CHAPTER 6

ENGINEERING PROPERTIES OF SOIL AND ROCK
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6.1 OVERVIEW

The purpose of this chapter is to identify, either by reference or explicitly herein, appropriate methods of soil and rock property assessment, and how to use that soil and rock property data to establish the final soil and rock parameters to be used for geotechnical design. The final properties to be used for design should be based on the results from the field investigation, the field testing, and the laboratory testing, used separately or in conjunction. Site performance data should also be used if available to help determine the final geotechnical properties for design. The Department Geotechnical Engineer’s responsibility is to determine which parameters are critical to the design of the project and then determine those parameters to an acceptable level of accuracy. See NYSDOT GDM Chapter 2, and the individual chapters that cover each geotechnical design subject area, for further information on what information to obtain and how to plan for obtaining that information.

The focus of geotechnical design property assessment and final selection shall be on the individual geologic strata identified at the project site. A geologic stratum is characterized as having the same geologic depositional history and stress history, and generally has similarities throughout the stratum in terms of density, source material, stress history, and hydrogeology. It should be recognized that the properties of a given geologic stratum at a project site are likely to vary significantly from point to point within the stratum. In some cases, a measured property value may be closer in magnitude to the measured property value in an adjacent geologic stratum than to the measured properties at another point within the same stratum. However, soil and rock properties for design should not be averaged across multiple strata. It should also be recognized that some properties (e.g., undrained shear strength in normally consolidated clays) may vary as a predictable function of a stratum dimension (e.g., depth below the top of the stratum). Where the property within the stratum varies in this manner, the design parameters should be developed taking this variation into account, which may result in multiple values of the property within the stratum as a function of a stratum dimension such as depth.

6.2 INFLUENCE OF EXISTING AND FUTURE CONDITIONS ON SOIL AND ROCK PROPERTIES

Many soil properties used for design are not intrinsic to the soil type, but vary depending on conditions. In-situ stresses, the presence of water, rate and direction of loading can all affect the behavior of soils. Prior to evaluating the properties of a given soil, it is important to determine the existing conditions as well as how conditions may change over the life of the project. Future construction such as new embankments may place new surcharge loads on the soil profile or the groundwater table could be raised or lowered. Often it is necessary to determine how subsurface conditions or even the materials themselves will change over the design life of the project. Normally consolidated clays can gain strength with increases in effective stress and overconsolidated clays may lose strength with time when exposed in cuts. Some construction materials, such as weak rock, may lose strength due to weathering within the design life of the embankment.
6.3 METHODS OF DETERMINING SOIL AND ROCK PROPERTIES

Subsurface soil or rock properties are generally determined using one or more of the following methods:

- in-situ testing during the field exploration program,
- laboratory testing, and
- back analysis based on site performance data.

In-situ test methods are discussed in NYSDOT GDM Chapter 4. The two most common in-situ test methods for use in soil are the Standard Penetration Test, (SPT) and the cone penetrometer test (CPT). The laboratory testing program generally consists of index tests to obtain general information or to use with correlations to estimate design properties, and performance tests to directly measure specific engineering properties. The observational method, or use of back analysis, to determine engineering properties of soil or rock is often used with slope failures, embankment settlement or excessive settlement of existing structures. With landslides or slope failures, the process generally starts with determining the geometry of the failure and then determining the soil/rock parameters or subsurface conditions that cause the safety factor to approach 1.0. Often the determination of the properties is aided by correlations with index tests or experience on other projects. For embankment settlement, a range of soil properties is generally determined based on laboratory performance testing on undisturbed samples. Monitoring of fill settlement and pore pressure in the soil during construction allows the soil properties and prediction of the rate of future settlement to be refined. For structures such as bridges that experience unacceptable settlement or retaining walls that have excessive deflection, the engineering properties of the soils can sometimes be determined if the magnitudes of the loads are known. As with slope stability analysis, the geometry of the subsurface soil must be adequately known, including the history of the groundwater level at the site.

The detailed measurement and interpretation of soil and rock properties shall be consistent with the guidelines provided in FHWA-IF-02-034, Evaluation of Soil and Rock Properties, Geotechnical Engineering Circular No. 5 (Sabatini, et al., 2002), except as specifically indicated herein.

