TEXAS HIGHWAY DEPARTMENT

FOUNDATION EXPLORATION AND DESIGN MANUAL

BRIDGE DIVISION

AUGUST 1956

FOUNDATION EXPLORATION AND DESIGN MANUAL

Prepared By The Bridge Division

Of

The Texas Highway Department

Austin, Texas

AUGUST 1956

COMMISSION

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September 19, 1956

IN REPLY REFER TO
FILE NO. D-5

Bridge Information Circular No. 3

Subject: Foundation Exploration and Design Manual

To: District Engineers, Engineer-Manager, and Resident Engineers.

Gentlemen:

Attached hereto is a copy of the "Foundation Exploration and Design Manual". It is essentially a revision of the second section of the Plan Preparation Book III.

The purpose of this manual is to present and explain the essential steps in the foundation exploration and foundation design of a structure. Consequently, it can be utilized to advantage by Resident Engineers and designers as well as on-the-job core drill loggers.

The major revision of this manual is a result of our recent research and study in the field of structure foundation design relative to drilled shafts in the utilization of side shear or skin friction and point bearing capacity.

Please study and note the design procedures and limitations which govern when utilizing skin friction as outlined in the manual on pages 53 and 54. Also see pages 62, 63 and 70. When frictional resistance is used to estimate the safe design on a straight shaft, the following note should be placed on the bridge layout sheets:

> "The sides of the drilled shafts below the Rock and/or Shale elevation shall be roughened to increase the frictional resistance."

The excellent co-operation of the personnel throughout the State in carrying out the standard foundation exploration and design procedures is greatly appreciated.

Yours truly,

D. C. Greer
State Highway Engineer

By Randle B. Alexander

Randle B. Alexander
Bridge Engineer

FOREWORD

This is a reprint of the second section of Book III of the Plan Preparation Manuals originally issued in December, 1952, slightly revised in February, 1953 and revised in August, 1956.

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GENERAL. As a general rule, the type, exact span lengths, cost, and to some extent, the appearance of a highway structure, are determined by a single factor: the natural foundation material available. The care used in foundation exploration should be commensurate, therefore, with the value of the information to be obtained. Foundation data should be sufficiently complete and accurate to provide the designer with a dependable basis for making a choice of structure type and an economic comparison of layouts and to permit planning on which construction may proceed with confidence of encountering a minimum of delays.

Many explorations have been made where a large number of test holes were drilled, but the overall factual information about the existing natural formations was very meager. The true evaluation of a foundation exploration should be made on the basis of the cost per foot of reliable information and not on the cost per foot of hole.

The science of Soil Mechanics and Foundation Engineering has made great advancement in the past decade. The ability of engineers to explore, sample, test and evaluate most earth formations in relation to substructure design has increased rapidly in the past few years. The natural result of this development has been the introduction of more economical substructure designs that better fit the existing conditions. It has made possible the wider use of the ''Drilled Shaft'' type of substructure both with and without "Underreaming".

Research and investigational work has shown that the shear strength of a soil is a reasonable measure of the load carrying capacity of a friction pile driven in that soil. This fact has made it possible to evaluate the accuracy of the generally accepted dynamic methods of determining pile capacities. The results of these evaluations indicate that we need a more realistic method of measuring pile capacities, and on large projects it generally will be found economically feasible to make pile load tests to establish the true pile capacity or to make complete soil strength tests as a basis for determining pile capacity. The actual testing of samples of the formations has resulted in more realistic allowable unit design loads being used when founding on shales, various rock formations and clays.

In summary it might be stated that the need for adequate explorations can be justified solely from an economic standpoint. Each project should be studied individually and the extent of the investigation based upon the magnitude of the project and the nature of the existing earth formation as related to the economics of the possible substructure designs.

OBJECTIVE:

The objective in structure foundation exploration is to determine, within the limits of the proposed structure, the elevation at which various earth strata exist, which information, together with the character, strength and description of the formations, will materially affect decisions on design.

Putting it plainly - the objective is to find out what is existing in order that the designing engineer can make a complete study and determine the most economical design. Simple as this sounds, it is amazing how often exploration work fails to accomplish this objective.

METHOD:

It is quite impossible to set forth a methodical rule to be followed in making foundation explorations due to the widely divergent job conditions encountered in the various parts of Texas. There are many instances where good foundation material is encountered at shallow depths and adequate investigations can be made by digging open pits. Then, those border line cases will be encountered where a good clay is available near the surface and rock also is available within easy reach. In these cases the engineer in charge of the exploration must be careful not to lose sight of the objective by making decisions for the designer and fail to furnish complete data upon which to make an impartial study of all possible design types. The size of the proposed structure will, of course, be a dominant factor of influence in deciding the method as well as the extent of the exploration.

Aside from the very shallow exploration work where the open pit method is adaptable, the majority of Highway Department exploration work is done with one of the rotary core drill rigs which operate out of the Camp Hubbard Shops. This equipment is routed by the Bridge Division and when operating in the field works under the District Engineer or his duly authorized representative. The equipment is constantly being improved as new problems are encountered. Constructive criticism of any part of the operation is always welcome and should be directed to the Bridge Engineer, File D-5, Austin.

Rotary core drill rigs operating out of Austin are mounted on 5 ton trucks with tandem rear axles. (Fig. 1) The rigs are powered by the truck engine through a "Power-Take-off" mechanism which utilizes the truck transmission and gives a wide range of power and speed at the drill head. Other features of the rigs include a reciprocating type of power mud pump, hydraulically powered pull down or "crowd", hydraulically retracting drill head and many other minor items that assist in obtaining core samples under very difficult conditions.

Exploration methods now in use on these rotary core drills can be divided into five main groups, namely: Wash Boring or Fish Tail Drilling, Dry Barrel or Single Wall Barrel Core Sampling, Wet Barrel or Double Wall Barrel Core Sampling, Push Barrel Sampling, and Cohesionless Sand Sampling.

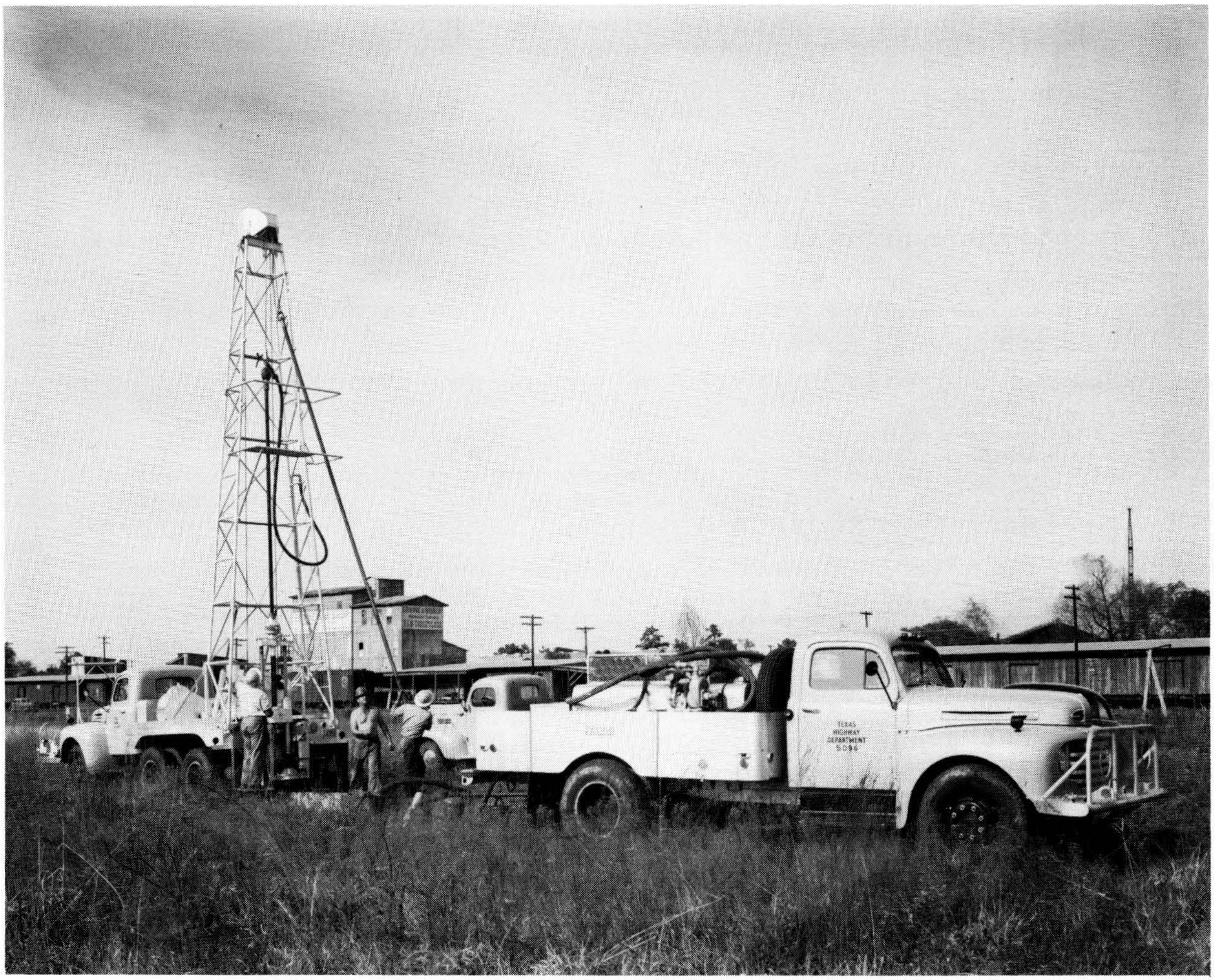


FIG. 1. ROTARY CORE DRILL RIG AND SUPPLY TRUCK - IN OPERATION

Wash boring or Fish Tail drilling should not be permitted until the classification of the strata has been definitely established and it is desired to drill a hole rapidly to establish the elevation at which a hard stratum exists below. The core drillers have been instructed not to use the wash boring method except when specifically directed by the district representative. Attempts to classify by watching the wash water leads to very erroneous conclusions and is to be avoided at all times. The wash boring method is a rapid way to make a hole through most all formations except rocks and hard shales, but when you are through, a hole in the ground is about all you have to show for your work.

Dry Barrel or Single Wall sampling is the method most generally used (Fig.2-A). The core sample obtained is generally in a disturbed condition due to the pressure applied to cut the core and pack it in the barrel so that it can be recovered. However, the core can be extracted from the barrel either by water pressure or by hydraulically powered piston extractor and a visual classification made. When used for sampling in practically all materials encountered except very soft mucks and cohesionless sands, the dry barrel sampler will give a sample containing all components in the original formation and the amount of disturbance will depend upon the softness of the formation. Although this method is called the dry barrel method, it should be pointed out that some cooling water is often used with this method and in the hard formations a small amount of water is circulated during the cutting of the core.

FIG. 2A DRY BARREL SAMPLER

Wet Barrel or Double Wall Barrel sampling is used in a wide range of formations when undisturbed core samples are desired. (Figs. 2-B, 2-C) The sampler used consists of an inner and outer barrel. The outer barrel is a thick wall tube with saw tooth cutter. The inner barrel is a thin wall tube connected to the head of the sampler on free running bearings. The outer barrel is rotated and cuts an annular ring around the core as the sample is received into the inner barrel. The inner barrel remains stationary due to friction between the core sample and the barrel wall. Water is circulated down the drill stem, thence between the inner and outer barrel picking up the cuttings from the annular ring, carrying them up around the outside of the outer barrel to the ground surface where they are deposited in a sump. A viscous mud slurry can be added to the circulating water to lift cuttings consisting of sands and gravels. There are several versions of the double wall barrel samplers. For formations other than rock we use a type that has a thin sheet metal liner that fits the inner core barrel and furnishes a handy method of removing the core as well as a protection to the undisturbed core while transporting same to the laboratory. For rock and hard shales the liner is omitted, as this type material has ample strength for handling without the protection of the liner. The relative projection of the inner and outer core barrel cutting bits can be varied by adding or subtracting collar washers. For rock and hard cutting materials it is necessary that the outer barrel cutter lead the inner cutter as the hard material cannot be penetrated by the knife edge cutter on the inner barrel. However, when taking a core in clays, sand clays, etc., the inner cutter is adjusted to lead the outer cutter and thereby protect the core from erosion by the circulating water. When the proper length core has been cut and received in the inner core barrel the circulating pump is shut off and the outer barrel rotated at a relatively high speed. This generates enough heat to cause the lower end of the core to expand and bind itself in the barrel while the sampler is withdrawn from the hole. This particular operation calls for a driller with skill, experience, and patience.

