Introduction to Laboratory Testing of Soils

Course No: T04-004

Credit: 4 PDH

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An Introduction to Laboratory Testing of Soils

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CONTENTS

1. INTRODUCTION

1.1 SCOPE. This course covers laboratory test procedures, typical test properties, and the application of test results to design and construction. Symbols and terms relating to tests and soil properties conform, generally, to definitions given in ASTM Standard D653, Standard Definitions of Terms and Symbols Relating to Soil and Rock Mechanics found in Reference 1, Annual Book of ASTM Standards, by the American Society for Testing and Materials.

1.2 LABORATORY EQUIPMENT. For lists of laboratory equipment for performance of tests, see Reference 2, Soil Testing for Engineers, by Lambe, Reference 3, The Measurement of Soil Properties in the Triaxial Test, by Bishop and Henkel, and other criteria sources.

1.3 TEST SELECTION FOR DESIGN. Standard (ASTM) or suggested test procedures, variations that may be appropriate, and type and size of sample are included in Tables 1, 2, 3, and 4. Table 5 lists soil properties determined from such tests, and outlines the application of such properties to design. ASTM procedures are found in Reference 1.

1.3.1 SAMPLE SELECTION. Samples to be tested should be representative, i.e. they should be similar in characteristics to most of the stratum from which they come, or be an average of the range of materials present. If this appears difficult because of variations in the stratum, it may be necessary to consider subdivisions of the stratum for sampling, testing, and design purposes. In general, tests on samples of mixed or stratified material, such as varved clay, should be avoided; usually such results are not indicative of material characteristics; and better data for analysis can be obtained by testing the different materials separately. Undisturbed samples for structural properties tests must be treated with care to avoid disturbance; an "undisturbed" sample found to be disturbed before testing normally should not be tested. Fine-grained cohesive samples naturally moist in the ground should not be allowed to dry before testing, as irreversible changes can occur; organic soils are particularly sensitive. Soils with chemical salts in the pore water may change if water is added, diluting the salt concentration, or if water is removed, concentrating or precipitating the salt. Organic soils require long-term low temperature (60deg.C) drying to avoid severe oxidation (burning) of the organic material.

Table 1
Requirements for Index Properties Tests and Testing Standards

Test	Reference for Standard Test Procedures (a)	Variations from Standard Test Procedures, Sample Requirements	Size or Weight of Sample for Test
Moisture content of soil	(1) ASTM D2216	None (test requires unaltered natural moisture content)	As large as convenient
Moisture, ash, and organic matter of peat materials	(1) ASTM D2974	None	
Dry unit weight	None	Determine total dry weight of a sample of measured total volume (requires undisturbed sample)	As large as convenient
Specific gravity			
Material smaller than No. 4 sieve size	(1) ASTM D854	Volumetric flask preferable; vacuum preferable for de-airing	25 to 50 for fine-grained soil; 150 gm for coarse-grained soils
Material larger than No. 4 sieve size	(1) ASTM C127	None	500 gm
Atterberg Limits		Use fraction passing No. 40 sieve; material should not be dried before testing	
Liquid limit	(1) ASTM D423	None	100 to 500 gm
Plastic limit	(1) ASTM D424	Ground glass plate preferable for rolling	15 to 20 gm
Shrinkage limit	(4)	In some cases a trimmed specimen of undisturbed material may be used rather than a remolded sample	30 gm

Table 1 (continued) Requirements for Index Properties Tests and Testing Standards			
Test	Reference for Standard Test Procedures (a)	Variations from Standard Test Procedures, Sample Requirements	Size or Weight of Sample for Test
Gradation			
Sieve analysis	(1) ASTM D422	Selection of sieves to be utilized may vary for samples of different gradation	500 gm for soil with grains to 3/8”; to 5,000 gm for soils with grains to 3”
Hydrometer analysis	(1) ASTM D422	Fraction of sample for hydrometer analysis may be that passing No. 200 sieve. For fine-grained soil entire sample may be used. All material must be smaller than No. 10 sieve.	65 gm for fine-grained soil; 115 gm for sandy soil.
Corrosivity			
Sulphate content	(5)	Several alternative procedures in reference	Soil/water solution prepared. See reference.
Chloride content	(5)	Several alternative procedures in reference	Soil/water solution prepared. See reference.
pH	(1) ASTM D1293	Reference is for pH of water. For mostly solid substances, solution made with distilled water and filtrate tested; standard not available.	
Resistivity (laboratory)	None	Written standard not available. Follow guidelines provided by manufacturers of testing apparatus.	
Resistivity (field)	(6)	In situ test procedure	

(a) Number in parenthesis indicates Reference number.

(b) Samples for tests may either be disturbed or undisturbed; all samples must be representative and non-segregated; exceptions noted.

(c) Weights of samples for tests on air-dried basis

Table 2 Requirements for Structural Properties			
Test	Reference for Suggested Test (a)	Variations from Suggested Test Procedures	Size or Weight of Sample for Test (undisturbed, remolded or compacted)
Permeability			
Constant head procedure for moderately permeable soil	(2) (4)		Sample size depends on max. grain size, 4 cm dia. By 35 cm height for silt and fine sand
Variable head procedure	(2) (4)	Generally applicable to fine-grained soils	Similar to constant head sample
Constant head procedure for coarse-grained soils	(4) (1) ASTM D2434	Limited to soils containing less than 10% passing No. 200 sieve size. For clean, coarse-grained soil the procedure in (4) is preferable.	Sample diameter should be ten times the size of the largest soil particle.
Capillary head	(2)	Capillary head for certain fine-grained soils may have to be determined indirectly	200 to 250 gm dry weight
Consolidation			
Consolidation	(2)	To investigate secondary compression, individual loads may be maintained for more than 24 hrs	Diameter preferably 2-1/2 in or larger. Ratio of diameter to thickness of 3 to 4.
Swell	(7) AASHTO T258		Diameter preferably 2-1/2 in or larger. Ratio of diameter to thickness of 3 to 4
Collapse potential	(8)		2 specimens for each test, with diameter 2-1/2 in or larger. Diameter to height ratio 3 to 4.

Table 2 (continued) Requirements for Structural Properties			
Test	Reference for Suggested Test (a)	Variations from Suggested Test Procedures	Size or Weight of Sample for Test (undisturbed, remolded or compacted)
Shear Strength			
Direct shear	(2) (1) ASTM D3080	Limited to tests on cohesionless soils or to consolidated shear tests on fine-grained soils	Generally 0.5 in thick, 3 in by 3 in or 4 in by 4 in plan, or equivalent circular cross section
Unconfined compression	(2) (1) ASTM D2166	Alternative procedure given in reference (4)	Similar to triaxial test samples
Triaxial Compression			
Unconsolidated-undrained (Q or UU)	(1) ASTM D2850	Consolidated-undrained tests may run with or without pore pressure measurements, according to basis for design	Ratio of height to diameter should be less than 3 and greater than 2. Common sizes are: 2.8 in dia, 6.5 in high. Larger sizes are appropriate for gravelly materials to be used in earth embankments.
Consolidated-undrained (R or CU)	(2) (4)		
Consolidated-drained (S or CD)			
Vane Shear			Block of undisturbed soil at least three times dimensions of vane.

(a) Number in parenthesis indicates Reference number.

Table 3 Requirements for Dynamic Tests			
Test	Reference for Test Procedure (a) (b)	Variations from Standard Test Procedure	Size or Weight of Sample for Test
Cyclic Loading			
Triaxial compression	(9)		Same as for structural properties triaxial
Simple shear	(9)		
Torsional shear	(10)	Can use hollow specimen	
Resonant Column	(10) (11)	Can use hollow specimen	Same as for structural properties triaxial; length sometimes greater.
Ultrasonic Pulse			
Soil	(12)		Same as for structural properties triaxial
Rock	(1) ASTM D2845		Prism, length less than 5 times lateal dimension; lateral dimension at least 5 times length of compression wave

(a) Number in parenthesis indicates Reference number.

(b) Except for the ultrasonic pulse test on rock, there are no recognized standard procedures for dynamic testing. References are to descriptions of tests and test requirements by recognized authorities in those areas.

Table 4 Requirements for Compacted Samples Tests			
Test	Reference for Standard Test Procedure (a) (b)	Variations from Standard Test Procedures	Size or Weight of Sample for Test (c)
Moisture-Density Relations			
Standard Proctor 5-1/2 lb hammer, 12 in drop	(1) ASTM D 698	Preferable not to reuse samples for successive compaction determinations	Each determination (typically 4 or 5 determinations per test): Method A: 6 lbs Method B: 14 lbs Method C: 10 lbs Method D: 22 lbs
Modified Proctor 10 lb hammer, 18 in drop	(1) ASTM D1557	Preferable not to reuse samples for successive compaction determinations	Method A: 7 lbs Method B: 16 lbs Method C: 12 lbs Method D: 25 lbs
Maximum and Minimum Densities of Cohesionless Soils	(1) (4) ASTM D2049		Varies from 10 to 130 lbs depending on max. grain size
California Bearing Ration	(1) ASTM D1883	Compaction energy other than that for Modified Proctor may be utilized	Each determination requires 15 to 25 lbs depending on gradation
Resistance R-value	(1) ASTM D2844		10-15 lbs depending on gradation
Expansion Pressure	(7) AASHTO T190	Alternatively, testing procedures of Table 2 may be utilized	10-15 lbs depending on gradation
Permeability and Compression	(13)	Best suited for coarse-grained soils. Alternatively, testing procedures of Table 2 may be utilized	15 lbs of material passing No. 4 sieve size

(a) Number in parenthesis indicates Reference number.

(b) For other sources of standard test procedures, see Table 1.

(c) Weight of samples for tests given on air-dried basis.

