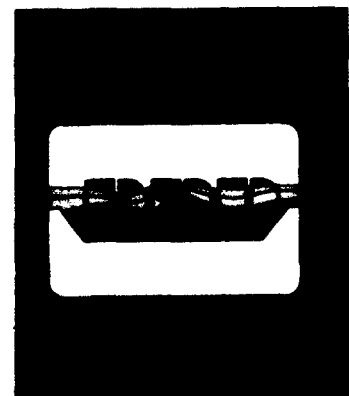
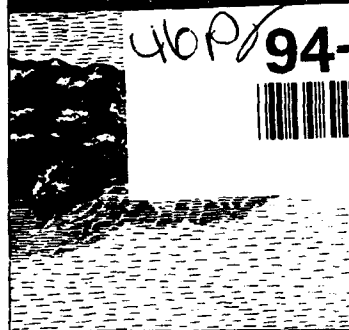
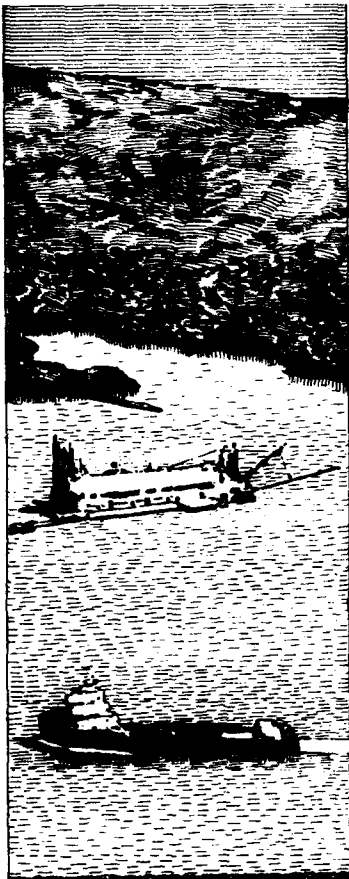


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DREDGING RESEARCH PROGRAM

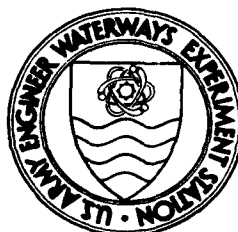
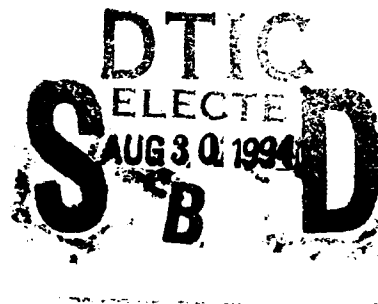
CONTRACT REPORT DRP-94-3

DESCRIPTORS FOR GRANULAR BOTTOM
SEDIMENTS TO BE DREDGED

by

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Newark, Delaware 19716



June 1994

Final Report

Approved For Public Release; Distribution Is Unlimited

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Prepared for DEPARTMENT OF THE ARMY
U.S. Army Corps of Engineers
Washington, DC 20314-1000

Under Work Unit 32471

Monitored by U.S. Army Engineer Waterways Experiment Station
3909 Halls Ferry Road, Vicksburg, Mississippi 39180-6199

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The Dredging Research Program (DRP) is a seven-year program of the U.S. Army Corps of Engineers. DRP research is managed in these five technical areas:

- Area 1 - Analysis of Dredged Material Placed in Open Water**
- Area 2 - Material Properties Related to Navigation and Dredging**
- Area 3 - Dredge Plant Equipment and Systems Processes**
- Area 4 - Vessel Positioning, Survey Controls, and Dredge Monitoring Systems**
- Area 5 - Management of Dredging Projects**

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Dredging Research Program Report Summary



Descriptors for Granular Bottom Sediments to be Dredged (CR DRP-94-3)

ISSUE: Existing soil descriptor systems are not universally used or even understood by all groups involved in designing, planning, and executing dredging projects. The disparities increase risk factors and thus the cost of such projects.

RESEARCH: The primary objectives of a Dredging Research Program (DRP) work unit entitled "Descriptors for Bottom Sediments to be Dredged" are as follows:

- Identify appropriate geotechnical engineering parameters, develop standard dredging material descriptors based on the parameters, and correlate the parameters with dredge equipment performance.
- Identify techniques suitable for measurement of appropriate geotechnical parameters.

A resource review was conducted that included technical literature and standard test methods to relate the physical properties of granular soils to descriptors of anticipated difficulty associated with dredge cutting. The

resulting procedures will be used as input to meet the objectives of the work unit.

SUMMARY: A step-by-step procedure was developed for use in determining descriptors to typify the dredgeability of granular sediments. The procedure included field testing and simple laboratory tests as well as two possible supplemental approaches to use to confirm the resulting descriptors. The extent of the site investigation and the subsequent soil characterization are identified as determining the reliability of the descriptors.

AVAILABILITY OF THE REPORT: The report is available through the Interlibrary Loan Service from the U.S. Army Engineer Waterways Experiment Station (WES) Library, telephone number (601) 634-2355. National Technical Information Service (NTIS) report numbers may be requested from WES Librarians.

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Descriptors for Granular Bottom Sediments to be Dredged

by **Dov Leshchinsky**

**Department of Civil Engineering
University of Delaware
Newark, DE 19716**

Final Report

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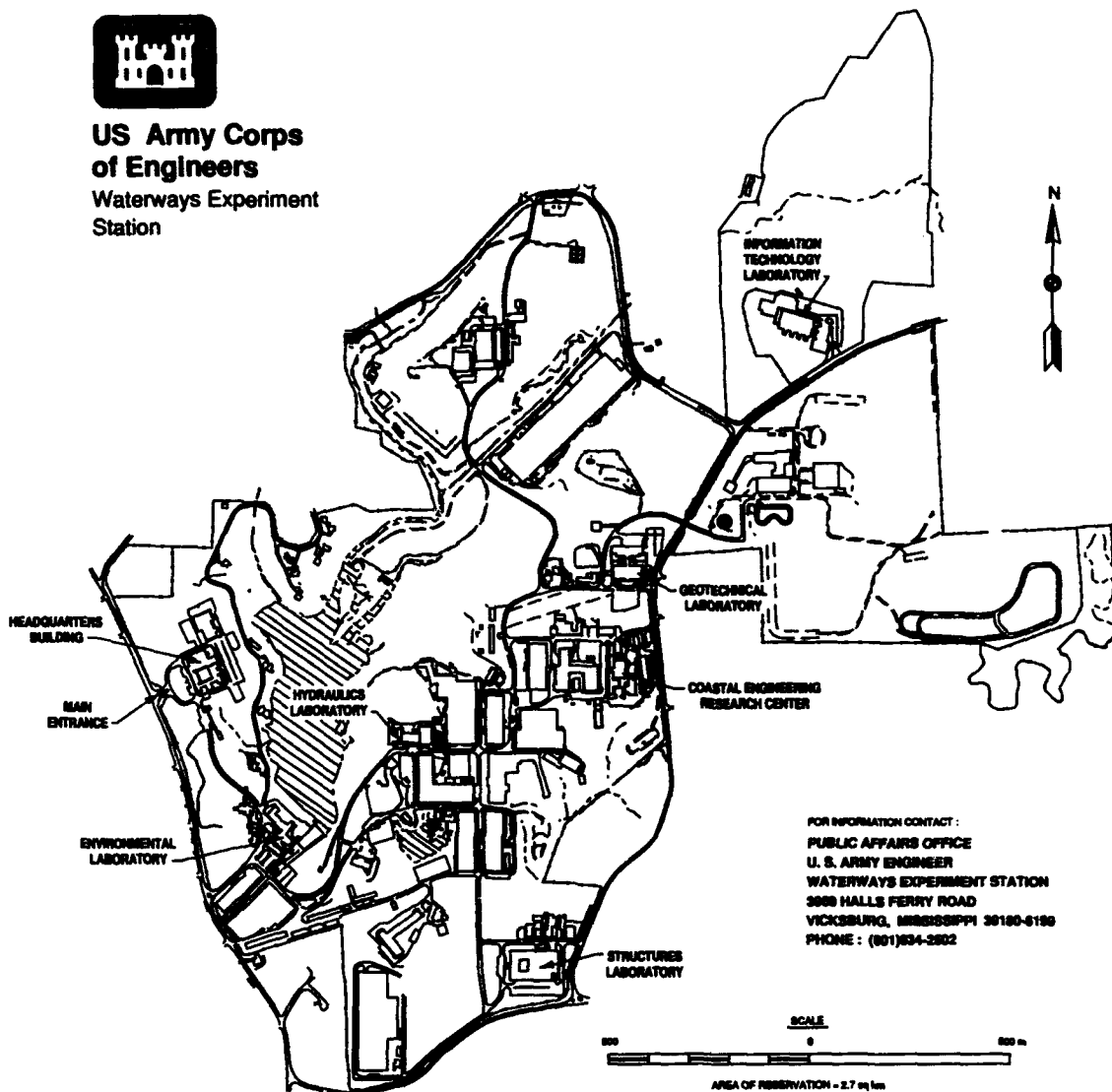
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Preface

This report was prepared under Intergovernmental Personnel Agreement No. 89-14-C for the US Army Engineer Waterways Experiment Station (WES) under Dredging Research Program (DRP) Technical Area 2, Work Unit No. 32471, "Descriptors for Bottom Sediments to be Dredged." The DRP is sponsored by Headquarters, US Army Corps of Engineers (HQUSACE). Technical Monitors for the DRP were Messrs. Glenn Drummond, Vince Montante, Rixie Hardy, and John Perez. HQUSACE Advisors were Messrs. M. K. Miles, Ben Kelly, and Don Pommer.

This report was written by Dr. Dov Leshchinsky, Professor of Civil Engineering, University of Delaware, Newark, DE, under the technical oversight of Dr. Jack Fowler, Principal Investigator, Soil Mechanics Branch (SMB), Soil and Rock Mechanics Division (S&RMD), Geotechnical Laboratory (GL); and Mr. Milton Myers, Chief, SMB, GL; Dr. Don C. Banks, Chief, S&RMD, GL; and Dr. W. F. Marcuson III, Director, GL. Dr. Banks was also the Manager for Technical Area 2, "Material Properties Related to Navigation and Dredging," of the DRP. Mr. E. Clark McNair, Jr., and Ms. Carolyn M. Holmes, Coastal Engineering Research Center (CERC), WES, were Manager and former Assistant Manager, respectively, of the DRP. Dr. James R. Houston and Mr. Charles C. Calhoun, Jr., were Director and Assistant Director, respectively, of CERC, which oversees the DRP.

At the time of publication of this report, Director of WES was Dr. Robert W. Whalin. Commander was COL Bruce K. Howard, EN.

For further information on this report or on the Dredging Research Program, please contact Mr. E. Clark McNair, Program Manager, at (601) 634-2070.

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Summary

A step-by-step procedure to establish an unbiased description of the difficulty associated with the dredge cutting of granular soils is presented. It is based on the effective shear strength of saturated cohesionless soils. The procedure accounts for the effective angle of internal friction and for the potential of the soil to rapidly dissipate excess pore-water pressures developing in response to dredge cutting. This potential is assessed from an estimate of the typical permeability of the soil at the site.

A practical procedure to estimate the friction angle (ϕ') and the coefficient of permeability (k) is detailed. It is based on the representative in-situ relative density. The relative density (D_r) is estimated from direct measurements of density and water content (a method is proposed) at the site, supplemented by laboratory tests to determine the possible minimum/maximum dry densities of the granular soil. However, since there is no established laboratory test standard for min/max density of silty soils, an appropriate test has been developed and verified as part of this work. Based on the Unified Soil Classification System (USCS), the soil is identified. Utilizing the USCS classification together with D_r , the coefficient of permeability k and the friction angle ϕ' are estimated from well-established correlations. Subsequently, the descriptors for the dredge cutting are evaluated. A qualitative scale of anticipated difficulty (i.e., easy to very difficult) is presented.

For sandy soils, it is recommended to also determine the descriptors by either the Standard Penetration Test (SPT) or the Cone Penetrometer Test (CPT). As indicated in the report, these field tests produce indirect information about D_r and therefore should be considered supplemental. However, these tests may provide a good indication about the difficulty of dredge cutting and thus be helpful in substantiating the descriptors. It is pointed out that the reliability of the descriptors depends mainly on the extent of site investigation and the subsequent characterization of the soils.

