CHAPTER 4.0
LABORATORY TESTING FOR FOUNDATION DESIGN

Laboratory testing is an important element in foundation engineering. The complexity of testing required for a particular project may range from a simple moisture content determination to specialized strength testing. However, testing can be expensive and time consuming. The foundation engineer should recognize the project problems to be solved so as to optimize testing; particularly strength and consolidation testing.

However, before describing various soil test methods, the behavior of soil under load will be examined and common soil mechanics terms introduced. The following discussion only includes basic concepts of soil deformation behavior and only deals with saturated soils. The engineer must grasp these concepts to understand why particular types of soil testing are necessary to solve particular highway problems. The terms and symbols shown will be used throughout this manual. Basic soils textbooks should be consulted for detailed explanation of terms.

A sample of soil may be composed of soil grains, water and air. The soil grains are irregularly shaped solids which are in contact with other adjacent soil grains. The weight and volume of a soil sample depends on the specific gravity of the soil grains (solids), the size of the area between soil grains (voids or pores) and the amount of void space filled with water. Common terms associated with weight-volume relationships are shown in Table 4 – 1. Of particular note is the void ratio (e) which is a general indicator of the relative strength and compressibility of the soil sample, i.e., low void ratios generally indicate strong, incompressible soils, high void ratios may indicate weak, compressible soils.

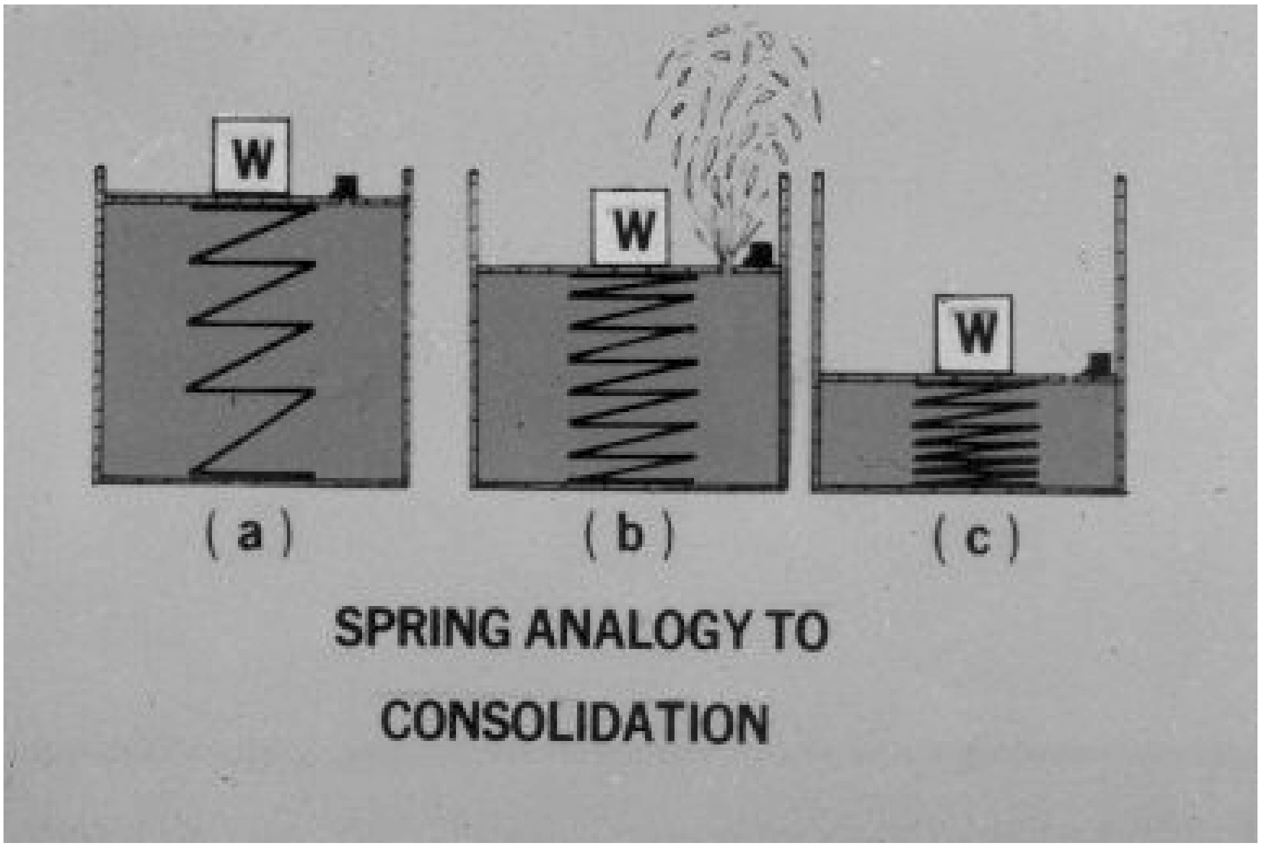
TABLE 4 – 1
WEIGHT VOLUME CHARACTERISTIC

Property	**Symbol**	**Units[1]**	**How To Obtained**	**Direct Applications**
Moisture content	W	D	Directly from test AASHTO T93	Classification and in volume-weight relations.
Unit weight	γ	FL^{-3}	Directly from test or from volume- weight relations AASHTO T38	Classification and for pressure computations.
Porosity	n	D	Computed from volume-weight relations	Parameters used to represent relative volume of solids to total volume of soil.
Void ratio	e	D	Computed from volume-weight relations.	
Specific gravity	G_S	D	Directly from AASHTO T100	Volume computations.

UNITS[1]: F=Force or weight; L=Length; T=Time; D=Dimensionless

When a load is applied to a soil sample, the deformation which occurs will depend on the grain to grain contacts (intergranular forces) and the amount of water in the voids (pore water). If no pore water exists, the sample deformation will be due to sliding between soil grains and deformation of individual, soil grains. Experience has shown that rearrangement of soil grains due to sliding accounts for the most deformation. Adequate deformation is required to increase the grain contact areas to take the applied load. As the amount of pore water in the void increases, the pressure exerted on soil grains will increase and reduce the intergranular contact forces. In fact, tiny clay particles may be forced completely apart by water in the pore space.

Deformation of a saturated soil is more complicated than dry soil as water molecules, which fill the voids, must be squeezed out of the sample before readjustment of soil grains can occur. The more permeable a soil is, the faster the deformation under load will occur. However, when the load on a saturated soil sample is quickly increased, the increase is carried entirely by the pore water until drainage begins. Then more and more load is gradually transferred to the soil grains until the excess pore pressure has dissipated and the soil grains readjust to a denser configuration. This process is called consolidation and results in a higher unit weight and a decreased void ratio.



4.1 PRINCIPLES OF EFFECTIVE STRESS

The consolidation process demonstrates the very important principle of effective stress, which will be used throughout this manual. Under an applied load, the total stress in a saturated soil sample is composed of the intergranular stress and pore pressure (neutral stress). As the pore water has no strength and is incompressible, only the intergranular stress is effective in resisting shear or limiting compression of the soil sample. Therefore, the intergranular contact stress is called the effective stress. When pore water drains from soil during consolidation, the area of contact between soil grains increases, which increases the level of effective stress. Stage construction of embankments is used to permit increase of effective stress in the foundation soil before subsequent fill load is added. In such operations effective stress increase is frequently monitored with piezometers to insure the next stage of embankment can be safely placed. Simply stated, the principle of effective stress states that the total stress on any plane within a soil mass is equal to the sum of the effective stress and the pore pressure.

In general, soil deposits below the water table will be considered saturated and the ambient pore pressure at any depth, may be computed by multiplying the unit weight of water by the height of water above that depth. The total stress at that depth may be found by multiplying the total unit weight of the soil by the depth. The effective stress is the total stress minus the pore pressure.

