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STRENGTH AND SETTLEMENT CHARACTERISTICS

OF A COMPACTED RESIDUAL SOIL

A THESIS

Presented to

the Faculty of the Graduate Division

by

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STRENGTH AND SETTLEMENT CHARACTERISTICS

OF A COMPACTED RESIDUAL SOIL

Approved:

George F. Sowers

A. B. Vesic

F. M. Hill

Date Approved by Chairman: June 5, 1961

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TABLE OF CONTENTS

		Page
ACKNOWLEDGEMENTS		ii
LIST OF ILLUSTRATIONS		iv
SUMMARY		v
Chapter		
I.	INTRODUCTION	1
	Fill Construction Problems Purpose and Objectives Geological Origin of Test Soils	
II.	EXPERIMENTAL PROCEDURES	7
	Classification Tests Full Scale Field Tests Laboratory Tests - Undisturbed Fill Samples	
III.	DISCUSSION OF TEST RESULTS	19
	Plate Load Tests Test Boring Data In Place Settlement Data Laboratory Tests Comparison of Field and Laboratory Data Compaction Characteristics	
IV.	CONCLUSIONS	45
V.	RECOMMENDATIONS	47
APPENDIX		49
	Illustrations	50
BIBLIOGRAPHY		72

LIST OF ILLUSTRATIONS

Figure No. Page

0. General Plan of Test Area 50
1. Classification Test Data 51
1A. Supplementary Laboratory Test Data 52
2. Laboratory Compaction Tests 53
3A. Photograph of Small Load Test Setup 54
3B. Photograph of Large Plate Load Test Setup 55
4. Plate Load Test - (8 Inch Diameter) 56
4A. Ultimate Bearing Capacity vs Per Cent Compaction .. 57
5. Plate Load Tests (7 S.F. Area) 58
5A. Limiting Settlement vs Per Cent Compaction 59
5B. Time vs Settlement Curves 60
6. Test Boring Data - Test Section 1 61
6A. Test Boring Data - Test Section 2 62
6B. Test Boring Data - Test Section 3 63
7. Inplace Settlement Plates - Section 1 64
7A. Inplace Settlement Plates - Section 2 65
7B. Inplace Settlement Plates - Section 3 66
8. Triaxial Shear Data - Test Section 1 67
8A. Triaxial Shear Data - Test Section 2 68
8B. Triaxial Shear Data - Test Section 3 69
8C. Compaction vs Strength Characteristics 70
9. Consolidation Test Data 71

SUMMARY

The purpose of this investigation was to determine the variations in the strength and settlement characteristics of a compacted residual soil with variations in its density. This objective was attained by actual field measurements with large scale load tests and laboratory tests performed on undisturbed samples of the compacted soil. This investigation was needed to assist in the development of a practical means of structurally evaluating an artificially compacted soil mass.

The test soil is a highly micaceous silt found in the Atlanta, Georgia area. The soil is more specifically classified as a brown-grey highly micaceous silty well graded sand or well graded sandy silt. The soil has low plasticity and is classified as ML by the United system or A-4 by the revised U. S. Bureau of Public Roads system.

The test results show that the bearing capacity of the remolded soil increases with an increase in dry density. The theoretical bearing capacity was consistent with the actual bearing capacity (as determined by eight inch diameter plate load tests), if the lower cohesion value (c") was used in computing bearing capacities. The bearing capacity factors determined by Terzaghi for insensitive soils were used.

The actual settlements, as measured by long termed load tests, were consistently one-half to three-fourths of the computed total settlements. The theoretical settlements were determined as consolidation settlement based on Westergaard's average stress distribution beneath a rectangular foundation. The actual settlement recorded in tests conducted on fill compacted to ninety-five per cent compaction (ASTM D-698) was one-fourth of the settlement recorded by tests conducted on similar fill compacted to eighty-five per cent compaction.

There is an abrupt change in the strength and settlement characteristics of the compacted soil between ninety and ninety-five per cent compaction. Approximately twice the compactive effort required to attain ninety per cent compaction is required to reach ninety-five per cent compaction.

The inherent springy nature of the test soil caused difficulties with laboratory tests. Undisturbed samples invariably expanded with lack of confinement especially in the less dense samples.

Further research should be conducted using different load test plate sizes and considering a surcharge load. Sufficient time should be allowed for complete consolidation settlement. Since the standard Proctor compactive effort is relatively low, further studies should include soils compacted to at least 100 per cent compaction.

The standard penetration test provides a reasonably accurate method of measuring in-place densities of a partially saturated fill soil mass.

CHAPTER I

INTRODUCTION

Fill Construction Problems

In recent years engineers have come to realize the importance of structural foundations in regard to providing a more practical and economical building. Along with the increase in attention to foundations has developed a need for a better understanding of the various media on which most structures rely for support. In those areas where hard, bed rocks exist within a few feet of the ground surface, foundation support is no problem. However, such ideal conditions are not common. While it is normal to support light to moderately heavy structures with shallow footings on virgin soil, many desirable building sites must be cleared, graded and often filled in with additional material to produce level ground. In such cases the designer is confronted with the problem of deciding if shallow spread foundations on fill may be safely used to support the building. To compound the problem, portions of the building area often rest directly on hard virgin soil while other portions are underlain with filled material. If the added fill soil was placed for the purpose of supporting a structure, the quality of the soil and the limits of the filled area are known. However, if the site has been filled over a period of time by unknown methods using any rubbish, debris, and uncompacted soil, the foundation problem is compounded.

Why then is it advisable to explore the possibility of utilizing a questionable soil mass such as a fill for structural support? Why not excavate all foundations to the virgin soil below the fill or abandon the site? The answers to these questions are many fold. First, there are economic considerations. As new businesses, industries and residences engulf a populous area, the better sites are used first. Subsequent expansion and added growth continues to demand acceptable property for new buildings. The maximum utilization of an area prompts the development of sites that were initially considered poor, undesirable locations for building construction. Many of these areas require extensive grading and additional fill material.

A second economical reason for exploring the use of foundations on fill involves the actual cost of the foundation. If foundations extend through all fill to the virgin soil, costly excavations, bracing of dug pits and added building materials are involved. If the fill depth exceeds eight feet to ten feet, some type of expensive drilled pier or pile will probably be necessary to reach the desired foundation material. Even deeper foundations will be needed if the exposed and covered virgin soils are weak or compressible. Footings placed on fill can sometimes be used to bridge over soft virgin soils to avoid costly deep foundations. Foundations supported on filled material often reduce building costs if ground water exists near the original ground surface.

Many structures supported by foundations on virgin soil or bedrock have integral sections or supplementary parts that are supported separate from the main structure. For example, many buildings have ground floor slabs that rest above the original ground level outside of the building. The floor must be structurally supported with costly beams, columns and walls or on fill placed inside the building. Filled material is also often required outside of buildings for pavements and ramps.

In addition to building construction, earth embankments are used for dams, bridge approaches and road way construction. These and other methods of using a remolded soil mass emphasize the importance of this portion of foundation engineering.

Purpose and Objectives

The purpose of this thesis is to determine the variations in the strength and settlement characteristics of a compacted residual soil with variations in its density.

This objective was attained by actual field measurements with full size tests and laboratory tests performed using undisturbed samples of the compacted soil. The residual soil used during this investigation is a highly micaceous sandy silt found in the Atlanta, Georgia area.

Geological origin of test soils.--A knowledge of the geological origin of the selected test soil contributes greatly to the understanding of its composition and physical characteristics. Atlanta and the surrounding area is located in the Piedmont region. Geographically, the Piedmont is a narrow undulating plateau that extends from east central Alabama through most of the northern half of Georgia and northward through central North Carolina and Virginia to southern Pennsylvania. The soils of the Piedmont in Georgia are primarily red-brown clayey sandy silts, brown and grey silty sands and sandy silts that have formed as a result of continuous inplace weathering of the underlying crystalline rock. The soils in some places may be only a few feet deep while in others they are over one hundred feet in depth. These residual soils gradually become stiffer with an increase in depth until a transition from stiff soil into hard rock or soft decomposed rock occurs. These stiff to hard residual soils often retain many of the physical characteristics of the original crystalline rock such as laminations, banding, and coloration.

The parent bedrocks of the residual soils are primarily metamorphosed igneous rocks that are very old geologically. These rocks have been altered over a long period of time under the combined effect of heat and pressure. This action along with the effects of different rates of rock cooling cause many of the minerals to segregate and to form dark bands or streaks of the different minerals in the rock structure. This is a distinct characteristic of the gneiss and schist rocks that underlie the Atlanta area.

Considerable variation in composition exists between the various virgin soil horizons of the Georgia Piedmont soils. The surface soils being the most weathered, are usually red-brown (oxidized) sandy clayey silts or sandy silty clays. The thickness of this surface layer usually does not exceed six feet. The second soil horizon is a sandy silt or silty sand that contains variations in mica content. The presence of biotite mica indicates the changes in weathering and variations in mineral content. High mica content indicates rapid oxidation in the presence of iron, aluminum and acids that have been accumulated through the downward leaching of these minerals. A third or intermediate horizon normally exists in most areas. This deeper soil may be generally described as a hard soil or partially decomposed rock that forms the transition from soil to hard crystalline rock.

The micaceous sandy silts and silty sands of the second soil horizon constitute the predominant soil. This soil study was conducted using a highly micaceous sandy silt of this zone. It is classified as a brown-grey highly micaceous silty well graded sand of low plasticity. It is classified as ML by the United system or A-4 by the revised U. S. Bureau Of Public Roads system.

CHAPTER II

EXPERIMENTAL PROCEDURES

Classification Tests

Based on the soil profile developed from a series of auger borings, the field test site was selected in the Atlanta, Georgia area. Representative composite samples of the micaceous silty sands were obtained for classification testing. Standard grain size and plasticity tests (Atterberg Limits) were conducted on four separate portions. The gradation limits are given on an included chart (Figure 1). The curves are similar with the maximum grain size variation below the U. S. standard sieve No. 60 (fine sand and silt sizes). These variations, expressed in per cent passing the finer sieves, are 15 per cent, 16 per cent, and 18 per cent for the standard 60, 100 and 200 sieves, respectively. In each case, however, smooth curves are indicated throughout the gradation limits.

The standard Atterberg Limits tests also show some variations. Three of the test samples were essentially non-plastic while the fourth was of low plasticity. These plasticity test data are tabulated on Figure 1A.

Major differences in the test soil gradation are caused by variations in the mica content and mica flake size. Each of five soil samples were tested for total mica content (by weight). Three determinations were made by normal visual and washing procedures. Two tests were conducted using a more exact method. This procedure involves the separation of minerals according to their specific gravity in a Bromoform (CH BR 3) solution. The mica flakes remain in suspension while the lighter minerals (Silica and Feldspar) float to the top. Any heavy minerals or impurities settle out of the solution. All of the tests indicate high but variable mica content. The percentages of mica (by weight) vary from 35 per cent to 54 per cent. A high per cent of the coarse particles (shown on the grain size chart -- Figure 1) is mica flakes.

A number of representative samples were tested to determine the soils compaction characteristics. All compaction tests were made as specified by the standard Proctor test (ASTM D-698-58T). The tests show some variations that may be attributed to variations in soil fines (-200 sieve) and mica content. The average of these moisture-density determinations is presented in Figure 2 along with each separate curve. An average maximum dry density of 94.5 PCF was determined with an optimum moisture of 23 per cent.

Full Scale Field Tests

A large quantity of the test soil was excavated from the virgin condition and stock piled for use in the fill structure. A pit approximately forty feet long, twelve feet wide, and eight feet in depth was excavated using conventional earth moving equipment. The top two feet of original earth was separated and wasted. (This material was a red brown sandy silty clay not to be used in the test series.) The dug pit was then covered by a large tent to protect it from inclement weather. The stock pile of loose soil was also covered. The pit was divided into three sections of equal volume thus providing sections twelve feet wide, approximately twelve feet long and eight feet deep. The stock piled soil was then manually replaced into each of the three test sections using construction and compaction procedures that produced soil panels six feet in thickness, one each at densities equivalent to 85 per cent, 90 per cent and 95 per cent of the Standard Proctor maximum dry density.

Prior to initiating actual construction of the soil panels, compaction procedures for each panel were trial tested. Two compactors were used. A small vibratory compactor (Jay tamp) was used to initially tighten the loose soil. This compaction device employs a fourteen inch square steel plate for compaction. The steel plate is actuated by an eccentric wheel powered by a small gasoline engine. During operation the fifty pound weight of the machine rests on the steel vibrating plate or partially on two rubber wheels. A small test area four feet square was used to check the compaction effectiveness of the Jay tamp. Soil layers (four inches compacted thickness) were densified to approximately 80 per cent compaction with one complete pass of the tamper. Three passes produced 85 per cent compaction but no appreciable increase in compaction above 85 per cent could be obtained until five or more passes were used. It was decided that the Jay tamper would be used exclusively in construction of Section 1 (85 per cent compaction). The Jay tamp was also used in the other sections to smooth and tighten the loose soil prior to using the Barco Rammer.

Other small test sections were compacted using the Barco Rammer compaction machine. This dynamic soil compactor employs a single stroke gasoline engine actuated by a magneto. With each manual stroke the 200 pound machine is lifted approximately twelve inches by a gasoline driven piston. A dynamic energy of approximately 200 foot-pounds per blow is thus applied to the soil surface over an area 9.5 inches in diameter. Through various trials it was found that soil layers of six inch compacted thickness could be tamped to 90 per cent compaction with two complete passes, and to 95 per cent compaction if four complete passes were used. Each impact blow overlapped the preceding blow approximately twenty per cent. If the soil moisture content exceeded the optimum moisture by more than two per cent, additional passes were needed to gain the required density

The compactive effort used in the three fill sections are as follows:

Section	Required Compaction	Required Dry Density	Jay Tamp	Barco Rammer	Layer Compacted Thickness
1	85 %	80.3 PCF	3 passes	None	4 inches
2	90 %	85.1 PCF	1 pass	2 passes	6 inches
3	95 %	89.8 PCF	1 pass	4 passes	6 inches

The virgin soil surface at the bottom of each test pit was eight feet below grade. Prior to initiating any filling a single settlement test plate was set on the virgin soil at the bottom of each test pit. Each test plate consisted of a one foot square one-fourth inch thick steel plate to which one-half inch diameter steel rods were welded in the vertical position at the center of each plate. Each rod was seven feet long and thus extended one foot above the top of the six feet of compacted fill. Each rod was loose fitted with a steel pipe sleeve throughout the entire rod length with the exception of the last four inches at the bottom. This sleeve prevented frictional drag of the fill soil against the steel rod which was used to measure only the plate movement.

Settlement plates were also placed on the in place fill in each section after three feet of fill was in place and at the top of the fill (six foot thickness).

Periodically throughout fill construction and later during the field tests, settlements of each of the nine plates were observed using a standard elevation rod and engineers level. The elevations were referenced to a bench mark located twenty-five feet or more away from the fill sections.

To insure that each soil layer was compacted to the desired density, a number of in place density tests were taken on each six inch layer. The compacted soil was sampled using a four inch diameter thin walled steel cylinder having a volume of exactly one-thirtieth cubic foot. During testing the steel tube was driven into the fill using an adaptor and sledge hammer. The soil sample was weighed in the field for immediate density determinations. The required beam balances, pans, hot plate and other necessary items were set up in a field laboratory to allow complete density testing at the site.

Two series of plate load tests were conducted to determine the strength and settlement characteristics of each of the three compacted fill conditions. The first test series was performed using one inch thick, eight inch diameter steel plates. Three tests were conducted in each fill section at different locations over the fill surface. The round test plate was placed on the smooth horizontal fill surface after a thin layer of standard uniform Ottawa sand was spread over the soil surface to fill any small holes or voids that could exist between the test plate and soil surface. A calibrated hydraulic jack reacting against the bottom chord of an aluminum truss provided the test load reaction. The truss was anchored with a series of earth anchors at each end of the truss. Plate movements were measured with two micrometer dial gages mounted on a beam supported independently of the truss or loading system. The dial gages measured plate movements of .0001 inch. This system of load reaction and plate settlement measurements was used throughout all of the plate load tests.

Each of the three round plate tests were made around the center of the fill sections but more than two feet from the edge of the fill mass. During each of the eight inch diameter plate tests the soil was subjected to pressures which were increased in increments of approximately 740 PSF (250 pounds total load) until the soil sheared. The settlement under each increment of load is shown on the included load versus settlement curve (Figure 4) for each test. A photograph of the load test plate is shown on Figure 3A.

The second load test series was performed near the center of each fill section using a square steel plate having an area of seven square feet. To prevent plate bending during loading, a series of smaller plates were used to stiffen the test plate and thus provide uniform pressure distribution. The load was applied in 500 PSF increments until a total load of 24,500 PSF was reached. This load was the maximum capacity of the loading system. Each increment of load was maintained until plate movement was less than .001 inches but for a minimum of twelve hours. Time increments of forty-eight hours were required for some load increments. This procedure was used to allow complete contact settlement to occur. The test results are shown on the included pressure versus settlement curves (Figure 5) for each test. A photograph of the load test plate setup is shown on page fifty five.