Conversion Factors. Non-SI to SI (Metric)

Units of Measurement

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

<u>Multiply</u>	<u>By</u>	<u>To Obtain</u>
feet	0.3048	metres
feet per minute	0.005080	metres per second
inches	0.02540	metres
pounds (force) per square foot	47.88026	pascals
pounds (mass)	0.4535924	kilograms
pound per cubic foot	16.01846	kilograms per cubic metre

DESCRIPTORS FOR GRANULAR BOTTOM SEDIMENTS TO BE DREDGED

Introduction

1. The scope of the problems associated with the dredging of soil sediments is covered comprehensively by Spigolon (1988). The report herein is limited to granular soils, i.e., silt, sand, gravel, or a combination of these cohesionless soils. Cohesive soils and clay balling are being addressed in another report titled "Behavior of Hydraulically Transported Clay Balls." A step-by-step procedure to establish unbiased descriptors characterizing the dredgeability of a particular site is developed. These descriptors convert physical properties of the granular soil into a qualitative scale of anticipated difficulty associated with dredge cutting; i.e., easy to very difficult. It should be stated initially that the scale sensitivity to soil properties has not been verified yet in dredge cutting operations. However, only experience with using the developed procedure will allow adjustment and refinement of the descriptor.

2. The ease of dredging a soil decreases as its effective shear strength increases. Ideally, laboratory shear tests on undisturbed specimens should provide direct information about the strength of the in situ soil. However, undisturbed samples of underwater granular soils are virtually impossible to obtain and test properly in the laboratory. Alternatively, sophisticated laboratory shear tests on remolded specimens can be conducted. These tests require speculative and complicated interpretations of results because of the little resemblance of the relevant behavior of the specimens in the laboratory to the field behavior. Practically, proper site investigation combined with appropriate soil classification and simple field and laboratory tests may provide the necessary information to characterize the dredgeability of a granular soil site.

Relative Density

3. Relative density is frequently used in problems of foundation engineering associated with clean sands. As a result, significant experience with regard to its value as related to soil strength has been gained. Therefore, while it is a difficult parameter to accurately determine for in situ sands, it appears that a useful description of dredgeability of granular soils may emerge from the in situ relative density concept. That is, a typical value of

relative density characterizing a site may provide an intelligent clue as to how difficult it will be to dredge it, and what equipment and procedure should be used.

4. Relative density is defined as

$$D_r = \frac{e_{\max} - e_{\text{in-situ}}}{e_{\max} - e_{\min}} \quad (1)$$

or, alternatively,

$$D_r = \frac{\gamma_{d\text{in-situ}} - \gamma_{d\min}}{\gamma_{d\max} - \gamma_{d\min}} \times \frac{\gamma_{d\max}}{\gamma_{d\text{in-situ}}} \quad (2)$$

where

D_r = relative density

$e_{\max}$, $e_{\min}$, $e_{\text{in-situ}}$ = maximum, minimum, and in-situ void ratios, respectively

$\gamma_{d\max}$, $\gamma_{d\min}$, $\gamma_{d\text{in-situ}}$ = maximum, minimum, and in-situ dry density, respectively

Note that the minimum and maximum void ratios correspond, respectively, to the maximum and minimum dry densities obtained in arbitrary standardized tests and are not necessarily the absolute extreme density values physically obtainable for the material. Strictly speaking, these standardized tests apply only to sand and gravel. Appendix A provides a procedure for silts as well as stating the standards for sands and gravels. Once the limiting densities (i.e., $\gamma_{d\max}$ and $\gamma_{d\min}$) for a particular soil have been determined following the procedure described in Appendix A, and a representative value of $\gamma_{d\text{in-situ}}$ has been selected as described in paragraph 8, one can calculate the typical relative density at a site using Equation 2.

5. The broad description of an in-situ state of compactness of soil, associated with a given relative density, is generally accepted as follows (Lambe and Whitman 1969):

<u>Relative Density Range, D_r, percent</u>	<u>Description of Compactness</u>
0-15	Very loose
15-35	Loose
35-65	Medium
65-85	Dense
85-100	Very dense

6. Density, and subsequently relative density, may be of critical importance when associated with strength of submerged silts and fine to medium sands when subjected to dredge cutting at a given rate. Loose granular soil exhibits an apparent decrease in its strength during rapid dredge cutting (i.e., when sheared under virtually undrained conditions as compared with shear under drained conditions). Conversely, dense soil will exhibit an apparent increase in strength if the cutting is rapid (e.g., Van Os and Van Leussen 1987). This phenomenon has to do with the concept of effective shear strength and with the development of pore pressures during undrained shear. That is, it depends on the tendency of soil to dilate/contract during shear and the drainage conditions which are controlled by the soil's permeability. The permeability, in turn, controls the rate of dissipation of excess pressures developing in response to volume change during shear. In other words, during the process of dredging saturated granular soil, the shear (or cutting) rate and permeability are important because of the effect of dilatancy/contraction: decrease in pore-water pressure, due to dilatancy under nearly undrained conditions, occurs in dense soil with low permeability when sheared rapidly; consequently, an increase in effective stresses occurs resulting in an increase in the strength. The higher effective stresses result in larger required cutting forces at high shear rates. This phenomenon is most notable with soils having a coefficient of permeability less than 0.01 cm/sec ($k < 0.01$ cm/sec). When $k > 0.1$ cm/sec there should not be a problem associated with an apparent increase in strength considering the possible rates of available dredging equipment (i.e., for $k > 0.1$ cm/sec the soil is considered to exhibit drained behavior and, therefore, no negative excess pore-water pressure develops during dredge cutting). Only GP and GW soils consistently fall in this category.

7. It should be pointed out that $\gamma_{d_{min}}$ can be used to estimate the required maximum containment volume for the dredged material. A brief discussion about this application is presented in Appendix B.

Assessment of Dredgeability

8. It appears that characterization of granular soils based on their relative density is useful in establishing a descriptor that defines the difficulty associated with the dredging at a particular site. However, the soil permeability also should be accounted for in establishing the descriptor. The following steps are required to attain this descriptor (see also Figure 1).

- a. Estimate the typical in-situ wet density (γ) and water content (w). This can be done using a nuclear probe (ASTM D 2922 and D 3017*). Alternatively, some samplers may cause only a slight disturbance of density; thus they might be suitable for the estimate of in-situ density and, possibly, water content. Typical γ and w should be estimated at depths relevant to the planned excavation.
- b. Identify visually the soil in the vicinity where γ and w were estimated. Use the guidelines given in NAVFAC (1982); see Table 1. This identification should be used to verify step c.
- c. Ship the soil (at least 50 lb** if silt or sand, and at least 100 lb if gravel), dug in the vicinity where step a was conducted, to the laboratory. Classify this soil using the Unified Soil Classification System (ASTM D 2487); see Table 2. Note that classification requires grain size analysis (ASTM D 422). Make sure that the conclusions obtained from steps b and c are compatible.
- d. Using ASTM D 854 determine the specific gravity of soil solids, G_s . As a guide in determining whether G_s is reasonable when considering the type of soil, use Table 3.
- e. Using the field values of γ and w obtained in step a, calculate the in-situ void ratio, i.e.,

(3)

$$e_{in-situ} = (1 + w) G_s \frac{\gamma_w}{\gamma} - 1$$

* Test methods referred to in this manner are from Annual Book of ASTM Standards, Vol 04.08.

** A table of factors for converting non-SI units of measurement to SI (metric) units is presented on page 4.

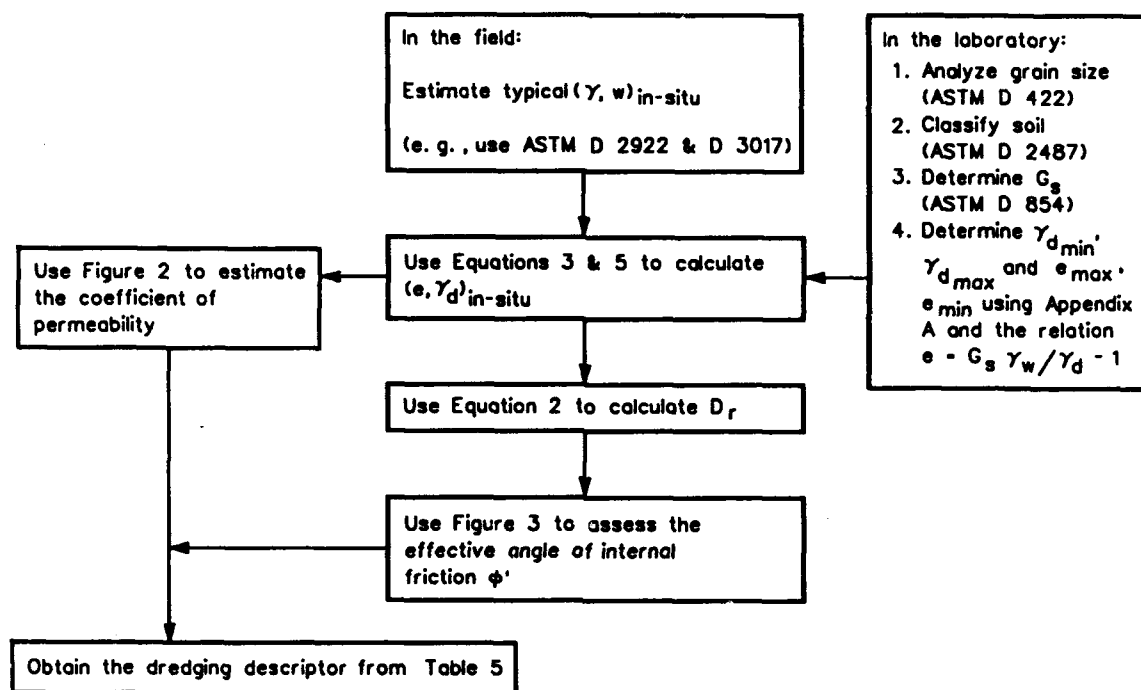


Figure 1. Flow chart to obtain the dredging descriptors using direct measurement of field density

where γ_w is the water unit weight (= 1 grams/cubic centimeter). To assess the reliability of results, calculate the degree of saturation

$$S = \frac{w G_s}{e_{\text{in-situ}}} \quad (4)$$

The degree of saturation, S , cannot be greater than 100 percent. Also, if gas bubbles were not observed at the site, S should be nearly 100 percent (typically $100\% > S > 95\%$). Calculate the in-situ dry density, $\gamma_{d \text{ in-situ}}$

$$\gamma_{d \text{ in-situ}} = \frac{\gamma}{1 + w} \quad (5)$$

Table 1
Visual Identification of Samples NAVFAC. 1982

Definitions of Soil Components and Fractions

1. Grain size

<u>Material</u>	<u>Fraction</u>	<u>Sieve Size</u>
Boulders		12"+
Cobbles		3" - 12"
Gravel	Coarse Fine	3/4" - 3" No. 4 to 3/4"
Sand	Coarse Medium Fine	No. 10 to No. 4 No. 40 to No. 10 No. 200 to No. 40
Fines (silt & clay)		Passing No. 200

2. Coarse- and fine-grained soils

<u>Descriptive Adjective</u>	<u>Percentage Requirement</u>
Trace	1 - 10%
Little	10 - 20%
Some	20 - 35%
And	35 - 50%

3. Fine-grained soils. Identify in accordance with plasticity characteristics, dry strength, and toughness as described in Table 2.

	<u>Descriptive Term</u>	<u>Thickness</u>
Stratified Soils	Alternating Thick	
	Thin	
	With Parting	- 0 to 1/16" thickness
	Seam	- 1/16 to 1/2" thickness
	Layer	- 1/2 to 12" thickness
	Stratum	- Greater than 12" thickness
	Varved clay	- Alternating seams or layers of sand, silt and clay
	Pocket	- Small, erratic deposit, usually less than 1 foot
	Lens	- Lenticular deposit
	Occasional	- One or less per foot of thickness
	Frequent	- More than one per foot of thickness

Table 2

Unified Soil Classification System (reproduced from NAVFAC 1982)

Primary Divisions for Field and Laboratory Identification		Group Symbol	Typical Names	Laboratory Classification Criteria	Supplementary Criteria for Visual Identification
Coarse-grained soils. (More than half of the coarse fraction is larger than 3-inch sieve is larger than No. 200 sieve size.)	Gravel. (More than half of the coarse fraction is larger than No. 4 sieve size about 1/4 inch.)	GW	Well graded gravels, gravel-sand mixtures, little or no fines.*	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4. $C_z = \frac{(D_{30})^2}{D_{10} D_{60}}$ between 1 and 3.	Wide range in grain size and substantial amounts of all intermediate particle size.
	Clean gravels. (Less than 5% of material smaller than No. 200 sieve size.)				
Coarse-grained soils. (More than half of the coarse fraction is larger than No. 4 sieve size about 1/4 inch.)	Gravel. (More than half of the coarse fraction is larger than No. 4 sieve size about 1/4 inch.)	GP	Poorly graded gravels, gravel-sand mixtures, little or no fines.*	Not meeting both criteria for GW.	Predominantly one size (uniformly graded) or a range of sizes with some intermediate sizes missing (gap graded).
	Gravels with fines. (More than 12% of material smaller than No. 200 sieve size.)*	GM	Silty gravels, and gravel-sand-silt mixtures.	Atterberg limits below "A" line, or PI less than 4	Nonplastic fines or fines of low plasticity.
Coarse-grained soils. (More than half of the coarse fraction is larger than 3-inch sieve is larger than No. 200 sieve size.)	Gravel. (More than half of the coarse fraction is larger than No. 4 sieve size about 1/4 inch.)	GC	Clayey gravels, and gravel-sand-clay mixture.	Atterberg limits above "A" line, and PI greater than 7.	Plastic fines.
	Gravels with fines. (More than 12% of material smaller than No. 200 sieve size.)*			Atterberg limits above "A" line with PI between 4 & 7 is borderline case GM-GC	

* Materials with 5 to 12 percent smaller than No. 200 sieve are borderline cases, designated: GW-GM, SW-SC, etc.