6.4 GEOTECHNICAL LABORATORY SERVICES

Laboratory testing is a fundamental element of a geotechnical investigation. The ultimate purpose of laboratory testing is to utilize repeatable procedures to refine the visual observations and field testing conducted as part of the subsurface field exploration program, and to determine how the soil or rock will perform under the imposed conditions. The ideal laboratory program will provide sufficient data to complete an economical design without incurring excessive tests and costs. Depending on the project issues, testing may range from simple soil classification testing to complex strength and deformation testing.
6.4.1 Types of Laboratory Testing

Laboratory testing of samples recovered during subsurface investigations is the most common technique to obtain values of the engineering properties necessary for design. A laboratory testing program consists of “index tests” to obtain general information on categorizing materials, and “performance tests” to measure specific properties that characterize soil behavior for design and constructability assessments (e.g., shear strength, compressibility, hydraulic conductivity, etc.).

Table 6-1 provides a listing of commonly-performed soil laboratory tests. Tables 6-2 and 6-3 provide a summary of typical soil index and performance tests, respectively.
### Table 6-1 Commonly Performed Laboratory Tests on Soils

(Samtani and Nowatzki, 2006)

<table>
<thead>
<tr>
<th>Test Category</th>
<th>Name of Test</th>
<th>Test Designation</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Practice for Description and Identification of Soils (Visual-Manual Procedure)</td>
<td>AASHTO</td>
</tr>
<tr>
<td>Index Properties</td>
<td>Test Method for Determination of Water (Moisture) Content of Soil by Direct Heating Method</td>
<td>T 265</td>
</tr>
<tr>
<td></td>
<td>Test Method for Specific Gravity of Soils</td>
<td>T 100</td>
</tr>
<tr>
<td></td>
<td>Method for Particle-Size Analysis of Soils</td>
<td>T 88</td>
</tr>
<tr>
<td></td>
<td>Test Method for Classification of Soils for Engineering Purposes</td>
<td>M 143</td>
</tr>
<tr>
<td></td>
<td>Test Method for Amount of Material in Soils Finer than the No. 200 (0.075 mm) Sieve</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils</td>
<td>T 89; T 90</td>
</tr>
<tr>
<td>Compaction</td>
<td>Test Method for Laboratory Compaction Characteristics of Soil Using Standard Effort (12,375 ft. lbs./ft$^3$)</td>
<td>T 99</td>
</tr>
<tr>
<td></td>
<td>Test Method for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,250 ft. lbs./ft$^3$)</td>
<td>T 180</td>
</tr>
<tr>
<td>Strength Properties</td>
<td>Test Method for Unconfined Compressive Strength of Cohesive Soil</td>
<td>T 208</td>
</tr>
<tr>
<td></td>
<td>Test Method for Unconsolidated, Undrained Compressive Strength of Cohesive Soils in Triaxial Compression</td>
<td>T 296</td>
</tr>
<tr>
<td></td>
<td>Test Method for Consolidated, Undrained Compressive Strength of Cohesive Soils in Triaxial Compression</td>
<td>T 297</td>
</tr>
<tr>
<td></td>
<td>Method for Direct Shear Test of Soils under Consolidated Drained Conditions</td>
<td>T 236</td>
</tr>
<tr>
<td></td>
<td>Test Methods for Modulus and Diameter of Soils by the Resonant-Column Method</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Test Method for Laboratory Miniature Vane Shear Test for Saturated Fine-Grained Clayey Soil</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Test Method for CBR (California Bearing Ratio) of Laboratory-Compacted Soils</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Test Method for Resilient Modulus of Soils</td>
<td>T 294</td>
</tr>
<tr>
<td></td>
<td>Test Method for Resistance R-Value and Expansion Pressure of Compacted Soils</td>
<td>T 190</td>
</tr>
<tr>
<td>Consolidation, Swelling, Collapse Properties</td>
<td>Test Method for One-Dimensional Consolidation Properties of Soils</td>
<td>T 216</td>
</tr>
<tr>
<td></td>
<td>Test Method for One-Dimensional Consolidation Properties of Soils Using Controlled-Strain Loading</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Test Methods for One-Dimensional Swell or Settlement Potential of Cohesive Soils</td>
<td>T 258</td>
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<tr>
<td></td>
<td>Test Method for Measurement of Collapse Potential of Soils</td>
<td>-</td>
</tr>
<tr>
<td>Permeability</td>
<td>Test Method for Permeability of Granular Soils (Constant Head)</td>
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<td></td>
<td>Test Method for Measurement of Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Permeameter</td>
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</tr>
<tr>
<td>Corrosivity (Electrochemical)</td>
<td>Test Method for pH for Peat Materials</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Test Method for pH of Soils</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Test Method for pH of Soil for Use in Corrosion Testing</td>
<td>T 289</td>
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<tr>
<td></td>
<td>Test Method for Sulfate Content</td>
<td>T 290</td>
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<tr>
<td></td>
<td>Test Method for Resistivity</td>
<td>T 288</td>
</tr>
<tr>
<td></td>
<td>Test Method for Chloride Content</td>
<td>T 291</td>
</tr>
<tr>
<td>Organic Content</td>
<td>Test Methods for Moisture, Ash, and Organic Matter of Peat and Other Organic Soils</td>
<td>T 194</td>
</tr>
</tbody>
</table>
## Table 6-2 Methods for Index Testing of Soils (Samtani and Nowatzki, 2006)

