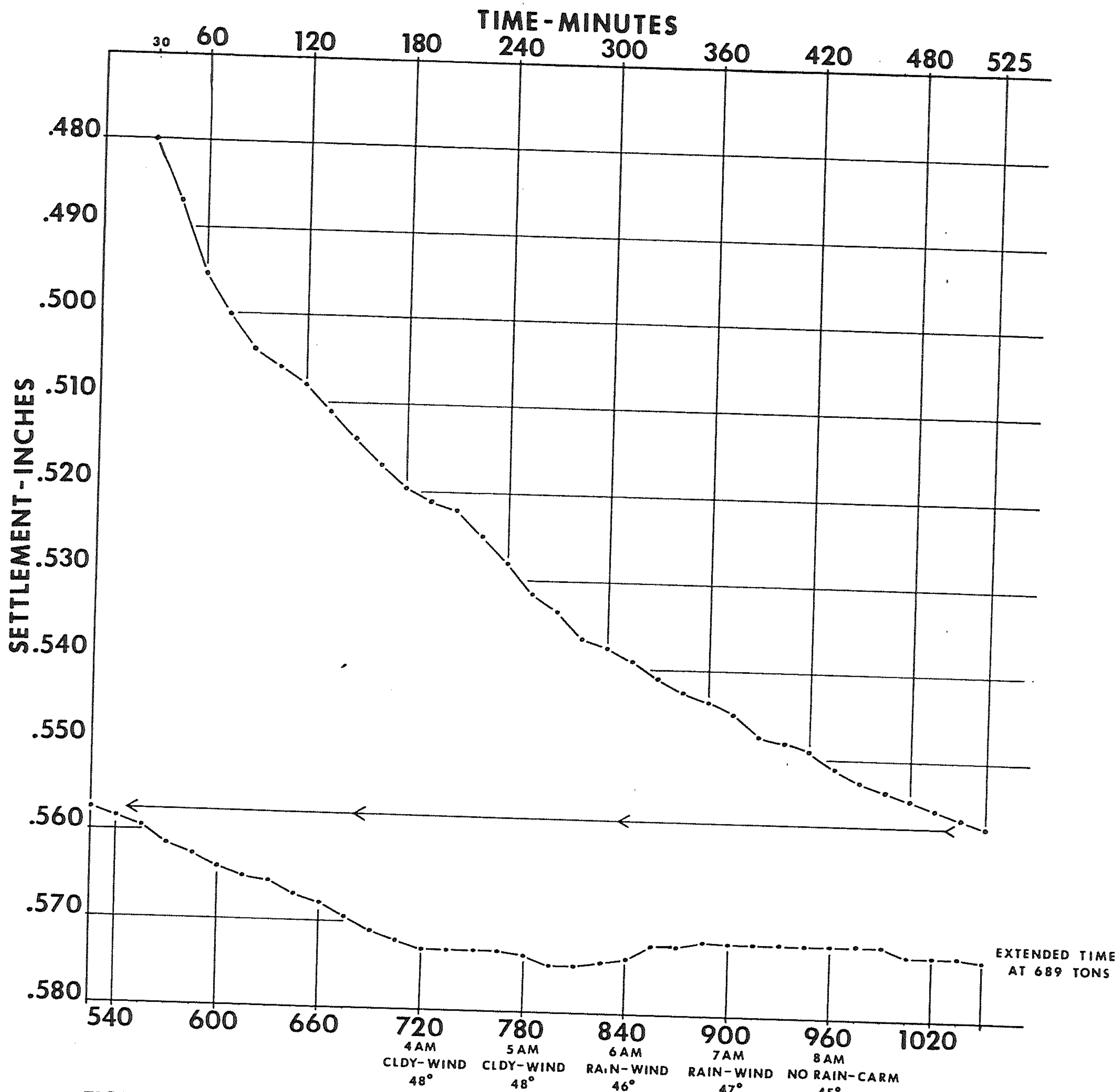


FIGURE 42 EXTENDED TIME SETTLEMENT CURVES 689 TONS

TPA—7

SERGENT, HAUSKINS & BECKWITH

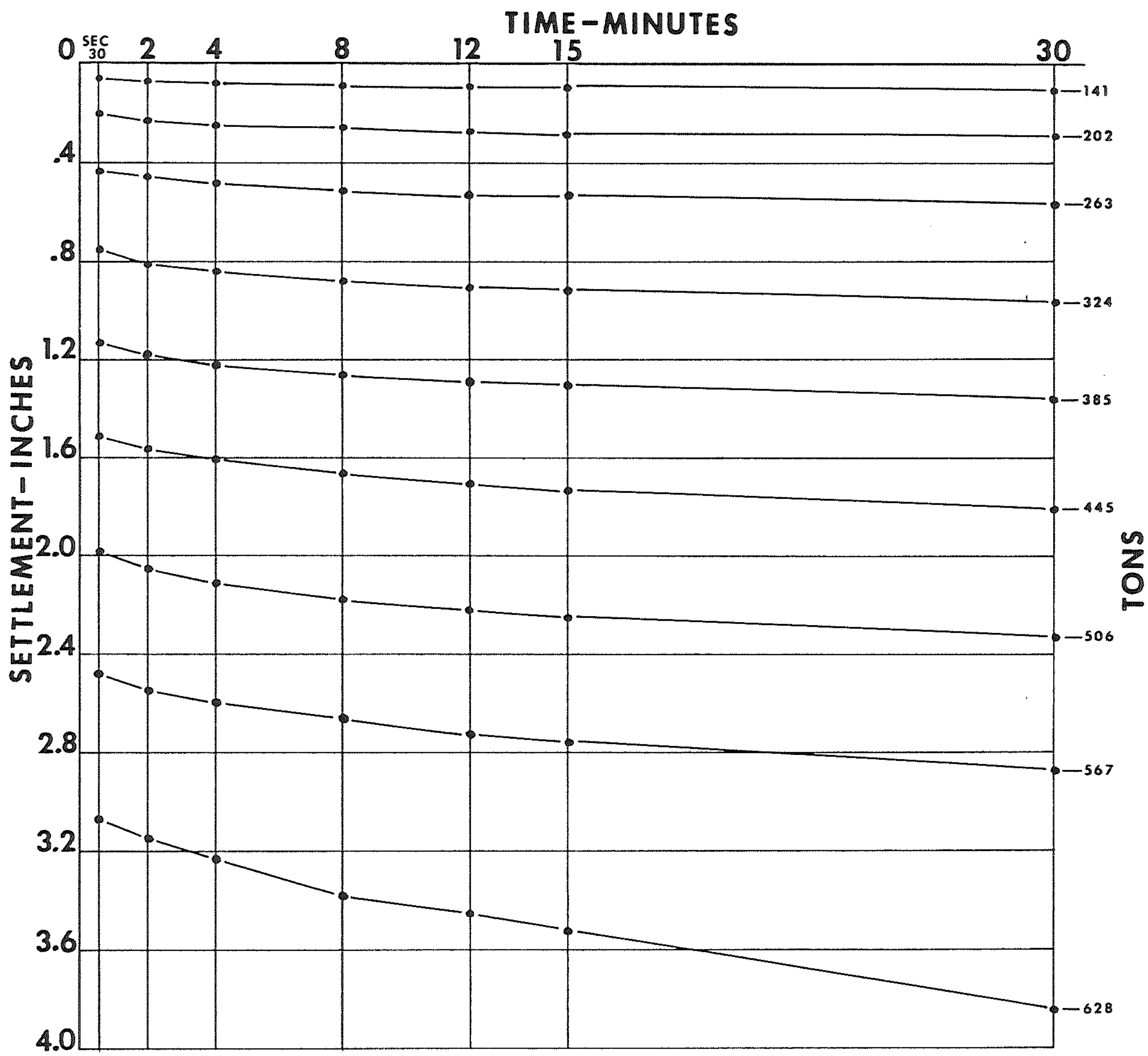
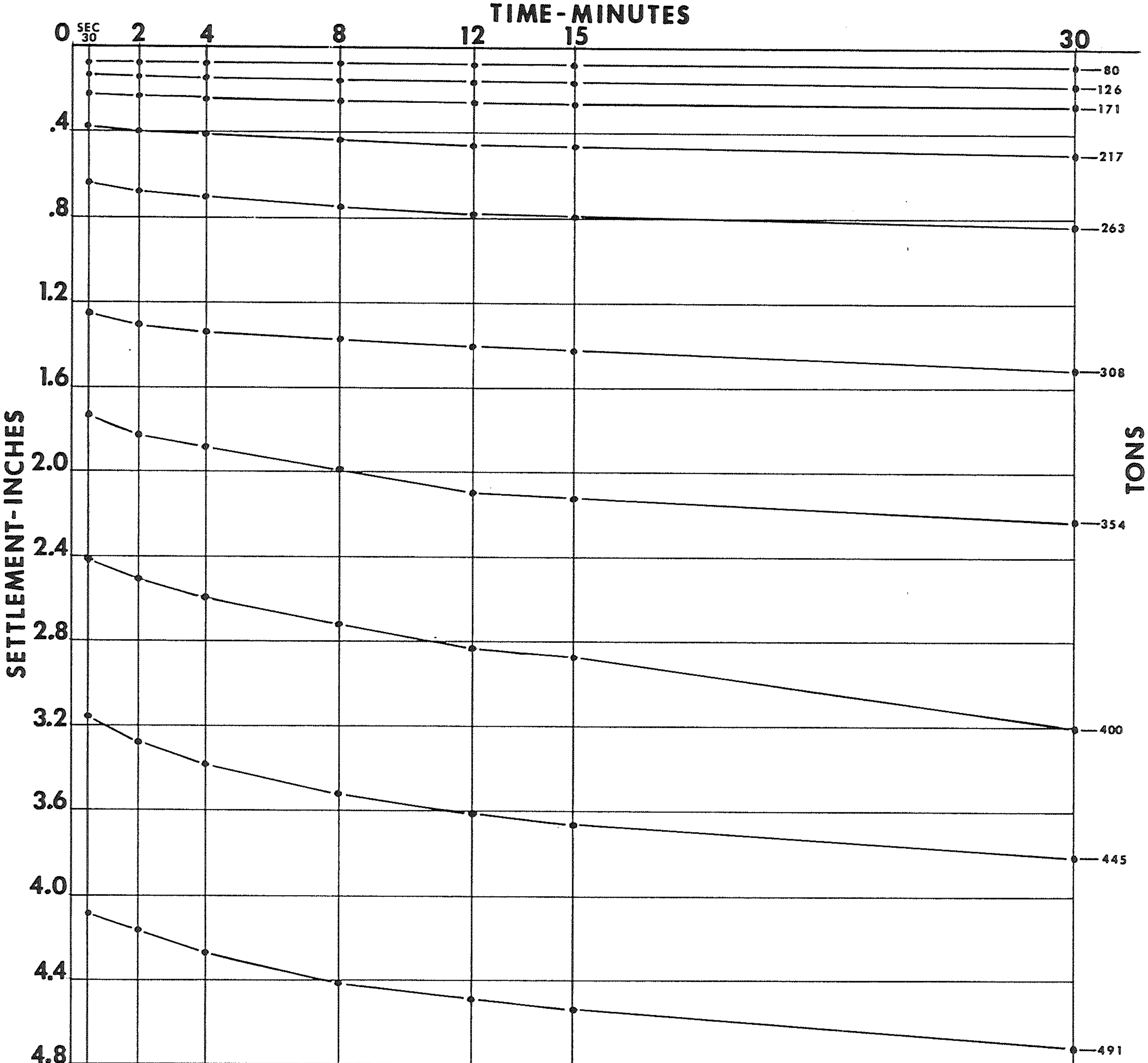


FIGURE 43 TIME SETTLEMENT CURVES

TPB—1

SERGENT, HAUSKINS & BECKWITH



TPB—2

FIGURE 44 TIME SETTLEMENT CURVES

SERGENT, HAUSKINS & BECKWITH

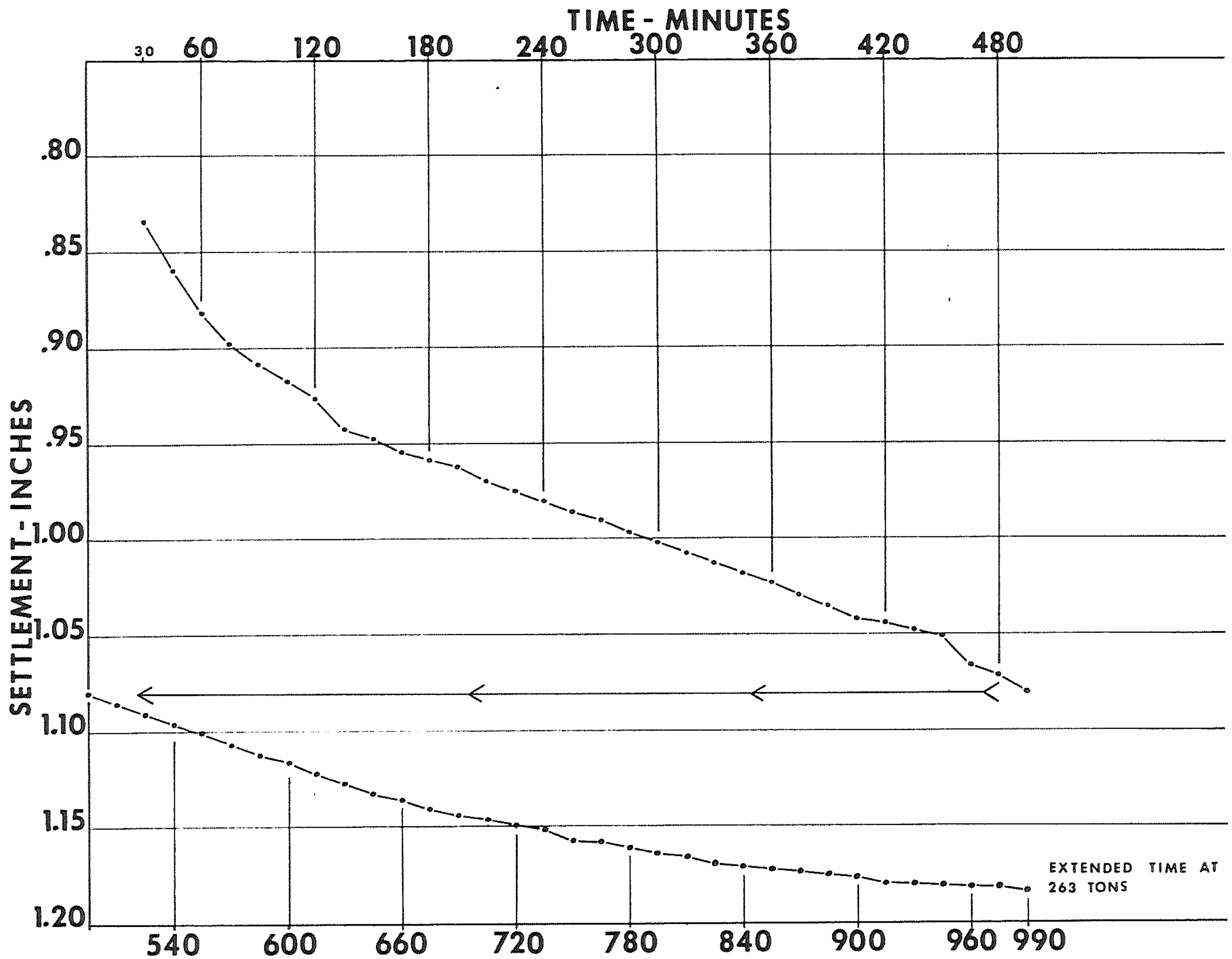
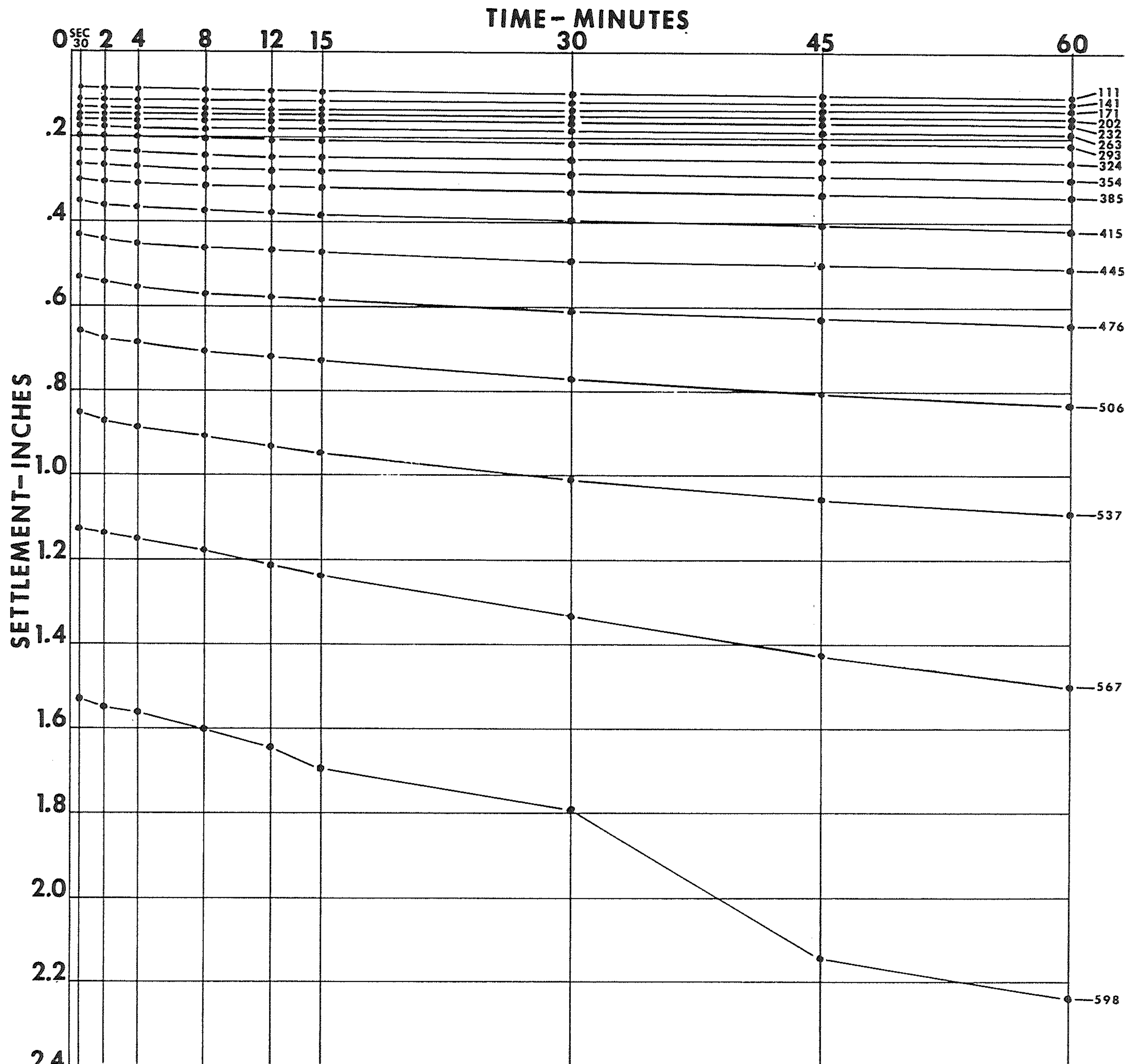


FIGURE 45 EXTENDED TIME SETTLEMENT CURVES

TPB—2

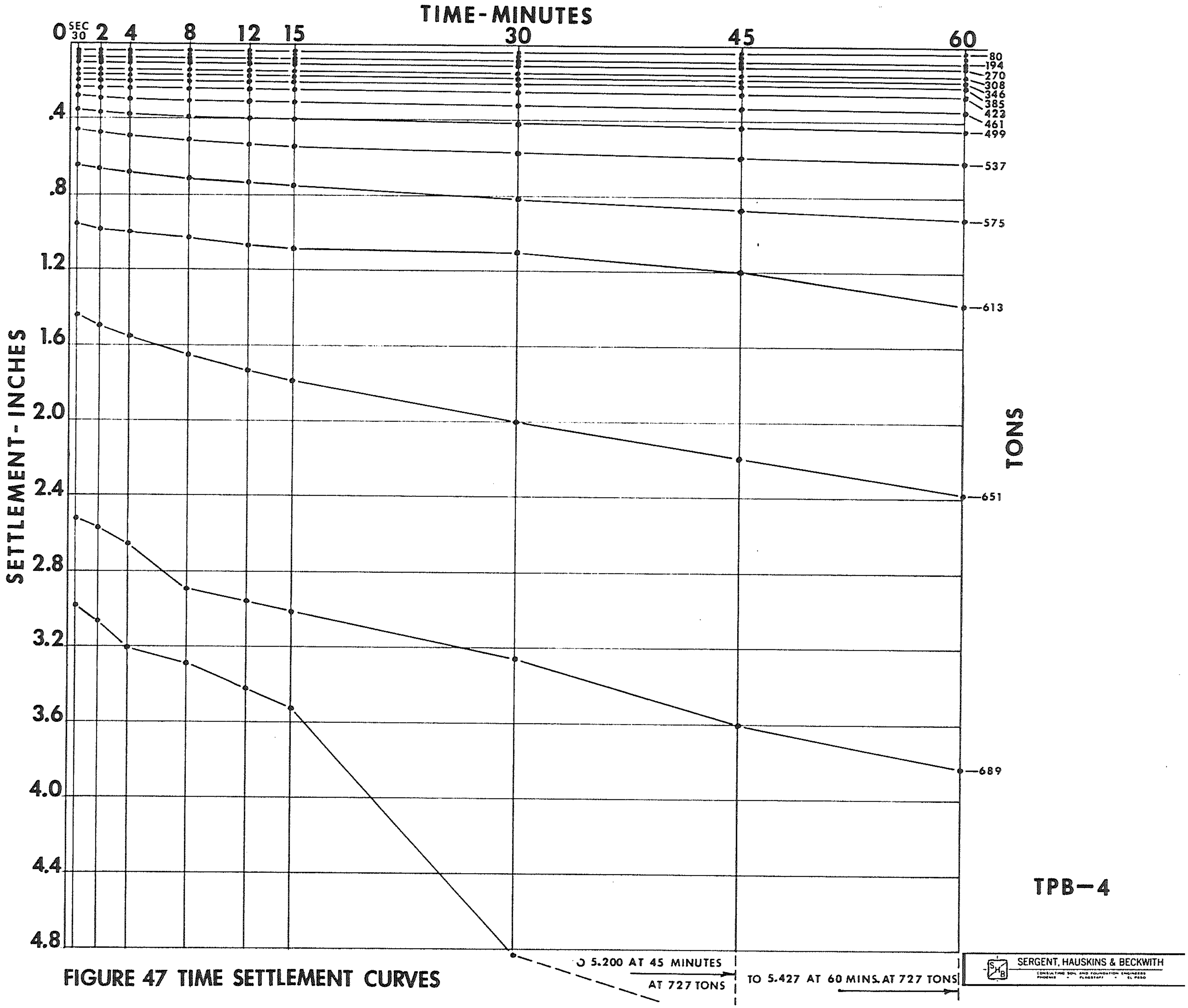
SERGENT, HAUSKINS & BECKWITH



TPB—3

FIGURE 46 TIME SETTLEMENT CURVES

SERGENT, HAUSKINS & BECKWITH



TPB—4

FIGURE 47 TIME SETTLEMENT CURVES

SERGENT, HAUSKINS & BECKWITH

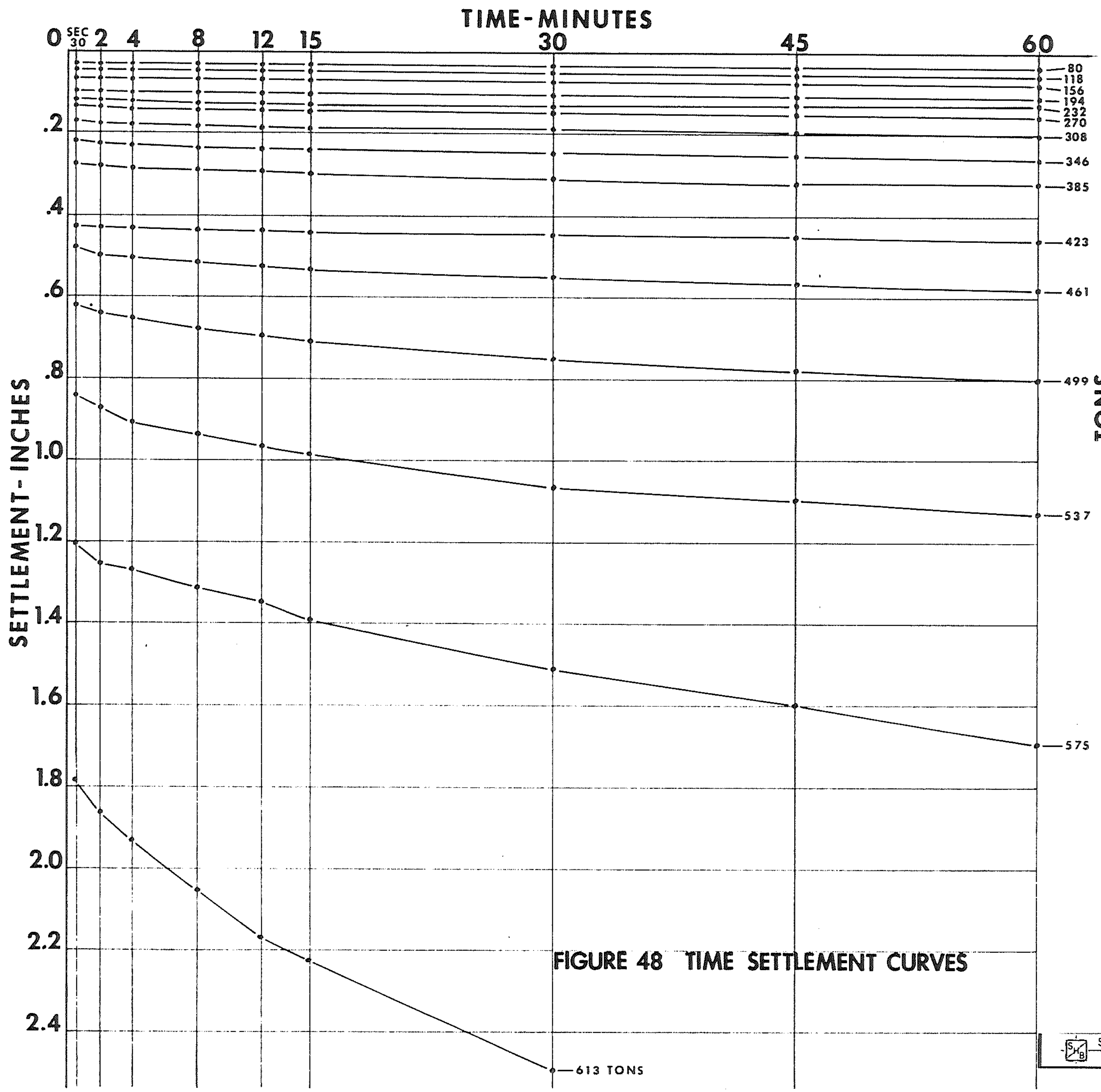


FIGURE 48 TIME SETTLEMENT CURVES

TPB—5

SERGENT, HAUSKINS & BECKWITH

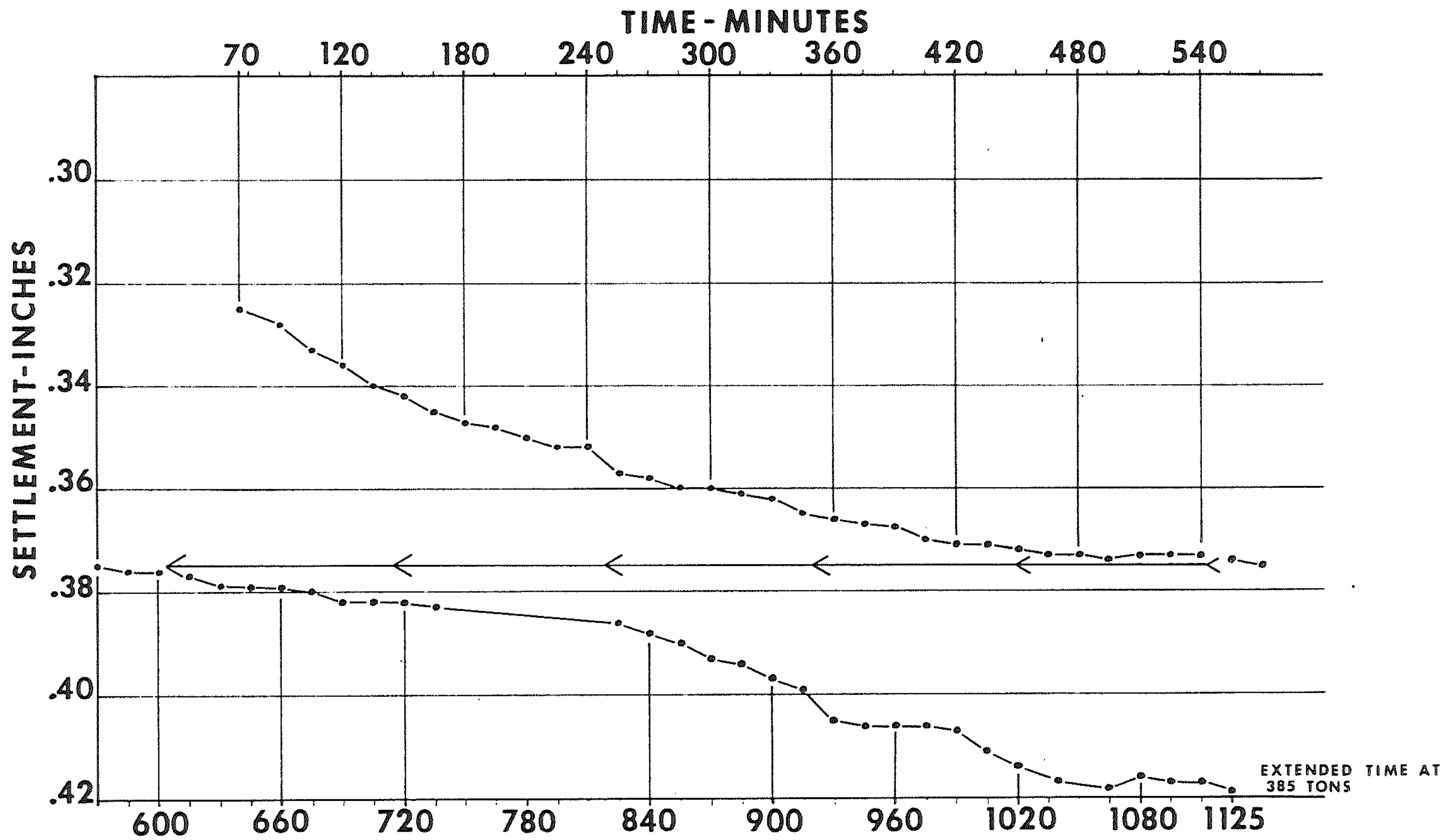


FIGURE 49 EXTENDED TIME SETTLEMENT CURVES

TPB—5

SERGENT, HAUSKINS & BECKWITH

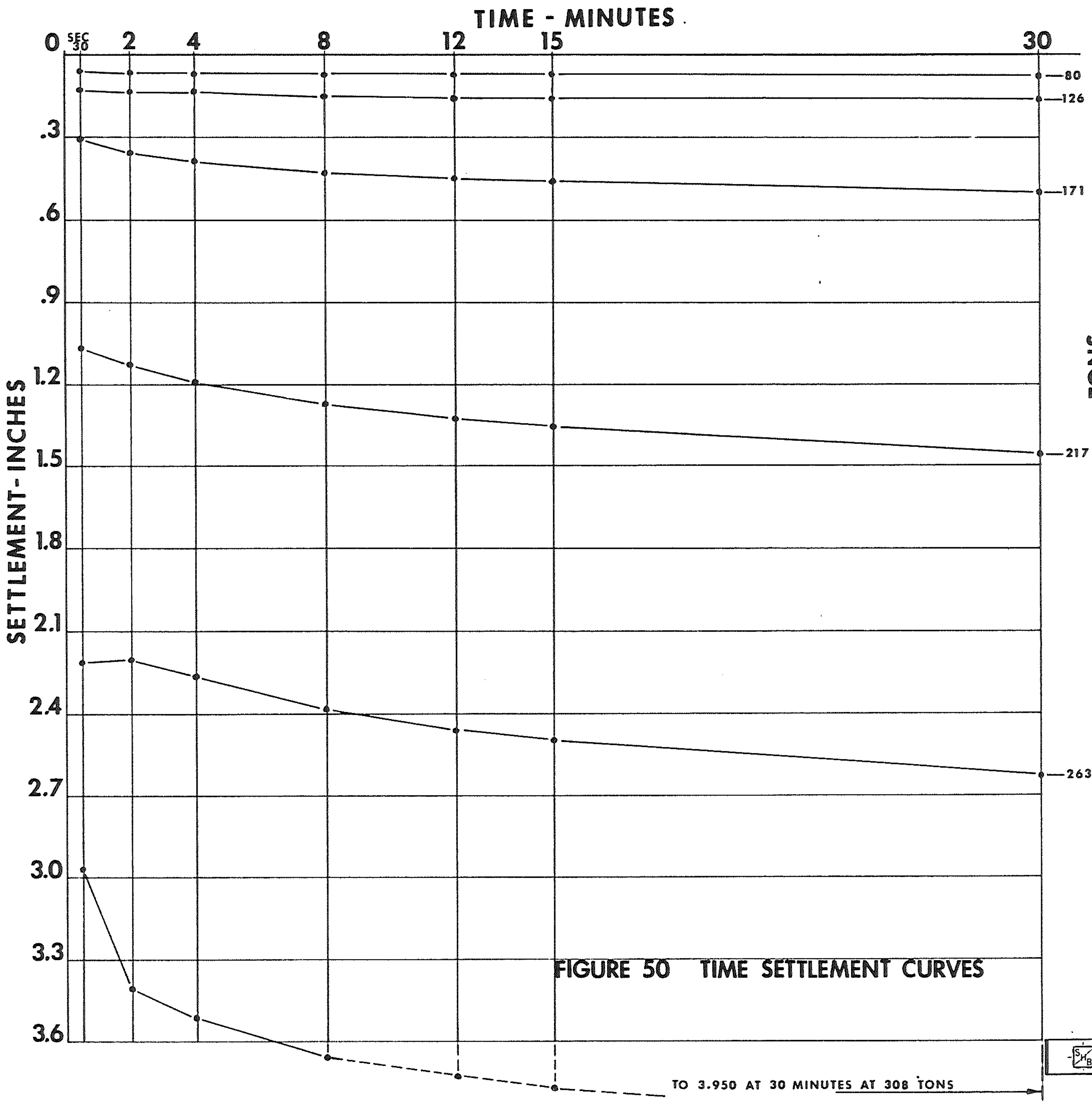


FIGURE 50 TIME SETTLEMENT CURVES

TPB—6

SERGENT, HAUSKINS & BECKWITH

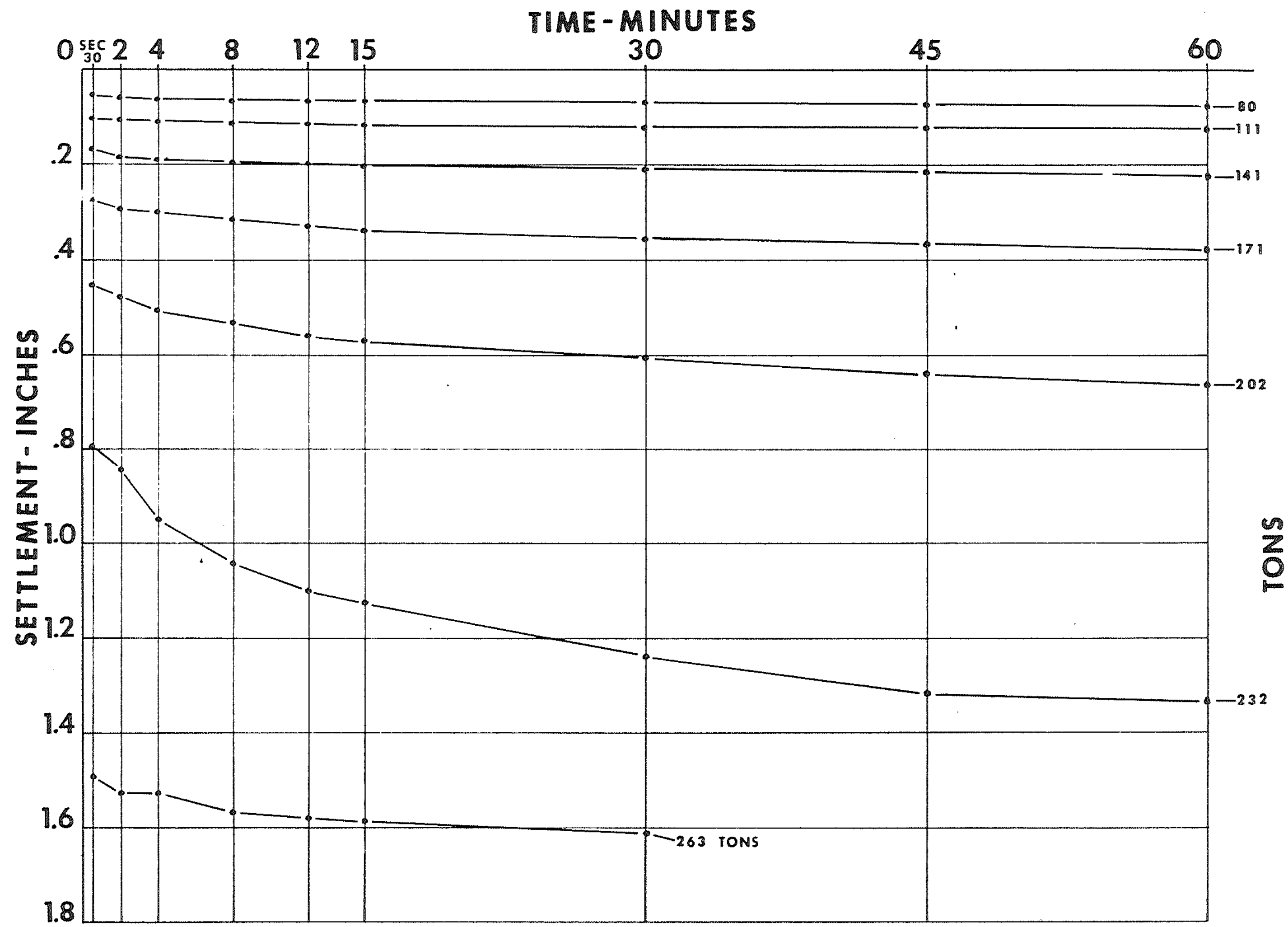


FIGURE 51 TIME SETTLEMENT CURVES

TPB—7

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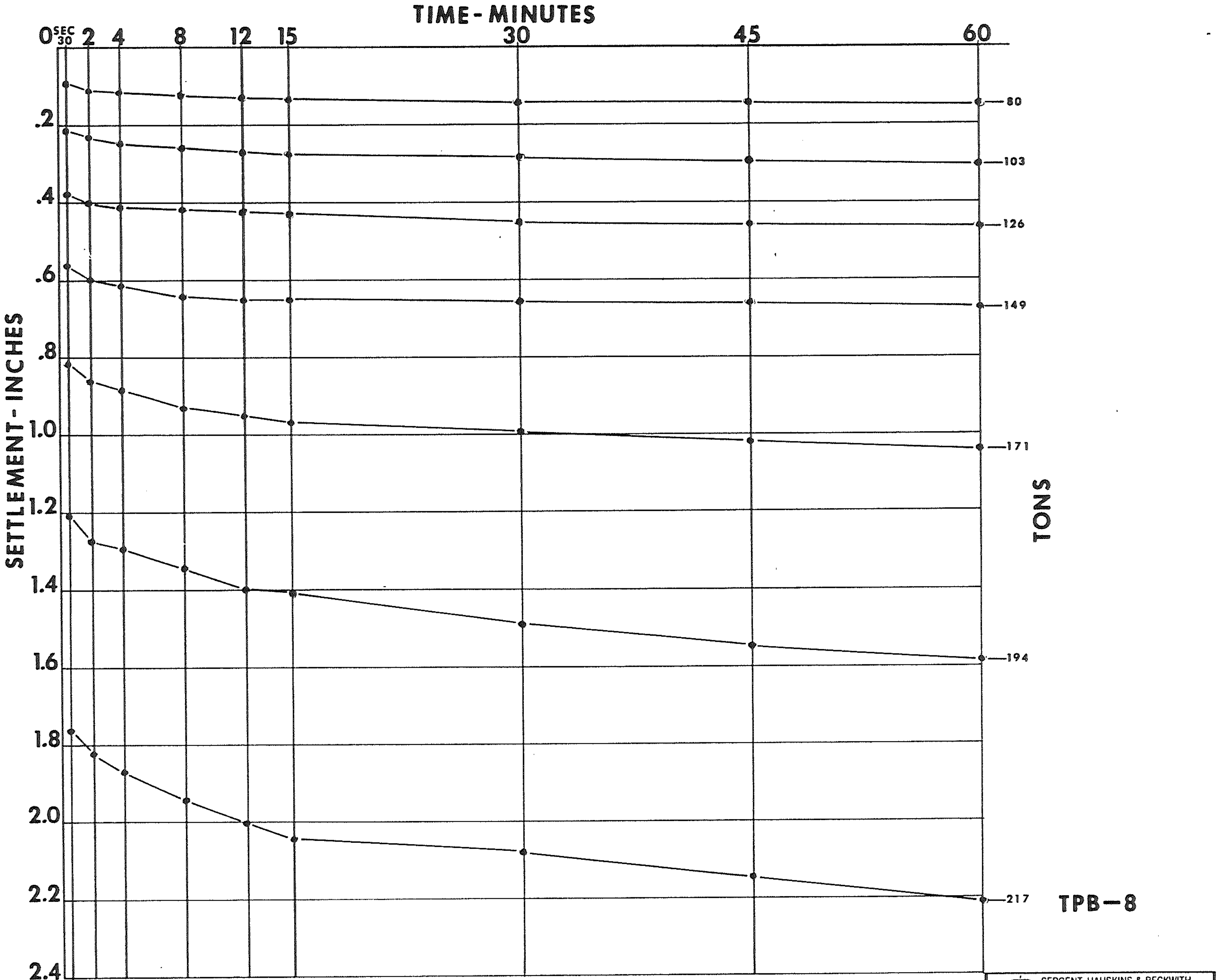


FIGURE 52 TIME SETTLEMENT CURVES

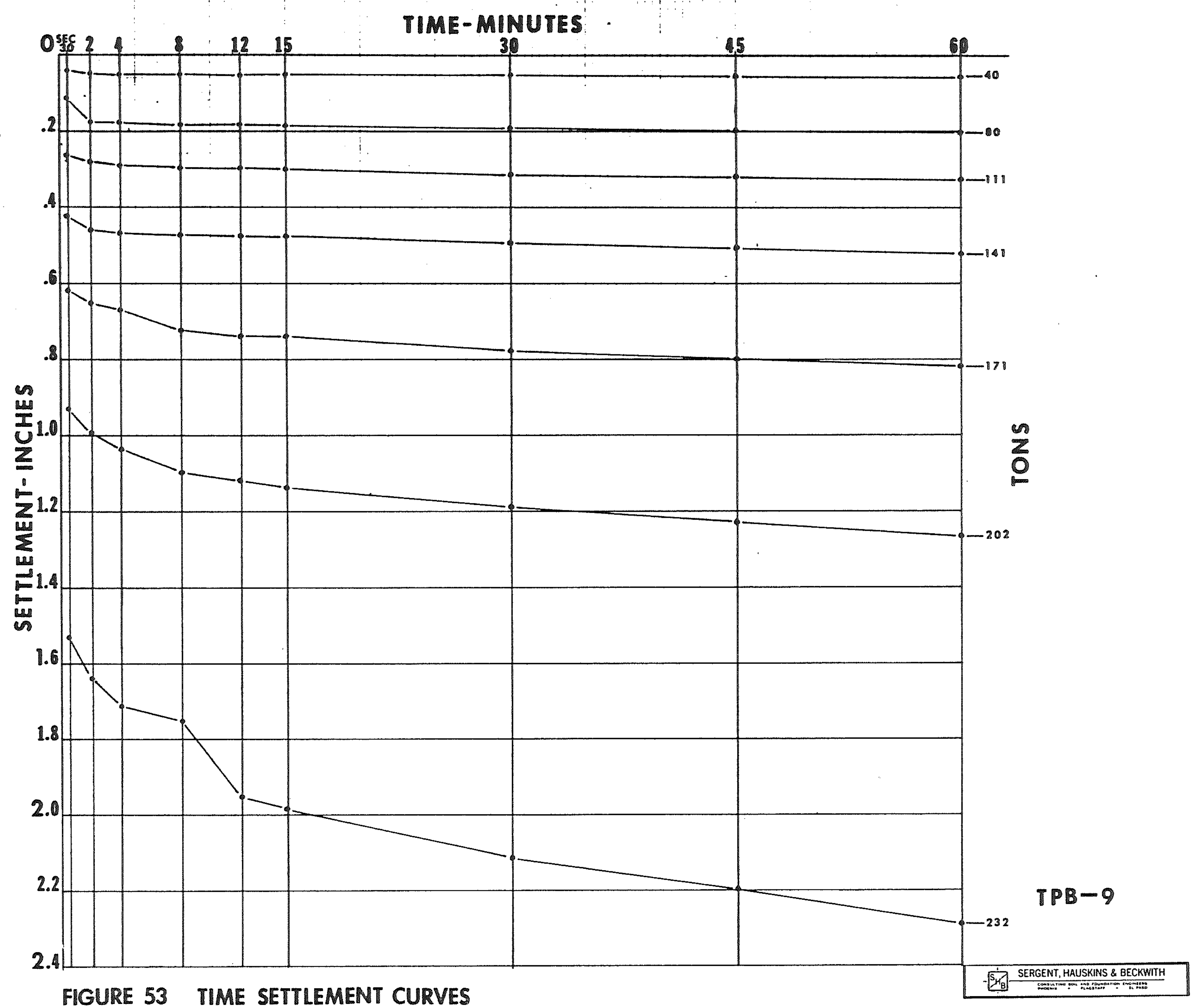


FIGURE 53 TIME SETTLEMENT CURVES

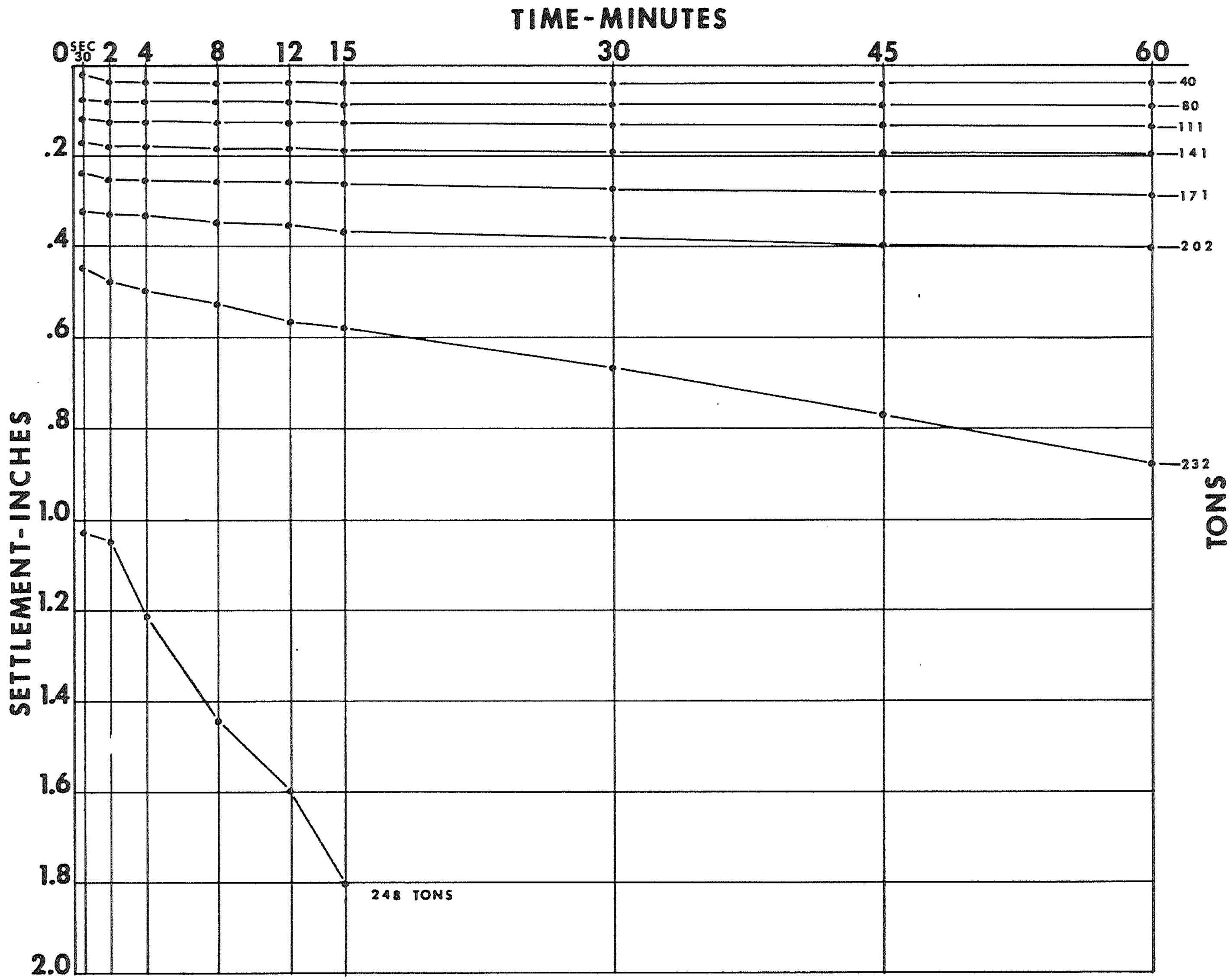


FIGURE 54 TIME SETTLEMENT CURVES

TPB—10

SERGENT, HAUSKINS & BECKWITH

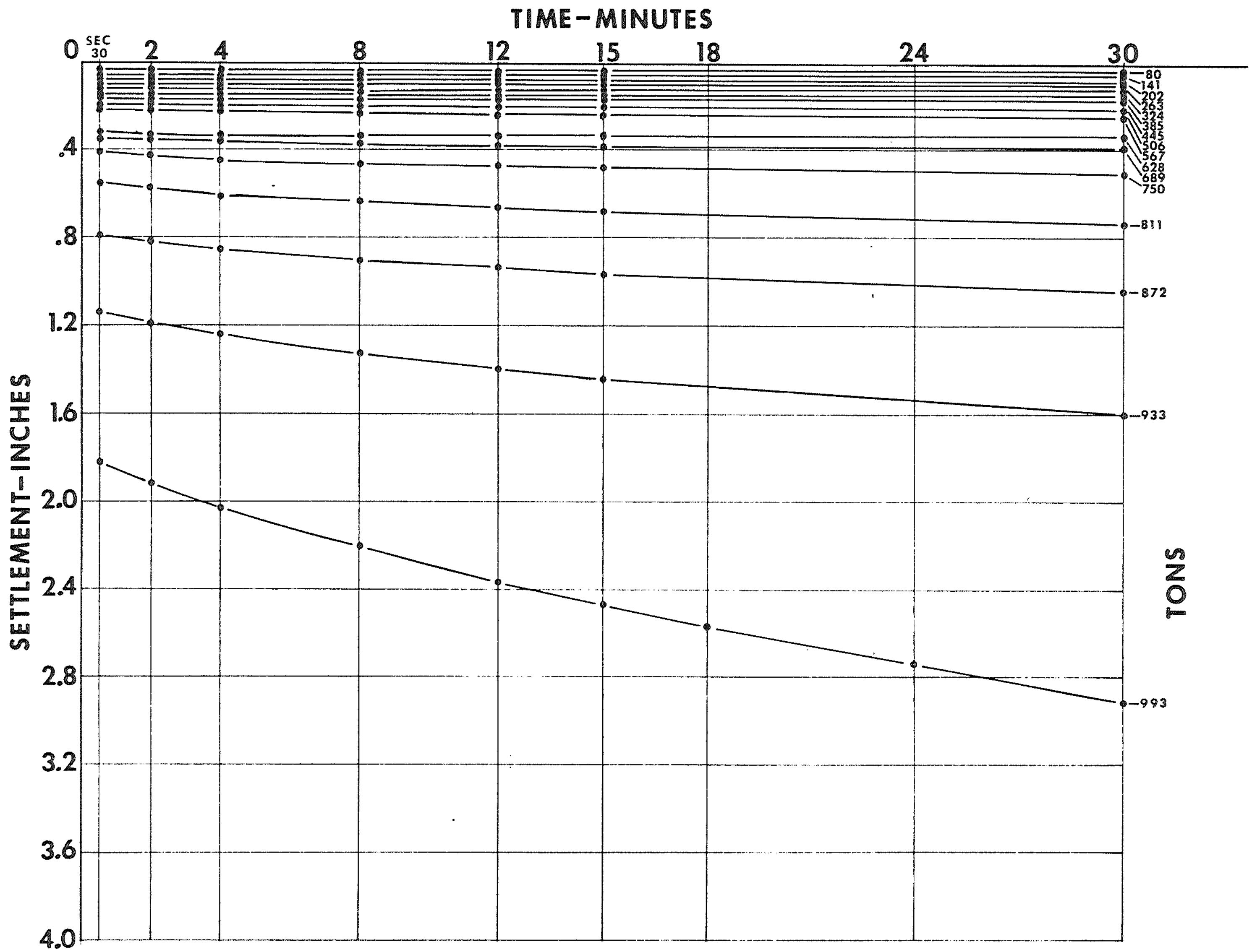


FIGURE 55 TIME SETTLEMENT CURVES

TPC—1

SERGENT, HAUSKINS & BECKWITH

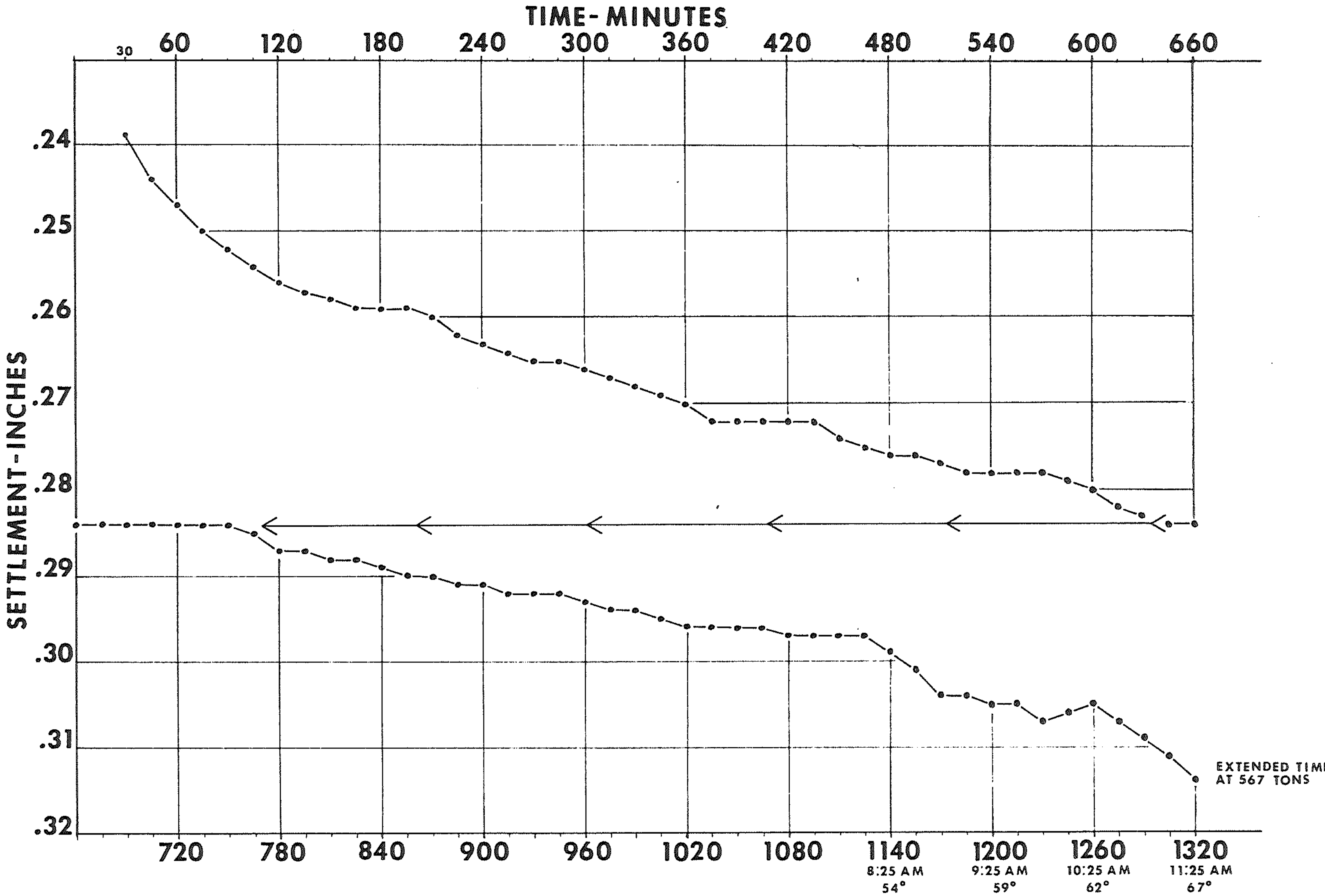


FIGURE 56 EXTENDED TIME SETTLEMENT CURVES

TPC—1

SERGENT, HAUSKINS & BECKWITH

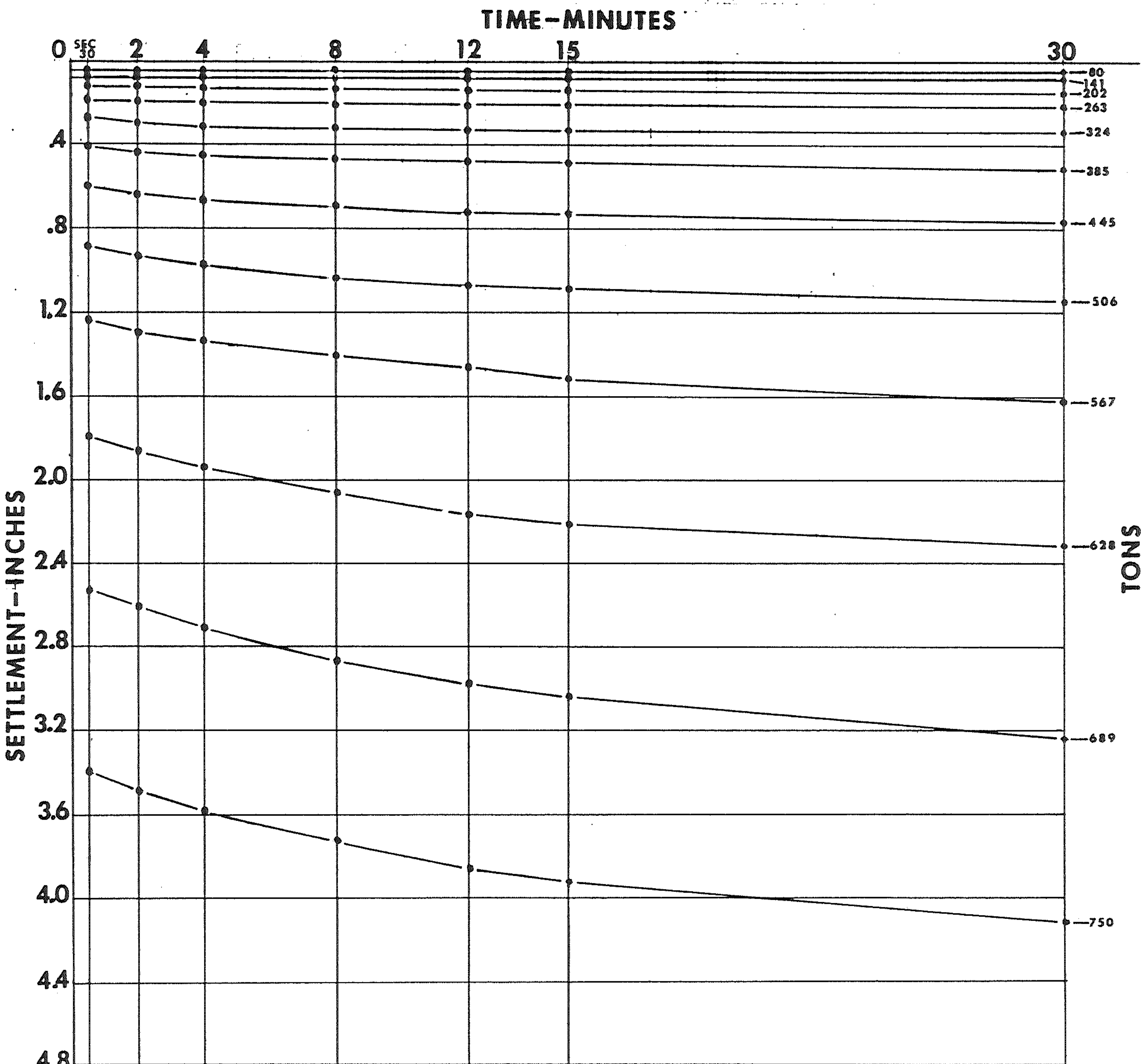


FIGURE 57 TIME SETTLEMENT CURVES

TPC—2

SERGENT, HAUSKINS & BECKWITH

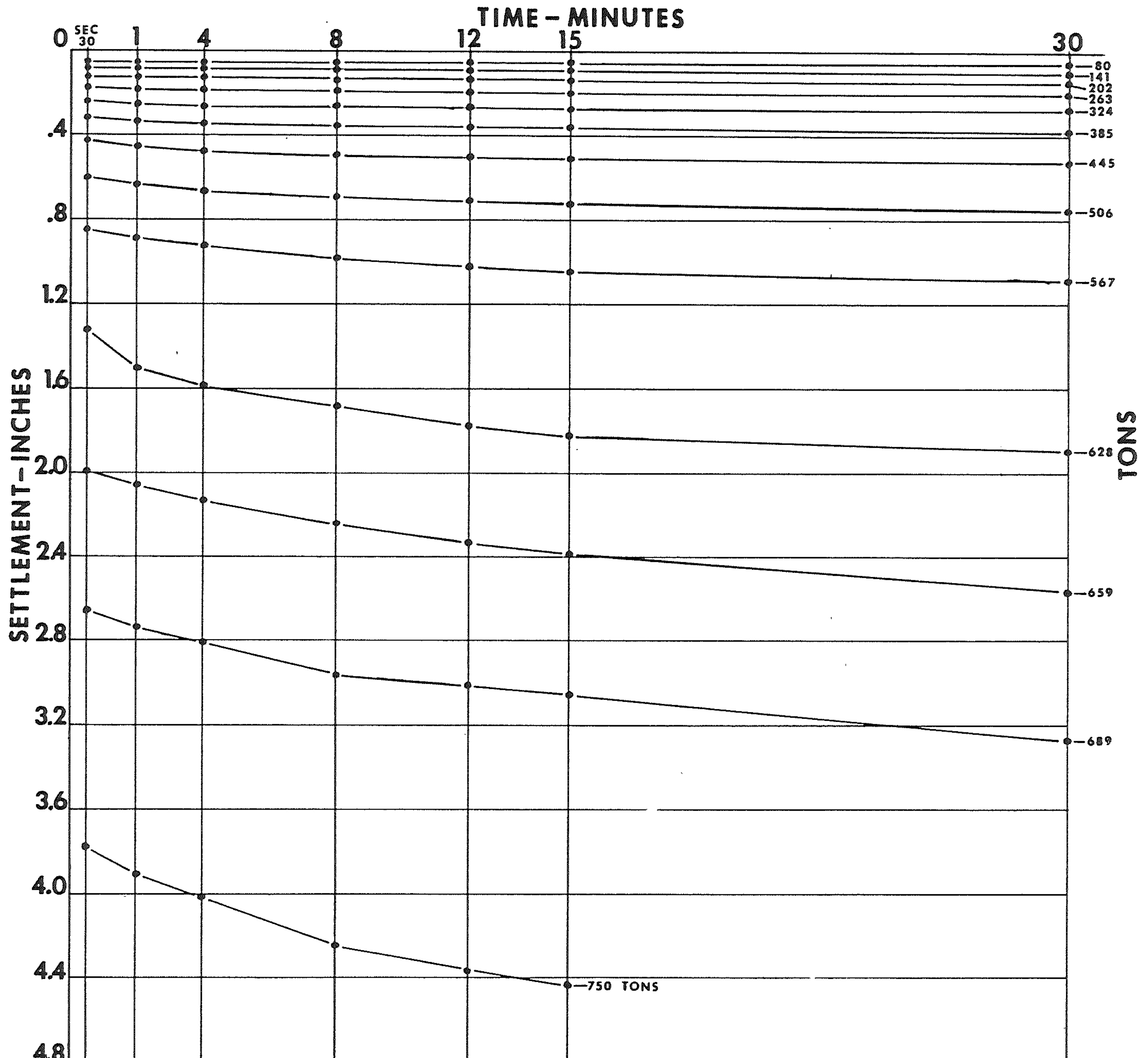


FIGURE 58 TIME SETTLEMENT CURVES

TPC—3

SERGENT, HAUSKINS & BECKWITH

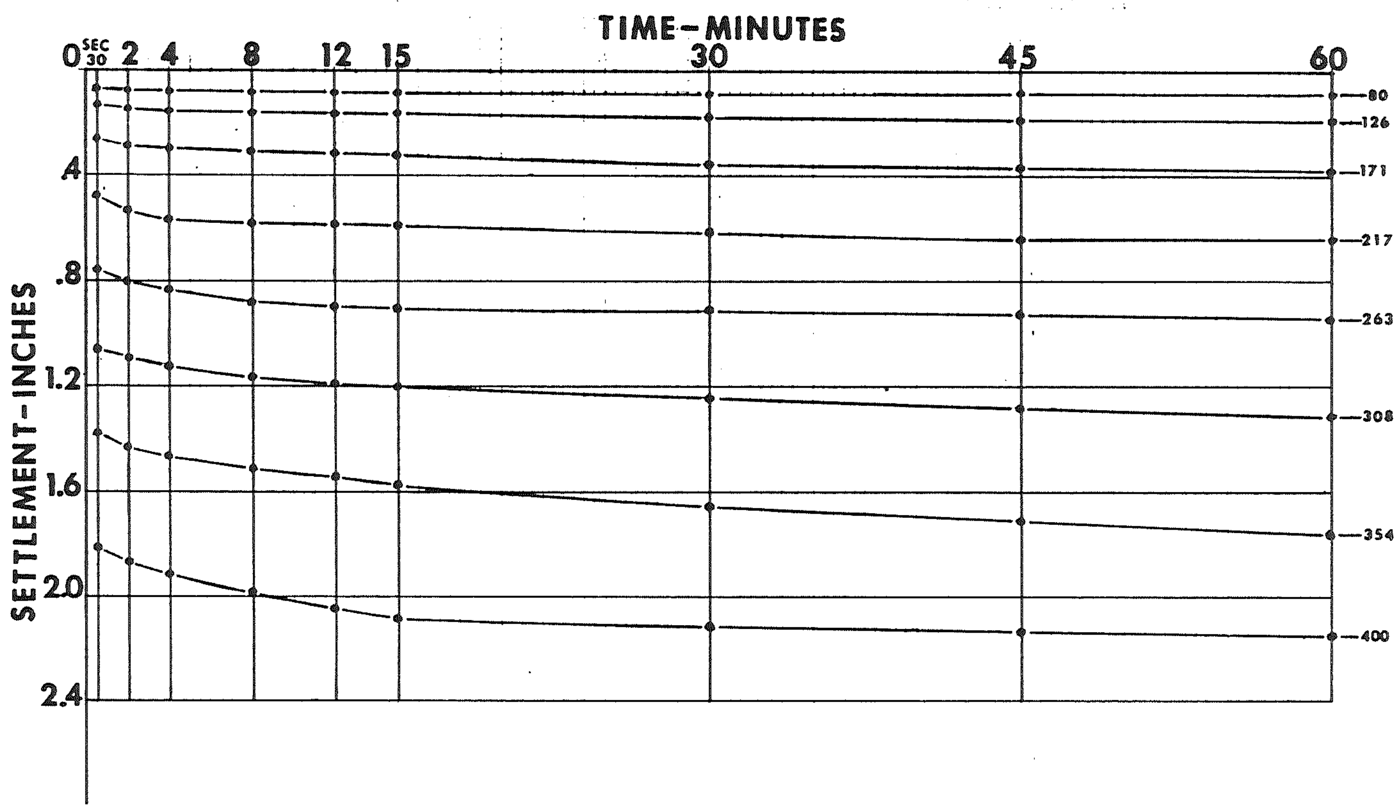


FIGURE 59 TIME SETTLEMENT CURVES

TPC—4

SERGENT, HAUSKINS & BECKWITH

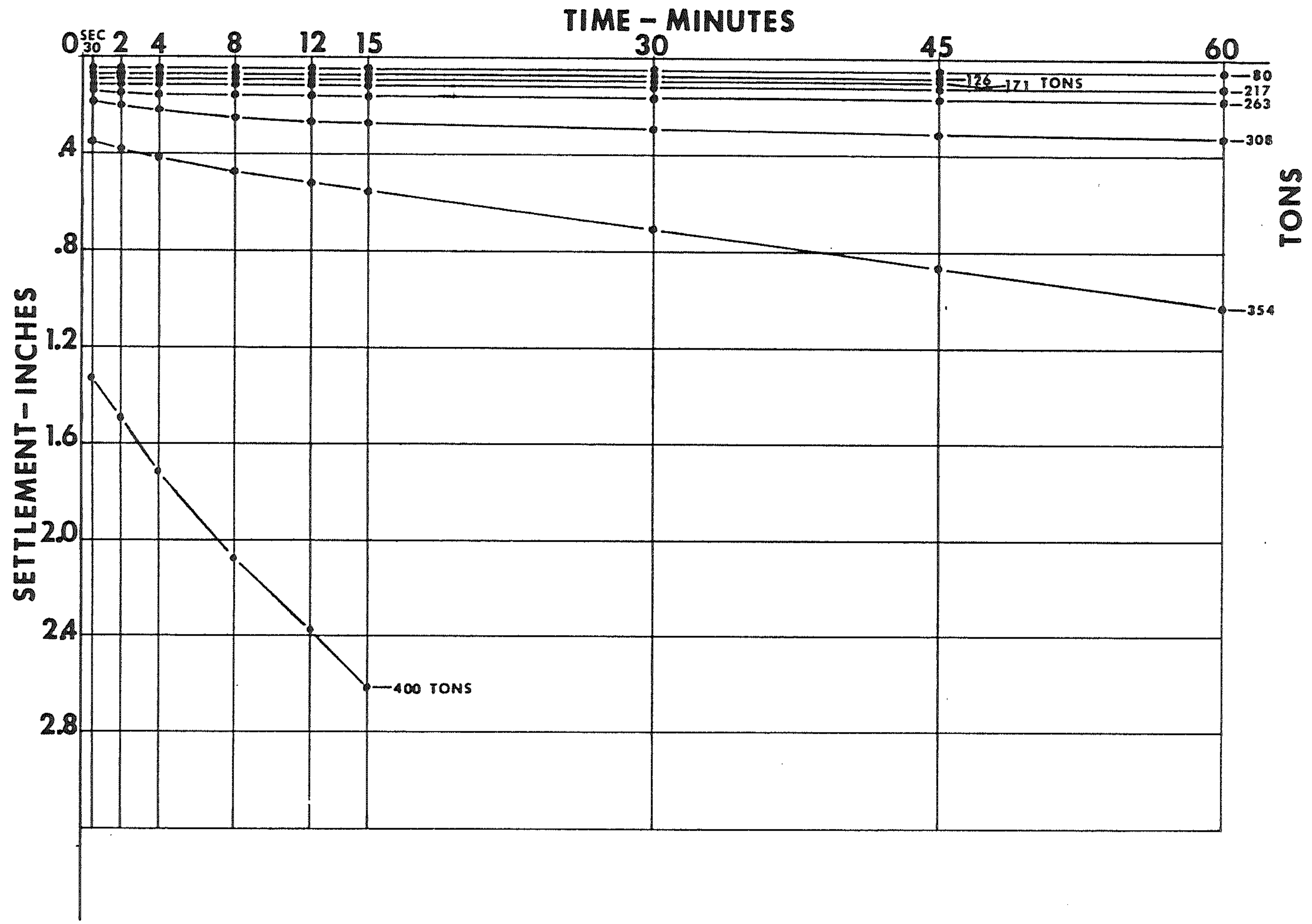


FIGURE 60 TIME SETTLEMENT CURVE

TPC—5

SERGENT, HAUSKINS & BECKWITH

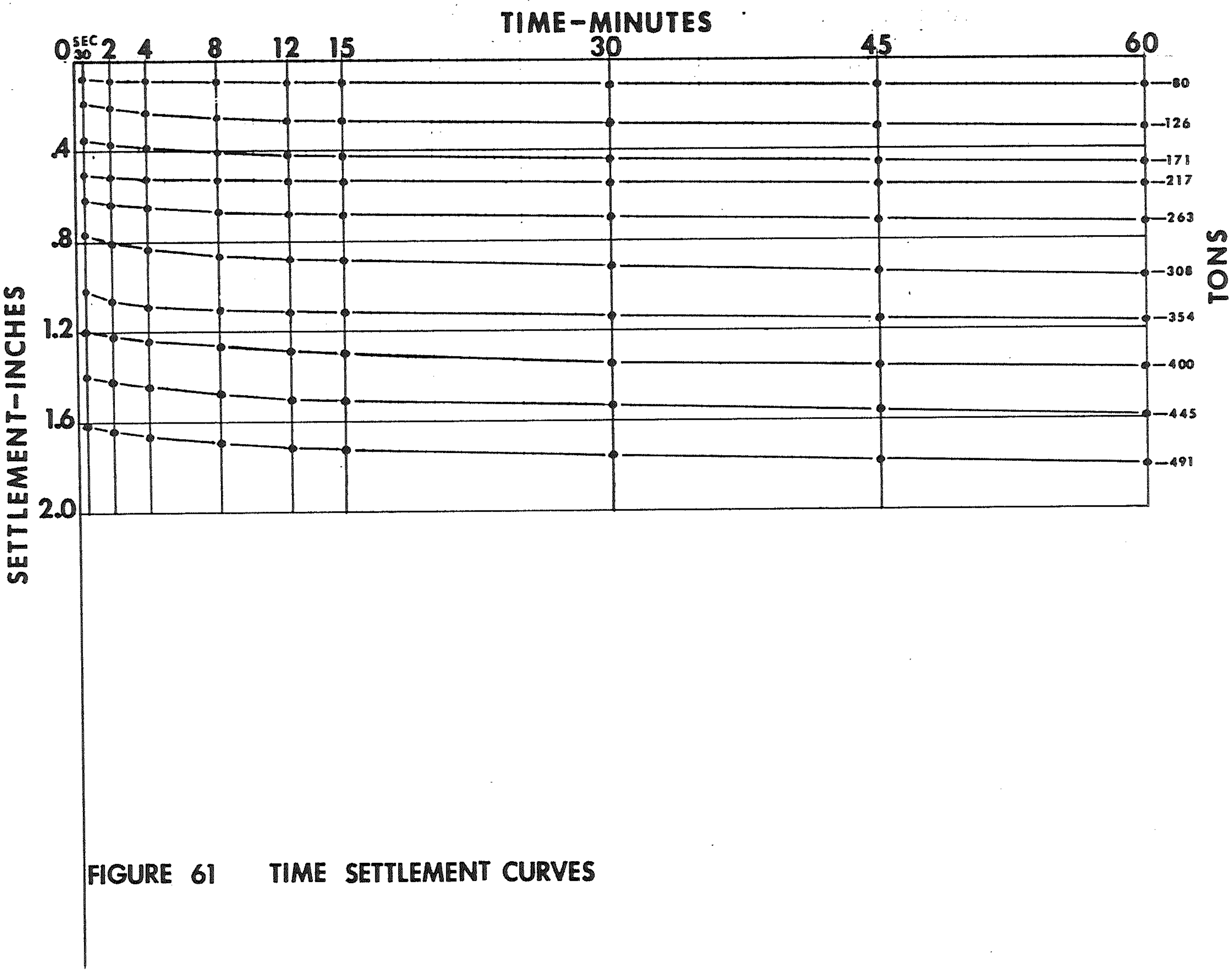


FIGURE 61 TIME SETTLEMENT CURVES

TPC—6

SERGENT, HAUSKINS & BECKWITH

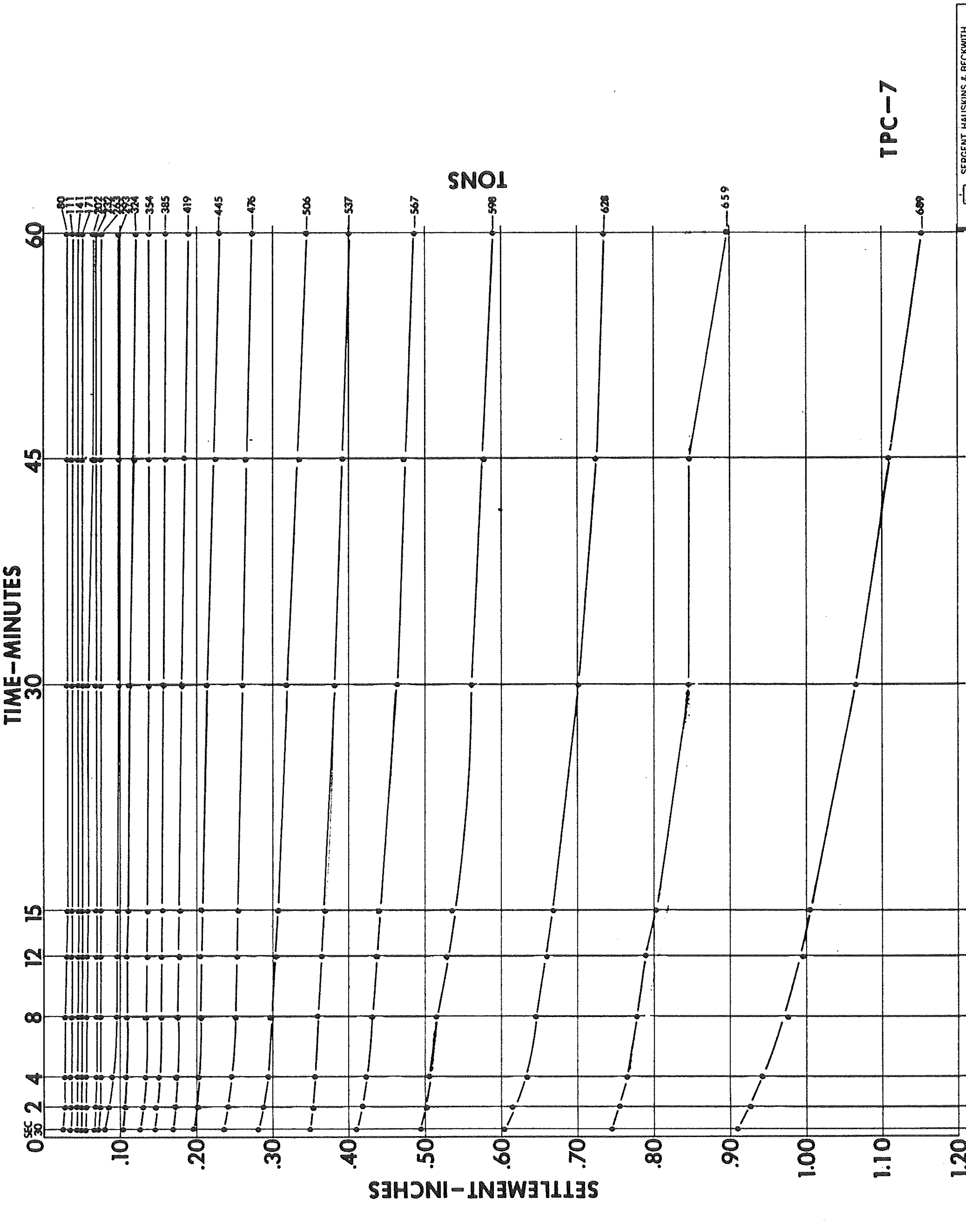


FIGURE 62 TIME SETTLEMENT CURVES

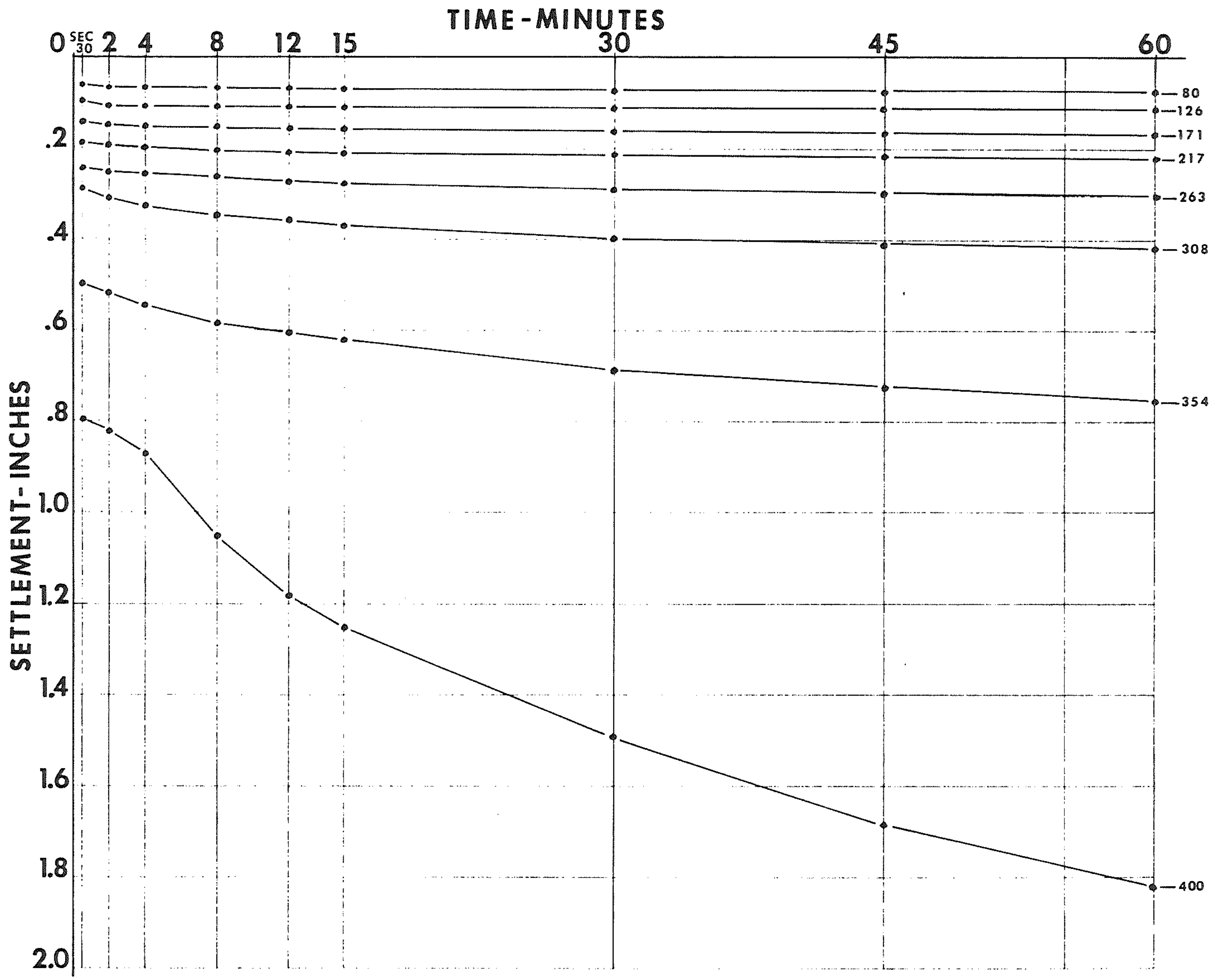


FIGURE 63 TIME SETTLEMENT CURVES

TPC —8

SERGENT, HAUSKINS & BECKWITH

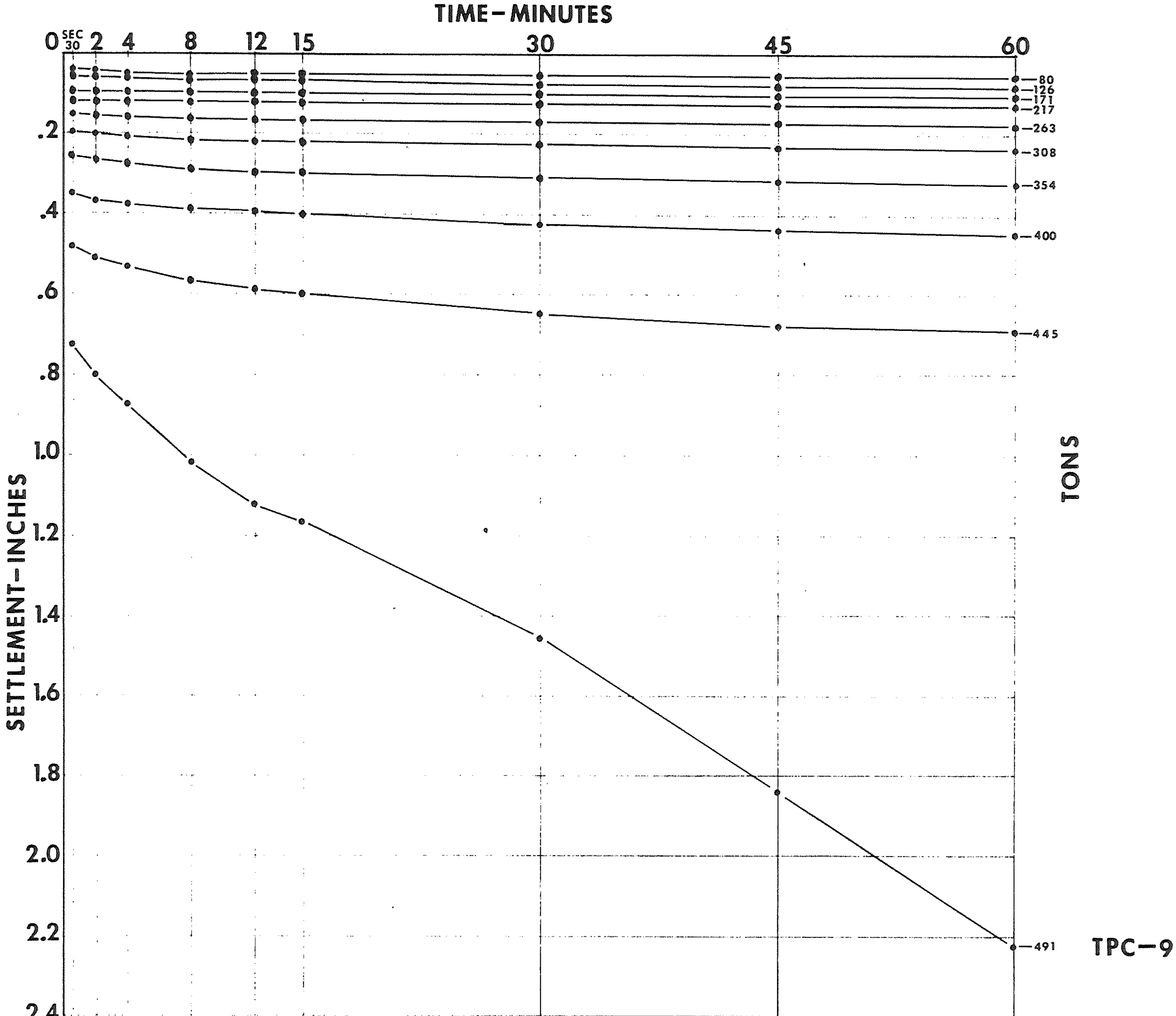
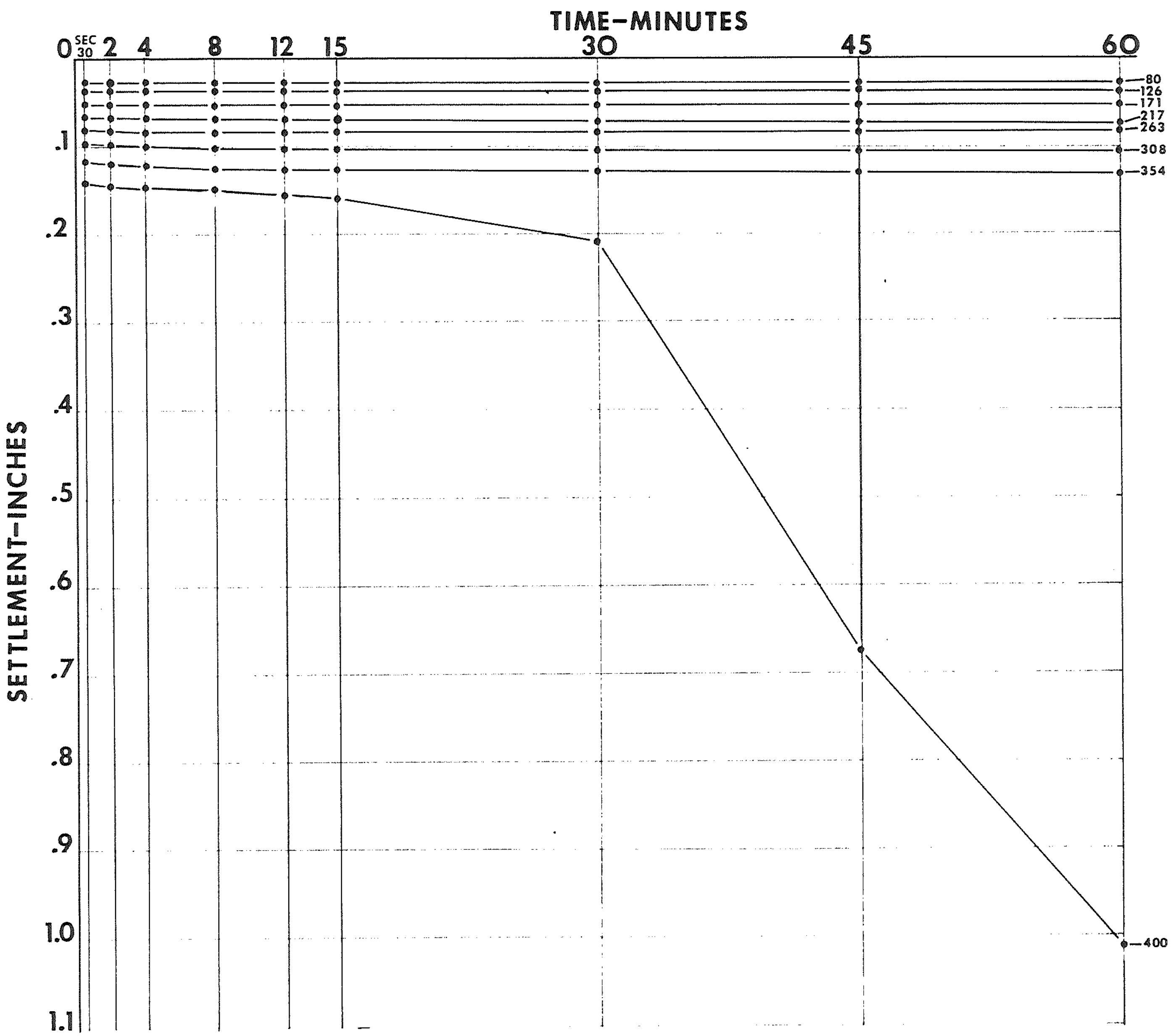


