Division of Highway Planning  
State Highway Department of Georgia  
Atlanta 3, Georgia  

Attention: Mr. Roy A. Flynt  
State Highway Planning Engineer  

Subject: Quarterly Progress Report No. 1 - Project B-189  
Contract No. HPS-1 (57)  
"A Study of Bearing Capacity of Pile Foundations"  
October 1 to December 31, 1960  

Gentlemen:  

The subject project work was not started until approximately October 1, 1960; therefore, the first "Quarterly Progress Report" covers the period to December 31, 1960.  

From the initiation of this project, the required general testing program indicated the necessity of extending the existing outdoor test area of the Soil Mechanics Laboratory. Plans for the annex to the existing area were made and a grant of $7,000 was obtained from the Engineering Experiment Station for the extension. The construction work, performed by the construction group of the Physical Plant at Georgia Tech, started in November and is expected to be completed very soon.  

During the quarter plans have been made for various equipment needed for the execution of the testing program: a test pit for large scale models and a test bin for small scale tests. Both the test pit and test bin are to have loading frames, pile driving rig equipment, and hoists. The work on the test pit and test bin, both with loading frame, is practically completed. Material has been purchased and a part of the work done on the other equipment. Studies and plans are being made for equipment for transportation and placing of materials for the models. Also, past research in this field is being reviewed and a detailed testing program laid out.  

In summary, after a somewhat late start, very good progress was made on the project during the past quarter.

Respectfully submitted,

Aleksandar B. Vesic  
Project Director  

Approved:  

Thomas W. Jackson, Chief  
Mechanical Sciences Division
May 23, 1961

Division of Highway Planning
State Highway Department of Georgia
Atlanta 3, Georgia

Attention: Mr. Roy A. Flynt
State Highway Planning Engineer

Subject: Quarterly Progress Report No. 2 - Project B-189
Contract No. HPS-1 (57)
"A Study of Bearing Capacity of Pile Foundations"
January 1 to March 31, 1961

Gentlemen:

During this second quarter of the project the construction of the laboratory annex for pile testing was practically completed. Also, the pile driving rig equipment, except hammer and lifting device, was nearly completed. Studies for equipment for transportation and placing of materials were continued and funds requested from the Engineering Experiment Station for construction of an installation for pneumatic handling of granular materials.

Plans were made and some work done on miscellaneous auxiliary equipment for small scale testing. Preliminary tests on colored-sand models for investigation of shear-patterns were made (optimum mix, vibration effect studies, etc.). Also, work was continued on study of literature of past research in this field as well as on detailed programs of investigation.

In summary, good progress was made on the project during the past quarter. In order to accelerate the work, two additional student assistants, Mr. D. C. Banks and Mr. O. S. Lord, both senior students in Civil Engineering, were hired and started working on approximately April 1.

Respectfully submitted,

Aleksandar B. Vesic
Project Director

Approved:

Thomas W. Jackson, Chief
Mechanical Sciences Division
July 10, 1961

Division of Highway Planning
State Highway Department of Georgia
Atlanta 3, Georgia

Attention: Mr. Roy A. Flynt
State Highway Planning Engineer

Subject: Quarterly Progress Report No. 3 - Project B-189
Contract No. HPS-1 (57)
"A Study of Bearing Capacity of Pile Foundations"
April 1 to June 30, 1961

Gentlemen:

During the past quarter experimental studies of placing and compaction of models were completed. Concerning the problem of handling of materials, investigations showed that a pneumatic conveying system would not be economically feasible. It was decided to provide mechanical conveying of materials for which equipment was purchased from the Engineering Experiment Station funds.

Tests with models of colored sand were continued. Equipment for testing 2 in. diameter-piles and 2 in. wide strips was completed. By the end of the quarter testing of 2 in. pile models was commenced and is currently in full progress. Other tests are being prepared to begin immediately after completion of the present series.

Respectfully submitted,

[Signature]
Aleksandar B. Vesic
Project Director

Approved:

[Signature]
Thomas W. Jackson, Chief
Mechanical Sciences Division
November 28, 1961

Division of Highway Planning
State Highway Department of Georgia
Atlanta 3, Georgia

Attention: Mr. Roy A. Flynt
State Highway Planning Engineer

Subject: Quarterly Progress Report No. 4 - Project B-189
Contract No. HPS-1 (57)
"A Study of Bearing Capacity of Pile Foundations"
July 1 to September 30, 1961

Gentlemen:

During the past quarter, testing of 2 inch pile models and 2 inch wide strips was completed. The results are being evaluated. Tests with models of colored sand were continued. Equipment for testing 4 inch diameter piles was completed and good progress was made in testing. Additional equipment for placing sand in the large pit was completed, and work started on 6 inch diameter piles.

Respectfully submitted,

Aleksandar B. Vesic
Project Director

Approved:

for Thomas W. Jackson, Chief
Mechanical Sciences Division
Division of Highway Planning
State Highway Department of Georgia
Atlanta 3, Georgia

Attention: Mr. Roy A. Flynt
State Highway Planning Engineer

Subject: Quarterly Progress Report No. 5 - Project B-189
Contract No. HPS-1 (57)
"A Study of Bearing Capacity of Pile Foundations"
October 1 to December 31, 1961

Gentlemen:

During the period of time indicated above, work continued toward completion of Phase I of the project.

The results of tests with 2 inch pile models and 2 inch wide strips were evaluated. They showed a trend in increase of bearing capacity with depth which is partly different from what was expected and which casts some doubt that silo-effect may have existed in test bins.

It was decided to check the actual vertical pressures in the test pit by introducing pressure cells in all future tests.

Most of the testing of 4 inch diameter piles was completed and part of the results evaluated. Equipment for testing 6 inch diameter piles was finished and the final series of tests, which have to be made in the large test pit, was started. First-filling of the 1000 ft³ test pit caused considerable delay. Some improvement of the handling equipment is being studied. During the next quarter work will continue on the 6" pile tests and on the theoretical analyses in progress.

Respectfully submitted,

Aleksandar B. Vesic
Project Director

Approved:

Thomas W. Jackson, Chief
Mechanical Sciences Division
May 17, 1962

Division of Highway Planning
State Highway Department of Georgia
Atlanta 3, Georgia

Attention: Mr. Roy A. Flynt
State Highway Planning Engineer

Subject: Quarterly Progress Report No. 6 - Project B-189
          Contract No. HFS-1(57)
          "A Study of Bearing Capacity of Pile Foundations"
          January 1 to March 31, 1962.

Gentlemen:

During the past quarter first tests of the final series (6" piles) were completed. As mentioned in the previous report the preparation of models in the large test pit is proceeding with considerable delay. Studies of handling equipment and operations showed that considerable improvement can be achieved by constructing a bucket elevator for vertical transport of sand. This piece of equipment was designed and ordered.

Work on evaluation of test results and miscellaneous theoretical analyses was continued. The main problem remains to estimate the possible silo-effect in the test bin. The measurements by means of earth pressure cells did not furnish reliable data. An independent method of determining silo-effect is being sought.

Respectfully submitted,

Aleksandar B. Vesic
Project Director

Approved:

Thomas W. Jackson, Chief
Mechanical Sciences Division
July 31, 1962

Division of Highway Planning
State Highway Department of Georgia
Atlanta 3, Georgia

Attention: Mr. Roy A. Flynt
State Highway Planning Engineer

Subject: Quarterly Progress Report No. 7 - Project B-189
Contract No. HPS-1(57)
"A Study of Bearing Capacity of Pile Foundations"
April 1 to June 30, 1962

Gentlemen:

During the reporting period, experiments with the 4 inch piles were completed and tests in process on the 6 inch piles were continued. Also during the quarter, a bucket elevator for vertical transport of sand was installed in the laboratory. This elevator will do much to facilitate continuation of the project at the scheduled rate.

Evaluation and analysis of the experimental results was continued and the final report for Phase I of the project is now in preparation.

Respectfully submitted,

Aleksandar B. Vesic
Project Director

Approved:

Thomas W. Jackson, Chief
Mechanical Sciences Division
Division of Highway Planning  
State Highway Department of Georgia  
Atlanta 3, Georgia

Attention: Mr. Roy A. Flynt  
State Highway Planning Engineer

Subject: Quarterly Progress Report No. 8 - Project B-189  
Contract No. HPS-1(57)  
"A Study of Bearing Capacity of Pile Foundations"

Gentlemen:

From July 1st to September 30th, work continued mainly on the following phases of the project:

1. Tests with 6.75 inch piles buried in sand of various densities.
2. Tests with 8 inch plates at the surface of sand of various densities.
3. Evaluation of test results and preparation of final report.

Tests with 6.75 inch piles and 8 inch plates were completed in August. The results confirm essentially the trend indicated in experiments with 2 and 4 inch piles. Namely, the bearing capacity of piles in homogeneous masses of land is not proportional to depth, as frequently contended.

Following completion of the tests mentioned, work started on testing driven piles of 4 inch diameter.

Respectfully submitted,

Aleksandar B. Vesic  
Project Director

Approved:  

Thomas W. Jackson, Chief  
Mechanical Sciences Division
Division of Highway Planning
State Highway Department of Georgia
Atlanta 3, Georgia

Attention: Mr. Roy A. Flynt
State Highway Planning Engineer

Subject: Quarterly Progress Report No. 9 - Project B-189
Contract No. HPS-1(57)
"A Study of Bearing Capacity of Pile Foundations"
October 1 to December 31, 1962

Gentlemen:

During the above indicated period, testing of 4-inch diameter driven piles was continued. The first results of this second phase of work indicate generally the same trend of increase of bearing capacity with depth as found in tests with buried piles.

The evaluation of test results of the first phase was completed. A theoretical explanation was found for the phenomena observed. The final report has been almost completed and will be submitted in about a month. A paper containing some of the results of the first phase has been prepared for presentation at the next Annual Meeting of the Highway Research Board. This paper has been submitted for approval to Mr. Roy A. Flynt.

Respectfully submitted,

Aleksandar B. Vesic
Project Director

Approved:

Thomas W. Jackson, Chief
Mechanical Sciences Division
Division of Highway Planning
State Highway Department of Georgia
Atlanta 3, Georgia

Attention: Mr. Roy A. Flynt
State Highway Planning Engineer

Subject: Quarterly Progress Report No. 10 - Project B-189
Contract No. HPS-1(57)
"A Study of Bearing Capacity of Pile Foundations"
January 1 to March 31, 1963

Gentlemen:

During the reporting period, testing of 4-inch diameter driven piles was completed. The results show generally the same trend of increase of bearing capacity with depth as that observed in tests with buried piles: at shallow depth increase is linear, at greater depths a constant final value is reached which depends only upon relative density of the sand.

In order to summarily check these findings in the field, the same 4-inch pile was driven and tested in stages on a selected site in Crawford County, Georgia. The soil profile at that site consists of a deep mass of homogeneous, moist sand. The results of these tests confirm the main findings of the model tests performed in the laboratory.

A proposal was prepared for continuation and completion of this research project by 1964.

Respectfully submitted,

Aleksandar B. Vesic
Project Director
Division of Highway Planning  
State Highway Department of Georgia  
Atlanta 3, Georgia  

Attention: Mr. Roy A. Flynt  
State Highway Planning Engineer  

Subject: Quarterly Progress Report No. 11 - Project B-189  
Contract No. HPS-1(57)  
"A Study of Bearing Capacity of Pile Foundations"  
April 1 to June 30, 1963  

Gentlemen:  

During the reporting period, the main effort was devoted to preparations for field tests with instrumented piles (Phase III). It has been decided to make these tests at the I-16 crossing of the Ogeechee River in Effingham County. A detailed testing program was prepared. Plans were drawn for the reaction structure, a very heavy steel box girder designed for a reaction of 1,000 kips over a span of 60 feet. This girder will be delivered by the American Bridge Division of United States Steel.  

At the same time, in cooperation with the State Highway Department, specifications were prepared for the erection of the reaction structure and the testing of piles. Studies were made also of instrumentation to be used for static and dynamic strain measurements.  

Finally, work was started on analysis of test results of Phase II.  

Respectfully submitted,  

Aleksandar B. Vesić  
Project Director
Division of Highway Planning
State Highway Department of Georgia
Atlanta 3, Georgia

Attention: Mr. Roy A. Flynt
State Highway Planning Engineer

Subject: Quarterly Progress Report No. 12 - Project B-189
Contract No. HPS-1(57)
"A Study of Bearing Capacity of Pile Foundations"
July 1 to September 30, 1963

Gentlemen:

During the reporting period preparations for field tests with instrumented piles were continued. Plans and specifications for second subcontract (driving and testing) were completed. Bids were requested and received for that subcontract. However, difficulties were encountered with the right-of-way which could not be secured, due to a pending court action. Since starting of field work at any date after October 1 would involve almost certain flooding of the site before the completion of the tests, it was decided to postpone the planned tests of Phase III until next spring.

During the same reporting period work was started on Phase IV of the project - investigation of pile groups. A detailed testing program was prepared. Material for model piles (4-inch aluminum tubes) was purchased and the necessary number of piles instrumented. Two tests with single piles forced by jacking to the desired depth were completed. The third test, in which a group of four piles spaced two diameters center-to-center is forced into a medium dense sand, is in progress.

Finally, work was continued on analysis of test results of Phase II.

Respectfully submitted,

Aleksandar B. Vesic
Project Director
Division of Highway Planning  
State Highway Department of Georgia  
Atlanta 3, Georgia

Attention: Mr. Roy A. Flynt  
State Highway Planning Engineer

Subject: Quarterly Progress Report No. 13 - Project B-189  
Contract No. HPS-1(57)  
"A Study of Bearing Capacity of Pile Foundations"  
October 1 to December 31, 1963

Gentlemen:

During the reporting period, work was continued on instrumentation for field tests with 18-inch piles (Phase III). In particular, measurements for recording dynamic strains during pile driving were prepared and studied. Preparations were also made for machining the pieces of 18-inch pipe that will be used in the tests.

Work was continued also on Phase IV - investigation of pile groups. Miscellaneous technical difficulties encountered in testing the first four-pile group were finally overcome and that test was successfully completed.

This is the last quarterly progress report on the project. According to instructions received by the sponsor, monthly progress reports will be submitted in the future, starting with February 1, 1964.

Respectfully submitted,

Aleksandar B. Vesic  
Project Director

ABV/c
Division of Highway Planning  
State Highway Department of Georgia  
Atlanta, Georgia  

Attention: Mr. H. H. Buckaba  
State Highway Planning Engineer  

Subject: Monthly Progress Report No. 1 - Project B-189  
Contract No. HP-1(57)  
"A Study of Bearing Capacity of Pile Foundations"  
January 1 to January 31, 1966  

Gentlemen:

During the month of January preparations were continued for field tests with 18-inch piles. Pieces of the 18-inch pipe were machined and some of the strain-gauges have been applied.

After completion of first four-group test, results were partly evaluated. The test pit was excavated and backfilled with sand for the next group test. To assure continuous operation, two new student assistants were hired.

Respectfully submitted,

Aleksandar S. Vesic  
Project Director

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March 4, 1964

Division of Highway Planning
State Highway Department of Georgia
Atlanta 3, Georgia

Attention: Mr. H. H. Hucksba
State Highway Planning Engineer

Subject: Monthly Progress Report No. 2 - Project B-139
Contract No. HPS-1(57)
"A Study of Bearing Capacity of Pile Foundations"
February 1 to February 29, 1964

Gentlemen:

During the reporting period, preparations were continued for field tests of 18 inch piles at the Ogeechee River site. The pieces of the future piles were partly equipped with strain gauges. Authorization was given to the Procurement Officer of Georgia Tech to renew the contract for pile driving and testing that was suspended last September because of prospects for too late a start in the fall.

Work on Phase IV, testing of pile groups, was successfully continued and the second model test with a four-pile-group was completed.

Respectfully submitted,

Aleksandar S. Vesic
Project Director

ASV/c
Division of Highway Planning
State Highway Department of Georgia
Atlanta 3, Georgia

Attention: Mr. H. H. Hulkebea
State Highway Planning Engineer

Subject: Monthly Progress Report No. 3 - Project E-189
Contract No. EPS-1(57)
"A Study of Bearing Capacity of Pile Foundations"
March 1 to March 31, 1964

Gentlemen:

During the past month, preparations for field tests of 18-inch instrumented piles at the Ogeechee River site have practically been completed. A meeting was held on March 23rd in the undersigned Project Director's office with the subcontractor of this project, Mr. Fraser. On that occasion, details of pile driving and testing were discussed. The general situation is such that actual work could be started at any date after April 1st. However, the poor weather conditions will probably cause a further delay of perhaps one month in starting date of the field tests.

After completing the second test with a 4-pile group (Phase IV), the test pit in the laboratory is being used for trial driving of sections of the 18-inch pile. In this way, the instruments for static and dynamic strain measurements are being tested prior to actual use in the field.

The results of the second group tests are being analyzed.

Respectfully submitted,

Aleksandar S. Vesic
Project Director

ASV/c

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and the Experiment Station Security Office.
May 8, 1964

Division of Highway Planning
State Highway Department of Georgia
Atlanta 3, Georgia

Attention: Mr. H. H. Hucksbe
State Highway Planning Engineer

Subject: Monthly Progress Report No. 4 - Project B-189
Contract No. HPS-1(57)
"A Study of Bearing Capacity of Pile Foundations"
April 1 to April 30, 1964

Gentlemen:

Poor weather conditions and high water level in the Ogeechee River prevented the start of field testing (Phase III) during the last month. This situation continues as of now. Predictions indicate that field work probably will not be possible before May 20th.

Meanwhile, the last few pieces of the 18" pile were instrumented and calibrated in the laboratory.

The sand model for the third test with a 4-pile group (Phase IV) has been prepared. After forcing the group into the soil, the concrete cap has been formed, and the model completed for testing.

The analysis of results of the second group test has been completed.

Respectfully submitted,

Aleksandar S. Vesic
Project Director

ASV/c
Division of Highway Planning  
State Highway Department of Georgia  
Atlanta 3, Georgia  

Attention:  Mr. H. H. Buckeba  
State Highway Planning Engineer  

Subject:  Monthly Progress Report No. 5 - Project E-189  
Contract No. HFC-1(57)  
"A Study of Bearing Capacity of Pile Foundations"  
May 1 to May 31, 1964  

Gentlemen:  

High water level at the Ogeechee River test site continued throughout the month of May, preventing the start of the planned tests of the Phase III. It is hoped that the field work will be possible sometime in June.  

Another set of trial measurements of dynamic strains, during pile driving, was performed with a 10 ft. section of the 10 in. pile. Encouraging results were finally obtained.  

The third test with a four-pile-group in the laboratory has been completed. The analysis of the results is in progress. The fourth test is being prepared.  

Respectfully submitted,  

Aleksandar S. Vasic  
Project Director  

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Prior to June 3, 1966  
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Division of Highway Planning  
State Highway Department of Georgia  
Atlanta 3, Georgia  

Attention:  Mr. H. H. Hucksba  
State Highway Planning Engineer  

Subject:  Monthly Progress Report No. 6 - Project B-189  
Contract No. HPS-1(57)  
"A Study of Bearing Capacity of Pile Foundations"  
June 1 to June 30, 1964  

Gentlemen:  

The field tests at the Ogeechee River test site have been further  
postponed because of unexpected complications with the easement for the  
access road to the site. It appears that it will be necessary to wait  
until the roadway contractor for the expressway builds his access roads.  

The fourth test with a four-pile group has been completed in the  
laboratory, thus finishing the first series of model tests.  

Additional piles for a nine-pile group have been instrumented and  
calibrated. First test in the second series is now being prepared.  

Respectfully submitted,  

[Signature]  
Aleksandar S. Vesic  
Project Director  

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Prior to July 8, 1964  
without permission of the Research Sponsor  
and the Experiment Station Security Office.
Division of Highway Planning  
State Highway Department of Georgia  
Atlanta 3, Georgia  

Attention: Mr. H. H. Huckeba  
State Highway Planning Engineer  

Subject: Monthly Progress Report No. 7 - Project E-189  
Contract No. HPS-1(57)  
"A Study of Bearing Capacity of Pile Foundations"  
July 1 to July 31, 1964  

Gentlemen:  

During the reporting period, the first test of the second series (nine-pile groups) was successfully completed. The analysis of results of this test is in progress. In the meanwhile, the second test with a nine-pile group has been prepared and almost completed (excepting final penetration tests). Work continued also on analysis of previous test results and on preparation of the final report for Phase II of the project.  

The situation at the Ogeechee test site remains unchanged: it was impossible to secure easement for the access road. There is reason to hope that construction of the access roads by the roadway contractor may start sometime in August. This would make testing at our site possible in early September. However, further delay in construction of the access roads may require postponement of our field testing until next spring.  

Respectfully submitted,  

[Signature]  

Aleksandar S. Vesic  
Project Director  

ASV/c  

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Prior to Aug 1, 1965  
without permission of the Research Sponsor and the Experiment Station Security Office.
Division of Highway Planning  
State Highway Department of Georgia  
Atlanta 3, Georgia  

Attention: Mr. H. H. Huckeba  
State Highway Planning Engineer  

Subject: Monthly Progress Report No. 8 - Project E-189  
Contract No. HPS-1(57)  
"A Study of Bearing Capacity of Pile Foundations"  
August 1 to August 31, 1964  

Gentlemen:

During the month of August, three further large-scale model tests were successfully completed. The results of all tests in Phase IV have been analyzed and presented at a meeting of the Highway Research Council.

There has been no change in status of Phase III - Ogeechee River Tests. It appears that this phase will have to be postponed until the spring or summer of 1965.

Respectfully submitted

[Signature]

Aleksandar S. Vesic  
Project Director  

ASV/0
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Prior to 10-6 1966

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and the Experiment Station Security Office.

\[
\text{October 6, 1964}
\]

Division of Highway Planning
State Highway Department of Georgia
Atlanta 3, Georgia

Attention: Mr. H. H. Huckeba
State Highway Planning Engineer

Subject: Monthly Progress Report No. 9 - Project B-189
Contract No. HPS-1(57)
"A Study of Bearing Capacity of Pile Foundations"
September 1 to September 30, 1964

Gentlemen:

During the month of September, the work on Phase IV (Pile Groups) was continued very actively. Two additional model tests were completed; bringing the total of completed model tests in this phase to thirteen. Work was continued also on analysis of test results of this phase as well as on the preparation of the final report for Phase II.

There was no improvement in the situation on the Ogeechee River site, where field tests with large 18-inch piles had to be postponed because of the access road problems. The long expected conference of State Highway Department officials and contractors involved in work on the Interstate 16, did not help materialize the hopes that an access road may be built sometime in September. In this situation, after consultations with Mr. Marmelstein, State Highway Bridge Engineer, it was decided to again postpone the field tests in question until late spring of 1965.

On September 15, the Project Director moved to Durham, North Carolina, to his new position as Professor of Civil Engineering at Duke University. He will continue directing this project until completion as a consultant of the Engineering Experiment Station of Georgia Tech.

Respectfully submitted,

[Signature]
Aleksandar S. Vesic’
Project Director

[Signature]

ASV/c

REVIEW

PATENT 10-13’ 67
FORMAT 10-13’ 1964
November 9, 1964

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Division of Highway Planning
State Highway Department of Georgia
Atlanta 3, Georgia

Attention: Mr. H. H. Huckeba
State Highway Planning Engineer

Subject: Monthly Progress Report No. 10 - Project B-189
Contract No. HPS-1(57)
"A Study of Bearing Capacity of Pile Foundations"
October 1 to October 31, 1964

Gentlemen:

During the month of October, another model test with a single pile
penetrating through loose sand to a firm stratum of dense sand was com-
pleted. Work was continued also on analysis of test results as well as
on the preparation of the final report for Phases II and IV.

The project work is generally progressing well and it is anticipated
that it will be finished on time, excepting Phase III (Ogeechee River Tests),
which have been postponed until next spring.

Respectfully submitted,

Aleksandar S. Vesic
Project Director

ASV/c

REWV
PATENT 11-18 1964 BY
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Division of Highway Planning
State Highway Department of Georgia
Atlanta 3, Georgia

Attention: Mr. H. H. Huckeba
State Highway Planning Engineer

Subject: Monthly Progress Report No. 11 - Project B-189
Contract No. HPS-1(57)
"A Study of Bearing Capacity of Pile Foundations"
November 1 to November 30, 1964

Gentlemen:

During the month of November, an additional model test with a four-pile group penetrating through loose sand to a firm stratum of dense sand was completed. The bearing capacity recorded was approximately equal to four times the bearing capacity of a previously tested single pile. Work was continued also on the preparation of final reports for Phases II and IV. It is found generally that the results are consistent enough to permit drawing very definite conclusions concerning the behavior of piles and pile groups in sand.

Early during the month the Project Director received a special invitation from the Organizing Committee for the North American Conference on Deep Foundations, an international meeting organized jointly by the American and Mexican Societies of Civil Engineers, to act as a panel member in Session II of the Conference, devoted to model tests on deep foundations. This may be considered as another recognition for the research work performed in this project, which is becoming internationally known because of the significance of its results.

The Project Director extends herewith his apologies for the delay of this progress report, caused by the extreme shortage of time he experienced in the period when the report was due (the mentioned Conference took place December 7 through 12).

Respectfully submitted,

Aleksandar S. Vesic
Project Director

ASV/c
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Division of Highway Planning
State Highway Department of Georgia
Atlanta 3, Georgia

Attention: Mr. H. H. Huckeba
State Highway Planning Engineer

Subject: Monthly Progress Report No. 12, Project B-169
Contract No. HP5-1(57)
"A Study of Bearing Capacity of Pile Foundations"
December 1 to December 31, 1964

Gentlemen:

During the reporting period the last model test, made with a single
pile in submerged sand, was successfully completed. Work was continued
also on analysis of data from previous tests as well as on preparation of the
final report.

Respectfully submitted,

Aleksandar S. Veselj
Project Director

ASV/6
Division of Highway Planning  
State Highway Department of Georgia  
Atlanta 3, Georgia

Attention: Mr. H. H. Huckeba  
State Highway Planning Engineer

Subject: Monthly Progress Report No. 13 - Project B-189  
Contract No. HPS-1(57)  
"A Study of Bearing Capacity of Pile Foundations"  
January 1 to January 31, 1965

Gentlemen:

During the month of January some work was done on analysis of data from just completed laboratory tests with groups of piles.

This project is now entering a dormant stage until the beginning of field tests at the Ogeechee River site. The only work anticipated in the coming months will be done by the Project Director himself (administration and analyzing the data in preparation for the final report).

Respectfully submitted,

Aleksandar Sedmak Vesic  
Professor and Project Director B-189

ASV:smc

ENGINEERING EXPERIMENT STATION  
GEORGIA INSTITUTE OF TECHNOLOGY  
ATLANTA, GEORGIA
March 25, 1965

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Division of Highway Planning
State Highway Department of Georgia
Atlanta 3, Georgia

Attention: Mr. H. H. Huckaba
State Highway Planning Engineer

Subject: Monthly Progress Report No. 14 - Project B-189
Contract No. HPS-1(57)
"A Study of Bearing Capacity of Pile Foundations"
February 1 to February 28, 1965

Gentlemen:

Excepting the minor administrative work in connection with the subject project, no other work on the project was performed during the reporting period.

Respectfully submitted,

[Signature]

Aleksandar Sedmak Vesic
Professor and Project Director B-189

ENGINEERING EXPERIMENT STATION
GEORGIA INSTITUTE OF TECHNOLOGY
ATLANTA, GEORGIA
Division of Highway Planning  
State Highway Department of Georgia  
Atlanta 3, Georgia

Attention:  Mr. H. H. Huckeba  
State Highway Planning Engineer

Subject:  Monthly Progress Report No. 15 - Project B-169  
Contract No. HPS-1(57)  
"A Study of Bearing Capacity of Pile Foundations"  
March 1 to March 31, 1965

Gentlemen:

Excepting the minor administrative work in connection with the subject project, no other work on the project was performed during the reporting period.

Respectfully submitted,

Aleksander Sedmak Vesic  
Project Director
May 6, 1965

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and the Experiment Station Security Office.

Division of Highway Planning
State Highway Department of Georgia
Atlanta 5, Georgia

Attention: Mr. H. H. Huckeba
State Highway Planning Engineer

Subject: Monthly Progress Report No. 16 - Project B-189
Contract No. HPS-1(57)
"A Study of Bearing Capacity of Pile Foundations"
April 1 to April 30, 1965

Gentlemen:

Excepting the minor administrative work in connection with the
subject project, no other work on the project was performed during the
reporting period.

Respectfully submitted,

[Aleksandar Sedmak Vesic]
Project Director

ASV/c

REVIEW
PATENT 5-18-1965 BY
FORMAT 5-18-1965 BY
Division of Highway Planning  
State Highway Department of Georgia  
Atlanta 3, Georgia

Attention: Mr. H. H. Huckeba  
State Highway Planning Engineer

Subject: Monthly Progress Report No. 17 - Project B-189  
Contract No. HPS-1(57)  
"A Study of Bearing Capacity of Pile Foundations"  
May 1 to May 31, 1965

Gentlemen:

Excepting the minor administrative work in connection with the subject project, no other work on the project was performed during the reporting period.

Respectfully submitted,

[Signature]

Aleksandar Sedmak Vesic  
Project Director

ASV/c
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Division of Highway Planning
State Highway Department of Georgia
Atlanta 3, Georgia

Attention: Mr. H. H. Huckeba
State Highway Planning Engineer

Subject: Monthly Progress Report No. 18 - Project B-189
Contract No. HP3-1(57)
"A Study of Bearing Capacity of Pile Foundations"
June 1 to June 30, 1965

Gentlemen:

During the month of June, Phase III of the subject research project
was reactivated. Mr. Robert P. Dennis, B.S. (Georgia Institute of Technology),
was hired as a Research Assistant to take charge of electronic measurements
in the field test. Contacts were re-established with R. F. Burton, Company,
subcontractor in charge of erection of steel structure and pile driving. This
company delegated Mr. Seth Ward, B.S. (University of Alabama), to be in
charge of their field work.

On the basis of a report by Mr. S. E. Smith, Jr., Resident Highway
Engineer at Savannah, Georgia, the date for starting the field tests has been
set for July 26, the earliest date when the access to the site might be available.
All preparations and administrative measures are being taken to assure the
start of operations by that date.

