EVALUATION OF DEEP COMPACTION OF COHESIONLESS SOILS USING THE STANDARD PENETRATION TEST

by

Dennis L. Hess

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ABSTRACT

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Three methods of estimating relative density of deep sand deposits from Standard Penetration Test data have been investigated to determine their utility in determining soil design parameters. The data from four field investigations and two laboratory experiments were used to compare correlation values of relative density with values obtained from in-place sampling. The effects of different samplers and sampling techniques on the relative density values obtained from the in-place samples were incorporated into the analysis. The results of the investigation show that the Gibbs and Holtz and Waterways Experiment Station correlations produced relative density estimates within eight percent of relative density values determined from recovered samples. The Gibbs and Holtz correlation is acceptable where non-uniform deposits are present or where grain size analysis is unavailable or considered unnecessary. The Waterways Experiment Station correlation provides the best results, and is warranted where uniform deposits are present, and where grain size analysis data can be developed from field samples.

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INTRODUCTION

Relative density is a commonly used parameter for correlation with the physical properties of cohesionless soils. In cohesionless soils, relative density can be used to determine a soil's in-situ shear strength, its susceptibility to liquefaction, and its general suitability for foundation uses (13). Relative density can be calculated in terms of void ratio or dry density. In either case, three quantities are needed; the values for a soil in its loosest, its densest, and its in-place conditions. Direct measurement of these quantities is imprecise and not easily duplicated, which makes the utility of relative density values sometimes questionable. Much research has been concentrated on determining relative density by indirect means, particularly by using the Standard Penetration Test. The purposes of this paper are to identify the present methods of estimating relative density of deep sand deposits and evaluate the current state-of-the-art regarding the use of the Standard Penetration Test for that purpose.

BACKGROUND

Concept of Relative Density. A soil's volume consists of solids, water, and air. The volume that the air and water occupy is collectively referred to as the volume of the voids, V_v.

The void ratio, e, is defined at the ratio of the volume of the voids to the volume of the solids for a given soil mass.

The relative density, D_r, for a soil of a given void ratio is defined as:

$$D_r = (e_M - e)/(e_M - e_m)^* \tag{1}$$

Where e_M = void ratio of a soil in its loosest state, and e_m = void ratio of a soil in its densest state.

A soil's dry density, γ_d, is defined as its weight per unit volume. Substituting the density equation into Equation (1), relative density of a soil with a given density, can be expressed as:

$$D_r = \gamma_M(\gamma_d - \gamma_M / \gamma_M - \gamma_m)/\gamma_d \tag{2}$$

Where γ_M = unit weight of soil in its densest state, and γ_m = unit weight of soil in its loosest state.

For cohesionless soils, relative density is difficult to use as an index parameter. Burmister (7) believed that relative density could provide a common basis for relating soil behavior phenomena and establishing physical relationships that influence soil behavior, but Holtz (13) illustrated the limitations of the relative density approach.

Holtz noted that accurate measurement of the parameters (i.e., maximum, minimum, and in-place densities) is necessary

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(*) Relative density is correctly expressed as a percentage. The ratio form will be used here to avoid misinterpretation in later calculations. Symbols are defined where they first appear, and in Section D.

to give relative density values meaningful reliability. Dissimilarity between specimens poses a problem for duplicating or comparing one's own work with others'. For cohesionless soils, particle segregation during placement into molds for minimum and maximum density tests can cause variations in the results. The range and quality of testing apparatus and operator expertise available can cause wide variations in test results. Inconsistent field sampling techniques (wide range and condition of test equipment, operator expertise) are other sources of variations. Combined, these errors can produce inconsistent relative density values.

A cohesionless soil's shear strength is influenced by friction, the ability of the grains to interlock, and the confining pressure. Wu's (39) studies of shear strength and relative density in sands showed that the angle of internal friction of a deposit is independent of particle size and increases with increasing relative density. He also observed that an increase in the soil deposit's mean particle size results in a decrease in the soil's compressibility. Consequently, for different natural soils, the soil with the lower mean particle size will exhibit a higher compressibility and be more dense; therefore, it should exhibit a greater shearing resistance.

Terzaghi (36) notes that increasing angularity and coarseness of soil grains contributes to frictional resistance. Also, that confining pressure has no practical effect on the angle of internal friction at effective pressures less than 70 psi

(98.1 kPa). For pressures greater than 70 psi (98.1 kPa), the angle of internal friction values decrease gradually by about ten degrees. He concluded that this decrease was associated with an increase in the percentage of grains that are crushed as the state of failure was approached. Langfelter's (16) study of factors influencing shear stresses in cohesionless soils supports both Wu's and Terzaghi's conclusions.

Travenas (37) investigated the measurement of relative density and determined that its accuracy is highly dependent upon the accuracy of maximum and minimum density measurements. Actual relative density values for the same soil can vary as much as 6% for laboratory tests and 13% for field tests when they are conducted by the same operators. These variations are about 12% larger if the results from different operators are compared. Travenas concluded that due to the wide range of values obtainable from in-situ density testing, the relative density concept should be carefully applied and used qualitatively, and not as a basic parameter in a calculation.

Indirect Methods of Measuring Relative Density. Several methods for indirectly measuring relative density are in current use: The Menard pressuremeter; the nuclear moisture-density gauge; the quasi-static cone penetrometer, also known as the 'Dutch' cone; and the Standard Penetration Test, which employs a split-spoon sampler.

The Menard pressuremeter was developed in 1956. As Baguelin (3) describes, it measures the force required to deform in-situ soil. With suitable calculations, this force can be used to determine soil properties, including strength. With appropriate correlations, the soil strength can be used to estimate relative density. The pressuremeter consists of three cells: A central, flexible rubber bladder, and two smaller rubber bladders, one on each end. A steel rod holds the bladders together and defines the pressuremeter's length. The assembly is lowered into a borehole for the test. The center bladder is pressurized by injecting water, forcing it against and deforming the borehole walls. The smaller bladders are inflated with gas to the same pressure as the center bladder. The center bladder's volume and pressure changes are recorded in a control unit on the ground surface. Using the Menard pressuremeter requires that the boring device be removed before the pressuremeter can be inserted. Advocates of pressuremeters do not have much theoretical or empirical evidence to justify their use in cohesionless soils, and their applications will not be discussed in this paper.

The nuclear moisture density gauge (31) has a probe rod which contains a small radioactive source. The probe is inserted into an access hole and the radiation absorbtion and reflections are recorded. In-situ density and water content is determined from this data. The test and the results can be quickly obtained on site, as opposed to conducting laboratory tests, and the compacted soil is not disturbed. Nuclear equipment

is very expensive compared to sand cone or rubber balloon test equipment, and operators must be specially trained and certified for using nuclear test equipment. Generally, its applicability is limited to shallow depths.

The static ('Dutch') cone penetrometer is widely used in Europe for soil exploration. Sanglerat (33) states that it is a simple, expedient, and economical tool. It provides a continuous record of penetration resistance, and the test values are a reliable index of consistency or relative density of substrate, since factors such as the size of the bore hole and disturbances at the base of the hole are eliminated. Alperstein (1) compared the cone penetrometer with the split-spoon sampler used in the United States, and agreed with the advantages Sanglerat claimed. However, he noted three disadvantages; no soil sample is obtained with the cone penetrometer, it is difficult to use in hard or bouldery soil, and there are no generally agreed upon standards of interpretation of results in engineering practice in the United States. Mitchell (28) cites Schmertmann's method for obtaining relative density of sand using the cone penetrometer, which employs cone bearing capacity and overburden pressure.

The Standard Penetration Test (SPT) with a split-spoon sampler is the most commonly used method in the United States for soil exploration. Developed by C.R. Gow (10) in 1902, it employed a one- or two- foot (0.305 or 0.610 m) long, two-inch (5.08 cm)

outside diameter cylinder with a beveled end. The cylinder was driven into the ground by striking rods attached to the cylinder with a falling weight. The number of blows required to drive the sampler twelve inches is referred to as the Standard Penetration Test blow count, N, and has units of blows/foot. Soil is forced up into the cylinder and a sample is retained. The cylinder is split lengthwise to allow sample retrieval.

Improvements to Gow's design included a check valve to prevent loose samples from falling out of the sampler as it was retracted. Initially, the sampler was driven by striking with a 110-pound (49.83 kg) weight. The present method uses a 140-pound (63.42 kg) weight. The dimensions of the tool and procedures for its use in obtaining representative samples for identification for and laboratory tests have been outlined by the American Society for Testing and Materials (2).

Schmertmann (35) noted that the SPT provides a rudimentary means of determining in-place shear strength. However, due to the empirical nature of the test and subsequent limitations of its usefulness, he recommended that 'N' values not be used as design or acceptance criteria.

In spite of Schmertmann's assertions, the test procedure is standardized within the United States, and there is much empirical data available that correlates SPT blow count with relative density, shear strength, and liquefaction potential.

Terzaghi and Peck (36) showed that the SPT blow count was qualitatively related to relative density, and used it to provide the basis for estimating the settlement of footings. However, this

relationship did not incorporate any possible effects of overburden pressure on the SPT blow count or relative density. Gibbs and Holtz (11) conducted experiments that provided a quantitative relationship between relative density and the SPT blow count curve. Their tests were conducted at overburden pressures from 0 to 40 psi (0 to 275.77 kPa). The result of their work is expressed by Equation (3):

$$N = 1.7D_r^2(10+p) \qquad (3)$$

Where N = SPT blow count, D_r = relative density, and p = effective overburden pressure, in pounds per square inch (1 psi = 6.89 kPa).

Meyerhof's (27) study of shallow foundations in sand and D'Appolonia's (8,9) investigation of spread footings on sand showed that the Terzaghi and Peck settlement curves overestimated settlement by 50 to 60 percent. Peck and Bazarra's (29) investigation of the Gibbs and Holtz (12) correlation indicated that the corrections suggested were not realistic because the laboratory conditions did not reflect field conditions, that the relative densities that were studied were higher than the relative densities considered by Terzaghi and Peck, and therefore were not applicable. Bazarra (4) presented a correction to the SPT N-value which correlated to field measurements of settlements of shallow foundations on dry sand. The results of his study are:

For overburden pressures less than 10.417 psi (71.818 kPa),

$$N = 20D_r^2(1+0.288p) \qquad (4a)$$

For overburden pressures greater than 10.417 psi (71.818 kPa),

$$N = 20D_r^2(3.25+0.072p) \qquad (4b)$$

where p = overburden pressure, in pounds per square inch (1 psi = 6.89 kPa).

