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THE BEARING CAPACITY OF DRILLED PIERS ON PARTIALLY DECOMPOSED ROCK

A THESIS

Presented to

the Faculty of the Graduate Division

by

James Vernon Walters

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THE BEARING CAPACITY OF DRILLED PIERS ON

PARTIALLY DECOMPOSED ROCK

Approved: ______________________
George F. Sowers

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Dr. David B. Comer III

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Don B. Jones

Date Approved by Chairman: May 29, 1959

FOREWORD

The purpose of this research has been to determine the load-bearing characteristics of drilled piers on partially decomposed rock. This objective can be more completely defined by four questions: First, what is the ultimate bearing capacity of a drilled pier upon partially decomposed rock? Second, what is the relationship between the load on such a pier and the pier's settlement? Third, is settlement of a pier a function of time? Fourth, what are the relationships between the physically measurable qualities of the unsound rock and the load-bearing characteristics of a pier supported on such rock? To answer these questions, three piers were constructed and subjected to load tests; then undisturbed samples of the partially decomposed rock which supported the piers were excavated and tested.

The construction and testing of the piers were relatively expensive, and I am deeply indebted to the people and organizations who loaned or gave the labor, materials, services, and equipment which made this research possible. To the Georgia Institute of Technology upon whose land the piers were constructed, to the State Highway Department of Georgia for the use of testing equipment, to the Southern Railway System and the Henry Newton Company for the loan of hydraulic jacking equipment, and to Calvert Iron Works, Inc., for the use of a loading beam, I am very grateful.

Without the help of Law Engineering Testing Company which furnished hydraulic jacking equipment and performed subsurface soil investigations on the test site, it would have been impossible to correlate the pier test results with information that is usually obtained during the subsurface investigations of a proposed building site. Those who contributed most materially to this research, however, are Mr. Jack R. McKinney, President of McKinney Drilling Comapny, and his staff. I am very appreciative of the efforts and time the staff expended in my behalf and of the expense Mr. McKinney has borne to finance the project.

To Professor George F. Sowers, thesis advisor, who has given encouragement and guidance in the planning and execution of this research; and to Professor Don B. Jones and Dr. David B. Comer III, who have served on my thesis reading committee, I am greatly indebted.

Finally, I dedicate this thesis to my wife, Barbara, who has worked with me so faithfully and patiently in all phases of the work.

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SUMMARY

Drilled piers are a type of deep foundation, *i. e.*, the piers are supported by a material of relatively high strength which is overlain by a soil of poorer quality that would not be capable of withstanding the load. In the Piedmont Region, for example, drilled piers can be utilized to support large loads at high contact pressures if the piers are founded upon the metamorphic bedrock of the area. However, the bedrock is often covered by a layer of partially decomposed rock which is difficult to remove. This material, a product of weathering of the bedrock, also has been observed to withstand heavy loads. The purpose of this research was to develop a method of accurately predicting the load-supporting ability of drilled piers founded upon this weathered material.

A subsurface soil exploration was performed to locate a suitable stratum of partially decomposed rock upon which test piers could be constructed. Both a hand auger and a wash-boring drill were used to locate the upper surface of the weathered rock. Then a pier-drilling machine was used to drill a hole three feet in diameter to allow visual examination of the weathered rock and to allow the recovery of chunk samples of the material for unconfined compressive strength tests. Full load tests were performed upon two piers eight inches in diameter and one pier twelve inches in diameter. Loads were applied in 10,000-pound increments, and settlement was recorded on a predetermined time schedule until the rate of settlement became less than 0.005 inches per hour.

An analysis of the results of pier load tests and of the results

of the unconfined compressive strength tests of the undisturbed rock samples indicate three significant relationships: First, the unconfined compressive strength of partially decomposed rock is a good index of the quality of the rock. Second, the ultimate bearing capacity of drilled piers on partially decomposed rock can be predicted with the use of an analysis developed by Bell. Third, the magnitude of pier settlement measured during the load tests indicates that statically indeterminate buildings can be constructed on such foundations without dangerous settlement.

It is recommended that further load tests of drilled piers on partially decomposed rock be performed. The piers to be tested should be founded on varying qualities of the weathered rock. Such tests should indicate the range of rock strengths over which the use of Bell's analysis is proper, and might also reveal a relationship between rock strength and pier settlement.

CHAPTER I

INTRODUCTION

Drilled piers are a relatively new type of deep foundation. The term "deep foundation" is generally used to describe a foundation which extends down through a layer of weak, or otherwise objectionable, soil and is supported by an underlying layer that has sufficient strength to support the given load at a relatively high contact pressure. Partially decomposed rock is one foundation material that has been observed to support satisfactorily the loads placed upon it by drilled piers. The presently reported study has been performed to develop an analysis that will allow the accurate prediction of the bearing capacity of drilled piers on partially decomposed rock.

The tests performed by the author were executed in Atlanta, Georgia, in the Piedmont Region, where partially decomposed rock is often available as a foundation material. The Piedmont Region is a broad strip extending from central Alabama across Georgia, the Carolinas, and Virginia, and tapering out to an end in the vicinity of Baltimore and Philadelphia. The western boundary of the Piedmont Region is the Blue Ridge range of the Appalachian Mountains. The eastern boundary is formed by the Coastal Plain sediments which overlap the rocks of the Piedmont Region in an irregular contact known as the Fall Line.

The features which characterize this region and distinguish it from surrounding regions are products of its geologic history. The bedrocks of the region are crystalline metamorphic rocks, princiaplly gneisses and schists, which have been jointed and faulted and have been intruded by igneous rock masses of various shapes. Since the metamorphism and subsequent diastrophic upheaval of the underlying rock, erosion has partially removed its overburden and weathering has created the soil profile characteristic of the Piedmont Region.

The top stratum consists of several feet of red clay which may include angular quartz particles, weathered mica, kaolinite clays, and iron oxides. The second stratum is a more granular soil which grades from sandy silt at the top to silty sand in the lower part. The third stratum is a layer of extremely variable thickness and composition. It contains strong, hard rock which is fractured in irregular, interlocking patterns, lenses of weaker rock, and even irregular soil lenses. The fourth stratum is the sound bedrock.

In the strata between the top, highly oxidized clay layer and the bedrock, preferential weathering along the faults and joints has allowed the weathering agents to attack the sound rock along a very irregular surface. As the weathering has continued along the joints and faults, it has left lenses or sheets of hard rock enclosed between layers of weathered rock or soil. Because of these lenses, foundation investigations are more involved and complicated in the Piedmont than in some other regions.

When a building is to be built on a typical site in the Piedmont, the designer has several alternatives in selecting the material upon which to place the foundations. A light structure may be erected on the hard, cemented, desiccated clay layer on top; but settlement of the softer, underlying silt must be considered. Very light buildings might be supported by large bearing surfaces upon the second stratum, which is principally sandy silts and silty sands. Extremely heavy buildings can be supported by deep foundations which rest upon the bedrock of the region; but to place foundations upon this rock, which is able to support very great loads on small areas, the builder must excavate the stratum of fractured-but-hard (unsound) rock which overlies it. In some cases this layer is as much as twenty feet thick.

The three alternative types of foundations already mentioned can be designed by the use of rational analyses which consider the usual measurable soil properties. A fourth alternative is to use the unsound rock stratum to support medium-sized and heavy buildings. This stratum is able to support larger loads and can withstand higher stresses than can the top layer of clay or the second layer of silts and sands, but it is not able to withstand stresses as great as those which the bedrock can withstand. Therefore, for the same magnitude of total load, foundations on the unsound rock must be larger than foundations which are designed to support the same total loads and rest on the bedrock. The increase in cost for the larger foundations on the unsound rock is compensated for, however, because both the excavation to allow the placing of deep foundations on the bedrock and the driving of piles through the unsound rock are difficult and costly.

The three deterrents to the use of foundations upon the fractured-but-strong, unsound rock are the difficulty of locating a satisfactory rock stratum, ascertaining its thickness and continuity; the difficulty of determining and correlating its strength properties; and the lack of a proved method of design for such foundations. The object of this research was to lessen some of these difficulties as they affect the design of one special type of deep foundation, the drilled pier. The drilled pier differs from other types of piles, piers, and caissons chiefly in its manufacture. Usually, a truck-mounted, gasoline-powered, rotary drilling machine is used to drill a straight, vertical shaft into the soil to the depth of the pier bottom (see Fig. 1). The drilling is done with a helical bit (see Fig. 2). These bits are usually available in sizes from one foot to eight feet in diameter. The drill is rotated and simultaneously forced into the earth until it is full of soil; then it is raised to the surface and rotated at a speed sufficient to sling the soil from the bit. Laborers shovel the soil away from the hole, and the process is repeated until the desired depth is reached.

If low allowable bearing stresses make a large contact between the underlying soil and the pier bottom desirable, a reaming bit may be used to ream a conical section at the bottom of the shaft. The bit is roughly cylindrical and has wings or vanes which are extended when the bit is forced against the bottom of the hole (see Fig. 3). By the use of this type of tapering bit, the bottom of the hole may be enlarged to about twice the shaft diameter. Casing is available where unfavorable water conditions or caving in of the sides are encountered; laborers with air hammers can cut through rock lenses; and various other equipment and techniques are available for use under other special conditions.

When the hole has been cleaned out and approved by the inspector, any required reinforcing steel is tied and placed in the hole, and the concrete is poured and vibrated into the desired dense condition in the hole. If the casing was used, it is removed just before or just after placing the concrete, as job conditions dictate.

Even a brief description of the manufacture of drilled piers suggests

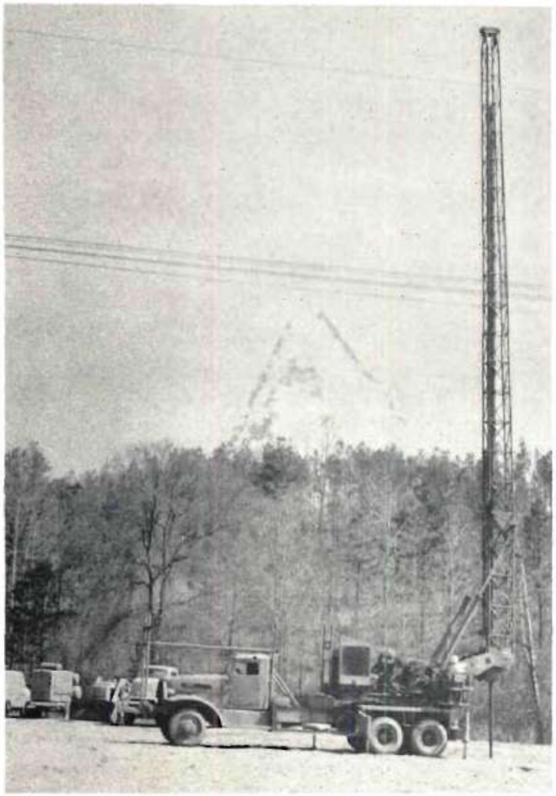


Fig. 1. Pier Drilling Machine

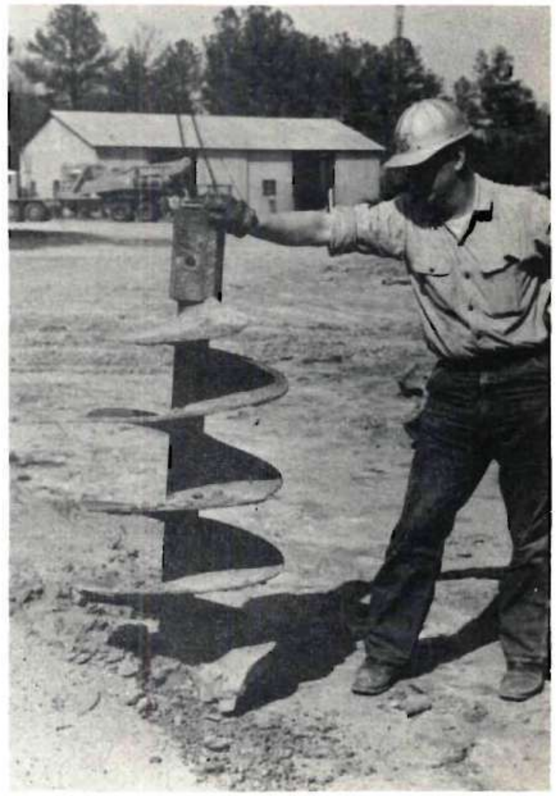


Fig. 2. Drilling Bit for Straight Shaft Piers

Fig. 3. Reaming Bit

many of their technical and economical advantages. First, on jobs where piers twenty-four inches in diameter or larger are used, the engineer may enter the hole and inspect, sample, and test the material on which the pier will rest. Driven, cast-in-place piles usually are too small to allow this close examination of the foundation supporting material. Of course, steel "H" piles or precast concrete piles preclude such examination.