Table 5 Soil Properties for Analysis and Design				
Property	Symbol	Unit (a)	How Obtained	Direct Applications
Volume-weight Characteristics (b)				
Moisture content	w	D	Direct from test	Classification and volume-weight relations
Unit weights	γ	FL^{-3}	Directly from test or from volume-weight relations	Calssification and pressure computations
Porosity	n	D	Computed from volume-weight relations	Parameters used to represent relative volume of voids with respect to total volume of soil or volume of solids
Void ratio	e	D		
Specific gravity	G	D	Directly from test	Volume computations
Plasticity Characteristics				
Liquid limit	LL	D	Directly from test	Classification and properties correlation
Plastic limit	PL	D	Directly from test	
Plasticity index	PI	D	LL - PL	
Shrinkage limit	SL	D	Directly from test	Classification and computation of swell
Shrinkage index	SI	D	PL - SL	
Activity	A_C	D	PI/%<2 microns	Identification of clay mineral
Liquidity index	LI	D	(W – PL)/PI	Estimating degree of preconsolidation, and soil consistency

Table 5 (continued) Soil Properties for Analysis and Design				
Property	Symbol	Unit (a)	How Obtained	Direct Applications
Gradation Characteristics				
Effective diameter	D_{10}	L	From, grain size curve	Classification, estimating permeability and unit weight, filter design, grout selection, and evaluating potential frost heave and liquefaction
Percent grain size	D_{30}, D_{60}, D_{85}	L	From, grain size curve	
Coefficient of uniformity	C_U	D	D_{60}/D_{10}	
Coefficient of curvature	C_Z	D	$(D_{30})^2/(D_{10)}xD_{60})$	
Clay size fraction		D	From grain size curve, % finer than 0.002 mm	
Drainage Characteristics				
Coefficient of permeability	K	LT^{-1}	Directly from permeability test or computed from consolidation test data	Drainage, seepage, and consolidation analysis
Capillary head	h_C	L	Directly from test	Drainage and drawdown analysis
Effective porosity	n_C	D	Directly from test for volume of drainable water	
Consolidation Characteristics				
Coefficient of compressibility	a_V	L^2F^{-1}	Determine from natural plot of e vs. p curve	Computation of ultimate settlement or heave in consolidation analysis
Coefficient of volume compressibility	m_V	L^2F^{-1}	$a_V/(1+ e)$	

Table 5 (continued)
Soil Properties for Analysis and Design

Property	Symbol	Unit (a)	How Obtained	Direct Applications
Consolidation Characteristics (continued)				
Compression index	C_c	D	Determined from e vs. log p curve	Computation of ultimate settlement or swell in consolidation analysis
Recompression index	C_r	D		
Swelling index	C_s	D		
Coefficient of secondary compression	C	D	Determined from semi-log time-consolidation curve	Computation of time rate of settlement
Coefficient of consolidation	C_V	L^2T^{-1}		
Preconsolidation pressure	P_C	FL^{-2}	Estimate from e vs. log p curve	Settlement analysis
Overconsolidation ratio	OCR	D	P_C/P_O	Basis for normalizing behavior of clay
Shear Strength Characteristics				
Apparent angle of shearing resistance	Ǿ	A	Determined from Mohr circle plot of shear test data for total stress	
Cohesion intercept	c	FL^{-2}		
Effective angle of shearing resistance	Ǿ’	A	Determined from Mohr circle plot of effective stress shear test data (drained tests with pore pressure measurements)	
Effective cohesion	c’	FL^{-2}		

Table 5 (continued) Soil Properties for Analysis and Design				
Property	Symbol	Unit (a)	How Obtained	Direct Applications
Shear Strength Characteristics (continued)				
Unconfined compressive strength	q_u	FL^{-2}	Directly from test	Analysis of stability, load carrying capacity of foundations, lateral earth load
Undrained shear strength	s_u	FL^{-2}		
Sensitivity	s_t	D	Undisturbed strength/remolded strength	
Modulus of elasticity or Young's modulus	E_s	FL^{-2}	Determined from stress-strain curve or dynamic test	Computation of elastic settlement or rebound
Compaction Characteristics				
Maximum dry unit weight	γ_{max}	FL^{-3}	Determined from moisture-dry unit weight curve	Compaction criterion
Optimum moisture content	OMC	D		
Maximum and minimum density of cohesionless soils	$\gamma_{d\ max}$ $\gamma_{d\ min}$	FL^{-3}	Directly from test	
Characteristics of Compacted Samples				
Percent compaction		D	γ / γ_{max}	Compaction control, properties correlation
Needle penetration resistance	p_r	FL^{-2}	Directly from test	Moisture control of compaction
Relative density	D_r	D	Determined from results of max. and min. density tests	Compaction control, properties correlation, liquefaction studies
California Bearing Ratio	CBR	D	Directly from test	Pavement design, compaction control

Table 5 (continued) Soil Properties for Analysis and Design				
Property	Symbol	Unit (a)	How Obtained	Direct Applications
Characteristics of Compacted Samples (continued)				
Shear modulus	G	FL^{-2}	Determined from resonant column, cyclic simple shear, ultrasonic pulse, or dynamic triaxial tests	Analysis of foundation and soil behavior under dynamic loading
Damping ratios		D	Determined from resonant column test, dynamic triaxial, or cyclic simple shear test	
Rod (longitudinal)	D_L			
Shear (torsional)	D_T			
Resonant frequency		T^{-1}	Determined from resonant column testing	
Longitudinal	f_L			
Torsional	F_T			

(a) Units: F=force or weight; L=length; T=time; D=dimensionless; A=angular measure

(b) For complete list of volume-weight relationships, see Table 6

1.3.2 INDEX PROPERTIES TESTS. Index properties are used to classify soils, to group soils in major strata, to obtain estimates of structural properties, and to correlate the results of structural properties tests on one portion of a stratum with other portions of that stratum or other similar deposits where only index test data are available. Procedures for most index tests are standardized (Table 1). Either representative disturbed or undisturbed samples are utilized. Tests are assigned after review of boring data and visual identification of samples recovered. For a simple project with 4 to 6 borings, at least 3 gradation and/or Atterberg tests should be made per significant stratum (5 to 15 feet thick). For complex soil conditions, thick strata, or larger sites with more borings, additional tests should be made. Moisture content tests should be made liberally on samples of fine-grained soil. In general, the test program should be planned so that soil properties and their variation can be defined adequately for the lateral and vertical extent of the project concerned.

1.3.3 TESTS FOR CORROSIVITY. The likelihood of soil adversely affecting foundation elements or utilities (concrete and metal elements) can be evaluated on a preliminary basis from the results of the tests referenced in Table 1. The tests should be run on samples of soil which will be in contact with the foundations and/or utilities in question; typically these will be only near-surface materials. For a simple project with uniform conditions, three sets of tests may be adequate. Usually the chemical tests are run only if there is reason to suspect the presence of those ions.

1.3.4 STRUCTURAL PROPERTIES TESTS. These must be planned for particular design problems. Rigid standardization of test programs is inappropriate. Perform tests only on undisturbed samples obtained as specified or on compacted specimens prepared by standard procedures. In certain cases, completely remolded samples are utilized to estimate the effect of disturbance. Plan tests to determine typical properties of major strata rather than arbitrarily distributing tests in proportion to the number of undisturbed samples obtained. A limited number of high quality tests on carefully selected representative undisturbed samples is preferred. In general, selecting design values requires at least three test values for simple situations of limited areal extent; larger and more complex conditions require several times these numbers.
Where instantaneous deformation characteristics of soils are to be evaluated, constitutive relationships of the materials in question must also be established.

1.3.5 DYNAMIC TESTS. Dynamic testing of soil and rock involves three ranges: low frequency (generally less than 10 hertz) cyclic testing, resonant column high frequency testing, and ultrasonic pulse testing. The dynamic tests are used to evaluate foundation support characteristics under repeated loadings such as a drop forge, traffic, or earthquake; a primary concern is often liquefaction. Young's modulus (E), shear modulus (G), and damping characteristics are determined by cyclic triaxial and simple shear tests. Resonant column can be used to determine E_s , G, and damping. From the resonant frequency of the material in longitudinal, transverse, and torsional modes, Poisson's ratio (γ) can be computed from test data. Foundation response to dynamic loading and the effect of wave energy on its surroundings is studied in the light of these test results. The ultrasonic pulse test also evaluates the two moduli and Poisson's ratio, but the test results are more reliable for rocks than for soils. Dynamic tests can be run on undisturbed or compacted samples, but should be run only if the particular project really requires them. The number of tests depends on project circumstances. Estimates of dynamic parameters can be obtained from correlations with other properties.

1.3.6 COMPACTION TESTS. In prospecting for borrow materials, index tests or compaction tests may be required in a number proportional to the volume of borrow involved or the number of samples obtained. Structural properties tests are assigned after borrow materials have been grouped in major categories by index and compaction properties. Select samples for structural tests to represent the main soil groups and probable compacted condition. At least one compaction or relative density test is required for each significantly different material (based on gradation or plasticity). Numbers of other tests depend on project requirements.

1.3.7 TYPICAL TEST PROPERTIES. Various correlations between index and structural properties are available showing the probable range of test values and relation of parameters. In testing for structural properties, correlations can be used to extend results to similar soils for which index values only are available. Correlations are of varying quality, expressed by standard deviation, which is the range above and below the average trend, within which about two-thirds of all values occur. These relationships are useful in preliminary analyses but must not supplant careful tests of structural properties. The relationships should never be applied in final analyses without verification by tests of the particular material concerned.

2. INDEX PROPERTIES TESTS

2.1   MOISTURE CONTENT, UNIT WEIGHT, SPECIFIC GRAVITY. Index properties tests are used to compute soil volume and weight components (Table 6). Ordinarily, determine moisture content for all the representative samples (disturbed or undisturbed) for classification and grouping of materials in principal strata. See Table 1 for test standards.

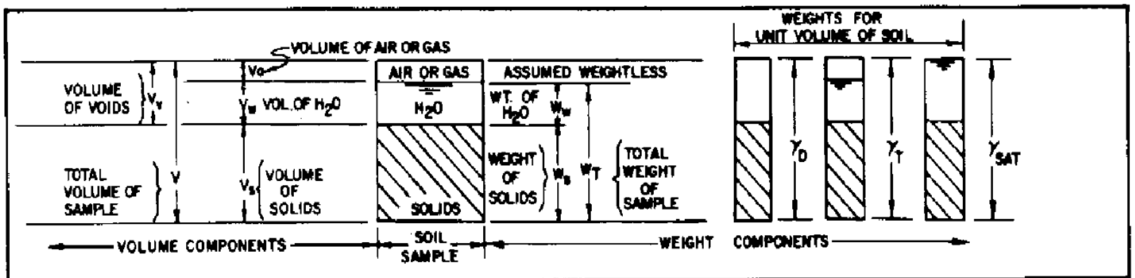
2.1.1 UNSATURATED SAMPLES. Measure moisture content, dry weight, specific gravity, and total volume of specimen to compute volume-weight relationships.

