THESIS ON

MODEL STUDY OF PILES DRIVEN INTO SAND

SUBMITTED IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR THE DEGREE OF BACHELOR OF SCIENCE IN CIVIL ENGINEERING FROM THE ROBERT ENGINEERING SCHOOL.

ISTANBUL - TURKEY

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INTRODUCTION

Importance of Model Studies

There is no science, but engineering, where experimenting is the best and most reliable method for obtaining enlightening results and drawing fruitful conclusions.

An engineer thinks in terms of two things mainly, that is safety and economy. So far experiments in many cases have proven to be inapplicable for the main reason of their being uneconomical. But today this disadvantage of experimenting is overcome by the introduction of models. Models serve the purpose of experimenting very well and are more advantageous due to their economy, ease of handling and availability at any time.

Further more Dr. Charles Terzaghi in one of his articles published in the Engineering News Record under the title of Future Development and Problems in Soil Mechanics, states the following:

For practical application, soil mechanics must be applied in the first instance to the following objects:

(a) A theory of models must be developed, without such a theory no valid conclusion can be drawn from the results of loading on soils or from model test (as on dams) with respect to full sized structure.

(b) <u>Classification of Soils</u> must be accomplished. It should be possible to determine and express quantitativel y the relation between two apparently similar soils found in different localities.

(c) <u>Adequate Design Data</u> must be developped. The new science must guide the way toward obtaining all the required data for the economical design of structure consis t-ing of soils in contact with soils.

<u>Pile Driving</u>:

Bed rock is the ideal case for foundations of any construciton, but in many of the sites bed rock is either encountered in great depths or not at all.

Take a case like this. Suppose that test borings made after the first inspection of the building showed that the solid top layer of soil rested on a deposit of soft silt of such a thickness that there is no chance to carry the foundations down to solid ground, then the only cure is to put the foundations on piles. In order to find out how many piles is needed, it is necessary to drive a couple of test piles and determine the amount of load one individual pile could stand. To follow the traditional procedure one must observe the average penetration of the pile under the last ten blows, compute the bearing capacity of the pile by some furmulae, so far among the formulae that are applied are the Engineering News formula, Good Rich formula, Kropf formula and many others. The applications of these formulae rests on two assumptions:

1. That the bearing capasity of a pile can be computed from the effect of the impact and penetration per blow.

2. That the bearing capacity of the complete foundation is equal to the sum of the bearing capacities of the individual piles. The first assumption takes the following into account:

R = Weight of hammer

G - Weight of the pile

L - Length of the pile

F - Area of the cross section of the pile

E = Modulus of elasticity of the pile material

h = Distance of hammer drops

s = Penetration produced by one blow

m = Coefficient of elasticity of the impact, and m = I for perfectly elastic impact.

Qd = Resistance against penetration of the pile under impact

Q = Ultimate bearing capacity of the pile under static load

C = Empirical constant depending on the nature of the pile and the resistance against penetration $= \frac{Q\ d\ L}{Z\ F\ E}$

The theory of semi elastic impact leads to the following equation:

$$Qd = \frac{F}{L} E \left(-s \pm \sqrt{S^2 + \frac{2Rh}{2} \cdot \frac{Rm^2}{RG} \cdot \frac{GL}{F}}\right)$$

The value, m, is usually assumed equal to 0.5 (semi elastic impact) For m = o, the equation becomes Redtenbacher's formula, which is quite extensively used in Europe.

On the other hand, if perfect elastic impact is assumed m = I then the above equation becomes

$$Rh = Q_d s \frac{1}{2} \frac{Q_d}{F} \frac{L}{E} = Q_d \left(S + \frac{1}{2} \frac{Q_d}{F} \frac{L}{E} \right)$$

$$Qd = \frac{Rh}{\frac{1}{2} \cdot s \cdot \frac{Q_d}{F} \frac{L}{E}}$$

If the fact $\frac{1}{2} \frac{Q_d}{F} \frac{L}{E}$ depending on both the nature of the pile and the resistance against penetration, is disregarded and if this variable term is replaced by an empirical constant, C, independent of all these factors,

$$Q_d = \frac{Rh}{S+C}$$

which is nothing else but the absurd Engineering News Formula. These two things, namely, the nature of the pile and the resistance to penetration, must never be neglected in the bearing capacities of piles.

Purpose of the Thesis

The aim of this thesis was to construct a model apparatus for driving, loading and piling test of piles and plot curves for each of the test carried for a circular and concical pile in dry and wet sand, and compare the necessary results for all of them and then draw a certain conclusion.

Foreword

The author, here wants to emphasize the importance of undertaking an experimental thesis. Although, much harder work is necessary in accomplishing an experimental thesis, it is the best and the most beneficial type of task for anybody who wants to learn something.

The author wishes that the Administration of the Engineering School of Robert College would urge or even oblige the students to take up an experimental thesis rather than designs in their Senior year.

SOIL MECHANICS

Introductory

The majority of the complicated problems that a civil engineer faces are encountered in relation with earthwork, his largest concern. Lack of sufficient knowledge, on sand and clay, which make up most of the soils for foundations give him endless trouble.

Besides the granular theory of Coulomb-Rankine, soils, the oldest construction material had no science of stress and mechanics. Dr. Charles Terzaghi, at the closing year of the World War, then Professor of Civil Engineering at Robert College, began his soil mechanics reasearches. His first experiments on sand to discover the nature, and laws of its internal friction, and volume changes under load and moisture, then reasearches on clay, reached a stage where their results proved to be fruitful.

Most of his interesting facts are carefully explained and analyzed by a series of articles that appeared in Engineering News Record of 1925, vol. 95 under title of Principles of Soil Mechanics.

The eight series of articles that covered his work were as follows:

1. Phenomena of cohesion of clay
2. Compressive strength of clay
3. Determination of permeability of clay
4. Settlement and consolidation of clay
5. Physical differences between sand and clay
6. Elastic behavior of sand and clay
7. Friction in sand and clay
8. Future developments and problems

Classifications

As stated by Dr. Glennon Gilboy the subject of soil mechanics is divided into two main groups.

1. Soil Physics. Comprising studies of the composition and grain distribution of soils, permeability, compressibility, consolidation, compressive strength, internal friction and cohesion.

2. Soil Engineering. Including investigations on the bearing capacity and settlement of foundations, on hydraulic-fill dams, on the frost heaving of highway sub-grades, and on the lateral pressure of earth against retaining walls.

The two groups are closely related in as much as a knowledge of the behavior of the soil is considered a knowledge of the physical properties of its component parts.

PRELIMINARY

Definitions

The term soil, as may be defined by engineers, includes not only the earth mold but also the raw soil, naturally or artificially deposited, ranging from soft mud and peat to solid rock.

A soil mass consists of a network of solid particles, including interspaces of varying size, where those interspaces are filled with air, with water, or partly with air and partly with water.

The mass of a soil has a certain total volume V and a certain weight W Part of the total volume is occupied by solid matter, which has a volume V_s and a weight W_s

Specific Mass Gravity. S_m may be defined as the ratio of the total weight W to the total volume V of the mass.

$$S_m = \frac{W}{V}$$

Specific Gravity, s which must be clearly distinguished from specific mass gravity, may be defined as the ratio of the part of the total weight occupied by the solid matter to the volume of the solid matter alone.

$$s = \frac{W_s}{V_s}$$

Volume of voids, is a certain percentage of the total volume not occupied by the solid matter, and is expressed as V_v . It may be occupied by air or by water.

Porosity, p is simply the ratio of volume of voids to the total volume of the mass;

$$p = \frac{V_v}{V}$$

Void ratio, e of a mass is the ratio of volume of voids to volume of solid;

$$e = \frac{V_v}{V_s}$$

Water content. If part of the volume of voids is filled with water and the total weight of the water in the mass is W_w, then the ratio of total weight of water in the mass to the total weight occupied by the solid matter is called the water content;

$$w = \frac{W_w}{W_s}$$

General. Unless specified otherwise the specific gravity of water is taken as unity. Hence V_w in cubic centimeters will be numerically equal to W_w in grams.

A soil is said to be saturated if all the void space is filled with water, where $e = ws$.

If the void space is partially filled with water, the degree of saturation G is the ratio of the total volume of water to the volume of voids.

The volume of G varies from zero for dry soil to 100 % for saturated soil.

In figure. I the various quantities above discussed are shown diagramatically.

Determination of the specific gravity by the pyonometer method is the most reliable in determinaing the specific gravity of all soils. The necessary measurements are:

1. Weight of bottle + water $= W_1$
2. Weight of bottle + water - soil $= W_2$
3. Dry weight of soil $= W_o$

For every accurate work, the specific gravity S_o of the water corresponding to the temperature of the test is also noted.

The relations involved are as follows:

$$W_1 = W_o + \frac{W_o s_o}{s} + W_2$$

where specific gravity is expressed as:

$$s = \frac{W_o s_o}{W_1 - W_o - W_2}$$

STRUCTURE[1]

Single-grained Structure. An accumulation of equal spheres, such as a box full of billiard balls, is the ideal prototype of single-grained structure, observed in materials in which there is little or no tendency for the grains to adhere to one another. Such materials are termed cohesionless, and are represented in soils by sands and gravels.

The porosity of a granular mass may vary within rather wide limits, depending upon the manner in which the grains are grouped together. As an illustration, circular disks arranged as in* Fig. 2a occupy a smaller proportion of the bounding area than when arranged as in Fig.* 2b. Theoretical solution of the analogous problem in space leads to the result that the porosity of an accumulation of equal spheres ranges from 47.6% to 26%.

Granular soils, such as sands are not accumulations of equal spheres. Nevertheless, the limiting values of porosity are remarkably close to those found theoretically. Such soils seldom have porosities above 50% or below 23%, corres-

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*All figures may be found at the end of this paper.

1. Notes On Soil Mechanics. By G. Gilboy 1930. Cambridge, Mass. Pages 3 - 6.

ponding to void-ratios of 1 and 0.3, respectively. The actual values for any given granular soil may be found experimentally; the upper by pouring the soil loosely into a container and determining its weight and the volume it occupies, the lower by tamping and shaking the soil until it has reached its minimum volume, and measuring weight and volume as before.

The density of a granular soil in any given state of compactness is defined with reference to the maximum, and minimum void-ratios the soil may assume. Consider a sample of sand which is in neither its densest nor loosest condition. Let void-ratio be e. Let the upper limiting voids ratio of the same sand, determined as outlined above, be e_u and the lower e_l. Then the <u>relative density</u> R of the sample is defined as

$$R = \frac{e_u - e}{e_u - e_l}$$

It is evident that a sand in its loosest state has a relative density zero, in its densest state unity.

<u>Honeycomb Structure</u>. <u>True Cohesion</u>. A solid body may be considered as a network of molecules held in definite positions by mutual attraction. When two such bodies come into contact with one another, the molecules of one body at the point of contact exert an attraction upon those of the other body. The forces involved are very small and their effects are negligible when the bodies themselves are of a higher

order of magnitude than that of molecules. Sand grains, for instance, are so large that the effect of intermolecular attraction at points of contact is quite negligible. The grains behave as though there were no force binding them together, hence the term cohesionless.

As the grains become finer, however, the effect of intermolecular attraction becomes more and more noticeable. Consider a sediment being formed at the bottom of a lake; let Fig. 3 represent a portion of the top surface of the sediment. Grain A is falling through the water, as shown at (a). At (b) the grain has come into contact with another grain B previously deposited. At the instant of contact, intermolecular attraction is set up at the point of contact between A and B. If the grains are relatively large, this attraction is insignificant, and grain A rolls into the position shown in (c), forming a single-grained structure.

Now consider another sediment, shown in Fig. 4, composed of grains so small that intermolecular attraction is appreciable in proportion to the weight of a grain. The grain A settling through the water as at (a) comes into contact with grain B, shown at (b). The tendency to overturn exists, but is restrained by intermolecular attraction, acting like a patch of glue at the point of contact. Grain A therefore remains in position. In this manner a porous structure, containing voids of a higher order of magnitude than the grains, is built up as indicated at (c). This is known as <u>honeycomb structure</u>, and is found in fine silts and clays.

The intermolecular attraction between grains at the point of contact is known as true cohesion.

Flocculent Structure. Colloids. Solid particles suspended in a liquid tend to settle to the bottom under the influence of gravity. When the particles are very small, however, suspensions exist in which the particles never settle out. These are known as colloidal suspensions or sols. The liquid is termed the continuous phase, the suspended particles the disperse phase.

The forces acting in all the suspensions are the same, but on account of the small size of colloidal particles, the effects are markedly different. The molecules of the liquid, being in constant vibration, strike against the particles of the disperse phase. If the particles are large in proportion to the size of a molecule, the impacts on any particle will, by the law of probability, be balanced. If the particles are small, approaching the size of a molecule, impacts will not be balanced; the vibrating molecules strike with sufficient force to produce motion of the particle. The motion is an irregular darting back and forth, which can be observed under the microscope. It is known as Brownian movement from the name of the person who first noted its existence.

Brownian movement is sufficient to counteract the tendency of an individual particle to settle under the force of gravity, in as much as the particle might shoot upward in a fraction of a second a distance through which it would take a minute to settle. However, the movement is not in itself sufficient to prevent sedimentation of a large group of particles, because if two particles should collide, true cohesion would come into play and the two particles would adhere. Others would inevitably join, finally forming an aggregate so large that molecular impacts would balance, Brownian movement would cease, and the aggregate would settle to the bottom.

This action is prevented in a colloidal suspension by the fact that the particles do not collide. They all carry a small but definite electric charge, of the same sign, while the liquid is charged with electricity of opposite sign. A discussion of the nature and character of this charge is beyond the scope of this work. It is sufficient to know that it exists, and serves, by causing mutual repulsion, to keep the individual particles from colliding. Hence Brownian movement persists, and the solid remains indefinitely in suspension.

The nature of the electric charge may be determined by immersing an anode and a cathode in the suspension. The particles will move toward the pole of opposite sign. This phenomenon is called <u>cataphoresis</u>.

If a small quantity of electrolyte is added to the suspension, the molecules of the electrolyte ionize. Suppose the disperse phase is charged negatively. The positive ions are attracted to the negatively charged particles, and neutralize the charge. The force preventing collision of the particles no longer exists. The particles collide, stick together, and form flocs, each floc having a honeycomb structure of solid particles. When the flocs become large enough so that Brownian movement ceases, they settle to the bottom under the force of gravity.

The flocs are still small enough so that when they form a sediment they will be arranged in honeycomb structure. Thus the sediment will have a honeycomb structure of second order, formed by flocs grouped around voids larger than the flocs themselves, each floc formed by grains grouped around voids larger than the grains themselves. This is termed flocculent structure, and is illustrated in Fig. 5 The sediment as a whole is called a gel.