Marcuson and Bieganousky conducted a two-phase study of SPT results and the accuracy of relative density predictions based upon SPT N-values. In the first phase (20), experiments were conducted on two fine, uniformly graded (Reid-Bedford Model and Ottawa) sands. In the second phase (21, 23), experiments were conducted on two coarse, poorly graded (Platte River and standard concrete) sands. A stacked ring soil container, a vibrating platform, a loading system for applying overburden pressure, and drilling and sampling equipment were used in the experiments. Density values for a given test were determined from the weight of sand placed in the volume of the container filled. Maximum and minimum dry densities were determined from laboratory procedures. Penetration resistance values were compiled for both sands at overburden pressures of 10 psi (68.94 kPa), 40 psi (275.77 kPa), and 80 psi (551.54 kPa). Undisturbed soil samples were taken at each pressure using a Hvorslev sampler to compare in-place densities obtained by this method with the known densities determined above. Marcuson (22) originally presented this data in tabular form, which is reproduced in Table 1. This data is used later in this paper, along with three case histories, to assist in interpreting relative density from in-place density measurements. The results of the study indicated that penetration resistance was sensitive to

TABLE 1A: Waterways Experiment Station Data, Platte River Sand (21)

Test (1)	Relative Density, in per-cent (2)	Adjusted Dry Dens-ity, in pounds per cubic foot (3)	SPT N, blows per foot (4)	Effective Overburden Pressure, in pounds per square inch (5)
27	19.2	106.1	3	10
27	19.2	106.1	2	10
27	24.2	107.0	7	40
27	24.2	107.0	8	40
27	32.9	108.6	11	80
27	32.9	108.6	12	80
28	53.7	112.8	11	10
28	53.7	112.8	12	10
28	56.2	113.1	22	40
28	56.2	113.1	26	40
28	58.1	113.5	33	80
28	58.1	113.5	35	80
29	91.4	120.7	53	10
29	91.4	120.7	52	10
29	91.4	129.7	47	10
29	91.4	129.7	46	10
29	91.4	129.7	73	40
29	91.4	120.7	66	40
29	91.4	120.7	94	80
29	91.4	120.7	78	80

Note: 1 pcf = 0.15 kN/m^3 ; 1 psi = 6.89 kPa: 1 ft = 30.48 cm.

TABLE 1B: Waterways Experiment Station Data, Standard Concrete Sand (21)

Test (1)	Relative Density, in per- cent (2)	Adjusted Dry Dens- sit, in pounds per cubic foot (3)	SPT N, blows per foot (4)	Effective Overburden Pressure, in pounds per square inch (5)
30	20.1	106.7	2	10
30	20.1	106.7	1	10
30	25.9	107.6	9	40
30	25.9	107.6	8	40
30	29.7	108.2	16	80
30	29.7	108.2	17	80
31	95.9	119.8	38	10
31	95.9	119.8	38	10
31	95.9	119.8	30	10
31	95.9	119.8	39	10
31	95.9	119.8	60	40
31	95.9	119.8	74	40
31	95.9	119.8	78	80
31	95.9	119.8	86	80
32	49.3	111.4	9	10
32	49.3	111.4	9	10
32	50.5	111.6	20	40
32	50.5	111.6	23	40
32	51.7	111.8	35	80
32	51.7	111.8	37	80

Note: 1 pcf = 0.15 kN/m^3 ; 1 psi = 6.89 kPa; 1 ft = 30.48 cm

changes in density, overburden pressure, and lateral stress conditions; a SPT N-value is reproducible in nearly homogeneous deposits, but heterogeneous field conditions made estimating relative density more difficult; the Standard Penetration Test is not sufficiently accurate for direct evaluation of relative density unless site specific correlations are developed; expressions derived from statistical analysis do not adequately address the range of subsurface conditions found in the field; and a simplified family of curves correlating SPT N-values, relative density, and overburden pressure for all cohesionless soils under all conditions are not possible to develop. The study produced the following correlation:

$$D_r = 11.7+0.76 \left|222N+1600-53p-50(C_u)^2\right|^{1/2} \qquad (5)$$

Where C_u is the coefficient of uniformity.

Despite the widespread usage of the Standard Penetration Test, it is not without limitations. Wu (39) states that the SPT should not be used as the sole method of determining relative density if the soil deposit contains wide and erratic variations in particle size and relative density. Fletcher (10) lists several reasons for obtaining different SPT blow counts in the same soil. The driving weight's height of drop is not always precisely 30 inches (76.2 cm). The drop is usually shorter, resulting in shorter strokes and higher blow counts per test. The rigging used to lift the weight runs through blocks and over catheads, inducing a drag on the weight as it falls, increasing the blow count. The sharpness of the sampling spoon tip can

affect the blow count, the duller spoon tip requiring more driving energy. In deep sampling (i.e., greater than 200 feet (61 m)), the energy absorbed by rod buckling under the driving force can result in an increased blow count. Alternately, deep excavation (i.e., greater than 15 feet (4.5 m)) before in-place testing tends to reduce the blow count because of the removal of overburden pressure. Brown (6) also investigated the effects different types and lengths of drill rods had on SPT values and arrived at similar conclusions. Ireland (16) contends that the sampler does not give representative samples with respect to in-situ density because the vibration of driving tends to densify loose soils and loosen dense soils. Kovacs (17, 18) evaluated the free and non-free fall driving hammers and recommended that the energy imparted to the sampler, rather than the height of fall, be standardized to give more consistent SPT results; but that the 'standard' energy selected not nullify the existing engineering correlations.

Even with all of these disadvantages, it is doubtful that the Standard Penetration Test will be discarded. The test gives a good general indication of subsurface conditions, and can be supplemented by other sampling methods when borings indicate more complete data collection is necessary (5).

UNDISTURBED SAMPLING OF SANDS

The phrase 'undisturbed sampling' of sands is misleading, since sampling methods usually have adverse effects on the recovered sample. For this paper, 'undisturbed' samples will be defined as samples obtained by sampling methods in which special efforts

have been made to minimize the detrimental effects of sampling.

In the following case studies described a sampler was used to recover samples for grain size analysis and in-place density measurements. Four different types of samplers were used, each having a different effectiveness in producing relatively undisturbed samples of cohesionless soils.

The Dames and Moore sampler used in the Red Wing case study is a thick-walled, split-barrel, open sampler. It is approximately 25 inches (60.96 cm) long, with an outside diameter of 3 1/4 inches (8.26 cm) and an inside diameter of 2 1/2 inches (6.35 cm). The interior of the sampler contains a thin brass liner to provide a smooth inside surface to minimize friction between the soil and the sampler during operation. The mouth of the sampler has eight flap valves which remain open during the initial placement of the sampler in the borehole. The valves have a small offset on the tips and are eccentically hinged, and are closed by the friction between the sample and the valves and the weight of the sample as the sampler is extracted. Once extracted, the spoon is opened and the sample, encased by the brass liner, is removed.

The Osterberg sampler used in the Treasure Island study is a thin-walled, fixed-piston sampler. It is approximately 36 inches (91.44 cm) long, with a sampler outside diameter of approximately 2 3/16 inches (5.56 cm) and an inside diameter of 2 inches (5.08 cm). The fixed-piston sampler differs from the open sampler in that prior to inserting the sampler into the borehole, the piston inside the sampler is lowered until it is flush with the sampler's

cutting edge. The sampler is lowered into the borehole and pushed into the base of the hole until the sampling depth is reached. While the piston is held stationary, the sampler tube is advanced, in this case hydraulically, past the piston into the soil to obtain a sample. The piston rod is clamped to prevent downward movement of the piston during withdrawal from the borehole. After withdrawal, the sample is retained in the tube for transportation. After arrival at the laboratory, the sample is pushed out of the tube.

The Hvorslev sampler used in the Marcuson case study and the three-inch (7.62 cm) thin-wall piston sampler used in the Boca Raton case study are also of the fixed-piston type, and their operation is very similar to the Osterberg sampler. The principal differences are that the Hvorslev sampler is hand-driven, yielding a sample approximately six inches (15.24 cm) long and approximately two inches (5.08 cm) in diameter, while the three-inch (7.62 cm) sampler is driven by a falling weight, producing a sample approximately 24 inches (60.96 cm) long and three inches (7.62 cm) in diameter.

Hvorslev (15) investigated various samplers and observed relationships between the design of the samplers, the method of driving, and the quality of samples recovered.

The principal dimensions of the sampler are used to determine the amount of recovered sample. The over-all condition of a soil sample is represented by the total recovery ratio, R, and is defined as

$$R = L/H \tag{6}$$

where H = the penetration of the sampler below the bottom of the borehole during the actual sampling, and L = the length of the sample before withdrawal.

Unfortunately, this ratio does not provide information on the change in thickness of the sample unless the entrance of excess soil is minimized or eliminated. However, it is possible to determine corresponding penetration and length values while the sampler is being forced into the soil. From these values, penetration and length increments can be measured, and the specific recovery ratio, r, can be determined as follows:

$$r = \Delta L / \Delta H \tag{7}$$

Where ΔL = the incremental sample length, and ΔH = the incremental penetration of the sample.

The effect of a cutting edge with an inside diameter smaller than the inside diameter of the sampling tube upon the recovered sample was also investigated. When the reduced inside clearance exceeds that which is required to compensate for elastic expansion of the soil after it enters the sampler, the sample may become shortened. To account for the influence of inside clearance, the inside clearance ratio, C_i, was defined as follows:

$$C_i = \frac{D_s - D_e}{D_e} \tag{8}$$

Where C_i = clearance ratio, D_s = the inside diameter of the sampler, and D_e = inside diameter of the cutting edge.

It is apparent that a specific recovery ratio of 1.0 indicates that the recovered length of sample is equal to the driven length of

the sampler. By considering the influence of inside clearance, a sample may have a specific recovery ratio as low as $(1-2C_i)$ and be considered undisturbed.

The effect of the sampler's wall thickness on sample quality was investigated. The cross-sectional area of the sampling tube represents the amount of soil which is displaced when the sampler is forced into the ground. The area ratio, C_a, is approximately equal to the ratio between the volume of displaced soil and the volume of the sampler. It is defined by the equation:

$$C_a = \frac{D_w^2 - D_e^2}{D_e^2} \qquad (9)$$

Where D_w = sampler's outer diameter, and D_e = sampler's inner diameter.

Hvorselev observed that samplers with low area ratios (i.e., thin walls) had less sample disturbance, less penetration resistance, and less admission of excess soil than thick-walled samplers. Piston samplers required less borehole preparation and cleaning than open samplers. Driving of samplers by falling weight tended to reduce the amount of excess soil admitted to samplers, but produced vibrations in the sampler that tended to loosen dense soils and densify loose soils. Pushing tended to produce a less disturbed sample than hammering, but the motion had to be uniform and continuous for the length of the sample to prevent development of wall friction and adhesion. For hand-pushed samplers, it was noted that rotation of the sampler should be avoided since it could cause failure of the sample as it enters the sampler. Marcuson (22) drew the same conclusions concerning the effects of fixed-piston samplers and vibrations on the

DESCRIPTIONS OF CASE HISTORIES

Four case histories were used to determine which method of interpreting indirect measurements from the Standard Penetration Test most closely determined the actual in-situ relative density of cohesionless soils. Two of the case histories involved measuring the effect of vibroflotation on increasing soil density. The third case history dealt with a subsurface exploration prior to construction, and the fourth case history was a laboratory investigation of the ability to determine in-situ relative density in assessing the liquefaction potential of cohesionless soils.