Second, visual examination during drilling and the use of men and air hammers in the hole allow the removal of thin, but strong, layers of rock which could stop, deflect, or break a driven pile.

Third, a very important economical advantage is that, for a foundation of given size and depth, minimum excavation is needed. Unlike a spread footing excavation, the pier excavation need only be made as large as the pier will be.

Fourth, since the drilling process depends chiefly upon the cutting action of the bit rather than upon percussion, or shock and displacement, to advance the hole, very little damage or none is done to a previously drilled hole or poured pier by drilling another hole close to it. This is in contrast to a phenomenon encountered in pile driving. Sometimes the driving of piles causes others previously driven to be pushed up out of the soil or, in the case of cast-in-place, cased piles, causes the collapse of driven, but unpoured, casings.

Fifth, the piers are made of common building materials. Few indeed are the localities in the United States where concrete is not available, and only a small amount of the important commodity, reinforcing steel, is used in drilled pier construction.

Sixth, drilled piers can be used economically on some projects where special conditions such as congested working areas or time limitations preclude the use of other types of foundations. A railroad construction project in Atlanta is a good example of such a use. This job consisted of lengthening a four-lane highway bridge over the railway while maintaining rail and highway traffic. The piers were drilled and poured in cylindrical cardboard forms. Then the deck for a highway bridge was built on the ground on the piers. Soil was subsequently excavated from beneath the bridge deck to make way for rail switching yards. The bridge is supported by its drilled-pier columns, which continue on into the soil as its drilled-pier foundations. The cardboard forms were removed to leave an almost perfectly smooth surface which required little finishing.

When one considers all the technical and economical advantages of the drilled pier and begins to analyze its value and utility, he immediately thinks of the high bearing capacities and the inherent stability of the sound bedrock of the Piedmont Region. However, the depth of the rock below the surface of the ground and the difficult excavation of the weathered rock overlying it sometimes make founding of the pier on bedrock too expensive for all but the heaviest of structures.

The next questions one asks are how much load can be placed on the unsound rock, how much settlement can be expected, and how can these two quantities be predicted. Generally, the purpose of this study is to attempt to answer these three questions. The strength of weathered rock, the relative ease of excavation of the soil above it, the occurrence of the material over such a large area, and the inherent economy of the drilled pier make it very desirable to formulate design criteria for the drilled pier on partially decomposed rock.

Because the drilled pier is a relatively new type of foundation, very little research has been performed in fields related to its use. In Texas and California, where the drilled pier has been used fairly extensively, tests have been performed to evaluate skin friction and end bearing capacity of the piers in plastic clays. Mr. Lawrence A. DuBose (1) performed such tests at the Agricultural and Mechanical College of Texas.

In the Union of South Africa, Mr. L. E. Collins (2) has formulated criteria for the design of drilled piers which must resist tensile forces caused by the swelling of the expansive clays of that area.

A search of the engineering literature reveals that, to date, no research has been performed upon the problem of determining the bearing capacity of drilled piers on partially decomposed rock. This is explained by the fact that, to date, research on drilled piers has been undertaken in localities where drilled piers have been used more extensively and for a longer time, and where the soil conditions happen to be different from those found in the Piedmont.

CHAPTER II

SITE LOCATION

Five main factors were considered in the choice of a site for the experimental part of this research. First, approximately a fifth of an acre of rent-free land was needed. Second, for the sake of convenience and economy, partially decomposed rock should be found between the depths of ten and twenty feet. Third, it was felt that the stratum of unsound rock upon which a test pier would rest should be fairly homogeneous below the pier for a distance about twice the diameter of the pier. Fourth, the ground water table should be below the pier bottoms to allow simple construction and rock sampling techniques. Fifth, it was desirable to have the test site near the laboratory for economy of time and for ease in transporting samples and equipment to and from the site.

It was found that two lots owned by the Georgia Institute of Technology were available for use as test sites. Hand-auger borings on the first lot indicated that any underlying rock was at a depth greater than twenty-five feet. The absence of a rock stratum near the surface ruled out the use of this lot, so exploratory hand-auger borings were begun on the other lot.

Seven hand-auger holes were bored in an area roughly forty feet in width and fifty feet in length. The holes were bored to refusal. An isoplethic map of the area prepared from data obtained from the hand-auger exploration indicated a rock layer which dips approximately 17 degrees in a southeasterly direction (see Fig. 4). In the area where borings were made, depth to the stratum varied from nine feet to sixteen feet.

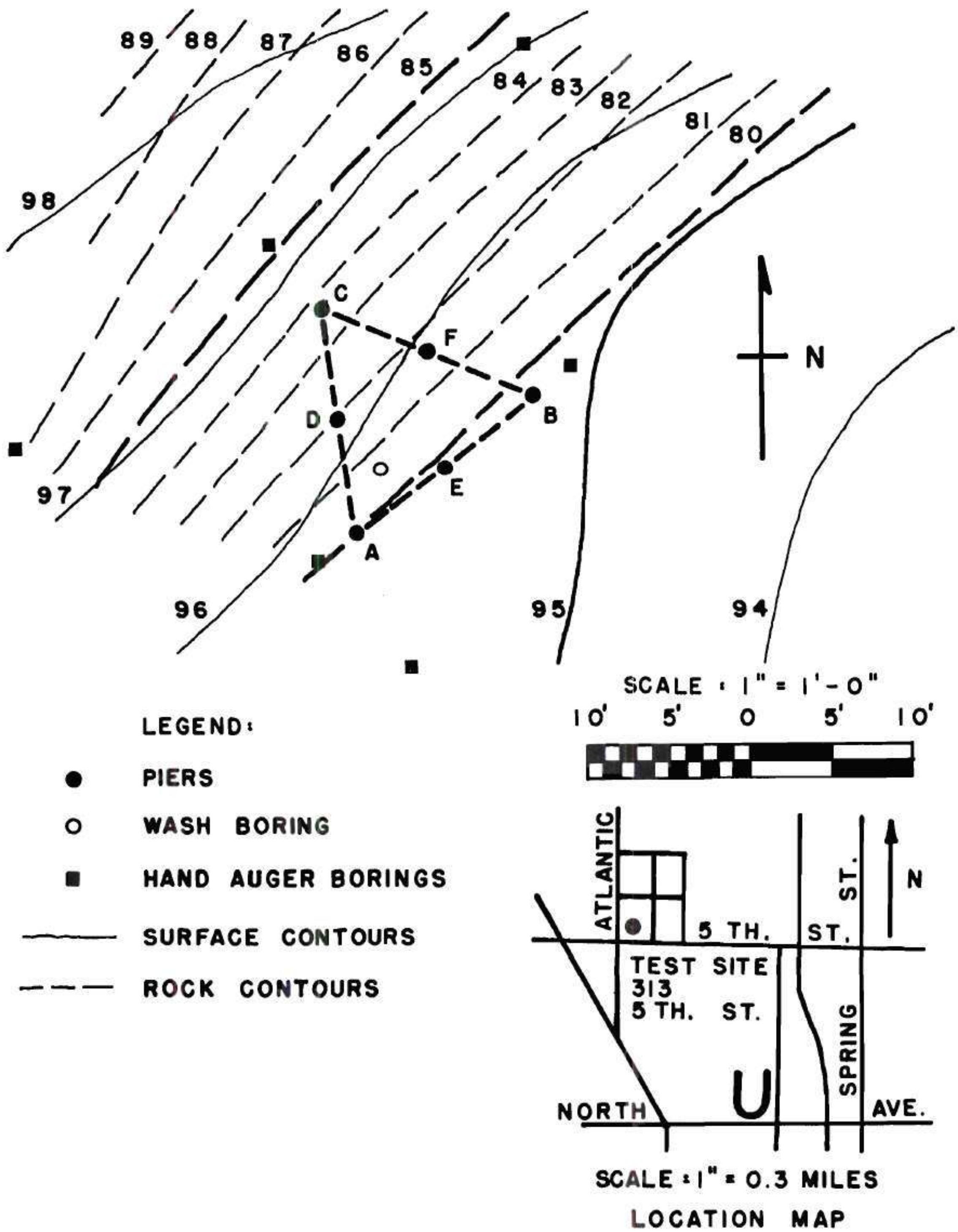


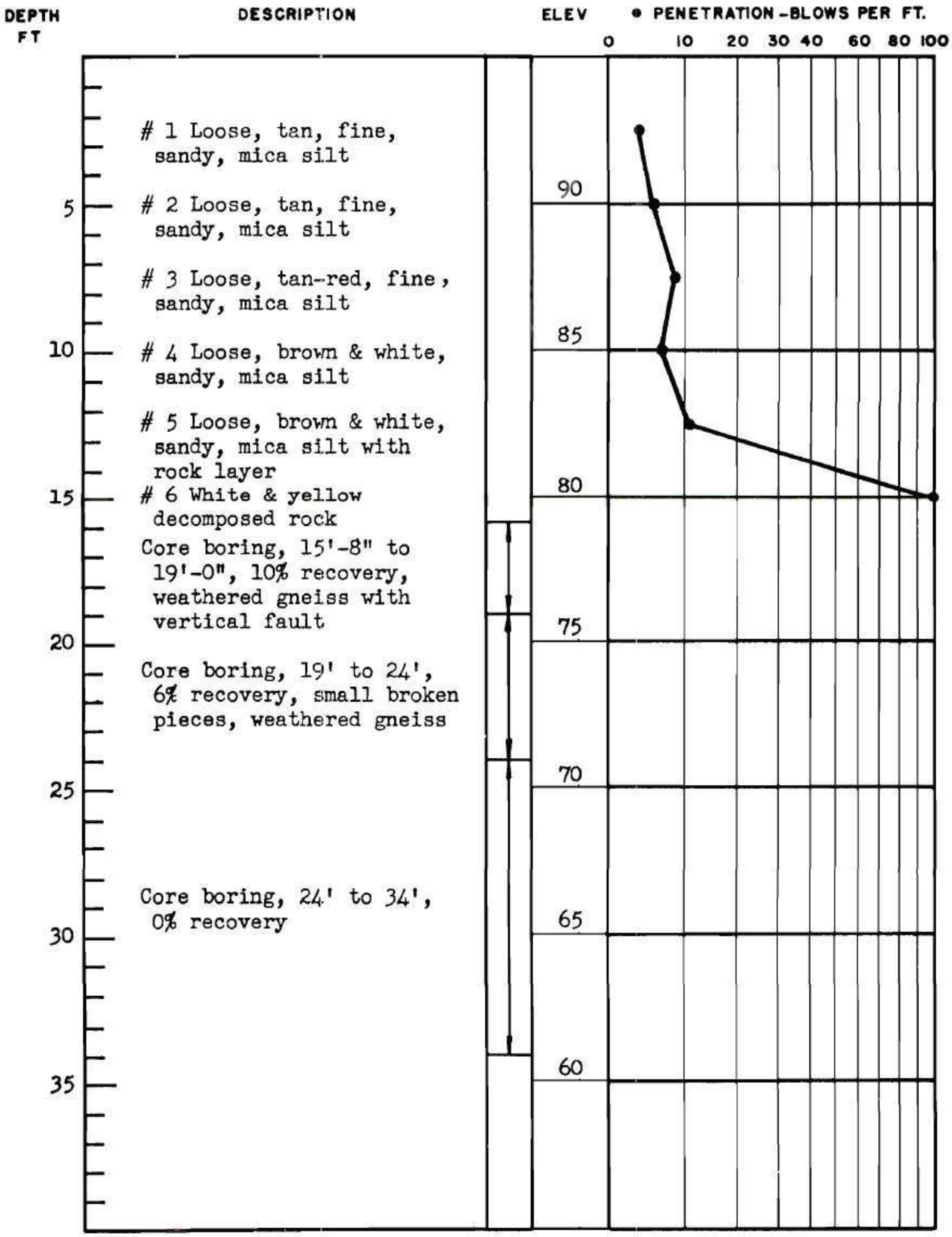
Fig. 4. Map of Test Site

To check the presence of such a rock stratum, a combination soil-sampling and diamond-core drilling rig was used to drill another exploratory hole (see "Soil Boring Log," Fig. 5). Split-spoon samples were taken at two-and-one-half-foot intervals. The one-and-one-half-inch inside-diameter sampler was driven by 30-inch free fall blows with a 140-pound hammer. As the sampling proceeded, the driving resistance (the number of blows of the hammer per foot of sampler advance) increased from 4 at 2.5 feet to 11 at 12.5 feet. Refusal was met at 15 feet where 60 blows were required to advance the sampler 7 inches.