<table>
<thead>
<tr>
<th>Test</th>
<th>Procedure</th>
<th>ASTM and/or AASHTO</th>
<th>Applicable Soil Types</th>
<th>Applicable Soil Properties</th>
<th>Limitations / Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moisture content, $w_c$</td>
<td>Dry soil in oven at 100 ± 5 °C</td>
<td>D 2216 T 265</td>
<td>Gravel, sand, silt, clay, peat</td>
<td>$e_o$</td>
<td>Simple index test for all materials</td>
</tr>
<tr>
<td>Unit weight and density</td>
<td>Extract a tube sample, measure dimensions and weight.</td>
<td>D 2216 T 265</td>
<td>Soils where undisturbed samples can be taken, i.e., silt, clay, peat</td>
<td>$γ_c$, $γ_d$, $γ_l$, $γ_m$, $γ_p$</td>
<td>Not appropriate for clean granular materials where undisturbed sampling is not possible. Very useful index test.</td>
</tr>
<tr>
<td>Atterberg limits, LL, PL, PI, SL, LI</td>
<td>LL - Moisture content associated with closure of the groove at 25 blows of specimen in Casagrande cup. PL - Moisture content associated with crumbling of rolled soil at 1/8-in (3mm)</td>
<td>D 4318 T 89 T 90</td>
<td>Clays, silts, peat, silty and clayey sands to determine whether SM or SC.</td>
<td>Soil classification and used in consolidation parameters</td>
<td>Not appropriate in non-plastic granular soil. Recommended for all plastic materials.</td>
</tr>
<tr>
<td>Mechanical sieve</td>
<td>Place air dry material on a series of successively smaller screens of known opening size and vibrate to separate particles of a specific equivalent diameter.</td>
<td>D 422 T 88</td>
<td>Gravel, sand, silt</td>
<td>Soil classification</td>
<td>Not appropriate for clay soils. Useful, particularly in clean and dirty granular materials.</td>
</tr>
<tr>
<td>Wash sieve</td>
<td>Flush fine particles through a U.S. No. 200 (0.075 mm) sieve with water.</td>
<td>C 117 D 1140 T 88</td>
<td>Sand, silt, clay</td>
<td>Soil classification</td>
<td>Needed to assess fines content in dirty granular materials.</td>
</tr>
<tr>
<td>Hydrometer</td>
<td>Allow particles to settle, and measure specific gravity of the solution with time.</td>
<td>D 422 D 1140 T 88</td>
<td>Fine sand, silt, clay</td>
<td>Soil classification</td>
<td>Helpful to assess relative quantity of silt and clay.</td>
</tr>
<tr>
<td>Sand Equivalent</td>
<td>Sample passing No. 4 (4.75 mm) sieve is separated into sand and clay size particles.</td>
<td>D 2419 T 176</td>
<td>Gravel, Sand, silt, clay</td>
<td>Aggregate classification</td>
<td>Useful for aggregates</td>
</tr>
<tr>
<td>Specific gravity of solids</td>
<td>The volume of a known mass of soil is compared to the known volume of water in a calibrated pycnometer</td>
<td>D 854 D 5550 T 100</td>
<td>Sand, silt, clay, peat</td>
<td>Used in calculation of $e_o$, $ρ_d$, $γ_m$, $γ_p$</td>
<td>Particularly helpful in cases where unusual solid minerals are encountered.</td>
</tr>
<tr>
<td>Organic content</td>
<td>After performing a moisture content test at 110°C (230°F), the sample is ignited in a muffle furnace at 440°C (824°F) to measure the ash content.</td>
<td>D 2974 T 194</td>
<td>All soil types where organic matter is suspected to be a concern</td>
<td>Not related to any specific performance parameters, but samples high in organic content will likely have high compressibility.</td>
<td>Recommended on all soils suspected to contain organic materials.</td>
</tr>
</tbody>
</table>