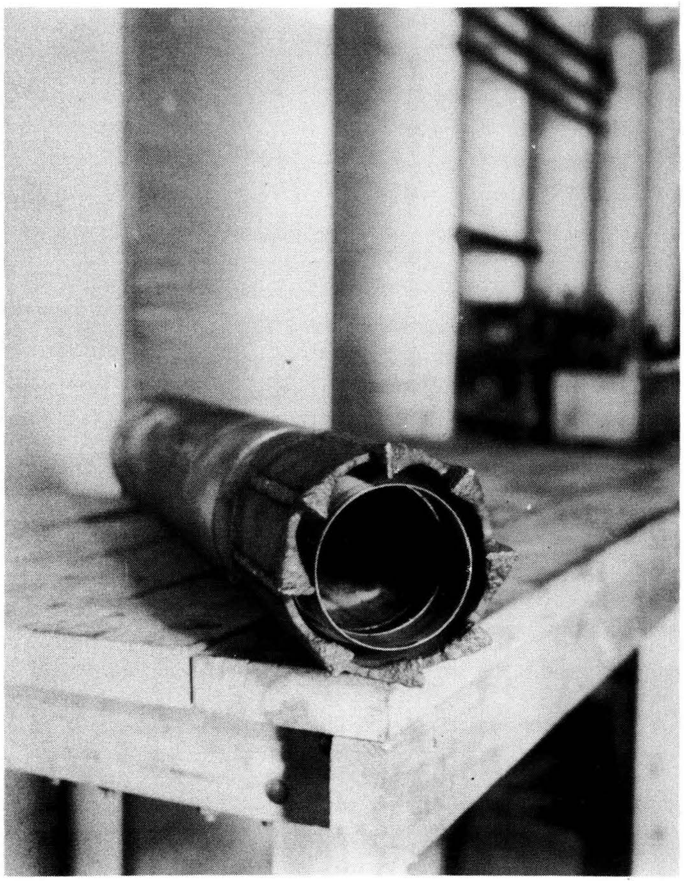


FIG. 2-B WET BARREL SAMPLER

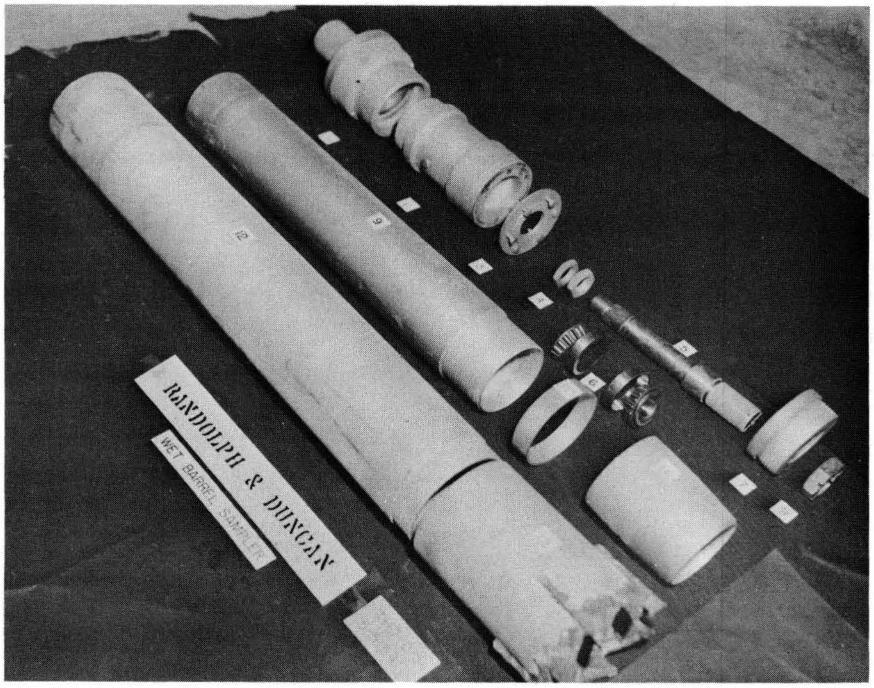


FIG. 2-C WET BARREL SAMPLER

The Push Barrel Sampler (Fig. 2-D) as the name implies, employs the simple principle of pushing a thin walled tube with a sharp cutting edge into the formation with the hydraulic push down on the drill rig. This type sampler recovers very good "Undisturbed" samples where it is adaptable but its usefulness is limited to materials into which it can be forced and which have sufficient cohesion to remain in the barrel while the sampler is withdrawn from the hole. The usual procedure is to force the sampler into the formation with a slow steady push and rotate it about two turns to break off the core before beginning the withdrawal. The push barrel sampler is faster than the double barrel sampler and is to be preferred where it is adaptable.

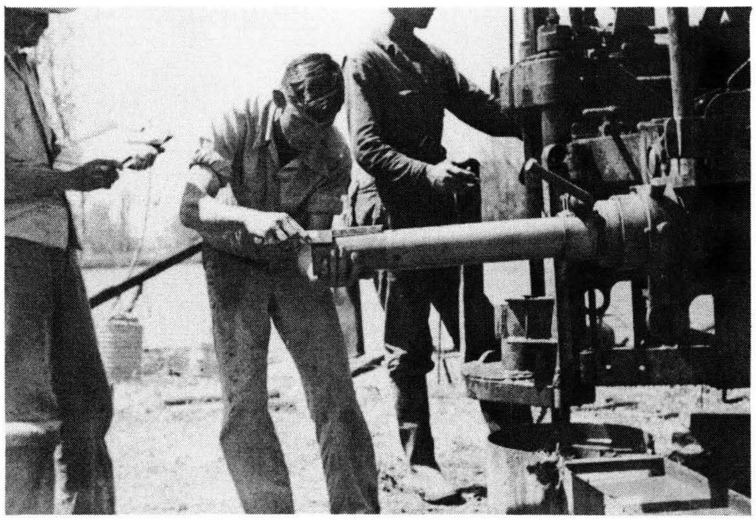


FIG. 2-D PUSH BARREL SAMPLER SHOWING CORE BEING EXTRACTED

The last and least used sampling tool is the cohesionless sand sampler. (Fig. 2-E) It is to some extent a combination of the last two named samplers. It consists of an outer barrel or air bell and inner barrel or sample tube. The use of this sampler is limited to very large projects where loose cohesionless sand exists and it is important that the density and,nature of the sand be determined. Due to the limited use of this tool, it is not considered desirable to spend the time describing its use. Complete description and details of the sand sampler can be obtained on request to the Bridge Division, File D-5, Austin.

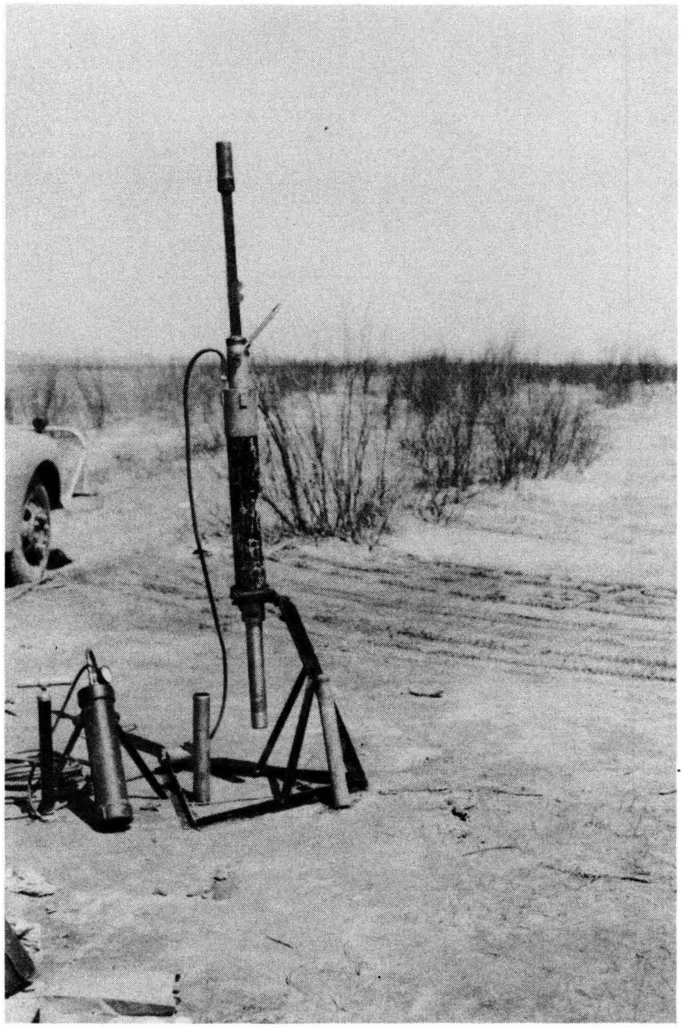


FIG. 2-E. COHESIONLESS SAND SAMPLER

FIG. 3. FIELD LABORATORY FOR TRIAXIAL TESTING

In addition to the above mentioned drilling tools each of the rotary core drills operating out of Austin are equipped to make the standard Penetrometer Test. (Figs. 4 & 5) This test consists of recording the number of blows of a 170 pound hammer dropping 24 inches that is required to force a 3 inch diameter steel cone 12 inches into a formation. In cases where hard formations are encountered, including rock, the instructions are to hit the pin 100 blows and accurately record the resulting penetration in inches for the first and second 50 blows. This test has been in use since 1950, and it is now standard procedure in all our exploration work to make the test at least each 10 feet of hole and more often if necessary in order that each significant formation is tested. Experience to date with the Penetrometer Test indicates that the number of blows of the hammer for the first 6 inches and the second 6 inches of penetration should be recorded separately as it is indicative of a granular material if the number of blows for the second 6 inches is significantly more than that for the first six inches. Curves based upon our experience to date with the Standard Penetrometer Test are attached to and supplement this paper. These curves show the relation between the test results and the shear strength of the soil as measured in the laboratory as well as the relation between the test results and measured dynamic pile resistance. The use of these charts will be discussed later under interpretation of results of subsurface explorations.

Reference is made to "Sample Test Boring Logs" as illustrated. It will be noted that where the Standard Penetrometer Test is run, the results are recorded showing the number of blows required for the first and second 6" of penetration separately. Whenever the penetrometer test deviates from this standard 12" penetration, it is necessary that the exact penetration in inches be shown on the log. The basic symbols for the various earth formations shown on this "Sample Test Boring Logs" as well as the above method for recording penetrometer test data is recommended as standard procedure for showing test boring data on plan layout sheets.