2.1.2 SATURATED SAMPLES. If moisture content and dry weight are measured, all volume-weight parameters may be computed by assuming a specific gravity. If moisture content and specific gravity are measured, all volume-weight parameters may be computed directly. Volume-weight of fine-grained soils below the water table may be determined with sufficient accuracy by assuming saturation.

2.2 GRADATION. In addition to their use in classification, grain-size analyses may be applied to seepage and drainage problems, filter and grout design, and evaluation of frost heave. See Table 1 for test standards.

2.2.1 GRAIN-SIZE PARAMETERS. Coefficient of uniformity C_u and coefficient of curvature C_z are computed from D_{60}, D_{30}, and D_{10}, which are particle size diameter corresponding respectively to 60%, 30%, and 10% passing on the cumulative particle size distribution curves. C_u and C_z indicate the relative broadness or narrowness of gradation. D_{10}, is an approximate measure of the size of the void spaces in coarse-grained soils.

Volume and Weight Relationships



		PROPERTY	SATURATED SAMPLE (W_s, W_w, G, ARE KNOWN)	UNSATURATED SAMPLE (W_s, W_w, G, V, ARE KNOWN)	SUPPLEMENTARY FORMULAS RELATING MEASURED AND COMPUTED FACTORS			
VOLUME COMPONENTS	V_s	VOLUME OF SOLIDS	$\frac{W_s}{G\gamma_w}$		$V-(V_a+V_w)$	$V(1-n)$	$\frac{V}{(1+e)}$	$\frac{V_v}{e}$
	V_w	VOLUME OF WATER	$\frac{W_w}{\gamma_w}$		$V_v - V_a$	SV_v	$\frac{SVe}{(1+e)}$	$SV_s e$
	V_a	VOLUME OF AIR OR GAS	ZERO	$V-(V_s+V_w)$	$V_v - V_w$	$(1-S)V_v$	$\frac{(1-S)Ve}{(1+e)}$	$(1-S)\ V_s e$
	V_v	VOLUME OF VOIDS	$\frac{W_w}{\gamma_w}$	$V-\frac{W_s}{G\gamma_w}$	$V - V_s$	$\frac{V_s n}{1-n}$	$\frac{Ve}{(1+e)}$	$V_s e$
	V	TOTAL VOLUME OF SAMPLE	$V_s + V_w$	MEASURED	$V_s + V_a + V_w$	$\frac{V_s}{1-n}$	$V_s(1+e)$	$\frac{V_v(1+e)}{e}$
	n	POROSITY	$\frac{V_v}{V}$		$1-\frac{V_s}{V}$	$1-\frac{W_s}{GV\gamma_w}$	$\frac{e}{1+e}$	
	e	VOID RATIO	$\frac{V_v}{V_s}$		$\frac{V}{V_s}-1$	$\frac{GV\gamma_w}{W_s}-1$	$\frac{W_w G}{W_s S}$	$\frac{n}{1-n}$; $\frac{wG}{S}$

Table 6
Volume and Weight Relationships

Volume and Weight Relationships

		PROPERTY	SATURATED SAMPLE (W_s, W_w, G, ARE KNOWN)	UNSATURATED SAMPLE (W_s, W_w, G, V, ARE KNOWN)	SUPPLEMENTARY FORMULAS RELATING MEASURED AND COMPUTED FACTORS			
WEIGHTS FOR SPECIFIC SAMPLE	W_s	WEIGHT OF SOLIDS	MEASURED		$\frac{W_T}{(1+W)}$	$GV\gamma_w(1-n)$	$\frac{W_w G}{eS}$	
	W_w	WEIGHT OF WATER	MEASURED		wW_s	$S\gamma_w V_v$	$\frac{eW_s S}{G}$	
	W_t	TOTAL WEIGHT OF SAMPLE	$W_s + W_w$		$W_s(1+w)$			
WEIGHTS FOR SAMPLE OF UNIT VOLUME	γ_D	DRY UNIT WEIGHT	$\frac{W_s}{V_s + V_w}$	$\frac{W_s}{V}$	$\frac{W_t}{V(1+w)}$	$\frac{G\gamma_w}{(1+e)}$	$\frac{G\gamma_w}{1+wG/S}$	
	γ_T	WET UNIT WEIGHT	$\frac{W_s+W_w}{V_s+V_w}$	$\frac{W_s+W_w}{V}$	$\frac{W_T}{V}$	$\frac{(G+Se)\gamma_w}{(1+e)}$	$\frac{(1+w)\gamma_w}{w/S+1/G}$	
	γ_{SAT}	SATURATED UNIT WEIGHT	$\frac{W_s + W_w}{V_s + V_w}$	$\frac{W_s + V_v\gamma_w}{V}$	$\frac{W_s}{V} + \left(\frac{e}{1+e}\right)\gamma_w$	$\frac{(G+e)\gamma_w}{(1+e)}$	$\frac{(1+w)\gamma_w}{w+1/G}$	
	γ_{SUB}	SUBMERGED (BUOYANT) UNIT WEIGHT	$\gamma_{SAT} - \gamma_w$		$\frac{W_s}{V} - \left(\frac{1}{1+e}\right)\gamma_w$	$\left(\frac{G+e}{1+e} - 1\right)\gamma_w$	$\left(\frac{1-1/G}{w+1/G}\right)\gamma_w$	
COMBINED RELATIONS	w	MOISTURE CONTENT	$\frac{W_w}{W_s}$		$\frac{W_t}{W_s} - 1$	$\frac{Se}{G}$	$S\left[\frac{\gamma_w}{\gamma_D} - \frac{1}{G}\right]$	
	S	DEGREE OF SATURATION	1.00	$\frac{V_w}{V_v}$	$\frac{W_w}{V_v \gamma_w}$	$\frac{wG}{e}$	$\frac{w}{\left[\frac{\gamma_w}{\gamma_D} - \frac{1}{G}\right]}$	
	G	SPECIFIC GRAVITY	$\frac{W_s}{V_s \gamma_w}$		$\frac{Se}{w}$			

Table 6 (continued)
Volume and Weight Relationships

2.2.2 TESTING PROGRAM. Gradations of a large number of samples usually are not required for identification. Samples should be grouped in principal strata by visual classification before performing grain-size analyses on specimens of major strata.

2.3 ATTERBERG LIMITS. For classification of the fine-grained soils by Atterberg Limits. In addition to their use in soil classification, Atterberg Limits also are indicators of structural properties, as shown in the correlations in this discussion. Atterberg Limit tests should be performed discriminately, and should be reserved for representative samples selected after evaluating subsoil pattern. Determine Atterberg Limits of each consolidation test sample and each set of samples grouped for triaxial shear tests. For selected borings, determine Atterberg Limits on samples at regular vertical intervals for a profile of Limits and corresponding natural water content. See Table 1 for test standards.

3. PERMEABILITY TESTS

3.1 APPLICATIONS. Permeability coefficient is used to compute the quantity and rate of water flow through soils in drainage and seepage analysis. Laboratory tests are appropriate for undisturbed samples of fine-grained materials and compacted materials in dams, filters, or drainage structures. See Table 2 for test standards and recommended procedures.

3.1.1 FINE-GRAINED SOILS. Permeability of fine-grained soils (undisturbed or compacted) generally is computed from consolidation test data or by direct measurement on consolidation or triaxial shear specimens. For soils with permeability less than 10 cm/sec, a sealant must be used between the specimen and the wall of the permeameter.

3.1.2 SAND DRAIN DESIGN. Sand drain design may require complete permeability data for soils to be stabilized, including determination of permeabilities in both vertical and horizontal direction.

3.1.3 FIELD PERMEABILITY TESTS. The secondary structures of in situ soils, stratification, and cracks have a great influence on the permeability. Results of laboratory tests should be interpreted with this in mind, and field permeability tests should be performed where warranted.

3.2 TYPICAL VALUES. Coefficient of permeability is a property highly sensitive to sample disturbance, and shows a wide range of variation due to differences in structural characteristics. Permeability of clean, coarse-grained samples is related to D_{10} size (Figure 1).

4. CONSOLIDATION TESTS

4.1 UTILIZATION. One-dimensional consolidation tests with complete lateral confinement are used to determine total compression of fine-grained soil under an applied load and the time rate of compression caused by gradual volume decrease that accompanies the squeezing of pore water from the soil.

4.2 TESTING PROGRAM. Consolidation tests require undisturbed samples of highest quality. Select samples representative of principal compressible strata. Determination of consolidation characteristics of a stratum requires from two to about eight tests, depending on the complexity of conditions. Select loading program to bracket anticipated field loading conditions.

4.2.1 INCREMENTAL LOADING (IL) WITH STRESS CONTROL. Ordinarily, apply loads starting at 1/4 tsf and increase them by doubling 1/2, 1, 2, 4, 8, etc., tsf. For soils with pronounced swelling tendency, it may be necessary to rapidly increase loading to 1/2 tsf or higher, perhaps to overburden pressure, to prevent initial swell. For soft, normally consolidated soils, start loading at 1/16 or 1/32 tsf and increase loads by doubling the previous value. To establish the reconsolidation index C_r, and swelling index C_s, include an unload-reload cycle, after P_c, has been reached. Unload must be to 1/8 the existing load, or preferably less. Reloads should be applied in the same manner as for the initial curve.

4.2.2 CONSTANT RATE OF STRAIN (CRS). The specimen is subjected to a constantly changing load while maintaining a constant rate of strain. Pore pressure is continuously monitored to ensure that the primary consolidation is completed at the applied strain rate. These tests can be performed in shorter time than IL tests and yield more accurate values of preconsolidation pressure P_c. Coefficient of consolidation c_v, values can be determined for very small load increments, but the test equipment is more complicated and requires that estimates of strain rate and P_c, be made prior to the start of the test. See Reference 15, Consolidation at Constant Rate of Strain, by Wissa, et al for guidance.