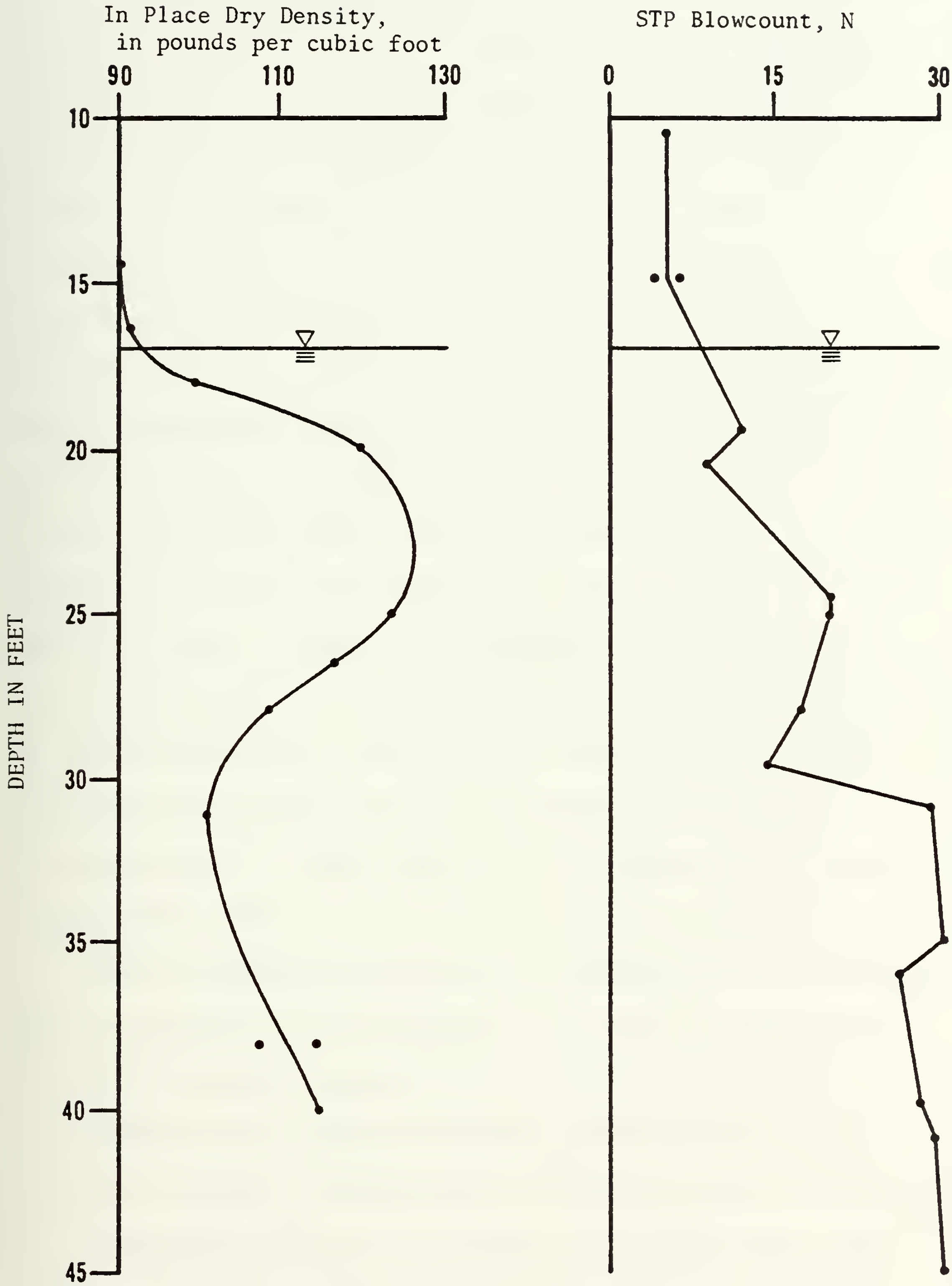
Red Wing, Minnesota. A study of sub-surface conditions was conducted in Red Wing, Minnesota, to determine the suitability of soil compaction by vibroflotation for foundation support of a nuclear power plant. The purpose of the compaction was to densify the natural soil to a depth of 45 feet (13.73 m) below the original grade to a relative density of 85% to provide a margin of safety against liquefaction. The natural soils were granular alluvial soils, deposited as glacial outwash or river sediments. The granular soil between the original grade and 25 feet (7.63 m) depth were loose fine to medium sandy soils. Soils below 25 feet (7.63 m) were mostly fine to medium sands, containing varying amounts of coarse sand and gravel. Between 25 feet (7.63 m) and 45 feet (13.73 m) the granular soil was medium dense to dense, but contained discontinuous layers of loose granular material. Below 45 feet (13.73 m) the granular

soil was dense to very dense. The water table was approximately 17 feet (5.19 m) below the original grade level.

The test procedure consisted of drilling control borings at or near the vibroflotation test patterns located in the vicinity of the proposed power house structure. These borings were representative of test borings drilled for the plant foundation study. Standard Penetration Tests (SPT) were performed in the control borings, while supposedly relatively undisturbed samples were recovered from the test borings using a Dames and Moore Sampler. The area ratio of the Dames and Moore sampler, as calculated from Equation (9), is 81%. Experiments (15) with a similar sampler with an area ratio of 39% showed that excess soil was introduced into the sampler. Based upon the higher area ratio, it is evident that samples recovered using the Dames and Moore sampler would be greatly disturbed, and in-place density values obtained from the sample would be artificially high, due to the introduction of excess soil. The results of this case history, therefore, should be used cautiously with the aforementioned limitations in mind.

The relative densities in the control borings were estimated by correlation with SPT blow counts, while the relative densities in the test borings were measured directly by computing the in-place densities of the relatively undisturbed soil samples. Figure 1 is a plot of the in-place density and SPT blow count with respect to depth.

FIGURE 1: Results of Standard Penetration Tests and In-place Density Determinations, Red Wing, Minnesota



1 ft = 0.305 m
1 pcf = 0.157 kN/m^3

Treasure Island, California. A study of subsurface conditions was conducted at Treasure Island, California, to determine the suitability of vibroflotation and compaction piles for the foundation support of a barracks building. The purpose of the compaction was to densify the sand fill in the building area to a depth of 30 feet (9.15 m) to reduce the possibility of liquefaction during an earthquake. The general conditions at Treasure Island consisted of a hydraulically placed sand fill deposit approximately 30 feet (9.15 m) deep, overlying 8 feet (2.44 m) of medium dense sand and 20 feet (6.10 m) of soft to medium gray silty clay. The clay deposit was underlain by layers of stiff clay and dense sand. The water table was at a depth of 6 feet (1.83 m). From grade level to a depth of 10 to 15 feet (3.05 to 4.76 m) the soil consisted of medium dense fine to medium grained sand, with occasional deposits of coarse sand. Beneath 15 feet (4.76 m), the fill consisted of loose to very loose silty fine to medium sands containing numerous inclusions of gray silty clay.

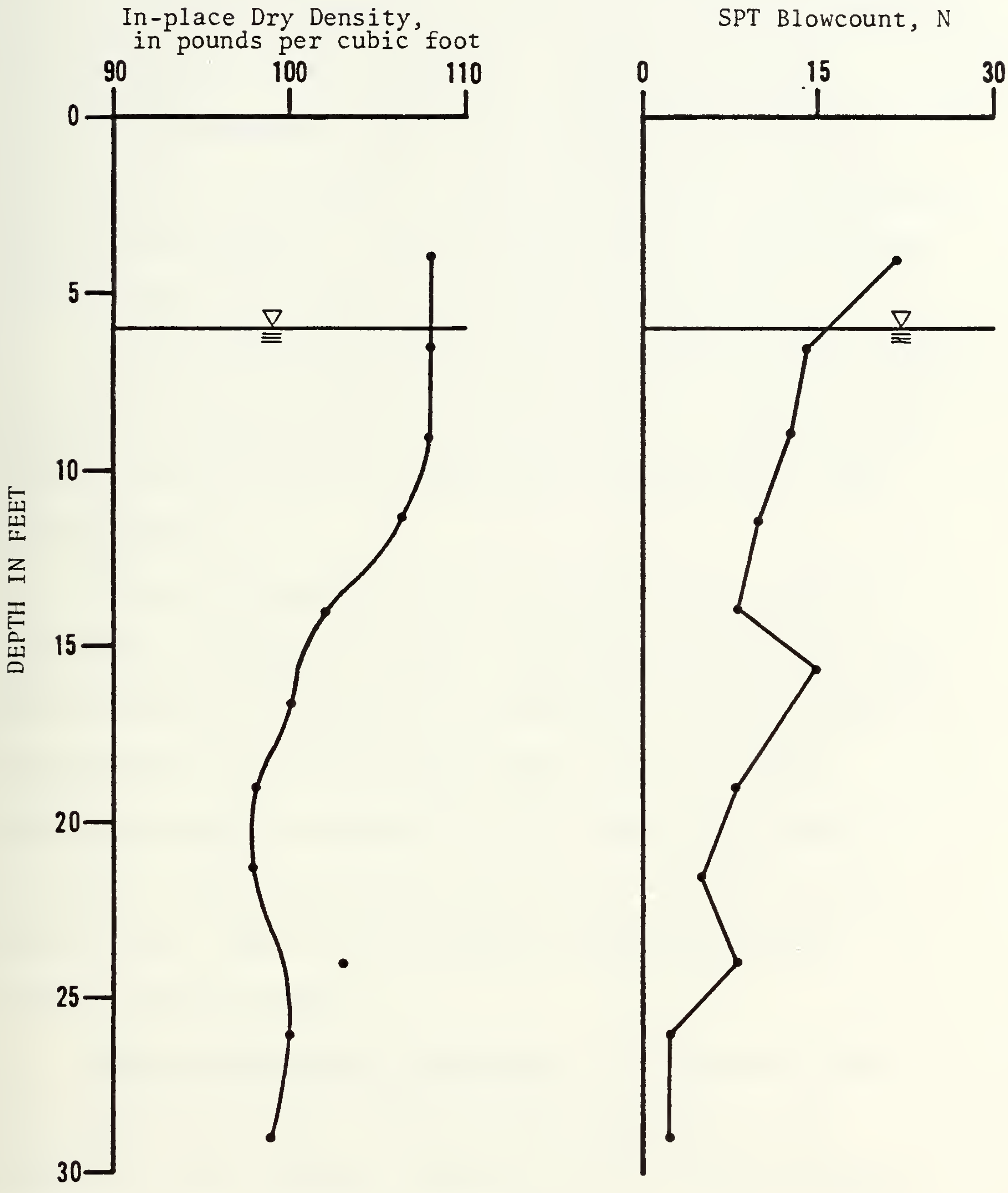
The test procedure consisted of conducting Standard Penetration Tests in borings at three locations on the site. Samples of the fill were recovered from one of the exploratory borings using a modified Osterberg sampler which was pushed into the soil by hydraulic pressure. The area ratio, as calculated from Equation (9), was approximately 19.6%. It is possible that excess soil could have been forced into the sampler, since Hvorselev (15) states that a sampler's area ratio should be less than 10% to ensure

recovery of an undisturbed sample. As the minimum specific recovery ratio, as calculated from Equation (7), is approximately 87.5%, and the description of the sampling procedure in the case history noted that "virtually complete recovery" of the samples was obtained during the exploration, it is reasonable to conclude that there was negligible disturbance to the sample, and that the laboratory values for in-place density were representative of actual field conditions.