FIGURE 64 TIME SETTLEMENT CURVES

SERGENT, HAUSKINS & BECKWITH

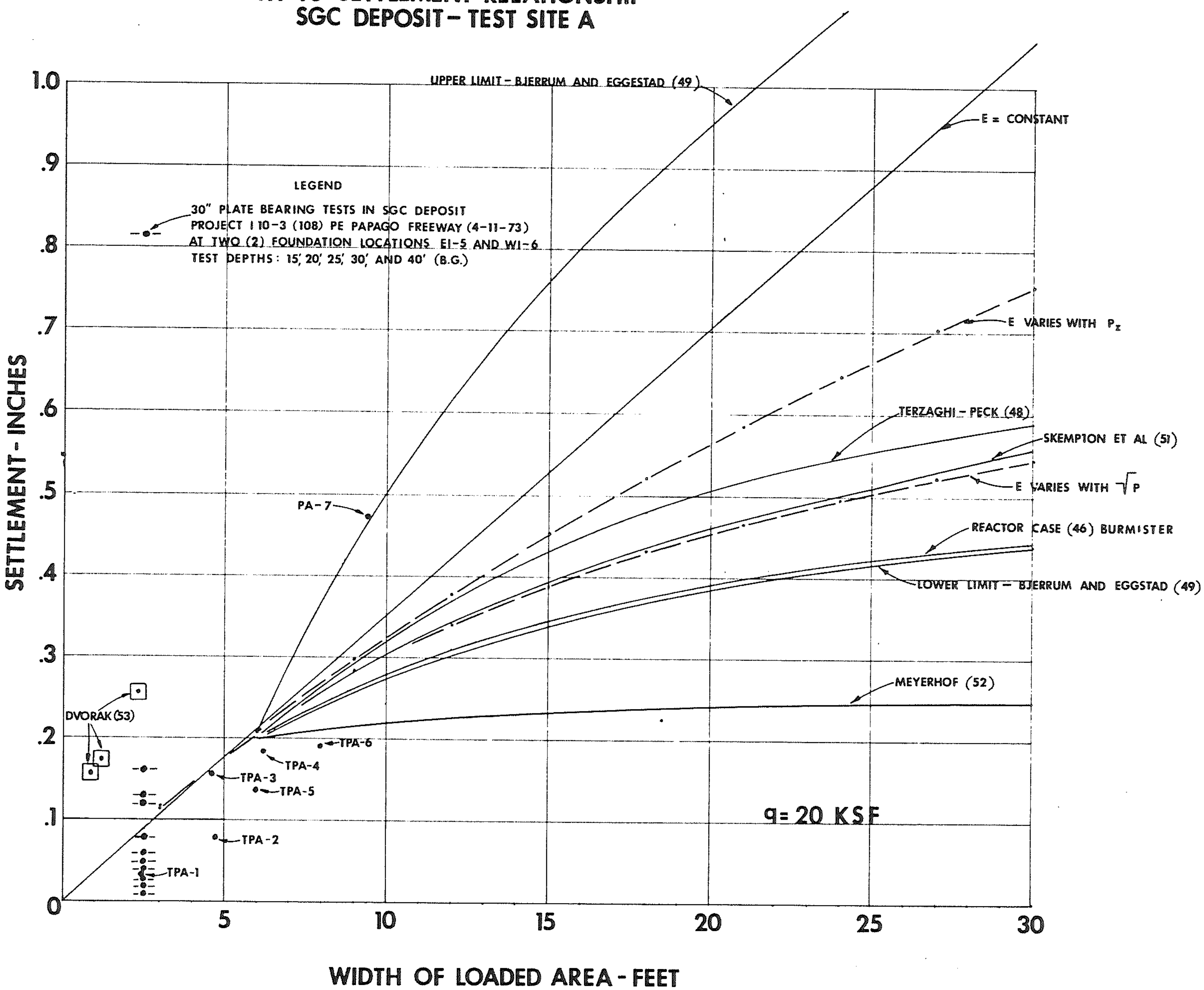


TPC—10

FIGURE 65 TIME SETTLEMENT CURVES

SERGENT, HAUSKINS & BECKWITH

FIGURE 66 WIDTH VS. SETTLEMENT RELATIONSHIP SGC DEPOSIT – TEST SITE A



SERGENT, HAUSKINS & BECKWITH

FIGURE 67 RELATIONSHIP BETWEEN MODULUS OF DEFORMATION AND BEARING PRESSURE SITE A

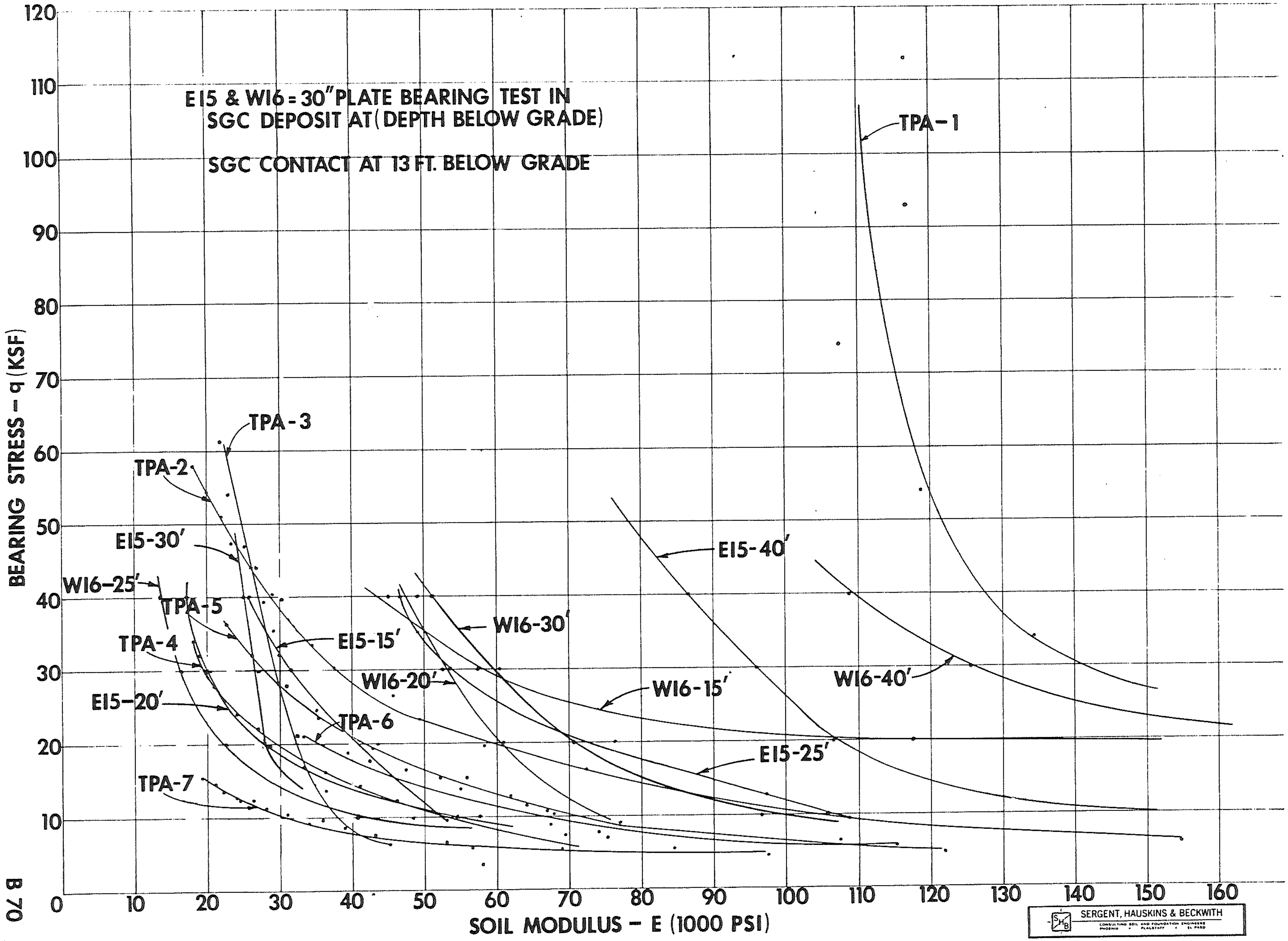
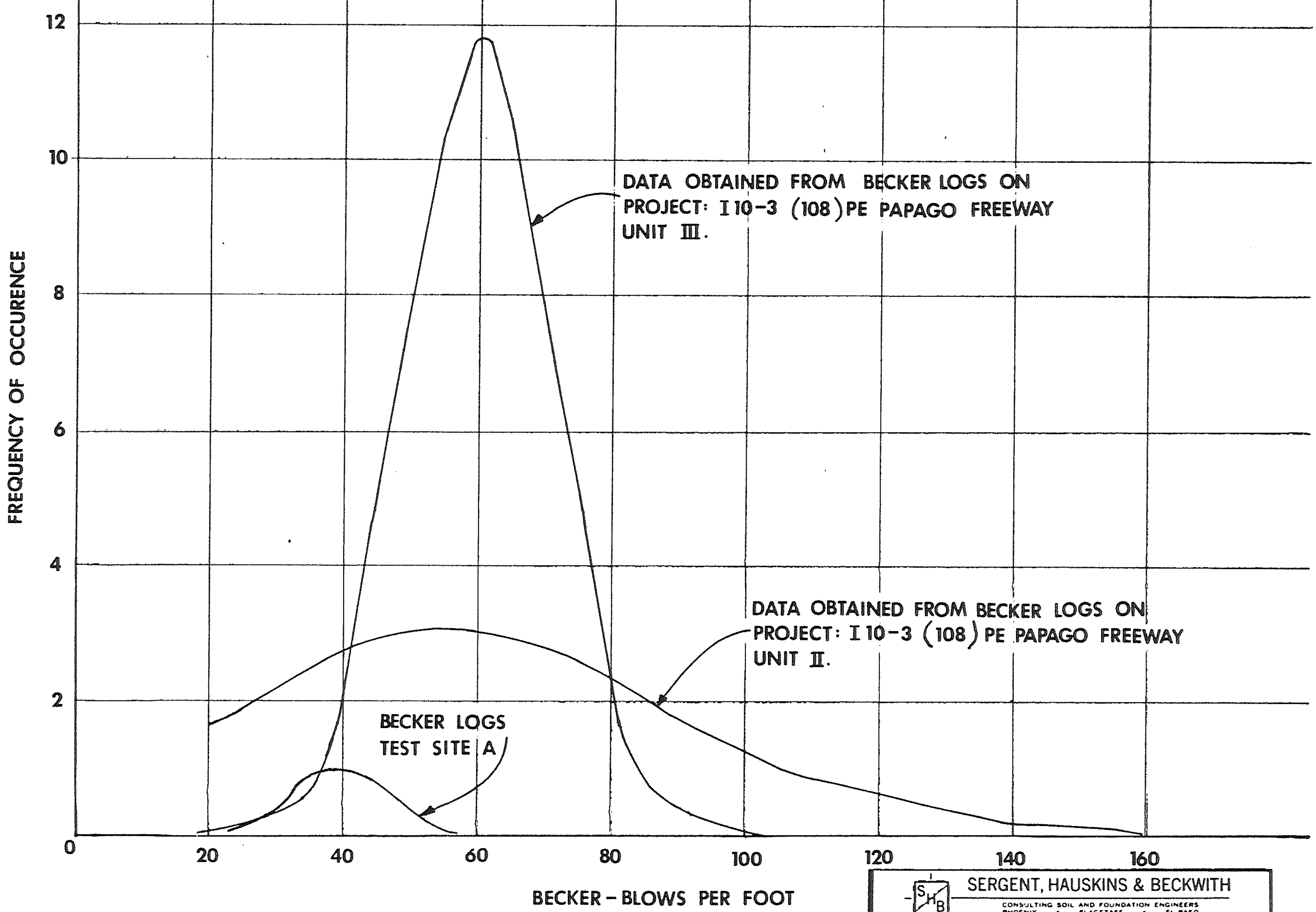


FIGURE 68

FREQUENCY OF MEAN BECKER BLOWS FOR TEST SITE A RELATED TO 155 BORINGS ON UNIT III AND 130 BORINGS ON UNIT II - PAPAGO FREEWAY



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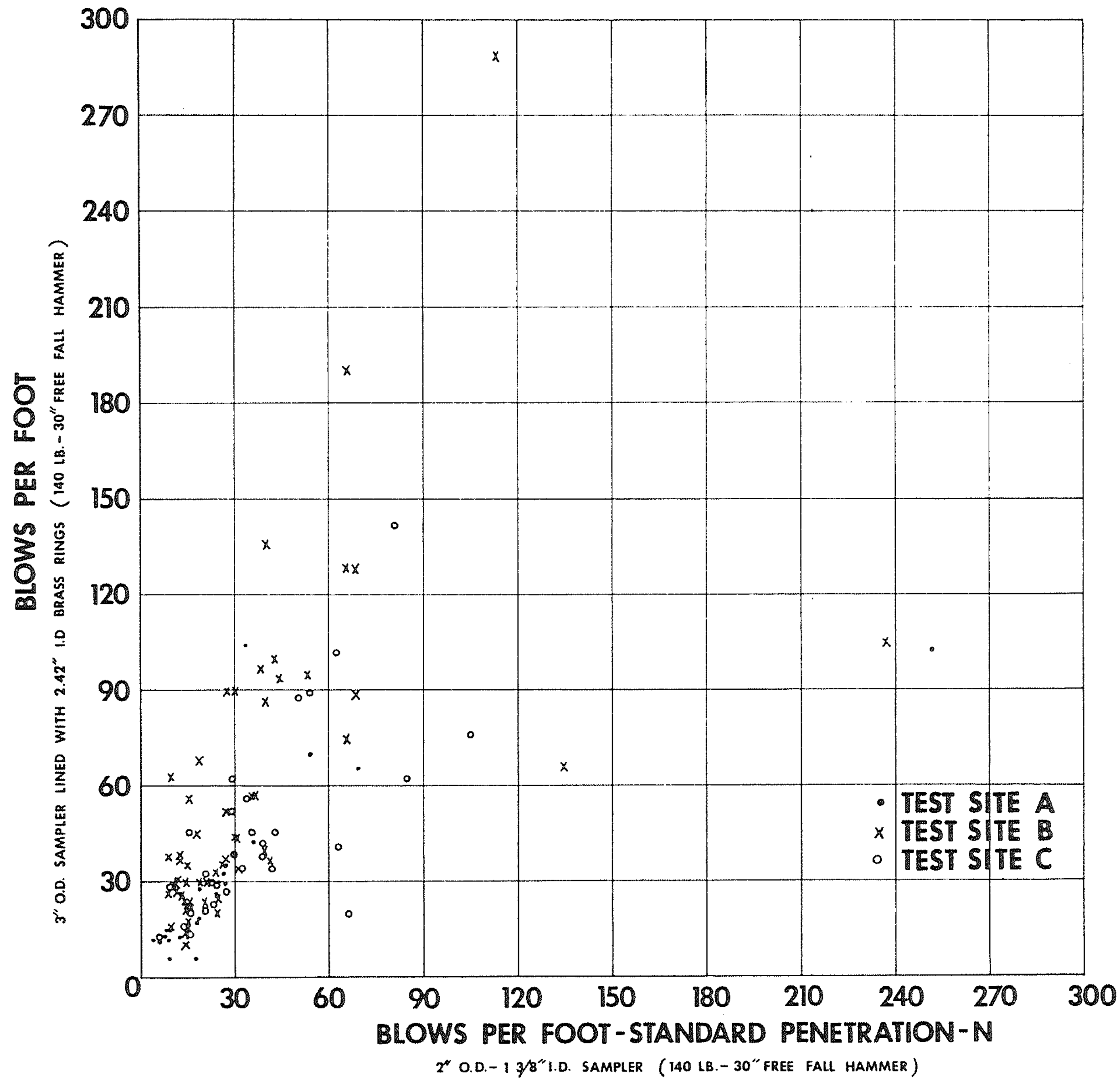


FIGURE 69 RELATIONSHIP BETWEEN BLOW COUNT ON 2" AND 3" TUBE SAMPLES

SERGENT, HAUSKINS & BECKWITH

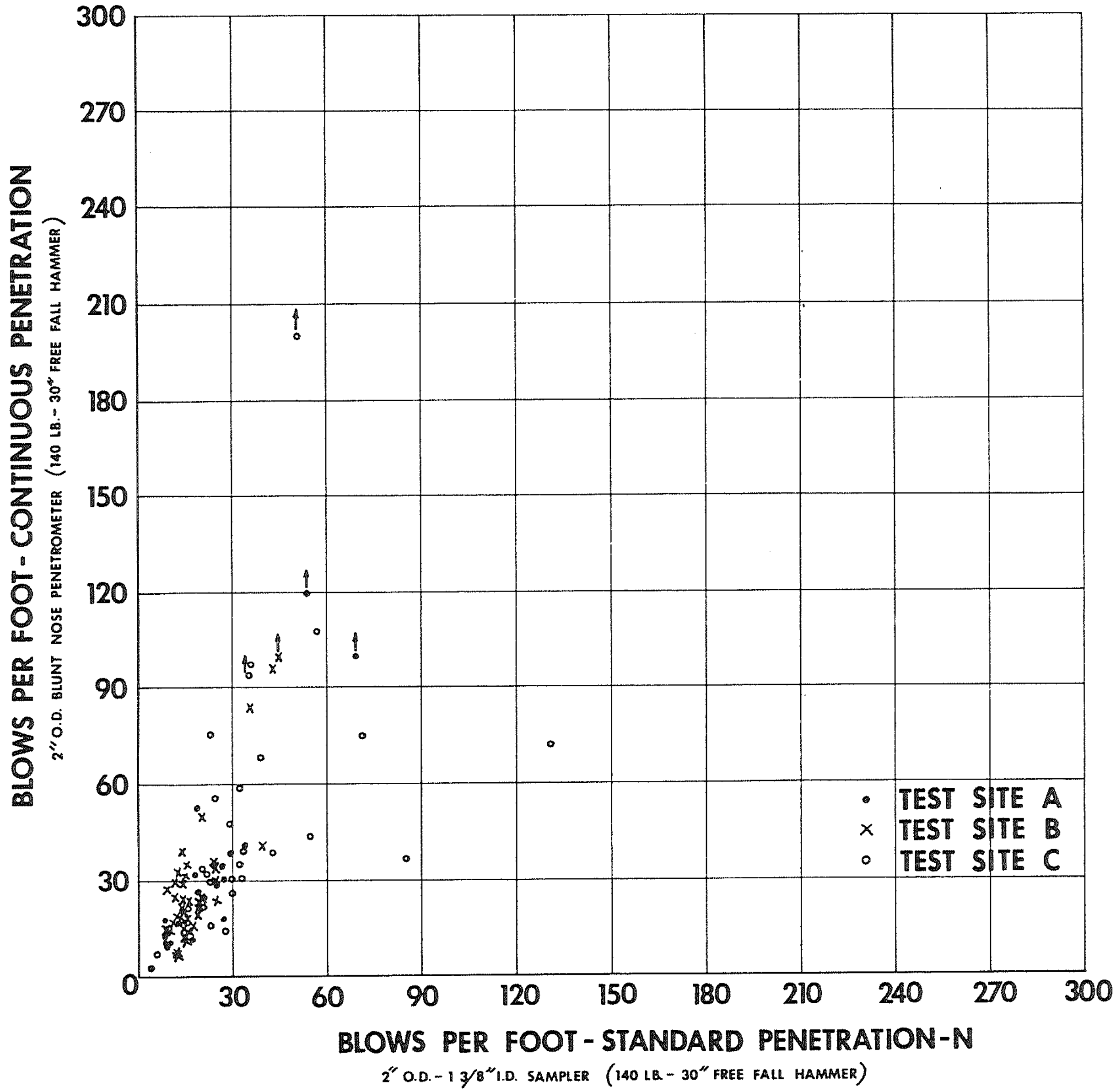


FIGURE 70 RELATIONSHIP BETWEEN STANDARD PENETRATION RESISTANCE AND CONTINUOUS PENETRATION BLOW COUNT

SERGENT, HAUSKINS & BECKWITH

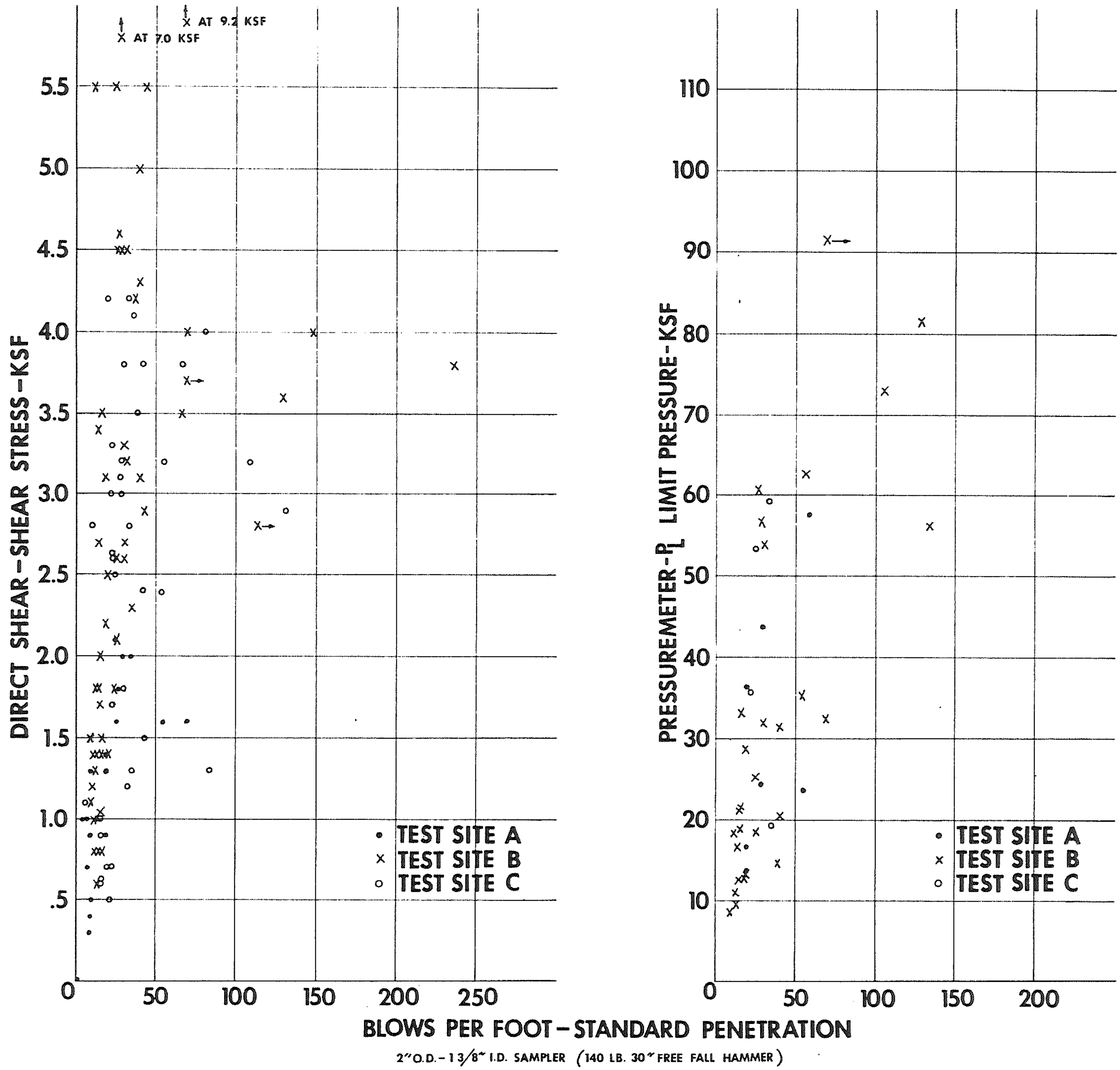


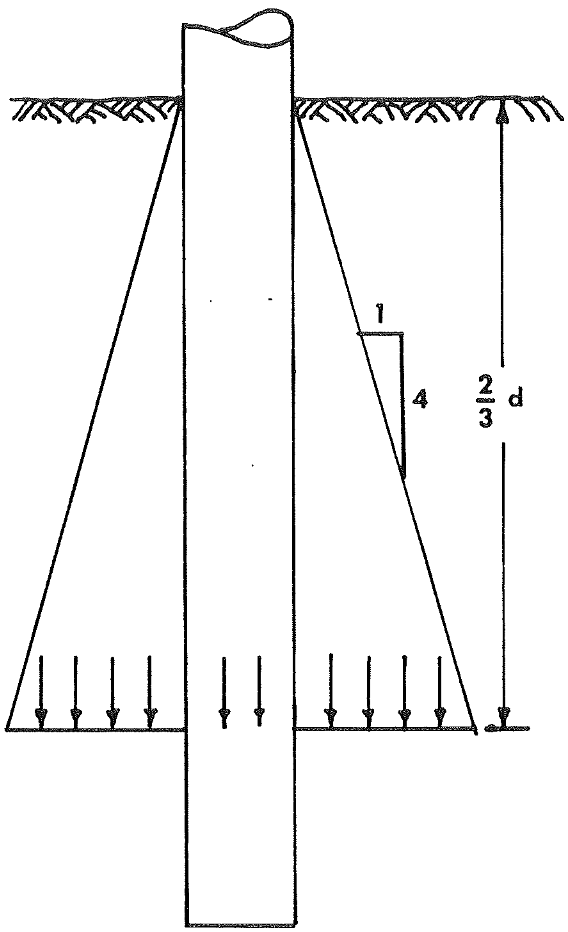
FIGURE 71 RELATIONSHIP BETWEEN STANDARD PENETRATION RESISTANCE, DIRECT SHEAR-SHEAR STRESS AND PRESSUREMETER P_L

SERGENT, HAUSKINS & BECKWITH

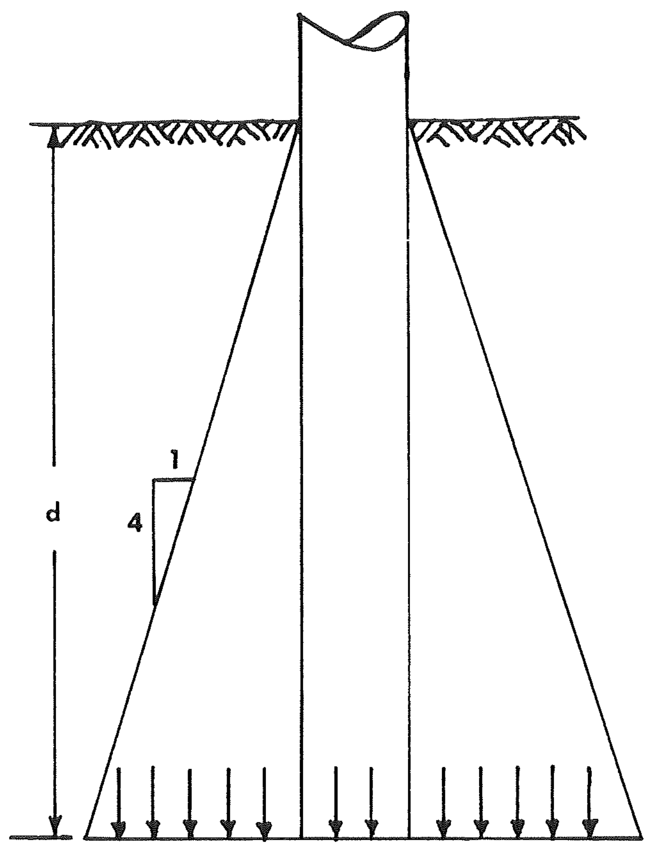
FIGURE 72

ASSUMPTIONS IN "EQUIVALENT PIER" SETTLEMENT COMPUTATIONS

TEST SITES B & C



All other test piles sites B & C - except TPC 4, 5 & 6



TPB - 1

FIGURE 73 DIRECT SHEAR TEST DATA VS. DEPTH – SITE A

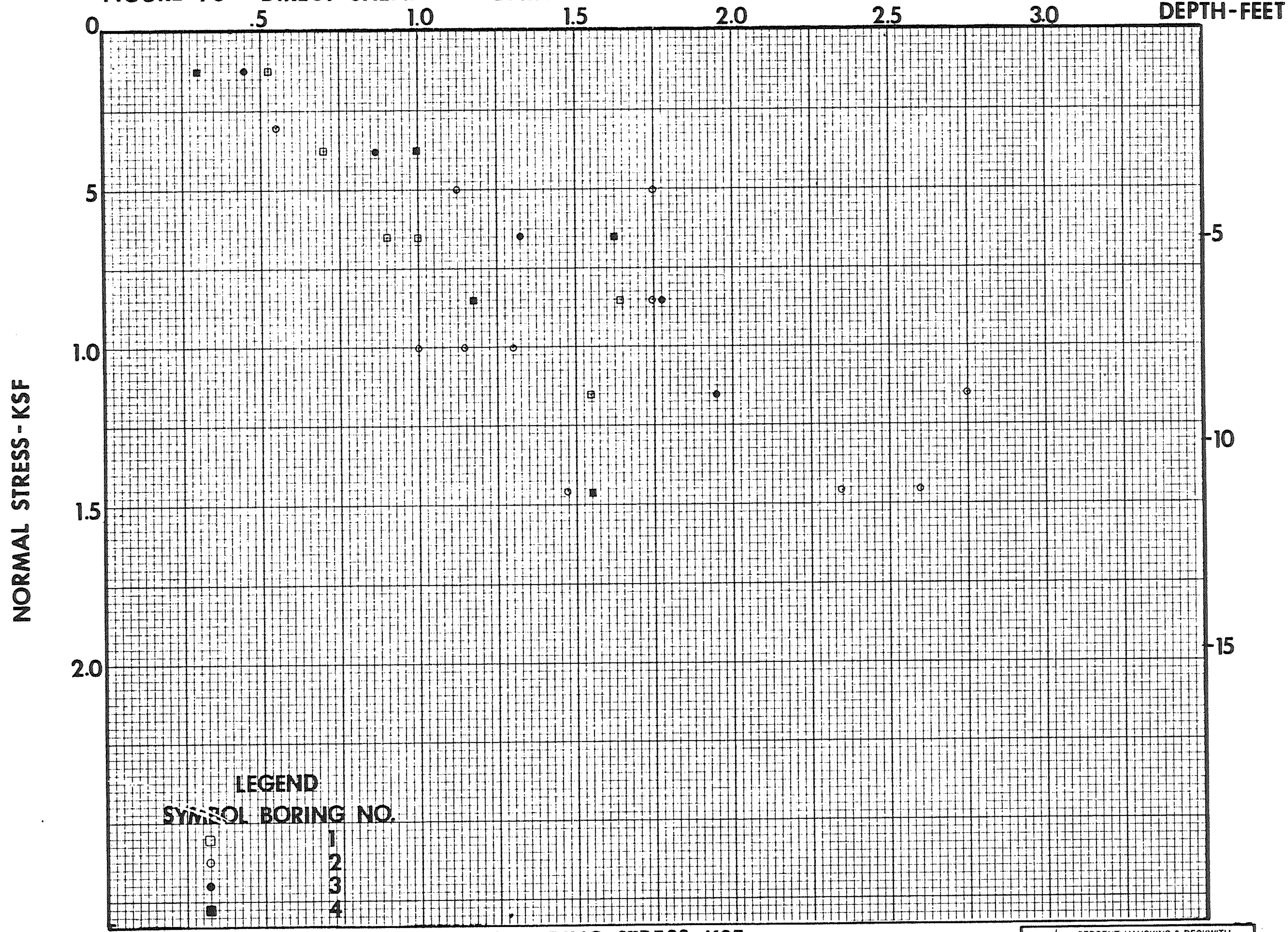


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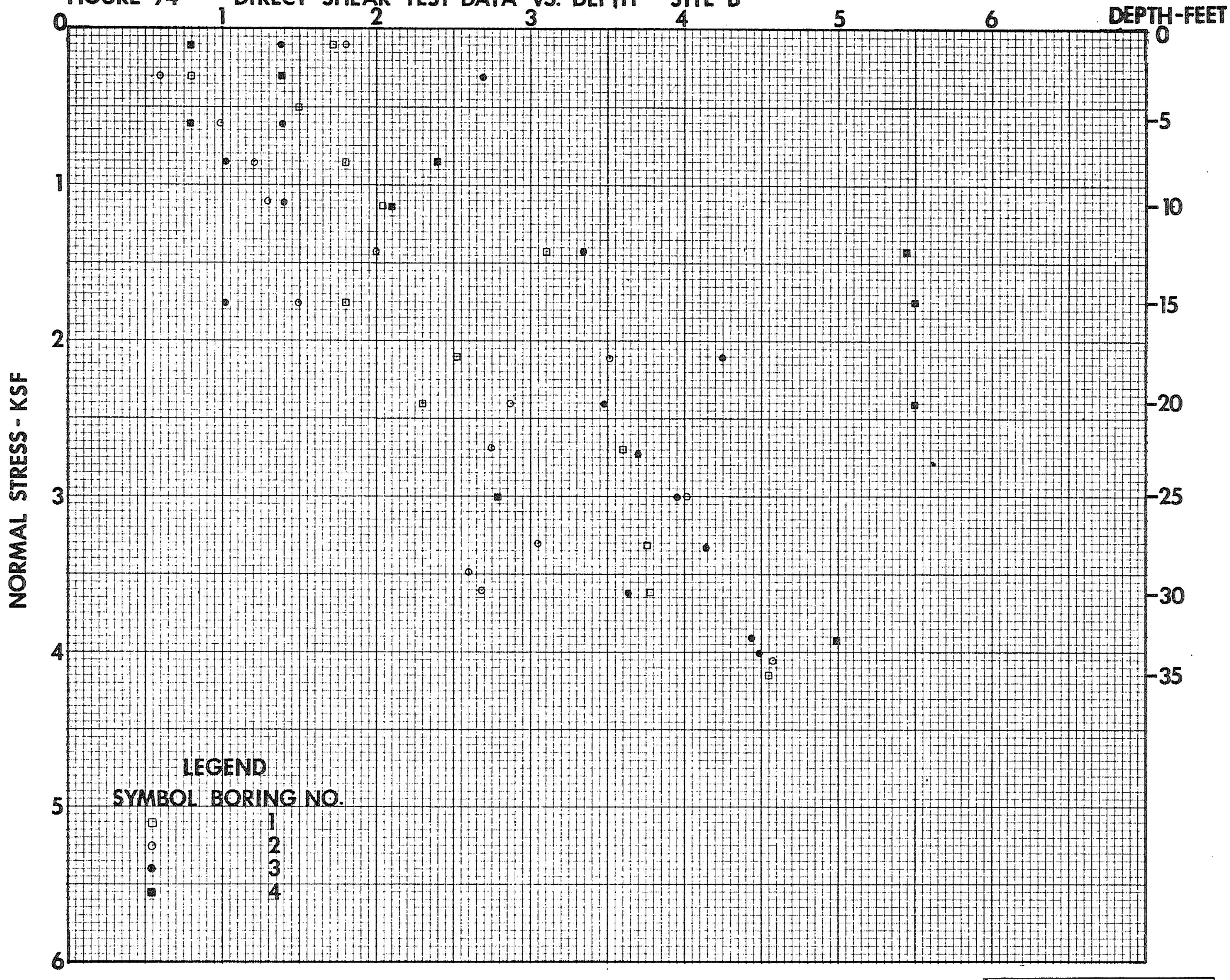
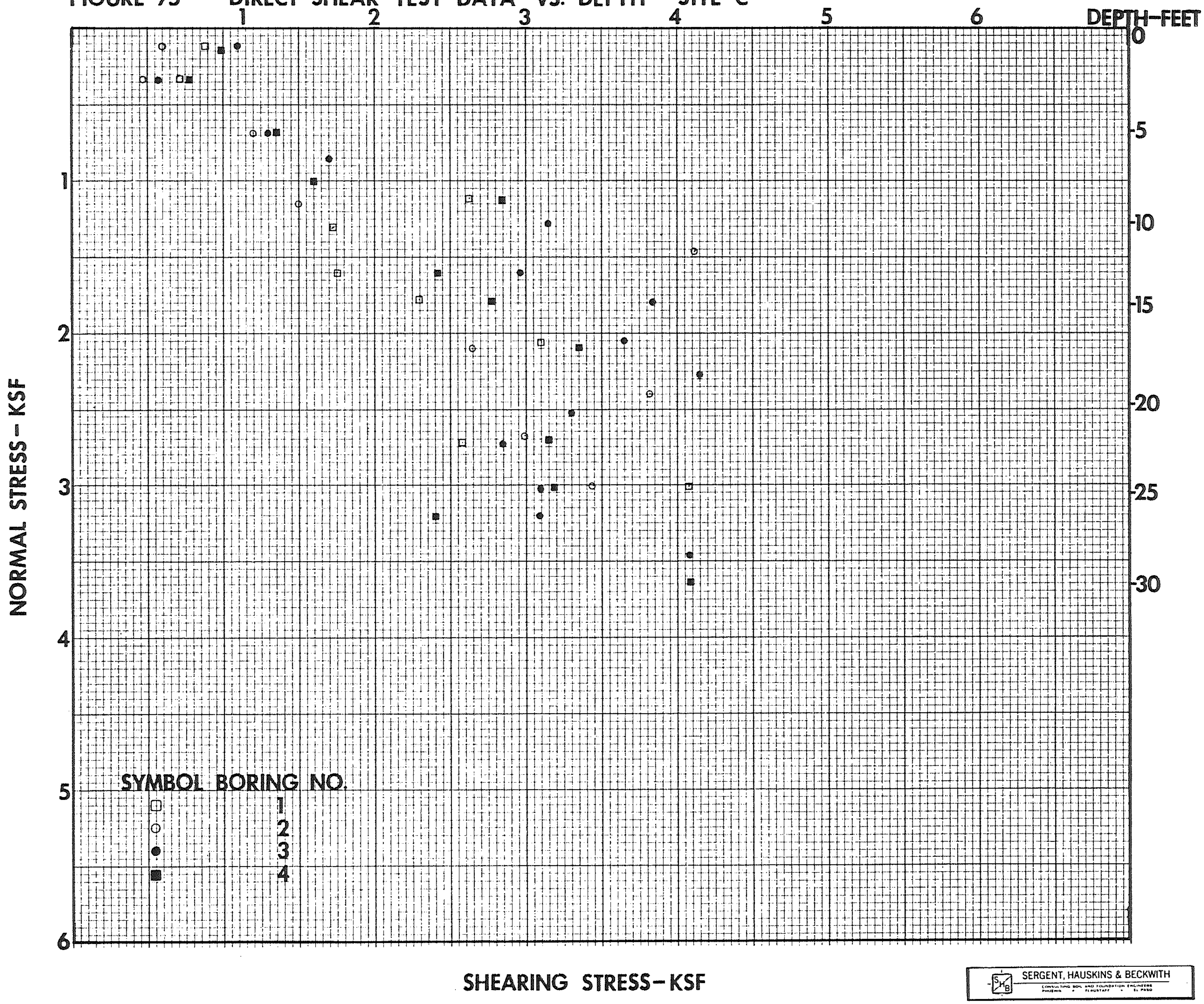
FIGURE 74 DIRECT SHEAR TEST DATA VS. DEPTH — SITE B



SERGENT, HAUSKINS & BECKWITH

FIGURE 75 DIRECT SHEAR TEST DATA VS. DEPTH – SITE C



SERGENT, HAUSKINS & BECKWITH

FIGURE 76 PRESSUREMETER AND UNCONFINED COMPRESSIVE STRENGTH TESTS VS. DEPTH

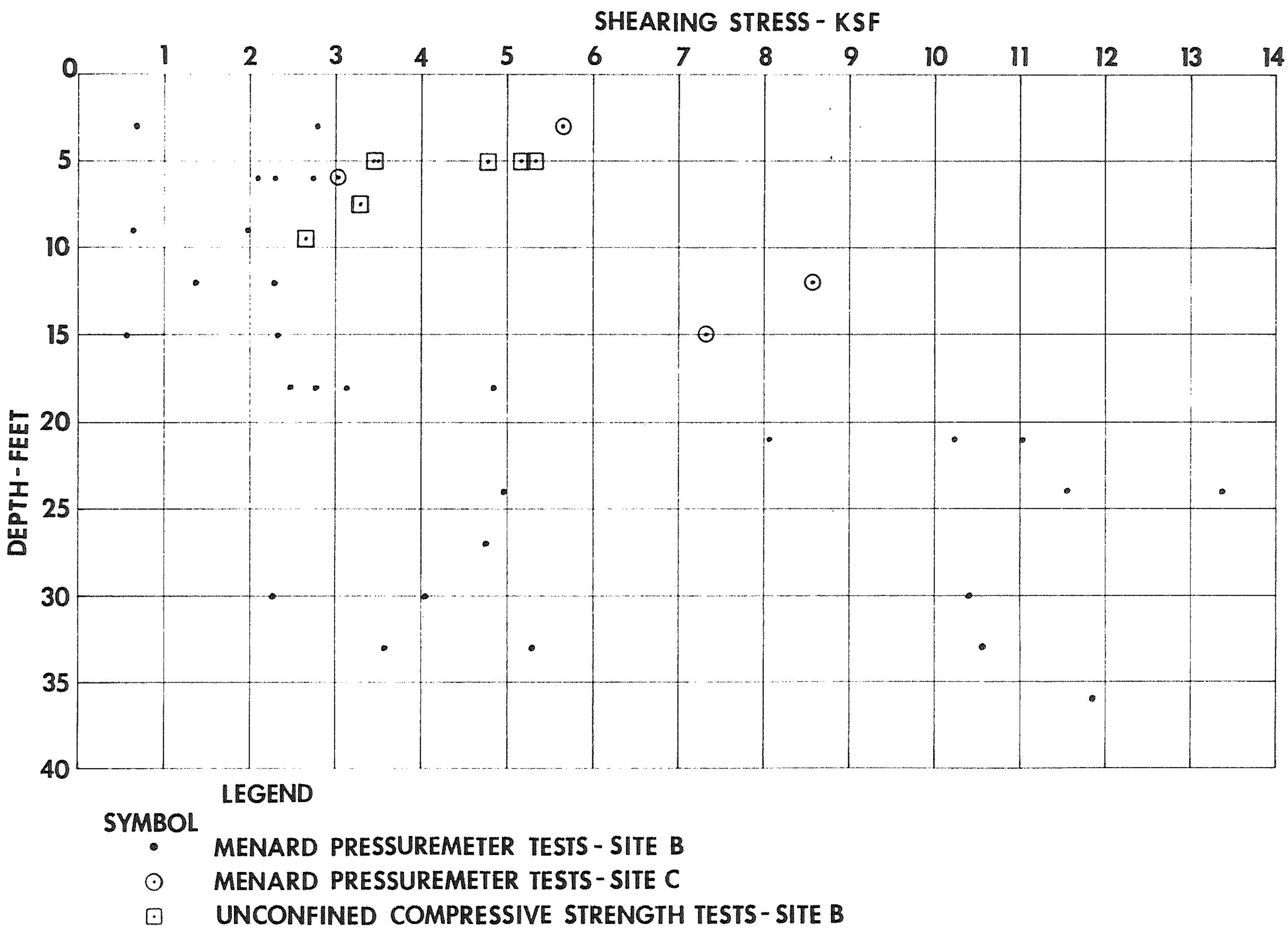


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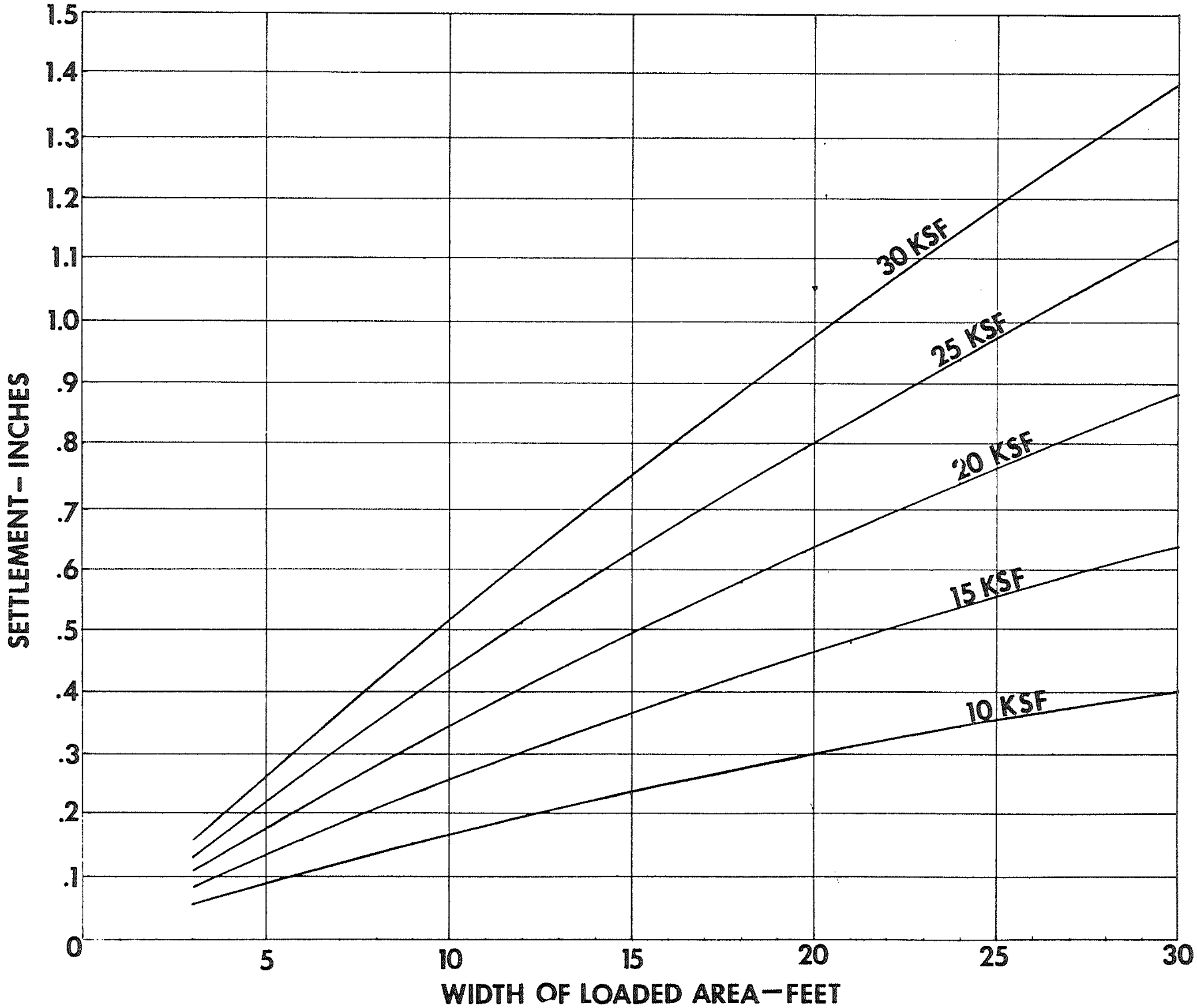
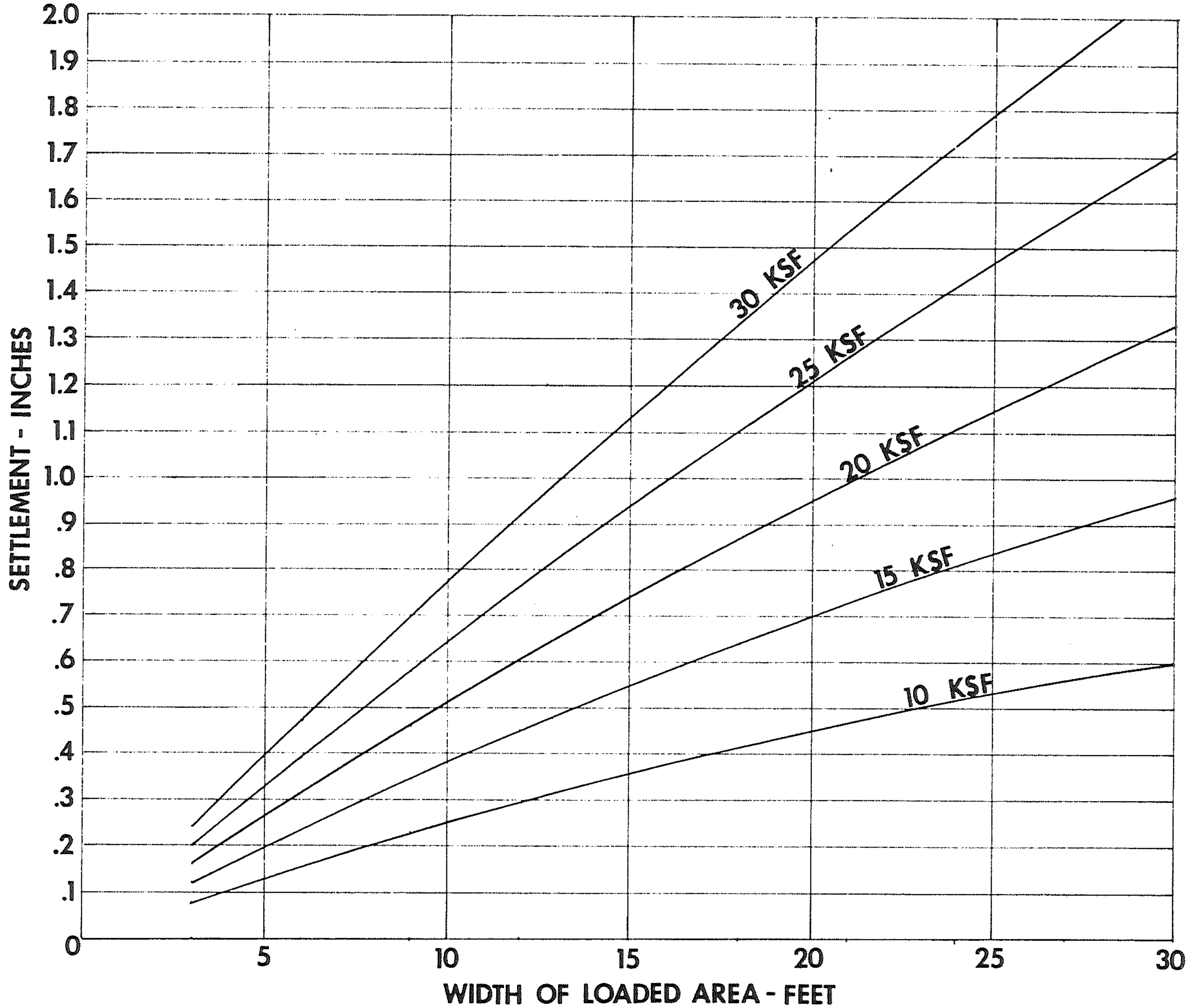
FIGURE 77 "AVERAGE" WIDTH VS. SETTLEMENT CURVES – SITE A



SERGENT, HAUSKINS & BECKWITH

FIGURE 78 RECOMMEND DESIGN CURVES - SGC - SITE A
WIDTH VS. SETTLEMENT AT VARIOUS PRESSURES (q)



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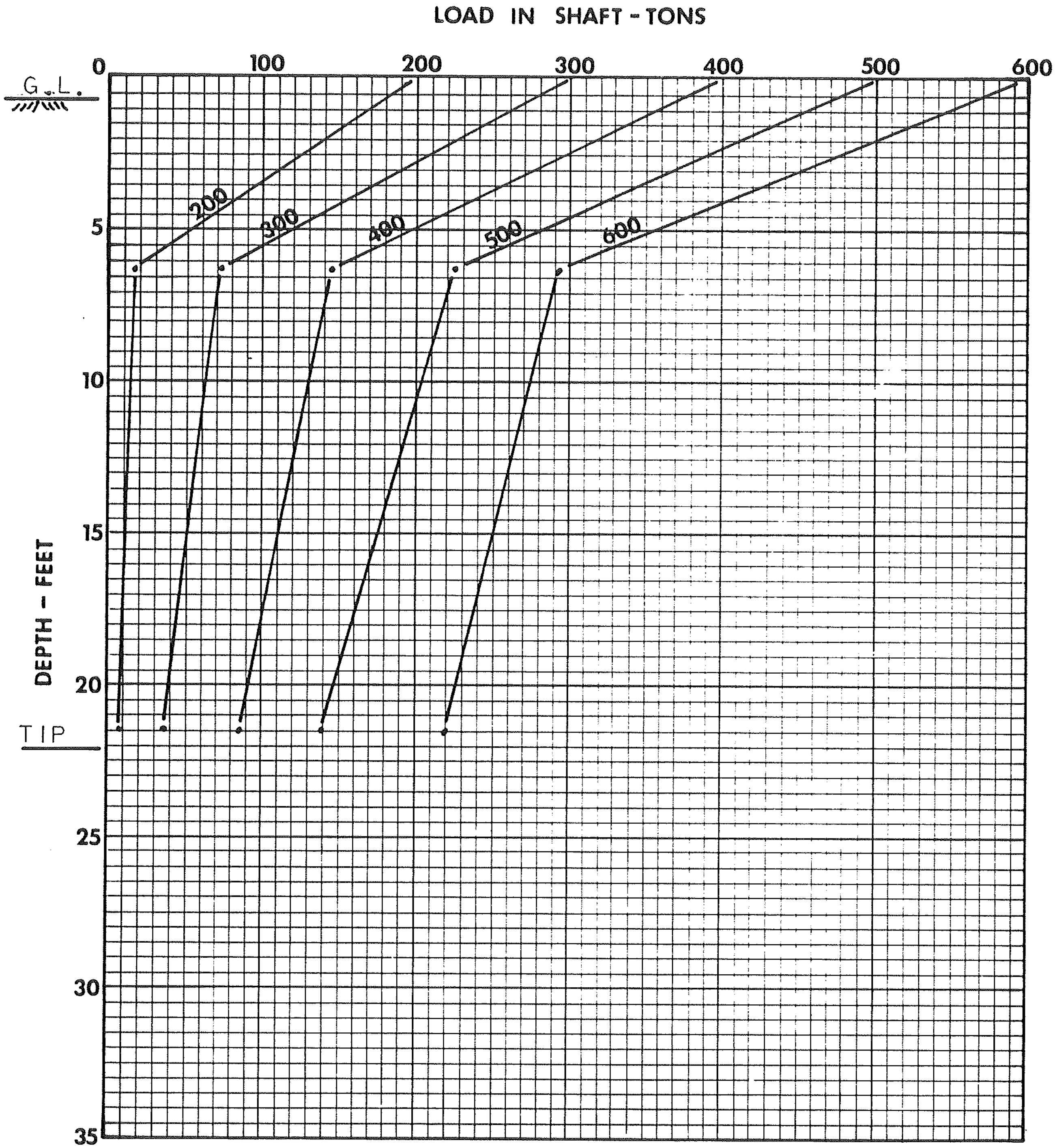
PROJECT SITE B - RESEARCH STUDY DRILLED CAST-IN-PLACE CONCRETE PILES JOB NO. E71-272

TPB-1

FIGURE 79 LOAD DISTRIBUTION ALONG SHAFT FOR VARIOUS LOADINGS



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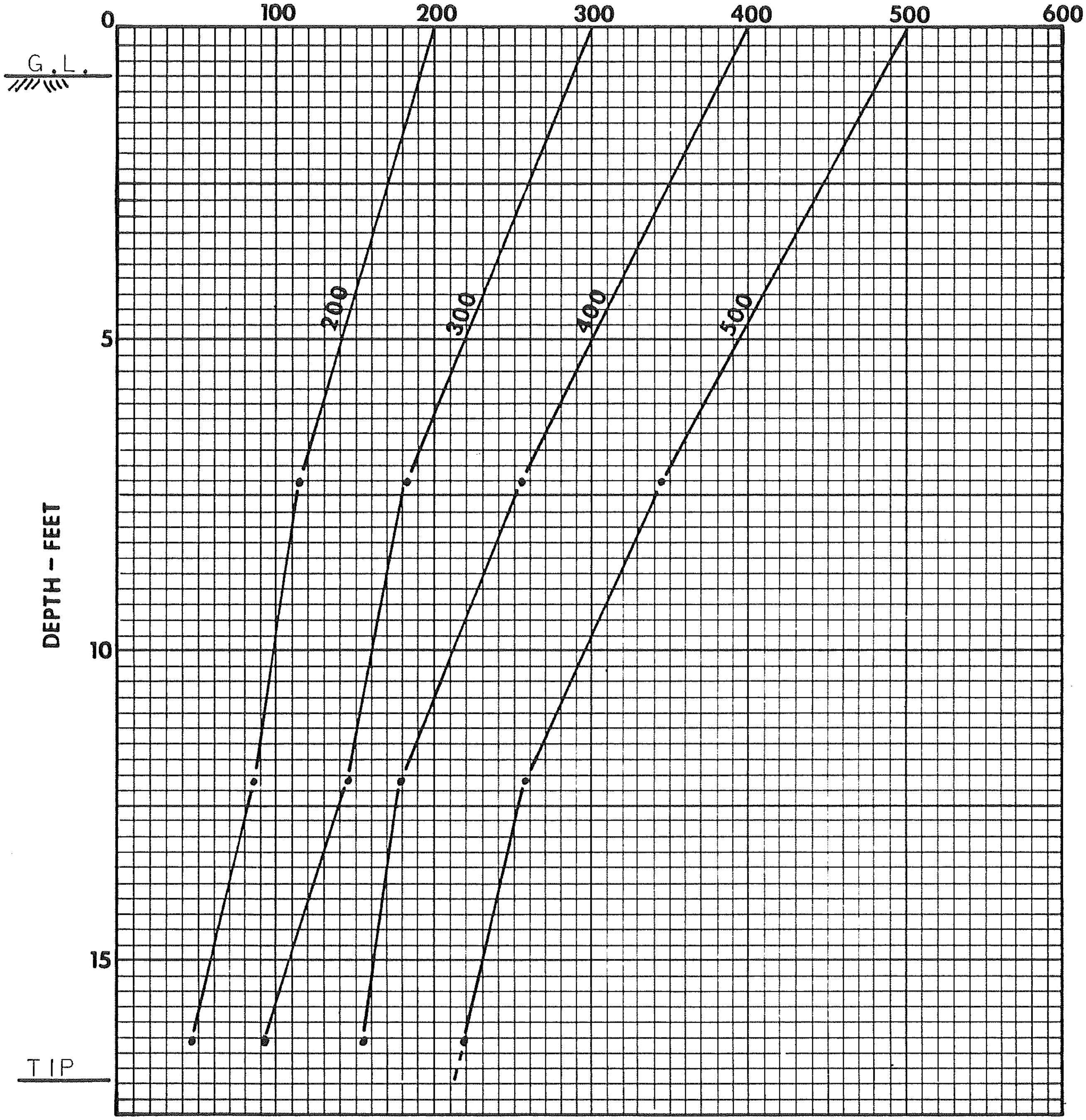
Drilled Cast-in-Place Concrete Piles JOB NO. E71-272

TPB-2

FIGURE 80 LOAD DISTRIBUTION ALONG SHAFT FOR VARIOUS LOADINGS



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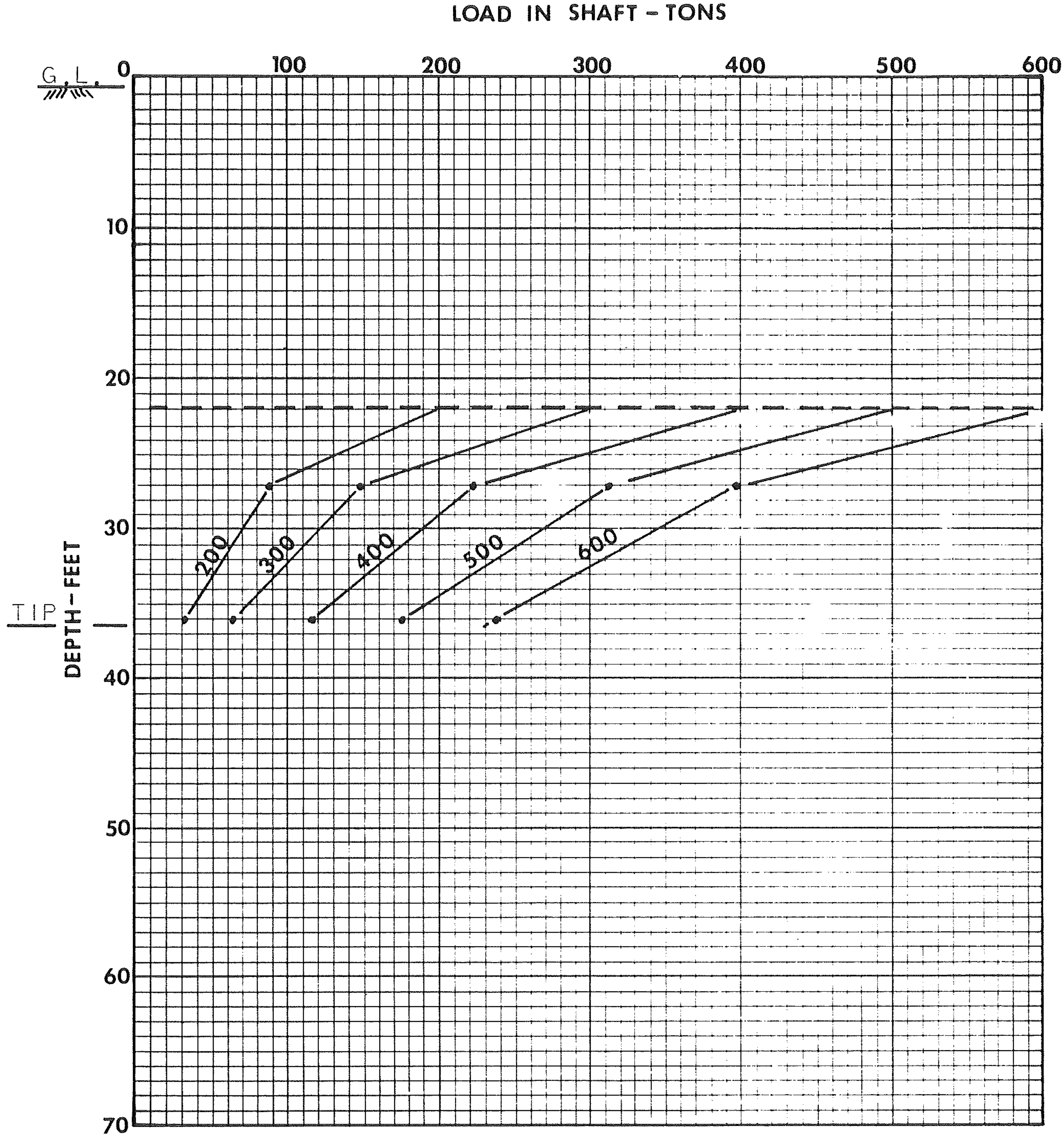
TPB-3

FIGURE 81 LOAD DISTRIBUTION ALONG SHAFT FOR VARIOUS LOADINGS



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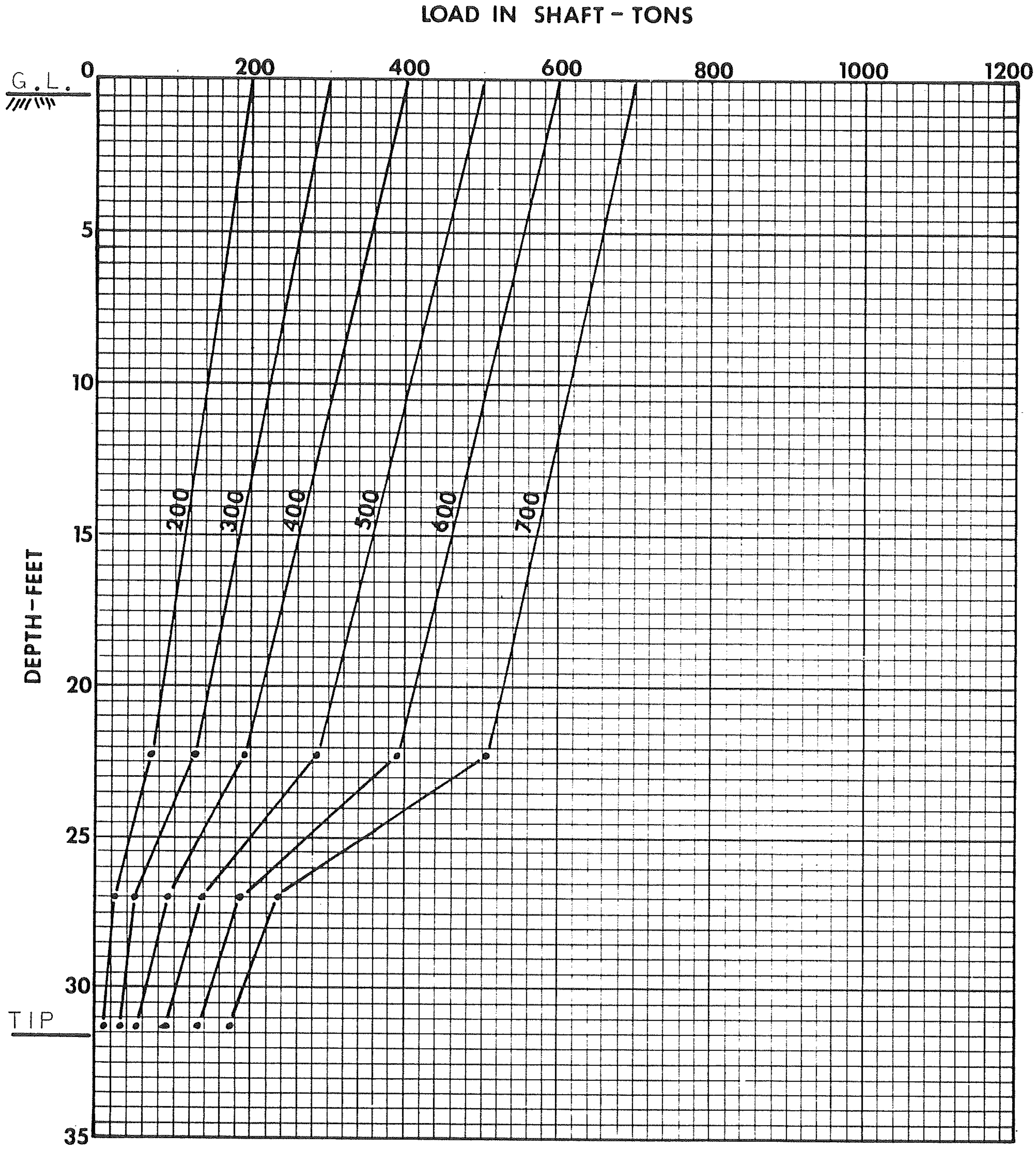
DRILLED CAST-IN-PLACE CONCRETE PILES JOB NO. E71-272

TPB-4

FIGURE 82 LOAD DISTRIBUTION ALONG SHAFT FOR VARIOUS LOADINGS



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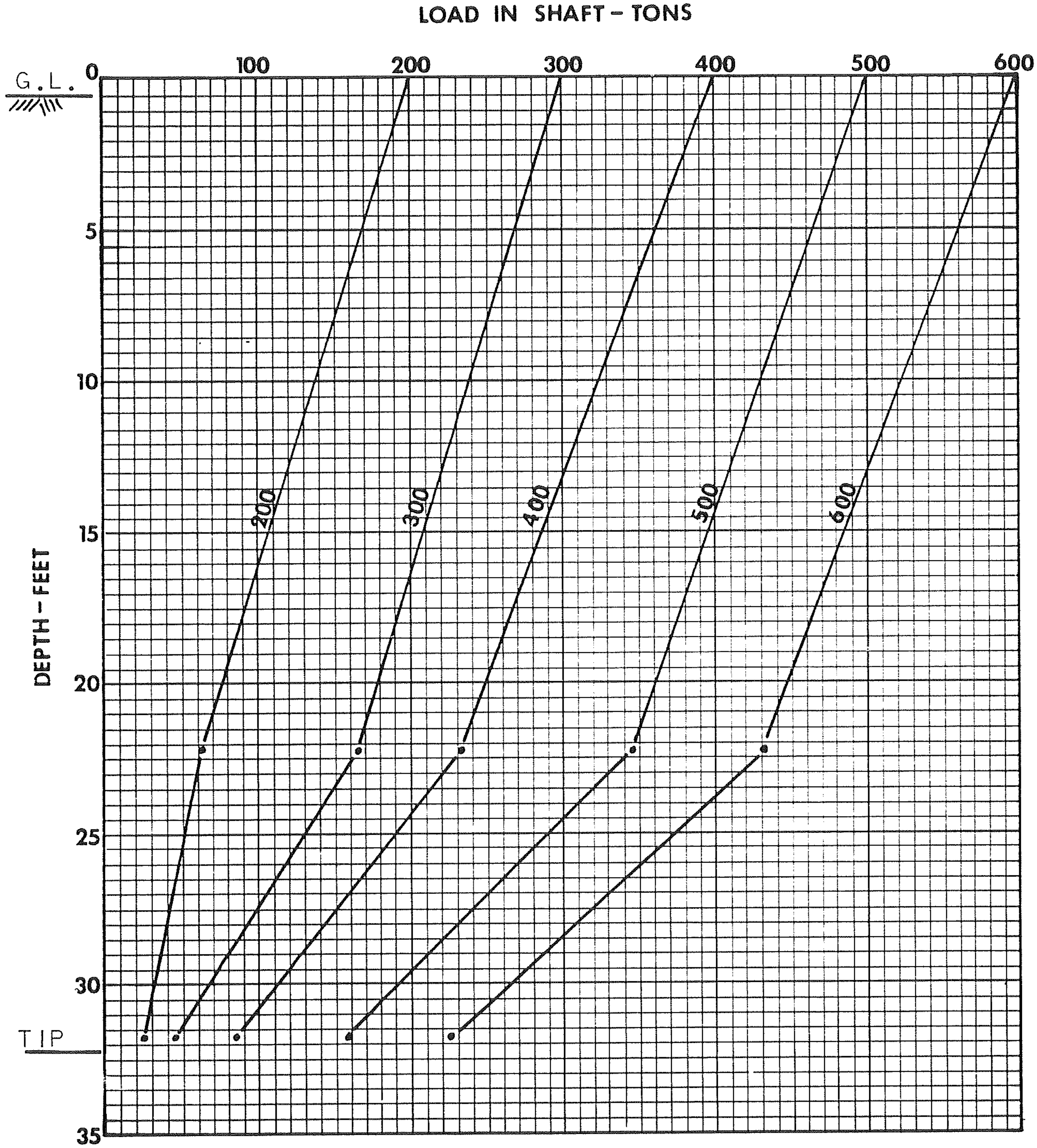
PROJECT SITE B - RESEARCH STUDY DRILLED CAST-IN-PLACE CONCRETE PILES. JOB NO. E71-272