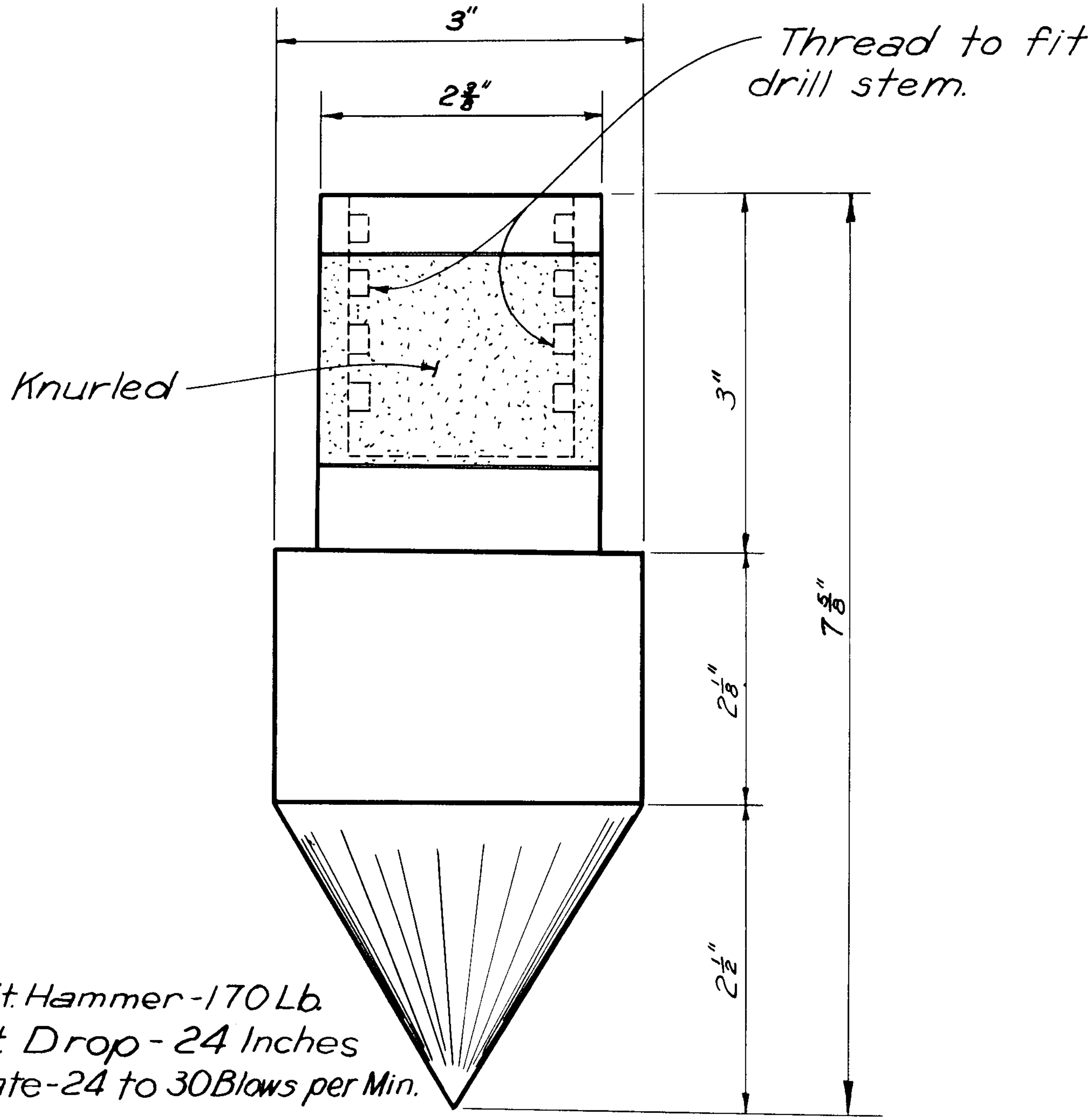
HAMMER UNIT

CLOSE-UP
TRIPPING MECHANISM

FIG. 4

AUTOMATIC TRIP HAMMER
USED IN MAKING THE STANDARD PENETROMETER TEST

STANDARD PENETROMETER TEST
CONICAL
DRIVING
POINT



Wt. Hammer - 170 Lb.
Ht. Drop - 24 Inches
Rate - 24 to 30 Blows per Min.

FIG. 5

BRIDGE DIVISION
JUNE 10, 1949

The first test hole at any Structure is generally made with very little advance knowledge of the subsurface materials which will be encountered. It should be made carefully and with 100% core recovery with the Dry Barrel Sampler or if necessary with the Wet Barrel Sampler. A good descriptive log of each significant stratum should be recorded. Penetrometer Tests should be made at each 10 foot interval of depth. This first hole should be carried well below the probable founding depth of the substructure. If so-called bed rock or a shale is encountered and is considered to be the probable formation on which the structure will be founded, it is recommended that this first hole be carried ten feet into the formation. For all formations other than rock or shale the hole should be carried to a depth below the probable founding elevation of approximately 50 feet. In applying this rule, where friction piles will probably be used, the founding elevation should be considered as the elevation corresponding to the center of resistance which may be assumed to be at the mid-point of pile penetration in the supporting earth strata. A rough determination of the founding elevation can be made from the results of Penetrometer Tests. A major structure is an exception to this rule in which case a more complete analysis should be made.

As a general rule it is suggested that no undisturbed sampling be attempted on the first hole. Upon completion of this first hole it should be possible to formulate a tentative plan of procedure for the remainder of the exploration. In formulating the tentative plan bear in mind that the completed exploration should contain the following:

1. Test holes at each end of the proposed structure plus a sufficient number of staggered intermediate test holes to determine the location of all significant earth formations well below the probable founding elevation. A reasonable correlation between adjacent holes should be obtained. The recommended maximum spacing of holes is 250 feet where the significant formations appear to be uniformly bedded as indicated by the descriptive log of each hole and the penetrometer tests. Geologic maps can be utilized to determine beforehand along with past experience the significant formations which are expected to be encountered.

2. At one test hole at each structure site the Penetrometer Test should be made at four foot intervals of depth in order to detect the presence of thin or obscure formations that may be missed in running the test at 10 foot intervals. In all other adjacent holes where the penetrometer test is run at 10 foot intervals, the pen test elevation should be varied 5 feet from one hole to the next in order to get a good test spread. <u>Penetrometer Tests should be taken as soon as Hard Clay, Shale or Rock stratum is encountered and at four foot intervals thereafter if it is likely that the proposed footing will be landed in the material.</u>

3. Undisturbed samples for strength tests where large structures are involved and also for small structures if the formations indicate that the final design may include any of the following:

(a) Friction piles to be driven in a formation showing less than 30 blows per foot with the Penetrometer.

(b) Underreamed drilled shafts to be founded in a material showing less than 30 blows per foot with the Penetrometer.

(c) Drilled shaft type of foundation without under-reaming if there is doubt about the safe allowable unit loads to be used.

4. A complete log record for each test hole on the Departmental Form 513, including the information called for at the top of the form. (See recommended logging terminology at end of this chapter.)

   On large jobs and on jobs where the formations are non-uniform, it is suggested that during foundation exploration a pencil profile be plotted showing the test data. A study of this profile will help in attaining the overall objective previously mentioned. See suggested sheet titled "Foundation Material Profile".

In addition to the rotary drill rig exploration, it is sometimes desirable to drill one or more large auger test holes to determine the feasibility of the drilled shaft type design. At the present time this is the only sure method for determining the presence of water bearing strata that may affect the design and should be resorted to where the information can be justified economically. The large auger hole exploration is also useful on underpass structures as it affords a convenient means for the engineers representing the railroad to make an inspection which may result in a more economical substructure design.

INTERPRETATION AND DESIGN:

The interpretation of the data obtained from subsurface explorations presents problems as complex as any encountered in the highway engineering field. The development of the technique of substructure design has lagged behind that in the field of superstructure design due to the difficulty in evaluating the strength and service characteristics of the subsurface materials.

However, significant progress has been made in the field of Soil Mechanics as it pertains to substructure design and much of the guess work, design changes during construction, and waste in overdesign can be avoided by application of recently proven techniques.

The interpretation of the test data is invariably tied in with the extent of the exploration and type of formation. For convenience the explorations are divided into four classes.

I. Plastic clay and sand-clay explorations where the taking of undisturbed samples appear unnecessary and the design is to be based upon observations and Penetrometer Tests.

II. Plastic Clay and Sand-Clay explorations where the use of undisturbed sampling and triaxial testing is indicated in addition to observed soil conditions and Penetrometer Tests.

III. Cohesionless Sand explorations both with and without undisturbed sampling but including visual classifications and Penetrometer Tests.

IV. Hard Clay, Shale and Rock explorations both with and without undisturbed sampling but including visual classifications and Penetrometer Tests.

Examples - Class I Explorations:

(a) Pile Foundation Design

Assume 14 inch square concrete Piles to be driven with No. 1 Vulcan Hammer. Required design load 28.0 tons per pile.

Test Hole Data:

0 feet - 15 feet Soft Gray Clay 4(6") 4(6")

15 feet - 38 feet Med. F. Tan Clay 8(6") 10(6")

38 feet - 75 feet Firm Tan Sandy Clay 19(6") 24(6")

Low water table at 8 feet depth.

Assume 8 feet Alignment hole. Using Correlation curve in Fig. 6, for dynamic resistance and correlation curve in Fig. 7 for estimating the static capacity of the pile the following table can be completed.

Depth Ft.	Estimated Dynamic Res. Tons (Fig. 6)	Frict. Area Pile Sq. Ft.	Static Resistance in Tons/Sq. Ft. (Fig. 7)	Estimated True Capacity Tons
8 -15 (7 Ft.)	4	32.7	0.13	4.
15-38 (23 Ft.)	18	107.4	0.24	25.
38-42 (4 Ft.)	7	18.7	0.56	10.
Total	29			39

This shows that we could expect to obtain the design capacity by the hammer formula with pile penetrations of about 42 feet, whereas if complete soil tests or a pile load test were made we probably would need only about 38 feet of penetration. The final decision is an economic one but ordinarily a saving of only 4 ft. per pile would not justify the time or expense of the more extensive investigation.

(b) Spread Footing or Drilled Shaft Design.

Assume same foundation condition as above example.

Past experience has shown that it is not safe practice to land a footing in a material with a penetrometer test of less than 30 blows per foot without making strength tests. We will assume for our example that the proposed structure is small and the soil strength tests cannot be justified. Therefore, we will not consider landing above the 38 ft. depth, and our problem is to determine the safe allowable unit load in the material showing 43 blows per foot with the penetrometer. From the correlation curves in Fig. 8, we find that 43 blows per foot on the lower curve shows an allowable bearing of 2.02 tons per square foot and the upper curve shows an allowable bearing of 3.54 tons per square foot. A visual inspection of the material indicates that a value of 2.8 tons per square foot of bearing would be conservative. The accuracy of this step in the solution is naturally dependent upon one's experience with soils. However, the descriptive terminology shown for the three curves in Figure 8 will make it possible to obtain a reasonably safe allowable bearing for a given soil condition. The use of the values from these curves will always give conservative design and where the proposed structure is of considerable size, sound engineering will dictate that soil strength tests be run.

The 2.8 tons per square foot obtained from the curves can then be used as a basis for making an economic study of this design as compared with the pile foundation design determined in the first example.

Examples - Class II Explorations

It is not considered within the scope of this manual to cover the details of triaxial testing. Reference is made to a paper entitled "Triaxial Testing: Its Adaption and Application to Highway Materials with Addenda No. 1" which was developed and reported by the Soils Section of the Materials and Test Laboratory and distributed by Administrative Letter 43-50 together with several good texts on the subject such as "Fundamentals of Soil Mechanics" by Donald W. Taylor, "Soil Engineering" by M. G. Spangler, and "Soil Mechanics in Engineering" by Terzaghi and Peck. In the following examples, it is assumed that adequate triaxial tests are complete and the "Rupture" or "Strength" line has been determined on the Mohr's diagram for each significant formation involved.

(a) Pile Foundation Design

For an example in estimating pile lengths based upon soil strengths refer to Fig. 9, which shows a complete study for 14 inch precast concrete pile lengths on a grade separation structure on U. S. 75 in Galveston County.

Fig. 10 shows the tabulated data for each significant strata based upon triaxial test results; also see sample sheet entitled "Calculated Static Capacity of Friction Piles" for a form on which this data may be tabulated from which graphs may be made.

Fig. 11 shows the rupture or strength line for the "Firm Silty Clay" stratum at 36 to 40 ft. depth. This Rupture or Strength line was the result of drawing a line tangent to the Mohr's strength circles which were obtained from a series of triaxial tests of undisturbed samples of this particular stratum. Following thru the computations for this 38 to 40 ft. depth material on Fig. 10, the submerged density is shown as 57.2 which was obtained by subtracting 62.5 from the average wet density of 119.7, all in lbs. per cubic foot. The average depth of 38 ft. is the mid-point of the 36 to 40 ft. depth. The overburden pressure is assumed to act hydrostatically and is calculated by the equation:

$$U = \frac{WD}{144} = \frac{57.2(38)}{144} = 15.1 \text{ p.s.i.}$$

in which
U = Overburden pressure in lbs. per sq. inch.
W = Submerged Density in lbs. per cu. ft.
D = Average Depth in feet.