At the point of refusal, core boring was begun. For the first eight feet of coring, only eight inches of rock sample was recovered. The core samples gave clear indication that the rock sampled is unsound or weathered. Below the depth of thirty-one feet, little resistance was met, and coring was discontinued at a depth of thirty-four feet. The slight resistance to coring after thirty-one feet indicates that the weathering agents have developed the erratic subsurface conditions characteristic of the Piedmont and that the partially decomposed rock layer contains soil lenses and pockets.

To allow undisturbed rock and soil sampling and to permit visual examination of the subsurface strata, two holes three feet in diameter were drilled near the proposed test location with a pier-drilling machine. The holes were drilled to refusal, and later one was excavated with hammer and chisel to obtain rock samples and to determine the thickness of the particular rock lens. The thickness of the rock which was free of soft seams was about two feet.

Undisturbed soil samples were taken from the side of the hole, and



Note: Penetration is number of blows of 140 lb. hammer falling 30 in. required to drive 1.5 in. sampler 1 ft.

Fig. 5. Soil Boring Log

triaxial shear tests were performed to establish design criteria for the reaction piers of the test apparatus. The rock samples were cut to size with a diamond saw and were tested for compressive strength, which was used as an index of rock quality.

All of the exploratory borings, corings, and tests indicated that the soil profile of the second site is the typical Piedmont soil profile. The rock stratum was found close enough to the surface and was of satisfactory quality, and no ground water was encountered during exploration. The lot thus satisfied all of the requirements and thus was selected as the site of tests.

CHAPTER III

EQUIPMENT

The experimental portion of this project was the full-scale load tests of two eight-inch-diameter piers and one twelve-inch-diameter pier. There were two major requirements to consider in the design of the equipment for these tests. First, the equipment must allow the application of a given load on a pier. Second, the apparatus must allow measurement of the settlement of that pier at given time intervals. Figure 6 shows the layout and pertinent details of the testing equipment.

Two 50-ton hydraulic jacks supplied the force for loading the pier. Side by side, the jacks were capable of supplying a 100-ton force through a distance of six inches. These jacks were jacked against a 36-inch, wide-flange, structural steel beam which in turn was anchored, at its ends, to tension piers.

The tension piers, or anchor piers, were reinforced, belled-bottom, concrete piers. Each was designed to resist 100 kips of upward force. For the design of the anchor piers, it was assumed that if the ultimate strength of the soil surrounding the pier were reached, the pier would raise with itself a mass of earth shaped roughly like the frustum of an inverted cone. The total uplift force which the pier could withstand would equal the sum of the weights of the pier and the accompanying soil mass. The size of a conic frustum is a function of two quantities, the height of the cone and the vertex angle of the cone. The latter, in the case of the anchor piers, was determined by the angle of internal friction, ϕ, of the

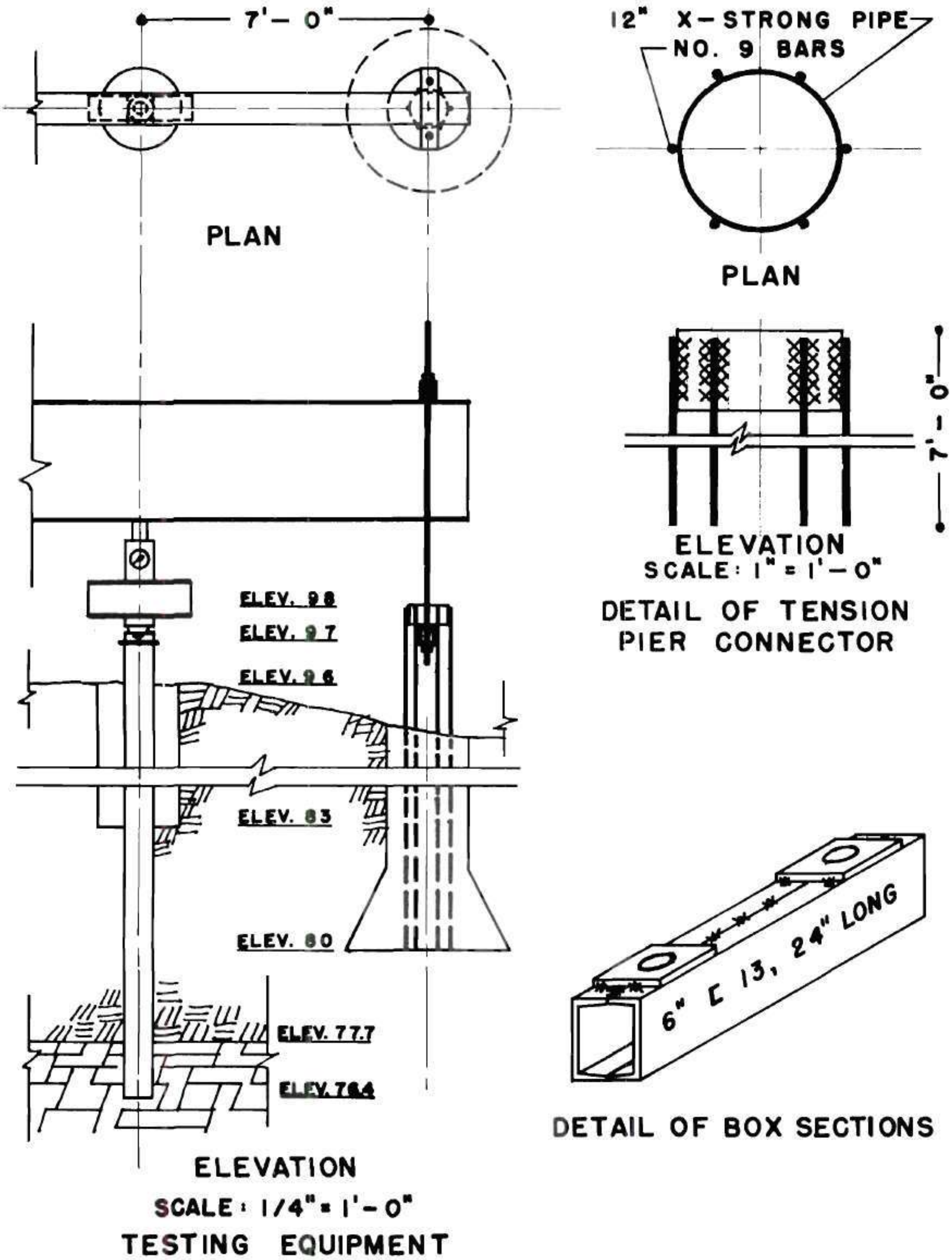


Fig. 6. Plan and Details of Testing Equipment

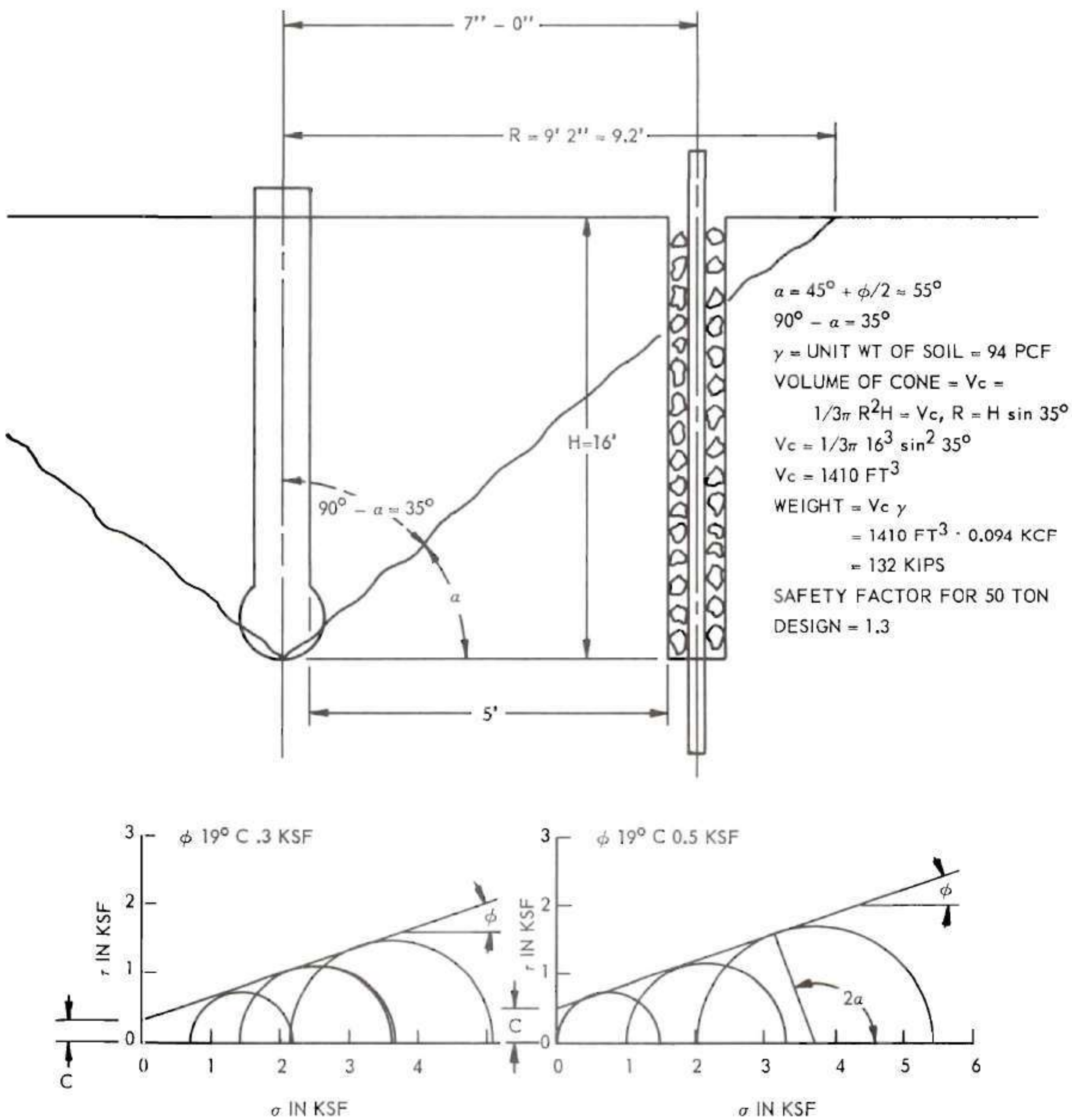
soil surrounding the pier. From Mohr's analysis, the surface of principal stress occurs at an angle of $45^\circ + \phi/2$ with the horizontal, or at an angle of α with the horizontal (3). Thus the central angle of the frustum is $2\ (90^\circ - \alpha)$ or $2\ (45^\circ - \phi/2)$. (See Fig. 7.)

The angle of internal friction of the soil surrounding the piers, as determined by triaxial tests of undisturbed samples, is 19 degrees, and the unit weight of the soil is 94 pounds per cubic foot. Computations shown on Fig. 7 indicate that the ultimate load of a 24-inch pier embedded in such soil to a depth of 16 feet is 132 kips. Thus the tension piers were designed with a safety factor of approximately 1.3.

Structurally, the anchor piers were designed so that the six No. 9 steel reinforcing bars resisted all the tensile force in the pier and the concrete merely held the steel together. Where the reinforcing was spliced, the bars were overlapped sufficiently for the concrete to transfer the force from the lower bar to the upper bar by bonding action.

Because of the desirability of testing several piers, and for the sake of economy, it was desirable to use the anchor piers as many times as possible. The final arrangement was to have the anchor piers form the vertices of an equilateral triangle whose sides were fourteen feet (see Fig. 4). Then the test piers were located at the midpoints of the sides. Thus, each anchor pier could be used twice, and only three anchor piers were needed to test three test piers.

Although the test piers and the anchor piers were separated by a clear distance of only five feet, it is felt that the possible movement of the anchor piers would not affect the settlement or failure of the test piers because the holes for the test piers were drilled oversized and were below the probable zones of failure of the anchor piers (see Fig. 7).



σ_1	σ_3	$\sigma_1 + \sigma_3/2$
0.72	2.20	1.46
1.44	3.70	2.57
1.44	3.61	2.52
2.16	5.05	3.60

σ_1	σ_3	$\sigma_1 + \sigma_3/2$
0	1.50	.75
1	3.29	2.15
2	5.45	3.72

Fig. 7. Tension Pier Analysis Diagram

Figures 8, 9, and 10 show the pier testing setup in detail. Figure 8 is an overall view of the loading facilities and the test piers. The reaction (tension) piers are A, B, and C. The test piers are D, E, and F. Figure 8 also shows the stakes and wooden beam that were used to support the dial micrometers to allow the measurement of pier settlement.