TPB-5

FIGURE 83 LOAD DISTRIBUTION ALONG SHAFT FOR VARIOUS LOADINGS

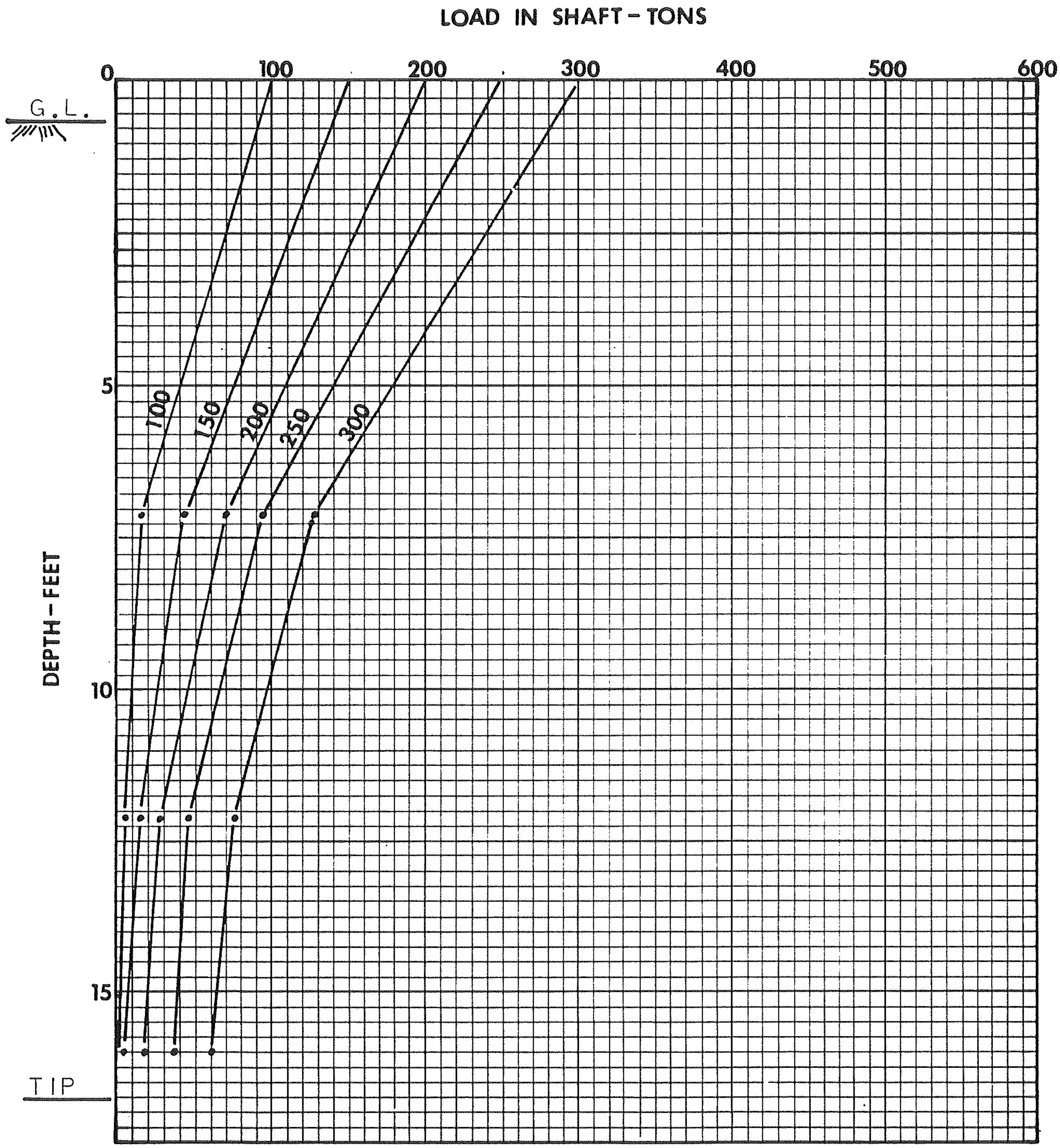


SITE B - RESEARCH STUDY

PROJECT DRILLED CAST-IN-PLACE CONCRETE PILES JOB NO. E71-272

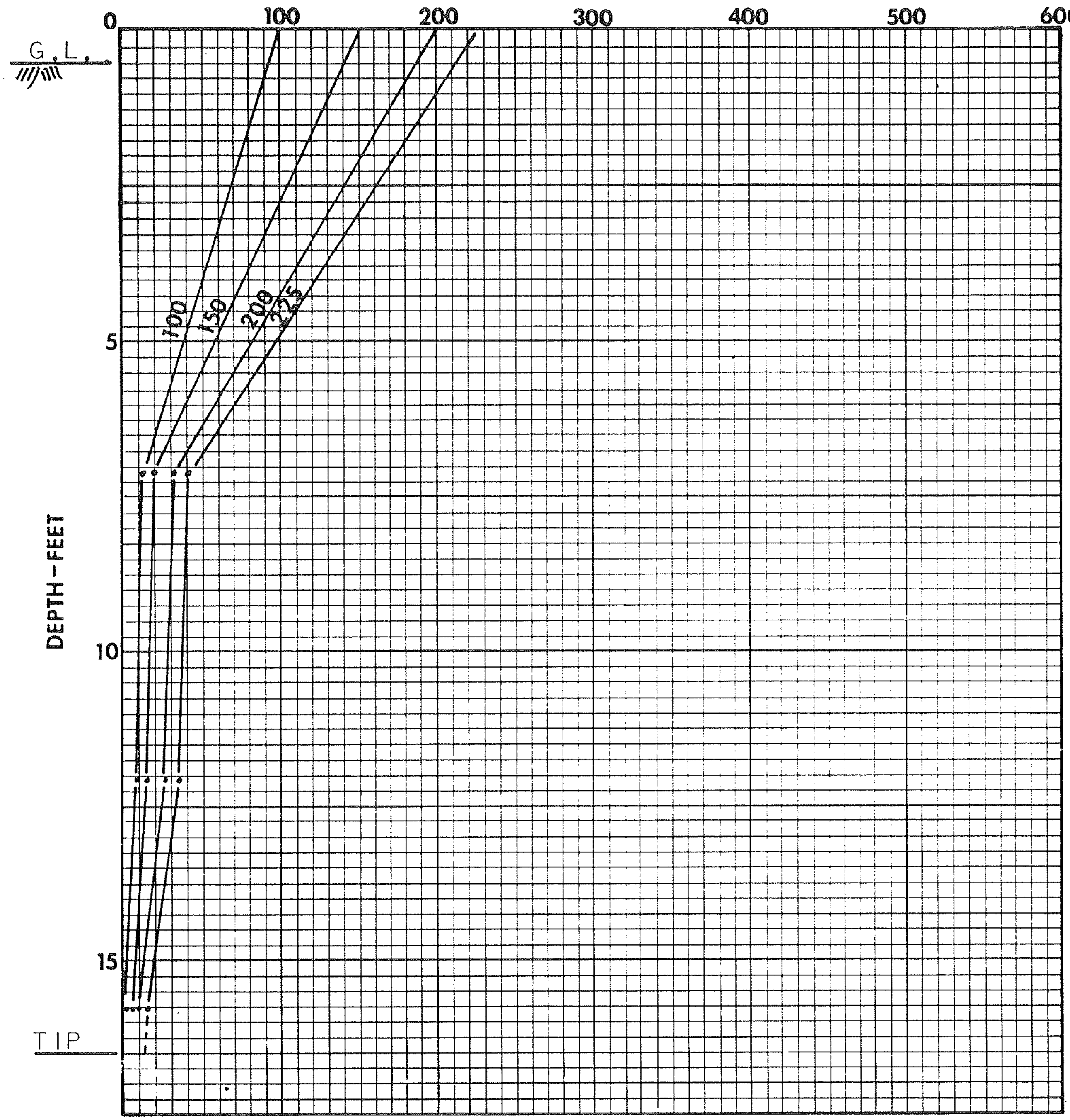
TPB-6

FIGURE 84 LOAD DISTRIBUTION ALONG SHAFT FOR VARIOUS LOADINGS



TPB-7

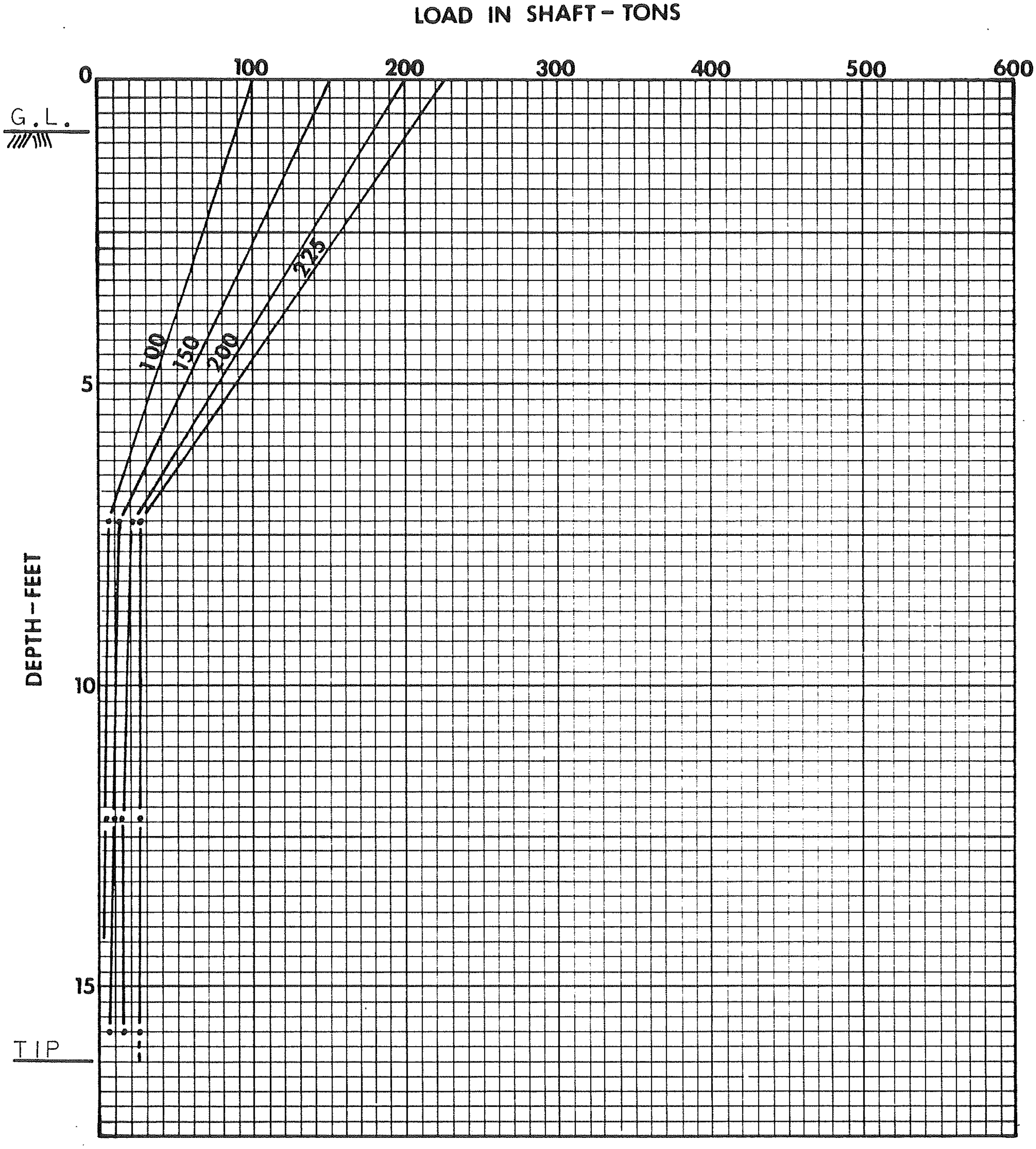
FIGURE 85 LOAD DISTRIBUTION ALONG SHAFT FOR VARIOUS LOADINGS



PROJECT SITE B - RESEARCH STUDY DRILLED CAST-IN-PLACE CONCRETE PILES JOB NO. E71-272

TPB-8

FIGURE 86 LOAD DISTRIBUTION ALONG SHAFT FOR VARIOUS LOADINGS



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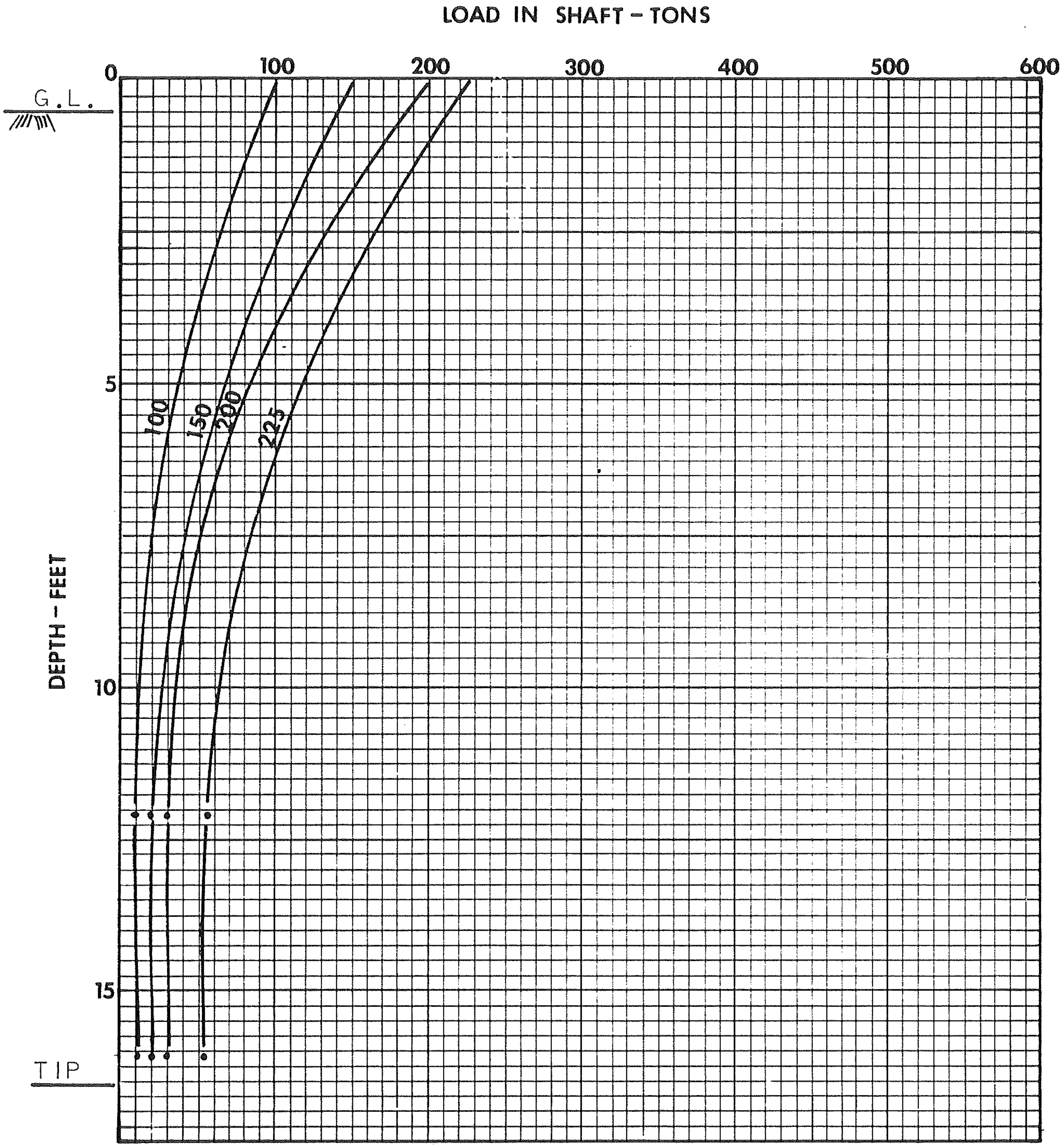
TPB-9

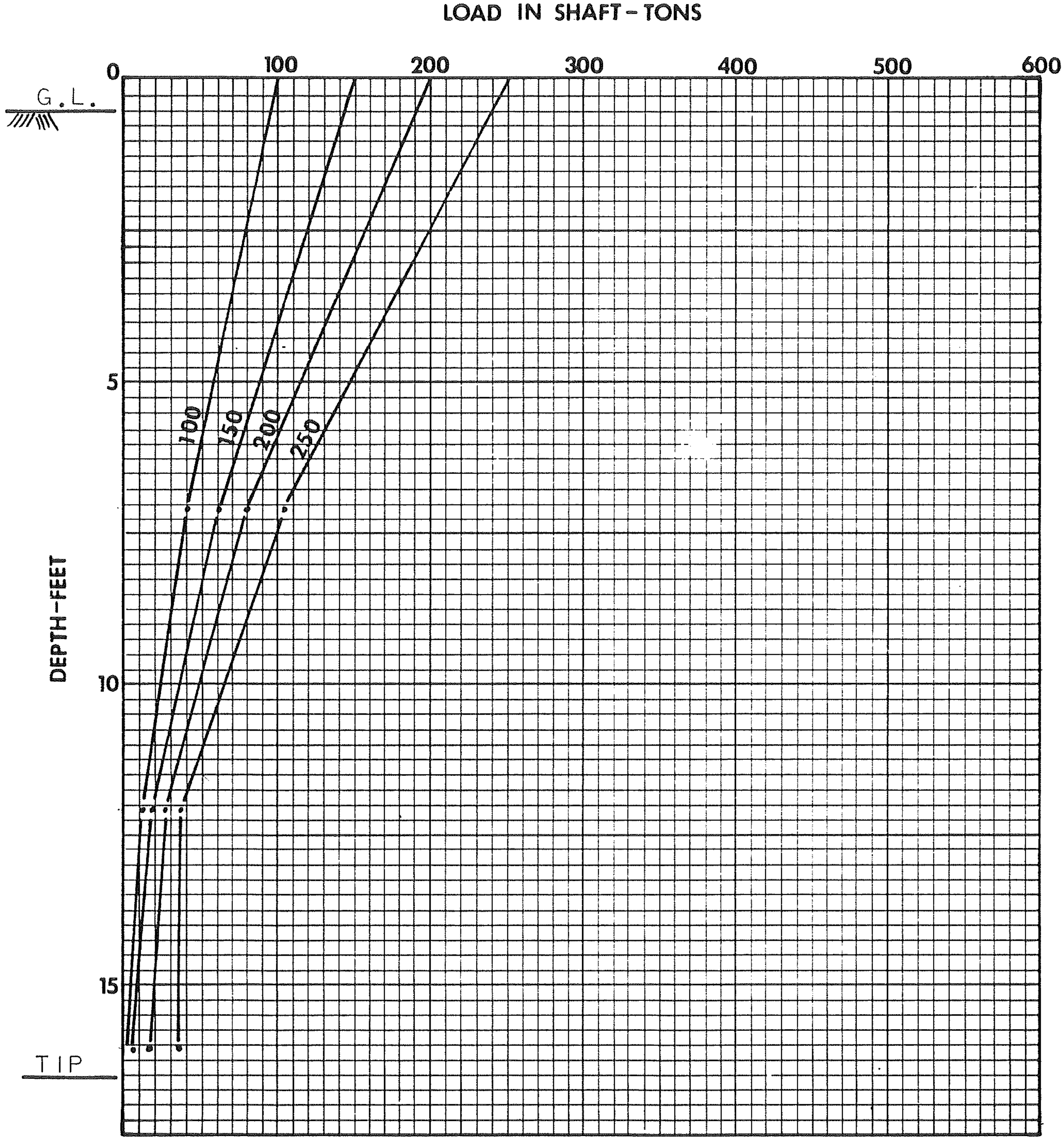
FIGURE 87 LOAD DISTRIBUTION ALONG SHAFT FOR VARIOUS LOADINGS



PROJECT Site B - Research Study Drilled Cast-in-Place Concrete Piles JOB NO. E71-272

TPB-10

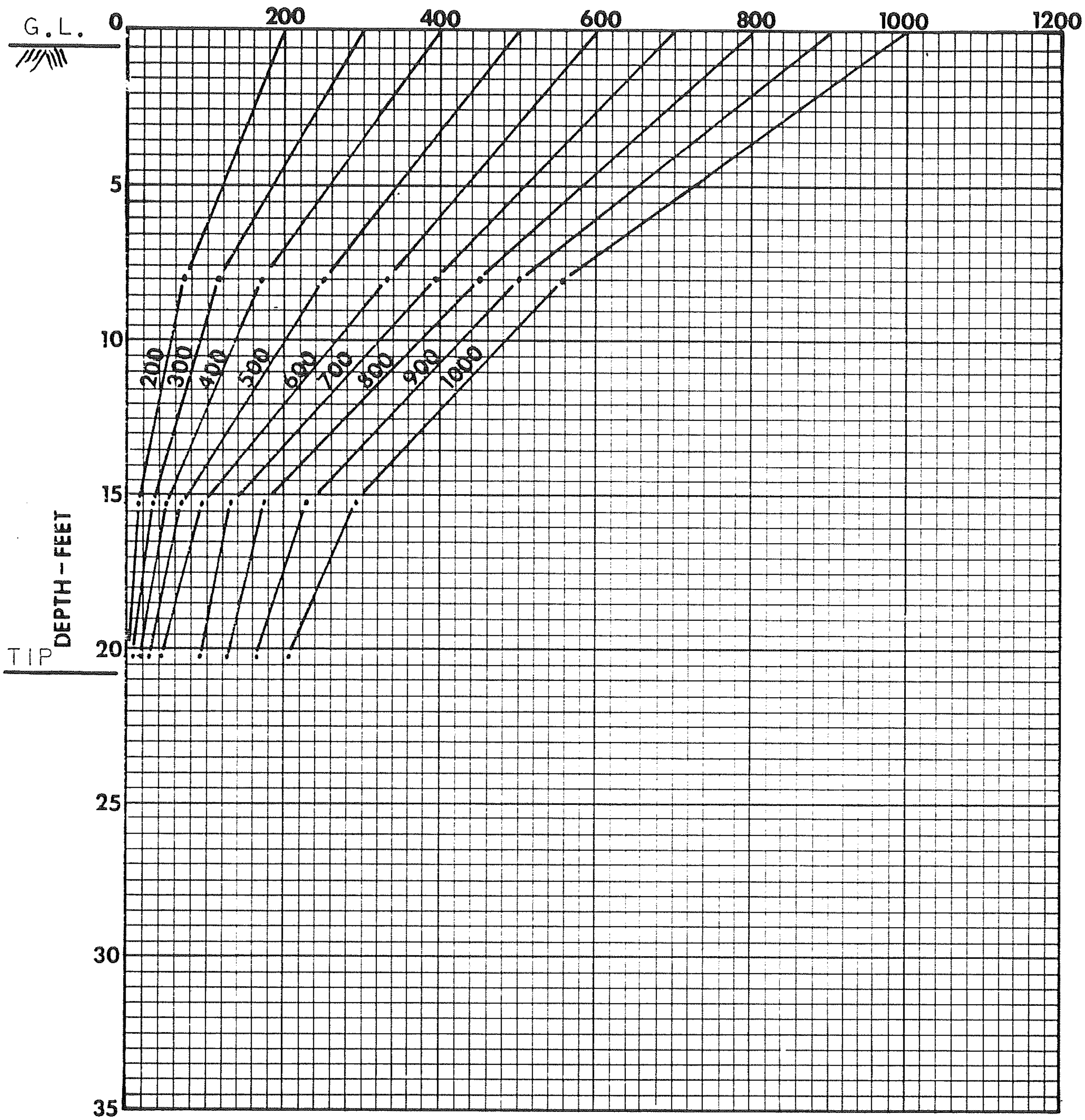
FIGURE 88 LOAD DISTRIBUTION ALONG SHAFT FOR VARIOUS LOADINGS



PROJECT SITE C - RESEARCH STUDY DRILLED CAST-IN-PLACE CONCRETE PILES JOB NO. E71-272

TPC-1

FIGURE 89 LOAD DISTRIBUTION ALONG SHAFT FOR VARIOUS LOADINGS

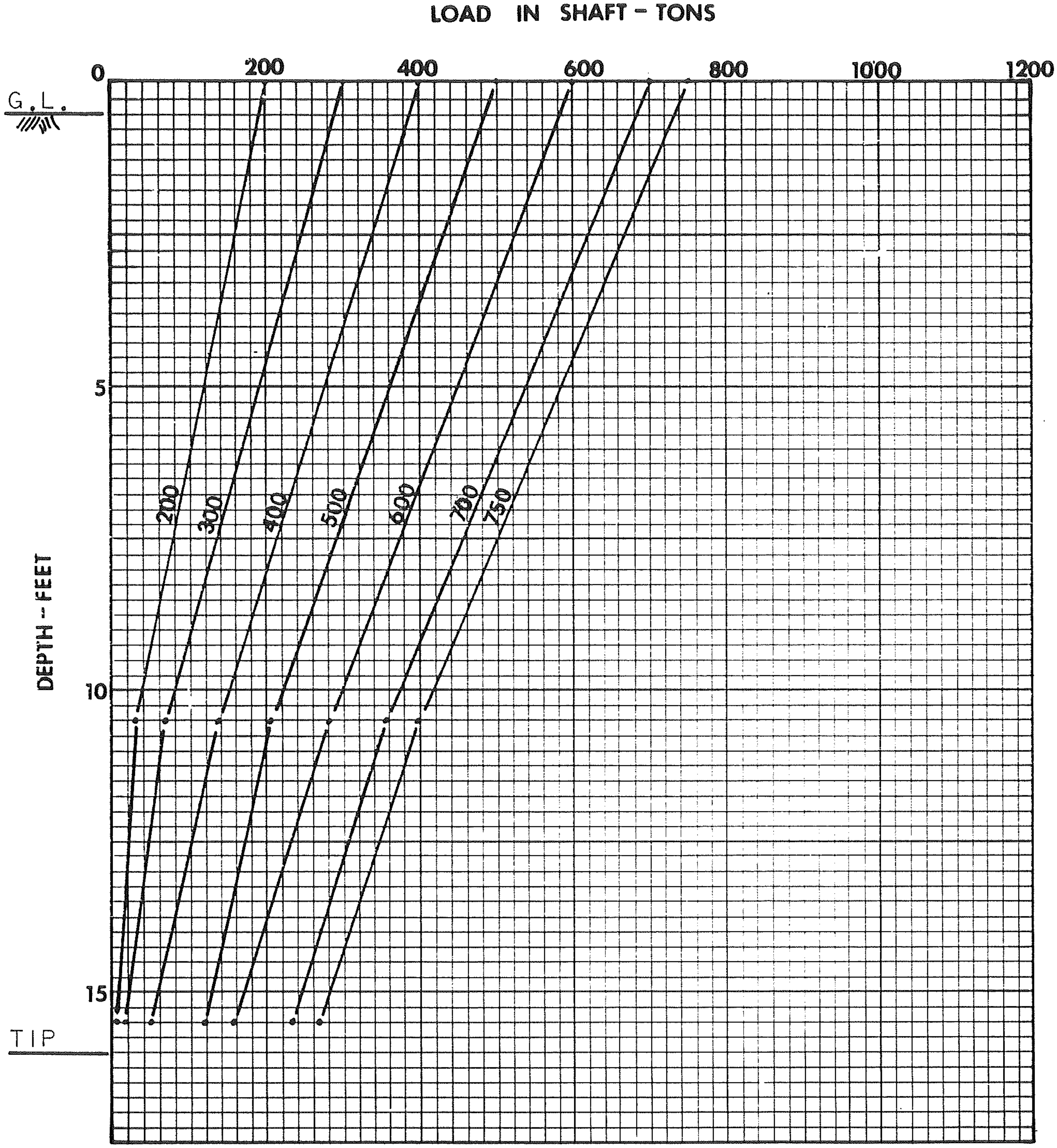


SITE C - RESEARCH STUDY

PROJECT DRILLED CAST-IN-PLACE CONCRETE PILES JOB NO. E71-272

TPC-2

FIGURE 90 LOAD DISTRIBUTION ALONG SHAFT FOR VARIOUS LOADINGS



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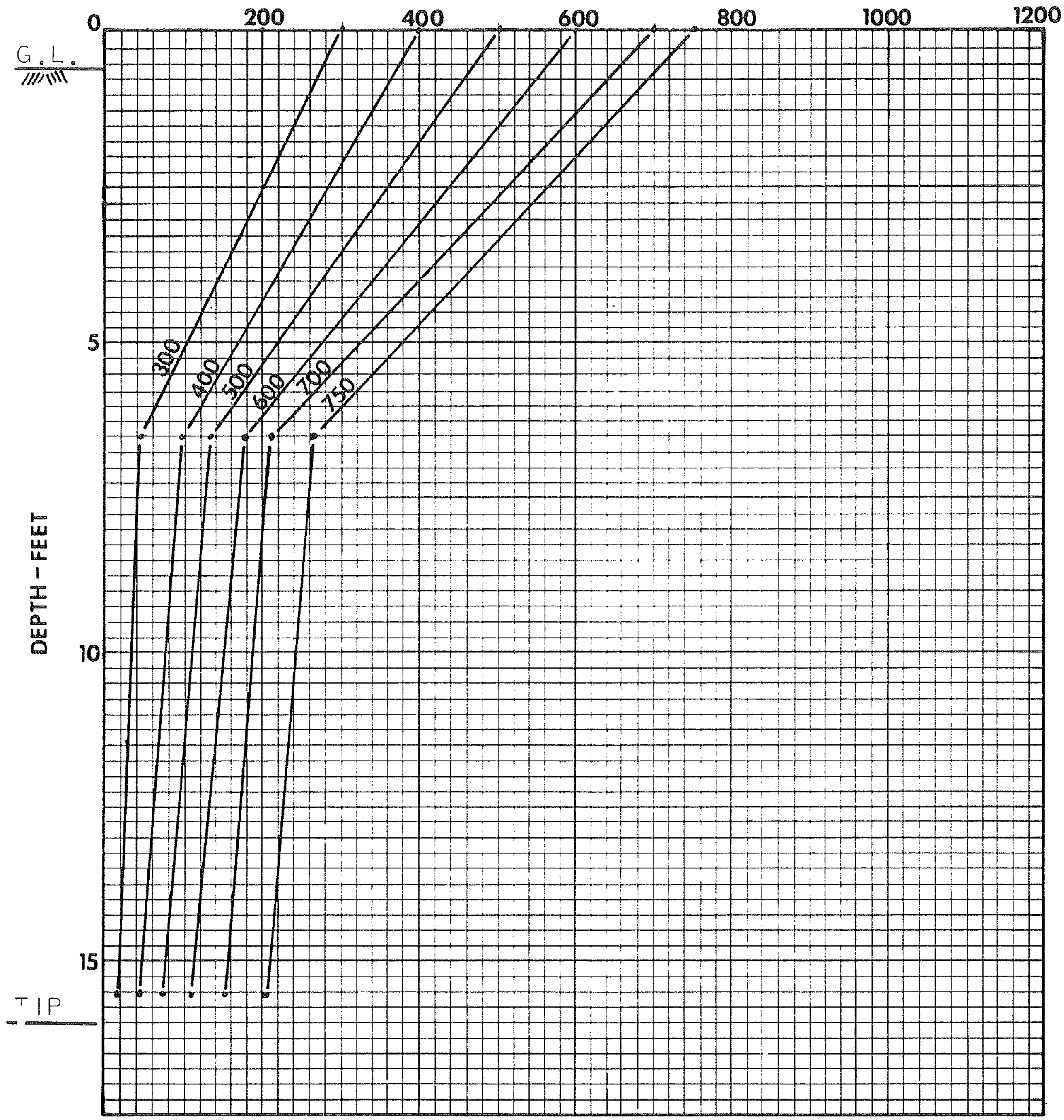
TPC-3

FIGURE 91 LOAD DISTRIBUTION ALONG SHAFT FOR VARIOUS LOADINGS



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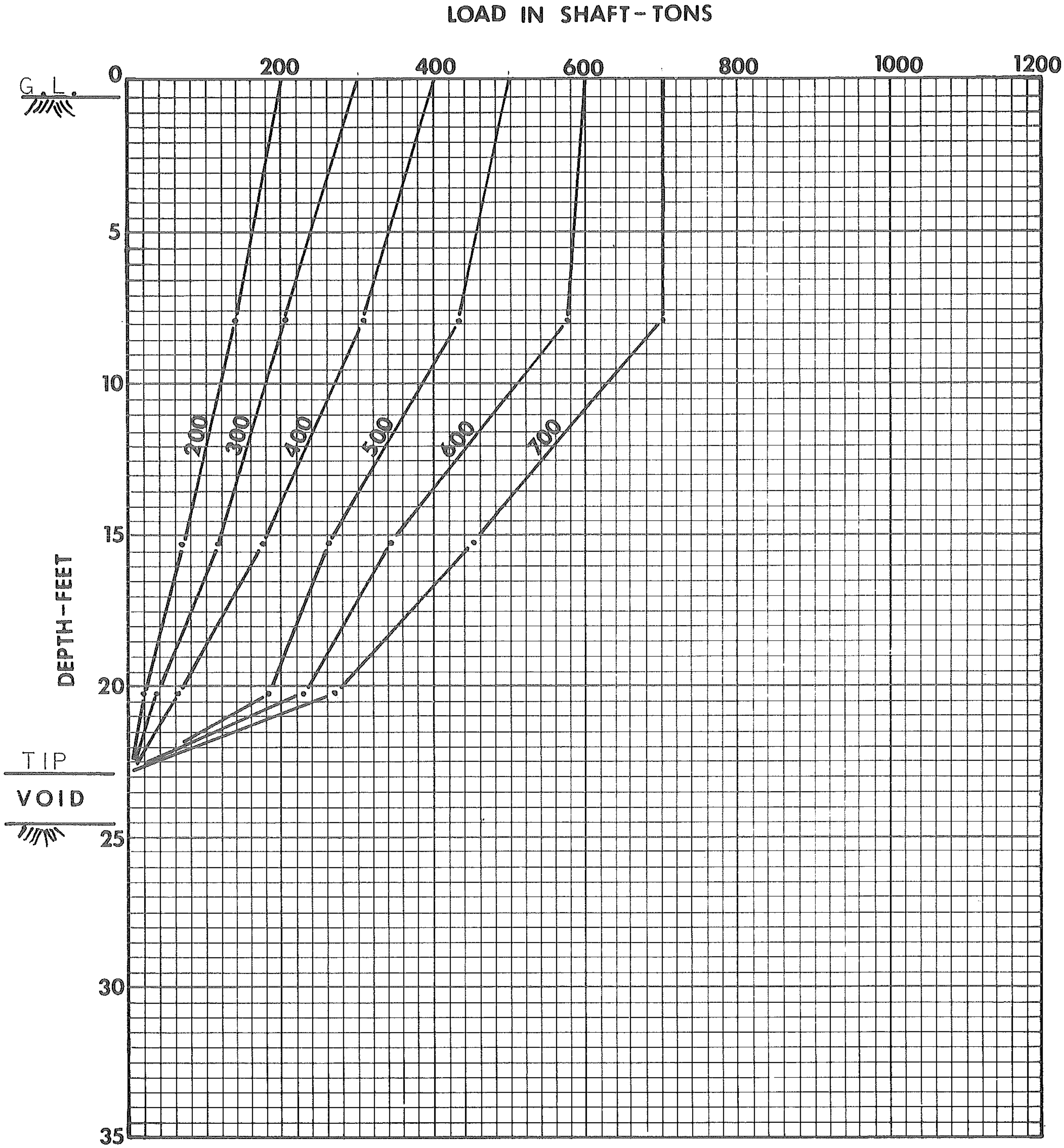
TPC-7

FIGURE 92 LOAD DISTRIBUTION ALONG SHAFT FOR VARIOUS LOADINGS



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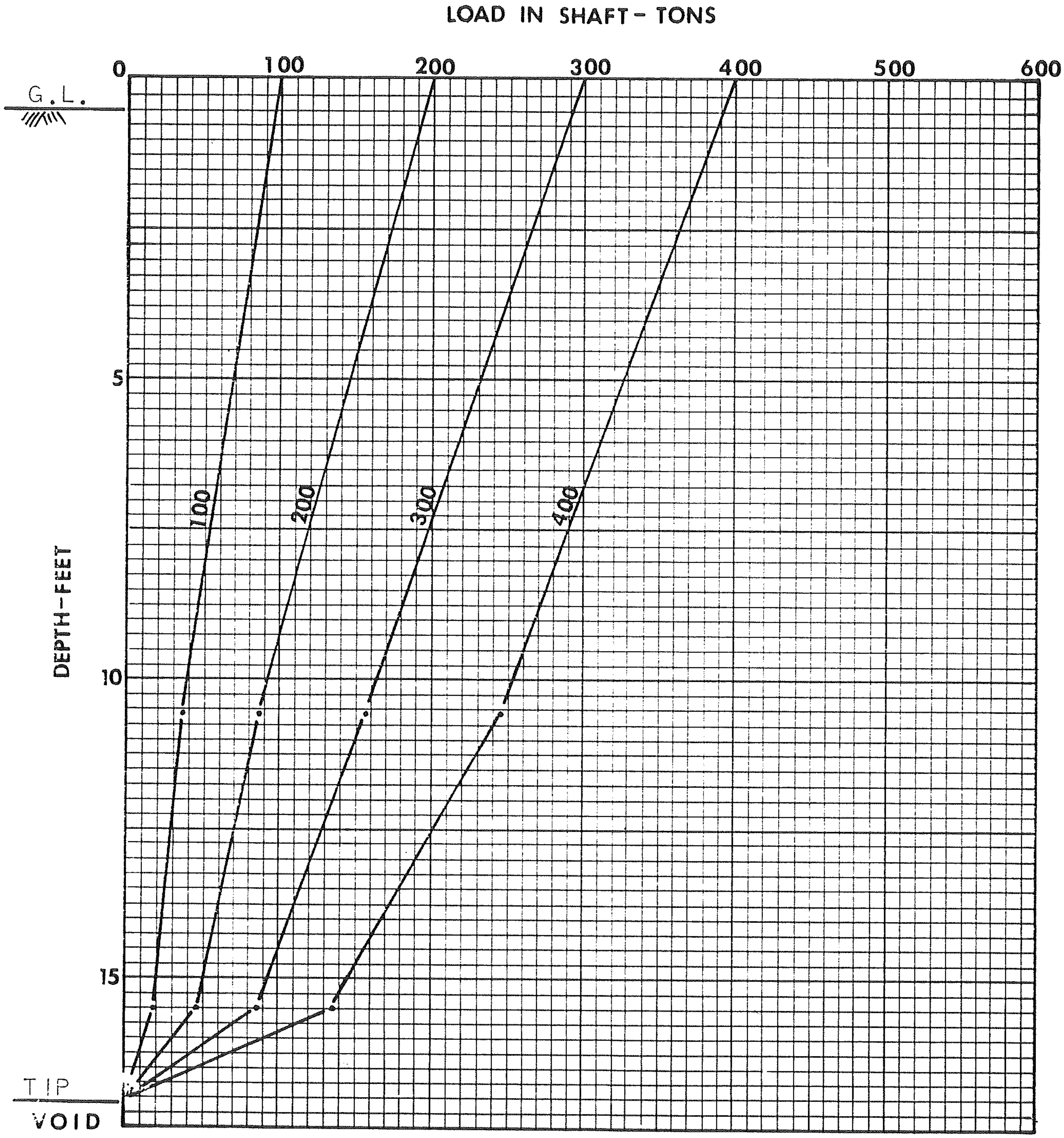
PROJECT SITE C - RESEARCH STUDY DRILLED CAST-IN-PLACE CONCRETE PILES JOB NO. E71-272

TPC-8

FIGURE 93 LOAD DISTRIBUTION ALONG SHAFT FOR VARIOUS LOADINGS



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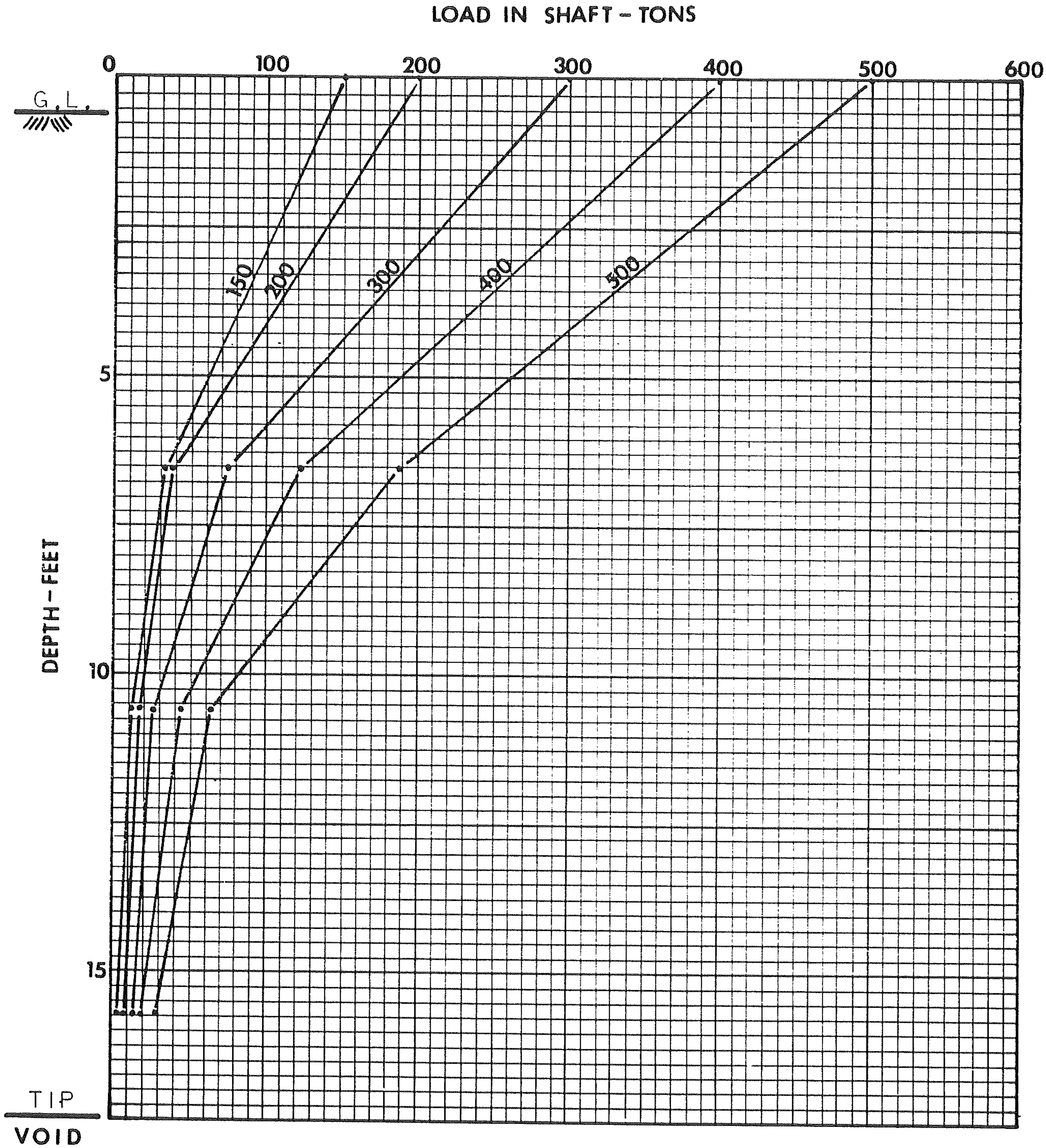
PROJECT SITE C - RESEARCH STUDY
DRILLED CAST-IN-PLACE CONCRETE PILES JOB NO. E71-272

TPC-9

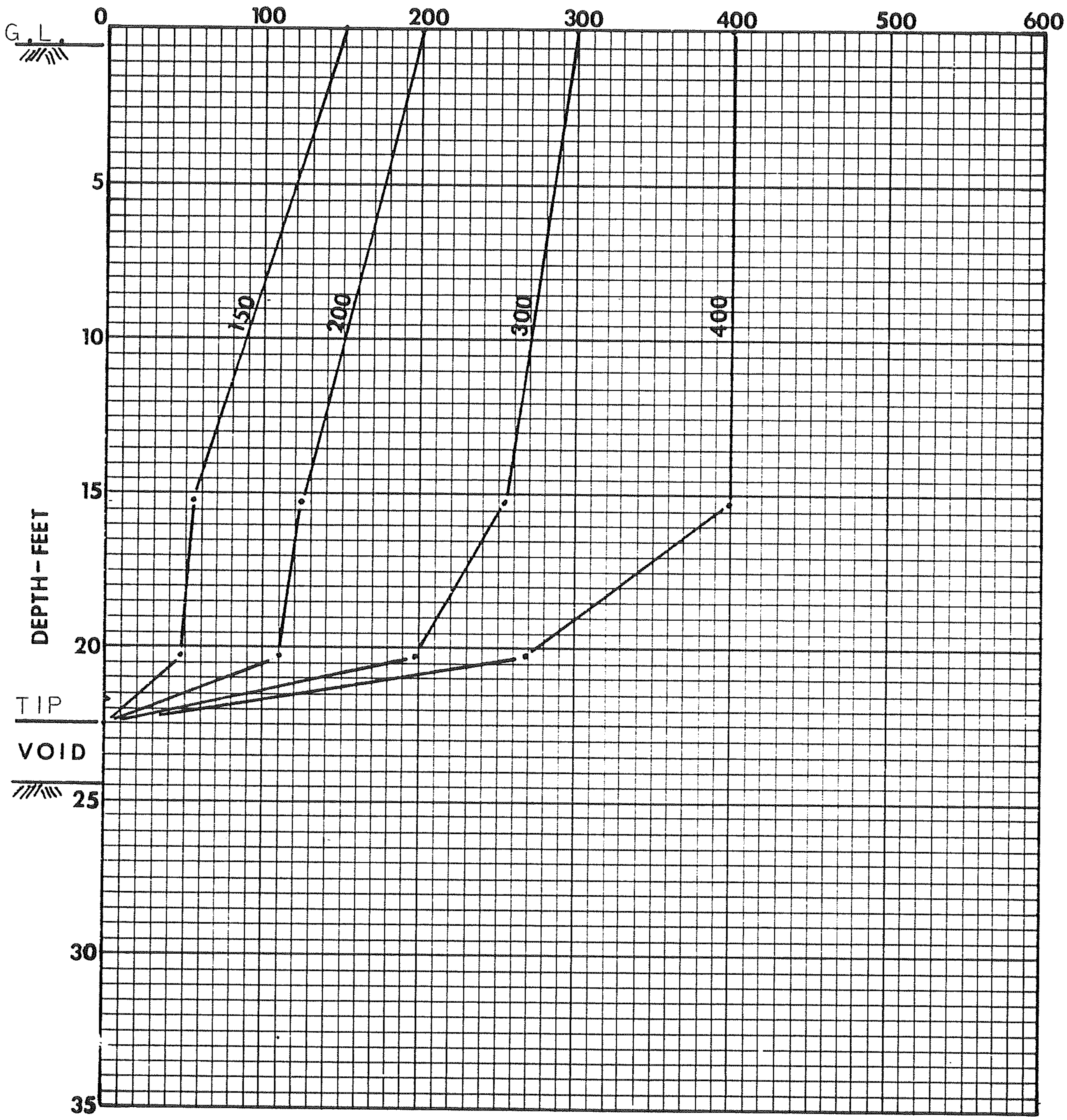
FIGURE 94 LOAD DISTRIBUTION ALONG SHAFT FOR VARIOUS LOADINGS



PROJECT Site C - Research Study
Drilled Cast-in-Place Concrete Piles JOB NO. E71-272

TPC-10

FIGURE 95 LOAD DISTRIBUTION ALONG SHAFT FOR VARIOUS LOADINGS



PROJECT Site A - Research Study Drilled Cast-in-Place Concrete Piles

LOG OF TEST BORING NO. 1A

JOB NO. E71-272 DATE 11-19-70

RIG TYPE CME-55
BORING TYPE 6½" Hollow Stem Auger
SURFACE ELEV.
DATUM

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows per foot 140 lb. 30" free-fall drop hammer	Dry Density Lbs. per cu. ft.	Moisture Content Per Cent of Dry Wt.	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
0	27			S	9		12	CL		SILTY CLAY, small amount of sand, medium plasticity, brown
	10									
	10			S	7		10			SILTY CLAY, small amount of gravel to ½", low to medium plasticity, light brown
	10									
	14									
5	15			S	9		11	CL		
	21									
	29			S	25		16		some clayey sand stratifi-cations	SANDY CLAY, moderately to strongly lime cemented, medium to high plasticity, light brown to tan
	71							CH-SC		
10	100/ 7 3/4"			S	69		12			
				S	35		15			CLAYEY GRAVEL, subrounded, moderately lime cemented, medium plasticity, light brown
				S	50/1¼" (no recovery)			GC		
15										Stopped auger at 14'6" Sampler refused at 14'7¼"

Penetrometer refused at 9'7 3/4"

GROUND WATER

DEPTH	HOUR	DATE
	NONE	

SAMPLE TYPE

A – Auger cuttings. B – Block sample
S – 2" O.D. 1.38" I.D. tube sample.
U – 3" O.D. 2.42" I.D. tube sample.
T – 3" O.D. thin-walled Shelby tube.

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PROJECT Site A - Research Study Drilled Cast-in-Place Concrete Piles

JOB NO. E71-272 DATE 12-3-70

LOG OF TEST BORING NO. 1A

RIG TYPE CME-55

BORING TYPE 6½" Hollow Stem Auger

SURFACE ELEV.

DATUM

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows per foot 140 lb. 30" free-fall drop hammer	Dry Density Lbs. per cu. ft.	Moisture Content Per Cent of Dry Wt.	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
0			☒	U	3-3-3-2-2-2 / 15	96	10	CL		
			☒	U	2-2-2-2-2-3 / 13	106	8			
5			☒	U	2-1-2-2-2-3 / 12	101	9	CL		
			☒	U	2-3-4-4-7-10 / 30	96	12			
10			☒	U	2-7-8-14-14-20 / 65	101	13	CH-SC		
			☒	U	2-4-9-10-18-46 / 43			GC		
15										Auger refused at 12' Stopped sampler at 13'

GROUND WATER

DEPTH	HOUR	DATE
	NONE	

SAMPLE TYPE

A – Auger cuttings. B – Block sample
S – 2" O.D. 1.38" I.D. tube sample.
U – 3" O.D. 2.42" I.D. tube sample.
T – 3" O.D. thin-walled Shelby tube.

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PROJECT Site A - Research Study
Drilled Cast-in-Place Concrete Piles

LOG OF TEST BORING NO. 2A

JOB NO. E71-272 DATE 11-19-70

RIG TYPE CME-55
BORING TYPE 6½" Hollow Stem Auger
SURFACE ELEV.
DATUM

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows per foot 140 lb. 30" free-fall drop hammer	Dry Density Lbs. per cu. ft.	Moisture Content Per Cent of Dry Wt.	Unified Soil Classification
0	12							
	6		X	S	4		13	CL
	8							
	10		X	S	6		11	
5	9							
	10		X	S	9		12	
	21							
	35		X	S	24		10	CL
	35							
10	33							
	36		X	S	29		10	
	14							
	12		X	S	17		12	SC
	96							
15	100/4"		X	S	251		4	GC-GP
20								

Penetrometer refused at 14'4"

VISUAL CLASSIFICATION

SILTY CLAY, some sand, low plasticity, brown

SANDY CLAY, small amount of gravel to ½", low to medium plasticity, light brown

note: becomes moderately cemented below 7'±

CLAYEY SAND, moderately lime cemented, medium plasticity, light brown to tan

CLAYEY GRAVEL, some sand, gravel to 3/4", subrounded, moderately to weakly lime cemented, medium plasticity, light brown to tan

Stopped auger at 14'6"
Stopped sampler at 16'

REMARKS

GROUND WATER

DEPTH	HOUR	DATE
	NONE	

SAMPLE TYPE

A – Auger cuttings. B – Block sample
S – 2" O.D. 1.38" I.D. tube sample.
U – 3" O.D. 2.42" I.D. tube sample.
T – 3" O.D. thin-walled Shelby tube.

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PROJECT Site A - Research Study Drilled Cast-in-Place Concrete Piles

JOB NO. E71-272 DATE 12-2-70

LOG OF TEST BORING NO. 2A

RIG TYPE CME-55

BORING TYPE 6½" Hollow Stem Auger

SURFACE ELEV.

DATUM

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows per foot 140 lb. 30" free-fall drop hammer	Dry Density Lbs. per cu. ft.	Moisture Content Per Cent of Dry Wt.	Unified Soil Classification
0								
			☒	U	2-2-2-2-2-2 12	99	11	CL
			☒	U	1-1-2-2-2-3 11			
5			☒	U	1-1-1-1-1-1 6	102	10	
			☒	U	4-4-4-4-4-6 26	104	11	CL
10			☒	U	3-3-5-7-9-11 38	100	11	
			☒	U	4 blows for 10" (no recovery)			SC
15			☒	U	6-19-20-34-24-44 103		8	GC
20								

REMARKS

VISUAL CLASSIFICATION

Stopped auger at 14'6"
Stopped sampler at 15'6"

GROUND WATER

DEPTH	HOUR	DATE
	NONE	

SAMPLE TYPE

A – Auger cuttings. B – Block sample
S – 2" O.D. 1.38" I.D. tube sample.
U – 3" O.D. 2.42" I.D. tube sample.
T – 3" O.D. thin-walled Shelby tube.

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PROJECT Site A - Research Study Drilled Cast-in-Place Concrete Piles

JOB NO. E71-272 DATE 11-20-70

LOG OF TEST BORING NO. 3A

RIG TYPE CMF-55
BORING TYPE 6½" Hollow Stem Auger
SURFACE ELEV.
DATUM

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows per foot 140 lb. 30" free-fall drop hammer	Dry Density Lbs. per cu. ft.	Moisture Content Per Cent of Dry Wt.	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
0	17									SILTY CLAY, medium plasticity, light brown
	18		X	S	8		4	CL		
	24									SANDY CLAY, weakly lime cemented, low to medium plasticity, light brown
	32		X	S	18		4			
5	34		X	S	19		6	CL		
	53									
	51									
	35		X	S	26		6			CLAYEY SAND, predominantly fine, moderately lime cemented, medium plasticity, some gravel, tan
	42									
10	39		X	S	34		7			
	41							SC		
	21									
	18		X	S	27		7			
	29		X	S	74		3	GC		CLAYEY GRAVEL, moderately lime cemented, medium plasticity, tan
15	86									
	100/ 9 3/4"									

Penetrometer refused at 15'9 3/4"

Stopped auger at 13'6"
Stopped sampler at 15'

GROUND WATER

DEPTH	HOUR	DATE
	NONE	

SAMPLE TYPE

A – Auger cuttings. B – Block sample
S – 2" O.D. 1.38" I.D. tube sample.
U – 3" O.D. 2.42" I.D. tube sample.
T – 3" O.D. thin-walled Shelby tube.

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Site A - Research Study
PROJECT Drilled Cast-in-Place Concrete Piles
JOB NO. E71-272 DATE 12-2-70

LOG OF TEST BORING NO. 3A

RIG TYPE CME-55
BORING TYPE 6½" Hollow Stem Auger
SURFACE ELEV.
DATUM

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows per foot 140 lb. 30" free-fall drop hammer	Dry Density Lbs. per cu. ft.	Moisture Content Per Cent of Dry Wt.	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
0			X	U	14	92	7	CL		
			X	U	17	95	7			
5			X	U	19	90	8	CL		
			X	U	32					
10			X	U	104	99	10	SC		
			X	U	3-5-5-5-11-6 35	103	7			
15			X	U	5-9-9-9-12-12 56 (no recovery)			GC		
20										

Stopped auger at 14'6"
Stopped sampler at 15'6"

GROUND WATER

DEPTH	HOUR	DATE
	NONE	

SAMPLE TYPE
A – Auger cuttings. B – Block sample
S – 2" O.D. 1.38" I.D. tube sample.
U – 3" O.D. 2.42" I.D. tube sample.
T – 3" O.D. thin-walled Shelby tube.

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PROJECT Site A - Research Study Drilled Cast-in-Place Concrete Piles

JOB NO. E71-272 DATE 11-20-70

LOG OF TEST BORING NO. 4A

RIG TYPE CME-55

BORING TYPE 6½" Hollow Stem Auger

SURFACE ELEV.

DATUM

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows per foot 140 lb. 30" free-fall drop hammer	Dry Density Lbs. per cu. ft.	Moisture Content Per Cent of Dry Wt.	Unified Soil Classification	Remarks	Visual Classification
0	15									
	13		X	S	8		5	CL		SILTY CLAY, low plasticity, light brown
	17									
	17		X	S	13		9			SANDY CLAY, low to medium plasticity, light brown
	3							CL		
5	27		X	S	19		7			
	32									
	31		X	S	27		8			CLAYEY SAND, some gravel, moderately to strongly lime cemented, medium plasticity, tan
	100/ 10½"							SC		
10			X	S	54		7			
			X	S	41		6			CLAYEY GRAVEL, weakly to moderately lime cemented, medium plasticity, tan
								GC		
15			X	S	60		2			
20										

Penetrometer refused at 8'10½"

Auger refused at 13'6"
Stopped sampler at 15'

GROUND WATER

DEPTH	HOUR	DATE
	NONE	

SAMPLE TYPE

A – Auger cuttings. B – Block sample
S – 2" O.D. 1.38" I.D. tube sample.
U – 3" O.D. 2.42" I.D. tube sample.
T – 3" O.D. thin-walled Shelby tube.

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PROJECT Site A - Research Study Drilled Cast-in-Place Concrete Piles

JOB NO. E71-272 DATE 12-2-70

LOG OF TEST BORING NO. 4A

RIG TYPE CME-55

BORING TYPE $6\frac{1}{2}$" Hollow Stem Auger

SURFACE ELEV.

DATUM -

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows per foot 140 lb. 30" free-fall drop hammer	Dry Density Lbs. per cu. ft.	Moisture Content Per Cent of Dry Wt.	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
0										
				U	15	88	7	CL		
				U	13	94	6			
5				U	28	94	8	CL		
				U	29	78	11			
10				U	70			SC		
				U	110/10"	97	6	GC		
15										Auger refused at 12' Sampler refused at 12'10"

GROUND WATER

DEPTH	HOUR	DATE
	NONE	

SAMPLE TYPE

A – Auger cuttings. B – Block sample

S – 2" O.D. 1.38" I.D. tube sample.

U – 3" O.D. 2.42" I.D. tube sample.

T – 3" O.D. thin-walled Shelby tube.

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TABULATION C. TEST RESULTS

Job No. E71-272 **Date** ______

Client:

Project Site A - Research Study

Drilled Cast-in-Place Concrete Piles

Material ______

Source ______

HOLE NO.	LOCATION	DEPTH	UNIFIED CLASS.	LL	PI	SIEVE ANALYSIS – ACCUM. % PASSING												LAB. NO.
						200	100	40	16	10	4	1/4	3/8	3/4	1	1½	2	
1A	See Site Plan	6"-2'	CL	29	13	64	80	90	96	97	98	99	100					2566-2
1A	See Site Plan	2½'-4'	CL	25	8	54	68	84	90	91	94	98	99	100				2566-3
1A	See Site Plan	4½'-6'	CL	34	17	58	70	80	87	90	94	96	98	100				2566-4
1A	See Site Plan	7'-8½'	CH	61	37	62	72	80	88	89	90	92	94	100				2566-5
1A	See Site Plan	9½'-11'	SC	47	25	46	57	63	76	78	84	86	92	100				2566-6
1A	See Site Plan	12'-12½'	SC	39	17	44	56	76	85	87	92	93	96	100				2566-7
1A	See Site Plan	12½'-13½'	GC	49	26	14	18	22	28	30	40	46	59	92	100			2566-8
2A	See Site Plan	6"-2'	CL	26	10	65	79	88	96	97		99	100					2566-9
2A	See Site Plan	2½'-4'	CL	27	10	56	70	86	94	95	97	98	100					2566-10
2A	See Site Plan	4½'-6'	CL	29	13	56	70	84	92	94	97	99	100					2566-11
2A	See Site Plan	7'-8½'	CL	39	20	52	64	76	84	86	98	100						2566-12
2A	See Site Plan	9½'-11'	CL	34	15	52	62	72	78	80	86	90	96	100				2566-13
2A	See Site Plan	12'-13½'	SC	38	22	44	57	70	80	81	87	90	93	100				2566-14
2A	See Site Plan	14½'-16'	GC-GP	38	17	12	15	24	34	38	48	51	60	84	100			2566-15
3A	See Site Plan	6"-2'	CL	29	13	62	74	86	93	94	95		96	100				2566-15

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TABULATION OF TEST RESULTS

Job No. E71-272 **Date** ______

Client:

Project Site A - Research Study

Drilled Cast-in-Place Concrete Piles

Material ______

Source ______

HOLE NO.	LOCATION	DEPTH	UNIFIED CLASS.	LL	PI	SIEVE ANALYSIS – ACCUM. % PASSING												LAB. NO.
						200	100	40	16	10	4	1/4	3/8	3/4	1	1½	2	
3A	See Site Plan	2½'-4'	CL	26	9	54	70	84	92			96	98	100				2566-17
3A	See Site Plan	4½'-6'	CL	33	16	56	68	82		90	93	95	99	100				2566-18
3A	See Site Plan	7'-8½'	SC	45	24	36	47	58	67	70	76	81	88	97	100			2566-19
3A	See Site Plan	9½'-11'	SC	37	20	42	56	73	88	91	94	96	98	100				2566-20
3A	See Site Plan	12'-13½'				41	53	70	83	84	87	89	92	95	100			2566-21
3A	See Site Plan	13½'-15'				12	17	22	28	30	36	40	46	60	100			2566-22
4A	See Site Plan	6"-2'				62	77	90	96	97	99	100						2566-23
4A	See Site Plan	2½'-4'				56	70	84	92			96	98	100				2566-24
4A	See Site Plan	4½'-6'				56	68	80	86		92	94	98	100				2566-25
4A	See Site Plan	7'-8½'				48	60	74	82	84	88	90	94	100				2566-26
4A	See Site Plan	9½'-11'				46	54	68	81	84	90	92	96	100				2566-27
4A	See Site Plan	12'-13½'				24	32	52	70	74	81	84	89	100				2566-28
4A	See Site Plan	13½'-15'				10	14	20	27	30	40	45	52	67	85	100		2566-29

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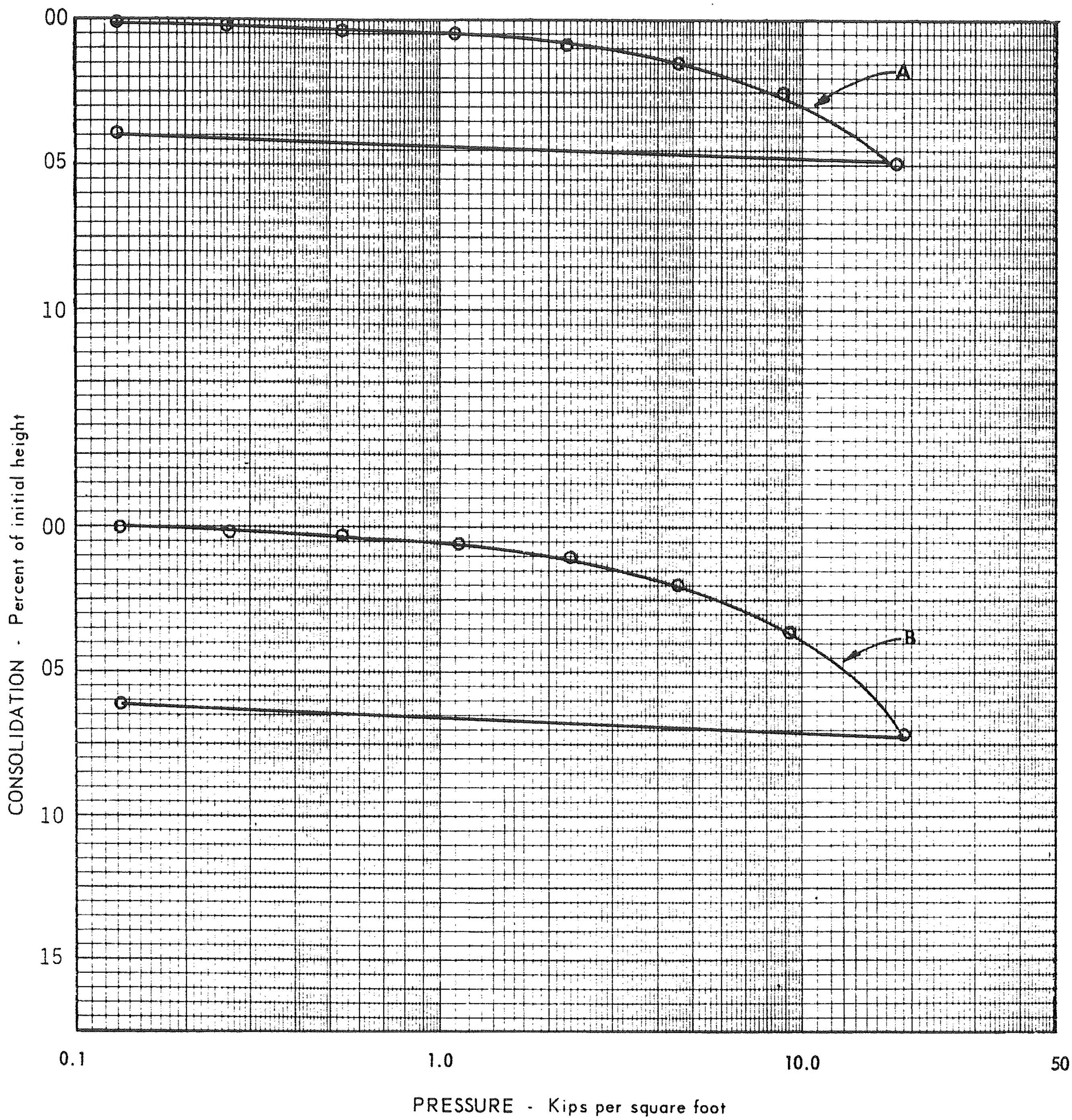
SUMMARY OF CONSOLIDATION TESTS

Site A - Research Study

PROJECT Drilled Cast-in-Place Concrete Piles **JOB NO.** E71-272



CURVE	SAMPLE	INITIAL DRY DENSITY LBS./CU. FT.	MOISTURE CONTENT % DRY WEIGHT INITIAL	MOISTURE CONTENT % DRY WEIGHT FINAL	UNIFIED SOIL CLASSIFICATION
A	1A @ 2½'-3½'	105.5	8.3	-	CL
B	1A @ 4½'-5½'	101.4	9.1	-	CL

SOIL MOISTURE CONDITION

——— INSITU

– – – – SUBMERGED

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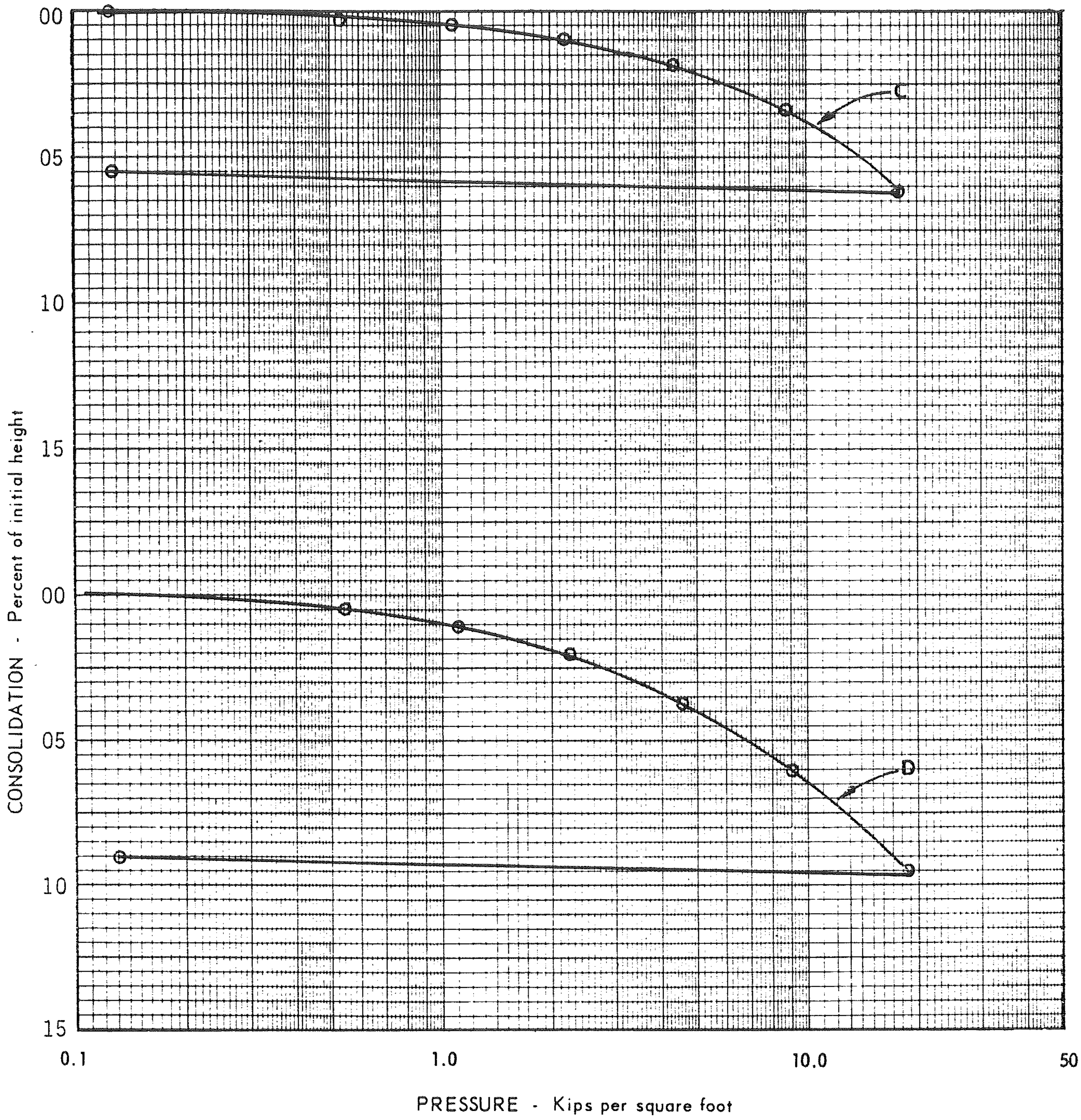
SUMMARY OF CONSOLIDATION TESTS

PROJECT Site A - Research Study Drilled Cast-in-Place Concrete Piles JOB NO. E71-272



CURVE	SAMPLE	INITIAL DRY DENSITY LBS./CU. FT.	MOISTURE CONTENT % DRY WEIGHT		UNIFIED SOIL CLASSIFICATION
			INITIAL	FINAL	
C	1A @ 9½'-10½'	101.1	12.9	-	SC
D	1A @ 12'-13'	102.7	7.3	-	SC

SOIL MOISTURE CONDITION

———	INSITU
— — — —	SUBMERGED

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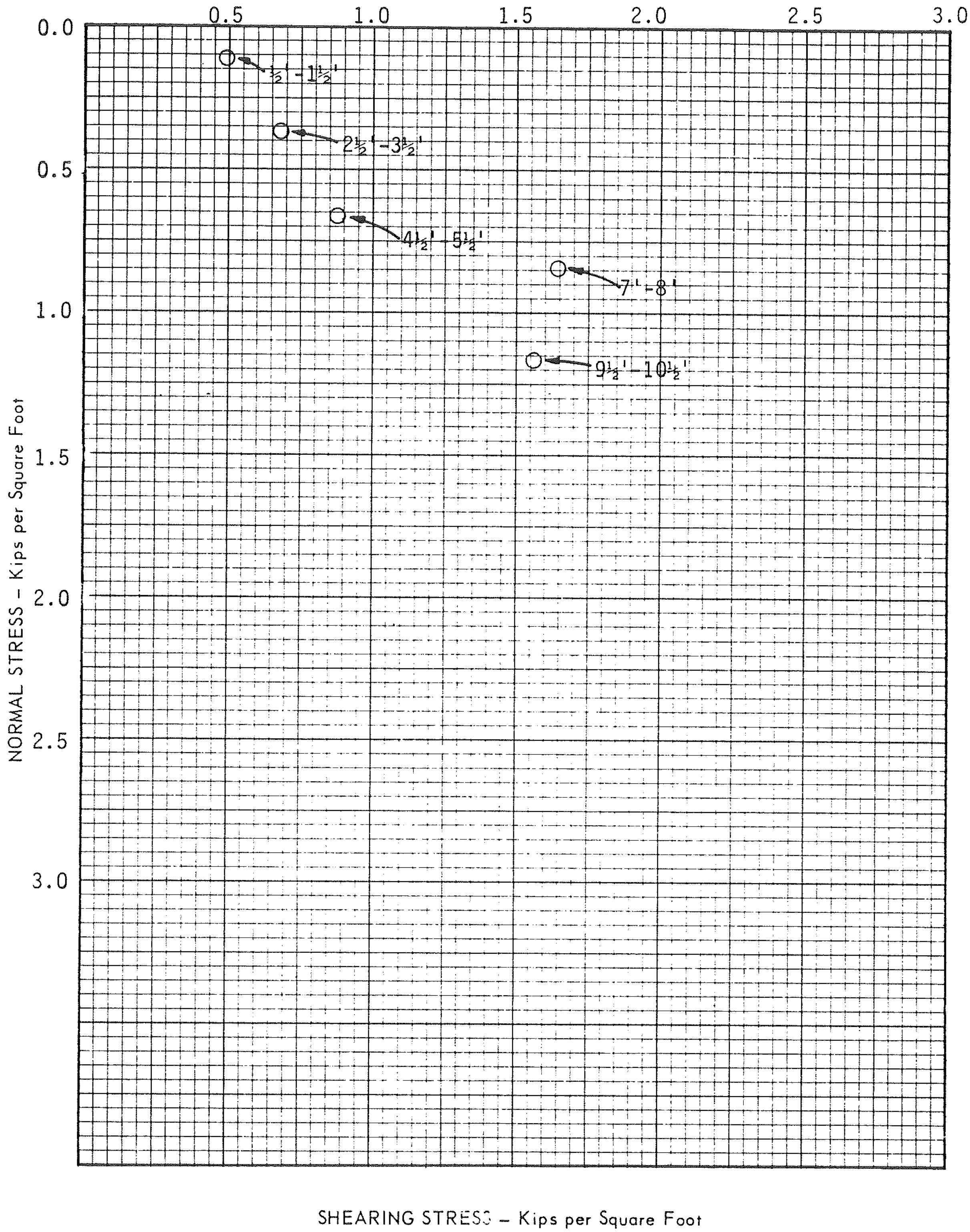
SUMMARY OF DIRECT SHEAR TESTS

PROJECT Site A - Research Study Drilled Cast-in-Place Concrete Piles JOB NO. E71-272

Test Boring No. 1A



SOIL MOISTURE CONDITION

○ – INSITU
● – SUBMERGED

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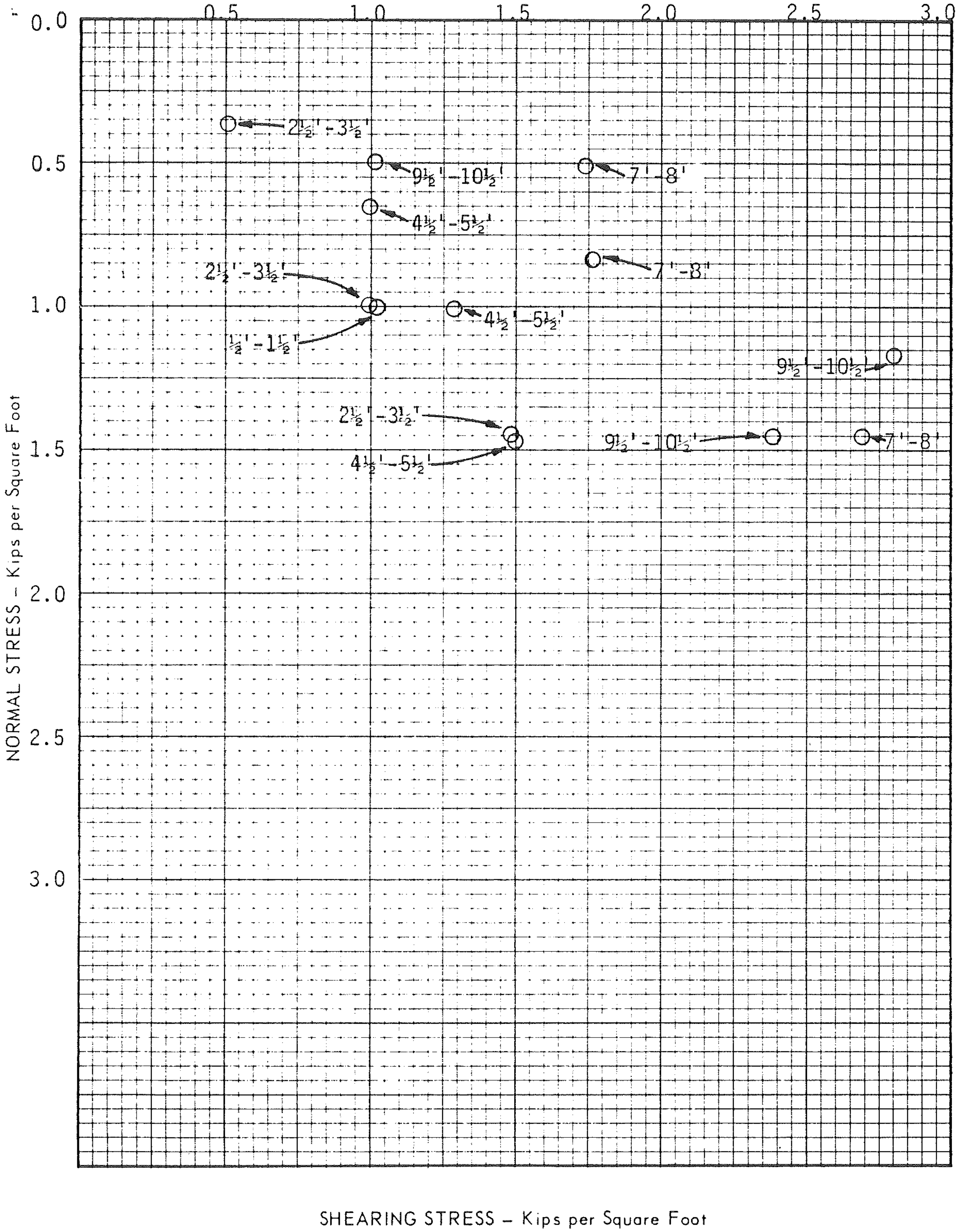
SUMMARY OF DIRECT SHEAR TESTS

PROJECT Site A - Research Study Drilled Cast-in-Place Concrete Piles JOB NO. E71-272

Test Boring No. 2A



SOIL MOISTURE CONDITION

○ – INSITU
● – SUBMERGED

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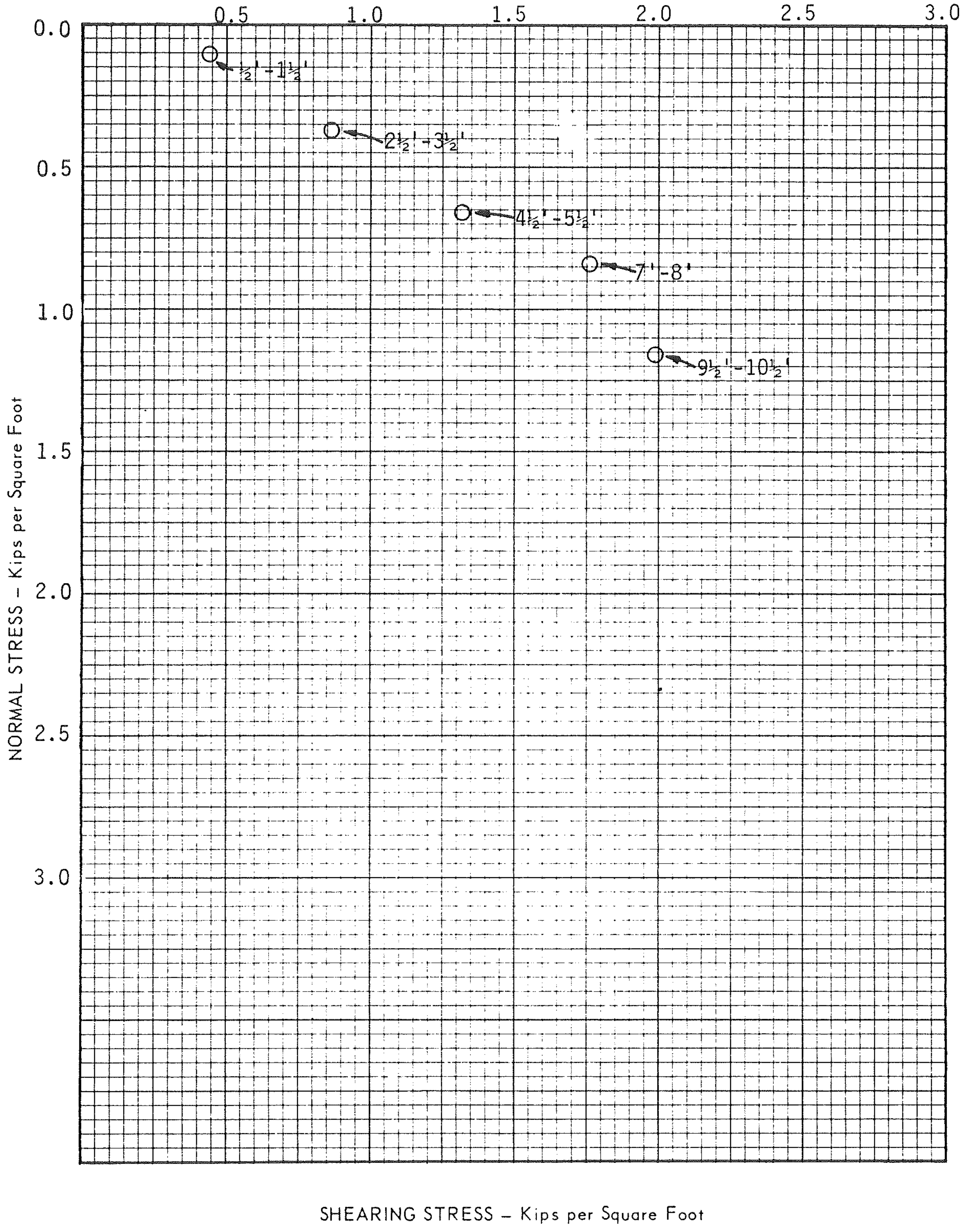
SUMMARY OF DIRECT SHEAR TESTS

PROJECT Site A - Research Study Drilled Cast-in-Place Concrete Piles JOB NO. E71-272

Test Boring No. 3A



SOIL MOISTURE CONDITION

○ – INSITU
● – SUBMERGED

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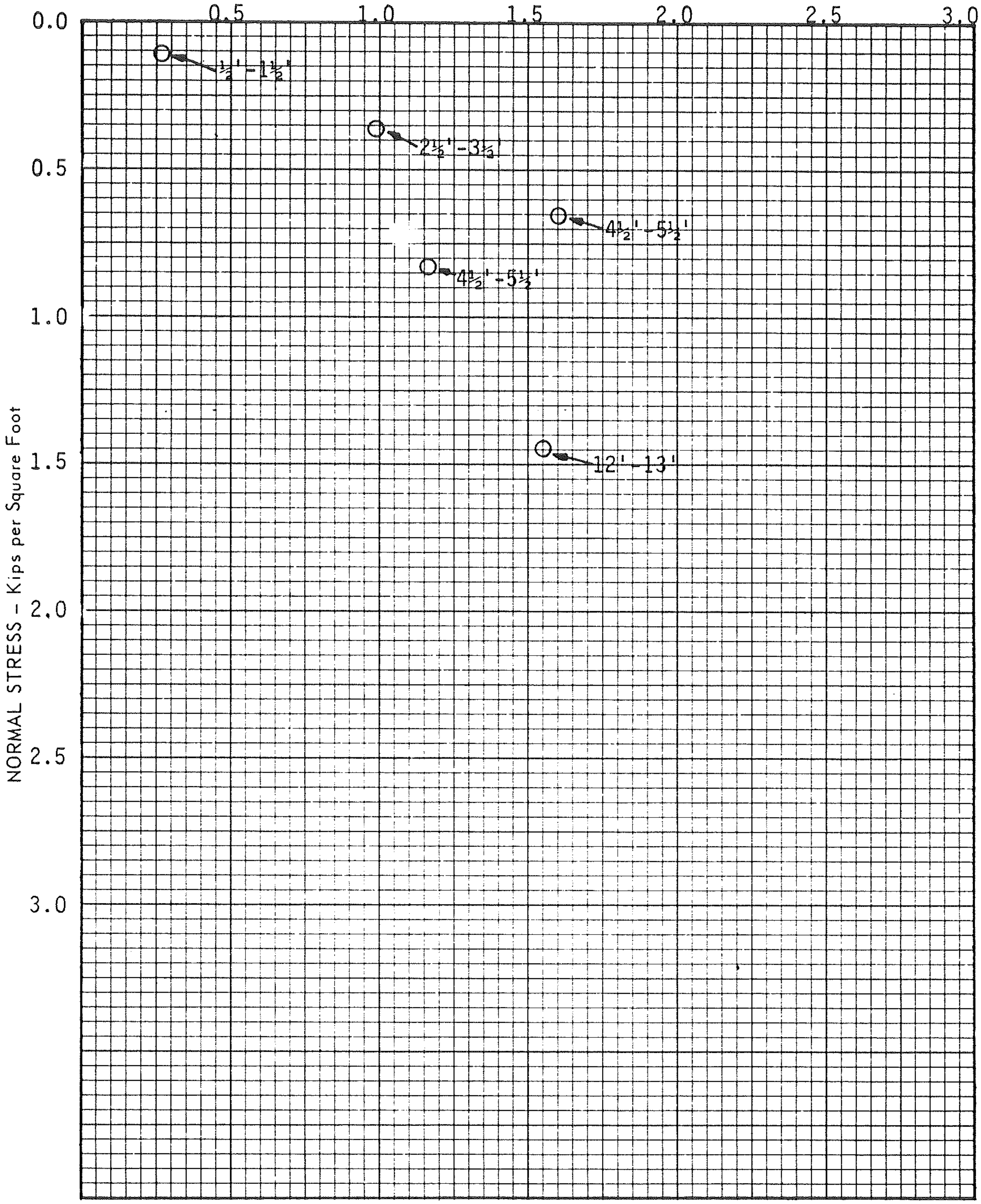
SUMMARY OF DIRECT SHEAR TESTS