The average shearing strength of the stratum can then be taken graphically from the diagram Fig. 11, which is found to be 10.0 p.s.i. or 1440 p.s.f. This value can be calculated if preferred by scaling the value of cohesion, c = 4.5 p.s.i. and the angle of internal friction, $\emptyset = 20$ degrees, from the same diagram and using the following Coulomb's equation:

$$S = c + U \tan \phi = 4.5 + 15.1\ (0.364).$$

$$S = 10.0 \text{ p.s.i.}$$

The pile surface area within the 4 ft. stratum is 4.67(4) = 18.7 sq. ft. The ultimate capacity of the pile within the 38 to 40 ft. stratum is calculated with the following equation:

$$P' = Sa = 1440\ (18.7) = 26,928 \text{ lbs.}$$

$$P' = 13.5 \text{ tons.}$$

A theoretically more exact answer may be obtained by using Mohr's theory for shearing strength which can be taken graphically from the diagram, Fig. 11 or calculated with the following equation:

$$S = (c + U \tan \phi)\ (1 + \sin \phi)$$

$$S = 13.4 \text{ p.s.i. or } 1930 \text{ p.s.f.}$$

$$P' = Sa = 1930\ (18.7) = 36,084 \text{ lbs.}$$

$$P' = 18.0 \text{ tons.}$$

A factor of safety of 2 is applied to this ultimate capacity for each of the strata and the accumulated static capacity curve using submerged densities is plotted as shown in Fig. 9. This curve shows that the design load of 31.4 tons will require that the pile tip be driven to 40 ft. depth. As an interesting follow-up of this problem, it will be noted that the dynamic driving resistance actually obtained was 17.2 tons as shown by the short dash curve in Fig. 9. This pile was load tested and proven adequate for a design load in excess of 45 tons. Time did not permit running the test to theoretical pile failure but the net settlement obtained indicated the pile could have been proven safe for a design load of over 52 tons which was the indicated capacity based upon using the wet density of the soil in computing the overburden pressure.

(b) Underreamed Drilled Shaft and Spread Footing Design

For this example reference is made to Fig. 11, showing graphically the results of triaxial tests on the 36.0 to 40.0 ft. depth stratum used in the above example. It is assumed that the footing is to be landed so that the point of maximum shear will occur at 38 ft. depth and it is desired to calculate the maximum safe unit design load using a factor of safety of 2. This is an approximate graphic solution which is based upon stress equations and assumptions which will give conservative results when used within the limitations noted.

Assumptions (1), (2) & (4) and Stress Equations (3-A), (3-B) & (3-C)

$$Z = 0.707\, r \qquad (1)$$

$$U = W(d + Z)\, 12 \qquad (2)$$

$$P' = \frac{V - U}{0.808} \qquad (3\text{-A})$$

$$P' = \frac{H - U}{0.23} \qquad (3\text{-B})$$

$$P' = \frac{X}{0.289 \operatorname{Cos} \frac{(\emptyset + 34^\circ)}{2}} \qquad (3\text{-C})$$

$$P = \frac{P'}{F.S.} \qquad (4)$$

Where:

Z = depth, in feet, from bottom of footing to point of maximum shear stress in soil.

r = radius of footing or radius of equivalent circular area for non-circular footings, in feet.

U = overburden soil pressure, assumed to act hydrostatically, in pounds per sq. inch.

W = average density of soil overburden in pounds per cu. inch. (Conservative practice requires that we use submerged density for substructures in stream beds, or where surface drainage is poor, and where the overburden soil is sandy).

d = depth, in feet, from surface of ground to bottom of footing. Where material is subject to scour, take $\underline{d}$ as distance from point of maximum scour to bottom of footing.

V = vertical unit stress or major principal stress expressed in pounds per sq. inch.

L or H = lateral unit stress or minor principal stress expressed in pounds per sq. inch.

P' = unit load on soil at footing elevation that will result in theoretical failure of soil in shear.

P = maximum safe unit design load in soil at footing elevation based upon a given Factor of Safety (F.S.) usually taken as 2.

X = point of intersection of Stress Line and Rupture Line read on vertical scale, Fig. 11.

P' and P can be expressed either in pounds per sq. inch or per sq. ft. Ultimately, P is usually expressed in Tons per sq. ft.

The angle of 33°-50' (use 34°) which the "Stress Line" makes with the horizontal axis and the influence values in the above stress equations are based upon as assumed Poissons Ratio of 0.5.

Solution:

Fig. 11 shows the Rupture or Strength Line of the 36.0 to 40.0 ft. stratum as plotted from the Triaxial test data.

The "Stress Line" is drawn in making an angle of 34° with the horizontal axis and passes thru the value of U = 15.1 p.s.i. on the horizontal axis which, as determined by the preceding example, is the overburden soil pressure for this example.

The maximum stress circle is then drawn in tangent to both the "Stress Line" and the "Rupture Line". Where the right side of this maximum stress circle cuts the horizontal axis, the value of V = 83.5 p.s.i. is obtained and where the left side of the maximum stress circle cuts the horizontal axis the value of H = 34.6 p.s.i. is obtained.

P' can then be obtained by substituting the value of V in equation (3-A) or the value of H in equation (3-B) as shown in Fig. 11, giving a value of P' = 84.8 p.s.i. by either equation. This problem can also be solved or checked by equation (3-C).

Using a factor of safety of 2, compute the value of P = 42.4 p.s.i. or --- P = 3.05 tons per sq. ft.

With this information, the designer can make an economic comparison of the pile and underreamed drilled shaft designs. On this particular project the pile foundation was chosen because of the reasonable doubt that existed as to the feasibility of underreaming due to the water bearing characteristics of the stratum. Also, the cost differential was small. If the cost differential had been significant, an auger test hole could have been justified to verify the feasibility of underreaming.

In addition to the assumptions stated above, this method of estimating the safe allowable design load on a soil is applicable only when:

1. The depth of the footing below the point of maximum scour is greater than the footing diameter.

2. The foundation soil is a plastic or semi-plastic type of material.

3. The foundation soil is of uniform or of increasing strength for a considerable depth below footing.

4. In case of rectangular footings, the length is not greater than 1 1/2 times the width.

5. The triaxial test results are based upon reasonably undisturbed samples of the strata involved and a sufficient number of tests were made to obtain representative soil strengths.

III. Cohesionless Sand Explorations

This type of formation does not lend itself to undisturbed sampling for Triaxial Testing. Undisturbed samples for density tests can be obtained with the Sand Sampler previously mentioned,but the operation is slow, tedious and costly and is not justified except on large projects. The usual procedure is to make an adequate number of Penetrometer tests upon which to base an interpretation. If the sand is known to be cohesionless and the Penetrometer shows less than about 30 to 45 blows per foot without much increase in the number of blows for the second 6 inches of penetration, then the sand is in a very loose state and will be a very poor foundation.

If the sand shows a marked increase in the number of blows for the second 6 inches of penetration under the Penetrometer and the number of blows per foot is above about 45, then the sand is reasonably dense and will become more dense when loaded. A conservative estimate of the static capacity of a friction pile, in such a sand, can be made by assuming an angle of internal friction of 30^{o} and applying the basic Coulomb equation:

$$R = \frac{(C+Wh \tan \phi)A}{F.S.} \qquad (5)$$

Where:

C = Cohesion = 0 (in case of sand)

W = Submerged density of sand (Use 50#/c.f.)

h = distance (in feet) below maximum scour depth to center of area of pile.

$\tan \phi$ = 0.577 (Assuming $\phi = 30^{o}$)

A = Surface area (sq. ft.) of pile in friction below the point of maximum scour.

R = design pile capacity in pounds

F.S. = Factor of Safety (Use 2)

Example

Find required penetration of a 15 inch square precast concrete pile to carry a design load of 35 tons in a deep cohesionless sand showing a Penetrometer Test value of 21 blows for first 6 inches of penetration and 48 blows for the second 6 inches of penetration. Assume maximum scour depth to be 15 feet.

Area of 15 inch sq. pile

= 5 sq. ft. per foot of penetration

Then A = 5p

where p = effective penetration of pile below point of maximum scour.

Substituting in equation (5)

$$35 \times 2000 = \frac{(0 + 50 \frac{(p)}{2} \; 0.5777)5p}{2}$$

p = 44 ft.

Required penetration would then be 44 ft. plus 15 ft. (scour depth) = 59 ft. total.

It is quite evident that the maximum scour depth is very important and good judgment must be exercised in its determination.

IV. Hard Clay, Shale and Rock Explorations.

Materials in this class of exploration will show less than 12 inch penetration with 100 blows under the Penetrometer Test.

(a) Pile Foundation Design

As shown in Fig. 6, a pile driven into materials of this class will reach refusal with a few feet of penetration and the maximum safe design load will be governed by the capacity of the pile as a structural member. Under this condition the pile will be a point bearing pile and sufficient penetration must be required to give adequate lateral support.

(b) Drilled Shaft and Spread Footing Design

The sampling and testing of materials of this class exploration have been sub-divided. Materials with Penetrometer Tests showing 3-1/2 inches and less per 100 blows are sampled and tested with unconfined compression tests. The maximum safe allowable unit pressure is taken as one-half the ultimate crushing strength for design purposes. This results in an ultra-conservative use of the material strength in its confined state and occasionally a more complete analysis is justified. Materials with Penetrometer Tests showing more than 3-1/2 inches per 100 blows are tested triaxially and the safe allowable design load is obtained as outlined in the problem of Figure 11.

On projects where undisturbed samples cannot be justified, a conservative estimate of the maximum safe allowable unit design pressures can be made from the results of Penetrometer Tests and the use of the correlation curves shown in Fig. 12A or Fig. 12B if applicable.

Precautions to be observed in the interpretation of results. Make sure that:

(1) The landing elevation of the footing is below the point of maximum scour and below significant moisture fluxuations.

(2) The earth stratum on which the footing is to be founded is of uniform or of increasing strength for at least five feet below the proposed founding elevation and is free from soft or yielding formations for at least 15 feet below the founding elevation.

(3) An adequate number of tests have been run to be certain they represent the actual condition of the material.

Recent research and study in the field of structure foundation design relative to cast-in-place concrete piles or drilled shafts indicate that the utilization of side shear or skin friction and point bearing capacity is a logical and safe design procedure if combined and used under certain limitations. If the formation involved is ROCK and/or SHALE, the skin friction capacity should be evaluated by triaxial tests, unconfined compression tests and/or penetrometer correlation curves, Figures 11 & 12B. (See sample sheet)

The following design procedures and limitations govern when utilizing skin friction:

(1) In general, Skin Friction should be considered only in the lower strata of material.

(2) Skin Friction should be utilized only below the point of maximum scour and below point of significant moisture fluxuations.

(3) Skin Friction should not be counted above the elevation at which casing is set.

(4) The earth stratum or strata in which the shaft is to be founded is of uniform or of increasing strength for at least five (5') feet below the proposed founding elevation and is free from soft or yielding formations for at least fifteen feet (15') below the founding elevation.

(5) An adequate number of tests have been run to ascertain the actual strength of the material involved. F.S. = 3 should be used.

(6) The intention and basis of the design should be clearly shown on the plans for checking purposes.

(7) The portion of the wall of the drilled shaft where frictional resistance is to be considered should purposely be left rough to obtain intimate bond between concrete and earth formation.

(8) Exposure to the air of a shale should be kept to a minimum. As soon as the shaft is drilled and cleaned, concrete and steel should be placed.

(9) Item 416.3 and pertinent special provisions thereto shall govern.