Laboratory tests were performed on the samples to determine the maximum, minimum, and in-place dry densities. Figure 2 is a plot of the in-place density and SPT blow count with respect to depth.

Boca Raton, Florida. Schmertmann (34) investigated the correlation between relative density and SPT values in conjunction with a subsurface exploration at Florida Atlantic University. Three undisturbed sample borings were made within the site, each of which was 4 to 5 feet (1.22 to 1.53 m) from a Standard Penetration Test boring. The standard borings were done in accordance with ASTM-D 1586, with the blow count during the last 12 inches (30.48 cm) of the 18 inch (45.72 cm) drive being recorded. Undisturbed sampling was done with special care using a three-inch (7.62 cm) diameter thin-walled fixed-piston sampler. The sampling stroke was 24 inches (60.96 cm), and the sampling was continuous with depth in a hole kept open with drilling mud. In-place density was assumed to be equal to the computed density of all the sand recovered in the sampler. This assumption would not be valid for samplers with large area ratios,

FIGURE 2: Results of Standard Penetration Tests and In-Place Density Determinations, Treasure Island, California



1 ft = 0.305 m
1 pcf = 0.157 kN/m^3

as the admission of excess soil into the sampler would yield an in-place density higher than actual field conditions. However, the three-inch (7.62 cm) thin-walled sampler has an area ratio of approximately 6.87%, which minimizes the admission of excess soil. Provided that there was no loss of soil from the sampler during recovery, this assumption would provide in-place density values consistent with actual field conditions.

Procedures approximating ASTM D2049-64T were used to determine maximum and minimum densities, using a 1/30 cu ft (944.64 cu cm) mold and vibrating wet for approximately 60 minutes for each maximum density.

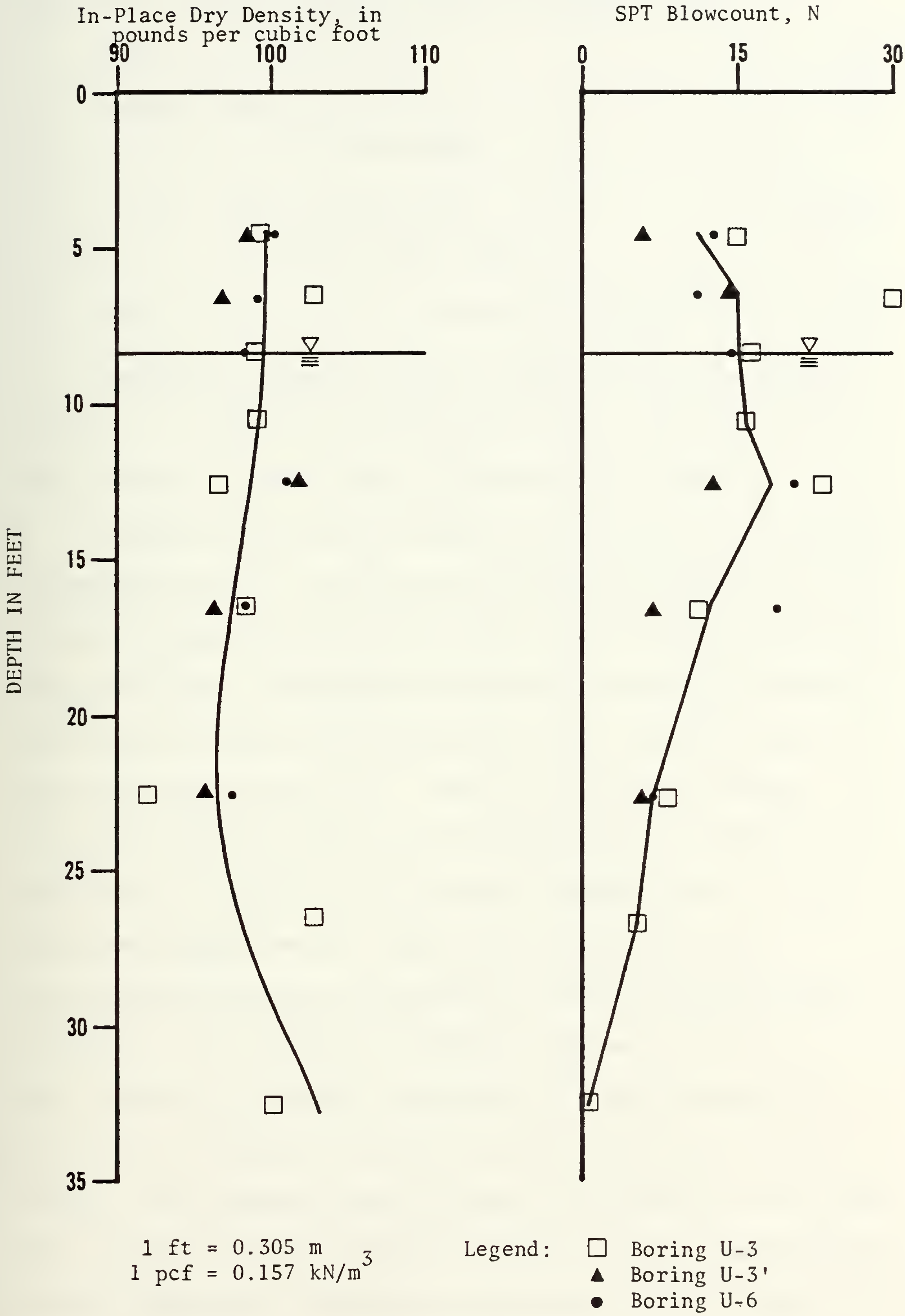
The sands tested were quartz, non-plastic and having no indications of organic materials. Occasional shells were found, but the sands did not appear to be cemented. A sieve analysis was performed on each undisturbed sample, and the maximum percentage passing the Number 200 U.S. Standard sieve was 5.5% by washing. The samples ranged from fine to very fine poorly graded sands. Twenty of the samples were matched with SPT tests as shown in Figure 3.

<u>Waterways Experiment Station Data</u>. The laboratory test data presented in Table 1 earlier was incorporated in the evaluation of SPT methods, along with the three case histories described in the foregoing paragraphs.

METHOD OF ANALYSIS

There are currently three indirect methods of estimating the relative density of deep sand deposits. They are Gibbs and Holtz (G-H) (11), Bazaraa (4), and Waterways Experiment Station

FIGURE 3: Results of Standard Penetration Tests and In-Place Density Determinations, Boca Raton, Florida.



(WES) (22). Each is summarized below in equation form.

G-H: $N = 1.7D_r^2(10+p)$ (3)

Bazaraa: For $p \leq 10.417$ psi (4a)

$N = 20D_r^2(1+0.288p)$

For $p \geq 10.417$ psi (4b)

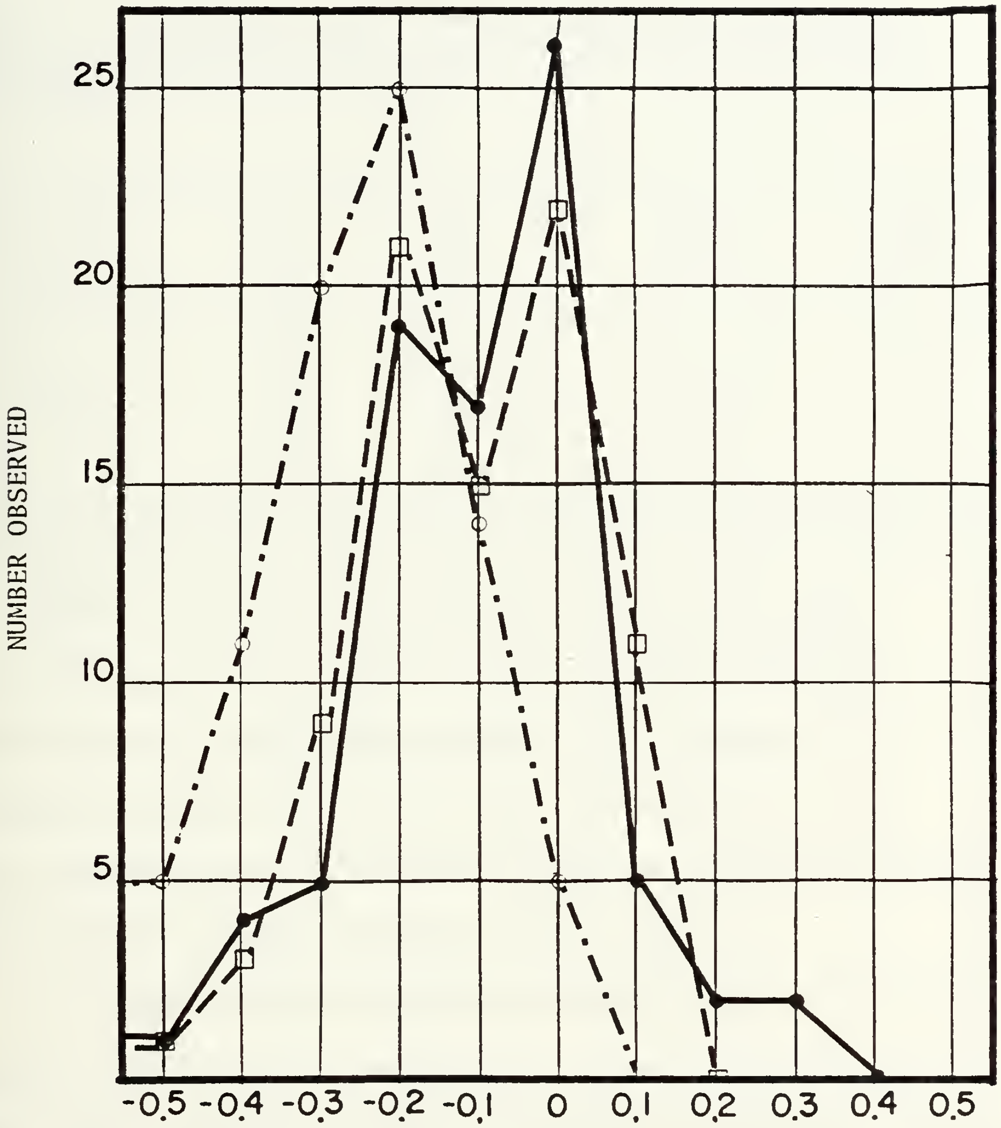
$N = 20D_r^2(3.25+0.072p)$

WES: $D_r = 11.7+0.76|222N+1600-53p-50(C_u)^2|^{\frac{1}{2}}$ (5)

Where D_r = relative density, N = SPT blow count in blows per foot, p = effective overburden pressure in pounds per square inch (1 psi = 6.9 kPa), and C_u = coefficient of uniformity.