Effect of Shape of Grains.

So far the structure of soil masses has been discussed on the implicit assumption that the individual grains were more or less spherical in shape. Many soils, however, contain mineral constituents with flakey grains, such as mica.

These grains have a tremendous influence upon the density of the material, and upon other important properties which will be considered later. It has been noted, for example, that the maximum void-ratio of clean sand is about 1. If a quantity of mica, coarse enough so that there can be no structural effect due to true cohesion, is mixed with the sand, the void-ratio may be made considerably greater than 1. Increasing the percentage of mica may raise the value to 10 or more. The influence of grain shape is too often neglected in analyses of soil behavior. It should be kept in mind, inasmuch as it is of equal importance with the influence of size.

<u>MECHANICAL ANALYSIS</u>

The oldest and most apparent method of analyzing soils is by separating a sample of any given soil into fractions on the basis of grain size. At the beginning it was believed that by determining the proportions of various constituents of a sample, the properties of the soil could generally be determined quite satisfactorily; however, many evidences proved that the situation was not so simple.

One must not depend on the results obtained by a mechanical analysis due to the following reasons:

1. As soon as a sample is broken into its various constituents the <u>original structure</u> of the soil is destroyed.

2. Mechanical analysis fails to determine the shape of the grains, which play an important part in the make up of different soils.

<u>Distribution diagram</u>. The method of expressing the results of a mechanical analysis is usually represented by a curve showing the weight of all grains smaller or coarser than any given diameter, expressed as a percentage of the total weight of the sample. A typical curve is shown in figure (3). Point A, indicates that 40% of the sample is made up of grains smaller than 0.1 mm. in diameter. Mechanical

analysis' objective is obtained after the curve is plotted.

<u>Effective Size and Uniformity Coefficient</u>. Hazen while experimenting with filter sands found out two constants from the distribution diagram, or mechanical analysis curve; first being effective size and the second uniformity coefficient.

1. <u>The effective size</u>, which is defined as a diameter such that the aggregate weight of all smaller grains is 10% of the total weight of the sample. In figure (3) the effective size is d_1.
2. <u>Uniformity coefficent</u>. Again from the figure (3) another diameter d_2 is determined such that the aggregate weight of all smaller grains is 60% of the total weight; and the ratio of this diameter, d_2 , to the effective size is defined as the uniformity coefficient.

The uniformity coefficient shows the range in the grain size. A uniformity coefficient of <u>one</u> would represent a soil in which the grains are all of the same size. An increasing uniformity coefficient corresponds to an increasing range in the grain size which makes up that particular sample.

For coarser materials, such as sand, grave, mechanical analysis is carried in a very simple way by passing the sample through a series of screens with known diameter of openings. For soils composed of grains roughly 0.2 and 0.0002 mm. in diameter, sedimentation methods is used.

These methods are based on the principles that the rate at which a sphere sinks in a liquid, indefinite in extent is directly proportional to the square of the diameter of the sphere.

Stocks' Law of Sedimentation, is the most convenient method for the mechanical analysis of very fine grained soils. The law states that, if a sphere is allowed to fall through a liquid indefinite in extent, at first its velocity will rapidly increase under the acceleration of gravity, until a definite terminal velocity is reached. The theoretical terminal velocity is then expressed:

$$V_t = \frac{2}{g} \quad \frac{s-s_o}{n} \quad \left(\frac{d}{2}\right)^2$$

where "s" and "s_o" are the specific gravities of sphere and liquid respectively, "n" is the viscosity of the liquid and d the diameter of the sphere.

Stocks' Law is inaccurate due to the following reasons:

1. The body of water is not indefinite so that terminal velocity is practically not reached.
2. The particles are not spherical.
3. An average specific gravity is used which may differ with the specific gravity of any given particle in the sample
4. The motion of any particular grain is influenced by the presence of other grains present in the container.

There are several other methods of conducting the mechanical analysis by sedimentation such as successive sedimentation, simultaneous sedimentation and Wiegner method which are not discussed here for they are not themost convenient methods for mechanical analysis of very fine grained soils.

MECHANICAL ANALYSIS OF "GOKSU" SAND

In order to have a clue to the effect of grain size on the penetration, and setlements in the model study of piles driven into "Göksu" sand, a mechanical analysis of that particular sand was carried.

The object of the experiment was to determine by sieving the percentages of the various sized particles in the sample of sand. These percentages may be expressed as follows:

1. Total percentage finer than any given sieve size
2. Total percentage coarser than any given sieve size
3. Percentage finer than a given sieve, but coarser than the next sieve in the series.

In carrying over the experiment the apparatus used were as follows:

1. A.S.T.M. series (for other than concrete aggregate) No: 200, 100, 80, 50, 40, 30, 20 and 10.
2. A Balance

The material was a fine sand obtained from "Göksu" delta.

After the sand was dried under the sun, a sample weighing 500.00 grams to the nearest 0.1 of a gram and placed in the nest of the sieves, having previously arranged with the largest mesh at the top.

The nest was shaken for 20 minutes and it was found out that no appreciable quantity was passing through the mesh.

To express the mechanical analysis in accordance with the first object stated above, the material passing each sieve was weighed, by first weighing the material passing finest and adding to it the material retained on each successive sieve. To express the mechanical analysis in accordance with the second object, the material retained on each successive sieve was added. And finally to express the mechanical analysis in accordance with the third object, the material retained on each sieve was weighed seperately.

The mechanical analysis of the "Göksu" sand was carried twice and the results obtained below are the averages of the two tests. The accompanying table gives the value of sieve openings corresponding to the sieve size.

Mesh Designation U.S. Standard Sieve Number (Series	Sieve Opening In millimeters
10	2.000
20	0.840
30	0.590
40	0.420
50	0.297
80	0.177
100	0.149
200	0.074

Data and Results for the
Mechanical Analysis
of Göksu Sand

Results of Mechanical Analysis

Sieve Size	First Method Finer Weight grms	First Method Finer Percent	Second Method Coarser Weight grms	Second Method Coarser Percent	Third Method Sieve Size Finer	Third Method Sieve Size Coarser	Weight grms	Percent
10	501.700	100 %	0.000	0.000 %	---	10	0	0.0 %
20	498.158	99.1 %	3.542	0.707 %	10	20	3.542	0.707 %
30	494.388	96.5 %	17.312	3.950 %	20	30	13.770	2.745 %
40	456.154	9I.0 %	45.546	9.050 %	30	40	28.234	5.640 %
50	360.099	7I.80%	I4I.60I	28.90 %	40	50	96.055	I9.I6 %
80	37.299	7.44%	464.40I	92.50 %	50	80	322.800	6.44 %
I00	4.432	0.884%	497.268	99.30 %	80	I00	32.867	6.54 %
200	0.556	0.0111%	50I.I44	I00.00 %	I00	200	3.826	0.775 %
Pan					200			

Plot

The aim of the mechanical analysis is reached after the curve is plotted. The results of the mechanical analysis were plotted as percent finer as ordinates and sieve oppenings as abscissa in millimeters.

Results

From the graph, the effective size, that is the diameter such that the aggregate weight of all smaller grain is 10% of the total weight, was found to be 0.20 millimeters.

Another diameter d_2, was found such that the aggregate weight of all smaller grains is 60% of the total weight which was found to be 0.260 millimeters; and the ratio of d_2 to d,

$$e = \frac{.26.000}{0.185} = 1.325$$

Conclusions:

1. Having an effective diameter of 0.200 millimeters the sand used was a fine sand.

2. Having a uniformity coefficient of 1.325 the grain sizes do not vary in a wide range.

INTERNAL FRICTION IN SAND

So far the Foundation Soils Committee of the American Society of Civil Engineers undertook many soil mechanics researches; among these the experiments which were carried in order to determine the interna~~tional~~l friction of Ottawa sand was the most extensive, (The reader is referred to see Transaction of American Society of Civil Engineers, 1917, 1920), but no general condusions were drawn out of them.

Charle Terzaghi in one of his articles that appeared in the Engineering News Record Vol. 95. No. 26, under the title of Friction in Sand and Clay states the followings:

"In fact, there is no definite single coefficient of friction. The value of the coefficient depends both on the manner in which slip is produced and on the changes in structure of the sand prior to the slip. On theoretical considerations as well as by the results of experimental work, the author has been led to distinguish between two fundamentally different cases:

(a) Separation of a mass of sand along a plane: Within the zone of slip there is complete rearrangement of the grains. Since Reynolds' classical experiment with the sand-bag we know that the grains of a mass of sand cannot change partners unless the volume of voids of the sand temporarily increases. Hence the separation along a plane requires a gradual loosening up of the structure of the sand in the vicinity of the plane of separation, i.e., the separation involves a tendency toward increasing the volume of voids. The coefficient of friction along a plane of separation is called the coefficient of internal friction.

(b) Continuous deformation of a mass of sand; If a mass of sand is uniformly stretched or compressed so that the displacement of the grains occurs throughout the whole mass, the stability of the structure increases and finally assumes a maximum. In this limiting stage the frictional resistance which determines the ratio between the extreme principal stresses is the coefficient of internal resistance. Its value represents the maximum value which the coefficient of friction can assume within a mass of sand of a given density. It may be considerably greater than the angle of repose.

Cases (a) and (b) are limiting cases; there is an infinite number of intermediate possibilities, each of which corresponds to another type of internal displacement, involving another coefficient of friction, limiting the state of equilibrium for that particular type of displacement. Thus the value of the coefficient of internal friction depends on whether the slip is confined to a zone only a few grains in thickness or whether it occurs within a layer ten times as thick. This seems to be the chief reason for the disagreement between the results of large-scale retaining-wall tests and miniature tests.

The fricition tests of the Foundation Soils committee are a notable example of the utter complexity of the frictional resistance in sands. It is timely to conclude from these and similar experiments that there can be no hope of exactly determining the value of the coefficient of friction required for calculating the outcome of more complicated earth pressure phenomena in sands. We can only guess at that value by choosing some intermediate value between the extreme ones or by indirectly computing it from what has been observed."

INTERNAL FRICTION TEST

As shown in figure (4) a box was filled with sand and a clean steel tape introduced into it laying in a perfectly horizontal plane, then by means of a thin and strong rope passing over a pulley it was attached to a bucket. The weight W consisting of partly sand and partly lead balls well distributed on top of the box. At first the author tried to find the force necessary to pull the tape by adding a few c.c of water to the bucket, but this was not satisfactory for the load was such that a very big container had to be put instead of smaller and the weighing would not be very accurate. The next attempt was to drop small lead balls diam. of about 3/4 of a millimeter in the bucket instead of water. This simplified the method of procedure. A curve was plotted with load on top as ordinates and force necessary to pull the tape as abscissas. Then the coefficient of resisting friction was commuted.

Observing the graph, the coefficient of resisting fricition increased with the increased load on sand, and a tangent drawn to curve at any point is the coefficient of resisting fricition. Another curve was plotted as coefficient of internal resisting friction as ordinate and load in Kglcm^2.

The curve is almost parabolic, the increase is rapid at first, then increases with a decreasing rate till it reached almost a constant value. The coefficient of internal fricition in that particular Gôksu sand has a value ranging between .3180 and .3250, as it may be observed from the graph.

Data for Internal

Friction Test (3 trical)

	Load "P"	F_1	F_2	F_2	Average Force
Units	Kg.	Grams	Grams	Grams	Grams
	1	138	116	121	125
	2	281	295	318	300
	3	386	382	392	390
	4	750	752	748	750
	5	989	1018	1003	1000
	6	1518	1476	1506	1500
	7	1770	1782	1773	1775
	8	2230	2278	2242	2250
	9	2740	2721	2789	2750
	10	3297	3218	3245	3250
	11	3500	3482	3518	3500
	12	4016	4078	4056	5050
	13	4125	4146	4125	4125
	14	4580	4544	4531	4580
	15	4781	4764	4815	4780

Data and Results for

Internal Friction Test

	Load "P"	F	$\frac{F}{P} = u$	Area	Intensity
Units	Kg	grams		cm^2	Kg/cm^2
	1	125	.125	20.675	.0483
	2	300	.150	"	.0966
	2	390	.175	"	.1249
	4	750	.198	"	.1932
	5	1000	.225	"	.2415
	6	1500	.250	"	.2898
	7	1775	.274	"	.3381
	8	2250	.280	"	.3864
	9	2750	.310	"	.4342
	10	3250	.325	"	.4830
	11	3500	.310	"	.5303
	12	4050	.330	"	.5796
	13	4125	.313	"	.6280
	14	4580	.325	"	.6760
	15	4760	.318	"	.7250

PİLE DRIVİNG

Piles, Classifications and Uses:

Any structure is divided into two parts, one of which is supported by the other, the one which is supported is called the superstructure and the one which supports is called the substructure respectively.

The substructure frequently is divided into two parts in turn, and those two parts differ from each other in form and character, the lower part being called foundation upon which the entire structure rests. Therefore a foundation of a structure may be defined as that part which is generally placed below the surface of the ground and which distributes the load upon the earth.

Foundations are divided into three main types:-

1. Simplest type, that is one which is obtained by widening the base of a wall or pier.
2. Spread footing, that is one in which the bearing area is enlarged, either by putting reinforcing the concrete or brick, or by putting extra tiers of steel beams in order to furnish enough bearing area.
3. Pile foundations which consist of a base of concrete or timber, supported by a number of

piles which distribute the load to the earth through a considerable depth, either by bearing or by friction combined with bearing.

A pile is put into the ground either vertically or nearly so in order to increase its power to carry weight of a structure or to resist a later force.

A bearing pile is one which carries a superimposed load, having different shapes, may be the same cross section throughout its length or tapered.

Bearing piles are used in foundations under two typical conditions.

1. When the piles are driven through soft material to a stratum of firm or practically unyielding material.
2. When no hard bottom can be reached by any reasonable length of pile and the friction of the pile in the ground is sufficient to support the load with safety.

In the first case the pile receives little if any lateral support and therefore acts a column while in the second case the true pile action occurs and the load is either limited by the adhesion of the ground to the surface of the pile or the compressive resistance of the material in the upper part of the pile.

The principal use of piles occurs in the foundations of bridges, buildings and other structures in which they act simply as bearing piles.