4.2 OVERBURDEN PRESSURE

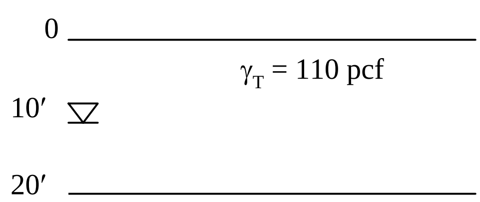
The laboratory testing required to solve soil-related problems involves simulating conditions naturally existing in the ground. Soils existing a distance below ground are affected by the weight of the soil above that depth. The influence of this weight, known generally as overburden pressure, causes a state of stress to exist which is unique at that depth, for that soil. When a soil sample is removed from the ground, that state of stress is relieved as all confinement of the sample has been removed. In testing, it is important to reestablish the in situ stress conditions and to study changes in soil properties when additional stresses representing the expected design loading are applied. As previously mentioned, the effective stress (grain to grain contact) is the controlling factor in shear and consolidation.

The test stresses are estimated from either the total or effective overburden pressure. The engineers' first task is determining the total and effective overburden pressure variation with depth. This relatively simple job involves determining the average total unit weight for each soil layer in the soil profile, and determining the depth of the water table. Unit weight may be accurately determined from density tests on undisturbed samples or estimated from standard penetration values and soil visuals. The water table depth, which is standardly recorded on boring logs, can be used to compute the pore pressure at any depth. The total overburden pressure (P_T) is found by multiplying the total unit weights of each soil layer by the layer thickness and continuously summing the results with depth. The effective overburden pressure (P_o) at any depth is determined by accumulating the weights of all layers above that depth with consideration of the water level conditions at the site as follows:

1. Soils above the water table - multiply the total unit weight by the thickness of each respective soil layer above the desired depth, ie, $P_o = P_T$.

2. Soils below the water table - subtract pore pressure (μ) from P_T or reduce the total unit weights by the weight of water (62.4 pcf), ie, use effective unit weights γ_b and multiply by the thickness of each respective soil layer between the water table and the desired depth $P_o = P_T - (\gamma_w \text{ x depth})$, or γ_b x depth.

Example 4-1: Find P_0 at 20 feet below ground in a sand deposit with a total unit weight of 110 pcf and the water table 10 feet below ground. Plot P_T and P_0 verses depth from 0′ – 20′.



Solution:

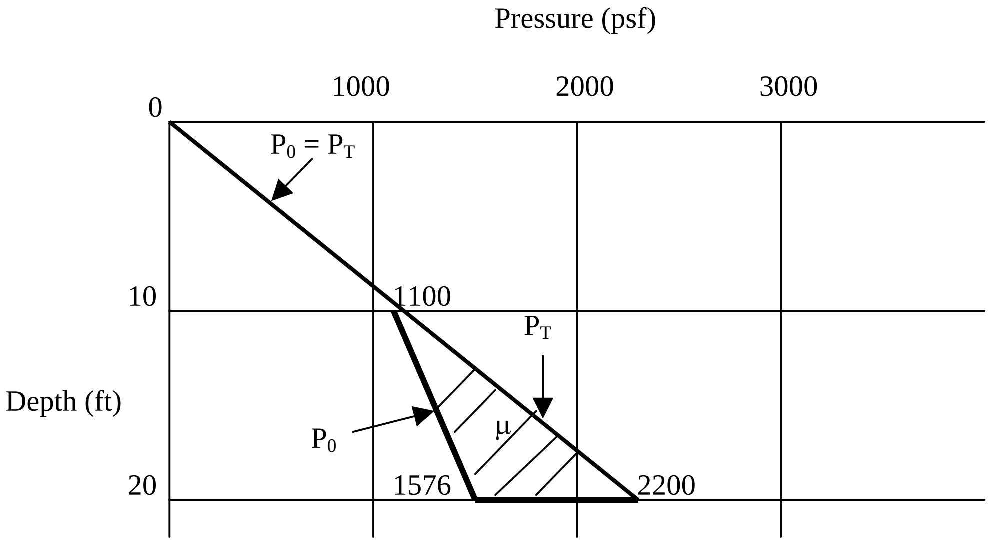
$$P_0 = P_T - \mu$$

$$P_T \text{ @ } 10' = P_0 \text{ @ } 10' = 10' \times 110 \text{ pcf} = 1100 \text{ psf}$$

$$P_T \text{ @ } 20' = P_T \text{ @ } 10' + (10' \times 110 \text{ pcf}) = 2200 \text{ psf}$$

$$\mu \text{ @ } 20' = 10' \times 62.4 \text{ pcf} = 624 \text{ psf}$$

$$P_0 \text{ @ } 20' = P_T \text{ @ } 20' - \mu \text{ @ } 20' = 2200 - 624 = 1576 \text{ psf}$$



Pressure Diagram

A PLOT OF EFFECTIVE OVERBURDEN PRESSURE VERSUS DEPTH IS CALLED A $\underline{P_0}$ <u>DIAGRAM</u> AND IS USED THROUGHOUT ALL ASPECTS OF FOUNDATION TESTING AND ANALYSIS.

4.3 USE OF ATTERBERG LIMITS

The following are the more important uses of Atterberg limits in determining engineering properties of soils:

1. Help identify and classify the soil.

2. PI (plasticity index) is an indicator of soil compressibility and potential for volume change. Estimate compression index (C_c) for normally consolidated and low sensitivity clay in preliminary design using:

$$C_c \cong 0.009 \, (LL-10)$$

3. PL (plastic limit) can indicate if clay has been preconsolidated. Most soils are deposited at or near their liquid limit. If the in situ natural water content (W) is near the plastic limit (PL), then the soil is probably preconsolidated. Some stress has been applied in the past to squeeze that water out.

4. Clay may also be assumed to be preconsolidated if the liquidity index (LI), which is the (moisture content minus plastic limit) divided by plastic index is less than 0.7.

   Atterberg limit formation from a specific site is frequently plotted on the "A-line" diagram to assess basic soil properties.