Respectfully submitted,

Aleksandar Sedmak Vesic
Project Director

ASV/c
DIVISION OF HIGHWAY PLANNING
STATE HIGHWAY DEPARTMENT OF GEORGIA
ATLANTA, GEORGIA

ATTENTION: MR. H. H. HUCKEBA
STATE HIGHWAY PLANNING ENGINEER

SUBJECT: MONTHLY PROGRESS REPORT NO. 19 - PROJECT B-189
CONTRACT NO. HPS-1(57)
"A STUDY OF BEARING CAPACITY OF PILE FOUNDATIONS"
JULY 1 TO JULY 31, 1965

GENTLEMEN:

Early in July the Project Director held a conference in Atlanta with the representatives of the State Highway Department, particularly Messrs. Vernon W. Smith, Jr. and Thomas D. Moreland in connection with the forthcoming start of field testing at the Ogeechee River site. He also discussed at length all the technical problems involved in planned testing with Mr. Seth Ward, the newly appointed representative of the R. F. Burton Construction Company, as well as with Mr. Robert P. Dennis, Assistant Research Engineer on the project. As a result of these conferences and other contacts between the interested parties, a firm schedule of events was established, indicating the start of field operations on August 2, and their completion on September 10, 1965. A copy of the schedule is enclosed.

The major part of the work during this month was done on recalibration of test pile sections. Because of unavailability of the Georgia Tech testing machine, the work was subcontracted with the Lockheed-Georgia Company in Marietta, Ga. This allowed at the same time to calibrate the instrumented pile up to its design load of 1,000,000 lb., whereas the former calibration was made only up to 450,000 lb., the limit of capacity of the Georgia Tech testing machine. One section of the test pile was damaged during the loading operations and had to be replaced. Other five sections performed correctly.

It was decided to try to perform a pull-out test with the instrumented pile at the end of the testing operations. For this purpose a pull-out testing assembly is being designed and constructed.

Respectfully submitted,

ALEKSANDAR SEDMAK VESTEC
PROJECT DIRECTOR

ASV/e
# OGEECHEE RIVER TESTS

## Revised Schedule of Events

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<td>Mon 26 July</td>
<td>Layout Group of Highway Department of site (Moreland, Smith)</td>
</tr>
<tr>
<td>Tue 27</td>
<td>Steel shipped from Birmingham, Alabama (Ward)</td>
</tr>
<tr>
<td>Wed 28</td>
<td>Calibrated pile shipped from Atlanta (Dennis &amp; V. Smith)</td>
</tr>
<tr>
<td>Thu 29</td>
<td>Reaction piles shipped from Foster Co. (Ward)</td>
</tr>
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<td>Fri 30</td>
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| Mon 2 August | Orientation on site                                                                 |
| Tue 3       |                                                                                   |
| Wed 4       | Driving reaction piles                                                             |
| Thu 5       |                                                                                   |
| Fri 6       |                                                                                   |

| Mon 9 August | Erecting reaction structure                                                        |
| Tue 10      |                                                                                   |
| Wed 11      |                                                                                   |
| Thu 12      |                                                                                   |
| Fri 13      | Driving (Stage 1)                                                                 |

| Mon 16 August | Testing (Stage 1)                                                                 |
| Tue 17       |                                                                                   |
| Wed 18       | " Stage 3                                                                          |
| Thu 19       | " Stage 4                                                                          |
| Fri 20       | " Stage 5                                                                          |

| Mon 23 August | " Stage 6                                                                          |
| Tue 24       | Changing to pulling assembly                                                        |
| Wed 25       | Pullout test                                                                        |
| Thu 26       | Moving the reaction structure                                                       |
| Fri 27       |                                                                                   |

| Mon 30 August |                                                                                   |
| Tue 31       |                                                                                   |
| Wed 1 September | Driving concrete pile                |
| Thu 2        | Testing concrete pile                                                              |
| Fri 3        | Disassembling the reaction structure                                               |

| Mon 6        | LABOR DAY                                                                         |
| Tue 7        | Disassembling the reaction structure                                               |
| Wed 8        |                                                                                   |
| Thu 9        | Clean up and miscellaneous                                                         |
| Fri 10       |                                                                                   |

July 28

ASV
Division of Highway Planning
State Highway Department of Georgia
Atlanta 3, Georgia

Attention: Mr. H. H. Huckeba
State Highway Planning Engineer

Subject: Monthly Progress Report No. 20 - Project B-189
Contract No. HPS-1(57)
"A Study of Bearing Capacity of Pile Foundations"
August 1 to August 31, 1965

Gentlemen:

During the reporting period, field operations at the Ogeechee River site were started and a significant progress in pile testing was achieved, though not exactly according to the previously made work schedule. There was some delay in actual start of the operations, because of late arrival of elements of the steel structure from Birmingham, Alabama. Also, transportation of steel elements from the railway station in Meldrin and erection of the structure took more time than anticipated.

The erection of the reaction structure started on August 13, and was completed on August 24. August 25 was used to correct some defects in the bracing of steel panels of the structure and to drive the first piece of instrumented pile. The first stage of testing was completed on August 26; and the second stage on August 27. The remaining four stages will be completed on a new schedule, that is by September 2. Barring unforeseen events, it can be expected that the field operations will be completed by, approximately, September 20.

The Project Director inspected the site on August 13 and August 24 keeping, in the meantime, contacts with the project personnel by telephone. On this latter date he was accompanied by Mr. John Bowman, of Troxler Laboratories, Inc., who performed nuclear measurements of density and water content of soil at three characteristic locations on the site and over a depth of approximately 50 ft. In this work, performed without charge to the project, he was assisted by Mr. Thomas D. Moreland, Soils Engineer of the State Highway Department of Georgia, and a member of his staff. The results obtained are expected to be most useful in interpretation of load test results, since they will provide an exact profile of relative density and moisture content of the sand mass through which the piles are being driven.

Respectfully submitted,

Aleksandar Sedmak Vesic
Project Director

ASV/c
October 8, 1965

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Division of Highway Planning
State Highway Department of Georgia
Atlanta 3, Georgia

Attention: Mr. Emory C. Parrish
State Highway Planning Engineer

Subject: Monthly Progress Report No. 21 - Project B-189
Contract No. HP6-1(57)
"A Study of Bearing Capacity of Pile Foundations"
September 1 to September 30, 1965

Gentlemen:

During the month of September the field tests at the Ogeechee River
were successfully completed. The 18-inch diameter steel pipe pile was
tested in five stages, to a depth of 50 ft. The sixth stage was not attempted
because of the fact that the bearing capacity exceeded the design load of the
reaction structure and of the loading jack: 500 tons. The 16-inch square
prestressed concrete pile was driven and tested at a total length of 50 ft.

Immediately after the completion of the tests, work was started on
evaluating and analyzing the results. This work is planned to continue until
approximately December 1 when writing of the final report of the Phase IV
has been scheduled to begin.

Respectfully submitted,

[Signature]

Aleksandar Sedmak Vesic
Project Director

ASV/c
Division of Highway Planning
State Highway Department of Georgia
Atlanta 3, Georgia

Attention: Mr. Emory C. Parrish
State Highway Planning Engineer

Subject: Monthly Progress Report No. 22 - Project B-189
Contract No. HPS-1(67)
"A Study of Bearing Capacity of Pile Foundations"
October 1 to October 31, 1965

Gentlemen:

During the reporting period work was continued on evaluation and
analysis of test data of the Ogeechee River tests. The Final Report for
Phase IV of the Project is also in the stage of final analyses and write-up.

Respectfully submitted,

Aleksandar Sedmak Vesic
Project Director

ASV/e
Division of Highway Planning  
State Highway Department of Georgia  
Atlanta 3, Georgia  

Attention: Mr. Emory C. Parrish  
State Highway Planning Engineer  

Subject: Monthly Progress Report No. 23 - Project B-189  
Contract No. HPS-1(57)  
"A Study of Bearing Capacity of Pile Foundations"  
November 1 to November 30, 1965  

Gentlemen:  

During the reporting period work was continued on evaluation and analysis of test data of the Ogeechee River tests. The Final Report for Phase IV of the Project is also in the stage of final analyses and write-up.  

Respectfully submitted,  

Aleksandar Sedmak Vesic  
Project Director  

ASV/o
January 9, 1966

Division of Highway Planning
State Highway Department of Georgia
Atlanta 3, Georgia

Attention: Mr. Emory C. Parrish
State Highway Planning Engineer

Subject: Monthly Progress Report No. 24 - Project B-169
Contract No. HPS-1(57)
"A Study of Bearing Capacity of Pile Foundations"
December 1 to December 31, 1965

Gentlemen:

During the reporting period work was continued on evaluation and
analysis of test data of the Ogeechee River tests. The Final Report for
Phase IV of the Project is also in the stage of final analyses and write-up.

Respectfully submitted,

Aleksandar S. Vesic
Project Director

ASV/c
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GEORGIA INSTITUTE OF TECHNOLOGY
ENGINEERING EXPERIMENT STATION
ATLANTA, GEORGIA 30332

February 9, 1966

Division of Highway Planning
State Highway Department of Georgia
Atlanta 3, Georgia

Attention: Mr. Emory C. Parrish
State Highway Planning Engineer

Subject: Monthly Progress Report No. 25 - Project B-109
Contract No. HP8-1(57)
"A Study of Bearing Capacity of Pile Foundations"
January 1 to January 31, 1966

Gentlemen:

During the month of January the analysis of data obtained in the Ogeechee River pile load tests was continued. Basic processing of all data has been completed. Work continues on detailed processing and interpretation.

Further progress was achieved in the preparation of the final report for the entire project. At present the effort is concentrated on interpretation of load tests with pile groups (Phase IV).

Respectfully submitted,

Aleksandar S. Vesic
Project Director

ASV/c
July 8, 1966

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Division of Highway Planning   
State Highway Department of Georgia   
Atlanta 3, Georgia

Attention:  Mr. Emory C. Parrish   
State Highway Planning Engineer

Subject:  Monthly Progress Report No. 26 - Project B-189
Contract No. HPS-1(67)
"A Study of Bearing Capacity of Pile Foundations"
June 1 to June 30, 1966

Gentlemen:

Following an interruption of several months, caused by foreign travel
and other commitments of the project director, the project work was resumed
again on June 6.

Preparations for calibration tests that would allow interpretation of
nuclear density and moisture content tests were started and completed. The
calibration tests were made during the last week of the month, with the
assistance of Mr. John Bowman of Troxler Laboratories, Inc. (It should be
noted that this company offered the use of their equipment as well as the
assistance of their engineer without any charge to this project.) Work is now
being continued on interpretation of density and moisture content data obtained.

The work on the final report for the entire project has now entered the
final stage. During the reporting period the final draft of the report pertaining
to the phases I and II has been revised and completed. The work still continues
on chapters pertaining to phases III and IV. Numerous new analyses have been
added in this last stage. First drafts of chapters dealing with deep foundations
in cohesive soils, as well as with recommended design procedures, are also
being prepared.

Respectfully submitted,

[Signature]

Aleksandar S. Vesic,  
Project Director

ASV/c
LIBRARY DOES NOT HAVE MONTHLY PROGRESS REPORT # 27
Division of Highway Planning  
State Highway Department of Georgia  
Atlanta 8, Georgia

Attention: Mr. Emory C. Parrish  
State Highway Planning Engineer

Subject: Monthly Progress Report No. 28 - Project B-189  
Contract No. HPS-1(67)  
"A Study of Bearing Capacity of Pile Foundations"  
August 1 to August 31, 1968

Gentlemen:

During the month of August the final report of the subject research project was completed. A delay in tracing and layout of figures made it impossible to deliver the draft of the report by the end of the month as originally scheduled. This work is being done at present and it is anticipated that the draft will finally be in sponsor's hands about September 20.

Respectfully submitted,

[Signature]

Aleksandar S. Vesic  
Project Director

ASV/c
Division of Highway Planning
State Highway Department of Georgia
Atlanta, Georgia

Attention: Mr. Emory C. Parrish
State Highway Planning Engineer

Subject: Monthly Progress Report No. 29 - Project B-189
Contract No. HPS-1(57)
"A Study of Bearing Capacity of Pile Foundations"
September 1 to September 30, 1966

Gentlemen:

The draft of the final report of the subject research project was completed and reproduced during this month. Four copies were sent to you for review on September 19. Tracing of some figures in ink still continues and will be completed early next month.

Respectfully submitted,

[Signature]

Aleksandar S. Vesic
Project Director

ASV/c
Division of Highway Planning  
State Highway Department of Georgia  
Atlanta 3, Georgia

Attention: Mr. Emory C. Parrish  
State Highway Planning Engineer

Subject: Monthly Progress Report No. 30 - Project B-189  
Contract No. HPS-1(57)  
"A Study of Bearing Capacity of Pile Foundations"  
October 1 to October 31, 1966

Gentlemen:

During the month of October the tracing of the remaining figures of the final report was completed. This concluded the work on the subject research project, excepting possible revisions of the draft of the final report and printing.

The Project Director wishes to use this opportunity to thank all the administrators concerned for their assistance and understanding during the execution of this project. He hopes that the results obtained will help improve the design procedures for deep foundations of highway bridges, resulting in savings that will pay many times the cost and effort associated with this project.

Respectfully submitted,

Aleksandar Sedmak Vesic  
Project Director

ASV/c
FINAL REPORT

PROJECT B-189

A STUDY OF BEARING CAPACITY OF DEEP FOUNDATIONS

By
Aleksandar S. Vesić
Professor of Civil Engineering

Contract with
STATE HIGHWAY DEPARTMENT OF GEORGIA
In Cooperation with
U. S. DEPARTMENT OF COMMERCE
BUREAU OF PUBLIC ROADS

March 31, 1967

School of Civil Engineering
GEORGIA INSTITUTE OF TECHNOLOGY
Atlanta, Georgia
A STUDY OF BEARING CAPACITY OF DEEP FOUNDATIONS

By

Aleksandar S. Vesic
Professor of Civil Engineering

Contract with
STATE HIGHWAY DEPARTMENT OF GEORGIA
In cooperation with
U. S. DEPARTMENT OF COMMERCE
BUREAU OF PUBLIC ROADS

March 31, 1967

The opinions, findings, and conclusions expressed in this publication are those of the author and not necessarily those of the State Highway Department of Georgia or the Bureau of Public Roads.
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NOTATIONS

Notations are accompanied by basic dimensions in English units (in., lb., sec.)
The following relationships can be used for conversion to other units.

\[
\begin{align*}
1 \text{ in.} & = 2.54 \text{ cm} = 1/12 \text{ ft.} \\
1 \text{ lb.} & = 0.454 \text{ kg} = 1/1,000 \text{ kip} = 1/2,000 \text{ ton} \\
1 \text{ ton/ft.}^2 & = 13.9 \text{ lb./in.}^2 = 0.98 \text{ kg/cm}^2
\end{align*}
\]

\[b_r = \text{cap rim width (in.)}\]
\[c = \text{shear strength intercept (cohesion) (lb./in.}^2)\]
\[c_1, c_2, c_3 = \text{temporary pile compressions (in.)}\]
\[c_a = \text{adhesion along pile shaft (lb./in.}^2)\]
\[e = \text{coefficient of restitution in Hiley formula (dimensionless)}\]
\[e = \text{void ratio of the soil (dimensionless)}\]
\[e = \text{pile spacing (in.)}\]
\[e_f = \text{driving energy efficiency (dimensionless)}\]
\[f_o = \text{unit skin (shaft) resistance (lb./in.}^2)\]
\[h = \text{ram stroke (in.)}\]
\[k_n = \text{coefficient of (axial) pile reaction (lb./in.)}\]
\[m = \text{number of pile rows (dimensionless)}\]
\[n = \text{soil porosity (percent)}\]
\[p_s = \text{normal pressure on foundation shaft (skin) (lb./in.}^2)\]
\[q = \text{effective overburden pressure (lb./in.}^2)\]
\[q_c = \text{cone point resistance (lb./in.}^2)\]
\[q_v = \text{effective vertical stress at the elevation of foundation base (lb./in.}^2)\]
\[q_o = \text{unit point (base) resistance (lb./in.}^2)\]
NOTATIONS (Continued)

\( q_s \) = effective vertical stress along the skin (lb./in.\(^2\))
\( s \) = pile set (in.)
\( s_1 \) = driving constant in the EN formula (in.)
\( w \) = pile settlement (in.)
\( \bar{w} \) = pile group settlement (in.)
\( w_p \) = settlement of pile point (in.)
\( w_s \) = settlement due to pile shaft (in.)
\( A \) = cross-sectional area of pile shaft (in.\(^2\))
\( A_p \) = bearing area of the point (base) (in.\(^2\))
\( A_r \) = effective bearing area of the pile cap rim (in.\(^2\))
\( A_s \) = bearing area of the skin (shaft) (in.\(^2\))
\( B \) = foundation width or pile diameter (in.)
\( \bar{B} \) = pile group width (in.)
\( C_w \) = settlement coefficient (dimensionless)
\( D \) = foundation depth or embedded pile length (in.)
\( D_r \) = relative density (dimensionless).
\( E \) = modulus of deformation of the soil (lb./in.\(^2\))
\( E' \) = plane-strain modulus of deformation = \( E/1-\nu^2 \) (lb./in.\(^2\))
\( E_b \) = modulus of deformation of the pile shaft (lb./in.\(^2\))
\( K_s \) = coefficient of skin pressure (dimensionless)
\( K_{s*} \) = \( \zeta_s K_s \) = coefficient of skin resistance for a circular or square shaft (dimensionless)
\( L \) = pile length (in.)
\( N \) = standard penetration test blow count (dimensionless)
NOTATIONS (Continued)

$N_c$, $N_q$, $N_\gamma$ = bearing capacity factors (dimensionless)

$N_c^*$, $N_q^*$ = bearing capacity factors for a deep circular or square foundation (dimensionless)

$Q$ = total foundation load (lb.)

$Q_c$ = ultimate cap load (lb.)

$Q_o$ = ultimate total load (lb.)

$\delta_o$ = ultimate group load (lb.)

$Q_p$ = ultimate point (base) load (lb.)

$Q_s$ = ultimate skin (shaft) load (lb.)

$R$ = ultimate dynamic load (lb.)

$W_p$ = weight of the pile (lb.)

$W_r$ = weight of the ram (lb.)

$\alpha$ = coefficient of distribution of skin friction (dimensionless)

$\beta$ = adhesion reduction coefficient (regeneration factor) (dimensionless)

$\gamma$ = effective unit weight of soil (lb./in.$^3$)

$\gamma_d$ = dry unit weight of the soil (lb./in.$^3$)

$\delta$ = angle of skin friction (dimensionless)

$\zeta$ = group settlement factor (dimensionless)

$\zeta_c$, $\zeta_q$, $\zeta_\gamma$ = shape factors for point resistance (dimensionless)

$\zeta_s$ = shape factor for skin resistance (dimensionless)

$\eta$ = pile group efficiency (dimensionless)

$\theta$ = apex angle of the plastic wedge

$\nu$ = Poisson's ratio (dimensionless)

$\pi$ = 3.1416 (dimensionless)
\[ \rho = \text{stress acting on rupture line (lb./in.}^2) \]
\[ \sigma = \text{normal stress (lb./in.}^2) \]
\[ \sigma_1, \sigma_2 = \text{major and minor principal stresses (lb./in.}^2) \]
\[ \tau = \text{shearing stress (lb./in.}^2) \]
\[ \phi = \text{angle of shearing resistance of the soil (dimensionless)} \]
Introduction

Highway bridges are frequently located in areas where the upper soil strata are too weak to sustain heavy structures and where the loads have to be transferred to suitable layers by means of deep foundations. Such foundations are also sometimes imposed by the requirement of safety against scour even on sites where relatively incompressible strata such as sands or gravels are found at shallow depth. In any case, the problem of bearing capacity of deep foundations is quite important or even critical for safe and economical bridge design.

By the method of construction, there are three general types of deep foundations: piles, piers, caissons. The varieties within each of these general types are very numerous (there are, for example, over 70 different types of piles described in the engineering literature). Still, from the point of view of soil mechanics, there are only two general types of deep foundations. The first type may be represented by a foundation installed by some process of excavation or drilling which does not induce significant changes in density or structure of the bearing soil. Practically all piers and caissons and some piles belong to this type. The other type may be represented by a deep foundation forced into the ground by driving or a similar operation, which induces significant changes in adjacent soil. Most of piles belong to this second type. The significant difference in bearing capacities of different deep foundation types under otherwise identical conditions should be recognized from the beginning.

The evaluation of bearing capacity of deep foundations has been based in the past mostly on previous experience, full-size load tests and the use of dynamic pile formulas. Numerous investigations performed during the past
twenty-five years have shown that dynamic formulas are not reliable and that even a full-size load test can give misleading results if not conducted and interpreted properly.

It is generally admitted today that the analysis of bearing capacity of deep foundations should be based on strength and compressibility characteristics of soil strata affected by construction operations and imposed loads. However, although formulas for bearing capacity based on strength considerations have been derived, there is little reliable experimental data about the actual values of bearing capacity factors and coefficients pertaining to such frequent soil types as sands or silts. Very little is known about actual shear phenomena around deep foundations, particularly around pile groups.

It is the object of this research project to investigate fundamental parameters influencing the bearing capacity of deep foundations, particularly piles and pile groups, with the final aim of improving the rational approach to design of such foundations.

The entire project was divided into four phases. Phase I, executed in 1960 through 1962, contained a study of past research and a large-scale model investigation of deep foundations in homogeneous masses of dry sand. Phases II and III, executed in 1962 through 1965, consisted of model studies of bearing capacity of driven piles in sand, as well as of full-scale tests of piles driven on selected sites in Georgia. Phase IV, executed in 1963/64, contained investigations of group action of driven piles in sand, as well as studies of effect of ground-water conditions and some special tests.

The present report is divided into five chapters. Chapter I contains a review of existing theories of bearing capacity of deep foundations. Chapter II discusses the bearing capacity of deep foundations in homogeneous masses of
sand. Chapter III is devoted to the experiments with piles driven into homogeneous masses of sand. Chapter IV reports on field tests with large instrumented piles driven into a natural sand deposit at the Ogeechee River site in Effingham County, Georgia. A report on experiments with pile groups in sand is presented in Chapter V. A discussion of bearing capacity of deep foundations in clays, silts and other soil types follows in Chapter VI.

Finally, Chapter VII contains general conclusions and recommendations for design of deep foundations.
CHAPTER I

Theories of Bearing Capacity of Deep Foundations

Statement of the Problem

The basic problem of bearing capacity of deep foundations can be formulated as follows (Fig. 1): A rigid foundation of known shape and dimensions is placed at a depth D in a homogeneous mass of soil of defined physical properties. A static, vertical, central load is applied on the top. What is the ultimate load $Q_0$ that this foundation can support?

The load is generally transmitted partially along the foundation shaft or skin, partially at the foundation base or point. The two bearing components of the load, the skin load $Q_s$ and the base or point load $Q_p$, are usually considered separately. The total ultimate load is then expressed as the sum of these two components:

$$Q_0 = Q_p + Q_s = q_o A_p + f_o A_s$$  (1)

Here $q_o$ represents the unit base resistance and $f_o$ unit skin resistance of the foundation (lb/in$^2$ or kg/cm$^2$). $A_p$ and $A_s$ are, respectively, bearing areas of the base and the skin.

In this way the problem is reduced to separate determination of base resistance $q_o$ and skin resistance $f_o$.

Bearing Capacity of the Base

The solution of the problem of bearing capacity of the base has been sought in the past primarily by an approach based on the classical work by Prandtl (1,2), and Reissner (3). They have presented a solution of the pro-
Figure 1. Basic Problem of Bearing Capacity of a Deep Foundation.

\[ Q = Q_p + Q_s = q_o A_p + f_o A_s \]
blem of penetration of a rigid stamp into an incompressible (rigid-plastic) solid (Fig. 2). That solution, first applied to the problem of bearing capacity of soils by Caquot (4) and Buisman (2) is usually written in the following general form (6):

\[ q_o = cN_c \zeta_c + qN_q \zeta_q + \frac{1}{2} \gamma BN_c \zeta_y \]  

(2)

In expression (2) \( c \) represents the shear strength intercept (cohesion) of the soil, \( q \) the overburden pressure, \( \gamma \) the unit weight of the soil involved in shear and \( B \) the foundation width. \( N_c, N_q, N_\gamma \) are bearing capacity factors for a strip foundation and \( \zeta_c, \zeta_q, \zeta_\gamma \) are shape factors. Both \( N \) and \( \zeta \) factors are generally, dimensionless functions of the angle of shearing resistance \( \varphi \).

In the case of cohesionless material, such as dry or submerged sand \( c = 0 \) and

\[ q_o = qN_q \zeta_q + \frac{1}{2} \gamma BN_c \zeta_y \]  

(3)

In the case of frictionless material such as saturated clays in undrained conditions of loading, \( \varphi = 0 \), \( N_q = \zeta_q = 1 \) and \( N_\gamma = 0 \), so that

\[ q_o = cN_c \zeta_c + q \]  

(4)

Using the same general approach but different shear patterns with rupture lines reverting to the shaft (Fig. 3) De Beer (7,8) and Jáky (2) have obtained another solution of the same problem with considerably higher bearing capacity factors. Their work has been further extended by Meyerhof (10, 11), and others. However, recognizing the effects of compressibility in real soils, De Beer used this solution only for evaluations of the lower limits of the shear
Figure 2. Prandtl-Reissner Solution as Applied by Caquot and Buisman.
Figure 3. Shear Pattern with Rupture Lines Reverting to the Shaft.
strength in situ from penetrometer tests.

It should be noted that all the mentioned solutions were obtained by considering a plane problem (long rectangular foundation). Some work in developing solutions for an axially symmetrical problem (circular foundation) has also been done in the USSR, primarily by Berezantsev (12). However, there is still a tendency to determine the shape factors empirically.

Another possible approach to the problem of base bearing capacity originated in the work by Bishop, Hill and Mott (13), who have considered the problem of expansion of a spherical or cylindrical cavity inside an infinite mass of an ideal solid. In such a case there exists around the cavity a highly stressed zone where the material, by assumption, behaves as a rigid-plastic solid. Outside that zone it behaves as an ideal elastic (or linearly deformable) solid.

The solutions of this kind have been applied to the problem of bearing capacity of deep foundations by Gibson (14), Skempton, Yassin and Gibson (15) and Ladanyi (16). According to these solutions the bearing capacity \( q_c \) can be computed by an expression analogous to (2) without the third term, however. It should be noted that according to previously mentioned solutions in conditions of infinite mass or of very great depth this third term becomes negligible compared with other two terms. Consequently, all theoretical approaches indicate that at greater depths the bearing capacity of the base should be practically independent of its size and proportional to the overburden pressure \( q \). Based on this conclusion and some limited experimental evidence it has been generally admitted that the point bearing capacity of pile foundations should be equal to the point resistance of a deep cone penetrometer.

Values of bearing capacity factors \( N_q \) found by different theoretical
solutions mentioned are presented in Fig. 4, diagram on the left. The diagram on the right-hand side of the same figure shows the corresponding factors \( q_1 N = N^* \) for circular foundations, as proposed by different investigators. This diagram contains also the empirical curves for \( N^* \) recommended by Brinch Hansen (17,19) and Caquot and Kerisel (18). The appreciable difference in numerical values proposed should be noted.

Bearing Capacity of the Skin

From the early beginnings of theory of deep foundations the unit skin resistance, \( f_0 \), has been evaluated as resistance to sliding of a rigid body in contact with soil. On the basis of classical concepts of friction of solids in contact, it has been considered that the mentioned resistance consists of two parts. The first part, independent of normal pressure acting on contact area, is commonly called adhesion and denoted by \( c_a \). The second part, called friction, is assumed to be proportional to the mentioned normal pressure, so that:

\[
f_0 = c_a + p_s \tan \delta
\]  

(5)

Here \( \delta \) denotes the angle of skin friction and \( p_s \) average normal pressure on the skin. This pressure is generally assumed to be proportional to the corresponding average overburden pressure along the skin, \( q_s \). Or:

\[
p_s = K_s q_s
\]  

(6)

where \( K_s \) is a dimensionless number, which can be called coefficient of skin pressure. With this, the following expression for skin resistance is obtained:

\[
f_0 = c_a + K_s \tan \delta q_s
\]  

(7)
Figure 4. Theoretical Bearing Capacity Factors.
In cohesionless materials $c_a = 0$ and

$$f_o = K_s \tan \delta q_s$$  \hspace{1cm} (8)

This expression suggests that, in homogeneous cohesionless soils, unit skin resistance $f_o$ should be proportional to the average overburden pressure $q_s$.

In frictionless materials $\delta = 0$ and

$$f_o = c_a$$  \hspace{1cm} (9)

suggesting that in homogeneous frictionless soils unit skin resistance should be a constant.

**Previous Experiments**

Among numerous experimental studies related to the considered problem, comparatively few have been of sufficiently general nature to permit drawing definite conclusions concerning the influence of the different parameters involved.

Successful field experiences with predetermination of ultimate bearing capacity of piles by means of deep cone penetration tests (5,17,20,21-25) have built a certain confidence in the general validity of the theoretical approaches described. It has been found in very many instances that the point bearing capacity of driven piles was indeed comparable to that of a deep cone penetrometer. One of the solutions proposed was being used with apparent success for evaluation of shear strength of soils in situ (8). Also, small-scale model tests (10,15) as well as several well documented full-scale tests on piles and piers (reviewed in 32) indicated $N_q$ values in the wide general range predicted by the theories.
Some experiences, however, were not that encouraging. For instance, scale effects of opposite nature than those predicted by the theories have been reported (26-27). Observations of shear patterns in sand around deep foundations (28-31) showed failure surfaces to be localized to the immediate vicinity of the foundation base. To our knowledge, not a single test ever indicated failure surfaces reverting to the shaft. The latest large scale experiments, undertaken by the Institut de Recherches Appliquées du Beton Armé (IRABA) near Paris, lead Kérisel (32) to conclude, without a rational explanation, that both foundation depth and size significantly influence the bearing capacity factor \( N_q \). He suggested that \( N_q \) was not a unique function of \( \varphi \) but a complex function of \( \varphi, D/B \) and \( B \). We shall return to discussion of this conclusion later in this report.