Wood, rather than steel or aluminum, was used to support the dial micrometers because of its lower coefficient of thermal expansion. The use of steel or aluminum would have greatly reduced the accuracy of the settlement measurements. Several coats of paint were used to reduce the warping which might have been caused by changes of the moisture content of the air.

Figure 9 shows the beam and the reaction pier connection assembly. With the use of this type of connection assembly, the only equipment needed to move to the next pier when the tests of one pier were completed was a truck with an "A" frame and a winch to move the heavy steel beam.

The jacking assembly is shown in detail in Fig. 10. The bottom flange of the loading beam is visible at the top of the figure. Directly under the loading beam is the 50-ton hydraulic jack. The small beam section upon which the jack rests was used so that two of the 50-ton jacks could be placed side by side to allow a total jacking force of 100 tons. Below the small beam is a ball-and-socket loading head used to eliminate any bending moment which might have been created by a lack of parallelism between the bottom of the jack and the top of the pier. The steel plate upon which the loading head rests was grouted onto the top of the test pier. Also, the portions of the steel plate which extended beyond the side of the pier were used as a smooth surface against which the dial micrometers

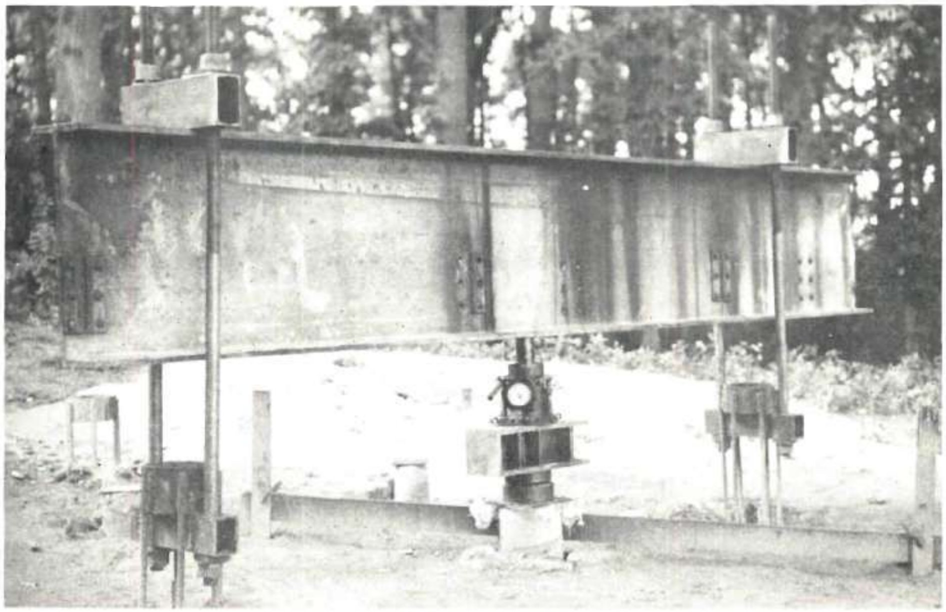


Fig. 8. Loading Assembly and Piers

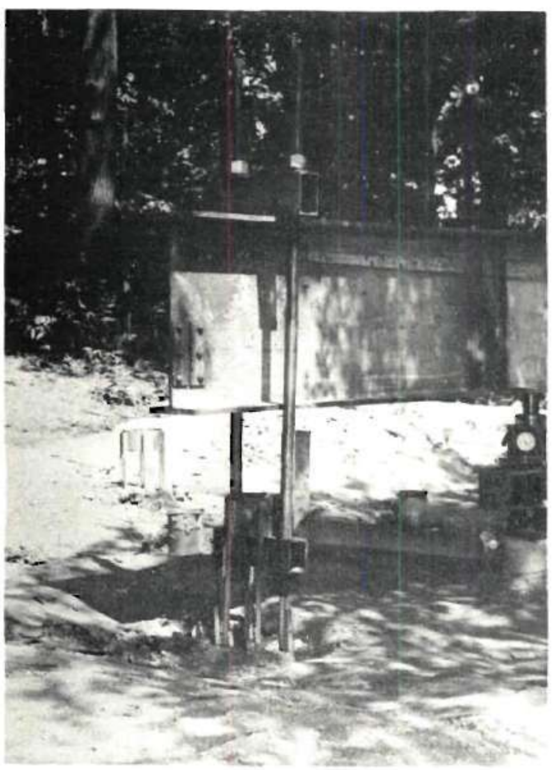


Fig. 9. Beam and Tension Pier Connection Assembly

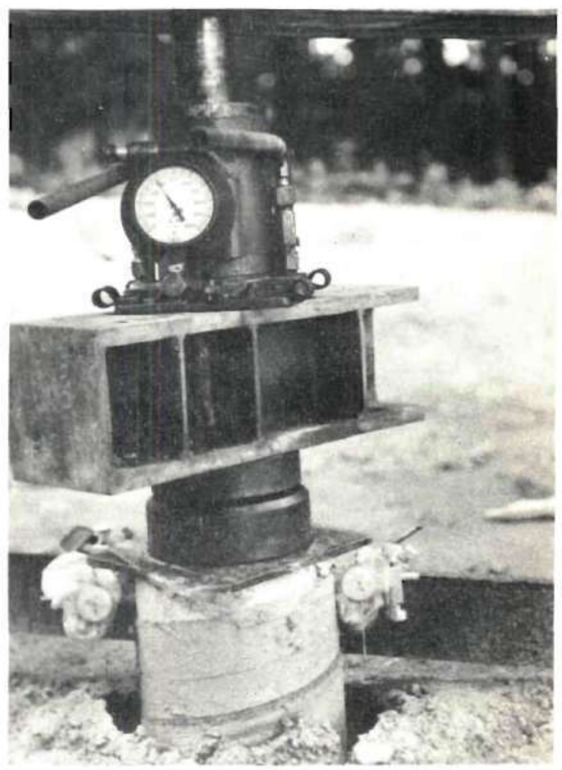


Fig. 10. Hydraulic Jack and Jacking Assembly

might rest. As is seen in the photograph, polyethylene bags were placed over the dial micrometers to protect them from the moisture of the atmosphere.

Three criteria were considered in the selection of the sizes of the test piers. First, the greatest load which could be applied with the available jacking equipment was 100 tons. Second, because of the minimum size of the drilling equipment available and the difficulty of pouring smaller piers, the smallest pier which could be practicably constructed was eight inches in diameter. Third, the subsurface exploration of the site had revealed that the unsound rock was very heterogeneous. Pieces of very strong rock were found within six inches of material not a third as strong.

Test piers "D" and "F" were made the minimum diameter of eight inches. Examination of the rock samples taken during the site exploration indicated that the maximum jacking force and the small end area of these piers would result in a contact pressure sufficient to cause failure of any of the unsound rock upon which the test piers might be founded. Pier "E" was made twelve inches in diameter, so that if rock weaker than that anticipated in the design of the eight-inch piers were encountered, a larger pier could be tested with the available loading equipment.

The test piers were designed as pure compression members, with full lateral support, in accordance with the design recommendations of The ACI Building Code of 1951 (4). The piers were designed to be built with concrete having a compressive strength of 5,000 pounds per square inch. The high strength of the concrete assured failure of the soil rather than a structural failure of the concrete.

Since this study was limited to evaluating the end bearing capacity of the piers, the holes for the test piers were drilled oversize to a depth several feet above the bottom and cardboard tubes were used as forms. The effect of such construction was the practical elimination of skin friction along the pier. The cardboard "Sonotubes" were seated in the bottom of the hole by the static weight of two men. That was sufficient to make the seat leak proof, as excavation of the piers revealed.

The piers were carefully constructed by an experienced crew. First, the holes for all six piers were dug. The first two test pier holes (holes "D" and "E") were bored with a two-foot diameter bit until drilling became difficult. This occurred at depths of about fourteen and fifteen feet respectively for the eight-inch and the twelve-inch piers. Then the holes were cleaned out by hand, after which a bit the size of the proposed pier was used to drill until refusal was met at a depth about one foot below the bottom of the two-foot diameter portion of the hole in each of the two cases. On the third pier, however, the resistant stratum was broken through, and the bottoms of the two-foot diameter and the eight-inch diameter sections of the hole were thirteen feet and nineteen feet, respectively, below the surface of the ground.

The "Sonotubes" for the test piers were seated in the smaller sockets of the test pier holes and were anchored in vertical alignment. Reinforcing steel was tied and placed in the anchor pier holes, and the special loading assemblies were tied to it. Then concrete was poured and vibrated into place in the piers, strength specimens of the concrete were molded, and the piers were left to cure.

When the curing was satisfactory, as indicated by compressive strength tests of the concrete cylinders, the loading beam was hoisted into position, centered over the first test pier, and bolted to the tension piers. The dial gage apparatus was assembled, the jack was placed upon the pier, and the equipment was ready for the beginning of tests.

CHAPTER IV

PROCEDURE

The actual procedure of testing the piers followed closely the procedure of a common pile load test. Usually for a pile load test, a uniform increment of load of approximately one-tenth of the probable ultimate load is selected. After each load increment is applied the settlement of the pile is periodically measured until several consecutive settlement measurements indicate that all appreciable settlement under that load has occurred. Ten kips was arbitrarily chosen as the loading increment which would probably be most satisfactory for these tests. Pier top deflections under each load were measured and recorded at fifteen seconds, thirty seconds, one minute, two minutes, four minutes, and so on after the load was applied with the elapsed time from loading to reading roughly being doubled for each succeeding reading until the settlement of the pier cap was less than 0.005 inches per hour. Then another load increment was applied and the deflection measuring process was repeated. The loading cycles continued until failure had definitely been observed.

The occurrence of failure was determined with the aid of a graph of final settlement versus load (see Fig. 11). Failure was considered to have occurred when increased settlement for the load increments caused an abrupt break in the smooth curve. The intersection of the tangents to the straight line sections adjoining the break in the curve was chosed as the point from which to obtain the load and settlement co-ordinates of failure.