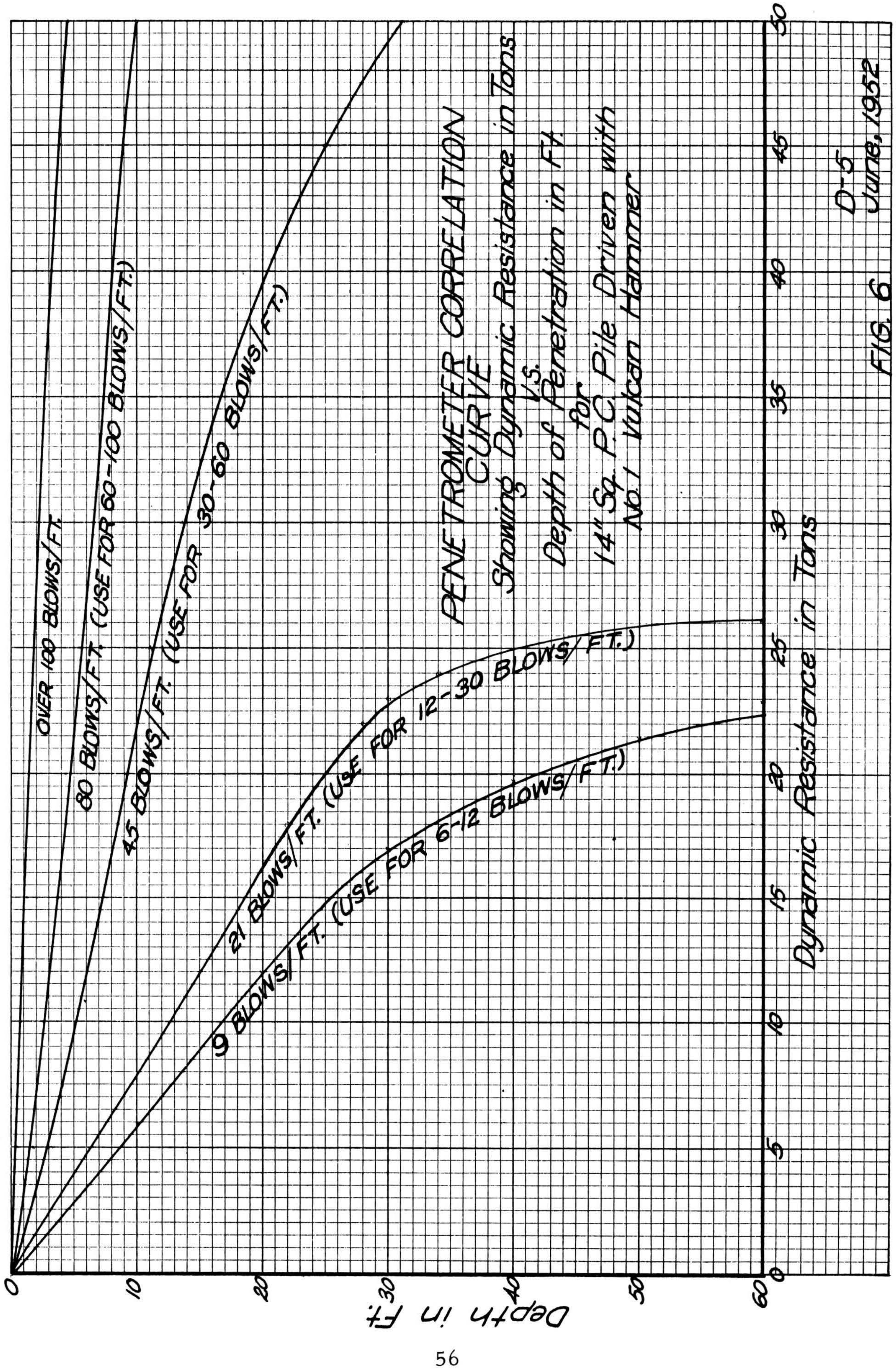
(10) Adequate site exploration program should be carried out as outlined on pages 37 & 38.

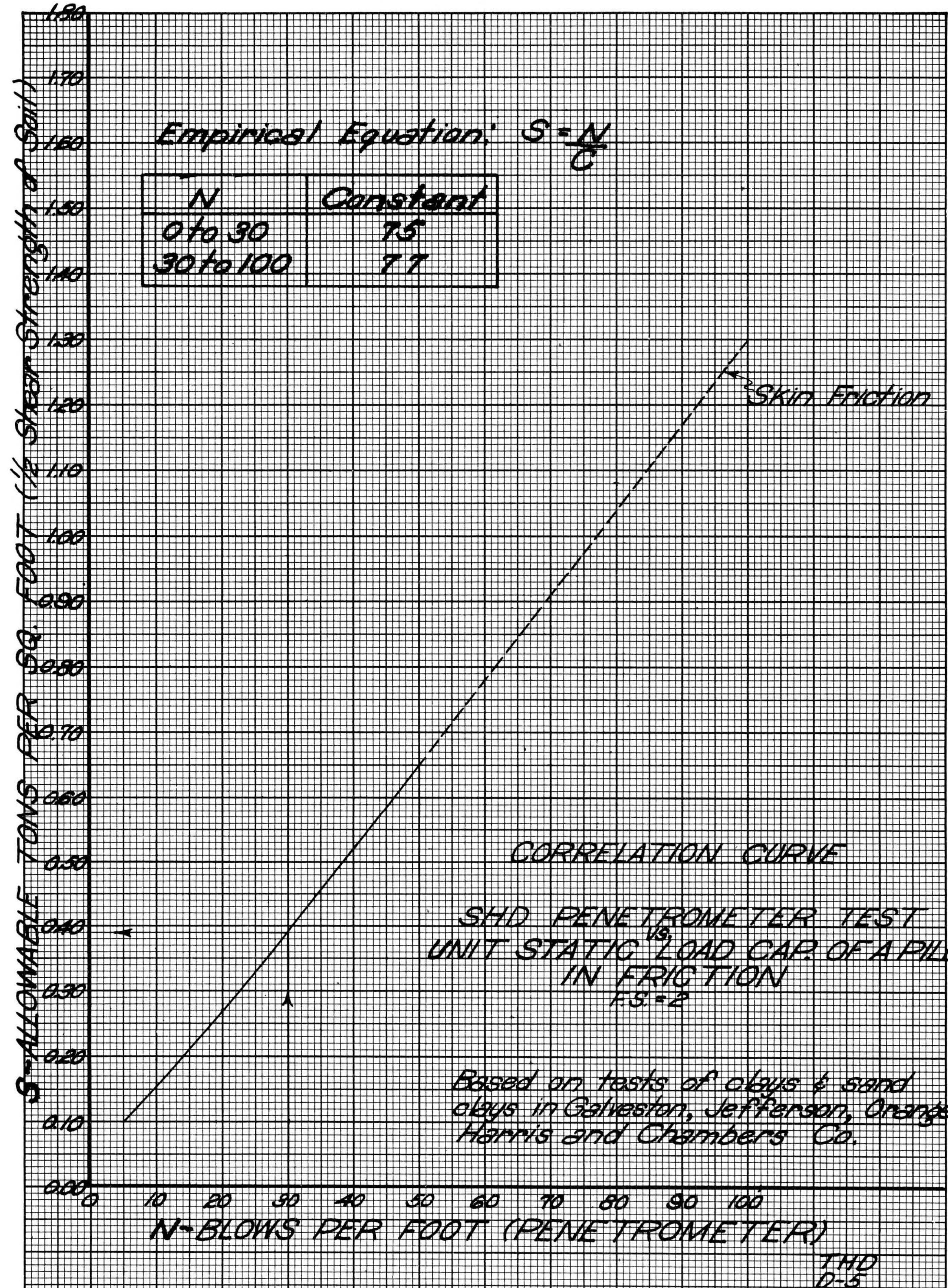
Conclusion:

The interpretative procedures outlined herein are believed to represent sound conservative engineering practice. However, the correlation curves are based upon experience of the Department to date and are subject to revision as more information is obtained. The use of the curve as well as the graphic method of interpreting the triaxial test data should be accompanied by good sound engineering judgment.

The true cost of an adequate foundation exploration is measured not by the preliminary cost but by the preliminary cost less the saving in construction cost as a result of the adequate exploration.

In addition to this saving in the design of the structure, reliable exploration data will result in better relations between the Contractors and the State, which will ultimately result in lower bid prices.


PENETROMETER CORRELATION CURVE
Showing Dynamic Resistance in Tons
Vs.
Depth of Penetration in Ft.
for
14" Sq. P.C. Pile Driven with
No. 1 Vulcan Hammer
D-5
June, 1952
FIG. 6
OVER 100 BLOWS/FT.
80 BLOWS/FT. (USE FOR 60-100 BLOWS/FT.)
45 BLOWS/FT. (USE FOR 30-60 BLOWS/FT.)
21 BLOWS/FT. (USE FOR 12-30 BLOWS/FT.)
9 BLOWS/FT. (USE FOR 6-12 BLOWS/FT.)
Dynamic Resistance in Tons
0 5 10 15 20 25 30 35 40 45 50
Depth in Ft.
0 10 20 30 40 50 60


Empirical Equation: S = N/C
N | Constant
0 to 30 | 75
30 to 100 | 77
Skin Friction
CORRELATION CURVE
SHD PENETROMETER TEST
vs.
UNIT STATIC LOAD CAP. OF A PILE
IN FRICTION
F.S. = 2
Based on tests of clays & sand clays in Galveston, Jefferson, Orange, Harris and Chambers Co.
S - ALLOWABLE TONS PER SQ. FOOT (½ Shear Strength of Soil)
N - BLOWS PER FOOT (PENETROMETER)
0.00
0.10
0.20
0.30
0.40
0.50
0.60
0.70
0.80
0.90
1.00
1.10
1.20
1.30
1.40
1.50
1.60
1.70
1.80
0
10
20
30
40
50
60
70
80
90
100
THD
D-5
Rev. 11-55
FIG. 7