The general aim of the present investigation is to contribute to the understanding of phenomena occurring beneath and around deep foundations in sand, which are the most complex and the least understood at present. This is to be achieved by large-scale model tests in the laboratory under highly controlled conditions that are never encountered in nature, as well as by field tests at selected sites. The studies conducted in different phases of this project are described and discussed in the following chapters.
CHAPTER II

BEARING CAPACITY OF DEEP FOUNDATIONS
IN HOMOGENEOUS MASSES OF SAND

Introduction

At the outset of this phase of investigations three important facts need to be stressed:

1) There are no homogeneous sand deposits in the nature. In addition, placing of deep foundations, particularly piles, often increases the inhomogeneity of the deposit. Yet, the problem of bearing capacity in a homogeneous soil mass is fundamental. All the theories are based on the assumption of homogeneity. Only repeated tests in a homogeneous mass under controlled conditions can yield meaningful results that stand a chance of being generalized.

2) It is not easy to prepare an indeed homogeneous mass of sand for model testing in the laboratory. Our experience of over ten years of continuous work of this kind has taught us that, despite the utmost of care, few models will be homogeneous enough to yield reliable quantitative data, unless the diameter of the model foundation is at least 1.5 in or about of the size of the Dutch cone penetrometer.

3) Because of unclarified scale effects, a range of foundation sizes must be tested. The lesson learned by using this approach is that the conventional dimensional analysis of model tests in sand often fails. Scale effects have to be assessed in a different way, because they are connected with changes in "intrinsic" properties of the material, and even with the changes in character of the investigated phenomena (33).

These three facts were dominant in the setting up of experimental program and in designing the equipment for this phase of the present investigation. It was decided to test model foundations varying in size from about 2 to 7 inches in sand masses varying in relative density from about 20% to about 80%.
Tests with still larger piles were planned to follow in the subsequent phases.

**Testing Facility and Loading Equipment**

For the purpose of this investigation a pile testing facility was constructed adjacent to the Soil Mechanics Laboratory at Georgia Institute of Technology. Characteristic sections of this facility in different phases of operation are shown in Fig. 5. The general view of the facility is given in Fig. 6.

The main feature of the facility is a cylindrical test pit 8 ft. 4 in. in diameter and 22 ft. deep (Fig. 5) in which models of deep foundations can be placed in any kind of soil under controlled conditions. The pit is connected to a 12 in. sump which allows regulation and control of water level in the models. A 200 ton capacity reaction frame permits vertical or horizontal loading of models by means of corresponding hydraulic jacks. An adjustable A-frame at the upper level serves as support for pile-driving equipment as well as for miscellaneous equipment used in placing and excavating sand for models. This frame permits driving of piles vertically or at any batter up to 3:1 by means of a drop-hammer sliding along the leads. The entire facility is served by a 1.5 ton service crane.

For small-scale tests a steel box 50- by 50-in. square and 70 in. deep with a 5-ton loading frame was constructed.

Loading of models was performed generally by means of hydraulic jacks of appropriate capacity (up to 200 tons). The load measurements were made by a corresponding set of proving rings and electronic load cells with a precision of less than one percent. Displacement measurements were made by ordinary micrometer dial gauges (0.0001 in. precision).
Figure 5. Testing Area in Different Phases of Operation.
Figure 6. General View of the Testing Area.
Model Foundations

Two types of foundations were generally built: cylindrical, with circular bases 2.13, 4 and 6.75-in. in diameter, and prismatical, with rectangular bases 2.44- by 12.44-in. The lengths varied according to the foundation depths (see Table 1) between 10 and 113 in.

The foundations were constructed on a principle similar to that of a deep-cone penetrometer. They consist of a steel casing inside which a steel shaft independently connects the loading head with the base. The general view of this arrangement is shown in Fig. 7. In the case of 2 and 4-in. diameter foundations (Fig. 7) separate loading of base and skin of the foundation is possible through exchange of the loading head. In the case of 6-in. diameter foundations (Fig. 7b) the loading head is constructed so as to allow separate registration on the strain-indicator of base and total (base + skin) load. An outside view of this special head is seen in Fig. 8.

The flat bearing surfaces of the bases are covered by sandpaper to assure perfect roughness. To prevent caving-in of sand while the base only is pushed, the latter is protected by a cap (see the figure), tightly fitted with the bottom of the casing. To minimize friction between the cap and casing the contact surface is kept clean and perfectly lubricated.

Properties of Sand

All the tests in this investigation were performed with a medium sand originating from the Chattahoochee River, near Atlanta. The sand was sieved through a window screen (equivalent to sieve opening 1.4 mm.). The grain size distribution curve (shown in Fig. 9) and microscopic examinations indicate a medium, uniform, sand composed mostly of sub-angular quartz particles, but rich in mica.
Table 1. Summary of Loading Tests of Phase I

<table>
<thead>
<tr>
<th>Series</th>
<th>Test Numbers</th>
<th>Foundation Shape</th>
<th>Size</th>
<th>Number of Different Sand Densities</th>
<th>Size of Container in Which Tests Were Performed</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>21-40</td>
<td>Circular</td>
<td>2.13</td>
<td>4</td>
<td>50x50x70&quot;</td>
<td>Skin Diameter 2&quot;</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>0,10&quot;,20&quot;,30&quot;,40&quot;</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B</td>
<td>1-20</td>
<td>Rectangular</td>
<td>2&quot;x12&quot;</td>
<td>4</td>
<td>50x50x70&quot;</td>
<td>Skin Dimensions 2.25&quot;x12.25&quot;</td>
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<tr>
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<td></td>
<td></td>
<td>2.44&quot;x12.44&quot;,10&quot;,20&quot;,30&quot;,40&quot;</td>
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<td></td>
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</tr>
<tr>
<td>C</td>
<td>41-52</td>
<td>Circular</td>
<td>3.94&quot;</td>
<td>4</td>
<td>50x50x70&quot;</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>4&quot;</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>4&quot;</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>80&quot;</td>
<td>3</td>
<td>φ100&quot;,264&quot;</td>
<td></td>
</tr>
<tr>
<td>D</td>
<td>61-70</td>
<td>Circular</td>
<td>6&quot;</td>
<td>4</td>
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<tr>
<td></td>
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<td>6.75&quot;</td>
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<td></td>
<td></td>
<td>60&quot;,110&quot;</td>
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<td></td>
</tr>
<tr>
<td>E</td>
<td>81-84</td>
<td>Circular</td>
<td>8&quot;</td>
<td>4</td>
<td>φ100&quot;,264&quot;</td>
<td></td>
</tr>
</tbody>
</table>
Figure 7. Model Foundations.
Figure 8. View of a Test in Progress.
Figure 9. Grain-Size Distribution Curve.
The material has been air-dried prior to use in tests. The water content, controlled throughout the investigation, varied between 0.2 and 0.3 percent.

Maximum and minimum densities of this sand, as determined by standard procedures are shown in Table 2. The same table contains corresponding minimum and maximum porosities \( n \) and void ratios \( e \).

Shear strength characteristics of the sand were determined by standard triaxial tests (constant cell pressure and positive deviator stress). A total of 54 air-dry samples 2.8 in. in diameter and approximately 6 in. high were prepared at four different densities, and tested using cell pressures varying from 5 to 80 lb/in\(^2\). A strain-controlled loading machine was used for all the tests with the axial strain rate 0.02 in. per minute. The result of the tests are presented in Table 3.

Assuming the Coulomb-Mohr criterion of failure to be valid, an ordinary plot of these results in \( \tau \) versus \( \sigma \) or \( \sigma_1 - \sigma_3 \) versus \( \sigma_1 + \sigma_3 \) presentation can be made. Such a plot indicated that the strength envelopes of the sand in question are slightly curved. To get a better insight into the nature of this curvature the results are also plotted as \( [\sigma_1 - \sigma_3 \text{ versus } 1/\sigma_3] \), (plot proposed by Hansen and Odgaard, \( \text{34} \)) (Fig. 10). In such a presentation a straight-line strength envelope appears as a straight line having the equation

\[
\frac{\sigma_1 - \sigma_3}{\sigma_3} = \frac{2 \sin \varphi}{1 - \sin \varphi} + \frac{2 \cos \varphi \, c}{1 - \sin \varphi \, \sigma_3}
\]  

An examination of Fig. 10 reveals that a reasonably good straight-line approximation of the actually curved envelopes can be obtained by separately considering two ranges of confining pressures \( \sigma_3 \), namely \( \sigma_3 < 10 \text{ lb/in}^2 \) and \( \sigma_3 > 10 \text{ lb/in}^2 \).

For \( \sigma_3 < 10 \text{ lb/in}^2 \) (\( \frac{1}{\sigma_3} > 0.10 \text{ in}^2/\text{lb} \)) the \( \sigma_1 - \sigma_3/\sigma_3 \) values are practically
Table 2. Maximum and Minimum Densities of Chattahoochee River Sand

<table>
<thead>
<tr>
<th>Density</th>
<th>Dry unit weight $\text{lb/ft}^3$</th>
<th>Void ratio $e$</th>
<th>Porosity $n$</th>
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<tr>
<td>Minimum</td>
<td>79.0</td>
<td>1.10</td>
<td>52.4%</td>
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<tr>
<td>Maximum</td>
<td>102.5</td>
<td>0.615</td>
<td>38.1%</td>
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Table 3. Triaxial Test Results

<table>
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<tr>
<th>Test No.</th>
<th>Dry Unit Weight (Y_d)</th>
<th>Void Ratio</th>
<th>Mean Void Ratio</th>
<th>Cell Pressure (\sigma_3)</th>
<th>Stress Difference at Failure (\sigma_1 - \sigma_3)</th>
<th>Axial Strain at Failure %</th>
<th>(\frac{\sigma_1 - \sigma_3}{\sigma_3})</th>
<th>Angle of Internal Friction (\phi_o)</th>
<th>Strength Intercept (c_o)</th>
<th>Angle of Internal Friction (\phi_1)</th>
<th>Strength Intercept (c_1)</th>
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<td>1</td>
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<td>0.957</td>
<td>5</td>
<td>13.3</td>
<td>2.7</td>
<td>2.65</td>
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<tr>
<td>2</td>
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<td>0.957</td>
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<tr>
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<td>0.830</td>
<td>20</td>
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Figure 10. Triaxial Test Results.
independent of $1/\sigma_3$, which means that the shear strength intercept $c_0$ is zero.

The angles of internal friction $\varphi_0$ corresponding to observed shear strengths are given in column 9 of Table 2 and also plotted in Fig. 11. This figure shows that $\varphi_0$ can be expressed as a function of void ratio $e$ approximately by:

$$\tan \varphi_0 = \frac{0.68}{e}$$  \hspace{1cm} (11)

For 80 lb/in$^2 > \sigma_3 > 10$ lb/in$^2$ the $\sigma_1 - \sigma_3/\sigma_3$ values can be approximated by linear functions of $1/\sigma_3$, which indicates that the shear strength intercept $c_1$ is different from zero. Columns 10 and 11 of Table 2 show the $c_1$ and $\varphi_1$ values indicated by Fig. 10, left part. The same $c_1$ and $\varphi_1$ values as functions of initial void ratio $e$ are shown in Fig. 11. The following analytical expression gives a good approximation of $\varphi_1$ as a function of $e$:

$$\tan \varphi_1 = \frac{0.59}{e}$$  \hspace{1cm} (12)

At higher pressures the envelope continues to be curved. The exact nature of this curvature has been established by high pressure triaxial tests (35).

**Placing of Sand and Control of Density**

All the models for this investigation have been constructed in the following way. First, sand of desired uniform density was placed up to the planned elevation of the foundation base. The model foundation was then brought to full contact with the carefully prepared horizontal sand surface and fixed in place so that it could not move during the subsequent operation of filling the sand above the base of the footing. Following former experiences (36) exceptionally uniform loose and medium dense sand models ($D_R < 0.70$) were built by pouring sand from containers with perforated bottoms (Fig. 5b). It was confirmed again that the
Figure 11. Shear Strength Parameters as a Function of Initial Void Ratio.
density of sand models so built is a unique function of the height of free fall of sand as long as other variables (rate of flow) remain the same (Fig. 12).

Denser sand models ($D_n > 0.70$) were built by surface vibration of 4 in. thick sand layers obtained by pouring sand 30 in. from the perforated container. Electric vibrators having a frequency of 3600 cycles per minute, attached to steel plates of appropriate shape were used for surface vibration. Lead surcharge was added as necessary to achieve maximum compaction. All possible care was exercised to obtain uniform density throughout a model.

The homogeneity of sand in models was checked by penetrometer soundings. For this purpose a simple static-cone micropenetrometer was constructed. This device has a point diameter of 1/2 in. and a shaft diameter of the casing of 3/8 in. The assembly can be pushed into the sand by means of a screw jack at about 4 in. per minute. Total resistance (point + skin) was recorded in several positions across the model and plotted versus depth for each test. To convert the measurements of this kind into density an empirical relationship was established between total resistance reduced to unit area of the point end dry unit weight of the material (Fig. 13). This was achieved by sounding sand models in a 24- by 16- by 60-in. box placed on a scale and filled by the same methods as used for building larger models.

Review of Tests Performed

Following the program outlined in the Introduction, six series of tests were performed. The main characteristics of five series of regular loading tests are presented in Table 1.

The sixth series of tests, Series M, numbered 101 to 105, were devoted to the study of failure phenomena under foundations. For this purpose models of soil were built of distinct layers of sand to which cements of two different
Figure 12. Relative Density of Sand as Function of Height of Fall.
Figure 13. Relationship Between Depth and Total Penetration Resistance for Different Sand Densities.
colors were added (10 percent by weight). After performing the loading test in the usual way, water was added to the models to cause setting of the layered mass. A few days later, the hardened block was cut through characteristic sections where shear patterns at failure were visible for observation and analysis.

**Loading Procedure**

The loading procedure for tests at the surface was similar to that followed in ordinary plate load tests. The load was applied in increments of about 1/20 of the estimated failure load at one minute intervals.

The loading procedure for tests beneath the surface was, in principle, the same. However, three separate loading stages existed in each test of series A, B and C. Namely, first the foundation base was pushed until failure was reached. The same was repeated with the foundation shaft. Finally after the shaft reached the base, both were pushed together and the total resistance was recorded.

Since the loading head of the 6.75-in. foundations was of different construction (Fig. 7) the loading stages in D-tests differed somewhat from those described above. By pushing the foundation base to failure, the loading head was brought to contact with the skin and the entire foundation was forced in the soil. Special proving rings (Fig. 7) registered base and skin loads separately during this second stage.

Displacements of foundation base and skip were recorded at one-minute intervals by two micrometer dial gauges placed near the loading head (Fig. 7). Also, in tests of series D, displacements of the sand surface were measured at different locations around the foundation.

**Test Results**

Significant results of the loading tests performed are given in Tables 4 through 9 and discussed in the following chapter.
Table 4. Significant Results of Loading Tests With Circular Plates at the Surface

<table>
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<tr>
<th>Test No.</th>
<th>Plate Diameter B in.</th>
<th>Dry Unit Weight of Sand $\gamma_d$ lb/ft$^3$</th>
<th>Ultimate Pressure $q_\infty$ lb/in$^2$</th>
<th>$\frac{q_\infty}{\gamma_B}$</th>
<th>Ultimate Settlement $w$ lb/in</th>
<th>$w_B$ %</th>
<th>Type of Failure</th>
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*G = General Shear, L = Local Shear, P = Punching Shear Numbers in parenthesis refer to first failure.
Table 5. Significant Results of Loading Tests
With Rectangular Plates at the Surface

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<th>Plate Size</th>
<th>Dry Unit Weight of Sand $\gamma_d$ lb/ft$^2$</th>
<th>Ultimate Pressure $q_o$ lb/in$^2$</th>
<th>$q_o/\gamma B$</th>
<th>Ultimate Settlement w in.</th>
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*G = General Shear, L = Local Shear, P = Punching Shear
Numbers in parenthesis refer to first failure.
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<th>Test No.</th>
<th>Depth D in.</th>
<th>Dry unit Weight of Sand $Y_d$ lb/ft³</th>
<th>Ultimate Base Resistance $q_o$ lb/in²</th>
<th>Ultimate Settlement $w$ in.</th>
<th>W/B</th>
<th>Ultimate Skin Resistance $f_c$ lb/in²</th>
<th>Ultimate Skin Displacement</th>
<th>Total Ultimate Load lb.</th>
<th>Ultimate Displacement for Total Load in.</th>
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<td>34</td>
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Table 7. Significant Test Results - Circular Deep Foundations

Base and skin diameter 4.00 inches

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<thead>
<tr>
<th>Test No. (1)</th>
<th>D in. (2)</th>
<th>$\gamma_d$ lb/ft$^3$ (3)</th>
<th>$q_o$ lb/in$^2$ (4)</th>
<th>w in. (5)</th>
<th>$w_B$ in. (6)</th>
<th>Ultimate skin resistance $f_o$ lb/in$^2$ (7)</th>
<th>Ultimate skin displacement in. (8)</th>
<th>Total ultimate load lb. (9)</th>
<th>Ultimate displacement for total load in. (10)</th>
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</thead>
<tbody>
<tr>
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<td>95.8</td>
<td>202.0</td>
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<td>1.002</td>
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<td>114.9</td>
<td>0.971</td>
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<td>0.484</td>
<td>0.348</td>
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<td>62.0</td>
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<td>0.462</td>
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Table 8. Significant Test Results – Circular Deep Foundation

Base and skin diameter: 6.75 inches

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<tr>
<th>Test No.</th>
<th>D in.</th>
<th>$\gamma_d$ lb/ft$^3$</th>
<th>$q_0$ lb/in$^2$</th>
<th>w in.</th>
<th>w/B %</th>
<th>Ultimate skin resistance $f_o$ lb/in$^2$</th>
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<th>Total ultimate load (lb.)</th>
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<td>27.8</td>
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<td>0.330</td>
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<td>30.6</td>
<td>1.26</td>
<td>18.7</td>
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<td>1.750</td>
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<td>43.0</td>
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Table 9. Significant Test Results - Rectangular Deep Foundations

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<th>Test No.</th>
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<th>Dry Unit Weight of Sand $\gamma_d$ lb/ft$^3$</th>
<th>Ultimate Pressure $q_o$ lb/in$^2$</th>
<th>Ultimate Displacement $w$ in.</th>
<th>Ultimate Skin Resistance $f_o$ lb/in$^2$</th>
<th>Ultimate Skin Displacement $\delta$ in.</th>
<th>Total Ultimate Load lb.</th>
<th>Ultimate Displacement for Total Load in.</th>
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<td>126.0</td>
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<td>52.8</td>
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<td>3,150</td>
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<td>735</td>
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Base Width 2.44 inches
Base Length 12.44 inches

Skin Width 2.25 inches
Skin Length 12.25 inches
Characteristic load-settlement diagrams of surface tests are shown in Figs. 14 through 16. The black points indicate ultimate and first failure loads. The criterion by which these loads were established follows a little later in the text.

Characteristic load-settlement diagrams of base and skin loading tests at greater depth are shown in Figs. 17 through 19 and 20 through 22. Black points indicate ultimate loads.

Characteristic failure patterns at greater depth obtained in tests with colored sand are shown in Figs. 23 through 25. Fig. 23 shows what happens when a circular shaft penetrates through dense sand \( D_R \sim 0.9 \) from a relative depth of \( D/B = 10 \) to a relative depth of \( D/B = 19 \). Fig. 24 shows the analogous phenomenon for a rectangular foundation penetrating from \( D/B = 10 \) to \( D/B = 11 \), and Fig. 25 for a rectangular foundation penetrating from \( D/B = 5 \) to \( D/B = 6 \).

**Types of Failure**

Three characteristic types of failure were observed in surface tests. Foundations on relatively dense sand \( (D_R > 0.70) \) fail suddenly with very pronounced peaks of base resistance (Fig. 26a) when the settlement reaches about 7 percent of the foundation width. The failure is accompanied by the appearance of failure surfaces at the sand surface and by considerable bulging of sheared mass of sand. The phenomenon corresponds exactly to that described earlier by Terzaghi (6) as **general shear failure**.

Foundations on sand of medium density \( (0.35 < D_R < 0.70) \) do not show a sudden failure. As the settlements exceed about 8 percent of the foundation width small sudden shears within the sand mass are apparent from observations of load and settlement gauges. Simultaneously, bulging of the sand surface starts. At settlements of about 15 percent of foundation width, a visible boundary of
Figure 14. Typical Results of Surface Tests.
Figure 15. Typical Results of Surface Tests.
Figure 16. Typical Results of Surface Tests.
Figure 17. Typical Results of Base Loading Tests at Greater Depth.
Figure 18. Typical Results of Base Loading Tests at Greater Depth.

<table>
<thead>
<tr>
<th>TEST NO.</th>
<th>$\gamma_d$ (LB/FT$^3$)</th>
<th>$D_a$ (IN)</th>
<th>DEPTH (IN)</th>
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<td>110</td>
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<td>0.61</td>
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<tr>
<td>66</td>
<td>85.8</td>
<td>0.35</td>
<td>110</td>
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</table>
Figure 19. Typical Results of Base Loading Tests at Greater Depth.
Figure 20. Typical Results of Skin Loading Tests.
Figure 21. Typical Results of Skin Loading Tests.
Figure 22. Typical Results of Skin Loading Tests.
TEST NO. 104 D/B=10 CIRCULAR FOUNDATION B=1 IN. D_R~0.9

Figure 23. Shear Pattern Under a Circular Foundation Placed at Greater Depth in Very Dense Sand.
TEST NO. 101 D/B=10 RECTANGULAR FOUNDATION B=1.5 IN. D_R~0.9

Figure 24. Shear Pattern Under a Rectangular Foundation Placed at Greater Depth in Very Dense Sand.
TEST NO. 102 D/B=5 RECTANGULAR FOUNDATION B=1.5 IN. D_R~0.9

Figure 25. Shear Pattern Under a Rectangular Foundation Placed at Shallow Depth in Very Dense Sand.
Figure 26. Types of Failure.
sheared zone at the sand surface appears. However, the peak of base resistance may never be reached.

The phenomenon is of the same nature as that described by Terzaghi (6) and by De Beer and Vesic (36) as local shear failure (rupture par refoulement incomplete). In the latter investigation, however, the tests were stress-controlled, so that the beginning of large shears in the soil mass was much more pronounced and was always recorded as the first failure of the foundation.

Finally, foundations on relatively loose sand ($D_R < 0.35$) penetrate into the soil without any bulging of the sand surface (Fig. 26c). The base resistance steadily increases as the settlement progresses. The rate of settlement, however, increases and reaches a maximum at a settlement of about 15 to 20 percent of foundation width. Sudden shears can be observed in sequence as soon as the settlement reaches about 6 to 8 percent of foundation width. The failure surface, which is vertical or slightly inclined and follows the perimeter of the base, never reaches the sand surface. The phenomenon is essentially punching shear failure (rupture par enfoncement) as described by De Beer and Vesic (36).

The same three characteristic types of failure are observed at shallow depths. However, as the relative depth $D/B$ increases, the limiting relative densities at which failure types change increase. The approximate limits of types of failure to be expected as relative depths $D/B$ and relative density of sand $D_R$ vary are shown in Fig. 27. The same figure shows that there is a critical relative depth below which only punching shear failure occurs. For circular foundations this critical relative depth seems to be around $D/B = 4$, and for long rectangular foundations around $D/B = 8$.

It is important to note that the limits of types of failure depend on the compressibility of the material. More compressible materials will generally have
Figure 27. Types of Failure at Different Relative Depth D/B of Foundations in Sand.
lower critical relative depths. Following this trend it is not difficult to explain why some materials may exhibit punching shear failure only.

**Criterion of Failure or Ultimate Load**

In accordance with observations just described the following criteria of failure or ultimate load were established.

In the case of general shear failure that criterion is very clear: a peak of base resistance is always reached, corresponding to the appearance of failure surfaces at the sand surface, and to an abrupt change of rate of settlement from positive to negative.

In the case of local shear failure there is not always a peak of base resistance, however the rate of settlement reaches a maximum at the same load at which failure becomes visible at the surface. This load is considered as ultimate. In addition, first failure, clearly distinguishable only in stress-controlled tests, can be noted when settlements reach magnitudes at which the general shear failure occurs in dense sand (36).

Finally, in the case of punching shear failure there is no peak of base resistance nor any appearance of failure surfaces. However, a peak of settlement rate can be noted. The corresponding load is considered as ultimate load.

Analogous criteria are adopted for skin loading tests.

**Foundations at Surface**

To compare observed bearing capacities in surface tests with corresponding theoretical values, Fig. 28 is presented. This figure shows measured values of \( q_o \sqrt{\frac{I}{yB}} \) (Column 5, Tables 4 and 5) as a function of dry unit weight of sand, \( y_d \), or relative density \( D_R \). To have a better basis for comparison, the ultimate pressures of rectangular foundation have been multiplied by a shape factor 0.60 (value recently confirmed by very extensive experiments, (37). Both first and
Figure 28. Observed Bearing Capacities of Foundations at Surface.
ultimate failure pressures are shown for medium and loose sands. The same figure shows theoretical bearing capacity factor $N_y$ after Caquot and Kérisel (38) multiplied by shape factor 0.60. To present the factor $N_y$ as a function of dry unit weight or void ratio, the experimentally established relationship (11) between the angle of internal friction $\phi$ and void ratio $e$ is used.

An examination of Fig. 28 shows that the observed ultimate bearing capacities are generally 1.2 to 4 times higher than corresponding theoretical values. This is, in general agreement with findings of earlier experiments of similar nature (36, 34, 39). It should be noted that a fully satisfactory explanation of this phenomenon has not yet been found.

The ranges of relative densities in which different types of failure occur, shown on the top of Fig. 28, also agree well with those found in an earlier investigation (36). It seems that the conventional classification of sands by relative density into loose ($D_R < 0.33$), medium ($0.33 < D_R < 0.67$) and dense ($D_R > 0.67$) has a certain meaning concerning the type of failure of shallow foundations on such materials.

The settlements at which ultimate loads were recorded, expressed as percentage of the foundation width, are given in Figure 29. This figure shows that general shear failure usually occurs at settlements not exceeding 10 percent of foundation width, while the other failure types take place at settlements of about 15 to 20 percent of the foundation width. This is in general agreement with former observations. However, slightly higher relative settlements at failure of rectangular foundations do not conform with some earlier findings (37). It should be noted that a similar trend was observed in tests with deep foundations (Fig. 32).
Figure 29. Settlement at Failure for Surface Foundations.
Base Resistance of Deep Foundations

The general trend of increase in bearing capacity of the base with increase of foundation depth can best be seen from Figs. 30 and 31. Fig. 30 shows the ultimate base resistance of 2.13-in. circular foundations as a function of foundation depth D. Fig. 31 is an analogous plot for 2.44- by 12.44-in. rectangular foundations. A practically linear increase of bearing capacity with depth can be observed only at shallow depths, not exceeding approximately D/B = 4 for circular and D/B = 6 for rectangular foundations. As the foundation depth increases further, the rate of increase of bearing capacity with depth decreases. At a relative depth of approximately D/B = 15 the bearing capacity reaches asymptotically final values which appear to be functions of sand density only.

Base displacements or settlements needed to reach the ultimate loads are shown in Fig. 32. It appears that there is a tendency of ultimate settlements to increase with both foundation size and depth. This tendency, however, is not pronounced. It may be stated, that in the range of foundations sizes and depth used in this investigation, ultimate loads are reached at settlements of about 20-30 percent of foundation depth. This figure is in general agreement with isolated former observations.

A comparison of final bearing capacities of circular and long rectangular foundations indicates that the former are approximately 1.50 times higher. Fig. 33 shows the average final bearing capacity of the base as a function of dry unit weight of sand, with bearing capacities of rectangular foundations multiplied by a shape factor of 1.50.

Plotted in the same figure are final bearing capacities observed in tests with 4- and 6.75-in. circular foundations. It appears that the final bearing capacities are independent of foundation size, at least for dense and medium dense sands.
Figure 30. Measured Bearing Capacities of the Base; Circular Foundation, B = 2.13 Inches.
Figure 31. Measured Bearing Capacities of the Base; Rectangular Foundation, 2.44 x 12.44 Inches.
Figure 32. Ultimate Settlement of Deep Foundations.
Figure 33. Ultimate Base Resistance at Greater Depth.
A similar conclusion can be reached by studying Fig. 6 of Kérisel's paper (32) although the numerical values obtained by the two investigations are not directly comparable due to differences in experimental approach used as well as in sand properties.

**Skin Resistance of Deep Foundations**

The variation of ultimate skin resistance $f_o$ with depth for 2-in. circular foundations is shown in Fig. 34. For models in dense, vibrated sand, a long initial linear increase of $f_o$ with depth (up to $D/B = 15$) is followed by a sharp turn into a final skin resistance which remains constant as the depth increases further. For models in loose and medium dense sand the shape of the initial part of the $f_o$ curve is not quite clear. It appears that there is also an initial linear increase limited to a depth of about four diameters. Beyond this depth the skin resistance turns sharply into a practically constant final value, varying with sand density only.

Fig. 35 shows the analogous diagrams for rectangular foundations 2.25- by 12.25-in. The trend is similar, however, the initial part along which $f_o$ increases linearly with depth seems to be longer. The slope of the initial linear part is approximately three to four times less than the corresponding slope in the case of circular foundations. The final skin resistance, however, appears to be approximately 1.5 times lower for the rectangular shape.

Fig. 36 shows the final average skin resistance as a function of dry unit weight of sand. Skin resistances of rectangular foundations multiplied by a shape factor of 1.5 are also plotted. The curve traced takes into account the fact that, due to method of placing of medium dense sand the density in immediate vicinity of the skin was lower than the average density of the entire model, particularly in the case of 2-in. foundations.
Figure 34. Ultimate Skin Resistance - Circular Foundation 2 Inches.
Figure 35. Ultimate Skin Resistance - Rectangular Foundation
2.25 x 12.25 Inches.
Figure 36. Final Skin Resistance at Greater Depth.
It is interesting to note that the general shape of the $f_o$ curves found in the present investigation differs from that found in IRABA tests (32). However, it agrees well with numerous former observations on full-scale piers and caissons, which usually show a linear increase of $f_o$ at shallow depths, but a practically constant $f_o$ at greater depths.