Symbols:
- $e_o$: in-situ void ratio
- $ρ_d$: dry density
- $γ_m$: dry unit weight
- $γ_p$: total unit weight
- $γ_c$, $γ_d$, $γ_l$, $γ_m$, $γ_p$: total vertical stress
# Table 6-3 Methods for Performance Testing of Soils

(Samtani and Nowatzki, 2006)

<table>
<thead>
<tr>
<th>Test</th>
<th>Procedure</th>
<th>Applicable Soil Types</th>
<th>Soil Properties</th>
<th>Limitations / Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-D oedometer</td>
<td>Incremental loads are applied to a soil specimen confined by a rigid ring. Deformation values are recorded with time, loads are typically doubled for each increment and applied for 24 hours each.</td>
<td>Primarily clays and silts; granular soils can be tested, but typically are not.</td>
<td>$p_0$, OCR, $C_v$, $C_w$, $k$</td>
<td>Recommended for fine grained soils. Results can be useful index to other critical parameters.</td>
</tr>
<tr>
<td>Constant rate of strain oedometer</td>
<td>Loads are applied such that $\Delta u$ is between 3 and 30 percent of the applied vertical stress during testing.</td>
<td>Clays and silts; not applicable to free draining granular soils.</td>
<td>$p$, $C_v$, $C_w$, $C_s$, $k$</td>
<td>Requires special testing equipment, but can reduce testing time significantly.</td>
</tr>
<tr>
<td>Unconfined compression (UC)</td>
<td>A specimen is placed in a loading apparatus and sheared under axial compression with no confinement.</td>
<td>Clays and silts; cannot be performed on granular soils or fissured and varved materials.</td>
<td>$s_{uc}$</td>
<td>Provides rapid means to approximate undrained shear strength, but disturbance effects, test rate, and moisture migration will affect results.</td>
</tr>
<tr>
<td>Unconsolidated undrained (UU) triaxial shear</td>
<td>The specimen is not allowed to consolidate under the confining stress, and the specimen is loaded at a quick enough rate to prevent drainage.</td>
<td>Clays and silts.</td>
<td>$s_{um}$</td>
<td>Sample must be nearly saturated. Sample disturbance and rate effects will affect measured strength.</td>
</tr>
<tr>
<td>Isotropic consolidated drained compression (CIDC)</td>
<td>The specimen is allowed to consolidate under the confining stress, and then is sheared at a rate slow enough to prevent build-up of pore water pressures.</td>
<td>Sands, silts, clays</td>
<td>$\phi'$, $c'$, $E$</td>
<td>Can be run on clay specimen, but time consuming. Best triaxial test to obtain deformation properties.</td>
</tr>
<tr>
<td>Isotropic consolidated undrained compression (CIUC)</td>
<td>The specimen is allowed to consolidate under the confining stress with drainage allowed, and then is sheared with no drainage allowed, but pore water pressures measured.</td>
<td>Sands, silts, clays, peats</td>
<td>$\phi'$, $c'$, $s_{u,ciuc}$, $E$</td>
<td>Recommended to measure pore pressures during test. Useful test to assess effective stress strength parameters. Not recommended for measuring deformation properties.</td>
</tr>
<tr>
<td>Direct shear</td>
<td>The specimen is sheared on a forced failure plane at a constant rate, which is a function of the hydraulic conductivity of the specimen.</td>
<td>Compacted fill materials; sands, silts, and clays</td>
<td>$\phi'$, $\phi''$</td>
<td>Requires assumption of drainage conditions. Relatively easy to perform.</td>
</tr>
<tr>
<td>Flexible Wall Permeometer</td>
<td>The specimen is encased in a membrane, consolidated, backpressure saturated, and measurements of flow with time are recorded for a specific gradient.</td>
<td>Relatively low permeability materials ($k \leq 1 \times 10^{-7}$ cm/s); clays &amp; silts</td>
<td>$k$</td>
<td>Recommended for fine grained materials. Backpressure saturation required. Confining stress needs to be provided. System permeability must be at least an order of magnitude greater than that of the specimen. Time needed to allow inflow and outflow to stabilize.</td>
</tr>
<tr>
<td>Rapid Wall Permeometer</td>
<td>The specimen is placed in a rigid wall cell, vertical confinement is applied, and flow measurements are recorded with time under constant head or falling head conditions.</td>
<td>Relatively high permeability materials; sands, gravels, and silts</td>
<td>$k$</td>
<td>Need to control gradient. Not for use in fine grained soils. Monitor for sidewall leakage.</td>
</tr>
</tbody>
</table>