In each case study the in-place relative density was computed using Equation (2). The maximum and minimum dry densities for the Treasure Island, Boca Raton case studies, and the WES experimental data were given. The maximum and minimum dry densities for the Red Wing case study were estimated from sieve analysis results and Winterkorn and Fang (37). The SPT N-values and effective overburden pressures (and 'C_u' for Equation (5)) were used in Equations (3), (4), and (5) to estimate relative density. The estimated relative density values were subtracted from the corresponding in-place relative density value to determine the difference. The frequency of difference values according to each equation was recorded and placed on a normal distribution curve shown in Figure 4. Negative values indicate the relative density value obtained from the indirect correlation was less than the in-place direct measurement of relative density, while positive values indicate that the indirect correlation relative density value was greater than the direct measurement of relative density.

FIGURE 4: Differences Between Measured Relative Density and Interpreted Relative Density from Indirect Correlation with the Standard Penetration Test



Difference (Interpreted D_r - Measured D_r)

Legend: ——— Gibbs and Holtz Correlation (Eq 3)

—·· Bazarra Correlation (Eq 4)

— —· Waterways Experiment Station Correlation (Eq 5)

The mean, standard deviation and 95% confidence level limits of relative density differences for each correlation are listed below:

Table 2: Differences Between Measured Relative Density and Interpreted Relative Density from Indirect Correlation With Standard Penetration Resistance

Correlation (1)	Mean (2)	Std. Dev. (3)	Lower Limit (4)	Upper Limit (5)
G-H:	-0.069	0.197	-0.111	-0.0271
Bazaraa:	-0.231	0.184	-0.270	-0.192
WES:	-0.083	0.178	-0.121	-0.045

Figure 4 shows that the Bazaraa correlation is quite conservative, yielding relative density values from 18% to 27% lower than in-place measurements. This discrepancy is mitigated somewhat by the fact that the Bazaraa correlation was intended for use in determining maximum, rather than actual settlements of shallow foundations in dry sand.

The Gibbs and Holtz and WES correlations show much better agreement with in-place measurements. While the Gibbs and Holtz mean value is slightly closer to zero than the WES mean, the WES standard deviation, and hence the confidence interval is smaller, indicating that the sample gradation has some influence on penetration resistance for a given relative density

The Waterways Experiment Station data in Table 1 comprises about 47% of all of the data used for this analysis. The preponderence of WES data raises the question of whether the

results of the distribution were biased towards the WES correlation. Of the two coarse sands Marcuson used, one was similar to the U.S. Bureau of Reclamation sand which was used by Gibbs and Holtz for their study. The other sand was a poorly graded sand available commercially for use in preparing concrete. To determine if the WES data had a significant influence on the test results, the distribution calculations were performed separately on the WES data and the data obtained from the case studies. The results of the calculations are shown in Tables 3 and 4 below, and graphically in Figures 5 and 6.

Table 3: Differences Between Measured Relative Density and Interpreted Relative Density from Indirect Correlations with Standard Penetration Resistance (Waterways Experiment Station data only)

Correlation (1)	Mean (2)	Std. Dev. (3)
G-H:	0.015	0.125
Bazaraa:	-0.127	0.094
WES:	0.033	0.088

Table 4: Differences Between Measured Relative Density and Interpreted Relative Density from Correlations with Standard Penetration Resistance (Red Wing, Treasure Island, and Boca Raton data)

Correlation (1)	Mean (2)	Std. Dev. (3)
G-H:	-0.145	0.218
Bazaraa:	-0.324	0.195
WES:	-0.187	0.174

FIGURE 5: Differences Between Measured Relative Density and Interpreted Relative Density from Indirect Correlation with the Standard Penetration Test (Waterways Experiment Station Data Only).

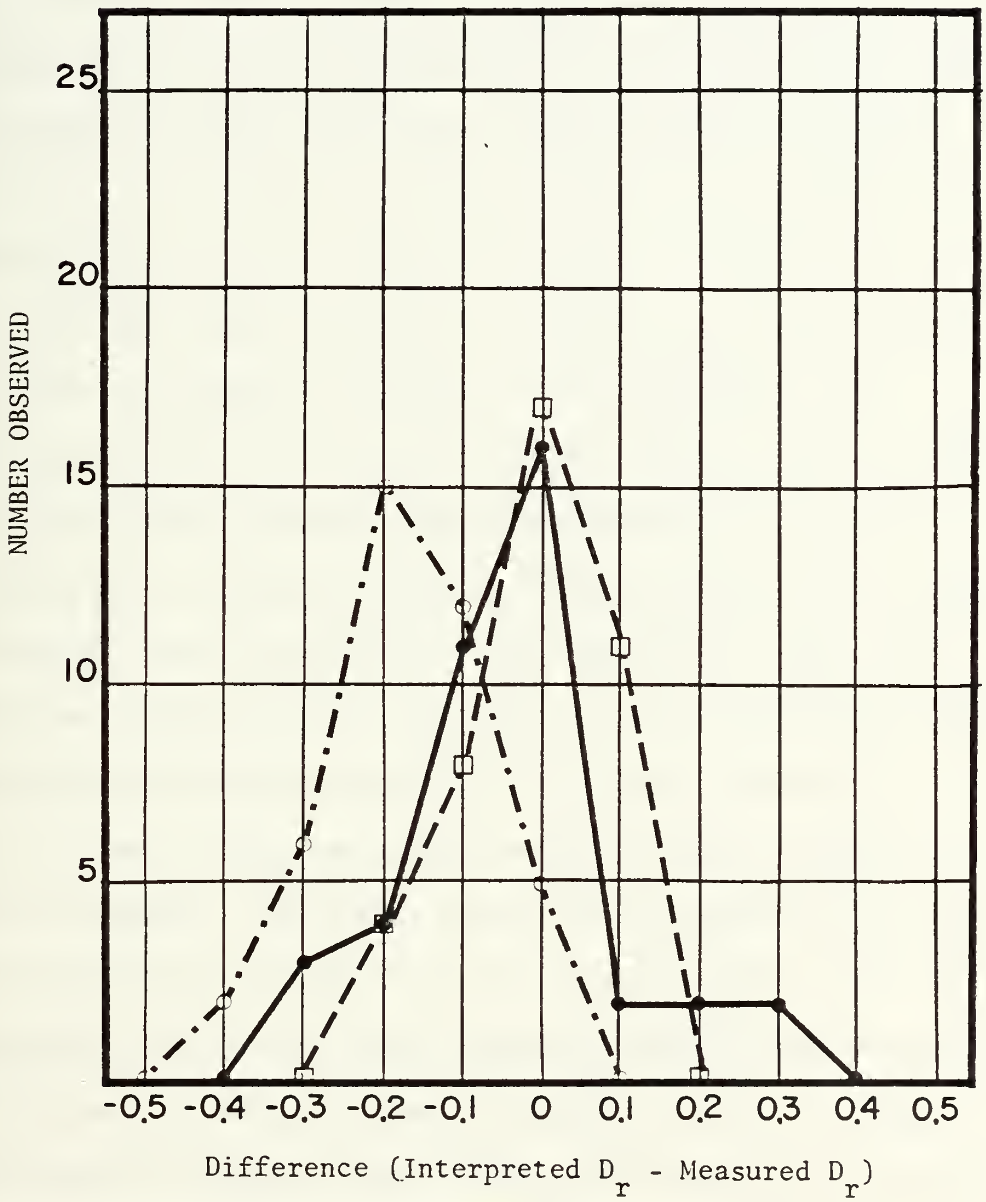
Difference (Interpreted D_r - Measured D_r)

Legend: —— Gibbs and Holtz Correlation (Eq 3)

—·— Bazaraa Correlation (Eq 4)

– – – Waterways Experiment Station Correlation (Eq 5)

FIGURE 6: Differences Between Measured Relative Density and Interpreted Relative Density from Indirect Correlation with the Standard Penetration Test (Red Wing, Treasure Island, and Boca Raton Data).



Legend: ——— Gibbs and Holtz Correlations (Eq 3)

–·– Bazarra Correlation (Eq 4)

– – Waterways Experiment Station Correlation (Eq 5)

It is clear that the inclusion of data from the WES experiments did not prejudice the results in favor of the WES correlation. A comparison of Figures 5 and 6 shows that while the mean values of the WES data were considerably closer to the vertical axis than the mean values of the case histories data, the relative positions of the means and the values of the standard deviations showed no change with each other or with the respective values from the combined data.

Marcuson's (22) observations of changes in in-place density as a result of sampling techniques were then applied to the data to determine if such corrections would have an effect on the statistical data. Marcuson suggested two density corrections, one based upon overburden pressure, and one based upon the height of the sample from the bottom of the sampling tube. His study noted that density increases as high as 3.4 pcf (0.53 kg/cm^3) for a measured relative density of 30%, and density decreases as low as 1.7 pcf (0.27 kg/cm^3) for a measured relative density of 86% were possible. Typically, these errors averaged from 1.2 pcf (0.19 kg/cm^3) for low density soils to 0.6 pcf (0.09 kg/cm^3) for high density soils. In order to maximize the effect on the data, the extreme corrections were applied proportionately through the entire relative density range, with no single correction exceeding -3.4 pcf (0.53 kg/cm^3) for low density soils, or +1.7 pcf (0.27 kg/cm^3) for high density soils. The overburden pressures were re-calculated based upon the corrected in-place densities. The differences between the corrected in-place and corrected indirect relative

density measurements were evaluated over a normal distribution curve. The mean and standard distribution from each correlation are listed in Table 5.

Table 5: Differences Between Measured Relative Density and Interpreted Relative Density from Indirect Correlations with Standard Penetration Resistance, with Allowances for Sampling Effects on Measured Relative Density

Correlation (1)	Mean (2)	Std. Dev. (3)
G-H:	-0.054	0.229
Bazaraa:	-0.219	0.239
WES:	-0.070	0.226

The effect on each correlation's mean and standard deviation with Marcuson's corrections are not materially different than without the correction. While the mean values did tend to migrate closer to zero, the most significant result is that for large data bases, the correction exacerbates data scatter, as evidenced by increased standard deviations. In individual tests or more detailed field studies where corrections could be more judiciously applied, the corrections might give a better indication of in-place density. However, for the above case studies the corrections had little impact and were not subsequently used.

The foregoing were also used to determine how well Standard Penetration Test correlations can be used to approximate angle of

internal friction. Meyerhof (26) compared the relationships between relative density and the angle of internal friction suggested by Peck (30) and show in Table 6, and recommended that the upper values for the friction angle were safe limits for well graded sands, but for silty sands the value of the angle should be reduced five degrees in the absence of shearing tests.