After all load tests were completed additional field tests were conducted. Two soil test borings were made into each fill mass at locations where the fill was undisturbed. Continuous split spoon samples were taken throughout the fill depth using methods specified by ASTM Specification D-1586-58T. Cylindrical spoon samples (1.5 inch diameter) were secured by driving the 2.0 inch O.D. split spoon sampler into the soil with blows from a 140 pound hammer falling thirty inches. The sampler was initially seated six inches into the undisturbed fill and then driven an additional one foot. The number of hammer blows required to penetrate the soil the last twelve inches was recorded and is designated the "Penetration Resistance". The penetration resistances for each foot of fill depth is shown on the included test boring records (Figure 6) for each fill section.

A second test boring was made in each fill area using a smaller penetrometer. At intervals of one foot of fill depth a two inch diameter steel pointed cone was inserted in the test hole and seated 1.5 inches into the undisturbed fill soil. The soils penetration resistance was again measured by the number of blows of a fifteen pound hammer with a free fall of twenty inches required to drive the steel cone 1.75 inches. The results are plotted on Figure 6 along with the other boring data from the same fill section.

A series of undisturbed samples of the fill in each section were taken for laboratory testing. The samples were secured by forcing a four inch diameter thin walled, seamless steel tubing into the undisturbed fill. The sampling tube was advanced by slow movement into the fill mass under pressure from a hydraulic jack which reacted against the load test truss. Each fill soil sample, still encased in the tubing, was carefully removed from the hole and sealed on each end with parafin. Each sample was transported to the laboratory for testing.

Samples of the spoon samples obtained in each test boring were also retained and later tested to determine the in place moisture content of the fill at the various depths.

Throughout the entire field testing operations, the elevations of each settlement plate were observed. The total time period of these recordings vary from four months for fill Section 1 to only one month for Section 3. The plate movements are recorded on Figure 7.

Laboratory tests - undisturbed fill samples:--The majority of all laboratory tests on the undisturbed soil samples were conducted in the central soil testing laboratory of the Law Engineering Testing Company located in Atlanta, Georgia. The tests were performed by trained technicians under the direct supervision of the writer. A few tests were conducted by the writer in the soil laboratory at the Georgia Institute of Technology.

The strength characteristics of the compacted soils at each density condition were determined by the triaxial shear test. Each undisturbed soil sample, still in its steel tube, was cut into six inch long sections on a high-speed abrasive saw. Each section was weighed and portions were tested for moisture content. From these data the soil void ratio, wet weight and dry weight were computed. Other portions were tested to determine the specific gravity of the soil particles.

Three sections of each undisturbed sample were extracted from the tubes for triaxial shear tests. Each was trimmed into cylinders approximately two inches in diameter and four inches long. Each soil cylinder was then encased in a thin rubber membrane and placed inside a compression chamber. Each of the three prepared samples was tested at different confining air pressures. The first sample was tested at zero confining pressure. The other two samples were tested at confining pressures of 2000 PSF and 4000 PSF, respectively. The axial load on each test sample was increased in small increments until the soil failed in shear. The test results are presented in the form of stress-strain curves and Mohr diagrams. The other supplementary test data for each sample are also shown on Figure 8.

Portions of each undisturbed sample were extruded from sampling tubes for consolidation testing. These tests were performed to determine the settlement characteristics of the fill at the three density conditions. Each soil specimen was cut into a disc 2.4 inches in diameter and one inch thick. This soil disc was then placed in a stainless steel ring between two porus rigid plates. The prepared sample was then subjected to incrementally increasing vertical loads. The vertical deformation of the soil disc was accurately measured with a micrometer dial gage. Each test continued until a total pressure of 30 KSF had been applied. At each increment of load, the soil sample time-deformation relationship was recorded. No additional load was added until all measurable deformation had ceased. The soils moisture content was preserved by moist cotton placed around the test sample. The results of the tests are presented in the form of pressure versus void ratio curves as shown in Figure 9 for each test.

CHAPTER III

DISCUSSION OF TEST RESULTS

Plate Load Tests

Figure 4 presents the average load versus settlement curves obtained in each of the three test sections. Each curve represents the average of three tests made at different points in the respective test sections. The point of failure from each curve is presented on Figure 4A, which shows the relationship between ultimate bearing capacity and soil density expressed as per cent compaction. Within the soil density range covered by the test series, the ultimate bearing capacity of the remolded soil increases linearly with increase in the amount of soil compaction. It was anticipated that a greater increase in ultimate bearing capacity would occur between ninety per cent compaction and ninety-five per cent compaction than between eighty-five per cent and ninety per cent compaction. The slope of the bearing capacity versus per cent compaction curve (Figure 4A) may change at densities greater than ninety-five per cent compaction, however.

It is noted that the total plate settlement at the point of failure increased slightly with increase in compaction. However, there was very little difference in the deflection at failure between the ninety per cent compaction and the ninety-five per cent compaction curves. This condition may be partially attributed to variations in the compaction of the top four to six inches of fill in the center section (ninety per cent compaction) which necessarily received more traffic during testing operations. The per cent of compaction of this top layer was probably increased and thus caused slightly better test results than would normally be obtained at exactly ninety per cent compaction. Two of the three load tests conducted in test section one (eighty-five per cent compaction) produced approximately the same load-settlement relationship. The third test produced a similar curve although the elastic deflection at the point of failure was about two-thirds of the deflection at failure for the other two load tests conducted in the same test section. However, all three tests indicate approximately the same failure pressure.

The load test series conducted in test sections two and three show very consistent load-settlement characteristics within each test group. Although each test group consistently shows a definite point of initial soil shear failure, the tests in both section two and three indicate a second partial shear occurs at pressures beyond the initial shear point. This is possibly caused by the slight variation in soil density within the depth zone of about 1.5 plate diameters below the bottom of the plate. The fact that the fill was constructed in layers presents a partial explanation to this phenomena. The results of all of the nine plate load tests are tabulated below:

Test Section	Percent Compaction	Load Test No.	Failure Pressure	Deflection @ failure	Progressive Failure
1	85 %	1	3,700 PSF	.27	
		2	3,300	.24	
		3	3,800	.19	
2	90 %	5	7,650	.32	yes
		6	7,000	.30	yes
		7	6,500	.25	yes
3	95 %	9	9,800	.37	yes
		10	10,500	.38	
		11	10,800	.41	yes

After each test plate was removed the test area was inspected. None of the load tests indicated any noticeable bulging adjacent to the test plate. All tests indicated a vertically sheared soil face around the edges. If each test had been continued to complete failure the mode of failure could have been better defined. In each case failure was designated as the point where initial shearing occurred.

The large plate load test results are shown on Figure 5 as a plate settlement versus pressure curve for each test. The tests were terminated when no additional load could be carried by the truss and earth anchor system. All three tests reached a total load of 21,000 pounds (3000 PSF) and the tests on sections one and two reached 24,500 pounds (3500 PSF) before the earth anchors failed.

The load test curves obtained for tests conducted on section two (ninety per cent) and section three (ninety-five per cent) indicate a straight line relationship between the soil bearing pressure and total deflection. Total deflections of .575 inches and .331 inches occurred beneath these two test sections respectively at a pressure of 3500 PSF. The load test data for the test conducted in test section one (eighty-five per cent) indicate a partial bearing capacity failure at a 2700 PSF pressure. The change in slope of the curve is not abrupt though there is a clear indication that shearing was initiated. There was some evidence of soil shear around the plate edges at this point during the test. A total settlement of 1.29 inches was obtained at the 3500 PSF pressure.

Considerable time was required to complete each of the large load tests (7 S.F. plate). Each increment of load remained intact until all measurable settlement was completed (with the exception of load test number four -- test section one). A total time of twenty-two days was allowed to complete the test conducted in test section one (eighty-five per cent). Each of the other two tests required seventeen days for completion. As a result of the long term load application during this test series, considerable consolidation settlement occurred. The time rate of settlement for each load test is shown on Figure 5A.

Although the load test reaction was not sufficient to produce a bearing capacity failure in test sections two and three, the settlements incurred during each test (below 3500 PSF) are considerable and thus are the governing design criteria for allowable bearing capacity determinations.

If a limiting total settlement of 0.5 inch is specified the following maximum bearing pressures are indicated by the load test data:

Test Section	Load Test No.	Per cent Compaction	Maximum Bearing Pressure
1	4	85 %	1600 PSF
2	8	90 %	2900 PSF
3	12	95 %	3840 PSF

These data are shown in graphical form (Figure 5A) as a plot of per cent compaction versus maximum bearing pressure at a limiting settlement of 0.5 inch. Although the curve is practically a straight line, it indicates a greater increase in pressure is allowed between eighty-five per cent to ninety per cent compaction than between ninety per cent and ninety-five per cent compaction. The curve further indicates that a maximum pressure between 4000 PSF and 4800 PSF should be possible at 100 per cent compaction, if the settlement is limited to 0.5 inch. This data, however, does not take into account additional long term settlements which will result caused by the weight of the fill. These figures can thus only be used as a basis of comparison. Many other factors must be considered in determining an allowable bearing pressure for foundations on a remolded soil.

A study of the time versus settlement curves (Figure 5B) shows that the slope of the curves for tests in sections two and three are almost zero (horizontal) at the final pressure of 3500 PSF. The lower curve of this plot (test section one) reached a total settlement of 1.3 inches but would seem to approach an asymptote of about 1.5 inches total settlement if the test had continued further. The shape of the curve for test section three indicates that the surface portion of the fill may have been slightly denser than the deeper fill. In addition, this curve shows that some additional settlement would have occurred at 3500 PSF if the test had continued for a longer period. The 3500 PSF pressure was maintained for only two days time as a result of a reaction failure on one end of the truss.

Test Boring Data

The included boring records, Figure 6 shows the relative density of each fill test section as measured using the penetrometer and the standard ASTM penetration test. These field tests were conducted immediately after each fill section was completed. The data are tabulated below:

Test Section	Per cent Compaction	Penetrometer Blows	Standard Penetration Blows	Blow Ratio
1	85 %	4 - 5	2 - 3	1.8
2	90 %	6 - 7	3 - 5	1.6
3	95 %	8 - 9	7 - 8	1.1

The above figures show that the penetrometer blows are consistently greater than the split spoon penetration blows but the ratio changes with the density of the fill. As the density increases the blow ratio decreases from 1.8 to 1.1. These test data indicate that there is no increase in the soils penetration resistance with increase in depth below the surface which reveals that the small differences in confinement produced by a surcharge weight do not greatly effect this soil's ability to resist shear. This conclusion is only valid in a general sense because of the limitations and inaccuracies involved in such expedient field tests. Assuming that the surcharge weight does not appreciably effect the penetration tests, the boring data indicate a very uniformly dense soil mass in each test section.

After all field tests were completed, loose, disturbed samples of the fill were secured with a hand auger at one foot intervals through the six feet of fill in each test section. The moisture content of each sample was determined and is tabulated below:

Depth	Test Section 1	Test Section 2	Test Section 3	Average Moisture Content
1 ft.	28.1	27.7	27.5	27.8
2	27.1	27.0	26.6	26.9
3	25.3	24.1	23.8	24.4
4	24.0	24.0	23.5	23.8
5	24.1	24.5	24.1	24.2
6	23.8	23.6	23.2	23.5

The optimum moisture content (twenty-three per cent), at which the fill was placed, was maintained reasonably well within the bottom four feet of fill. However, the moisture content of the top one foot of fill was 4.8 per cent higher than optimum moisture. This data shows that some moisture was added during the wet, rainy months which occurred while fill construction and testing were done. This data further substantiates the reasons for some erratic test data obtained from undisturbed samples secured near the fill surface.

<u>In place settlement data</u>.--The records of the settlement plate movements during fill construction and testing are

shown on Figure 7. These charts indicate the amount of plate movement relative to its zero level which was referenced to a bench mark as the plate was installed. The point at which fill construction was completed and each large plate load test was conducted is also indicated on each chart.

The data, obtained from the settlement plates placed in test section one, cover the longest period of time since this test section was constructed first. The data indicate that the bottom plate (six feet) initially moved up but then settled gradually as the fill weight was added. Movements of the center plate reflect accurately the settlement of the bottom three feet of fill and the underlying virgin soil. The gradual increase in settlements was abruptly increased while the large plate load test was conducted. The load test increased the total plate movement from 0.20 inches at the beginning of the load test to .34 inches after the test was completed. The affects of the load tests are also reflected in the movements of the bottom plate. A summation of these two settlement graphs shows that the bottom three feet of fill compressed approximately .05 inch as a result of the plate load test. However, almost half of this compression rebounded after the load had been released for two months. It is noted that the slope of the time settlement curves for both settlement plates prior to the load test can be projected in a straight line to intersect the rebounding curve which occurred after the load test was completed.

The time-settlement data recorded in test section two show total movements that are approximately one-half the movement recorded in test section one. After all the fill was placed the center plate settlement was negligible even during the plate load test. After the load test was completed the center and bottom plates rebounded a small amount.

The settlement plates placed in test section three moved even less than in section two. Since the bottom plate settled almost as much as the center plate during the entire time interval, the compression of the bottom three feet of fill can be considered zero.

The surface plate in all three sections gradually moved up rather than down. This information indicates that the highly micaceous silts rebound considerably if it is unconfined. Even greater swelling probably would have occurred if the soils had been saturated.

The time-settlement data give a general picture of the fill movements although no direct correlation with the load test settlements can be established. The test plates were located in the fill around the center section where the load tests were conducted. The effects of the additional load test pressures will therefore be dependent on the radial distance as well as the depth and density of the fill.

Laboratory tests.--The included laboratory data were determined by tests made on undisturbed soil samples taken from the three fill test sections. A number of tests were conducted on samples that were not representative of the average fill condition from which the samples were taken. The major variations in the condition of these samples occurred in samples secured from the top two feet of the fill where slight changes in moisture content and density existed after the fill had been in place for a considerable time. In each case the poor (non representative) samples were slightly wetter and less dense than the typical condition. Other variations occurred as a result of the sampling procedures and the disturbances incurred through normal handling of samples after they were in the sampling tube. The samples secured from test section one (eighty-five per cent) were particularly hard to extrude from the sampling tube and trim into test specimens. Many samples were destroyed before representative samples could be prepared for testing.

A second major variation was caused by the layered structure of the fill. Many presumably good samples were tested but gave erratic results because of the plane of weakness between soil layers. Other variations were caused by swelling of the soil samples after extrusion from the sampling tube. If a sample was not tested immediately after removal from the tube, a small decrease in density was inevitable. This condition was verified by the data obtained from the surface settlement plates.

The results of the strength tests are presented on Figures 8, 8A and 8B. The average shear strength characteristics for each of the three soil densities were determined by reconstruction of the Mohr diagrams using the average stress-strain relationship established for each condition. In each case the average data were determined using three or more actual triaxial shear tests conducted on different typical samples secured from the same fill test section. Although a range or band of stress-strain data was established for each density condition, only the average data are presented for this evaluation.

A study of the three reconstructed triaxial shear tests presents a clear picture of the strength variations which occur with variations in soil density. The important strength characteristics from these plots are tabulated below for comparison:

Test Section	Compaction	Dry Density	M.C.	Void Ratio	C'	c"	Ø
1	85 %	80.3	23 %	1.08	.800	.180	21°
2	90 %	85.0	23.8%	.98	.800	.250	22°
3	95 %	89.8	24.0%	.86	.800	.400	25°

The major strength variation with variation in density occurs at low confining pressures. The unconfined compressive strength of the remolded soil increased with an increase in density. This variation is reflected in the curved section of the failure envelope which produced the variable c" intercepts. This value (c") is the minimum failure shear stress and is only obtained at a low or zero confinement.

Because of the c" variations the amount of effective rebonding of the smaller silt particles appears to be governed to some extent by the remolded density. Assuming the moisture content is held reasonably constant, the amount of bond (cohesion) is proportional to the remolded density but the permanence of this bond strength may depend greatly on the seasoning (age of the remolded sample) as well as its initial density and its confinement. The ageing of the remolded soil could thus tend to increase the effective cohesion when confined and decrease it when confinement is lacking. This is particularly true in these highly micaceous soils which swell or expand if unconfined. The high mica content tends to separate the soil matrix through expansion which invariably occurs in the unconfined condition.

There is no measurable variation in the maximum cohesion (apparent cohesion c') with variation in soil density. Since the c' intercept is established by a projection of the straight portion of the failure envelope, its value in the interpretation of the intrinsic strength characteristics of such semi-heterogeneous remolded soils would seem questionable. However, it is possible that the apparent cohesion (c') approaches the actual shear strength of these sandy micaceous silts in the undisturbed condition. The cohesive portion of the shear strength is ordinarily greatly reduced with remolding because the natural bond of the virgin soil is lost.