Symbols:
- $\phi'$: peak effective stress friction angle
- $\phi''$: residual effective stress friction angle
- $c'$: effective stress cohesion intercept
- $s_{um}$: undrained shear strength
- $p$: preconsolidation stress
- OCR: overconsolidation ratio
- $c_v$: vertical coefficient of consolidation
- $E$: Young's modulus
- $k$: hydraulic conductivity
- $C_v$: compression index
- $C_r$: recompression index
- $C_m$: modified compression index
- $C_r'$: recompression index
- $C_m'$: modified recompression index
- $C_s$: secondary compression index
- $C_s'$: modified secondary compression index
6.4.2 Quality Control for Laboratory Testing

Improper storage, transportation and handling of samples can significantly alter the material properties and result in misleading test results. The requirements provided in NYSDOT GDM Chapter 4 regarding these issues shall be followed.

Laboratories conducting geotechnical testing shall be either AASHTO accredited or fulfill the requirements of AASHTO R18 for qualifying testers and calibrating/verifications of testing equipment for those tests being performed. In addition, the following guidelines (Mayne, et al., 1997) for laboratory testing of soils should be followed:

1. Protect samples to prevent moisture loss and structural disturbance.
2. X-ray soil samples in Shelby tubes to determine amount of disturbance.
3. Carefully handle samples during extrusion of samples; samples must be extruded properly and supported upon their exit from the tube.
4. Avoid long term storage of soil samples in Shelby tubes.
5. Properly number and identify samples.
6. Store samples in properly controlled environments.
7. Visually examine and identify soil samples after removal of smear from the sample surface.
8. Use pocket penetrometer or miniature vane only for an indication of strength.
9. Carefully select “representative” specimens for testing.
10. Have a sufficient number of samples to select from.
11. Always consult the field logs for proper selection of specimens.
12. Recognize disturbances caused by sampling, the presence of cuttings, drilling mud or other foreign matter and avoid during selection of specimens.
14. Always perform organic content tests when classifying soils as peat or organic. Visual classifications of organic soils may be very misleading.
15. Do not dry soils in overheated or underheated ovens.
16. Discard old worn-out equipment; old screens for example, particularly fine (< No. 40) mesh ones need to be inspected and replaced often, worn compaction mold or compaction hammers (an error in the volume of a compaction mold is amplified 30x when translated to unit volume) should be checked and replaced if needed.
17. Performance of Atterberg Limits requires carefully adjusted drop height of the Liquid Limit machine and proper rolling of Plastic Limit specimens.
18. Do not use tap water for tests where distilled water is specified.
19. Properly cure stabilization test specimens.
20. Never assume that all samples are saturated as received.
21. Saturation must be performed using properly staged back pressures.
22. Use properly fitted o-rings, membranes, etc. in triaxial or permeability tests.
23. Evenly trim the ends and sides of undisturbed samples.
25. Also do not mistakenly identify failures due to slickensides as shear failures.
26. Do not use unconfined compression test results (stress-strain curves) to determine elastic
modulus values.

27. Incremental loading of consolidation tests should only be performed after the completion of each primary stage.

28. Use proper loading rate for strength tests.

29. Do not guesstimate $e$-$

\log p$ curves from accelerated, incomplete consolidation tests.

30. Avoid “Reconstructing” soil specimens, disturbed by sampling or handling, for undisturbed testing.

31. Correctly label laboratory test specimens.

32. Do not take shortcuts: using non-standard equipment or non-standard test procedures.

33. Periodically calibrate all testing equipment and maintain calibration records.

34. In variable material, always test a sufficient number of samples to obtain representative results.

### 6.4.3 Developing the Testing Plan

The amount of laboratory testing required for a project will vary depending on availability of preexisting data, the character of the soils, and the requirements of the project. Laboratory testing must be intelligently planned in advance but flexible enough to be modified based on test results. Laboratory tests should be selected to provide the desired and necessary data as economically as possible. The ideal laboratory testing program will provide the Departmental Geotechnical Engineer with sufficient data to complete an economical design, yet not tie up laboratory personnel and equipment with superfluous testing. The cost for laboratory testing is insignificant compared to the cost of an over-conservative design.