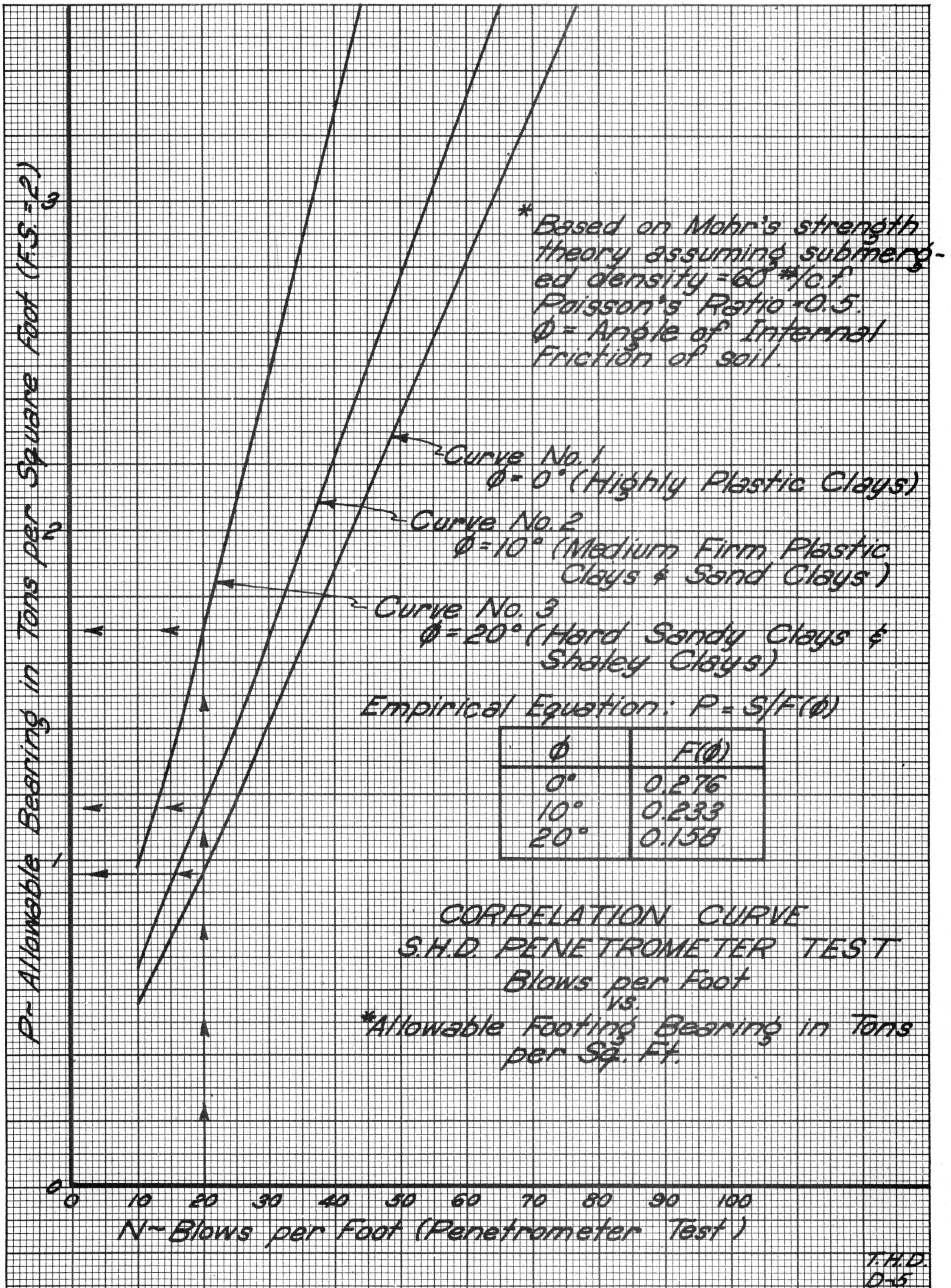

P- Allowable Bearing in Tons per Square Foot (F.S.=2)
0
1
2
3
0
10
20
30
40
50
60
70
80
90
100
N~Blows per Foot (Penetrometer Test)
*Based on Mohr's strength theory assuming submerged density = 60 #/c.f. Poisson's Ratio = 0.5. Ø = Angle of Internal Friction of soil.
Curve No. 1
Ø = 0° (Highly Plastic Clays)
Curve No. 2
Ø = 10° (Medium Firm Plastic Clays & Sand Clays)
Curve No. 3
Ø = 20° (Hard Sandy Clays & Shaley Clays)
Empirical Equation: P = S/F(Ø)
Ø | F(Ø)
0° | 0.276
10° | 0.233
20° | 0.158
CORRELATION CURVE
S.H.D. PENETROMETER TEST
Blows per Foot
vs.
*Allowable Footing Bearing in Tons per Sq. Ft.
T.H.D.
D-5
Rev. 11-55

Fig. 8

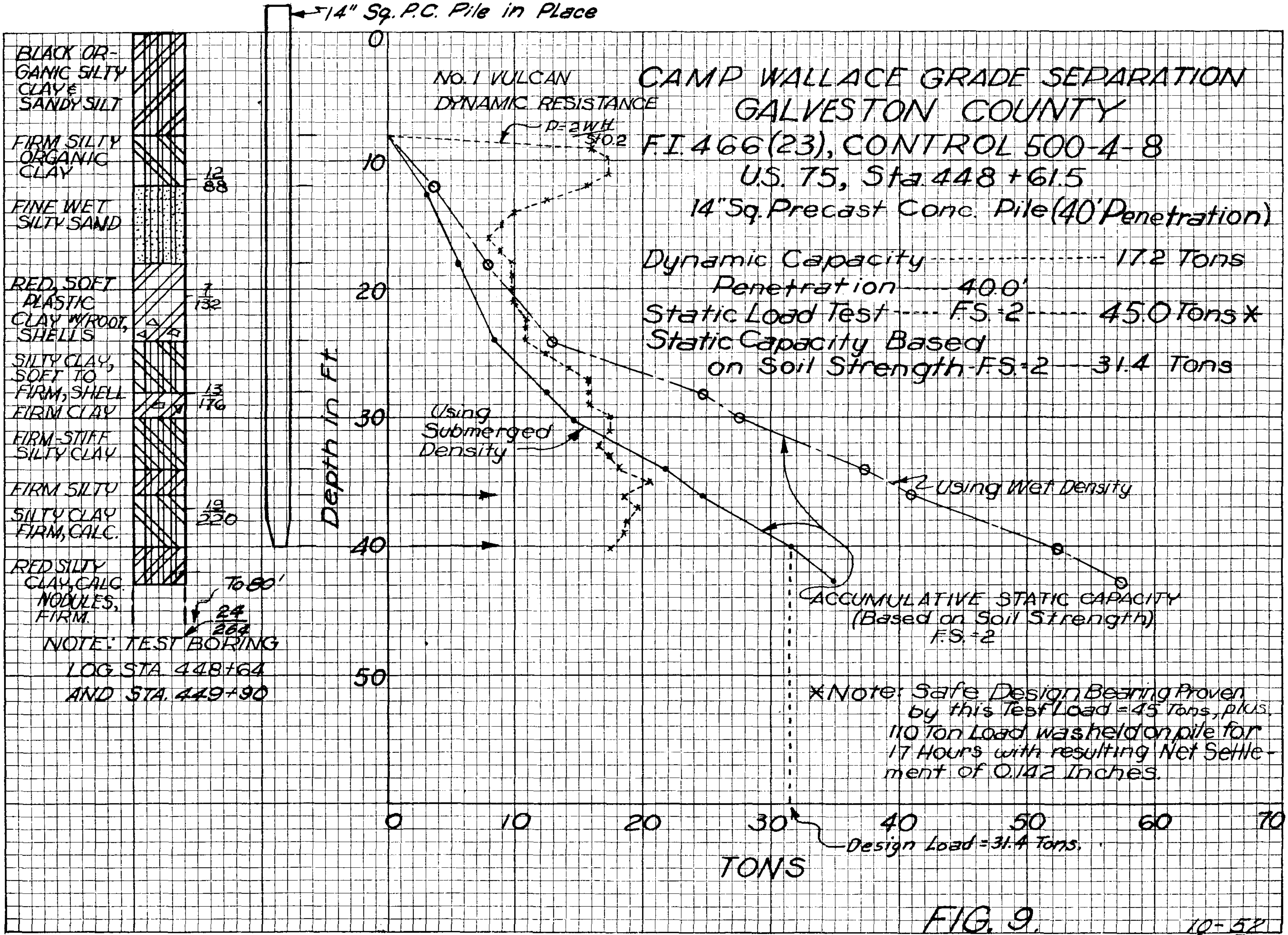

14" Sq. P.C. Pile in Place
CAMP WALLACE GRADE SEPARATION
GALVESTON COUNTY
F.I. 466(23), CONTROL 500-4-8
U.S. 75, Sta 448 + 61.5
14" Sq. Precast Conc. Pile (40' Penetration)
Dynamic Capacity ---------- 17.2 Tons
Penetration ---- 40.0'
Static Load Test ---- F.S.=2 ---- 45.0 Tons *
Static Capacity Based
on Soil Strength - F.S.=2 --- 31.4 Tons
No. 1 VULCAN
DYNAMIC RESISTANCE
P=2WH/S+0.2
BLACK ORGANIC SILTY CLAY & SANDY SILT
FIRM SILTY ORGANIC CLAY
FINE WET SILTY SAND
RED, SOFT PLASTIC CLAY W/ROOT, SHELLS
SILTY CLAY, SOFT TO FIRM, SHELL
FIRM CLAY
FIRM-STIFF SILTY CLAY
FIRM SILTY
SILTY CLAY FIRM, CALC.
RED SILTY CLAY, CALC. NODULES, FIRM.
12/88
7/132
13/176
19/220
24/264
To 80'
NOTE: TEST BORING LOG STA. 448+64 AND STA. 449+90
Depth in Ft.
Using Submerged Density
Using Wet Density
ACCUMULATIVE STATIC CAPACITY (Based on Soil Strength) F.S.=2
*Note: Safe Design Bearing Proven by this Test Load = 45 Tons, plus. 110 Ton Load was held on pile for 17 Hours with resulting Net Settlement of 0.142 Inches.
Design Load = 31.4 Tons.
TONS
10-52
D-5

FIG. 9.