Some serious consideration or weight must be given to the validity of using the apparent cohesion (c') in actual strength determinations. The curved portion of the Mohr failure envelope is established solely from the unconfined compressive strength which is plotted on each of the included charts (Figures 8, 8A and 8B). Although the samples are only partly saturated at the beginning of the compression load, the degree of saturation increases rapidly with decrease in void ratio as the test progresses. The variable effects of pore water pressure could be substantial although no definite data were determined during this test series.

A more influential element involved in the unconfined testing of the partly saturated micaceous silts is the composition and structure of the soil. Even using the best remolded samples and testing procedures can not eliminate the inherent expansion which occurs during sample preparation and testing. It is reasonable to assume that some reduction in the initial intrinsic bond of the soil particles accompanies expansion. The unconfined test thus may give overly conservative results because of the lack of lateral support which is needed to arrest the initial expansion and prevent excessive expansion during testing. The boundary conditions of the unconfined test would therefore seem too severe for the test soil in question.

The Mohr failure envelopes established by the reconstructed diagrams show only slight changes in the angle of internal friction for test samples compacted to eighty-five per cent to ninety-five per cent of the standard maximum dry density. The following data were taken from the included stress-strain relationships for each density condition:

Figure	Per cent Compaction	Normal Stress @ Failure (σ_1) σ_3=1000 PSF	σ_2=4000 PSF
8	85.0	4400 PSF	10,600 PSF
8A	90.0	4500 PSF	11,000 PSF
8B	95.0	5000 PSF	12,500 PSF

The above data indicate that the increase in σ_1 at failure with an increase in density is small at a confining pressure of σ_3=1000 PSF. However, at the 4000 PSF confining pressure (σ_3) there is a pronounced increase in the normal pressure required to produce failure between ninety per cent compaction and ninety-five per cent compaction. The angle of internal friction does not appear to change appreciably, but the effectiveness of a small change in the ϕ

angle is emphasized in bearing capacity computations discussed later. It is apparent from the above discussion and the included data that the angle of internal friction is a function of the initial dry density and the magnitude of the confinement. The relationships discussed above are shown on Figure 8C.

The variations in the consolidation test data obtained from the many tests on undisturbed fill samples presented a problem of determining the average void ratio versus normal stress relationship for each fill test section. This was done by averaging the initial void ratios, the slope of the recompression curves and virgin curves and omitting the test data which were obtained using poor samples or non representative samples. A number of the samples secured near the fill surface were slightly wetter and less dense than the typical condition. The reasons for this condition were previously discussed. The consolidation curves presented on Figure 9 were constructed using the average data obtained from three or more consolidation tests conducted on separate undisturbed fill samples from the same test section.

The pertinent test results are tabulated below:

Test Section	Per cent Compaction	Initial Void Ratio	Compression Index (C_c)	Average Moisture Content	Degree of Saturation (S)
1	85 %	1.10	.282	25 %	61 %
2	90 %	.96	.260	25 %	70 %
3	95 %	.84	.210	25 %	80 %

During consolidation testing of the eighty-five per cent compacted samples an initial change in the void ratio occurred before any normal load was applied. Under the small weight of the testing apparatus the void ratio was reduced from an average of 1.10 to 1.03. This information further emphasizes the inherent springy structure of the test soil especially at low density. Some reduction in the initial void ratio may have similarly occurred within the more dense samples but none was recorded.

A measure of the initial slope of the recompression curves reveals some interesting facts. The initial slopes of the eighty-five per cent and ninety per cent curves are almost exactly the same. No appreciable variation in slope of these two curves occurs until a vertical pressure of 1500 PSF is reached. Above a vertical pressure of 1500 PSF the rate of change in slope of the less dense sample (eighty-five per cent) increases much faster than the ninety per cent density curve until pressures in excess of 10 KSF are exceeded. It is noted that all three consolidation curves converge toward a void ratio of 0.60 at a vertical pressure of 40 KSF. At this point the upper curves have approximately the same slope while the bottom curve continues at a lower slope. This analysis shows that theoretically a considerable change in the settlement characteristics of the test soil occurs between ninety per cent and ninety-five per cent compaction.

<u>Comparison of field and laboratory data</u>.--An evaluation of the theoretical bearing capacity of each fill test section was made using the applicable triaxial shear data. The following general bearing capacity equation was used:

$$q_c = \frac{\gamma b}{2} N_\gamma + c N_c + q' N_q$$

During the initial analysis the surcharge component ($q' N_q$) was deleted since the actual plate load tests had no surcharge. Although the above general equation is widely accepted, the bearing capacity factors N_γ, Nc and Nq vary or cover a wide range depending on the type of soil and the theoretical approaches and studies of a number of authorities. The early studies of Terzaghi (1) established the first factors for use in a rational computation of the bearing capacity of partially saturated soils that derive shear strength from both intrinsic bond (cohesion) and internal friction. Later research by Sowers (2) indicated a need for some variation from the Terzaghi bearing capacity factors to include consideration for types of foundation. Similar factors were established by Bell (3) that are more conservative than originally found by Terzaghi. Recent works of Meyerhof (4), however, essentially agree with the findings of Terzaghi. During this evaluation the writer has used the bearing capacity factors of Terzaghi for insensitive soils.

Before any conclusions or comparisons between actual and theoretical bearing capacity can be drawn, additional consideration must be given to the range of cohesive shear strength values previously discussed. Large variations in the computed bearing capacity can be found depending on the exact cohesive strength used. The following tabulation presents the range of allowable bearing capacities that were computed along with the actual bearing capacities that were measured by the eight inch diameter plate load tests. Both the theoretical and the actual bearing capacities include a safety factor of 2.5 against the point of initial shear failure:

Test Section	Per cent Compaction	Theoretical Bearing Capacities c'	Theoretical Bearing Capacities c''	Actual Bearing Capacity (Average of 3 tests)
1	85 %	5820 PSF	1355 PSF	1400 PSF (1080 PSF)
2	90 %	6460	2100	2750
3	95 %	7970	4040	4100

The theoretical bearing capacities include an allowance for the round plate by using an effective width of .9 D. The above data show close agreement between the theoretical and actual bearing capacities if the lower cohesive shear strength values (c'') are used. Since a partial shear failure was obtained at 2700 PSF by the large plate load test conducted in section one (eighty-five per cent), an allowable bearing capacity of 1080 PSF (S.F.=2.5) checks closely with the 1400 PSF value determined using the smaller plate. This information allows a conclusion that the lower cohesion values (c") should be used in determining the bearing pressure at which local shearing is initiated.

An analysis of the theoretical settlements that could occur under a seven square foot plate was conducted using the applicable consolidation data. A fill mass 6.0 feet deep with a pressure of 3500 PSF applied at the surface was used in each computation. The theoretical settlements were based on the pressure distribution as determined by Westergaard's (5) formula for square foundations. The average stress values (b/4 from center) were used in the settlement computations.

The distribution of the theoretical settlements is noted. The following data show the per cent of the theoretical total settlement in each two foot thick fill layer of each test section:

Test Section	Top 2 Feet	Middle 2 Feet	Bottom 2 Feet	Total Settlement
1	75 %	17 %	8 %	1.760 inches
2	65 %	25 %	10 %	1.275 inches
3	66 %	23 %	11 %	.574 inches

The figures show that a higher percentage of the total settlement occurs in the upper portion of the fill at lower density. There is an indication that the settlement distribution is more evenly distributed in depth at higher density. The inconsistences in the settlement distribution pattern are caused by variations in the slope of the consolidation recompression curves.

The above tabulated theoretical settlements do not include the elastic deflection although some elastic deflection may be reflected in the small void ratio change which occurs along the recompression portion of the consolidation curve. However, for comparison some of the initial elastic deflection (which occurred immediately under the weight of the settlement plates alone) was not recorded. It is difficult to differentiate between the elastic deflection and consolidation settlement unless the elastic rebound can be accurately measured. Although the elastic rebound of each load test was not measured, considerable upward movement of the test plate occurred as the test load was released.

Additional computations were made to determine the theoretical elastic deflections that could occur under each density and loading condition. The modulus of elasticity at each soil density was determined from the applicable stress-strain curve. The following elastic deformations were computed using the general equation: $\rho = \frac{.6qb}{E}$

Test Section	Modulus of Elasticity (E)	Elastic Deflection (P)	Actual Settlement
1	350 psi	1.33 inches	1.29
2	425 psi	1.10 inches	0.58
3	560 psi	0.84 inches	0.33

These deflection data include some consolidation settlement and to that extent partly reflect the consolidation curves. However, to establish and account for the total movements of the load test plate would require a detailed study of the deflection and rebound characteristics of the test soil at different densities. It is estimated that at least one half of the load test plate movements is elastic deflection.

The actual settlements that occurred under the large plate load tests are presented in Figure 5 and Figure 5B. The time versus settlement curves (Figure 5B) for each plate test show the rate of plate movement during the entire loading cycles. Since the load was applied in 500 PSF increments, each load test reasonably simulates the actual loading cycle that occurs during building construction. With the exception of the load test conducted in test section one (eighty-five per cent compaction) the slopes of the time-settlement curves are zero. Therefore, any additional settlements for a longer period of time at the 3500 PSF pressure would have been small. However, the bottom curve of Figure 5B shows that the load test in section one should have continued until the curve was essentially horizontal. Since a partial shear failure is indicated on Figure 5 for this same test, it is possible that movement would have continued indefinitely.

The theoretical consolidation settlements and the actual plate settlements are tabulated below for comparison:

Test Section	Per cent Compaction	Theoretical (T)	Actual (A)	Ratio T/A
1	85 %	1.76 inches	1.29 inches	1.37
2	90 %	1.27 inches	.58 inches	2.19
3	95 %	.57 inches	.33 inches	1.73

It is noted that the actual measured settlements also include any movement that may have occurred in the virgin soil below the fill. The in place settlement plate readings given in Figure 7 show that settlements of the bottom plates during the load test period are less than .05 inches in test sections two and three and .09 inches in section one. The total measured settlements could be reduced by these applicable figures but the ratio of theoretical to actual settlements would not change appreciably. The average settlement ratio is 1.75 which is a reasonable value considering the type of soil used in this test series. The great differences in the theoretical and actual settlements are attributed to the following conditions:

1. The inherent springy nature of the test soil.
2. Expansion of test samples during sampling and

test preparation.

3. The variations in the drainage conditions of the test sample and the in place fill.

4. Errors in the testing procedures--this should be minor.

Compaction characteristics.--The compaction characteristics of the test soil were recorded during construction of the test panels. The laboratory compaction characteristics are presented in Figure 2 as the standard moisture density relationship. This standard Proctor test (ASTM D-698) subjects the test soil to 12,500 foot pounds energy in a small confined mold. The type and amount of compactive effort applied to the test soil is extremely different in the field. The compactive effort exerted by the Jay tamper in construction of test section one can not be realistically evaluated. However, the compactive effort of the Barco Rammer can be measured. During construction of test sections two and three the amount of work required to reach the required density was recorded. The required compactive effort using the Barco Rammer is tabulated below:

Test Section	Per cent Compaction	Dry Density Required	Number of Impact Blows Per 6 Inch Layer	Compactive Effort Foot-Pounds per Cubic Foot
2	90	85.1 PCF	600	1750
3	95	89.8 PCF	1200	3500

This data shows that a considerable increase in the compactive effort is required to produce a small increase in density. This phenomena is not unusual, since others have established that the compactive effort versus dry density relationship is not linear.

The type of compactive effort produced by the Barco Rammer is considerably different from that produced in the laboratory for two major reasons. First, the compaction under the standard Proctor method employs a much smaller striking surface which produces a high impact pressure in a confined mold. This tends to shear the soil with each hammer blow while the Barco Rammer produces a low impact pressure at a high energy input per blow. The Barco compaction is thus a more efficient application of work. Secondly, the size of the Barco Ram gives much better confinement of the soil under compaction and therefore causes greater efficiency in moving the soil grains closer together. The tamp foot size also effects the density of the soil at greater depths than does smaller tampers. It is noted that considerable care was necessary to insure that full energy was applied per blow since lower work output with the Rammer is possible if the operator does not cause the Rammer to develop full lifting height with each compression stroke. This variable was taken into consideration in developing the above energy data.

The inherent physical characteristics of any soil require that more energy per unit of volume be expended in obtaining very high densities. For example, the micaceous silts may be loose dumped at eighty per cent compaction, may be easily tamped to ninety per cent compaction but must be thoroughly compacted to reach higher densities. The included data indicate that approximately twice the energy required for ninety per cent compaction is required to obtain ninety-five per cent compaction.

CHAPTER IV

CONCLUSIONS

The following conclusions have been made based on the results of the field and laboratory tests:

1. The high mica content causes the compacted soil to expand or rebound when unconfined.
2. The shear strength of the test soil increases with an increase in dry density (per cent compaction).
3. The allowable bearing capacity will be governed primarily by the limiting settlement.
4. The actual settlement will be one half to three fourths of the theoretical settlements computed as consolidation settlement based on Westergaard's average st ess distribution beneath a rectangular foundation.
5. Considerable elastic deflection occurred and was recorded as a part of the total test plate settlement.
6. The magnitude of actual settlement decreases rapidly as the dry density increases from ninety to ninety-five per cent compaction.

7. The theoretical bearing capacity should be based on the lower cohesion value (c").
8. The ratio of penetrometer blows to the standard penetration resistance varies from 1.8 at low density to 1.1 at high density.
9. Approximately twice the compactive effort is required to increase the compaction from ninety to ninety-five per cent.
10. There is considerable difference in the laboratory and field compactive effort. The field compaction is much more efficient.
11. The triaxial shear test is a good method of determining the shear strength of the test soil.
12. The in-place density of a partially saturated homogeneous fill soil can be estimated using the standard penetration test (ASTM D-1586).

CHAPTER V

RECOMMENDATIONS

The major problems involved in this testing program were the limitations of the field testing equipment and the extremely long periods of sustained testing. If the load reaction had been sufficient to produce failure beneath the large plates in all three test sections, more accurate strength determinations could have been made. The use of a few settlement plates to measure in place settlements is not satisfactory unless settlement readings are also made directly under the loaded area. If similar investigations are made in the future the following recommendations should be considered:

1. Provide sufficient load test reaction to produce complete failure.
2. Provide a means of automatically regulating the hydraulic pressure used to produce the test load.
3. Allow sufficient time to conduct each load test in order that a maximum amount of consolidation settlement will be recorded.
4. Secure undisturbed soil samples from the fill mass at different periods throughout the field testing program in order that any changes in the in place

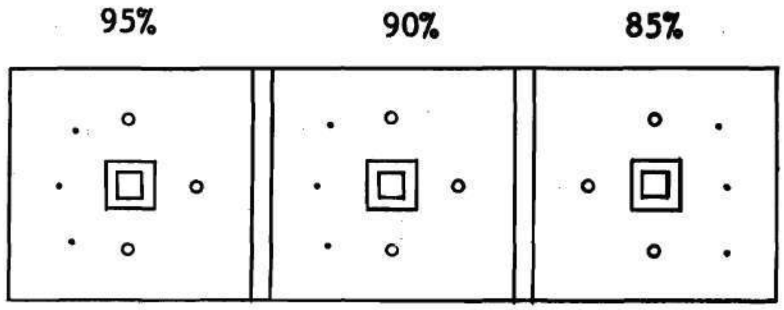
density with time may be determined.

5. Test samples from different depths to determine if variations exist with change in fill depth.
6. If settlement plates are used, devise a method of determining the movements immediately under the load tests at various depths.
7. Determine the variations in the strength characteristics of the fill with variations in the size of the loaded area.
8. Eliminate the testing of fill compacted to only eighty-five per cent compaction and add a denser fill test section at one hundred per cent compaction.

The major problems encountered in the laboratory work involved extrusion of undisturbed samples and preparation for testing. If a more resilient soil is used some of this trouble will be eliminated. If the same type of highly micaceous soil is used no attempt should be made to test this material at densities corresponding to less than eighty-eight to ninety per cent compaction. All undisturbed samples should be dug samples that are secured in tubes greater than six inches in diameter. All undisturbed samples should be tested instantly after they are removed from the sample container to obtain the best results.