From this, Bowles (5) developed the following equations:

For less than 5% passing #200 sieve,

$$\phi = 30+25D_r \tag{10}$$

For more than 5% passing #200 sieve,

$$\phi = 25+25D_r \tag{11}$$

Where ϕ = angle of internal friction, degrees, and D_r = relative density.

Equations (10) and (11) can be used to show a difference in the angle of internal friction as a function of the difference in relative density. If ϕ_1 is the value of the angle of internal friction obtained from the in-place relative density measurement D_{r1}, and ϕ_2 is the internal friction angle value obtained from a relative density value obtained from an indirect measurement D_{r2}, insertion of these values into Equation (10), for example, and subtracting one from the other results in:

$$\phi_2 - \phi_1 = 25(D_{r2}-D_{r1}) \tag{12}$$

Similar treatment of Equation (11) will give the same results.

TABLE 6: Empirical Values for Angle of Internal Friction and Relative Density Based Upon the Standard Penetration Test for Sands (26)

Condition (1)	SPT N, in blows per foot (2)	Relative Density (3)	Angle of internal friction in degrees (4)
Very loose	5	0.15	30
Loose	5 - 10	0.15 - 0.35	30 - 35
Medium	10 - 30	0.35 - 0.65	35 - 40
Dense	30 - 50	0.65 - 0.85	40 - 45
Very dense	50	0.85	45

Note: 1 ft = 30.48 cm

The differences between direct and indirect relative density measurements for each correlation obtained previously were inserted into equation (12).

The mean, standard deviation and 95% confidence level intervals of the differences between the angles of internal friction for each correlation are shown in Table 7.

Table 7: Differences Between Angles of Internal Friction Using Measured Relative Density and Interpreted Relative Density from Indirect Correlations with Standard Penetration Resistance

Correlation (1)	Mean (2)	Std. Dev. (3)	Lower Limit (4)	Upper Limit (5)
G-H:	-1.724	4.931	-2.772	-0.676
Bazaraa:	-5.770	4.602	-6.748	-4.792
WES:	-2.069	4.451	-3.015	-1.123

The same general characteristics of the Gibbs and Holtz, Bazaraa, and WES correlations observed in Figure 4 are present in Figure 5: Bazaraa shows considerable conservatism in calculated angle of internal friction values while the Gibbs and Holtz and WES correlations show fair agreement and less conservatism. The Gibbs and Holtz correlation gave angle of internal friction values that are typically one-half degree closer to the internal friction angle value obtained from direct relative density measurements than does the WES correlation, indicating that its use in obtaining angle of internal friction values for design purposes would be more appropriate than the WES correlation.

DISCUSSION

The Bazaraa correlation was originally developed to predict maximum settlements of shallow foundations on dry sand. As shown in Figure 4, the general use of this correlation to determine relative density of deep sands produces, understandably, excessively conservative values. Therefore, its use in such an application is not recommended.

The Gibbs and Holtz and WES correlations are both reliable conservative methods for estimating relative density from SPT blow count data. Gibbs and Holtz (12) noted that the overburden pressure applied in their experiments did not exceed 40 psi (275.77 kPa), and suggested that their correlation is not valid beyond 40 psi (275.77 kPa). However, in the Marcuson case study, 12 of his 40 experiments were performed with an overburden pressure of 80 psi (551.54 kPa). In only 2 of the 12 experiments did the Gibbs and Holtz correlation yield a relative density value higher than the in-place relative density value. The balance of the values averaged 11 percent lower than the in-place relative density values, suggesting that even above 40 psi (275.77 kPa), the Gibbs and Holtz correlation will furnish reasonable, if conservative, relative density values.

The WES correlation, while on the average being slightly more conservative than Gibbs and Holtz (eight percent lower than actual as opposed to about seven percent for G-H), provides greater consistency in its values as shown by the smaller standard deviation (0.178 compared to 0.197). The inclusion of the coefficient of uniformity into the correlation

requires that the sample be subjected to grain size analysis before a determination concerning relative density can be made. This may cause a preference for the Gibbs and Holtz correlation for use in the field, as a general indication of relative density is quickly obtainable from knowing the depth of the sample and a reasonable approximation of the density of the overburden. However, with laboratory tests available, the WES correlation would render a more accurate relative density value with higher confidence levels than would the Gibbs and Holtz correlation.

Peck (30) and Meyerhof (26) have shown that there is a direct relationship between the angle of internal friction and relative density; the internal friction angle increases as relative density increases. It is possible to substitute relative density - SPT N-value correlations into a relative density - angle of internal friction correlation to obtain a relationship which expresses angle of internal friction as a function of the SPT blow count and overburden pressure. Marcuson's (22) study indicated that a simplified family of curves correlating SPT N-values, relative density and overburden pressure for all cohesionless soils under all conditions is not valid. Likewise, de Mello (25) states that a universal correlation for relative density, angle of internal friction and overburden pressure is not possible. However, de Mello also noted that a correlation between SPT N-values, angle of internal friction and overburden pressure, independent of relative density, is possible and can be considered valid in general application. Of the three

methods available for the indirect measurement of relative density in terms of SPT blow count and overburden pressure, the Waterways Experiment Station correlation appears to be the one best suited for use with other correlations to produce a correlation for angle of internal friction directly, without an intermediate step of determining relative density.

The data seems to indicate that the Waterways Experiment Station correlation developed by Marcuson and Bieganousky is a more accurate method of determining relative density by indirect means than the more popular Gibbs and Holtz method. Use of this correlation would reduce the number of field tests necessary to obtain a reliable indication of subsurface in-place relative density.

CONCLUSIONS

Circumstances concerning the nature and extent of subsurface exploration vary with the type of deposit encountered and the extent of available resources. In regard to the selection of a method by which the relative density and angle of internal friction of a cohesionless soil can be ascertained with reasonable accuracy, the following conclusions are offered as a guide:

(1) For non-uniform, or variable, deposits, or instances where grain size analysis is not available or considered unnecessary, use of the Gibbs and Holtz correlation will provide relative density values averaging about seven percent lower than actual values, and angle of internal friction values averaging about 1.7 degrees lower than actual values.

(2) For uniform deposits, or where grain size analysis is considered necessary to augment other knowledge of subsurface conditions, the Waterways Experiment Station correlation should be used, as it will render relative density and internal friction angle values at least equivalent to the Gibbs and Holtz correlation, with fewer samples necessary to obtain the same level of confidence.