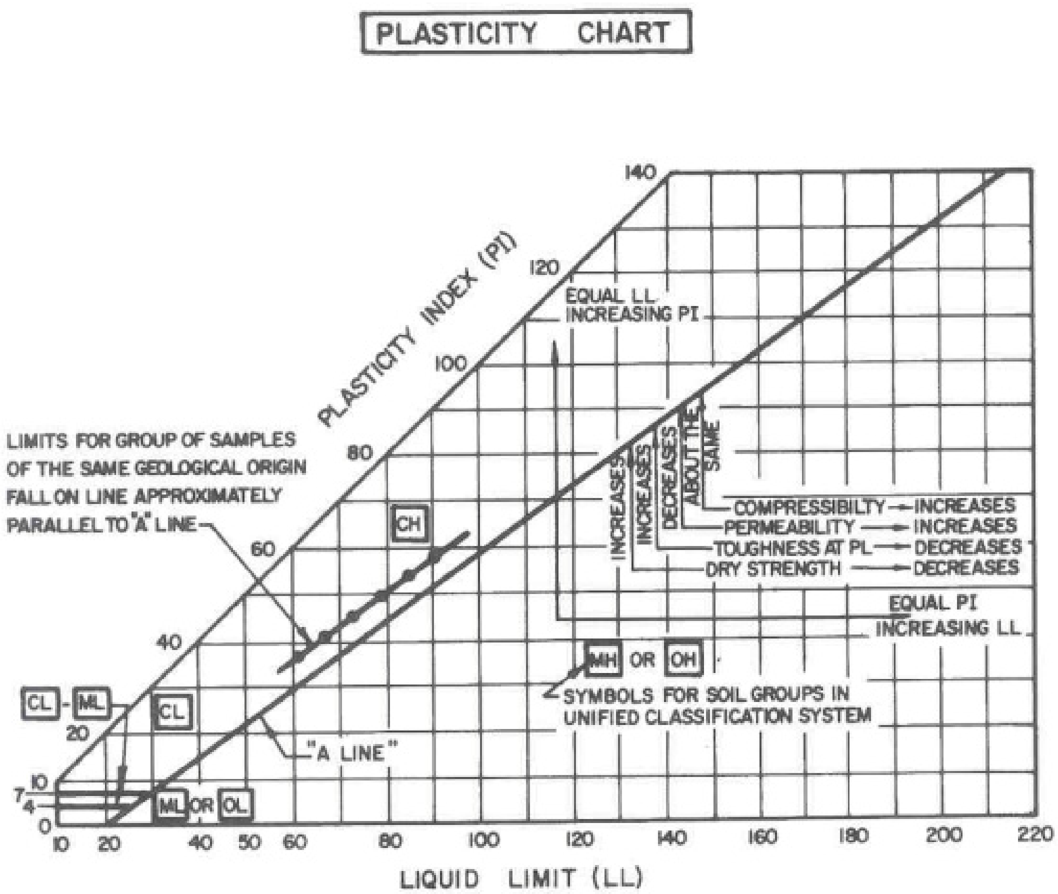


Figure 4-1 Plasticity (A-Line) Chart

The value of this simple procedure is great as noted by Arthur Casagrande in his "Discussion of Requirements for the Practice of Applied Soil Mechanics", in the first Pan American Conference on Soil Mechanics and foundation Engineering, September 1959.

> I consider it essential that an experienced soils engineer should be able to judge the position of soils, from his territory, on a plasticity chart merely on the basis of his visual and manual examination of the soils. And more than that, the plasticity chart should be for him like a map of the world. At least for certain areas of the chart, that are significant for his activities, he should be well familiar. The position of soils within these areas should quickly convey to him a picture of the significant engineering properties that he should expect.

4.4 EFFECTS OF TEMPERATURE EXTREMES

As soil samples contain a percentage of water, exposure to temperature extremes after the sample has been removed from the ground will produce permanent undesirable changes in the soil's engineering properties. The two primary causes of temperature extremes are poor transport and storage of samples prior to testing. Freezing and thawing cycles will destroy soil structure which results in reduced shear strength and increased compressibility test results. Heating will dry soil samples and result in artificially high strength and low compressibility test results. Undisturbed samples should be tested as soon as possible after extraction and only stored in temperature/humidity controlled areas.

The life of properly stored samples varies but recommended practical maximums are two months for sensitive soils (sensitivity > 4). Watch for telltale warning signs of sample shrinkage or oxidation when extruding older samples.

4.5 LABORATORY TESTING GUIDELINES

An experienced geotechnical engineer can only decide certain considerations regarding laboratory testing, such as when, how much, and what type. The following guidelines are presented.

1. Perform a laboratory visual identification on all soil samples extracted from the borings.

2. Perform a moisture content analysis on all samples (cohesionless samples may be excluded if the number of samples becomes great). Classification tests may be performed on selected samples as requested by the designer.

*3. Perform an adequate number of consolidation tests on cohesive soil samples to determine variation of preconsolidation pressure with depth. Estimate one test every 5 feet for the top 20 feet of a cohesive deposit and one test each 10 feet thereafter. Normally consolidated soils may be tested at intervals of 10 feet throughout.

*4. Perform shear strength tests in each definable soil deposit. Each cohesive deposit should have at least one 3-point consolidated undrained test with or without pore pressure measurements at 10 to 15-foot intervals. The 3-point tests should be consolidated at equal pressure increments between existing effective overburden pressure, and the final effective pressure developed under the embankment loading. Unconsolidated, undrained tests may be performed on remaining undisturbed samples at confining pressures above total overburden. All unconsolidated undrained tests should only be done on samples extruded directly from the sampling tube and tested untrimmed at full diameter.

* Note that these tests may be costly and time consuming. An experienced geotechnical engineer must schedule or review all testing requests before implementation. Laboratory testing is not required on many routine projects.

A list of common soil properties routinely used in design, which can be determined by laboratory testing, is presented in Table 4-3.

TABLE 4 – 2
COMMON SOIL PROPERTIES

Many other soil properties are determined by testing and routinely used in design. A list of some common properties is presented below.

PROPERTY	SYMBOL	DIM.	HOW OBTAINED	USE
Gradation characteristics Effective diameter...	D_{10}	L	AASHTO T88. From Grain-size curve	Classification, estimating permeability and unit weight, filter design, grout selection, & evaluating potential frost heave
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	Cz	D	$(D_{30})^2$ / (D_{10} x D_{60})	
Clay size fraction	…	D	From grain-size curve, % finer than 0.002 mm	Classification
Consolidation characteristics: Coefficient of compressibility	a_v	L^2 F^{-1}	Determined from arith. e vs. p curve	Computation of ultimate settlement or swell in consolidation analysis
Coefficient of volume Compressibility	m_v	L^2 F^{-1}	a_v / 1 + e	
Compression index	C_c	D	Determined from semilog e vs p curve. AASHTO T216	
Recompression index	C_r	D		
Swelling index	C_s	D		
Coefficient of secondary compression	C_α	D	Determined from semilog time-consolidation curve	
Coefficient of consolidation	C_v	L^2 T^{-1}		Computation of time rate of settlement
Preconsolidation pressure	P_c	FL^{-2}	Estimated from semilog e vs p curve	Consolidation analysis
Shear strength characteristics: Angle of internal friction	ϕ	D	Determined from Mohr envelope for total normal stress. AASHTO T234	Analysis of stability and load carrying capacity of foundations.
Cohesion intercept	c	FL^{-2}		
Angle of internal friction	ϕ'	D	Determined from Mohr envelope for effective normal stress	
Cohesion intercept	c'	FL^{-2}		
Unconfined compressive strength	q_u	FL^{-2}	Directly from test AASHTO T208	
Shear strength	s	FL^{-2}		
Sensitivity	S_t	D	q_u(undisturbed) ÷q_u(remolded)	Estimating effect of disturbance in driving piles
Modulus of elasticity	E_s	FL^{-2}	Determined from stress - strain curve	Computation of elastic settlement or rebound
Characteristics of compacted samples: Maximum dry unit weight	γ_{max}	FL^{-3}	Determined from moisture-density curve AASHTO T99 AASHTO T180	Compaction control and computation of weights and forces in stability analysis
Optimum moisture content	OMC	D		
Relative density	D_d	D	Determined from results of max and min density tests	Compaction control
California bearing ratio	CBR	D	Directly from test AASHTO T193	Pavement Design

Reference: NAVFAC DM-7 F=Force, L=Length, T=Time, D=Dimensionles

4.6 PROCESS OF CONSOLIDATION

Consolidation is a decrease in the volume of a soil due to static loading or vibrational forces applied to the soil mass. In highway design, static loading is represented by the permanent load placed on the soil by embankments and structures. As most compressible foundation soils are below the water table, all the voids are filled with water. An applied load will cause soil grains to readjust to a more compact position in order to carry the load. This readjustment cannot take place until the water, which is incompressible, escapes from the voids. In impervious soils which have small voids, water will travel very slowly.

Years may be needed for the water to drain away from the loaded soil area so the settlement can go to completion. The general consolidation characteristics of various major soil types are used to determine if a highway settlement problem may be anticipated. The following three groupings portray these characteristics:

1.	Gravels, sands, and non-plastic silts (granular soils)	Relatively incompressible. Will consolidate immediately under load when fill is placed. These soils do not present embankment settlement problems
2.	Plastic silt-clay mixtures (cohesive soils)	Soft silts and clays are more compressible than stiff silts and clays. Settlement may continue long after construction.
3.	Organic soils	Very compressible, and settlement of large magnitude will continue for years.