After the three piers had been tested to failure, a hole was dug

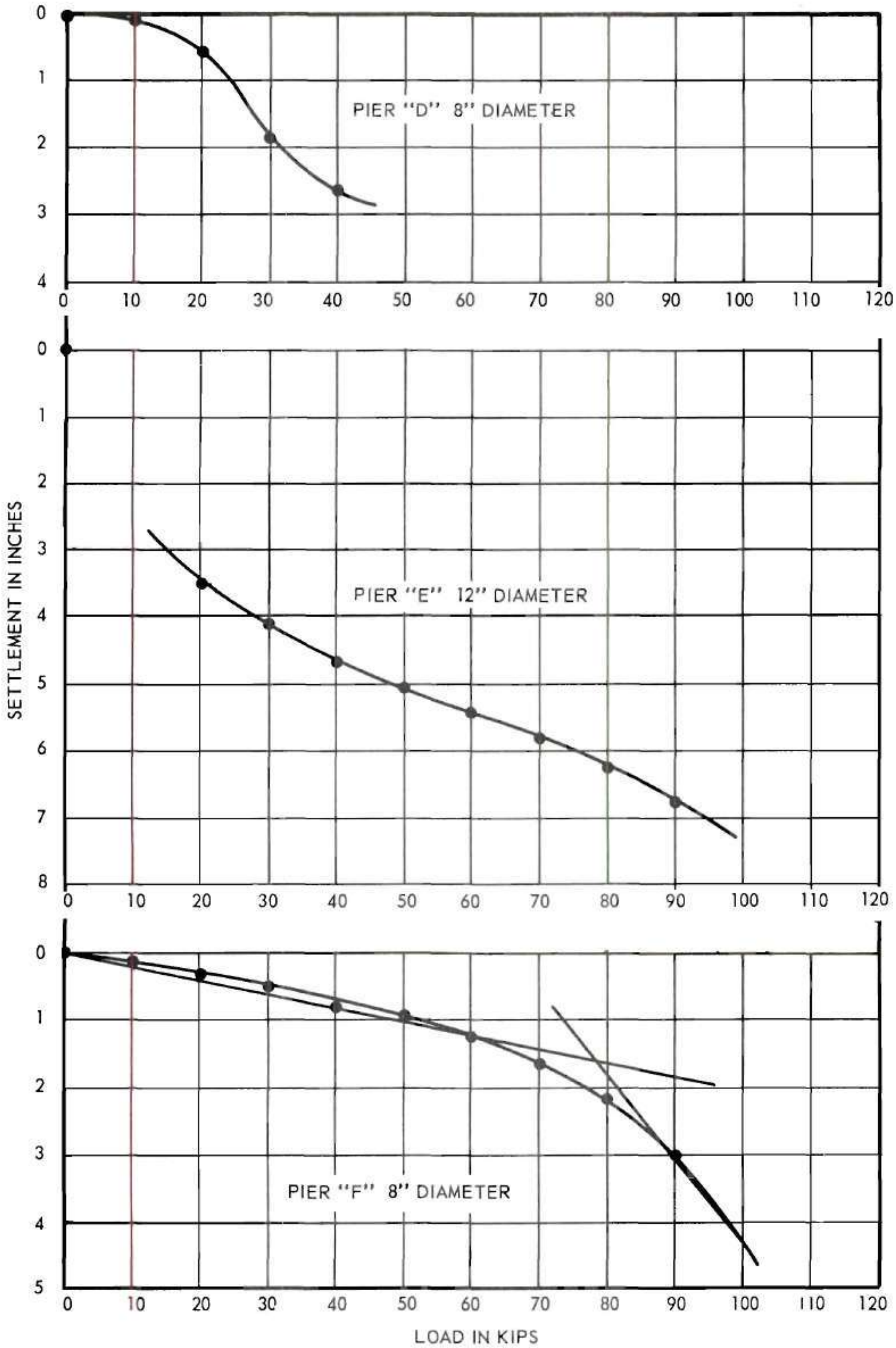


Fig. 11. Graph of Settlement Versus Load for Test Piers

centrally among them to allow visual examination and sampling of the soil surrounding and supporting them. Figure 12 is a full-scale photograph of the bottom of pier "F" (eight inches in diameter) and the partially decomposed rock upon which it rests. The flexible, steel measuring tape at the side of the pier provides a size reference. At a later date, another hole was dug adjoining pier "F" to allow sampling of the material directly beneath the pier. These examinations revealed that pier "F" was supported by a stratum of partially decomposed rock which is slightly more than two pier diameters thick.

Fig. 12. Bottom of Pier "F" and Supporting Foundation Material

CHAPTER V

RESULTS

The most important data expected to come from this research are the magnitudes of the ultimate bearing capacities of the test piers on the partially decomposed rock. Unfortunately, two of the three test piers (pier "D," eight inches in diameter, and pier "E," twelve inches in diameter) were supported by very thin seams of the partially decomposed rock, and the ultimate bearing capacities of those piers were dependent upon the strength of the soil below the rock as well as the strength of the rock itself. Although the results of the tests of these piers cannot be considered in the formulation of a procedure for predicting the bearing capacity of drilled piers on partially decomposed rock, they do demonstrate the importance of visual and manual examination to determine the continuity of the foundation stratum in partially decomposed rock.

Pier "F" (eight inches in diameter) was supported by a stratum of unsound rock which was slightly more than two pier diameters thick (see Fig. 12). The observation and analysis of the failures of numerous foundations similar to the test piers indicate that the bearing capacity of a pier on a stratum of that relative thickness is principally a function of the strength of the material of which that stratum consists. Therefore, it is felt that the results of the load test of pier "F" are significant and valid.

A graph of settlement versus load for the load tests of the piers (Fig. 11) has been employed to determine the values of load and settlement at failure. The point which represents the failure condition is assumed to be the intersection of the tangents to the practically straight segments of the settlement-load curve. The co-ordinates for load and settlement for pier "F" at failure as determined by this graphical method are 78 kips and 1.6 inches respectively; and the ultimate bearing capacity, q_c, is 224 kips per square foot or 112 tons per square foot.

Since friction along the sides of the piers was virtually eliminated by use of construction techniques described above (p. 21), the ultimate bearing capacities of the piers were principally a function of the strength of the materials directly beneath each pier. To evaluate the strength of the rock stratum directly beneath pier "F," unconfined compressive strength specimens were cut from three rock samples. Rock sample "A" was located about two feet south of pier "F," whereas samples "B" and "C" were taken north of the pier and within four inches of it. Table 1 contains the values of cohesion, the values of unconfined compressive strengths of the specimens, and the location from which each sample was taken. (From Mohr's analysis, the cohesion, C, for a material such as the partially decomposed rock is approximately equal to one-half of the unconfined compressive strength of the material (3).)

The unconfined compressive strength of the specimens taken from locations within three feet of each other attests the extremely heterogeneous nature of the partially decomposed rock. On the other hand, the unconfined compressive strengths of the specimens taken from a single piece of rock fall into a small range, a fact which indicates that handling, cutting, and testing of the samples were reasonably uniform and accurate.

Since there is a wide variation of the unconfined compressive

Table 1. Record of Sample Location, Unconfined Compressive Strength, and Cohesion C for Samples of the Unsound Rock Below Pier "F"

Sample Number & Location	Specimen Number	Unconfined Compressive Strength (ksf)	Cohesion C (ksf)
A. South of Pier, Two Feet Away	A1	780	390
	A2	920	460
	A3	1135	567
	A4	1425	712
	A5	1102	551
	A6	1752	846
	A7	935	467
	A8	925	462
B. North of Pier, Four Inches Away	B1	140	70
	B2	129	65
	B3	35	18
	B4	73	37
C. North of Pier, Four Inches Away	C1	20	10
	C2	56	28

strengths of the specimens cut from rock sample "A" (the sample located about two feet from pier "F") and the unconfined compressive strengths of the specimens cut from rock samples "B" and "C" (the samples located less than four inches from pier "F"), it was thought to be desirable to use the average of the strengths of the specimens from samples "B" and "C" as the index of the strength of the material that supported the pier. The average of those strengths is 76 ksf, and the corresponding value of C is 38 ksf.

If the Mohr's envelope of rupture of the material supporting the pier is assumed to be a straight horizontal line, i. e., if C is assumed to be constant for all values of confining pressure within the range which one wished to consider, the failure of the pier on such a material may be analyzed in the same manner as that of a pier on a saturated clay, for which the Mohr's envelope of rupture is horizontal (5). Such an assumption is believed to be reasonable in the case of this partially decomposed rock because the failure of a pier on this material involves the shearing of relatively large pieces of rock rather than the breaking of a cementing agent and/or the sliding of unbroken particles over each other, as in the cases of sands and weak sandstones.

The presence of a plane of weakness in the foundation material could invalidate any analysis based on this assumption, but the occurrence of such a plane of weakness is unpredictable and therefore cannot be considered in this analysis. The absence of such a stratum of weak material in the rock under pier "F" was ascertained by hand excavation of the rock under the pier after the load tests were completed.

A series of analyses developed by Bell (6) is based upon Mohr's concept of the failure of materials and takes into consideration the factors mentioned below. Allowance is made for the shape of the foundations by separating foundations into three classes: round, square, and long (wall type footings). The angle of internal friction (ϕ) and the cohesion (C) are used as indices of the quality of soil beneath them. The depth of the bottom of the foundation below the surface of the ground (D_f) and the unit weight (γ) of the overburden are used to compensate for depth at which the foundations are located. The following are the values of these factors for pier "F":

Foundation Shape	Round
Angle of Friction (ϕ)	0
Cohesion (C)	38 ksf.
Foundation Depth (D_f)	19 ft.
Unit Weight of Surcharge (γ)	94 pcf.

The solution for the critical stress under pier "F" by the Bell analysis applicable to this case is as follows:

$$\text{Critical Stress } (q_c) = 5.2\ C + \gamma D_f$$

$$q_c = 5.2\ (38) + (0.094)(19)$$

$$q_c = 198 \text{ ksf}$$

The critical stress (q_c) indicated by Bell's analysis is 198 kips per square foot or 88 per cent of the measured q_c of 224 ksf for pier "F." Such close agreement between the measured and the computed values for the ultimate bearing capacity of pier "F" is encouraging indeed. However, it makes doubly regrettable the fact that it was impossible to obtain for pier "D" and pier "E" reliable data which could help to corroborate or refute the accuracy of the testing procedures and analysis implied by such close agreement.

Besides the measurement of the ultimate bearing capacity of pier "F" and the analysis of the failure of that pier, another product of this research is the information obtained by the measurement of pier settlement and the correlation of settlement with other variables. The amount which pier "F" had settled at the graphically determined failure load is 1.6 inches. When the pier had been subjected to only one-third of the failure load (a safety factor of 3.0 is commonly used in the design of soil structures) the pier had settled approximately 0.5 inches.

Studies based on experience and on structural analysis indicate that between two columns of a statically indeterminate reinforced concrete structure approximately twenty feet apart a differential settlement of that magnitude is allowable (7). Since the total settlement of the test pier at a third of the failure load did not exceed the differential settlement allowable for an indeterminate steel or reinforced concrete building having spans of twenty feet or more, one could build almost any of the usual commercial or industrial buildings on such pier foundations without fear of costly damage due to settlement.

A study of the relationship between settlement and time provides information of considerable theoretical interest. When the settlement caused by each of the load increments was plotted against the logarithm of the elapsed time after the application of that load, the resulting curves had the same general shape as those obtained in laboratory consolidation tests of soils (see Appendix). One dissimilarity between these two series of curves is that the former curves do not approach an asymptote as do the consolidation curves. When a uniform procedure was being considered for the pier load tests, it was arbitrarily decided that the settlement caused by a given load would be measured until the rate of settlement became less than 0.005 inch per hour. If the settlement caused by each load had been measured over a longer period of time, _i. e._, when the rate of settlement was less than 0.005 inch per hour, the curves for pier settlement versus the logarithm of elapsed time might also have approached an asymptote. If this theory is correct, the similarity of the shapes of the curves indicates that the settlement of the pier probably was, in agreement with Terzaghi's (8) consolidation theory, caused by consolidation of the partially decomposed rock which supported it.

CHAPTER VI

CONCLUSIONS

Some structures which have formerly been built on other types of foundations can be built more economically on drilled piers supported on partially decomposed rock. The ultimate bearing capacity of such piers can be predicted from the unconfined compressive strength of the foundation material with the use of Bell's (6) analysis. It is essential, however, to ascertain the continuity of the foundation material for a depth of two pier diameters by manual examination. Furthermore, where partially decomposed rock is of sufficient strength that drilled piers founded upon it are the most economical foundation for a common industrial or office building, it is unlikely that such a structure will suffer damage caused by foundation settlement. It is regrettable that, because piers "D" and "E" were founded on strata of insufficient thickness, these conclusions are based upon the load tests of only one pier, pier "F."

In addition to the readily usable practical information, from this research has come the conclusion of theoretical interest that the settlement of the test pier probably was caused by consolidation (as defined by Terzaghi's (8) theory of consolidation) of the foundation material beneath the pier. Such a conclusion was drawn after a comparison of settlement-log time curves for the test piers and those for laboratory consolidation tests of soil samples.

CHAPTER VII

RECOMMENDATIONS

The design of drilled piers on partially decomposed rock must be based upon the unconfined compressive strength of specimens cut from hand-excavated, chunk samples. Even after this precaution has been taken, it is imperative that a test hole approximately two inches in diameter and about twice the pier diameter in depth be drilled in the bottom of each pier excavation so that a hooked steel rod may be used to determine the continuity of the foundation material.

Further study of the bearing capacity of drilled piers on partially decomposed rock is needed. First, a greater number of tests upon which to base conclusions about design criteria would greatly increase the confidence an engineer could place in his design. Second, the load testing of piers on varying qualities of rock might indicate a significant relationship between rock strength and settlement. Finally, further study should result in better understanding of the mechanics of pier settlement and in more economical design of drilled piers on this foundation material which is so common in the Piedmont Region.