(Continued)

(Sheet 1 of 3)

Table 2. (Continued)

Primary Divisions for Field and Laboratory Identification			Group Symbol	Typical Names*	Laboratory Classification Criteria	Supplementary Criteria for Visual Identification
Coarse-grained soils. (More than half of material finer than 3-inch sieve is larger than No. 200 sieve size.)	Sands. (More than half of the coarse fraction is smaller than No. 4 sieve size.)	Clean sands. (Less than 5% of material smaller than No. 200 sieve size.)				
			SW	Well graded sands, gravelly sands, little or no fines*	$C_u = \frac{D_{60}}{D_{10}}$ greater than 6. $C_c = \frac{(D_{30})^2}{D_{10}D_{60}}$ between 1 and 3.	Wide range in grain sizes and substantial amounts of all intermediate particle sizes.
			SP	Poorly graded sands, and gravelly sands, little or no fines*	Not meeting both criteria for SW.	Predominantly one size (uniformly graded) or a range of sizes with some intermediate sizes missing (gap graded).
			SM	Silty sands, sand-silt mixtures.	Atterberg limits below "A" line, or PI less than 4	Atterberg limits above "A" line with PI between 4 & 7 is borderline case SM-SC.
			SC	Clayey sands, sand-clay mixtures.	Atterberg limits above "A" line with PI greater than 7.	Nonplastic fines or fines of low plasticity. Plastic fines.

* Materials with 5 to 12 percent smaller than No. 200 sieve are borderline cases, designated: GW-GM, SW-SC, etc.

(Continued)

(Sheet 2 of 3)

Table 2. (Concluded)

Primary Divisions for Field and Laboratory Identification		Group Symbol	Typical Names	Laboratory Classification Criteria		Supplementary Criteria for Visual Identification		
Fine grained soils. (More than half of material is smaller than No. 200 sieve size.) (Visual: half of particles are so fine that they cannot be seen by naked eye.)	Silts and clays. (Liquid limit less than 50.)	ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands.	Atterberg limits below "A" line, or PI less than 4.	Atterberg limits above "A" line with PI between 4 and 7 is borderline case ML-CL	Dry Strength	Reaction to Shaking	Toughness Near Plastic Limit
	Silts and clays. (Liquid limit less than 50.)	CL	Inorganic clays of low to medium plasticity; gravelly clays, silty clays, sandy clays, lean clays.	Atterberg limits above "A" line, with PI greater than 7.		None to slight	Quick to slow	None
	Silts and clays. (Liquid limit less than 50.)	OL	Organic silts and organic silt-clays of low plasticity.	Atterberg limits below "A" line		Medium to high	None to very slow	Medium
Fine grained soils. (More than half of material is smaller than No. 200 sieve size.) (Visual: half of particles are so fine that they cannot be seen by naked eye.)	Silts and clays. (Liquid limit greater than 50.)	MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts.	Atterberg limits below "A" line.		Slight to medium	Slow	Slight
	Silts and clays. (Liquid limit greater than 50.)	CH	Inorganic clay of high plasticity, fat clays.	Atterberg limits above "A" line.		Slight to medium	Slow to none	Slight to medium
	Silts and clays. (Liquid limit greater than 50.)	OH	Organic clays of medium to high plasticity.	Atterberg limits below "A" line		High to very high	None	High
Fine grained soils. (More than half of material is smaller than No. 200 sieve size.) (Visual: half of particles are so fine that they cannot be seen by naked eye.)	Highly organic soils	PT	Peat, muck and other highly organic soils.	High ignition loss, LL and PI decrease after drying.		Medium to high	None to very slow	Slight to medium
						Organic color and odor, spongy feel, frequently fibrous texture.		

Table 3
Typical Range of G. (Bowles 1986)

Type of Soil	Specific Gravity, G.
Sand	2.65-2.67
Silty sand	2.67-2.70
Inorganic clay	2.70-2.80
Soils with micas or iron	2.75-3.00
Organic soils	Variable, but may be under 2.00.

- f. Use Appendix A as a guide to conduct laboratory tests to determine the minimum ($\gamma_{d_{min}}$) and maximum ($\gamma_{d_{max}}$) densities of the granular soil. Combine Equations 2 and 5 to estimate the relative density, D_r , of the in-situ soil. Estimate the reasonableness of the laboratory-obtained $\gamma_{d_{min}}$ and $\gamma_{d_{max}}$, using the general guide given in Table 4. Note that values in Table 4 do not stem from any standard test procedure but rather signify range of feasible values in the literal sense of minimum and maximum properties. Subsequently, for some soils it contains values for which no standard test is available and the term relative density has no known meaning. However, these soils are not relevant to the scope of this report. Utilizing the relations $e = G_s \gamma_w / \gamma_d - 1$, calculate e_{max} (corresponding to $\gamma_d = \gamma_{d_{min}}$) and e_{min} (corresponding to $\gamma_d = \gamma_{d_{max}}$), and check against typical values given in Table 4.
- g. Utilize Figure 2 to estimate the coefficient of permeability, k , that corresponds to $e = e_{in-situ}$. Rate the permeability using the ranges shown in the figure: High, Medium, or Low Permeability.
- h. Combine the soil classification (step c) and relative density (step f) to locate a point in Figure 3. This point defines the effective angle of internal friction ϕ' . It also defines (γ_d , e , n)_{in-situ}; however, these are not needed since their value has already been determined directly. Only ϕ' should be read off the chart.
- i. Use the rating obtained in step g and the estimated angle of friction ϕ' (step h) to determine the dredging descriptor defined and presented in Table 5. Note in this table that the descriptor is a function of both effective angle of friction and the ability of the soil to dissipate excess pore pressure (i.e., its permeability). That is, it is a function of the effective shear strength associated with the dredge cutting activity.

Table 4

Typical Values of Soil Index Properties (NAVFAC 1982)

Particle Size and Gradation				Voids				Unit Weight (lb/cu ft)				
	Approximate Size Range (mm)		Approximate D_{10} (mm)	Approximate Range Uniform Coefficient C_u	Void Ratio		Porosity %		Dry Weight γ_d		Submerged Weight $\gamma' = \gamma_{sat} - \gamma_w$	
	D_{max}	D_{min}			e_{max}	e_{min}	n_{max}	n_{min}	γ_{dmin}	γ_{dmax}	γ'_{min}	γ'_{max}
GRANULAR MATERIALS Uniform Materials a. Equal spheres (theoretical values) b. Standard Ottawa SAND c. Clean, uniform SAND (fine or medium) d. Uniform, inorganic SILT Well-graded Materials a. Silty Sand b. Clean, fine to coarse SAND c. Micaceous SAND d. SILTY SAND & GRAVEL	-	-	-	1.0 1.1	0.92 0.80	0.35 0.06	47.6 44	26 33	- 92	- 110	- 57	- 69
	0.84	0.59	0.67									
	-	-	-	1.2 to 2.0 1.2 to 2.0	1.0 1.1	0.40 0.40	50 52	29 29	83 80	118 118	52 51	73 73
	0.06	0.006	0.012									
MIXED SOILS Sandy or Silty CLAY Slip-graded SILTY CLAY with stone or rock fragments Well-graded GRAVEL, SAND, SILT & CLAY mixture	2.0	0.001	0.003	10 to 30	1.8	0.25	64	20	60	136	38	86
	250	0.001	-	-	1.0	0.20	50	17	84	140	53	89
	250	0.001	0.002	25 to 1000	0.70	0.13	41	11	100	148	62	94

(Continued)

Table 4. (Concluded)

Particle Size and Gradation				Voids				Unit Weight (lb/cu ft)				
	Approximate Size Range (mm)		Approximate D ₁₀ (mm)	Approximate Range Uniform Coefficient C _u	Void Ratio		Porosity %		Dry Weight γ _d		Submerged Weight γ' = γ _{sat} - γ _w	
	D _{max}	D _{min}			e _{max}	e _{min}	n _{max}	n _{min}	γ _{dmin}	γ _{dmax}	γ' _{min}	γ' _{max}
					loose	dense	loose	dense	loose	dense	loose	dense
CLAY SOILS CLAY (30%-50% clay sizes) Colloidal CLAY (-0.002 mm: 50%)	0.06	0.5μ	0.001	-	2.4	0.50	71	33	50	112	31	71
	0.01	10μ	-	-	12	0.60	92	37	13	106	8	66
ORGANIC SOILS Organic SILT Organic CLAY (30% - 50% clay sizes)	-	-	-	-	3.0 4.4	0.55 0.70	75 81	36 41	40 30	110 100	26 16	69 62

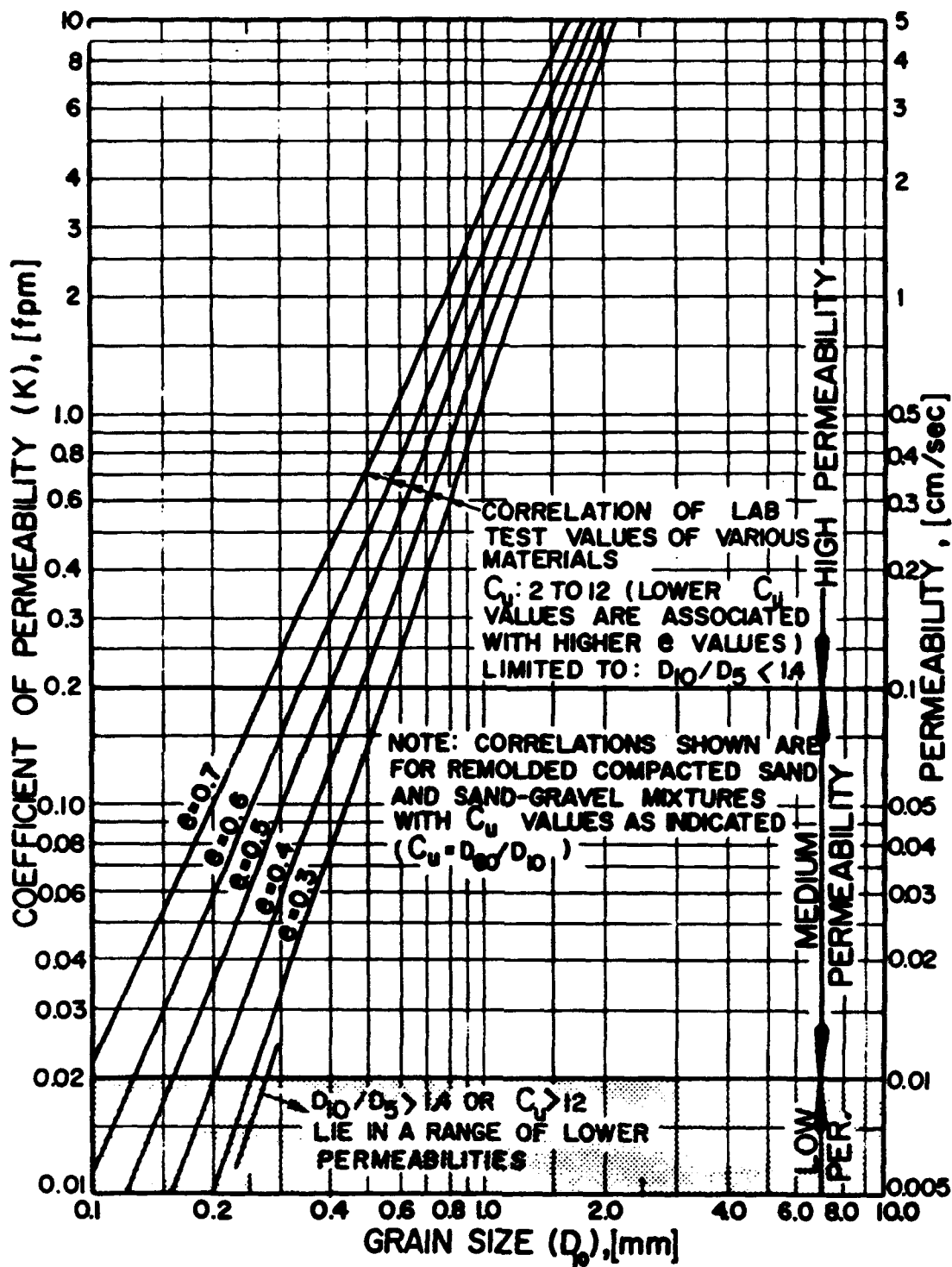


Figure 2. Permeability of sands and sand-gravel mixtures (modified after NAVFAC 1982)