APPENDIX

COMPACTION



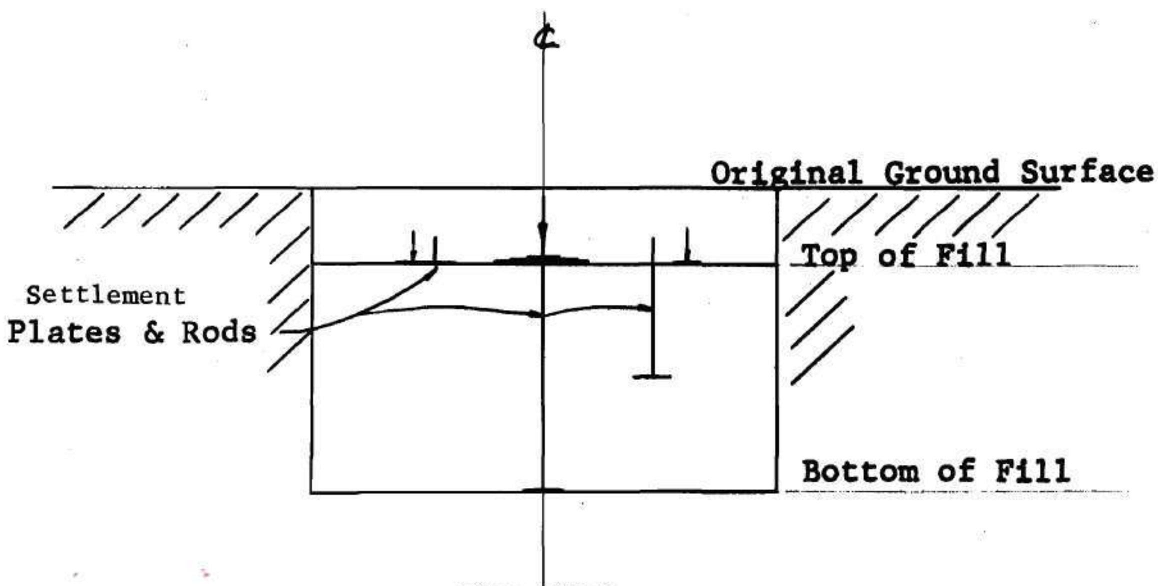
PLAN VIEW

Scale 1 in. = 10 ft.

Large Plate Load Tests ___ ⧈

Small Plate Load Tests ___ o

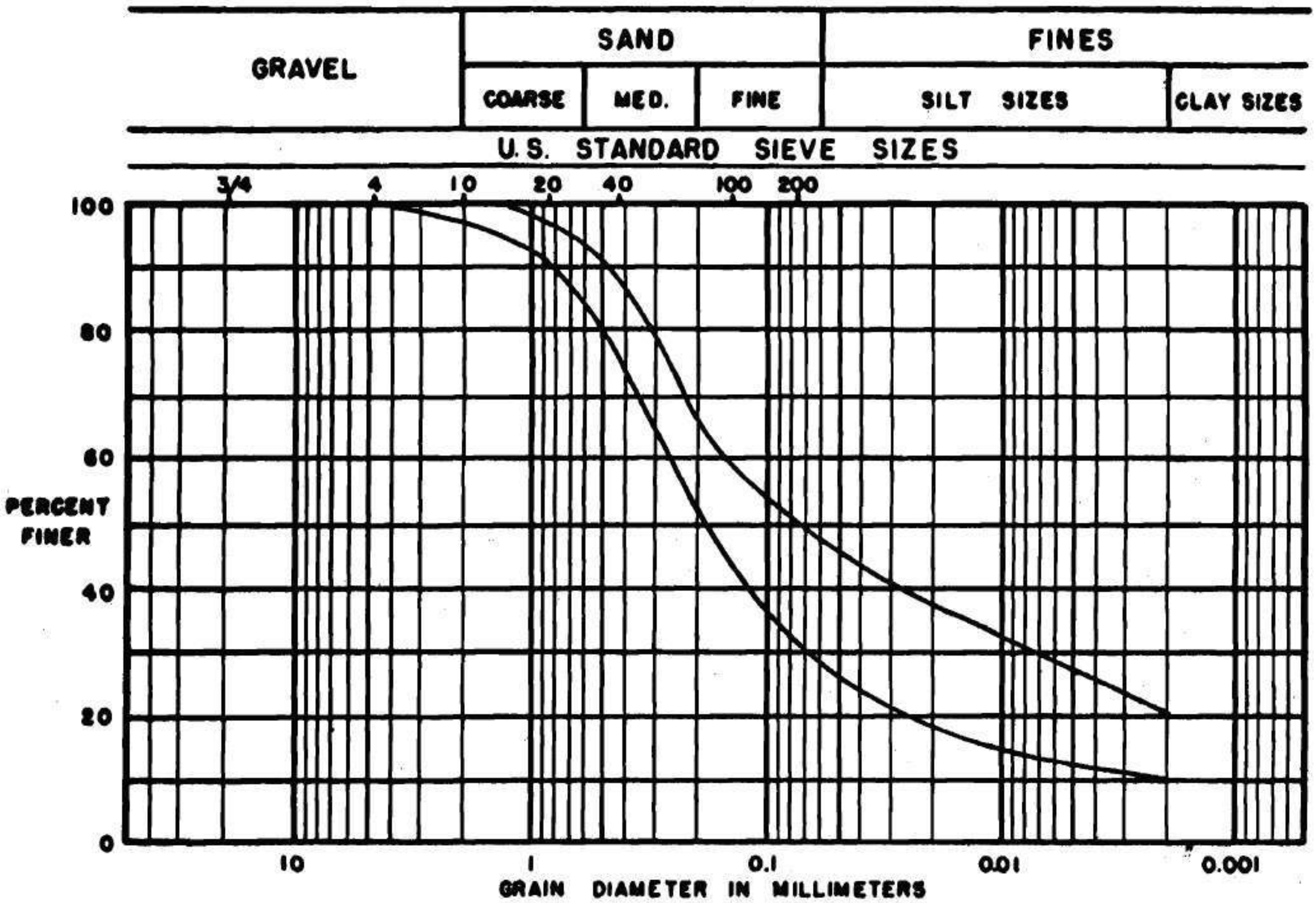
In-place Settlement Plates _ •



END VIEW

Scale 1 in. = 5 ft.

Fig. 0 GENERAL PLAN OF TEST AREA



Soil Description

from above chart - very micaceous silty medium and fine sand

actual description - brown-grey medium and fine sandy very micaceous silt

Fig. 1 Classification Test Data

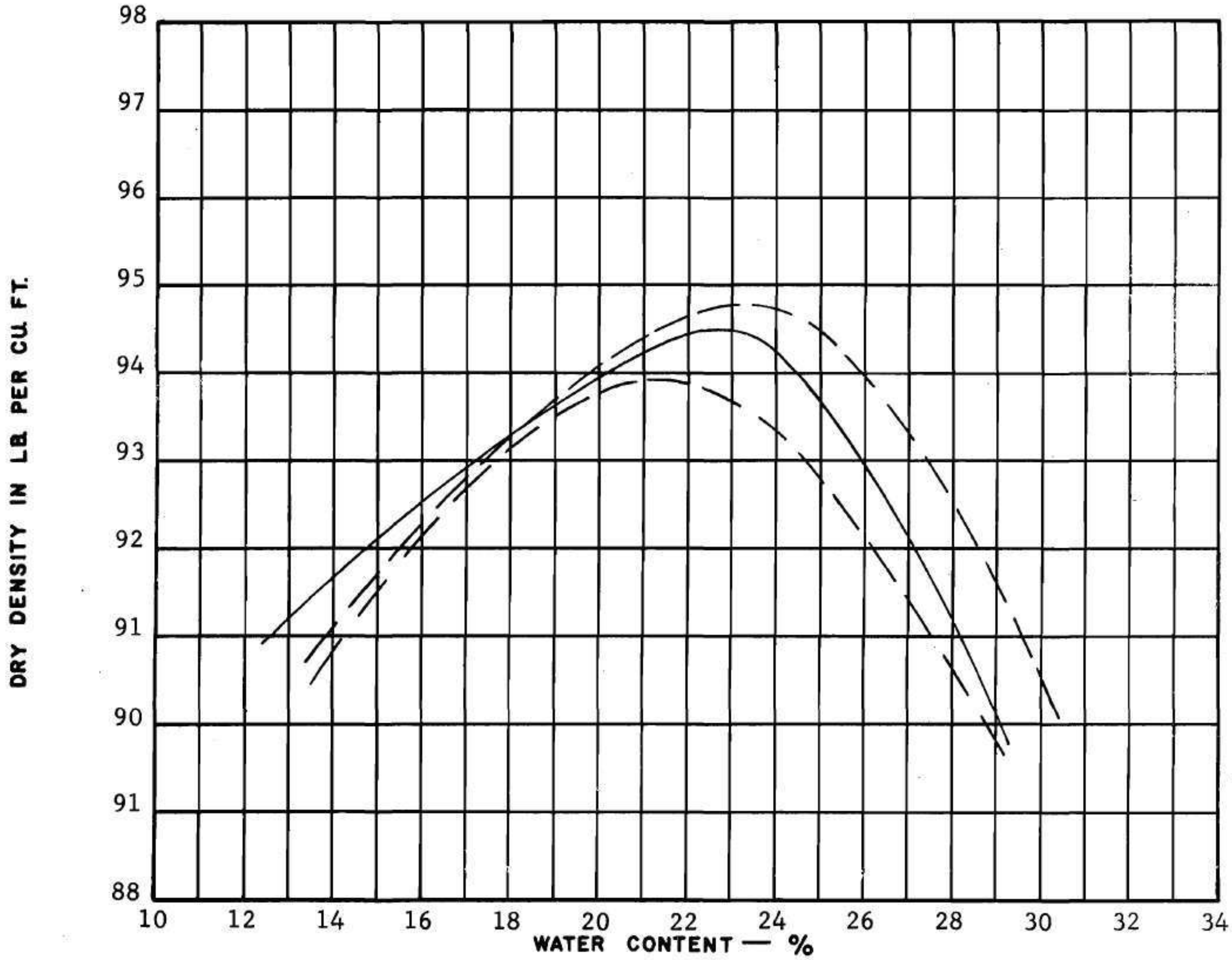
PLASTICITY TESTS:

Sample	Liquid Limit	Plastic Limit	Plasticity Index
1	29	16	13
2	Non - Plastic		
3	Non - Plastic		
4	Non - Plastic		

MICA CONTENT:

Sample	Method	Per Cent (by weight)
1	Mechanical	47
2	Mechanical	35
3	Mechanical	36
4	Bronoform	44
5	Bronoform	54

Fig.1A Supplementary Laboratory Test Data



MAXIMUM DRY DENSITY 94.5 **LB. PER CU. FT.**

OPTIMUM MOISTURE 23.0 **%**

METHOD OF TEST- ASTM D-698-58T

TYPE MATERIAL- Brown-Grey Fine to Medium Sandy Very Micaceous Silt

Fig. 2 Laboratory Compaction Tests

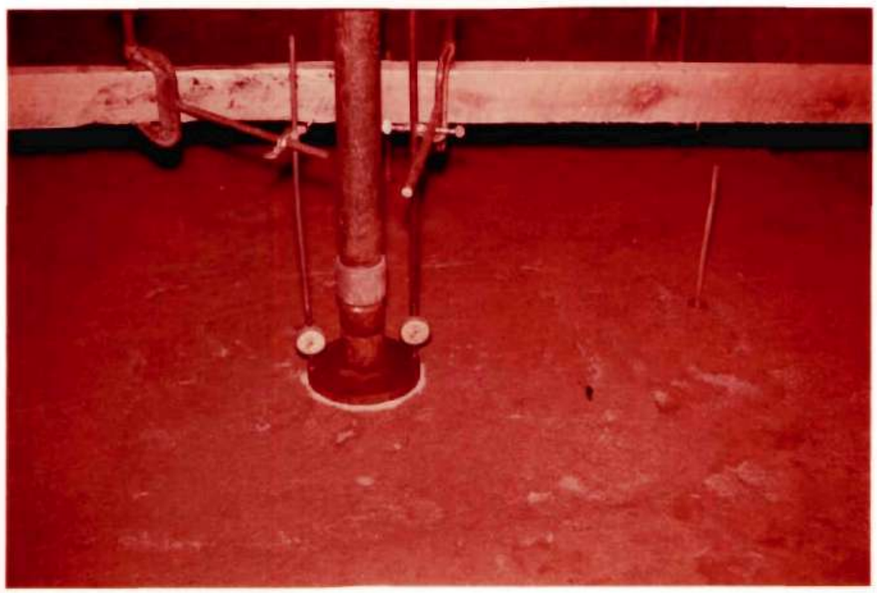


Fig. 3a SMALL PLATE LOAD TEST SETUP

(Note: Settlement Plate Rods Also Shown)

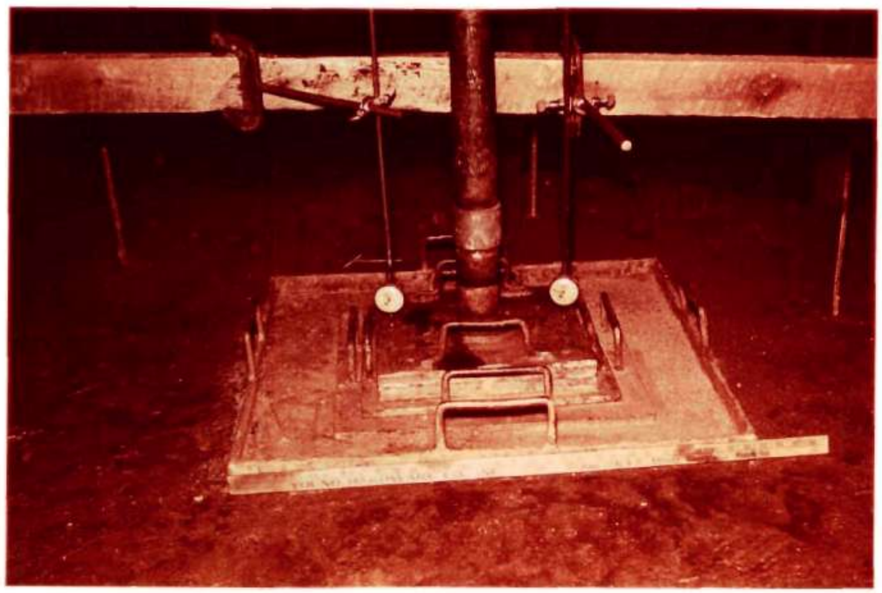
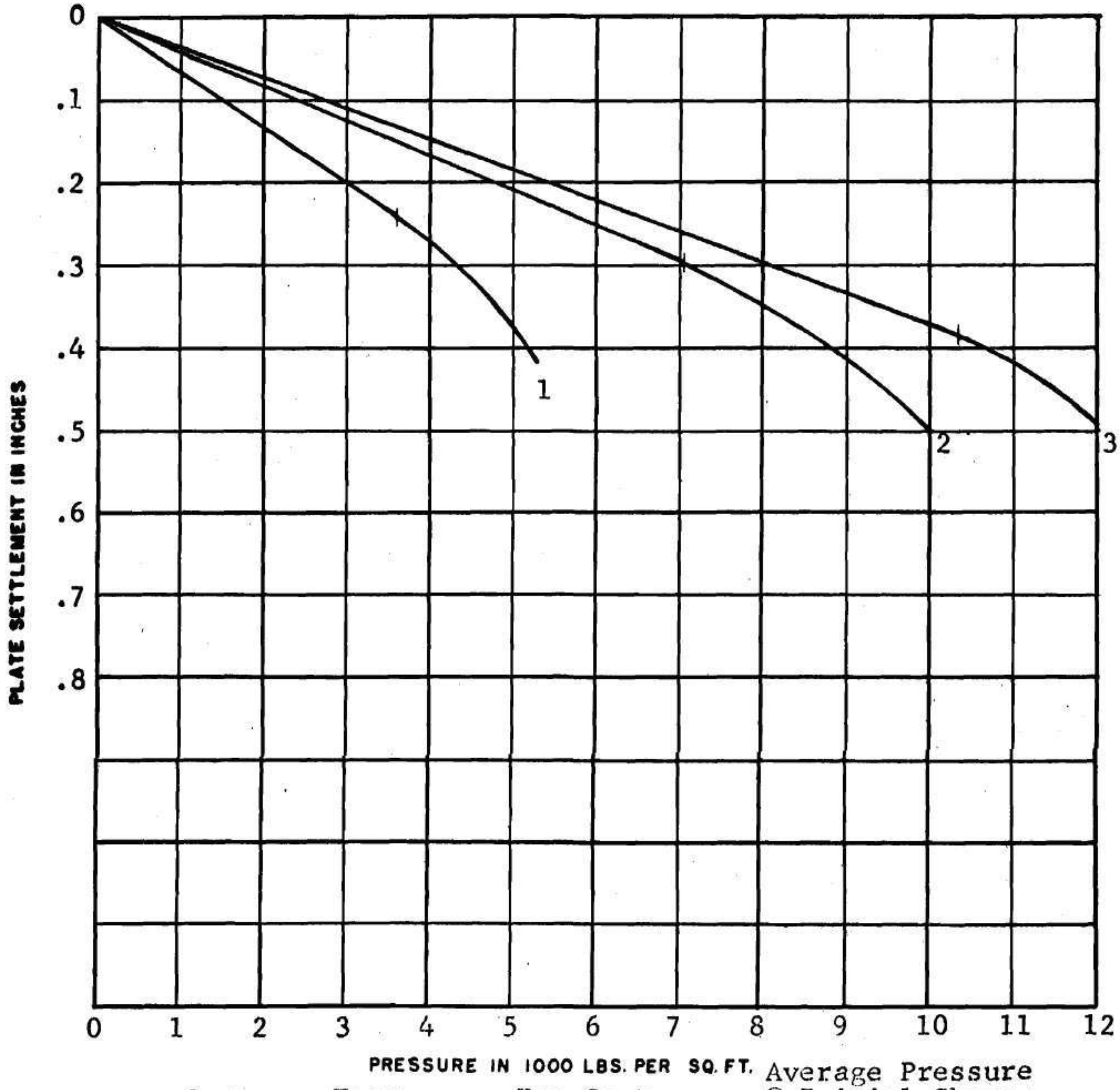