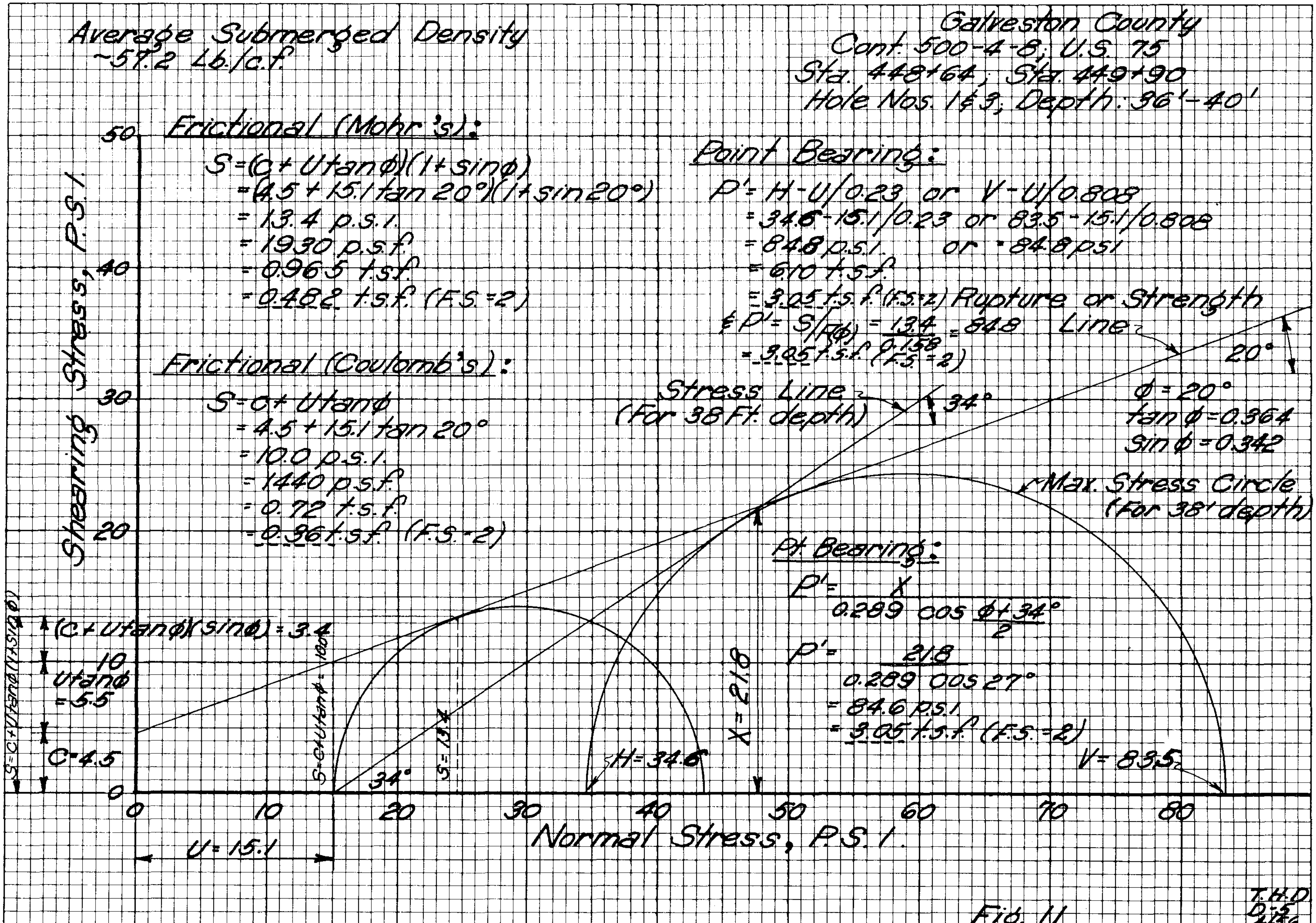
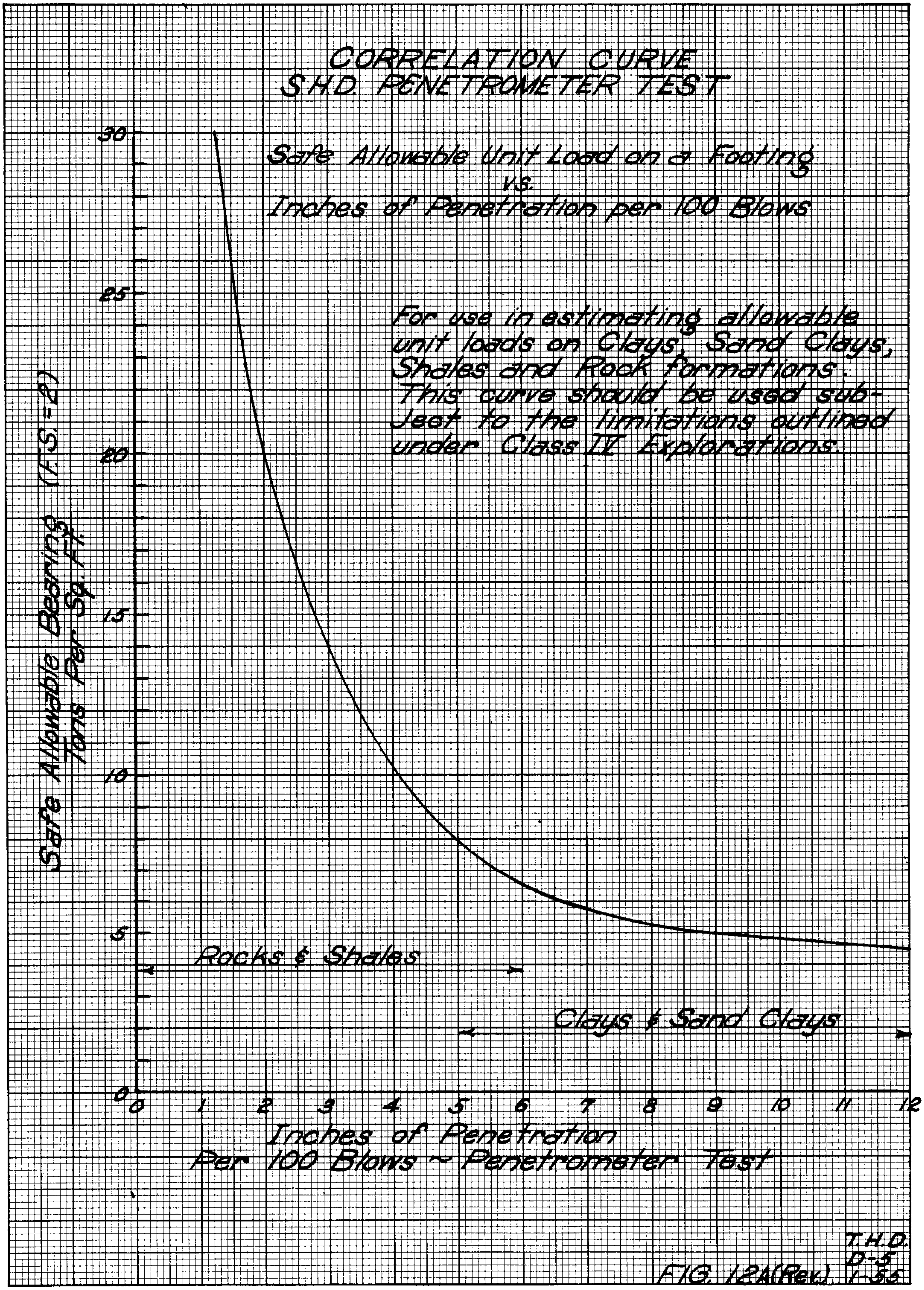
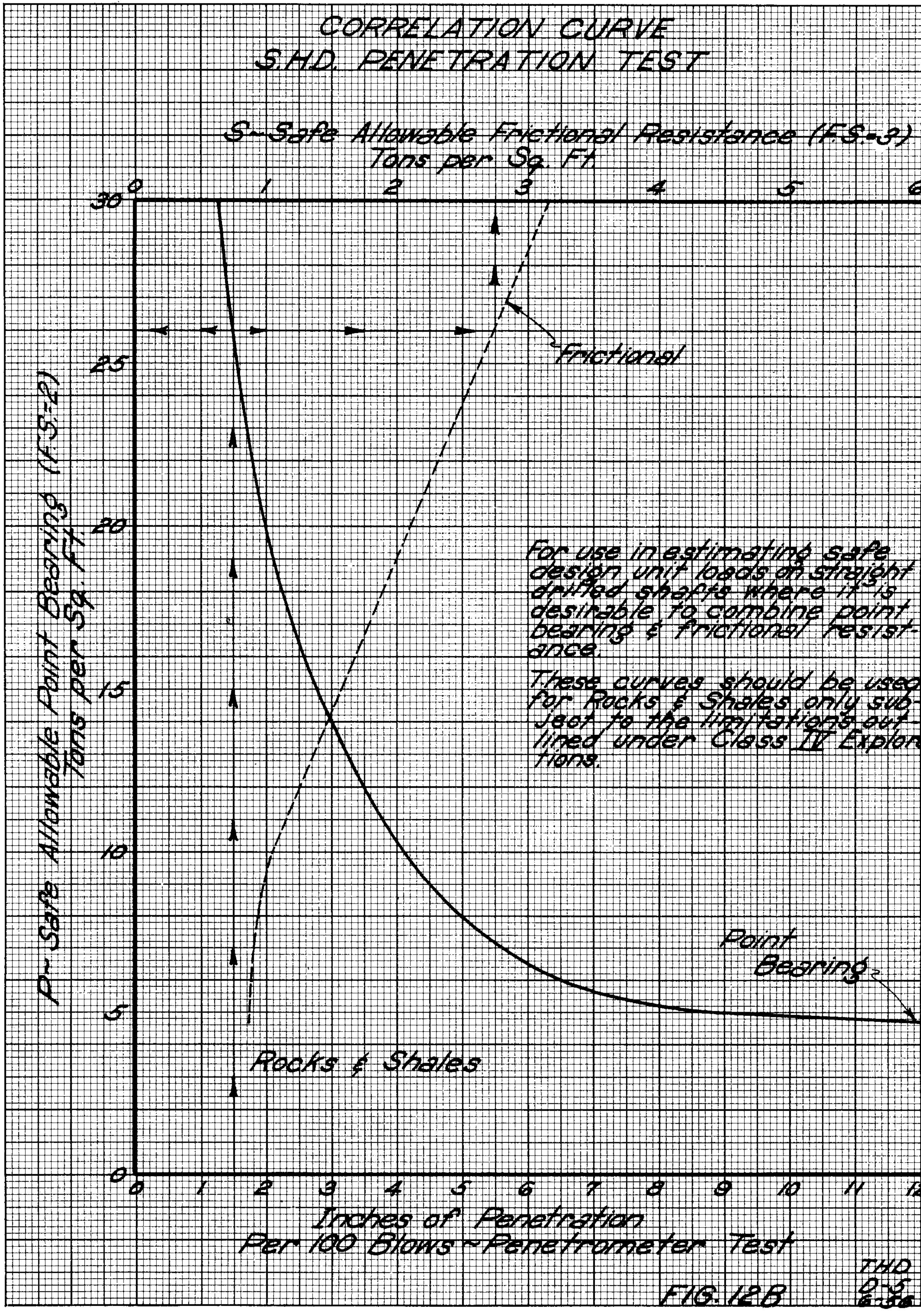

Average Submerged Density
~57.2 Lb./c.f.
Galveston County
Cont. 500-4-8; U.S. 75
Sta. 448+64; Sta. 449+90
Hole Nos. 1 & 3; Depth: 36'-40'
Frictional (Mohr's):
S=(c+U tan φ)(1+sin φ)
=(4.5+15.1 tan 20°)(1+sin 20°)
=13.4 p.s.i.
=1930 p.s.f.
=0.965 t.s.f.
=0.482 t.s.f. (F.S.=2)
Frictional (Coulomb's):
S=c+U tan φ
=4.5+15.1 tan 20°
=10.0 p.s.i.
=1440 p.s.f.
=0.72 t.s.f.
=0.36 t.s.f. (F.S.=2)
Point Bearing:
P'=H-U/0.23 or V-U/0.808
=34.6-15.1/0.23 or 83.5-15.1/0.808
=84.8 p.s.i. or =84.8 psi
=6.10 t.s.f.
=3.05 t.s.f. (F.S.=2)
& P'=S/F(φ)=13.4/0.158=84.8
=3.05 t.s.f. (F.S.=2)
Rupture or Strength Line
20°
Stress Line (For 38 Ft. depth)
34°
φ=20°
tan φ=0.364
sin φ=0.342
Max. Stress Circle (For 38' depth)
Pt. Bearing:
P'= X / 0.289 cos (φ+34°)/2
P'= 21.8 / 0.289 cos 27°
=84.6 p.s.i.
=3.05 t.s.f. (F.S.=2)
X=21.8
H=34.6
V=83.5
S=c+U tan φ=10.0
S=13.4
(c+U tan φ)(sin φ)=3.4
U tan φ=5.5
C=4.5
S=c+U tan φ(1+sin φ)
U=15.1
Shearing Stress, P.S.I.
Normal Stress, P.S.I.
0
10
20
30
40
50
60
70
80
T.H.D.
D-5
4/56

Fig. 11


CORRELATION CURVE
S.H.D. PENETROMETER TEST
Safe Allowable Unit Load on a Footing
vs.
Inches of Penetration per 100 Blows
For use in estimating allowable unit loads on Clays, Sand Clays, Shales and Rock formations. This curve should be used subject to the limitations outlined under Class IV Explorations.
Safe Allowable Bearing (F.S.=2)
Tons Per Sq. Ft.
30
25
20
15
10
5
0
Rocks & Shales
Clays & Sand Clays
0 1 2 3 4 5 6 7 8 9 10 11 12
Inches of Penetration
Per 100 Blows ~ Penetrometer Test
T.H.D.
D-5
FIG. 12A(Rev.) 1-55

DRILLED SHAFT PROPERTIES

Nominal Diameter (Inches)	Shaft Crossectional Area (Ft.^{2})	Shaft Surface Area Per Lin. Ft. (Ft.^{2}/Ft.)
18	1.77	4.71
24	3.14	6.28
30	4.91	7.85
36	7.07	9.42
42	9.62	10.99
48	12.57	12.57
54	15.90	14.14
60	19.64	15.71
66	23.76	17.28
72	28.27	18.85
78	33.18	20.42
84	38.48	21.99
90	44.18	23.56
96	50.27	25.13


CORRELATION CURVE
S.H.D. PENETRATION TEST
S~Safe Allowable Frictional Resistance (F.S.=3)
Tons per Sq. Ft.
0
1
2
3
4
5
6
30
25
20
15
10
5
0
Frictional
For use in estimating safe design unit loads on straight drifted shafts where it is desirable to combine point bearing & frictional resistance.
These curves should be used for Rocks & Shales only subject to the limitations outlined under Class IV Explorations.
Point Bearing
Rocks & Shales
P~Safe Allowable Point Bearing (F.S.=2)
Tons per Sq. Ft.
0
1
2
3
4
5
6
7
8
9
10
11
12
Inches of Penetration
Per 100 Blows~Penetrometer Test
FIG. 12B
THD
D-5
6-36

CALCULATED DATA FOR PILE BEARING***
(Using Submerged Density)

Control 500-4-8 Project FI 466(23) Camp Wallace Grade Separation Sta. 448+61.5 Hwy. US 75

14 inch concrete pile (precast)

STRATA** ft.	DENSITY* "W" #/cf	AV. DEPTH "D" ft.	HYDRO. PRESS. "U" psi	SHEARING STRENGTH "S" psi	psf	PILE SURFACE "A" AREA. ft.	ULTIMATE CAPACITY "P" #	tons
8-12	61.0	10.5	4.45	3.6	519	14.0	7,260	3.63
12-18	61.0	15.0	6.35	3.0	432	28.0	12,100	6.05
18-24	42.0	21.0	6.12	2.8	403	28.0	11,300	5.65
24-28	45.9	26.0	8.29	6.1	880	18.7	16,450	8.23
28-30	64.1	29.0	12.92	5.7	820	9.33	7,650	3.87
30-34	63.4	32.0	14.1	11.0	1585	18.7	29,600	14.80
34-36	58.0	35.0	14.1	8.7	1250	9.33	11,650	5.82
36-40	57.2	38.0	15.1	10.0	1440	18.7	26,928	13.5
40-42.8	55.0	41.3	15.7	7.6	1095	12.5	13,700	6.85
TOTAL FRICTIONAL RESISTANCE								68.40

*Submerged density calculated by subtracting 62.5 from Av. Wet Density

**Strata measured from original ground elevation of 22.0 ft. Bottom of pile is 42.8 ft. below this datum.

***Coulomb's Equation

FIG. 10

RECOMMENDED LOGGING TERMINOLOGY

7 BASIC GROUPS OF MATERIAL WITH DEFINITIONS

1. ROCK is a solidified, unyielding material which is not subject to change of form, volume or supporting value under wide changes in moisture content.