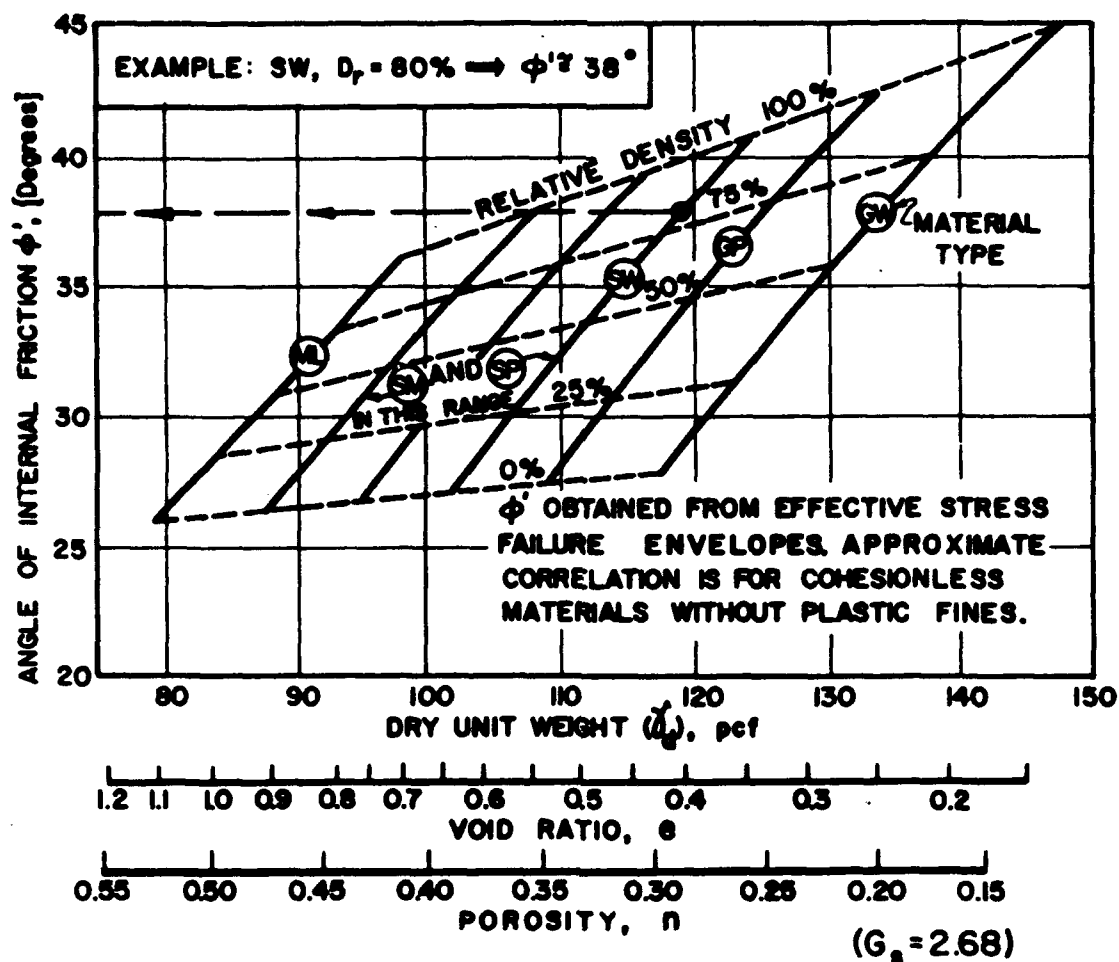


Figure 3. Correlations of strength characteristics for granular soils (NAVFAC 1982)

It should be pointed out that Table 5 is limited to normal rates of dredge cutting. Slowing down the process may ease the dredging for the low and medium permeability types of soil. The descriptor can be used as a predictor to assess the difficulties associated with a dredge cutting operation at a particular site. As presented, its reliability as a predictor depends on the extent of the site investigation. Table 5 categorizes soils, relative to each other, based on trends related to effective shear strength and it provides a qualitative descriptor using a logical scale. Because of insufficient information related to dredging, however, it is not clear at present how sensitive the chosen scale is when applied to dredge cutting. Subsequently, only five categories were selected. Experience should allow refinement of the descriptor's scale.

Table 5
Descriptors Associated with Dredge Cutting Difficulty

Angle of Internal Friction ϕ'	<u>Dredging Difficulty Rating*</u>			<u>Condition</u>
	<u>Permeability</u>			
	<u>High</u>	<u>Medium</u>	<u>Low</u>	
Less than 25°	1	1	1-2	Very Loose
25° - 30°	2	2	2-3	Loose
30° - 35°	3	2-3	3-4	Medium
35° - 39°	4	3-4	4-5	Dense
Greater than 39°	5	5	5	Very Dense

* Descriptors Equivalent to Dredging Rating:

- 1 - Very Easy
- 2 - Easy
- 3 - Normal
- 4 - Difficult
- 5 - Very Difficult

Supplemental Information

9. Often, site investigation includes the Standard Penetration Test (SPT) or the Cone Penetrometer Test (CPT). The results of these tests are indirect but are useful in substantiating the descriptor developed in paragraph 8. They are considered, therefore, supplemental and should be used only when the site mainly consists of sandy soil.

Standard Penetration Test (SPT)

10. The resistance to the penetration of a standard sampler in borings is measured in this test (ASTM D 1586). The method is rapid, and when tests are properly conducted, they yield useful data. Test results, however, are affected by operational procedures, by the presence of gravel, or cementation. The effective angle of internal friction or density is not measured directly. Once the number of blows, N , at a certain depth has been determined, follow these steps (see also Figure 4):

- a. Use Figure 5 to estimate the relative density, D_r , of sands. In this figure, use the counted typical value N at its appropriate depth. Note that the correlation given in Figure 5 is oriented toward large SPT depths which may not be relevant for dredging. For shallow depths there are no reliable correlations available, mainly because of the likely soil disturbance near the surface. Note the right-hand side ordinate in

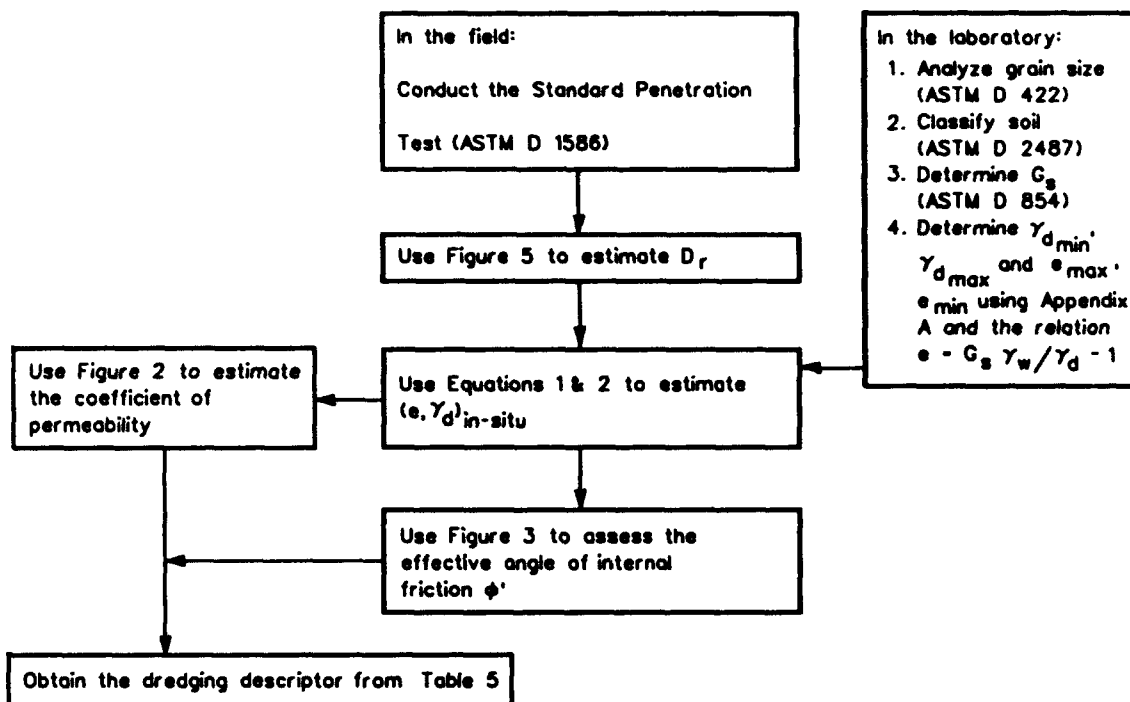


Figure 4. Flow chart to obtain the dredging descriptors using the SPT

Figure 5. Though it did not appear in the original reference, the depth ordinate has been added here for convenience. This ordinate is valid for the following assumptions: (1) water table is at or above ground surface, and (2) total unit weight of soil is constant with relevant depth and its average value is about 120-122 pcf.

- b. Repeat steps **b**, **c**, **d**, **f**, and **h** in paragraph 8 to fully define the soil. Obtain the angle of internal friction, ϕ' .
- c. Use Equation 1 together with results obtained from steps **a** and **b** to assess in-situ void ratio, i.e., $e_{in-situ}$. Use this void ratio to estimate the coefficient of permeability, k , from Figure 2.
- d. Utilize ϕ' (step **b** and the permeability rating (step **c**) to determine the dredging descriptor presented in Table 5. Verify against the descriptor obtained in paragraph 8.

Cone Penetrometer Test (CPT)

11. In this test (ASTM D 3441), a cone-shaped tip is jacked from the ground surface to provide a continuous resistance record. The speed of

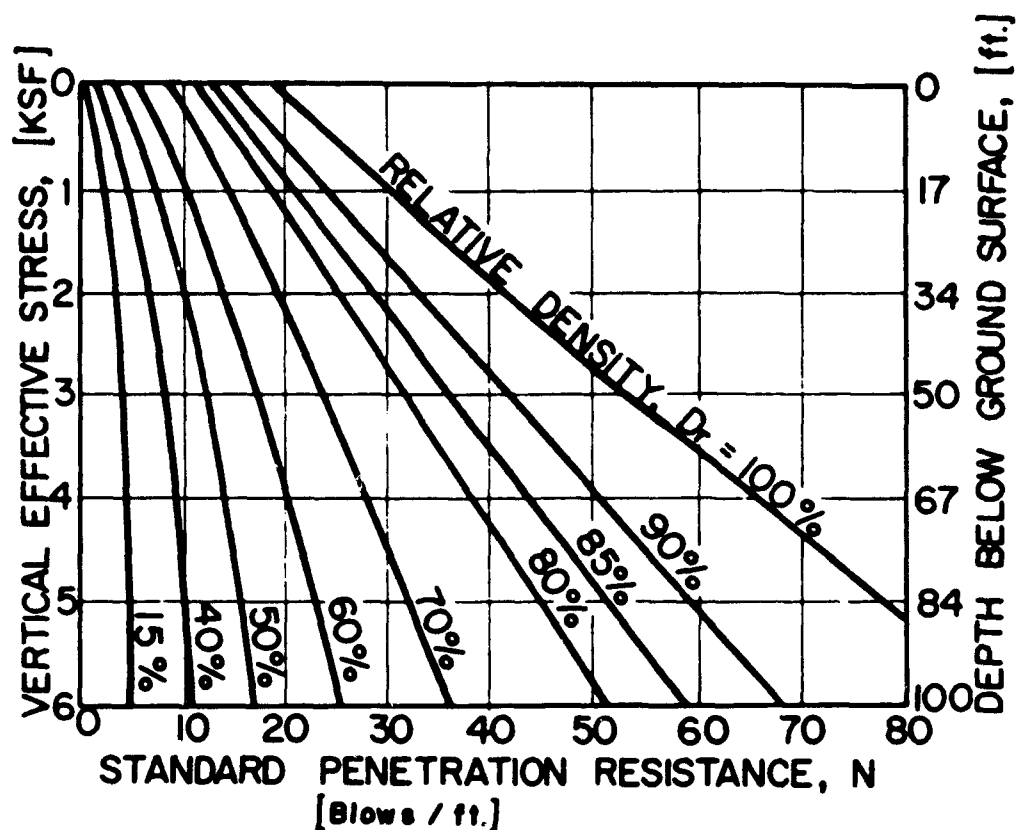


Figure 5. Correlations between relative density and standard penetration resistance in accordance with Gibbs and Holtz (NAVFAC 1982)

operation allows considerable data to be obtained in a short period of time. Major drawbacks are the nonrecoverability of samples for identification, difficulty in advancing the cone in dense or hard deposits, and need for stable and fairly strong working surface to jack the rig against. Once the cone tip resistance, q_a , at a certain depth has been determined, follow these steps (see also Figure 6):

- a. Use Figure 7 to estimate the relative density, D_r , of sands. In this figure, use the measured typical q_a and its corresponding depth. As is the case in step a in paragraph 10 also here the correlations are oriented toward large CPT depths. Note that similar to the modification in step a, paragraph 10, a depth ordinate was added to Figure 7 for convenience. Same assumptions were used in establishing this ordinate.
- b. Same as step b, paragraph 10.
- c. Same as step c, paragraph 10.
- d. Same as step d, paragraph 10.