The selection of representative specimens for testing is one of the most important aspects of sampling and testing procedures. Selected specimens must be representative of the formation or deposit being investigated. The Departmental Geotechnical Engineer will study the subsurface exploration logs, understand the geology of the site, and visually examine the samples before selecting the test specimens. Samples are selected on the basis of their color, physical appearance, structural features and an understanding of the disturbance of the samples.

Laboratory testing should be performed on both representative and critical test specimens obtained from geologic layers across the site. Critical areas correspond to locations where the results of the laboratory tests could result in a significant change in the proposed design. In general, a few carefully conducted tests on samples selected to cover the range of soil properties with the results correlated by classification and index tests is the most efficient use of resources.

The following should be considered when developing a testing program:

- Project type (bridge, embankment, rehabilitation, buildings, etc.)
- Size of the project
- Loads to be imposed on the foundation soils
- Types of loads (i.e., static, dynamic, etc.)
- Whether long-term conditions or short-term conditions are in view
- Critical tolerances for the project (e.g., settlement limitations)
- Vertical and horizontal variations in the soil profile as determined from boring logs and
visual identification of soil types in the laboratory
• Known or suspected peculiarities of soils at the project location (i.e., swelling soils, collapsible soils, organics, etc.)
• Presence of visually observed intrusions, slickensides, fissures, concretions, etc in sample – how will it affect results
• Project schedules and budgets
• Property data needed for specific design procedures

6.4.4 Laboratory Soil Performance Test Requests

The type and number of laboratory tests to determine soil performance will vary with each project and will depend on:
• Project requirements and anticipated construction sequence,
• Character of soils, and
• Previous tests performed.

Soil performance tests are typically conducted on cohesive soils (soils containing a noticeable amount of clay) or organic soils with the majority of the testing done on undisturbed samples. Typically, soil performance tests consist of Consolidation Tests and Shear Strength Tests.

Prior to any soil performance testing, the Departmental Geotechnical Engineer is to:
• Review undisturbed hole drill logs, examine jar and tube samples,
• Review all x-rays taken of tubes. The x-rays are used for:
  o Determining if sample disturbance (e.g. arched layers) has taken place. This is important in data interpretation.
  o Selecting specimens for laboratory testing. Unless the soil deposit is homogeneous (not likely), test samples are to be preselected (it is not to be a random process where whatever is extruded out of the tube is selected first).
• Prepare an Effective Overburden (Pₒ) diagram as shown in Figure 6-1. This chart is required in establishing test requirements.
Figure 6-1 Effective Overburden Diagram

The Departmental Geotechnical Engineer prepares the laboratory test request as shown in Figure 6-2.
### Figure 6-2 Laboratory Test Request

<table>
<thead>
<tr>
<th>CONSOLIDATION TESTS</th>
<th><strong>STRENGTH TESTS</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td>SAMPLE DIAMETER</td>
<td><strong>TYPE OF TEST AND CONSOLIDATION PRESSURE</strong></td>
</tr>
<tr>
<td>SIZE STATUS</td>
<td>CONS. PRESS. (PSI)</td>
</tr>
<tr>
<td>LOAD CYCLE</td>
<td>1</td>
</tr>
<tr>
<td>SAMPLE NUMBER</td>
<td>L4</td>
</tr>
</tbody>
</table>

**ADDITIONAL REQUESTS**

**TYPES OF STRENGTH TESTS**

- CU - CONSOL. - UNDRAINED
- CD - CONSOL. - DRAINED
- CU/UP - CONSOL. - UNDRAINED with PORE PRESSURE
- CAU - ANISTROPICALLY CONSOL. UNDRAINED
- UU - UNCONSOL. UNDRAINED
6.4.4.1 Consolidation Tests

The Departmental Geotechnical Engineer is responsible for:

- Selecting loading time increments (time intervals between application of successive load increments. 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% 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.
- Specifying if and how many recycles are required. 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.
- Designating loads for $C_v$ determinations. Generally, values are computed starting at the load below the effective overburden pressure ($P_o$) at the depth of the tube sample.