A P P E N D I X

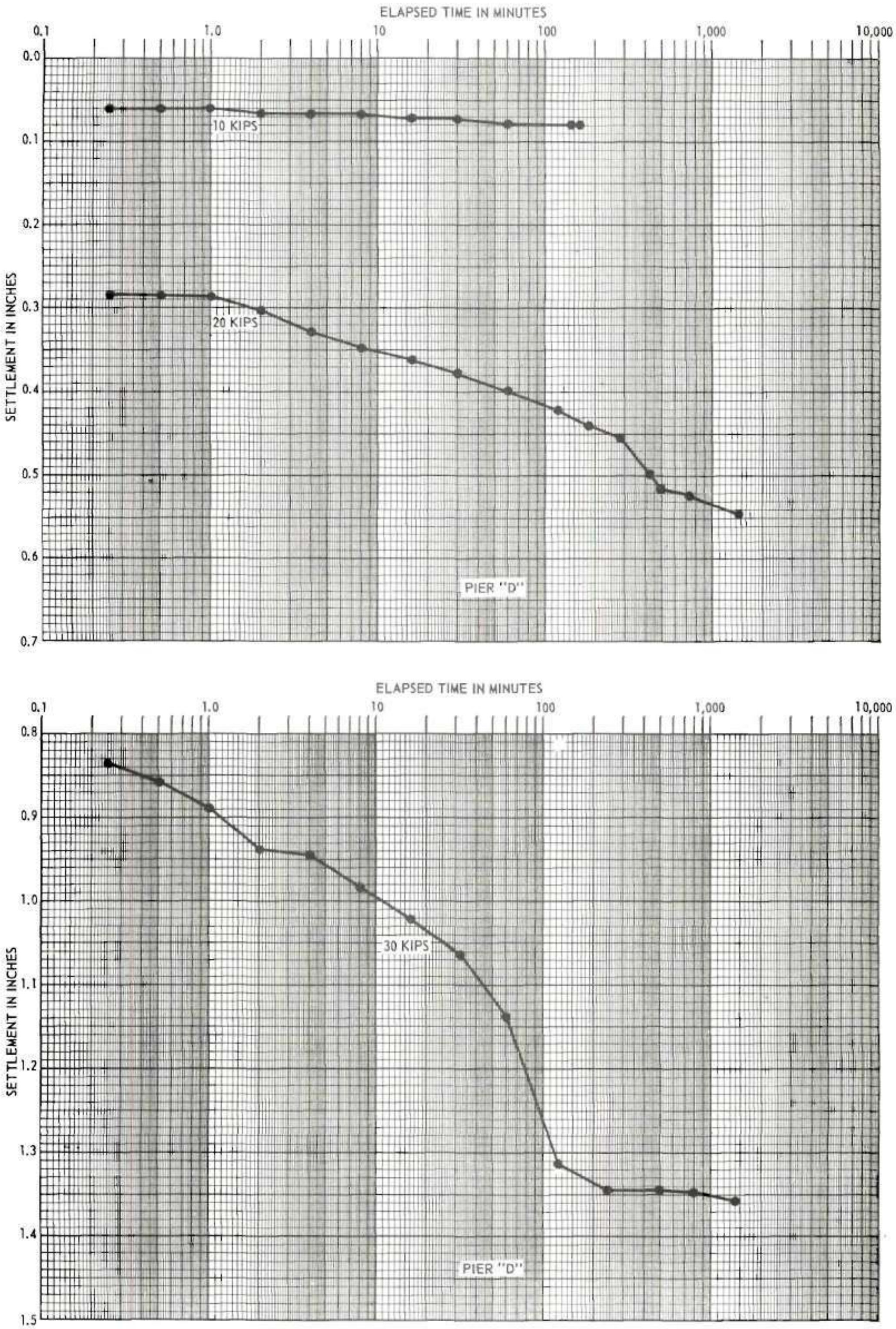
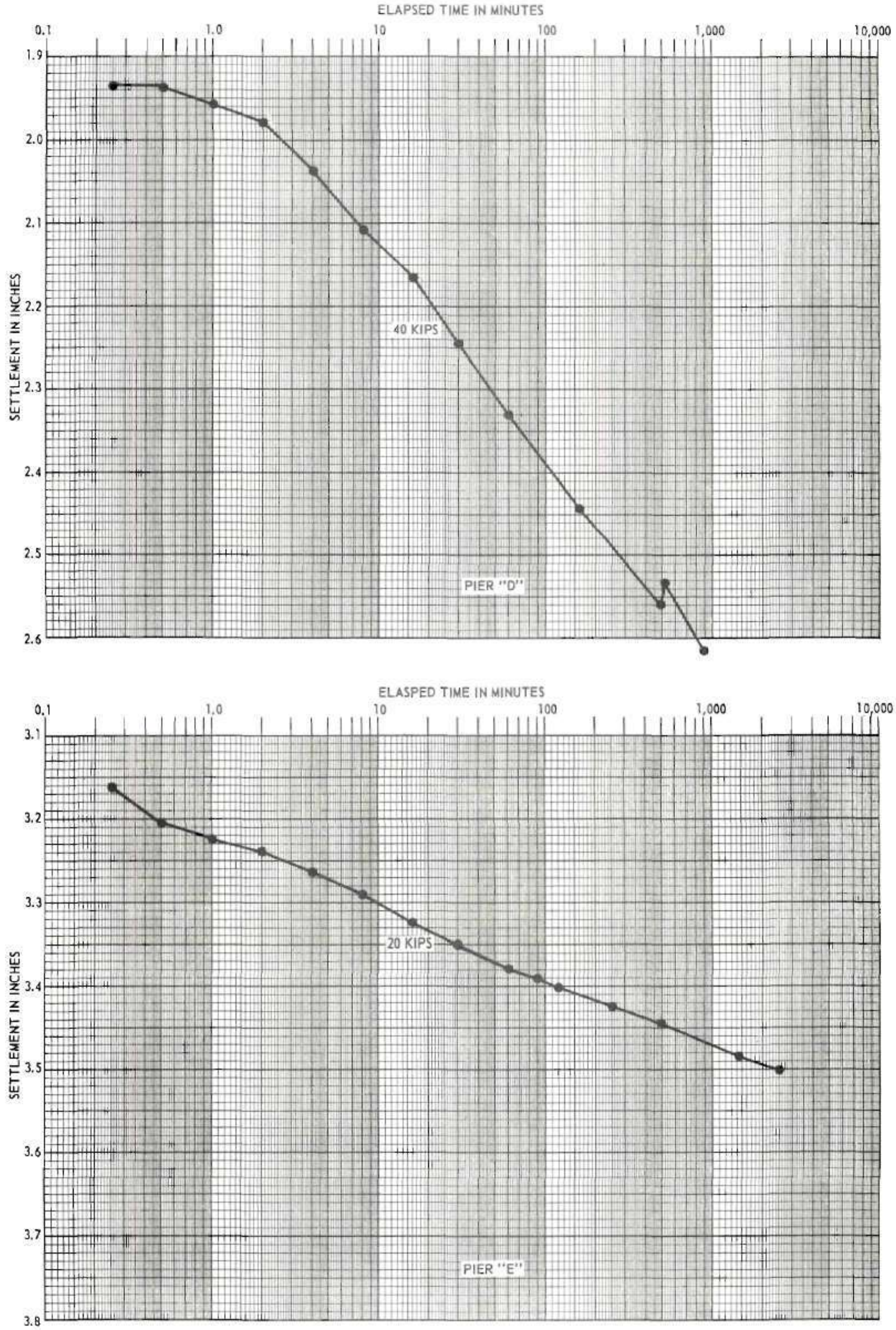


Fig. 13. Graphs of Settlement Versus Time for Test Piers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2.1
2.2
2.3
2.4
2.5
2.6
40 KIPS
PIER "D"
ELAPSED TIME IN MINUTES
0.1
1.0
10
100
1,000
10,000
SETTLEMENT IN INCHES
3.1
3.2
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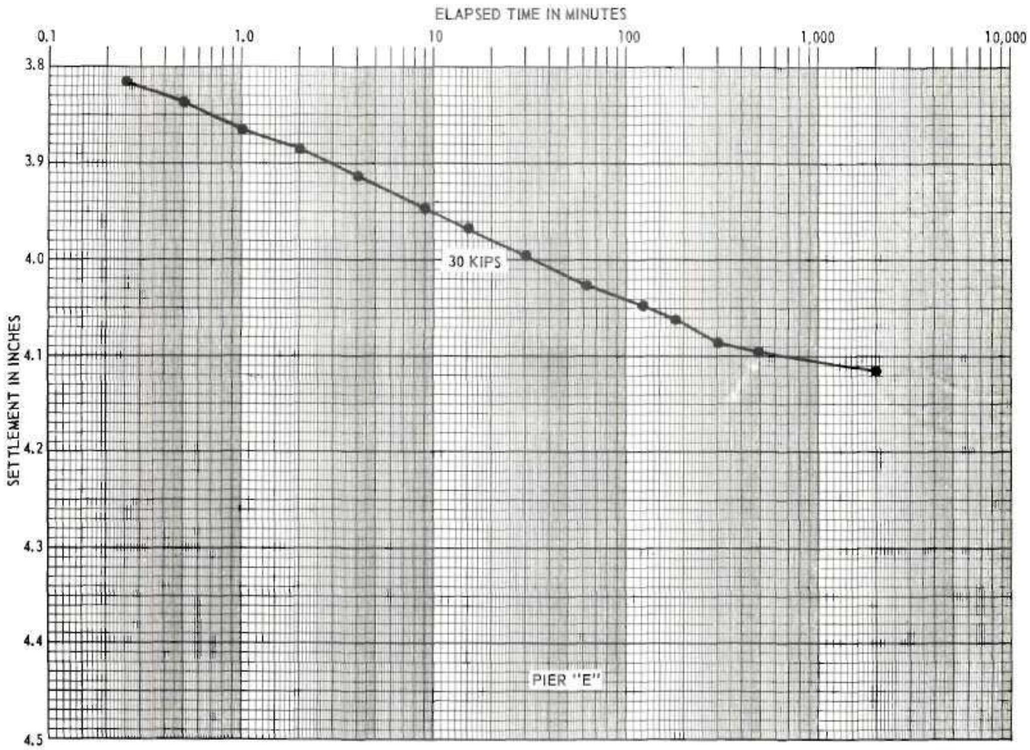
3.6
3.7
3.8
20 KIPS
PIER "E"

Fig. 13. (Cont.)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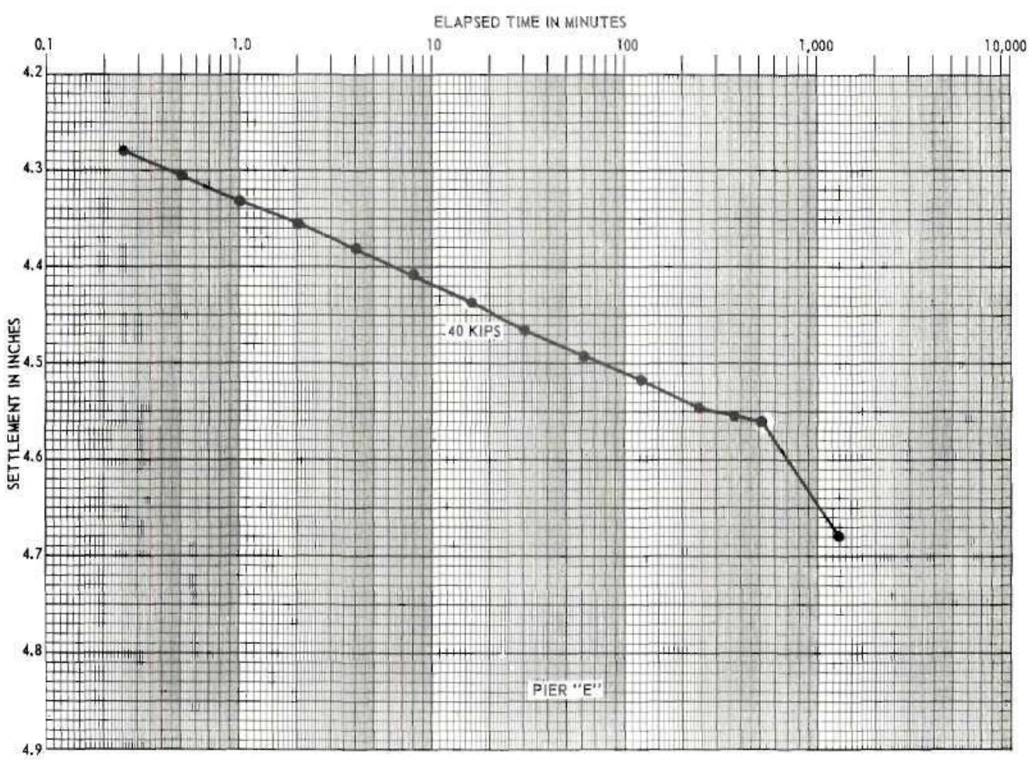
4.4
4.5
30 KIPS
PIER "E"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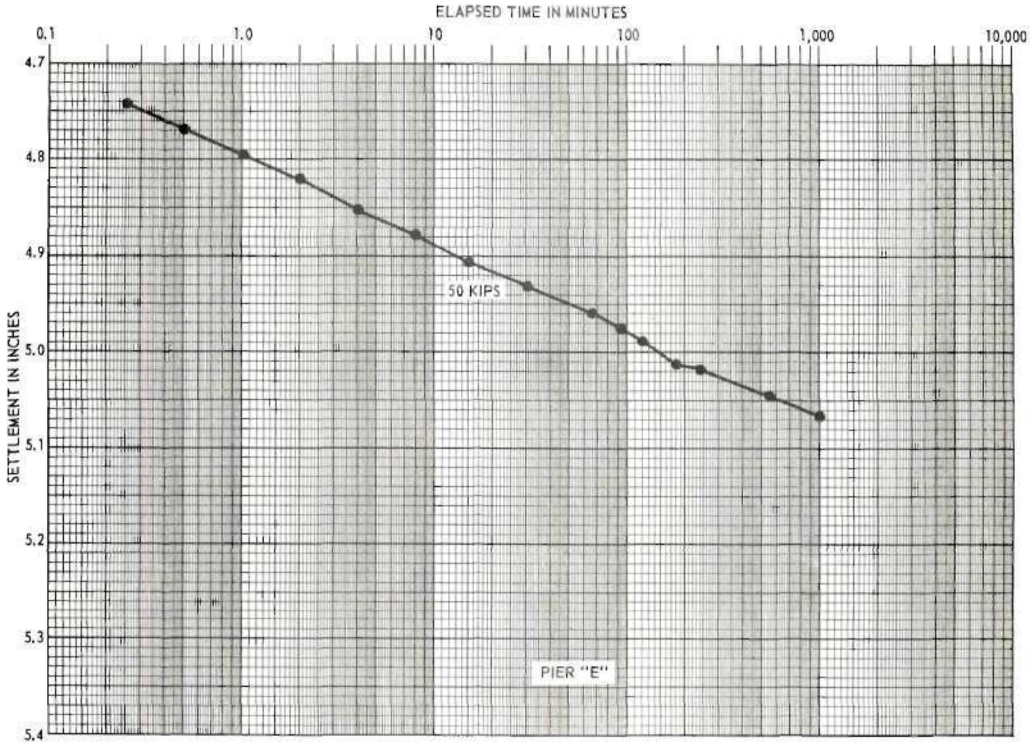
4.7
4.8
4.9
40 KIPS
PIER "E"