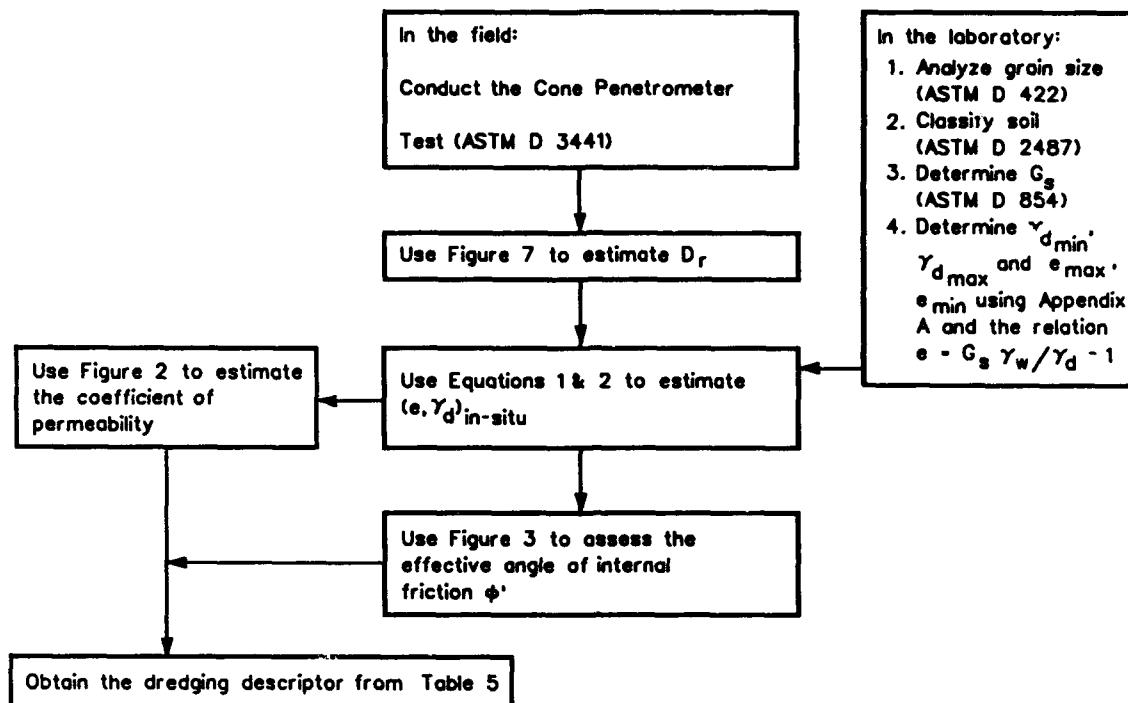


Figure 6. Flow chart to obtain the dredging descriptors using the CPT

12. It should be pointed out that if the cone is equipped with a pore-water pressure transducer, direct information can then be collected. If negative pore pressure develops during shear, an apparent increase in shear strength should be expected during dredge cutting--see discussion in paragraph 6. The soil then will be classified as having low (or medium) permeability, resulting with a descriptor predicting difficulty in dredging. It should be pointed out, however, that interpretation of measured data regarding pore water measurement may be difficult. Parameters affecting the measured values include the location of the porous element on the penetrometer, system compliance, as well as the density and permeability of the soil (see ASTM D3441 for more details).

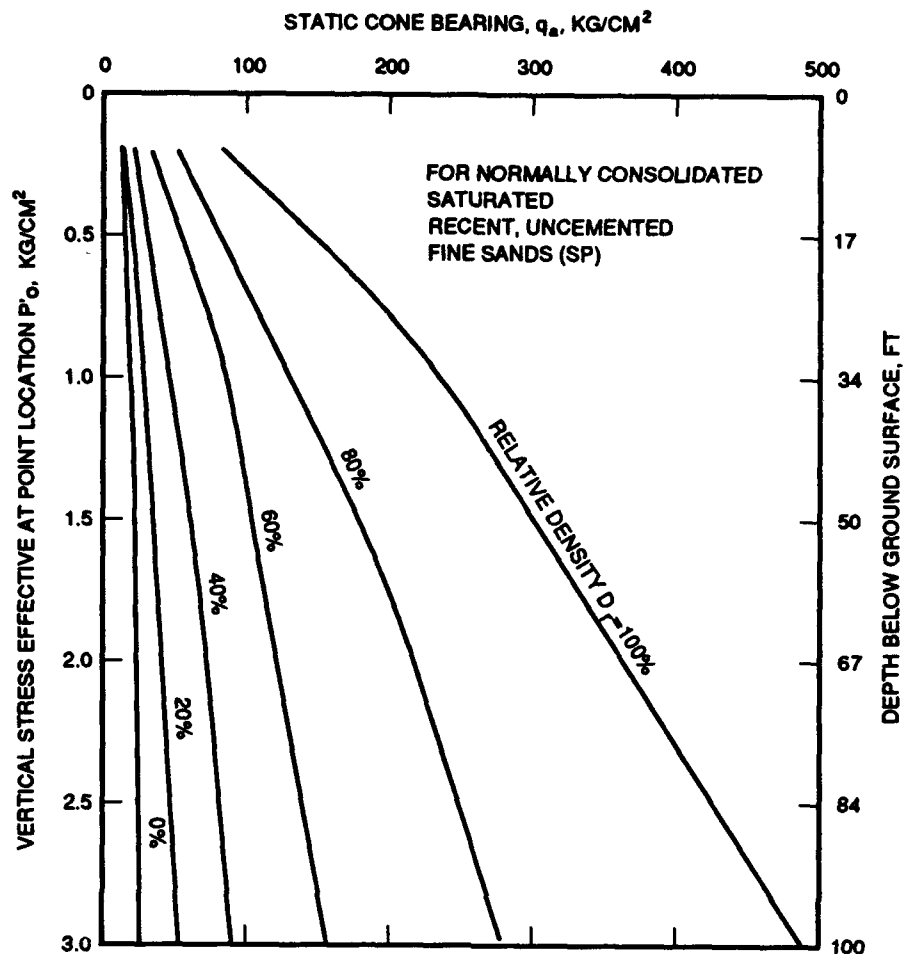


Figure 7. Approximate relationship between q_a and D_r (after Schmertmann 1978)

Conclusions and Recommendations

13. Descriptors characterizing the dredgeability of granular soils have been developed. They are related to the effective shear strength of granular soils. This strength is a result of both the effective angle of friction and, indirectly, the coefficient of permeability of the soil. The permeability is used as a measure indicating the ability of the soil to dissipate excessive pore-water pressures developing during dredge cutting. Consequently, it affects the shear strength of the soil when rapid shear (i.e., dredge cutting) is applied and thus influences the dredgeability.

14. A step-by-step procedure to determine the descriptors is presented and recommended. It includes field tests to estimate the in-situ density and water content, as well as simple laboratory tests to identify the soil and its

minimum/maximum densities. As a result, the relative density of the soil, including gravel, sand, and silt, can be estimated. By modifying existing correlations commonly used in foundation engineering, the shear strength and subsequently the descriptor for dredgeability were established. To verify the value of this descriptor in sand soils, it is recommended to conduct either the Standard Penetration Test (SPT) or the Cone Penetrometer Test (CPT). Since these two tests are less direct in defining the descriptor as compared to the recommended main approach (which is based on field measurement of density), either one of these tests is considered to provide supplemental information only. Either the SPT or CPT should be used only if the site consists of sand. The descriptors' characterization of a site depends mainly on the extent of site investigation.

15. The descriptors have been developed based on fundamental concepts in soil mechanics. However, they contain a conversion which is based on judgement; i.e., physical properties of granular soils are converted into a qualitative scale of anticipated difficulty associated with dredge cutting. It should be pointed out that there is insufficient relevant experience in the dredging discipline to verify the accuracy of the scale chosen for the descriptor. Therefore, it is recommended that the descriptors be used as a basis for future adjustment and refinement in conjunction with dredge cutting operations considering the suggested procedure for its determination. Special attention should be given then to silty soils.

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- D 854: Standard Test Method for Specific Gravity of Soils.
- D 1557: Standard Test Method for Moisture-Density Relations of Soils and Soil-Aggregate Mixture Using 10 lb. Rammer and 18 in Drop.
- D 1586: Standard Method for Penetration Test and Split-Barrel Sampling of Soils.
- D 2487: Standard Test Method for Classification of Soils for Engineering Purposes.
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- D 3017: Standard Test Method for Water Content of Soil and Rock in Place by Nuclear Methods (Shallow Depth).
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Appendix A: Minimum and Maximum Density Tests for Granular Soils

Introduction

1. Review of the existing test standards to determine the minimum and maximum densities of granular soils reveals that their applicability is limited to soils containing less than 15 percent of particles passing a No. 200 sieve. Consequently, for soils containing less than 15 percent fines, it is recommended to use ASTM D 4254 for minimum density and D 4253 for maximum density. Both tests are conducted on dry sand or gravel and are well established. However, in dredging one encounters silty soils quite often.

2. Silty sands and silts cause difficulties in the maximum density test due to "boiling out" of the finest particles. This results in a density lower than the true maximum density being measured; in some cases measured "maximum" densities less than the dry densities corresponding to optimum moisture contents in a standard compaction test have been reported (Head 1980). Furthermore, the conventional tests on dry silts often do not result in the lowest possible density due to tendency of fine grains to "lock" into denser structure. Consequently, the following two sections describe "wet" procedures to determine the minimum and maximum densities. The minimum density test utilizes a modified deposition-in-water procedure which slows the rate of particles deposition as well as simulating the sedimentation process occurring in the field. The maximum density test gives density which is very close to the one possible to achieve without particles breakage and it can be relied upon to give repeatable results at a high density.

Minimum Density Test: Silty Soils

3. The procedure is based on Kolbuszewski's (1948) extensive research work on minimum density test in water. Essentially, it follows the step-by-step procedure described by Head (1980) with modifications to include vacuum to reduce air entrapment. This modification further simplifies an already simple test. It is applicable to silty soil.

Apparatus

4. The apparatus consists essentially of a 2000-ml clear glass graduated cylinder, approximately 80 mm in diameter, arranged as shown in Figure A1. It also includes a source of vacuum (at least 20-in. Hg) and a balance with an accuracy of at least 0.1 g. It should be pointed out that the stopper in Figure A1 must provide an airtight seal. Subsequently, if the top of the commercially available glass cylinder contains a spout, simple glass-work may be needed to remove it.