Site A - Research Study

PROJECT Drilled Cast-in-Place Concrete Piles **JOB NO** E71-272

Test Boring No. 4A



SHEARING STRESS – Kips per Square Foot

SOIL MOISTURE CONDITION

○ – INSITU
● – SUBMERGED

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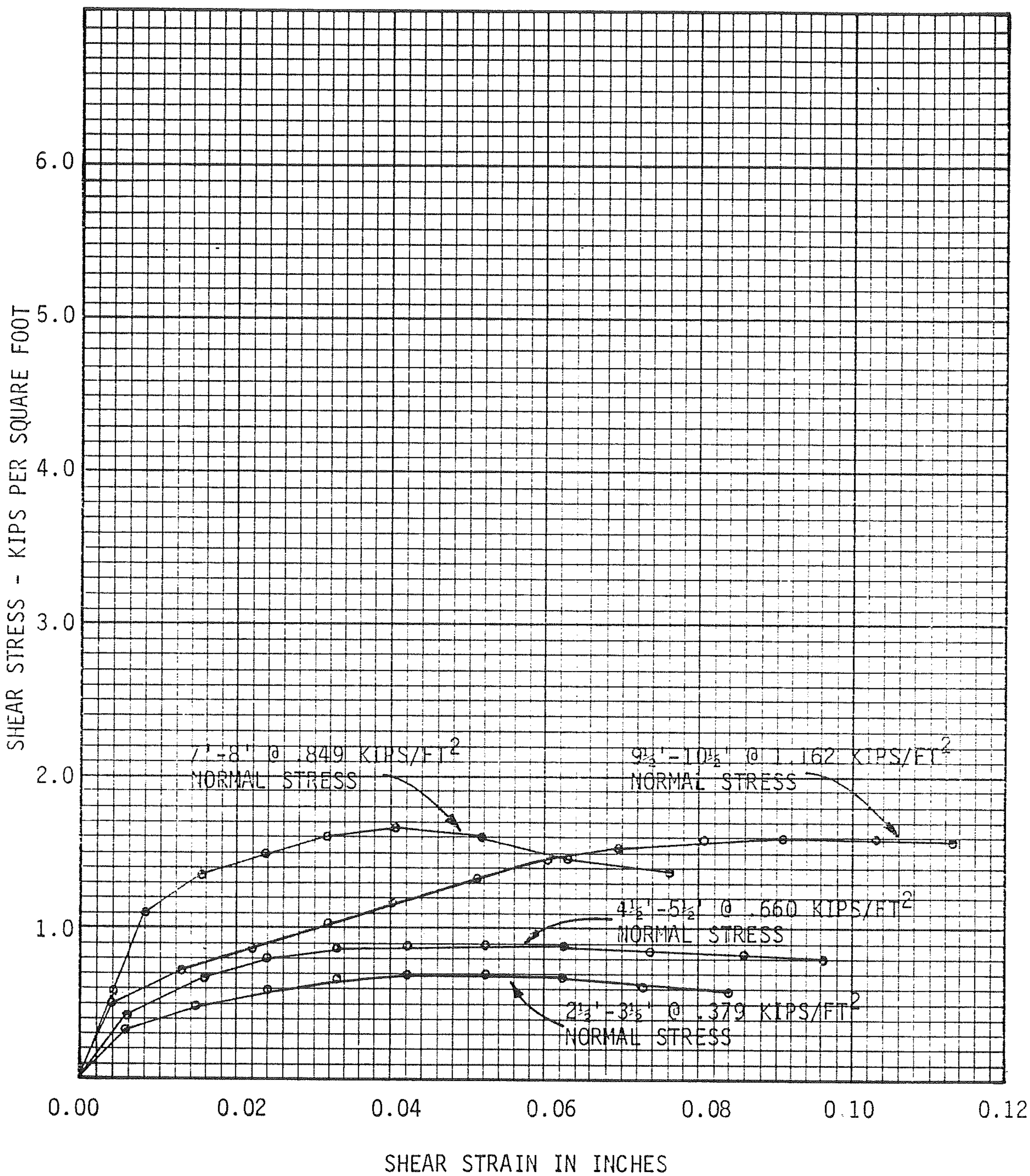
PROJECT Site A - Research Study
Drilled Cast-in-Place Concrete Piles JOB NO. E71-272

DIRECT SHEAR TEST DATA

Test Boring No. 1A



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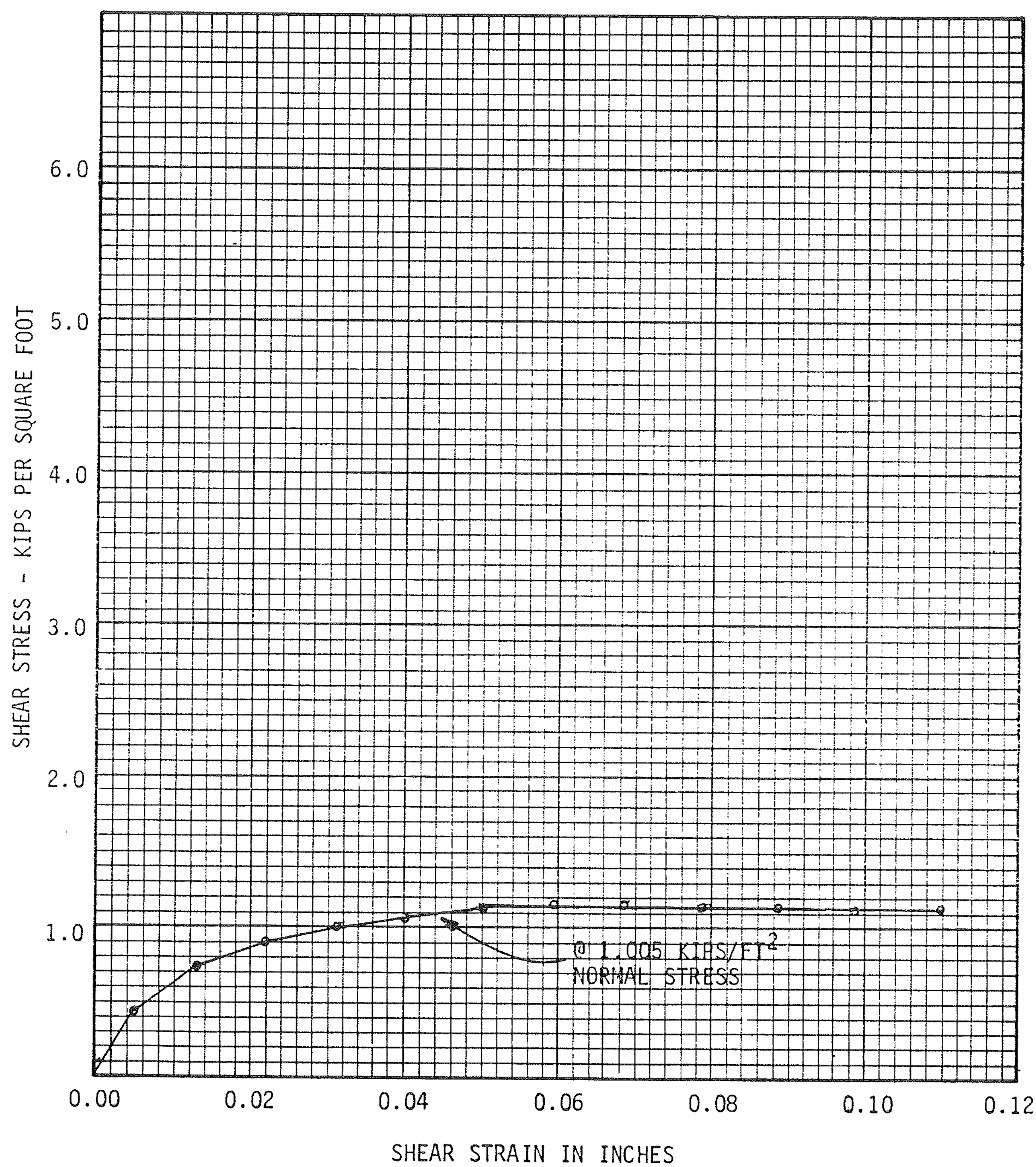
PROJECT Site A - Research Study
Drilled Cast-in-Place Concrete Piles JOB NO. E71-272

DIRECT SHEAR TEST DATA

Test Boring No. 2A, ½'-1½'



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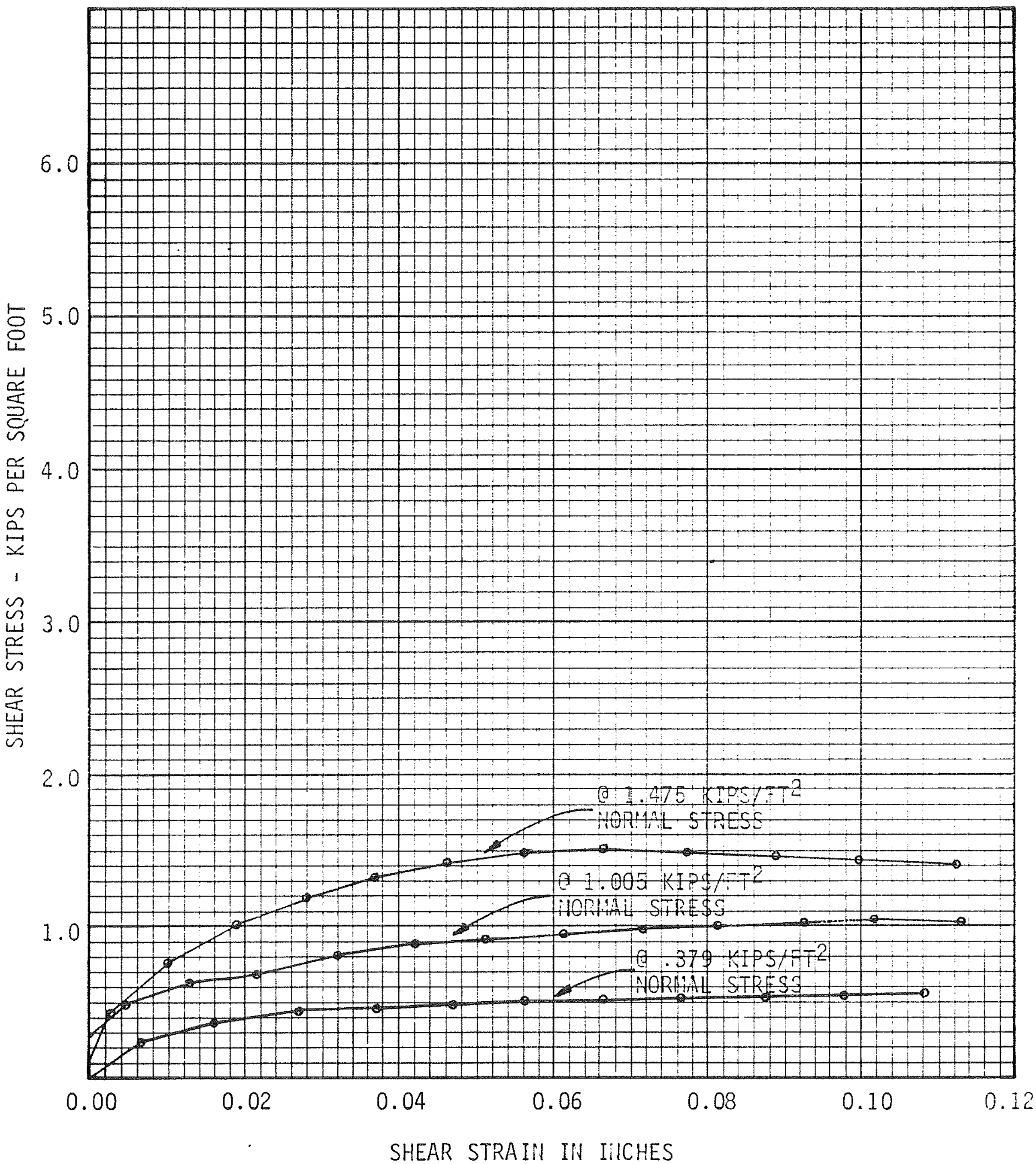
Site A - Research Study

PROJECT Drilled Cast-in-Place Concrete Piles **JOB NO.** E71-272

DIRECT SHEAR TEST DATA

Test Boring No. 2A, 2½'-3½'



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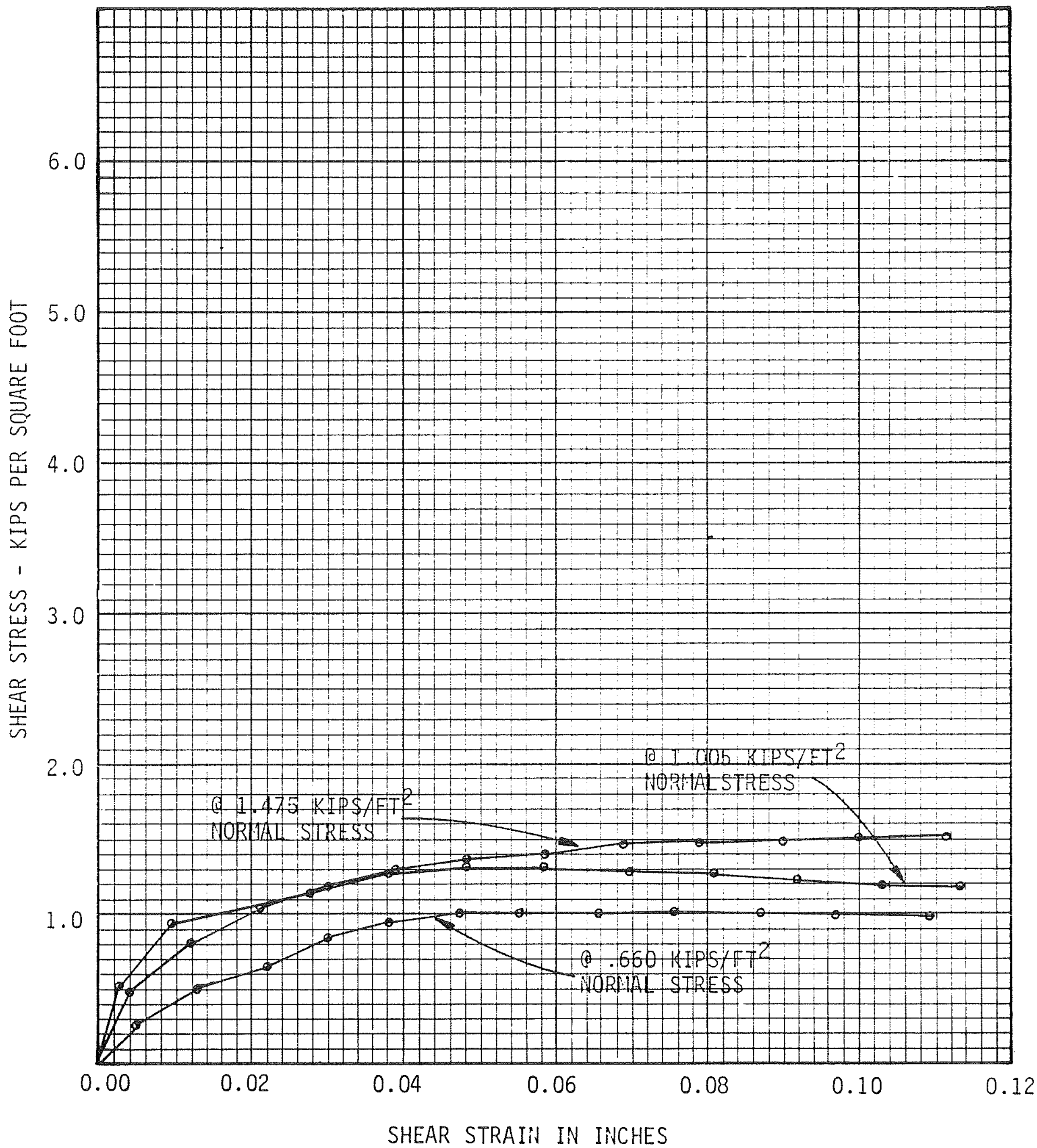
PROJECT Drilled Cast-in-Place Concrete Piles **JOB NO.** E71-272

DIRECT SHEAR TEST DATA

Test Boring No. 2A, 4½'-5½'



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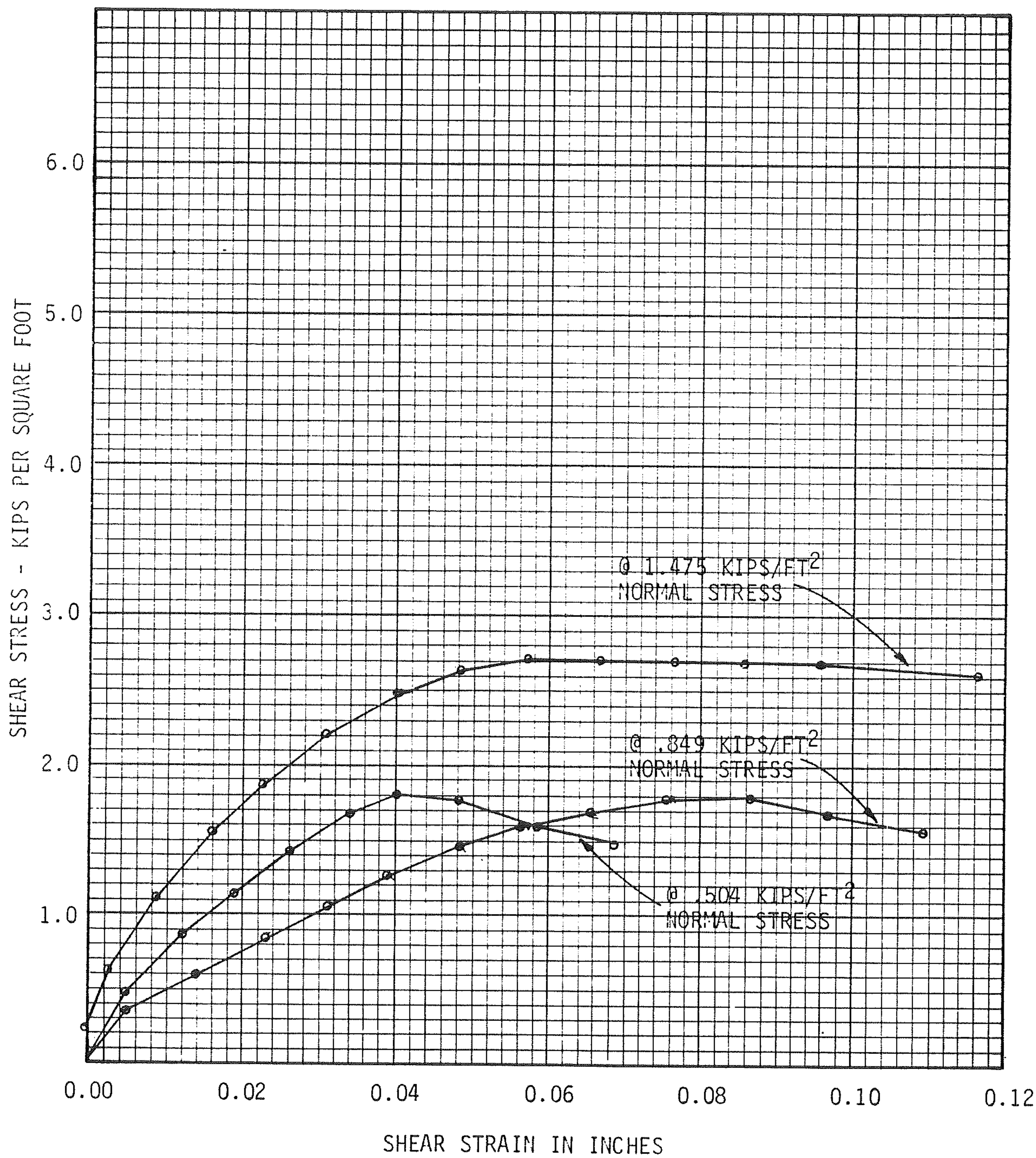
PROJECT Drilled Cast-in-Place Concrete Piles **JOB NO.** E71-272

DIRECT SHEAR TEST DATA

Test Boring No. 2A, 7'-8'



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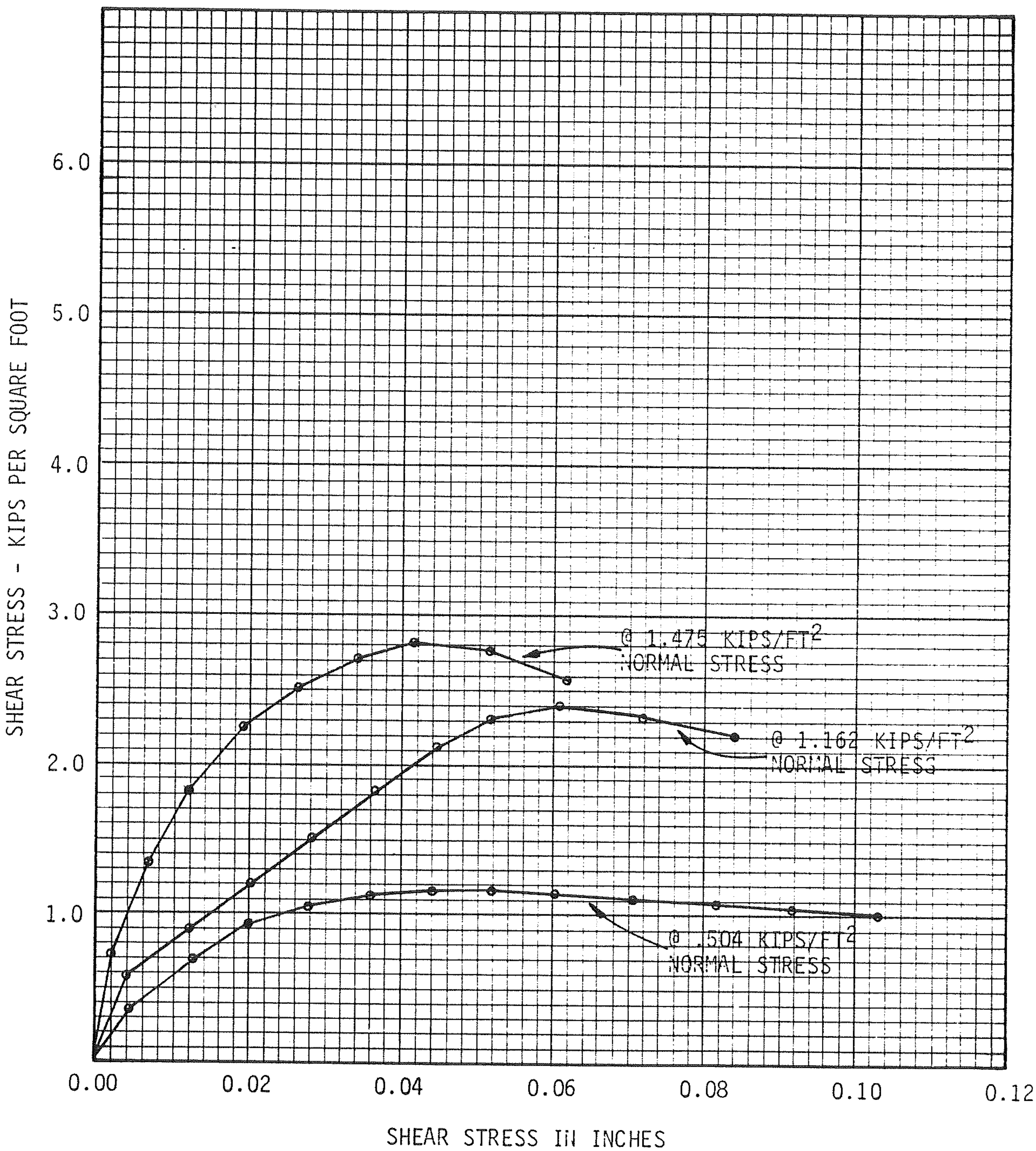
PROJECT Site A - Research Study
Drilled Cast-in-Place Concrete Piles JOB NO. E71-272

DIRECT SHEAR TEST DATA

Test Boring No. 2A, 9½'-10½'



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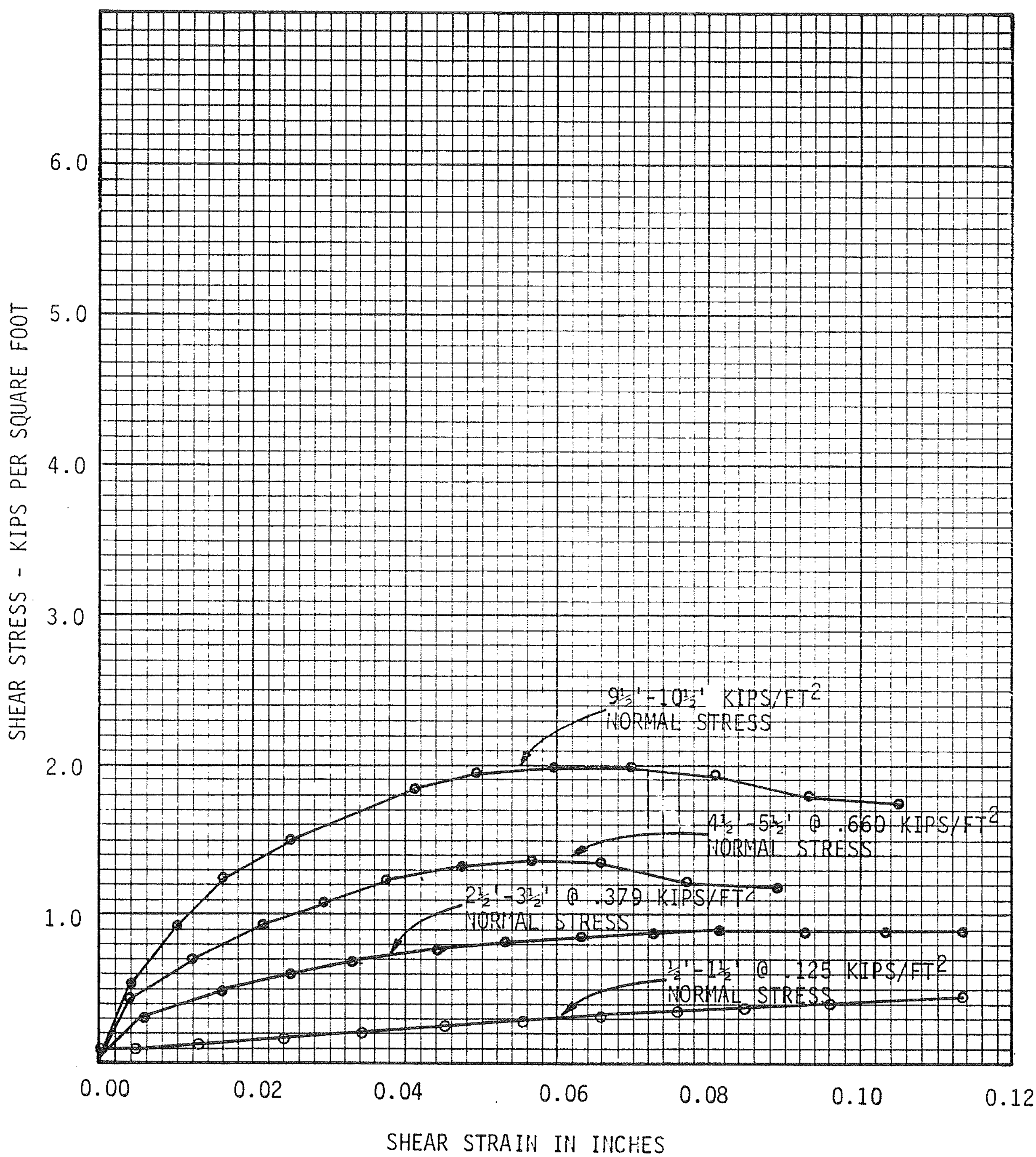
PROJECT Site A - Research Study
Drilled Cast-in-Place Concrete Piles JOB NO. E71-272

DIRECT SHEAR TEST DATA

Test Boring No. 3A



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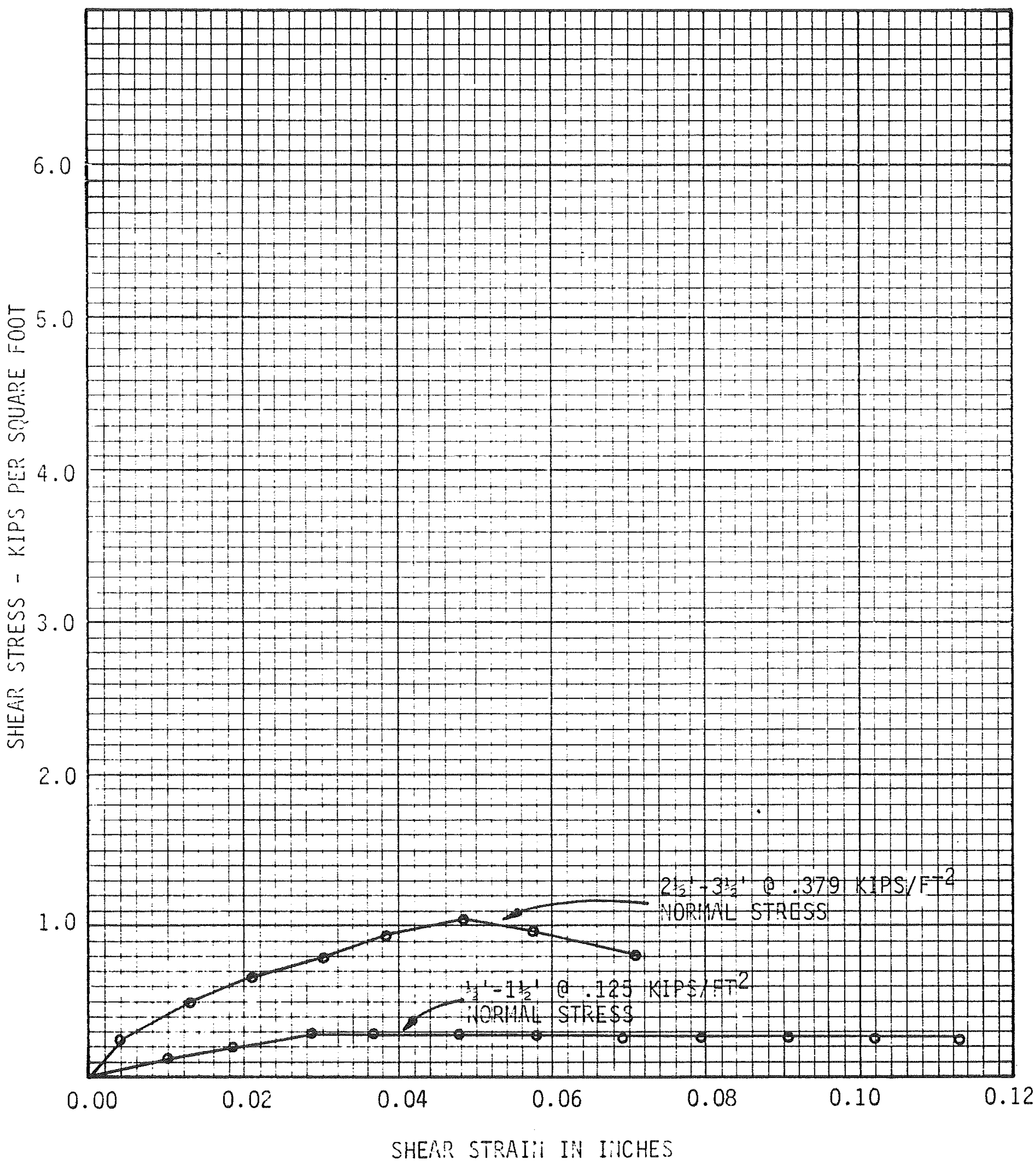
Site A - Research Study
PROJECT Drilled Cast-in-Place Concrete Piles **JOB NO.** E71-272

DIRECT SHEAR TEST DATA

Test Boring No. 4A



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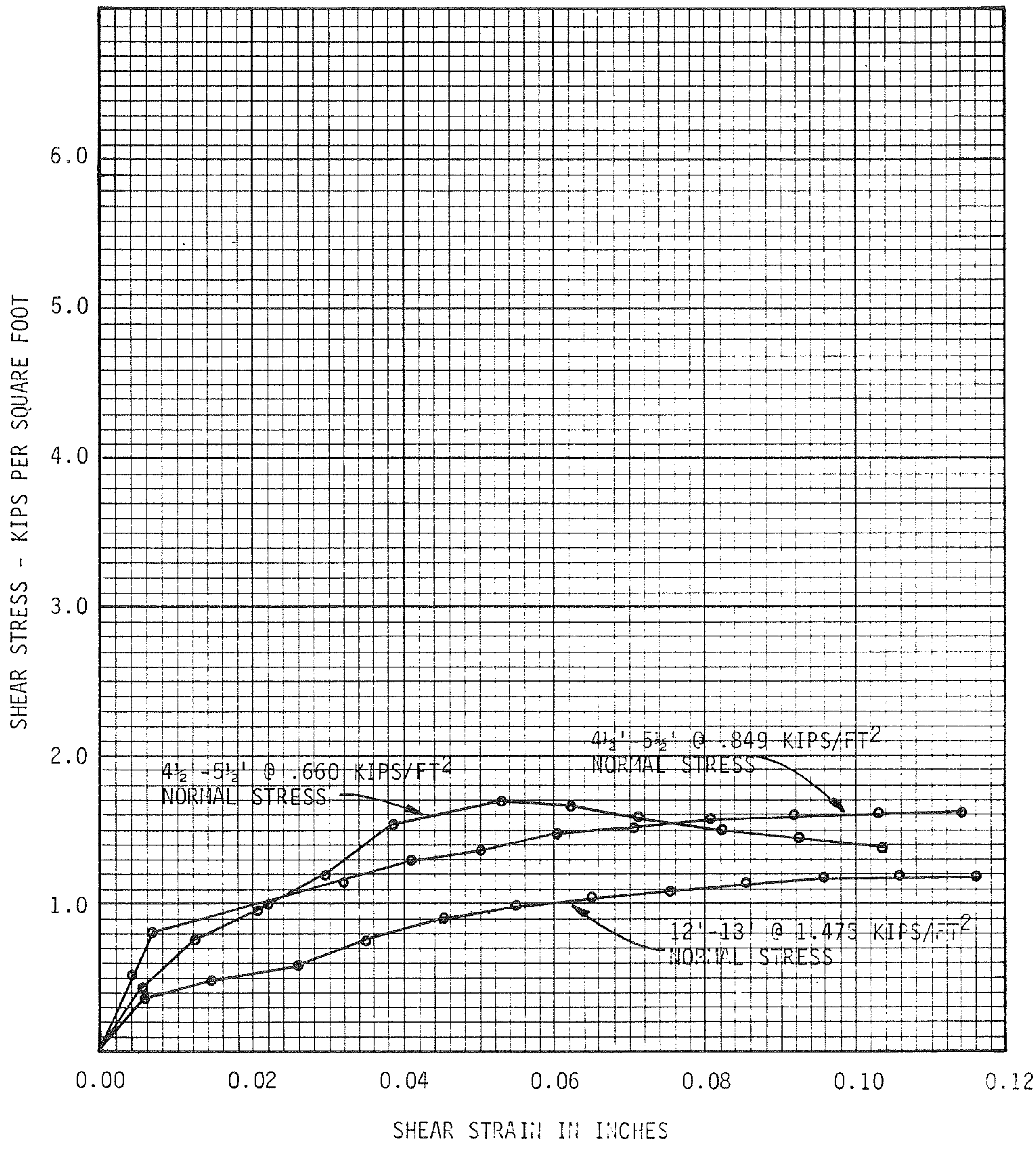
Site A - Research Study

PROJECT Drilled Cast-in-Place Concrete Piles **JOB NO.** E71-272

DIRECT SHEAR TEST DATA

Test Boring No. 4A



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PROJECT Site B - Research Study
Drilled Cast-In-Place Concrete Piles

JOB NO. E71-272 **DATE** 12-15-70

LOG OF TEST BORING NO. 1B

RIG TYPE CME-55
BORING TYPE 6½" Hollow Stem Auger
SURFACE ELEV.
DATUM

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows per foot 140 lb. 30" free-fall drop hammer	Dry Density Lbs. per cu. ft.	Moisture Content Per Cent of Dry Wt.	Unified Soil Classification	Remarks
0	10, 13		X	S	1-1-1-2-3-2-2-3-3 / 15		13	CL	
	21, 24		X	S	1-2-1-2-2-2-2-3-3 / 14		8	SC	
5	35, 23		X	S	3-2-2-2-3-3-3-2-3 / 16		6	SM	
	35, 36		X	S	3-3-2-3-3-4-5-4-5 / 24		11		
10	25, 35		X	S	3-3-2-4-4-3-4-5-5 / 25		14		
	27, 24		X	S	1-2-3-2-3-3-3-4-4 / 19		23		
	19, 22		X	S	1-2-2-2-3-3-3-3-4 / 18		27	CL	
15	25, 29		X	S	1-1-2-2-2-2-2-3-3 / 14		15		
	36, 30		X	S	2-1-2-3-3-5-4-5-5 / 25		22		
	32, 84		X	S	2-3-3-5-6-5-7-6-6 / 35		14	CH	
20	98, 89		X	S	7-11-10-13-18-23-27-25-28 / 134		10		
	100/11½"		X	S	4-11-12-15-25-21-23-21-23 / 128		12	CL	
25			X	S	2-5-8-8-14-28-50/1½" / 65		9	ML	
			X	S	34-38-33-33-48-50/1" / 236+		15		
			X	S	3-3-3-3-4-5-5-6-7 / 30		24		
30			X	S	1-2-3-5-4-4-5-7-8 / 33		29		
			X	S	2-6-5-4-4-4-5-6-5 / 28		25	CL	
35			X	S	2-2-3-2-4-4-5-6-5 / 26		26		
			X	S	5-4-7-7-7-7-8-7-8 / 44		7		
			X	S	6-6-6-8-9-9-11-10-9 / 56		5	SC	dense to very dense
40									

Penetrometer refused at 22'11½"

	VISUAL CLASSIFICATION
	SILTY CLAY, medium to high plasticity, brown
	CLAYEY SAND, predominantly fine, weakly cemented, low to medium plasticity, brown
	SILTY SAND, trace of gravel, predominantly medium to fine, non-plastic, brown
	SILTY CLAY, weakly lime cemented, medium plasticity, brown
	CLAY, some sand, trace of fine gravel, strongly lime cemented, high plasticity, brown
	SILTY CLAY, strongly lime cemented, medium plasticity, brown
	SANDY SILT, trace of gravel, moderately to strongly lime cemented, very low plasticity, brown
	SILTY CLAY, trace gravel, moderaly lime cemented, medium plasticity, brown
	CLAYEY SAND, some gravel, well graded, moderately lime cemented, medium plasticity, brown
	Stopped auger at 38'6" Stopped sampler at 40'

GROUND WATER

DEPTH	HOUR	DATE
	NONE	

SAMPLE TYPE

A – Auger cuttings. B – Block sample
S – 2" O.D. 1.38" I.D. tube sample.
U – 3" O.D. 2.42" I.D. tube sample.
T – 3" O.D. thin-walled Shelby tube.

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PROJECT Site B - Research Study Drilled Cast-in-Place Concrete Piles

JOB NO. E71-272 DATE 12-18-70

LOG OF TEST BORING NO. 1B

RIG TYPE CME-55

BORING TYPE 6½" Hollow Stem Auger

SURFACE ELEV.

DATUM

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows per foot 140 lb. 30" free-fall drop hammer	Dry Density Lbs. per cu. ft.	Moisture Content Per Cent of Dry Wt.	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
0			X	U	1-3-3-3-4-3 / 17	97	20	CL		
			X	U	1-1-1-2-5-3 / 10	100	8	SC		
5			X	U	3-4-5-4-4-4 / 24	106	7	SM		
			X	U	3-4-5-7-7-7 / 33	93	20			
10			X	U	1-3-4-5-5-6 / 20	98	18			
			X	U	6-7-7-8-8-9 / 45	94	30	CL		
15			X	U	2-3-2-4-4-6 / 21	98	9			
			X	U	3-3-4-5-5-6 / 26	93	26			
20			X	U	5-7-11-11-11-12 / 57	105	8	CH		
			X	U	10-15-8-8-10-15 / 66	107	12	CL		
25			X	U	22-50 for 1 3/4" / 72+			ML		
			X	U	6-3-9-13-27-42 / 100					
30			X	U	17-17-14-12-12-18 / 90	93	32			
			X	U	4-9-13-15-25-24 / 90	114	19	CL		
35			X	U	3-4-5-7-8-8 / 35	89	30			
40										Stopped auger at 34'6" Stopped sampler at 35'6"

GROUND WATER

DEPTH	HOUR	DATE
	NONE	

SAMPLE TYPE

A – Auger cuttings. B – Block sample

S – 2" O.D. 1.38" I.D. tube sample.

U – 3" O.D. 2.42" I.D. tube sample.

T – 3" O.D. thin-walled Shelby tube.

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PROJECT Site B - Research Study Drilled Cast-in-Place Concrete Piles

JOB NO. E71-272 DATE 12-11-70

LOG OF TEST BORING NO. 2B

RIG TYPE CME-55

BORING TYPE 6½" Hollow Stem Auger

SURFACE ELEV.

DATUM

Continuous Penetration Resistance (by foot of depth, from 0): 4, 8, 16, 21, 16, 15, 14, 14, 15, 19, 14, 7, 11, 13, 15, 15, 21, 35, 65, 96, 100/11"

Depth in Feet	Sample	Sample Type	Blows per foot 140 lb. 30" free-fall drop hammer	Dry Density Lbs. per cu. ft.	Moisture Content Per Cent of Dry Wt.	Unified Soil Classification	Remarks	Visual Classification
0	X	S	1-2-2-2-2-3-2-2-2 13		13	CL		SILTY CLAY, medium plasticity, brown
	X	S	2-2-2-2-2-3-2-3-2 14		7	SC		CLAYEY SAND, medium plasticity, predominantly fine to medium, weakly lime cemented, light brown
5	X	S	2-2-2-3-3-2-2-3-2 15		8			
	X	S	1-2-1-1-3-1-2-1-2 10		14	CL-CH	sandy stratification at 8½-9'+	SILTY CLAY, some sand, medium plasticity, weakly lime cemented, brown
	X	S	1-1-2-2-2-2-2-2-2 12		22			
10	X	S	1-2-1-2-2-2-2-2-3 13		27			
	X	S	2-2-1-2-2-2-3-3-3 15		22			
15	X	S	1-1-1-2-1-1-2-1-2 9		24			
	X	S	1-1-2-2-2-2-3-3-4 16		24			
	X	S	1-2-2-5-5-5-9-10-11 43		13			
20	X	S	8-12-15-20-16-15-27-25-25 128		18	CH		CLAY, some sand, high plasticity, medium lime cemented, light brown
	X	S	6-6-7-10-10-10-8-8-7 53		16			
25	X	S	10-12-14-20-21-23-25-46-50/1½" 149		15	ML		SANDY SILT, trace gravel, very low plasticity, moderately to strongly lime cemented, very light brown
	X	S	4-6-6-6-5-6-5-6-12 40		28			
	X	S	4-4-4-3-3-3-2-7-12 30		12			
30	X	S	3-3-3-4-4-5-7-6-5 31		14	CL		SILTY CLAY, moderately lime cemented, medium to low plasticity, brown
	X	S	2-1-3-2-3-3-3-4-4 19		17	ML		SANDY SILT, low plasticity, brown
35	X	S	3-1-4-5-6-5-4-3-4 27		10	CL		SILTY CLAY, trace of gravel, moderately lime cemented, medium to low plasticity, light brown
	X	S	3-3-4-4-4-4-6-4-6 28					
40	X	S	5-4-5-6-6-6-5-5-4 32		7	SC		CLAYEY SAND, generally well graded, some gravel, low plasticity, brown

Penetrometer refused at 20'11"

Stopped auger at 38'6"
Stopped sampler at 40'

GROUND WATER

DEPTH	HOUR	DATE
	NONE	

SAMPLE TYPE

A – Auger cuttings. B – Block sample
S – 2" O.D. 1.38" I.D. tube sample.
U – 3" O.D. 2.42" I.D. tube sample.
T – 3" O.D. thin-walled Shelby tube.

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Site B - Research Study
PROJECT Drilled Cast-in-Place Concrete Piles

JOB NO. F71-272 DATE 12-21-70

LOG OF TEST BORING NO. 2B

RIG TYPE CME-55
BORING TYPE 6½" Hollow Stem Auger
SURFACE ELEV.
DATUM

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows per foot 140 lb. 30" free-fall drop hammer	Dry Density Lbs. per cu. ft.	Moisture Content Per Cent of Dry Wt.	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
0			X	U	2-3-6-5-5-5 26	106	21	CL		
			X	U	3-3-2-2-2-3 15	98	5	SC		
5			X	U	1-2-3-2-3-4 15	114	7			
			X	U	2-2-2-3-3-4 16	100	13			
10			X	U	4-5-5-5-6-6 31	100	24			
			X	U	3-4-3-4-4-5 23	91	29	CL-CH		
15			X	U	4-5-4-4-4-5 26	95				
			X	U	7-8-9-9-10-13 56	109	19			
20			X	U	8-11-18-17-19-27 100	116	6			
			X	U	15-30-50/1½" 95+	95	14	CH		
25			X	U	13-15-12-21-50/1 3/4" 111+	88	19			
			X	U	4-12-9-7-6-9 47	87	23	ML		
30			X	U	4-3-7-9-10-11 44	97	27			
			X	U	1-4-5-6-7-7 30	105	16	CL		
35			X	U	4-5-9-9-12-13 52	101	16	ML CL		
										Stopped auger at 34'6" Stopped sampler at 35'6"
40										

GROUND WATER

DEPTH	HOUR	DATE
	NONE	

SAMPLE TYPE
A – Auger cuttings. B – Block sample
S – 2" O.D. 1.38" I.D. tube sample.
U – 3" O.D. 2.42" I.D. tube sample.
T – 3" O.D. thin-walled Shelby tube.

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Site B - Research Study

PROJECT Drilled Cast-in-Place Concrete Piles

JOB NO. E71-272 DATE 12-11-70

LOG OF TEST BORING NO. 3B

RIG TYPE CME-55

BORING TYPE 6½" Hollow Stem Auger

SURFACE ELEV.

DATUM

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows per foot 140 lb. 30" free-fall drop hammer	Dry Density Lbs. per cu. ft.	Moisture Content Per Cent of Dry Wt.	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
0	11, 18		X	S	1-2-2-3-2-7-3-3-3	16	13	CL		SILTY CLAY, medium to high plasticity, brown
	29, 39		X	S	2-2-2-2-2-2-3-2-3	14	16			
5	46, 50		X	S	2-2-2-3-3-3-4-3-4	20	10	SC	6" sand stratification at 5½'	CLAYEY SAND, predominantly fine, weakly cemented, low to medium plasticity, brown
	34, 27		X	S	2-2-2-2-2-1-1-2-1	9	9	SM		SILTY SAND, trace of gravel, predominantly fine to medium, nonplastic, brown
10	19, 17		X	S	1-1-1-1-2-1-2-2-3	11	20			
	17, 14		X	S	1-1-2-2-2-2-3-3-4	16	24			SILTY CLAY, weakly lime cemented, medium plasticity, brown
	17, 24		X	S	1-1-3-2-2-2-2-3-3	14	24			
15	21, 19		X	S	2-1-2-2-2-2-2-2-3	13	16	CL		
	30, 41		X	S	2-3-3-4-4-7-7-8-10	40	16			
	100/ 10"		X	S	2-6-9-10-10-12-10-12-12	66	13			
20			X	S	6-7-7-7-8-10-9-12-10	56	16	CH		CLAY, some sand, trace of fine gravel, strongly lime cemented, high plasticity, brown
			X	S	2-3-5-5-9-19-25/½"		15			
25			X	S	14-15-7-8-7-11-11-12-22	69	13	CL		SILTY CLAY, moderately to strongly lime cemented, medium plasticity, brown
			X	S	2-2-2-5-5-6-7-6-8	37	22			
			X	S	2-2-2-3-4-3-4-5-12	31	19			
30			X	S	3-3-2-4-3-4-5-6-6	28	13	ML		SANDY SILT, trace of gravel, moderately lime cemented, very low plasticity, brown
			X	S	2-4-4-4-4-6-4-4-5	27	14	CL		SILTY CLAY, trace of gravel, moderately lime cemented, medium plasticity, brown
35			X	S	2-3-3-4-4-6-5-6-6	31	23			
			X	S	2-2-3-4-6-6-6-7-7	36	10	SC		CLAYEY SAND, some gravel, well graded, moderately lime cemented, medium plasticity, brown
40			X	S	1-2-7-7-5-6-7-7-8	40	7			

Penetrometer refused at 18'10"

Stopped auger at 38'6"
Stopped sampler at 40'

GROUND WATER

DEPTH	HOUR	DATE
	NONE	

SAMPLE TYPE

A – Auger cuttings. B – Block sample
S – 2" O.D. 1.38" I.D. tube sample.
U – 3" O.D. 2.42" I.D. tube sample.
T – 3" O.D. thin-walled Shelby tube.

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PROJECT Site B - Research Study Drilled Cast-in-Place Concrete Piles

JOB NO. E71-272 DATE 12-18-70

LOG OF TEST BORING NO. 3B

RIG TYPE CME-55

BORING TYPE 6½" Hollow Stem Auger

SURFACE ELEV.

DATUM

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows per foot 140 lb. 30" free-fall drop hammer	Dry Density Lbs. per cu. ft.	Moisture Content Per Cent of Dry Wt.	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
0			X	U	1-2-2-4-5-7 / 21	101	23	CL		
			X	U	2-4-4-6-6-8 / 30	106	9			
5			X	U	2-4-4-4-5-5 / 24	107	6	SC		
			X	U	2-5-6-7-8-10 / 38	98	13	SM		
10			X	U	3-3-3-5-5-7 / 29	87	21			
			X	U	5-6-7-6-6-7 / 35	95	29	CL		
15			X	U	5-5-6-6-7-9 / 38	94	15			
			X	U	13-13-18-22-33-37 / 136	116	17			
20			X	U	25-17-19-32-47-50 / 190+	93	16			
			X	U	3-9-15-24-27-50 / 128+	106	14	CH		
25			X	U	6-10-13-10-13-18 / 70	87	25			
			X	U	9-9-8-10-10-11 / 57	103	21	CL		
30			X	U	9-3-5-5-6-6 / 34	92	31	ML		
			X	U	4-5-6-6-7-9 / 37	111	13			
35			X	U	4-7-7-8-8-9 / 43	108	17	SC		
40										Stopped auger at 34'6" Stopped sampler at 35'6"

GROUND WATER

DEPTH	HOUR	DATE
	NONE	

SAMPLE TYPE

A – Auger cuttings. B – Block sample

S – 2" O.D. 1.38" I.D. tube sample.

U – 3" O.D. 2.42" I.D. tube sample.

T – 3" O.D. thin-walled Shelby tube.

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PROJECT Site B - Research Study Drilled Cast-in-Place Concrete Piles

LOG OF TEST BORING NO. 4B

JOB NO. E71-272 DATE 12-14-70

RIG TYPE CME-55
BORING TYPE 6½" Hollow Stem Auger
SURFACE ELEV.
DATUM

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows per foot 140 lb. 30" free-fall drop hammer	Dry Density Lbs. per cu. ft.	Moisture Content Per Cent of Dry Wt.	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
0	17			S	3-2-3-2-2-3-2-3-2			CL		SILTY CLAY, high plasticity, brown
	22				14		14			
	31			S	2-1-2-2-1-2-2-2-2			CL		SANDY CLAY, some fine gravel, medium plasticity, reddish-brown
	30				11		6			
5	29			S	2-1-1-2-2-2-2-2-3					
	33				13		6	SM		SILTY SAND, trace of gravel, well graded, nonplastic, brown
	23			S	2-1-2-3-3-3-4-3-4					
	25				20		17			
	21			S	2-1-3-3-2-3-4-3-4					
10	18				19		22			SILTY CLAY, weakly lime cemented, low to medium plasticity, brown
	23			S	1-2-1-2-2-3-3-4-3					
	16				17		15			
	14			S	2-2-3-3-4-4-4-5-5			CL		
	24				25		16			
15	25			S	2-1-2-2-1-2-2-2-2					
	25				11		15			
	24			S	1-2-2-2-2-3-2-3-3					
	31				15		17			
	61			S	3-4-4-4-6-5-8-8-13					
20	100/10"				44		12			
				S	12-16-24-21-19-18-16-15-16					SILTY CLAY, moderately to strongly lime cemented, medium to high plasticity, brown
					105		13	CH		
				S	2-2-3-4-4-8-17-14-18					
					65		14			SANDY CLAY, moderately to strongly lime cemented, medium plasticity, light brown
25				S	6-18-15-24-50 for 1½"					
					113+		14			
				S	5-9-10-10-11-11-11-11-14					
					68		21			
				S	3-2-3-4-3-7-10-7-7			CL		
30					38		25			
				S	3-2-4-4-4-5-5-5-6					SILTY CLAY, some fine gravel, weakly lime cemented, medium plasticity, brown
					29		22			
				S	3-4-4-5-7-7-8-8-5					
					40		19	CL		
35				S	3-2-3-5-5-5-6-10-10					
					41		21			
				S	4-5-4-5-4-4-5-5-6					CLAYEY SAND, some gravel, well graded, moderately lime cemented, medium plasticity, brown
					29		9			
				S	4-4-5-7-7-7-6-7-6			SC		
40					40		6			

Penetrometer refused at 19'10"

Stopped auger at 38'6"
Stopped sampler at 40'

GROUND WATER

DEPTH	HOUR	DATE
	NONE	

SAMPLE TYPE

A – Auger cuttings. B – Block sample
S – 2" O.D. 1.38" I.D. tube sample.
U – 3" O.D. 2.42" I.D. tube sample.
T – 3" O.D. thin-walled Shelby tube.

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Site B - Research Study
PROJECT Drilled Cast-in-Place Concrete Piles

LOG OF TEST BORING NO. 4B

JOB NO. E71-272 DATE 12-17-70

RIG TYPE CME-55
BORING TYPE 6½" Hollow Stem Auger
SURFACE ELEV.
DATUM

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows per foot 140 lb. 30" free-fall drop hammer	Dry Density Lbs. per cu. ft.	Moisture Content Per Cent of Dry Wt.	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
0				U	1-3-2-3-2-4 15	88	10	CL		
				U	10-8-8-13-13-11 63	103	7	CL		
5				U	3-5-7-7-7-8 37	109	5	SM		
				U	2-2-3-5-4-5 21	109	6			
10				U	13-13-10-8-10-14 68	111	16			
				U	2-3-5-5-7-8 30	100	22	CL		
15				U	3-3-3-4-5-8 26					
				U	2-2-3-4-5-6 22					
20				U	6-7-15-16-22-28 94	99	11	CH		
				U	5-18-20-19-23-43 128	118	11			
25				U	25-75-38-51-100+ 289+	92	10	CL		
				U	10-15-14-15-16-19 89	105	12			
30				U	6-6-8-17-29-31 97	101	16			
				U	7-8-12-15-15-30 87	89	28	CL		
35				U	3-5-4-7-8-10 37					

Stopped auger at 34'6"
Stopped sampler at 35'6"

GROUND WATER

DEPTH	HOUR	DATE
	NONE	

SAMPLE TYPE
A – Auger cuttings. B – Block sample
S – 2" O.D. 1.38" I.D. tube sample.
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T – 3" O.D. thin-walled Shelby tube.

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TABULATION .· TEST RESULTS

Job No. E71-272   Date

Client:

Project Site B - Research Study

Drilled Cast-in-Place Concrete Piles

Material

Source

HOLE NO.	LOCATION	DEPTH	UNIFIED CLASS.	LL	PI	SIEVE ANALYSIS – ACCUM. % PASSING												LAB. NO.
						200	100	40	16	10	4	1/4	3/8	3/4	1	1½	2	
2B	See Site Plan	2½'-4'	SC	33	19	46	55	74	90	94	97	98	99	100				2585-83
2B	See Site Plan	6½'-8'	CL	40	20	75	90	95	98	99	100							2585-85
2B	See Site Plan	12½'-14'	CH	54	31	90	95	98	99		100							2585-88
2B	See Site Plan	16½'-18'	CL	44	24	91	95	98	99		100							2585-90
2B	See Site Plan	20½'-22'	CH	69	44	74	80	88	94	96	98	100						2585-92
2B	See Site Plan	24½'-26'	ML	30	1	52	62	75	88	93	98	99	100					2585-94
2B	See Site Plan	30½'-32'	CL	28	10	64	76	88	96	98	100							2585-97
4B	See Site Plan	6"-2'	CL	50	33	88	92	96	98	99	100							2585-122
4B	See Site Plan	4½'-6'	SM		NP	33	43	67	86	92	97	98	100					2585-124
4B	See Site Plan	18½'-20'	CL	30	10	74	87	95	99		100							2582-131
4B	See Site Plan	20½'-22'	CH	51	30	79	84	90	94	95	97	98	99	100				2582-132
4B	See Site Plan	22½'-24'	CL	34	14	69	80	90	93	94	97	98		100				2585-133
4B	See Site Plan	38½'-40'	SC	30	15	17	20	29	49	63	83	87	92	100				2585-141

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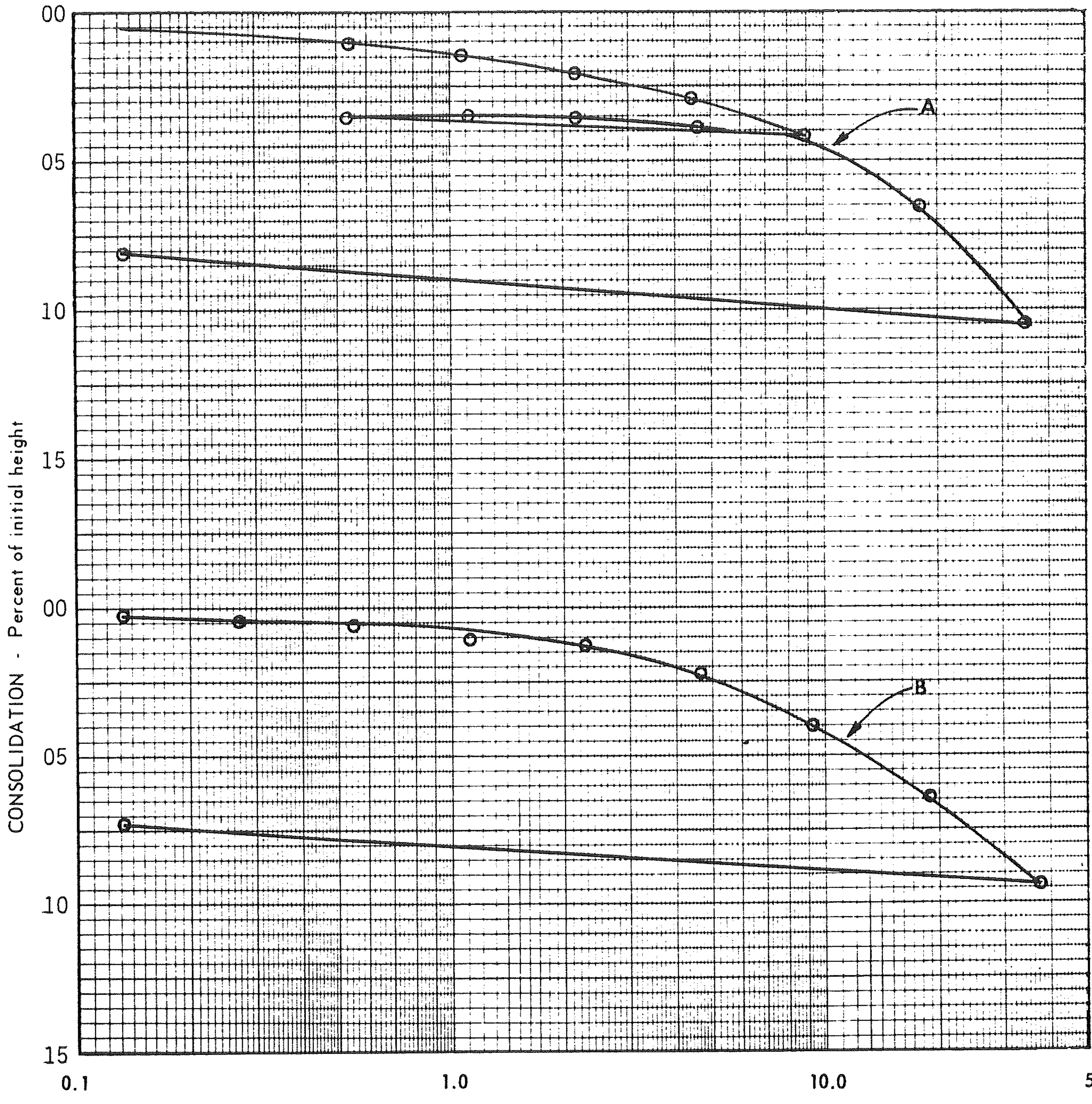
SUMMARY OF CONSOLIDATION TESTS

PROJECT Site B - Research Study Drilled Cast-in-Place Concrete Piles JOB NO. E71-272



CURVE	SAMPLE	INITIAL DRY DENSITY LBS./CU. FT.	MOISTURE CONTENT % DRY WEIGHT		UNIFIED SOIL CLASSIFICATION
			INITIAL	FINAL	
A	2B @ 7'-8'	97.0	6.9	-	CL
B	2B @ 9½'-10½'	113.4	18.0	-	CH

SOIL MOISTURE CONDITION

——— INSITU
– – – – SUBMERGED

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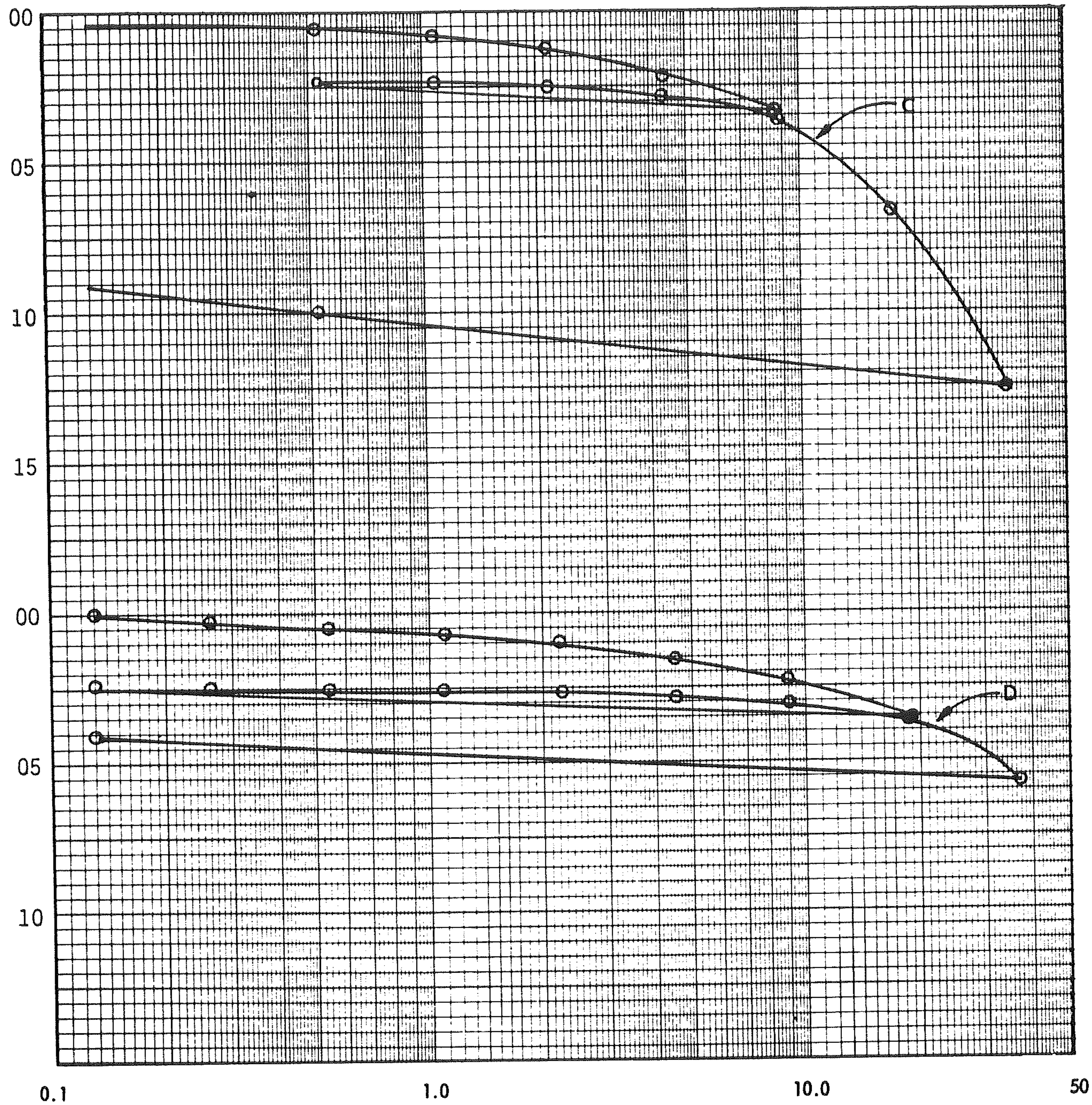
SUMMARY OF CONSOLIDATION TESTS

PROJECT Site B - Research Study Drilled Cast-in-Place Concrete Piles JOB NO. E71-272



CURVE	SAMPLE	INITIAL DRY DENSITY LBS./CU. FT.	MOISTURE CONTENT % DRY WEIGHT		UNIFIED SOIL CLASSIFICATION
			INITIAL	FINAL	
C	2B @ 12'-13'	89.0	22.8	-	CH
D	2B @ 17'-18'	112.15	13.8	-	CL

SOIL MOISTURE CONDITION

————	INSITU
— — — —	SUBMERGED

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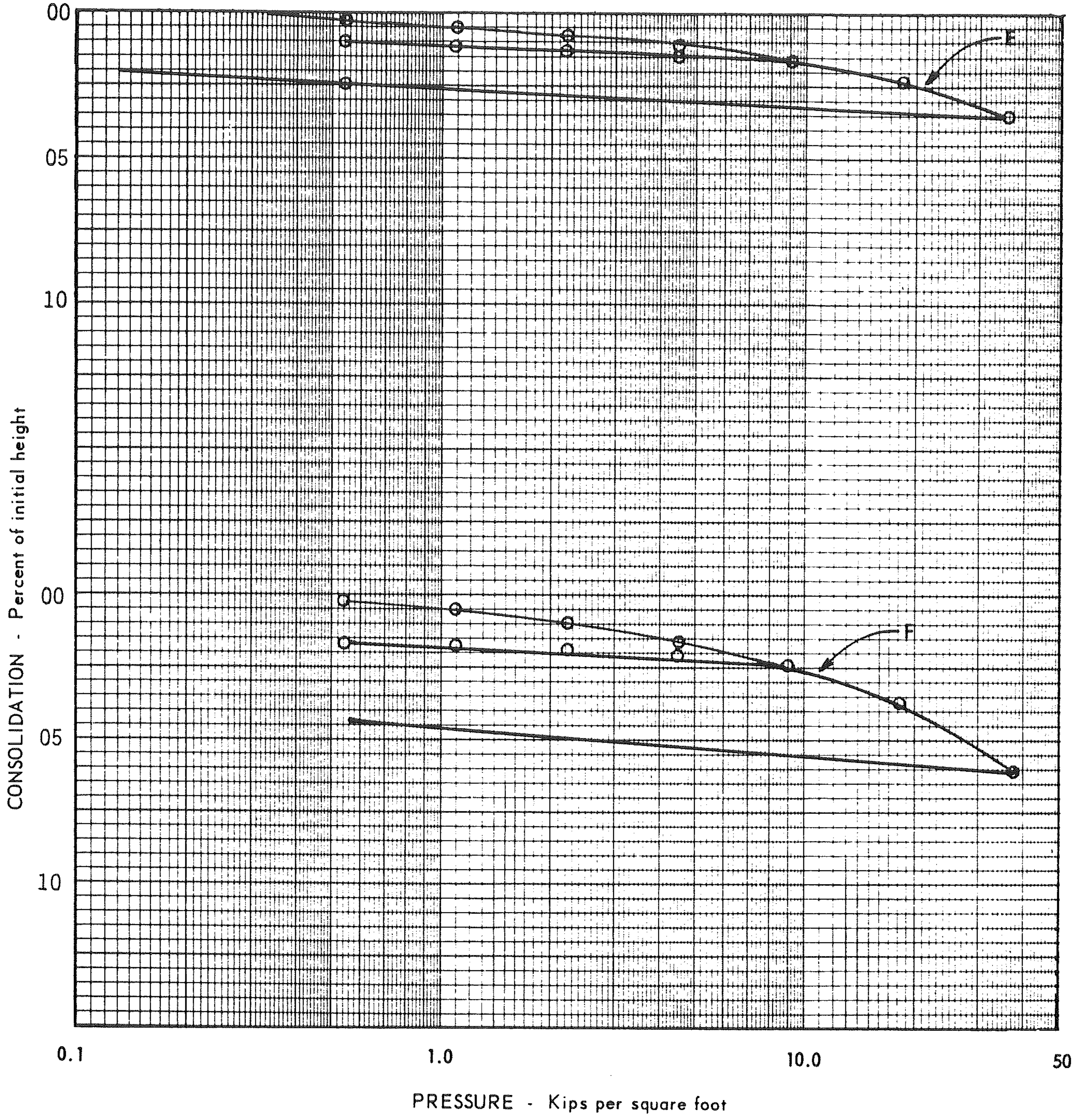
SUMMARY OF CONSOLIDATION TESTS

PROJECT Site B - Research Study Drilled Cast-in-Place Concrete Piles JOB NO. E71-272



CURVE	SAMPLE	INITIAL DRY DENSITY LBS./CU. FT.	MOISTURE CONTENT % DRY WEIGHT		UNIFIED SOIL CLASSIFICATION
			INITIAL	FINAL	
E	2B @ 19½'-20½'	115.43	11.6	-	CL
F	2B @ 24½'-25½'	85.6	19.6	-	ML

SOIL MOISTURE CONDITION

———	INSITU
– – – –	SUBMERGED

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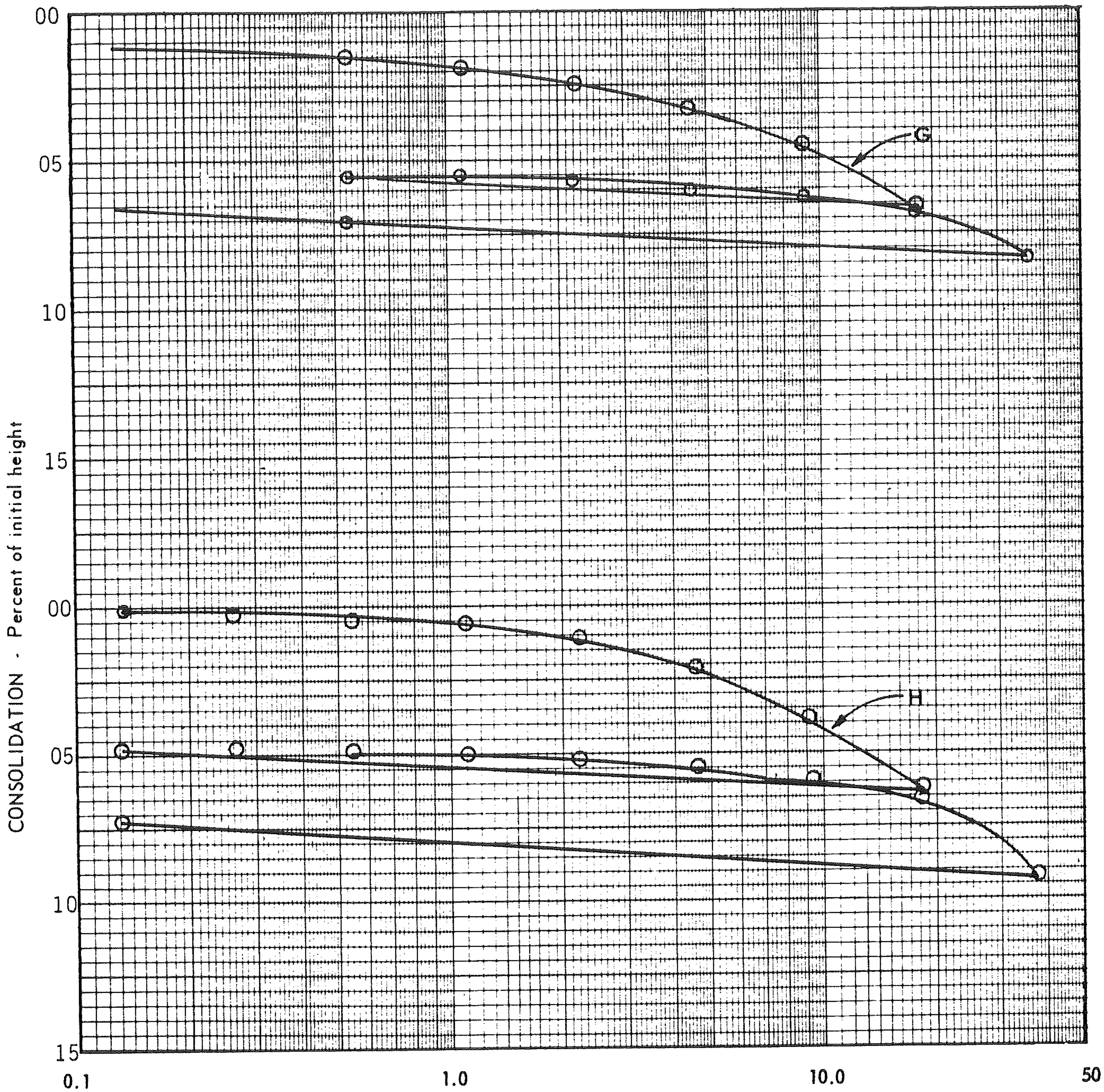
SUMMARY OF CONSOLIDATION TESTS

PROJECT Site B - Research Study Drilled Cast-in-Place Concrete Piles JOB NO. E71-272



CURVE	SAMPLE	INITIAL DRY DENSITY LBS./CU. FT.	MOISTURE CONTENT % DRY WEIGHT		UNIFIED SOIL CLASSIFICATION
			INITIAL	FINAL	
G	2B @ 27'-28'	96.6	16.3	-	CL
H	2B @ 32'- 33'	104.6	11.8	-	CL

SOIL MOISTURE CONDITION

——— INSITU

– – – – SUBMERGED

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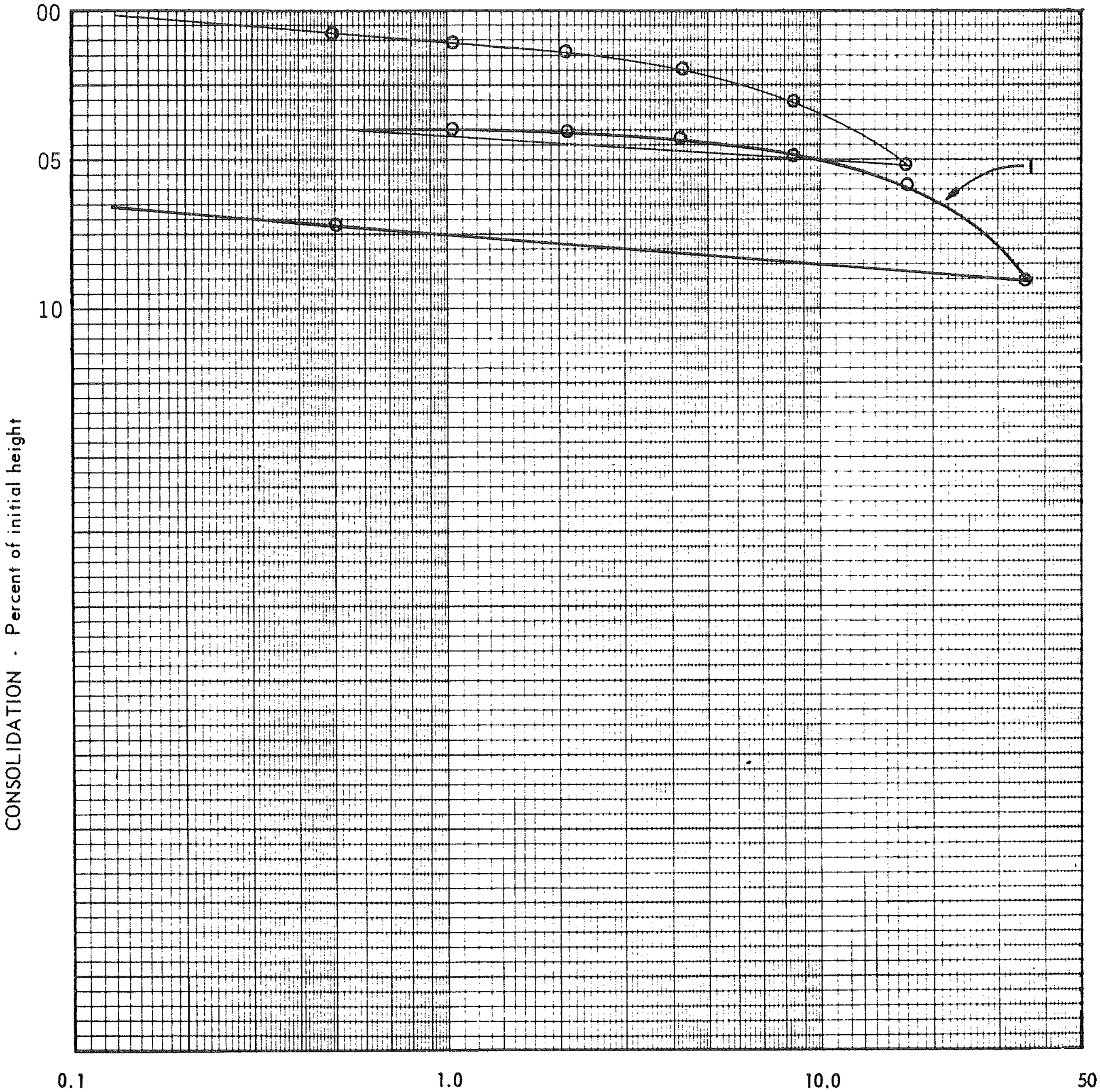
SUMMARY OF CONSOLIDATION TESTS

PROJECT Site B - Research Study Drilled Cast-in-Place Concrete Piles JOB NO. E71-272



CURVE	SAMPLE	INITIAL DRY DENSITY LBS./CU. FT.	MOISTURE CONTENT % DRY WEIGHT INITIAL	MOISTURE CONTENT % DRY WEIGHT FINAL	UNIFIED SOIL CLASSIFICATION
I	2B @ 32'-33'	90.2	29.7	-	CL

SOIL MOISTURE CONDITION

————	INSITU
– – – –	SUBMERGED

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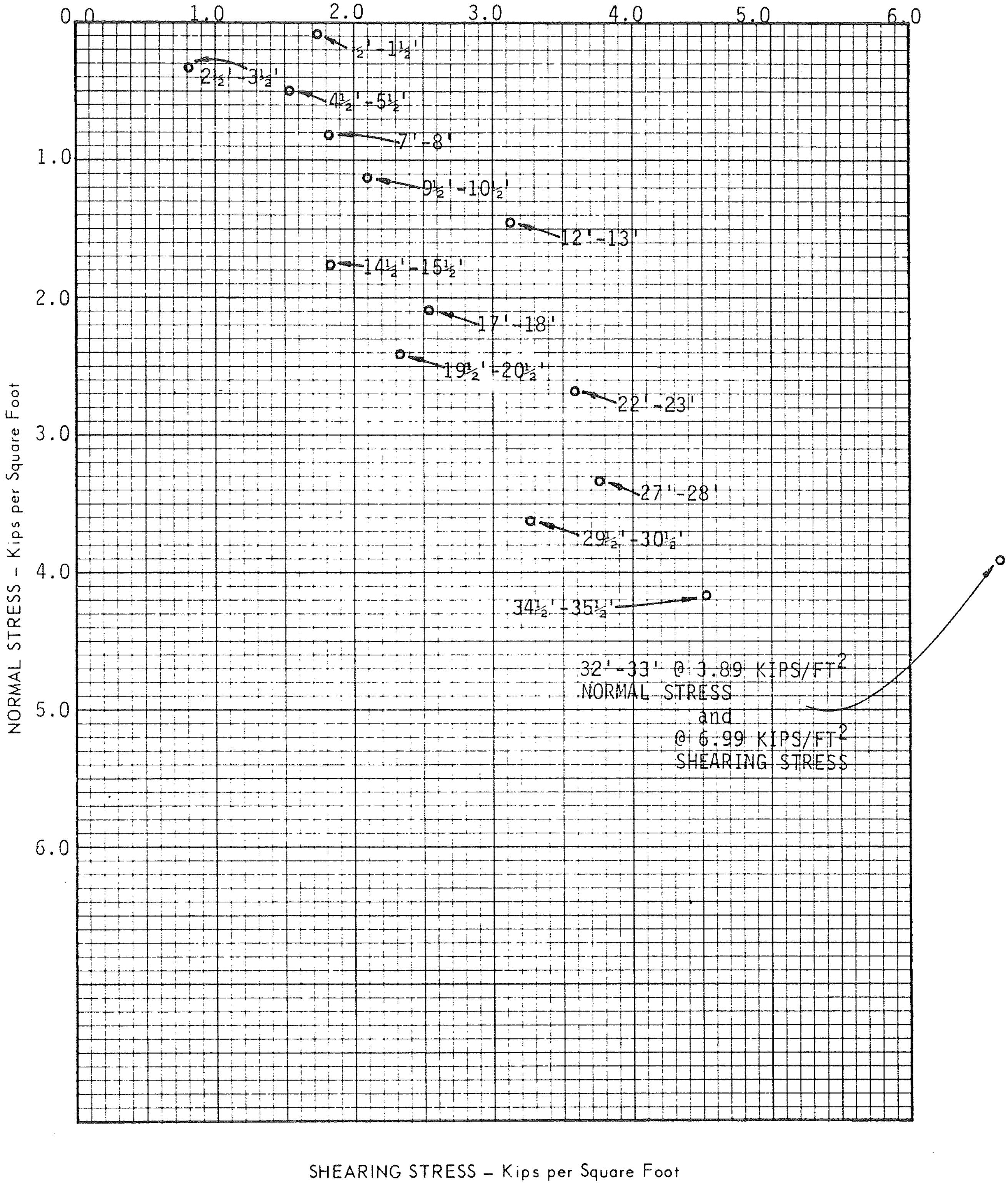
SUMMARY OF DIRECT SHEAR TESTS

PROJECT Site B - Research Study Drilled Cast-in-Place Concrete Piles **JOB NO** E71-272