Consolidation occurs in three stages; initial, primary, and secondary. Initial (elastic) compression occurs simultaneously with application of load and is usually quite small. It is due primarily to compression of air and gas in the soil voids. Primary compression normally is the largest part of the total compression that will occur. Water has to be squeezed out of the soil voids for primary consolidation to take place. Therefore, the amount of primary consolidation will depend on the initial void ratio of the soil. The greater the initial void ratio, the more water that can be squeezed out, and the greater the primary consolidation. The rate at which primary consolidation occurs is dependent on the rate at which the water is squeezed out of the soil voids. Secondary compression (creep) occurs after primary consolidation is complete. Secondary compression is not dependent on water being squeezed out of the soil. Secondary compression can occur under constant load. It is caused by the soil particles reorienting or deforming under constant load.

Initial compression accounts for the major portion of consolidation in granular soils. Primary compression accounts for the major portion of consolidation in cohesive soils. Primary and secondary compression both contribute significantly to organic soil consolidation.

Some natural deposits of cohesive soils have undergone heavy compression in geologic history (due to the weight of glaciers, due to the weight of overlying soil that has been eroded off, or due to desiccation) and are therefore relatively incompressible Such soils are called preconsolidated or overconsolidated and have been subjected to greater stresses in the past than at present. This is important because these soils can be reloaded (such as by weight of an embankment or bridge footing) and will not settle appreciably until the reapplied load exceeds the preconsolidation load. Cohesive deposits, which have never been subjected to previous compression, are called normally consolidated. This means the soil has never been subjected to an overburden stress any greater than the stress existing at the present time.

4.6.1 Consolidation Testing

In order to predict the amount of consolidation in cohesive and organic soils, adequate testing must be performed. The undisturbed soil sample to be tested should be obtained in the field with a thin wall tube sampler. The designer should instruct the laboratory as to how many tests should be performed and modifications to standard procedures. The test request (Figure 4 – 2) should include the following information.

1. Clear designation of which samples are to be tested. Usually consolidation testing is done in close intervals (5') near the layer top and at wider intervals (10') at greater depths.

2. Loading time increments should be specified to optimize production. Minimal time increments may be used for adding test loads up to one load before P_o. Thereafter, three hour-increments may be used for soils with a moisture content less than 50 percent while twenty-four hour -increments are needed for highly organic soils and very plastic clays. Longer load durations may be needed in highly organic soils to define the coefficient of secondary compression accurately.

3. The load range where the coefficient of consolidation is to be computed should be specified. Generally, values are computed starting at the load below effective overburden pressure at the depth of the tube sample.

4. The recycle loads, (if needed for very accurate settlement prediction) should be specified to start at one load beyond the preconsolidation pressure and return to one load below effective overburden pressure before reloading to the requested maximum test load.

The consolidation results are generally presented graphically as shown in Figure 4 – 3.

The pressure-void ratio plot is used to find the preconsolidation pressure and other values pertaining to the compression of the soil sample. Both the arithmetic and semi log pressure-void ratio plots have been shown although the semi log plot is recommended and will be used in subsequent sections of this manual. On the semi log pressure-void ratio plot (e vs. log P), the engineer can readily see the sharp break in the curve at P_c which indicates compression will increase rapidly for additional increases in load beyond the preconsolidation pressure. The semi log time-compression curve (t_{50}) is used to find the secondary compression and the time rate of consolidation for the soil sample.

Some geotechnical engineers prefer to use a plot of percent strain versus log of pressure is used instead of the e vs. log P plot. In this case the interpreted values of compression and recompression indices reflect the relationship between strain and void ratio, i.e., strain = $\Delta e/(1+e_0)$. To convert the strain based indices to the void ratio based indices (C_c and C_r) multiply the strain based values by $1 + e_o$. Void ratio based values (e vs. log P) will be used in the remainder of this book.

SOIL MECHANICS LABORATORY TEST REQUEST

PROJECT ________ PROJ. NO. ________ HOLE NO. ________

DATE ________ REQUEST BY ________ APPROVED BY ________ STATION ________ OFFSET ________

ORIGINAL REQUEST ☐ SUPPLEMENTAL REQUEST ☐

CONSOLIDATION TESTS						STRENGTH TESTS											SPECIAL AND ADDITIONAL TESTS**		
SAMPLE NUMBER	MACHINE CAPACITY	1ST LOAD FOR C_V	LOAD TIME	RECYCLE	*STATUS	SAMPLE NUMBER	SAMPLE DIAM.		TYPE OF TEST AND CONSOLIDATION PRESSURE							*STATUS	% ORG.	% INERT	
							1.4"	OTHER	U	UU	CONF. PRESS (PSI)	CU	CONS. PRESS (PSI)						
													1	2	3				

ADDITIONAL REQUESTS:

* CHECK MARK WHEN TEST IS PUT IN PROGRESS ✓ ; CIRCLE WHEN COMPLETED ; CROSS HATCH WHEN COMPUTED

** TO SUPPLEMENT ROUTINE CLASSIFICATION TESTS ON CONSOLIDATION AND STRENGTH TEST SAMPLES.

Figure 4-2: Laboratory Test Request Form

Analyze consolidation test data to determine:

1. Preconsolidation Pressure (P_c)

   The maximum pressure to which a soil has been loaded in the past will have a major influence on the amount of settlement to be expected under a proposed loading. In fact, 10 times more settlement may occur in an unconsolidated soil than a preconsolidated soil for equal load increments. These preconsolidation values should be carefully established for the entire depth of the cohesive deposit under consideration. Normally, a maximum and minimum value of P_c will be established and plotted as a range with depth.

2. Compression Index (C_c) and Recompression Index (C_r)

   The slope of virgin compression and recompression portions of the e vs. log P curve is respectively C_c and C_r. In general, C_c is approximately 10 times greater than C_r. The point where lines drawn tangent to the slopes intersect is the minimum preconsolidation pressure. The C_c and C_r values are respectively estimated by dividing the soil moisture content by 100 and 1000. As their names imply, the values are a direct measure of soil compression.

3. Initial Void Ratio (e_o)

   The value of e_o is determined prior to application of load. The value e_o is used in settlement computations to determine settlement magnitude.

4. Coefficient of Consolidation (C_v)

   This parameter is an indicator of the rate of drainage during consolidation; or in the case of pile driving an indicator of the time required for remolded soil to gain strength and reconsolidate around the pile. The value may be determined by t_{50} (as previously shown) or the t_{90} (square root of time) method which is described in many soil textbooks. A plot of C_v versus log P will show a sharp decrease at the preconsolidation pressure.

5. Coefficient of Secondary Compression (C_α)

   Of great importance in organic materials, this value may account for the majority of strata consolidation. The C_α value is determined from the t_{50} semi log time-compression curve.