4.2.3 GRADIENT CONTROLLED TEST (GC). Drainage is permitted at the upper porous stone while pore pressure is measured at the lower porous stone. A loading control system regulates the application of load so that a predetermined hydrostatic excess pressure is maintained at the bottom of the specimen. This method as well as CRS has similar advantages over IL, but does not require a prior estimate of strain rate. However, the equipment is more complex than for CSR.

4.3 PRECONSOLIDATION PRESSURE. This pressure value, P_c, forms the boundary between recompression and virgin compression ranges and is approximately the maximum normal effective stress to which the material in situ has been consolidated by a previous loading. Desiccation produces a similar effect. The preconsolidation pressure cannot be determined precisely, but can be estimated from consolidation tests on high quality undisturbed samples.

4.3.1 GRAPHICAL DETERMINATION. Estimate preconsolidation pressure from semi-logarithmic pressure-void ratio curve using the procedure given in the central panel of Figure 2. Alternative methods are given in Reference 17, Foundation Engineering, by Leonards, and Reference 18, The Undisturbed Consolidation of Clay, by Schmertmann. Maximum test pressures should exceed preconsolidation by an amount sufficient to define the slope of virgin compression. Generally, this requires application of three or more load increments exceeding the preconsolidation value.

4.3.2 APPROXIMATE VALUES. See Figure 3 for a relationship between preconsolidation pressure and liquidity index. For samples with natural moisture at the liquid limit (liquidity index of 1), preconsolidation ranges between about 0.1 and 0.8 tsf depending on soil sensitivity. For natural moisture at the plastic limit (liquidity index equal to zero), preconsolidation ranges from about 12 to 25 tsf.

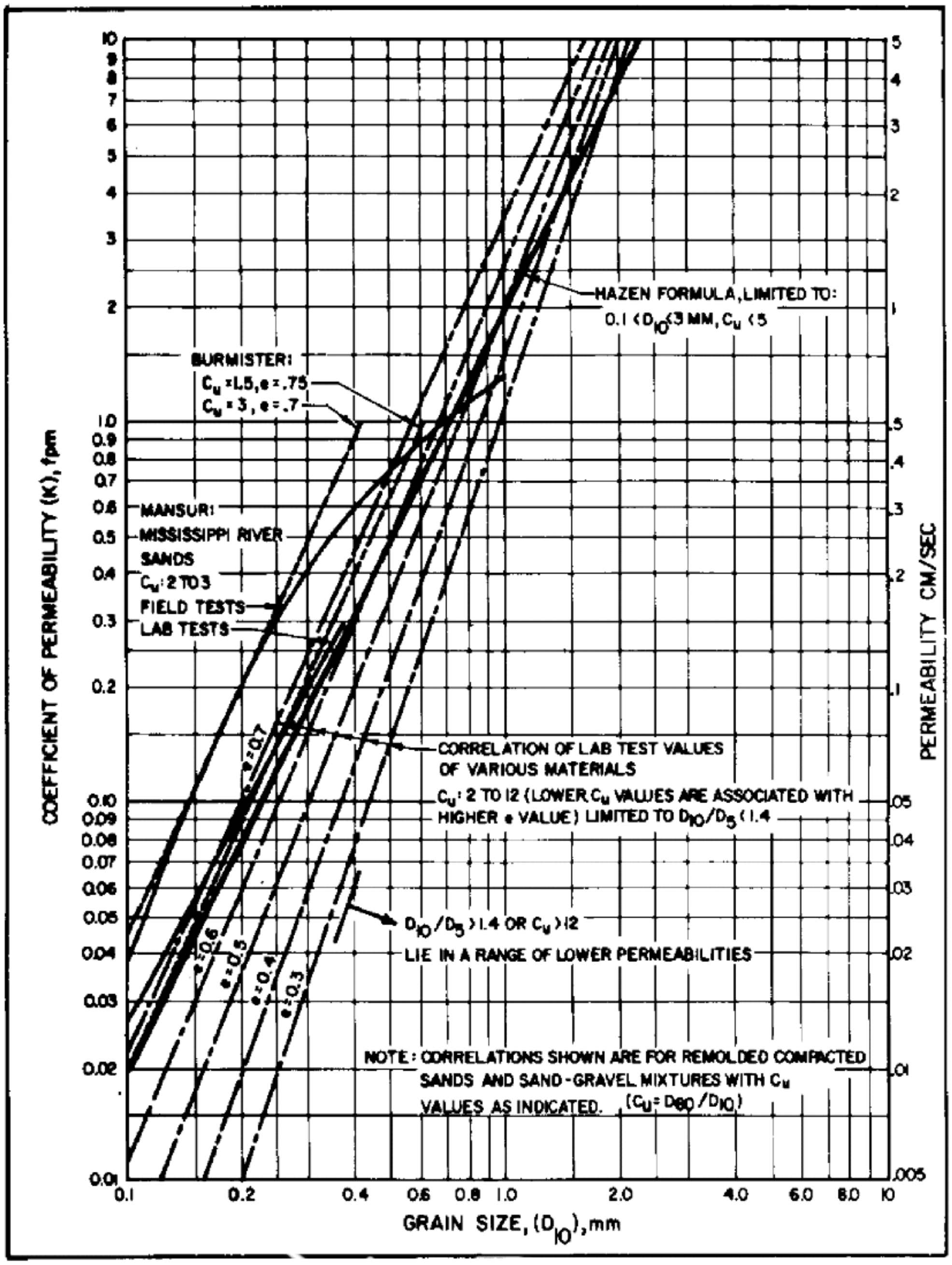
4.4 VIRGIN COMPRESSION. Virgin compression is deformation caused by loading in the range of pressures exceeding that to which the sample has been subjected in the past.

4.4.1 COMPRESSION INDEX. The semilogarithmic, pressure-void ratio curve is roughly linear in the virgin range. The semilogarithmic, straight line slope for virgin compression is expressed by the compression index C_c (See Figure 2.)

4.4.2 APPROXIMATE VALUES. The compression index of silts, clays, and organic soils has been correlated with the natural water content, initial void ratio and the liquid limit. The approximate values of C_c, for uniform sands in the load range of 1 to 4 tsf may vary from 0.05 to 0.06 (loose condition), and from 0.02 to 0.03 (dense condition).

4.5 RECOMPRESSION AND SWELL. Depending on the magnitude of preconsolidation, pressures applied by new construction may lie partly or wholly in the recompression range. If the load is decreased by excavation, fine-grained soil will undergo a volumetric expansion in the stress range below preconsolidation.

4.5.1 SWELLING INDEX. The slope of straight-line rebound of the semilogarithmic pressure-void ratio curve is defined by C_s (see Figure 2). The swelling index is generally one-fifth to one-tenth of the compression index except for soils with very high swell potential.



Permeability of Sands and Sand-Gravel Mixtures

Figure 1

Permeability of Sand and Sand-Gravel Mixtures

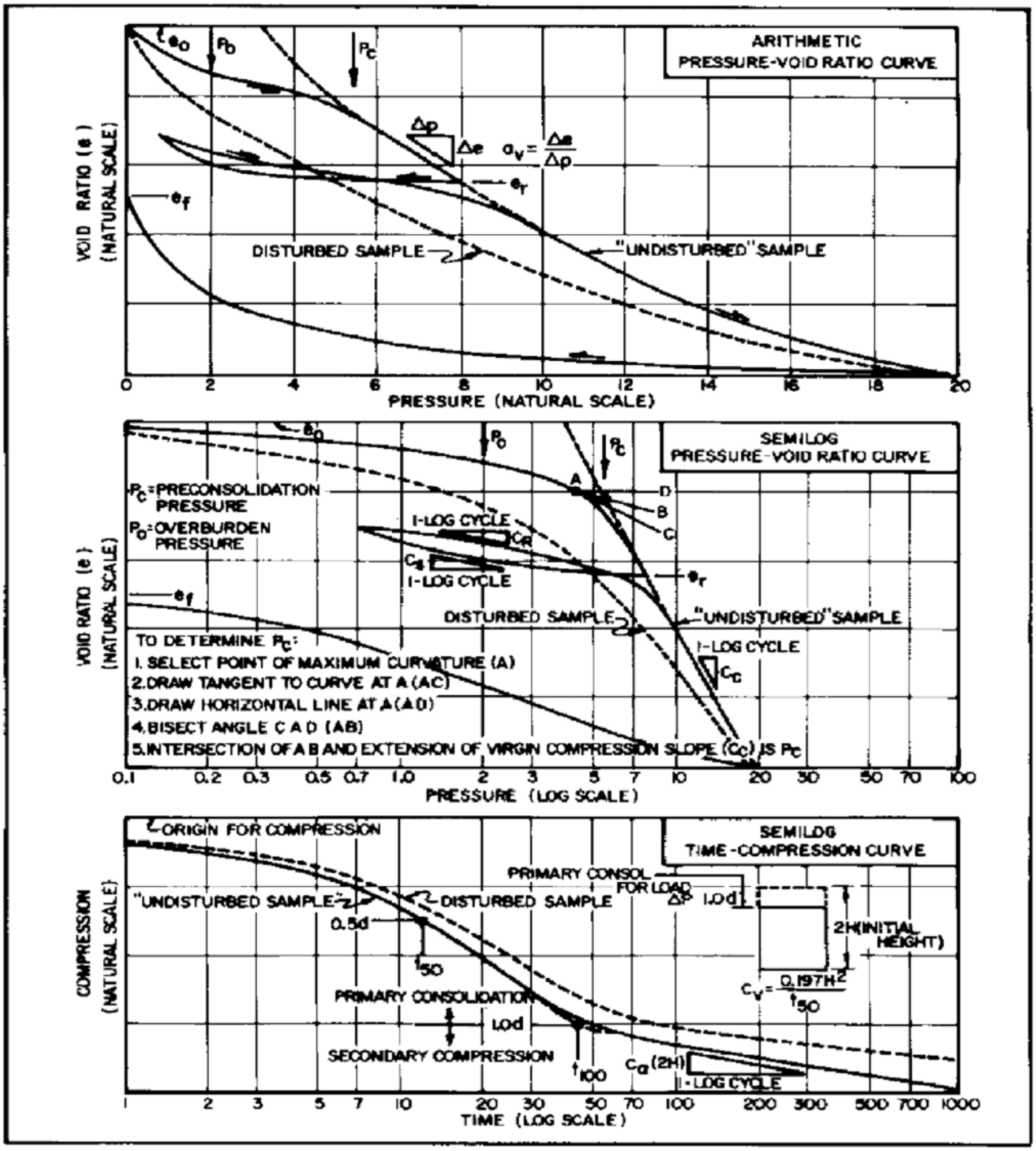


Figure 2

Consolidation Test Relationships

4.5.2 RECOMPRESSION INDEX. The slope of the straight line in the recompression range of the semilogarithmic pressure-void ratio curve is defined by C_r, where C_r is equal to or less than C_s. (See Figure 2).