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APPENDIX A

GRAIN SIZE ANALYSIS

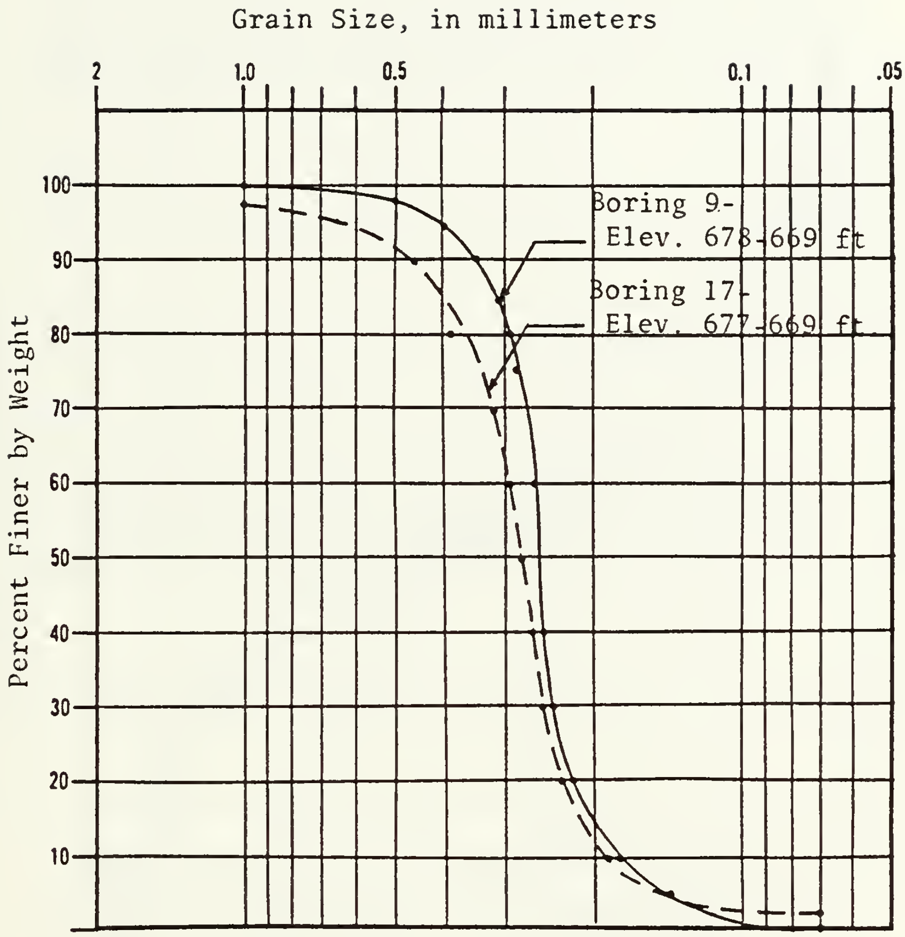

Figure A-1 Nuclear Power Plant, Red Wing, Minnesota

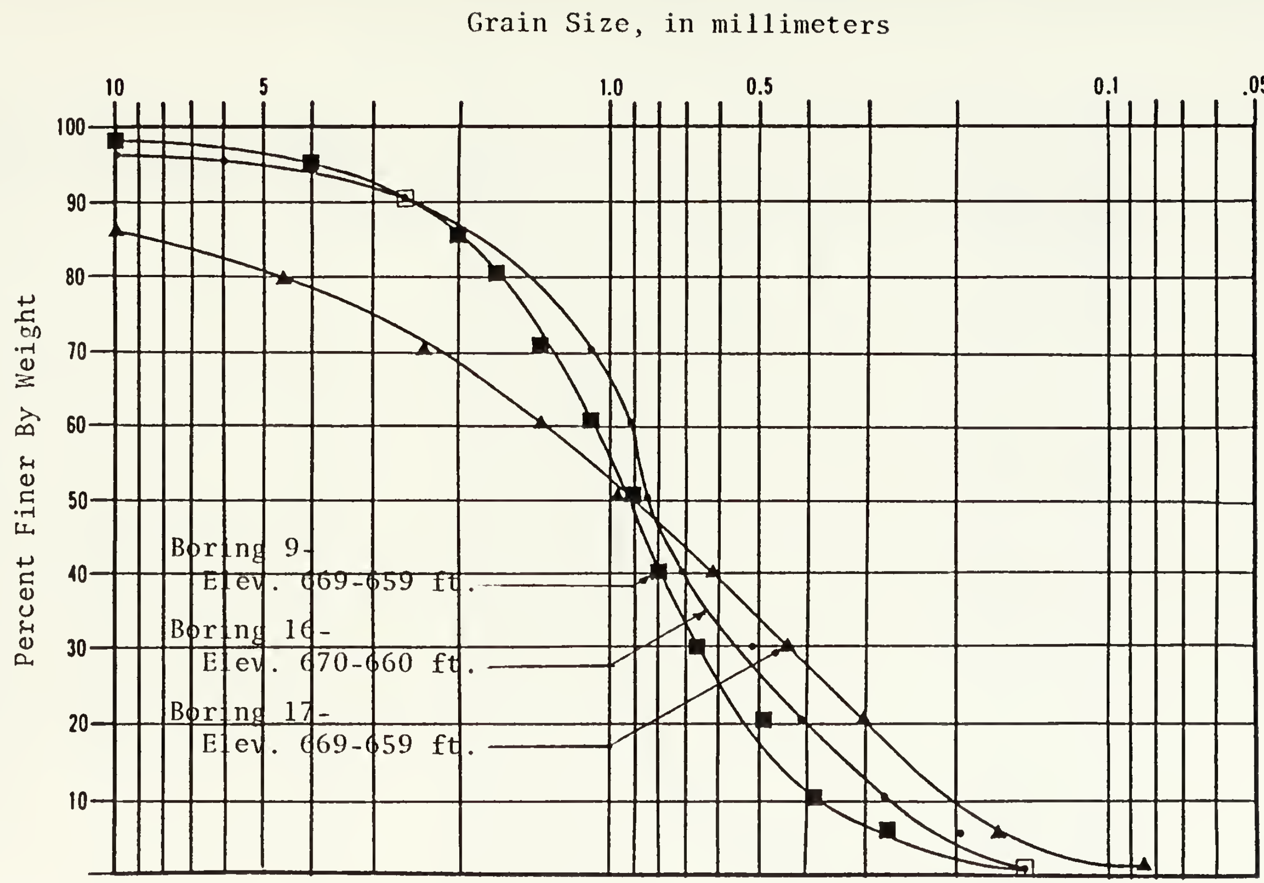

Figure A-2 Nuclear Power Plant, Red Wing, Minnesota

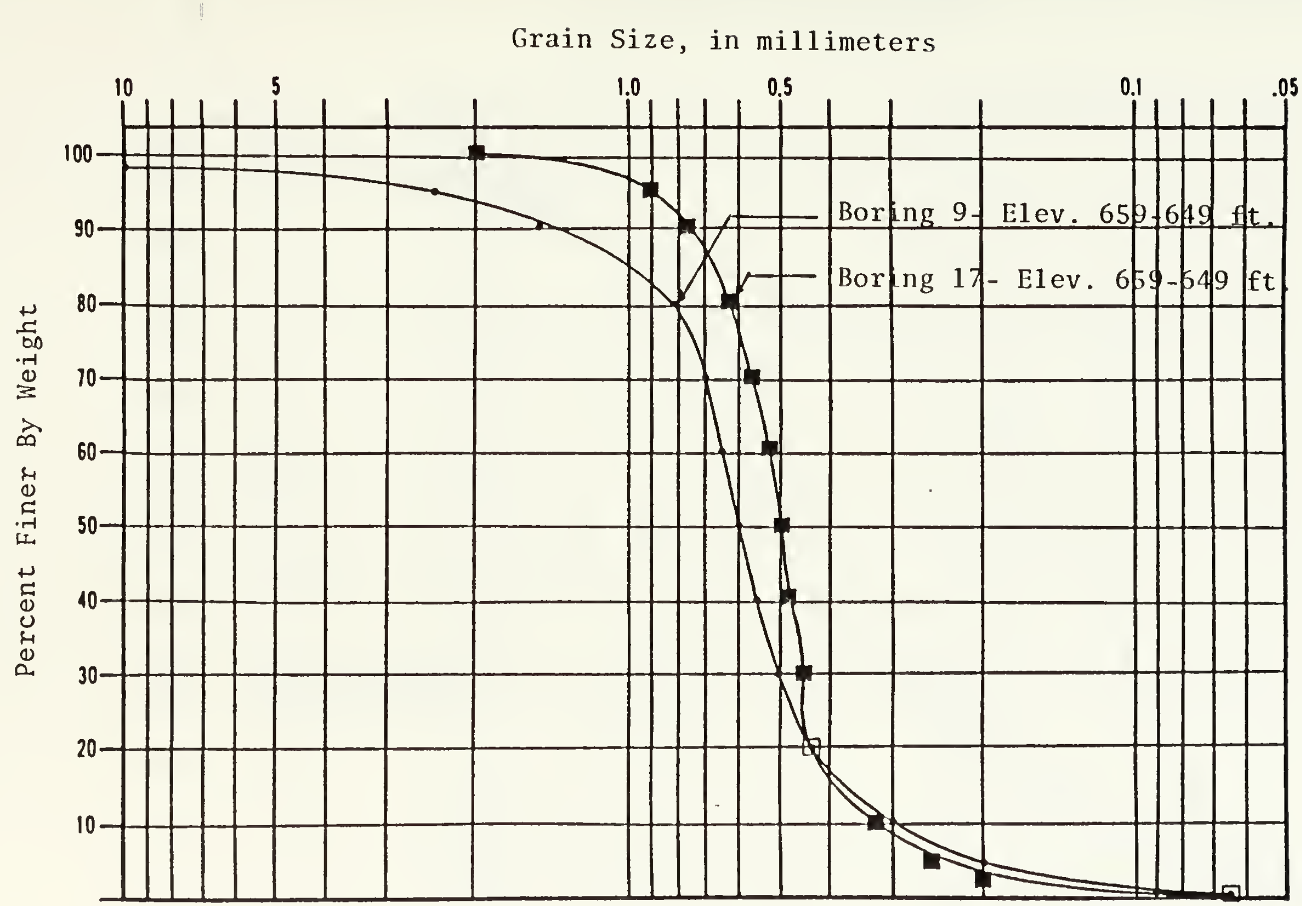

Figure A-3 Nuclear Power Plant, Red Wing, Minnesota

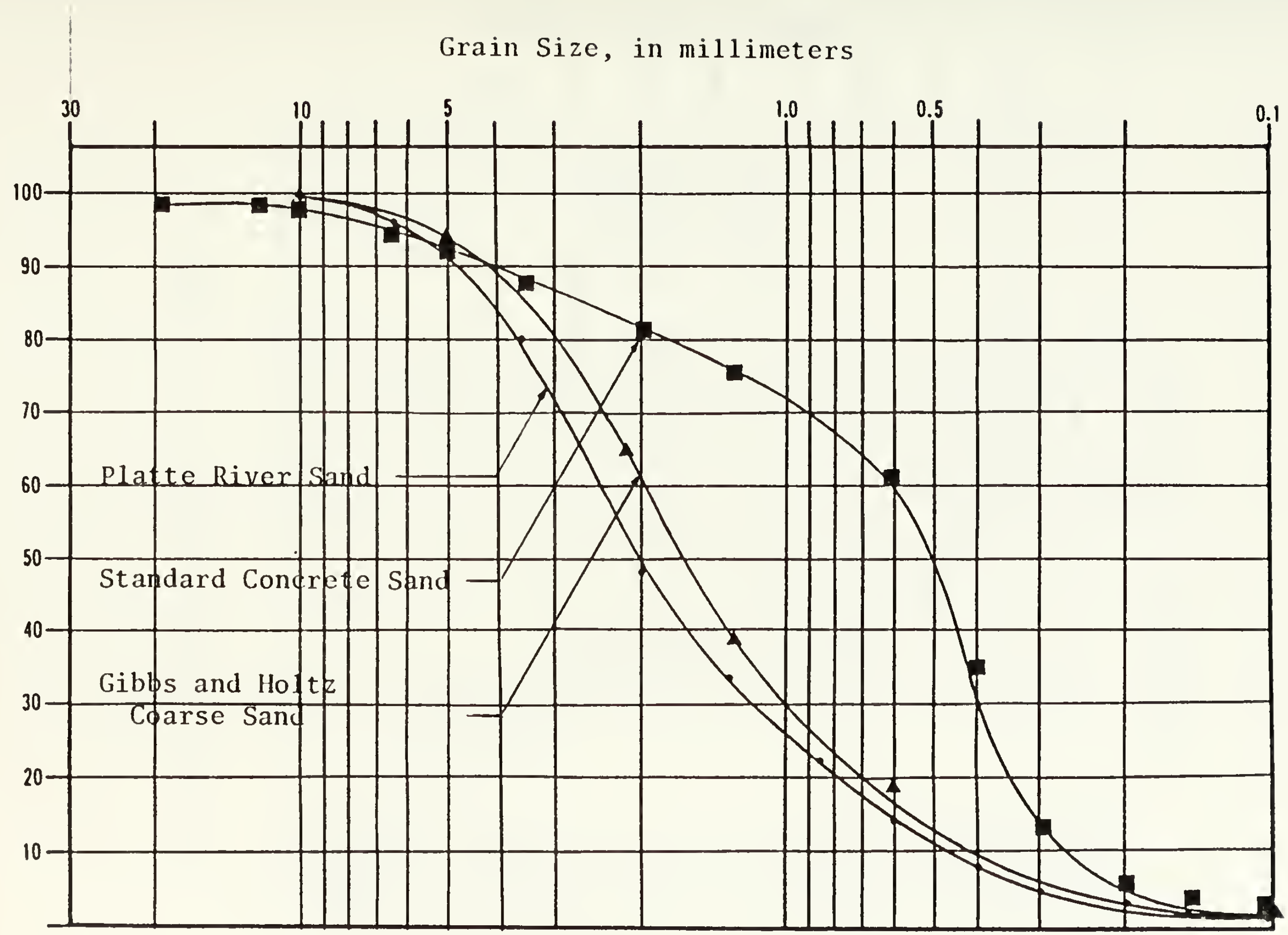

Figure A-4 Waterways Experiment Station Data

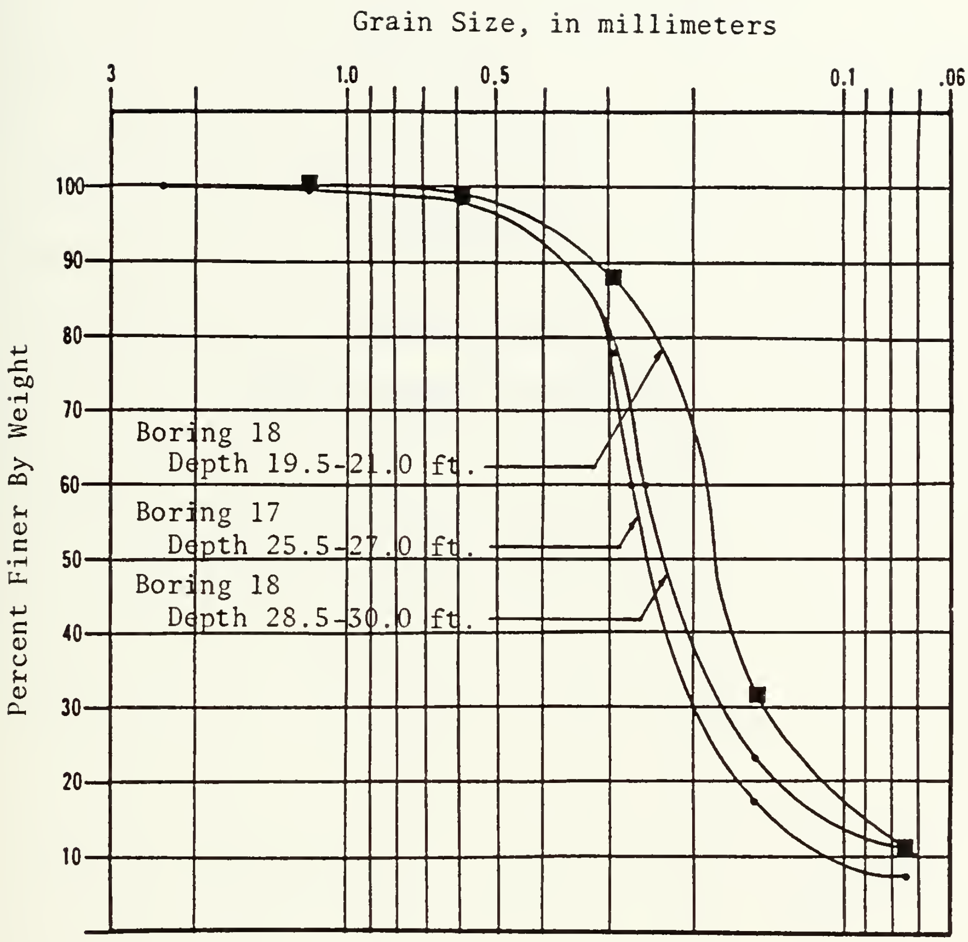

Figure A-5 Treasure Island, California

APPENDIX B

DISTRIBUTION OF

RELATIVE DENSITY COMPARISONS

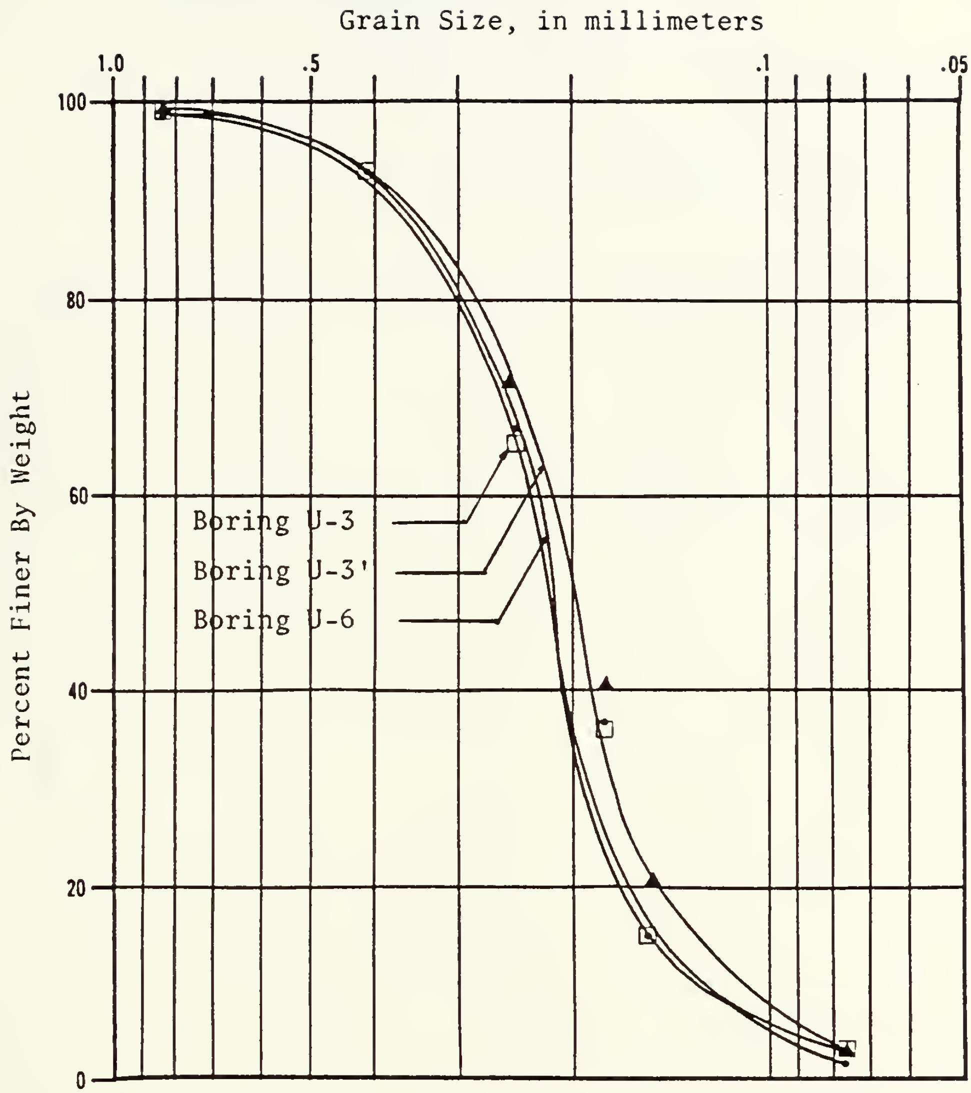

Figure A-6 Florida Atlantic University,
Boca Raton, Florida

RELATIVE DENSITY COMPARISONS

DIFFERENCES BETWEEN
INDIRECT MEASUREMENTS AND
IN-PLACE MEASUREMENTS

ALL DATA POINTS

VALUES BETWEEN	G-H	BAZ	WES
-.50	4	5	3
-.5 & -.45	0	3	0
-.45 & -.4	1	2	1
-.4 & -.35	3	4	1
-.35 & -.3	1	7	2
-.3 & -.25	2	16	4
-.25 & -.2	3	4	5
-.2 & -.15	6	10	9
-.15 & -.1	13	15	12
-.1 & -.05	7	10	6
-.05 & 0	10	4	9
0 & .05	13	5	14
.05 & .1	13	0	8
.1 & .15	3	0	7
.15 & .2	2	0	4
.2 & .25	1	0	0
.25 & .3	1	0	0
.3 & .35	2	0	0
.35 & .4	0	0	0
.40	0	0	0
TOTAL	85	85	85
MEAN	-.069	-.231	-.083
STD DEV	.197	.184	.178

Legend: G-H Gibbs and Holtz Correlation (Eq 3)
BAZ Bazaraa Correlation (Eq 4)
WES Waterways Experiment Station Correlation (Eq 5)

RELATIVE DENSITY COMPARISONS

DIFFERENCES BETWEEN
INDIRECT MEASUREMENTS AND
IN-PLACE MEASUREMENTS

WATERWAYS EXPERIMENT STATION DATA

VALUES BETWEEN	G-H	BAZ	WES
-.50	0	0	0
-.5 & -.45	0	0	0
-.45 & -.4	0	0	0
-.4 & -.35	0	0	0
-.35 & -.3	0	2	0
-.3 & -.25	0	6	0
-.25 & -.2	3	0	0
-.2 & -.15	0	7	1
-.15 & -.1	4	8	3
-.1 & -.05	2	8	2
-.05 & 0	9	4	6
0 & .05	10	5	10
.05 & .1	6	0	7
.1 & .15	2	0	7
.15 & .2	0	0	4
.2 & .25	1	0	0
.25 & .3	1	0	0
.3 & .35	2	0	0
.35 & .4	0	0	0
.40	0	0	0
TOTAL	40	40	40
MEAN	.015	-.127	.033
STD DEV	.125	.094	.088

RELATIVE DENSITY COMPARISONS

DIFFERENCES BETWEEN
INDIRECT MEASUREMENTS AND
IN-PLACE MEASUREMENTS

RED WING, TREASURE IS., AND BOCA RATON DATA