THE PHENOMENA OF PILE DRIVING

The term "pile driving" is applied to the operation of taking a pile and forcing it into a definite position in the ground without previous excavation. A number of methods are employed for this purpose, which require different kinds of equipment. Historically, the oldest method of driving a pile is by means of a hammer. While very small bearing piles, or posts, were doubtless driven at first by hand with a maul or beetle, those of larger size, usually designated as piles, required the use of a machine by which a hammer was raised with the aid of a pulley and rope and allowed to drop on the head of the pile. A weight used in this manner was hence called a drop-hammer. At first men, then horses, and afterward the steam engine were used to raise the hammer.

After the invention of the steam engine, steam-hammers were designed in which the driving weight is lifted a short distance by steam pressure and allowed to fall by gravity, the rapidity of action being greatly increased. Subsequently, steam-hammers were invented in which steam pressure reinforces the action of gravity on the down stroke. At one time pressure due to the explosion of gunpowder was used to drive piles, but that method is now regarded as antiquated.

To a very limited extent pile driving has been accomplished by placing a static weight upon a pile and rocking it to and fro in soft ground, to which condition this method is practically limited.

Another method of more recent discovery, which has greatly advanced the art of pile driving, consists in the use of the water-jet to aid in displacing the earth at the foot of the pile and to lessen the friction of the pile as it descends through the surrounding material. This method is generally employed in conjunction with the use of a hammer, although occasionally the hammer may serve merely as a static weight during a portion of the time required to sink the pile.

The phenomena of pile driving may perhaps be most readily understood by the student by considering the case in which a timber pile is driven vertically into the ground by means of a drop-hammer. After the piles are delivered on the site within reach of one of the lines of the pile-driver which is used to handle the piles, the line is made fast to a pile near its head and first dragged, if necessary, close to the front of the pile-driver, and then hoisted until it is suspended in the air. It is next placed and held laterally between the pair of tall parallel members of the pile-driver known as the leads and between which the hammer is guided in its movements. After lowering the pile until its foot rests on the ground, the line is released. The hammer, being held at the top of the leads by the other line, is now released and in falling strikes the head of the pile. It is then raised again and released for the second blow, and so on for successive blows until the required penetration of the pile is obtained.

During its fall, the velocity of the hammer is accelerated until the instant when the hammer and the pile, in connection with a certain mass of earth beneath and around it, move together. When the hammer strikes the head of the pile the pressure between the pile and hammer increases from zero up to a certain value when the pile as a whole begins to move. After all the compression in both hammer and pile has taken place, they will move together. Their velocity is then gradually reduced to zero by the varying resistance of the earth during the time of penetration for the pile. Some of the work done by the falling hammer is consumed in overcoming friction, in crushing and heating the head of the pile, and in compressing the pile and hammer while the remainder causes the penetration of the pile.

In careful experimental investigations conducted by ERNEST P. GOODRICH, with an apparatus designed to show the exact vertical motion of the pile, the time occupied by this motion, the velocity of the hammer as it strikes the pile, the velocity of the pile at each instant of its movement and the amount of compression suffered by the head of the pile from the blow of the hammer, it was found that on the average the penetration, measured from the deepest point, varies practically as the square of the time measured from the final instant. The autographic records showed also that, in the majority of cases, the final magnitude of the force acting on the pile is the same as its initial magnitude when the pile and hammer move together; and prove conclusively that the hammer remains in contact with the pile until the motion of the latter has ceased.

Small-sized experiments on pressing sticks with blunt tips into sand and other kinds of earth, as well as observations of regular piles, show that a conical mass is formed at the tip and pushed along, while curved-flow lines of earth appear as the material is pushed aside and compressed. The extent of the movement depends upon the compressibility of the earth. Often some of the material near the sides of the pile will move upward slightly. It is thus seen that the supporting power of the ground penetrated is one of the elements which determines the load which a pile can bear. In most cases this supporting power of the ground increases more or less with the depth, and hence the load depends upon the total depth of penetration. Sometimes the larger part of the superimposed load is transmitted by the pile through its foot to a hard substratum, and threfore acts like a column. When the pile is supported entirely by the frictional resistance between its sides and the earth, the load is transmitted to a deep ground level in a conoid of pressure through the earth above it. Usually these two methods of transferring a load from a pile to the earth act together in varying proportions.

This discussion of "The Phenomena of Pile Driving" is taken from:

Foundation of Bridges and Buildings by H. S. Jacoby and R. P. Davis

DRIVING TIMBER PILES

Observations in Practice

As a general rule, a heavy hammer with a low fall secures greater penetration with less expenditure of power than a light one with a high fall; it is also less injurious to the equipment. More blows can be given in the same time with a low fall and hence less time is given between blows for the ground to compact itself around the pile. In quicksand it is especially necessary to have the blows follow each other as rapidly as the operation of the hammer permits. In silt the rapidity of blows need not be quite so great as for quicksand.

When a pile sinks at a uniform rate it is less apt to jam, buckle or split than when driven with heavier blows and with marked intervals of time between them. This statement is confirmed by observations in putting down steel sounding rods by hand. For example, through soft gravel mixed with quicksand, one man may be able to push a rod down 5 or 6 feet, and if quick enough may pull the rod up again with the same expenditure of energy. If, however, the rod is allowed to rest no longer than 15 seconds, the sand packs against it so that two men are scarcely able to pull it up. A pile which is left standing for a few minutes in some kinds of sand may be packed so hard as to resist further penetration, or at least to require a much larger impact to start it again.

A very slight bounce of a drop-hammer occurs at every blow under good conditions for driving, but decided bouncing of the hammer may occur when the penetration ceases, or when the hammer is too light, or the fall too great, or ~~both~~ both; or when the head of the pile is crushed or broomed so as to cushion the blow.

In certain kinds of soil a pile may sink some distance and then refuse to go further, but will resume penetration when driven after an interval of rest; or it may refuse to sink under a heavy hammer and yield under the more rapid blows of a lighter one. The driving of one pile may cause adjacent piles to rise, and in soft ground or mud often causes an adjacent pile previously driven to move away slightly.

This is a resumé of the discussion of the 2Driving of Timber Piles"2 as given in:

Foundations of Bridges and Buildings by H. S. Jacoby and R. P. Davis

BEARING POWER OF PILES

The computation of the bearing capacity of a pile by the relation of drop and penetration rests on two assumptions.

1. That the bearing capacity of a pile can be computed from the effect of the blow.
2. That the bearing capacity of the complete fonndation is equal to the sum of the bearing capacities of the individual piles.

So far among the formulas that have been developed there are three which needs mentioning here. They are Goodrich, Engineering News and a formula depending on the semi-elastic as Redtenbacher's, and Kropf formula.

THE GOODRICH FORMULA

The most elaborate attempt which has been made to deduce a general theoretical formula for the final resistance of a timber pile when subjected to the blow of a drop-hammer is that of Ernest P. Goodrich, the results of which are contained in a paper entitled, The Supporting Power of Piles, published in the Transactions of the American Society of Civil Engineers, vol. 48, page 180, August, 1902. The phenomena of pile driving which are taken into account mathematically in deducing the formula, in accordance with the principles of physics and mechanics.

It is then shown how 14 other pile-driving formulas are derived from this general one, by stating the various assumptions with respect to its elements or terms which are made in each case. That some of the assumptions are seriously in error is proved conclusively by the wide variations in results obtained by the application of the formulas. The true values of some of the terms can be determined only by experimental investigation.

The general formula consists of 25 terms besides several numerical coefficients and exponents, and is therefore too complicated and unwieldy for practical use. For this purpose, a number of terms were evaluated with the aid of experiments conducted under proper conditions for pile driving in good practice. By substituting the values and inserting suitable numerical values for the dimensions and weights of the pile and hammer, an expression was derived giving a direct relation between the pressure on the head of the pile when it comes to rest, and the penetration. From this relation, it was found that for an allowance of 3 percent error in the observation (which, for example, is a variation of 1/8 inch for penetration of 4 inches), the corresponding error involved in the pressure on the pile is 3.1 percent when the penetration is 4 inches and 23 percent when the penetration is 1 inch. Hence, any terms in the formula which involves a change of less than 3 percent in the pressure on the pile may be advantageously omitted, and no penetration much less than 1 inch can be trusted to give the corresponding pressure within a reasonable percentage of error.

An extreme variation in the elastic shortening of the hammer is found to produce a change of only 0.07 percent in the pressure on the pile, and hence the four terms relating to the deformation are omitted. The difference between the elastic shortening of a long softwood pile and that of a short hardwood pile may cause an extreme variation in the pressure on the head of the pile of about 25 percent, and hence this term is retained.

After introducing the experimental values, and making the other changes mentioned, the formula for the final pressure on the head of the pile, as it comes to rest, is reduced to

$$F = \frac{-p}{C} + \frac{1}{C}\sqrt{p^2 + 1.15CW_h h(R_w - v)} \qquad (1)$$

in which p denotes the penetration of the pile under a single blow, C the elastic shortening of the pile due to longitudinal compression, W_h the weight of the hammer, h the fall of the hammer, R_w the ratio of the weight of the hammer to the combined weight of hammer, pile and earth moved in connection with the pole and v' the ratio of the work done in crushing and heating the head of the pile to the total work done by the hammer exclusive of losses before it strikes the pile.

The coefficient 1.15 in this expression relates to the velocity of the hammer, it being found by experiment that, when the hammer is operated in the customary manner with the line from the engine attached to it, $v^2 = 1.15gh$, instead of $v^2 = 2gh$ for a free fall.

This loss of energy may be computed by equation (1) from two sets of observations on the same pile for falls of the hammer which do not differ widely, provided it be assumed that both the total pressure or resistance of the pile and the loss of energy are the same in the two cases. From observations made on a number of piles Goodrich found that the loss of energy v' rarely exceeded 5 percent and in most cases was nearly 2 percent for piles that were sound and well driven. From a given numerical example in which $W_h = 3000$ pounds and $h = 180$ inches, the value of F is found by computation to be 134,400 or a reduction of about 7 percent. Without such observations and computations, it is absolutely impossible to form any reasonable judgment of the value of v', or of its effect.

To eliminate the value of C in equation (1), the same data are used and the values of v' computed from the formula but with C omitted. The value v' thus obtained for each pile includes losses due to the compression of the pile, as well as to heating and crushing its head. The values thus found vary greatly but average less than 10 percent, even with some very badly broomed piles. By plotting the percentage of energy losses due to all causes for the different falls of the hammer used in the experiment, the curve shows that the loss of energy increases with the height of the fall. The author of the formula states, however, that his observations tend to show that the terms involving the compression of the pile can be neglected and proper compensation be made by taking v' as 2 percent in the formula, provided thepiles are sound and well driven; but the formula is liable to be in error about 20 percent, if the piles are poorly driven and the fall is much less than 15 feet.

By making these further substitutions, the formula becomes $F = 0.575\ W_h h\ (R_W - 0.02)/p$. The term R_W involves the unknowable quantity W_g or the weight of the ground moved in connection with the pile. It wasestimated that for piles 700 inches long and weighing 2000 pounds, W_g should not be taken less than 1000 pounds, this estimate being based on observations of minature piles driven in a box of sand with glass sides, and of the ground found clinging to actual piles withdrawn from the earth. In special cases, such an assumption may involve an error of 33 percent and if combined with other cumulative errors the final value of F given by the formula may be

50 percent in error. The opinion wasexpressed by its author however, that if a sound well-driven pile weighing somewhat less than the hammer be tested by a fall of about 15 feet and shows a penetration of about 1 inch, the formula in its final shape will give the supporting power of the pile immediately after driving, with a probable error of considerably less than 10 percent. Inserting the value of $R_W = 0.5$, the formula finally reduces to the expression $F = 0.276\ W_h h/p$, or by changing the height of the fall from inches to feet, it becomes

$$F = \frac{10 W_h\ H}{3p} \tag{2}$$

in which F denotes the ultimate bearing power in pounds immediately after driving, W_h the weight of the drop-hammer in pounds, H the restrained height of fall in feet, the line being fastened to the hammer and p the final penetration per blow, expressed in inches.

Goodrich recommends "that in making tests for the supporting power of piles, a standard fall of hammer be adopted and specified for making all determinations, and that 15 feet be adopted for the following reason: (a) This height of fall produces good observable penetration with any but very light hammers, or for piles in extremely compact soils; (b) the penetration is not excessive for any but very heave hammers or for piles in very light soils; (c) all frames are large enough to afford this fall; (d) the lost energy is comparatively small (e) nearly all formulas give nearly the same values through this region of variation; (f) the writer's formula is especially built for this fall." After recommending a specification relating to the weight of hammer, height of fall and final penetration, he adds "that designers can more easily determine the necessary pile spacing and the most desirable factor of safety to be used in individual cases, and make the pile-drivers follow a standard specification, than otherwise."

FORMULAS DEPENDING ON THE THEORY OF ELASTIC AND SEMI ELASTIC BLOWS

$$Q_d = \frac{F}{L} E \left(-s \pm \sqrt{s^2 + \frac{2Rh}{E} \cdot \frac{R + m\,G}{R + G} \frac{L}{F}} \right) \tag{1}$$

R - Weight of hammer

G - Weight of the pile

L - Length of the pile

F - Area of the cross section of the pile

E - Modulus of elasticity of thepile material

h - Distance the hammer drops

s - Penetration produced by one blow

n - Coefficient of elasticity of the impact, mso for perfectly non elastic impact; and m - 1 for perfectly elastic impact.

Q_d- Resistance against penetration of the pile under impact

Q - Ultimate bearing capacity of the pile under static load.

C - Empirical constant depending on the nature of the pile and the resistance against penetration $= \frac{QdL}{LEF}$

The value m, is usually assumed equal to 0.5 (semi-elastic impact. For mso, the equation becomes Redtenbacher's formula, which is quite extensively used in Europe. On the other hand, if perfect elastic impact is assumed, msl, equation (1) becomes

$$Rh = Q_d \left(s - \frac{1}{2} \frac{Q_d^{\,2}}{F} \cdot \frac{L}{E}\right) = Q_d \left(s - \frac{1}{2} \frac{Q_d}{F} \frac{L}{E}\right) \tag{2}$$

$$Q_d = \frac{Rh}{s - \frac{1}{2}\frac{Q_d}{F}\frac{L}{E}} \qquad (3)$$

If the fact fact that the term $\frac{1}{2}\frac{Q_d}{F}\frac{L}{E}$, depending on both the nature of the pile and the resistance against penetration, is disregarded and if this variable term is replaced by an empirical constant, C, independent of all these factors

$$Q_d = \frac{R\,h}{s+c} \qquad (4)$$

which is nothing else but the well known Engineering news formula.