Skin displacements needed to reach ultimate skin resistance are shown in Fig. 37. It appears that these displacements are not dependent on foundation width and depth nor on sand density. For circular foundations they vary in the range of 0.30 to 0.40 in; for rectangular foundations they are about half of that magnitude. It is shown later, in the next chapter that a similar observation has been made in tests with driven piles.

**Bearing Capacity and Shape Factors at Shallow Depth**

As mentioned earlier, and shown in Figs. 30 and 31, at shallow depths not exceeding $D/B = 4$ the increase of bearing capacity with depth appears to be linear as proposed by expression (3). Therefore the initial slopes of curves in Figs. 30 and 31 indicate the experimental values of bearing capacity factor $N_q$ at shallow depths. The $N_q$ factors evaluated from these slopes are shown in Fig. 38. In order to take into account the effect of shape, the depth term of circular foundations was reduced by an assumed shape factor of $C = 2.00$. Good agreement resulting from such an assumption indicates that shape factor $C$ for a circular foundation in sand cannot be much different from 2. It should be remembered that Terzaghi (6) proposed for that factor a value of 1.30 and Brinch Hansen (19) values increasing with $\varphi$ from about 1.30 for $\varphi = 35^\circ$ to 2.20 for $\varphi = 45^\circ$.

As a basis for comparison the same Fig. 38 presents a curve of theoretical $N_q$ values after Prandtl-Reissner. To trace this curve the relationship (11)
Figure 37. Ultimate Skin Displacement of Deep Foundations.
Figure 38. Measured Bearing Capacity Factors $N_q$ at Shallow Depth.
between $\phi$ and $e$ was assumed valid. This is justified by the fact that the average normal stress along a rupture surface under foundations probably does not exceed 10 percent of the foundation pressure.

Higher $N_q$ values for two very dense models can easily be explained. At high relative sensitivities general shear failure still occurs at shallow depths (cf. Figs. 25 and 27). As failure surfaces extend above foundation level, bearing capacity must be higher than indicated by Prandtl-Reissner theory which neglects shear resistance of the overburden (Fig. 2). If this resistance is taken into account a depth factor of approximately 2 should be introduced for $D/B = 4$ and $\phi = 42^\circ$ (10, 19). Therefore, excellent agreement of existing theory and experiments can be stated if the sand is dense.

However, lower $N_q$ values observed for medium and loose sand models cannot be explained by the existing theories, which consider general shear failure only.

Evaluation of Bearing Capacity Factor $N_q$ in the Case of Local or Punching Shear Failure

To evaluate the bearing capacity factor $N_q$ in the case of local or punching shear failure of a long rectangular foundation the following shear pattern, based on observations on colored sand models, will be considered (Fig. 39). It consists of an elastic zone $ABC$ with two adjoining plastic zones $BCD$. The extent of development of these zones is determined by the angle $\theta$ at the apex.

It will be assumed for analysis that the overburden pressure $q$ is great enough to allow neglecting the own weight of the soil $\gamma$. Under such circumstances, solutions for weightless soil (3) can be applied to analyze stress conditions along $CD$. It is easy to show that the stress $p_D$ acting on rupture line at $D$ and the analogous stress $p_C$ at $C$ are connected by expression
\[ p_0 = p_c e^{-2 \theta \tan \phi} \]
\[ q_o = q \tan (45 + \phi/2) \]

Figure 39. Analysis of Punching or Local Shear Failure.
However:

\[ \rho_D = \rho_C e^{-2\theta\tan\varphi} \]  \hspace{1cm} (13)

Also, assuming that the minor principal stress along BD is equal to overburden pressure \( q \):

\[ \rho_C = \rho_A = q_0 \tan(45 - \varphi/2) \]  \hspace{1cm} (14)

Eliminating \( \rho_C \) and \( \rho_D \) from expressions (10) through (12) we find:

\[ q_o = q \tan^2(45 + \varphi/2)e^{2\theta\tan\varphi} \]  \hspace{1cm} (16)

By introducing \( \theta = 1.9\varphi \), on the basis of observations, the following expression for \( N_q \) is obtained:

\[ N_q = e^{3.8\theta\tan\varphi}\tan^2(45 + \varphi/2) \]  \hspace{1cm} (17)

Numerical values for different angles \( \varphi \) are presented in Table 10. They are lower than classical Prandtl-Reissner values. Reasonable agreement between \( N_q \) values computed by expression (17) and observed experimentally can be seen in Fig. 38.

**Bearing Capacity at Greater Depth**

Earlier discussion of base and skin resistance \( q_o \) and \( f_o \) has shown (Figs. 30 through 36) that, beyond some limiting relative depth \( D/B \), the increase of \( q_o \) and \( f_o \) with depth is not linear. As \( D/B \) increases over 15, \( q_o \) and \( f_o \) do not
Table 10. Bearing Capacity Factor $N_q$ in the Case of Local or Punching Shear Failure

<table>
<thead>
<tr>
<th>Angle of Internal Friction $\varphi$ (Degrees)</th>
<th>Bearing Capacity Factor $N_q$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>1.0</td>
</tr>
<tr>
<td>5</td>
<td>1.2</td>
</tr>
<tr>
<td>10</td>
<td>1.6</td>
</tr>
<tr>
<td>15</td>
<td>2.2</td>
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<tr>
<td>20</td>
<td>3.3</td>
</tr>
<tr>
<td>25</td>
<td>5.3</td>
</tr>
<tr>
<td>30</td>
<td>9.5</td>
</tr>
<tr>
<td>35</td>
<td>18.7</td>
</tr>
<tr>
<td>40</td>
<td>42.5</td>
</tr>
<tr>
<td>45</td>
<td>115</td>
</tr>
<tr>
<td>50</td>
<td>422</td>
</tr>
</tbody>
</table>
increase any more. Final values of \( q_0 \) and \( f_0 \) appear to be functions of density of sand only. (Figs. 33 and 36).

These observations seem to contradict the fundamental structure of bearing capacity equations (3) and (6) derived by using theories of plastic or elastic-plastic equilibrium. As mentioned in the Introduction, similar observations made recently by Kérisel (32) have lead him to conclude that the bearing capacity factor \( N_q \) is a complex function of \( \varphi \), \( D/B \) and \( B \). As long as no explanation of these findings is offered this appears to be the only possible conclusion.

However, our attempts to explain the obtained results by an appropriate rational analysis leave serious doubts as to the correctness of the conclusion. No matter how limited an extent of plastic zone adjacent to the foundation base is assumed, there still must be a certain increase of \( q_0 \) as overburden pressure increases. Therefore, \( N_q \) cannot be zero for any increment of loading as long as we deal with the same material. When loosening of sand structure or significant crushing of sand grains occurs, there is still a lower limit of angle of internal friction of the newly formed material. Consequently, sooner or later, there must be an increase of \( q_0 \) if overburden pressure continues to increase.

On the basis of these and other considerations the conclusion was reached that constant values of \( q_0 \) at greater depth do not result from decrease in \( N_q \) alone as suggested.

The explanation of the phenomena observed must, therefore, be sought through the assumption that \( q \) is not proportional to initial overburden pressure, as conventionally assumed. In connection with this, the true meaning of \( q \) in different theories should be remembered. In Prandtl-Reissner theory \( q \) is defined as normal stress at failure on horizontal plane of the foundation base (Fig. 2). In De Beer-Jáky-Mayerhof theories it is defined as normal stress at failure on the lower portion of the foundation shaft (Fig. 3). There is no good
reason to take these stresses a priori equal or proportional to the initial overburden pressure, if foundation is deeply embedded in sand.

To demonstrate the meaning of our results, let us for the moment assume that both $q_o$ and $f_o$ increase linearly with $q$ as indicated by equations (3) and (6), but that $q$ is strictly $q_f$ or effective normal stress at failure acting on an elemental horizontal plane next to the foundation (Fig. 40). Consider a plane problem, or a rectangular foundation placed at greater depth. Equations (3) and (8) can be rewritten in the following form:

$$q_o = q_f N_q$$  \hspace{1cm} (18)  

$$f_o = q_f K_s \tan \delta$$  \hspace{1cm} (19)  

Eliminating $q_f$ from the preceding equation we obtain

$$N_q = \frac{q_o}{f_o} K_s \tan \delta$$  \hspace{1cm} (20)  

In analogous way the following expressions can be written for a circular foundation:

$$q_o = q_f N_q \zeta_q$$  \hspace{1cm} (21)  

$$f_o = q_f K_s \tan \delta \zeta_s$$  \hspace{1cm} (22)  

$$N_q = \frac{q_o}{f_o} K_s \tan \delta \frac{\zeta_s}{\zeta_q}$$  \hspace{1cm} (23)  

Thus, it is possible to evaluate $N_q$ from results of tests at greater depth under mentioned assumptions without really knowing $q_f$, $K_s \tan \delta$ or $K_s \tan \delta \zeta_s$.  

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Figure 40. Stress Conditions in Vicinity of the Base of a Deep Foundation.
may be evaluated from the initial straight line part of the $f'_o$ line.

The results of such an evaluation are shown in Fig. 41, where individual results from tests at greater depth with 2-4- and 6.75-in. circular foundations as well as with 2- by 12-in. rectangular foundations are plotted. To plot comparable magnitudes both shape factors $\zeta_s$ and $\zeta_q$ for circular foundations were taken equal to 3 although $\zeta_s$ appeared to be slightly higher. The same figure shows, for comparison, bearing capacity factor $N_q$ after Prandtl-Reissner and after expression (17). It is evident that experimental values obtained by using expressions (20) and (23) are primarily functions of sand density and that they are independent of absolute magnitude of $q_F$. Their numerical values in dense sand are reasonably well estimated by using the Prandtl-Reissner classical expression for $N_q$ with a shape factor of approximately 3 for circular foundations. In medium and loose sands experimental values are lower, and comparable to those estimated by expression (17).

Some details of the analysis presented undoubtedly need further clarification, particularly the choice of shape factors which, due to limited number of tests performed, could not have been determined very accurately. One fact appears certain, however: that both base resistance $q_o$ and skin resistance $f'_o$ are linear functions of vertical stress at failure, $q_F$. This stress is not necessarily equal nor proportional to the overburden pressure $q$. The nonlinear increase of base or skin resistance with depth can be explained by a similar increase of $q_F$ with depth. If the base and skin resistance reach constant values at greater depth, it is because $q_F$ also becomes constant at greater depth.

**Analysis of Vertical Stress Around the Foundation**

According to the preceding discussion, curves presented in Figs. 30 and 31 indicate the nature of variation with depth of vertical stress at the base level,
Figure 41. Bearing Capacity Factors at Greater Depth.
q_f. At shallow depths (D/B < 4) q_f is equal to the overburden stress q; at greater depths (D/B > 15) q_f reaches a constant value independent of overburden stress.

In a similar way, curves in Figs. 34 and 35 indicate the variation of average vertical stress along foundation shaft, q_z, with foundation depth. From the shape of these curves it may be concluded that the distribution of vertical stresses q_z at any point z along the shaft must follow a curve similar to that shown in Fig. 42c. Namely, there should be a linear increase of q_z along a certain depth z_o, followed by a peak and gradual decrease to the final magnitude q_f. It is to be understood, however, that the foundation depth, sand density and some other factors may have influence on the shape of curves in question. Therefore, the peak mentioned may be more or less pronounced, or even nonexisting, leading to q_z curves of shapes between those shown in Fig. 42c and 42b.

Looking for an explanation of this general trend of variation of q_z with depth we arrived to the idea that the nonlinear increase of bearing capacity with depth could be attributed to "arching" in sand above the foundation base. There exists, indeed, a striking similarity between curves shown in Fig. 42 and curves of vertical pressure in a mass of sand above a yielding horizontal support (Fig. 18d).

On the basis of all the observations made, the following explanation of stress conditions around a deep foundation is suggested: When the foundation is loaded (Fig. 42a) the mass of sand beneath is compressed downward. At the same time sand around the foundation tends to follow the general downward movement of the mass. As a consequence of this, the originally horizontal stresses on a vertical plane n-n at a certain distance from the foundation become inclined.
Figure 42. Stress Distribution Around a Deep Foundation in Sand.
The inclination of these stresses is a function of the amount of displacement \( w \) of the foundation and of the distance \( z' \) from the base level. If the foundation depth \( D \) is great enough, and if the base displacement \( w \) remains limited, there may be a distance \( z'_0 \) beyond which the effect of downward movement is not felt any more. Above that distance stresses on vertical planes may remain horizontal and the vertical stress \( q_z \) may be equal to overburden stress \( q \).

The following arguments can be added in support of the mentioned explanation:

1. Measurements of displacements of sand surface during loading tests on foundations placed at greater depth showed downward movement of soil adjacent to the foundation, even in the case of foundations in very dense sand (See Fig. 43).
2. Measurements of sand density around the foundation after load testing to failure indicated, in the case of models made of dense sand, considerable loosening in a zone immediately above base level, but a slight densification below that level.

The existence of tractions above the level of foundation base has also been observed in IRABA tests (40). The study of recent X-ray graphs of displacements around a deep foundation model in sand, shown in Fig. 44a (41), also reveals tensile strains immediately above the foundation base amounting to 50% of maximum compressive strains below the base (Fig. 44b).

3. Final resistances of the base and skin of rectangular foundations are found to be 1.50 times lower than corresponding resistances of circular foundations of the same diameter. However, measurements of real shape factor for base and skin resistances at greater depth indicate values of approximately 3. This leads to the conclusion that the final vertical stresses \( q_f \) around rectangular foundations are two times higher than corresponding stresses around
Figure 43. Surface Deflections.
Figure 44. Strain Pattern around Point of a Deep Foundation.
circular foundations. The same ratio of final vertical stresses is found in the case of rectangular versus circular bins of the same diameter, where a similar phenomenon of arching occurs.

(4) Under similar conditions there is less arching under smaller foundations, resulting in higher bearing capacity for the same relative depth D/B. This can be explained by the fact that base displacements at failure increase proportionally to the foundation width. Displacement of a larger foundation will mobilize friction along relatively longer distance z.'

(5) Finally, measurements of skin resistance along model foundations in sand have indicated distributions similar to curves shown in Fig. 42 (42,43).

Numerous other observations on actual deep foundations as well as on models can be cited in favor of the explanation offered. However, it should be realized that the problem in question is very complex, and that variables such as roughness of foundation skin or method of construction (to mention just a few) may also be of significant influence.

Conclusions

(1) Shear patterns observed underneath buried model foundations in sand indicate that, depending on relative density of sand, all three types of failure previously described in the literature may occur at shallow depth: General shear failure, local shear failure and punching shear failure. However, at greater depths only punching shear failure occurs, irrespective of relative density of sand. Limits of types of failure to be expected vary with relative density or compressibility as well as with relative depth D/B of the foundation (Fig. 27).

(2) At shallow depths, not exceeding four foundation widths (D/B < 4) the increase of point bearing capacity with depth is linear. In dense sand (D_R > 0.70) the bearing capacity factor N_q can be estimated with sufficient accuracy using
an analysis based on conventional theory of general shear failure in a rigid-
plastic solid. In loose or medium dense sand, failure surfaces being localized,
better agreement with test results may be stated if an expression for $N_q$ derived
under the assumption of local or punching shear failure is used.

(3) At greater depths, generally exceeding 15 foundation widths, both base
resistance $q_o$ and skin resistance $f_o$ reach constant final values. These values
are independent of overburden pressure $q$ and appear to be functions of relative
density of sand only. This is explained by the "arching" of sand above the
foundation base. It is demonstrated that both $q_o$ and $f_o$ are proportional to
the effective vertical stress at failure, $q_f$ at the level of foundation base.

The bearing capacity factor $N_q$ at greater depth, defined as the ratio of
base resistance $q_o$ to vertical stress $q_f$, is practically independent of founda-
tion size and is a function of relative density or angle of internal friction
of sand. Observed bearing capacity factors $N_q$ for long rectangular foundations
at greater depth do not differ from those at shallow depth. However the shape
factor for circular foundations appears to be somewhat higher at greater depths.

(5) Skin resistance along the foundation shaft is not necessarily increas-
ing linearly with depth. Instead, it is proportional to the vertical stress $q_z$
at the corresponding elevation. That stress increases linearly only at shallow
depths. If the foundation is deeply embedded in sand, the distribution of
vertical stress, as well as of skin resistance, is likely to be similar to that
shown in Fig. 42.

(6) The fundamental fallacy of conventional analyses of bearing capacity
of deep foundations in sand consists in the tacit assumption that $q$ is always
equal to the initial overburden stress at the level of foundation base. This
may be correct if a deep foundation penetrates only slightly into a sand stratum
overlain by compressible soil. However, it may be entirely wrong if a deep foundation is completely embedded in sand.
CHAPTER III

Piles Driven into Homogeneous Masses of Sand

Introduction

The findings of the preceding chapter, reached from model studies of behavior of deep foundations in homogeneous masses of sand, have direct implications on design of foundations such as drilled piers or caissons, which are placed in such a manner that the initial density or structure of the bearing soil are not substantially changed. As mentioned in the introduction, the other important type of deep foundations are piles, which are forced into ground by driving or some other similar operation that usually produces significant changes in structure and density of the bearing soil.

In view of the fact that hundreds of thousands of loading tests have been performed on this type of deep foundations in the past, it may be astounding to find how little has been actually known only a few years ago about the basic mechanics of piles in sand. One major reason for this has certainly been the lack of large-scale investigations in controlled conditions. The second and third phases of this project were thus planned as an attempt to fill this gap. Tests of the second phase, all performed with 4-inch-diameter piles, are described in the present chapter.

Test Description

The tests of the second phase have been performed partly in the laboratory pile testing facility, partly on a field site in Crawford County, 5 miles south of Roberta, Georgia.

The pile testing facility, constructed for the purpose of this research
project, has been described in detail in the previous chapter. Its main feature is a large cylindrical test pit 8 ft. 4 in. in diameter and 22 ft. deep in which soil models of any composition can be built and tested under controlled conditions. The pit is served by a 200 ton reaction frame and an adjustable A-frame which permits pile driving by means of a drop hammer sliding along the leads. (Figs. 7 and 45)

All the laboratory model tests were performed with air-dry Chattahoochee River sand, described in more detail in the previous chapter. Its main characteristics have been summarized in Table 1.

The tests at the field site were made within the confines of the sand quarry of Atlanta Sand and Supply Co. in a very thick deposit of naturally moist medium to fine sand, very similar in composition and particle size to the Chattahoochee River sand. Its main characteristics are shown in Table 11. Its particle size distribution curve can be seen in Fig. 46, where the analogous curve for the Chattahoochee River sand is also shown for comparison. The results of standard penetration tests performed at the site are shown in Fig. 51.

The arrangement for field tests is shown in Fig. 47. Two steel augers set 10 ft. apart and connected by a horizontal reaction beam served as reaction frame for the load tests. Piles were driven again by a drop hammer, with the help of a drilling rig of the State Highway Department of Georgia. Both field and laboratory tests were performed with 4-inch diameter steel pipe piles constructed on a principle similar to that of a deep cone penetrometer (Fig. 7c). Separate loading of point and skin of these piles is possible through exchange of the loading head (Fig. 7c). The flat bearing surface of the pile points is covered with sandpaper to assure perfect roughness.
Table 11. Characteristics of Roberta Sand

<table>
<thead>
<tr>
<th>Characteristic</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean particle size</td>
<td>0.29 mm</td>
</tr>
<tr>
<td>Coefficient of uniformity</td>
<td>2.2</td>
</tr>
<tr>
<td>Specific gravity</td>
<td>2.67</td>
</tr>
<tr>
<td>Maximum void ratio</td>
<td>1.18</td>
</tr>
<tr>
<td>Minimum void ratio</td>
<td>0.60</td>
</tr>
<tr>
<td>Water content</td>
<td>4.2 to 6.8%</td>
</tr>
</tbody>
</table>
Figure 45. Test Arrangement in the Laboratory During Driving and Loading Operations.
Figure 46. Particle Size Distribution Curve, Roberta Sand.
Figure 47. General Arrangement of the Field Test.
All the sand models in the laboratory have been constructed by the two techniques described in detail in the preceding chapter. Loose and medium dense models ($D_R < 0.70$) were built by pouring sand from a fixed height over the horizontal surface of the model. Dense sand models were built by surface vibration of 4-inch thick sand layers which in turn were prepared by pouring sand from a height of 30 in. The homogeneity of sand models was checked by penetrometer soundings, for which a $\frac{1}{2}$ inch micropenetrometer was used.

Piles were driven by a drop hammer, the weight of which was 204 lb. in the laboratory tests and 350 lb. in the field tests. The height of fall of the hammers was varied according to sand density and depth. Continuous driving record was maintained, including the measurements of displacements of the sand surface. A general view of the experimental setup for driving and testing can be seen in Fig. 45.

**Testing Program and Procedure**

Four series of tests were conducted, each series consisting of testing of a pile in a specific sand mass. Series G-1, G-2 and G-3 were performed in the laboratory on dry sand, in loose, medium dense and dense state, respectively. Series G-4 was performed in the field on natural moist, loose sand. In each series the pile was successively driven and tested at six different depths, ranging from approximately 20 in. (5 pile diameters) to 120 in. (30 pile diameters). A review of tests performed is presented in Tables 12 through 15.

Three separate loading stages existed in each test, corresponding to the stages of operation of a conventional deep-cone penetrometer, namely point loading, skin loading and loading of the entire assembly. In each stage loads were applied in increments of about 1/10 of the estimated failure load in two-minute intervals.
Displacement of the pile point and skin were recorded in the laboratory by ordinary micrometer dial gauges (0.0001 in precision) and in the field by using a precision level and level rod (0.01 in precision). Also, displacements of the sand surface during driving and loading were measured by twelve micrometer dial gauges arranged in four perpendicular directions, as visible in Fig. 45.

Test Results

Driving records of the four test series are shown in Figs. 48 through 51. These figures show the sets for all hammer blows during driving as well as the number of blows per foot of penetration as functions of penetration depth. The variable height of fall of the hammers and the hammer weights are also shown.

Typical load-settlement diagrams recorded are shown in Figs. 52 through 54. They are very similar in shape to those observed earlier for deeply buried foundations in homogeneous masses of sand, and consist of initial linear sections which gradually turn to final linear sections, with steadily increasing load. Ultimate loads, marked in these diagrams by black points, have been defined, as before, as loads at which the displacement rate first reaches its maximum.

Significant results of the loading tests performed are assembled in Tables 12 through 15.

Ultimate Loads and Displacements

The diagrams of ultimate point and skin resistances versus depth are presented in Figures 55 and 56 respectively. These diagrams show a marked similarity with analogous diagrams for buried foundations, as well as with
Figure 48. Pile Driving Record, Test Series G-1 (Dry, Loose Sand).
Figure 49. Pile Driving Record, Test Series G-2 (Dry, Medium Dense Sand).
Figure 50. Pile Driving Record, Test Series G-3 (Dry, Dense Sand).
Figure 51. Pile Driving Record, Test Series G-4 (Moist, Loose Sand).
Figure 52. Typical Load-Displacement Diagrams, Point Loading Tests.
Figure 53. Typical Load-Displacement Diagrams, Skin Loading Tests.
Figure 54. Typical Load-Displacement Diagrams, Pile Loading Tests.
Table 12. Significant Results of Loading Tests - Series G-1

Laboratory Tests on Dry, Loose Sand

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Embedded Depth D in.</th>
<th>Dry Unit Weight of Sand ( \gamma_d ) lb./ft.</th>
<th>Ultimate Point Resistance ( q_o ) lb./in.²</th>
<th>Ultimate Point Displacement ( w ) in.</th>
<th>Ultimate Skin Resistance ( f_o' ) lb./in.²</th>
<th>Ultimate Skin Displacement ( D_f ) in.</th>
<th>Total Ultimate Load ( Q ) kip</th>
<th>( \frac{q_o}{f_o'} )</th>
<th>( N^* = \frac{q_o}{f_o' K_s \tan \delta} )</th>
<th>Initial Modulus of Deformation ( E' ) lb./in.²</th>
<th>( \frac{E'}{q_o} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>G-11</td>
<td>20.1</td>
<td>86.2 (89.5)</td>
<td>57.0</td>
<td>0.276 (0.202)</td>
<td>0.87</td>
<td>0.289</td>
<td>0.94</td>
<td>65.5</td>
<td>103</td>
<td>1,812 (31.8)</td>
<td></td>
</tr>
<tr>
<td>G-12</td>
<td>40.4</td>
<td>85.0 (89.6)</td>
<td>67.2</td>
<td>0.447 (0.396)</td>
<td>0.98</td>
<td>0.327</td>
<td>1.23</td>
<td>68.6</td>
<td>108</td>
<td>1,965 (29.2)</td>
<td></td>
</tr>
<tr>
<td>G-13</td>
<td>60.5</td>
<td>84.0 (88.2)</td>
<td>64.8</td>
<td>0.283 (0.392)</td>
<td>0.86</td>
<td>0.493</td>
<td>1.24</td>
<td>75.5</td>
<td>118</td>
<td>1,950 (30.1)</td>
<td></td>
</tr>
<tr>
<td>G-14</td>
<td>80.4</td>
<td>84.7 (88.7)</td>
<td>71.2</td>
<td>0.297 (0.158)</td>
<td>1.01</td>
<td>0.257</td>
<td>1.76</td>
<td>71.0</td>
<td>111</td>
<td>1,660 (23.3)</td>
<td></td>
</tr>
<tr>
<td>G-15</td>
<td>101.9</td>
<td>84.6 (88.7)</td>
<td>78.5</td>
<td>0.355 (0.114)</td>
<td>1.18</td>
<td>0.337</td>
<td>2.11</td>
<td>66.5</td>
<td>104</td>
<td>3,140 (40.1)</td>
<td></td>
</tr>
<tr>
<td>G-16</td>
<td>120.1</td>
<td>85.5 (89.0)</td>
<td>79.8</td>
<td>0.230 (0.173)</td>
<td>1.20</td>
<td>0.260</td>
<td>2.45</td>
<td>68.4</td>
<td>104</td>
<td>2,280 (28.5)</td>
<td></td>
</tr>
</tbody>
</table>

Remarks:
(a) \( K_s \tan \delta = 1.57 \) (from the initial portion of the \( f_o'/D \) curve)
(b) Unit weights in parentheses show densities after pile driving
(c) Point displacements in parentheses refer to the third stage of the tests - pushing the entire assembly (point + skin)
Table 13. Significant Results of Loading Tests - Series G-2

Laboratory Tests on Dry, Medium Dense Sand

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Embedded Depth D in.</th>
<th>Dry Unit Weight of Sand $Y_d$ lb./ft.³</th>
<th>Ultimate Point Resistance $q_o$ lb./in.²</th>
<th>Ultimate Point Displacement $w$ in.</th>
<th>Ultimate Skin Resistance $f_o$ lb./in.²</th>
<th>Ultimate Skin Displacement $\delta$ in.</th>
<th>Total Ultimate Load $Q$ kip</th>
<th>$q_o/f_o$</th>
<th>$N^* = q_oK_s\tan\delta/f_o$</th>
<th>$E' = \frac{q_o}{q_o} + \frac{E_{deformation}}{E_{modulus}}$</th>
<th>Initial Modulus of Deformation $E'$ lb./in.²</th>
</tr>
</thead>
<tbody>
<tr>
<td>G-21</td>
<td>21.2</td>
<td>88.9 (93.3)</td>
<td>119.0</td>
<td>0.596 (0.362)</td>
<td>1.10</td>
<td>0.201</td>
<td>1.73</td>
<td>108</td>
<td>198</td>
<td>4,180</td>
<td>35.2</td>
</tr>
<tr>
<td>G-22</td>
<td>40.7</td>
<td>91.7 (93.4)</td>
<td>155.1</td>
<td>0.479 (0.409)</td>
<td>1.46</td>
<td>0.395</td>
<td>2.63</td>
<td>106</td>
<td>194</td>
<td>5,040</td>
<td>32.4</td>
</tr>
<tr>
<td>G-23</td>
<td>63.9</td>
<td>92.6 (93.0)</td>
<td>193.5</td>
<td>0.600 (0.285)</td>
<td>1.64</td>
<td>0.387</td>
<td>3.48</td>
<td>118</td>
<td>216</td>
<td>6,280</td>
<td>32.4</td>
</tr>
<tr>
<td>G-24</td>
<td>79.7</td>
<td>93.5 (92.2)</td>
<td>195.2</td>
<td>0.629 (0.288)</td>
<td>1.96</td>
<td>0.394</td>
<td>4.08</td>
<td>100</td>
<td>182</td>
<td>6,080</td>
<td>31.2</td>
</tr>
<tr>
<td>G-25</td>
<td>103.3</td>
<td>92.3 (92.9)</td>
<td>136.6</td>
<td>0.419 (0.352)</td>
<td>1.79</td>
<td>0.386</td>
<td>4.03</td>
<td>76.4</td>
<td>140</td>
<td>4,600</td>
<td>33.8</td>
</tr>
<tr>
<td>G-26</td>
<td>124.3</td>
<td>89.7 (93.7)</td>
<td>161.4</td>
<td>0.353 (0.283)</td>
<td>1.55</td>
<td>0.388</td>
<td>4.60</td>
<td>104</td>
<td>191</td>
<td>5,700</td>
<td>35.3</td>
</tr>
</tbody>
</table>