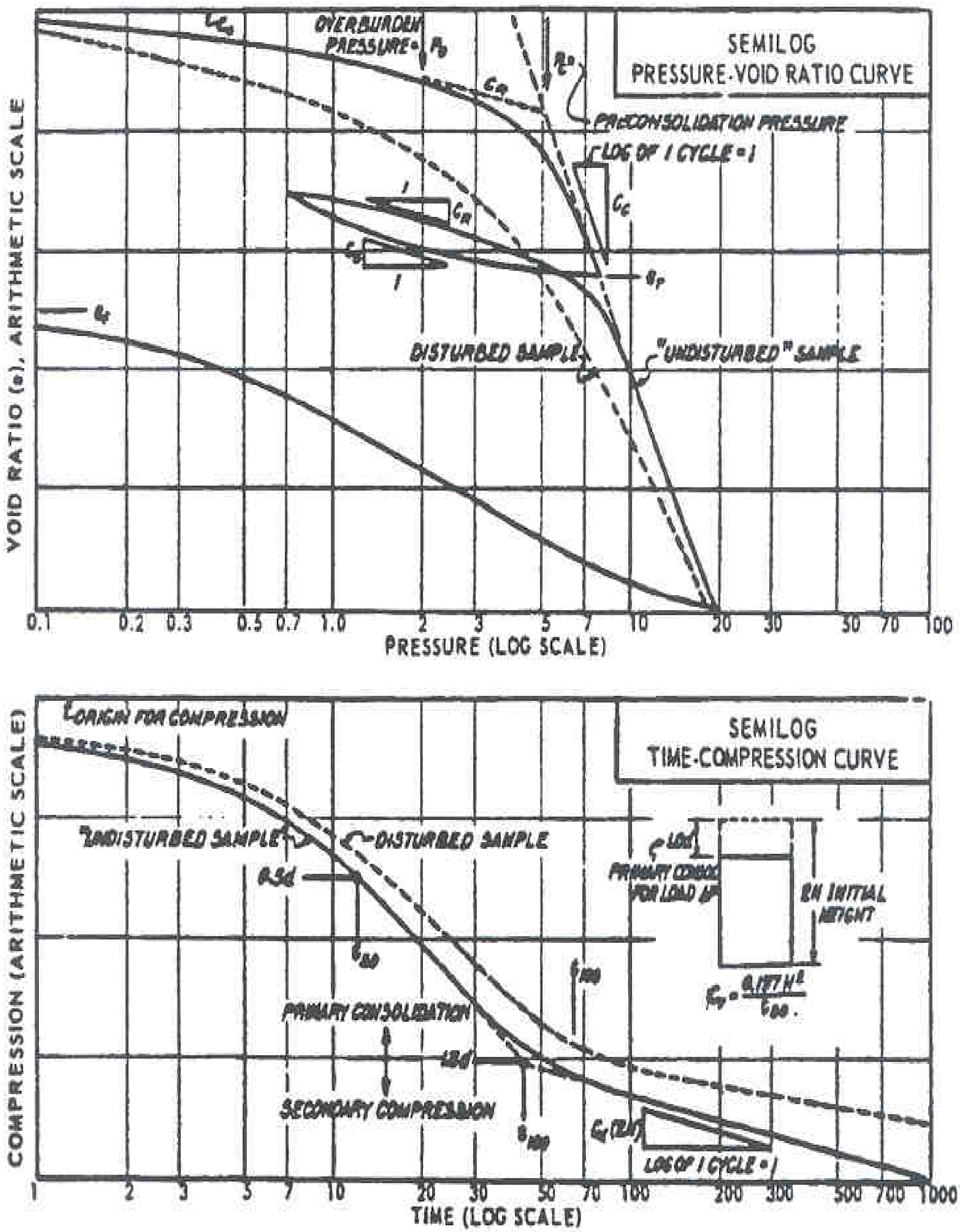
6. Effects of Sample Disturbance on Consolidation Test Results

   In previous pages the subject of undisturbed soil sampling was addressed. The importance of obtaining good quality samples was stressed. The influence of disturbance on consolidation test values is as follows and as shown on Figure 4 – 3.

   a. Eliminates the distinct break in the e vs. log P curve at the preconsolidation pressure.

   b. Lowers the estimate of preconsolidation pressure and the measured compression index.

   c. Decreases measured C_v values and eliminates the sharp break in e vs. log P at the preconsolidation pressure.



*Reference NAVFAC DM-7

Figure 4 – 3: Consolidation Test Relationships

d. Increases the recompression index.

e. Decreases the secondary compression coefficient.

The general effects of disturbance are under or over-prediction of the magnitude of expected settlement and over-prediction of the time for its occurrence.

The general importance of the consolidation test results applied to design are shown below. The test results may be applied to project design after a series of tests have been completed to represent the total depth of the soil deposit. The two most important predictions are:

1. The amount of settlement which may be determined by analyzing the test sample compression between the overburden pressure and the final pressure induced by the highway load at various depths. The compression may vary dramatically depending on the maximum past pressure to which the soil has been loaded.

2. The time for settlement may be estimated from the results of the compression versus time plots at loads between the overburden pressure and final pressure induced by the highway load. The important factors in the time-settlement relationship are:

   a. Time required is proportional to the square of the longest distance required for water to drain from the deposit. This distance is the thickness of the layer if all water drains only vertically to the surface, and one-half the layer thickness if more permeable soils also exist below the layer.

   b. Time required for consolidation varies inversely as the coefficient of consolidation.

   c. Rate of settlement decreases as time increases.

4.7 SOIL STRENGTH

The most important property of soils is strength. Slopes of all kinds, including hills, river banks, and man-made cuts and fills, stay in place only because of the strength of the material of which they are composed. Knowledge of soil strength is important for the design of structure foundations, embankments, retaining walls, pavements, and cuts.

Basic concepts indicate a soil can derive strength from two sources; friction between particles and cohesion between particles.

1. Cohesionless soils, such as gravel, sand, and silt, derive strength from friction between particles.

2. Cohesive soils, composed mainly of clay, derive strength from the attraction, or bond, between particles.

3. Mixtures of cohesionless and cohesive soils derive strength from both friction between particles and cohesion.

The frictional resistance between soil particles is dependent on the overburden pressure above the particles and the angle of internal friction between the particles. The total available shear strength (frictional resistance) is equal to the normal force times the tangent of ϕ (tangent of ϕ is equal to the coefficient of friction between the soil particles). The equation for frictional resistance is commonly written: $S = N \tan \phi$. A pile of dry sand will have a maximum angle of repose of about 30^o. This is approximately equal to the friction angle between soil particles.

The coefficient of friction between individual particles depends on both their mineral hardness and the surface roughness. However, the measured friction angle of a soil sample or deposit will also depend on the density of the mass caused by interlocking of particles. Angular particles, which interlock better than round particles, are specified for base courses and flexible pavement due to their higher strength. Care must be taken in estimating the coefficient of friction between dissimilar materials such as pile-soil systems as the interlock contribution is not mobilized.

The concept of cohesive strength is more difficult to explain as the cohesion is dependent on nebulous quantities such as the ionic bond between soil mineral grains. However, the practical aspects are easily understood in the relation to granular soils. Dry granular soils are unable to stand at slopes steeper than their angle of repose. However, clay deposits can be cut vertically to some limiting depth. Clay particles maintain their position in vertical slopes due to attractive forces between adjacent clay particles. This attractive force is commonly called the cohesion of the clay. The magnitude of the cohesion is dependent on the distance between individual clay mineral particles. The greater the separation, the lower the attractive force, and the smaller the cohesion;the closer the particles, the higher the cohesion. Separation of adjacent clay particles is maintained by water molecules which fill the void spaces between particles. As water is squeezed out due to external applied loads, separation decreases and cohesion increases. A unique relationship exists between the shear strength and water content of a clay. Great importance is associated with developing a plot of shear strength versus moisture content for major projects. Then moisture content variations can be used to assess shear strength variations in drill holes where undisturbed samples are not extracted.

The time required for water to be squeezed out from between soil particles varies generally with the size of the particles. The shear strength of granular soil increases immediately as the load increases. The strength of a pure cohesive soil increases very slowly after load is applied since consolidation is required for strength gain. Therefore, placement of highway embankments on cohesive soils must be controlled to prevent the applied load exceeding the initial soil cohesive strength.