4.6 COMPRESSION OF COLLAPSIBLE SOILS. Such soils require a special test for determining their collapse potential.

4.7 COEFFICIENT OF CONSOLIDATION (c_v). Those soil properties that control the drainage rate of pore water during consolidation are combined in the coefficient of consolidation.

4.7.1 DETERMINATION. Compute c_v from the semilogarithmic time-compression curve for a given load increment (bottom panel of Figure 2). Correct the origin for compression for the effect of air or gas in void spaces by the procedure given in Reference 2.

4.7.2 APPROXIMATE VALUES. Figure 4 may be used to determine approximate values of c_v.

4.8 SECONDARY COMPRESSION. After completion of primary consolidation under a specific load, the semilogarithmic time-compression curve continues approximately as a straight line. This is termed secondary compression (Figure 2). It occurs when the rate of compression is no longer primarily controlled by the rate at which pore water can escape; there are no excess pore pressures remaining.

4.8.1 ORGANIC MATERIALS. In organic materials, secondary compression may dominate the time-compression curve, accounting for more than one-half of the total compression, or even obliterating the change in slope used to establish the limit of primary compression.

4.8.2 APPROXIMATE VALUES. The coefficient of secondary compression C_α is a ratio of decrease in sample height to initial sample height for one cycle of time on log scale. See bottom panel of Figure 4 for typical values.

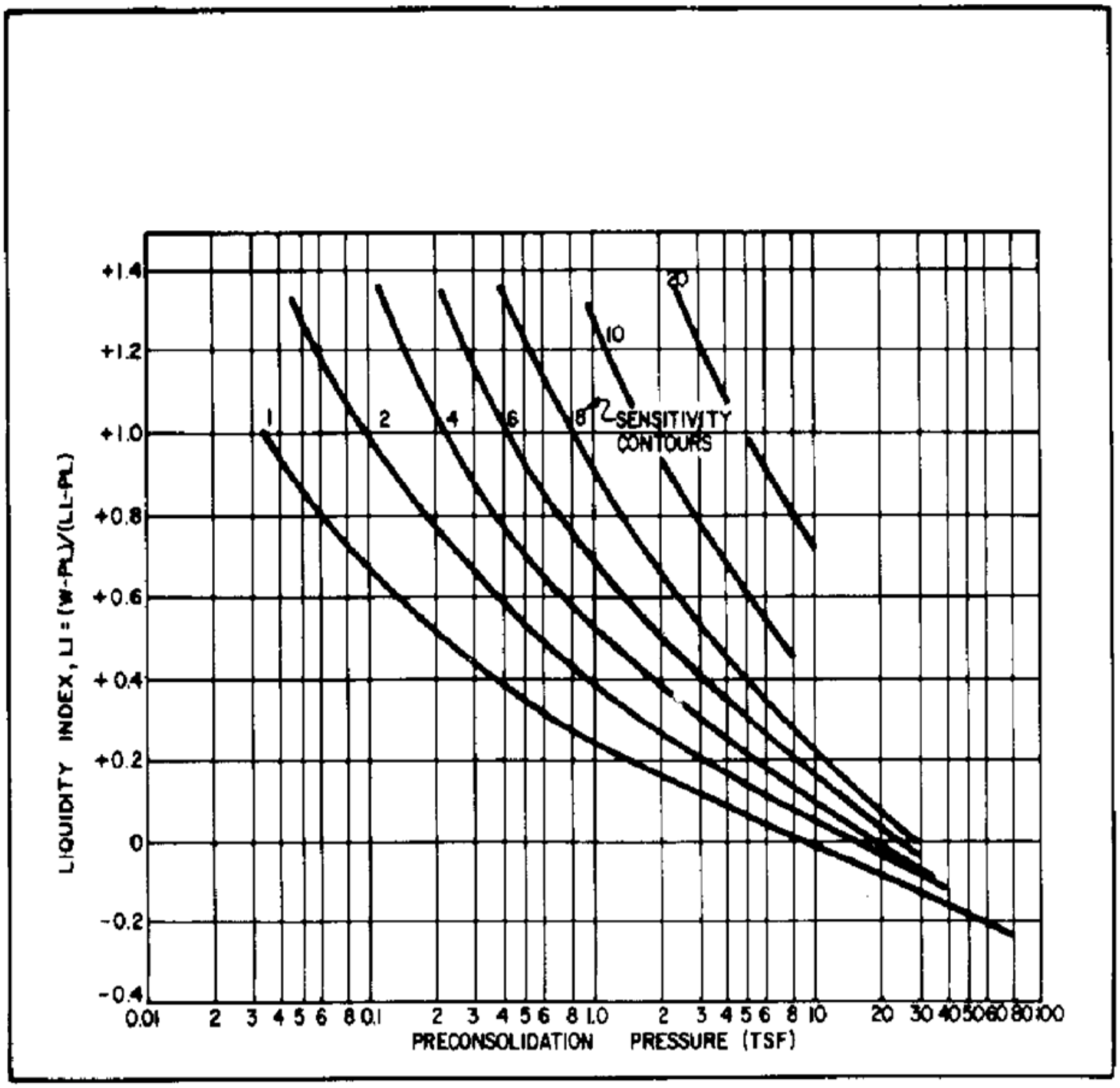


Figure 3

Preconsolidation Pressure vs. Liquidity Index

4.9 SAMPLE DISTURBANCE. Sample disturbance seriously affects the values obtained from consolidation tests as shown in Figure 2 and below.

4.9.1 VOID RATIO. Sample disturbance lowers the void ratio reached under any applied pressure and makes the location of the preconsolidation stress less distinct.

4.9.2 PRECONSOLIDATION PRESSURE. Sample disturbance tends to lower the compression index (C_c) and the preconsolidation pressure (P_c) obtained from the test curve.

4.9.3 RECOMPRESSION AND SWELLING. Sample disturbance increases the recompression and swelling indices.

4.9.4 COEFFICIENT OF CONSOLIDATION. Sample disturbance decreases coefficient of consolidation for both recompression and virgin compression. For an undisturbed sample, c_v usually decreases abruptly at preconsolidation stress. This trend is not present in badly disturbed samples.

4.9.5 COEFFICIENT OF SECONDARY COMPRESSION. Sample disturbance tends to decrease the coefficient of secondary compression in virgin compression loading range.

5. SHEAR STRENGTH TESTS

5.1 UTILIZATION. The shear strength of soil is required for the analysis of all foundation and earthwork stability problems. Shear strength can be determined by laboratory and field tests, and by approximate correlations with grain size, water content, density, and penetration resistance.

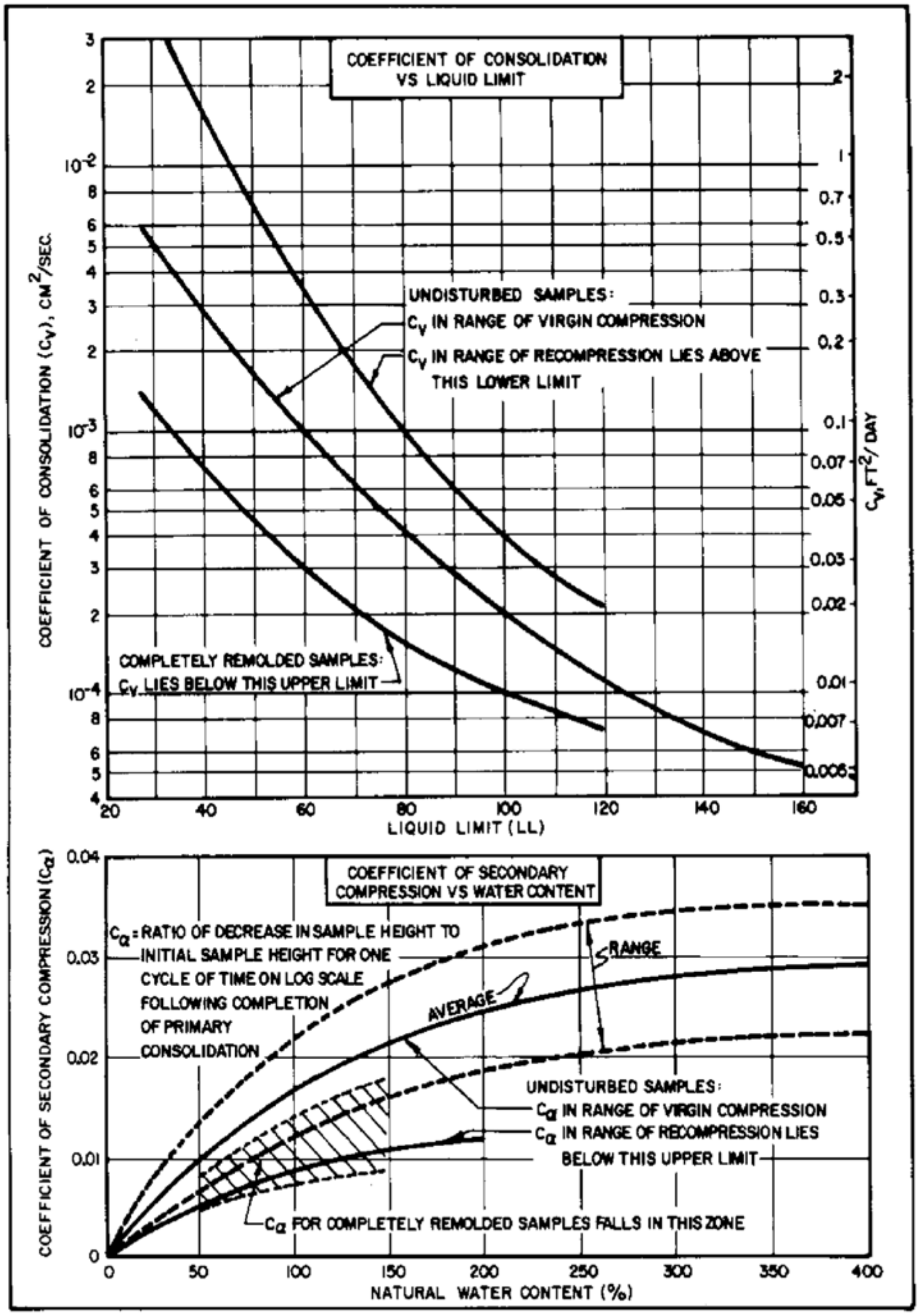


Figure 4

Approximate Correlation for Consolidation Characteristics of Silts and Clays

5.2 TYPES OF SHEAR TESTS. Many types and variations of shear tests have been developed. In most of these tests the rate of deformation is controlled and the resulting loads are measured. In some tests total stress parameters are determined, while in others effective stress strength parameters are obtained. The following are the most widely used testing procedures:

5.2.1 DIRECT SHEAR TEST. A thin soil sample is placed in a shear box consisting of two parallel blocks. The lower block is fixed while the upper block is moved parallel to it in a horizontal direction. The soil fails by shearing along a plane assumed to be horizontal. This test is relatively easy to perform. Consolidated-drained tests can be performed on soils of low permeability in a short period of time as compared to the triaxial test. However, the stress, strain, and drainage conditions during shear are not as accurately understood or controlled as in the triaxial test.

5.2.2 UNCONFINED COMPRESSION TEST. A cylindrical sample is loaded in compression. Generally failure occurs along diagonal planes where the greatest ratio of shear stress to shear strength occurs. Very soft material may not show diagonal planes of failure but generally is assumed to have failed when the axial strain has reached a value of 20 percent. The unconfined compression test is performed only on cohesive soil samples. The cohesion (c) is taken as one-half the unconfined compressive strength.

5.2.3 TRIAXIAL COMPRESSION TEST. A cylindrical sample is confined by a membrane and lateral pressure is applied; pore water drainage is controlled through tubing connected to porous discs at the ends of the sample. The triaxial test (Figure 5) permits testing under a variety of loading and drainage conditions and also allows measurement of pore water pressure. For details on testing procedures, see Reference 2. Triaxial shear test relationships are shown graphically in Figure 6.

5.2.3.1 Unconsolidated-Undrained (UU) or Quick Test (Q). In the UU test the initial water content of the test specimen is not permitted to change during shearing of the specimen. The shear strength of soil as determined in UU tests corresponds to total stress, and is applicable only to situations where little consolidation or drainage can occur during shearing. It is applicable primarily to soils having permeability less than 10^{-3} cm per sec.

5.2.3.2 Consolidated-Undrained (CU) or R Test. In the CU test, complete consolidation of the test specimen is permitted under the confining pressure, but no drainage is permitted during shear. A minimum of three tests is required to define strength parameters c and [phi], though four test specimens are preferable with one serving as a check. Specimens must as a general rule be completely saturated before application of the deviator stress. Full saturation is achieved by back pressure. Pore water pressure is measured during the CU test, thus permitting determination of the effective stress parameters c' and [phi]'. In the absence of pore pressure measurements CU tests can provide only total stress values c and φ.

5.2.3.3 Consolidated-Drained (CD) or S Test. In the CD test, complete consolidation of the test specimen is permitted under the confining pressure and drainage is permitted during shear. The rate of strain is controlled to prevent the build-up of pore pressure in the specimen. A minimum of three tests are required for c' and φ determination. CD tests are generally performed on well draining soils. For slow draining soils, several weeks may be required to perform a CD test.

5.2.3.4 Factors Affecting Tests. Triaxial test results must be appropriately corrected for membrane stiffness, piston friction, and filter drains, whenever applicable. The shear strength of soft sensitive soils is greatly affected by sample disturbance. The laboratory-measured shear strength of disturbed samples will be lower than the in-place strength in the case of UU tests. In the case of CU or CD tests, the strength may be higher because of the consolidation permitted.

5.2.4 OTHER PROCEDURES. In certain instances, more sophisticated tests are warranted. These may include triaxials with zero lateral strain conditions, simple shear tests, and tests inducing anisotropic stress conditions.

5.3 TEST SELECTION. In determining the type of test to be employed, considerations must be given to soil type and the applications for which the test data is required.

5.3.1 SOIL TYPE.

5.3.1.1 Clean Sands and Gravels. Undisturbed samples are very difficult to obtain and test properly, therefore sophisticated shear tests are usually impractical. For simple foundation problems, the angle of internal friction can be satisfactorily approximated by correlation with penetration resistance, relative density, and soil classification (Figure 7). Confirmation of the potential range of the angle of internal friction can be obtained from shear tests on the sample at laboratory densities bracketing conditions anticipated in the field. For earth dam and high embankment work where the soil will be placed under controlled conditions, triaxial compression tests are warranted.

5.3.1.2 Clays. For simple total stress applications where the immediate stability of foundations or embankments is of concern, the unconfined compression test or UU triaxial test is often adequate. For very soft or sensitive soils, difficult to sample, the field vane test is useful. For long-term stability problems requiring effective stress analysis, such as landslides, CU triaxial tests with pore pressure measurements should be used. Long-term stability problems in some highly overconsolidated clays may require the CD test (see Reference 19, Stability of Natural Slopes and Embankment Foundations State-of-the-Art Report, by Skempton and Hutchinson).

5.3.1.3 Silts and Mixed Soils. The choice of test is governed by whether total stress analysis or effective stress analysis is applicable. In cases of very soft silts, such as in marine deposits, the in-place vane shear test is especially helpful in evaluating the shear strength and its increase with depth. For some thinly layered soils, such as varved clay, direct shear tests or simple shear tests are well suited for determining the strength of the individual layers. Where partial drainage is anticipated, use CU tests with pore water pressure measurements to obtain effective strength parameters.

5.3.1.4 Overconsolidated Soils. Frequently overconsolidated soils have defects such as jointing, fissures, etc. The laboratory values of strength which are obtained from a small test specimen are generally higher than the field strength values which are representative of the entire soil mass. The release of stress due to excavation and exposure to weathering reduces strength over a long period of time. This effect cannot be assessed by any of the laboratory tests currently in use. Most overconsolidated clays are anisotropic and the degree of anisotropy may also be influenced by their age. Effect of anisotropy can be determined in the laboratory. In highly overconsolidated soil which may not be fully saturated, unusually high back pressure may be necessary to achieve full saturation, thus making it difficult to perform CU tests. CD tests are more appropriate.

5.3.2 TYPE OF APPLICATION.

5.3.2.1 Total Stress Analysis. It is appropriate for the immediate (during and end of construction) safety of foundations and structures (embankments) consisting of or resting on clays where permeability is low. It is also applicable to embankment stability where rapid drawdown can occur. Use of unconfined compression tests or UU test is appropriate. Sample disturbance has significant effect on shear strength in these types of tests.

5.3.2.2 Effective Stress Analysis. Evaluation of long-term stability of slopes, embankments, and earth supporting structures in cohesive soil requires the use of effective stress strength parameters, and therefore CU tests with pore water pressure measurements or CD tests are appropriate. Tests must be run at a slow enough strain rate so that pore pressures are equalized during the CU test or are dissipated throughout the CD test. Essentially all analyses of granular soils are made using effective stress.

5.3.2.3 Stress Path Method. The stress path method is based on modeling the geological and historical stress conditions as they are known to influence soil behavior. To apply the method, stress history is determined and future stresses are computed based on actual construction plans. The stresses are modeled in a set of triaxial or similar strength tests (see Figure 6). Details of this procedure are found in Reference 20, Stress Path Method, Second Edition, by Lambe and Marr.

6. DYNAMIC TESTING

6.1 UTILIZATION. Capabilities of dynamic soil testing methods and their suitability for various motion characteristics are shown in Table 7. Dynamic testing is needed for loose granular soils and soft sensitive clays in earthquake areas, for machine foundation design, and for impact loadings. Only a brief description of tests follows.

6.2 RESONANT COLUMN TEST. The resonant column test consists of the application of sinusoidal vibration to one end (termed the active end) of asolid or hollow cylindrical soil specimen. The other end is known as the passive end. Compression waves or shear waves are propagated through the soil specimen to determine either Young's modulus (E) or shear modulus (G). Moduli are computed from the resonant frequency of the cylinder. For example, in the case where passive end platen is fixed, the lowest frequency for which the excitation force is in phase with the velocity at the active end is termed the resonant frequency. Damping is determined by turning off the excitation at resonant frequency and recording the decaying vibration.

6.3 CYCLIC TESTS. Currently, these are the most commonly used methods of evaluating the Young's modulus, shear modulus, damping, and liquefaction potential of coarse-grained soils.

6.3.1 CYCLIC TRIAXIAL COMPRESSION TEST. In triaxial testing of saturated soils, cell pressure is maintained constant while the axial stress is varied.

6.3.2 CYCLIC SIMPLE SHEAR TEST. Simple shear equipment has also found wide use in cyclic testing. The non-uniform stress conditions in simple shear may cause failure at a lower stress than that which would cause failure in situ. Measurement or control of lateral pressure is difficult in simple shear tests.

6.3.3 CYCLIC TORSIONAL SHEAR. Cyclic torsional simple shear tests on hollow samples offer the capability of measuring lateral confining pressure. In hollow cylinders stresses within the specimen are more uniform, though the specimens are difficult to produce. Also, tapered hollow cylinders have been used in torsional cyclic tests.