Remarks:  
(a) $K_s\tan\delta = 1.83$ (from the initial portion of the $f_o/D$ curve)  
(b) Unit weights in parentheses show densities after pile driving  
(c) Point displacements in parentheses refer to the third stage of the tests - pushing the entire assembly (point + skin)
<table>
<thead>
<tr>
<th>Test No.</th>
<th>Embedded Depth D in.</th>
<th>Dry Unit Weight of Sand ( V_d ) lb./ft. (^3)</th>
<th>Ultimate Point Resistance ( q_o ) lb./in. (^2)</th>
<th>Ultimate Point Displacement ( w ) in.</th>
<th>Ultimate Skin Resistance ( f_o ) lb./in. (^2)</th>
<th>Ultimate Skin Displacement ( \delta o ) in.</th>
<th>Total Ultimate Load ( Q ) kip</th>
<th>( q_o / f_o )</th>
<th>( N^* = q_o K_s \tan \delta )</th>
<th>Initial Modulus of Deformation ( E' ) lb./in. (^2)</th>
<th>( E' / q_o )</th>
</tr>
</thead>
<tbody>
<tr>
<td>G-31</td>
<td>21.3</td>
<td>97.0 (96.5)</td>
<td>303.5</td>
<td>0.518</td>
<td>1.77</td>
<td>0.132</td>
<td>3.96</td>
<td>171</td>
<td>488</td>
<td>13,850</td>
<td>45.7</td>
</tr>
<tr>
<td>G-32</td>
<td>39.8</td>
<td>97.3 (97.3)</td>
<td>479.2</td>
<td>0.568</td>
<td>2.87</td>
<td>0.334</td>
<td>7.59</td>
<td>167</td>
<td>476</td>
<td>20,400</td>
<td>42.6</td>
</tr>
<tr>
<td>G-33</td>
<td>61.6</td>
<td>97.2 (97.3)</td>
<td>600.2</td>
<td>0.636</td>
<td>3.74</td>
<td>0.459</td>
<td>10.68</td>
<td>161</td>
<td>459</td>
<td>22,400</td>
<td>37.4</td>
</tr>
<tr>
<td>G-34</td>
<td>79.6</td>
<td>97.4 (97.9)</td>
<td>785.2</td>
<td>0.658</td>
<td>4.56</td>
<td>0.356</td>
<td>14.70</td>
<td>172</td>
<td>490</td>
<td>30,200</td>
<td>38.4</td>
</tr>
<tr>
<td>G-35</td>
<td>104.0</td>
<td>98.3 (98.7)</td>
<td>891.2</td>
<td>0.636</td>
<td>5.33</td>
<td>0.428</td>
<td>17.96</td>
<td>167</td>
<td>476</td>
<td>39,600</td>
<td>44.4</td>
</tr>
<tr>
<td>G-36</td>
<td>120.1</td>
<td>98.2 (99.0)</td>
<td>862.5</td>
<td>0.439</td>
<td>5.01</td>
<td>0.351</td>
<td>19.35</td>
<td>172</td>
<td>490</td>
<td>42,000</td>
<td>48.6</td>
</tr>
</tbody>
</table>

Remarks:
(a) \( K_s \tan \delta = 2.85 \) (from the initial portion of the \( f_o / D \) curve)
(b) Unit weights in parentheses show densities after pile driving
(c) Point displacements in parentheses refer to the third stage of the tests - pushing the entire assembly (point + skin)
### Table 15. Significant Results of Loading Tests - Series G-4

**Field Tests on Moist, Loose Sand**

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Depth D in.</th>
<th>Standard Pen. Test Blow Count N</th>
<th>Ultimate Point Resistance $q_o$ lb./in.$^2$</th>
<th>Ultimate Point Displacement w in.</th>
<th>Ultimate Skin Resistance $f_o$ lb./in.$^2$</th>
<th>Ultimate Skin Displacement in.</th>
<th>Total Ultimate Load Q kip</th>
<th>$q_o/f_o$</th>
<th>$N^* = q_o/K_s \tan \delta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>G-42</td>
<td>46.6</td>
<td>4</td>
<td>328</td>
<td>0.41 (0.42)</td>
<td>-</td>
<td>-</td>
<td>4.55</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>G-43</td>
<td>59.0</td>
<td>6</td>
<td>417</td>
<td>0.24 (0.36)</td>
<td>1.13</td>
<td>0.48</td>
<td>6.40</td>
<td>370</td>
<td>599</td>
</tr>
<tr>
<td>G-44</td>
<td>80.1</td>
<td>6</td>
<td>336</td>
<td>0.38 (0.47)</td>
<td>0.97</td>
<td>0.36</td>
<td>6.41</td>
<td>346</td>
<td>560</td>
</tr>
<tr>
<td>G-45</td>
<td>98.5</td>
<td>6</td>
<td>409</td>
<td>0.54 (0.60)</td>
<td>1.30</td>
<td>0.36</td>
<td>7.96</td>
<td>314</td>
<td>508</td>
</tr>
<tr>
<td>G-46</td>
<td>119.5</td>
<td>9</td>
<td>515</td>
<td>0.48 (0.50)</td>
<td>1.21</td>
<td>0.24</td>
<td>9.61</td>
<td>425</td>
<td>688</td>
</tr>
</tbody>
</table>

**Remarks:**

(a) $K_s \tan \delta = 1.62$ (estimated by interpolation)

(b) The high values of $q_o/f_o$ and $N^*$ in columns (9) and (10) can easily be explained by assuming that moist sand at the field site had a capillary cohesion of about 1.5 lb./in.$^2$

(c) Point displacements in parentheses refer to the third stage of the tests - pushing the entire assembly (point + skin)
Figure 55. Variation of Point Resistance with Pile Length.
Figure 56. Variation of Skin Resistance with Pile Length.
those obtained later for large 18-in. piles. The linear increase of bearing
capacity with depth can be noticed only at relatively shallow depths, not
exceeding about five pile diameters. As the pile length increases further,
the rate of increase of bearing capacity decreases. For very long piles both
point and skin resistances reach constant final values. The relative depth
at which this occurs varies apparently with the relative density of the
sand. In loose sand (Series G-1) this occurred at about ten pile diameters,
in dense sand (Series G-3) only at about thirty pile diameters. In analogous
tests with buried foundation the observed relative depths were about the same
for loose and medium dense sand. However, for dense sand constant resistances
were reached at about twenty pile diameters of depth.

The average displacements needed to reach ultimate loads are shown in
Table 16. The same table contains similar data from tests with buried foun­
dations in homogeneous sand. It is seen that ultimate point loads of driven
piles are reached at considerably smaller displacements than in the case of
buried foundations. In connection with this it should be added that the
ultimate point displacements from the first stage of the tests (point loading
only) are somewhat larger than those from the third stage of the test (point +
skin loading, shown in parentheses), indicating the effects of repeated load
application.

There appears to be some tendency toward lower ultimate settlements in
the case of piles in loose sand. A similar tendency was noticed in tests
with buried foundations, which also indicated a trend toward larger relative
settlements of larger and deeper foundations. This tendency can also be
detected by comparing ultimate displacements of large, 18-in. diameter piles
(Table 19).
### Table 16. Average Displacements Needed to Reach Ultimate Loads.

<table>
<thead>
<tr>
<th>Series</th>
<th>Piles</th>
<th>Soil</th>
<th>Average Ultimate Point Settlement ( w ) in.</th>
<th>( w/B )</th>
<th>Average Ultimate Skin Displacement in.</th>
</tr>
</thead>
<tbody>
<tr>
<td>G-1</td>
<td>driven</td>
<td>loose, dry sand</td>
<td>0.31 (0.24)</td>
<td>7.8 (6.0)</td>
<td>0.33</td>
</tr>
<tr>
<td>G-2</td>
<td>driven</td>
<td>medium dense, dry sand</td>
<td>0.51 (0.33)</td>
<td>12.8 (8.2)</td>
<td>0.36</td>
</tr>
<tr>
<td>G-3</td>
<td>driven</td>
<td>dense, dry sand</td>
<td>0.58 (0.38)</td>
<td>14.5 (9.5)</td>
<td>0.34</td>
</tr>
<tr>
<td>G-4</td>
<td>driven</td>
<td>loose, moist sand</td>
<td>0.41 (0.47)</td>
<td>10.2 (11.7)</td>
<td>0.36</td>
</tr>
<tr>
<td>C</td>
<td>buried</td>
<td>dry sand</td>
<td>1.09</td>
<td>27.2</td>
<td>0.34</td>
</tr>
</tbody>
</table>

**Remark:** Numbers in parentheses refer to point displacements in the third stage of the tests (pushing the entire assembly - point + skin)
It is significant to note that the skin displacements needed to reach ultimate resistances appear to be independent of initial density of the sand and equal to about 0.35 in. In tests on buried circular foundations, discussed in previous chapter, ultimate skin displacements of about 0.30 to 0.40 inches were recorded. They appear to be independent of both the sand density and the foundation dimensions (diameters from about 2 to 7 inches, and lengths from about 10 to 120 inches). About the same displacements were found in a tension test with the large 18-in. diameter pile (see Table 19). It will be interesting to further explore the possible general validity and implications of this finding, which strengthens the previously expressed beliefs that the mobilization of shear strength along a fixed rupture surface is governed by the absolute displacement along that surface.

Analysis of the Driving Record

Since a number of organizations are still using pile driving formulas to determine the ultimate pile loads, it was considered of interest to compare the predictions of at least one of these formulas with actually observed ultimate loads. In view of the fact that the piles tested in this phase are steel pipes driven by a drop hammer, the Engineering News formula has been selected and used in the form:

\[ R = \frac{W_r h}{s + s_1} \]  \hspace{1cm} (24)

In this expression \( R \) is the pile resistance, \( W_r \) the ram weight, \( h \) ram stroke, \( s \) pile set and \( s_1 \) a driving constant having the dimension of length, to be taken equal 1.0 inch for drop hammers.

By using the above formula with sets as presented in Figs. 48 through
51 the ultimate loads shown in Fig. 57 have been obtained. The latter
figure, a direct comparison of computed and observed ultimate loads, indicates
a reasonable agreement for all piles driven in dry sand, one rare case for
which all experts agree that the dynamic formulas should be valid for. The
same figure shows that the presence of a slight amount of moisture in sand
is sufficient to change the picture; the ultimate loads computed by the driving
formula were up to 50% lower than the actual loads, in all probability because
of pore pressures induced by driving. It is interesting to note, however,
that a satisfactory agreement can be obtained by taking the driving con-
stant \( s_1 \) to be equal 0.4. A similar experience with large, 18-in. diameter
piles in submerged sand is discussed in the following chapter (cf. Fig. 85).

**Surface Deflections**

The surface deflection measurements during loading tests showed, as in
the previous phase of this investigation, rapid increases of deflections
during the early stages of the load application. This was followed, beyond
ultimate loads, by almost steady deflections in the case of piles in dense
sand, and by continually increasing deflections in loose sand. (Fig. 58)
There was, however, a major difference in the present test series with driven
piles. The surface deflections of piles in loose and medium dense sand were
directed downward, similarly to those observed on buried foundations, however
in the case of piles in dense sand (Series G-3) they were directed mostly
upwards. This can be seen nicely in Figs. 59 and 60, which show the shapes
of deflected surfaces on the basis of the averages of measured deflections
at twelve points around the loaded piles at failure. It is very important
to note, by comparison with Figs. 55 and 56, that the downward movements of
EN FORMULA  \[ Q = \frac{W}{s} + s_1 \]

\[ s_1 = 1.0 \text{ IN} \]

Figure 57. Comparison of Actual Ultimate Loads with Those Obtained by the Engineering News Formula.
Figure 58. Typical Surface Deflection versus Pile Settlement Diagrams.
Figure 59. Shapes of Deflected Surfaces at Failure, Medium Dense Sand.
Figure 60. Shapes of Deflected Surfaces at Failure, Dense Sand.
the adjacent sand surface are observed exactly in those tests in which the point and skin resistance data indicates departure from linear increases. This represents one significant additional argument in support of the explanation of non-linear behavior at greater depths by "arching".

**Bearing Capacity Factors and Coefficients**

In the preceding chapter substantial arguments were presented to prove that the non-linear increase of bearing capacity with depth cannot be attributed to the decrease of the bearing capacity factors $N_q$ alone. This, of course, suggests that the pressure $q$ in Eq. 3 is not necessarily equal to the overburden pressure. It was also demonstrated that, in the range of pile sizes investigated (2 to 7 in. in diameter) those bearing capacity factors appeared to be unique functions of the relative density or the angle of internal friction of sand. The same conclusion can be reached from data presented in Fig. 61. Here the measured values of

$$\frac{q_o}{f_o} \cdot (K_s \tan \delta) = N_q^*$$

are plotted against the relative density of sand, along with previous similar data for buried foundations.

It appears again that all the points define a single function. Note that the theoretical curve after Berezantsev (44), which is plotted for reference, fits exceptionally well the measured data. From this comparison it may be concluded that the bearing capacity factors $N_q^*$ for driven and for buried foundations are the same if driven piles are analyzed by using an increased sand density equal to the mean density prior and after pile driving.

Thus, it appears that the point bearing capacities of driven piles are
Figure 61. Observed Bearing Capacity Factors.
higher than those of bored piles of the same diameter only inasmuch as the sand
density is increased by pile driving. This is, however, not true for the
bearing capacities of the skin, as evident from comparisons in the following
Table 17.

Table 17 contains the values of the coefficient of skin pressure $K_s$, which all were found from measured values of $K_s \tan \delta$ corresponding to the
initial straight-line part of the skin resistance-versus-depth curve. Based
on earlier friction tests with steel plates on sand (36) it was assumed that
$\tan \delta = \tan \phi_{\text{min}} = 0.625$.

It is seen that the values for driven piles (column 3) are comparable
to passive earth pressure coefficients, while those for rectangular buried
foundations (column 5) are of the order of magnitude of at rest pressure
coefficients. Both of these findings are reasonable and agree to a certain
degree with limited former experience (45) (46). However tests with larger
piles, described in the following chapter indicate that $K_s$ may be affected
by pile size. Also, the values of $K_s$ for circular buried foundations are
somewhat surprising. It is not easy to explain why they should be so much
higher than those for rectangular foundations. Additional research will be
needed to clarify these questions.

Before concluding, it might be of interest to point out to the indications,
that the ratio of point and skin resistances of the pile is a parameter
directly related to the relative density of sand and the method of placement
of the pile and independent of pile size. This is evident from Fig. 62 in
which data from first two phases of the current research project has been
presented. Comparisons with other similar data from investigations on silts
(47) and clays, presented in Fig. 63 indicate that the parameter $q_c/f_c$, for
Table 17. Measured Coefficients of Skin Pressure $K_s$

<table>
<thead>
<tr>
<th>Sand</th>
<th>Relative Density $D_R$ (1)</th>
<th>Driven Piles (2)</th>
<th>Buried Circular Foundations (3)</th>
<th>Buried Long Rectangular Foundation (4)</th>
</tr>
</thead>
<tbody>
<tr>
<td>loose (backfilled)</td>
<td>0.2 to 0.4</td>
<td>2.5</td>
<td>1.6</td>
<td>0.4</td>
</tr>
<tr>
<td>medium dense (backfilled)</td>
<td>0.5 to 0.7</td>
<td>3.0</td>
<td>2.2</td>
<td>0.5</td>
</tr>
<tr>
<td>dense (vibrated)</td>
<td>0.8 to 0.9</td>
<td>4.5</td>
<td>3.3</td>
<td>1.1</td>
</tr>
</tbody>
</table>

Remark: Assumed $\delta = \phi_{\text{min}} = 32^\circ$
Figure 62. Ratio of Point and Skin Resistances.
Figure 63. Variation of $q_0/f_0$ with $\phi$.
piles in homogeneous masses of soil, may be a continuous function of the angle of internal friction $\phi$. Further evidence of this kind, found in tests with large, 18-inch diameter piles, has been presented in the following Chapter IV, Table 19.*

**Conclusions**

1. Experiments with 4-in.-diameter piles driven into homogeneous masses of sand indicate basically the same pattern of behavior and failure under load as found in the previous phase of this investigation dealing with buried foundations. At shallow depths there is linear increase of unit point and skin resistances with depth. At greater depths both resistances turn to quasi-constant final values. The relative depth at which this happens, however, is larger for driven piles, particularly in dense and very dense sands.

2. The relative displacements needed to reach ultimate point loads are proportional to the pile diameter. They are relatively smaller than those observed in the case of buried foundations (10% of pile diameter for driven piles versus > 25% of foundation width for buried foundations). However, the displacements needed to reach ultimate loads along the pile shaft appear to be independent of foundation size and method of construction. In the present test series they varied from about 0.3 to 0.4 inches irrespective of the relative density of sand.

3. The point bearing capacity of a driven pile is higher than that of

*It should not be overlooked that the curves in Figs. 62 and 63 have been drawn with data from tests in dry sand and verified with data in submerged sand only. In moist sands, the presence of capillary cohesion significantly affects $q$, without altering $f$. The net result is a substantial increase of the $q/f$ ratio. In the G-4 field test series, for instance, the average value of this ratio is 364, to be compared with only 69 in the comparable G-1 series.
a buried pile of the same diameter only inasmuch as the sand density is increased by pile driving. However, this cannot be said for the bearing capacity of pile skin, which is substantially higher for a driven pile because of increased lateral pressure on the skin.

(4) With a proper choice of "driving constants" dynamic formulae may give reasonable estimates of ultimate loads of piles in dry sand. However, the presence of a small amount of moisture in the sand mass is sufficient to alter this picture.

(5) The ratio of point and skin resistances of a pile or a deep foundation in general is a parameter directly related to the relative density of sand and independent of foundation size. While this ratio appears to be the same for dry or submerged sands, it may be significantly affected by the capillary "cohesion" of moist sand.
CHAPTER IV

Field Tests with Large Instrumented Piles

Introduction

With continued advancement of the foundation building technology there is a growing tendency toward utilization of larger and longer piles for bridge foundations. In view of the fact that conventional scaling laws are not generally valid in the case of deep foundations in sand, it was considered necessary to verify the principal findings of the two previous phases of this investigation by performing field tests similar to those described in the preceding chapter, however with substantially larger instrumented piles.

These tests have been performed at the site of future Ogeechee River bridge on Interstate Highway 16, in Effingham County, approximately eighteen miles west of Savannah, Georgia. General geographical location of the site is shown in Fig. 64, while Fig. 65 shows the site location on a detailed map. This site is characterized by deep deposits of medium dense to dense sand, typical for this area of the Atlantic Coastal Plain. In its upper horizons this sand is similar in composition to the Chattahoochee River and Roberta sands used in the previous two phases of the project.

Site Exploration

Detailed soil exploration of the site included exploratory borings with standard penetration tests, static deep cone penetration tests, as well as nuclear density and moisture content tests. The relative position of five exploratory borings, four deep cone penetration tests and three nuclear tests with respect to the two pile testing points is shown in Fig. 66.
Figure 6.4. General Geographical Location of the Ogeechee River Site.
Figure 65. Detailed Location of the Test Site.
Figure 66. Position of Exploratory Borings, Deep Cone Penetration Tests and Nuclear Tests.
Standard penetration tests were made according to the existing ASTM standards, using a 2-inch OD sampling spoon driven by a 140-lb. hammer falling free 30 inches for each blow. Deep cone static penetration tests were made with a 1.6-inch OD 60° cone penetrometer of the State Highway Department of Georgia, very similar in construction to the standard Dutch and Belgian devices of this kind.

Nuclear density and water content tests were made by using depth probes models 104A and 504 manufactured by Troxler Laboratories, Inc. inside 2-in. OD AX-casings driven by a 340-lb. hammer. Model 104A is a moisture depth gauge containing a 100 millicurie american-berilium source with a detector of thermal neutrons. Model 504 is a depth density gauge containing a 3 millicurie radium source, and an appropriate low energy gamma photon detecting element. They were used in conjunction with a Model 200B scaler and Model G-100 rate meter.

A brief summary of results of exploratory borings is presented in Table 18. Ranges of particle size distribution curves are shown in Fig. 67. The fine sand curves in this figure refer to materials found mostly in the top 5 ft. of soil profile as well as in pockets, mostly at elevations -10 to -20 ft.

The results of standard penetration tests are presented in Fig. 68. It is interesting to note the practically linear increase of the number of blows per foot N with overburden pressure. The results of static cone penetration tests are shown in Fig. 69. The nearly constant average resistance below the elevation +10 should be noted in this figure.

The results of nuclear density tests are shown in Fig. 70. They disclose the existence of slightly looser deposits in the top 10 ft. of the soil profile, as well as in pockets around the elevation -20 ft.
Table 18. Summary of Results of Exploratory Borings

<table>
<thead>
<tr>
<th>Boring No.</th>
<th>Depth ft.</th>
<th>Soil Description</th>
<th>Classification (AC)</th>
<th>Average SP Blow Count</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-1</td>
<td>0-11</td>
<td>Medium dense, brown-grey fine sand, slightly silty</td>
<td>SM</td>
<td>6</td>
</tr>
<tr>
<td></td>
<td>11-28</td>
<td>Dense grey sand, slightly gravelly</td>
<td>SP</td>
<td>14</td>
</tr>
<tr>
<td></td>
<td>28-30</td>
<td>Dense grey, well graded sand</td>
<td>SW</td>
<td>.</td>
</tr>
<tr>
<td></td>
<td>30-41</td>
<td>Dense grey sand</td>
<td>SP</td>
<td>21</td>
</tr>
<tr>
<td></td>
<td>41-52</td>
<td>Dense grey fine sand, slightly silty</td>
<td>SM</td>
<td>37</td>
</tr>
<tr>
<td>B-2</td>
<td>0-12</td>
<td>Medium dense, brown and grey fine sand</td>
<td>SP</td>
<td>6</td>
</tr>
<tr>
<td></td>
<td>12-19</td>
<td>Medium dense, grey sand</td>
<td>SP</td>
<td>15</td>
</tr>
<tr>
<td></td>
<td>19-32</td>
<td>Dense, grey, well graded sand</td>
<td>SW</td>
<td>32</td>
</tr>
<tr>
<td></td>
<td>32-35</td>
<td>Dense, grey fine sand</td>
<td>SP</td>
<td>23</td>
</tr>
<tr>
<td></td>
<td>35-40</td>
<td>Medium dense, silty fine sand</td>
<td>SM</td>
<td>23</td>
</tr>
<tr>
<td></td>
<td>40-42</td>
<td>Dense, grey well-graded sand</td>
<td>SW</td>
<td>34</td>
</tr>
<tr>
<td></td>
<td>42-44</td>
<td>Medium dense, grey sand, slightly micaceous</td>
<td>SP</td>
<td>17</td>
</tr>
<tr>
<td></td>
<td>44-52</td>
<td>Very dense green, fine, silty sand and grey, well graded sand, in layers</td>
<td>SM/SW</td>
<td>52</td>
</tr>
<tr>
<td>B-3</td>
<td>0-12</td>
<td>Loose, brown and grey, silty fine sand</td>
<td>SM</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>12-15</td>
<td>Dense, grey sand</td>
<td>SP</td>
<td>17</td>
</tr>
<tr>
<td></td>
<td>15-33</td>
<td>Dense, grey, well-graded sand</td>
<td>SW</td>
<td>19</td>
</tr>
<tr>
<td></td>
<td>33-52</td>
<td>Dense, grey fine silty sand, slightly micaceous</td>
<td>SM</td>
<td>35</td>
</tr>
</tbody>
</table>

(Continued)
Table 18 (Concluded)

<table>
<thead>
<tr>
<th>(1)</th>
<th>(2)</th>
<th>(3)</th>
<th>(4)</th>
<th>(5)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-4</td>
<td>0-1</td>
<td>Brown fine silty sand</td>
<td>SM</td>
<td>.</td>
</tr>
<tr>
<td></td>
<td>1-5</td>
<td>Very soft, brown-grey, highly plastic clay</td>
<td>CH</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>5-10</td>
<td>Medium dense, grey fine sand, slightly silty</td>
<td>SM</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>10-17</td>
<td>Medium dense, grey sand</td>
<td>SP</td>
<td>9</td>
</tr>
<tr>
<td></td>
<td>17-32</td>
<td>Dense, grey, well graded sand, slightly gravelly</td>
<td>SW</td>
<td>19</td>
</tr>
<tr>
<td></td>
<td>32-37</td>
<td>Dense, grey sand</td>
<td>SP</td>
<td>23</td>
</tr>
<tr>
<td></td>
<td>37-39</td>
<td>Dense, grey, well graded sand</td>
<td>SW</td>
<td>23</td>
</tr>
<tr>
<td></td>
<td>39-52</td>
<td>Dense, grey, fine silty sand</td>
<td>SM</td>
<td>32</td>
</tr>
<tr>
<td>69</td>
<td>0-10</td>
<td>Medium dense, grey, fine sand</td>
<td>SM</td>
<td>8</td>
</tr>
<tr>
<td></td>
<td>10-13</td>
<td>Dense, brown-grey well-graded sand</td>
<td>SW</td>
<td>17</td>
</tr>
<tr>
<td></td>
<td>13-34</td>
<td>Dense, grey sand</td>
<td>SP</td>
<td>20</td>
</tr>
<tr>
<td></td>
<td>34-45</td>
<td>Dense, grey, fine sand</td>
<td>SP</td>
<td>33</td>
</tr>
</tbody>
</table>
Figure 67. Particle Size Distribution of Sands at the Test Site.
Figure 68. Results of Standard Penetration Tests.
Figure 69. Results of Deep Cone Penetration Tests.
Figure 70. Results of Nuclear Density Tests.
Reaction Structure and Testing Equipment

For the purpose of this investigation a special reaction structure consisting of a steel box girder of 60 ft. span, supported at its ends by reaction pile groups was constructed at the site. A schematic plan of the entire assembly is shown in Fig. 71, together with a test pile in position. A photographic view is shown in Fig. 72.

The box girder was constructed of high-strength steel in the shops of the American Bridge Division, U. S. Steel Corporation in Birmingham, Alabama. It was transported by rail in three sections that were assembled at the site by high-strength bolts. Following the test series at testing point 1, the box girder weighing about 50 tons was moved to the location of testing point 2, which lay 60 ft. apart. The interior of the box girder was made watertight to hold approximately 60,000 gallons of water. This amount of water, pumped from the river, provided a ballast of 250 tons (50,000 lb.) for reaction of the test pile. The remaining reaction of about 200 tons was provided by reaction pile groups at the ends. Each of three such groups consisted of four 3/8 in. thick closed steel pipe piles, having outside diameters of 12.75 in. The driven lengths of the piles ranged from 42 to 66 ft. These piles, all driven with a McKiernan-Terry DB-40 diesel hammer (ram weight 2 tons), performed very satisfactorily in carrying their design loads of 25 tons per pile in tension.

The detail of the loading assembly is shown in Fig. 73. The test piles were loaded by a 500-ton hydraulic jack equipped with a sensitive pressure gauge for measurement of loads (accuracy 0.5% of measured pressure). Loads smaller than 200 tons were also recorded on a Baldwin-Lima electronic load cell having an accuracy of 0.5% of measured load. Axial loads along the shaft were measured by Budd 141-C6 foil strain-gauges connected in bridges of four
Figure 71. Schematic Plan and View of the Reaction Structure.
Figure 72. A Photograph of the Reaction Structure. Reaction Piles are Visible in the Foreground. The Inside of the Box-girder is Filled with Water.
Figure 73. Detail of the Loading Assembly.
gauges to eliminate possible effects of bending and improve the accuracy of load measurement to about 1 ton. A scheme of the arrangement of these gauges in bridges can be seen in Figs. 74 and 75.

Pile displacements were measured by micrometer dial gauges (0.0001 in. precision) against a 60 ft. long WF 45 reference beam supported at its ends by 30 ft. long 10 in. BP 42 piles. The displacements of the reaction piles were checked by a precision level (0.01 in. accuracy), which was also used during the tests to check the displacement readings made on dial gauges, as well as the elevation of the water table.

**Test Piles**

Two test piles were used. Test pile no. 1 was an instrumented steel pipe 18 inches in diameter, with ½ inch thick walls. This pile consisted of five sections, each approximately 10 ft. long, allowing driving and successive testing in five stages at depths of approximately 10, 20, 30, 40, and 50 ft. The bottom section had a 2 inch thick flat steel plate at the base that served as point for the pile during all stages of the test. Following the completion of the first stage of the test, the second section was butt-welded to the first one, thus forming the pile to be driven in the second stage of the test, and so on.

A layout of the test pile 1, showing all its essential dimensions is given in Fig. 74. The same figure shows the position of 28 141-C6 strain gauges, grouped in seven bridges of 4, that were attached to the interior walls of the pile sections for the purpose of measuring the axial load in the pile at different elevations. Fig. 75 shows the details of gauge layout within a bridge designed to register axial load in the pile shaft only, thus cancelling
Figure 74. Layout of Test Pile No. 1, Showing the Position of Strain-gauge Bridges.
Figure 75. Details of a Strain-gauge Bridge.
strains resulting from possible pile bending.

Test pile no. 2 was a 16 inch square 55 ft. long prestressed concrete pile designed by the Bridge Division of the Georgia State Highway Department and manufactured by Macon Prestressed Concrete Co. A layout of this pile, which has been selected for foundations of the future bridge, is shown in Fig. 76.

Both piles have been driven by a McKiernan-Terry DE-40 diesel hammer having a ram weight of 2 tons (4,000 lb.) The stroke was varied between 5 and 8 ft. The entire driving operation proceeded very smoothly, in spite of comparatively large size of piles and high resistance of the dense sandy soil. A photograph of the pile driver in operation is shown in Fig. 77. Driving records of both test piles are presented in Fig. 78. This figure shows the pile resistance as expressed in the number of blows per foot as a function of pile penetration in feet. The variation of hammer stroke with depth is also marked on the side.

**Loading Procedure**

As mentioned earlier, test pile 1 (18 in. OD steel pipe), was driven and tested in five stages. Tests with this pile, made at driving depths of 10, 20, 30, 40 and 50 ft. were numbered H-11 through H-15, respectively. The test with 16 in. square prestressed concrete pile was numbered H-2.

After completion of each test stage, a new pile section was added by welding and pile driving was continued the same day. The next stage followed no sooner than the following day. In this manner there was at least a 12-hour waiting period between driving and testing, assuring the dissipation of excess pore-water pressures created by pile driving.

Following the five tests with pile 1, a specially designed pull-out
Figure 76. Layout of Test Pile No. 2, Showing Prestressing Cables and Reinforcement.
Figure 77. Pile Driver in Operation.
Figure 78. Driving Record of the Test Piles.
assembly was mounted on the reaction structure and the test pile was subjected to a tension test (numbered H-16) by using the same hydraulic jack and the same displacement-recording arrangement.