For practical purposes most cohesive clay deposits contain some non-cohesive silt or sand. Hence under an increased load some increase in soil strength can be expected. The shear strength of any soil is commonly denoted as:

Shear Strength (S) = Cohesion (C) + Normal Force (N) x Tangent of Friction Angle (ϕ)

4.7.1 Strength Testing

The majority of strength tests are conducted on cohesive soils, as obtaining undisturbed samples of non-cohesive soils is difficult. Strength tests on cohesive soils are conducted on high quality undisturbed samples obtained from thin wall tubes. The number and type of test must be selected by the designer to suit the project conditions. For each test the designer should clearly indicate the consolidation or confining pressure to be used. These pressures are determined from the P_o diagram for each specific project. The range usually extends from the effective overburden pressure to the pressure induced by highway loading. The program objective should be to establish a profile of soil strength with depth. Soil strength parameters are frequently expressed as a ratio of shear strength over the effective overburden pressure (S/P_o).

The most common soil strength tests are as follows:

1. The Unconfined Compression Test is the simplest and quickest laboratory method used to measure the shear strength of a cohesive soil. Test results, especially with increasing depth, are conservative and misleading due to the release of confining stress when the sample is removed from below ground and tested.

2. The Triaxial Compression Test is a strength test where the sample is subjected to confining pressures similar to those which existed in the ground before sampling. In general triaxial tests may be done on soil samples which have either been consolidated in the lab to the effective overburden pressure before testing or left unconsolidated and tested at total overburden pressure. In either case, the tests try to produce the in situ effective stress condition. Unconsolidated tests must be done soon after sampling to insure no changes have occurred in the amount of pore water in the sample. The consolidated triaxial compression test duplicates as accurately as possible the sample's conditions in the ground and gives an accurate indication of shear strength. A series of tests on samples consolidated under various confining pressures may be run in order to determine the amount of strength increase with consolidation under embankment loads. The triaxial test procedure may be varied to account for short term (undrained) load application or long term (drained) load application.

3. The Vane Shear Test is a field test made in conjunction with drill hole explorations in soft clays. This is a test for determining shear strength rapidly without laboratory testing. A post-test soil sample should be extracted from the test depth to permit correlation of strength with other physical properties. The vane test strength results are accurate in soft silts and clays. Miniature laboratory vane tests are not as dependable due to sample confinement in the tube, size effects, and sampling disturbance.

4. The Direct Shear Test is a relatively simple test used to measure the shear strength of fine granular soils. This test is not recommended for silts and clays as test sample drainage cannot be controlled during the test. Sophisticated direct simple shear testing is appropriate for fine grained soils but the necessary equipment is only available at a few specialized soil labs.

4.7.2 Discussion of Shear Strength Testing

The shear strength of a soil is the maximum shear stress that the soil structure can resist before failure. Shear stresses are carried by the structure of soil grains as the water filling the pores has no shear strength. However, the shear strength of the soil structure is indirectly dependent on the pressure in the pore water which influences the N term in $S = C + N \tan \phi$. Foundation designers must consider the effects of expected construction operations on the subsoils when planning a test program. For example, when a highway embankment or structure footing is suddenly placed on a soft clay deposit, the pore water initially carries all the load and the available shear strength does not increase until drainage begins and the pore pressure decreases. In planning a test program for such a situation the designer would request unconsolidated undrained triaxial tests to determine the critical strength values, i.e., the initial shear strength before consolidation begins. Additional consolidated undrained or drained tests would also be used to determine the increase in shear strength as consolidation occurs and pore pressures dissipate. These results can be used to determine alternate methods of safely applying the loads, especially if the critical unconsolidated undrained strength is insufficient to sustain the proposed loading. Stage construction involves placement of an increment of load and a waiting period to allow strength gain so the soil deposit can safely support the next load increment.

4.7.3 Strength Test Results

Strength testing results are generally reported either as a shear strength (S) or in terms of cohesion (C) and friction angle (ϕ). Certain tests produce results that are limited in application. The following summary is generally applicable to tests on saturated cohesive soils unless otherwise stated.

1. Unconfined Compression (U)

   This test is widely used as a quick economical means of obtaining the approximately in situ shear strength of cohesive soils at shallow depths. The test results are presented in the form of a stress-strain plot where the shear strength is computed to be the maximum compressive stress divided by two. In cohesive samples this shear strength should approximately equal the cohesion as the test is performed at atmospheric pressure, i.e., N equals 0 in $S = C + N \tan \phi$. The reliability of this test is particularly poor with increasing sample depth (below about 30') because the sample tends to swell after removal from tube. Swelling causes greater particle separation and reduced shear strength. Swelling can be minimized by testing as soon as possible after removal from the tube and at full diameter. This reduces disturbance and preserves natural moisture content.

2. Unconsolidated Undrained (UU) Triaxial

   This test is also dependent on the soil sample retaining its original structure until testing occurs. All UU tests should only be done on samples extruded directly from the sampling tube and tested untrimmed at full diameter. The results are the shear strength existing at that depth. Theoretically, the test confining pressure may be varied substantially above the usually applied total overburden pressure without changing the test results as all increases in N are carried by the pore water. Practically, slight increases in shear strength will be noted due to sample drying (nonsaturation) and disturbance of original structure. These increases should not be interpreted as shear strength gain with increasing load. The UU shear strength may be used in quick loading situations such as rapid construction of a highway embankment where all the load is applied before the deposit can consolidate and gain strength.

3. Consolidated Undrained (CU) Triaxial

   This test is generally performed on sets of three soil samples taken from the same tube. Each sample is consolidated to a different effective stress. The shear strength of each sample is determined and plotted on a Mohr diagram. The result is a line (called an envelope) which intercepts the Y-axis at the cohesion value and has an inclination measured from the X-axis equal to the friction angle. The undrained shear strength values may be estimated from the envelope for any loading within the range of test pressures.

4. Consolidated Drained (CD) Triaxial

   This test is interpreted similar to the CU test except the envelope will usually have a smaller cohesion (typical value of <100 psf) and a larger friction angle. The results are used to duplicate long term loadings when excess pore pressures do not develop. CU tests with pore pressure measurements are used in place of CD tests due to savings in testing time.

5. Direct Shear (DS)

   This test is suitable for granular soils (and clays if proper equipment is used). Stress-strain curves are usually produced for at least three soil samples, each at a different test pressure. The stress-strain measurements are usually extended substantially beyond the peak to determine the residual shear stress, i.e., the stress which remains constant with increasing strain. The peak and residual shear strengths are plotted versus confining pressure to determine rate of increase of strength with applied load.

4.7.4 Comparison of Laboratory and Field Strengths

Laboratory soil samples are obtained from the ground by sampling from boreholes and sealing and transporting these samples to the laboratory. The degree of disturbance affecting the samples will vary according to the type of soil, sampling method and the skill of the driller. At best some disturbance will occur from the removal of in situ stresses during sampling and laboratory preparation for testing. In general, disturbance tends to reduce the shear strength obtained from unconfined or unconsolidated tests and increase the strength obtained from consolidated tests. There is, therefore, considerable attraction for measuring shear strength in the field, in situ. The vane shear test is the most commonly used field test for obtaining shear strength in soft to medium clays. As the test is performed rapidly, the strength measured is indicative of the undrained shear strength. In reviewing different types of field and lab testing in clays to determine the undrained-shear strength, the designer should expect the vane shear test to provide the most accurate value with U and UU tests yielding lower results and CU tests yielding slightly higher results.

4.7.5 Selection of Design Shear Strength

Frequently, on a large project the designer will receive a huge quantity of undrained shear strength test results from both the field and lab. This mountain of data must be concisely summarized to permit rational interpretation of results. The tests should be analyzed on a hole-by-hole basis. All tests from one hole should be reviewed and the existing undrained shear strengths selected. The results for each type of test should be plotted versus depth to determine the pattern of strength variation for each test type with depth and to assess the reliability of the data, i.e., a CU test result that is lower than the U test result at the same depth should be considered suspect. The general pattern of in situ shear strength results should be to increase with depth in a normally consolidated clay deposit. Clays which have been overconsolidated may only exhibit this increase at greater depths as the amount of preconsolidation increases shear strength in upper portions of the soil deposit.