Test Boring 1B



SOIL MOISTURE CONDITION

○ – INSITU
● – SUBMERGED

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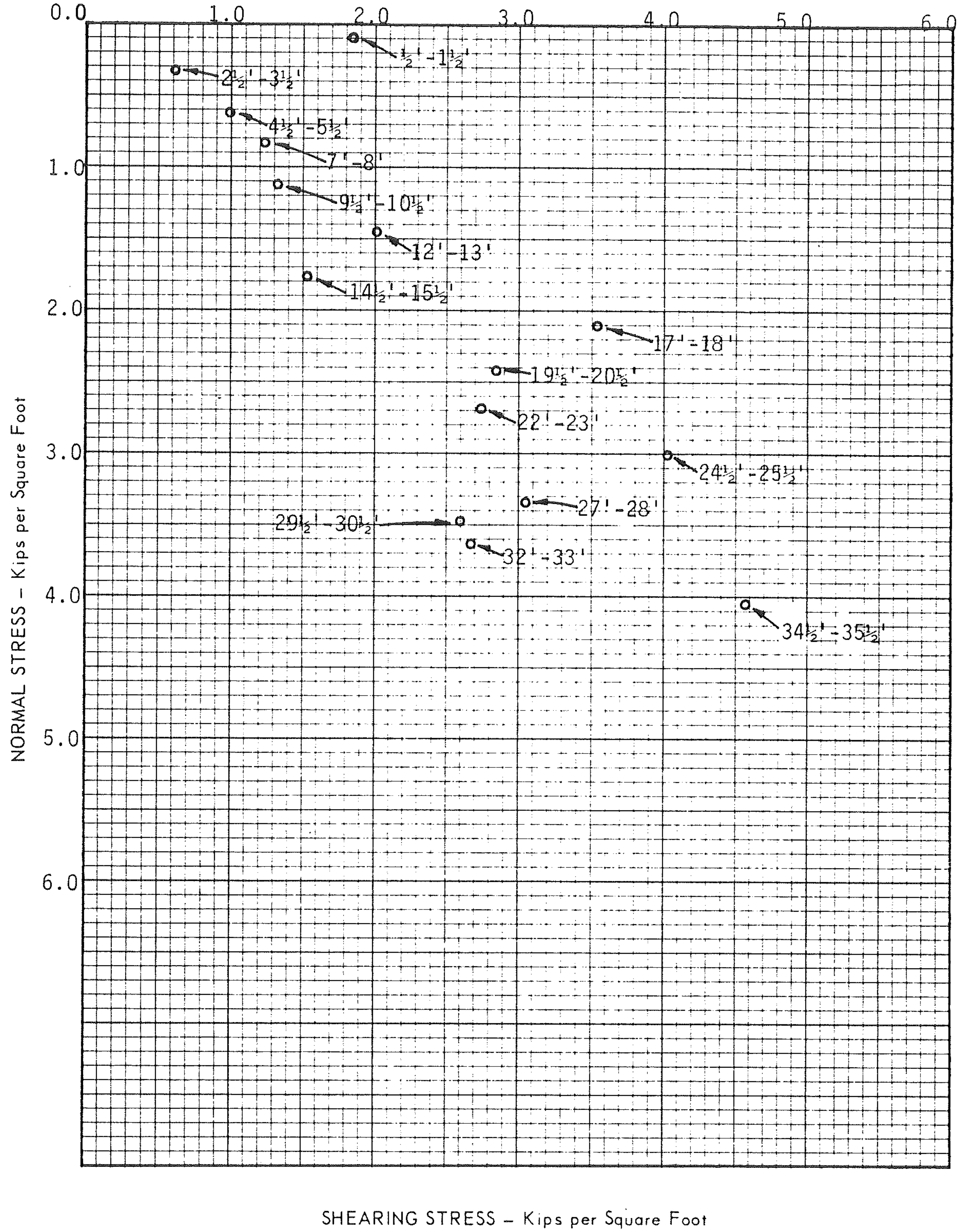
SUMMARY OF DIRECT SHEAR TESTS

PROJECT Site B - Research Study Drilled Cast-in-Place Concrete Piles **JOB NO** E71-272

Test Boring No. 2B



SOIL MOISTURE CONDITION

o – INSITU
● – SUBMERGED

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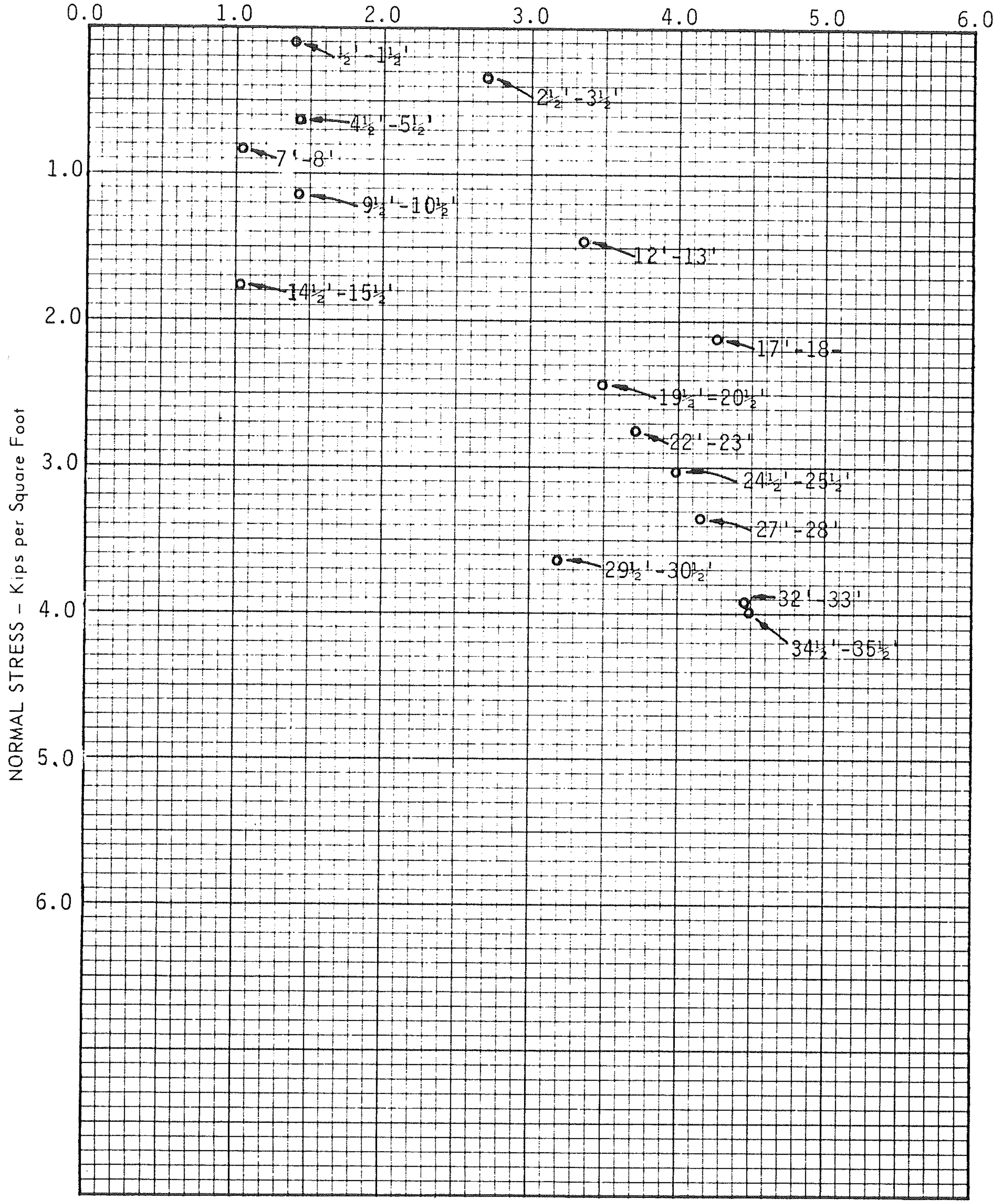
SUMMARY OF DIRECT SHEAR TESTS

Site B - Research Study

PROJECT Drilled Cast-in-Place Concrete Piles JOB NO E71-272

Test Boring No. 3B



SOIL MOISTURE CONDITION

O – INSITU
● – SUBMERGED

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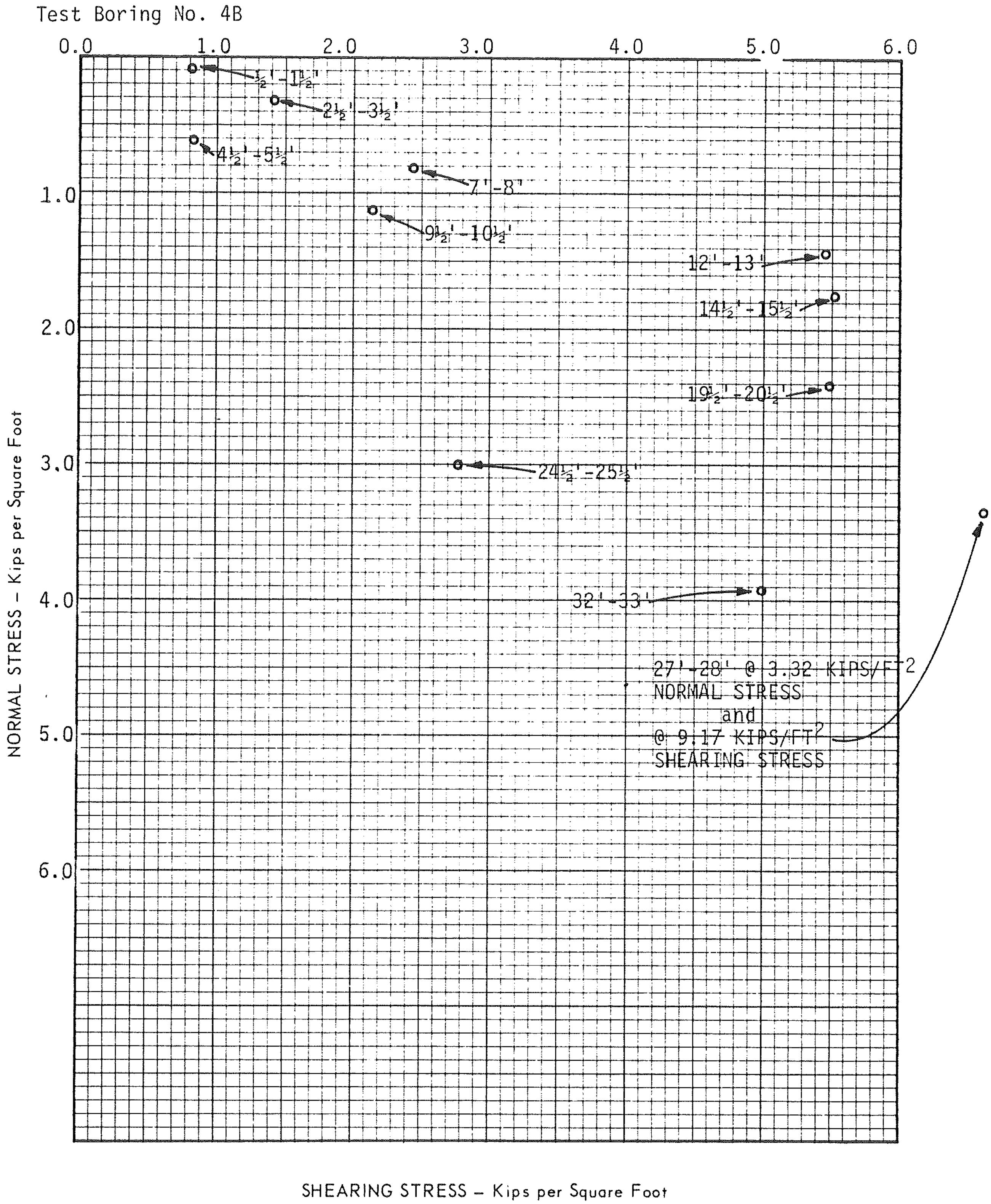
SUMMARY OF DIRECT SHEAR TESTS

Site B - Research Study

PROJECT Drilled Cast-in-Place Concrete Piles **JOB NO** E71-272



SOIL MOISTURE CONDITION

○ – INSITU
● – SUBMERGED

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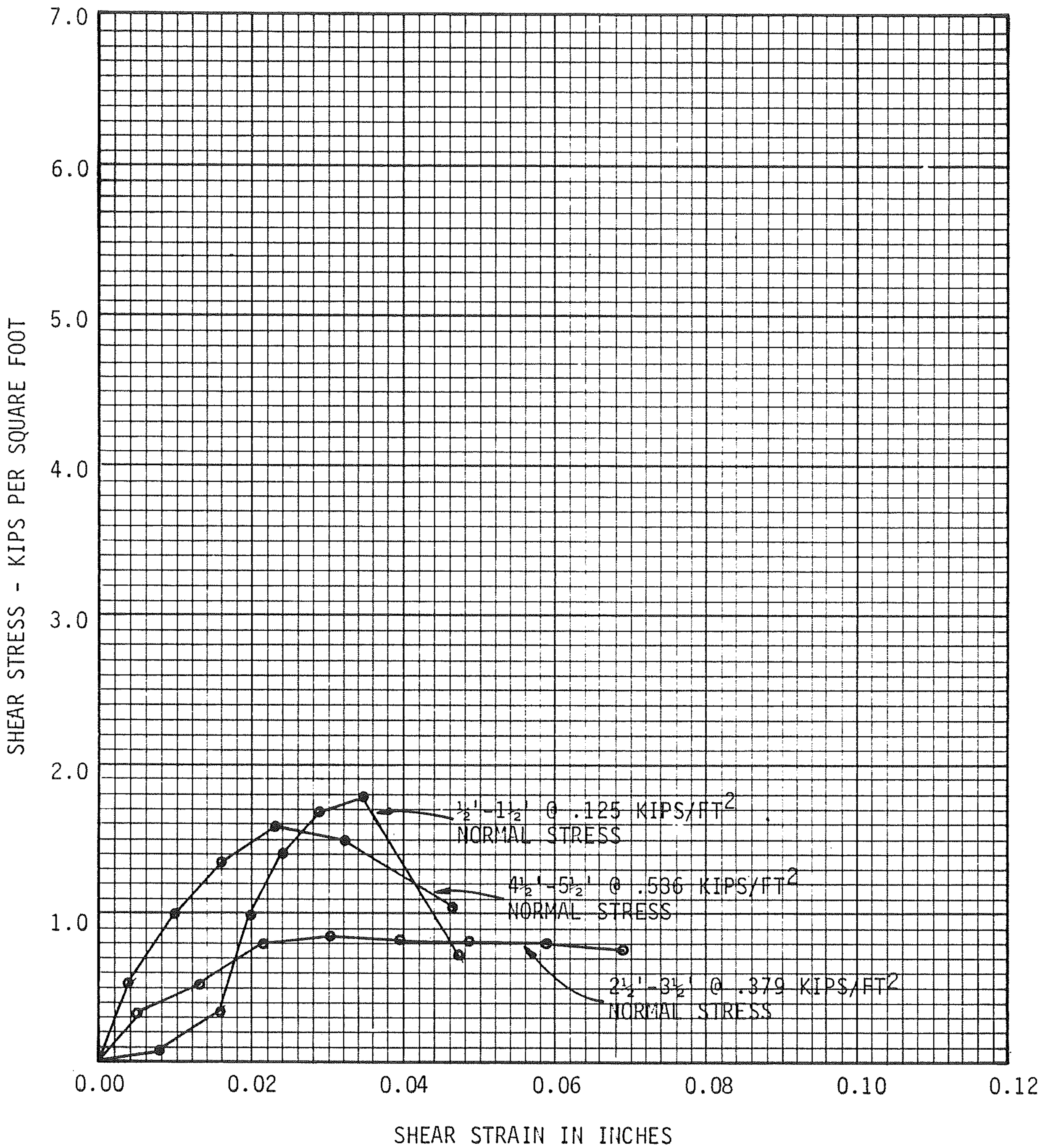
Site B - Research Study

PROJECT Drilled Cast-in-Place Concrete Piles **JOB NO.** E71-272

DIRECT SHEAR TEST DATA

Test Boring No. 1B



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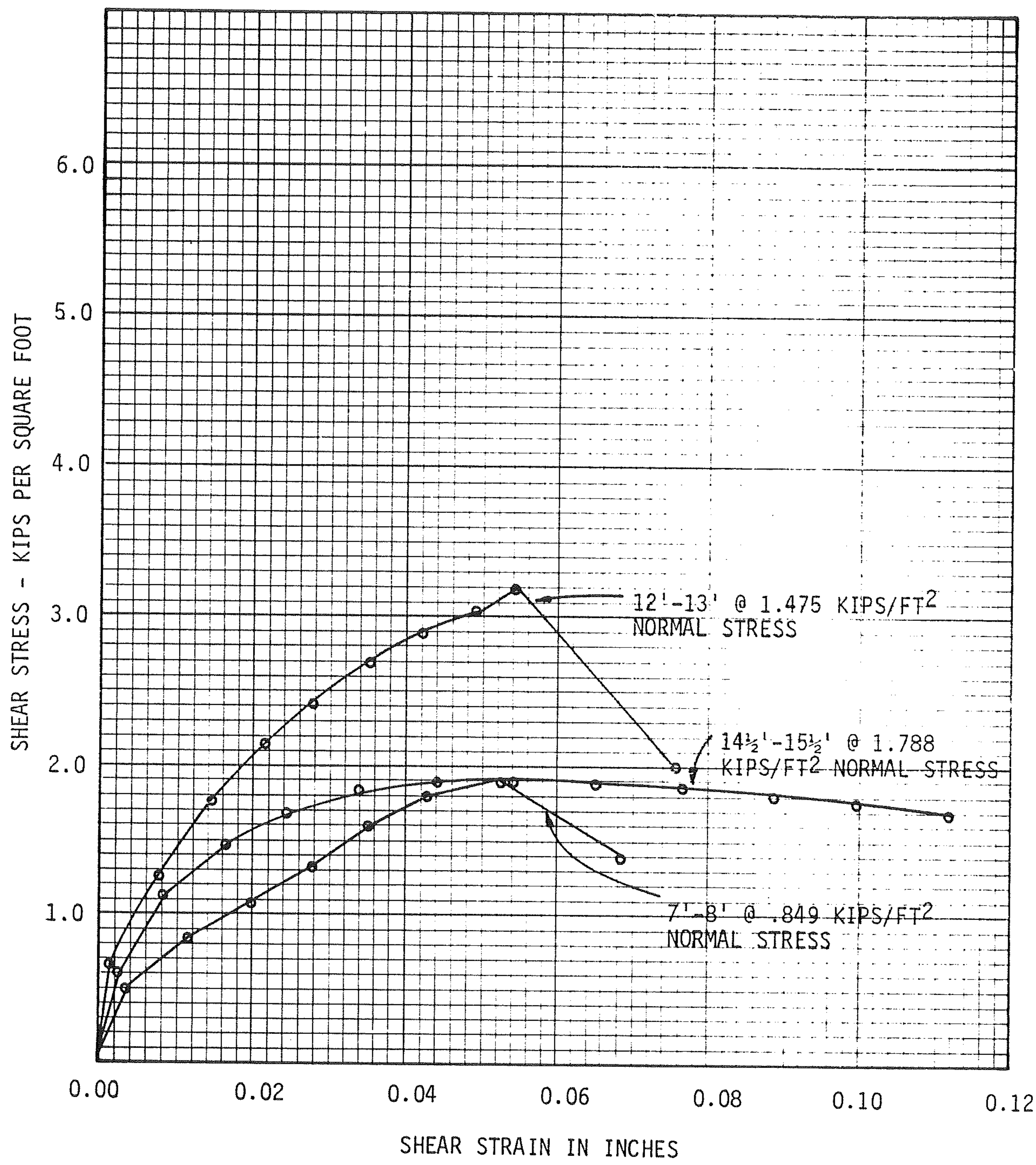
PROJECT Site B - Research Study
Drilled Cast-in-Place Concrete Piles JOB NO. E71-272

DIRECT SHEAR TEST DATA

Test Boring No. 1B



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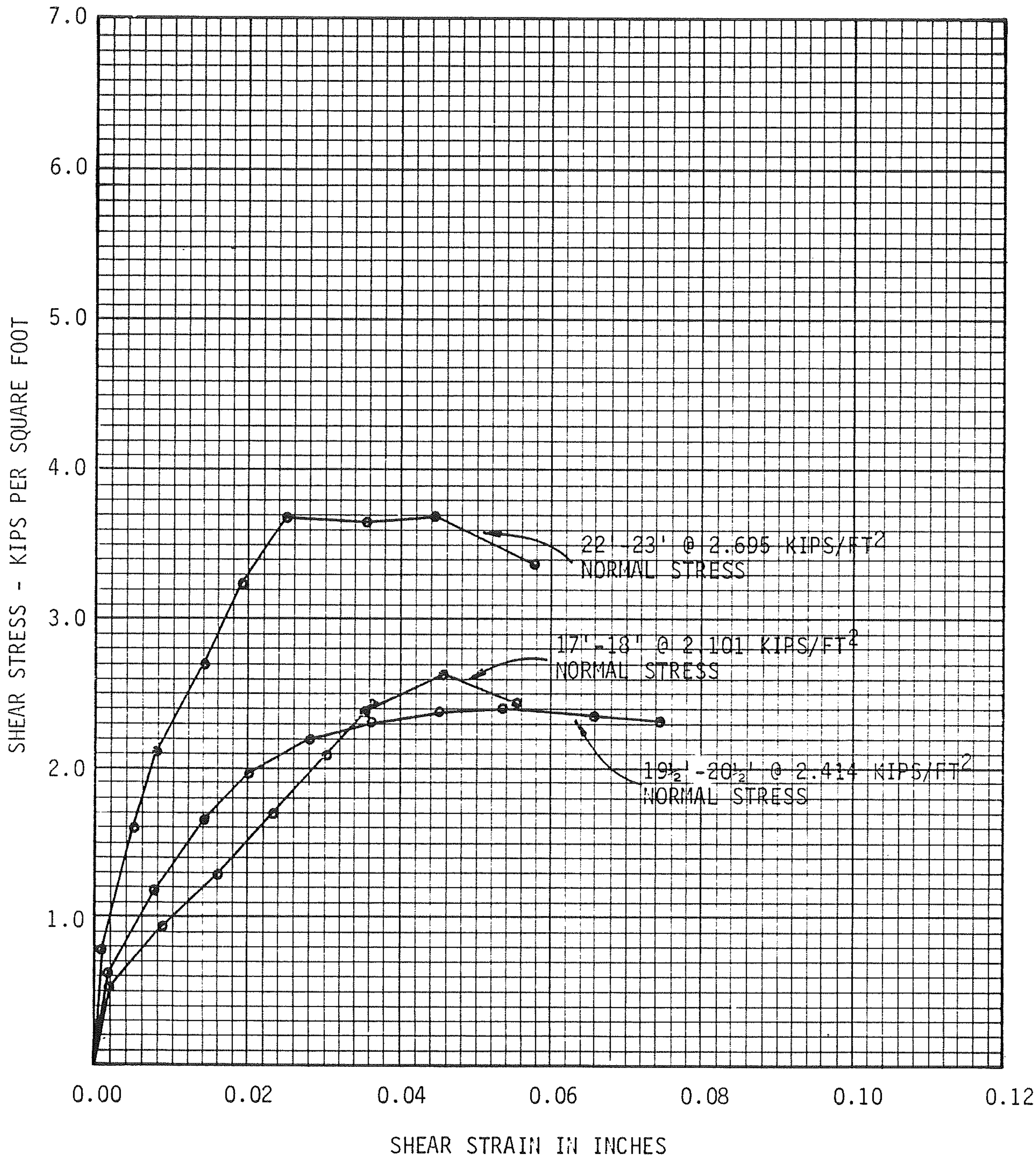
PROJECT Site B - Research Study
Drilled Cast-in-Place Concrete Piles JOB NO. E71-272

DIRECT SHEAR TEST DATA

Test Boring No. 1B



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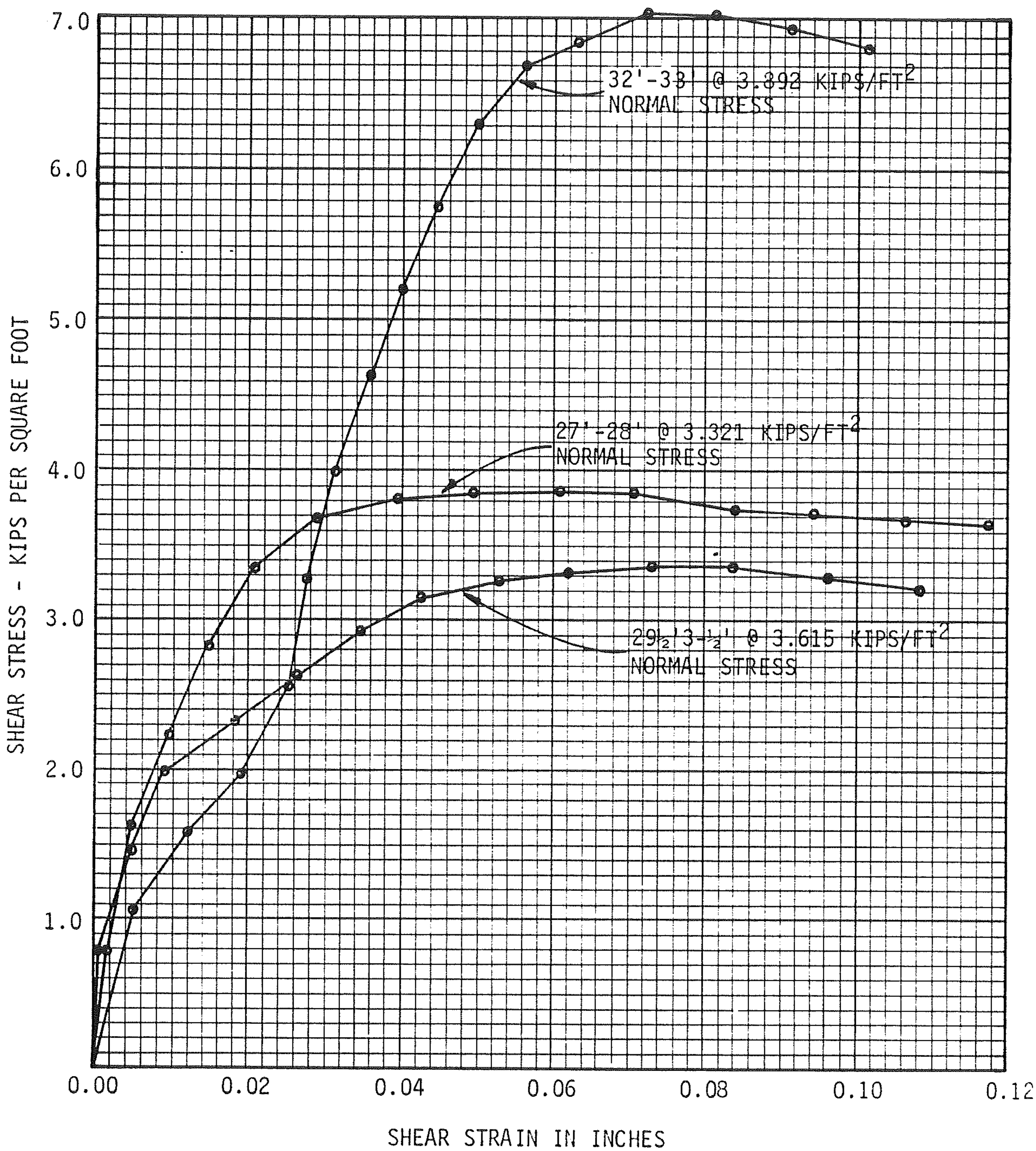
PROJECT Site B - Research Study
Drilled Cast-in-Place Concrete Piles
JOB NO. E71-272

DIRECT SHEAR TEST DATA

Test Boring No. 1B



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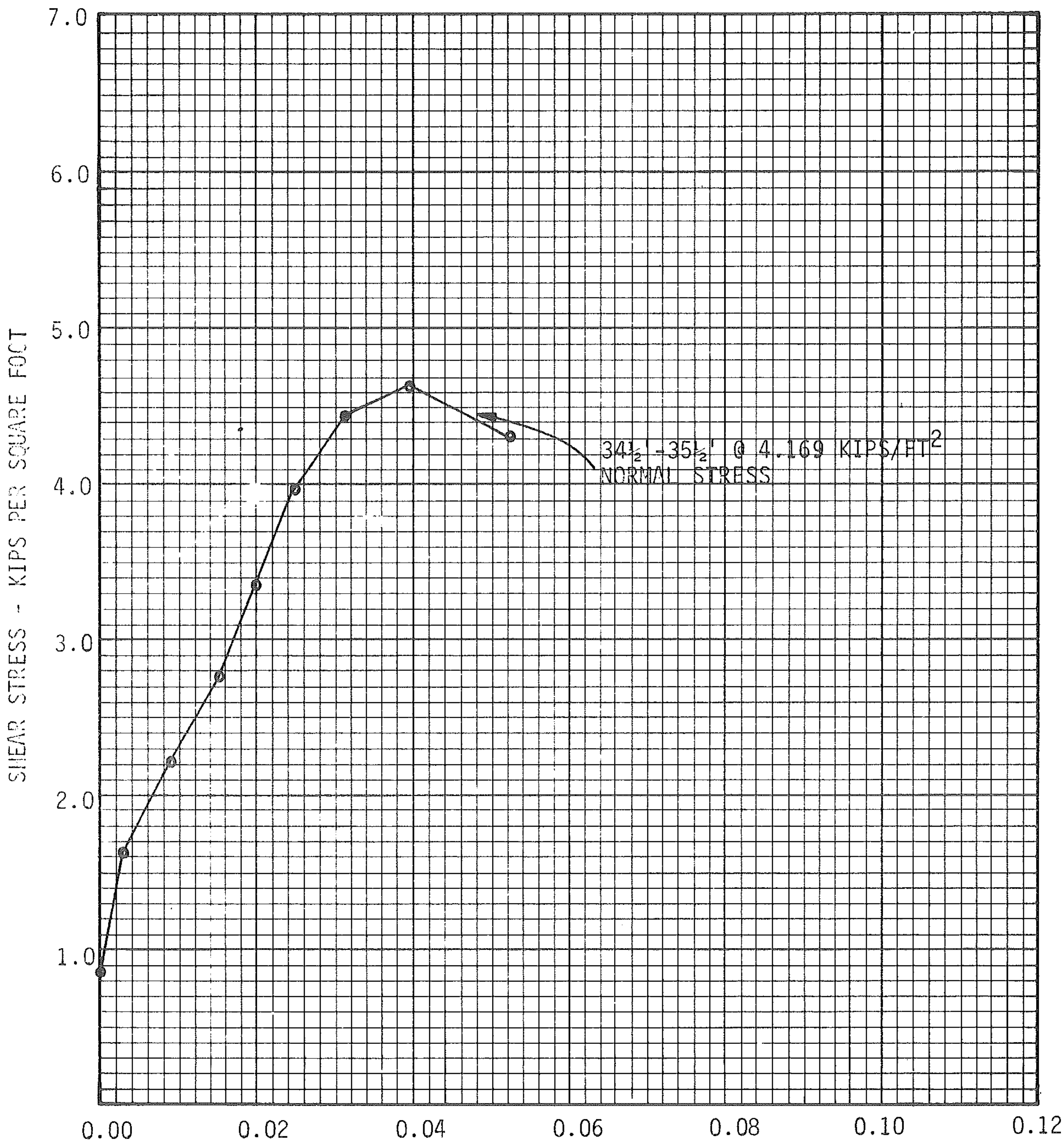
PROJECT Drilled Cast-in-Place Concrete Piles **JOB NO.** E71-272

DIRECT SHEAR TEST DATA

Test Boring No. 1B



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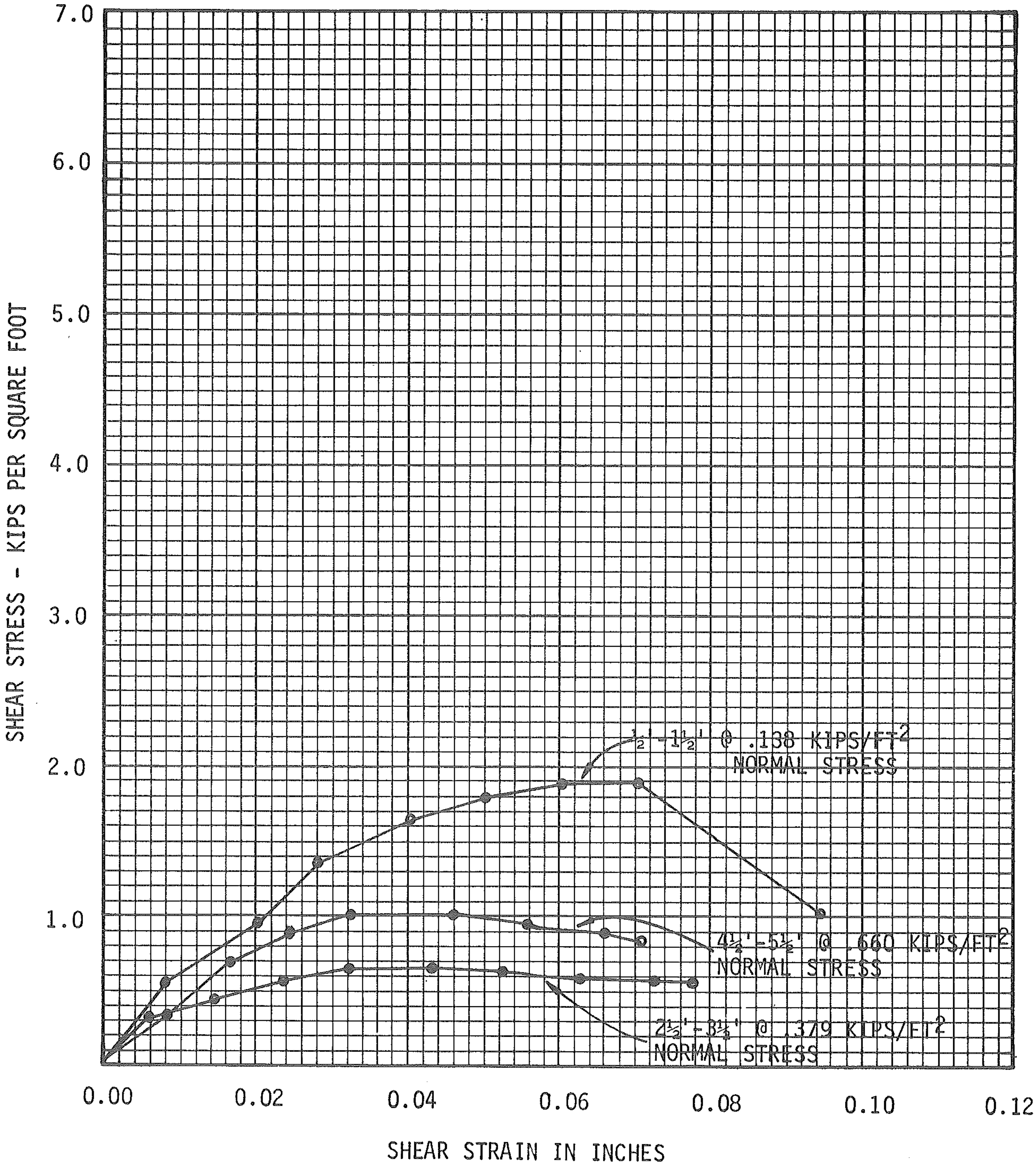
PROJECT Site B - Research Study
Drilled Cast-in-Place Concrete Piles JOB NO. E71-272

DIRECT SHEAR TEST DATA

Test Boring 2B



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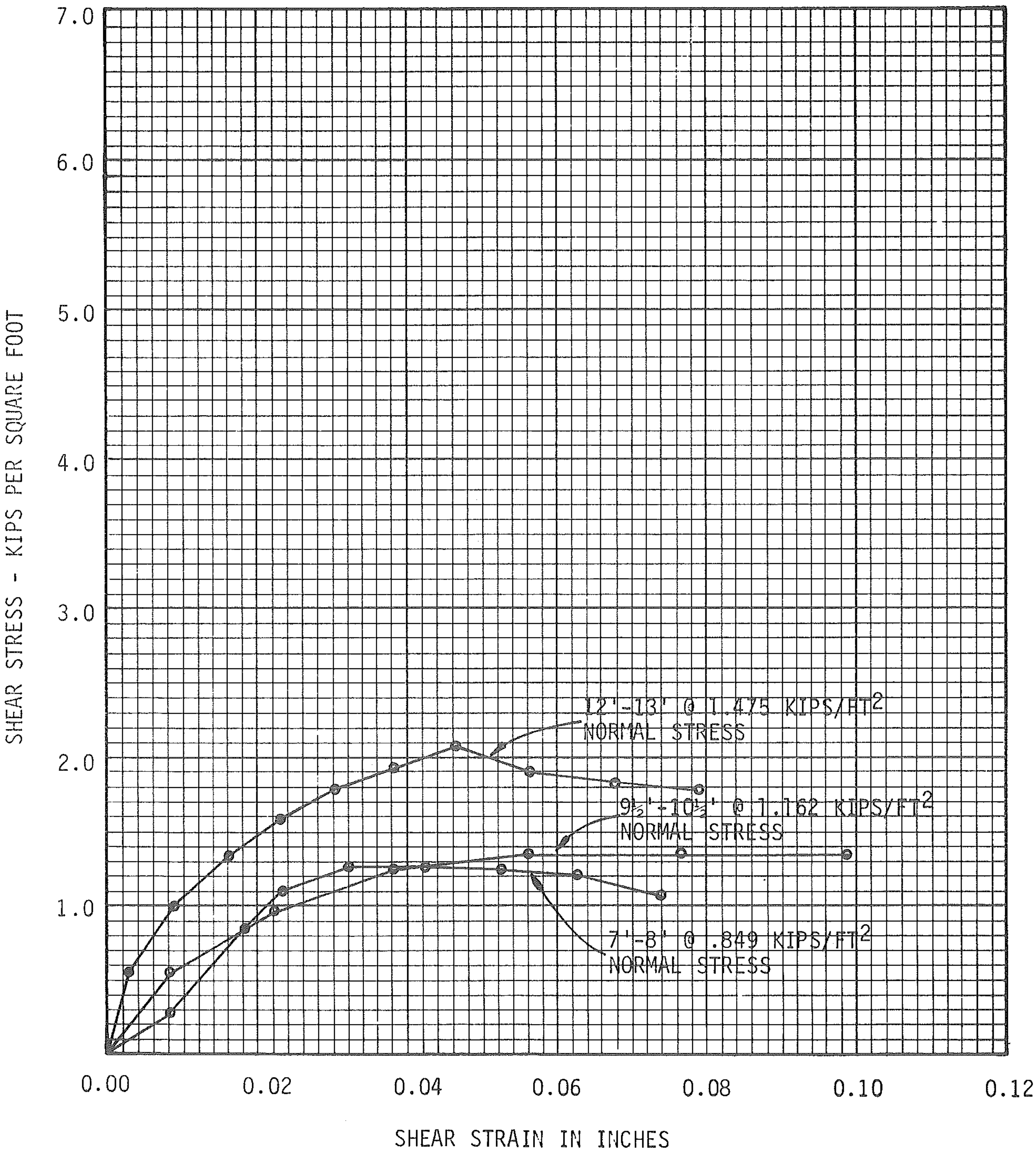
Site B - Research Study

PROJECT Drilled Cast-in-Place Concrete Piles **JOB NO.** E71-272

DIRECT SHEAR TEST DATA

Test Boring No. 2B



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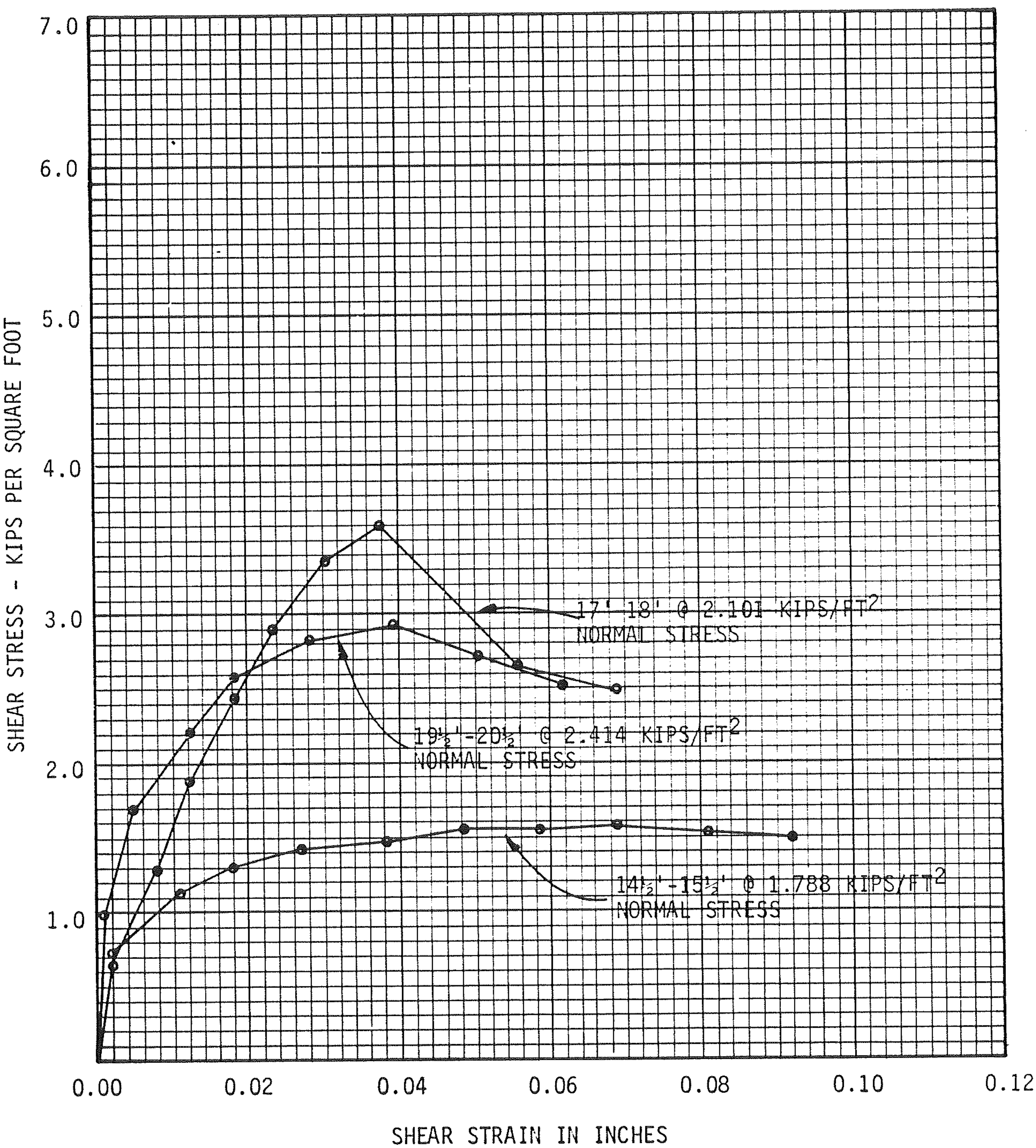
PROJECT Site B - Research Study
Drilled Cast-in-Place Concrete Piles JOB NO. E71-272

DIRECT SHEAR TEST DATA

Test Boring 2B



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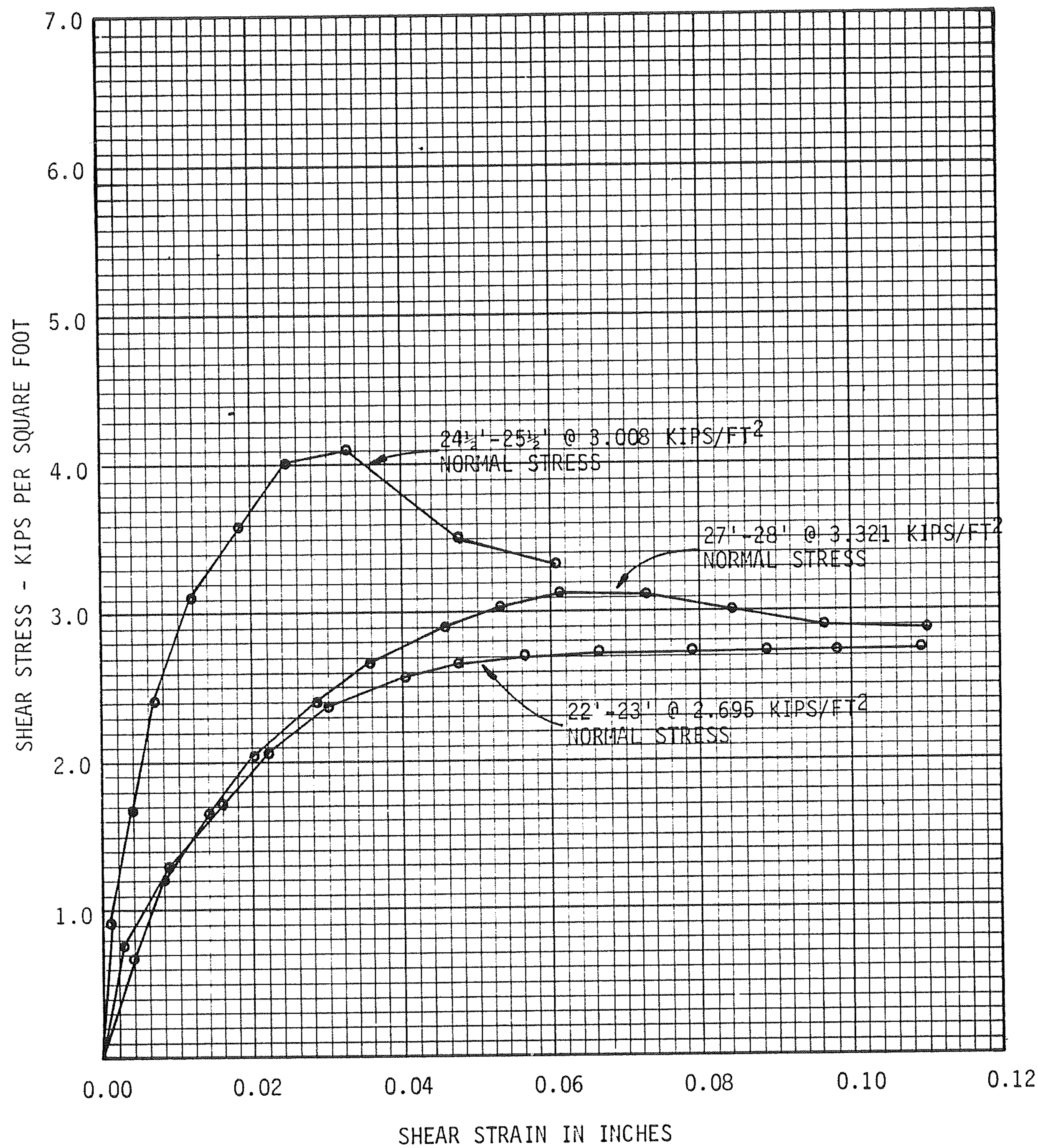
Site B - Research Study

PROJECT Drilled Cast-in-Place Concrete Piles **JOB NO.** E71-272

DIRECT SHEAR TEST DATA

Test Boring No. 2B



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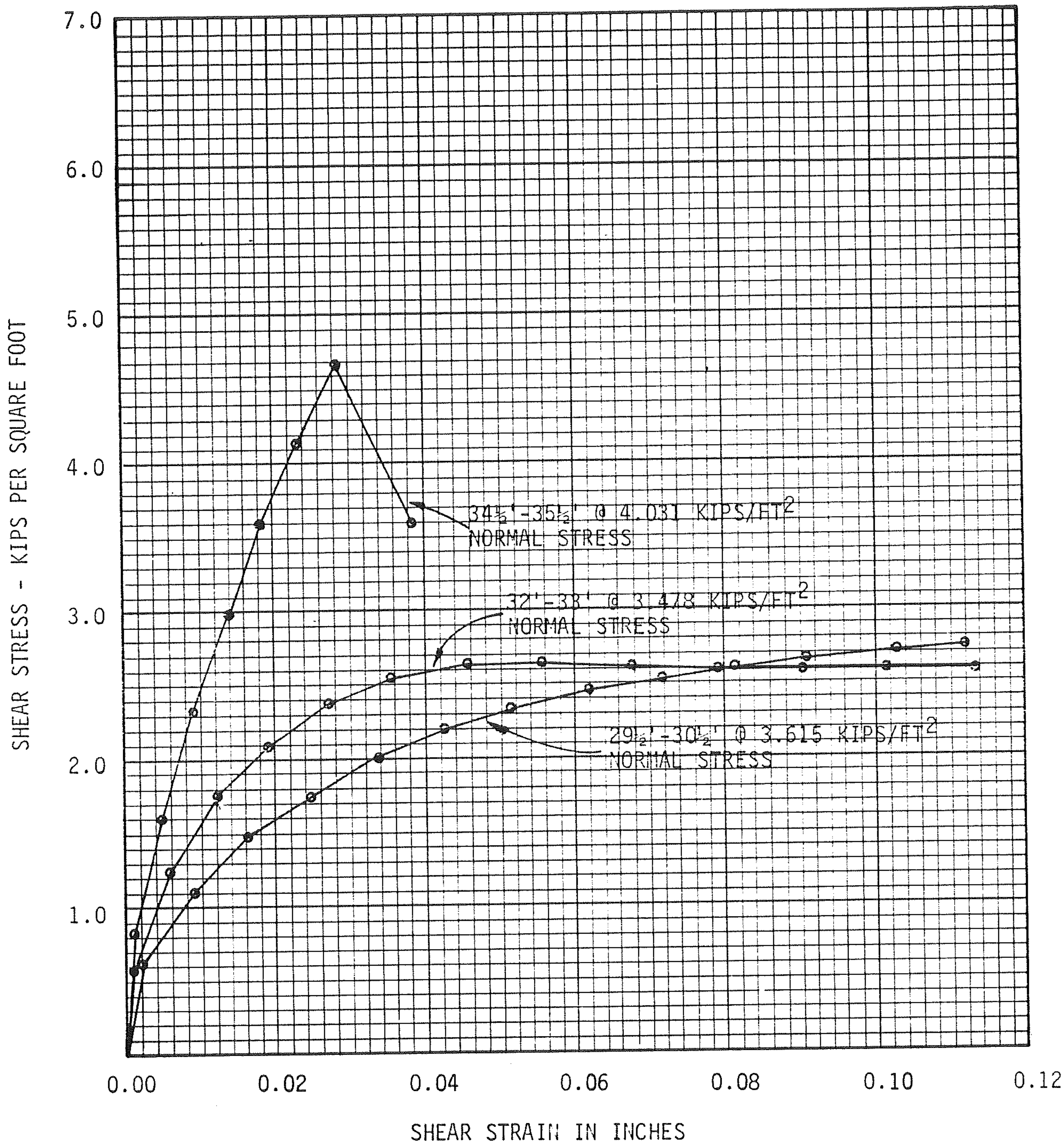
PROJECT Site B--Research Study
Drilled Cast-in-Place Concrete Piles

JOB NO. E71-272

DIRECT SHEAR TEST DATA

Test Boring No. 2B



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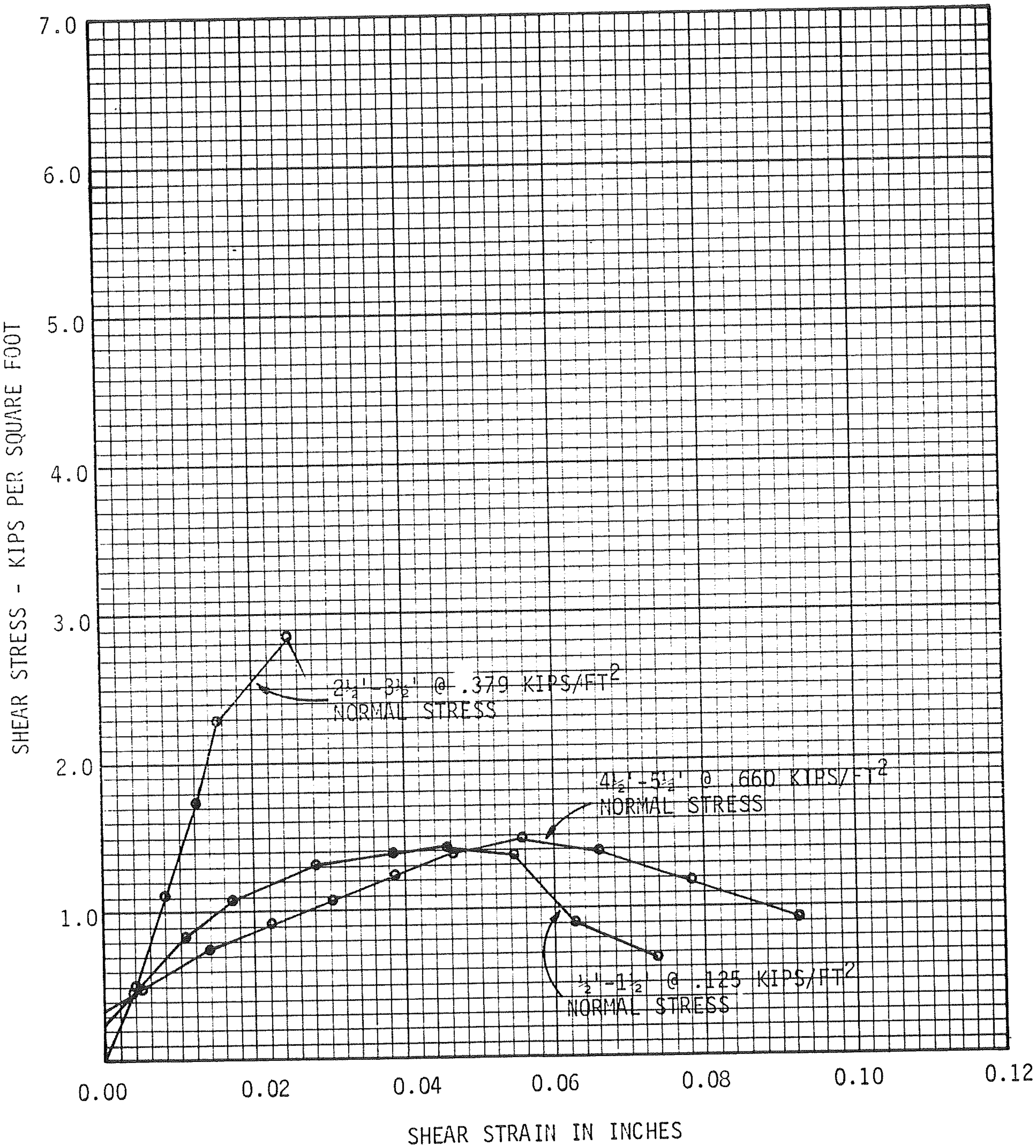
PROJECT Site B - Research Study Drilled Cast-in-Place Concrete Piles JOB NO. E71-272

DIRECT SHEAR TEST DATA

Test Boring No. 3B



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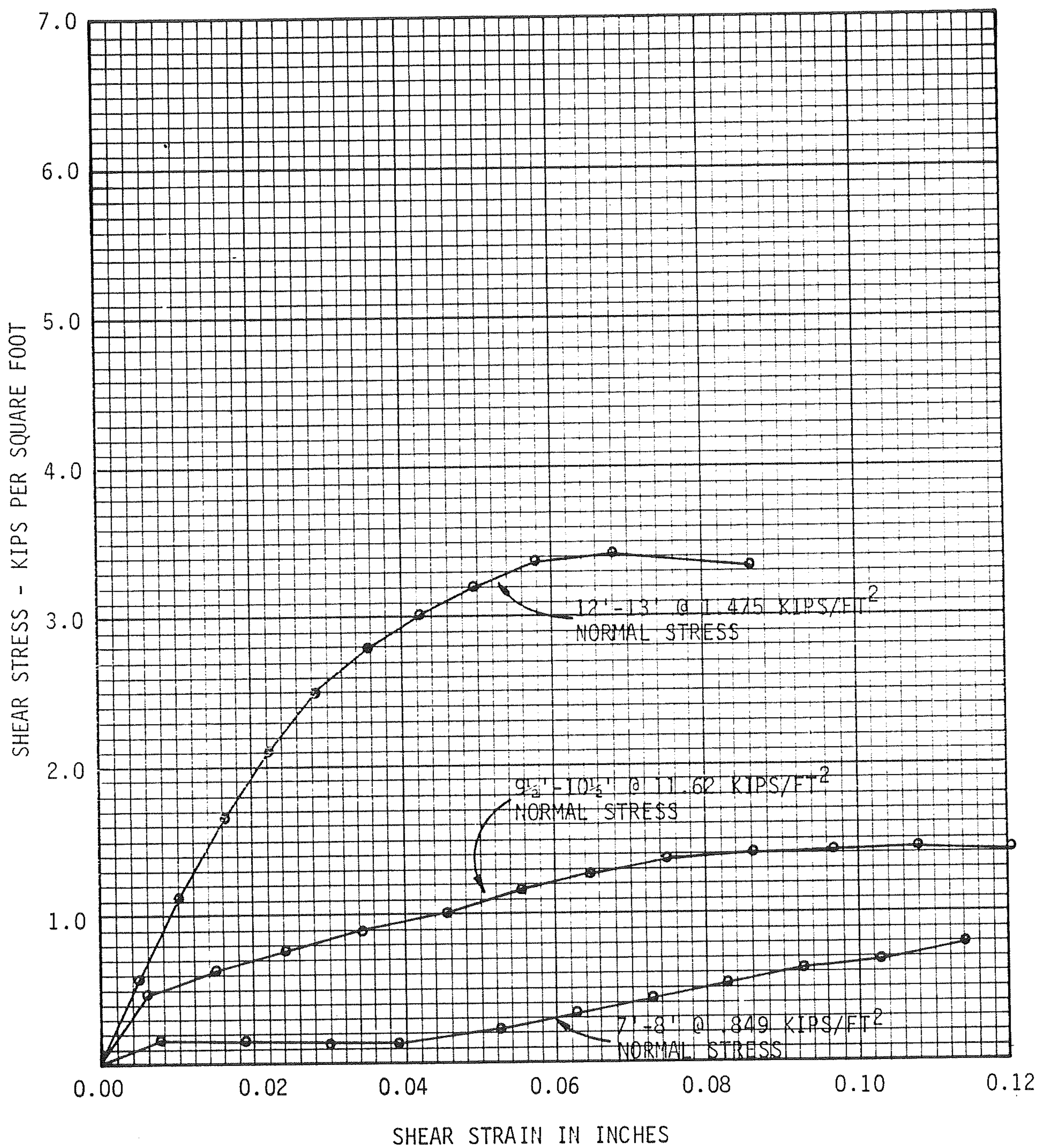
Drilled Cast-in-Place Concrete Piles JOB NO. E71-272

DIRECT SHEAR TEST DATA

Test Boring No. 3B



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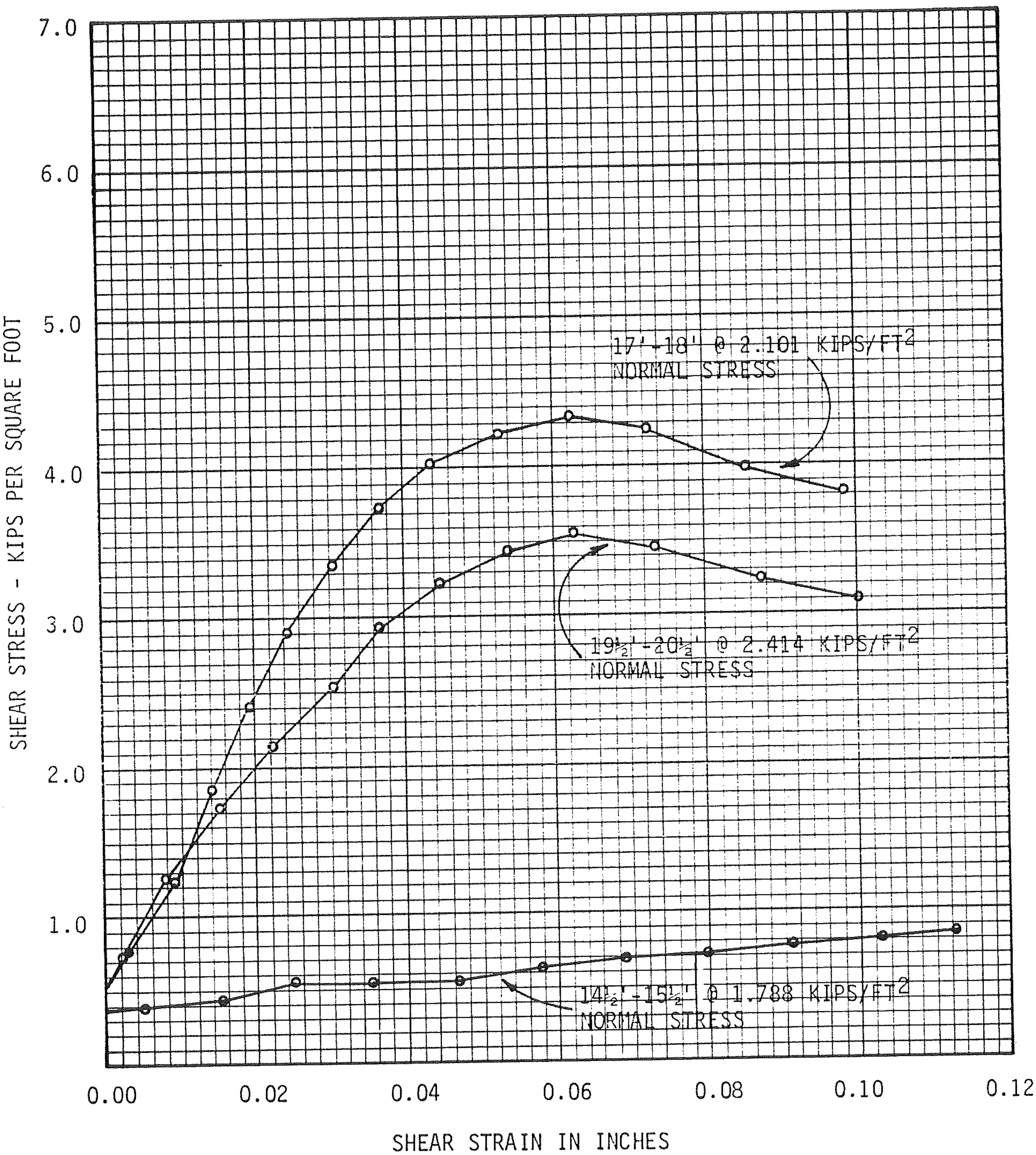
Drilled Cast-in-Place Concrete Piles JOB NO. E71-272

DIRECT SHEAR TEST DATA

Test Boring No. 3B



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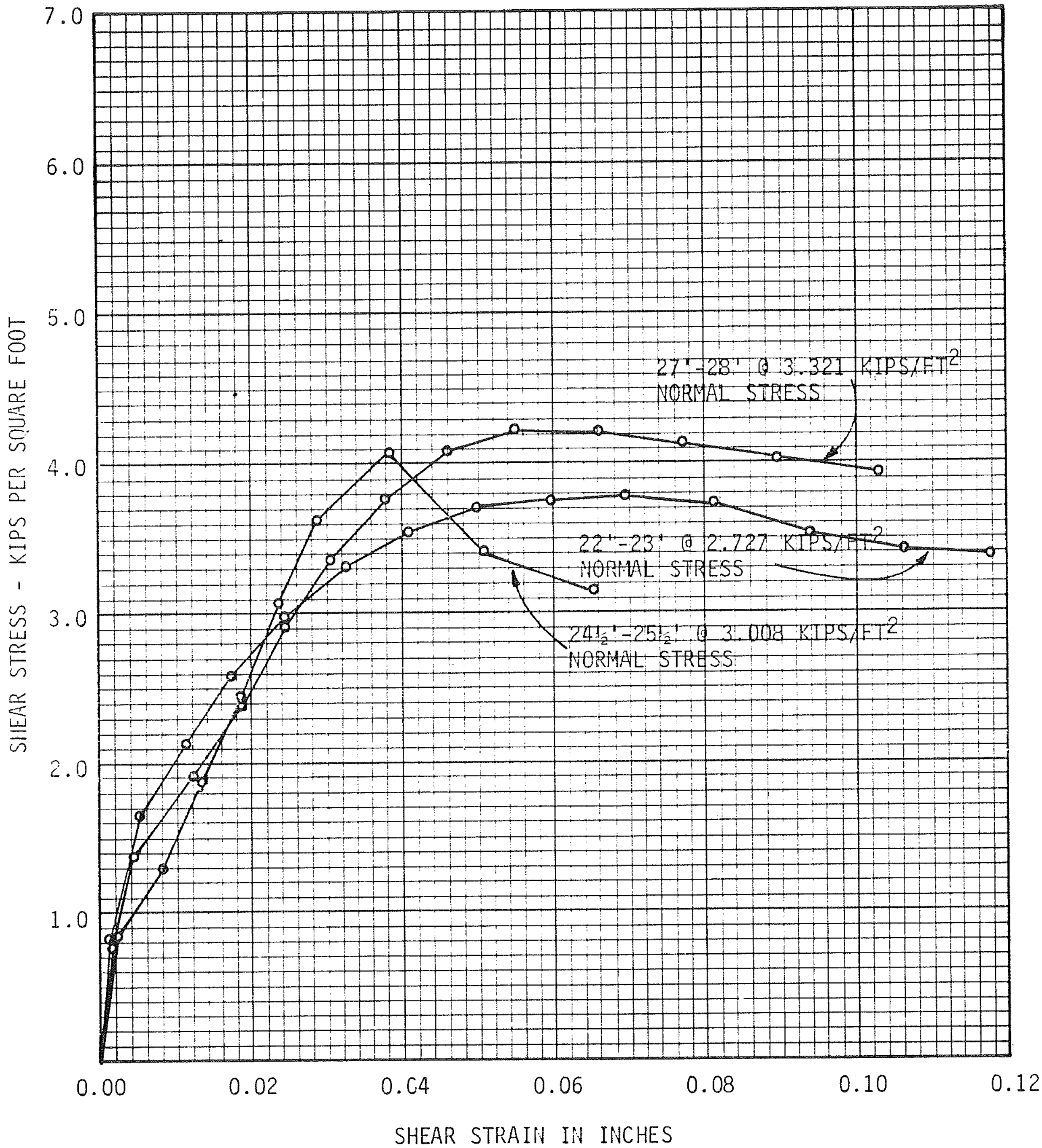
Site B - Research Study
PROJECT Drilled Cast-in-Place Concrete Piles **JOB NO.** E71-272

DIRECT SHEAR TEST DATA

Test Boring No. 3B



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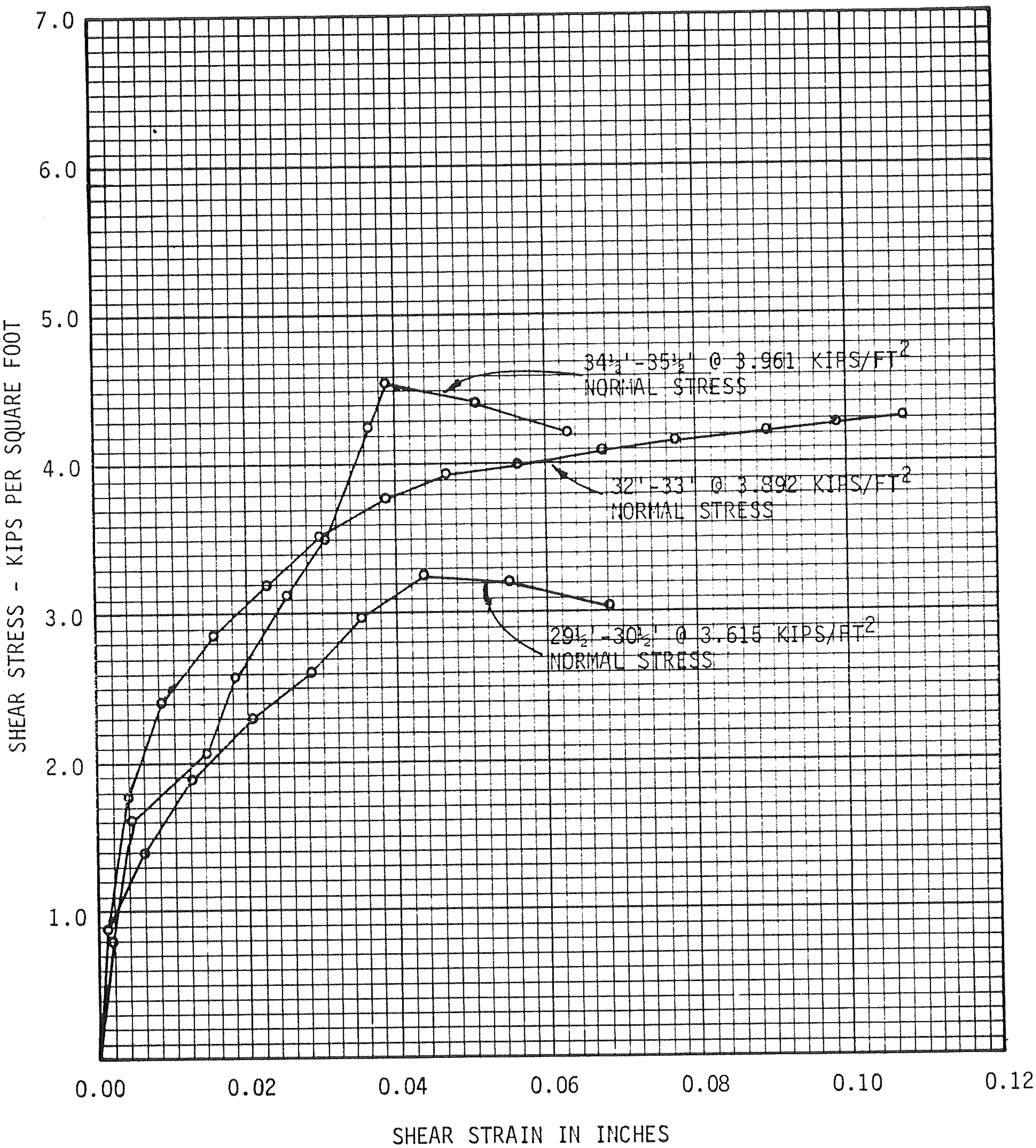
Drilled Cast-in-Place Concrete Piles JOB NO. E71-272

DIRECT SHEAR TEST DATA

Test Boring No. 3B



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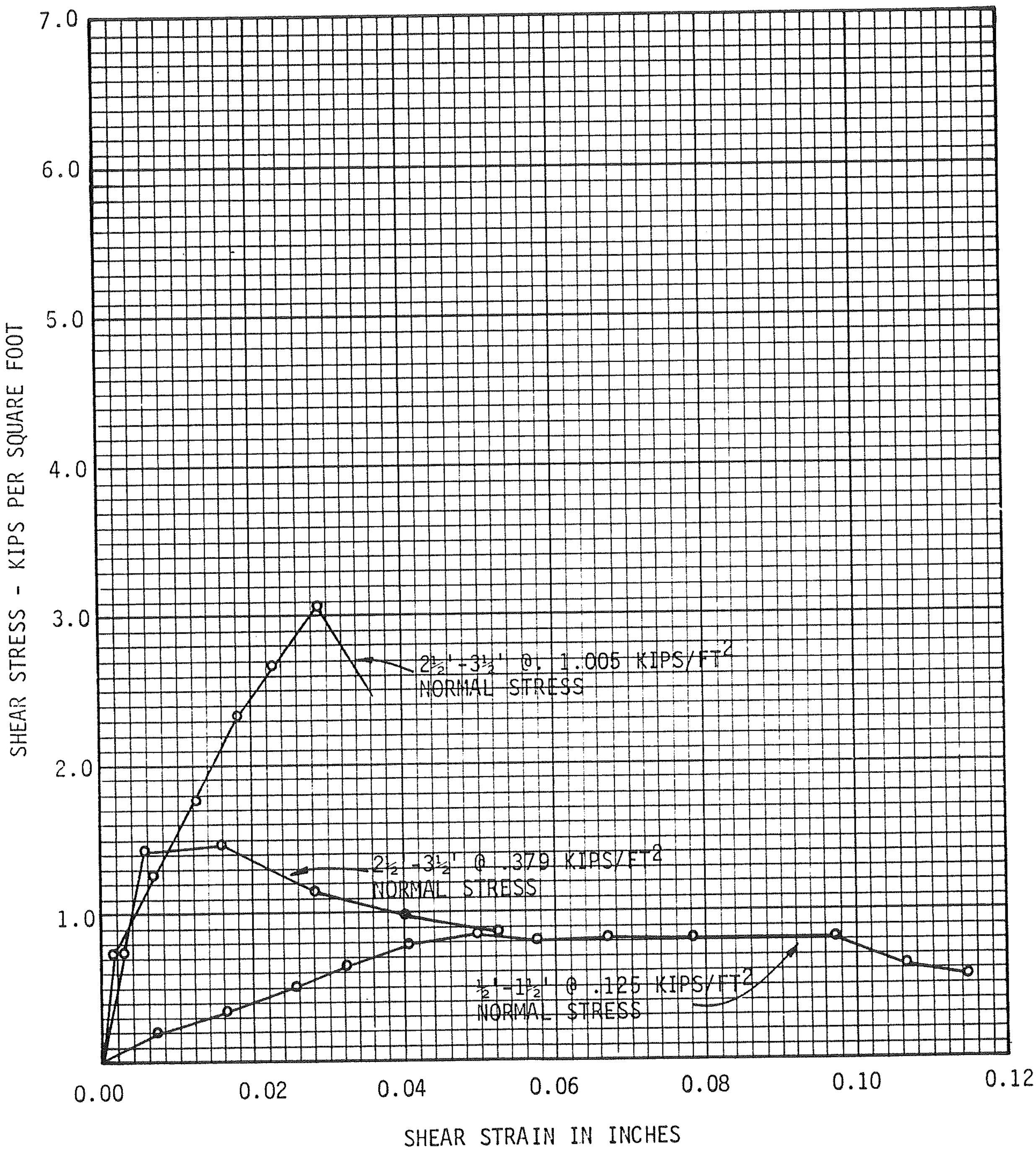
PROJECT Drilled Cast-in-Place Concrete Piles **JOB NO.** E71-272

DIRECT SHEAR TEST DATA

Test Boring No. 4B



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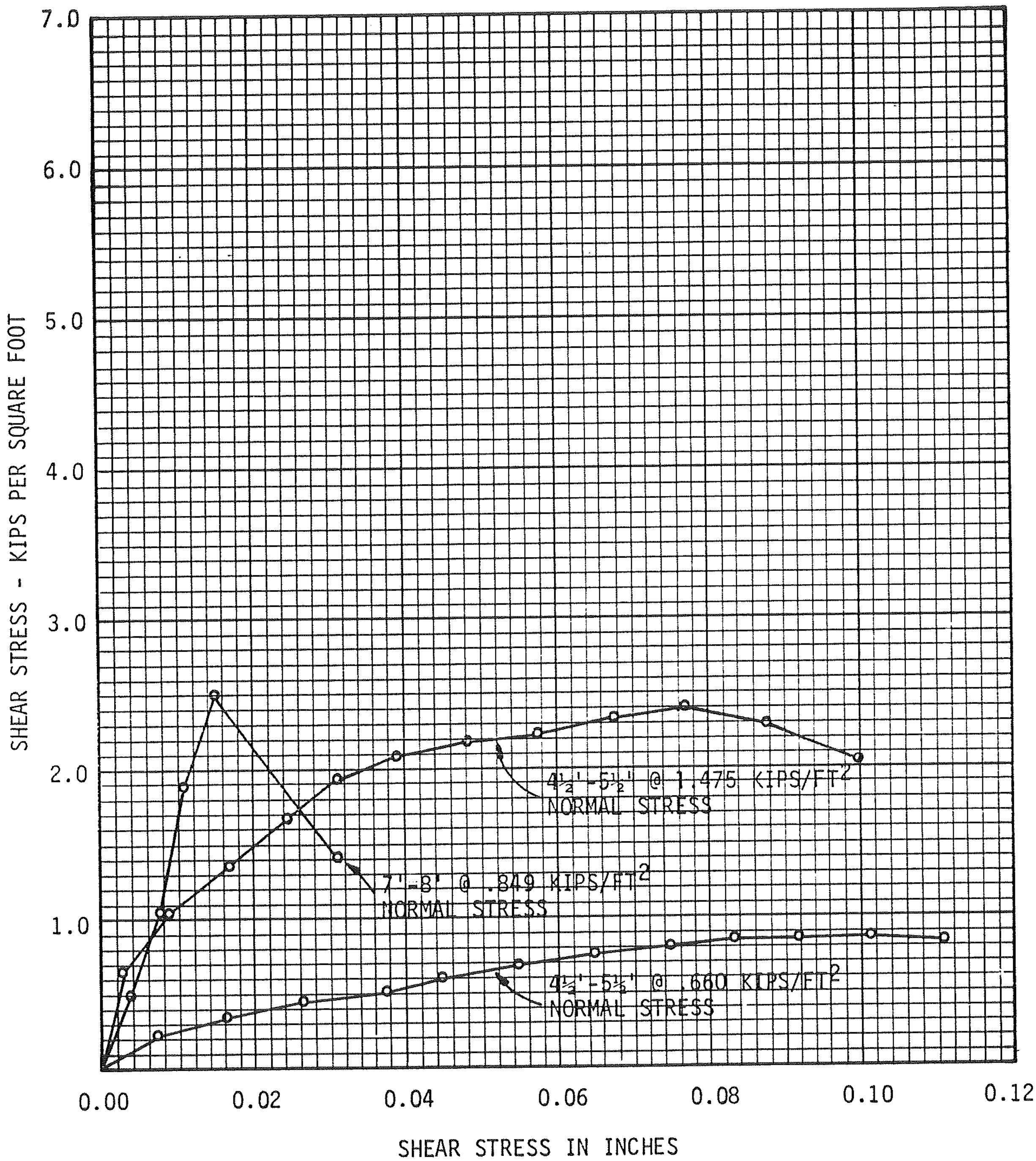
Drilled Cast-in-Place Concrete Piles JOB NO. E71-272

DIRECT SHEAR TEST DATA

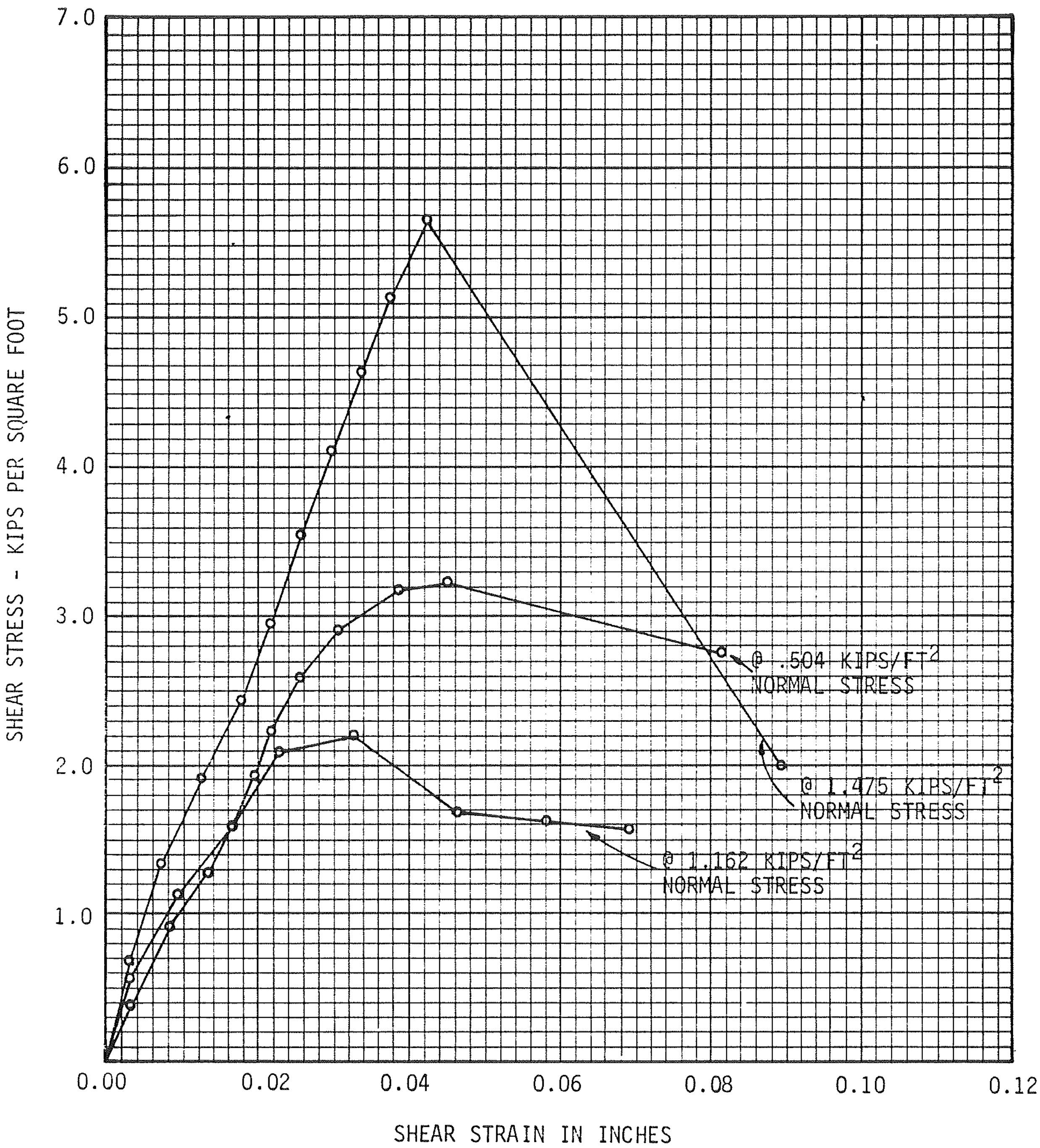
Test Boring No. 4B



Site B - Research Study
PROJECT Drilled Cast-in-Place Concrete Piles JOB NO. E71-272

DIRECT SHEAR TEST DATA

Test Boring No. 4B, 9½'-10½'



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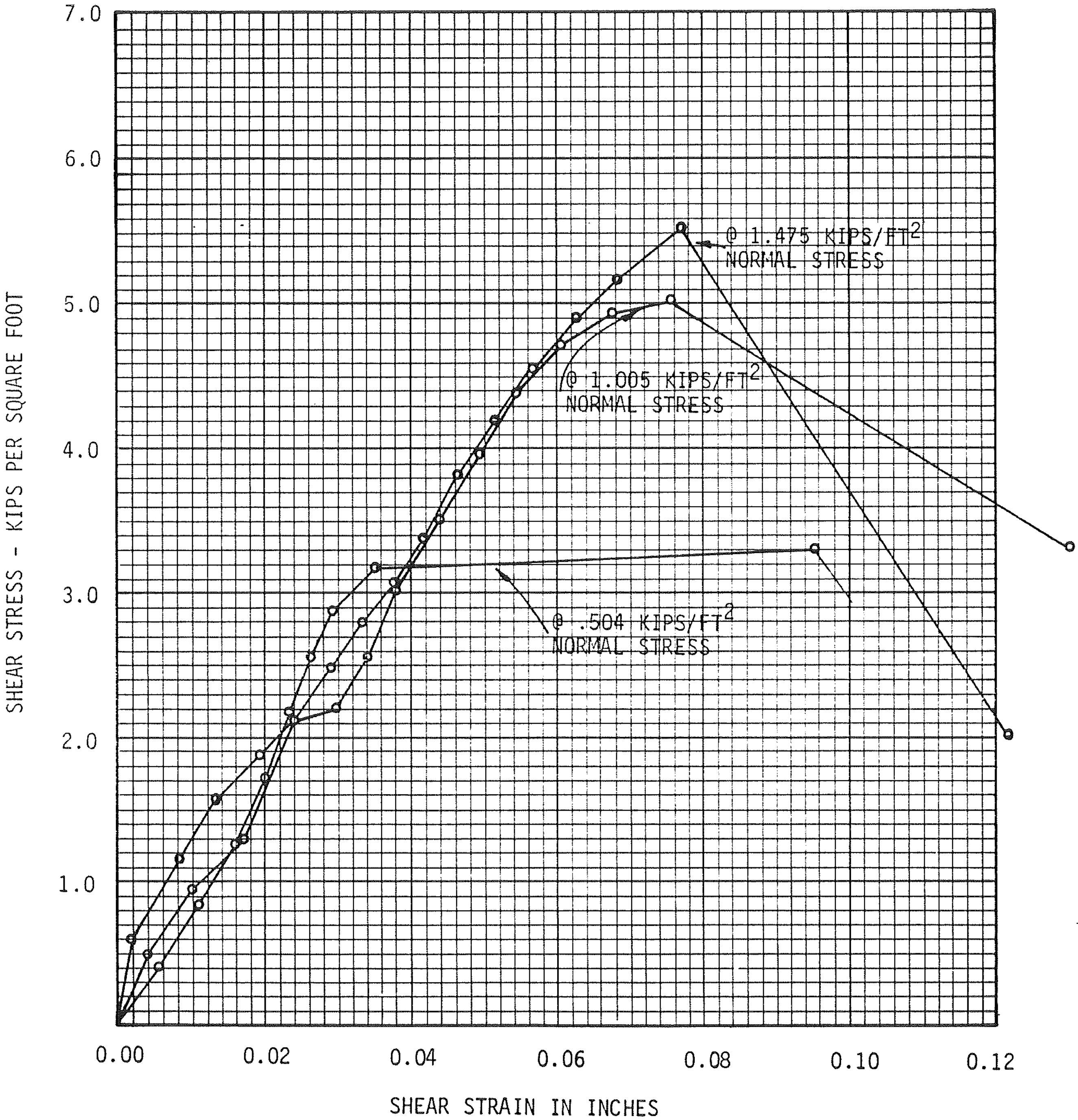
PROJECT Site B - Research Study Drilled Cast-in-Place Concrete Piles JOB NO. E71-272