A constant-rate-of-penetration procedure, (91), was followed in each test. The load was increased in increments corresponding to a rate of approximately 0.05 inches of pile displacement per minute. Loading was continued until a pile displacement of at least 5 inches was reached. The load was then released in two increments and applied again in two increments to the previous intensity. In this manner each loading stage took approximately 2 hours.

Strain-gauge and dial-gauge readings were made generally at five minute intervals, corresponding to intervals in which load increments were applied. The test operators followed the variation of load in the lowest positioned strain-gauge over the five-minute period by making readings at 0.5, 1, 2, 3, 4, and 5 minutes from the application of the new load increment. The observed difference in load between last two readings was consistently very small, never exceeding 1% of the total load recorded by that particular gauge.

**Test Results**

The load-displacement diagrams obtained in individual test stages of pile 1, as well as the test H-2 are shown in Fig. 79. They are very similar in shape to those obtained earlier with smaller piles, and consist of initial linear sections which gradually turn to final linear sections, with steadily increasing load. The black dots indicate the ultimate loads, defined, as in the previous phases of this investigation, as loads at which the displacement rate first reached its maximum.

The distributions of axial load in the piles as recorded by strain-
Figure 79. Load-Displacement Diagrams for Test Piles.
gauges along the shaft, is shown in Figs. 80 through 82, where a few missing readings from non-recording gauges were interpolated. The distributions at first loading are shown by solid lines; those in unloading by dotted lines and those in reloading by dashed lines. The distributions at ultimate loads are marked also by black dots.

Significant numerical data resulting from loading tests have been assembled in Table 19. The ultimate point displacements given are based on shaft displacements computed from actual skin load distributions. The elasticity moduli of steel and concrete used in these computations are, respectively, $30 \cdot 10^6$ lb./sq. in. and $4.5 \cdot 10^6$ lb./sq. in.

The initial modulus of deformation of the soil $E'$ given in column (11) has been evaluated assuming a stress distribution under pile point due to the point load equivalent to that in an elastic solid under surface loading (Boussinesq stress distribution). It should be noted that the corresponding value assuming stress distributions similar to those under deep loads in an elastic solid (Mindlin) would be about 40% of the values shown. The ultimate point load of the test pile no. 2 was estimated from unloading and reloading portions of the load-settlement curve. A discussion of all the significant results of loading tests follows in the next chapter.

**Ultimate Loads**

The ultimate point and skin resistances measured in pile load tests are plotted in Fig. 83 by black dots. It is seen from this figure that both point and skin resistances increase approximately linearly with depth over a limited zone, not exceeding ten pile diameters. Beyond a depth of approximately twenty pile diameters both point and skin resistances reach nearly constant
Table 19. Significant Results of Loading Tests

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Embedded Depth In.</th>
<th>Ultimate Load Qp, Tons</th>
<th>Ultimate Skin Load Qs, Tons</th>
<th>Ultimate Point Resistance (4) Qp/ft.²</th>
<th>Ultimate Skin Resistance (5) Qs/ft.²</th>
<th>Ultimate Total Load Q = Qp + Qs Tons</th>
<th>Ultimate Point Displacement In.</th>
<th>Ultimate Displacement In.</th>
<th>qo</th>
<th>f0</th>
<th>Initial Modulus of Deformation E' Ton/ft.²</th>
<th>E' qo</th>
</tr>
</thead>
<tbody>
<tr>
<td>H-11</td>
<td>119</td>
<td>61</td>
<td>34.5</td>
<td>15</td>
<td>0.32</td>
<td>76</td>
<td>1.43</td>
<td>1.60</td>
<td></td>
<td></td>
<td>108</td>
<td>650</td>
</tr>
<tr>
<td>H-12</td>
<td>241</td>
<td>173</td>
<td>97.8</td>
<td>59</td>
<td>0.62</td>
<td>232</td>
<td>1.66</td>
<td>1.78</td>
<td></td>
<td></td>
<td>158</td>
<td>4,450</td>
</tr>
<tr>
<td>H-13</td>
<td>349</td>
<td>212</td>
<td>119.8</td>
<td>85</td>
<td>0.62</td>
<td>297</td>
<td>1.57</td>
<td>1.81</td>
<td></td>
<td></td>
<td>193</td>
<td>6,380</td>
</tr>
<tr>
<td>H-14</td>
<td>472</td>
<td>214</td>
<td>121.0</td>
<td>133</td>
<td>0.72</td>
<td>347</td>
<td>1.39</td>
<td>1.74</td>
<td></td>
<td></td>
<td>168</td>
<td>5,180</td>
</tr>
<tr>
<td>H-15</td>
<td>591</td>
<td>258</td>
<td>145.9</td>
<td>163</td>
<td>0.70</td>
<td>421</td>
<td>1.33</td>
<td>1.87</td>
<td></td>
<td></td>
<td>208</td>
<td>6,300</td>
</tr>
<tr>
<td>H-25</td>
<td>591</td>
<td>-</td>
<td>-</td>
<td>173</td>
<td>0.74</td>
<td>173</td>
<td>-</td>
<td>0.38</td>
<td></td>
<td></td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>H-2</td>
<td>600</td>
<td>172*</td>
<td>97.2*</td>
<td>131*</td>
<td>0.50*</td>
<td>303</td>
<td>1.34</td>
<td>1.62</td>
<td></td>
<td></td>
<td>194*</td>
<td>4,250*</td>
</tr>
</tbody>
</table>

*Values based on analysis from unloading and reloading data.
Figure 80. Distribution of Axial Pile Load, Tests H-11, H-12, H-13.
Figure 81. Distribution of Axial Pile Load, Test H-14.
Figure 82. Distribution of Axial Pile Load, Test H-15.
Figure 83. Variation of Point and Skin Resistances with Depth.
final values. These findings are in agreement with previous observations on smaller piles in the laboratory and in the field. They depart from the established concepts of linear increase of bearing capacity of deep foundations with depth.

The same Fig. 83 shows, for comparison, the maximum and minimum values of point and shaft resistances measured in static cone penetration tests, the results of which have been presented in Fig. 69. As mentioned earlier, extensive previous experience with penetrometers has indicated that the point resistance of piles should be comparable to that of penetrometers, whereas the skin resistance of piles should be approximately the double of that measured by the penetrometers. An examination of Fig. 83 reveals that the Ogeechee River test results are in accord with this previous experience. It should be noted that point resistances of larger piles normally correspond to the average of minimum penetrometer point resistances.

A comparison of measured pile point and skin resistances with those predicted by standard penetration tests is presented in Fig. 84. This figure shows maximum and minimum standard penetration resistances $N$ (blows per foot) converted into point and skin resistances $q_0$ and $f_0$ according to empirical relationships

$$q_0 = 4N \text{ (ton/ft.}^2\text{)}$$

$$f_0 = 0.02N \text{ (ton/ft.}^2\text{)}$$

proposed by Meyerhof (48). In analyzing this figure one may find reasonable agreement between predetermined and observed point and skin resistances at greater depths (40 to 50 ft.). At the same time there is a discrepancy of the two sets of data at depths below 30 ft. A better fit of available data
Figure 84. Point and Skin Resistances Compared with Standard Penetration Data.
can be obtained by changing the factors in formulas (26) and (27). However, since the trends of increase of the standard and pile penetration resistances with depth are basically different, (the former being approximately linear with depth) an attempt to improve the proposed formulas in their present form does not appear to be justified.

**Tension Resistance**

The ultimate load of test pile 1 in tension, measured in test H-16 is slightly larger than the ultimate skin load in the preceding test H-15 with the same pile, and slightly smaller than the maximum skin load recorded in that test. It may be stated that for all practical purposes, the results of the tension test H-16 indicate that the ultimate skin loads in tension and compression are the same.

This result is very different from those obtained in the laboratory on 2-inch diameter steel pipe piles in dry, dense Chattahoochee River sand. The latter indicated ultimate skin loads in tension as small as 30% of the ultimate skin loads in compression. The reasons for this apparent discrepancy are not known. However it is significant to note that the unit skin resistance in tension of 0.74 ton/ft.\(^2\) measured in test H-16, is almost the same as the average 0.73 ton/ft.\(^2\) from five tension tests reported by Ireland (42). These tests were made with step-tapered Raymond piles of average diameter of 10 inches, average embedded length of 16.5 ft., in soil profiles consisting of submerged, fine sands of Florida and exhibiting practically the same standard penetration resistances as those of the Ogeechee River site. It should be added that other observations on full-scale piles do not contradict the above finding either. It is believed that the differences must be due to some kind of scale effect.
Analysis by Pile Driving Formulas

For the reasons given previously in Chapter III, the ultimate loads were compared with those determined from pile driving records by the use of dynamic pile formulas. Two driving formulas were used for this purpose: the very simple Engineering News formula and the more elaborate Hiley formula. The EN formula has been used in the common form given in the preceding chapter (expression 24). The Hiley formula has been used in the following, well known form:

\[
R = \frac{e_p W_r h}{s + 0.5 (c_1 + c_2 + c_3)} \frac{W_r + e^2 W'}{W_r + W_p}
\]  (28)

Here \( R \), \( W_r \), \( h \) and \( s \) have the same meaning as in expression (24). \( e_p \) (dimensionless) is the driving energy efficiency, assumed to be 100% for our diesel hammer. \( e \) (dimensionless) is the coefficient of restitution taken 0.55 for steel hitting steel, and 0.25 for steel hammer hitting wooden cushion over precast concrete pile. \( c_1 \), \( c_2 \), \( c_3 \) are temporary compressions, having the dimension of length. In our case \( c_1 = 0 \) for steel pile without cushion and \( c = 0.10 \) for concrete pile with cushion. A nominal value of 0.1 inch was taken for \( c_3 \) while \( c_2 \) (temporary compression of the pile itself) was computed from known pile axial loads, lengths and cross sections assuming values of elasticity moduli of steel and concrete to be, respectively, \((30) \times (10^6)\) lb./sq. in. and \((4.5) \times (10^6)\) lb./sq. in.

The results of this evaluation are presented, together with basic computational data, in Table 20 and also shown graphically in Fig. 85. It is evident from these results that the pile resistances determined by the Hiley formula appear to be well scattered around the line of actual resistances,
<table>
<thead>
<tr>
<th>Test No.</th>
<th>Driving Energy Wh</th>
<th>Set s in.</th>
<th>Effective Pile Length in.</th>
<th>Pile Weight Wp Tons</th>
<th>Pile Resistance EN Formula Tons</th>
<th>Pile Resistance Hiley Formula Tons</th>
<th>Actual Pile Resistance Q\textsubscript{R} Tons</th>
<th>Pile Resistance Corrected EN Formula (6) Tons</th>
</tr>
</thead>
<tbody>
<tr>
<td>H-11</td>
<td>168</td>
<td>1.500</td>
<td>138</td>
<td>0.6</td>
<td>67</td>
<td>90</td>
<td>76</td>
<td>87</td>
</tr>
<tr>
<td>H-12</td>
<td>144</td>
<td>0.177</td>
<td>248</td>
<td>1.1</td>
<td>123</td>
<td>367</td>
<td>232</td>
<td>241</td>
</tr>
<tr>
<td>H-13</td>
<td>168</td>
<td>0.172</td>
<td>346</td>
<td>1.6</td>
<td>144</td>
<td>330</td>
<td>297</td>
<td>284</td>
</tr>
<tr>
<td>H-14</td>
<td>180</td>
<td>0.100</td>
<td>442</td>
<td>2.1</td>
<td>164</td>
<td>388</td>
<td>347</td>
<td>347</td>
</tr>
<tr>
<td>H-15</td>
<td>192</td>
<td>0.033</td>
<td>541</td>
<td>2.6</td>
<td>186</td>
<td>316</td>
<td>421</td>
<td>424</td>
</tr>
<tr>
<td>H-2</td>
<td>192</td>
<td>0.032</td>
<td>542</td>
<td>7.6</td>
<td>186</td>
<td>178</td>
<td>303</td>
<td>303</td>
</tr>
</tbody>
</table>

Remark: Ram weight in all tests \( W_r = 2 \) tons
Figure 85. Comparison of Actual Ultimate Loads with Those Obtained by Pile Driving Formulas.
varying from about 60% to 160% of the actual loads. At the same time pile resistances determined by the ordinary Engineering News formula appear to have a general tendency of remaining lower than the actual resistances, being in the extreme case as low as 44% of the actual loads.

Looking only at the quotients of computed and actual pile resistances, it may appear that the agreement exhibited by the Hiley formula is satisfactory. However one should not overlook the fact that, according to that formula there would be practically no increase of pile resistance as the pile length is increased beyond 20 ft. In other terms the analysis by this formula does not indicate at all the actual differences in bearing capacities of piles at different depths. This is, in our opinion, an indication of a basic shortcoming of the formula itself.

It is interesting to note that the Engineering News formula, though apparently less accurate, indicates well the relative bearing capacities of the piles at different depths. Actually, if the constant $s_1$ in that formula is taken to be 0.42 inches, almost perfect agreement of computed and observed loads can be achieved for steel pipe piles. For precast concrete pile this constant appears to be equal 0.60 inches. A similar finding resulting from experiments with smaller steel pipe piles in dry and moist sand, described in the preceding chapter should be recalled.

Two conclusions can be drawn from the preceding analysis. First, the Hiley formula applied to this case does not indicate well the relative bearing capacities of different piles. Second, it appears that by proper selection of the constant $s_1$ in the Engineering News formula a reasonable estimate of relative bearing capacities of different piles can be achieved. These conclusions support the philosophy advocated by many leading foundation engineers,
that the simpler pile driving formulas are not less accurate than the more elaborate ones, and that the principal question we have to ask ourselves always remains whether the pile driving formulas are at all applicable to our particular soil conditions.

**Ultimate Displacements**

An examination of columns (8) and (9) in Table 20 indicates that the ultimate pile loads have been reached at pile head displacements very close to 10% of the pile diameter. The corresponding displacements of the pile points were, in average 8.2% of the pile diameter. These figures are slightly lower, but not significantly different from those presented in Table 16 for smaller driven piles in dry and moist sand. They are also in agreement with extensive experience with ordinary-size piles in sand which has lead to establishment of similar ultimate load criteria in England and the Low Countries.

Thus, on the basis of the present investigation, and in accordance with a number of similar field data, the following criterion for determination of ultimate loads of driven piles in sand is recommended:

Ultimate load of a driven pile in sand can be defined as load corresponding to total pile settlement equal to 10% of pile diameter, or pile point settlement equal to 8% of pile diameter, whichever is smaller.

For bored piles or piers the ultimate settlements are significantly larger and can be conservatively set at 25% of the pile or pier diameter.

**Distribution of Load along Pile Shaft**

Data presented in Figs. 80 through 82 indicate the nature of variation of load distribution along the pile shaft. While there exists a reasonable similarity of load distribution curves in loading and reloading, the curves
in unloading are distinctly different, indicating the presence of negative skin friction along the upper portions of the shaft.

It is interesting to note to what extent the relative magnitudes of point and skin loads vary during pile loading. Fig. 86 shows the point load expressed as percentage of total pile load plotted versus total pile load expressed as percentage of ultimate pile load. It is seen that the participation of pile shaft in carrying the total pile load is relatively greater during the early stages of the loading. The bearing capacity of the pile skin is mobilized faster, at smaller displacements, than that of the pile point. For example, it is seen in that figure that the pile point in test H-15 carries only 40% of the total at one-third of the ultimate load, however it carries 60% of the total at the ultimate load. It is also seen that the participation of the pile point is smaller in longer piles. Thus, in test H-11 (10 ft. long pile) the point carried ultimately over 80% of the total load, while in test H-15 (50-ft. long piles) it carried only 60% of the total load. All these findings are in full agreement with those of the preceding chapters, as well as with previous experience with piles in similar situations.

To obtain the actual distribution of skin friction along the shaft, the curves from Figs. 80 through 82 referring to ultimate loads were differentiated graphically. Although this procedure does not offer great accuracy in view of the relatively limited number of points along the shaft at which load measurements were taken, it still provided a fair picture of the character of skin friction distribution. As evident from Fig. 87, it is found that this distribution was generally parabolic. The skin friction appears to be concentrated more toward the upper portion of the pile in the case of shorter piles and more toward the pile tip in the case of longer piles. Similarity between
Figure 86. Relative Magnitude of Point and Skin Loads at Various Stages of Loading.
Figure 87. Distribution of Skin Friction Along Pile Shafts.
these curves and those observed in the laboratory on small piles in sand \(^{(42)}\), as well as in some field tests in clay and silty clay \(^{(51)}\) is indeed striking. Maximum skin frictions observed appear to be about 1.3 ton/sq. ft., while the averages at greater depth are about 0.65 ton/sq. ft. The variation of average skin resistances with depth, shown in Fig. 83, resembles closely similar curves obtained in experiments with smaller piles, described in preceding chapters.

**Pile Settlements**

As mentioned in previous discussion, the load-settlement curves of the test piles, shown in Fig. 79, exhibited long initial linear parts. To enable idealization of this linear behavior for settlement computations, the slopes of the initial linear parts of load-settlement curves were determined analytically from available data and corrected to exclude the contribution of pile shafts to settlements. This was relatively easily achieved in view of the fact that the magnitude and distribution of axial loads in the piles were exactly known. In this manner the settlements of pile points were analyzed, as in the preceding chapter, by assuming linear stress-strain behavior of the soil and Boussinesq stress distribution under pile points. The initial deformation moduli obtained in this manner are presented in Table 19, column 11.

It is found that the plane-strain deformation modulus \(E' = \frac{E}{1 - \nu^2}\) increases in proportion with ultimate point resistance of the pile, \(q_0\). From column (12) of Table 19, an average \(E'/q_0\) ratio for the present test series (large piles in very dense sand) of 45.8 was found. This value is in good agreement with those obtained earlier for smaller piles in loose to dense sands and reproduced in Fig. 88 together with similar data from tests with
Figure 88. $E/q_0$ as a Function of Relative Density of Sand.
jacked piles described in Chapter V of this report. The experimental points from that figure are best fitted by the empirical expression:

\[ \frac{E'}{q_o} = 25 \left( 1 + D_r^2 \right) \]  

(29)

where \( D_r \) is the relative density of the sand. This equation applies to driven piles in dry or submerged sand, while tests with jacked piles suggest slightly lower values. For piles driven into moist sand a correction for the effect of capillary cohesion, which considerably alters \( q_o \), must be introduced.

On the same Fig. 88 are plotted the \( \frac{E'}{q_o} \) ratios determined by tests of Phase I with buried piles (simulating piers and bored piles). It is seen that they can be approximated by the empirical expression:

\[ \frac{E'}{q_o} = 5.5 \left( 1 + D_r^2 \right) \]  

(30)

On the basis of expressions (29) and (30) the following single expression for determination of point settlements \( w_p \) of deep foundations in sand can be proposed:

\[ w_p = \frac{C_w q_p}{(1 + D_r^2) B q_o} \]  

(31)

In this expression \( q_p \) is the point load (tons), \( B \) foundation diameter (ft.), \( q_o \) the ultimate point (base) resistance (ton/ft.\(^2\)) and \( C_w \) is a settlement coefficient that depends on the method of construction of the foundation and, possibly, some other factors. For the tests of the present research project, performed on three different normally consolidated sands with piles ranging in size from 2 to 18 inches, the coefficient \( C_w \) is equal to 0.04 for driven piles, 0.05 for jacked piles, and 0.18 for buried piles (simulating piers or bored piles resting on undisturbed sand).
It should be emphasized that \( q_0 \) in this expression represents the ultimate point resistance of the foundation in question, which is not necessarily the same for different foundation types and which may be affected by scale effects as well.

To determine the total settlement \( w \) of a deep foundation, the settlement caused by the deformation of the foundation shaft, \( w_s' \), must be added. For this purpose one can use the expression:

\[
\begin{align*}
  w_s = (Q_p + \alpha q_s) \frac{L}{AE_b}
\end{align*}
\]  

where \( L \) is the pile length, \( A \) the cross-sectional area and \( E_b \) the modulus of deformation of the foundation shaft. The coefficient \( \alpha \) depending on the distribution of skin friction along the shaft can be taken to be equal 0.6.

It also should be noted that the expression (31) is free of any assumption concerning stress distribution under pile points. In contrast to this, expressions (29) and (30) are connected with the assumption of Boussinesq-type stress distribution. If the effect of deep burial is introduced according to Mindlin’s solution, lower deformation moduli amounting to about 40% of those given by the mentioned expressions should be used.

A final remark in this discussion is related to determination of settlements of shallow foundations in sand on the basis of deep cone penetration tests. For this purpose Buisman (52) suggested many years ago the use of the expression:

\[
E = 1.5 q_c
\]  

where \( q_c \) is the cone point resistance. Extensive field experience with this approach, particularly in Belgium (8,53,54) and Holland has shown that the
settlements determined by means of expression (33) are generally about twice as large as the actual ones. Thus field experience suggested that, in average,

\[ E = 3q_c \]  \hspace{1cm} (34)

Keeping in mind that the state of stress under a penetrometer point is probably closer to that predicted by the Mindlin's solution, while Boussinesq solution most likely applies to the average case of a shallow foundation, one obtains from (30) the following expression for potential use in settlement analysis by Buisman's approach:

\[ E = 2 (1 + D_r^2) q_c \]  \hspace{1cm} (35)

Thus, the \( E/q_c \) ratio should vary from 2 for very loose sands to 4 for very dense sands. These figures agree quite well with mentioned field experience, expressed by equation (34).

The empirical expression (35) may be suggested as an improvement of the original Buisman's expression (33) to fulfill an urgent need for some reasonably reliable practical method for prediction of settlements of foundations on sand. At the same time expression (31), resulting from the present investigation, may be proposed for trial use in analyses of settlements of piles and piers in sand.

**Bearing Capacity Factors and Coefficients**

In the two previous chapters, it was shown that the bearing capacity factors \( N_q^* \) for smaller (2-to 7 in. diameter) piles appeared to define a unique function of the relative density or the angle of internal friction of sand. A plot of individual test results suggesting the variation of \( N_q^* \) with
sand density was given in Fig. 61. The results of present test series with larger 16 and 18-inch diameter piles analyzed in the same manner yielded a $N_{q}^*$ value of 272 for an average relative density of 0.87. The point representing this set of data on Fig. 61, although within the range of scattering of individual test data, fell lower than the average of data representing tests with smaller piles.

Similarly, an analysis of the present test data indicated a skin pressure coefficient $K_s^*$ of approximately 2.5, a number somewhat lower from those observed in previous test series on smaller piles (see Table 17). It appears that both $N_{q}^*$ and $K_s^*$ are affected by pile size.

This discrepancy can only in part be explained by the fact that the upper few feet of soil in the present test series were made of looser and finer sand than the rest of the mass. If the analysis of test results is revised to take care of this difference a $N_{q}^*$ value of 320 and a $K_s^*$ value of 3.0 is obtained. Thus, it appears that both $N_{q}^*$ and $K_s^*$ show a slight decrease with increase of pile size. Data from a number of previous pile investigations seem to indicate a similar trend. A comprehensive explanation of this phenomenon has been presented elsewhere (55, 33, 56).

It is interesting to point out that the parameter $q_o/f_o$ introduced in the previous chapter appears to remain independent of pile size and dependent on the relative density of sand only. The present test series furnished an average $q_o/f_o$-value of 171, which fits very nicely the data presented in Figs. 62 and 63. It is not difficult to see that the variation of $q_o/f_o$ with $\phi$ for both test series with driven piles can be well represented by the expression:

$$q_o/f_o = 9 (10)^{1.3} \tan \phi$$ (36)
where $\phi$ is the angle of shearing resistance of the soil determined by conventional triaxial tests at 10 lb./sq. in. (0.70 kg./sq/cm.) confining pressure. It should be remembered that this expression results from tests with driven piles in normally consolidated dry or submerged sands. As evident from Figs. 62 and 63, the $q_o/f_o$ ratios should be higher for piers, as well as for bored or jacked piles. They also may be substantially higher in moist sands. In such situations as shown in the preceding chapter, the presence of capillary cohesion significantly increases $q_o$ without altering $f_o$.

Finally, two other quantities that appear to remain independent of pile size are the final point and skin resistances at greater depths. To demonstrate this, the curves from Figs. 33 and 36, indicating final resistances in function of relative density of sand for buried foundations, were replotted in Fig. 89 together with similar curves for driven piles drawn with data from Figs. 52 and 53. It is seen that the average values of final resistances for large, 16- and 18-inch diameter piles (shown by black dots) agree exceptionally well with curves obtained by tests on smaller, 4-inch piles. Analogous curves drawn by using data from IRABA tests (40) seem to also follow very well the same trend. The conclusion is that, beyond a critical depth, varying from about 10 pile diameters in very loose sand to about 20 pile diameters in very dense sand, both point and skin resistances reach final values which appear to be functions of relative density of sand and method of placement of the piles only.

It must be emphasized that the preceding conclusion applies to dry or submerged sands only and that the effect of negative pore-water stresses (capillary cohesion) in moist sand must be taken into account separately. The significance of this effect can be seen nicely in the same Fig. 89, where the
Figure 89. Final Point and Skin Resistances as Functions of Relative Density of Sand.
point resistance from field tests in moist sand, also marked by a black dot, is well off the line of resistances for dry or submerged sand.

It will be interesting to complete this curve with data coming from other investigations under controlled conditions. However, even in its present form, this figure supports the soundness of the principle of static cone penetration testing for predetermination of pile lengths in sand. This statement is made in full awareness of the existence of scale effects in deep foundations which may become significant not only in the case of very large piers and caissons, but also whenever a layer of sand is first reached (26, 27, 55, 56).

Conclusions

(1) Field tests with large (16- and 18-in. diameter) instrumented piles confirm most of the major findings of previous experiments with smaller, (2 to 7 in. diameter) piles. Point and skin resistances increase approximately linearly with depth over a limited zone, not exceeding ten pile diameters, reaching nearly constant final values beyond depths of approximately twenty pile diameters. These final values appear to be functions of relative density of sand only.

(2) While scale effects can be detected in bearing capacity factors and skin pressure coefficients, the ratio of point and skin resistances of a pile appears to be independent of pile size.

(3) An excellent correlation can be established between point and skin resistances measured by static cone penetration tests and actual pile resistances. However, the proposed empirical relationships (26) (27) between standard penetration test blow count N and static penetration resistances
show significant discrepancies. At the same time, it appears that the blow count $N$ can be related, at least in very dense sand, directly to the existing overburden pressure.

(4) The distribution of friction along pile shafts is generally parabolic. There appears to be concentration of friction toward the upper portion of the pile in the case of shorter piles, and toward the pile tip in the case of longer piles. The overall participation of pile shaft in carrying the total pile load is relatively greater during the early stages of the loading.

(5) Ultimate loads along the pile shaft appear to be the same in tension as in compression. Since this observation does not agree with previous findings on small-scale pile models, it is suspected that the scale effects affecting pile behavior in tension may be very pronounced.

(6) A simple empirical formula (31) relating pile settlement under working loads to the ultimate point resistance of the same pile has been proposed. Also an improved empirical relationship (35) between the modulus of deformation of sand and static cone point resistance has been suggested for evaluation of settlements of shallow foundations in sand.
CHAPTER V
EXPERIMENTS WITH PILE GROUPS IN SAND

Introduction

The discussion of the preceding chapters was mainly concerned with behavior of a single deep foundation, such as a pier or an isolated pile. However, piles are normally made in groups, spaced only a few diameters apart, center to center. Most frequently such groups are unified by concrete caps or cross-beams that may or may not be in contact with soil. Since the spacing of piles is usually predetermined by practical and economical considerations, the principal problems faced in design of pile groups remain the same as those for individual foundations, namely:

1) to determine the ultimate load of the group, \( \overline{Q}_o \);

2) to determine the settlement of the group, \( \overline{w} \) under a working load \( \overline{Q} \).

It is well known that the ultimate load of the group is generally different from the sum of ultimate loads of individual piles \( \Sigma q_o \). The factor

\[
\eta = \frac{\overline{Q}_o}{\Sigma q_o}
\]  

(37)

is called group efficiency. It is believed that this factor depends on parameters such as soil types in the profile, pile spacing and relative length method of group construction.

It is also known that the settlement of the group, \( \overline{w} \), is normally larger than the settlement of a single pile at comparable working load. Thus, it may be written

\[
\overline{w} = \xi w
\]  

(38)
Factor $\zeta$ may be called group settlement factor. It depends on a number of still unclarified parameters, the most important of which appear to be again the soil profile, size and shape of the group, and the method of construction.

There is at present no acceptable rational theory of bearing capacity of pile groups. A number of so-called "efficiency formulas" for group bearing capacity, such as Feld rule or Converse-Labarre formula have been proposed, largely on an empirical basis (for a comprehensive review of these formulas, reference (21) may be consulted). All give efficiency factors smaller than unity.

Terzaghi and Peck (49) suggested that the bearing capacity of a pile group cannot be greater than that of a block foundation defined by the exterior perimeter of the group. Meyerhof (11) generalized this idea, suggesting that the bearing capacity of any pile group could be computed by adding the skin resistance along the outer perimeter of the group and the resistance of the imaginary base defined by that perimeter. In this manner efficiency factors greater than unity can be obtained for pile groups in sand.

There exists no special theory dealing with analysis of settlements of pile groups. However, analyses similar to those made for shallow foundations have been made for years to determine consolidation settlements of deep compressible strata underlying pile groups. Such analyses of necessity include some assumptions on load transmission from the group to the soil, as well as on stress distribution in the soil mass.

There have been only a few attempts to analyze immediate settlements of pile groups. Based on experimental results published by Peagin (58) and Vargas (59), Skempton, Yassin and Gibson (15) proposed for driven piles in sand an empirical curve for group settlement factor $\zeta$. 

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Past Experiments with Pile Groups in Sand

For a number of years the results of investigations by Press in Germany (60) have been the chief source of information on pile groups. In these investigations tests were made on groups of two to eight driven and bored piles in natural, medium grained, moist, dense sand. The diameters of driven piles were 5 and 6 in.; their lengths, respectively, 6 and 10 ft. (D/B = 15 and 20). The diameters of bored piles, tested in groups of two, were 16 in; their length was 23 ft. (D/B = 17). Evaluations of data from these investigations indicate a difference in behavior of large, bored piles, as compared to that of small, driven piles. The efficiencies of large bored piles appeared to be less than unity; at pile spacings of three diameters they averaged only about 0.60. On the other hand, efficiencies of smaller, driven piles were all larger than unity.