4.8 PRACTICAL ASPECTS FOR LABORATORY TESTING

A poor understanding sometimes exists among geologists, structural engineers, and some foundation engineers about the type and amount of laboratory testing required for a structure foundation design. This weakness may render subsequent foundation design analyses useless. Organizations which have neither the proper testing facilities nor trained soils laboratory personnel may contract testing to private consultants. This solution can only be effective if the organization's foundation engineer can confidently request the necessary testing and review the results to select design values. A fair estimate of consultant testing costs may be obtained by assuming the following number of man-days (md) per test and multiplying by current costs; visual description of an SPT sample including moisture content (0.05 md), visual description of a tube sample including moisture content and unit weight (0.1 md), classification tests (0.7 md), undrained triaxial tests (0.9 md), drained triaxial tests (2.0 md), consolidation tests (2.0 md). These values include all work required to present a completed test result to the foundation designer.

Blanket consultant contracts "to perform testing necessary for design" usually result in unnecessarily large quantities of testing being performed, much of which does not apply to the project foundation problems. For example, if your multi-span structure is crossing a soft clay deposit underlain by sands, do not spend inordinate amounts of time and money to determine all strength and consolidation parameters of the soft clay layer at pier locations. Realize that the pile foundation will be designed using SPT values found in the underlying granular soils and that the only possible laboratory testing needed in the soft clay layer may be to estimate drag forces on the abutment piles. Also do not permit non-standard strength testing such as torvanes, penetrometers, etc. which are not covered by ASTM or AASHTO standards. Such devices should only be used as field index tests for consistency determination.

4.9 APPLE FREEWAY DESIGN EXAMPLE – LABORATORY TESTING

In this chapter the Apple Freeway Design Example is used to demonstrate the preparation of a P_0 diagram to prepare a laboratory test request for consolidation and strength tests. Typical consolidation and strength tests results are included at the end of the chapter.

Given: Preliminary soil profile and soil unit weights (determined in chapter 3)

Required: Prepare test request for consolidation and strength testing

Solution:

Step 1: Construct P_0 diagram at boring UDH BAF – 4. The boring where the samples for strength and consolidation tests were obtained

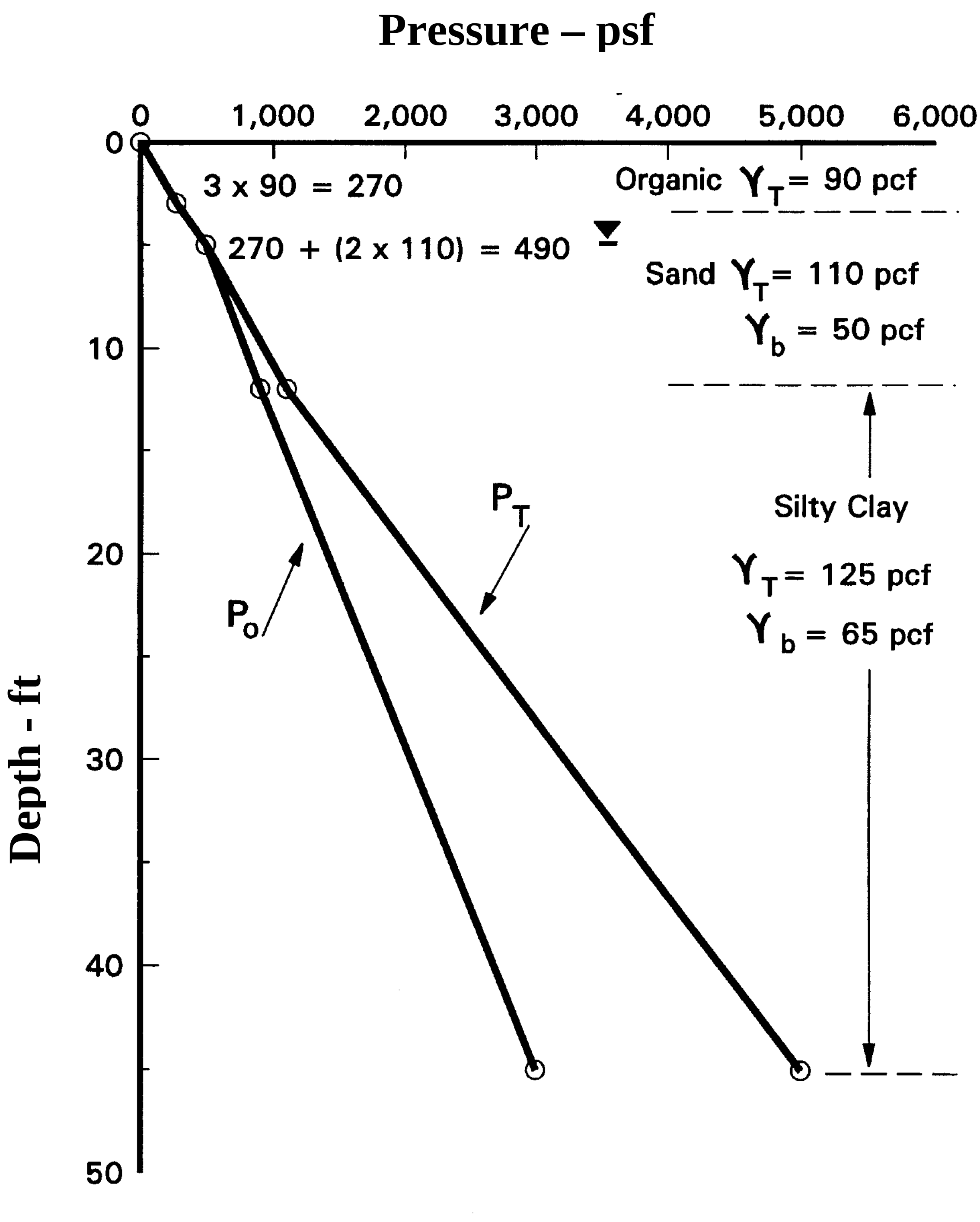
Step 2: Based on the pressure at each depth (P_0) specify loads, test duration and loading pattern for consolidation test and the confining and consolidation pressure for the UU and Cu tests.

Step 3: Use laboratory test results to obtain consolidation and strength parameters for design.

Site Exploration	Terrain Reconnaissance Site Inspection Subsurface Borings	
Basic Soil Properties	Visual Description Classification Tests Soil Profile	
→	**Laboratory Testing**	P_o Diagram Test Request Consolidation Results Strength Results
Slope Stability	Design Soil Profile Circular Arc Analysis Sliding Block Analysis Lateral Squeeze	
Embankment Settlement	Design Soil Profile Settlement Time – Rate Surcharge Vertical Drains	
Spread Footing Design	Design Soil Profile Pier Bearing Capacity Pier Settlement Abutment Settlement Vertical Drains Surcharge	
Pile Design	Design Soil Profile Static Analysis – Pier Pipe Pile H – Pile Static Analysis – abutment Pipe Pile H – Pile Driving Resistance Abutment Lateral Movement	
Construction Monitoring	Wave Equation Hammer Approval Embankment Instrumentation	

Apple Freeway Design Example – Laboratory Testing
Exhibit A

Pressure Diagram (P_0 & P_T)