2. GRAVEL is a non-plastic, cohesionless, granular material composed of fine to coarse fragments of one or more kinds of rock. (Particle size: 100% retained on No. 10 sieve.)

3. SAND is a non-plastic, cohesionless, granular material composed of fine rock particles. (Particle size: 100% Passing No. 10 sieve and 100% retained on the No. 270 sieve.)

4. CLAY is an earthy material composed of the smallest particles of land waste. Its stability and plasticity varies widely with moisture changes. Particle sizes are all smaller than 0.005 millimeters.

5. SHALE is a fine grained material of highly compressed layers of clay, or silt and has a characteristic laminated structure such that it can be split into thin layers which usually run horizontal. Shale is highly affected by changes in moisture and loses much of its strength when not supported laterally.

6. ORGANIC MATERIAL covers a wide range of materials which cannot be suitably classified under the other 6 groups. It is composed of decayed vegetable, animal, or marine life. Characteristic formations in this group are mucks, peat and lignite.

7. SILT is a fine grained material (Particle size: 100% passing No. 270 sieve and minimum size of 0.005 millimeter) with little or no plasticity except when organic or clayey fractions are present. For the purpose of logging, loess is placed in the Silt Group on account of particle size. Loess is a wind borne deposit while silt, in the strict sense, is deposited by water action.

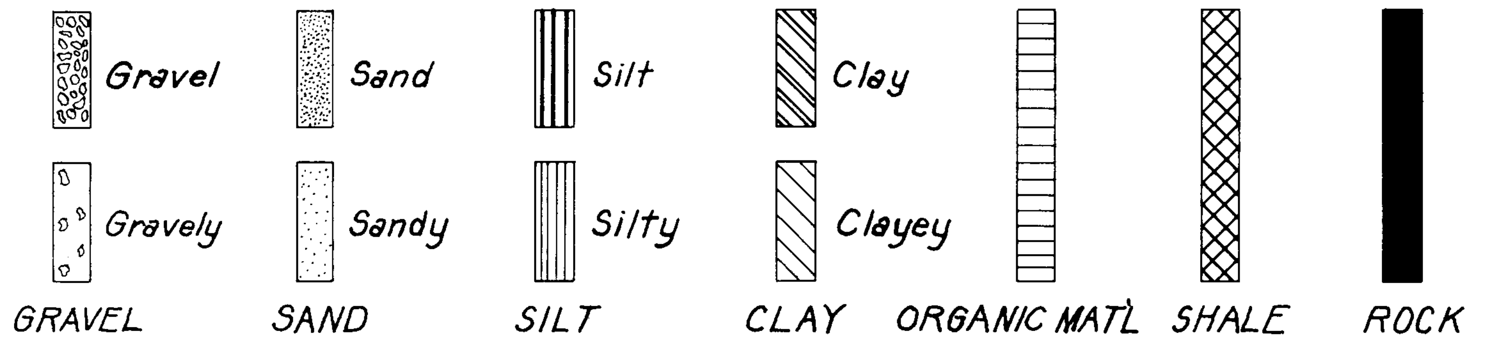
   It is suggested that all formations be classified under one of the 7 basic groups. However, in addition there should be as many descriptive terms used as necessary to clearly cover the KIND and CONDITION of the formation. Also, all logs on a project should be reviewed collectively to be sure that similar materials are described similarly and that unnecessarily detailed logging is avoided.

   The suggested descriptive terms are only a few of the commonly used ones. Additional descriptive terms should be used freely in actual practice.

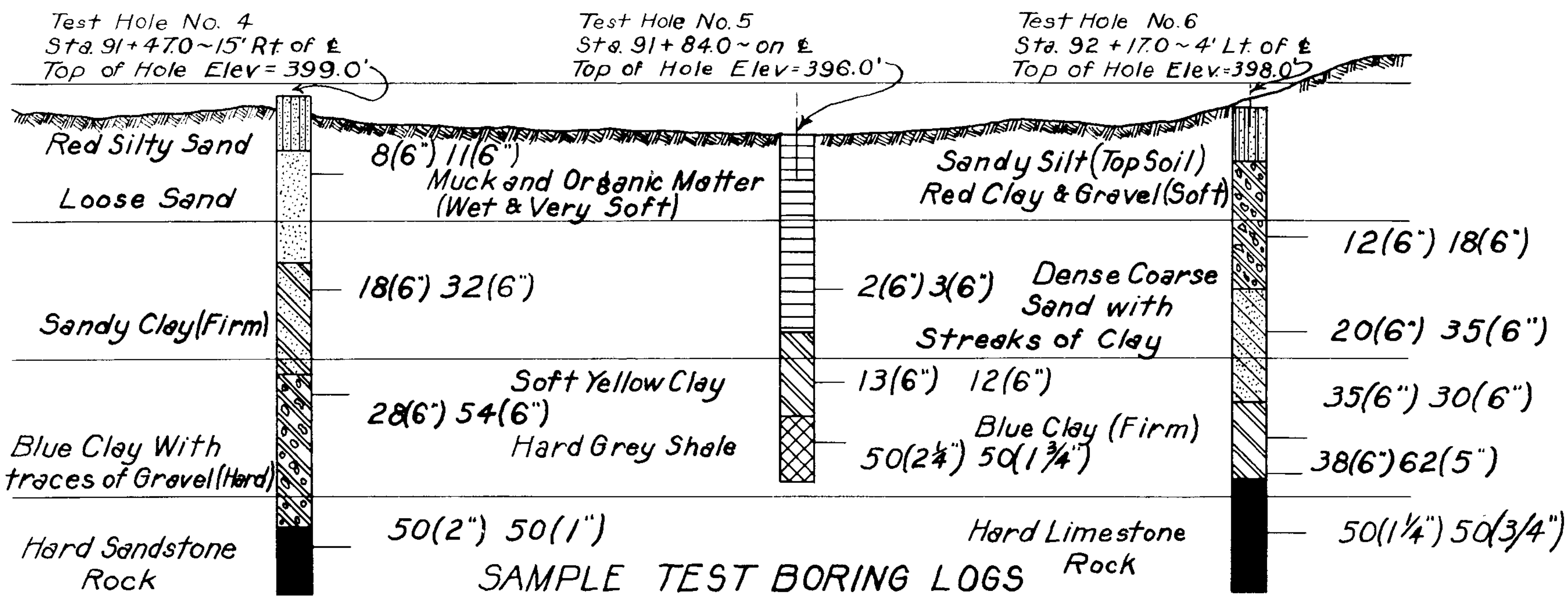
7 BASIC GROUPS WITH SUGGESTED DESCRIPTIVE TERMS

BASIC GROUP	KIND OF FORMATION		CONDITION OF FORMATION		
Rock	Sandstone Limestone Chalk Conglomerate	Slate Granite Flint Gypsum	Soft Medium Hard Hard Loosely Cemented	Firmly Cemented Laminated Nodular	
Gravel	Limestone Flint Caliche Sandy	Clayey Silty	Fine Coarse Loose Compact	Dense Well graded Water Bearing Clean	
Sand	Clayey (Loam) With Clay Lenses Gravelly With Sandstone Lenses Silty		Fine Coarse Well Graded Water Bearing Loose	Compact (Pack) Dense Cohesionless	
Clay	Sandy Gravelly Shaley Joint With Sand Lenses	Silty Organic Calcareous Loamy	Very Soft Soft Plastic Stiff Hard	Mucky Slickensided Friable Fissured Crumbly	Varved Marly Marbelized
Shale	Sandy Silty With Clay Lenses With Sandstone Lenses		Soft Medium Hard Hard Fissured		
Organic Material	Lignite Peat Muck Silty With Clay Lenses With Sand Lenses		Odorous		
Silt	Organic Inorganic Clayey Sandy Gravelly Loess		Loose Dense Water Bearing		

NOTE: Log observed moisture condition of material in natural state by terms of Dry, Moist or Saturated. Whenever necessary, supply additional appropriate descriptive or classifying terms.
In addition to the description of the KIND and CONDITION of a formation the log should include an accurate color description based upon the appearance of the formation with its natural moisture content.



BASIC SYMBOLS FOR TEST BORINGS



SAMPLE TEST BORING LOGS

SAMPLE SHEET

FOUNDATION MATERIAL PROFILE

COUNTY____________ STREAM________________________ HWY.NO.________

Cont.No.______ Date_______

Sta.No.						
Hole No.						Elev.

SAMPLE SHEET

CALCULATED STATIC CAPACITY OF FRICTION PILES
(Based on Coulomb's Theory)

Elev. Ft.	d Ft.	h Ft.	w #/Ft.^{3}	$\emptyset^\circ$	Tan$\emptyset$	c #/Ft^2	whTan$\emptyset$ #/Ft.^{2}	c+ whTan$\emptyset$ #/Ft.^{2}	Working Stress Each Strata Tons/Ft.^{2} SF of 2	Accumulated Bearing in Tons/Ft. of Perimeter of Piling

d(depth of layer);h(depth to centroid of strata); c(cohesion of soil)= c from TAT x 144; $\emptyset$(angle of internal friction); w=wet density of soil (For submerged conditions use Wet Density minus 62.4); a(perimeter of piling x depth of strata).
FORMULA: $p = (c + wh \tan \emptyset)a$.

Remarks: Accumulated Bearing in Tons/Ft. of perimeter of piling is based on a safety factor of 2.0.

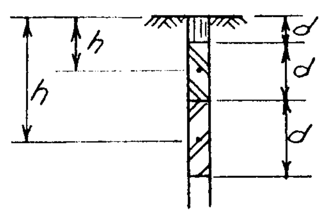
Name of job: ______________________

Control: ______________ I.P.E. __________

County: ______________ Hwy: __________

Hole No.: ______________________

Station: ______________________



T.H.D.
Date:

Sample Sheet

DRILLED SHAFT DESIGN
in
ROCKS & SHALES
(Based on Penetrometer Correlation Curves, Fig. 12B.)
(Triaxial & Unconfined Compression Tests)

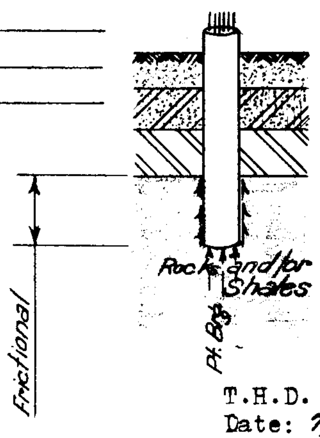
Bent No.	Design Shaft Load	Size Shaft	Penetrometer Correlation				Penetration		Shaft Tip Elev.	Remarks
			Point Brg. F.S.=2		Frictional F.S.=3		Need	Use		
	(T/s)	(Ft.)	(T/Ft^2)	(Tons)	(T/Ft^2)	(Tons)	(Ft.)	(Ft.)	(Ft.)	
1	49 T/s	30"	10 T/ft	49 T	1.05 T/ft	0 T	0'	3'	645'	
2	63	30"	10	49	1.05	14	1.7	3'	645'	
3	94	30"	10	49	1.05	45	5.4	6'	642'	
4	97	30"	8	39	0.95	58	7.8	8'	640'	
5	94	30"	12	59	1.25	35	3.6	4'	644'	
6	63	30"	14	69	1.50	0	0	3'	645'	
7	49	30"	14	69	1.50	0	0	3'	645'	

Remarks:

Name of job: Clear Fork Creek Bridge

Control: 193-2-5 I.P.E. 53-636

County: Neches Hwy: U.S. 89



T.H.D.
Date: 7/60