From a mechanical point of view, the assumption on which these formulas are based, are sound and there is not much doubt about their giving a fairly accurate conception of the resistance one has to overcome while driving the pile by a succession of impacts. Therefore, if experience shows that in certain cases the values furnished by the pile driving formulas have practically nothing in common with these determined by loading; tests being either far too small or far too large, the cause cannot be a deft in the formulas, but must be due to the fact that in these particular instances the forces Q_d resisting the penetration of the pile under impact are fundamentally different from the forces Q, which resist the penetration of the pile under static load.

This is precisely the situation a theoretical study of the pile driving phenomanon has disclosed. It is said that the bearing capacity of pile depends on two different factors, namely the frictional resistance acting along the sides of the pile and the point resistance or the resistance of the soil against being compressed and displaced by the pile. If these two resistances were dependent only on the character of the ground and on nothing else there could be no question about the resistance, Q_d, against driving the pile, and the bearing capacity Q, of the pile being identical. However, it is easy to prove that either one of them may be very different according to whether the pile is slowly faced or driven by impact.

If a friction test is made on a layer of sand by loading it and then measuring its resistance against shear, it will be found that the shearing resistance of the loaded layer, immediately after the application of the load is practically the same as is three days later. If however precisely the same test is performed with a layer of wet clay it is found that immediately after the application of the load the frictional resistance is very small, so small that one has the impression that the material is lubricated. The full frictional resistance does not develop for a couple of days in clay. If, however, laterally confined mass of sand is compressed, the speed with which the compression is performed has very little influence on the amount of work required to compress the material. On the other hand, the amount of work required for rapidly reducing the volume of 1 cu. ft. of laterally confined clay by 2 cu. in may be even 200 times as great as the amount of work required for producing the same volume change slowly.

The physical causes of these phonomena are clearly understood.

Applied to the mechanics of pile driving, knowledge of the hydrodynamic stress phenomena has led to classify soils into two main groups. In certain material (particularly in sand, gravel, and permeable artificial fills,) the resistance acting while the pile is being driven, are practically identical with those acting on the pile under static load. Under such conditions pile driving formulas can be expected to furnish good results.

In other materials (very fine grained silts, soft clays) the friciton acting on the pile during the driving is very much less than that which develops after a couple of days rest. while the resistance of the point of the pile under impact is very much greater than its resistance under static load. Due to these facts the total resistance against penetration of the pile into such material is:

(1) Dynamic resistance, Q, against penetration under impact which is the sum of a very small frictional resistance and a very considerable point resistance.

(2) Static resistance (ultimate bearing capacity, Q, under static load, which is the sum of a full frictional resistance and a very small point resistance.

Since the pile driving formulas furnish the value Q_d, there is no assurance whatsoever that for this class of materials the value, Q , may be of the same order of magnitude. The value Q_d, may by chance be equal to Q , the deficiency in static friction being compensated by an excess in point resistance; but condition is by no means necessary. It could as well be very much greater or very much less, depending on the material. The following analogy demonstrates the error committed when applying the pile driving formulas to resistance against the penetration of pile in this second class of materials.

The best way to distinguish whether a material belongs to the first or to the second class in that of comparing the penetration per blow immediately below and after a period of rest at least 24 hours. If these two penetrations are indentical, one can be quite sure that the material belong to the first class, and the pile driving formulas furnish reliable results.

PILE DRIVING TESTS ON MODEL PILES.

This experiment had in general 4 parts:

1. Driving a circular pile in dry sand
2. Driving a circular pile in wet sand
3. Driving a conical pile in dry sand
4. Driving a conical pile in wet sand

Let us take each item separately

Driving a circular pile in dry sand

The simple home made apparatus as shown in the figure was constructed the tank being filled with the fine sand. A pile 2 cm. in cross-section and 47 1/2 cm. long was placed in the leads. The ratios of length,cross-section of the pile,the fineness of the sand and the weight of drop hammer were so made,in order to keep the corresponding ratios that are existing in actual practice. The weight of the pile was 75 grams and the weight of the drop hammer,which was a steel pipe 2 cm. in cross-section and filled with lead,was exactly twice the weight of the pile that is 150 grams. A string was attached the end of the drop hammer,and a string was allowed to pass over a pulley with allowing very little friction.

The aim of the first test was 70 determine the effect of height of drop to penetration.

A shown in the Figure (a) the pile was divided into eleven sections,the first ten sections being five centimeters apart

Same as fig b except the two diameters are equal. 2 cm ϕ throughout the length of the pile.

from each other while the 11th section was 2 i/2 centimeters apart from former section.

A section 3 was chosen this meant that the pile was driven,before the experiment,up to section three into the sand by big blows corresponding to big falls; then the drop hammer was left to drop freely (note that the fall was not a restained fall) on the head of the pile from a height of 5 centimeters for five times,and the enlarged penetration on the sca was read, and it was divided by the number blows and penetration per blow was noted.

The same process was repeated for a fall of 10,15, 20,25,30,cm. respectively and the penetrations noted.

This experiment was carried three times,and different results were obtained this is simply due to the jercking of the sand and the changing of its density each time after driving and pulling the pile out of the sand container.

The same proceedure as was stated in the case of dry sand was carried with the wet sand water content being 1.26% also with three different densities. The plots shown in Figure (7) was obtained.

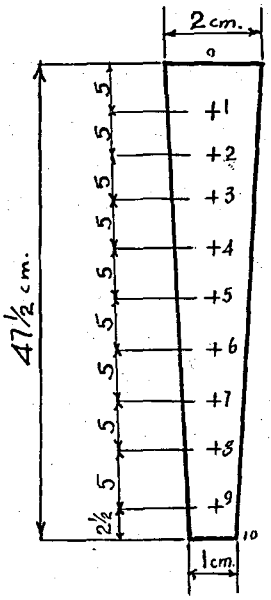
The line joining the points in all of the six cases were all straight lines,continuous driving loosened the sand particles and the penetration was increased by a small amount, the curves A,B, and C in the case of dry sand were the results of the first second and third trials respectively. The increase penetration ~~small~~ was small falls and large in bigger falls respectively.

In wet sand again the lines joining the points were

were again a straight lines and the rate of increase were the same as in the first case,only penetration for the same fall being twice as much as in the case of dry sand this shows that in wet sand the piles have little resistance against penetration that is t say the piles frictional resistance decrease with the water content

The fact that the lines lie on a straight line accor- ding to the theory of pile driwing by impact,indecates that the resistance of sand remained unchanged daring the driving period.

In the case of a conical pile the same experiment was repeated in the same manner for the same section 3 corresponding to the Figure (b) The conical pile had a diameter of two centimeters at the top and one centimeter at the bo- ttom.



Conical Pile.
(b)

The increase in the penetration was from 23 to 34/ both in dry and wet sand and average of 28/ increase is said to be the average increase due to the addition of water of about 1.26/

The second part of the drivind experiment was to the drive the pilethrought its lenghh and determine the penetration at each of these 10 sections.

The first test was a trial test,just to see roughly whether what the approximate penetrations are at each section. There it was observed that in section o,1, and 2, one drop had to be applied in order to get the results where as in the lower section, the hammer was dropped for 5,7, and 10 times in order to get enough and accurate penetrations.

The results obtained show that the penetration per blow decreased in all cases in dry and wet sand for both shapes.

If the results are compared , in wet sand penetration incfeases about 100% or more if the sand is loose, if the sand is well compacted so that the voids is minimum the penetration per blow decreases, and even it is less than it is in dry sand, same data were obtained but the author is not sureof their accuracy so they are not mentioned here but it is safe to state that the penetration per blow at different section decreases about 45% in well compacted sand.

The penetration per blow in conical piles is much higher than it is with circular and also increases with the water content if sand is loose, and decreases with respect to the penetration in dry sandiif well compacted. The curve plotted show this difference very clearly.

The author found it very difficult to drive both circular and conical pile in well compacted wet sand. One way to render it easy was by means of water jetting. Lack of time and availability of apparatus were the reasons why these tests were neglected.

Conclusions that are drawn from the driving tests are: In the first case penetration depends upon the degree of compactness in wet sand, and also on the shape of the pile. The curves obtained suggest that pile driving formulas such a as Engineering News and Redtenbaher formulas give good results.

According to this assumption, as was stated by Dr. Terzaghi, the bearing capacity of a pile depends upon the effect of impact , arrangement of sand particles that is to say the degree of its compactness in moist sand.

PULLING TEST

The apparatus shown in Figure (8) was constructed. The experient was similar to the Friction Test, the only difference was in the pull, which was a vertical pull instead of a horizontal one.

The results obtained and the plot ~~in Figure~~ (7) show the pull increase rapidly at first and then increase is gradual as the depth penetrated increase.

This show that especially in sand the bearing power of pile increases with the depth penetrated where the pile is a friction pile.

In wet sand the friction action along the sides of the pile decrease very much, so that the bearing capacity decrease provided the sand is not very well compacted.

In conical piles the friction is greater when the upward motion of the pile is impending, and it is very small when that point is reached. Herefore one might just as well say that the bearing capacity of friction pile depends on the shape of the pile, on the degree of compactness and the depth penetrated. Formulas derived along that line furnishes quite accurate results for there are not many cases where the foundation resting on a pile is apt to be pushed upwards.

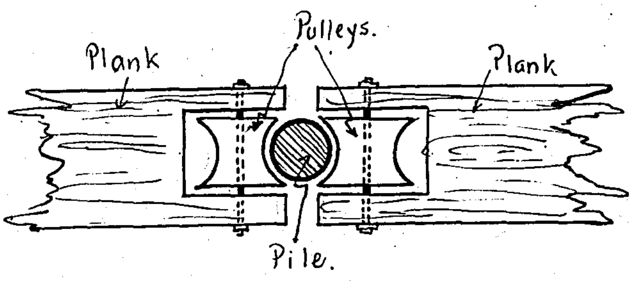
LOADING TEST OF PILES

The simple home made apparatus as shown in Figure (9) was constructed. The loading test for the round and conical pile was carried out. Several trials were made, but the tabulated results here are the accurate and the best ones.

The author found great difficulty in keeping the pile and the box which carried the load the pile vertical and without swaying in any direction. That difficulty was overcome in this way:

Two posts as shown in the sketch of the apparatus were rigidly nailed to the sand container with guides so that the roller attached to sand box and weight on top were alowed to slide while the pile settled under the load.

To keep the pile vertical at the same time two small planks were rigidly attached to the posts the ends of which were grooved and pulleys made of hard wood were placed in such a way. So that when two of these arrangements placed opposite each other as shown below:



the pile was able to fit it nicely and smoothly and while settling the pulleys simply rotated; this provided safety against sideway inclining of the pile while the load was applied on it.

The measurement was taken by the use of the pointer hinged to a stand as shown on the loading apparatus,00 as the ratios of the settlement was $\frac{1}{10}$, that is to say on actual settlement of .1 of a millimeter would register 1 millimeter on the scale. Therefore, results accurate to one tenth of a millimeter were available.

In carrying out the experiment, settlement reading for the same load was taken right at the time when the load was superimposed and also a while, in dry sand or both types, that is, conical and circular pile the time had no effect on the settlement in dry sand, the settlement was instantaneous.

This proves one of the characteristics of loading a pile in sand.

Sand particles as soon as displaced or pushed away from the sides of the pile come back immediately to their old position that group themselves again quickly along the pile and prevent it from further settlement. This shows that there is only instantaneous settlement in dry sand.

Furthermore settlement under the same loads for a conical pile was much greater, showing that a circular pile has a greater bearing capacity.

The next loading test was carried with both round and conical pile in dry and wet sand the water content in sand being 1.26%. A little water affected the settlement greatly. For the same loads in partly and very little compacted sand the settlement increased rapidly. The inefficiency of the experimental equipment did not allow me to put the datas obtained then.

There was one thing, anyhow, that the author feels safe in stating, besides the immediate settlement in wet snad there is also a gradual settlement especially in poorly compacted sand, and also in well compacted wet sand.

The loading test proved the followings:

1. In dry sand settlement occurs immediately, irrespective of the shape of the pile
2. Settlement depends upon the water content and the degree of compactness of the sand.
3. In wet sand besides immediate settlement there exists also a gradual settlement which does not last more than a few seconds only.

Besides the tests carried out on the sand the loading and the driving tests suggested the following characteristics of sand used:

1. Shrinks in drying (is due to dirt in the ~~besides~~ sand)
2. The volume of voids is much less than the average of 30-40%, it is about 1/4 of those.
3. Has very little cohesion
4. Compresses immediately when load is applied on it.

D A T A S & R E S U L T S

Relation Between Drop and Penetration in Dry
Sand of Three Different Densities
For a Circular Pile at Section III

Weight of pile = 75 grams   Length of pile = 47 1/2 cm.
Weight of drop hammer = 150 grams
Diameter of pile = 2 cm.

Units	h cm.	Initial Reading cm.	Final Reading cm.	Penetration per 5 blows mm.	Penetration per blow mm.
		Density =		d_1	
	5	37.25	37.4o	1.500	.30
	10	"	37.575	3.3251	.65
	15	"	38.050	5.325	1.10
	20	"	37.815	6.25	1.25
	25	"	38.100	8.400	1.700
	30	"	38.275	10.25o	2.05
		Density =		d_2	
	5	38.00	38.175	1.750	0.35
	10	"	38.375	3.750	0.77
	15	"	38.575	5.750	1.150
	20	"	38.750	7.50	1.500
	25	"	38.950	9.570	1.900
	30	"	39.150	11.500	2.300
		Density =		d_3	
	5	37.00	37.175	1.75	.900
	10	"	37.450	4.50	1.200
	15	"	37.600	6.00	1.65
	20	"	37.875	8.75	2.o50
	25	"	38.025	10.25	2.42
	30	"	38.200	12.10	

Relation Between Drop and Penetration in
Wet Sand of Three Different Densities
(Water 1.26/)
For a Circular Pile a Section III

		Intial Reading	Final Reading	Penetration per 5 blows	Penetration per blow
Units	cm.	cm.	cm.	mm.	mm.
		Density =		d_1	
	5	35.00	35.325	3.25	.650
	10	"	35.650	6.50	1.30
	15	"	35.9750	9.750	1.95
	20	"	36.250	12.50	2.50
	25	"	36.625	16.25	3.25
	30	"	36.950	19.50	3.90
		Density =		d_2	
	5	35.00	35.250	3.400	.680
	10	"	35.650	6.50	1.30
	15	"	36.00	10.00	2.00
	20	"	36.350	13.50	2.70
	25	"	36.700	17.00	3.40
	30	"	37.250	20.25	4.05
		Density =		d_3	
	5	35.00	35.400	3.400	.680
	10	"	35.750	7.50	1.50
	15	"	36.050	10.50	2.10
	20	"	36.500	14.50	2.90
	25	"	36.800	18.00	3.60
	30	"	37.175	21.75	4.35

Data for the Penetration per Blow
in Conical Pile in Dry Sand

Beg Diameter = 2 cm.
Small Diameter = 1 cm.
Length = 47 1/2 cm.
Weight = 60 grams
Weight of drop hammer = 150 grams.