TABLE 7

Capabilities of Dynamic Testing Apparatus

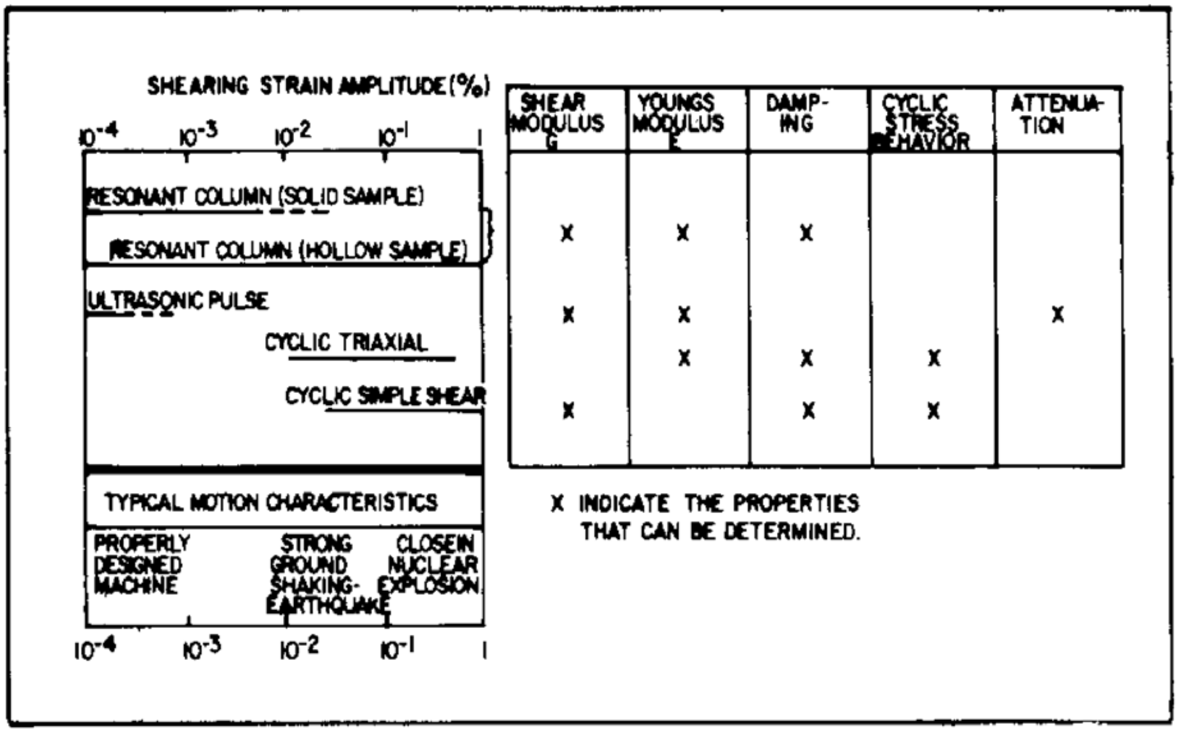


Table 7

Capabilities of Dynamic Testing Apparatus

6.3.4 FACTORS AFFECTING TESTS. Various testing and material factors that may affect cyclic strength as determined in the laboratory are method of specimen preparation, difference between reconstituted and intact specimens, prestressing, loading wave form, grain size and gradation, etc. For details on cyclic testing, see Reference 21, A Review of Factors Affecting Cyclic Triaxial Tests, by Townsend. For the nature of soil behavior under various types of dynamic testing see Reference 22, The Nature of Stress-Strain Behavior for Soils, by Hardin.

6.4 EMPIRICAL INDICATORS. The empirical relationships given here are to be used only as indicators and not in final design. Design involving dynamic properties of soil must be done only under the direction of experienced personnel.

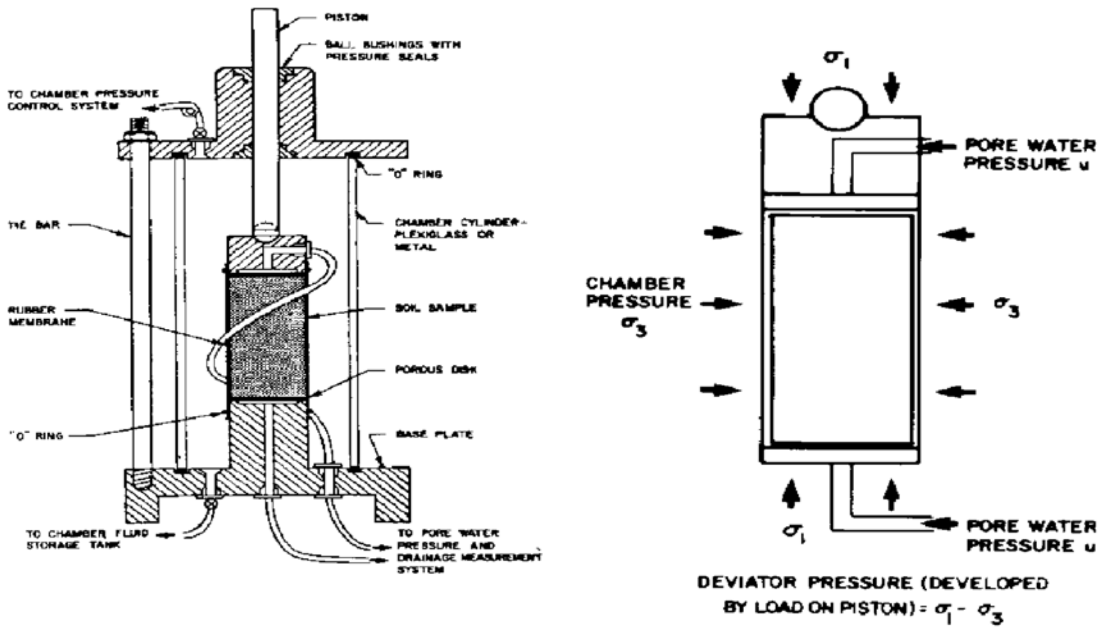
6.4.1 SHEAR MODULUS. In the absence of dynamic tests initial estimates of shear modulus, G, may be made using the relationships found in Reference 23, Shear Modulus and Damping in Soils: Design Equations and Curves, by Hardin and Drnevich, and Reference 24, Soil Moduli and Damping Factors for Dynamic Response Analyses, by Seed and Idriss.

6.4.2 POISSON'S RATIO. Values of Poisson's ratio (γ) are generally difficult to establish accurately. For most projects, the value does not affect the response of the structure sufficiently to warrant a great deal of effort in their determination. For cohesionless soils $\gamma = 0.25$ and for cohesive soils $\gamma = 0.33$ are considered reasonable assumptions. See Reference 25, Foundation Vibration, by Richart.

6.4.3 LIQUEFACTION OF COARSE-GRAINED SOILS. Liquefaction has usually occurred in relatively uniform material with D_{10}, ranging between 0.01 and 0.25 mm, C_u between 2 and 10, and standard penetration resistance less than 25 blows per foot. Liquefaction is more likely to be triggered by higher velocity than by higher acceleration. These characteristics may be used as a guide in determining the need for dynamic testing. The potential influence of local soil conditions (depth of stratum, depth of groundwater table, variation in soil density, etc.) on shaking and damage intensity must be carefully evaluated. See References 26, Earthquake Effects on Soil Foundation Systems, by Seed, and Reference 27, A Practical Method for Assessing Soil Liquefaction Potential Based on Case Studies at Various Sites in Japan, by Iwasaki, et al. A surcharge reduces the tendency of a deposit to liquefy.

7. TESTS ON COMPACTED SOILS

7.1 UTILIZATION. Compaction is used to densify soils during placement to minimize post-construction consolidation and to improve strength characteristics. Compaction characteristics are determined by moisture density testing; structural and supporting capabilities are evaluated by appropriate tests on samples of compacted soil.



	UNCONSOLIDATED UNDRAINED TEST	CONSOLIDATED TESTS		
		CONSOLIDATION PHASE	CU	CD
σ_3	HELD CONSTANT	HELD CONSTANT	HELD CONSTANT	HELD CONSTANT
σ_1	GRADUALLY INCREASED FROM σ_3	EQUAL TO σ_3*	GRADUALLY INCREASED FROM σ_3	VERY GRADUALLY INCREASED FROM σ_3
u	DRAINAGE LINES CLOSED	DRAINAGE LINES OPEN **	NO WATER PERMITTED TO ESCAPE. PORE PRESSURE MEASURED FOR EFFECTIVE STRESS TESTS.	DRAINAGE LINES OPEN

* UNLESS ANISOTROPIC CONSOLIDATION IS TO BE EFFECTED
** IN BACK PRESSURED TESTS, PRESSURE IS SUPPLIED TO PORE LINES, BUT DRAINAGE IS PERMITTED