Fig. 13. (Cont.)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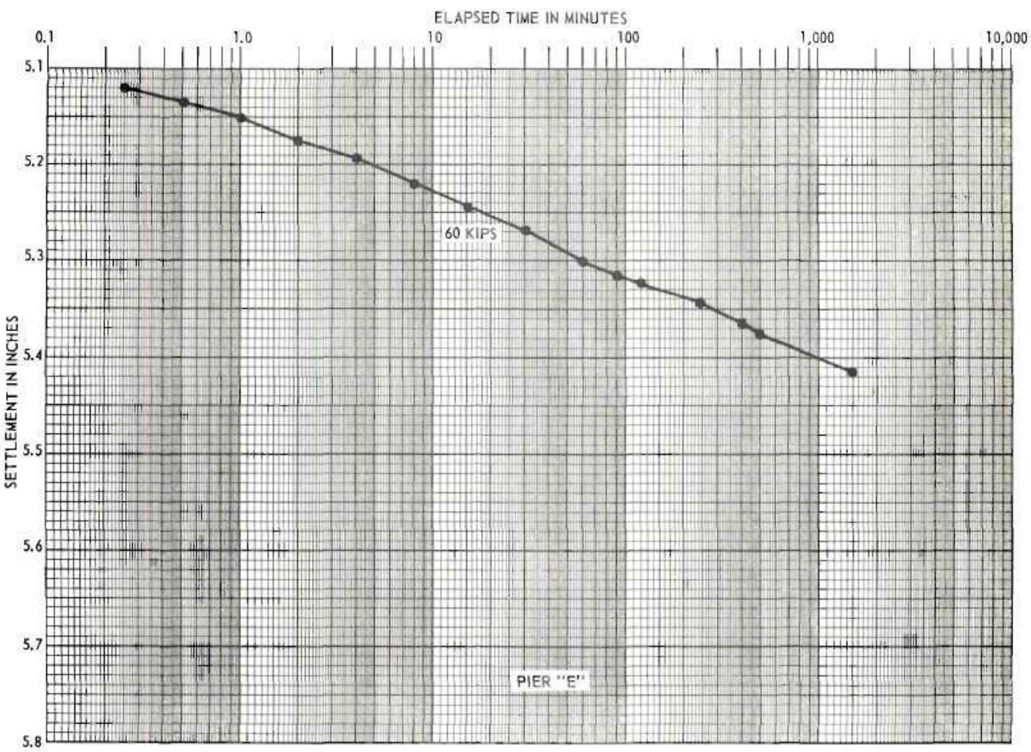
5.3
5.4
SETTLEMENT IN INCHES
50 KIPS
PIER "E"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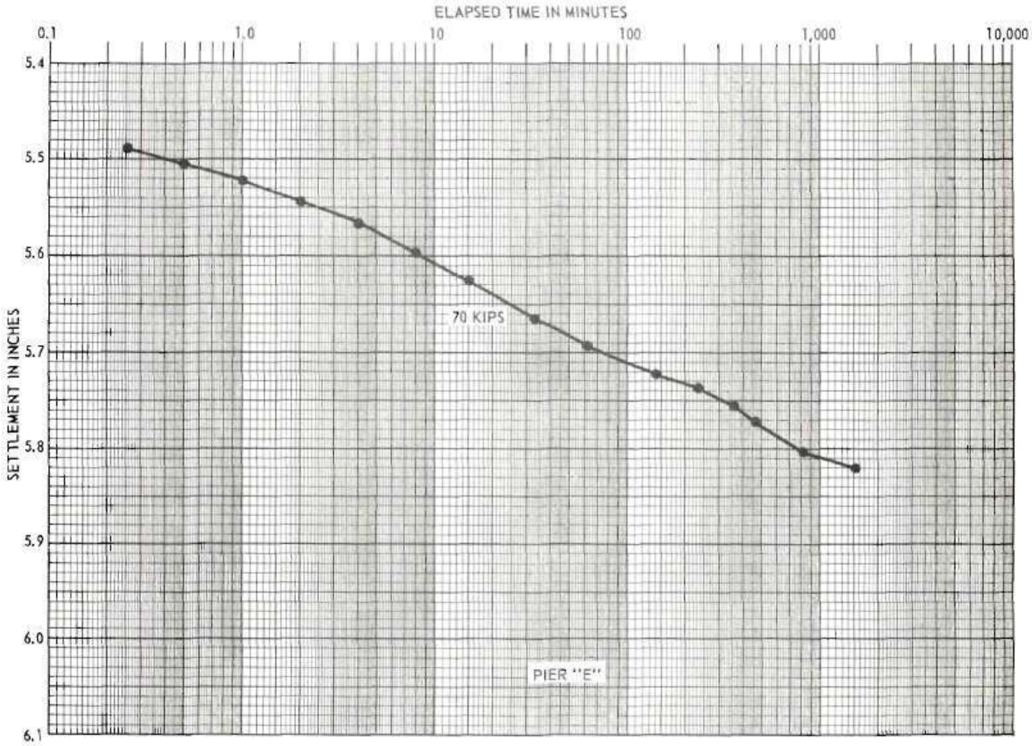
5.8
SETTLEMENT IN INCHES
60 KIPS
PIER "E"

Fig. 13. (Cont.)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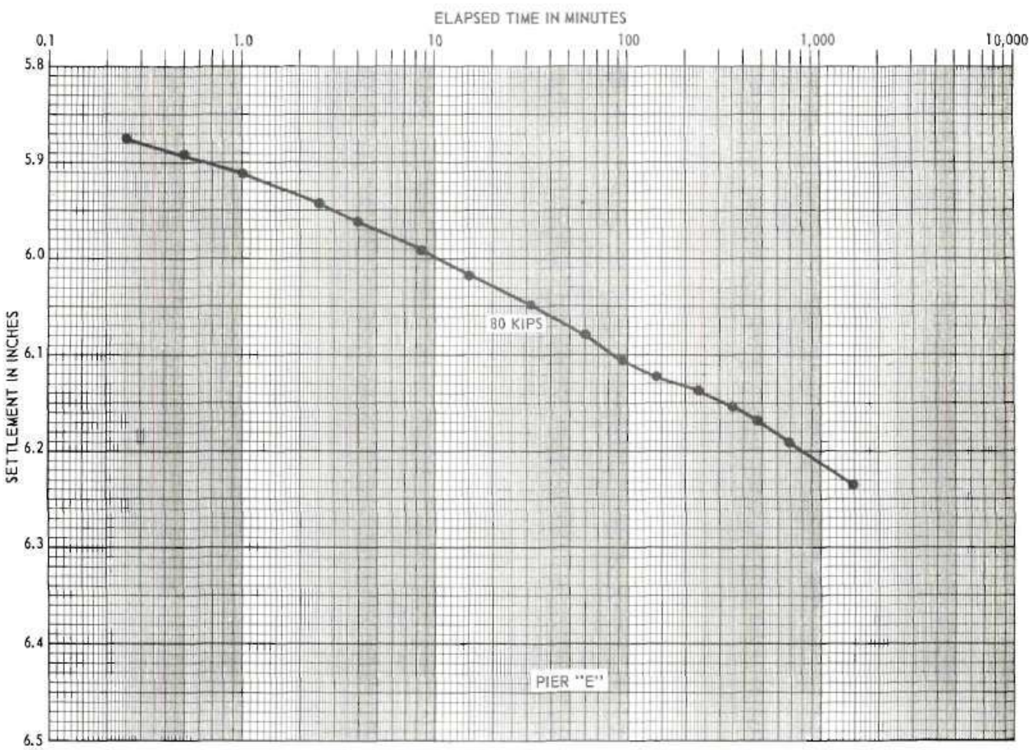
6.1
SETTLEMENT IN INCHES
70 KIPS
PIER "E"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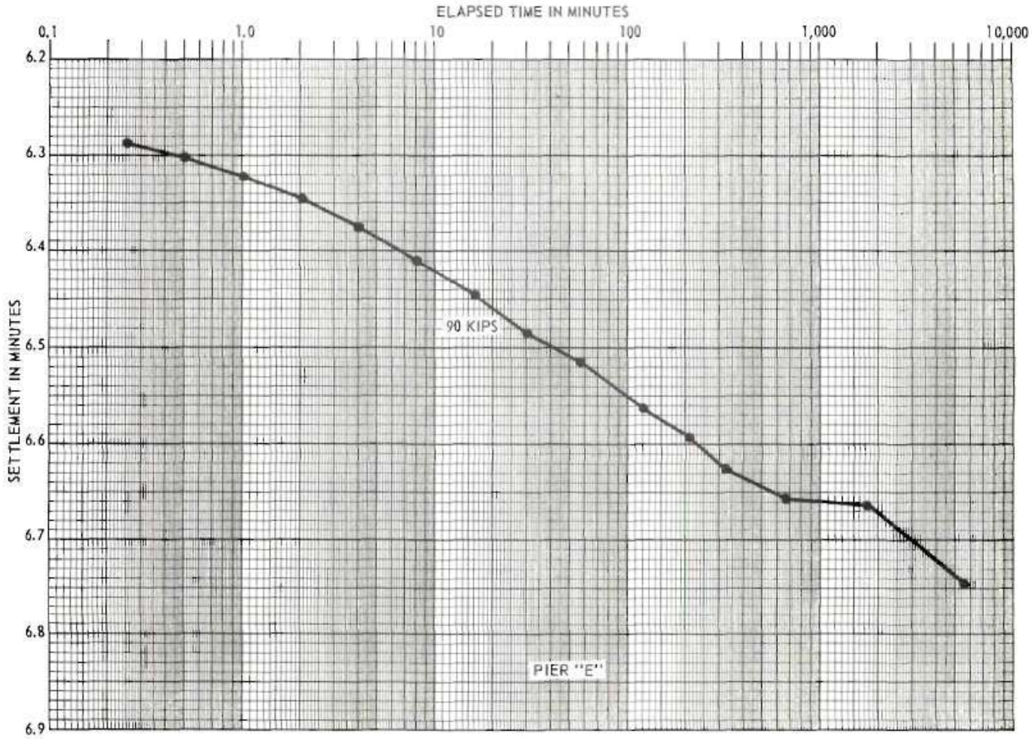
6.5
SETTLEMENT IN INCHES
80 KIPS
PIER "E"