Fig. 3b LARGE PLATE LOAD TEST SETUP

(Note: Settlement Plate Rods Also Shown)



Curve No.	Test Section	Per Cent Compaction	Average Pressure @ Initial Shear Failure
1	1	85 %	3600 PSF
2	2	90 %	7050 PSF
3	3	95 %	10,366 PSF

Fig. 4 Eight Inch Diameter Plate Load Tests

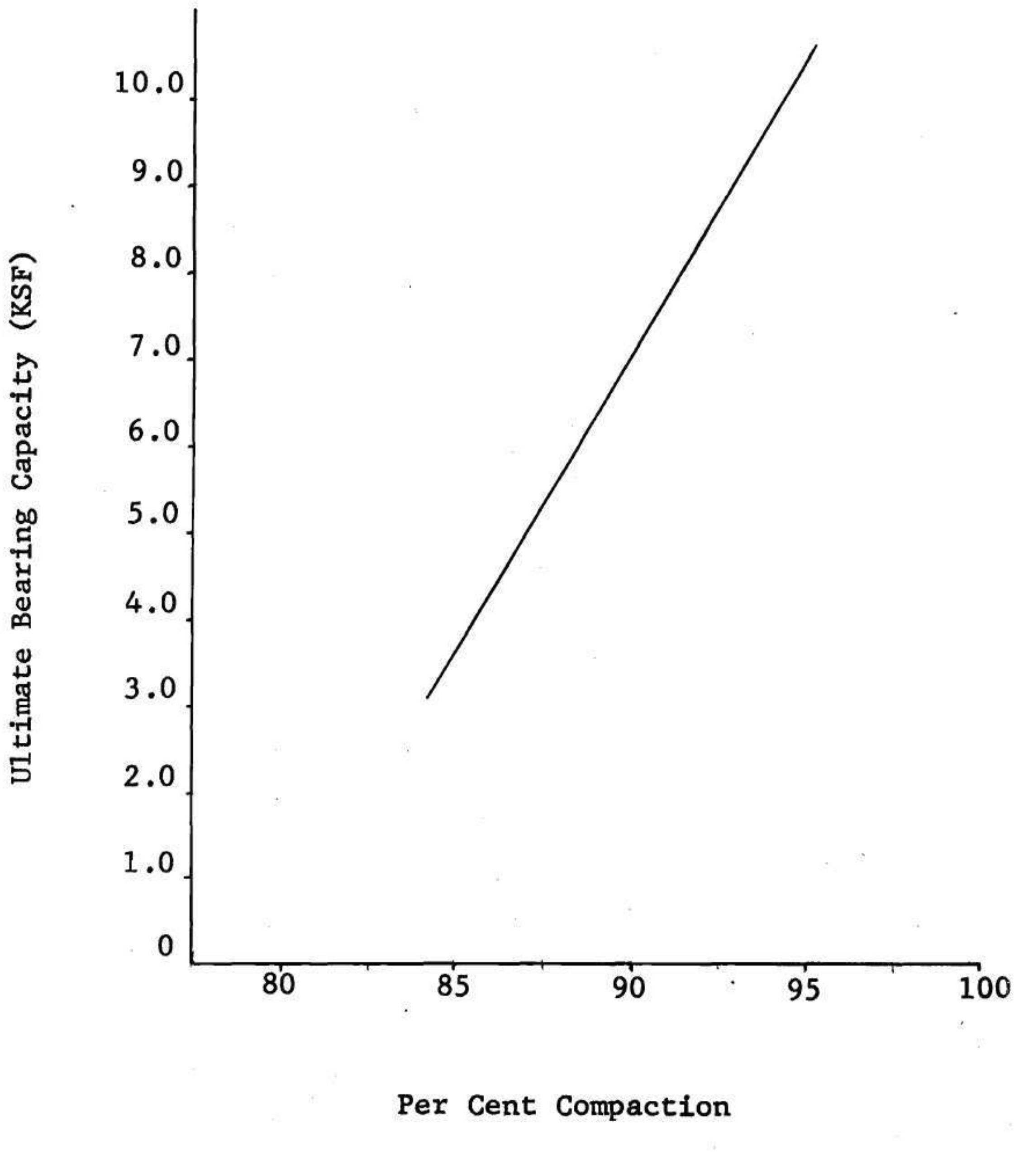


Fig. 4A Ultimate Bearing Capacity vs Per Cent Compaction
(Eight inch diameter plate load tests)

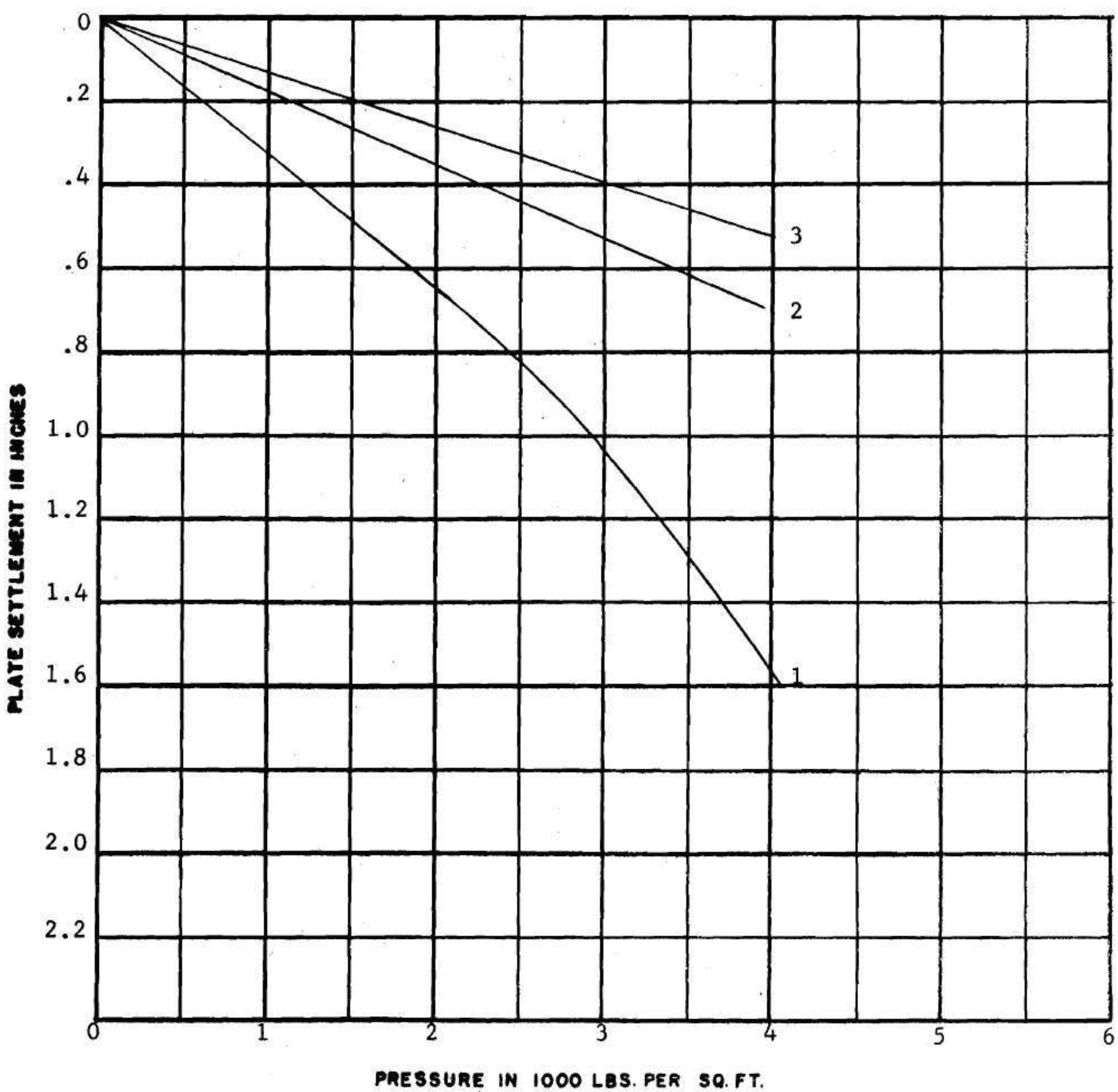


Fig. 5 Plate Load Tests (7 S. F. Area)

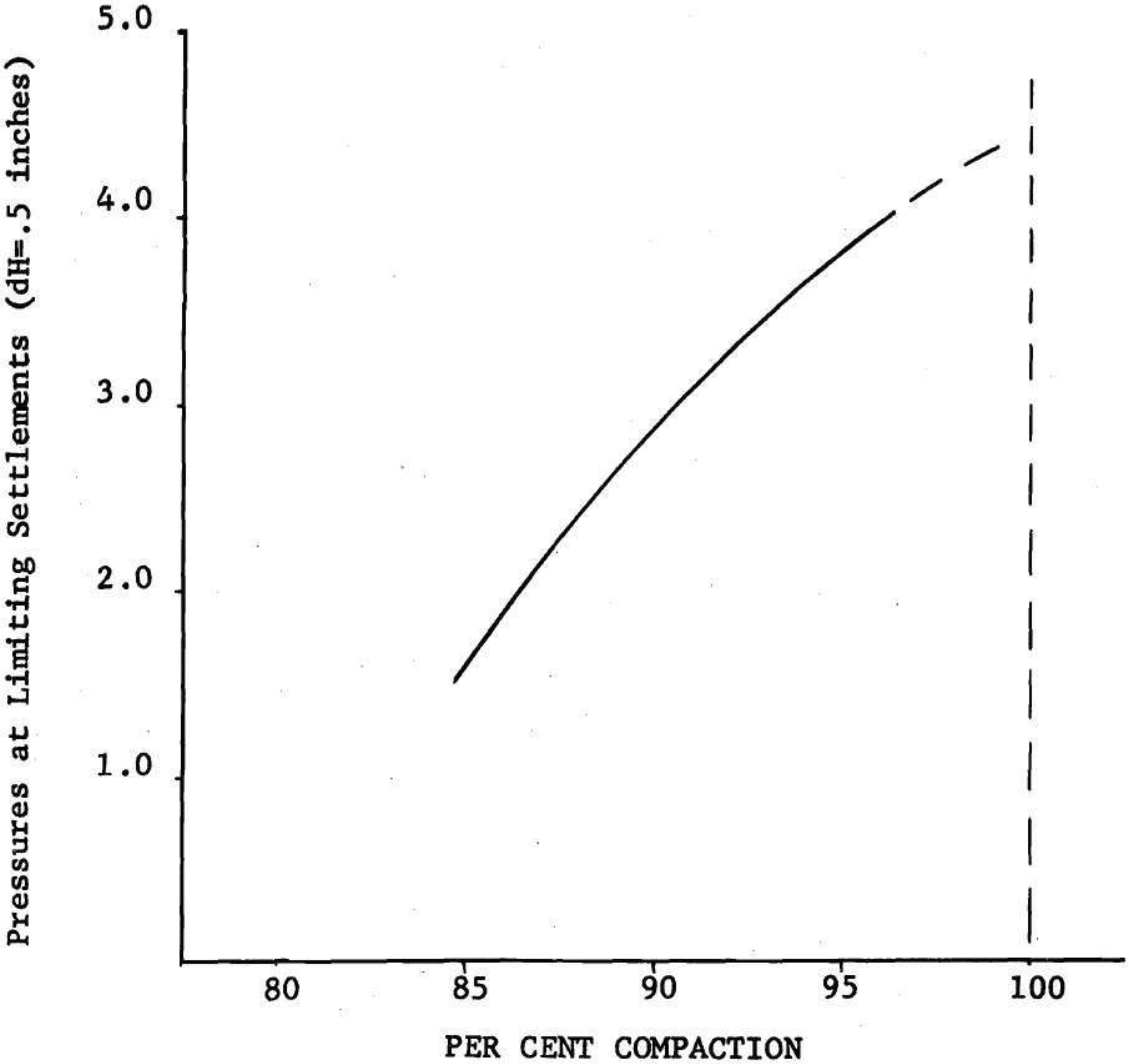
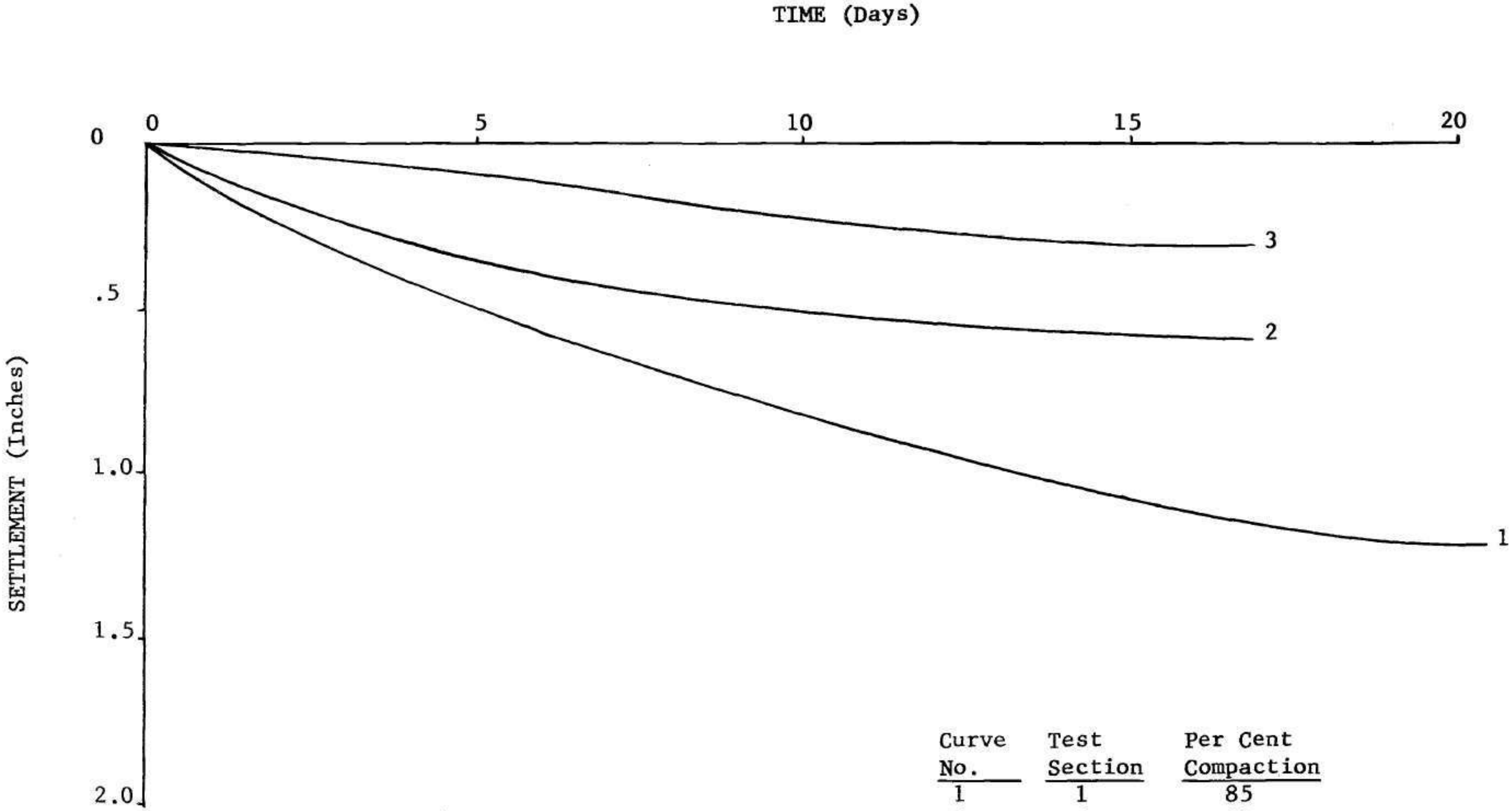


Fig. 5A Limiting Settlement vs. Per Cent Compaction
Plate Load Tests (Area-7 S.F.)