	Drop	Penetration (1)	Penetration (2)	Penetration (3)
Units	cm	mm.	mm.	mm.
	5	0.438	.513	.528
	10	o.888	.893	.916
	15	1.487	1.510	1.560
	20	1.675	1.683	1.810
	25	2.183	2.206	2.230
	30	2.568	2.93o	3.060

Data for the Penetration per Blow
on Conical Pile in Wet
Sand(1.26% water Content)
Pile the Sand Pile as the one
used in Dry Sand.

	Drop	Penetration (1)	Penetration (2)	Penetration (3)
Units	cm	mm.	mm.	mm.
	5	0.758	o.780	0.810
	10	1.63o	1.712	1.800
	15	2.513	2.600	2.670
	20	3.413	3.560	3.716
	25	4.308	4.443	4.571
	30	5.138	5.210	5.352

Data for the Penetration per Blow
at Different Sections
throughout the Entire
Length of the Circular
Pile Driven into Dry Sand

Drop being 20 cmtrs.

Section	No. of Drops	Penetration per Blow in mm.
0	1	12.25
1	1	5.000
2	3	2.670
3	4	1.900
4	6	1.800
5	8	1.450
6	10	1.125
7	10	0.775
8	10	0.525
9	10	0.210
10	--------------	Hammer Bounced

Data for the Penetration per Blow at Different Sections throughout the Entire Length of the Circular Pile Driven into Wet Sand (Water Content 1.26/) Drop Being 20 cm.

Section	No. of Drops	Penetration per Blow mm.
0	1	26.250
1	2	11.100
2	3	6.123
3	4	3.980
4	5	3.460
5	6	2.870
6	7	2.260
7	7	1.870
8	7	1.310
9	10	.563
10	10	.350

Data for the Penetration per Blow at
Different Sections throughout
The Entire Length of the
Conical Pile Driven into

Dry Sand.
ϕ = 2 & 1 cm. Weight = 60 grams
L = 47 1/2 cm. Drop = 20 cm.

Section	No.of Drops	Penetration per Blow in mm.
0	1	34.50
1	2	16.250
2	3	8.910
3	4	4.310
4	5	2.670
5	6	1.850
6	8	1.250
7	10	0.900
8	10	0.530
9	10	0.225
10	10	0.113

Data for the Penetration per Blow at Different Sections throughout the entire length of the conical pile Driven into wet Sand (Water Content I.26/) Drop = 20 cm.

Section	No.of Drops	Penetration per Blow in mm.
0	1	48.50
1	1	31.00
2	2	19.25
3	2	7.630
4	3	5.43
5	5	4.120
6	7	2.670
7	7	1.920
8	10	1.610
9	10	.610
10	10	.325

DATA FOR PULLING TEST

FOR CIRCULAR PILE IN DRY SAND

	Depth Penetrated	Pull (1)	Pull (2)	Pull (3)	Pull (4)	Average Pull
Units	cm.	Grams	Grams	Grams	Grams	Grams
	0.0					0
	12.5	228.0	221.0	218.7	221.60	223.10
	17.5	718.0	723.0	720.0	716.0	719.98
	22.5	1008.0	1000.10	1017.0	1010.0	1008.75
	27.5	1251.0	1311.50	1298.0	1308.0	1292.50
	32.5	2180	2162.0	2195.0	2203.0	2189.50
	37.5	4116	4113	4128.70	4110.8	4115.33

Results of the Pulling Test on a
Conical Pile Same Pile used
in all the Preceeding Tests

	Section Penetrated in Sand	Pull
Units		grams
	0	9025
	1	8431
	2	7353
	3	5350
	4	4726
	5	2374.0
	6	875.30
	7	750.00
	8	5o1.8
	9	124.80
	10	6.110

DATA FOR LOADING TEST

FOR 2 cm. ROUND PILE IN DRY SAND

Units	Load	Pressure	Settlement	Settlement	Settl.	Settle.
	Kg.	Kg. cm^2	mm.	mm.	mm.	mm.
	1	.320	3.22	3.14	3.09	3.15
	2	.680	11.18	11.32	11.25	11.25
	3	.960	13.10	13.10	13.40	13.10
	4	1.280	15.25	16.25	16.50	16.00
	5	1.600	16.47	16.75	16.70	16.63
	6	1.920	18.67	18.28	18.41	18.42
	7	1.344	17.16	17.83	17.96	20.10
	8	2.560	20.41	20.28	20.31	20.36
	9	2.880	20.82	20.76	20.66	20.78
	10	3.20	22.00	22.29	22.19	22.16
	11	3.54	22.91	22.88	27.76	27.85
	12	3.84	23.72	23.58	23.65	23.65
	13	4.16	24.09	24.28	24.11	24.11
	14	4.42	24.86	24.97	25.08	24.97
	15	4.80	25.43	25.49	25.58	25.52

Data for the Loading Test of
Conical Pile in Dry
Sand Sand Pile as
the one used in Previous Experiments

	Load	Area	Load/Kg cm^2	Settlement
Units	grams	cm^2	Kg/ cm^2	mm.
	255	.785	.325	12.91
	510	"	.650	18.75
	725	"	920	21.54
	14	"	1.260	24.00
	1220	"	1.550	26.27
	1488	"	1880	28.30
	1880	"	2208	30.40

T I M E   S H E E T

From the start up to the completion of my thesis I kept a careful record of the time that I spen~~d~~t.

The total time spent in purchasing,carrying and drying the Göksu sand as well as the time given to tests whose results were not successful are excluded from the following list:

The total number of hours spent was 181 hours,which is much more then the 72 hours that was set for the thesis,this including holidays and spring vacation.

I hope at least,the Professor in charge of this thesis will take the time element into account very seriously and limit his expectations.

TIME SHEET

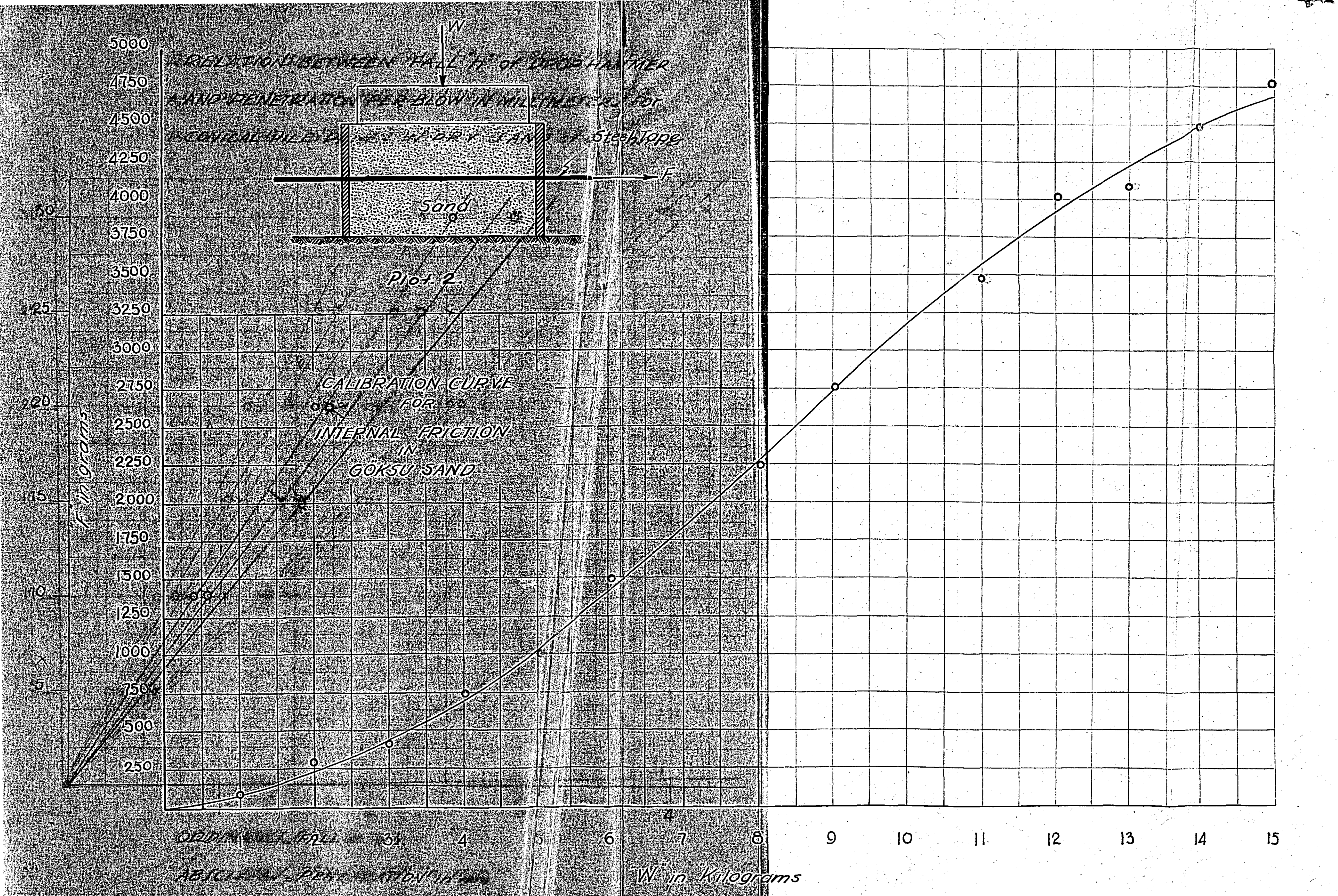
1. Soil mechanics study	20	Hours
2. Reading about theory of pile driving	15	"
3. Preliminary steps in model construction	3	"
4. Mechanical Analysis of Göksu Sand	6	"
5. Construction ~~in~~ of internal friction apparatus	5	"
6. Internal friction test	9	"
7. Construction of pile driving apparatus	12	"
8. Construction of pulling apparatus	3	"
9. Pulling test for conical & circular pile	8	"
10. Construction of the loading apparatus	15	"
11. Driving test for conical & circular pile in dry and wet sand	18	"
12. Loading test for circular & conical pile	10	"
13. Typing	40	"
14. Drawing sketches for the apparatus used	3	"
15. Plotting & drawing the curves	11	"
16. Reading the thesis	2	"
17. Arranging the thesis	1	"
Total	181	"

C O N C L U S I O N

I expected to do more than Ihave done so far, in the way of testing piles not only in one type of sand but some other types too, also with different water contents, and draw conclusion as the effect of grain size and water content on bearing power of piles, but the time in the least was not sufficient to reach that far.

Each of the data obtained are the results of many tests, and the close agreement of each test is a proof of their accu-racy and correctness.

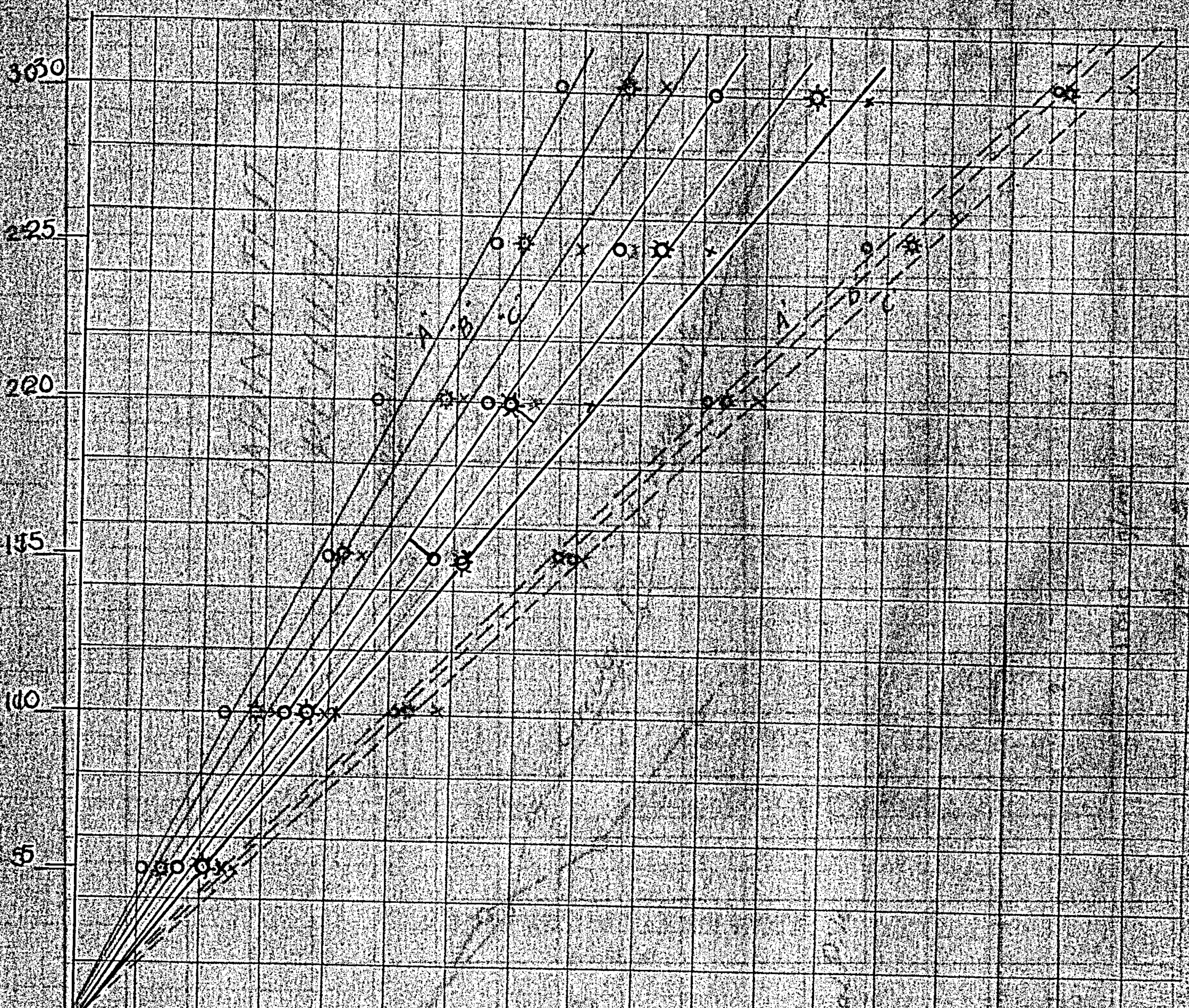
I want to express my gratefulness to Prof. E. F. Sheiry, the head of the department, who, by his useful suggestions hel-ped me in carrying over my thesis successfully.