6.4.4.2 Strength Tests

The Departmental Geotechnical Engineer is responsible for:

- Selecting test type:
  - CIU = Consolidated – Isotropic – Undrained.
  - CD = Consolidated – Drained.
  - CU/P = Consolidated Isotropic Undrained with Pore Pressure Measurements.
  - CAU = Consolidated Anisotropic Undrained.
  - UU = Unconsolidated – Undrained.
  - Direct Shear Test
  - Unconfined Compressive Strength Test
- Determining confining or consolidation pressure. Typically, the first test point (one of the three test samples obtained after trisecting the extruded tube sample) is to be consolidated to $P_o$. The second test point is consolidated to $P_F$ and the third test point is consolidated to the minimum (safe) pressure that the laboratory can attain (approximately 60 psi). The third point is used to determine the $c/p$ ratio.

Unless otherwise specified, triaxial strength tests are run to 10% strain or to failure, whichever occurs first. The Mohr circles are drawn at seven percent strain, or failure, whichever occurs first. Atterberg Limits, Hydrometer and Specific Gravities are performed on the sample trimmings.
6.4.4.2.1 Relevance of Laboratory Strength Tests to Field Conditions

Note: Plane strain tests (PSC/PSE) used for long features
Triaxial tests (TC/TE) used for near symmetrical features
Direct shear (DS) normally substituted for DDS to evaluate $\phi$

Figure 6-3 Relevance of Laboratory Strength Tests to Field Conditions
Total Stress Analysis (Undrained Strengths):
- Approximates performance of undrained saturated (or nearly) soil conditions under either relatively quick or typical construction loading scenarios.
- For stage construction analysis, calculate increase in shear strength from c/p
- Use pore pressure measurements only to verify that consolidation and strength gain in the field are proceeding as anticipated.
- Use mainly for short-term or quick loading conditions on soft clay. Unloading is critical only in very sensitive clay.

Effective Stress Analysis (Drained Strengths):
- Approximates effective stress parameters and field pore pressures under very slow (i.e., long-term) loading conditions or soil performance at very shallow depths.
- Shear failure pore pressures are difficult to calculate and often not included in the test results because the value is generally unconservative for most typical loading conditions.
- May need to rely on piezometers to catch development of possible shear pore pressures in the field. Problem: when noticeable shear pore pressures appear, failure may not be far away.
- Use for most soil performance analysis other than loading on soft clay.

6.4.4.2.2 Types of Triaxial Strength Tests

There are three types of triaxial tests:
- Consolidated Undrained (CU) test – test in which the sample is first consolidated to a predetermined pressure, $\sigma_{1c}$ and $\sigma_{3c}$, and no drainage is permitted during shearing of the sample;
- Consolidated Drained (CD) test – test in which the sample is first consolidated to a predetermined consolidation pressure, $\sigma_{1c}$ and $\sigma_{3c}$, and then drainage is permitted during shearing; and
- Unconsolidated Undrained (UU) test – test in which the sample in the pressure chamber is subjected to a confining pressure without allowing the sample to consolidate (drain).

No drainage is permitted during application of the axial load for the undrained tests.
Figure 6-4 Triaxial Shear Test Relationships
6.4.4.2.3 Selection of Final Design Shear Strengths

The in-situ strength of the soil must be selected carefully with full consideration given to the many complex facets that may have affected the laboratory determination of shear strength. Table 6-4 attempts to present a quantitative appraisal of the many factors that may lead to an overestimation (unconservative) or an underestimation (conservative) of the in-situ shear strength.

Johnson (1974) states “Evaluations of this type have little meaning unless they are done for a specific site and conditions, but even then required data are usually not available to permit reliable conclusions”. Thus, the Departmental Geotechnical Engineer should consider each of the factors listed in Table 6-4 and assign a quantitative factor of confidence or uncertainty to each factor, as it may have influenced the reported laboratory test data.