Curve No.	Test Section	Per Cent Compaction
1	1	85
2	2	90
3	3	95

Fig. 5B Time vs Settlement Curves
Plate Load Tests - (Area= 7 S.F.)

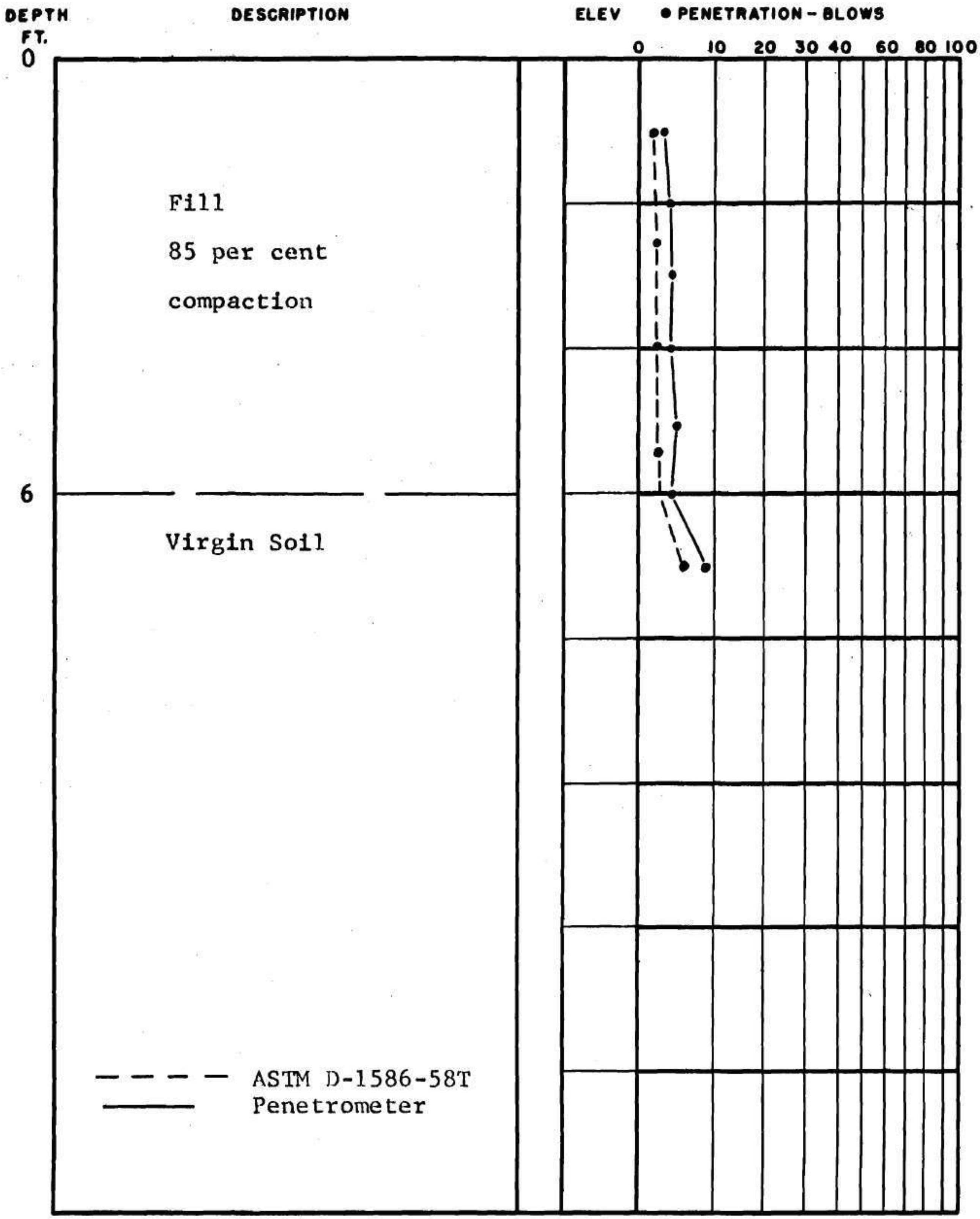


Fig. 6 Test Boring Data - Test Section 1

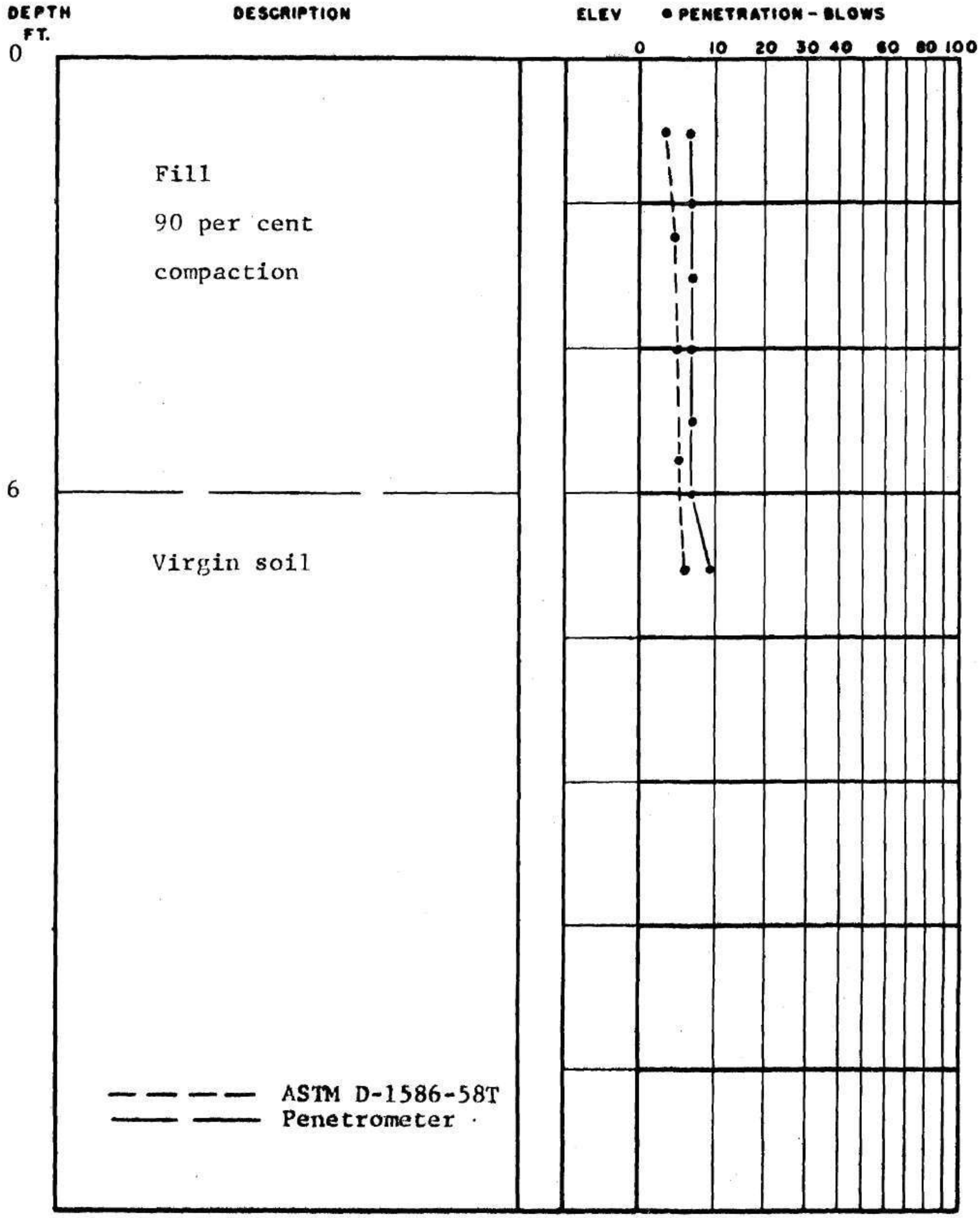


Fig. 6A Test Boring Data - Test Section 2

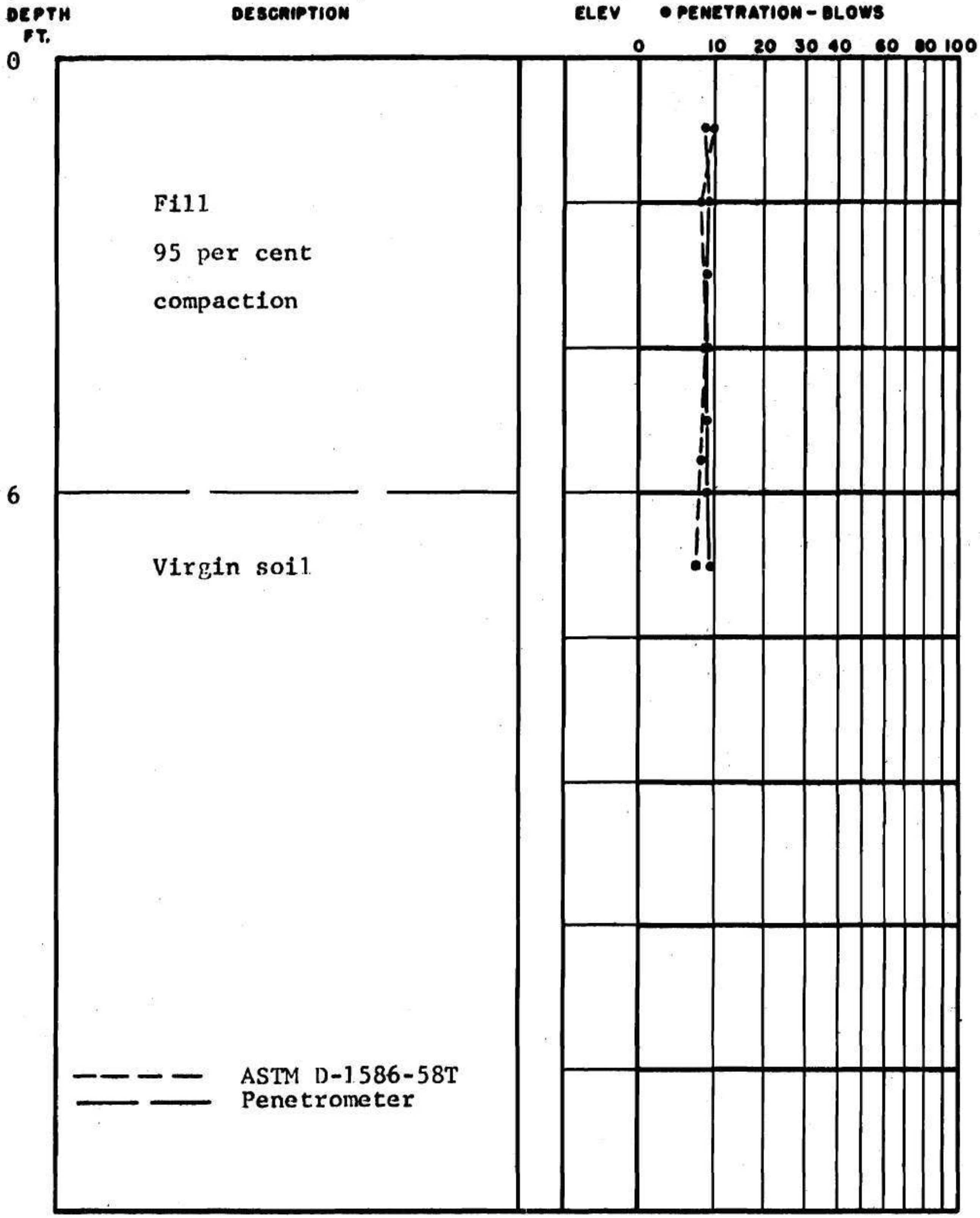


Fig. 6B Test Boring Data - Test Section 3

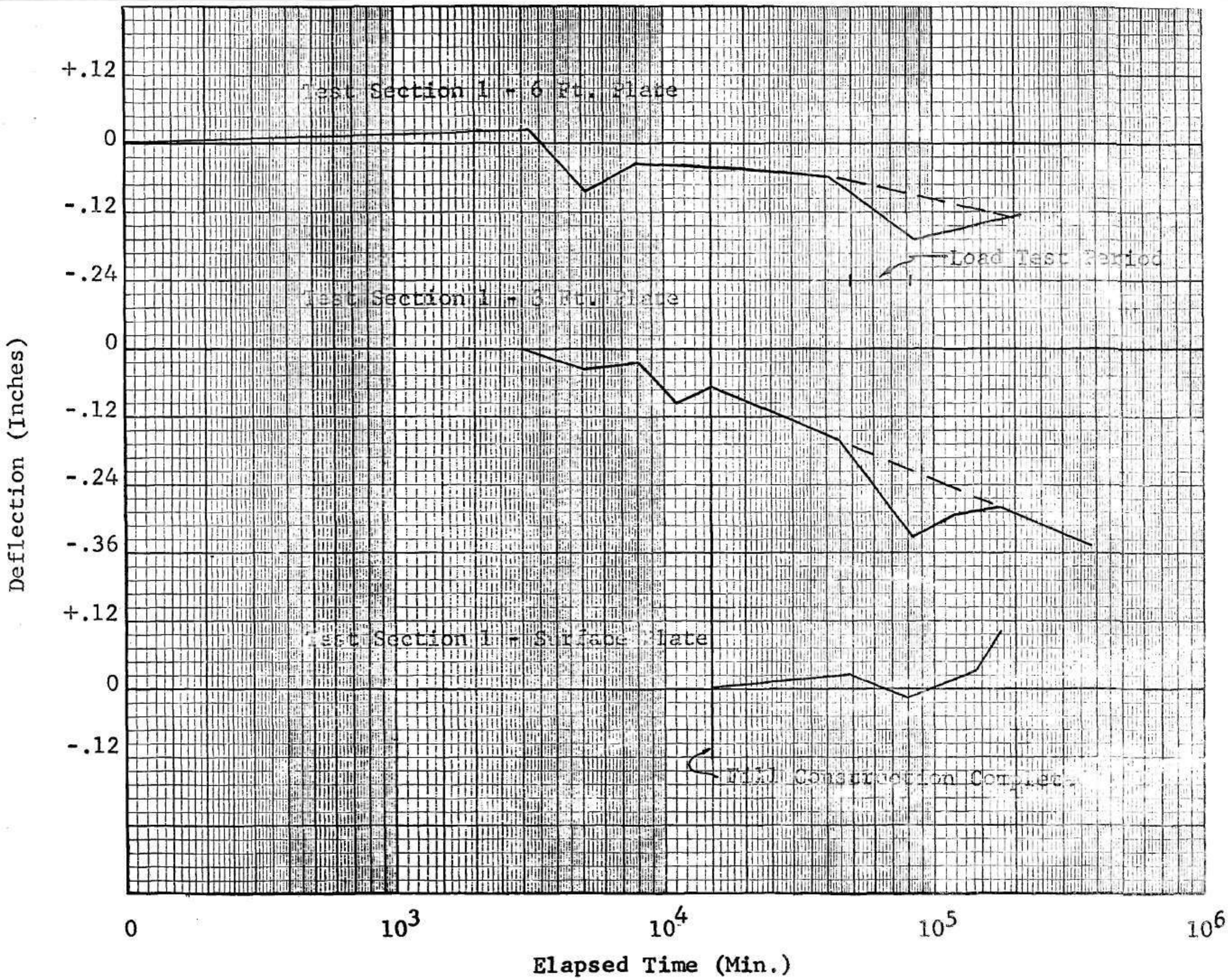


Fig. 7 Inplace Settlement Plates