W
Steel Tape
F
Sand
Plot 2.
CALIBRATION CURVE
FOR
INTERNAL FRICTION
IN
GÖKSU SAND
F in grams
W in Kilograms
5000
4750
4500
4250
4000
3750
3500
3250
3000
2750
2500
2250
2000
1750
1500
1250
1000
750
500
250
4
5
6
7
8
9
10
11
12
13
14
15

RELATION BETWEEN FALL "h" of DROP HAMMER

AND PENETRATION PER BLOW IN MILLIMETERS for

CONICAL PILE DRIVEN IN DRY, WET SAND of 3 Densities



ORDINATES = FALL in cm.

ABSCISSAS = PENETRATION in mm.

TOE 2.0 cm Ø x 47½ cm LONG.

$\emptyset_2$ = 1.0 cm. FOR SECTION 3

FIG.

RELATION BETWEEN FALL "h" of DROP HAMMER AND PENETRATION PER BLOW IN MILLIMETERS FOR PILE DRIVEN INTO DRY AND WET SAND

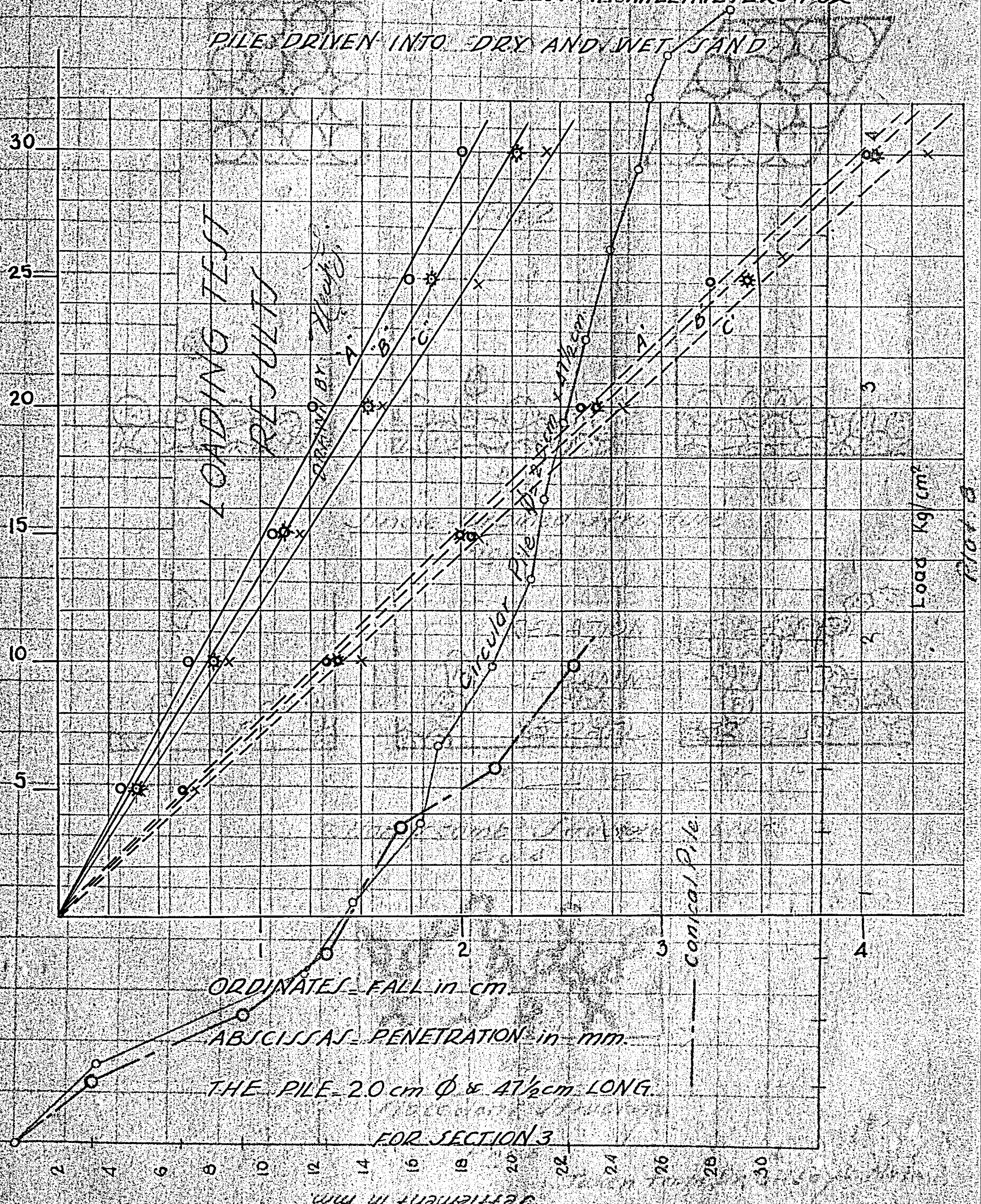


FIG 4

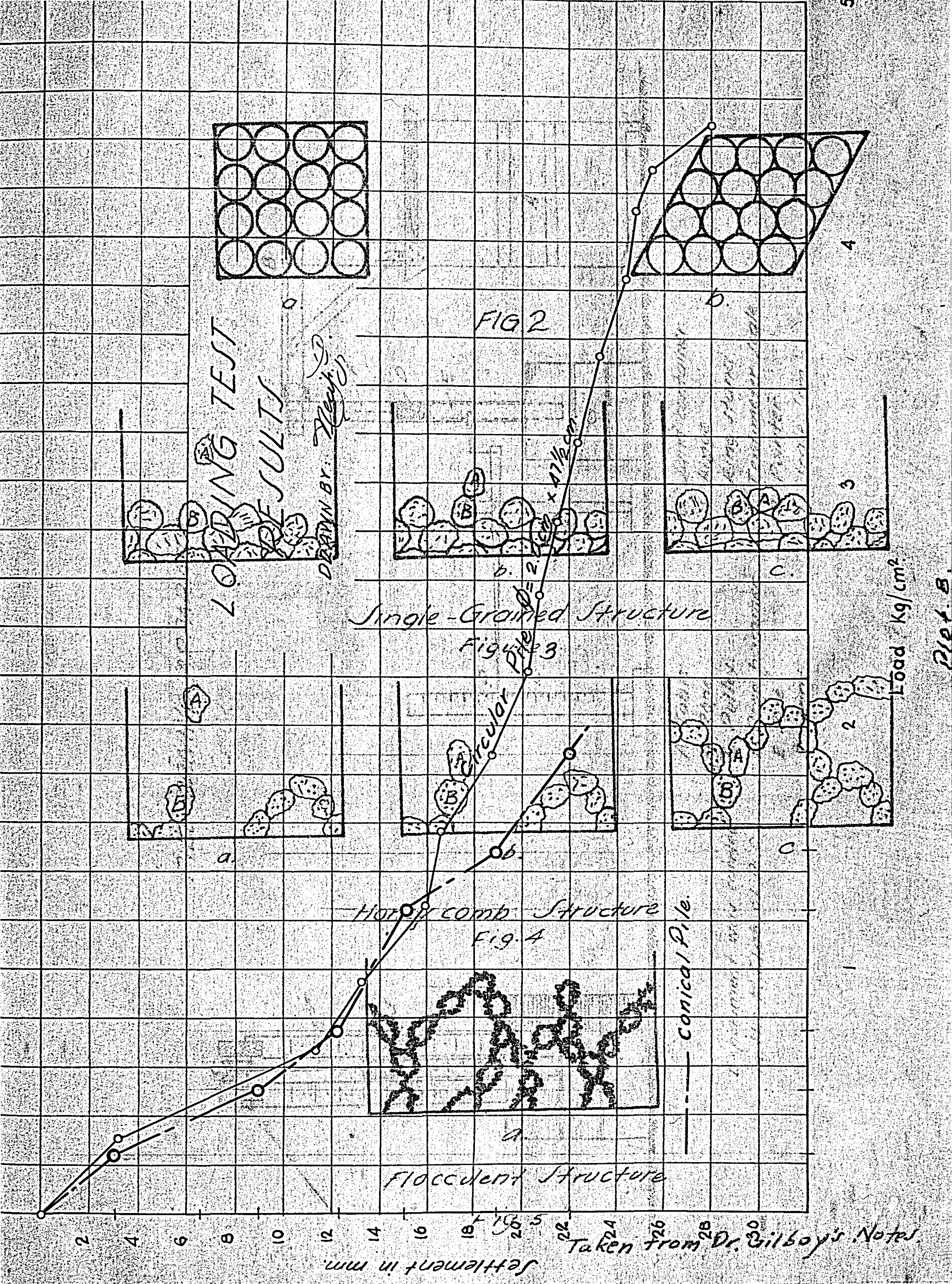


FIG 2
a
b.
Single-Grained Structure
Figure 3
a.
b.
c.
Honeycomb Structure
Fig. 4
Flocculent Structure
Fig. 5
a.
LOADING TEST RESULTS
DRAWN BY:
Circular Pile Ø = 2.2 cm × 41½ cm
Conical Pile
Load Kg/cm²
Settlement in mm
Taken from Dr. Gilboy's Notes
Plat. B

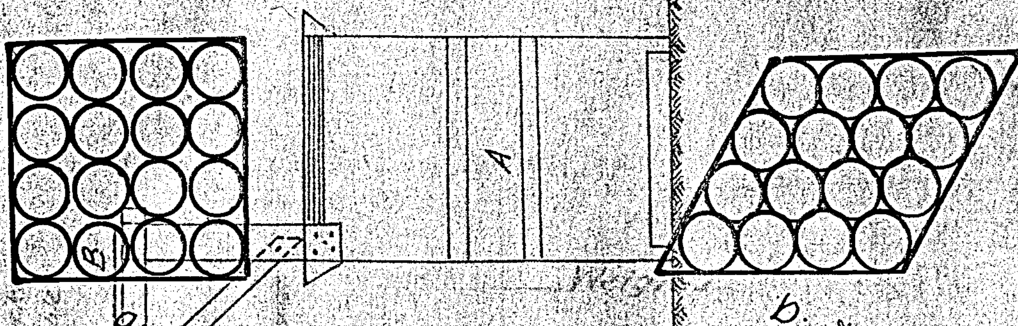
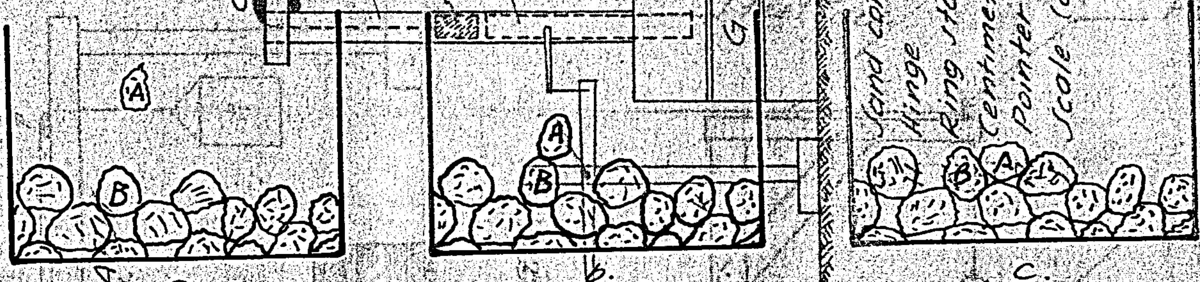
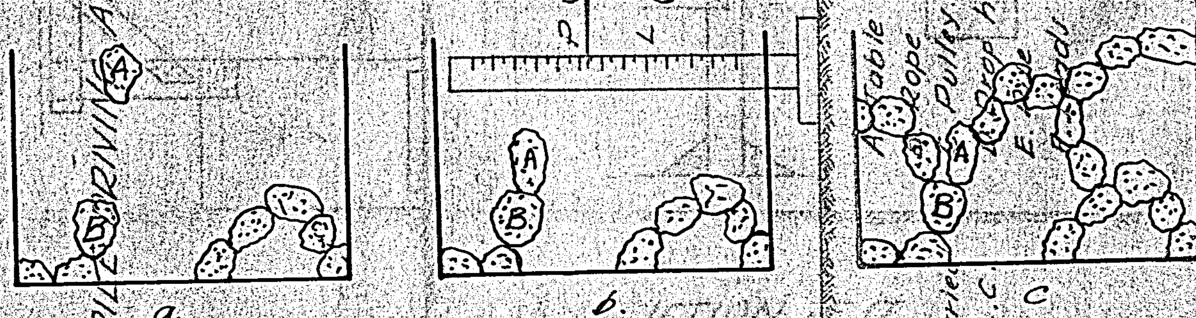


FIG. 2



Single-Grained Structure

Figure 3



Honey comb Structure

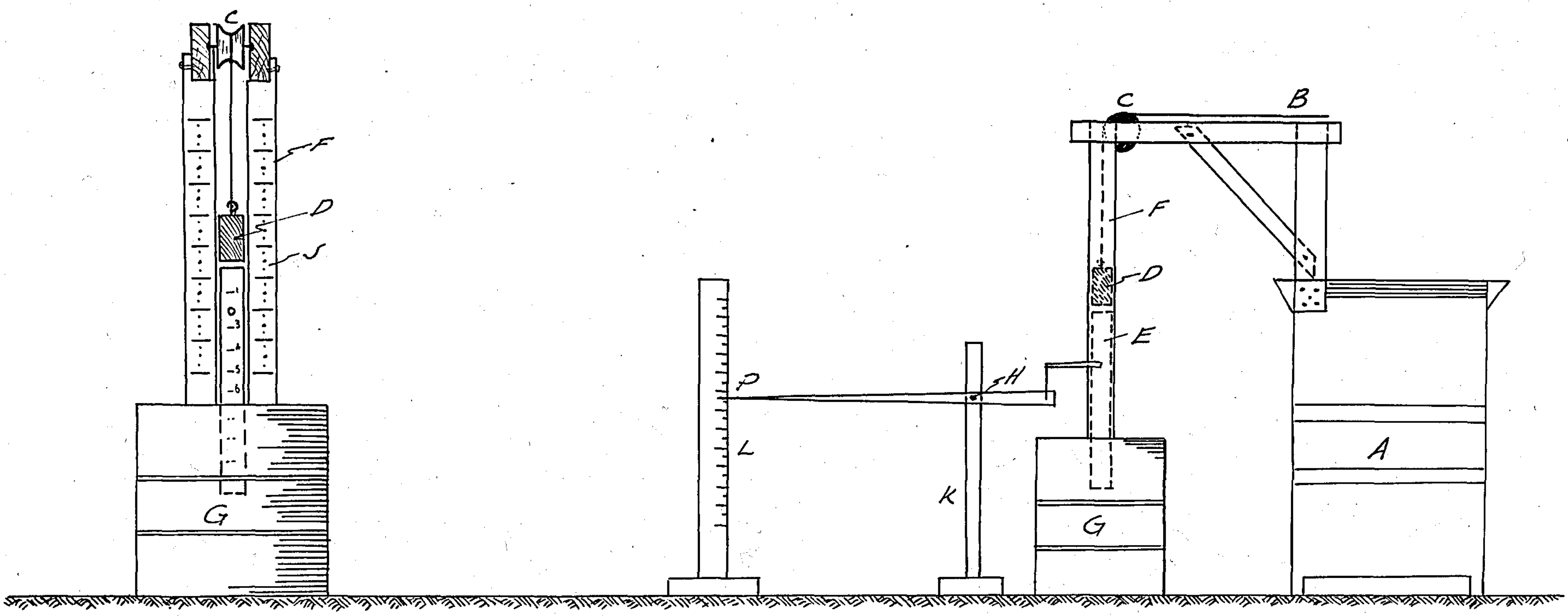
Fig. 4

Flocculent Structure

Fig. 5

Taken from Dr. Gilboy's Notes.