Fig. 13. (Cont.)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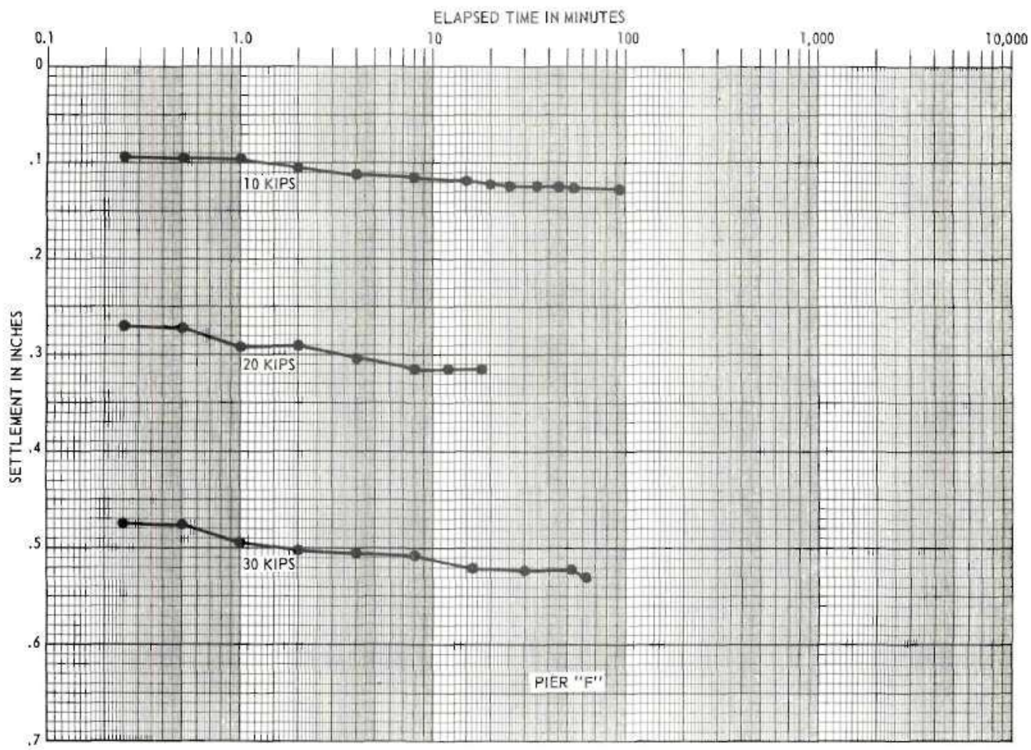
6.7
6.8
6.9
SETTLEMENT IN MINUTES
90 KIPS
PIER "E"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.1
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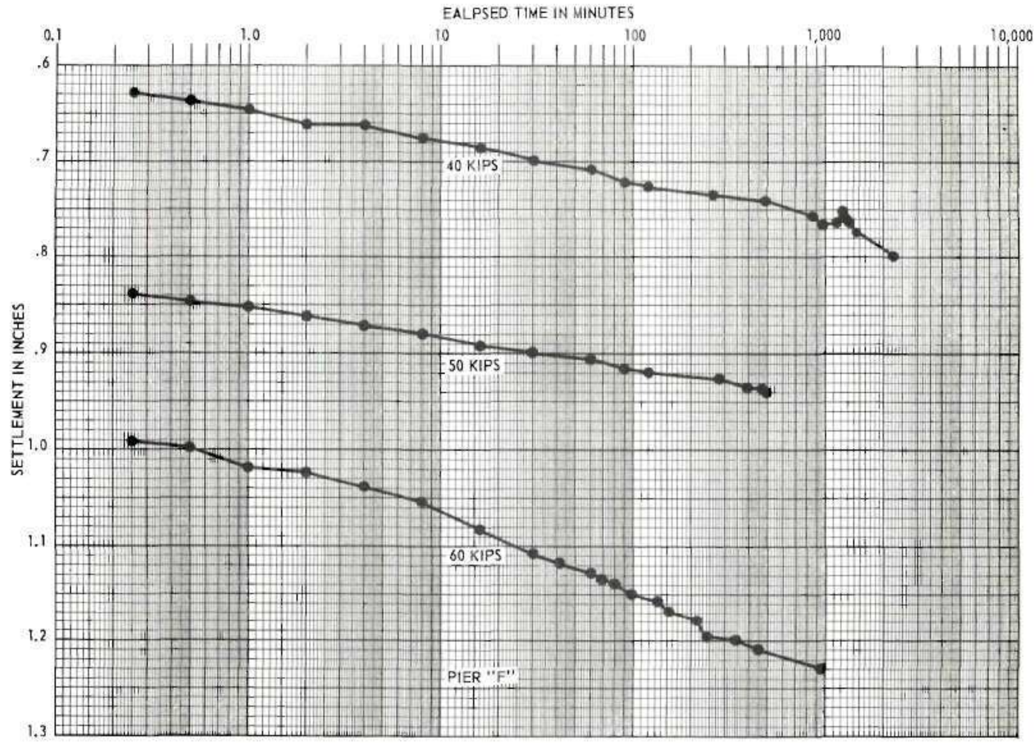
.7
SETTLEMENT IN INCHES
10 KIPS
20 KIPS
30 KIPS
PIER "F"

Fig. 13. (Cont.)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1.3
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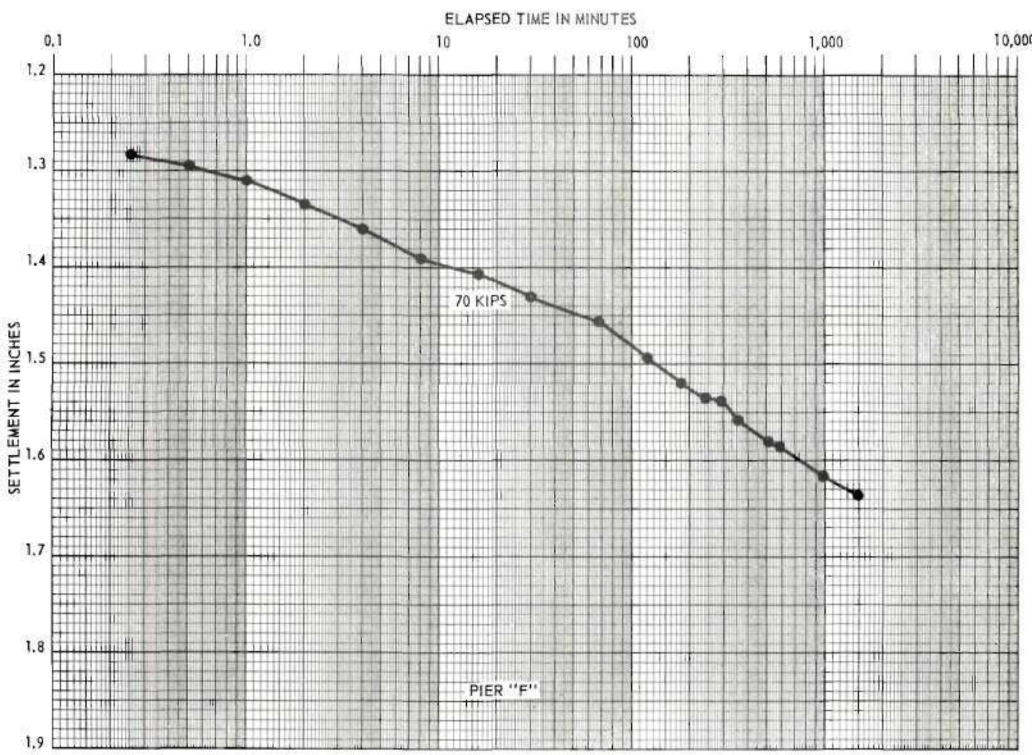
50 KIPS
60 KIPS
PIER "F"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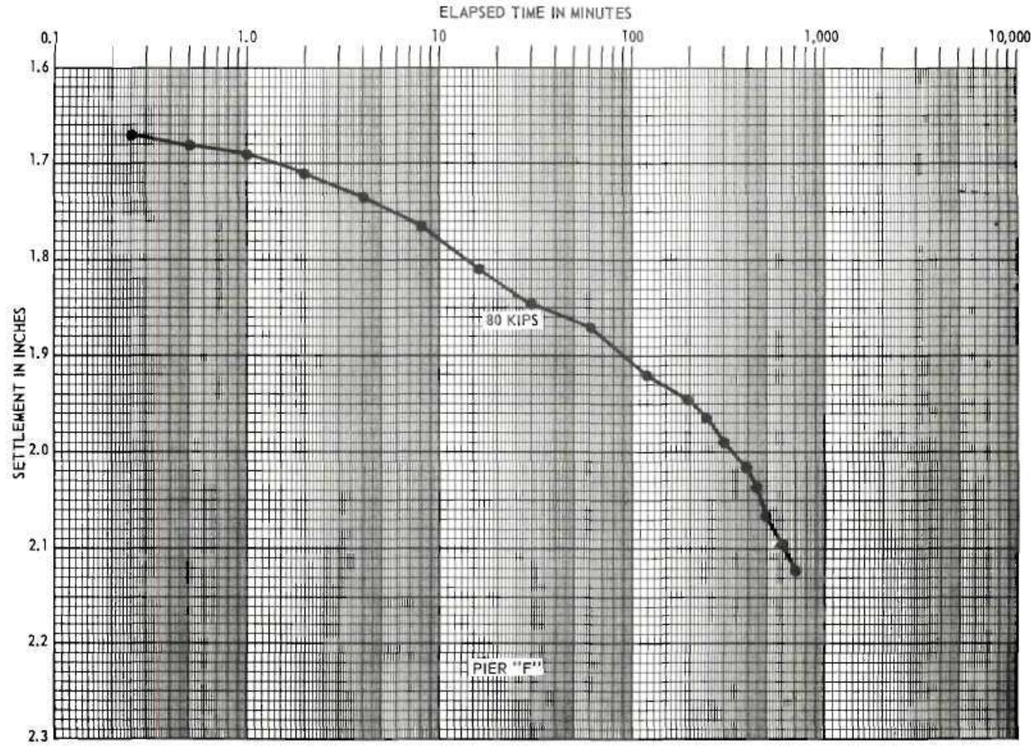
70 KIPS
PIER "F"

Fig. 13. (Cont.)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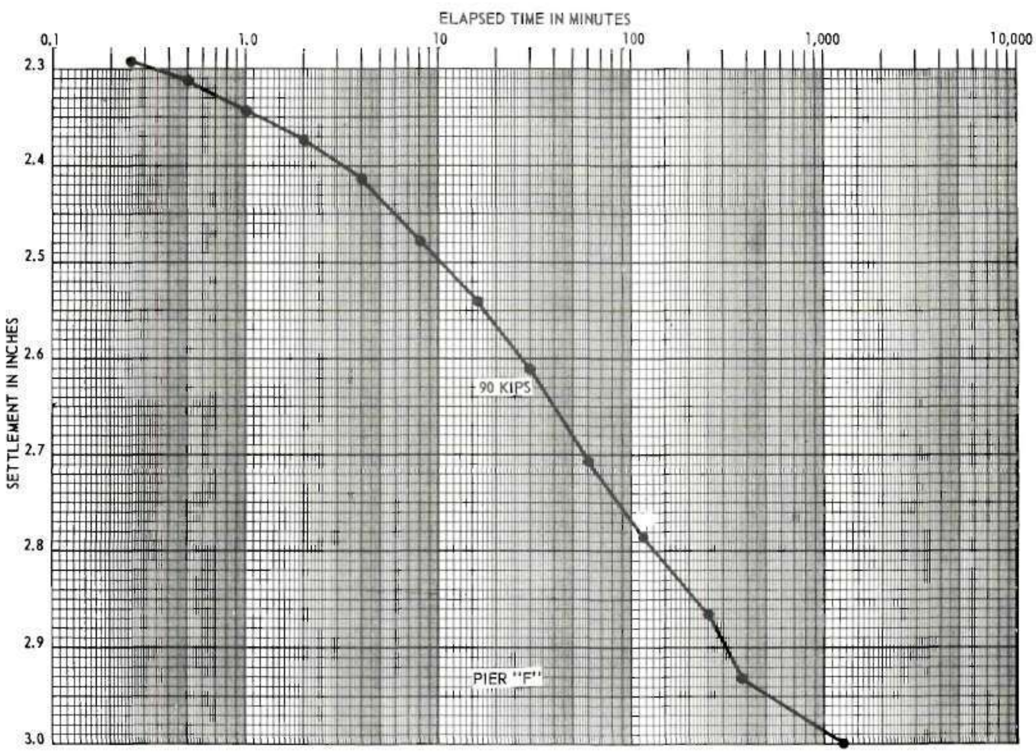
2.2
2.3
80 KIPS
PIER "F"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2.6
2.7
2.8
2.9
3.0
90 KIPS
PIER "F"

Fig. 13. (Cont.)

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