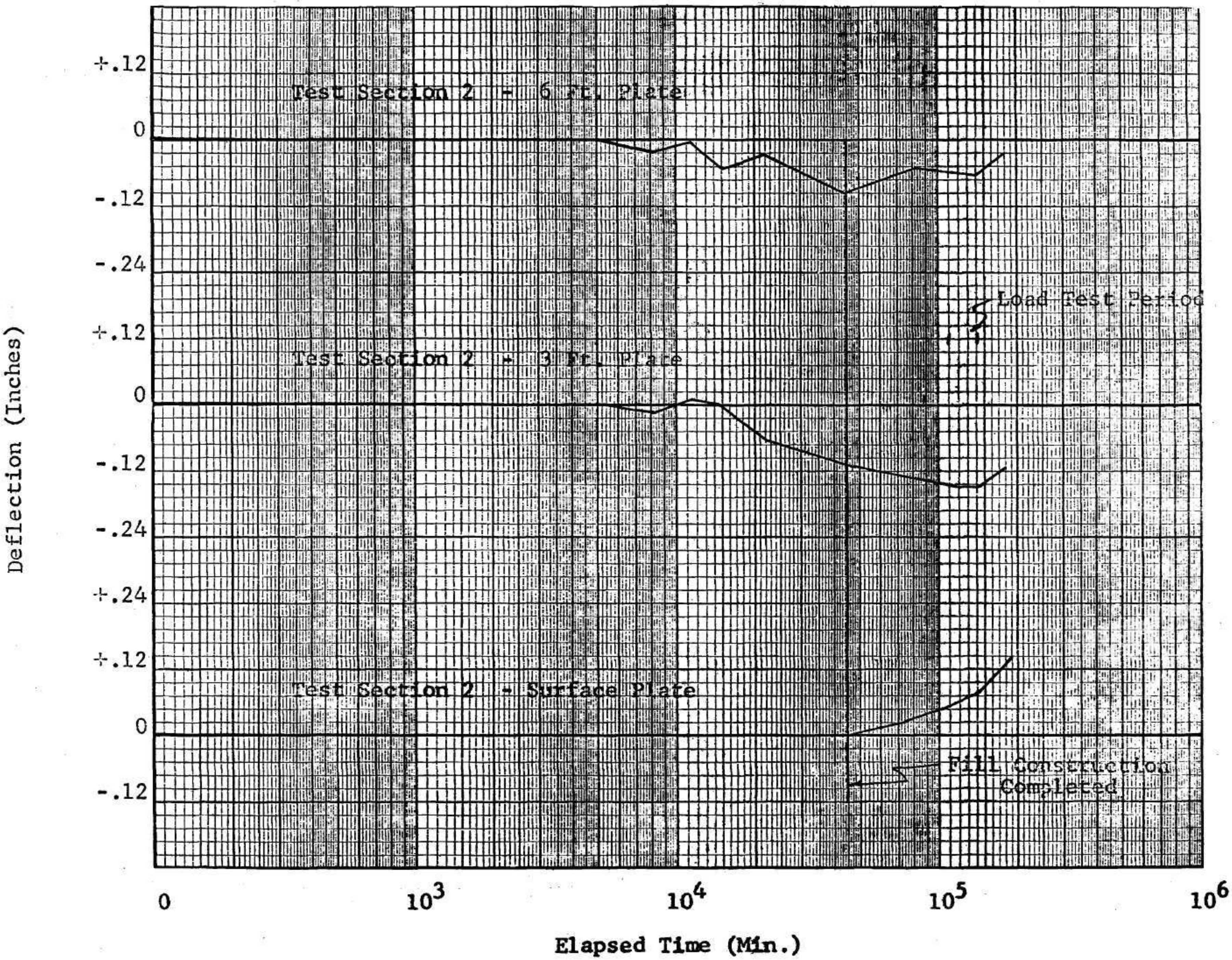


Fig. 7a Inplace Settlement Plates

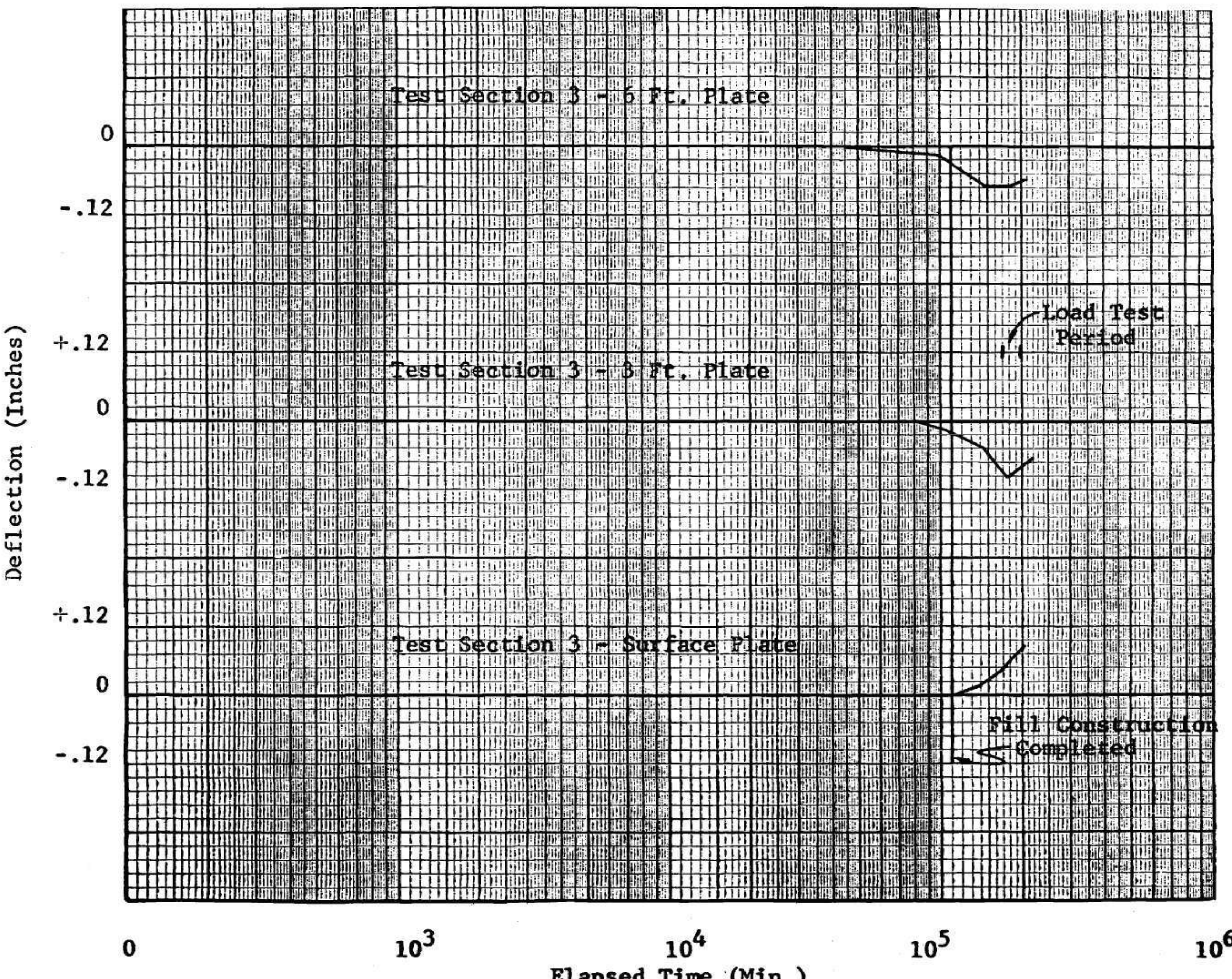
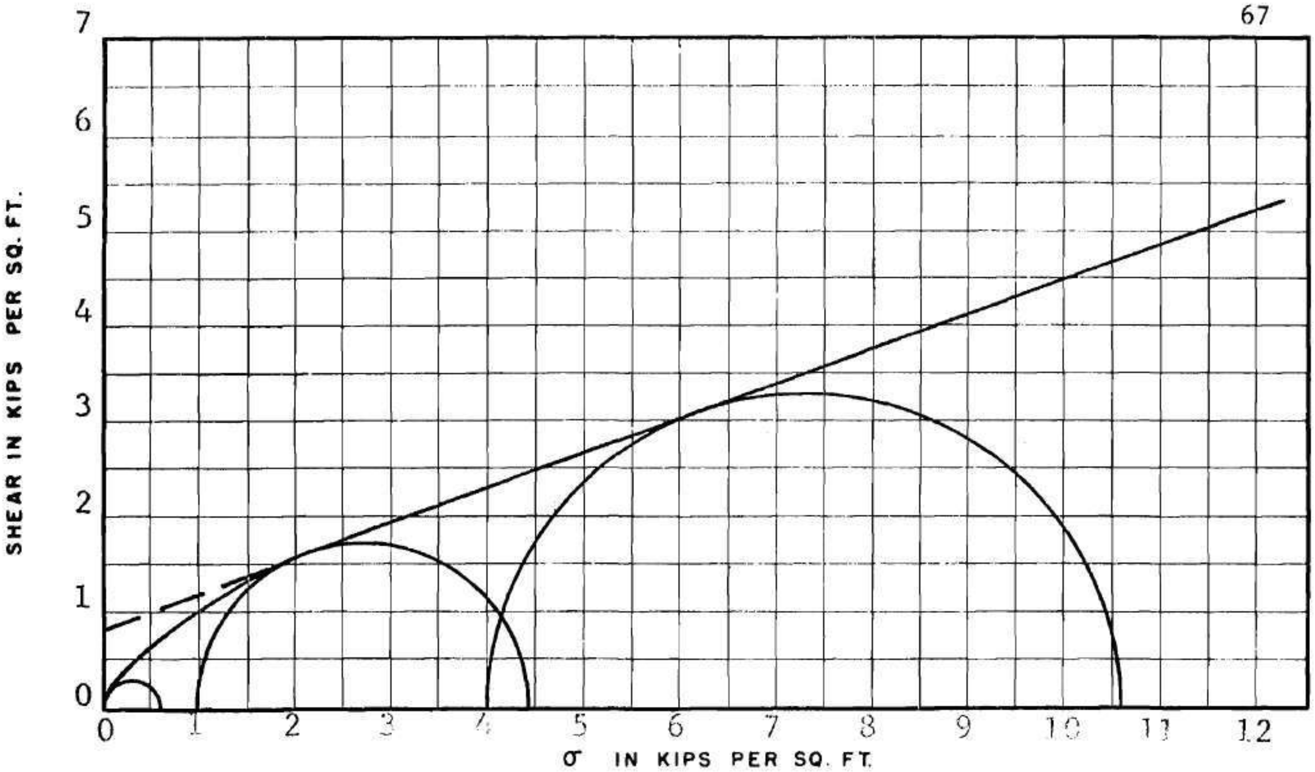
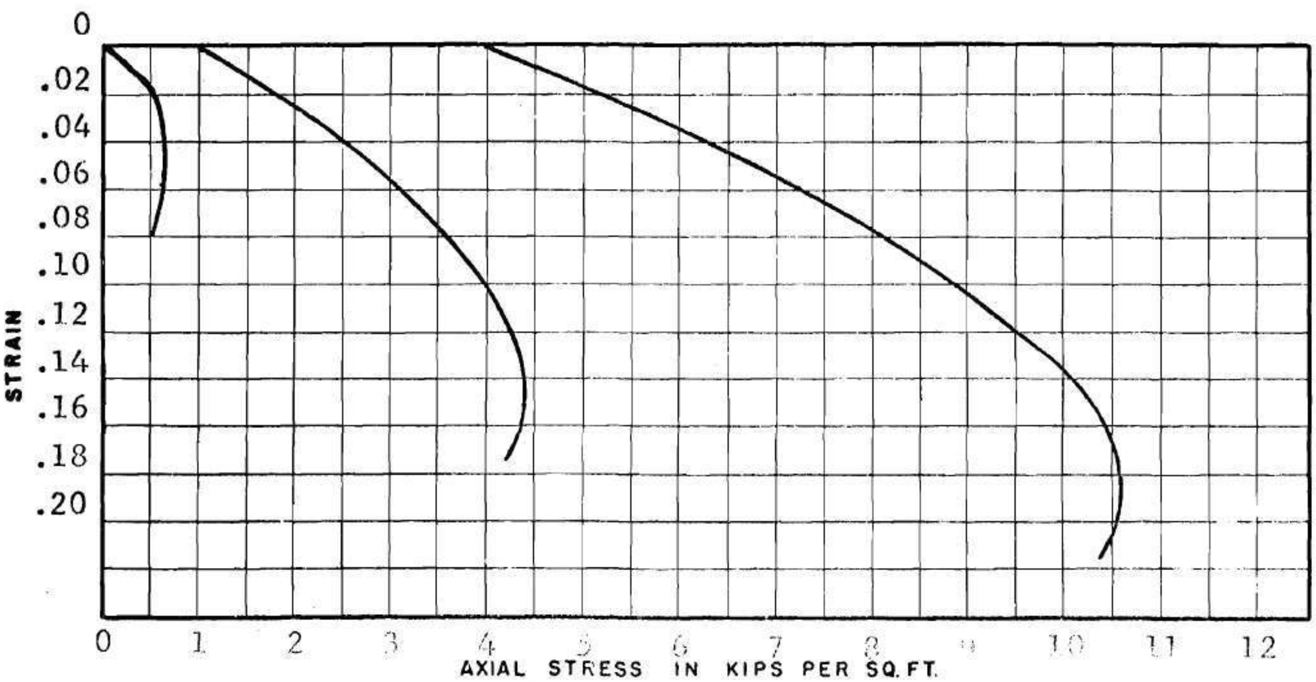


Fig. 7b Inplace Settlement Plates



MOHR DIAGRAMS — ϕ



STRESS-STRAIN CURVES

c″ = .100 KSF

"COHESION", c′ .800 KSF

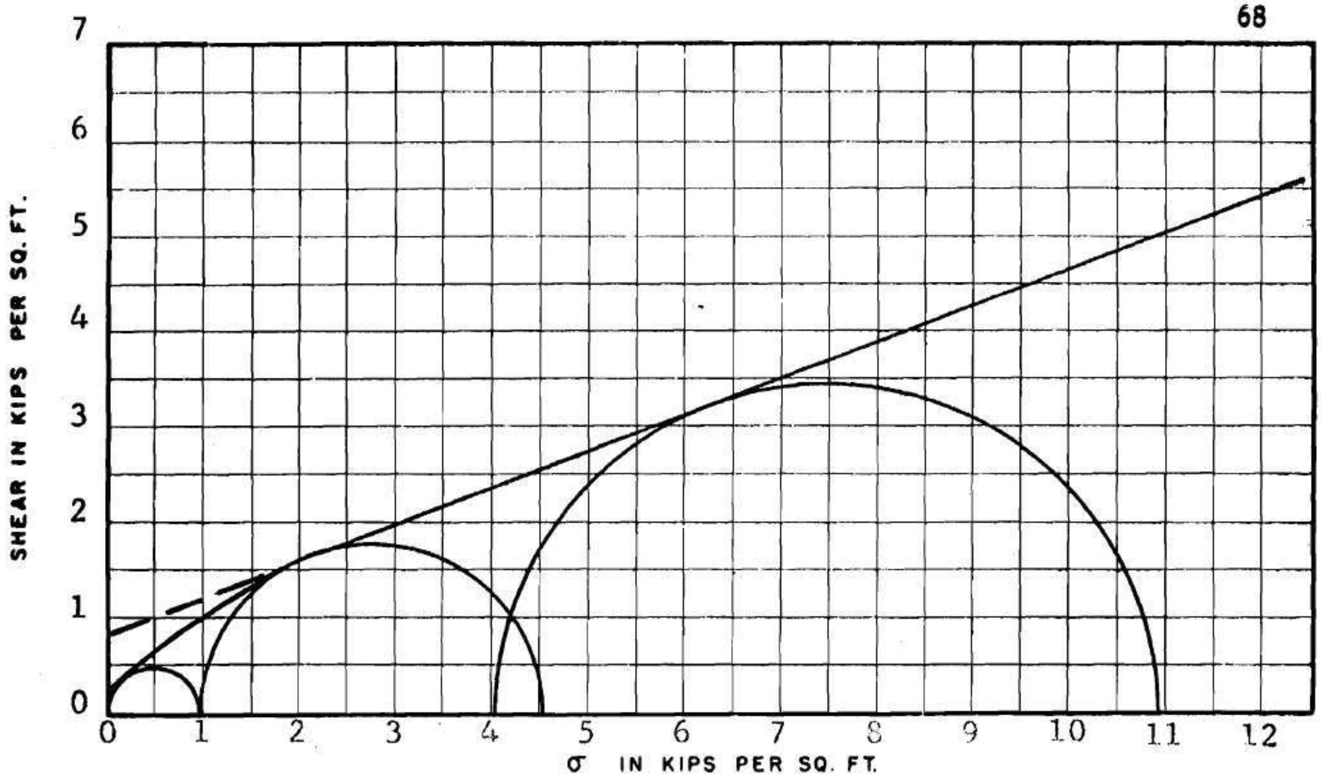
ANGLE OF SHEAR RESISTANCE, ϕ 21°

UNIT WEIGHT, γ 98.6 (γ_o = 80.3 PCF)