PILE DRIVING APPARATUS



A. Table
B. Rope
C. Pulley.
D. Drop hammer
E. Pile
F. Leads
G. Sand container
H. Hinge
K. Ring stand
L. Centimeter scale
P. Pointer
S. Scale (cm.)

Experiment was carried in the M.T. Lab. of R.C.

FIG. 7

Apparatus for Pulling Test

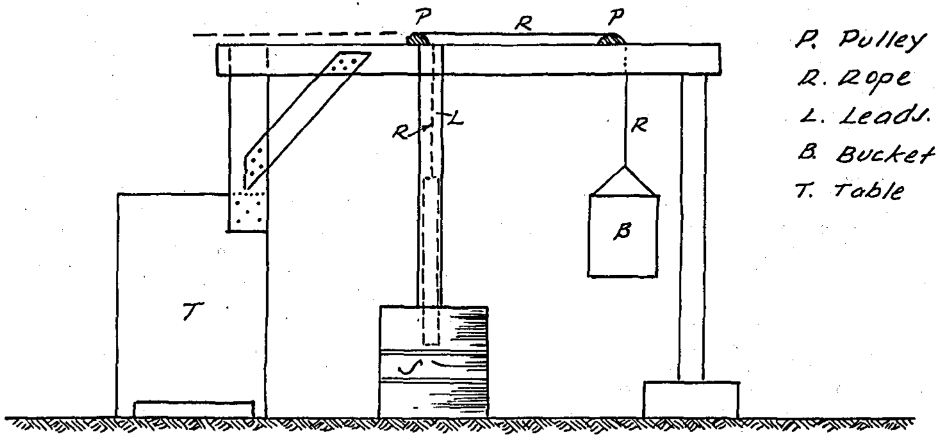
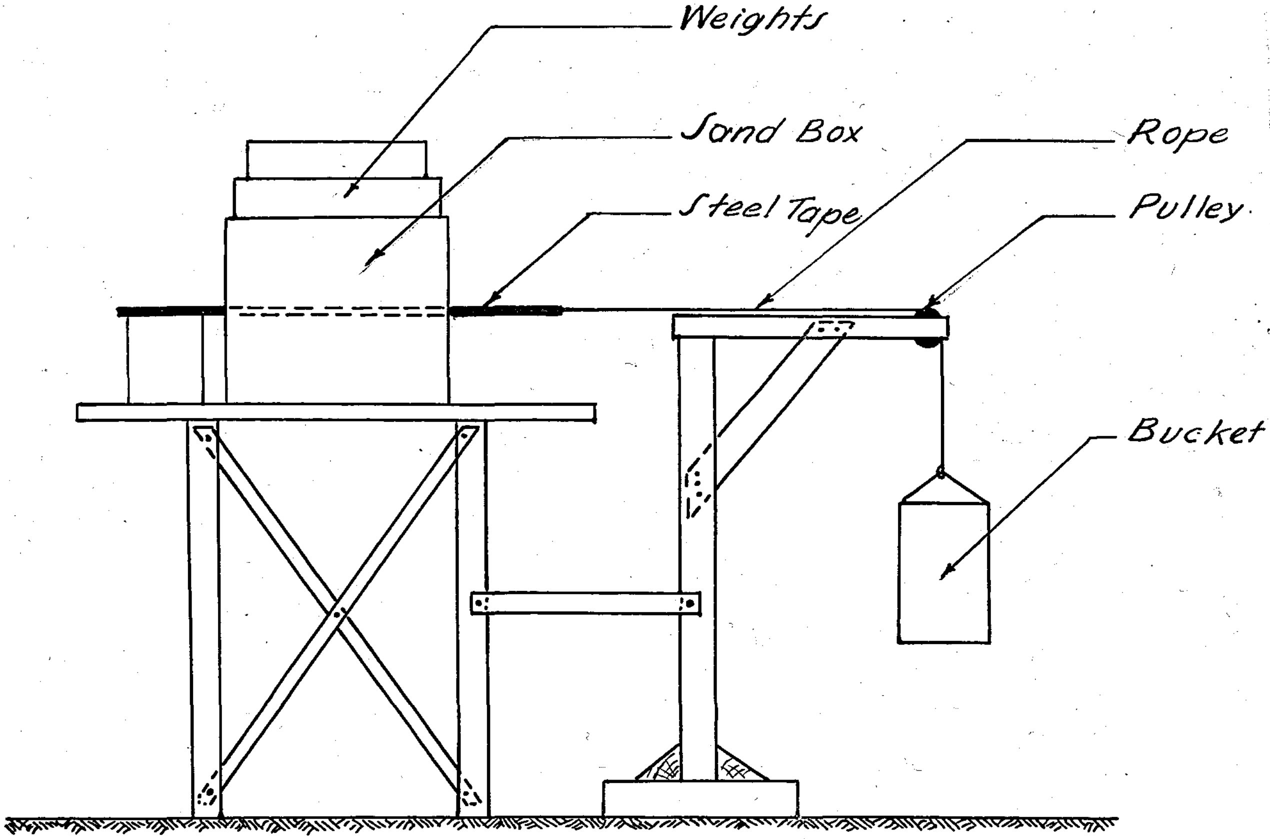
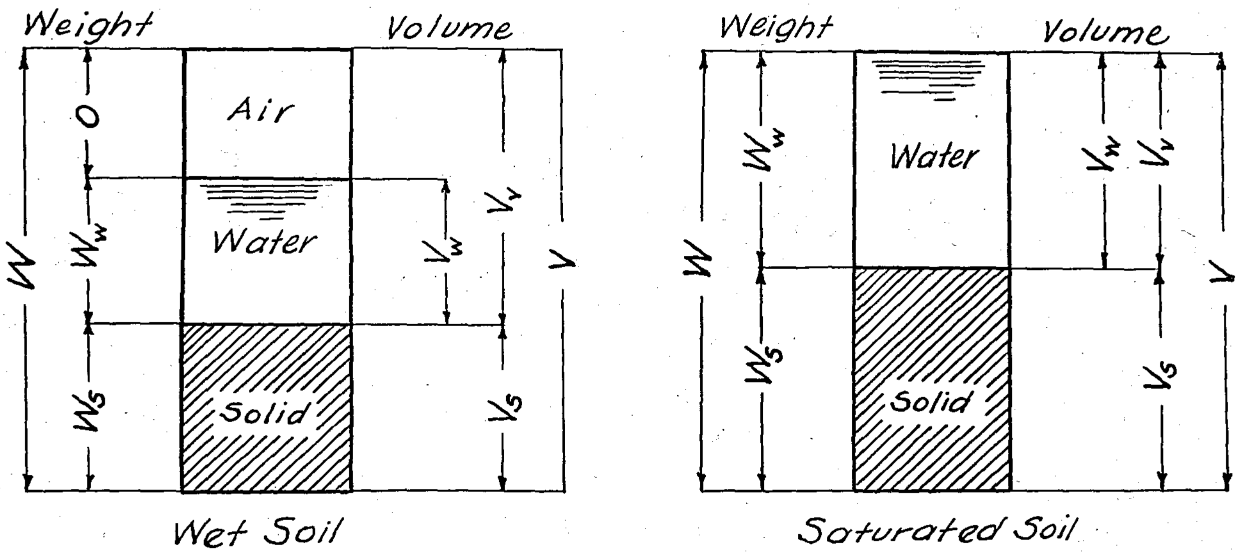


FIG. 8



INTERNAL FRICTION TEST
APPARATUS

FIG. 6.



Specific Gravity	$s = \frac{W_s}{V_s}$
Specific Mass Gravity	$s_m = \frac{W}{V}$
Porosity	$n = \frac{V_v}{V}$
Void-ratio	$e = \frac{V_v}{V_s}$
Water Content	$w = \frac{W_w}{W_s}$
Degree of Saturation	$G = \frac{V_w}{V_v}$

FIG. 1

Taken from Dr. Gilboy's Notes.