For two 5-in. diameter piles, the maximum efficiency was about 1.8 at spacings of 1.3 diameters; it dropped to about 1.2 at spacings of three diameters. For two 6-in. diameter (longer) piles the maximum efficiency was about 1.5 at spacings of two diameters; it dropped to about 1.3 at spacings of three diameters. The largest group of eight 5-in. piles at spacings of 4.3 diameters in somewhat looser sand indicated an efficiency of about 1.5.

Cambefort (61) tested groups of two to seven steel piles, 2-in. in diameter and 100 inches long. These long piles were driven into natural ground consisting of about 20 in. of humus, followed by 40 in. of stiff clay and 40 in. of fine sand underlain by gravel. Because of the non-homogeneity of the soil profile the results of this investigation cannot be compared directly with others. It is, however, interesting to mention that these results indicate maximum efficiencies of about 1.6 at spacings of three pile diameters.

Kézdi (62) reports on tests with groups of four concrete piles, 4 in.
square and 80 in. long \(D/B = 20\). His piles were driven in a row or as a square group at spacings varying from two to six pile diameters. The natural soil profiles consisted of fairly homogeneous, moist fine sand. This investigation indicated in loose sand a peak efficiency of about 2.0 at spacings of two pile diameters. However, in dense sand a peak efficiency of about 1.3 at spacings of three pile diameters was reported.

Similar or even higher efficiencies have been reported from investigations with small nail-size piles of 1/2 to 1-inch in diameter \((63)(64)(65)\). However, the results of such investigations have, in our opinion, only a limited meaning since it is virtually impossible to build sand models of homogeneity needed to obtain reliable quantitative data at this scale \((56)\).

**Testing Program**

In view of the limited information available about the behavior of pile groups under controlled conditions, it was decided to incorporate in the testing program of this research project a phase devoted to comprehensive experiments on large-scale models of pile groups in sand. The program included testing of groups of four and nine piles in a symmetrical square array. The piles were placed by jacking at spacings ranging from two to six pile diameters (center-to-center) into artificial deposits of dry sand in two soil situations:

a) homogeneous, medium dense mass (relative density \(D_r\) about 0.65).

b) a two-layer mass consisting of an upper stratum of very loose sand \((D_r \approx 0.20)\) underlain by a stratum of dense sand \((D_r \approx 0.80)\).

The two test series dealing with described soil situations were named, consecutively P- and Q-series. In addition to these, one test was performed in submerged, medium dense sand (S-series). The main features of all the performed tests are presented in Table 21.
Table 21. Summary of Pile Group Loading Test Results

All piles φ 4 in., 60 in. long, closed aluminum pipe

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Number of Piles in the Group</th>
<th>Pile Spacing c-c</th>
<th>Soil Profile</th>
<th>Dry Unit Weight of Soil</th>
<th>Ultimate Load per Pile Total</th>
<th>Ultimate Load Point</th>
<th>Ultimate Load Displacement</th>
<th>Ultimate Point Deflection at Failure</th>
<th>Surface Deflection at Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>P-12</td>
<td>1</td>
<td>-</td>
<td>dry, medium dense sand</td>
<td>92.7 lb./ft.³</td>
<td>3,050 lb.</td>
<td>2,320 lb.</td>
<td>-</td>
<td>0.30 in.</td>
<td>0.29 in.</td>
</tr>
<tr>
<td>P-13</td>
<td>1</td>
<td>-</td>
<td>medium dense sand</td>
<td>95.0 lb./ft.³</td>
<td>4,960 lb.</td>
<td>4,040 lb.</td>
<td>-</td>
<td>0.43 in.</td>
<td>0.40 in.</td>
</tr>
<tr>
<td>P-14</td>
<td>8</td>
<td>4</td>
<td>dry, medium dense sand</td>
<td>93.8 lb./ft.³</td>
<td>3,610 lb.</td>
<td>2,780 lb.</td>
<td>-</td>
<td>0.35 in.</td>
<td>0.33 in.</td>
</tr>
<tr>
<td>P-41</td>
<td>12</td>
<td>4</td>
<td>dry, medium dense sand</td>
<td>96.5 lb./ft.³</td>
<td>10,050 lb.</td>
<td>7,945 lb.</td>
<td>-</td>
<td>0.52 in.</td>
<td>0.47 in.</td>
</tr>
<tr>
<td>P-42</td>
<td>16</td>
<td>4</td>
<td>dry, medium dense sand</td>
<td>94.2 lb./ft.³</td>
<td>5,260 lb.</td>
<td>3,850 lb.</td>
<td>5,440 lb.</td>
<td>0.41 in.</td>
<td>0.39 in.</td>
</tr>
<tr>
<td>P-43</td>
<td>24</td>
<td>4</td>
<td>dry, medium dense sand</td>
<td>93.8 lb./ft.³</td>
<td>4,915 lb.</td>
<td>2,640 lb.</td>
<td>5,450 lb.</td>
<td>0.43 in.</td>
<td>0.41 in.</td>
</tr>
<tr>
<td>P-44</td>
<td>8</td>
<td>4</td>
<td>dry, medium dense sand</td>
<td>94.3 lb./ft.³</td>
<td>5,230 lb.</td>
<td>2,570 lb.</td>
<td>4,300 lb.</td>
<td>0.41 in.</td>
<td>0.39 in.</td>
</tr>
<tr>
<td>P-45</td>
<td>8</td>
<td>4</td>
<td>dry, medium dense sand</td>
<td>92.1 lb./ft.³</td>
<td>3,520 lb.</td>
<td>1,895 lb.</td>
<td>2,240 lb.</td>
<td>0.35 in.</td>
<td>0.34 in.</td>
</tr>
<tr>
<td>P-46</td>
<td>8</td>
<td>4</td>
<td>dry, medium dense sand</td>
<td>95.3 lb./ft.³</td>
<td>5,140 lb.</td>
<td>3,640 lb.</td>
<td>1,140 lb.</td>
<td>0.35 in.</td>
<td>0.33 in.</td>
</tr>
<tr>
<td>P-91</td>
<td>9</td>
<td>8</td>
<td>dry, medium dense sand</td>
<td>91.8 lb./ft.³</td>
<td>3,050 lb.</td>
<td>1,980 lb.</td>
<td>2,350 lb.</td>
<td>0.40 in.</td>
<td>0.39 in.</td>
</tr>
<tr>
<td>P-92</td>
<td>9</td>
<td>8</td>
<td>dry, medium dense sand</td>
<td>94.5 lb./ft.³</td>
<td>5,830 lb.</td>
<td>3,640 lb.</td>
<td>5,780 lb.</td>
<td>0.44 in.</td>
<td>0.41 in.</td>
</tr>
<tr>
<td>P-93</td>
<td>9</td>
<td>8</td>
<td>dry, medium dense sand</td>
<td>95.2 lb./ft.³</td>
<td>6,040 lb.</td>
<td>4,490 lb.</td>
<td>640 lb.</td>
<td>0.45 in.</td>
<td>0.42 in.</td>
</tr>
<tr>
<td>Q-11</td>
<td>1</td>
<td>-</td>
<td>dry, loose sand (48&quot;) + dry dense sand</td>
<td>93.2 lb./ft.³</td>
<td>8,730 lb.</td>
<td>8,010 lb.</td>
<td>-</td>
<td>0.50 in.</td>
<td>0.45 in.</td>
</tr>
<tr>
<td>Q-41</td>
<td>4</td>
<td>8</td>
<td>dry, medium dense sand</td>
<td>97.8 lb./ft.³</td>
<td>9,400 lb.</td>
<td>8,400 lb.</td>
<td>250 lb.</td>
<td>0.57 in.</td>
<td>0.52 in.</td>
</tr>
<tr>
<td>S-11</td>
<td>1</td>
<td>-</td>
<td>dry, medium dense sand</td>
<td>90.8 lb./ft.³</td>
<td>2,460 lb.</td>
<td>1,990 lb.</td>
<td>-</td>
<td>0.41 in.</td>
<td>0.40 in.</td>
</tr>
</tbody>
</table>
Testing Facility, Equipment and Materials

All the tests have been performed in the laboratory testing facility described earlier in Chapter II. The previously described cylindrical testing pit, 8 ft. 4 in. in diameter and 22 ft. deep has been used to build sand models under highly controlled conditions. The techniques used have been described in detail in Chapter II. Loose and medium dense strata have been built by pouring sand from a distribution pan from a fixed height over the horizontal surface of the model. Dense sand strata have been built by surface vibration of 4-in. thick sand layers brought into the pit by pouring from a 30-in. height. The variation of dry density of deposits so made was checked by cone-penetrometer soundings for which a 1/2-in. micropenetrometer, described earlier, was used. Penetration tests were made both prior and after the test, thus enabling observations of density changes during the test.

All tests of series P and Q have been made with air-dry sand from Chattahoochee River, the properties of which have been described in detail in the earlier text. Tests of series S have been made in a sand mass placed at the desired density in air-dry condition, and subsequently saturated by slow addition of water through the 12-in. sump built for this purpose (see Fig. 5) until a stationary water table at sand surface was established.

The piles used in all tests have been made of structural aluminum tubes 4.00 in. in diameter and 0.05 in. thick. All piles had embedded lengths of 60 in. or the D/B ratios of 15. The pile tips were closed by flat end plates covered with sandpaper to assure perfect roughness of the base. Each pile was equipped with two strain-gauge bridges placed in immediate vicinity of the top and the bottom of the pile to register the total and point loads in the pile. These bridges were similar in arrangement to those used with large piles in Phase III and described
in the preceding Chapter IV (Fig. 74). They consisted of four active Budd
gauges, type 141-C12 connected in series so as to cancel strains resulting from
possible bending, and accompanied by four temperature compensating gauges.

Piles have been forced into the soil in entire groups by jacking. A
photograph of the arrangement used during this operation in the four-pile-
group (P-4) series can be seen in Fig. 90. The forcing load from the 200-ton
hydraulic jack was transmitted through an electronic proving ring to stiff steel
plate, which served as a cap to distribute the load to the individual piles.
The pile heads were protected by heavy top plates with cylindrical collars
which were fitted to the pile walls from inside as well as from the outside
to prevent local plastic buckling. Not shown in this figure are the steel
templates that were used during the early stages of forcing to insure proper
spacing and alignment. These can be seen in Fig. 91, which shows the arrange-
ment used in the nine-pile-group (P-9) and subsequent Q series. In these series
the concrete cap was cast on a scaffolding over the pile heads at the outset.
High early strength cement (Type III) being used for the concrete mix, the cap
served after two days to transmit loads onto pile heads during the forcing
operation. This proved to be a far superior method of forcing the groups into
soil that eliminated most of the problems encountered in using a steel plate as
load distributing element.

Loading of models was performed, as previously, by means of hydraulic jacks
of appropriate capacity (up to 200 tons). Load measurements were made by using
a corresponding set of proving rings and electronic load cells. Displacement
and sand surface deflection measurements were made by ordinary micrometer dial
gauges. The axial loads in the pile shaft were measured, as described in one
of the preceding paragraphs, by means of Budd 141-C12 strain-gauge bridges
connected through a Baldwin 20-channel switching and balancing unit to a Baldwin
Figure 90. A Four-pile Group during Placing Operation.
Figure 91. A Nine-pile Group during Placing Operation.
Type N strain indicator.

**Test Procedure**

Following the preparation of sand model for a test, a cone penetration test was made, usually in the center of the pit to check the model density prior to placing of pile groups. The placing of groups into soil was achieved by static pressure of a hydraulic jack applied in increments corresponding to 2-in. of pile penetration in about eight minute intervals. Following each increment of pressure all strain, load and displacement gauges were read, thus securing complete information on load-displacement relationships for both point and skin of each pile, as well as for the entire group. Sand surface phenomena were observed during forcing, however, detailed measurements of surface topography were made prior to and immediately after forcing the group into the soil.

After the group had been properly placed (and the concrete cap cast in series P-4) surface deflection gauges were added and the assembly was ready for the actual test at planned depth. A photograph of a test at this stage is shown in Fig. 92.

The procedure followed during the actual loading tests was very similar to that practiced in previous phases of the project. The load was brought in increments of about 1/20 of the estimated failure load. After each load increments all strain, load and displacement gauges were read always in the same order. The average loading rates were about 0.01 in/min. up to a total displacement of 2-in. Beyond that each test was continued at a three times faster rate until a total displacement of 6-in. was achieved. Unloading and reloading was done usually in two increments at displacements between 1 and 2-in.

Following the loading tests detailed measurements of deformed surface topography and three additional cone penetration tests were made. The concrete cap
Figure 92. General View of Arrangements for a Loading Test.
was then broken and removed, thus freeing the piles to be removed, inspected, repaired and recalibrated for the next test. (A number of piles were damaged in the beginning to the extent that they had to be replaced. However, after improving the pile top collars and particularly after deciding to cast the concrete caps prior to placing of pile groups, all piles were found to be in excellent condition for repeated use.) Finally, the sand was removed from the test pit, making it ready for the subsequent test.

Test Results

Load-settlement diagrams of all the loading tests performed are presented in Figs. 93 through 102. Full lines in these figures show total loads transmitted by the groups or individual piles, heavy dashed lines indicate loads transmitted by pile points only, whereas light dashed lines indicate loads transmitted by the entire groups, including caps. Ultimate loads, defined as in the preceding phases of this investigation as loads at which displacement rates first reach their maximum, are marked in all the figures by black dots. The average sand densities for all the tests performed are given in figure legends.

Characteristic load-displacement diagrams recorded during forcing of piles and pile groups into soil are given in Figs. 103 through 109. Black dots in these figures indicate the ultimate loads obtained in actual load tests with the same piles or groups. The average sand densities are also shown in figure legends.

Characteristic surface deflection-versus-pile displacement diagrams recorded during loading tests are shown in Figs. 110 through 113. The points shown are average readings of two micrometer dial gauges placed symmetrically with respect to the pile or pile group center. The exact positions of these gauges are shown in the legend. Black dots indicate, as before, points corresponding to the ultimate loads.
Figure 93. Load-displacement Diagrams, Single Piles.
Figure 94. Load-displacement Diagrams, Tests P-41, P-45, P-46.
Figure 95. Load-displacement Diagrams, Test P-42.
Figure 96. Load-displacement Diagrams, Test P-43.
Figure 97. Load-displacement Diagrams, Test P-44.
Figure 98. Load-displacement Diagrams, Tests P-91, P-93.
Figure 99. Load-displacement Diagrams, Test P-92.
Figure 100. Load-displacement Diagrams, Test Q-11.
Figure 101. Load-displacement Diagrams, Test Q-41.
Figure 102. Load-displacement Diagrams, Test S-11.
Figure 103. Penetration Diagrams, Single Piles.
Figure 104. Penetration Diagrams, Test P-46.
Figure 105. Penetration Diagrams, Test P-42.
Figure 106. Penetration Diagrams, Test P-93.
Figure 107. Penetration Diagrams, Test Q-11.
PLACING A FOUR-PILE GROUP IN A TWO-LAYER MASS

TEST Q-41
¢4 IN PILES D = 60 IN
PILE SPACING 8 IN C-C
48 IN OF LOOSE SAND (88.7 LB/FT³)
UNDERLAIN BY DENSE SAND
DRY UNIT WEIGHT 97.8 LB/FT³

Figure 108. Penetration Diagrams, Test Q-41.
Figure 109. Penetration Diagrams, Test S-11.
Figure 110. Typical Surface Deflection Diagrams, Single Pile.
Figure 111. Typical Surface Deflection Diagrams, Four-pile Group.
Figure 112. Typical Surface Deflection Diagrams, Nine-pile Group.
Figure 113. Surface Deflection Diagrams, Four-pile Group in Layered Soil.
Significant numerical results of all the tests performed are assembled in Table 21 and discussed in the following paragraphs under appropriate sub-headings.

**Ultimate Loads**

The ultimate point and skin loads obtained in single-pile tests are plotted in Fig. 114 versus the dry unit weight of sand prior to the tests. Points representing individual tests define curves shown by heavy solid lines. The corresponding curves resulting from previous tests with driven and buried piles are shown, for comparison, by light solid lines. It appears that there is little difference between the ultimate point loads of jacked and driven piles at low and intermediate densities (up to \( D_r \) of about 0.65). In this range the ultimate point loads of buried piles are almost three times lower. However, at high relative densities the ultimate point loads of jacked piles are more comparable to those of buried piles and up to 20% lower than those of driven piles. In contrast to this the skin loads of jacked piles are much lower than those of driven piles, the difference generally decreasing with increased sand density. They are consistently about 35 to 40% higher than those of buried piles.

**Group Efficiencies**

Ultimate loads of pile groups are compared to those of single piles in Table 22. Since the sand densities varied from test to test, curves from Fig. 114 were used to find ultimate loads of single piles at exactly the desired densities for comparison with groups. The efficiencies were then computed for point and skin loads separately (Table 22, columns 13 and 14), as well as for total pile loads (column 12) and the entire pile groups with caps (column 15).

The group efficiencies so found are plotted versus pile spacing in diameters in Fig. 115. Black points and solid lines in this figure show total pile
Figure 114. Ultimate Loads of Single Piles.
# Table 22. Pile Group Efficiencies

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Number of Piles in the Group</th>
<th>Pile Spacing</th>
<th>Dry Unit Weight of Soil</th>
<th>Total Load Per Pile</th>
<th>Point Load Per Pile</th>
<th>Skin Load Per Pile</th>
<th>Cap. Contribution per Pile</th>
<th>Efficiencies</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(1)</td>
<td>(2)</td>
<td>(3)</td>
<td>(4)</td>
<td>(5)</td>
<td>(6)</td>
<td>(7)</td>
<td>(8)</td>
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<tr>
<td>P-41</td>
<td>4</td>
<td>2</td>
<td>96.5</td>
<td>10.05</td>
<td>7.80</td>
<td>7.94</td>
<td>6.23</td>
<td>2.10</td>
</tr>
<tr>
<td>P-45</td>
<td>4</td>
<td>2</td>
<td>92.1</td>
<td>3.52</td>
<td>2.74</td>
<td>1.89</td>
<td>2.14</td>
<td>1.62</td>
</tr>
<tr>
<td>P-46</td>
<td>4</td>
<td>2</td>
<td>95.3</td>
<td>5.14</td>
<td>5.29</td>
<td>3.64</td>
<td>4.20</td>
<td>1.50</td>
</tr>
<tr>
<td>P-42</td>
<td>4</td>
<td>3</td>
<td>94.2</td>
<td>5.26</td>
<td>3.96</td>
<td>3.35</td>
<td>3.10</td>
<td>1.91</td>
</tr>
<tr>
<td>P-43</td>
<td>4</td>
<td>4</td>
<td>93.8</td>
<td>4.91</td>
<td>3.64</td>
<td>2.64</td>
<td>2.85</td>
<td>2.27</td>
</tr>
<tr>
<td>P-44</td>
<td>4</td>
<td>6</td>
<td>94.3</td>
<td>5.23</td>
<td>4.06</td>
<td>2.57</td>
<td>3.18</td>
<td>2.66</td>
</tr>
<tr>
<td>P-91</td>
<td>9</td>
<td>2</td>
<td>91.8</td>
<td>3.05</td>
<td>2.52</td>
<td>1.98</td>
<td>2.02</td>
<td>1.07</td>
</tr>
<tr>
<td>P-93</td>
<td>9</td>
<td>2</td>
<td>95.2</td>
<td>6.04</td>
<td>5.17</td>
<td>4.49</td>
<td>4.10</td>
<td>1.55</td>
</tr>
<tr>
<td>P-92</td>
<td>9</td>
<td>3</td>
<td>94.5</td>
<td>5.83</td>
<td>4.26</td>
<td>3.64</td>
<td>3.35</td>
<td>2.19</td>
</tr>
<tr>
<td>Q-41</td>
<td>4</td>
<td>2</td>
<td>97.8</td>
<td>9.40</td>
<td>8.73</td>
<td>8.40</td>
<td>8.01</td>
<td>1.00</td>
</tr>
</tbody>
</table>
Figure 115. Pile Group Efficiencies.
efficiencies, whereas open points and dashed lines refer, respectively, to point load, skin load and total group load efficiencies (with cap).

It is seen in this figure that the overall efficiency of a four-pile group with cap in homogeneous, medium dense sand increases with pile spacing to a maximum of about 1.7 at spacings of three to four pile diameters, becoming somewhat lower with further increase of spacing. A sizable part of the increased bearing capacity comes from the caps. If the loads transmitted by the caps are deduced, the group efficiency drops to a maximum of about 1.3.

The average point load efficiencies for all the tests of this (P-4) series is 1.01. (In view of the range of scattering of individual test results marked in the figure at 2 diameters spacings there is probably no meaning in the apparent trend toward lower point load efficiencies at larger pile spacings.) In contrast to this the skin load efficiencies are much larger and show a definite trend of increase with pile spacing from about 1.8 at two pile diameters to a maximum of about 3 at five pile diameters and beyond.

Very similar results are indicated from tests with nine-pile groups in the soil conditions, however, these tests have been performed at spacings up to three pile diameters only. The only important difference in this test series appears to be in the cap contribution, which was relatively smaller for the nine-pile groups.

The tests of series Q on piles resting with their tips in dense sand overlain by very loose sand indicate efficiencies very close to unity.

All the results just exposed show conclusively, for the first time, that the increased bearing capacity of a pile group in sand comes primarily from the increase in skin loads. The point loads seem to be virtually unaffected by group action.
The Equivalent Pier Concept

The findings of the preceding paragraph obviously do not support the concept that pile groups in sand can be analyzed as foundation blocks or "equivalent piers" defined by the exterior perimeter of the group, no matter how small the pile spacing. The point load of the group is approximately equal to the sum of point loads of individual piles and its magnitude is significantly different from the ultimate base load of the equivalent pier.

Nevertheless, it should be noted, that according to the present test results, the equivalent pier concept appears to have some merit in considerations of ultimate skin loads. This can be seen from some results in Table 23. Data presented in column (7) of this table indicate that at spacings of two pile diameters the average skin resistance of groups of four and nine piles, computed along the group perimeter, is approximately the same, around 2.5 lb/sq. in. This is a much higher resistance than that of single jacked piles (about 1.1 lb/sq. in.) and is approximately equal to the average skin resistance of driven piles at the same initial sand density.

The Influence of Pile Caps

As evident from column (8) of Table 21, the pile caps contributed significantly to the bearing capacity, particularly in the case of smaller, four pile groups. An analysis of this contribution is made in Table 24.

It has been suggested (65) that the contribution of pile cap to the bearing capacity of a pile group results from a general shear failure under the outer rim of the cap contact surface (shown in Fig. 16 as shaded area) if the group fails as an equivalent pier. According to this same suggestion, the cap would contribute by its entire contact surface, just as a shallow foundation of the same size, if the pile spacing is great enough that piles fail individually.
Table 23. Comparison of Skin Resistance of Groups Considered as "Equivalent Piers"

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Average Dry Unit Weight (lb./ft.³)</th>
<th>Number of Piles in the Group (3)</th>
<th>Pile Spacing Diameters (4)</th>
<th>Average Ultimate Skin Load of the Group kips (5)</th>
<th>Perimeter Area of the Skin in² (6)</th>
<th>Average Perimeter Skin Resistance lb./in.² (7)</th>
</tr>
</thead>
<tbody>
<tr>
<td>P-41</td>
<td>94.6</td>
<td>4</td>
<td>2</td>
<td>6.96</td>
<td>2,674</td>
<td>2.60</td>
</tr>
<tr>
<td>P-45</td>
<td>95.3</td>
<td>4</td>
<td>3</td>
<td>7.64</td>
<td>3,634</td>
<td>2.10</td>
</tr>
<tr>
<td>P-46</td>
<td>94.2</td>
<td>4</td>
<td>4</td>
<td>9.08</td>
<td>4,594</td>
<td>1.98</td>
</tr>
<tr>
<td>P-91</td>
<td>93.5</td>
<td>9</td>
<td>2</td>
<td>11.65</td>
<td>4,594</td>
<td>2.54</td>
</tr>
<tr>
<td>P-93</td>
<td>94.5</td>
<td>9</td>
<td>3</td>
<td>19.71</td>
<td>6,514</td>
<td>3.03</td>
</tr>
<tr>
<td>P-92</td>
<td>93.8</td>
<td>1</td>
<td>-</td>
<td>8.56</td>
<td>754</td>
<td>1.14</td>
</tr>
<tr>
<td>P-12</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>P-13</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>P-14</td>
<td></td>
<td></td>
<td></td>
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<td></td>
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</table>
Table 24. Analysis of Contribution of Pile Caps

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Average Sand Density Prior to Test</th>
<th>Cap Dimensions</th>
<th>Effective Bearing Area (Outer Rim) $A_r$ (in.$^2$)</th>
<th>Outer Rim Width $b_r$ (in.)</th>
<th>Ultimate Cap Load $Q_c$ (lb.)</th>
<th>Ultimate Load Per Unit of Rim Surface $q_o = \frac{Q_c}{A_r}$ (lb./in.$^2$)</th>
<th>Bearing Capacity Factor $N_y = \frac{q_o}{\gamma b_r}$</th>
<th>Bearing Capacity Factor Corresponding to Average Sand Density</th>
</tr>
</thead>
<tbody>
<tr>
<td>P-42</td>
<td>94.2</td>
<td>20x20</td>
<td>144</td>
<td>2</td>
<td>5,440</td>
<td>37.7</td>
<td>345</td>
<td>330</td>
</tr>
<tr>
<td>P-43</td>
<td>93.8</td>
<td>24x24</td>
<td>176</td>
<td>2</td>
<td>5,450</td>
<td>31.0</td>
<td>285</td>
<td>282</td>
</tr>
<tr>
<td>P-44</td>
<td>94.3</td>
<td>32x32</td>
<td>240</td>
<td>2</td>
<td>4,300</td>
<td>17.9</td>
<td>164</td>
<td>343</td>
</tr>
<tr>
<td>P-45</td>
<td>92.1</td>
<td>15x15</td>
<td>81</td>
<td>1.5</td>
<td>2,270</td>
<td>27.7</td>
<td>346</td>
<td>158</td>
</tr>
<tr>
<td>P-46</td>
<td>95.3</td>
<td>15x15</td>
<td>81</td>
<td>1.5</td>
<td>1,140</td>
<td>14.1</td>
<td>177</td>
<td>532</td>
</tr>
<tr>
<td>P-91</td>
<td>91.8</td>
<td>23x23</td>
<td>129</td>
<td>1.5</td>
<td>2,350</td>
<td>18.2</td>
<td>228</td>
<td>147</td>
</tr>
<tr>
<td>P-92</td>
<td>94.5</td>
<td>31x31</td>
<td>177</td>
<td>1.5</td>
<td>5,780</td>
<td>32.6</td>
<td>398</td>
<td>373</td>
</tr>
<tr>
<td>P-93</td>
<td>95.2</td>
<td>23x23</td>
<td>129</td>
<td>1.5</td>
<td>640</td>
<td>5.0</td>
<td>60</td>
<td>510</td>
</tr>
<tr>
<td>Q-41</td>
<td>88.7</td>
<td>15x15</td>
<td>81</td>
<td>1.5</td>
<td>250</td>
<td>3.1</td>
<td>40</td>
<td>72</td>
</tr>
</tbody>
</table>
A quick analysis of ultimate cap loads given in column (8) of Table 21 shows that the concept of cap contribution around the rim appears to be sound. At the same time the ultimate cap loads obtained for groups with larger spacings on assumption of total cap bearing area participation are out of any proportion with the observed ones. The interesting fact is that even for these pile groups the concept of outer rim support seems to give quite good estimates of cap loads. This is particularly evident from data analyzed in Table 24 and shown graphically in Fig. 116. In this table the ultimate cap loads, reduced to loads per unit of rim contact surface defined surface bearing capacity factors \( N_Y \), which could be compared directly with those obtained earlier by numerous experiments with plates (Fig. 28). In view of the extreme sensitivity of \( N_Y \) as a function of sand density, the agreement of the two sets of \( N_Y \) values, with exception of one test (P-93), is very good.

**Distribution of the Load Among Piles**

Measurements of axial loads in individual piles during group placing as well as during loading tests made it possible to analyze load distributions among individual piles. In the case of four-pile groups the analyses show a high degree of uniformity of loads carried by individual piles, maximum deviations in these tests from the average amount typically 3 to 7% of the average pile loads.

In the case of nine-pile groups the differences were much larger, primarily because of existence of three different pile positions in the group (center, corner and edge). The typical distribution according to pile position can be seen in Fig. 117. Plotted versus vertical group displacement are average total and skin loads for the three mentioned positions. It is seen that total loads maintain an almost constant distribution ratio during the entire test. The center pile carries about 36% more load than the average, the edge piles...
Figure 116. Analysis of Contribution of Pile Caps.
Figure 117. Distribution of Load among Piles, Test P-93.
about 3% more than the average, and the corner piles about 12% less than the average. Other tests of the same series show a similar trend, with center piles carrying as much as 50% and as little as 20% more than the average.

While the total pile loads maintained a constant distribution ratio, this was not always the case with componental point and skin loads. It is evident from Fig. 117 that some redistribution of load between point and skin of the center pile was taking place during the test. Probably because of high compression created in the narrow space between that pile, the cap and surrounding edge and corner piles, the skin load contribution in the pile increased faster than point load contribution. A very similar redistribution took place in other two tests of the same series.

The distribution of individual pile loads at failure, shown in the lower right corner of Fig. 117 shows, beside the points already mentioned, that there is more difference in load between piles in the same position, than in the case of four-pile groups. Here the maximum difference from the average of the piles in the same position is about 18%. The corresponding differences in two other tests of the same series were 8% and 23%. The latter amount was, however, caused by pile tilting during group placing and should not be considered as typical.