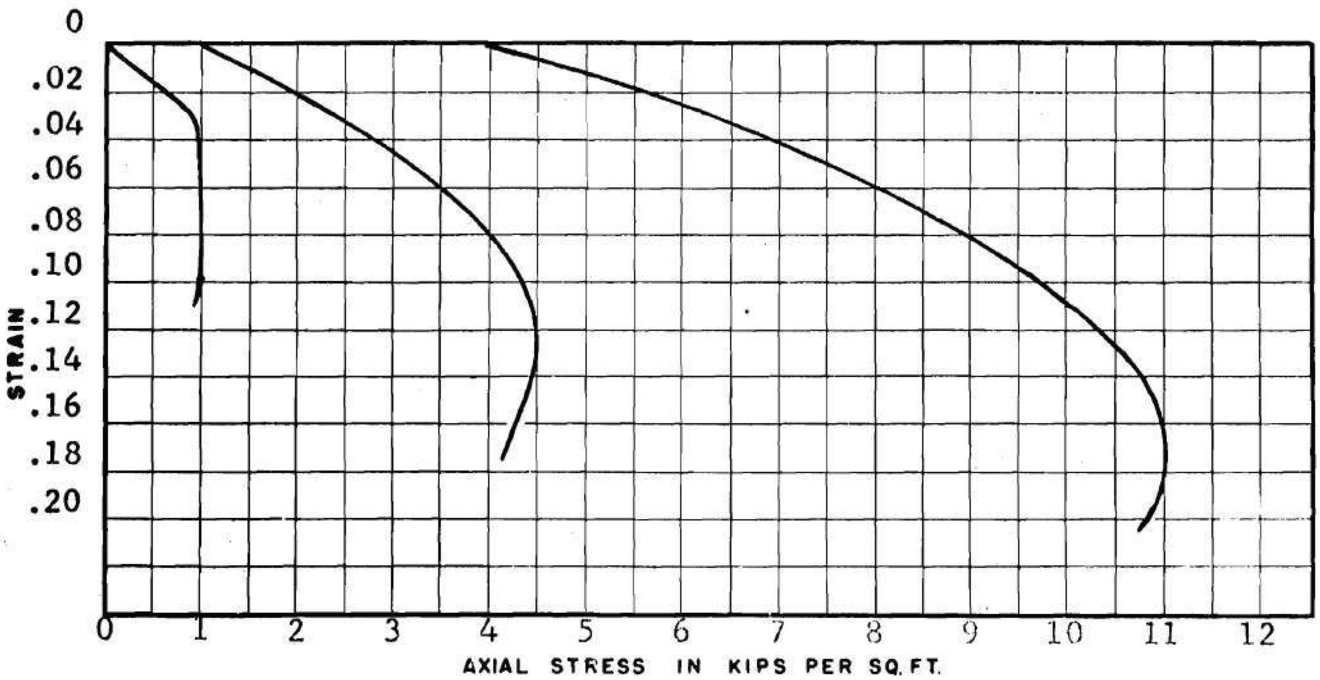
WATER CONTENT, w 23.0%

VOID RATIO, e 1.08 s= 58%

Fig. 8 Triaxial Shear Data
Test Section 1



MOHR DIAGRAMS — ϕ



STRESS-STRAIN CURVES

c'' = .250 KSF

"COHESION", c' .800 KSF

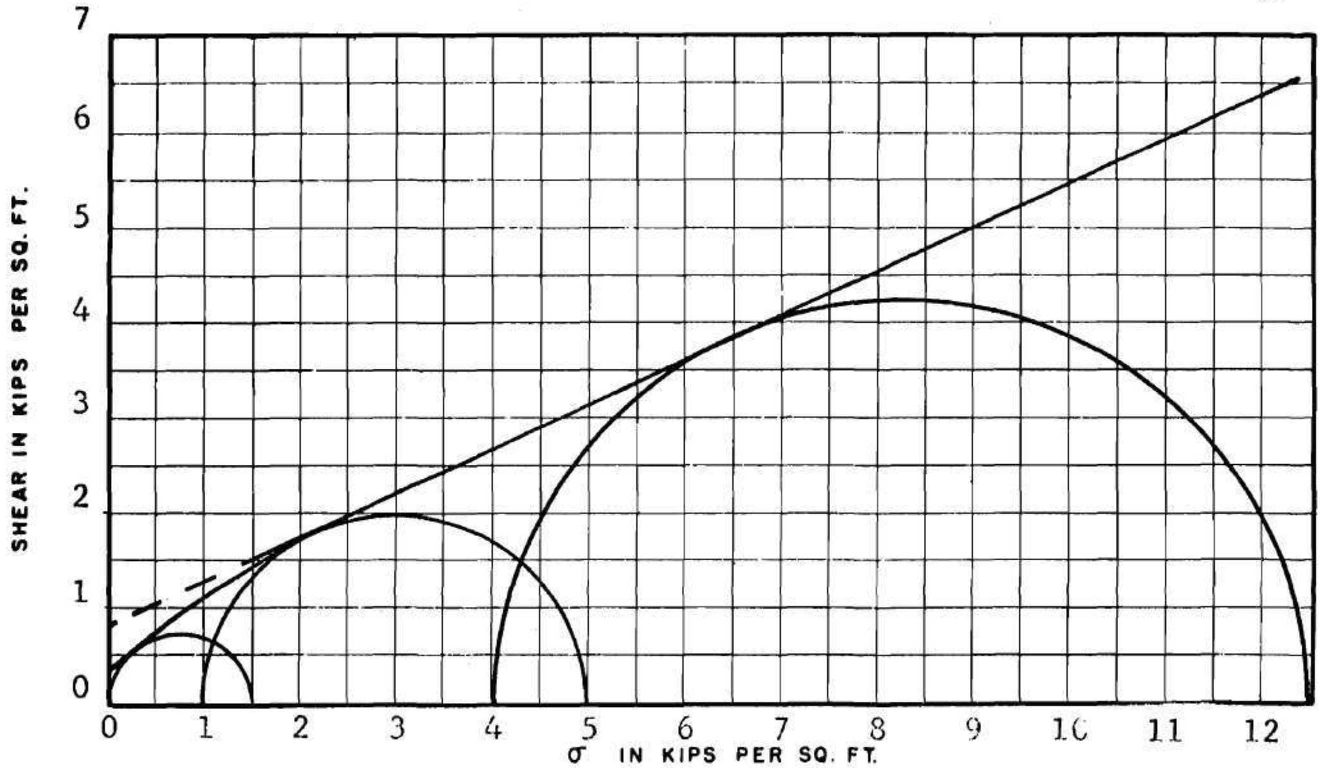
ANGLE OF SHEAR RESISTANCE, ϕ 22°

UNIT WEIGHT, γ = 105.2 (γ_D = 85.0 PCF)

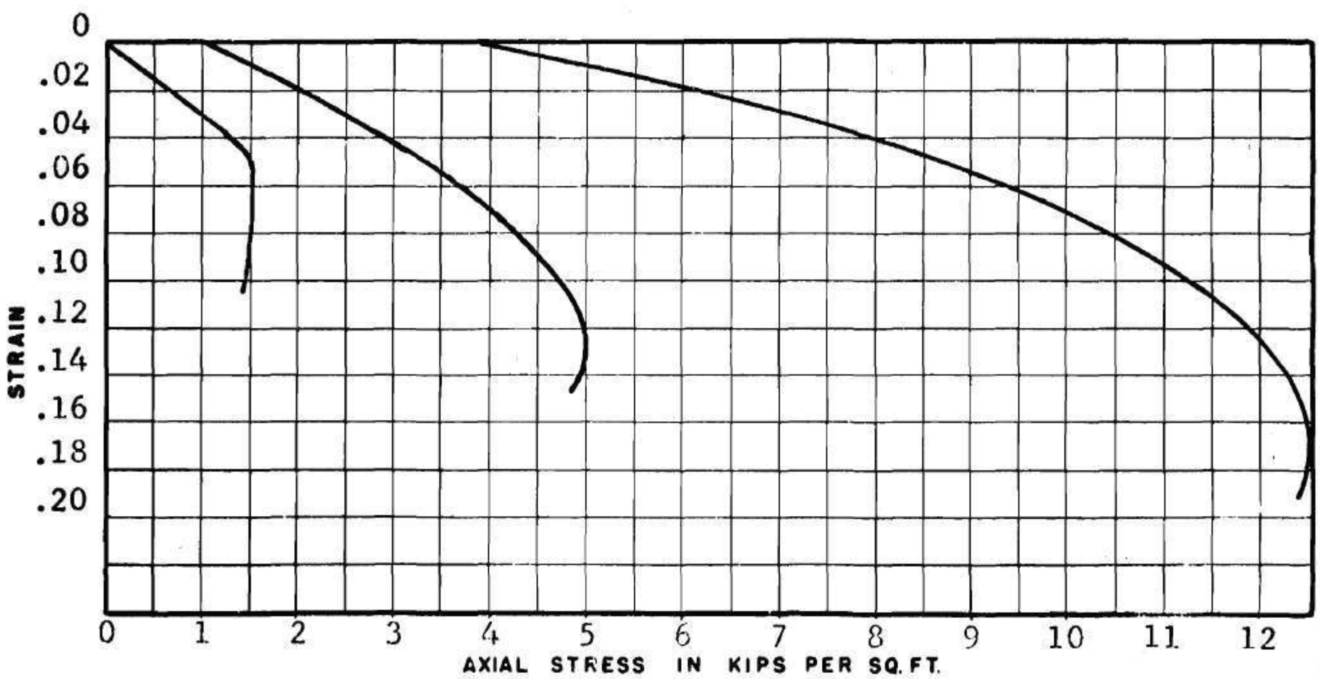
WATER CONTENT, W 23.8

VOID RATIO, e .98 s = 66%

Fig. 8A Triaxial Shear Data
Test Section 2



MOHR DIAGRAMS — ϕ



STRESS-STRAIN CURVES

c" = .400 KSF

"COHESION", c′ = .800 KSF

ANGLE OF SHEAR RESISTANCE, ϕ 25°

UNIT WEIGHT, γ 111.3 (γ_o = 89.8 PCF)

WATER CONTENT, w 24.0%

VOID RATIO, e .86 s = 77%

Fig. 8B - Triaxial Shear Data
Test Section 3

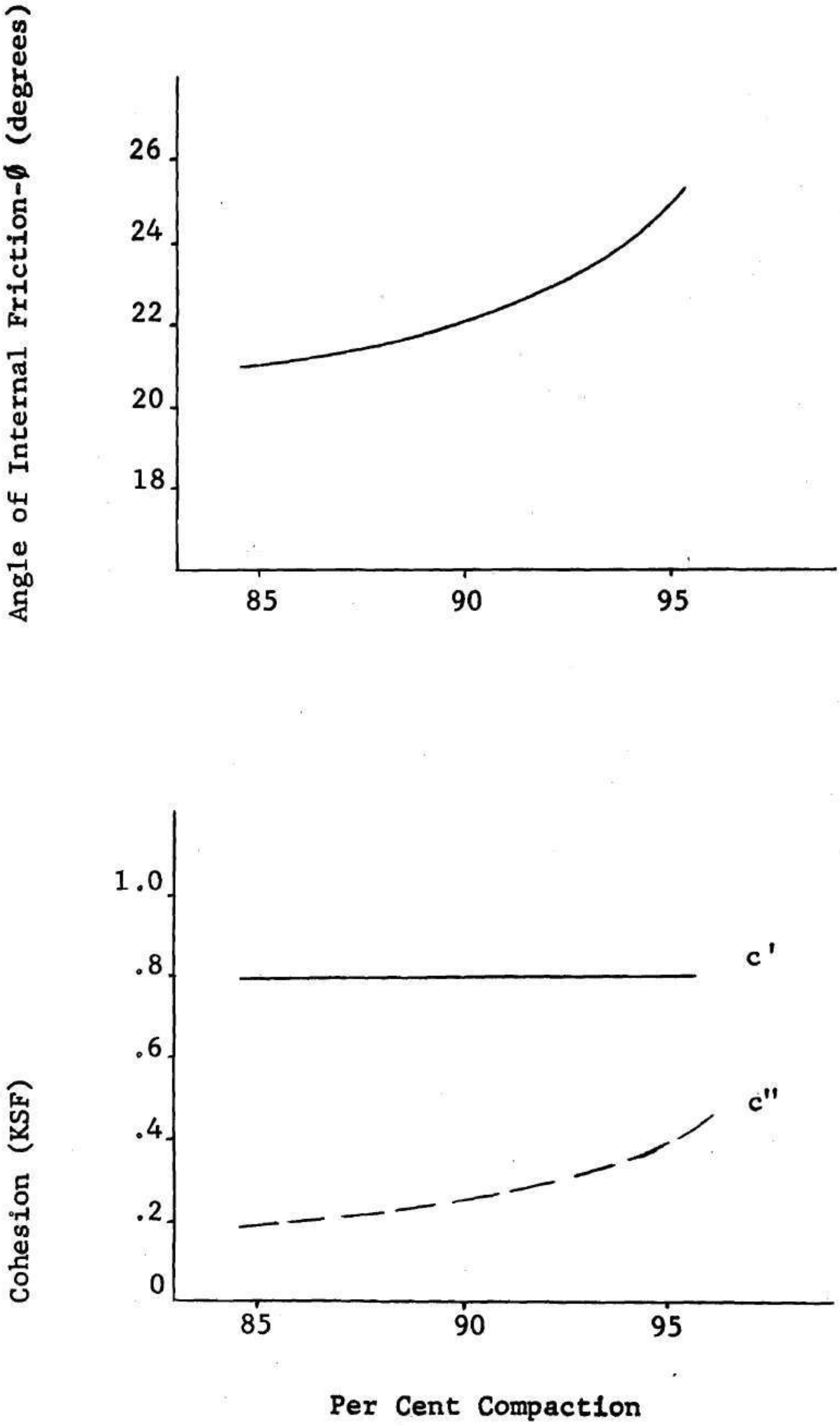


Fig. 8c Compaction vs Strength Characteristics

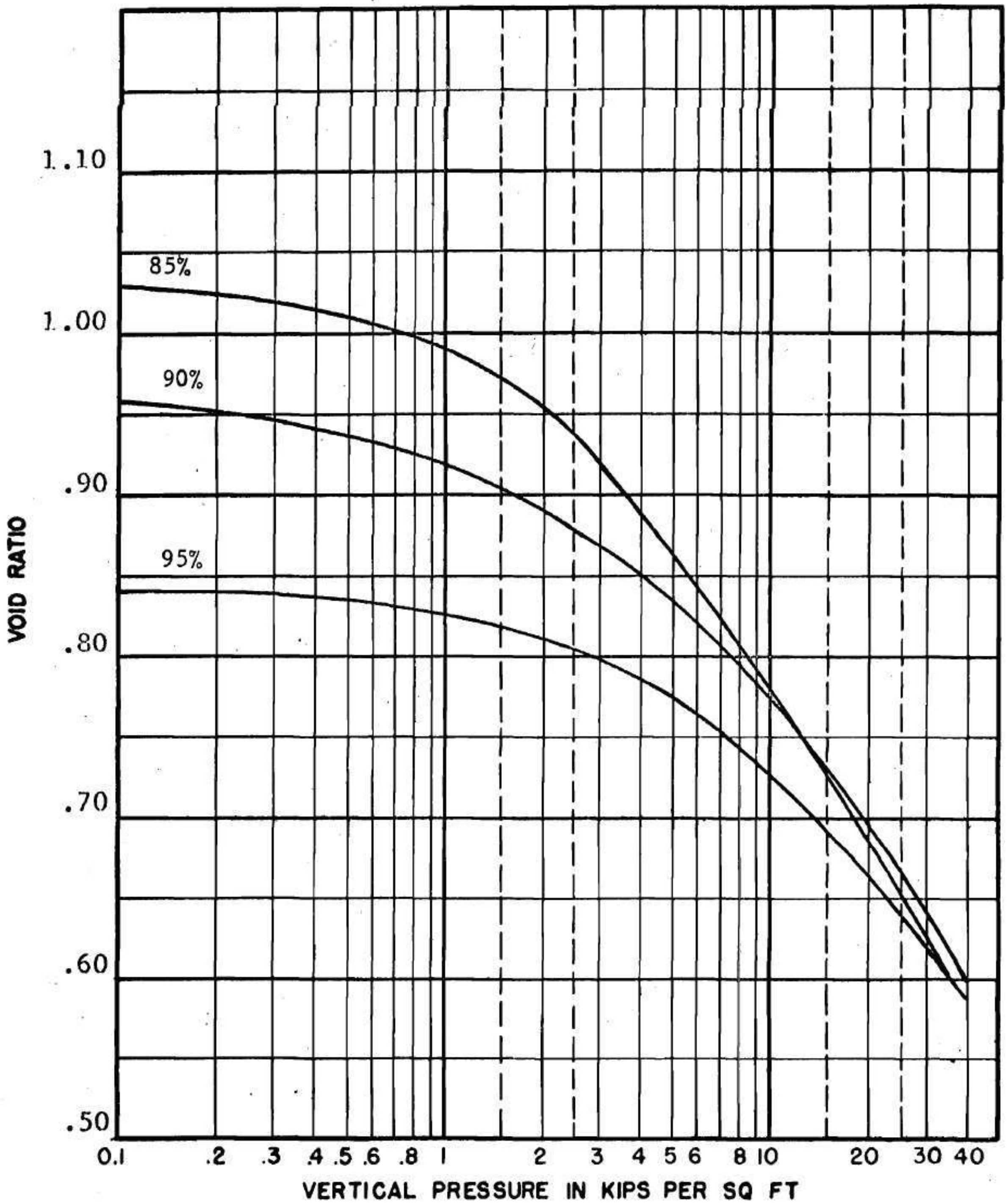


Fig. 9 Consolidation Test Data

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