Soil Mechanics Laboratory Test Request

SOIL MECHANICS LABORATORY TEST REQUEST

PROJECT I-O APPLE FREEWAY PROJ. NO. ______ HOLE NO. BAF-4
DATE 6-3-82 REQUEST BY ______ APPROVED BY ______ STATION 93+27 OFFSET 50' LT
ORIGINAL REQUEST ☒ SUPPLEMENTAL REQUEST ☐

CONSOLIDATION TESTS

SAMPLE NUMBER	MACHINE CAPACITY	1st LOAD FOR C_v	LOAD TIME	RECYCLE	* STATUS
T3	32K	500	3 hr.	YES	
T4		1000		YES	
T5				NO	
T6				YES	
T7				NO	
T9				NO	

STRENGTH TESTS

SAMPLE NUMBER	SAMPLE DIAM. 1.4"	SAMPLE DIAM. OTHER	U	UU *	CONF. PRESS (PSI)	CU	CONS. PRESS (PSI) 1 **	2	3 ***	* STATUS
T4		FULL		✓	12					
T4	✓					✓	8	21	36	
T5		FULL		✓	17					
T5	✓					✓	10	24	38	
T6		FULL		✓	21					
T6	✓					✓	12	26	40	
T7		FULL		✓	26					
T7	✓					✓	15	29	43	
T9		FULL		✓	33					
T9	✓					✓	19	33	47	
				* P_{TOTAL}			** P_o		*** P_f	

SPECIAL AND ADDITIONAL TESTS**

% ORG.	% INERT	

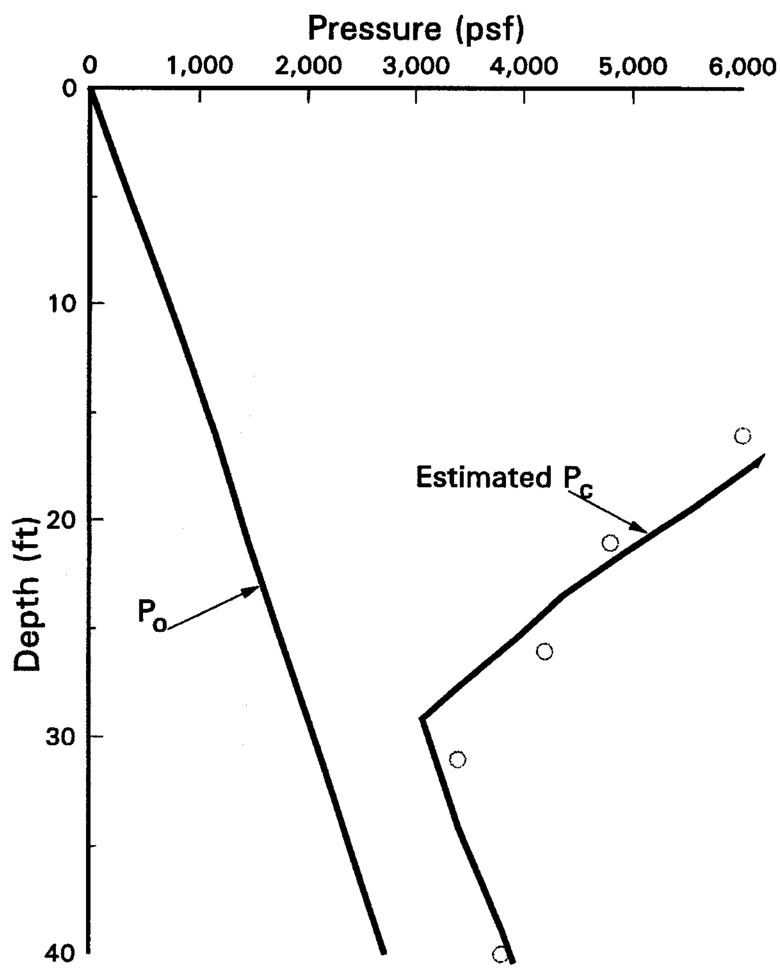
ADDITIONAL REQUESTS: CONSOLIDATION TESTS - SPECIFIC GRAVITY ON ALL TESTS
STRENGTH TESTS - ATTERBURG LIMITS AND HYDROMETER ON ALL TESTS

* CHECK MARK WHEN TEST IS PUT IN PROGRESS ✓ ; CIRCLE WHEN COMPLETED (✓) ; CROSS HATCH WHEN COMPUTED
** TO SUPPLEMENT ROUTINE CLASSIFICATION TESTS ON CONSOLIDATION AND STRENGTH TEST SAMPLES.

CONSOLIDATION TEST RESULTS SUMMARY

Hole UDH BAF-4

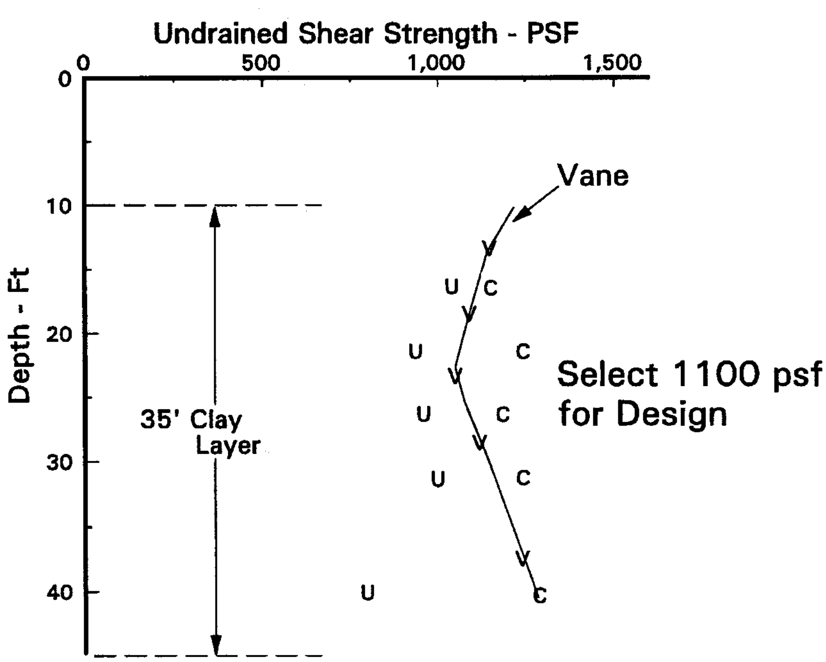
Depth Ft.	Tube No.	w %	P_o, psf	e_o	P_c, psf	C_r	C_c	c_v
11	T3	33	800	0.91	6500	0.033	0.35	0.6
16	T4	35	1150	0.89	6000	0.031	0.32	0.4
21	T5	31	1450	0.96	4800	0.040	0.36	0.8
26	T6	36	1790	1.01	4200	0.035	0.34	0.6
31	T7	38	2130	0.98	3400	0.037	0.34	0.8
40	T9	37	2720	1.02	3800	0.032	0.35	0.4



SHEAR STRENGTH TEST RESULTS SUMMARY

Hole UDH BAF-4

			Undrained Strength – psf			
Depth Ft.	**Tube No.**	**w %**	**U_u (U)**	**C_u @ P_o (C)**	**Vane (V)**	
					Undisturbed	**Remolded**
13		34			1150	550
16	T4	34	1050	1150		
18		36			1100	600
21	T5	35	950	1250		
23		38			1050	500
26	T6	39	975	1200		
28		37			1125	550
31	T7	40	1000	1250		
37		35			1250	600
40	T9	38	800	1300		



Summary of the Laboratory Testing Phase for Apple Freeway Design Problem

- Construct: P_0 Diagram

  Increase of pressure in the soil with depth.

- Prepare: Test Request

  Test pressures represent range of increase due to the embankment.

- Consolidation Results

  Compressibility, precompression and drainage rate of clay deposit.

- Strength Results

  Cohesion and increase of shear strength with confining pressure found.