DIRECT SHEAR TEST DATA

Test Boring No. 4B, 12'-13'



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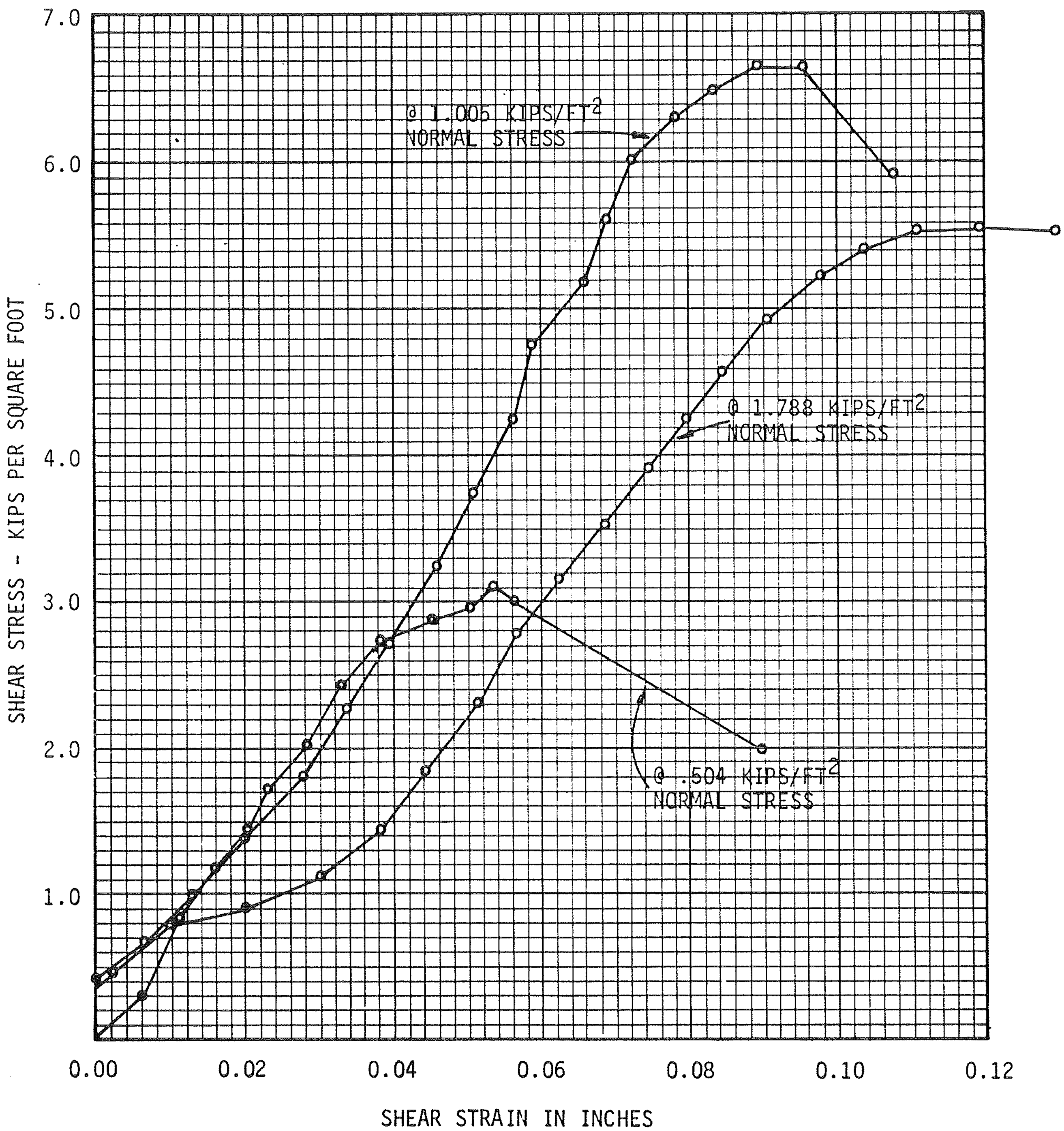
Site B - Research Study

PROJECT Drilled Cast-in-Place Concrete Piles JOB NO. E71-272

DIRECT SHEAR TEST DATA

Test Boring No. 4B, 14½' - 15½'



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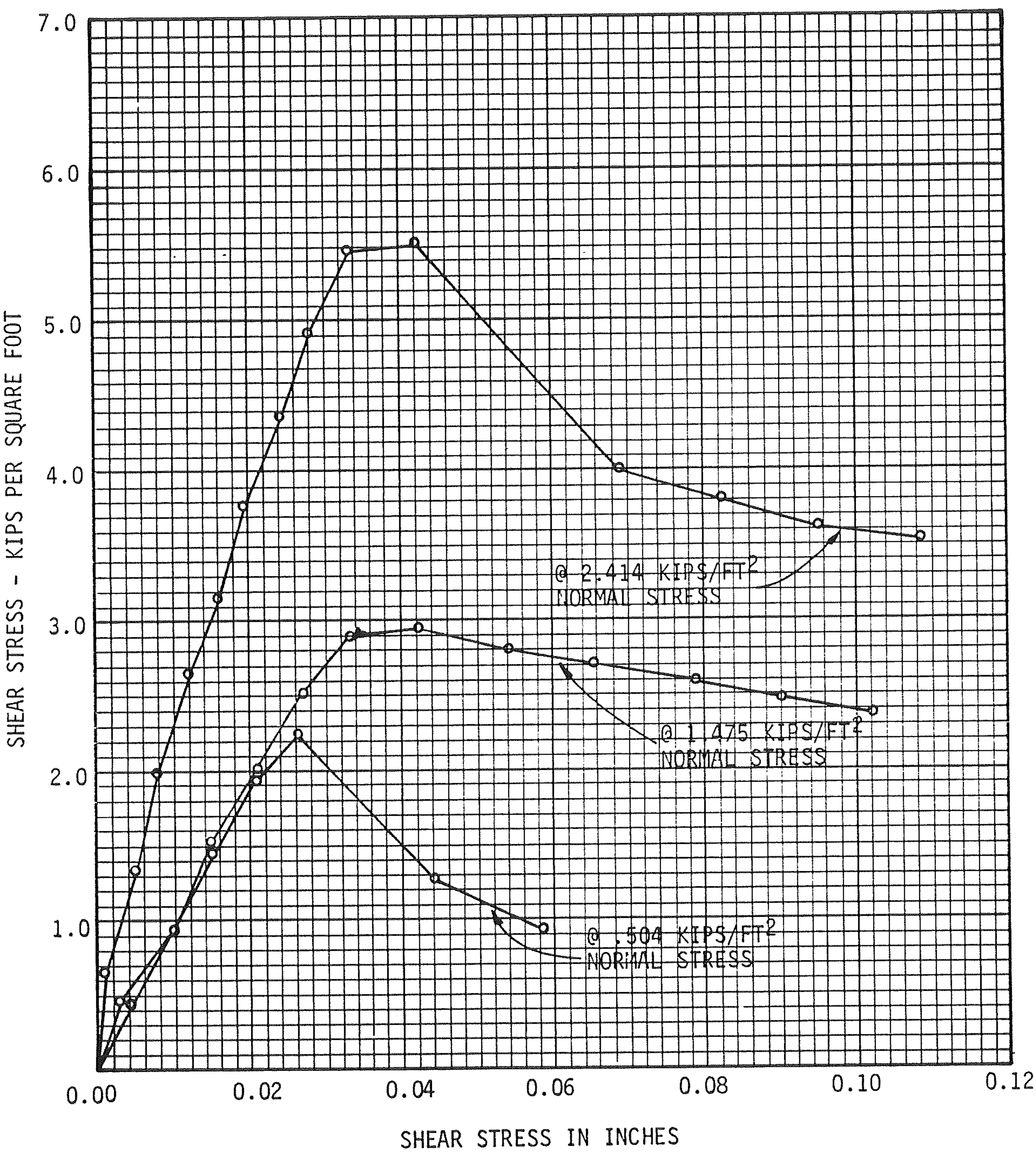
PROJECT Site B - Research Study Drilled Cast-in-Place Concrete Piles JOB NO. E71-272

DIRECT SHEAR TEST DATA

Test Boring No. 4B, 19½'-20½'



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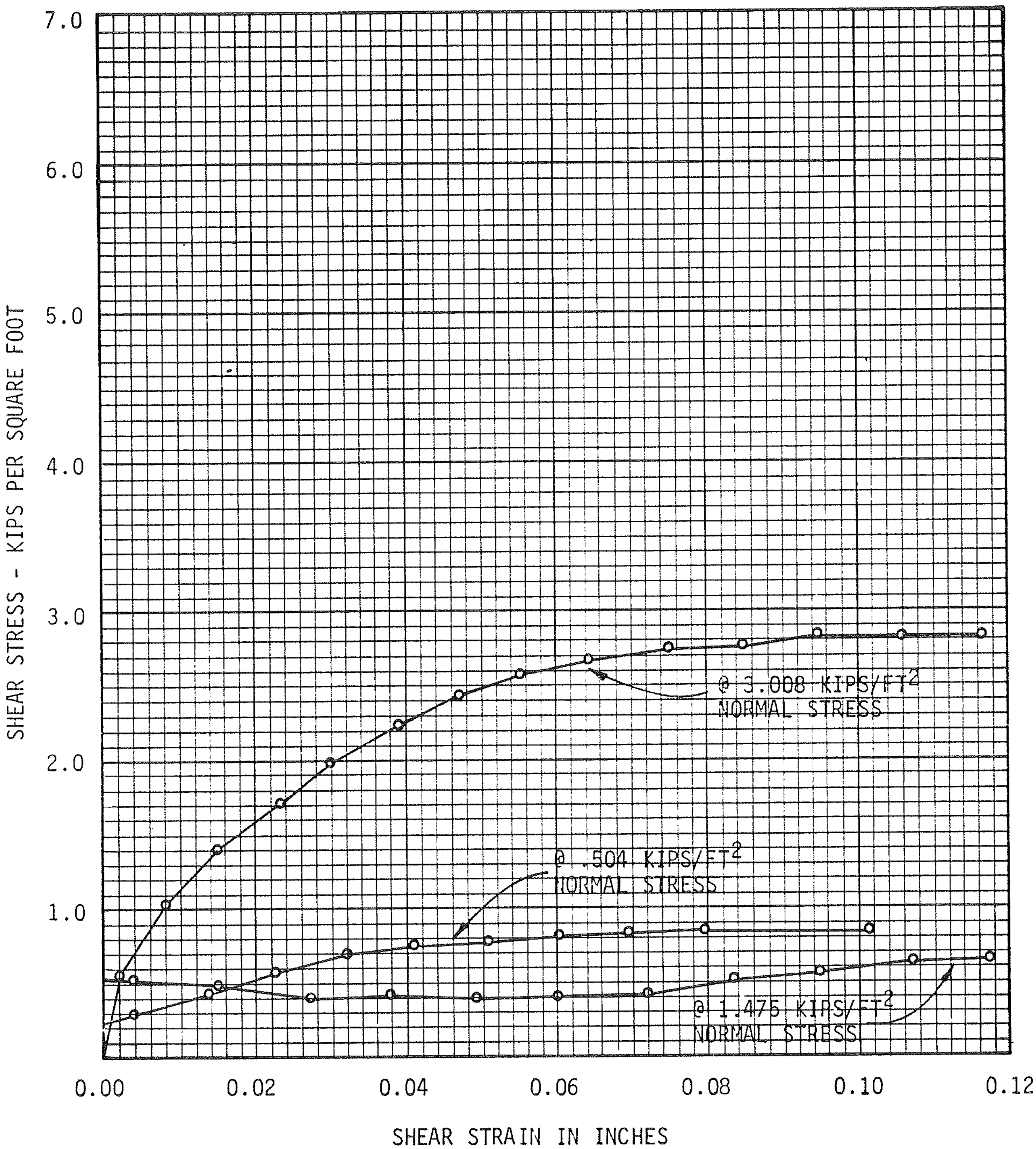
Site B - Research Study

PROJECT Drilled Cast-in-Place Concrete Piles **JOB NO.** E71-272

DIRECT SHEAR TEST DATA

Test Boring No. 4B, 24½'-25½'



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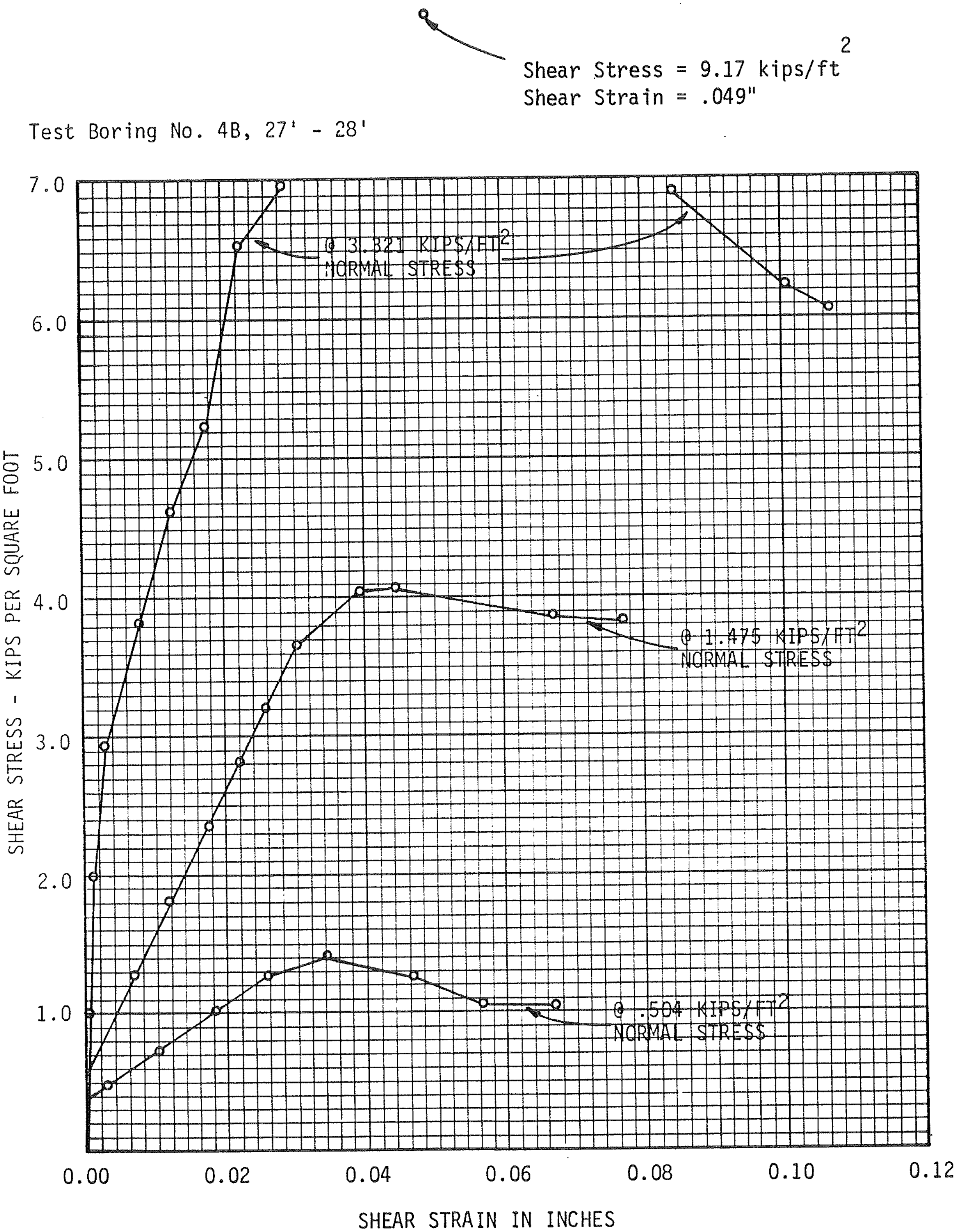
PROJECT Site B - Research Study
Drilled Cast-in-Place Concrete Piles JOB NO. E71-272

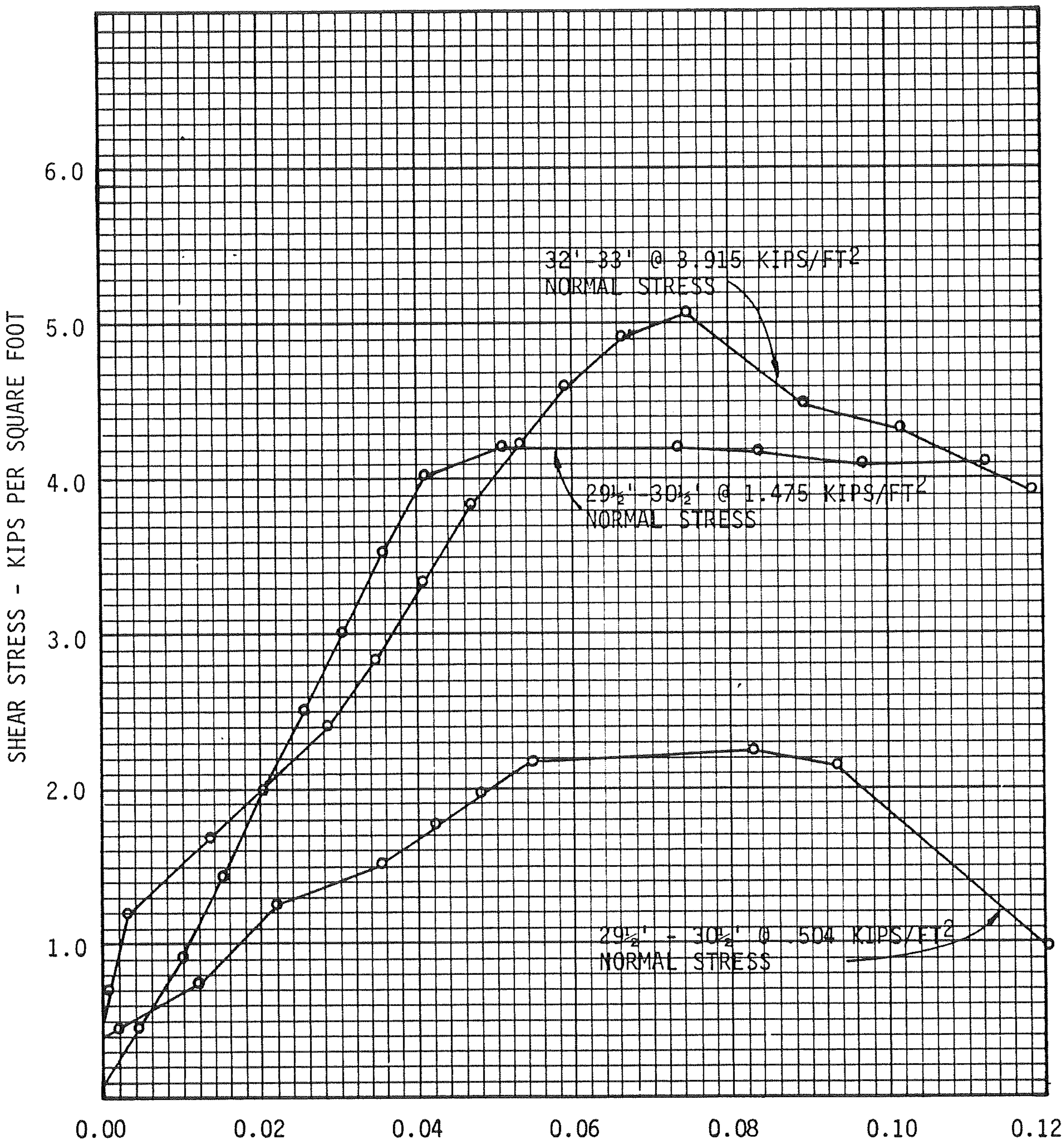
DIRECT SHEAR TEST DATA



Site B - Research Study
PROJECT Drilled Cast-in-Place Concrete Piles JOB NO. F71-272

DIRECT SHEAR TEST DATA

Test Boring No. 4B



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MATERIALS TESTING ENGINEERS
ENGINEERING ANALYSIS • PHYSICAL TESTING • QUALITY CONTROL • FIELD EXPLORATION

SUMMARY OF UNCONFINED COMPRESSION TEST DATA

DATE 2-9-73

PROJECT Research Study Drilled Cast-in-Place Concrete Piles

JOB NO. E71-272

LOCATION

LAB NO.

CLIENT

ADDRESS

SOURCE OF SAMPLE Site B

MATERIAL Block Samples

SAMPLED BY

SUBMITTED BY

REQUESTED BY

TESTED

DATE RECEIVED

TEST RESULTS

Depth	Unconfined Compressive Strength (psf)
4½'-5½'	10,335
	9,569
	10,615
	6,874
	6,902
7'-8'	6,585
9'-10'	5,272

PHOENIX (602) 272-6848 FLAGSTAFF (602) 774-4433 EL PASO (915) 772-3088 ALBUQUERQUE (505) 344-9940

Site C - Research Study

PROJECT Drilled Cast-in-Place Concrete Piles

JOB NO. E71-272 DATE 3-15-71

LOG OF TEST BORING NO. 1C

RIG TYPE CME-55

BORING TYPE 6½" Hollow Stem Auger

SURFACE ELEV.

DATUM

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows per foot 140 lb. 30" free-fall drop hammer	Dry Density Lbs. per cu. ft.	Moisture Content Per Cent of Dry Wt.	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
0	8			S	1-2-2-2-2-3-3-3-3					SILTY CLAY, weakly to moderately lime cemented, medium plasticity, light brown
	14				16		7	CL		
	22			S	3-3-3-2-3-4-3-4-4					
	34				20		7			
5	58			S	2-2-4-5-6-7-9-12-11			CL		SANDY CLAY, moderately to strongly lime cemented, medium plasticity, light brown
	91				50		6			
*	100/4½"			S	50/1"					
							4			
	23			S	4-4-3-4-4-3-4-4-4			CH		SANDY CLAY, some gravel, moderately lime cemented, high plasticity, very light brown
10	30				23		18			
	17			S	1-2-4-4-4-3-4-4-4					
	16				23		10			CLAYEY SAND, some gravel, moderately to strongly lime cemented, medium to high plasticity, very light brown
	15			S	8-6-6-13-7-5-5-4-5					
	20				30		15			
15*	100/9"			S	5-5-4-2-4-2-7-10-10					
					54		14	SC		
	69/8"			S	7-6-5-7-15-9-7-9-16					
					63		15			
20	313			S	11-18-14-13-15-10-6-9-9					
*	250/5"				62		9			
	42/6"			S	3-5-4-4-6-5-3-3-3					
					24		17			
	97			S	5-2-3-4-5-5-7-6-9			CH		SANDY CLAY, moderately lime cemented, high plasticity, light brown
25	150/6"				36		19			
30										Stopped auger at 24' Stopped sampler at 25'6"

*Penetrometer refused at 6'4½"
Auger advanced to 8'

*Penetrometer refused at 14'9"
Auger advanced to 18'4"

*Penetrometer refused at 20'5"
Auger advanced to 22'6"

Penetrometer refused at 24'6"

GROUND WATER

DEPTH	HOUR	DATE
	NONE	

SAMPLE TYPE

A – Auger cuttings. B – Block sample
S – 2" O.D. 1.38" I.D. tube sample.
U – 3" O.D. 2.42" I.D. tube sample.
T – 3" O.D. thin-walled Shelby tube.

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PROJECT Site C - Research Study Drilled Cast-in-Place Concrete Piles

LOG OF TEST BORING NO. 1C

JOB NO. E71-272 DATE 3-16-70

RIG TYPE CME-55
BORING TYPE 6½" Hollow Stem Auger
SURFACE ELEV.
DATUM

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows per foot 140 lb. 30" free-fall drop hammer	Dry Density Lbs. per cu. ft.	Moisture Content Per Cent of Dry Wt.	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
0			☒	U	1-2-2-2-3-4 / 14	87	8	CL		
			☒	U	3-3-4-4-5-4 / 23	78	6			
5			☒	U	8-14-18-17-17-14 / 88			CL		
			☒	U	49-46-20-21-31-38 / 209	104	6			
			☒	U	6-7-8-7-8-7 / 43	94	13	CH		
10			☐	U	2-5-6-7-7-7 / 34	87	22			
			☐	U	2-3-3-6-7-9 / 30	89	27			
15			☒	U	6-13-19-17-16-18 / 89	108	7			
			☒	U	4-4-4-4-5-16 / 37	88	30	SC		
			☒	U	4-5-7-8-7-10 / 41	97	19			
20			☒	U	4-8-18-25-31-16 / 102	93	18			
			☒	U	2-4-3-6-6-8 / 29	89	25			
25			☒	U	2-5-5-8-7-8 / 35	114	13	CH		
										Stopped auger at 24'6" Stopped sampler at 25'6"
30										

GROUND WATER

DEPTH	HOUR	DATE
	NONE	

SAMPLE TYPE

A – Auger cuttings. B – Block sample
S – 2" O.D. 1.38" I.D. tube sample.
U – 3" O.D. 2.42" I.D. tube sample.
T – 3" O.D. thin-walled Shelby tube.

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PROJECT Site C - Research Study Drilled Cast-in-Place Concrete Piles

JOB NO. F71-272 DATE 3-17-71

LOG OF TEST BORING NO. 2C

RIG TYPE CME-55
BORING TYPE 6½" Hollow Stem Auger
SURFACE ELEV.
DATUM

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows per foot 140 lb. 30" free-fall drop hammer	Dry Density Lbs. per cu. ft.	Moisture Content Per Cent of Dry Wt.	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
0	12 17		☒	S	1-1-3-2-3-3-3-2-3 16		6	CL		SILTY CLAY, weakly to moderately lime cemented, medium plasticity, light brown
	18 25		☒	S	3-3-4-4-3-4-4-3-3 21		7	CL		SANDY CLAY, moderately lime cemented, medium plasticity, very light brown
5	35 59		☒	S	3-4-5-5-5-4-5-5-8 32		4			SANDY CLAY, some gravel, moderately to strongly lime cemented, high plasticity, very light brown
* * *	100/0" 100/0" 100/0"		☒	S	12-26-32-28-20-18-22-23-25 136		6	CH		
10			☒	S	6-7-8-9-8-7-7-7-5 43		10			CLAYEY SAND, some gravel, moderately to strongly lime cemented, medium to high plasticity, very light brown
	52 65 31		☒	S	3-4-4-6-6-6-5-5-5 33		11			
15	56 31		☒	S	2-2-3-4-4-5-5-6-6 30		18	SC		
	48 26 47		☒	S	2-2-4-10-21-9-7-10-9 66		10			
20	75 39 124		☒	S	1-4-5-10-9-15-14-11-12 71		9			
	55 31		☒	S	6-5-2-4-3-4-5-6-7 29		24			SANDY CLAY, moderately lime cemented, high plasticity, light brown
25	65		☒	S	4-5-7-7-6-6-6-6-8 39		14	CH		

*Penetrometer refused at 7'0"
Auger advanced to 8'0"

*Penetrometer refused at 8'0"
Auger advanced to 9'0"

*Penetrometer refused at 9'0"
Auger advanced to 11'0"

Penetrometer stopped at 25'

Stopped auger at 24'6"
Stopped sampler at 26'

30

GROUND WATER

DEPTH	HOUR	DATE
	NONE	

SAMPLE TYPE

A – Auger cuttings. B – Block sample
S – 2" O.D. 1.38" I.D. tube sample.
U – 3" O.D. 2.42" I.D. tube sample.
T – 3" O.D. thin-walled Shelby tube.

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PHOENIX • FLAGSTAFF • ALBUQUERQUE • EL PASO

PROJECT Site C - Research Study Drilled Cast-in-Place Concrete Piles

JOB NO. E71-272 DATE 3-17-71

LOG OF TEST BORING NO. 2C

RIG TYPE CME-55

BORING TYPE 6½" Hollow Stem Auger

SURFACE ELEV.

DATUM

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows per foot 140 lb. 30" free-fall drop hammer	Dry Density Lbs. per cu. ft.	Moisture Content Per Cent of Dry Wt.	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
0				U	1-2-3-4-5-5- 20	92	7	CL		
				U	2-3-4-4-4-4 21	88	5	CL		
5				U	3-5-5-6-7-8 34	92	6			
				U	100/1½" (no recovery)			CH		
10				U	2-8-11-11-18-20 70	86	13			
				U	10-11-11-9-8-10 59	99	14			
15				U	5-5-7-8-7-7 39	98	13	SC		
				U	4-2-3-2-3-6 20	79	33			
20				U	2-1-2-2-3-3 13	109	14			
				U	7-9-10-13-14-9 62	80	33			
25				U	3-5-5-7-8-14 42	114	14	CH		
										Stopped auger at 26'
30										

GROUND WATER

DEPTH	HOUR	DATE
	NONE	

SAMPLE TYPE

A – Auger cuttings. B – Block sample
S – 2" O.D. 1.38" I.D. tube sample.
U – 3" O.D. 2.42" I.D. tube sample.
T – 3" O.D. thin-walled Shelby tube.

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PROJECT Site C - Research Study
Drilled Cast-in-Place Concrete Piles

JOB NO. E71-272 DATE 3-23-71

LOG OF TEST BORING NO. 3C

RIG TYPE CME-55
BORING TYPE 6½" Hollow Stem Auger
SURFACE ELEV.
DATUM

Depth in Feet	Continuous Penetration Resistance	Sample	Sample Type	Blows per foot 140 lb. 30" free-fall drop hammer	Dry Density Lbs. per cu. ft.	Moisture Content Per Cent of Dry Wt.	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
0	7	☒	S	1-1-1-1-1-1-1-1-1			CL		SILTY CLAY, medium plasticity, light brown
	12			6		7			
	16	☒	S	2-2-2-3-2-2-3-3-3					SANDY CLAY, moderately lime cemented, medium plasticity, very light brown
	21			16		8	CL		
5	34	☒	S	5-7-10-13-15-15-16-14-12					SANDY CLAY, some gravel, strongly lime cemented, high plasticity, very light brown
	37			85		6			
	57						CH		
*	150/7"	☒	S	14-14-15-18-18-17-15-17-20					
				105		7			CLAYEY SAND, some gravel, moderately to strongly lime cemented, medium to high plasticity, very light brown
10		☒	S	4-6-7-12-15-14-24-23-21					
	48			109		11			
	78								
	33	☒	S	3-3-3-3-3-4-4-4-3					
	22			21		21			
15	18	☒	S	5-6-5-6-6-7-7-7-10			SC		
	39			43		20			
	34								
	33	☒	S	3-2-2-3-3-3-3-4-4					
	85			20		20			
20	56	☒	S	5-6-8-8-3-3-3-2-4					
	36			23		14			
	19								
	11	☒	S	3-2-1-1-2-1-2-2-2					
	14			10		18			
25	15	☒	S	2-2-4-3-3-5-5-6-6			CH		SANDY CLAY, moderately lime cemented, high plasticity, light brown
				28		22			

*Penetrometer refused at 7'7"
Auger advanced to 10'

30

Penetrometer stopped at 25'

Stopped auger at 24'6"
Stopped sampler at 26'

GROUND WATER

DEPTH	HOUR	DATE
	NONE	

SAMPLE TYPE
A – Auger cuttings. B – Block sample
S – 2" O.D. 1.38" I.D. tube sample.
U – 3" O.D. 2.42" I.D. tube sample.
T – 3" O.D. thin-walled Shelby tube.

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PROJECT Site C - Research Study Drilled Cast-in-Place Concrete Piles

JOB NO. E71-272 DATE 11-2-71

LOG OF TEST BORING NO. 3C

RIG TYPE CME-55

BORING TYPE 6⅓" Hollow Stem Auger

SURFACE ELEV.

DATUM

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows per foot 140 lb. 30" free-fall drop hammer	Dry Density Lbs. per cu. ft.	Moisture Content Per Cent of Dry Wt.	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
0			☒	U	1-2-2-2-3-3 / 13	86	10	CL		
			☒	U	7-7-6-8-10-8 / 46	89	9	CL		
5			☒	U	6-8-8-12-13-15 / 62	93	10			
			☒	U	7-11-13-14-16-15 / 76	95	10	CH		
				U	~~100/1" (no recovery)~~					
10			☒	U	7-8-19-23-21-20 / 98	101	11			
			☒	U	3-4-13-49-40/0"					
15			☒	U	4-5-6-7-10-11 / 43	104	18			
			☒	U	8-8-7-8-7-8 / 46	103	16	SC		
			☒	U	4-4-4-5-8-7 / 32	90	26			
20			☒	U	4-6-6-5-4-4 / 29	94	19			
			☒	U	1-2-3-6-6-10 / 28	83	37			
25			☒	U	2-3-4-5-6-7 / 27	109	21			
			☒	U	4-14-20-32-41-47	104	23	CH		
30			☒	U	1-3-8-8-7-7 / 34	110	8	SC		CLAYEY SAND, considerable gravel, moderately lime cemented, medium plasticity, light brown
										Stopped auger at 28'6" Stopped sampler at 29'6"

GROUND WATER

DEPTH	HOUR	DATE
	NONE	

SAMPLE TYPE

A – Auger cuttings. B – Block sample
S – 2" O.D. 1.38" I.D. tube sample.
U – 3" O.D. 2.42" I.D. tube sample.
T – 3" O.D. thin-walled Shelby tube.

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PROJECT Site C-Research Study Drilled Cast-in-Place Concrete Piles

JOB NO. E71-272 DATE 10-15-71

LOG OF TEST BORING NO. 4C

RIG TYPE CME-55
BORING TYPE 6½" Hollow Stem Auger
SURFACE ELEV.
DATUM

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows per foot 140 lb. 30" free-fall drop hammer	Dry Density Lbs. per cu. ft.	Moisture Content Per Cent of Dry Wt.	Unified Soil Classification
0								
	9			S	1-2-1-2-3-2-2-3-2			
	16				14		9	CL
	20			S	2-3-3-3-3-3-4-4-5			
	32				22		6	
5	65			S	2-3-5-4-4-5-6-7-9			CL
*	100/5"				35		7	
				S	11-22-91-100/1 1/4"			
							6	CH
				S	8-19-84-5-6-31-16-9-9-10			
10	36/6"				131		7	
	51			S	3-4-3-4-4-3-5-4-5			
	56				25		14	
	25			S	4-5-5-5-5-6-5-5-6			
	35				32		14	
15	30			S	7-8-9-7-5-5-6-5-6			
	40				34		11	SC
	39			S	7-8-9-9-6-23-83-100/1 3/4"			
	117						20	
	97			S	9-8-10-11-9-6-10-12-9			
20	108				57		13	
	57			S	3-4-4-7-7-9-10-10-12			
	44				55		24	
	38			S				
	40						14	
25	48			S	5-4-4-4-6-4-4-6-5			
					29			
				S	3-2-4-4-5-5-6-9-13			CH
					42		14	
				S	19-12-20-13-15-14-15-11-13			
30					81		4	SC

*Penetrometer refused at 5'5"
Auger advanced to 9'6"

35

Penetrometer stopped at 25'

REMARKS	VISUAL CLASSIFICATION
	SILTY CLAY, weakly to moderately lime cemented, medium plasticity, light brown
	SANDY CLAY, moderately lime cemented, medium plasticity, very light brown
	SANDY CLAY, some gravel, strongly lime cemented, high plasticity, very light brown
	CLAYEY SAND, some gravel, moderately to strongly lime cemented, medium to high plasticity, very light brown
	SANDY CLAY, moderately lime cemented, high plasticity, light brown
	CLAYEY SAND, some gravel, strongly lime cemented, medium to high plasticity, very light brown
	Stopped auger at 28'6" Stopped sampler at 30'

GROUND WATER

DEPTH	HOUR	DATE
	NONE	

SAMPLE TYPE
A – Auger cuttings. B – Block sample
S – 2" O.D. 1.38" I.D. tube sample.
U – 3" O.D. 2.42" I.D. tube sample.
T – 3" O.D. thin-walled Shelby tube.

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Site C - Research Study

PROJECT Drilled Cast-in-Place Concrete Piles

JOB NO. E71-272 DATE 10-23-71

LOG OF TEST BORING NO. 4C

RIG TYPE CME-55

BORING TYPE 6½" Hollow Stem Auger

SURFACE ELEV.

DATUM

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows per foot 140 lb. 30" free-fall drop hammer	Dry Density Lbs. per cu. ft.	Moisture Content Per Cent of Dry Wt.	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
0			☒	U	2-2-2-3-3-4 16	87	9	CL		
			☒	U	4-5-5-6-6-6 32	79	7			
5			☒	U	5-7-7-8-9-10 46	92	7	CL		
			☒	U	14-16-64-100/1/4" (no recovery)			CH		
10			☒	U	6-5-4-7-6-6 34	91	15			
			☒	U	2-3-3-5-5-6 24	89	20			
15			☒	U	3-4-7-9-16-23 62	100	21			
			☒	U	9-10-8-9-5-15 56	99	20	SC		
				U	118/1" (no recovery)					
20										
			☒	U	6-6-6-7-8-10 43	87	27			
25			☒	U	6-7-7-8-12-12 52	110	13	CH		
			☒	U	3-4-4-6-6-11 34	104	11			
30			☒	U	11-25-28-29-24-25 142	116	5	SC		
										Stopped auger at 29'6" Stopped sampler at 30'6"
35										

GROUND WATER

DEPTH	HOUR	DATE
	NONE	

SAMPLE TYPE

A – Auger cuttings. B – Block sample
S – 2" O.D. 1.38" I.D. tube sample.
U – 3" O.D. 2.42" I.D. tube sample.
T – 3" O.D. thin-walled Shelby tube.

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TABULATION OF TEST RESULTS

Job No. E71-272 **Date**

Client:

Project Site C - Research Study

Drilled Cast-in-Place Concrete Piles

Material

Source

HOLE NO.	LOCATION	DEPTH	UNIFIED CLASS.	LL	PI	SIEVE ANALYSIS – ACCUM. % PASSING												LAB. NO.
						200	100	40	16	10	4	¼	⅜	¾	1	1½	2	
1C	See Site Plan	6"-2'	CL	39	19	80	89	95	96	96	98	98	98	100				70-130-1
1C	See Site Plan	4½'-6'	CL	34	16	65	70	75	81	86	95	98	99	100				70-130-3
1C	See Site Plan	8½'-10'	CH	75	48	53	56	60	65	70	79	82	94	100				70-130-5
1C	See Site Plan	14½'-16'	SC	38	20	42	46	53	60	64	74	77	82	94	100			70-130-8
1C	See Site Plan	20'-21½'	SC	49	30	36	41	48	53	56	63	66	74	82	100			70-130-10
1C	See Site Plan	24'-25½'	CH	60	39	53	75	96	99	99	100							70-130-12
2C	See Site Plan	2½'-3'	CL	31	14	75	86	94	96	97	100							70-130-14
2C	See Site Plan	12½'-14'	SC	74	49	46	49	54	59	63	72	77	83	100				70-130-18
2C	See Site Plan	17½'-19'	SC	69	45	48	50	56	65	71	83	89	97	100				70-130-20
2C	See Site Plan	22½'-24'	CH	70	51	72	84	95	99		100							70-130-22

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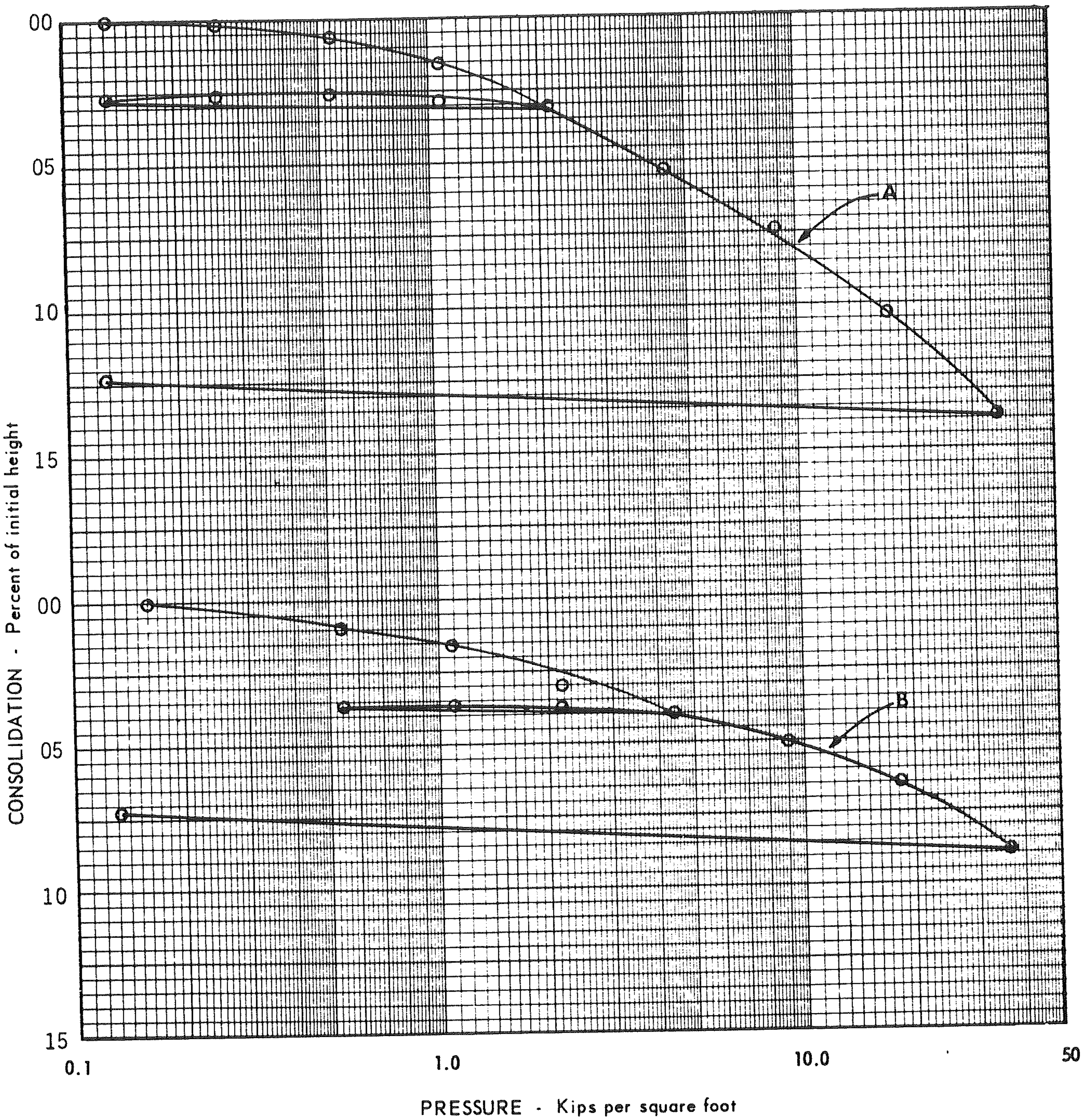
SUMMARY OF CONSOLIDATION TESTS

PROJECT Site C - Research Study Drilled Cast-in-Place Concrete Piles JOB NO. E71-272



CURVE	SAMPLE	INITIAL DRY DENSITY LBS./CU. FT.	MOISTURE CONTENT % DRY WEIGHT		UNIFIED SOIL CLASSIFICATION
			INITIAL	FINAL	
A	1C @ 2½' - 3½'	78.3	5.5	-	CL
B	1C @ 6½'-7½'	104.4	5.6	-	CL

SOIL MOISTURE CONDITION

——— INSITU
– – – – SUBMERGED

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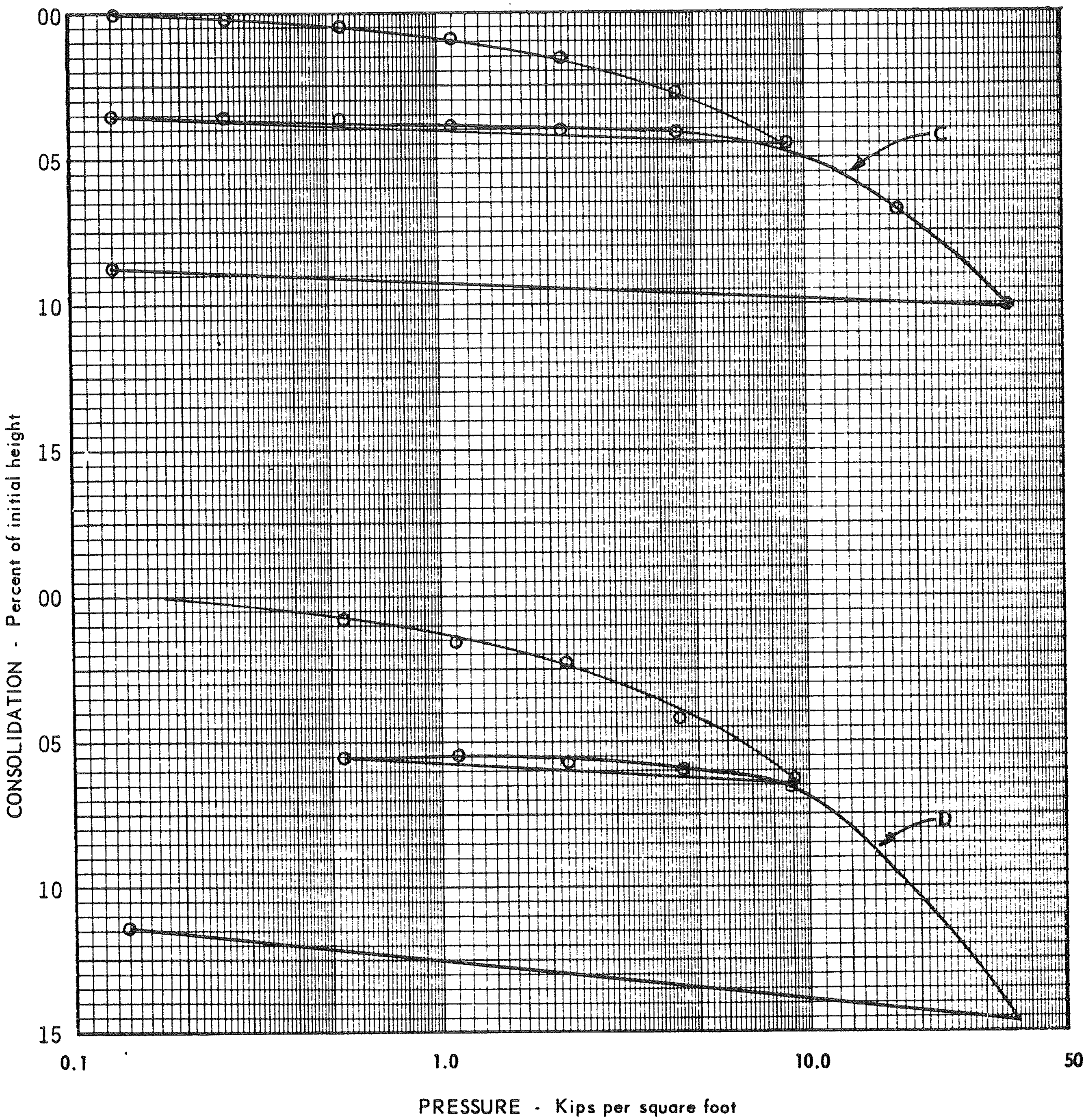
SUMMARY OF CONSOLIDATION TESTS

PROJECT Site C - Research Study Drilled Cast-in-Place Concrete Piles **JOB NO.** E71-272



CURVE	SAMPLE	INITIAL DRY DENSITY LBS./CU. FT.	MOISTURE CONTENT % DRY WEIGHT		UNIFIED SOIL CLASSIFICATION
			INITIAL	FINAL	
C	1C @ 8½'-9½'	93.9	13.4	-	CH
D	1C @ 10½'-11½'	86.6	22.2	-	CH

SOIL MOISTURE CONDITION

———	INSITU
– – – –	SUBMERGED

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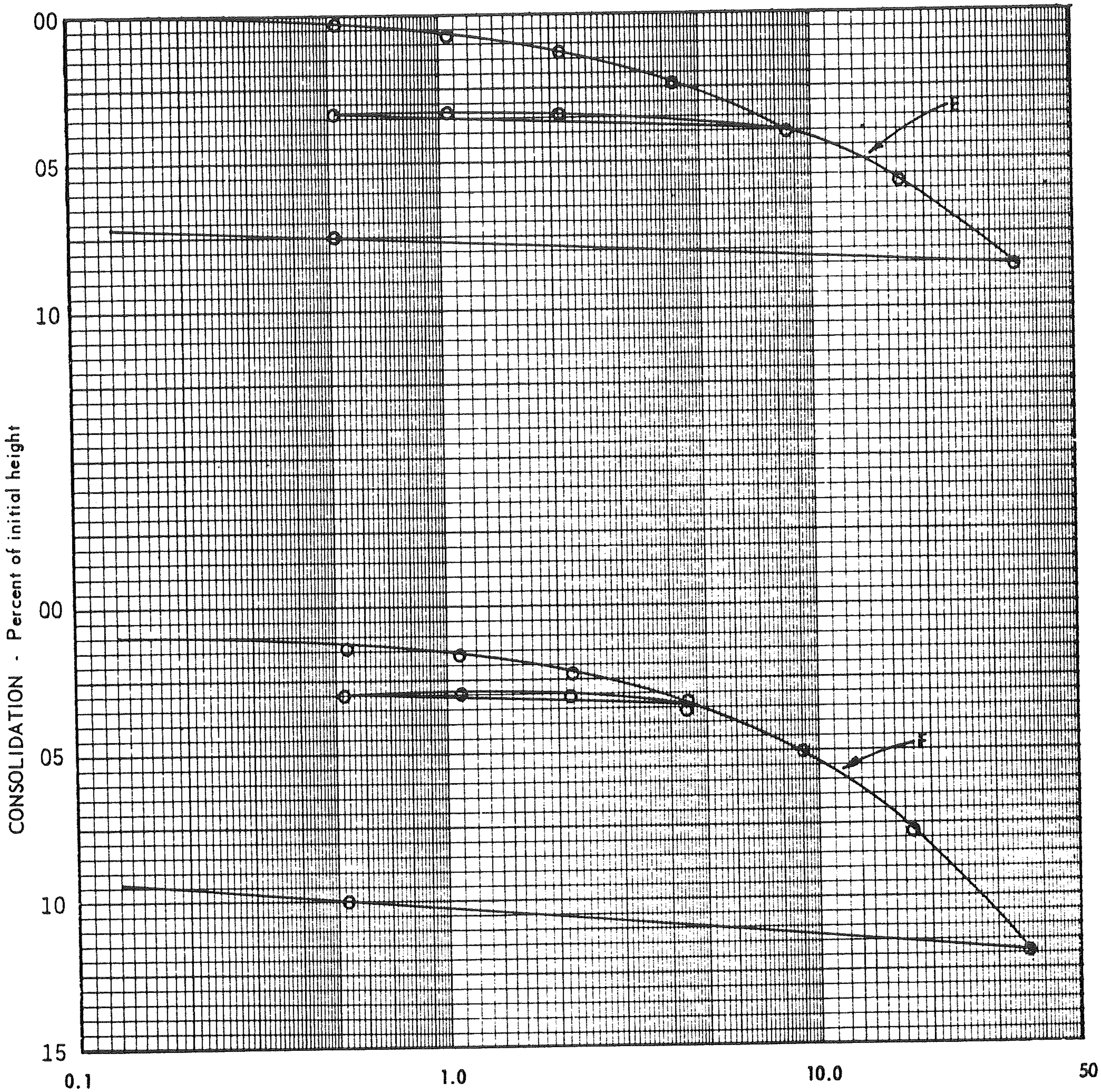
SUMMARY OF CONSOLIDATION TESTS

PROJECT Site C - Research Study Drilled Cast-in-Place Concrete Piles **JOB NO.** E71-272



CURVE	SAMPLE	INITIAL DRY DENSITY LBS./CU. FT.	MOISTURE CONTENT % DRY WEIGHT		UNIFIED SOIL CLASSIFICATION
			INITIAL	FINAL	
E	1C @ 14½'-15½'	107.9	7.1	-	SC
F	1C @ 18½'-19½'	96.5	18.9	-	SC

SOIL MOISTURE CONDITION

————	INSITU
– – – –	SUBMERGED

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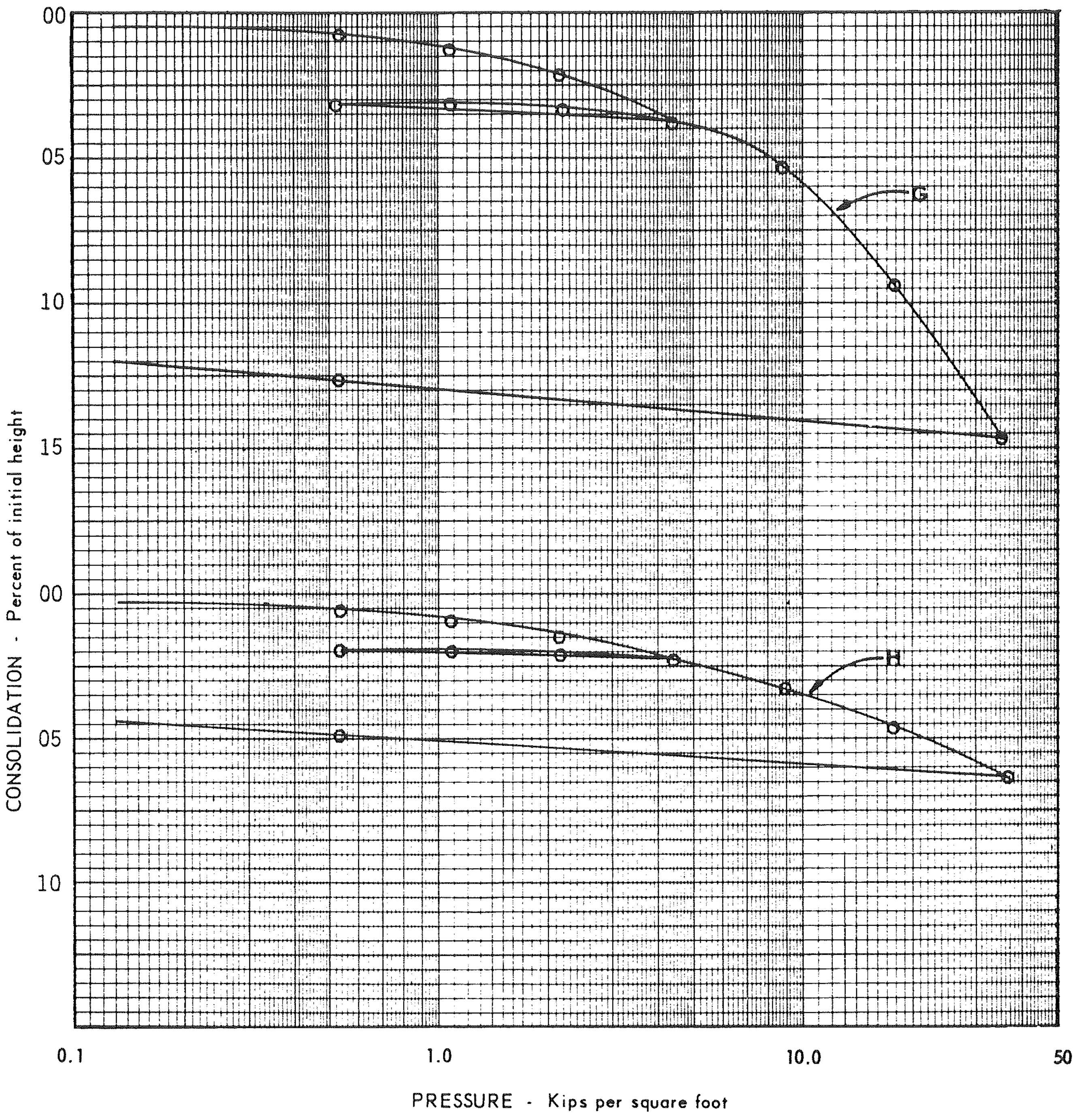
SUMMARY OF CONSOLIDATION TESTS

PROJECT Site C - Research Study
Drilled Cast-in-Place Concrete Piles **JOB NO.** E71-272



CURVE	SAMPLE	INITIAL DRY DENSITY LBS./CU. FT.	MOISTURE CONTENT % DRY WEIGHT		UNIFIED SOIL CLASSIFICATION
			INITIAL	FINAL	
G	1C @ 20½'-21½'	92.6	18.4	-	SC
H	1C @ 24½'-25½'	113.8	12.5	-	CH

SOIL MOISTURE CONDITION

———	INSITU
– – – –	SUBMERGED

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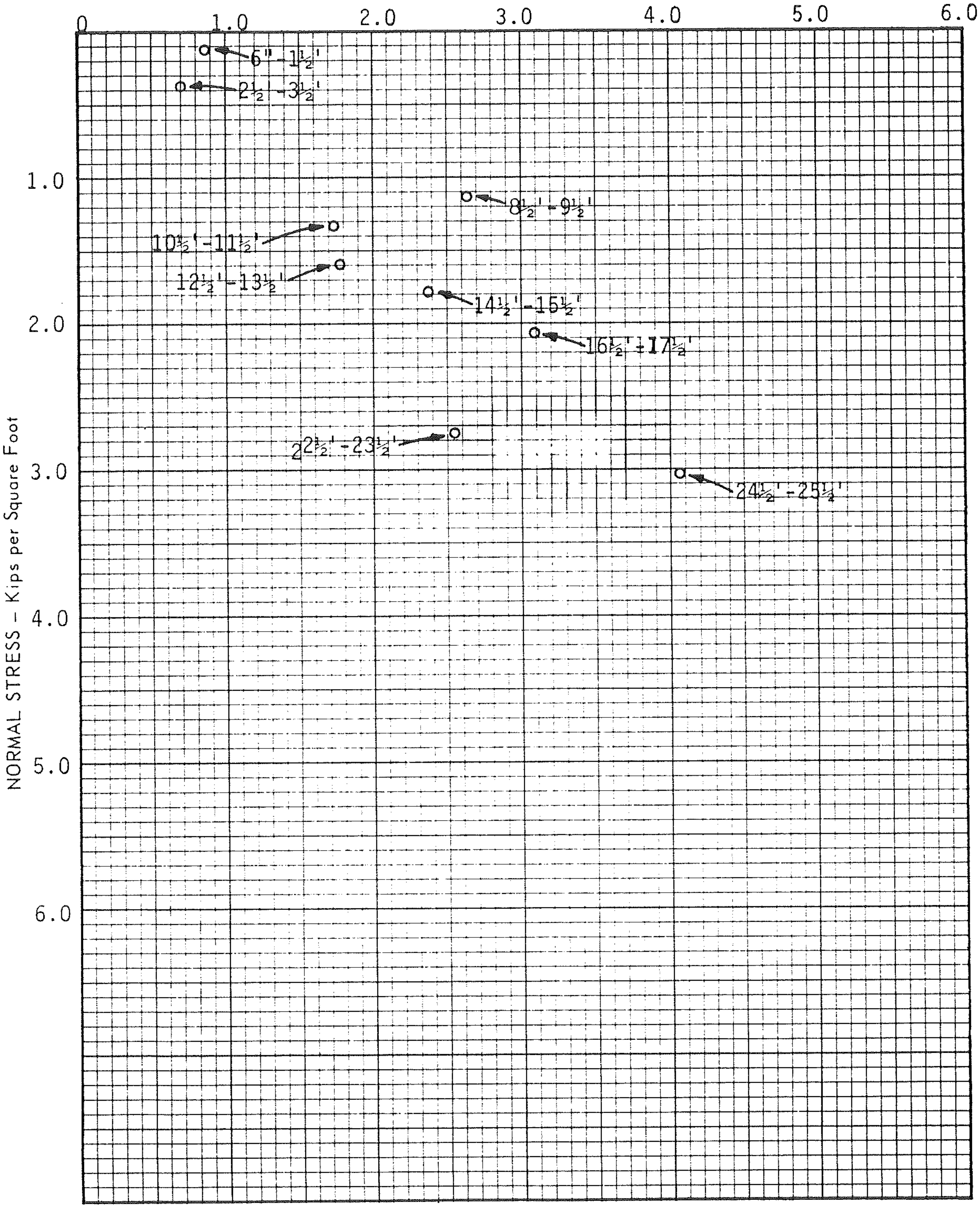
SUMMARY OF DIRECT SHEAR TESTS

Site C - Research Study

PROJECT Drilled Cast-in-Place Concrete Piles **JOB NO.** ~~E71-272~~

Test Boring No. 1C



SHEARING STRESS – Kips per Square Foot

SOIL MOISTURE CONDITION

○ – INSITU

● – SUBMERGED

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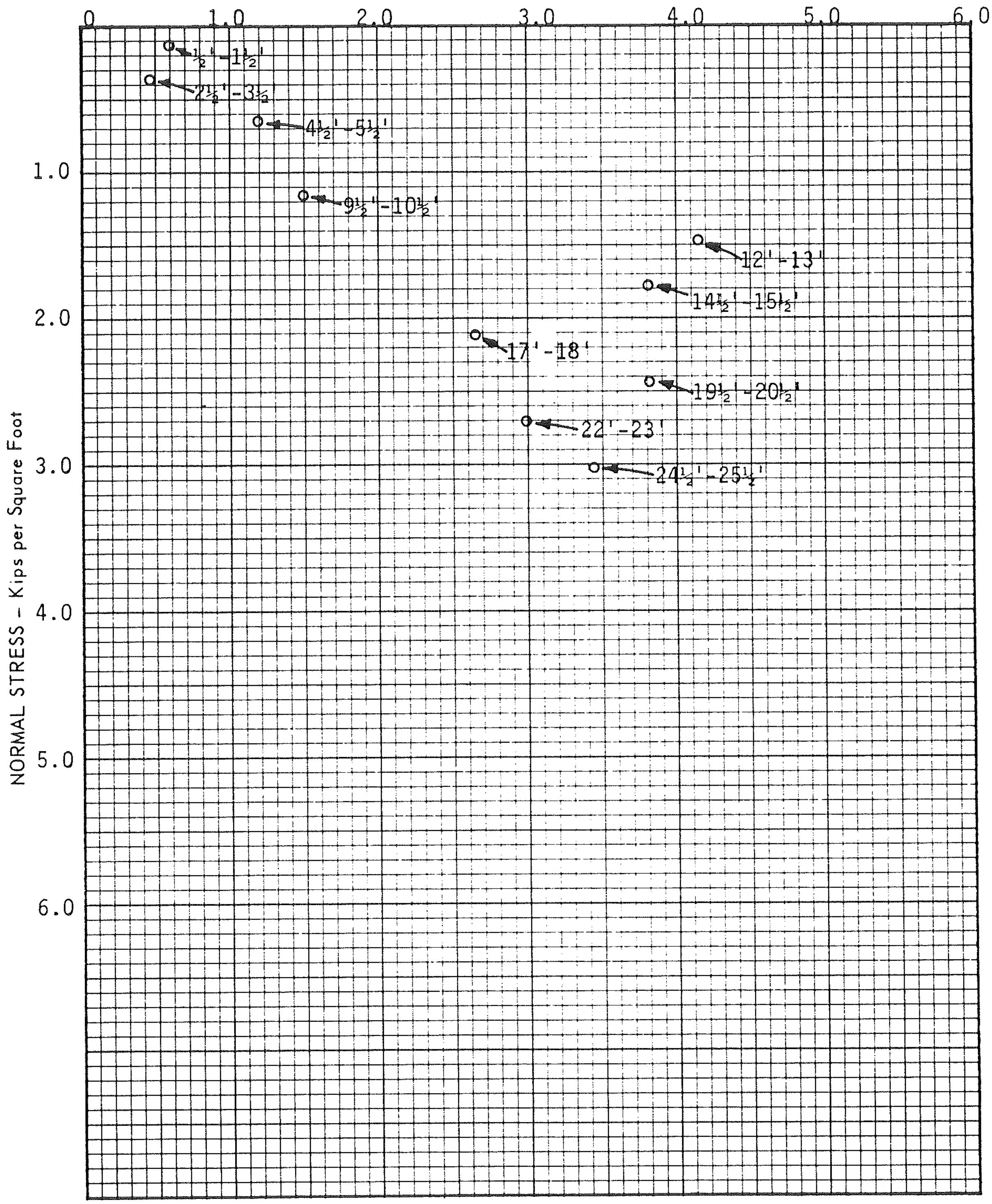
SUMMARY OF DIRECT SHEAR TESTS

PROJECT Site C - Research Study
Drilled Cast-in-Place Concrete Piles **JOB NO.** E71-272

Test Boring No. 2C



SOIL MOISTURE CONDITION

○	– INSITU
●	– SUBMERGED

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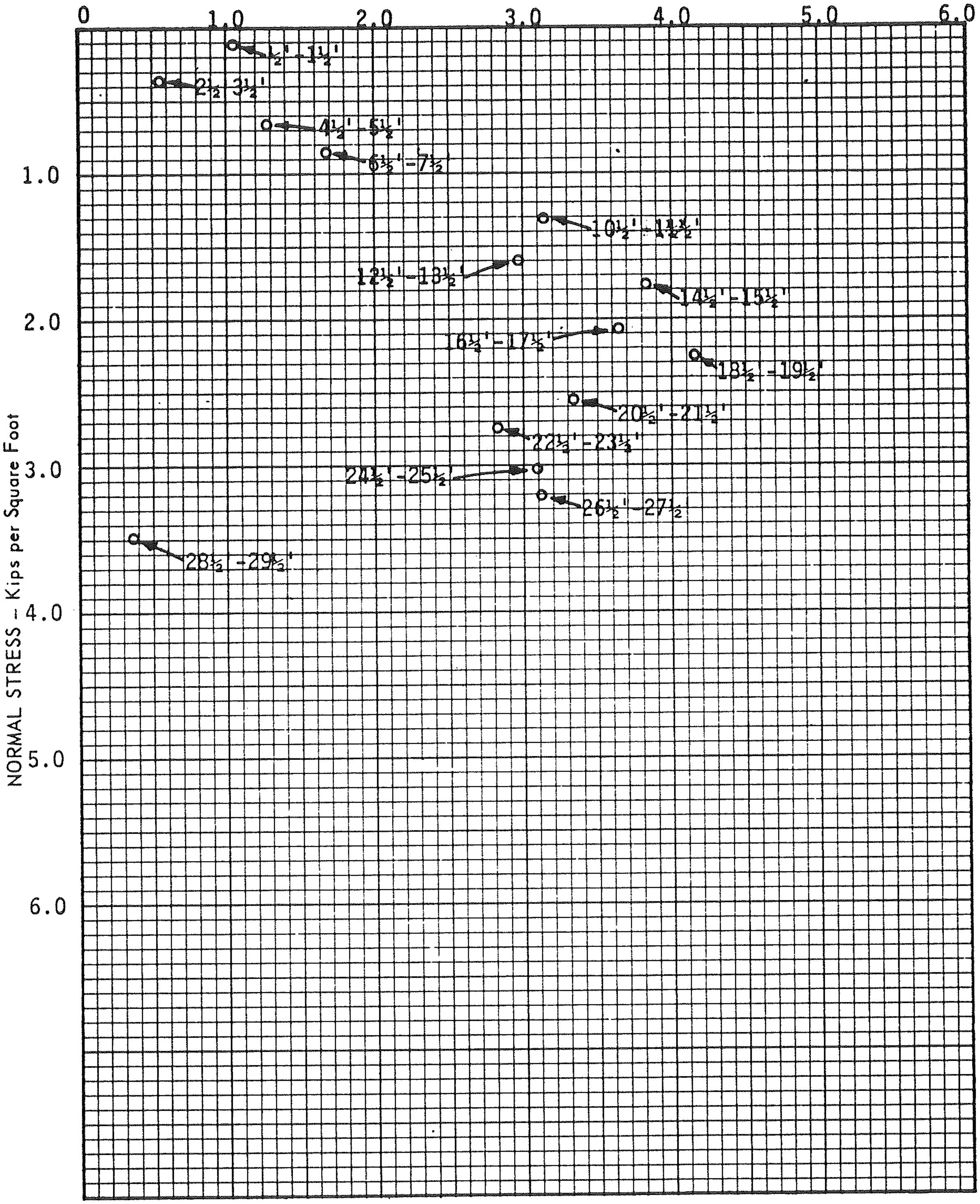
SUMMARY OF DIRECT SHEAR TESTS

PROJECT Site C - Research Study Drilled Cast-in-Place Concrete Piles JOB NO. E71-272

Test Boring No. 3C



SHEARING STRESS – Kips per Square Foot

SOIL MOISTURE CONDITION

○ – INSITU
● – SUBMERGED

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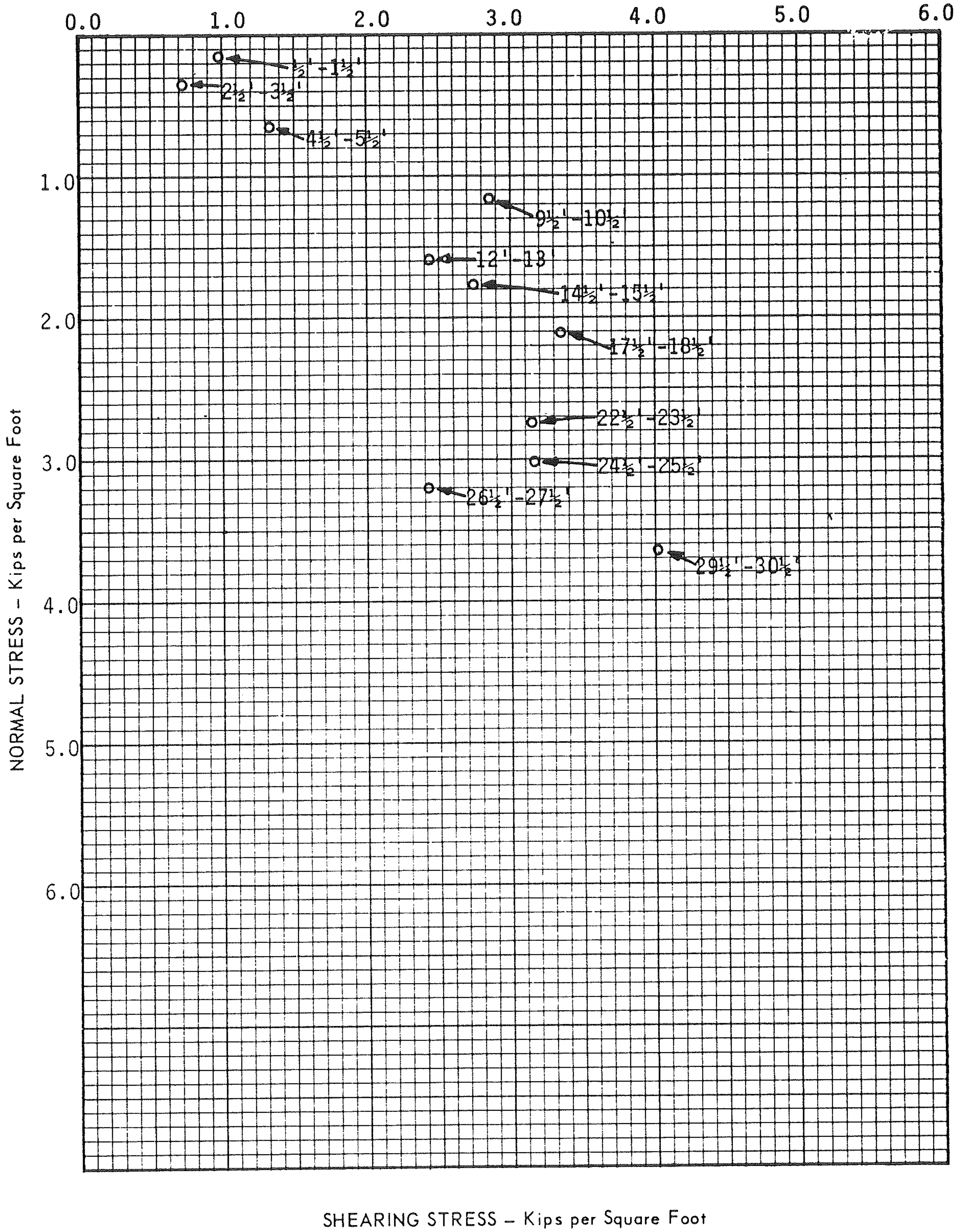
SUMMARY OF DIRECT SHEAR TESTS

PROJECT Site C - Research Study
Drilled Cast-in-Place Concrete Piles **JOB NO.** E71-272

Test Hole No. 4C



SOIL MOISTURE CONDITION

○ - INSITU
● - SUBMERGED

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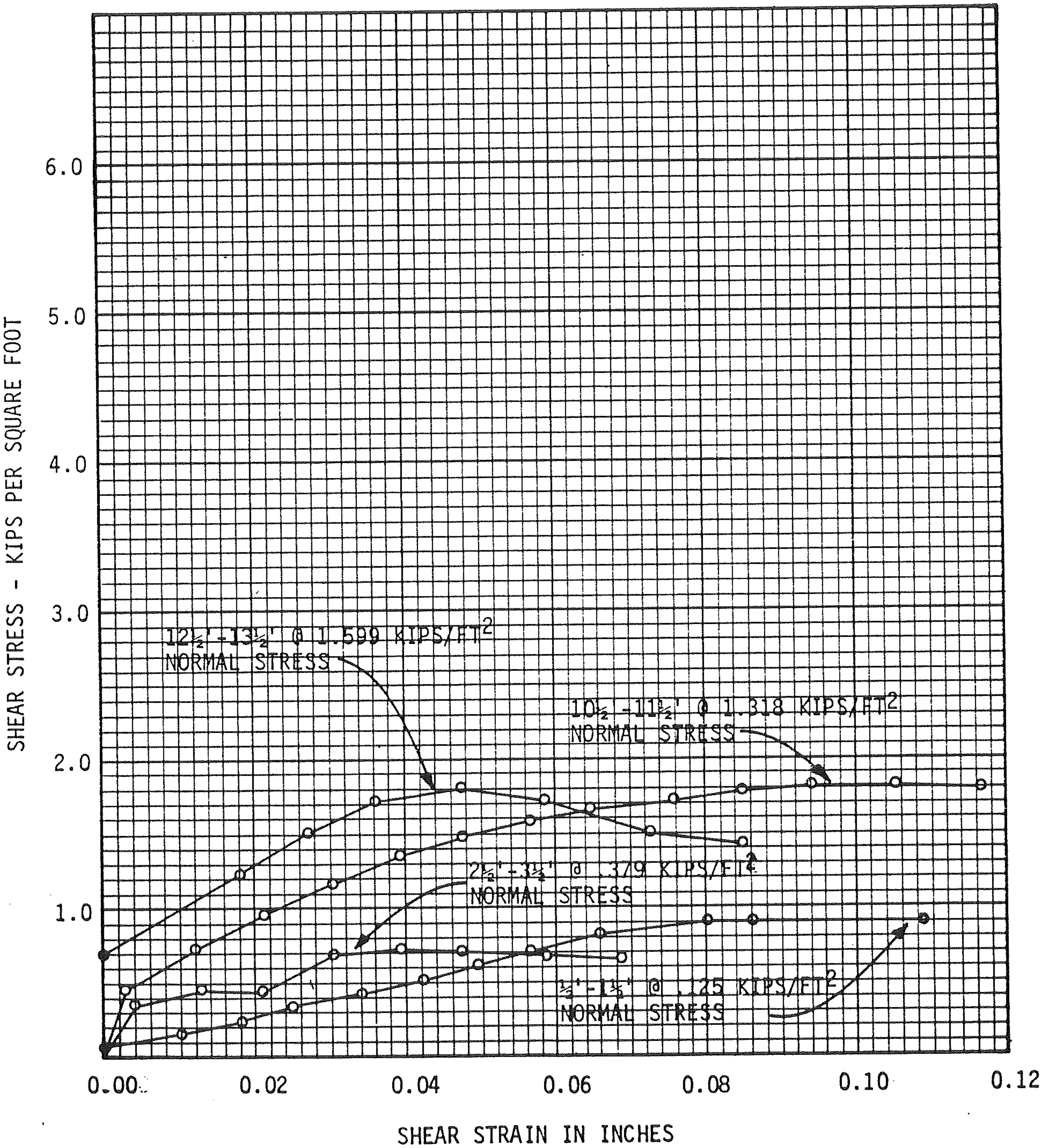
PROJECT Site C - Research Study Drilled Cast-in-Place Concrete Piles JOB NO. E71-272

DIRECT SHEAR TEST DATA

Test Boring No. 1C



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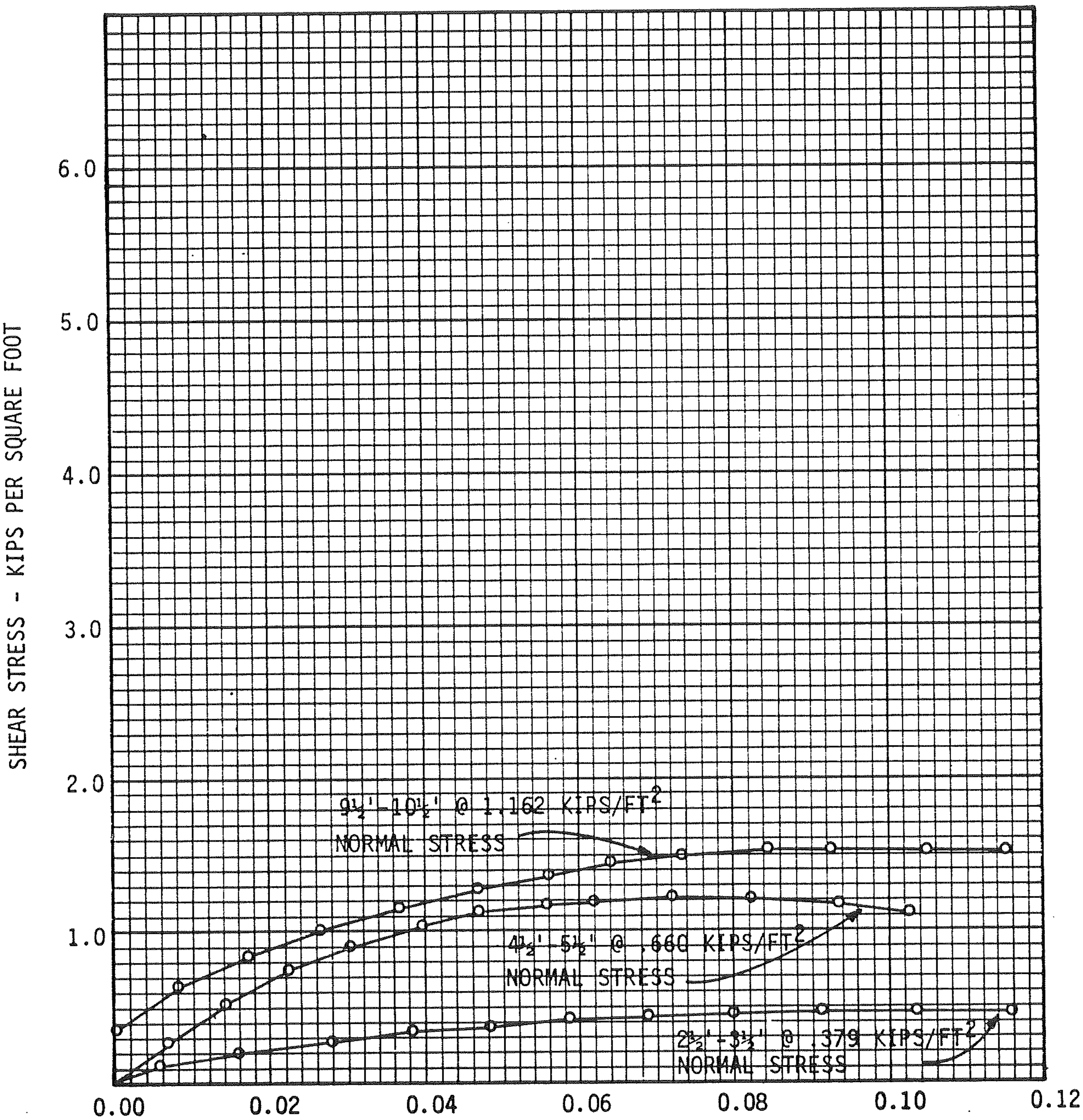
PROJECT Site C - Research Study
Drilled Cast-in-Place Concrete Piles JOB NO. E71-272

DIRECT SHEAR TEST DATA

Test Boring No. 2C



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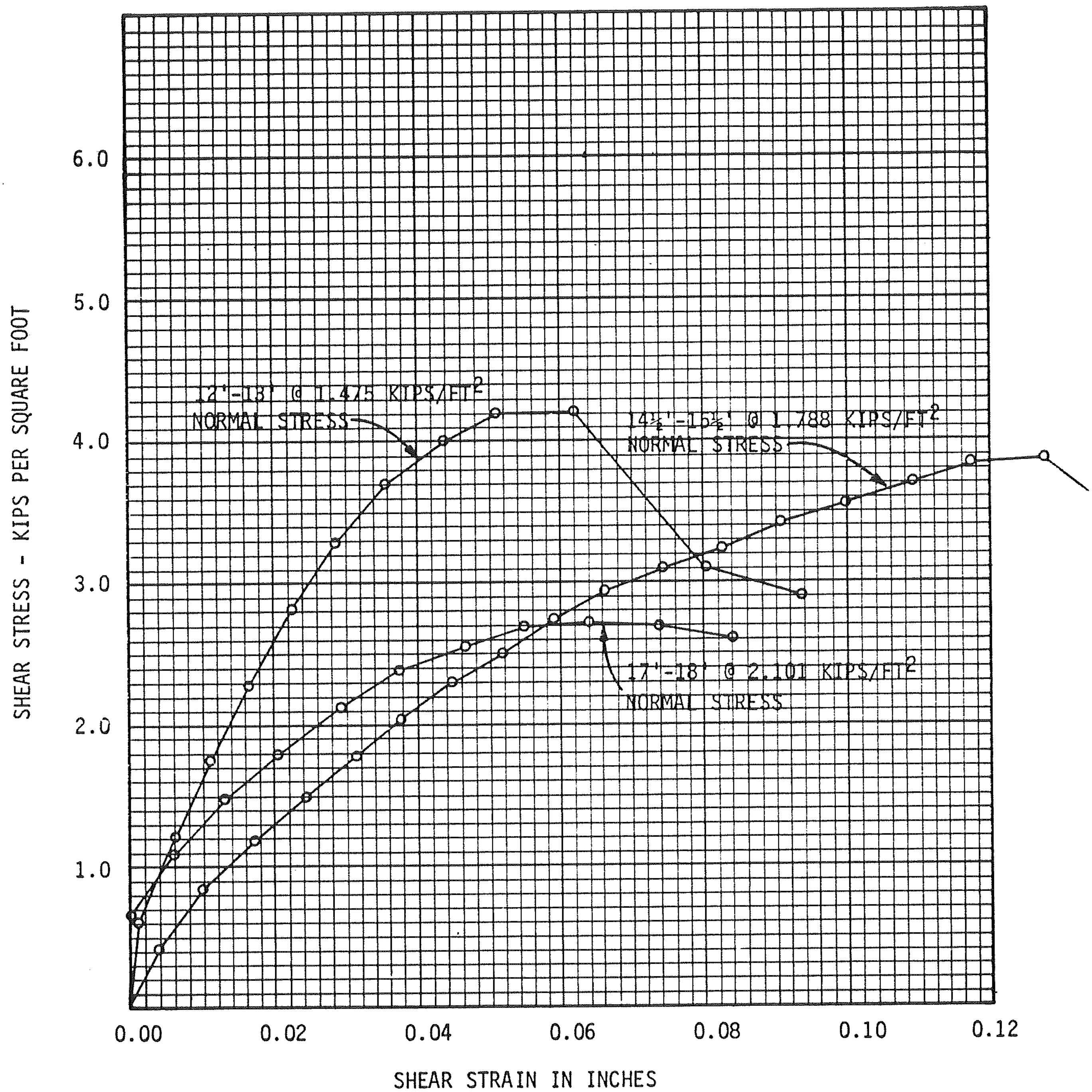
PROJECT Site C - Research Study
Drilled Cast-in-Place Concrete Piles JOB NO. E71-272