**Surface Deflections**

Surface deflection measurements during loading tests on single piles, presented in Fig. 110, bear similarity with those observed earlier on single buried and driven piles. All the surface gauges showed a continued downward movement until failure (marked in Fig. 110 by black dots) was reached. Following failure, there was practically no increase in deflection at distances as great as seven pile diameters from pile center. Deflections still continued at much slower rate at smaller distances; however all the gauges eventually approached steady
non-increasing readings. This indicates that pile penetration phenomena, from a certain point on, were limited to a zone within the mass of sand, not affecting the surface at all. In terms of explanations offered earlier for nearly-constant point and skin resistances at greater depth this means that complete arching has been achieved after a sufficiently great pile displacement.

The surface deflections measured during group tests (Fig. 111 through 113) also showed the initial downward movement of all surface gauges. Following failure (also marked by black dots) a trend toward continued steady deflection was observed in most of the tests, at least for a while. However, as the group was pushed much further, the trend of deflections reversed to an upward direction. This was obviously caused by the action of pile caps, which caused lateral and upward movements of the adjacent sand mass very similar to those observed with surface foundations failing in general shear failure. It appears from data in Fig. 112 that this trend eventually reverses again.

Pile Settlements

As in the previous phases of this investigation, the load-settlement curves of individual piles as well as those of pile groups, shown in Figs. 5-4 through 5-13 have the characteristic shape, consisting of initial linear sections which gradually turn to final linear sections, with steadily increasing loads. The initial linear sections are of such length that they represent the expected behavior under working loads as long as the safety factors are at least equal to 2.

Similarly to the settlement analyses in the preceding chapters, the initial linear parts were numerically represented by a plane-strain deformation modulus $E' = E/l - \nu^2$, which is, under assumption of Boussinesq-type stress distribution under pile points, related to the pile diameter $B$, load $Q$, and settlement $w$ by
the expression:

\[ w = \frac{Q}{BE_f} \]  

(39)

The deformation moduli \( E' \) resulting from analysis of individual pile tests by this approach are presented in Fig. 118. The data from the present test series with jacked piles (P-1) are marked by double circles. Also shown in the same figure are all similar data obtained in the preceding phases of this investigation.

It is seen that all data resulting from tests with driven piles define a single curve, in spite of the differences in pile sizes (4 to 18 in.). The present data resulting from tests with jacked piles differ little from those obtained for driven piles, appearing to be relatively lower as the relative density of sand increases. There is, at the same time, a substantial difference between settlements of driven piles as compared to those of buried piles in sand. At low relative densities (under 60%) the settlements of driven piles are over 10 times smaller than those of buried foundations. The mentioned ratio drops to about eight at relative densities of 80%. There are no data for buried foundations at higher densities; however, it appears that the ratio may tend to an ultimate value of about 5. No data is available either for jacked piles at such high densities (the point in the figure shown in this range refers to test Q-11 where the pile penetrated into very dense sand only for about 3 diameters).

To analyze the settlements of pile groups, the initial linear parts of the load-settlement curves of group tests were expressed in terms of coefficients of axial pile reaction \( k_n \) (kips per inch) in Table 25, column 5. The analogous coefficients for single piles are given in column 6 of the same table. The
Figure 118. Modulus of Deformation under Pile Points.
Table 25. Comparison of Settlements of Pile Groups with Those of Single Piles

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Average Dry Unit Weight ( \text{lb./ft.}^3 )</th>
<th>Number of Piles in the Group</th>
<th>Pile Spacing ( e/B )</th>
<th>Coefficient of Axial Pile Reaction ( k_n ) Group kip/in.</th>
<th>Coefficient of Axial Pile Reaction ( k_n ) Single Pile kip/in.</th>
<th>Group Settlement Ratio ( \zeta )</th>
</tr>
</thead>
<tbody>
<tr>
<td>P-41</td>
<td>96.5</td>
<td>4</td>
<td>2</td>
<td>40.7</td>
<td>56.4</td>
<td>1.38 (Av. 1.61)</td>
</tr>
<tr>
<td>P-45</td>
<td>92.1</td>
<td>4</td>
<td>2</td>
<td>15.5</td>
<td>23.4</td>
<td>1.51 (Av. 1.61)</td>
</tr>
<tr>
<td>P-46</td>
<td>95.3</td>
<td>4</td>
<td>2</td>
<td>20.8</td>
<td>42.8</td>
<td>2.05 (Av. 2.07)</td>
</tr>
<tr>
<td>P-42</td>
<td>94.2</td>
<td>4</td>
<td>3</td>
<td>20.0</td>
<td>33.8</td>
<td>1.69 (Av. 1.69)</td>
</tr>
<tr>
<td>P-43</td>
<td>93.8</td>
<td>4</td>
<td>4</td>
<td>15.4</td>
<td>31.5</td>
<td>2.07 (Av. 2.07)</td>
</tr>
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<td>2.25 (Av. 2.25)</td>
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group settlement ratios \( \zeta \), as defined by expression (38), follow in column (7) of the same table. They are also plotted in Figs. 119 and 120 against the relative width \( \bar{B}/B \) of the pile group, defined as the ratio of distance between the outer piles in the group and the pile diameter (see legend in Fig. 119). Also plotted in the two figures are similar data collected from two other investigations: the already mentioned Mississippi River tests (58) and group tests reported by Berezantsev, Khristoforov and Golubkov (67). An empirical relationship proposed by Meyerhof for groups of square piles (66) is also included in Fig. 120 for comparison.

In view of the narrow range of relative width over which the data of the present investigation were assembled and wide scattering of individual test results, it is felt that an attempt to establish a definite relationship between the relative width and the group settlement ratio at this time would be premature. However, the collected data seem to suggest that the general trend of the investigated relationship can be expressed by:

\[
\zeta \approx \sqrt{\frac{\bar{B}}{B}}
\]  

(40)

Actually the individual test results indicate \( \zeta \)-values as high as the double and as low as the half of those indicated by expression (40). In view of what is known about the nature of sand deposits, including those prepared in the laboratory under controlled conditions, the scattering of this extent should not be considered unusual.

In evaluating the data presented in Fig. 120 it should be remembered that, excepting one test, all experimental data presented in that figure have been obtained with groups in medium dense sand. It is conceivable that groups in much looser or much denser deposits will show somewhat different behavior. Also,
Figure 119. Variation of Group Settlement Ratio.
Figure 120. Comparison of All Data on Group Settlements.
the group settlement ratio is very likely affected by the ratio of pile point settlement, \( w_p \), to total pile settlement \( w \), and possibly by some other factors. Further studies of this problem both in the laboratory on large models in controlled conditions and in the field on actual piles and pile groups are highly desirable.

Conclusions

(1) Experiments with large-scale models of jacked pile groups in medium dense and dense sand indicate practically no group effect on ultimate point loads. However, a significant increase of ultimate skin loads is detected. The pile caps contribute to the overall bearing capacity of the group inasmuch as they are supported by sand along their outer rim.

(2) The overall group efficiency under the conditions just described increases to a maximum of about 1.7 at spacings of three to four pile diameters, becoming somewhat lower with further increase of pile spacing. A sizable part of the increase in bearing capacity comes from pile caps. If the loads transmitted by the caps are deduced, the group efficiency drops to a maximum of about 1.3.

(3) Measurements of axial loads in individual piles of the group indicate a high degree of uniformity of ultimate loads of piles in the same position. In the case of nine-pile groups, the center pile was able to pick up from 20% to 50% more load than the average. There was little variation of total load distribution in different loading stages. Some redistribution of load between point and skin of the same pile has been detected.

(4) The settlements of pile groups in sand are larger than those of individual piles carrying the same load. While a definite relationship between
the group size and the group settlement ratio has yet to be established, presently available data suggest that this ratio increases roughly as the square root of the relative width of the pile group with respect to pile diameter. However, it is conceivable that this ratio must be affected by the relative length of the pile (or the relative magnitude of point and skin loads) and, possibly, also by the relative density of sand.
CHAPTER VI

BEARING CAPACITY OF DEEP FOUNDATIONS
IN CLAY, SILT, AND OTHER SOIL TYPES

Introduction

There were two principal reasons behind the decision to devote the investigations of the present research project primarily to deep foundations in sand. On one hand it was felt that the phenomena occurring beneath and around deep foundations in sand are the most complex and by far the least understood. At the same time, since a fair understanding of mechanical behavior of foundations in saturated clays has been achieved, it was felt that clarification of principal problems of behavior of foundations in sand would be the next natural step and a sound basis for study of foundations in intermediate soil types.

Such a study will need to be, of necessity, the object of a separate research project. The present chapter is primarily a review of existing knowledge on deep foundations in clay, silt and intermediate soil types, written for orientation of the bridge designer in his daily problems of this kind. The findings of the preceding chapters and some new ideas are interjected only inasmuch as they affect the current beliefs and design practices.

Deep Foundations in Saturated Clay

It is now generally admitted that saturated clays in undrained conditions of loading behave as frictionless materials, having an angle of (apparent) shearing resistance $\phi = 0$, and a shearing strength $s = c$ independent of newly applied stresses. According to the basic theory of bearing capacity, exposed in Chapter I of this report, the unit point and skin resistances of a deep foundation in such a material can be expressed by Eqs. (4) and (9). For a
circular or square foundation these equations can be rewritten in the form:

\[ q_0 = cN_c^* + q \]  \hspace{1cm} (4a)

\[ f_0 = c_a \]  \hspace{1cm} (9)

In a simplified manner the problem bearing capacity is reduced to that of determination of the bearing capacity factor \( N_c^* \) and the adhesion \( c_a \) along the pile shaft. In addition to this remains the usual problem of the practicing foundation engineer: how to best estimate the actual undrained shear strength \( c \) of the soil under the foundation base. In the case of deep foundations the latter is aggravated by the fact that the soil properties may be affected by operations of placing (such as pile driving).

On the basis of theory, model studies and actual field tests, the value \( N_c^* = 9 \) has been proposed (68) (10) and verified many times in different locations (69) (70) (71). However, \( N_c^* \)-values as low as 5, and as high as 24 have been reported in the literature, (72) (73) (40).

A systematic explanation of the observed discrepancies in the \( N_c^* \)-values has not yet been offered. However, the low values observed may be caused by the unusual compressibility of the montmorillonite clay in question. Also, there are indications that the high values might be attributed to deviations from purely frictionless behavior of the particular clay. Additional research is needed to verify this. In the meantime, it appears reasonable to use the proposed \( N_c^* \)-value of 9, at least for not too sensitive clays at water contents in the plastic range. The errors induced by variations of this factor will be small, since the point load of a deep foundation in such clays normally represent only a minor fraction of the total load.
It should be added that static cone penetration tests are a very reliable tool for direct determination of point resistances of deep foundations in clay in general.

The determination of adhesion along the shaft of a deep foundation in clay presents, for several reasons, a more challenging problem. The shaft load is in most instances a major portion of the total ultimate load of the foundation. The adhesion along the shaft is, however, greatly affected by remolding of the adjacent clay during the foundation placing and it varies sometimes sharply with time during the process of regeneration of that clay.

Numerous studies during the past fifteen years have been devoted just to this aspect of the bearing capacity problem (69 through 88). In all these studies the shaft adhesion \( c_a \) has been compared with the undrained shear strength \( c \) of the adjacent clay prior to foundation placing. A summary of some comparisons of this kind has been presented in Fig. 121. Different types of piles to which test data refer are shown in this figure by different symbols explained in the legend. The source of data is marked next to each point by a letter, the explanation of which can also be found in the legend.

By inspecting carefully Fig. 121 one conclusion can be accepted without reservation. Namely, most of the investigators of this problem find that the shaft adhesion is equal to the undrained shear strength (cohesion) of clay in question as long as the latter does not exceed about 0.5 ton/sq. ft. or 1,000 lb./sq. ft. However, the attempts to interpret the presented data in the range of stiffer clays by introduction of a reduction coefficient (or regeneration factor) \( \beta = c_a / c \) do not seem to be justified. The only direct conclusion that should result from inspection of the right-hand portion of the diagram presented in Fig. 121 is that there is not any direct correlation between the shaft adhesion
Figure 121. Comparison of Pile Shaft Adhesion and Undrained Shear Strength of Adjacent Soil.
and the undrained shear strength of adjacent soil, as soon as that strength exceeds 0.50 ton/sq. ft.

In support of this conclusion it should be recalled that the undrained shear strengths higher than 0.50 ton/sq. ft. most likely come from negative pore-water stresses. Field tests on the Roberta test site described in Chapter III of this report show conclusively that negative pore-water stresses do not contribute to the skin resistance of fine sands. If the failure along the skin occurs directly at the contact surface, as in the case of stiff or hard clays, it is hard to see any reason for different behavior of cohesive soils in this respect. Thus, it is suggested that the skin resistance of deep foundations in stiff or hard clays should be compared with the frictional component of their drained shear strength, and analyzed in terms of an equation such as (8).

A few arguments can immediately be added in support of this idea. Studies of skin resistance of bored piles in London clay (83) seem to indicate lower resistances for shorter piles in very much the same manner as in the case of shorter piles in sand. This would concur with assumption of an initial linear increase of $q_s$ from Eq. (8) with depth. Detailed observations at the Bagnolet test site (40) made on penetrometers 1.8 to 4.4 in. in diameter as well as on a large 12 by 17 in. caisson pile show without any doubt that there is an initial linear increase of $f_0$ with depth, very similar to that observed in frictional materials such as sands. The slope of the initial part of the $f_0$ curve indicates a $K_s \tan \delta$ value of 1.1. Assuming that the angle $\delta$ may be somewhere between 20 and 30 degrees yields quite reasonable values of $K_s$ between 3 and 2. A quick analysis of $f_0$ values measured in other investigations in stiff or hard clay also yields $K_s \tan \delta$ values that appear to be quite reasonable.
New systematic research is, of course, needed to examine all the parameters that may affect skin resistance of deep foundations in stiff and hard clays. In the meantime the use of static penetration tests to predetermine the resistances along pile shafts can be recommended as the only reliable tool for that purpose. Analyses based on assumption of equality of pile adhesion and undrained shear strength of adjacent clay should give sensible results as long as the clay in question exhibits positive pore-water stresses in undrained shear.

Deep Foundations in Silt and Other Intermediate Soil Types

There is little systematized information on the behavior of deep foundations in silt, and practically no such information on intermediate soil types such as sandy clays or clay-gravels. Experience with static cone penetrometers in silts (8) (47) confirms the belief that they respond to deep foundation loadings in a similar manner as frictional materials such as sand (85). However, their sometimes great compressibility probably has a very significant effect on bearing capacity factors and coefficients (90). According to a theoretical analysis (56), the point bearing capacity of a compressible silt can easily be only a fraction of the corresponding capacity of a relatively incompressible material, such as sand, although both soils may have the same angle of internal friction. Also, the skin resistance of a compressible silt may be significantly lower because the skin pressure coefficient $K_s$ may be much smaller in such a material. Fortunately, there is an evidence that the static cone penetration test registers well the effects of compressibility on bearing capacity, so that the measured point and skin resistances in silt from such a test can be extrapolated to full size foundations.
In partially saturated silts great care should be exercised in properly evaluating the effects of negative pore-water pressures on point-resistances measured by a penetrometer. However, in view of the findings of experiments with partially saturated sands in Phase II of this research project (see Chapter III), these pressures should not affect the skin resistances of neither penetrometers nor piles. Systematic research is urgently needed to clear this and other questions concerning the behavior of deep foundations in intermediate soil types that is indeed so little understood.
CHAPTER VII

Conclusions and Recommendations

Principal conclusions resulting from different phases of this research project, presented at the ends of the corresponding chapters, can be summarized as follows:

(1) Large plastic deformations of the soil mass adjacent to deep foundations are confined to a cylindrical region around the foundation shaft and a spherical region beneath the foundation base. The size and shape of these zones are greatly affected by the relative compressibility of the soil. The shear pattern beneath the base is essentially that of punching shear failure even in the densest materials, such as sands at relative density 1.

(2) Laboratory and field tests with buried, driven and jacked piles 2- to 18-inch in diameter in dry, moist or saturated sand indicate the same general pattern of variation of point and skin resistance with depth. At shallow depths of penetration, there is a linear variation of these resistances with depth. However, at greater depths, both resistances turn to quasi-constant final values. The relative depths at which the changes in behavior occur depend on initial density of sand and the method of pile construction, they are the largest for driven piles in very dense sand. The final resistances appear also to be functions of relative density and method of placement of piles only.

(3) The distribution in sand of skin friction along pile shafts in sand is generally parabolic. The amount of shaft displacement needed to mobilize this friction appears to be independent of pile size. In contrast to this, the amount of displacement of the foundation base needed to develop ultimate loads
appears to be proportional to the diameter of the base. Consequently, the larger the size of the foundation, the greater is the participation of the shaft during the early stages of pile loading. Also, in connection with conclusions listed under (2), the greater the relative depth of the foundation, the greater is the participation of the shaft in carrying the total foundation load.

(4) In soft to firm saturated clays the resistance along pile skin is approximately equal to the undrained shear strength of the adjacent soil. However, in stiff to hard clays there is no direct relationship between pile adhesion and the undrained shear strength of the adjacent soils. It appears that a relationship should be sought between skin resistance and drained strength of such soils.

(5) Scale effects can be detected in bearing capacity factors and skin pressure coefficients of piles in sand. However, the ratio point and skin resistances of a pile in a homogeneous soil mass appears to be independent of pile size. For piles in sand, this ratio is a function of relative density of sand and method of placement of piles. Whereas the ratio appears to be the same in dry or submerged sand, it may be significantly affected by the "capillary cohesion" of moist sand.

(6) An excellent correlation can be established between point and skin resistances measured by static cone penetration tests and actual pile resistances. However the proposed empirical relationship between standard penetration test blow count N and pile resistances may show significant discrepancies.

(7) With a proper choice of "driving constants" dynamic formulae may give reasonable estimates of ultimate loads of piles in dry sand. However, the presence of a small amount of moisture in sand alters this picture. The
discrepancies in cohesive soils are particularly pronounced because of additional effects of structural changes of the material.

(8) Limited test data seem to indicate that the ultimate shaft loads of piles in sand are the same in compression as in tension, in very much the same manner as they are in the case of piles in clay or silt-type soils. However, there appears to exist a significant scale effect in this phenomenon, since all tests with smaller pile models in the laboratory indicate lower resistances in tension.

(9) Experiments with large-scale models of jacked pile groups in medium dense sand indicate practically no group effect on ultimate point loads. The bearing capacity of the group is somewhat larger than the sum of bearing capacities of individual piles primarily because of increased skin resistance. The pile caps may also contribute to the bearing capacity of the group inasmuch as they are supported by a bearing stratum around their rim.

(10) Simple empirical relationships can be established between point settlements and ultimate point resistances of piles in sand. Thus, it is justified to use the static cone penetration resistances to predict immediate settlements of deep foundations.

(11) Settlements of pile groups in sand are larger than those of individual piles carrying the same load, roughly in proportion to the square root of the relative width of the pile group.

(12) In spite of significant progress made during recent years in understanding of bearing capacity phenomena around deep foundations there is still no satisfactory theory that could be recommended without reservation to the practicing foundation engineer for analysis of ultimate loads. Such a theory must include considerations of compressibility and volume change of the soils involved in carrying the load imposed by the foundation.
Penetration tests, and particularly the static cone penetration test still remain as the best tool available at present for predetermination of ultimate loads and settlements of deep foundations. When dealing with larger piles, piers and caissons, an effort must be made to interpret the penetration data taking into account the scale effects. High pressure triaxial tests may be necessary to properly assess these effects.

The knowledge of phenomena around pile groups in sand is still very limited. It is not advised to rely on efficiencies greater than 1 unless they can be demonstrated by full-scale tests in actual field conditions.

Very little is still known about the behavior of deep foundations in silt and other soils having mechanical properties between those of sands and saturated, soft to firm clays. Laboratory and field studies are urgently needed to fill this gap.
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Introduction

The design of a deep foundation requires a variety of skills, much experience and a considerable amount of study based on knowledge of engineering sciences. No set of simple rules and procedures similar to those developed in some phases of structural design can be expected to cover the variety of conditions and forms of instability that a deep foundation may be endangered with. The following are the guidelines that a design engineer may find useful to respect, along with other considerations, to arrive at safe and economical dimensions of his foundations.

Basic Design Criteria

Principal dimensions of a deep foundation are determined so as to satisfy two basic design criteria:

a) The nominal design load \( Q_a \) should not exceed a specified fraction \( a \) of the ultimate load \( Q_o \), so that at any time

\[
Q_a \leq \frac{Q_o}{F}
\]

where \( F \) is the nominal safety factor.

b) The settlement \( w \) of the foundation under working loads should not exceed a specified limit set by the usage and structural tolerances of the supported structure.

Definition of Ultimate Load

The ultimate load, \( Q_o \), of a deep foundation can be defined as load at which the displacement/load rate first reaches its maximum (cf. Fig. 26
and pp. 56, 96, 164). In cases where the above definition causes ambiguities in interpretation, the ultimate load of a driven pile can be taken as load corresponding to total pile settlement of 10% of the pile point diameter or pile point settlement of 8% of the pile point diameter, whichever is smaller.

The same definitions can be used to make estimates of ultimate loads of bored piles or piers. However the ultimate settlements of such foundations may be higher. For bored piles or piers on sand they can be set as high as 25% of the base diameter.

Predetermination of the Ultimate Loads

For design purposes, the ultimate loads of deep foundations can be predetermined from the expression:

\[ Q = q A + f A \]

\[ \text{where } q \text{ represents the unit base (point) resistance, } f \text{ the unit skin (shaft) resistance, } A \text{ the bearing area of the foundation point and } A \]

\[ \text{the bearing area of the foundation shaft.} \]

The unit point and shaft resistance \( q \) and \( f \) can best be estimated by means of static cone penetration tests. The prototype penetrometer built in recent years by the Georgia State Highway Department can be used with slight improvements to develop simple and meaningful exploration programs for most bridge projects across the State. Considerable savings can be expected by gradual elimination of all but a few conventional borings on a particular site, with corresponding sizable reduction in sampling and laboratory testing. Further, long range, savings can be expected from gradual reduction of the number of needed full-size load tests on piles.
after acquiring sufficient experience in correlation of ultimate loads of penetrometers and full-size piles.

The following facts should be kept in mind in interpreting the results of static cone penetration tests:

a) The penetrometer is a small jacked pile. The point resistance of driven piles is, in principle, comparable to that of jacked piles (cf. Fig. 89 and 114). However, the point resistance of bored piles or piers in sand may be significantly lower. At the same time, shaft resistance of jacked piles in sand may be only slightly higher than that of bored piles or piers and is significantly lower than that of driven piles. The mentioned differences tend to disappear in finer and more compressible cohesionless soils.

b) There are scale effects in penetration resistance particularly in cohesionless soils. The point resistance of a penetrometer or a pile depends on strength and compressibility of a zone of soil extending a few diameters around the point. The size of this zone is proportional to the pile size. A penetrometer will register all the local peaks and lows of density and strength at its scale. Since failure in soil mass occurs always through the weaker elements in the mass, the resistance of a larger pile will be affected by the average of low (rather than by average) penetrometer resistances in the corresponding zone (cf. Fig. 83). Also, as a pile first enters a layer of dense sand, its point resistance increases linearly with relative depth of penetration into the new layer (depth expressed in pile diameters). Thus, in all such situations the pile point resistance is comparable to that of a penetrometer at the corresponding relative depth (cf. Ref. 55).

c) The point resistance of a penetrometer in a cohesive soil is, in principle, equal to that of a pile under analogous conditions of loading.
However, the shaft resistances may easily be different, because of differences in degree of remolding and subsequent thixotropic strength regain in the two cases. It is, thus, advisable to evaluate shaft resistances $f_0$ of piles in cohesive soils indirectly from the undrained shear strengths, $\sigma_0$, which in turn can be deduced from measured cone point resistances. (For theoretical considerations that allow this conversion see Ch. VI.)

**Ultimate Loads of Piles in Group**

Ultimate load of a pile group is not necessarily equal to the sum of ultimate loads of individual piles. The following basic findings can be used for determination of ultimate loads of pile groups; provided the pile spacing (center-to-center) is at least 2.5 pile diameters:

a) The ultimate point load of a group of piles can be taken to be equal to the sum of ultimate point loads of individual piles.

b) The ultimate skin load of a group of piles in clay or silt cannot be greater than the sum of ultimate skin loads of individual piles multiplied with the ratio of outer perimeter of the group to the sum of perimeters of individual piles.

c) The ultimate skin load of a group of piles in sand may be greater than the sum of individual skin loads. (This is probably caused by increased compaction and lateral compression caused by driving a pile group.) However, in view of scarce information available on full-size groups in sand it is not advisable to take advantage of this increased bearing capacity without a positive (preferably experimental) proof.

d) Pile caps in firm contact with surrounding sand contribute to the bearing capacity of the group as much as a bearing surface equal to their outer rim (see Fig. 116). However, in view of the potential danger of erosion by surface water, or settlement of the overburdened sand (which
often contains lenses and thin strats of compressible soil), the use of this increased bearing capacity in design is not generally recommended.

**Choice of the Safety Factors**

The choice of the safety factors to arrive at nominal design loads should in principle depend on the character of the structure and the loads, the probability of occurrence of maximum design load, as well as on the character of the soil profile and the extent of soil exploration or degree of knowledge of soil conditions on the site.

The following minimum safety factors are recommended for deep foundations of ordinary highway bridges placed in reasonably regular soil profiles that have been thoroughly explored by static cone penetration tests:

a) For nominal design loads obtained by most unfavorable combination of primary (dead and live vertical loads) $F_s = 2.5$;

b) For nominal design loads obtained by most unfavorable combination of both primary and secondary loads (thus, including wind, braking forces, etc.) $F_s = 2.0$.

These factors should be increased to at least 3.5, respectively 3.0 in cases where only limited soil exploration has been made or the principal bearing strata are of erratic composition.

**Predetermination of Settlements**

For design purposes, the settlements of deep foundations can be predetermined from the expression

$$ w = w_s + w_p + \rho_c $$

where $w$ = total (immediate + long term) settlement of the foundation;

$w_s$ = settlement due to deformation of foundation shaft;

$w_p$ = immediate settlement of the foundation point (base);

$\rho_c$ = long term (consolidation) settlement of the foundation point.
The settlement due to deformation of the foundation shaft can be estimated by the expression:

\[ w = \left( Q + \alpha Q' \right) \frac{L}{AEB} \]

where
- \( Q \) = point load (tons)
- \( Q' \) = shaft load (tons)
- \( L \) = length of foundation shaft (feet)
- \( A \) = cross-sectional area of the foundation shaft (ft²)
- \( E \) = modulus of deformation of the foundation shaft (ton/ft²)
- \( \alpha \) = a coefficient, depending on distribution of skin friction along the shaft; for uniform distribution (soft clay) \( \alpha = 0.5 \); for piles in sand, silt or stiff clay \( \alpha = 0.6 \).

The immediate settlement of the foundation point can be estimated by using the expression

\[ w = \frac{0.36 Q}{B E} \]

where \( B \) is the diameter of the foundation base (point) and \( E \) the modulus of deformation of the soil underneath the foundation base.

With known limitations, the modulus \( E \) can be determined from triaxial tests on soil samples. However, if the soil exploration for deep foundations is based primarily on static cone penetration tests it is desirable to rely on empirical relationships between the modulus \( E \) and the point resistance \( q \). When these are introduced, the following general expression for immediate settlement of the foundation point is obtained:
where $C'_w$ is a coefficient. According to presently available information for foundations in sand (pp. 168-172) $C'_w = C_w/(1 + D^2)$, where $D$ is the relative density of sand and $C_w$ another coefficient equal to 0.04 for driven piles, 0.05 for jacked piles and 0.18 for bored piles or piers. Similar information on foundations in piles indicates $C'_w$ values between 0.03 and 0.05 for driven piles and between 0.10 and 0.12 for bored piles or piers. For saturated clays the values between 0.02 and 0.06 appear to be reasonable for most types of deep foundations. However, for the latter soil type, the long-term (consolidation) settlement is often the most important.

The estimates of long-term settlement of deep foundations should be made by approaches similar to those used in the case of shallow foundations. These require consolidation testing of samples in the laboratory. It should be mentioned that studies have been made in recent years to investigate the empirical relationships between the consolidation modulus $M = 1/m$ and the cone point resistance $q_c$. Values of $M/q_c$ varying from 3 to 6 have been cited for a number of soft to stiff clays. It would be of interest to pursue these studies for clays commonly encountered in Georgia. In the long run considerable savings could be expected in sampling and laboratory testing if relationships of this kind could be established.

**Settlements of Piles in Group**

The settlement of a group of piles is generally larger than the settlement of a single pile under equal loads per pile. This is particularly apparent in the case of pile groups in sand, where the zone of significant stress in the soil due to imposed loads increases in direct proportion with
the size of the group, while the zone of increased sand density caused by pile driving remains limited in depth to a few pile diameters under pile points. Data assembled in this investigation indicate that the ratio of group settlement to the settlement of an individual pile under the same unit load increases roughly as the square root of the relative width of the group with respect to pile diameter.

Concerning pile groups in fine-grained soils (clays and silts) it is reasonable to admit that group settlements can be evaluated by conventional approaches of settlement analysis.

Settlement Tolerance Criteria

Settlement tolerances of highway bridges should generally depend on the structural system and material of the bridge superstructure and supports, type of foundation and foundation soil as well as type of pavement and construction details of the end joints. Although most design organizations and consulting engineers have some working rules concerning "admissible settlement" for this type of structures, there are no generally accepted criteria for the simple reason that no serious research was ever recorded on this subject in the engineering literature.

An analysis using data from similar research performed on frame buildings indicates that ordinary continuous girders and frames in reasonably regular soil profiles would be safe from structural damage as long as their supports settled no more than 0.3% of their span, if the bearing stratum is a cohesionless soil, and no more than 0.5% of their span if the bearing stratum is a cohesive soil. Also, for structural and/or psychological reasons the nominal settlements of bridge supports (piers or bents) should be kept at a safe limit of 0.4% of their width (measure perpendicularly to the bridge axis).
An additional criterion may be set for nominal settlement of end supports, involving primarily the difference in settlement between the bridge and the embankment after construction of the pavement.