Procedure

5. The test procedure is as follows.

- a. Pour about 1000 ml of water into the glass cylinder.
- b. Weigh out 1000 g of oven-dry soil.
- c. Place the dry soil in the cylinder using a funnel. Ensure that no soil particles adhere to the funnel by flushing small amount of water from a squeeze bottle through the funnel.
- d. Add water to the cylinder up to about 2000 ml.
- e. Let the submerged soil absorb water in the glass cylinder for about one hour.
- f. Attach stopper to glass cylinder and apply vacuum of 20-in. Hg (Figure A2) for at least 5 minutes and until visible movement of air bubbles entrapped in the soil-water mixture ceases.
- g. Disconnect the vacuum source by turning off the valve attached to the stopper. Tip the glass cylinder upside down, allow the soil to sink all the way to the stopper, and then quickly tilt it back to its original vertical position.
- h. Some of the soil particles may stick to the glass sides in the unsubmerged portion of the cylinder as well as to the bottom of the stopper (Figure A3). After releasing the vacuum, carefully remove the stopper, and gently flush the soil particles off it and off the unsubmerged glass sides using a squeeze bottle (Figure A4). Make sure that it is flushed into the glass cylinder.
- i. Let the suspended soil settle (Figure A5) so that a clear surface can be seen (Figure A6). This process may take an hour or more depending on the amount and size of fine particles. The level of the top of the soil may be taken as the loose volume reading, because the effect of the very fine silt particles, which are still suspended, in unsegregated loose soil is to hold the larger grains apart as well as to occupy the voids between them (Head 1980). However, to reflect the sedimentary process which typically forms soils in dredging operations and where segregation occurs, let the particles settling continue for 24 hours (Figure A7). At that time record the volume of the silty soil, V , in cubic centimeters.

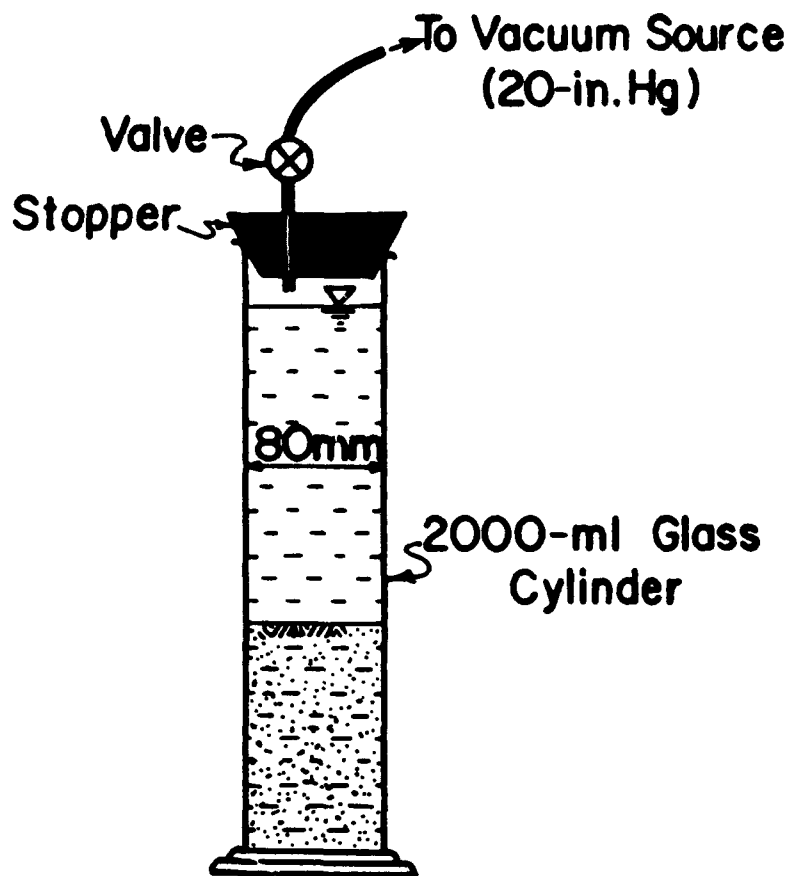


Figure A1. Setup for minimum density test on sand in water

- j. Calculate the minimum density, $\gamma_{d_{\min}}$, as

$$\gamma_{d_{\min}} = \frac{1000}{V} \text{ [gr/cc]}$$

Report the result to the nearest 0.001 gr/cc, and repeat the procedure as in in-water method.

- k. Using the same soil in the glass cylinder, return to step f, and repeat the procedure. The results of three such tests should fall within ± 0.5 percent of their average $\gamma_{d_{\min}}$.

Verification

6. To verify the consistency of the procedure, tests were conducted on a reddish brown silty sand (SM) supplied by WES. The typical gradation for this soil is illustrated in Figure A8 (50 percent passing sieve No. 200;