VALUES BETWEEN	G-H	BAZ	WES
-.50	4	5	3
-.5 & -.45	0	3	0
-.45 & -.4	1	2	1
-.4 & -.35	3	4	1
-.35 & -.3	1	5	2
-.3 & -.25	2	10	4
-.25 & -.2	0	4	5
-.2 & -.15	6	3	9
-.15 & -.1	9	7	0
-.1 & -.05	5	2	4
-.05 & 0	1	0	3
0 & .05	3	0	4
.05 & .1	7	0	1
.1 & .15	1	0	0
.15 & .2	2	0	0
.2 & .25	0	0	0
.25 & .3	0	0	0
.3 & .35	0	0	0
.35 & .4	0	0	0
.40	0	0	0
TOTAL	45	45	45
MEAN	-.145	-.324	-.187
STD DEV	.218	.195	.174

APPENDIX C

DISTRIBUTION OF
ANGLE OF INTERNAL FRICTION COMPARISONS

ANGLE OF INTERNAL FRICTION COMPARISONS

DIFFERENCES BETWEEN
INDIRECT MEASUREMENTS AND
IN-PLACE MEASUREMENTS

ALL DATA POINTS

VALUES BETWEEN	G-H	BAZ	WES
-13	4	5	3
-13 & -12	0	2	0
-12 & -11	0	2	0
-11 & -10	1	1	1
-10 & - 9	3	3	0
- 9 & - 8	1	6	2
- 8 & - 7	0	8	1
- 7 & - 6	3	11	4
- 6 & - 5	1	3	5
- 5 & - 4	7	4	9
- 4 & - 3	7	17	6
- 3 & - 2	7	10	0
- 2 & - 1	7	4	10
- 1 & 0	9	4	3
0 & 1	8	5	12
1 & 2	13	0	8
2 & 3	7	0	6
3 & 4	2	0	4
4 & 5	1	0	3
5 & 6	0	0	0
6 & 7	2	0	0
7 & 8	0	0	0
8 & 9	2	0	0
9 & 10	0	0	0
10 & 11	0	0	0
11 & 12	0	0	0
12 & 13	0	0	0
13	0	0	0
TOTAL	85	85	85
MEAN	-1.724	-5.770	-2.069
STD DEV	4.93	4.602	4.451

APPENDIX D

NOTATION

NOTATION

The following symbols are used in this paper:

C_a = area ratio

C_i = inside clearance ratio

C_u = coefficient of uniformity

cm = centimeter

D_e = inside diameter of the sampler cutting

D_r = relative density

D_s = inside diameter of the sampler tubing

D_w = outside diameter of the sampler

e = void ratio

ft = foot

H = sampler penetration below the bottom of the borehole

in. = inch

kg = kilogram

kPa = kiloPascal

L = length of the sample before withdrawl

m = meter

N = Standard Penetration Test blow count

p = effective overburden pressure

pcf = pounds per cubic foot

psf = pounds per square foot

psi = pounds per square inch

R = total recovery ratio

r = specific recovery ratio

Vv = Volume of voids

γ = unit weight

γ_d = dry unit weight

ϕ = Angle of internal friction

Subscripts

M = maximum

m = minimum

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c.1
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