Figure 5

Triaxial Apparatus Schematic

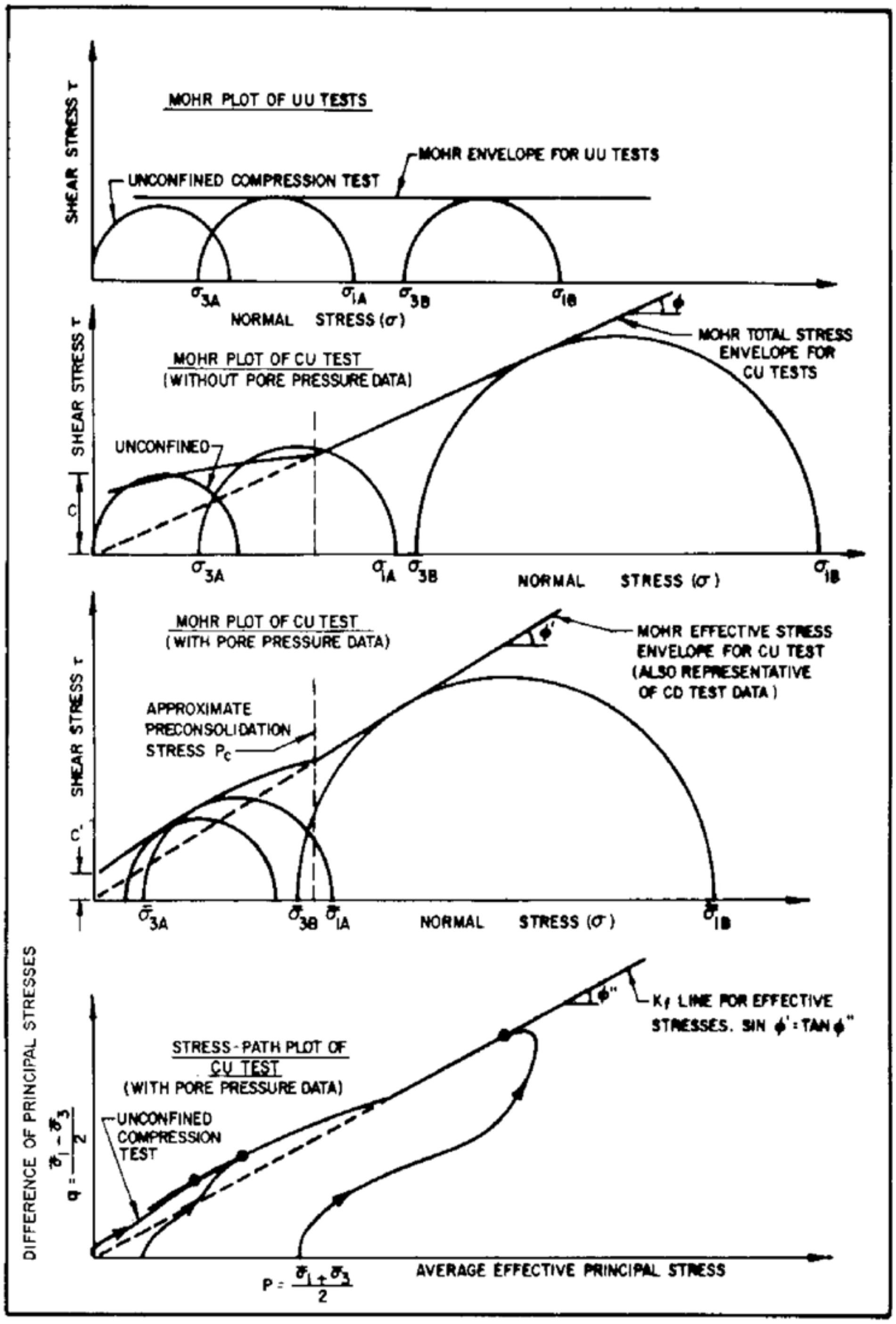


Figure 6

Triaxial Shear Test Relationships

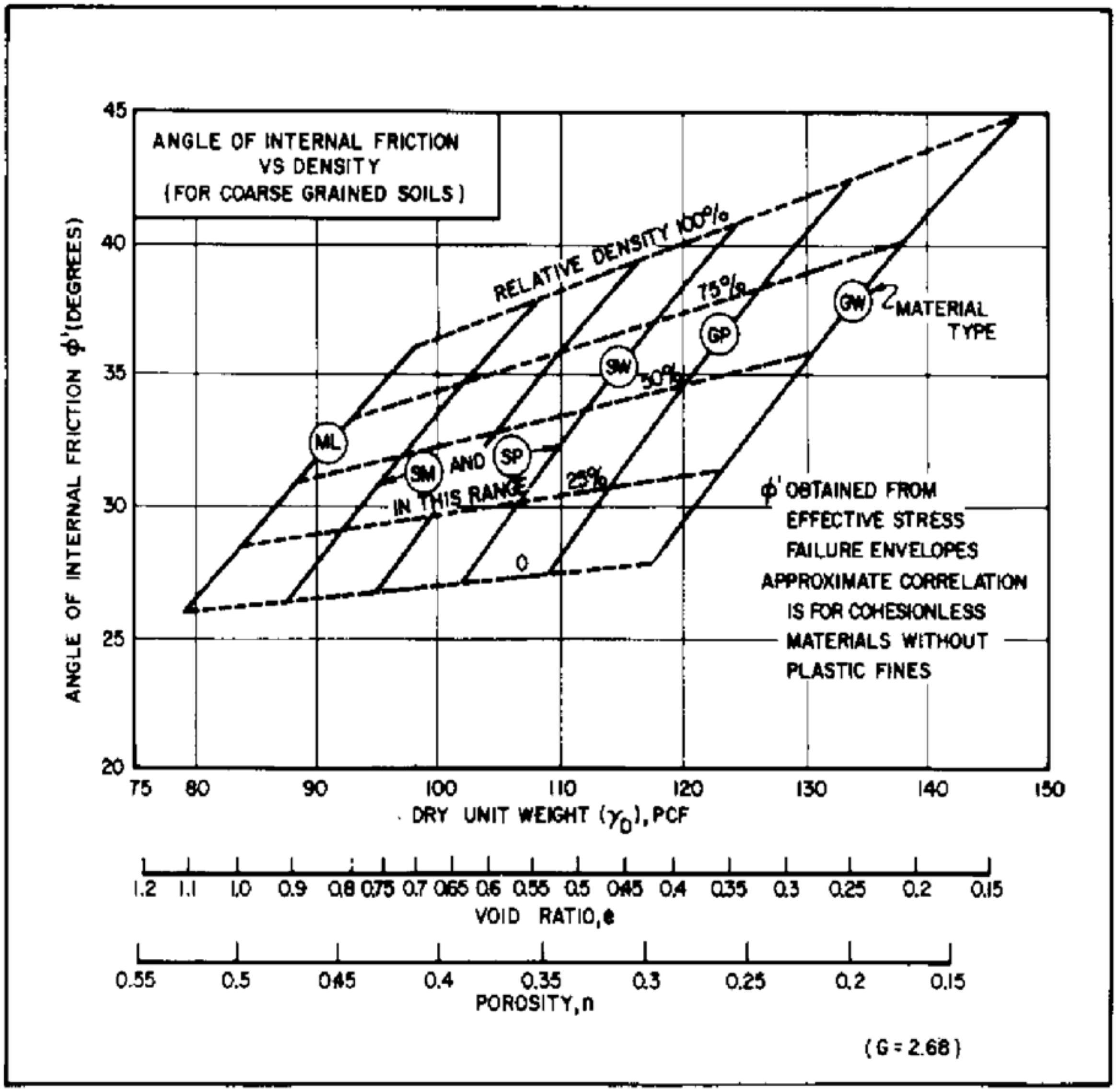


Figure 7

Correlations of Strength Characteristics for Granular Soils

7.2 MOISTURE-DENSITY RELATIONSHIPS. The Proctor test or a variation is employed in determining the moisture-density relationship. For cohesionless soils, Relative Density methods may be more appropriate.

7.2.1 STANDARD PROCTOR TEST. Use standard Proctor tests for ordinary embankment compaction control. In preparing for control, obtain a family of compaction curves representing principal borrow materials.

7.2.2 MODIFIED PROCTOR TEST. Especially applicable to either a heavily compacted base course or a subgrade for airfield pavement and may also be used for mass earthwork.

7.2.3 RELATIVE DENSITY OF COHESIONLESS SOILS. Proctor tests are often difficult to control for free-draining cohesionless soils and may give erratic compaction curves or density substantially less than those provided by ordinary compaction in the field (see Reference 28, Soil Mechanics, by Lambe and Whitman). Thus, relative density methods may be preferred. Tests for maximum and minimum densities should be done in accordance with ASTM Standard D2049, Relative Density of Cohesionless Soils (Table 3).

7.3 STRUCTURAL PROPERTIES. Structural properties of compacted-fill materials classified in the Unified System are listed in the literature.

7.4 CALIFORNIA BEARING RATIO (CBR). This test procedure covers the evaluation of subgrade, subbase, and base course materials for pavement design for highways and airfields. The resistance of a compacted soil to the gradual penetration of a cylindrical piston with 3 square inches in area is measured. The load required to cause either 0.1 inch or 0.2 inch penetration of the piston is compared to that established for a standard compacted crushed stone to obtain the bearing ratio. (See DM-21.03 for approximate relationships between soil type and CBR.) For guidance for design of subbase and bases, see DM-5.04 and DM-21.03.

8 TESTS ON ROCK

8.1 STRUCTURAL TESTS. Standard methods of testing rock in the laboratory for structural characteristics are only for intact rock. See Table 8 for testing procedures. Behavior of in situ rock, which typically has bedding planes, joints, etc., and may contain discontinuities filled with weaker material, is found to be very different from that of intact rock. In situ tests of joint strengths and compressibility are, therefore, more appropriate. The use of data from laboratory tests for bearing and settlement calculations of shallow and deep foundations is shown in DM-7.02 Chapters 4 and 5. Factors which correlate intact rock sample parameters to realistic field parameters are RQD (Rock Quality Designation) or the ratios of field values to laboratory values of compression or shear wave velocities.

8.2 ROCK QUALITY TESTS.

8.2.1 STANDARDS. Quality is normally evaluated by visual examination of the state of weathering and number and condition of discontinuities. RQD provides the best currently available basis for establishing overall rock quality. Relative measurements of rock quality can be made by comparing ratios of field values of compression or shear wave velocities to laboratory values.

8.2.2 AGGREGATE TESTS. While intended for roadway construction and asphalt and concrete aggregates, there are several standard tests which provide methods for measuring certain aspects of rock quality (see Table 9).

Table 8 Test Procedures for Intact Rock		
Test	Reference for Standard Procedure (a)	Size of Sample for Test
Unconfined compressive strength of core specimen	(1) ASTM D2938	Right circular cylinder with length to diameter ratio of 2 to 2.5, and a diameter not less than 2 inches
Elastic constants of core specimen	(1) ASTM D3148	Right circular cylinder with length to diameter ratio of 2 to 2.5
Direct tensile strength of intact rock core specimen	(1) ASTM D2936	Right circular cylinder with length to diameter ratio of 2 to 2.5
Triaxial strength of core specimen	(1) ASTM D2664	Right circular cylinder with length to diameter ratio of 2 to 2.5
Dynamic properties of core specimen at small strains	(1) ASTM D2845	Variable, dependent on properties of specimen and test apparatus
(a) Number in parenthesis indicates Reference number.		

Table 9 Test Procedures for Aggregate		
Test	Reference for Standard Procedure (a)	Applicability to Rock Cores
Weathering resistance	(1) ASTM C88	Applicable in principle, can be used directly by fracturing core
Visual evaluation of rock quality	(1) ASTM C295	Direct
Resistance to freezing	(1) ASTM C666	Applicable in principle, but only with significant procedure changes
Hardness	(1) ASTM C851	Direct
(a) Number in parenthesis indicates Reference number.		

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