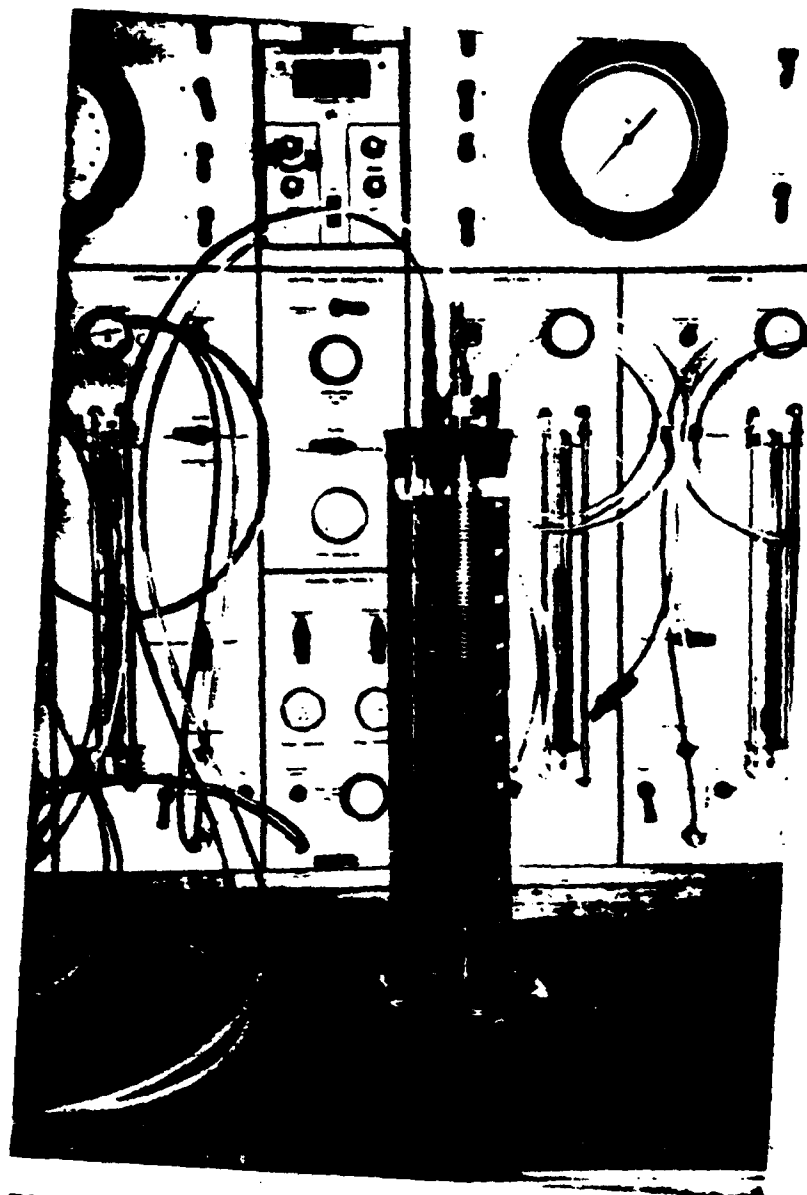


Figure A2. Vacuum applied to soil-water mixture

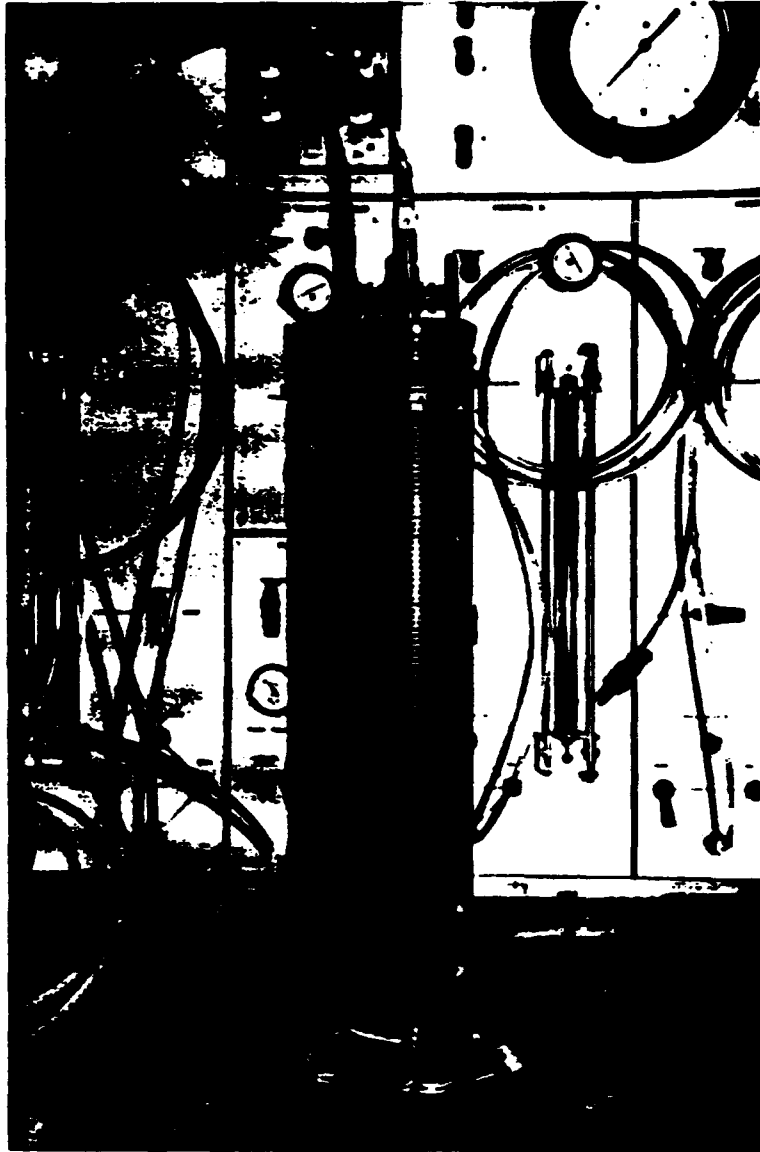


Figure A3. Mixed silty sand with deaired water

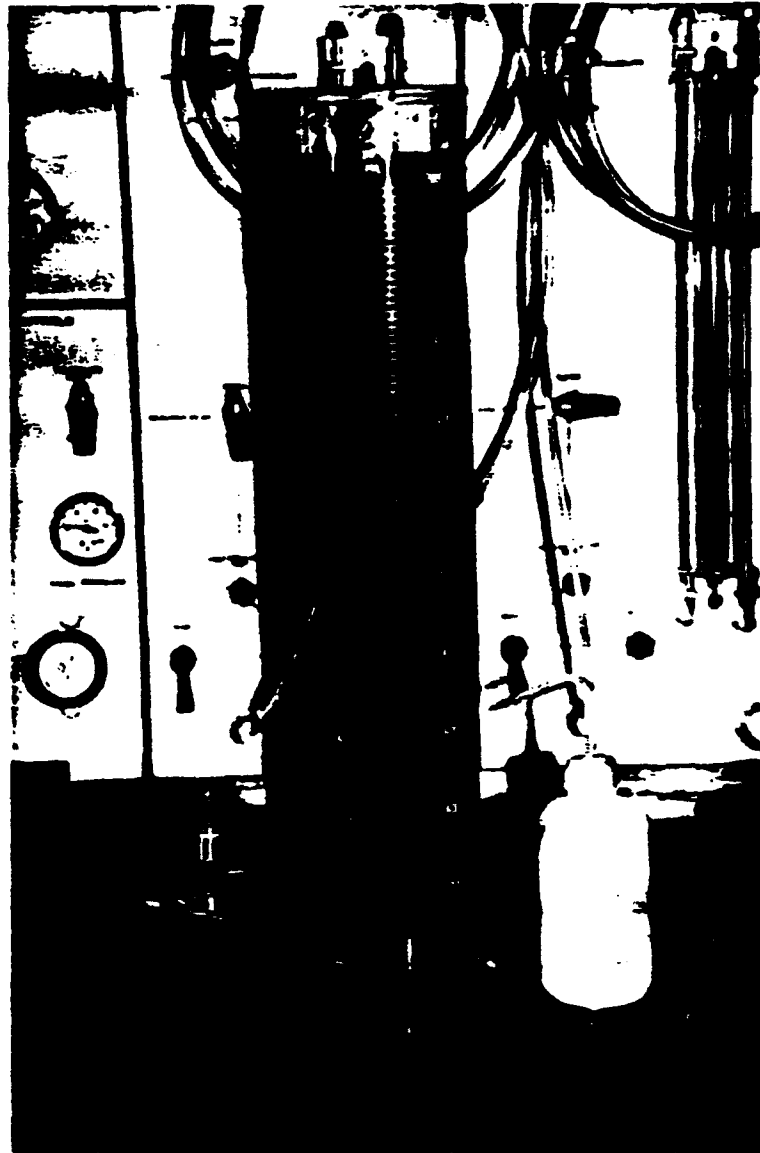


Figure A4. Soil suspension shortly after stopper removal

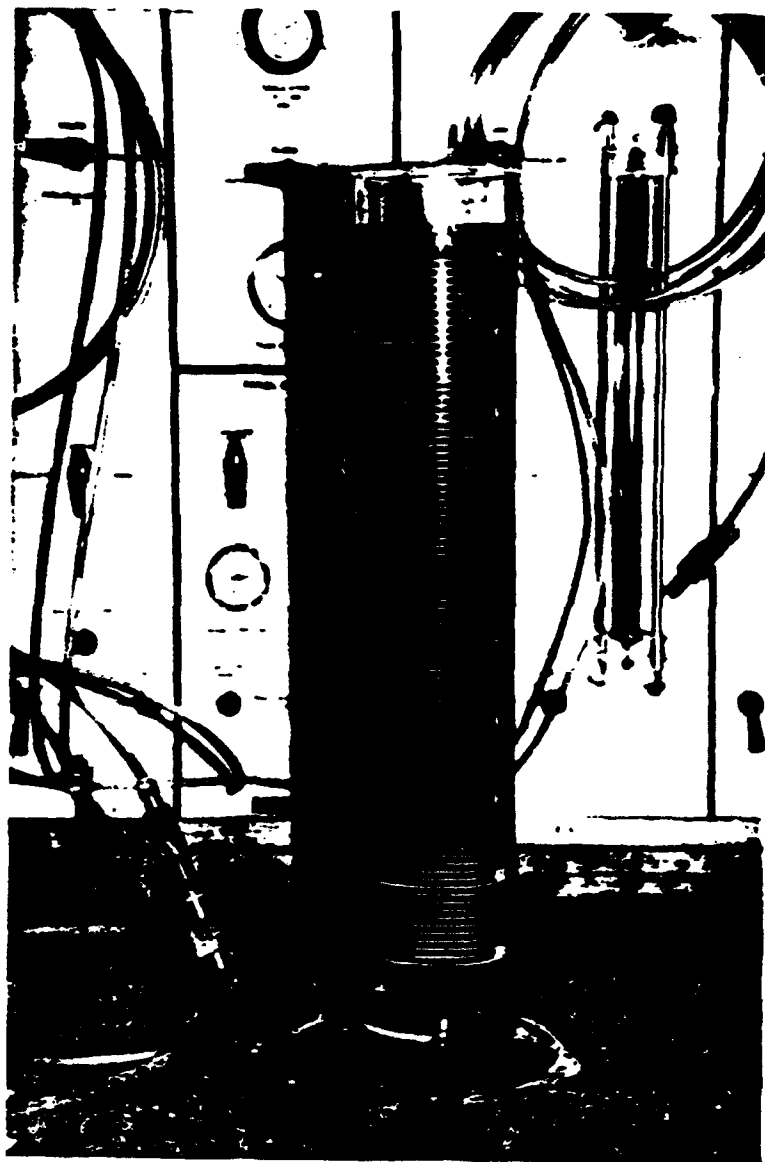


Figure A5. Emergence of soil surface

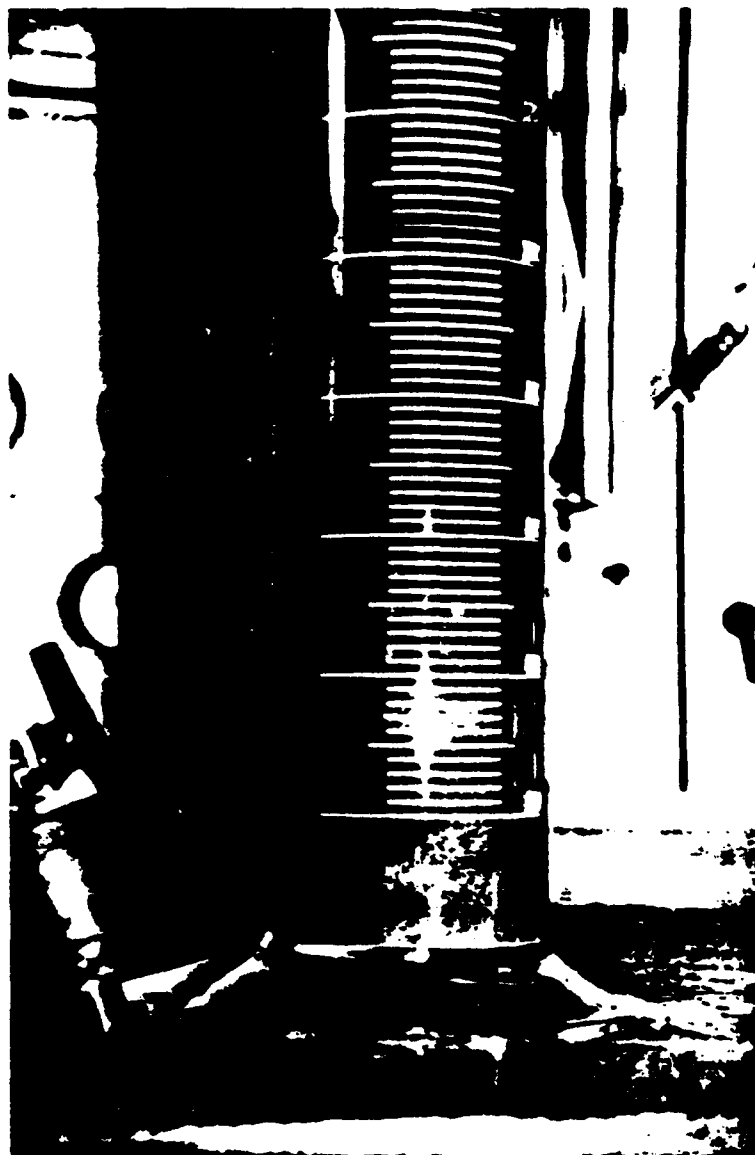


Figure A6. Close-up of soil surface

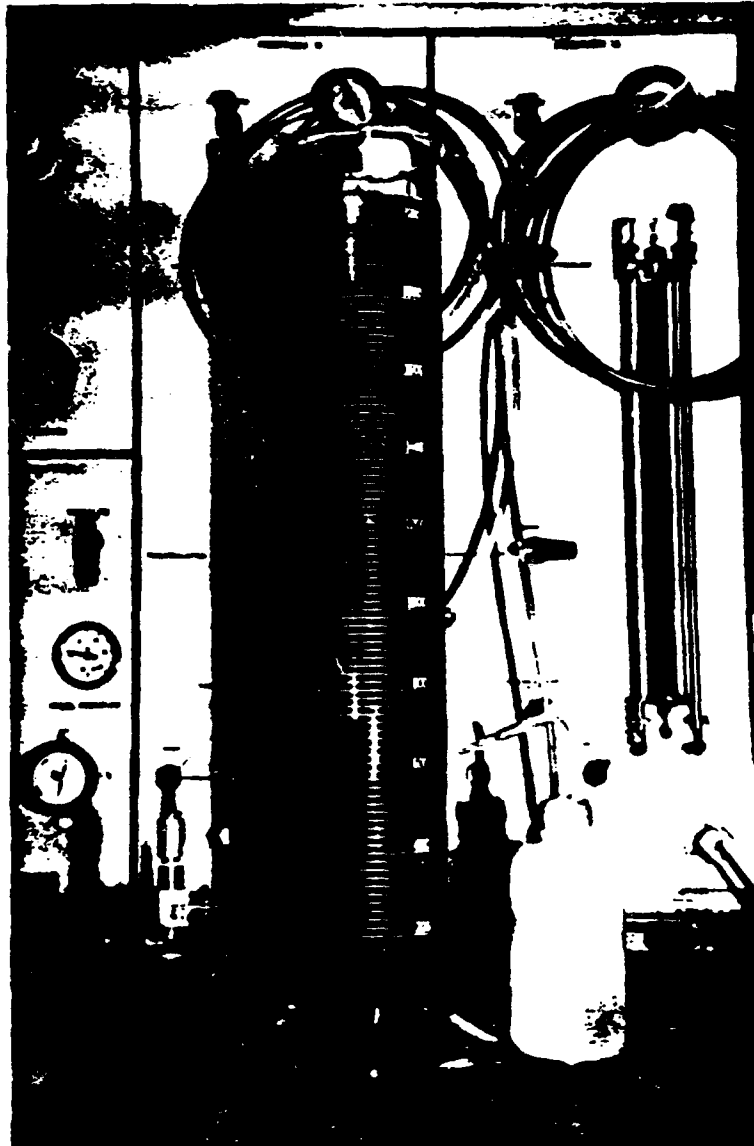


Figure A7. Completion of sedimentary process

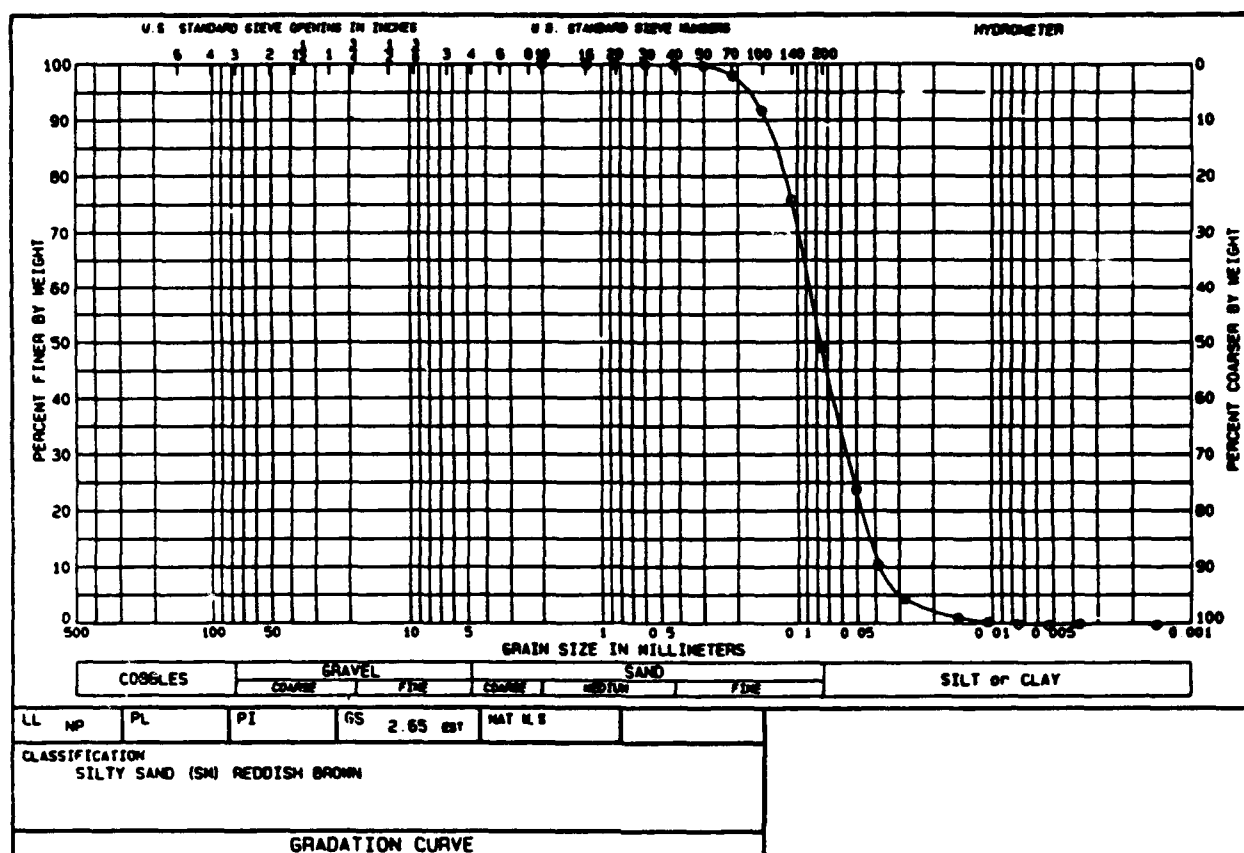


Figure A8. Gradation curve for tested soil

nonplastic fines). Three tests, each using soil weighing 1000 g, yielded in-water volumes of $V = 725$ cc, 720 cc, and 720 cc. It is interesting to note that one hour after the beginning of each test, the volumes defined by a clearly visible soil surface were $V = 720$ cc, 715 cc, and 715 cc, respectively. Consequently, the additional volume produced by the accumulation of fine grains sediments over an extra 23 hours was about 5 cc (i.e., about 0.7 percent). Calculating the minimum dry density in-water one gets $\gamma_{d_{\min}} = 1.385 \pm 0.3$ percent gr/cc. It should be pointed out that at the end of the settling process the soil appeared to be of uniform consistency, without any visible irregular voids or archings at its interface with the cylinder glass.