DIRECT SHEAR TEST DATA

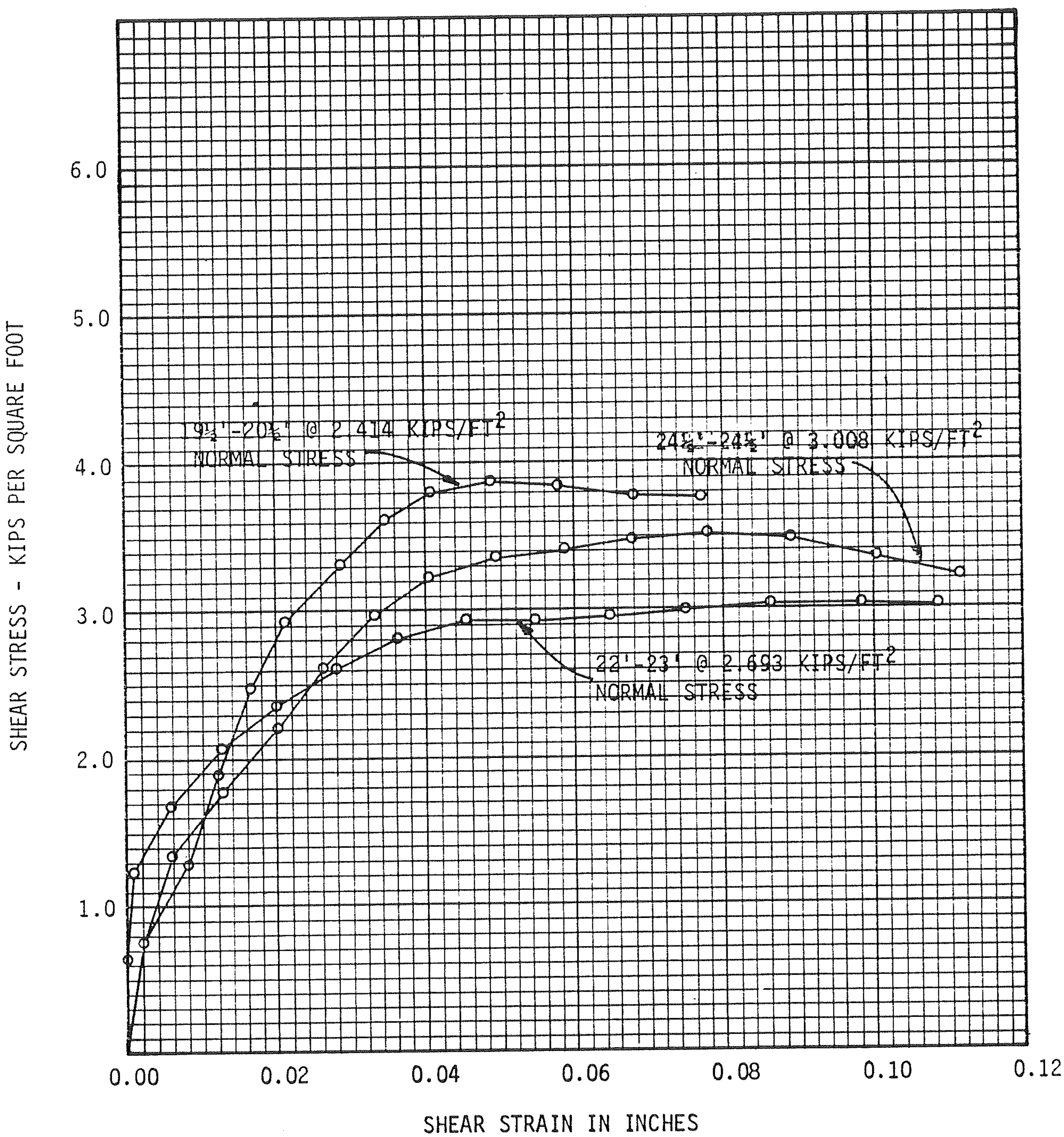
Test Boring No. 2C



PROJECT Site C - Research Study
Drilled Cast-in-Place Concrete Piles JOB NO. E71-272

DIRECT SHEAR TEST DATA

Test Boring No. 2C



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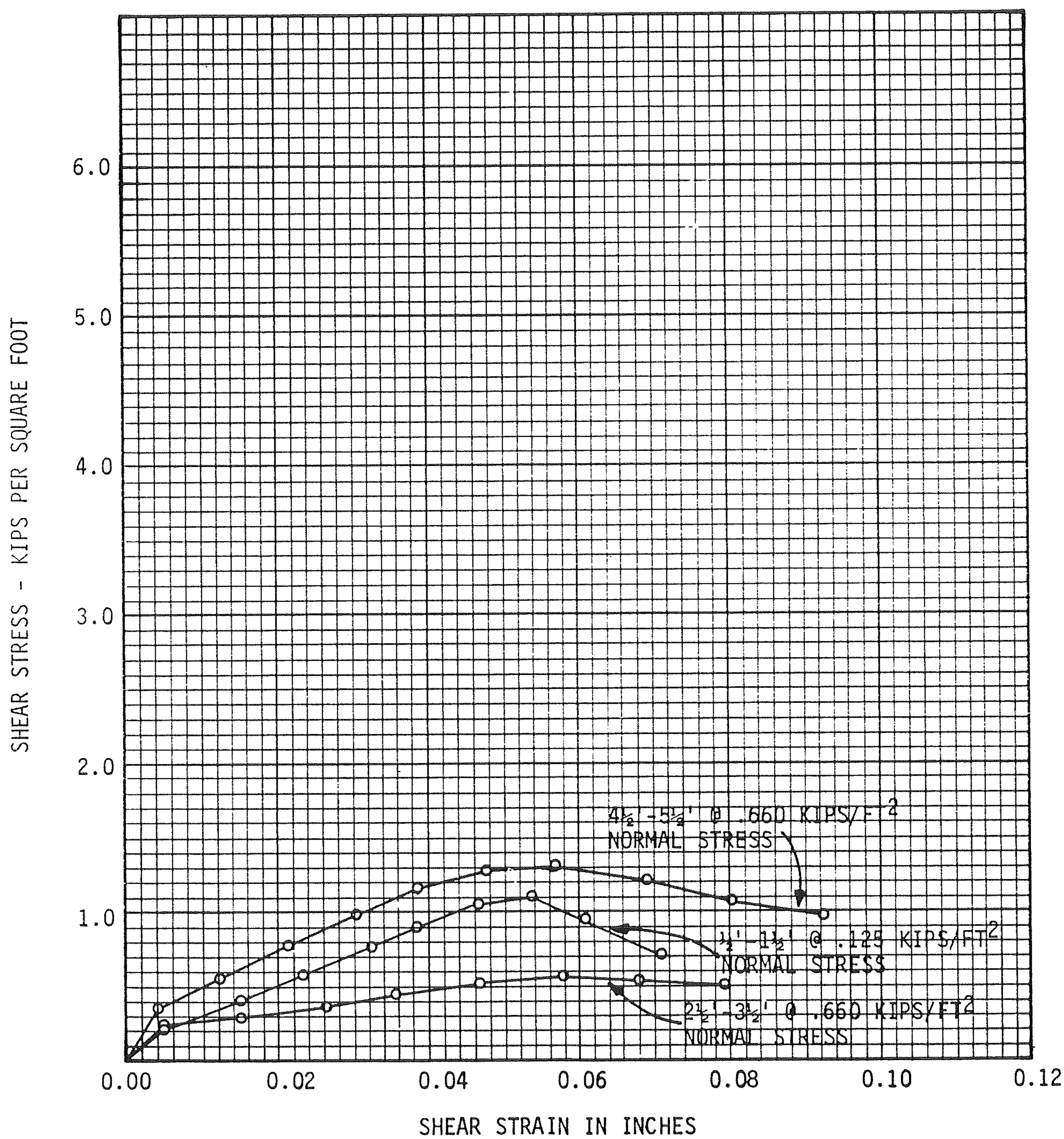
PROJECT Site C - Research Study
Drilled Cast-in-Place Concrete Piles JOB NO. E72-272

DIRECT SHEAR TEST DATA

Test Boring No. 3C

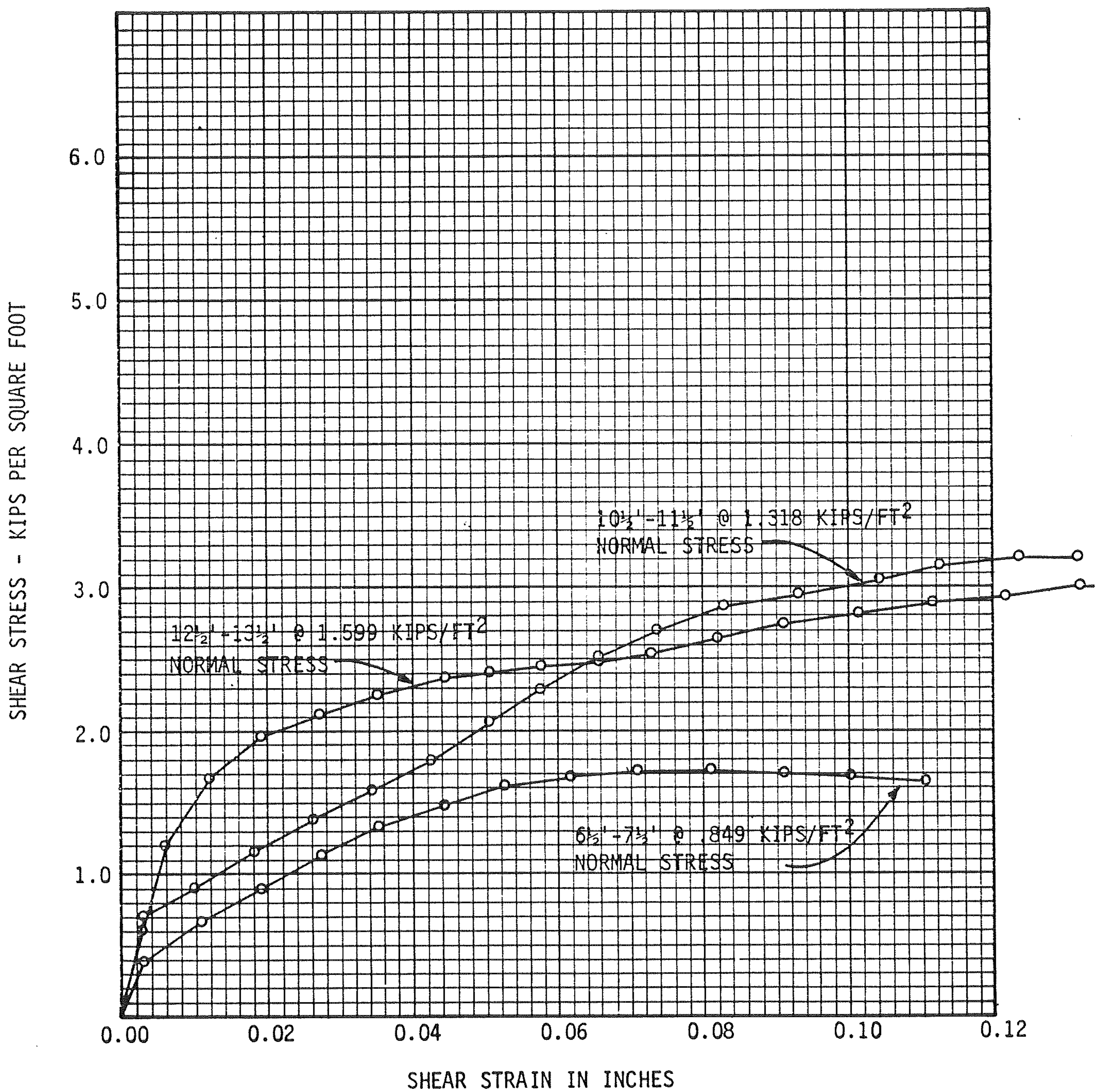


PROJECT Site C - Research Study
Drilled Cast-in-Place Concrete Piles

JOB NO. E71-272

DIRECT SHEAR TEST DATA

Test Boring No. 3C



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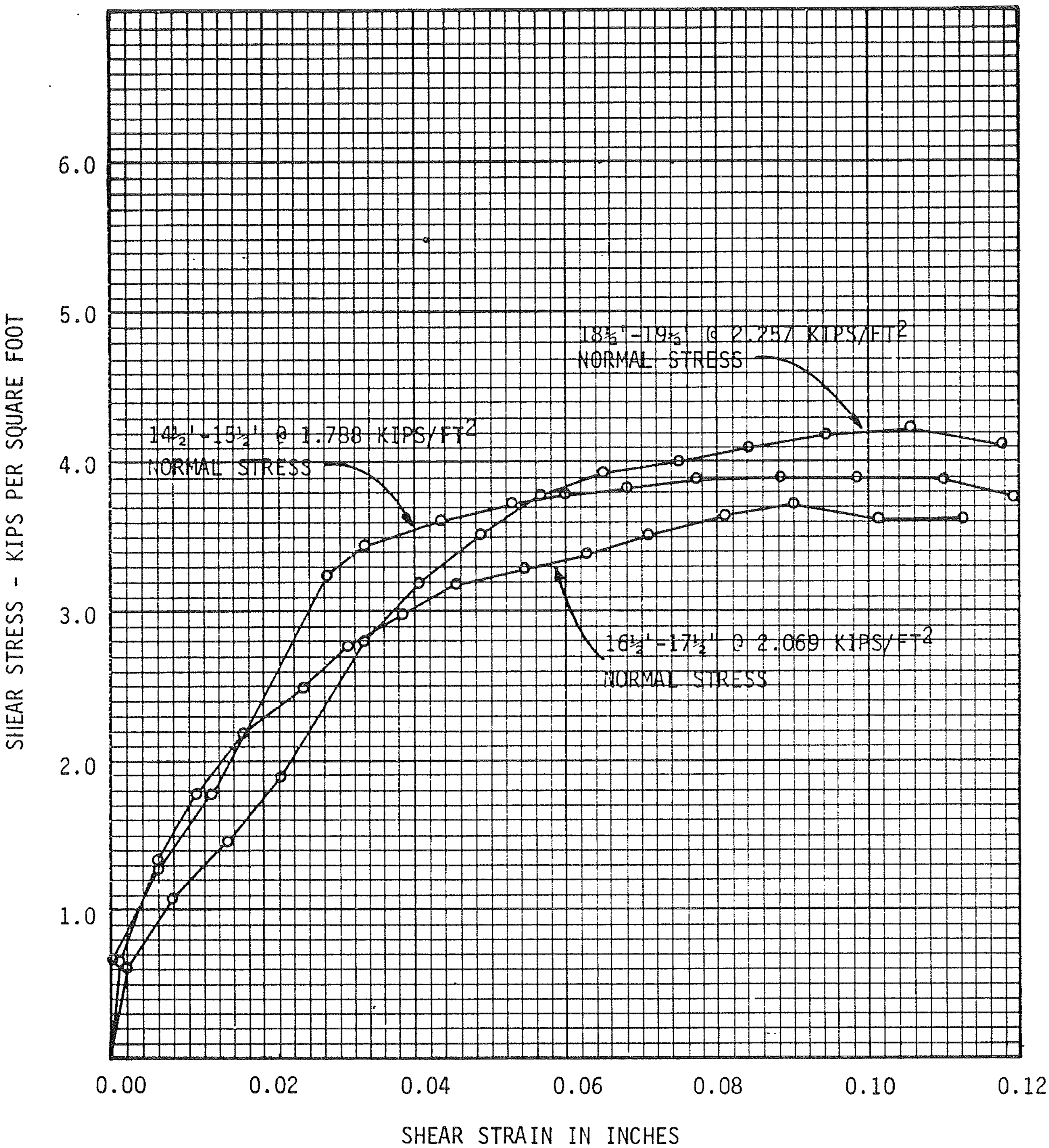

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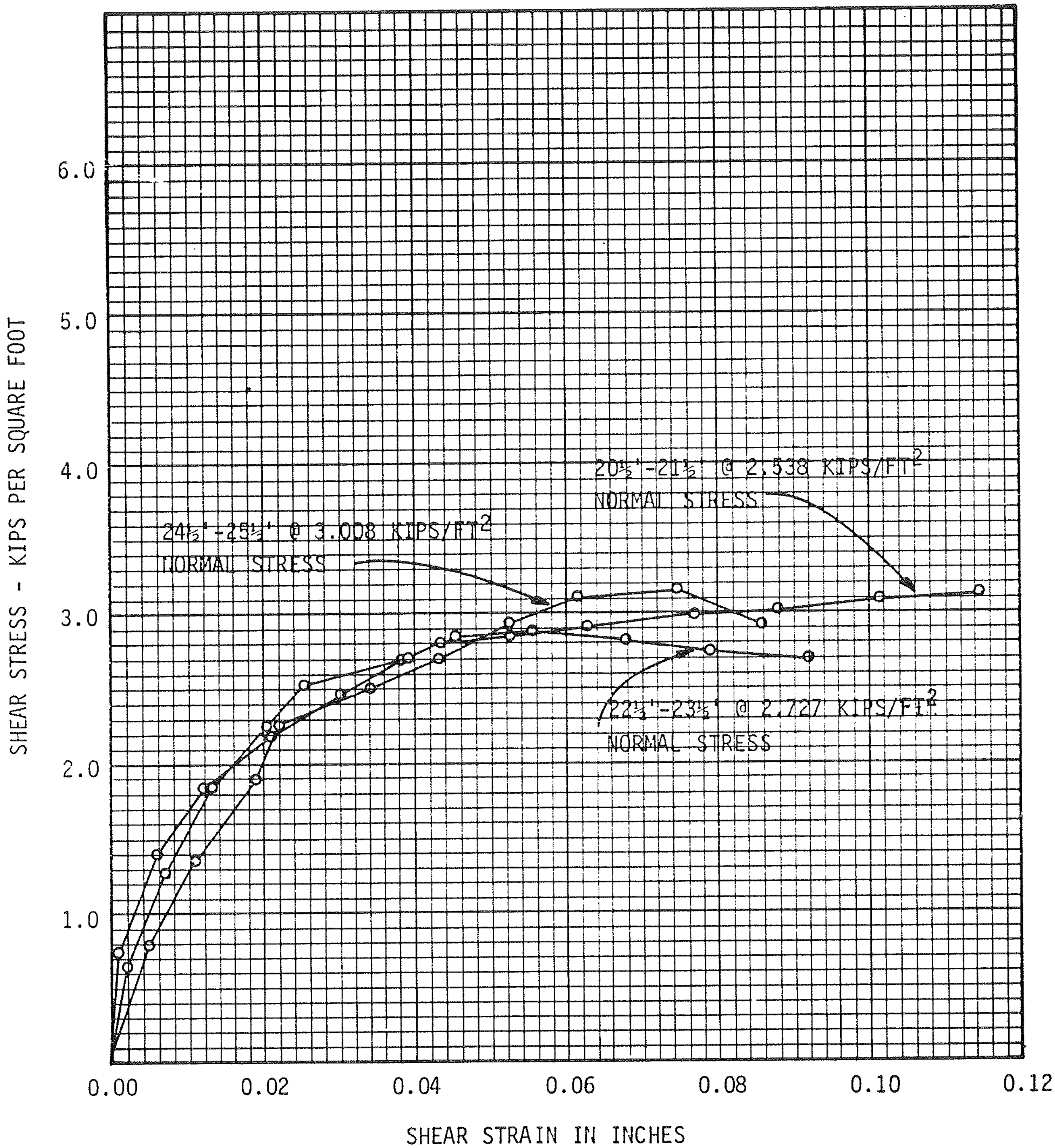
Test Boring No. 3C



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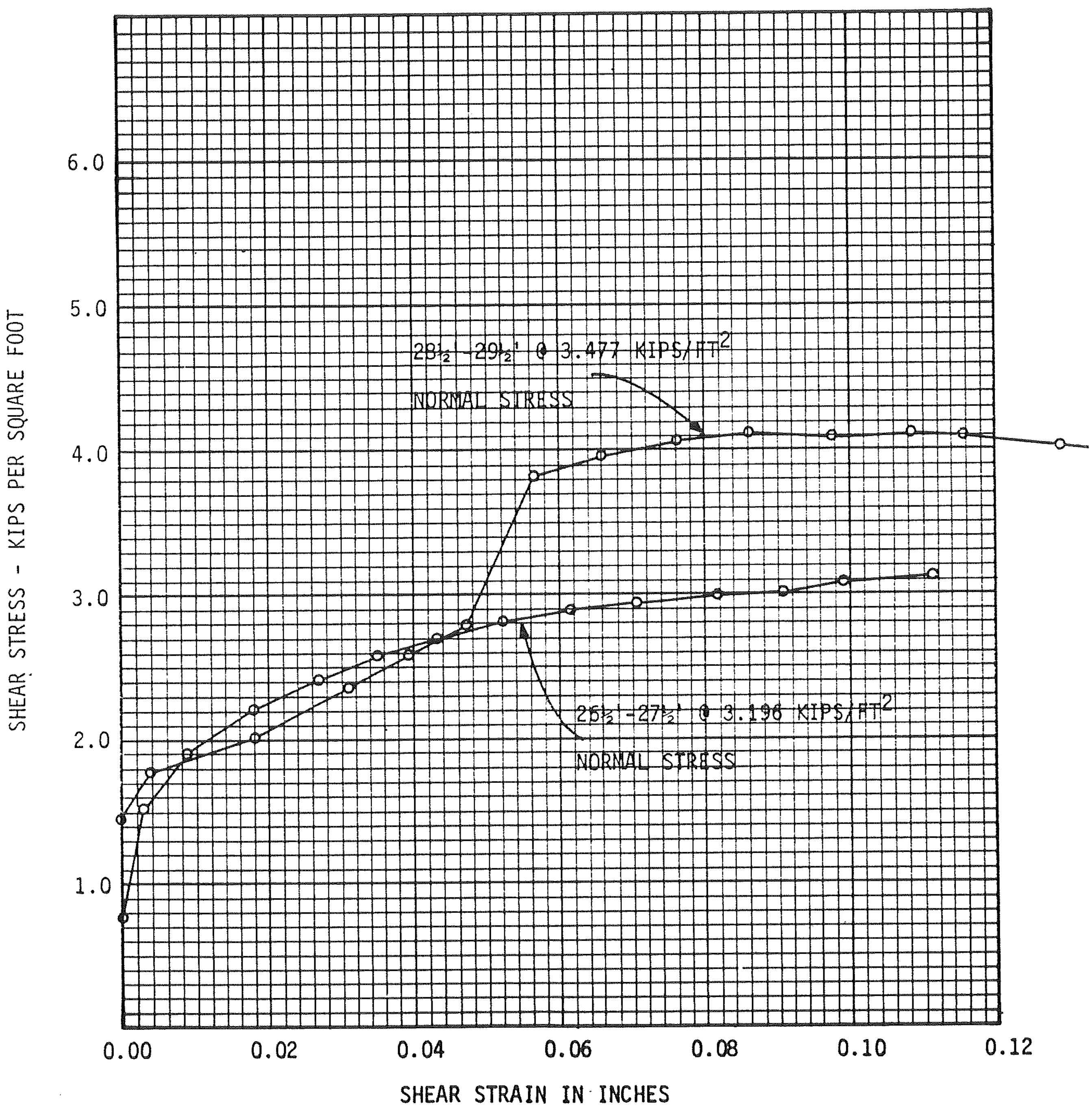
DIRECT SHEAR TEST DATA

Test Boring No. 3C



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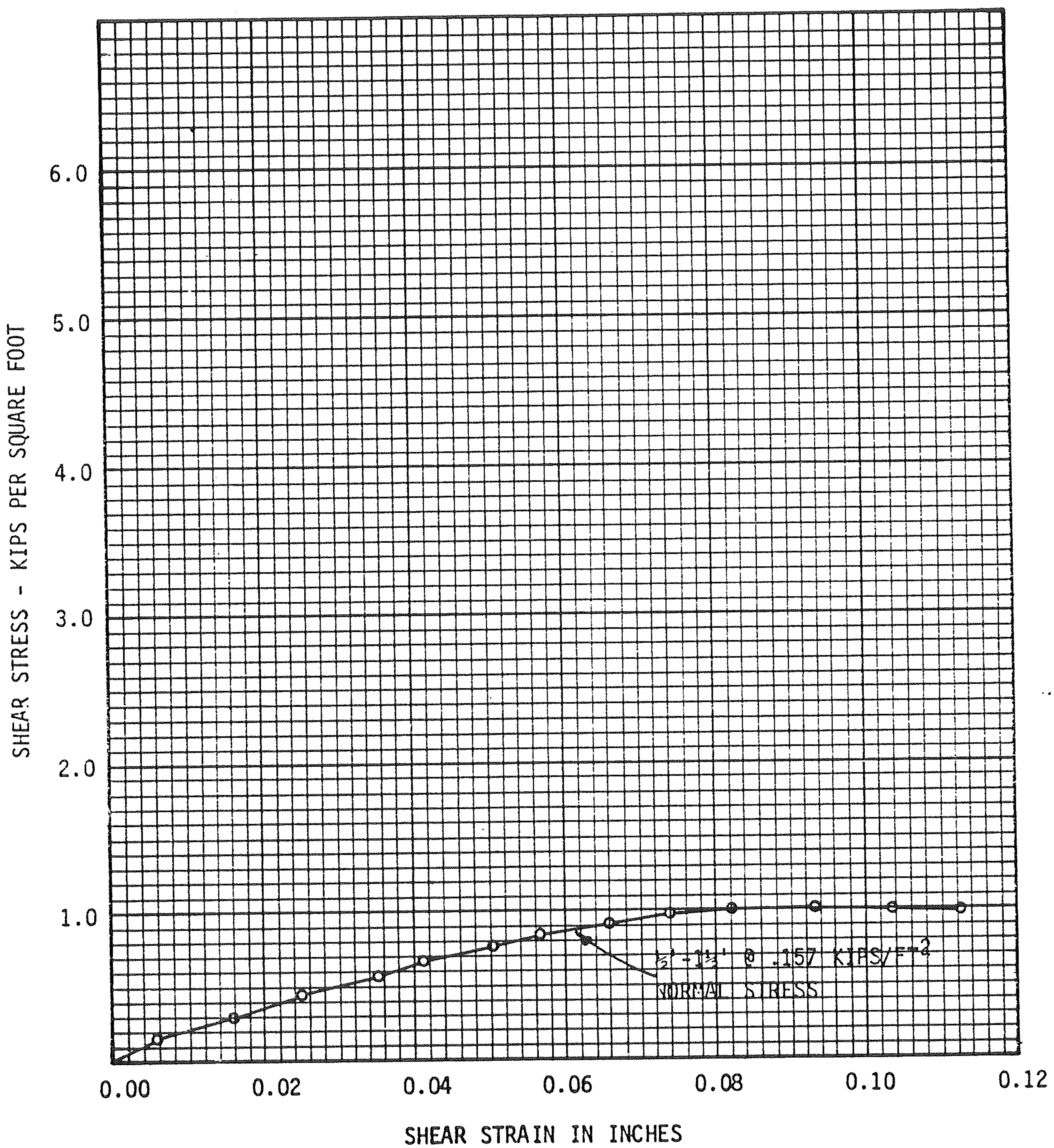
Drilled Cast-in-Place Concrete Piles JOB NO. E71-272

DIRECT SHEAR TEST DATA

Test Boring No. 4C



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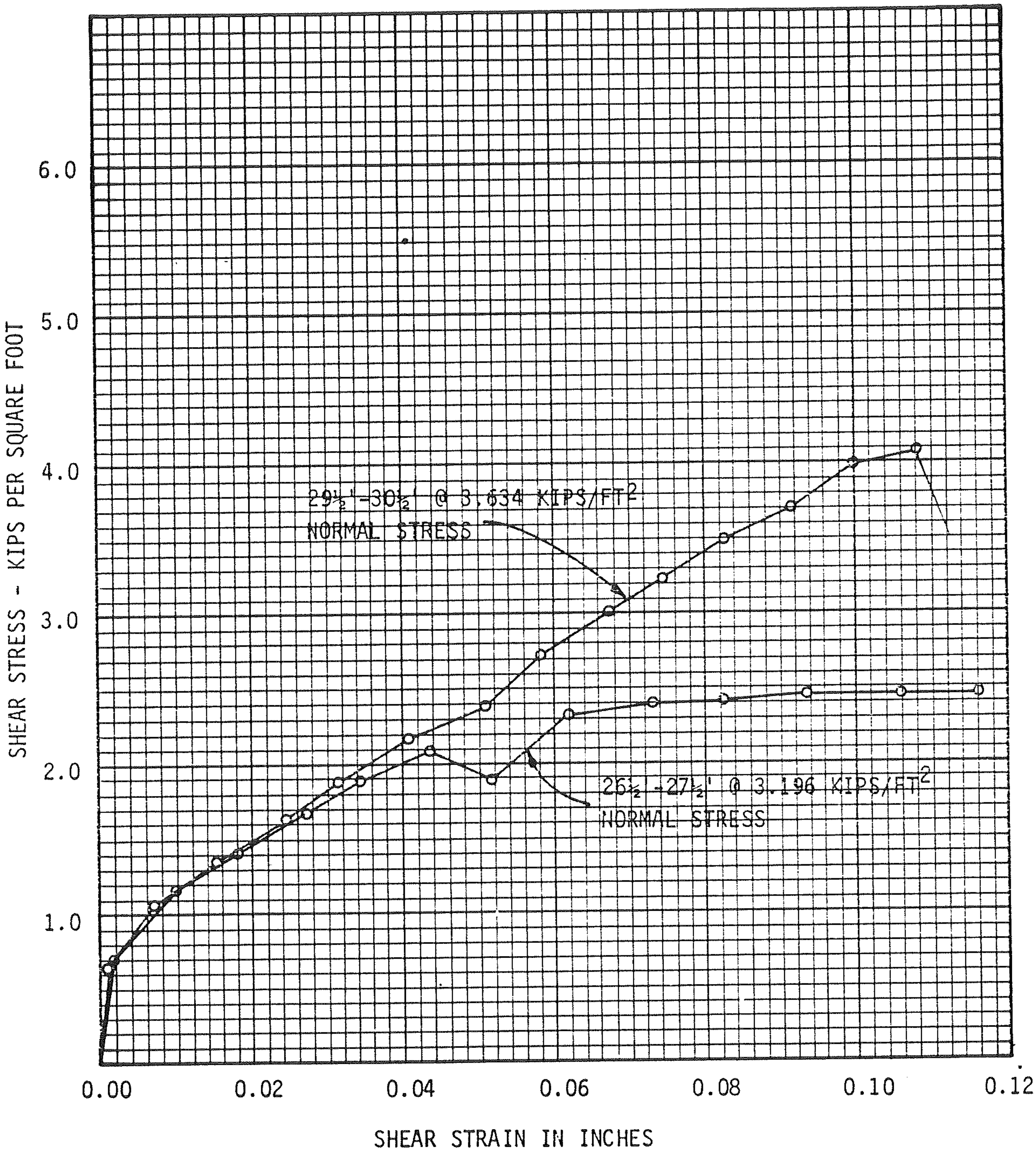
Drilled Cast-in-Place Concrete Piles JOB NO. E71-272

DIRECT SHEAR TEST DATA

Test Boring No. 4C



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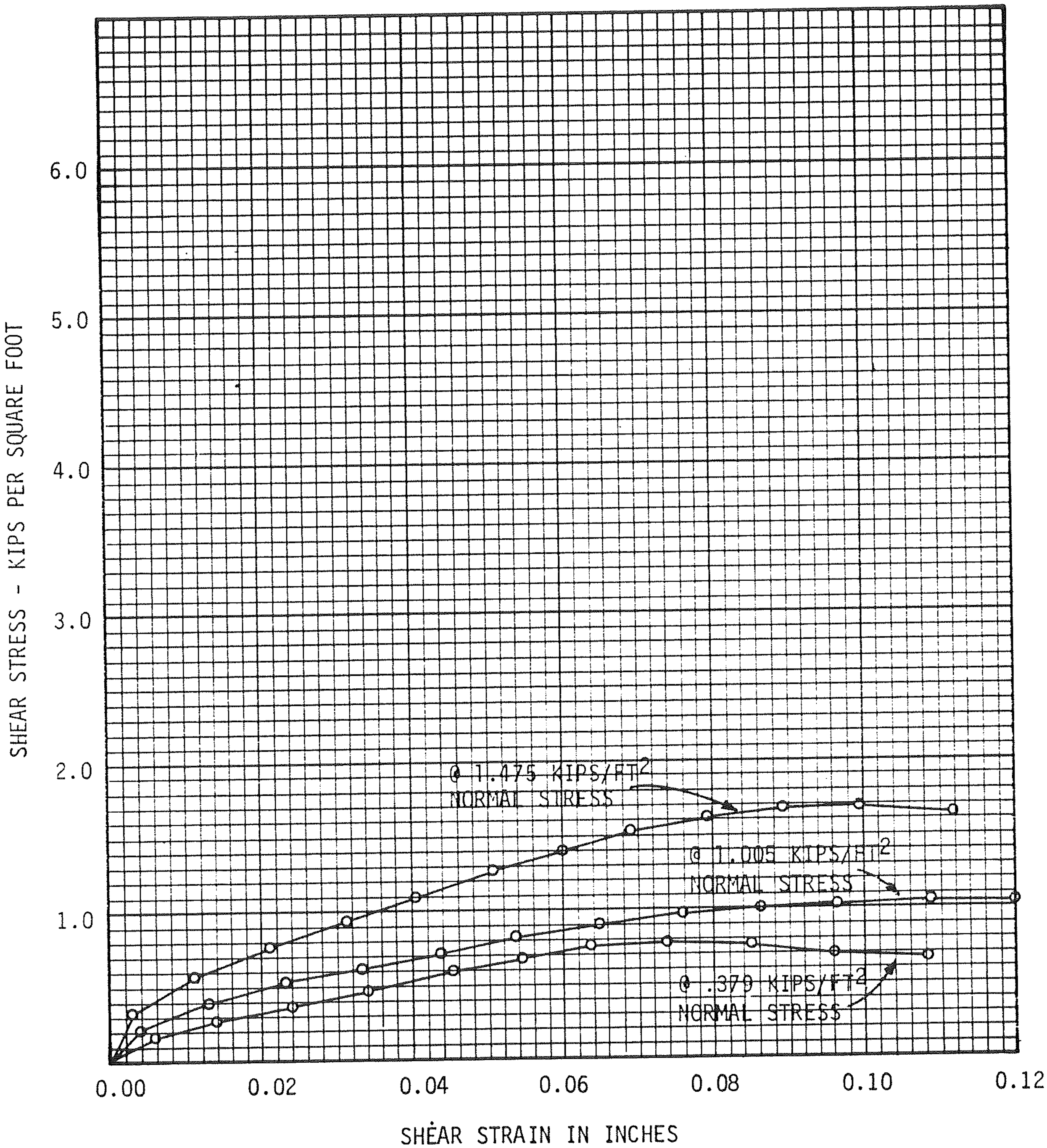
DIRECT SHEAR TEST DATA

Test Boring No. 4C, 2½'-3½'



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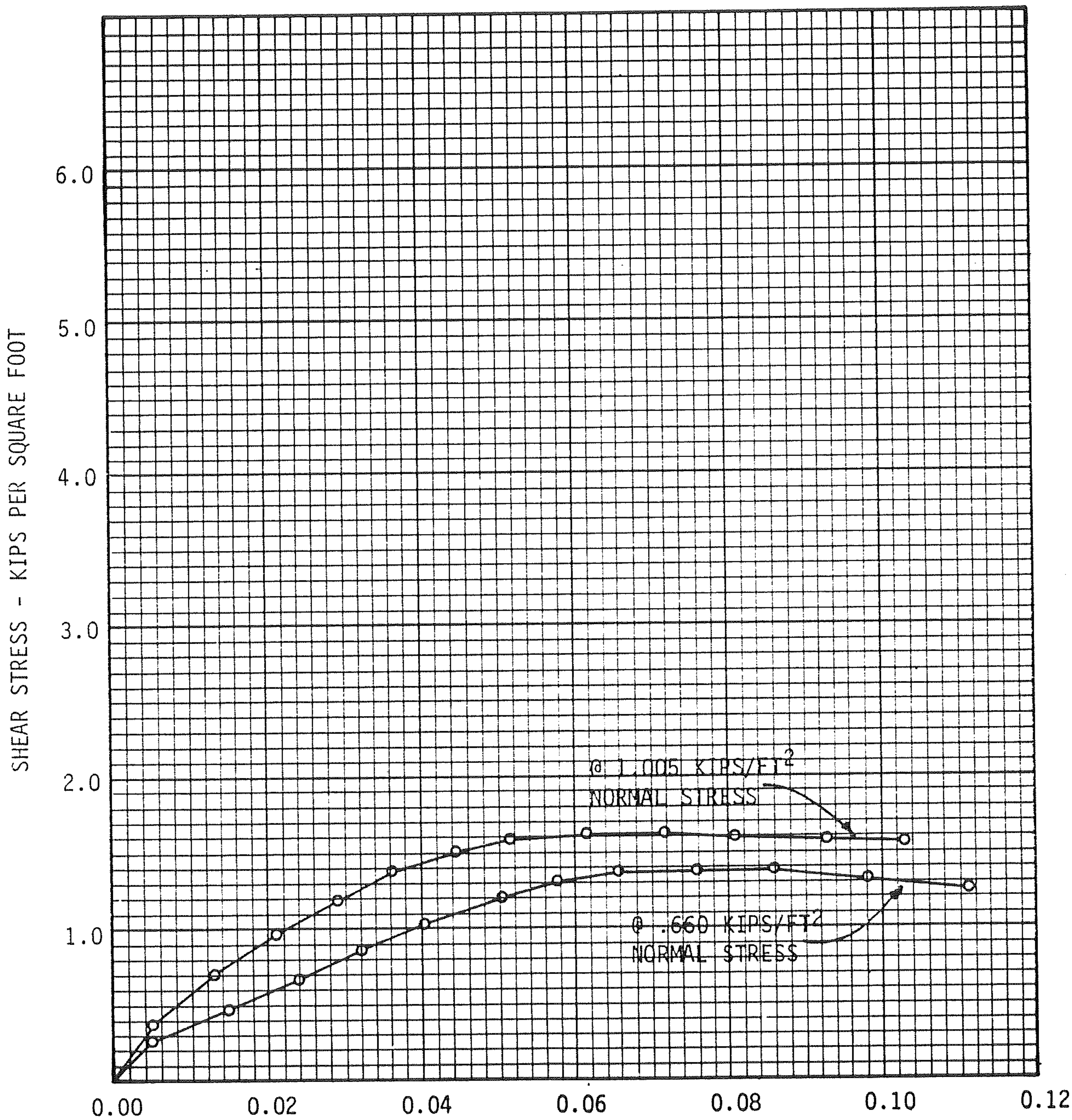
DIRECT SHEAR TEST DATA

Test Boring No. 4C, 4½'-5½'



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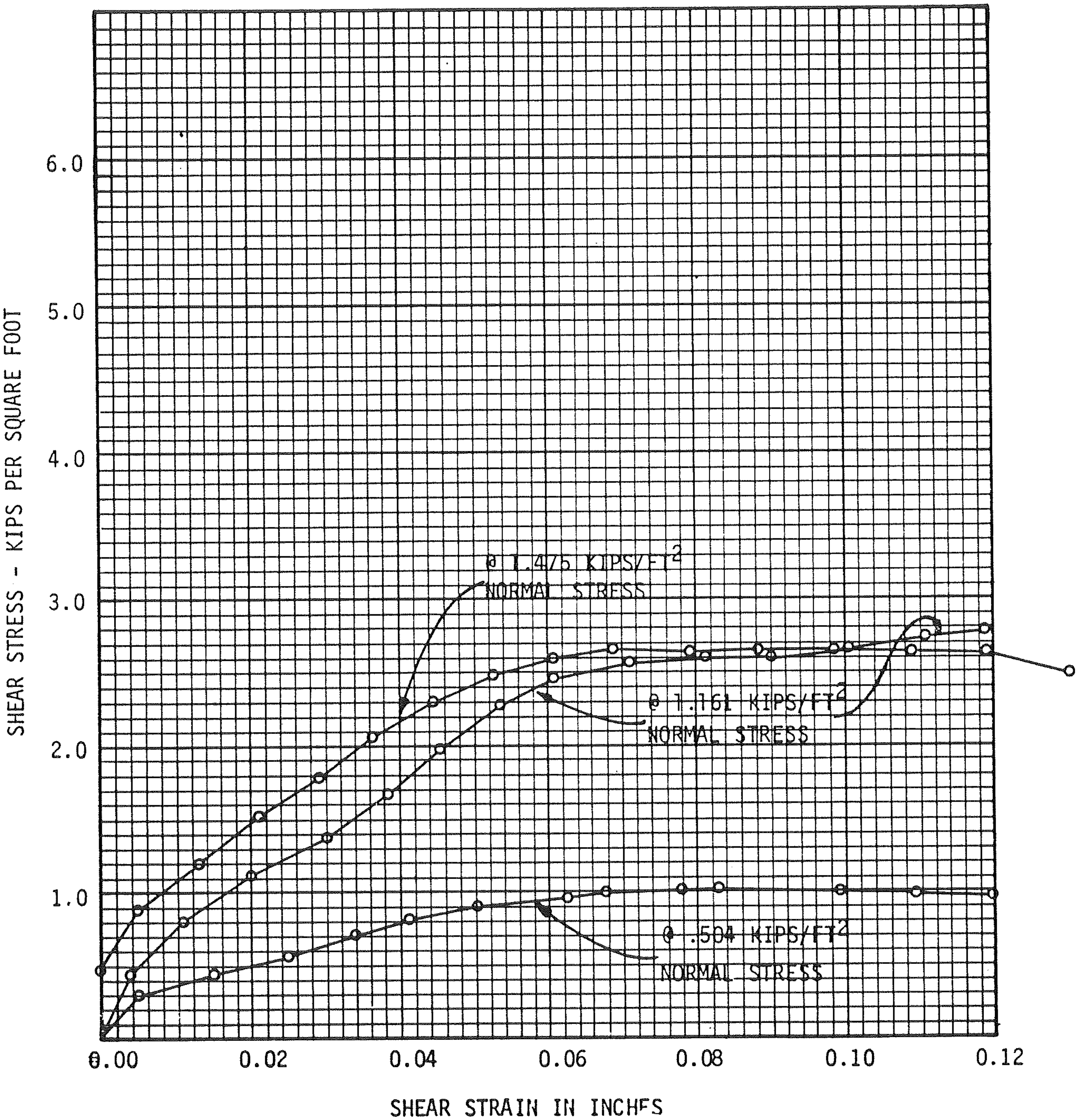
DIRECT SHEAR TEST DATA

Test Boring No. 4C, 9½'-10½'



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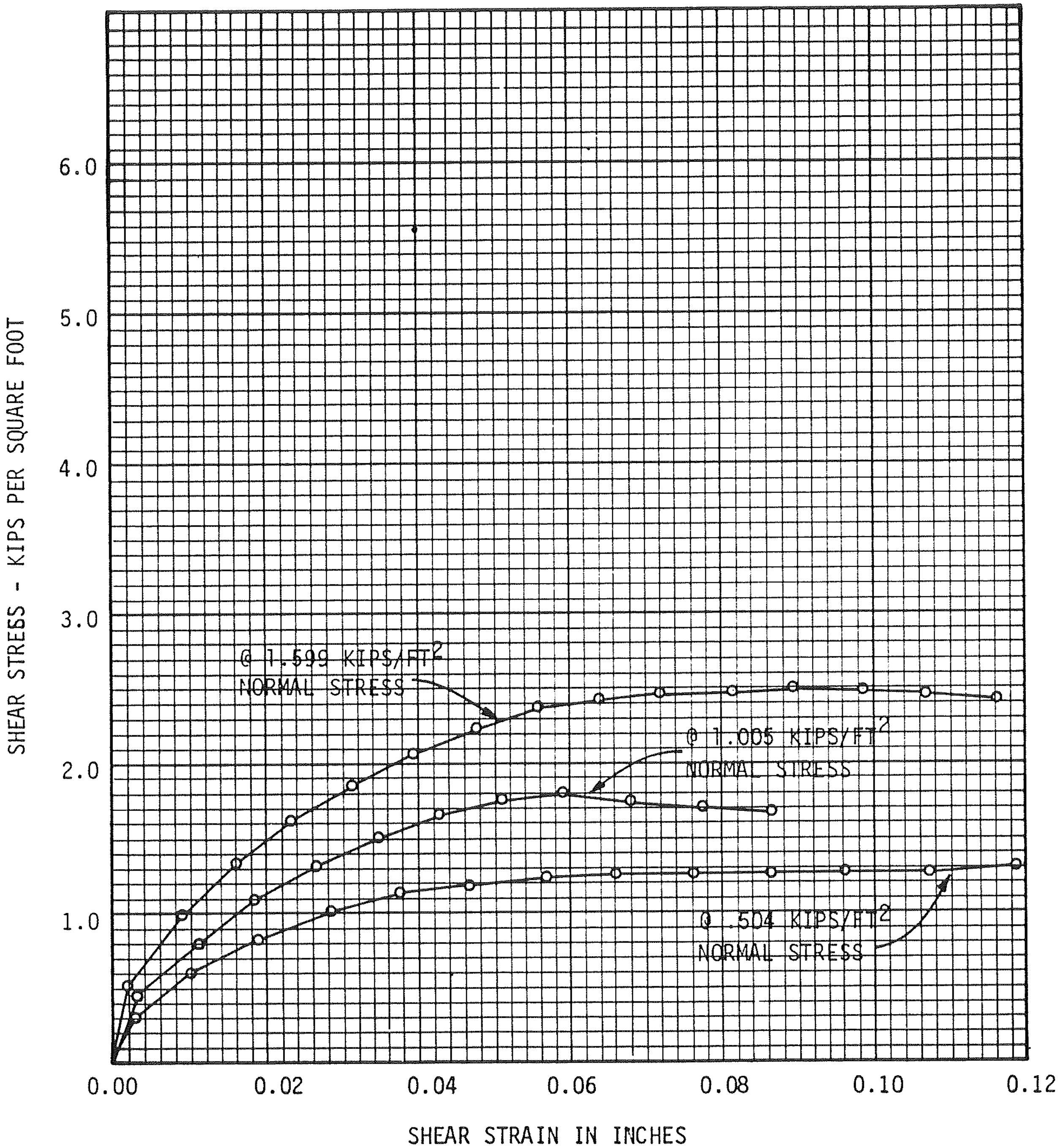
DIRECT SHEAR TEST DATA

Test Boring 4C, 12'-13'



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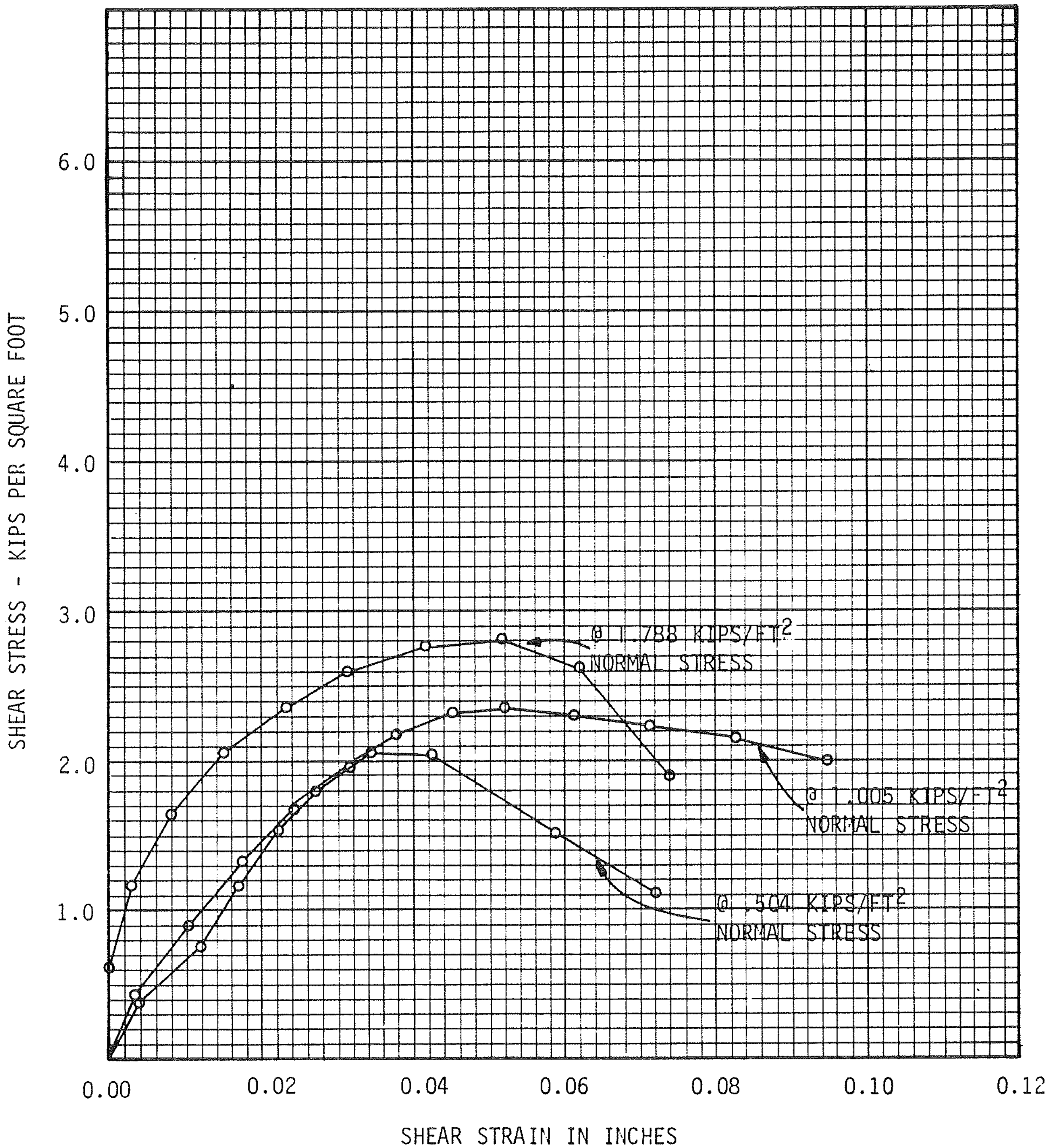
PROJECT Drilled Cast-in-Place Concrete Piles **JOB NO.** E71-272

DIRECT SHEAR TEST DATA

Test Boring No. 4C, 14½'-15½'



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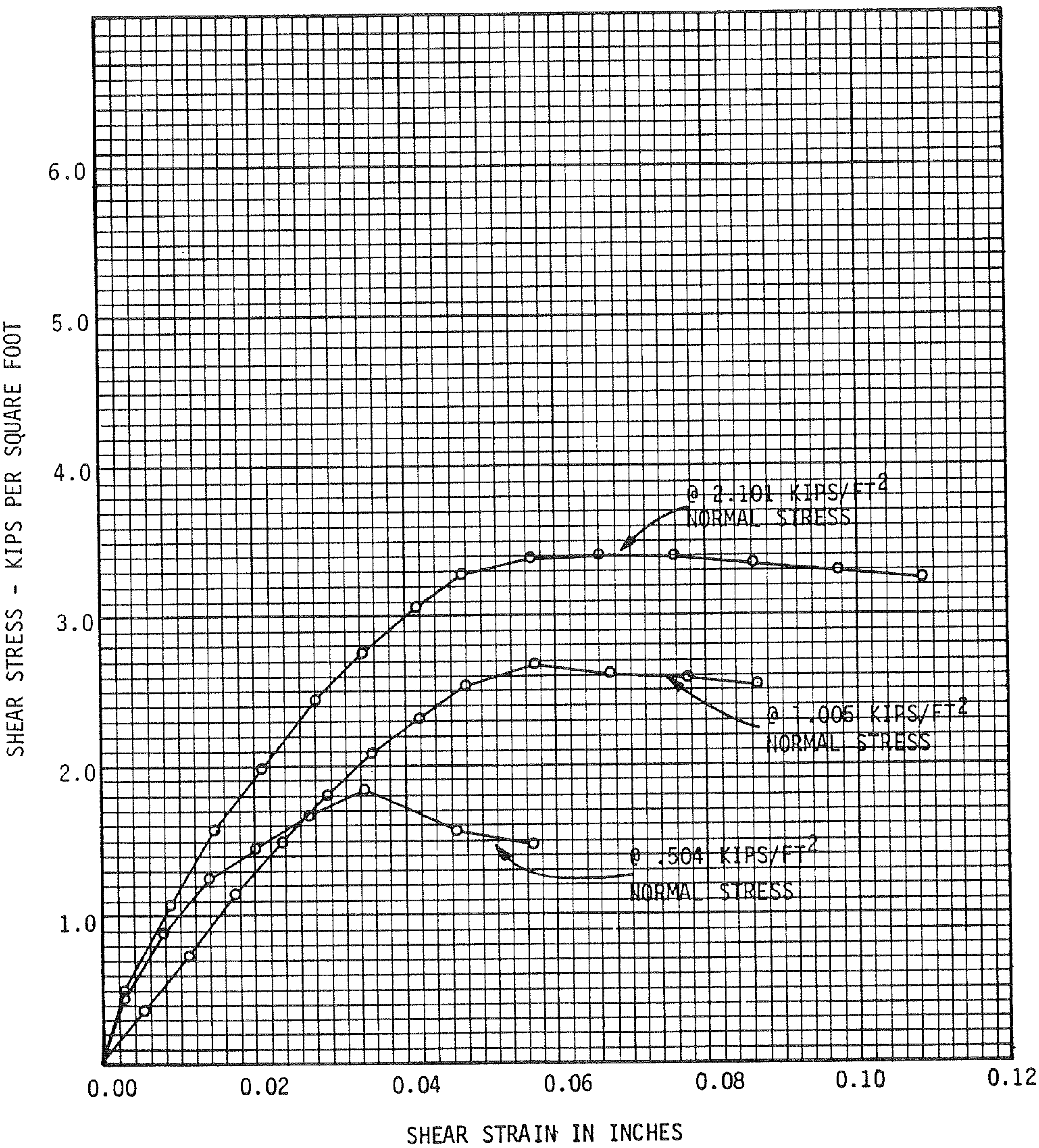
Drilled Cast-in-Place Concrete Piles JOB NO. E71-272

DIRECT SHEAR TEST DATA

Test Boring No. 4C, 17½'-18½'



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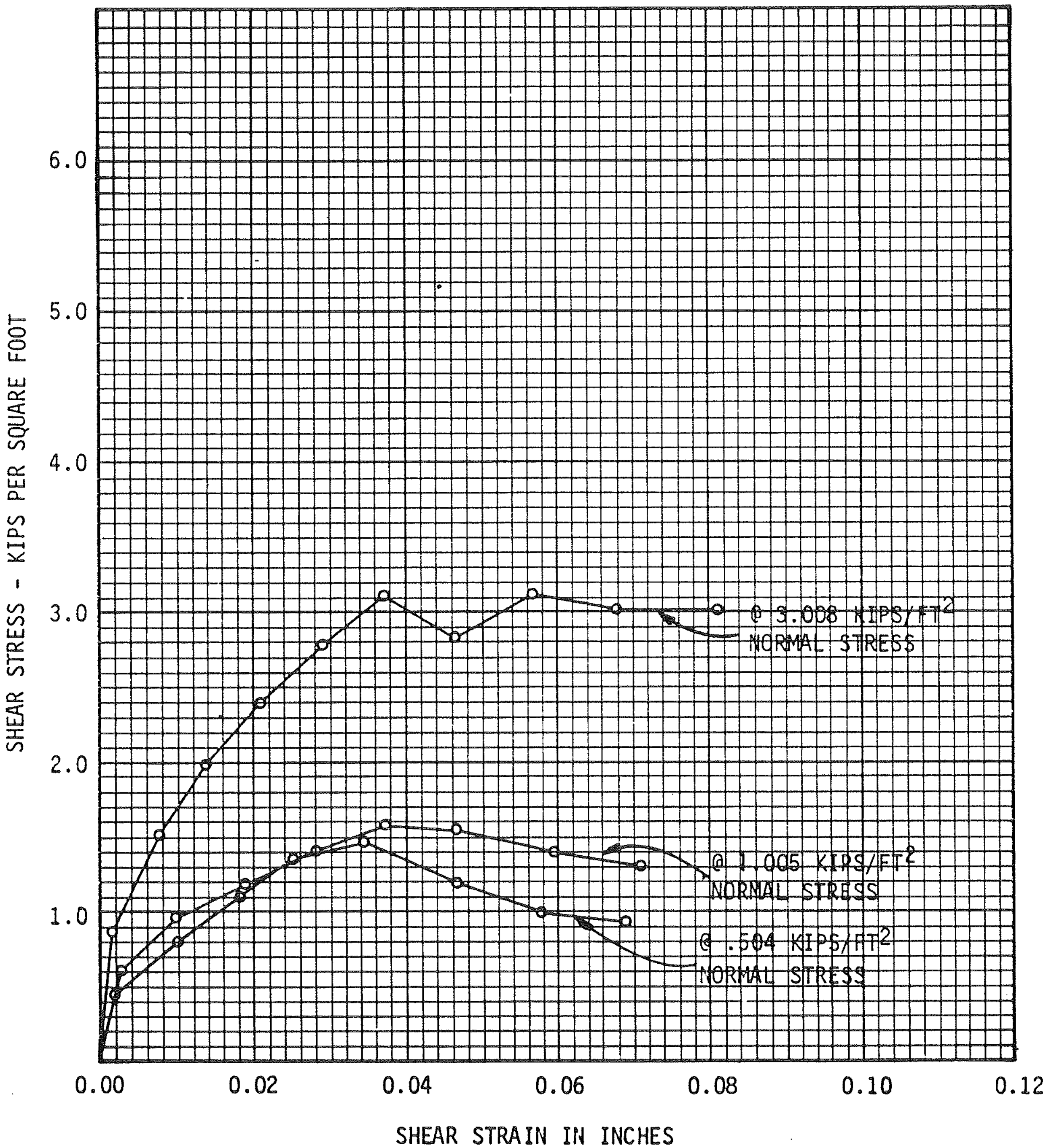
Drilled Cast-in-Place Concrete Piles

PROJECT **JOB NO.** E71-272

DIRECT SHEAR TEST DATA

Test Boring No. 4C, 24½'-25½'



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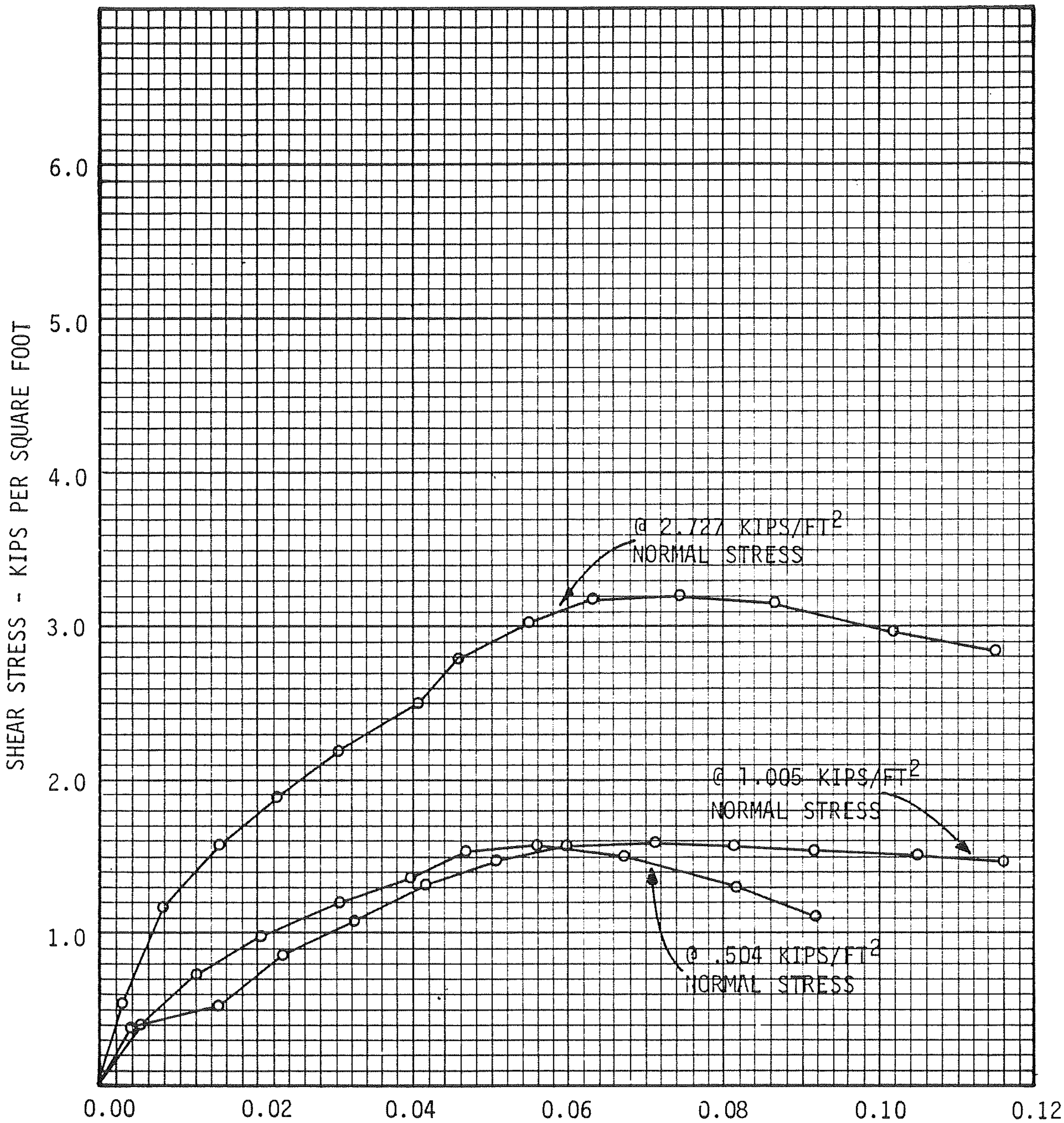
Drilled Cast-in-Place Concrete Piles
JOB NO. E71-272

DIRECT SHEAR TEST DATA

Test Boring No. 4C, 22½'-23½'



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MATERIALS TESTING ENGINEERS

ENGINEERING ANALYSIS • PHYSICAL TESTING • QUALITY CONTROL • FIELD EXPLORATION

RESULTS OF COMPRESSION TESTS

DATE

PROJECT Drilled Cast-in-Place Concrete Piles JOB NO. E70-130

CLIENT Arizona Highway Department ADDRESS 1739 West Jackson Phoenix, Arizona

CONTRACTOR ARCHITECT/ENGINEER

SOURCE OF SAMPLE Test Piles - TPA-1, TPA-3, TPA-4, TPA-6, TPA-7

SOURCE OF MATERIAL United Metro DESIGN STRENGTH, PSI 5,000 @ 28 days

TICKET NO. MAX. SIZE AGGREGATE, IN. 3/4

MIX ADMIXTURE Pozzolith 300R

BATCH SIZE NO. OF SPECIMENS MOLDED 3 - each set

TIME IN MIXER WATER ADDED ON JOB, GAL.

SLUMP, IN. See below SAMPLED BY SHB tested ATL-ETL

SUBMITTED BY SHB/Yates DATE RECEIVED

REMARKS:

SITE A

LAB NO.	IDENTIFICATION NO.	SLUMP (INCHES)	DATE MADE	DATE TESTED	AGE (DAYS)	STRENGTH (LBS./SQ. IN.)
*24-3672	TPA-1	3 1/4	7-29-71	8-5-71	7	5130
				8-26-71	28	6080
**24-3798	TPA-3	4 3/4	8-5-71	8-12-71	7	4030
				9-2-71	28	4970
24-3798	TPA-4	3 3/4	8-5-71	8-12-71	7	4490
				9-2-71	28	5920
3710	TPA-6	6	11-1-71	11-8-71	7	4560
3711				11-29-71	28	6440
3704	TPA-7	2 5/8	11-1-71	11-8-71	7	4400
3705				11-29-71	28	6490

RESPECTFULLY SUBMITTED,

*Modulus of Elasticity = 4.96×10^6 psi

**Modulus of Elasticity = 4.388×10^6 psi

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(602) 774-4433

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(915) 772-3088

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RESULTS OF COMPRESSION TESTS

DATE

PROJECT Drilled Cast-in-Place Concrete Piles JOB NO. E70-130

CLIENT Arizona Highway Department ADDRESS 1739 West Jackson Phoenix, Arizona

CONTRACTOR ARCHITECT/ENGINEER

SOURCE OF SAMPLE Anchors - Site A - TPA-6, TPA-7

SOURCE OF MATERIAL DESIGN STRENGTH, PSI 5,000 @ 28 days

TICKET NO. MAX. SIZE AGGREGATE, IN. $\frac{3}{4}$

MIX ADMIXTURE Pozzolith 82

BATCH SIZE NO. OF SPECIMENS MOLDED 3 - each set

TIME IN MIXER WATER ADDED ON JOB, GAL.

SLUMP, IN. See below SAMPLED BY SHB tested ATL

SUBMITTED BY SHB/Yates DATE RECEIVED

REMARKS:

SITE A

LAB NO.	IDENTIFICATION NO.	SLUMP (INCHES)	DATE MADE	DATE TESTED	AGE (DAYS)	STRENGTH (LBS./SQ. IN.)
3701	Anchors	4	11-1-71	11-8-71	7	4770
3702				11-29-71	28	6670
3707	TPA-7	$2\frac{3}{4}$	11-1-71	11-8-71	7	4900
3708				11-29-71	28	6720
3713	TPA-6 &	4	11-1-71	11-8-71	7	4930
	TPA-7			11-29-71	28	6760

RESPECTFULLY SUBMITTED,

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RESULTS OF COMPRESSION TESTS

DATE 2-5-73

PROJECT Drilled Cast-in-Place Concrete Piles JOB NO. E71-272

CLIENT Arizona Highway Department ADDRESS 1739 West Jackson Phoenix, Arizona 85007

CONTRACTOR ARCHITECT/ENGINEER

SOURCE OF SAMPLE Test Piles - TPB-1, TPB-2

SOURCE OF MATERIAL United Metro DESIGN STRENGTH, PSI 5,000 @ 28 days

TICKET NO. 6-21452, 6-21482 MAX. SIZE AGGREGATE, IN. $\frac{3}{4}$

MIX 1-28880 ADMIXTURE

BATCH SIZE 5 cubic yards NO. OF SPECIMENS MOLDED 3 each set

TIME IN MIXER WATER ADDED ON JOB, GAL. 10, 0, 5

SLUMP, IN. See below SAMPLED BY SHB/Davis

SUBMITTED BY DATE RECEIVED

REMARKS:

TEST SITE B

LAB NO.	IDENTIFICATION NO.	SLUMP (INCHES)	DATE MADE	DATE TESTED	AGE (DAYS)	STRENGTH (LBS./SQ. IN.)
4723	TPB-1	4	12-30-71	1-6-72	7	3980
				1-27-72	28	5980
4726	TPB-2	5	12-30-71	1-6-72	7	3530
				1-27-72	28	5750
4729	TPB-6	$3\frac{3}{4}$	1-4-72	1-11-72	7	4540
				2-1-72	28	6280

RESPECTFULLY SUBMITTED,

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DATE 2-5-73

PROJECT Drilled Cast-in-Place Concrete Piles JOB NO. E71-272

CLIENT Arizona Highway Department ADDRESS 1739 West Jackson, Phoenix, Arizona 85007

CONTRACTOR ARCHITECT/ENGINEER

SOURCE OF SAMPLE Test Piles - TPC-1, TPC-2, TPC-3, Anchor B3

SOURCE OF MATERIAL United Metro DESIGN STRENGTH, PSI 5,000 @ 28 days

TICKET NO. 6-22204, 6-17563 MAX. SIZE AGGREGATE, IN. $\frac{3}{4}$

MIX ADMIXTURE

BATCH SIZE 5, $8\frac{1}{2}$, $9\frac{1}{2}$ cubic yards NO. OF SPECIMENS MOLDED 3 each set

TIME IN MIXER WATER ADDED ON JOB, GAL. 0, 5, 0

SLUMP, IN. See below SAMPLED BY SHB/Davis, Yates

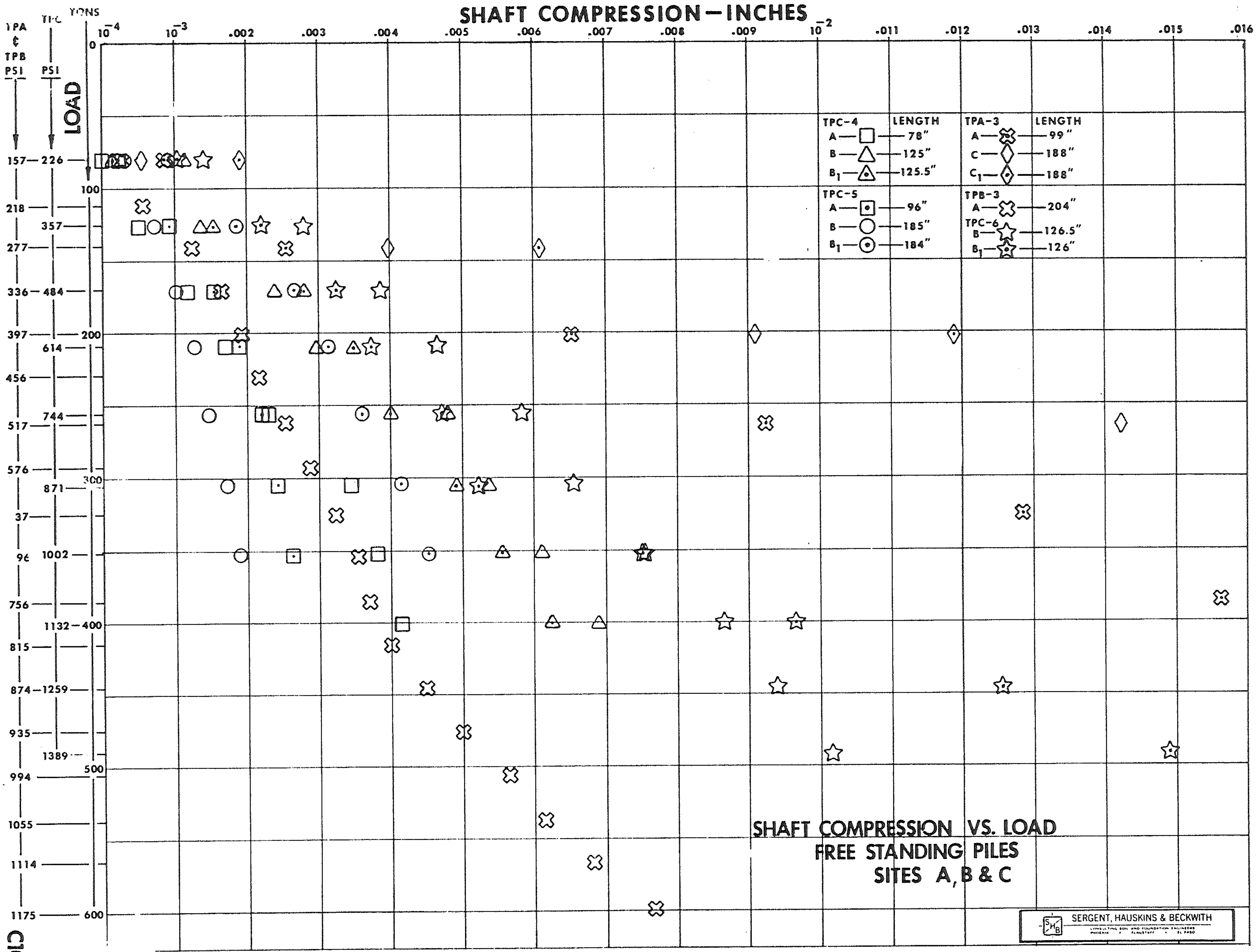
SUBMITTED BY DATE RECEIVED

REMARKS:

TEST SITE C

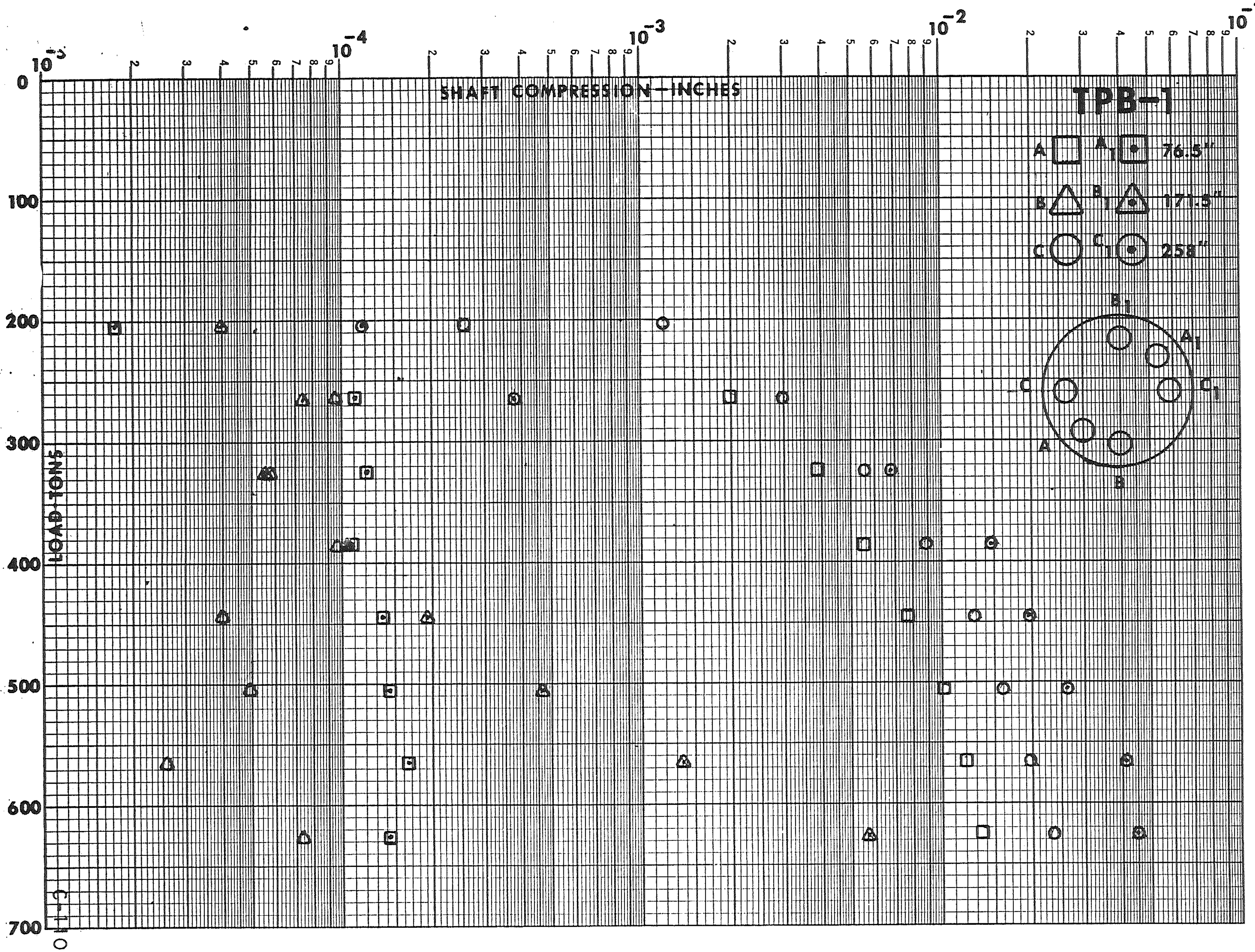
LAB NO.	IDENTIFICATION NO.	SLUMP (INCHES)	DATE MADE	DATE TESTED	AGE (DAYS)	STRENGTH (LBS./SQ. IN.)
4787	TPC-1	$5\frac{1}{4}$	1-7-72	1-14-72	7	4690
				2-4-72	28	6210
4799	TPC-2, 3	$4\frac{1}{4}$	1-10-72	1-17-72	7	4760
				2-7-72	28	6290
4168	Anchors	$4\frac{1}{4}$	11-19-71	11-29-71	10	5060
				12-17-71	28	5640

RESPECTFULLY SUBMITTED,

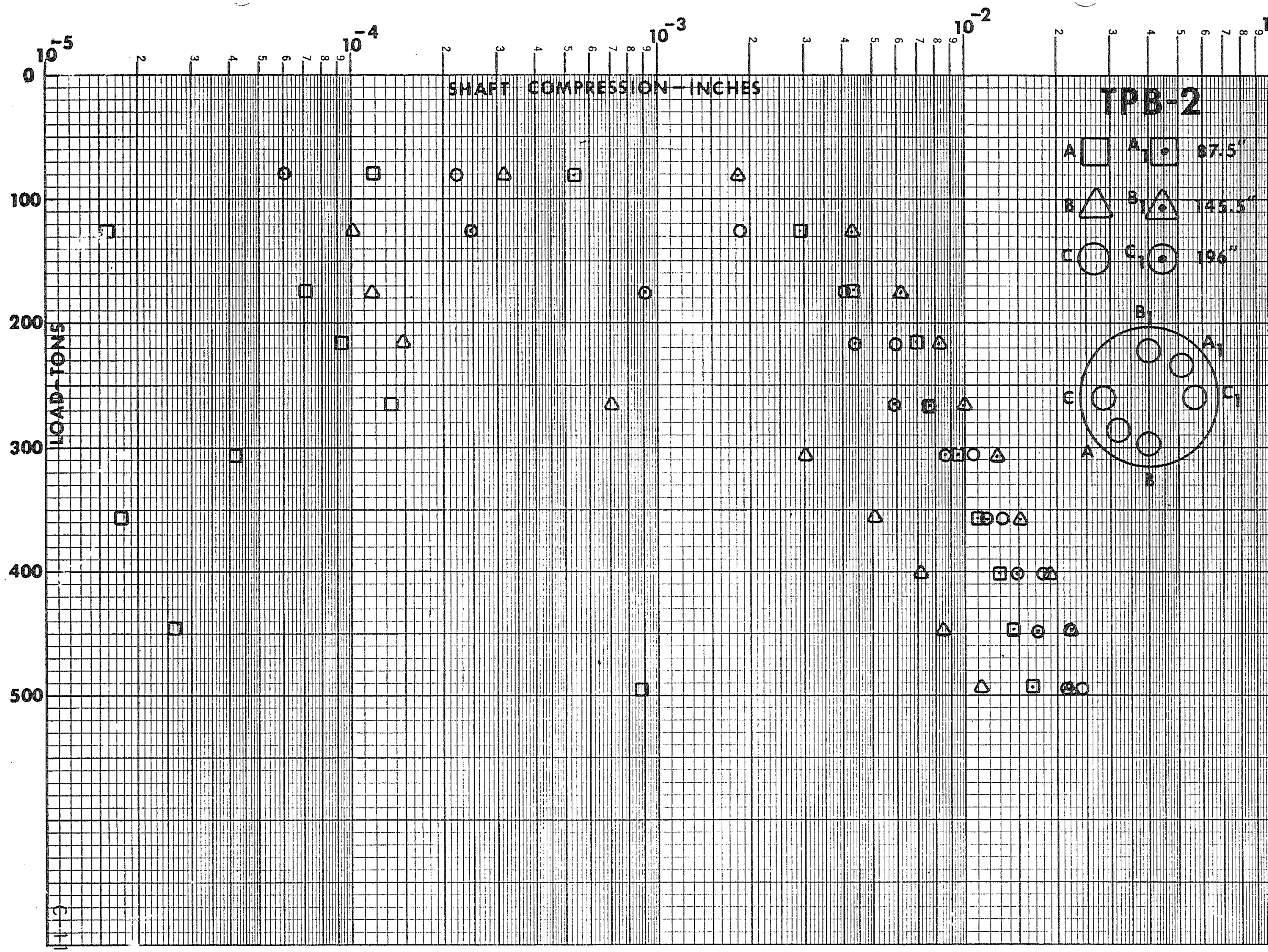


SHAFT COMPRESSION VS. LOAD
FREE STANDING PILES
SITES A, B & C

C109

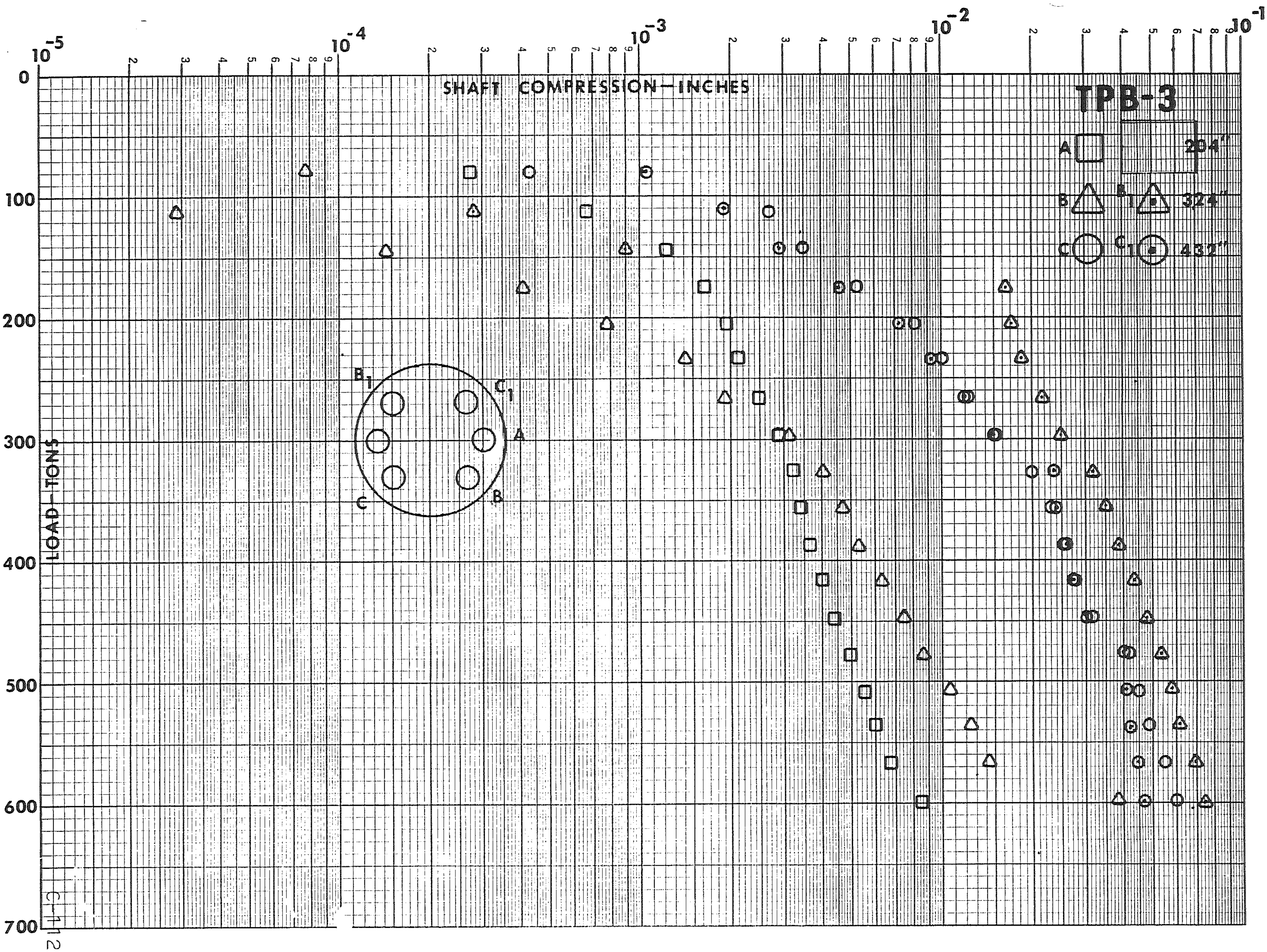

TPB-1
SHAFT COMPRESSION—INCHES
LOAD-TONS
A
A1 76.5″
B
B1 171.5″
C
C1 258″
A
A1
B
B1
C
C1
10^-5
10^-4
10^-3
10^-2
10^-1
0
100
200
300
400
500
600
700