LOADING APPARATUS

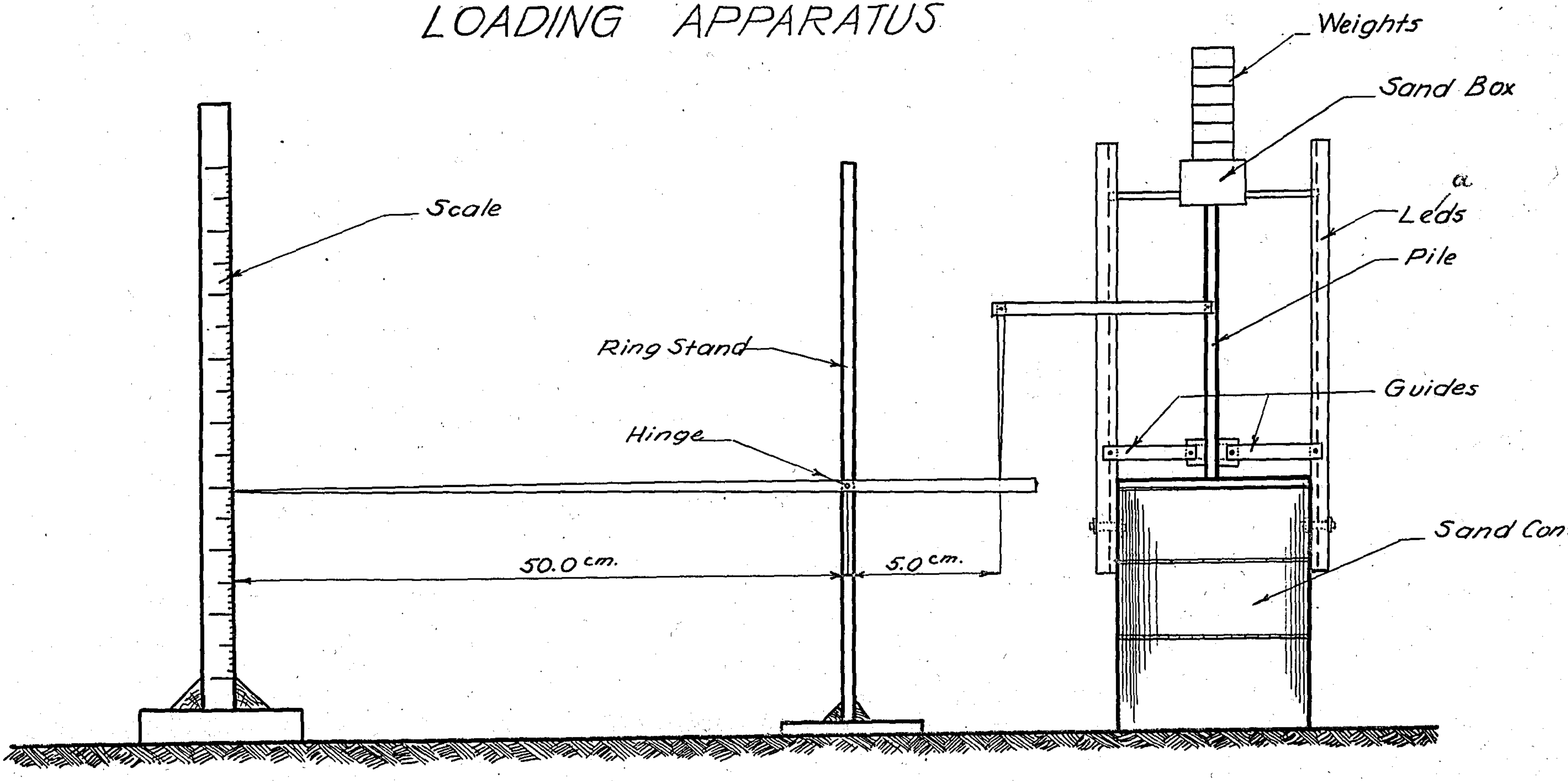
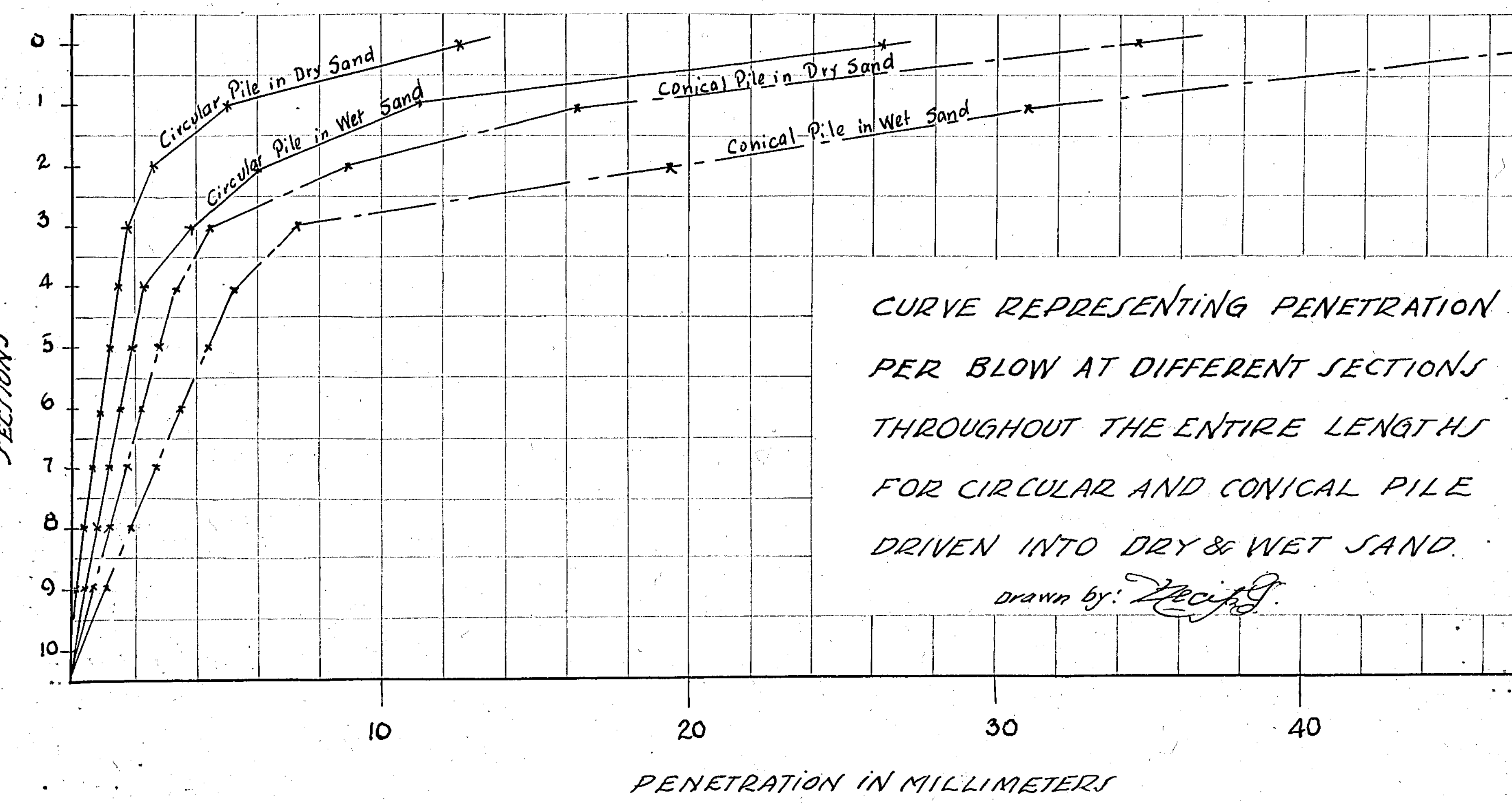
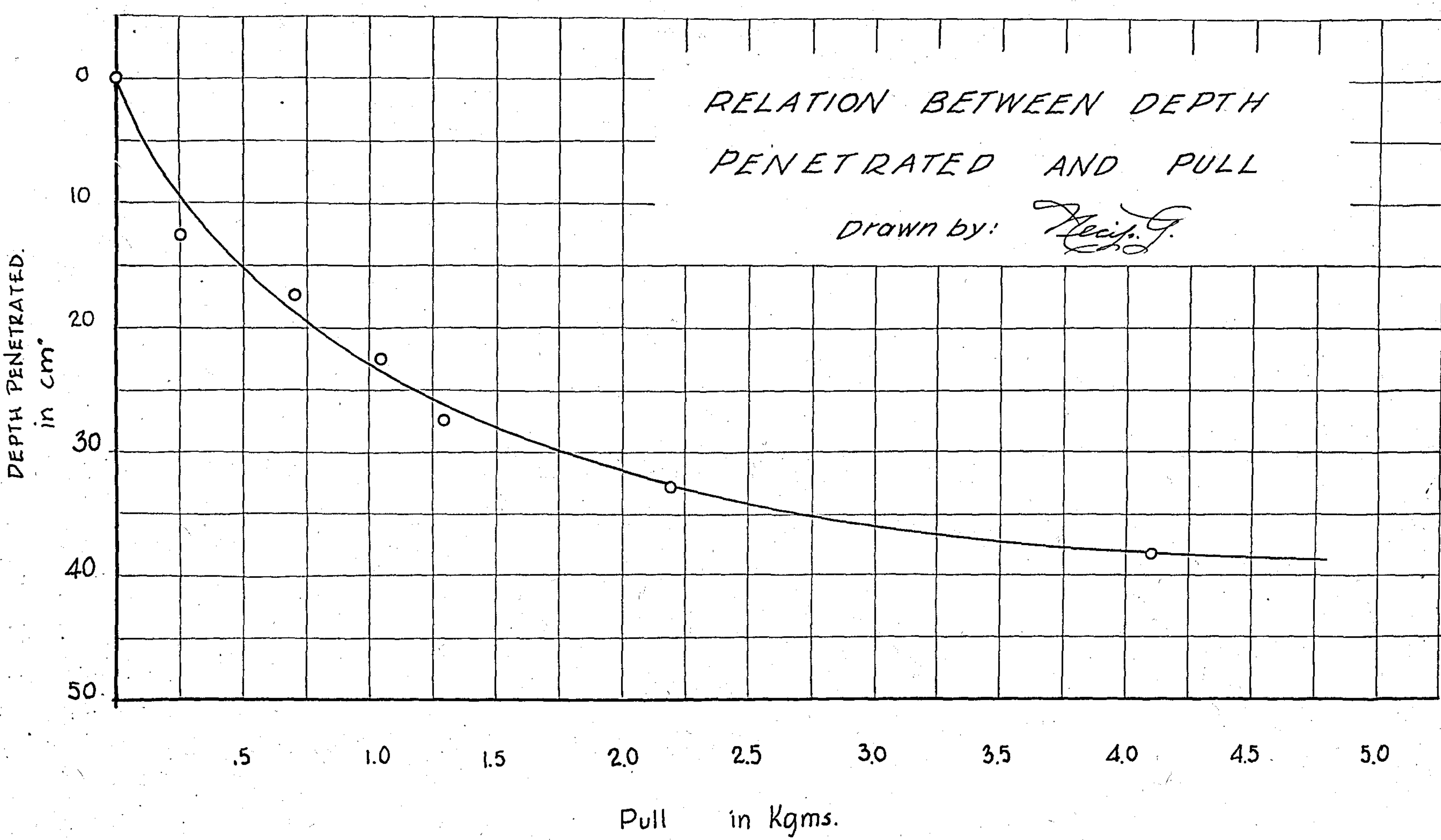


FIG. 9

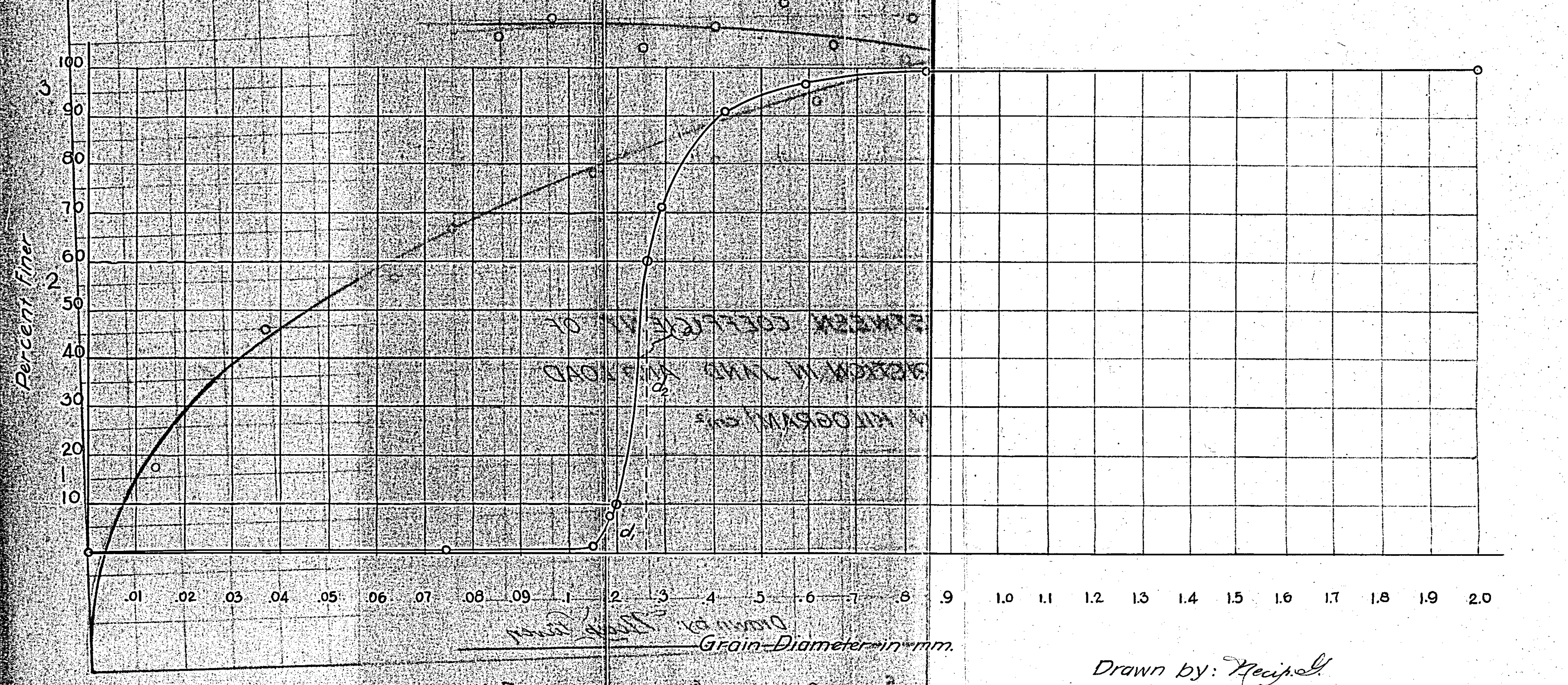

Circular Pile in Dry Sand
Circular Pile in Wet Sand
Conical Pile in Dry Sand
Conical Pile in Wet Sand
CURVE REPRESENTING PENETRATION PER BLOW AT DIFFERENT SECTIONS THROUGHOUT THE ENTIRE LENGTHS FOR CIRCULAR AND CONICAL PILE DRIVEN INTO DRY & WET SAND.
Drawn by:
SECTIONS
0
1
2
3
4
5
6
7
8
9
10
10
20
30
40
PENETRATION IN MILLIMETERS

Plot. 6.

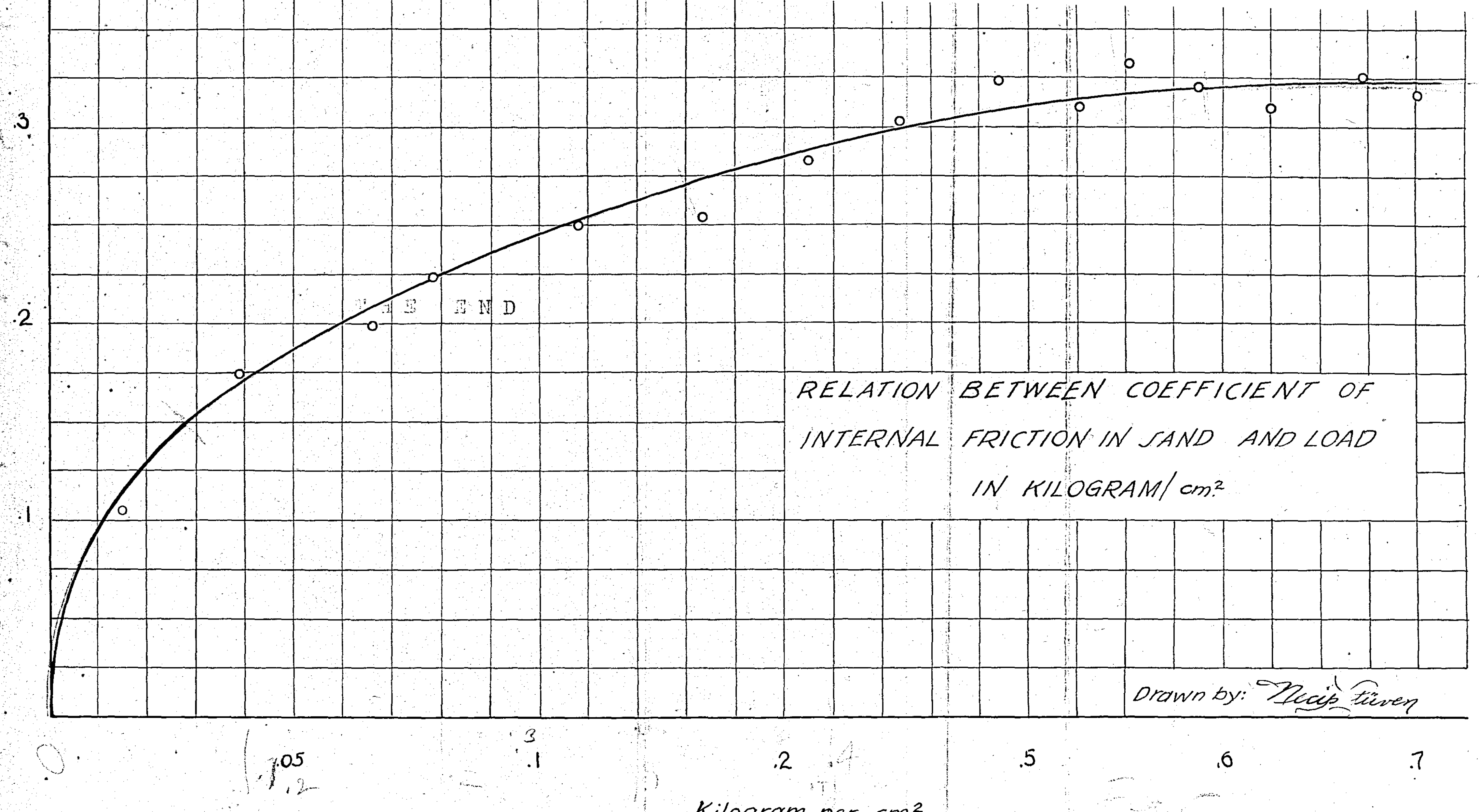

RELATION BETWEEN DEPTH
PENETRATED AND PULL
Drawn by:
DEPTH PENETRATED
in cm
0
10
20
30
40
50
.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0
Pull in Kgms.

Plot. 7

Mechanical Analysis of "Göksu" Sand



Plot. 1.



.3
.2
.1
0
.05
.1
.2
.5
.6
.7
THE END
RELATION BETWEEN COEFFICIENT OF INTERNAL FRICTION IN SAND AND LOAD IN KILOGRAM/cm.²
Drawn by:
Kilogram per cm.²

Plot. 3.

THE END