Maximum Density Test: Silty Soils

7. The suggested method is essentially a modification of Head's (1980) procedure for silty sands. For this type of soil the procedure can generally be relied upon to give a repeatable result at a high density.

Apparatus

8. The apparatus is the same as that specified in ASTM D 698 and D 1557 (or Standard Compaction Test and Modified Compaction Test, Appendix VI in Dept. of the Army, Office of the Chief of Engineers, EM 1110-2-1906).

Procedure

9. The procedure is as follows:

- a. Conduct two sets of compaction tests following ASTM D 698 (termed Standard Compaction Test) and ASTM D 1557 (termed Modified Compaction Test). In principle, the curves of Dry Density versus Moisture Content shown in Figure A9 will be obtained.
- b. Determine the maximum dry density at the optimal water content for the standard and modified compaction test.
- c. If $\left\{ \frac{[\gamma_{d_{\max}}(\text{modified}) - \gamma_{d_{\max}}(\text{standard})]}{\gamma_{d_{\max}}(\text{modified})} \right\} \leq 0.05$, report $\gamma_{d_{\max}}$ based on the modified compaction test to the nearest 0.001 gr/cc, and the procedure as ASTM D 1557 method.
- d. In case the ratio stated in step c exceeds 5 percent (i.e., maximum density is sensitive to compactive effort), repeat the D 1557 test method, but apply this time 80 blows per layer rather than 25. This modified test is termed Heavy Compaction Test (Figure A9), and $\gamma_{d_{\max}}$ based on it should be reported to the nearest 0.001 gr/cc, and the procedure as Modified ASTM D 1557 - Heavy Compaction.

Verification

10. Tests were conducted on the same type of silty sand used in the Minimum Density Test. The standard and modified compaction test results are presented in Figure A10. Results obtained for the standard compaction test were $\gamma_{d_{\max}} = 1.590$ gr/cc at $w = 14.6$ percent and for the modified test were $\gamma_{d_{\max}} = 1.670$ gr/cc at $w = 14.6$ percent. Since the difference in maximum densities for both tests is less than 5 percent, there is no need for a heavy compaction test, and the reported result is $\gamma_{d_{\max}} = 1.670$ gr/cc based on ASTM D 1557 procedure.

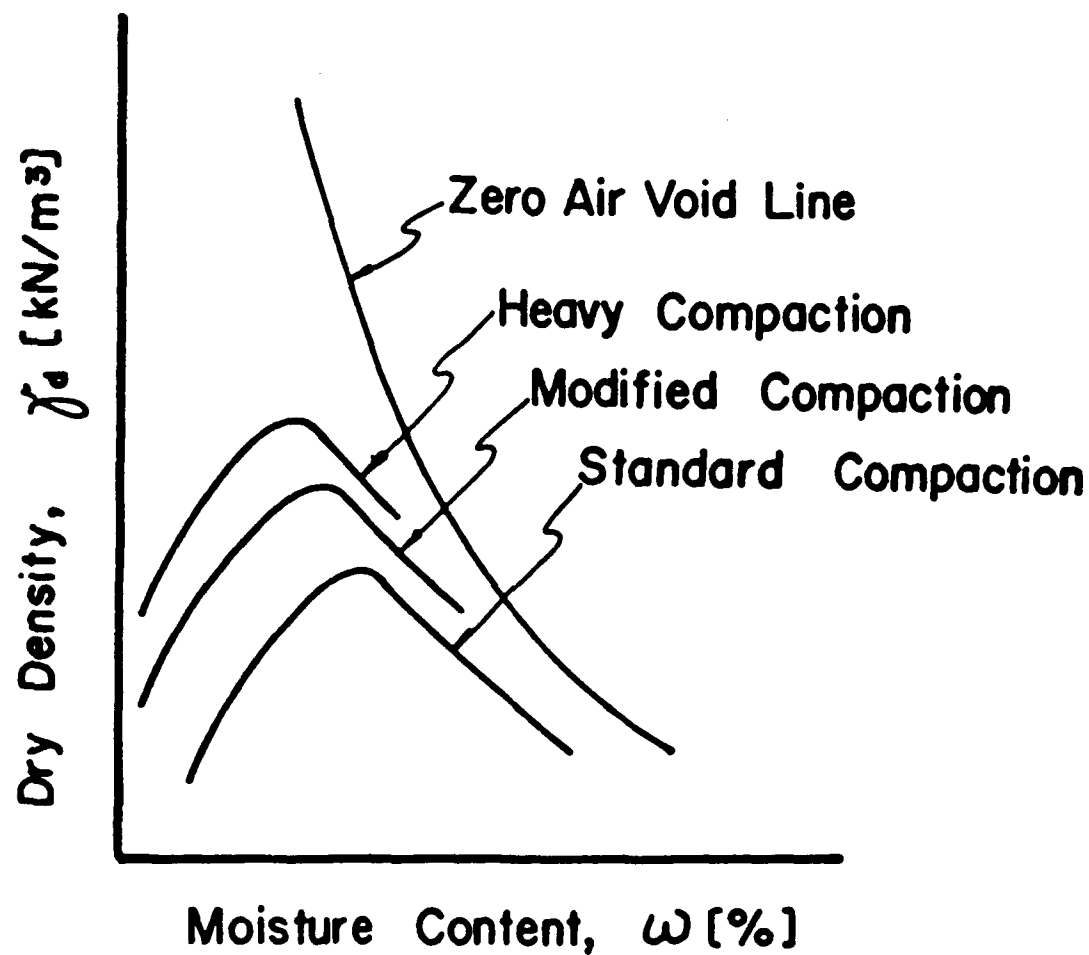


Figure A9. Typical dry density versus moisture content curves as a function of compaction energy

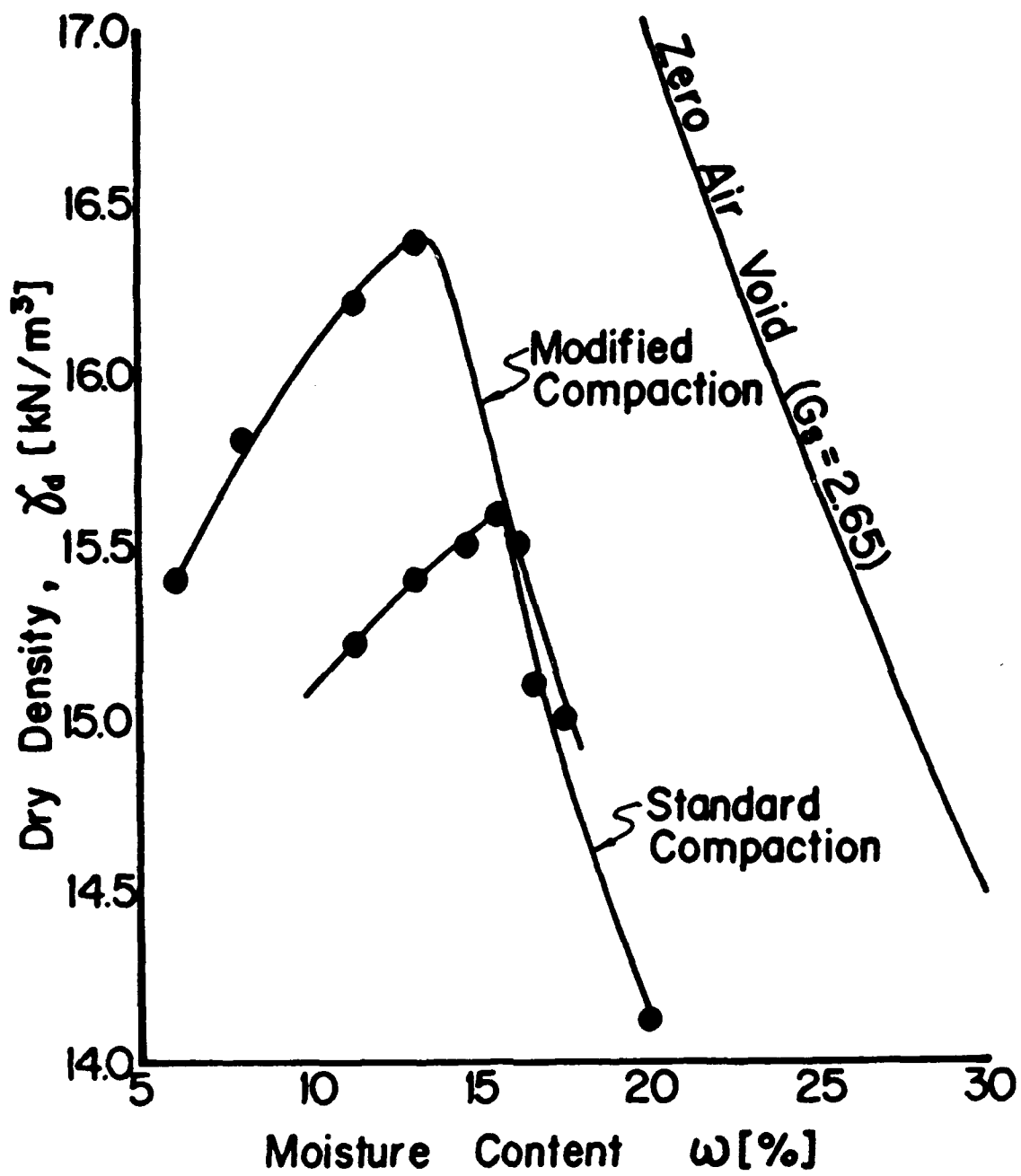


Figure A10. Compaction test results for silty sand

Appendix B: Minimum Density and Volume of Dredged Soil

1. The minimum density, obtained in Appendix A, can be used to estimate the maximum volume the dredge soil will occupy in the fill containment area. If the volume of the soil to be dredged is $V_{in-situ}$ and its average dry density is $\gamma_{d_{in-situ}}$, then the maximum volume of the fill is limited to

$$V_{fill} = V_{in-situ} \times \frac{\gamma_{d_{in-situ}}}{\gamma_{d_{min}}} \quad (B1)$$

Note that in Equation B1 no compaction of the dredged soil, placed in the fill area, is assumed (i.e., $\gamma_{d_{min}}$ is used). This assumption is approximately valid immediately after transporting and dumping the soil. However, due to consolidation occurring under self-weight, this volume will decrease as follows

$$V_{fill-final} = V_{fill} \times \frac{\gamma_{d_{min}}}{\gamma_{d_{fill-final}}} \quad (B2)$$

2. It is known from experience that the ratio $\gamma_{d_{min}} / \gamma_{d_{fill-final}}$ in Equation B2 can generally be 0.9 or less. Subsequently, the fill volume will decrease over time by 10 percent or more. This volume decrease is beneficial since it provides additional storage capacity. With fine soils, however, this decrease may occur over a period of years unless methods to facilitate drainage are being utilized.

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13. ABSTRACT (Maximum 200 words) Descriptors typifying the dredgeability of granular soils are presented. The descriptors are related to the effective shear strength of the soil. They account for the effective angle of internal friction and relative permeability; that is, the ability of the soil to dissipate excess pore-water pressures developing during dredge cutting. A step-by-step procedure to determine the descriptors is detailed. The procedure includes field tests such as the estimation of in-situ density and either the Standard Penetration Test or the Cone Penetrometer Test. It also includes simple laboratory procedures, some of which were modified to deal with silty soils. It is recommended to confirm the descriptor, which signifies the difficulty associated with the dredging of a particular site, using one of two possible supplemental approaches. The reliability of the descriptor largely depends on the extent of the site investigation and the subsequent characterization of the soils.				
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