C-110

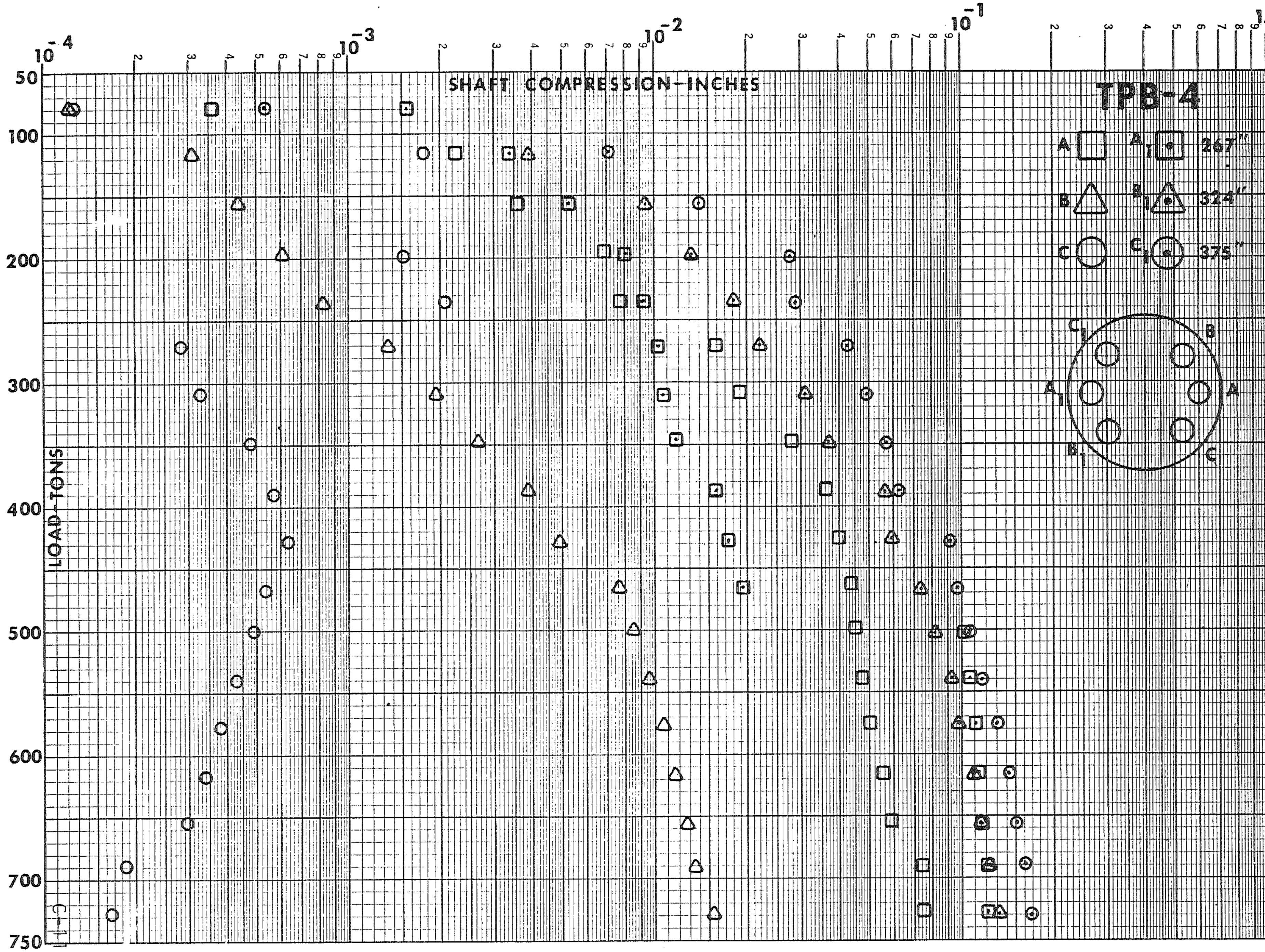


TPB-2
SHAFT COMPRESSION—INCHES
LOAD-TONS
A
A_1
87.5"
B
B_1
145.5"
C
C_1
196"

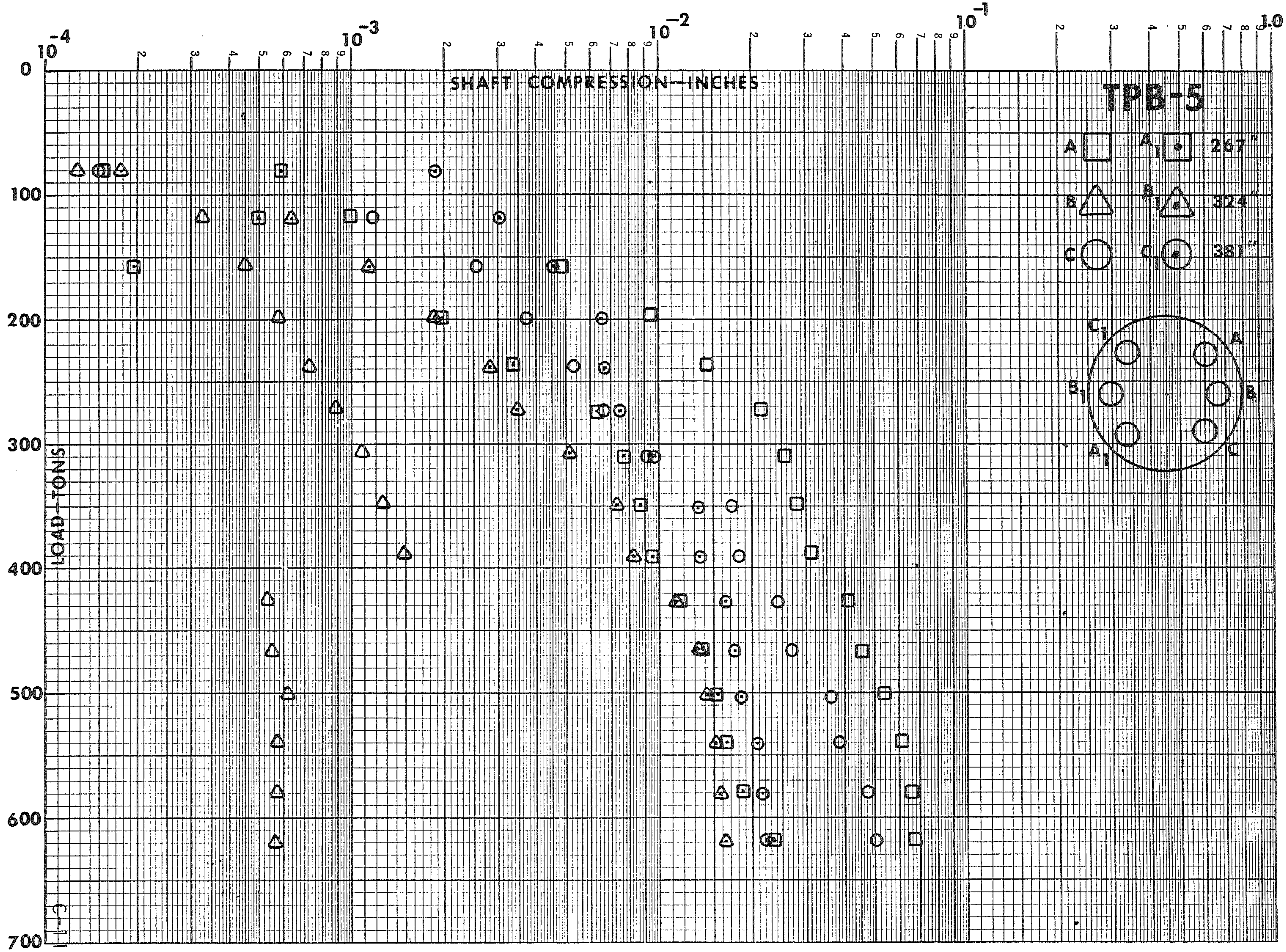
C-111

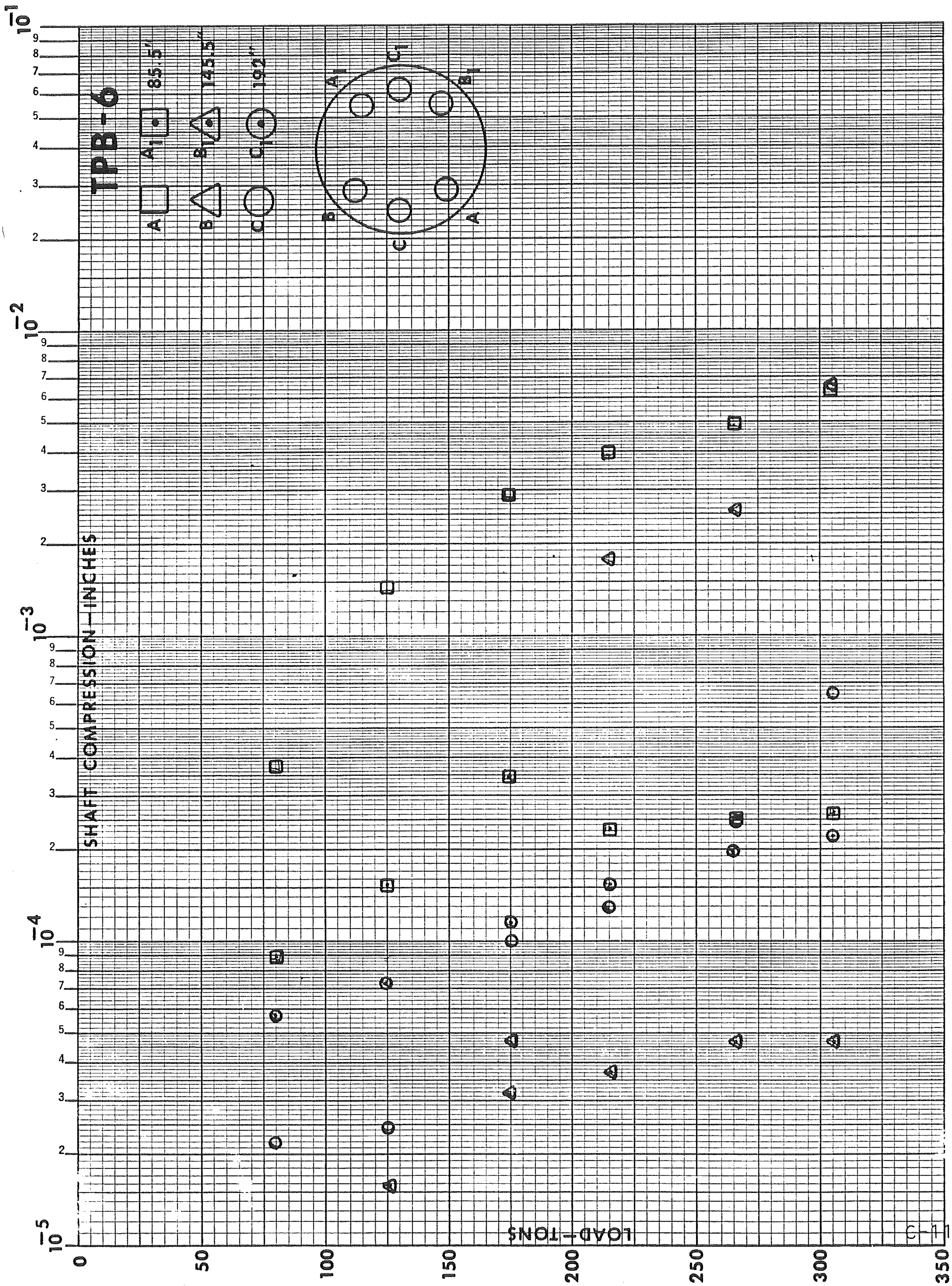


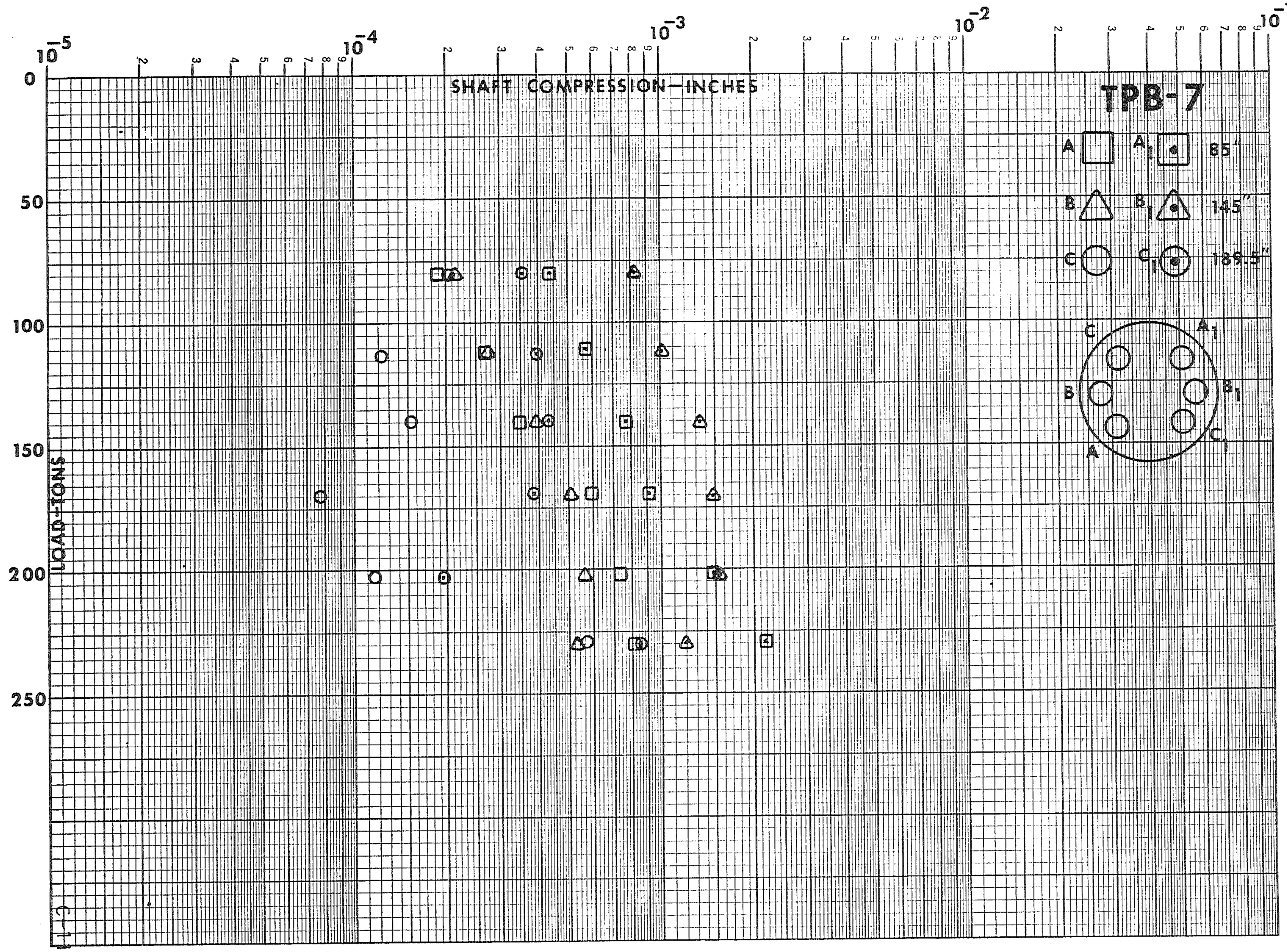
TPB-3
A .204"
B B1 .324"
C C1 .432"
SHAFT COMPRESSION—INCHES
LOAD—TONS
A
B
B1
C
C1

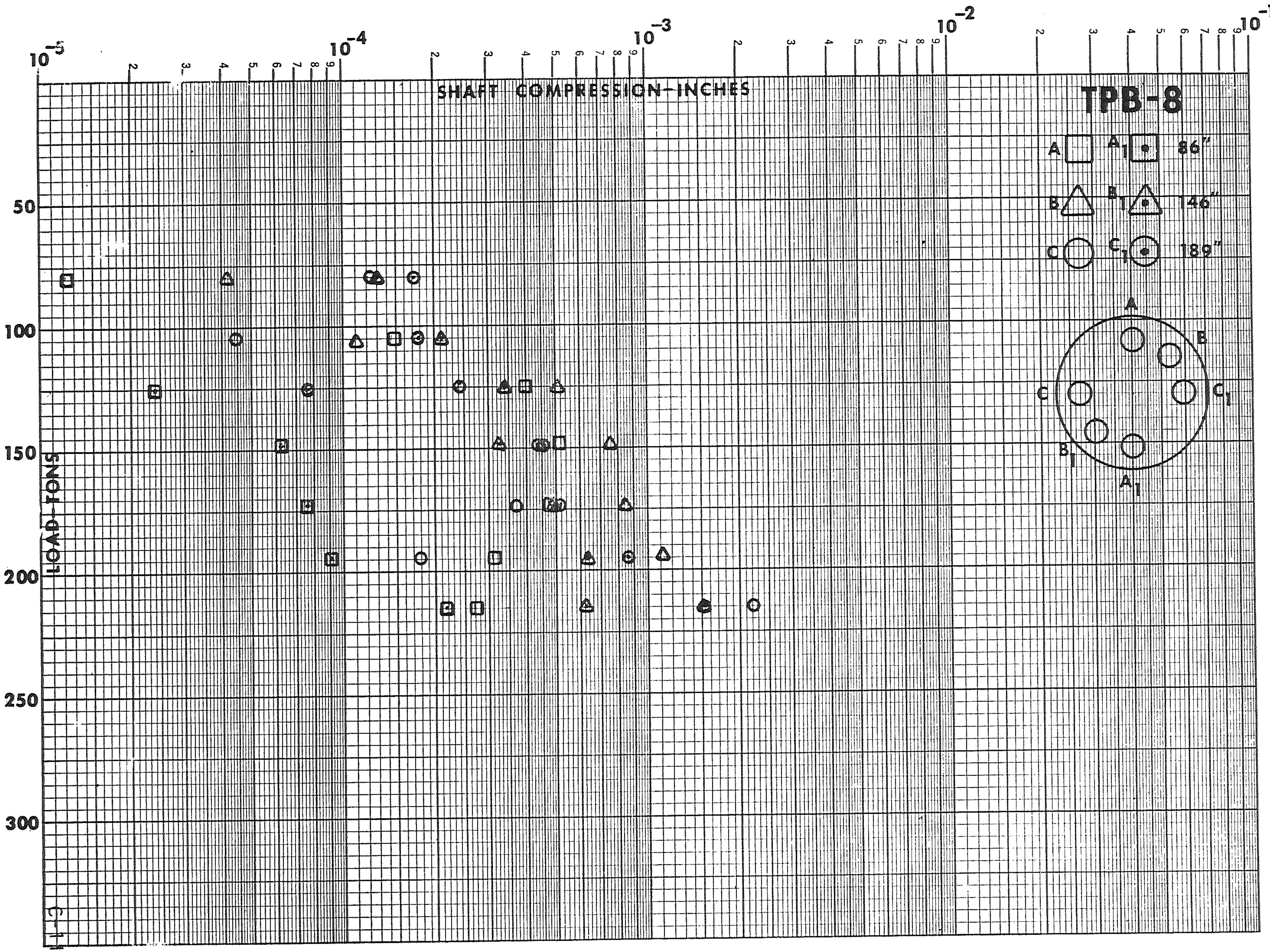


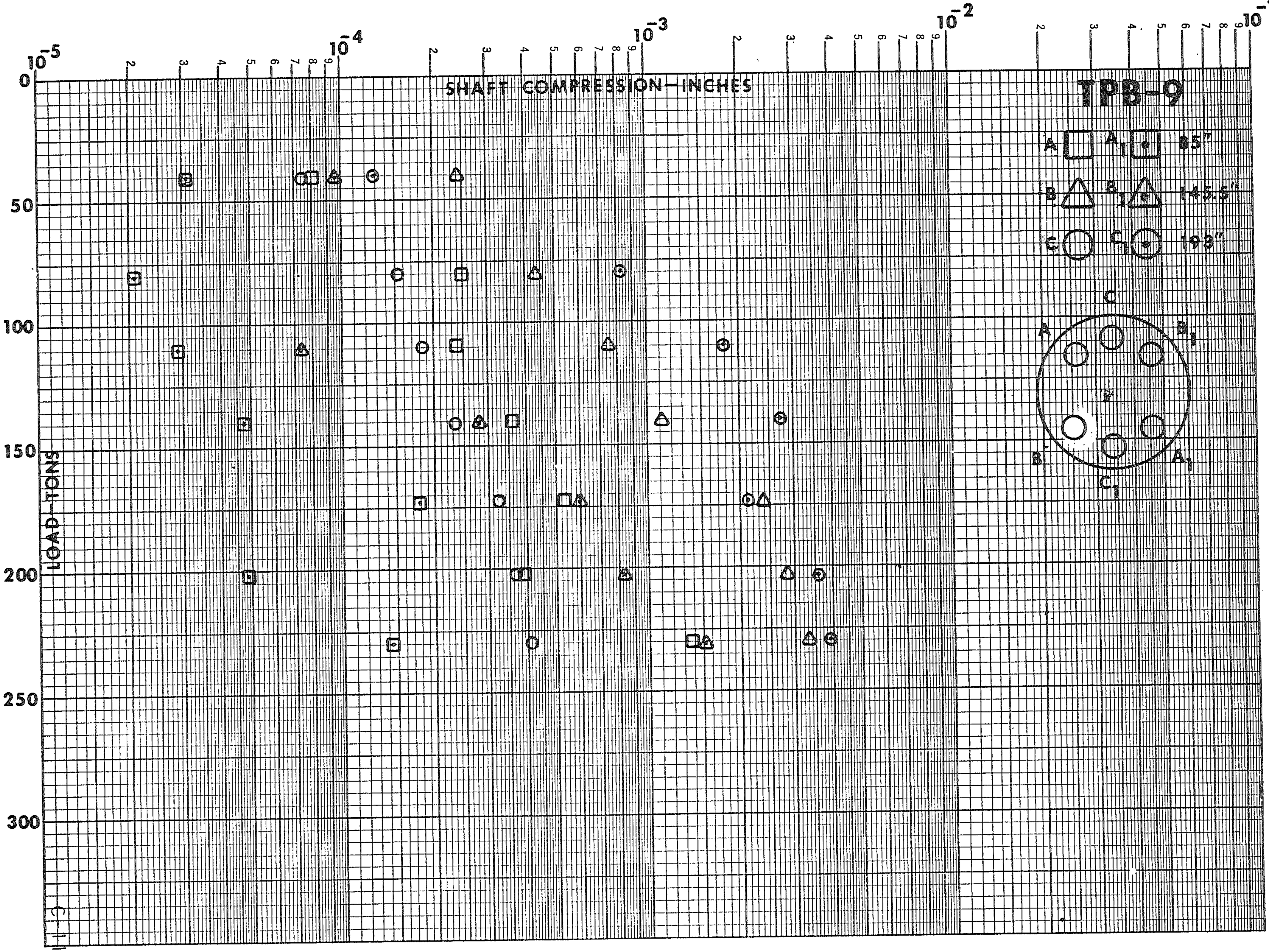
TPB-4
A
A1
.267"
B
B1
.324"
C
C1
.375"
A
B
C
A1
B1
C1
SHAFT COMPRESSION-INCHES
LOAD-TONS
10^-4
10^-3
10^-2
10^-1
1.0
50
100
200
300
400
500
600
700
750

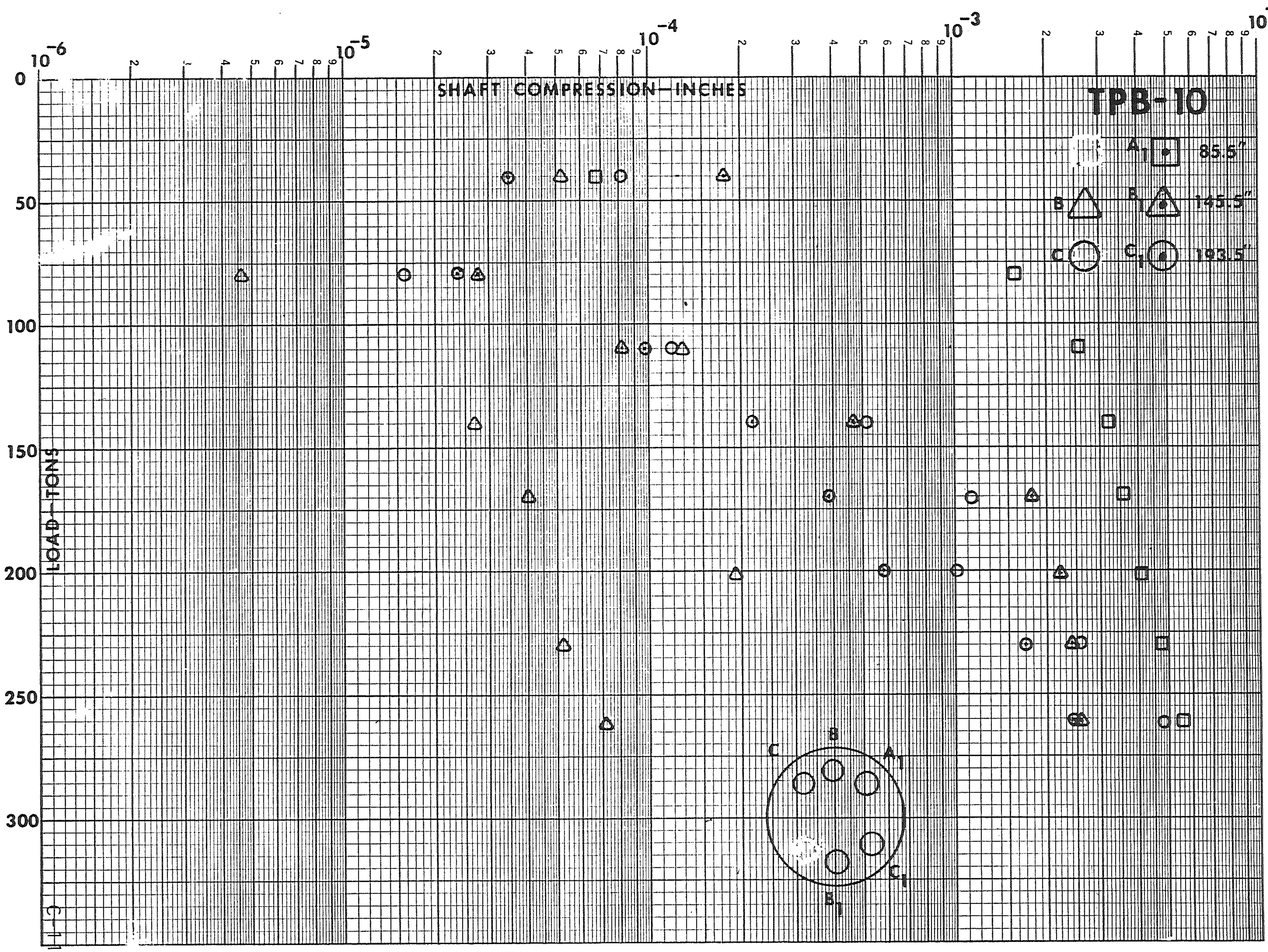

TPB-5
SHAFT COMPRESSION—INCHES
LOAD—TONS
A
A_1
.267"
B
B_1
.324"
C
C_1
.381"
A
B
C
A_1
B_1
C_1

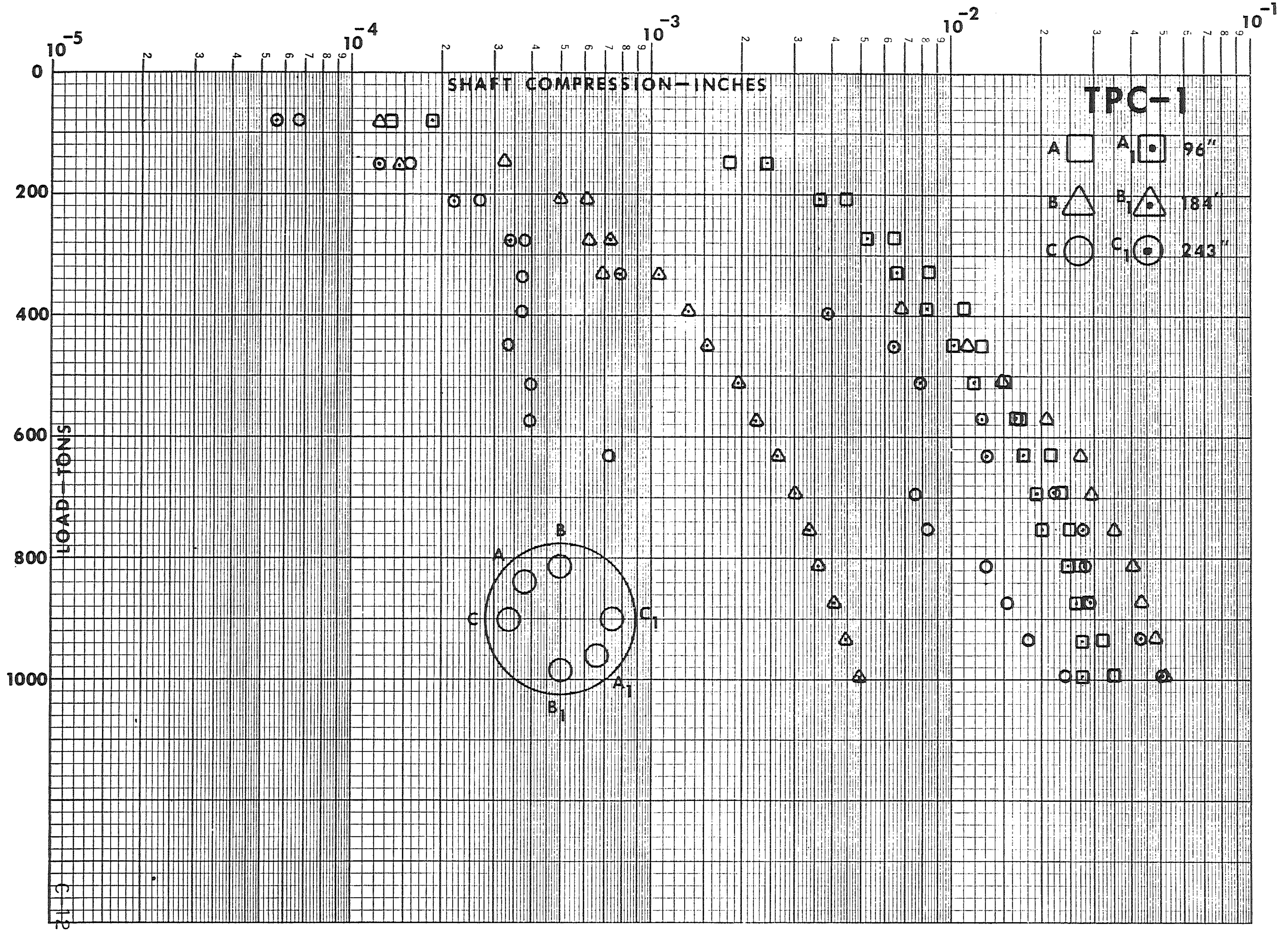

TPB-6
A
A1 85.5"
B
B1 145.5"
C
C1 192"
A1
C1
B1
B
C
A
SHAFT COMPRESSION—INCHES
LOAD—TONS
0
50
100
150
200
250
300
350


TPB-7
A
A1 85"
B
B1 145"
C
C1 189.5"
SHAFT COMPRESSION—INCHES
LOAD-TONS

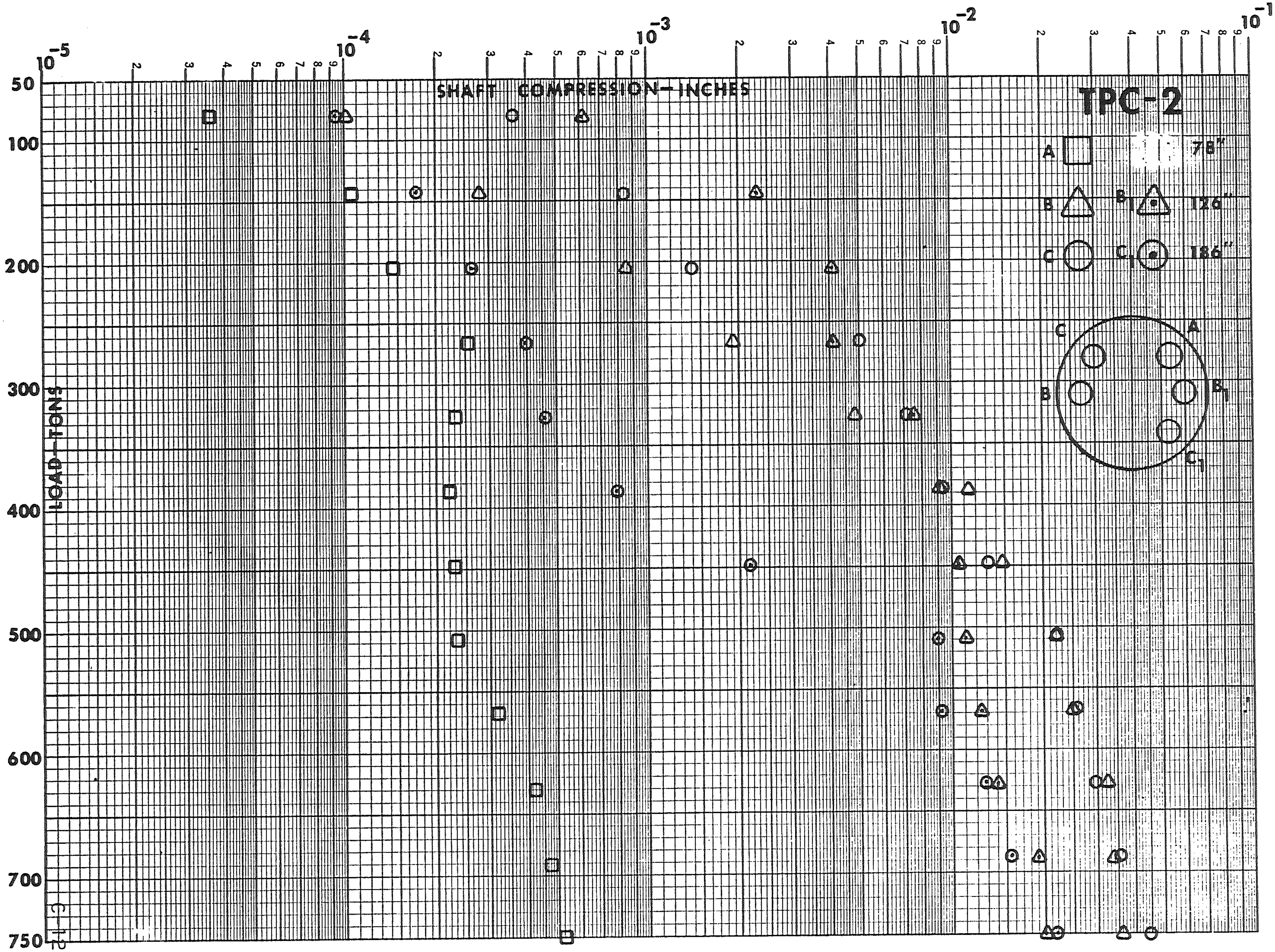

TPB-8
A A1 86"
B B1 146"
C C1 189"
A B C C1 B1 A1
SHAFT COMPRESSION-INCHES
LOAD-TONS
10^-5 2 3 4 5 6 7 8 9 10^-4 2 3 4 5 6 7 8 9 10^-3 2 3 4 5 6 7 8 9 10^-2 2 3 4 5 6 7 8 9 10^-1
50
100
150
200
250
300

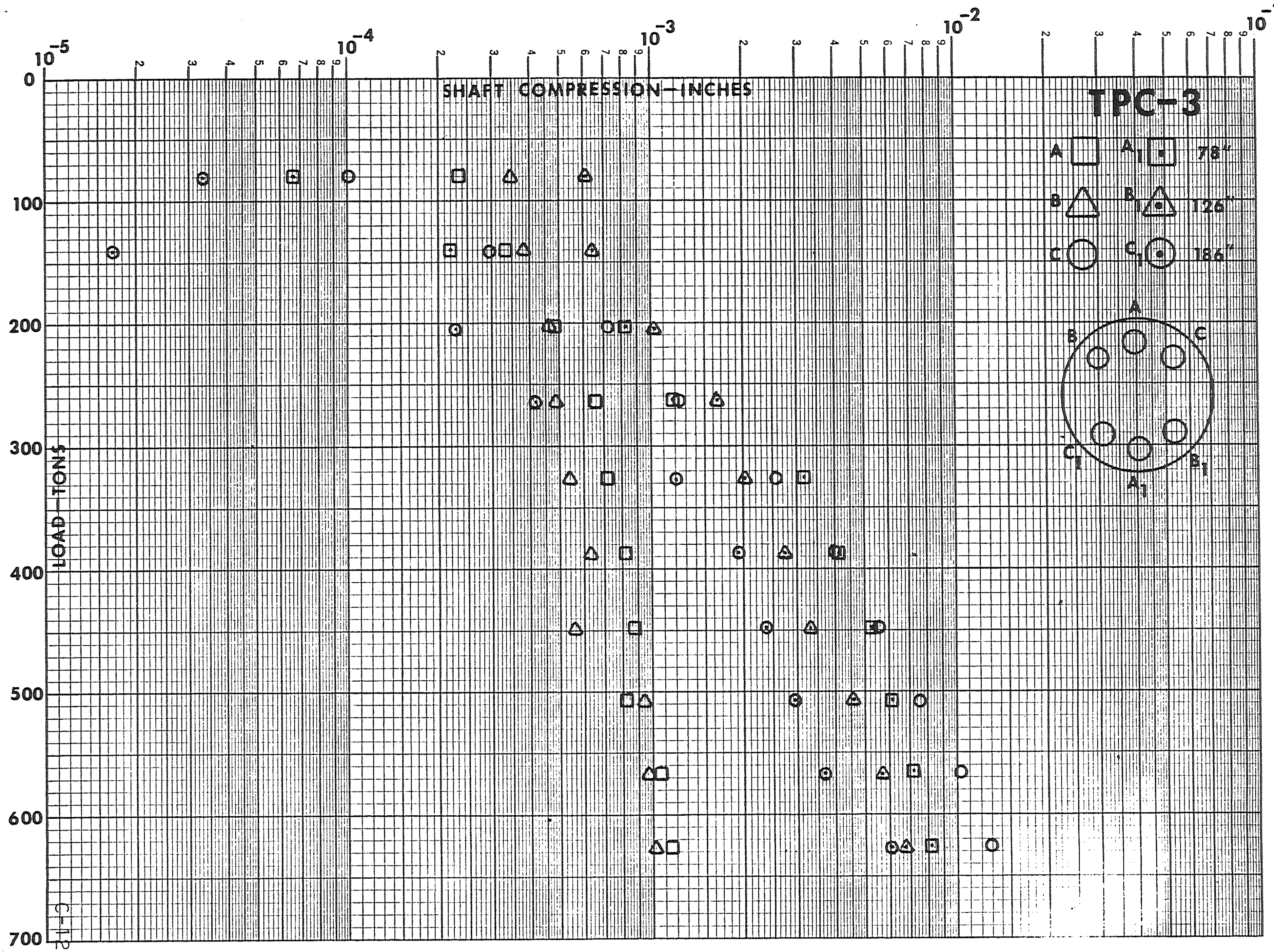

TPB-9
SHAFT COMPRESSION—INCHES
LOAD—TONS
A A1 85"
B B1 145.5"
C C1 193"
A C B1
B C1 A1

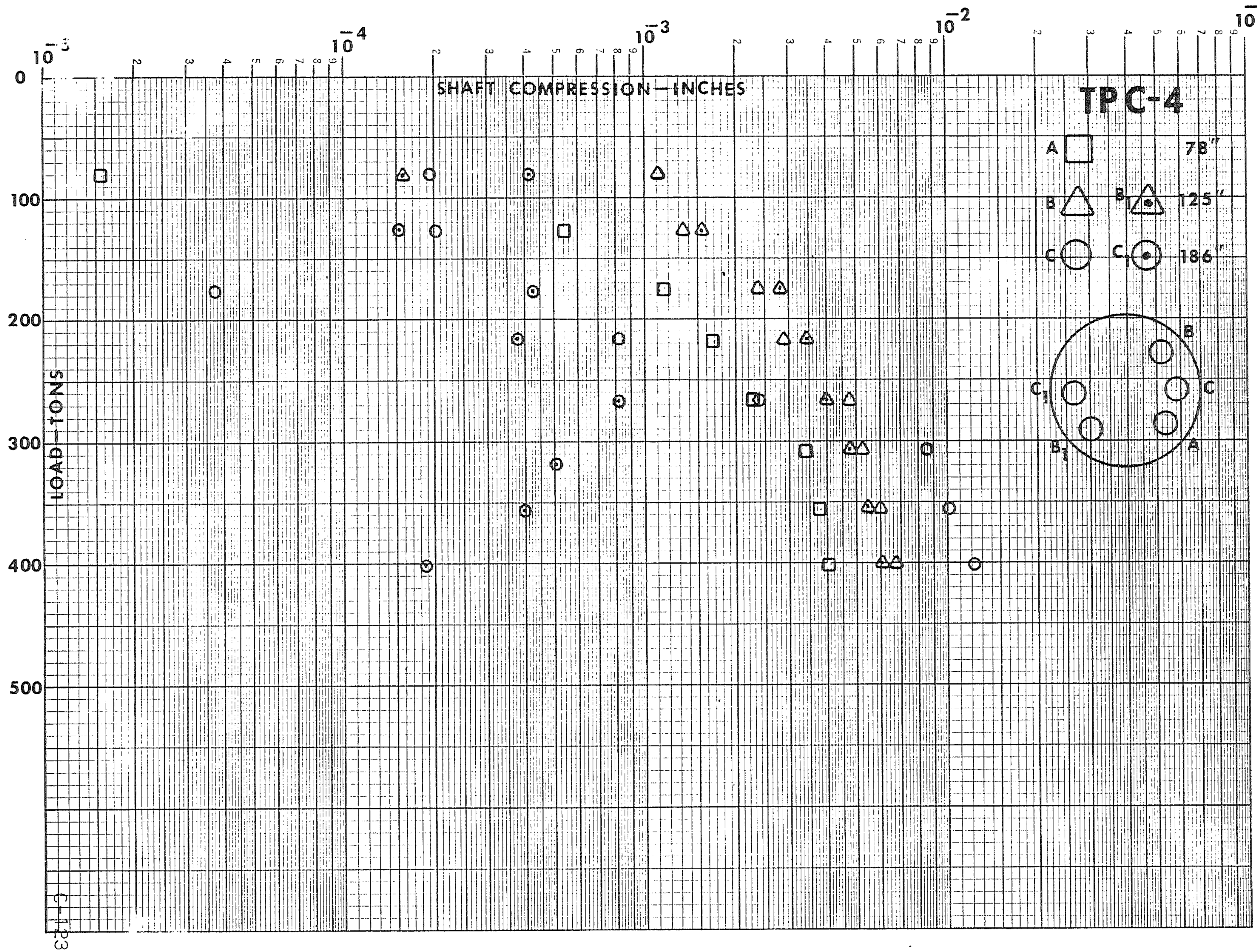

TPB-10
SHAFT COMPRESSION—INCHES
LOAD—TONS
A₁ 85.5″
B B₁ 145.5″
C C₁ 193.5″
C B A₁
C₁ B₁
0
50
100
150
200
250
300
10^{-6}
10^{-5}
10^{-4}
10^{-3}
10^{-2}

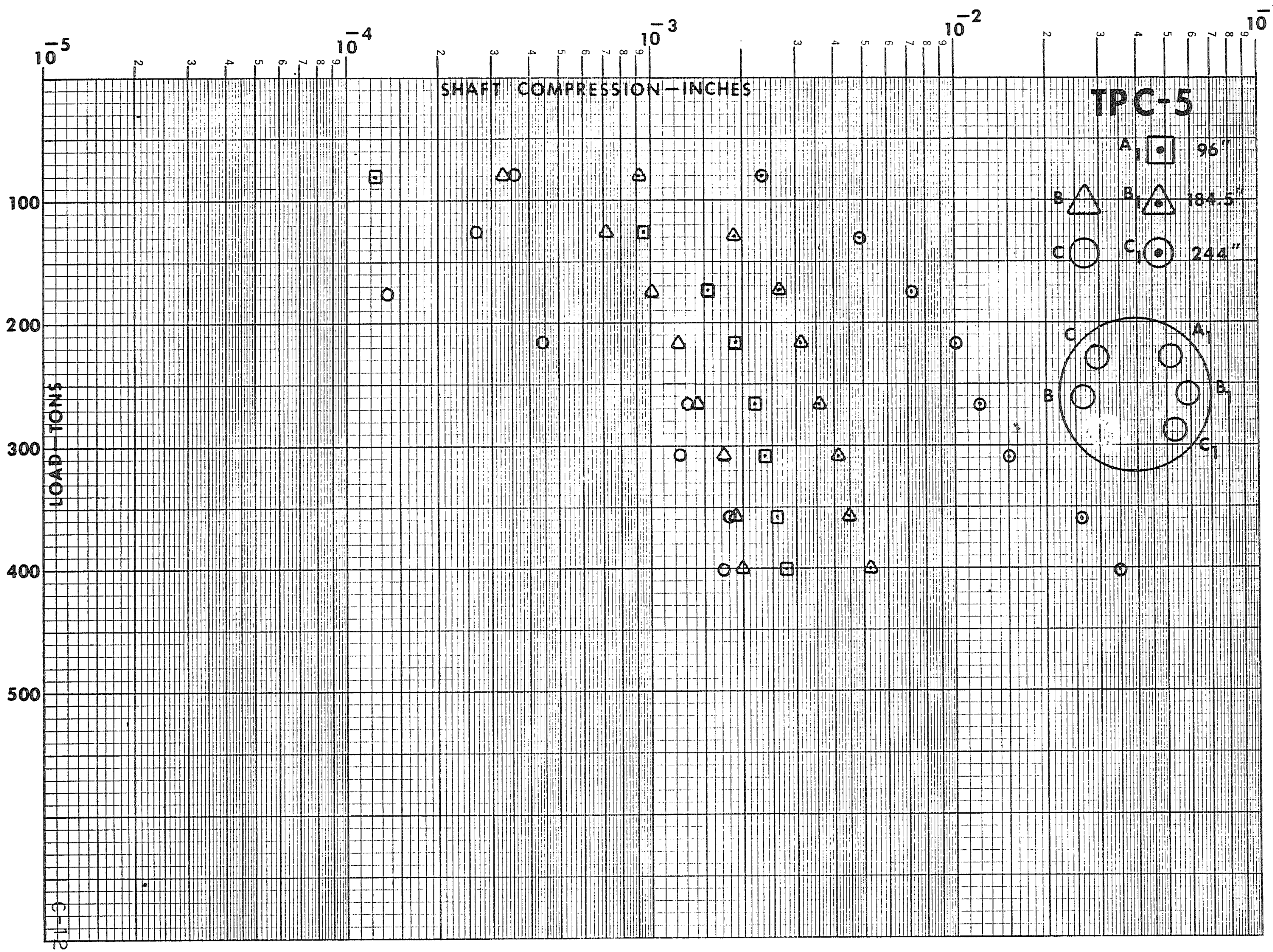


TPC-1
A
A1 96"
B
B1 184"
C
C1 243"
SHAFT COMPRESSION—INCHES
LOAD—TONS
10^{-5}
10^{-4}
10^{-3}
10^{-2}
10^{-1}
0
200
400
600
800
1000

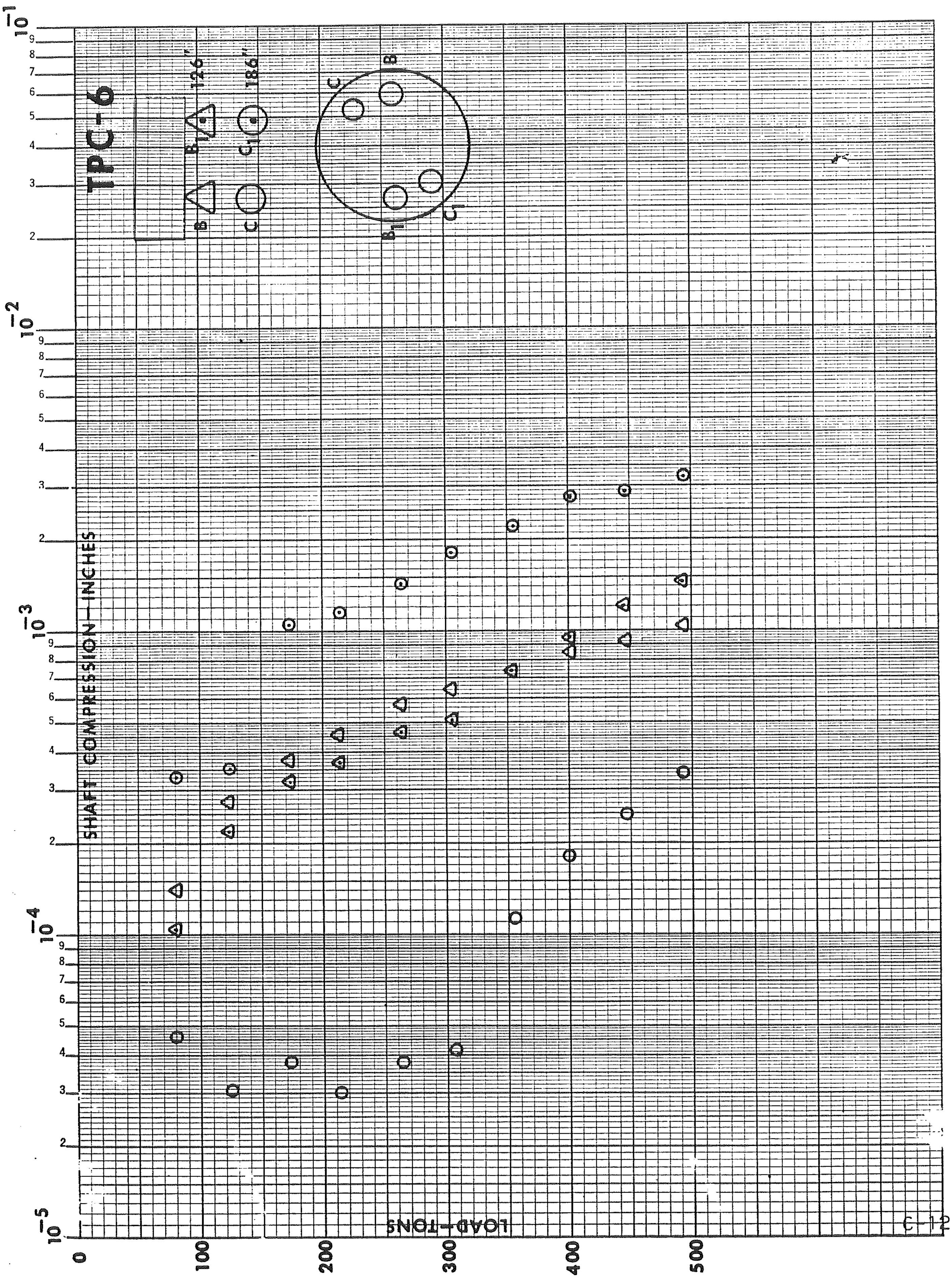

TPC-2
A 78"
B B1 126"
C C1 186"
A B1 C1 B C
SHAFT COMPRESSION—INCHES
LOAD—TONS
10^{-5} 10^{-4} 10^{-3} 10^{-2} 10^{-1}
50 100 200 300 400 500 600 700 750


TPC-3
A
A1
78″
B
B1
126″
C
C1
186″
A
B
C
C1
B1
A1
SHAFT COMPRESSION—INCHES
LOAD—TONS
10^-5
10^-4
10^-3
10^-2
10^-1
0
100
200
300
400
500
600
700

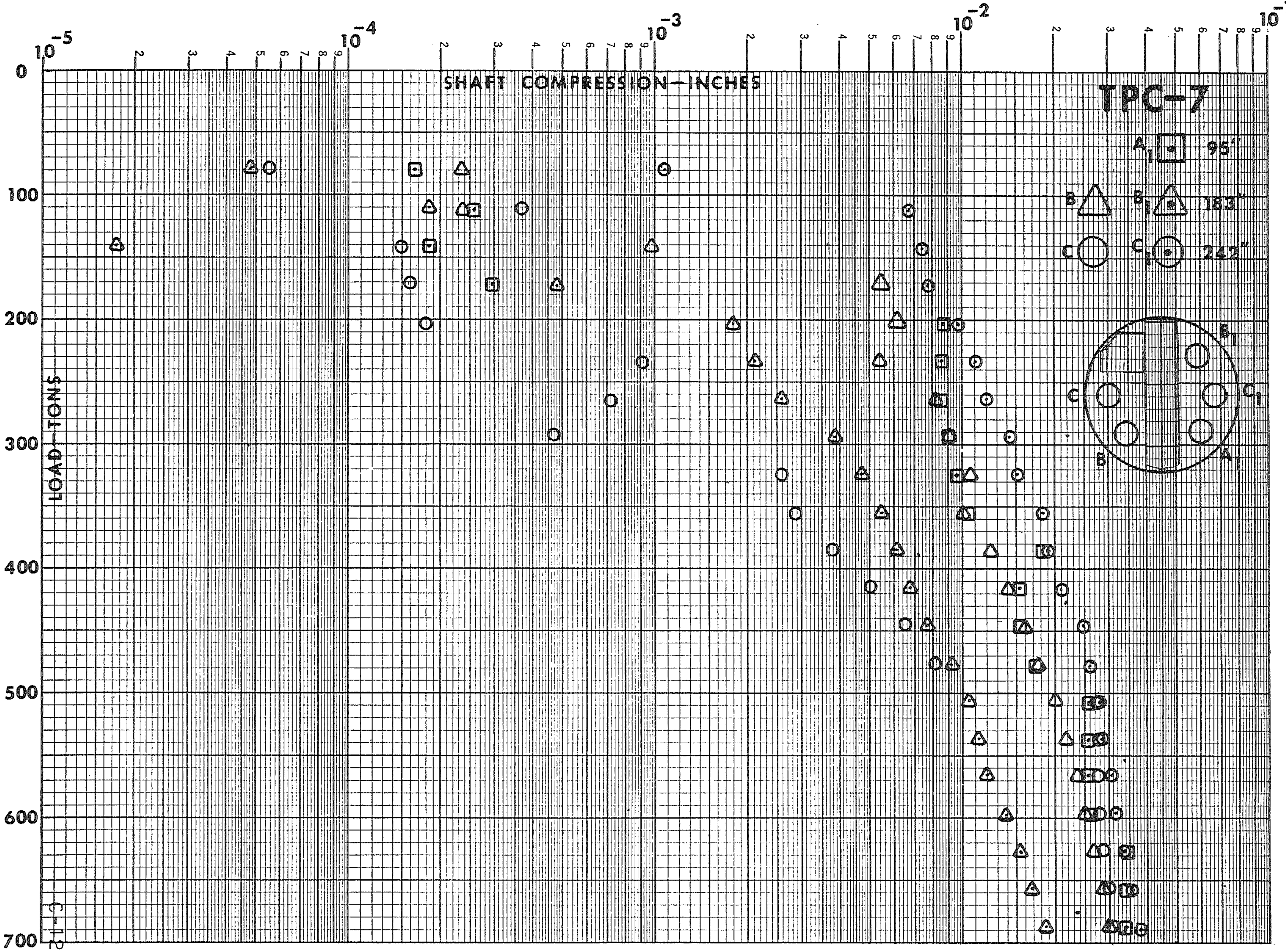

TPC-4
A 78"
B
B_1 125"
C
C_1 186"
B
C
A
C_1
B_1
SHAFT COMPRESSION—INCHES
LOAD—TONS
10^{-5}
10^{-4}
10^{-3}
10^{-2}
10^{-1}
0
100
200
300
400
500

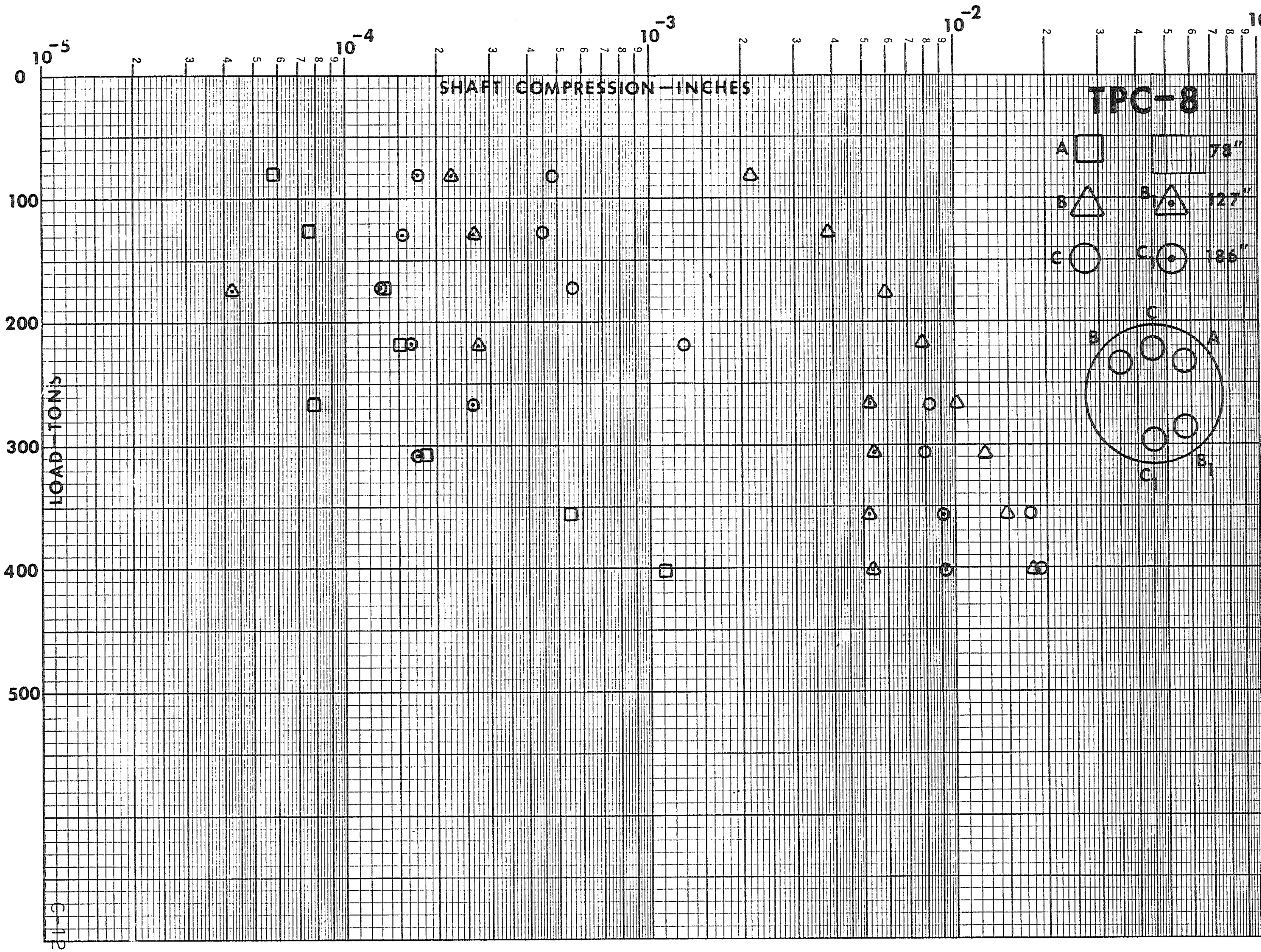


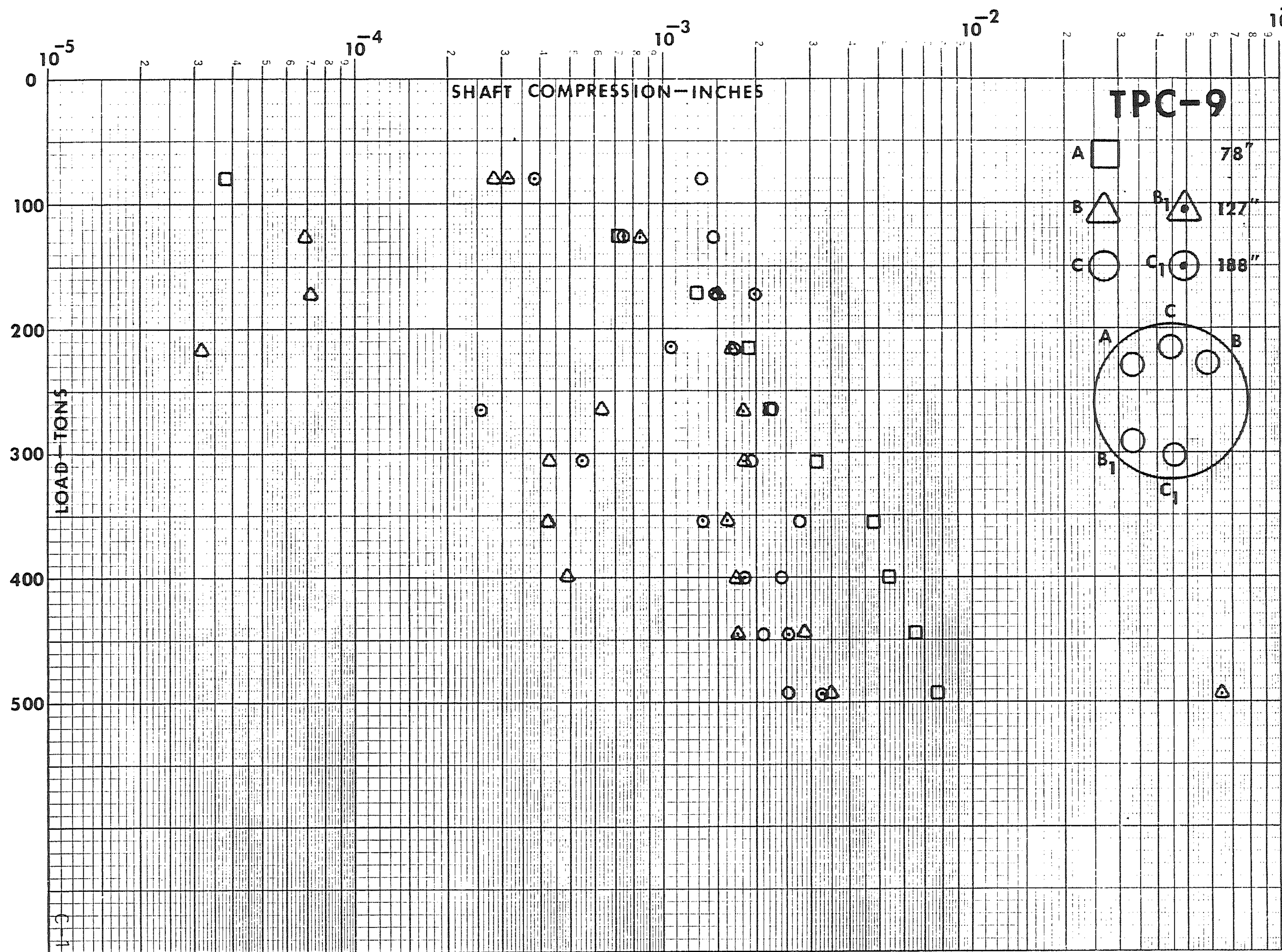
SHAFT COMPRESSION—INCHES
LOAD—TONS
TPC-5
A_1 96″
B B_1 184.5″
C C_1 244″
C A_1 B B_1 C_1
100
200
300
400
500
10^{-5}
10^{-4}
10^{-3}
10^{-2}
10^{-1}

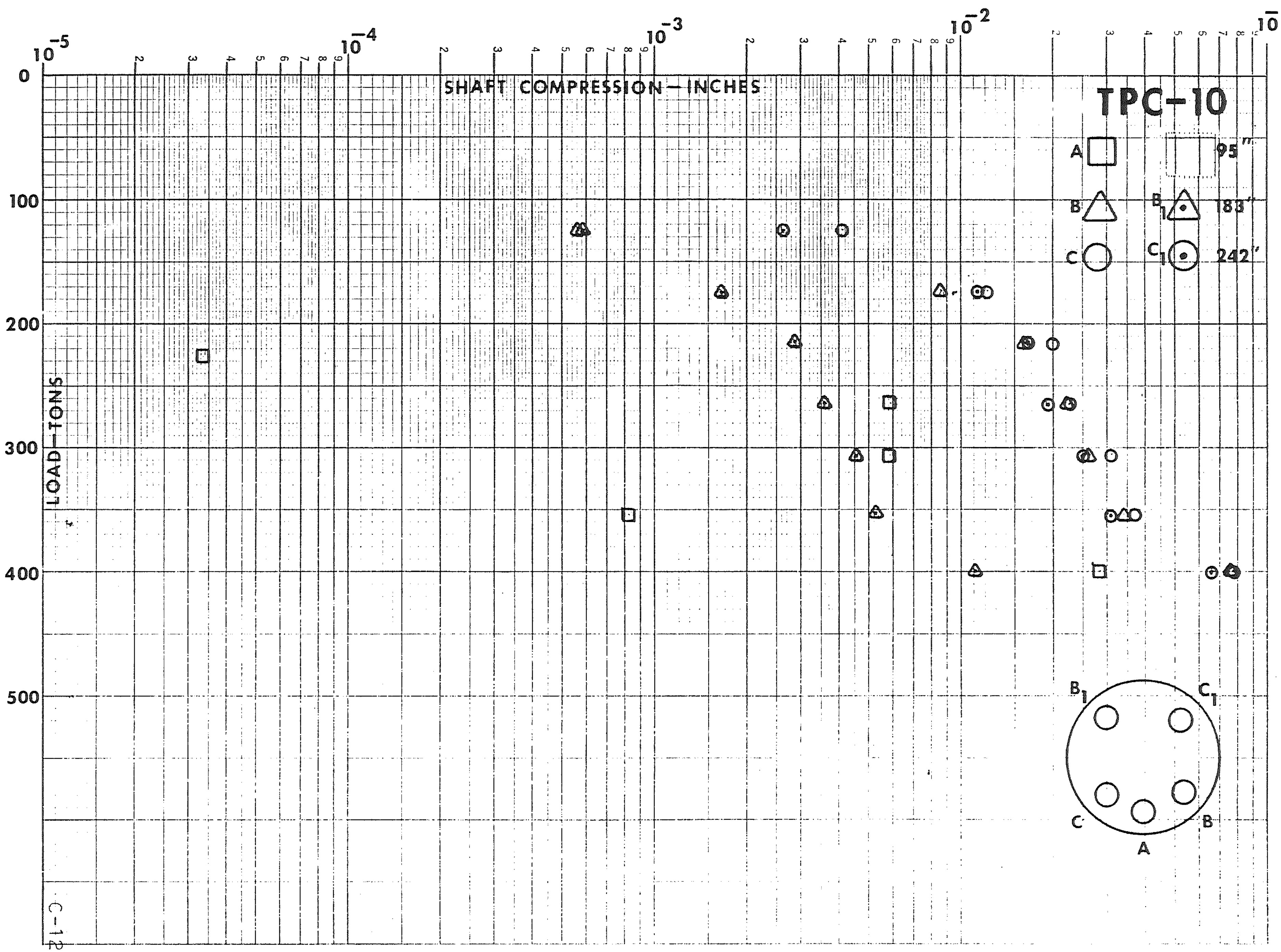


TPC-6
B
B1
126"
C
C1
186"
SHAFT COMPRESSION—INCHES
LOAD—TONS
0
100
200
300
400
500
10^{-1}
10^{-2}
10^{-3}
10^{-4}
10^{-5}


TPC-7
A_1 95"
B B_1 183"
C C_1 242"
SHAFT COMPRESSION—INCHES
LOAD—TONS
10^{-5}
10^{-4}
10^{-3}
10^{-2}
10^{-1}
0
100
200
300
400
500
600
700


TPC-8
SHAFT COMPRESSION—INCHES
LOAD—TONS
A
B
C
B_1
C_1
78″
127″
186″
0
100
200
300
400
500
10^{-5}
10^{-4}
10^{-3}
10^{-2}
10^{-1}


TPC-9
SHAFT COMPRESSION—INCHES
LOAD—TONS
A 78"
B B1 127"
C C1 188"
A B C B1 C1
0
100
200
300
400
500
10^{-5}
10^{-4}
10^{-3}
10^{-2}
10^{-1}


TPC-10
SHAFT COMPRESSION—INCHES
LOAD—TONS
A
B
C
B_1
C_1
95″
183″
242″
0
100
200
300
400
500
10^{-5}
10^{-4}
10^{-3}
10^{-2}
10^{-1}

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PROJECT ______ JOB NO. ______
LOCATION ______
SUBJECT Sample Computations SHEET NO. 1
COMPUTED BY ______ DATE ______ CHECKED BY ______ DATE ______

Example calculations for bearing capacity and settlement calculations for a sample design problem:

Given: A 2000 K load to be supported on a pile with 1/2 inch allowable settlement

Case Example 1

Assume from field or lab tests we have determined an internal friction angle of 35°, a γ of 120 pcf and a cohesion value of 2.0 kips/ft^2

Bearing Capacity

Total bearing capacity of a pile is

$Q_T = Q_S + Q_B$ $Q_S \equiv$ shear load capacity
$Q_B \equiv$ end bearing load cap.

$$Q_T = A_S(Ac + K\gamma z \tan\phi) + A_B(cN_c + D\gamma N_q)$$

$A_S \equiv$ side area of pile
$A \equiv$ cohesion reduction coefficient
$c \equiv$ soil cohesion
$K \equiv$ earth pressure coefficient
$\gamma \equiv$ unit weight of soil
$z \equiv$ depth of center of section desired from ground surface
$\phi \equiv$ angle of internal friction of soil
$D \equiv$ depth of pile base
$N_c, N_q \equiv$ bearing capacity coefficients

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COMPUTED BY ________ DATE ________ CHECKED BY ________ DATE ________

assume a homogeneous, semi-infinite layer

try a pile 2' diameter, 20' long

$A_S = \pi d l = \pi(2)20 = 125.7\ ft^2$

$A_B = \frac{\pi d^2}{4} = 3.14\ ft^2$

$Q_T = 125.7[1(2.0) + 1.0(.120)\frac{20}{2}(0.7002)] + 3.14[2.0(58) + 2(.12)55]$

$= 125.7[2.0 + 0.84] + 3.14[116 + 13.2]$

$= 357 + 406 = 763$ kips

This is the ultimate bearing capacity for a pile 2' ø and 20' long. Since the actual load is 2000 k, try a pile 4' ø by 60' long.

$Q_T = \pi(4)60[1(2.0) + 1.0(.120)\frac{60}{2}0.70] + \frac{\pi 4^2}{4}[2(58) + 4(.12)55]$

$= 754.0[2.0 + 2.52] + 12.6[116 + 26.4]$

$= 3408 + 1794 = 5202$ k for a 2.5 factor of safety the allowable working load would be $5202/2.5 = 2080$ k which is very close to the desired 2000 k load.

Now check settlement for this pile 4' ø x 60'.

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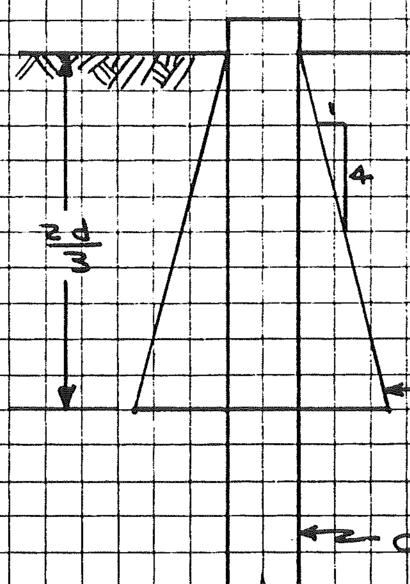
COMPUTED BY DATE CHECKED BY DATE

Use Janbu's method (ref. J.E. Bowles, Foundation Analysis and Design, p 91) for computing the immediate settlement of the pile (disregard any consolidation settlement for this example). Use the "equivalent pier" method as shown below.



$$S = \mu_0 \mu_1 q B \frac{1-\mu^2}{E}$$

Values of μ and E should come from some type of field or lab test, e.g. consolidation, Pressuremeter, or field loading test. If these values are

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COMPUTED BY ______ DATE ______ CHECKED BY ______ DATE ______

not available from lab or field tests, an average value as given in soil mechanics literature may be used to get a rough estimate value.

For this example use $\mu = 0.25$ and $E = 20{,}000$ psi

Now from Bowles, fig 2-21

$L/B = 1$: circle $\quad H/B \geq 10$

$$D/B = \frac{2/3(60)}{4+2(\frac{1}{4})\frac{2}{3}(60)} = \frac{40}{4+20} = \frac{40}{24} = 1.7$$

therefore, $\mu_0 = 0.62 \quad \mu_1 = 0.62$

$$S = \mu_0 \mu_1 q B \frac{1-\mu^2}{E} \quad \frac{lb}{ft^2}\, ft \,\frac{1}{\frac{lb}{in^2}}\, 144 \frac{in^2}{ft^2} \rightarrow ft$$

$$= 0.62(0.62) \frac{2{,}000{,}000}{\pi \frac{24^2}{4}}\, 24 \,\frac{1-0.25^2}{20{,}000(144)}$$

$= 0.0132$ ft.

$= 0.16$ in. Since this settlement is less than the maximum allowable amount of 1/2 inch the 4'ø x 60' long pile meets the design requirements.

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SUBJECT Sample computations SHEET NO. 5

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For a check on the above settlement analysis, use the Poulis and Davis method as given in Geotechnique, Vol XVIII, No. 3, Sep 68, p 351

Using fig 7 for a smooth pile with $d/L = 4/60 = .07$ $I_p = 0.11$

$$\rho = \frac{P}{dE} I_p$$

$$\rho = \frac{2,000,000}{4(20,000)144} 0.11 = 0.019 \text{ ft}$$

$= 0.23$ in close to the 0.16 in from the equivalent pier method above. Therefore, the dimensions 4' ø by 60' long are O.K.

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LOCATION ______

SUBJECT Sample computations SHEET NO. 6

COMPUTED BY ______ DATE ______ CHECKED BY ______ DATE ______

Case Example 2 - Using the design curves - fig 78

Load 2000 kips

as in the above example, for a 4'ø x 60' pile we calculated an allowable load of 2080 k

$$q \text{ ksf} = \frac{2000}{\pi \frac{24^2}{4}}$$ using the base area from the equivalent pier method

$q = 4.42$ ksf

enter figure 78 w/ width of loaded area = 4' and $q = 4.42$ ksf

get $\delta \approx 0.05$ inches

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PROJECT Pile Load Study JOB NO.

LOCATION

SUBJECT Typical Settlement Calculations SHEET NO. 1

COMPUTED BY L.T DATE 5/14 CHECKED BY DATE

Poulis & Davis Method

TPB-6 - E from Pressuremeter

$S = \frac{Q}{DE} I_w$

I_w from Geotechnique
Vol XVIII No. 3 Sep '68
Figure 7 p 358

$I_w = 0.22$ w/ $h/L = 0.19$

$S = \frac{303{,}000 \text{ lb.}}{3.02 \text{ ft } 6000 \text{ psi } (144 \frac{\text{in}^2}{\text{ft}^2})} 0.22$

$= 0.0255$ ft.

$= 0.31$ in.

change value of E →
E from consolidation

$S = \frac{303000}{3.02 (9400) 144} 0.22$

$= 0.016$ ft

$= 0.20$ in.

Equivalent Pier Method
- Pressuremeter E - average
TPB-6

$d = 16.16$ ft $\quad \frac{2d}{3} = 10.77$

$\frac{1}{4}(\frac{2d}{3}) = 2.69 \times 2 = 5.38$

$+ \; 3.02$

$8.40' = B$

PROJECT Pile Load Study JOB NO.

LOCATION

SUBJECT Typical Settlement Calculations SHEET NO. 2

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Equivalent Pier Method (cont)

From Bowles, Foundation Analysis and Design, Fig 2-21, p 92

$H/B \geq 5$, L/B = circle $\therefore \mu_1 = 0.70$

$D/B = 16.16/8.4 = 1.9$, $L/B = 1$ $\therefore \mu_0 = 0.62$

$s = \mu_0 \mu_1 q B \dfrac{1-\mu^2}{E_s}$ $E_s = 6000$ psi = 864,000 psf

$s = 0.62(0.70) \dfrac{303{,}000}{\frac{\pi}{4}(8.4)^2} 8.4 \dfrac{1-0.4^2}{864{,}000}$

= 0.019 ft

= 0.23 in.

Equivalent Pier Method

Pressuremeter - E - max

same q, B, μ_0, μ_1, μ as above

$s = \mu_0 \mu_1 q B \dfrac{1-\mu^2}{E} = 0.62(0.70) \dfrac{303{,}000}{\frac{\pi}{4}(8.4)^2} 8.4 \dfrac{1-0.4^2}{\cancel{864{,}000}\ 1353600}$

= 0.012 ft

= 0.15 in.

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Equiv. Pier Meth (cont)

Consolidation E - ave 1st cycle
same μ, μ_0, μ_1, q, B as above

$$s = \mu_0 \mu_1 q B \frac{1-\mu^2}{E} = 0.62(0.70) \frac{303000}{\frac{\pi}{4}(8.4)^2} 8.4 \frac{1-0.4^2}{3,100,400}$$

$= 0.0053$ ft
$= 0.063$ in.

Consolidation E - ave 2nd cycle
same μ, μ_0, μ_1, q, B as above

$$s = \mu_0 \mu_1 q B \frac{1-\mu^2}{E} = 0.62(0.70) \frac{303,000}{\frac{\pi}{4}(8.4)^2} 8.4 \frac{1-0.4^2}{\cancel{3,110,400}\ 1,959,400}$$

$= 0.0083$ ft
$= 0.1005$ in

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PROJECT Pile Load Study JOB NO.

LOCATION

SUBJECT Bearing Capacity Calcs SHEET NO. 4

COMPUTED BY L.T. DATE 5/14 CHECKED BY DATE

For TPB-6 - using values as given in Pile Load Study Report

Method 1

$Q_T = A_B q_B + A_s q_s$

$= A_B(cN_c + D\gamma N_q) + A_s(AC + K\gamma z \tan\phi)$

$= 7.16[1.9(48) + 16.2(.125)41] + 153.6[1.(1.9) + 1.(.125)\frac{16.2}{2}(.65)]$

$= 1631$ kips

Method 2

$Q_T = A_B q_B + A_s q_s$

$A_B q_B \rightarrow$ same as method 1

q_s from upper limit of direct shear

$Q_T = 1247 + 3.2(153.6) = 1739$ kips

Method 3

$Q_T = Q_s + 10 s A_B = 384 + 10(3.1)7.16$

$= 384 + 222 = 606$ kips

Q_s : same as method 1

s : from average of direct shear test

PROJECT Pile Load Study JOB NO.
LOCATION
SUBJECT Bearing Capacity Calcs SHEET NO. 5
COMPUTED BY DATE CHECKED BY DATE

Method 4

$Q_T = Q_s + 10\,s\,A_B$

Q_s: same as method two

s: from upper limit of direct shear

$Q_T = 492 + 10(3.8)7.16 = 492 + 272$

$= 764^{k}$

Method 5

$Q_T = Q_s + Q_B = s_o A_s + 1.4\,P_L\,A_B$

s_o: shearing stress from Pressuremeter

P_L: Limit pressure from Pressuremeter

$Q_T = 2.23(153.6) + 1.4(14.74)7.16$

$= 343 + 148$

$= 491^{k}$

Method 6

$$Q_T = A_s S_o + A_B P_L$$
$$= 2.23(153.6) + 7.16(14.74) = 343 + 106$$
$$= 449^{K}$$

Method 7

$$Q_T = A_s Q_s + A_B Q_B \qquad Q_s = N/10 \text{ Ksf} \leq 5.0 \text{ ksf}$$
$$Q_B = N \text{ Ksf} \leq 75 \text{ ksf}$$

$$Q_T = 153.6\left(\frac{15}{10}\right) + 7.16(15) = 230 + 107 = 337^{K}$$

Method 8

$$Q_T = A_s \frac{q_u}{2} + A_B(4.5\, q_u)$$
$$= 153.6\left(\frac{4.0}{2}\right) + 7.16(4.5)(4.0)$$
$$= 307 + 129 = 436^{K}$$

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PROJECT Pile Load study JOB NO.

LOCATION

SUBJECT Typical SHB Calcs. SHEET NO. 7

COMPUTED BY DATE CHECKED BY DATE

Settlement for TPA-3 - SGC Deposit

TPA-3

depth 16.16' base diam = 54.8"

Using Janbu's method for calculating settlement, J.E. Bowles, Foundation Analysis and Design, p 91, 92

$$\delta = \mu_0 \mu_1 q B \frac{1-\mu^2}{E}$$

$L/B = 1$ $H/B = 5$ $\frac{D}{B} = \frac{16}{4.5} = 3.5$

$\therefore \mu_1 = 0.75$ $\mu_0 = 0.58$

$\mu = 0.30$

For $q = 30$ ksf

$E = 20{,}000$ psi : Bowles p90 - dense sand and gravel

$$\delta = 0.75(0.58)30{,}000(5)\frac{1-0.30^2}{20{,}000(144\ \frac{in^2}{ft^2})}$$

$= 0.0206$ ft

$= 0.25$ in This compares well with the 'average' value given in Fig 77 i.e. 0.26 in.