After evaluating the quality of laboratory data, it is strongly recommended that the Departmental Geotechnical Engineer compare the appraised shear strength values with available correlation and local experience.
<table>
<thead>
<tr>
<th>Factor</th>
<th>Influence (percent)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sample disturbance of foundation materials (for relatively good, undisturbed samples).</td>
<td>- (5 to 20)</td>
<td>Remolding may increase strength of slickensided specimens. Disturbance is greatest for deep borings and soft soils.</td>
</tr>
<tr>
<td>Effect of fissures in clays, especially highly overconsolidated clays and clay shales – effects not reflected in small samples.</td>
<td>+ (25 to 1000)</td>
<td>Generally a factor for highly overconsolidated soils only.</td>
</tr>
<tr>
<td>Rough caps and bases in laboratory tests.</td>
<td>+ 5</td>
<td></td>
</tr>
<tr>
<td>Triaxial compression tests instead of compression, simple shear and extension tests.</td>
<td>+ (20 to 30)</td>
<td>Especially important for foundation soils.</td>
</tr>
<tr>
<td>Triaxial instead of plane strain tests.</td>
<td>- (5 to 8)</td>
<td></td>
</tr>
<tr>
<td>Back-pressure saturation.</td>
<td>Depends on field conditions.</td>
<td>May cause grossly excessive strengths in CU tests at low confining stresses; conservative at high confining stresses.</td>
</tr>
<tr>
<td>Conventional plotting of CU test data as total stress envelopes.</td>
<td>- (15 to 20)</td>
<td>Effect may be eliminated by plotting data according to Taylor’s Method.</td>
</tr>
<tr>
<td>Isotropic, instead of anisotropic, consolidation in CU triaxial compression tests:</td>
<td></td>
<td>Values shown assume test envelopes for isotropic consolidation interpreted as $t_f$ versus $\sigma_{ci}$’; i.e. as used by designers in stability analysis.</td>
</tr>
<tr>
<td>(a) $A_f &gt; \frac{1}{4}$</td>
<td>(a) – (0 to 30)</td>
<td></td>
</tr>
<tr>
<td>(b) $A_f &lt; \frac{1}{4}$</td>
<td>(b) + (0 to 20)</td>
<td></td>
</tr>
<tr>
<td>Anisotropic material behavior – use of vertical instead of suitably inclined test samples.</td>
<td>+ (10 to 40)</td>
<td></td>
</tr>
<tr>
<td>Conventional rates of shear in the laboratory.</td>
<td>+ (5 to 200)</td>
<td>Effect depends on rate of testing, soil type, rate of consolidation in field, etc.</td>
</tr>
<tr>
<td>Progressive failure.</td>
<td>+ (0 to 20)</td>
<td>Depends on soil; mainly a factor for foundation soils. May be more serious than shown for some soils.</td>
</tr>
</tbody>
</table>

Table 6-4 Factored Adjustments on Laboratory Strength Test Results
6.5 LABORATORY TESTING FOR THE DETERMINATION OF INDEX PROPERTIES

Attributes on which their classification and identification are based on are known as index properties. Index properties are used to discriminate between the different kinds of soil and rock within a broad category (e.g. clay will exhibit a wide range of engineering properties depending upon its composition).

6.5.1 Grain Size Analysis

There are two types of tests: Grain-Size with wash No. 200 and Hydrometer test. Grain size with wash No. 200, also known as Sieve Analysis, is for coarse-grained soils (sand, gravels). The hydrometer analysis is used for fine-grained soils (clays, silts).

![Figure 6-5 Soil Description by Components]
The results of the analyses are presented in a semilogarithmic plot known as particle-size distribution curves. In the semilogarithmic scale, the particle sizes are plotted on the log scale. The percent finer is plotted in arithmetic scale. Therefore, the graph is easy to read the percentages of gravel, sand, silt, and clay-size particles in a sample of soil.

The shape of the grain size curve is indicative of the grading. A “uniformly” graded soil has a grain size that is nearly vertical (curve B) and a “well-graded” soil has a flatter curve (curve A) that extends across several log cycles of particle size.

The grain-size analysis can also be used for obtaining three basic soil parameters from the curves. These parameters are: effective size ($D_{10}$), Coefficient of Uniformity ($C_u$), and Coefficient of Curvature ($C_c$).

The Coefficient of Uniformity represents the uniformity of a soil and is expressed as:

**Equation 6-1**

$$c_u = \frac{D_{60}}{D_{10}}$$

Where:

$D_{60}$ = the particle size for which 60% of the sample weight is finer

$D_{10}$ = the particle size for which 10% of the sample weight is finer.
Another quantity that may be used to judge the gradation of a soil is the Coefficient of Curvature which is expressed as:

Equation 6-2

\[ c_c = \frac{(D_{30})^2}{(D_{10})x(D_{60})} \]

Where:

- \( D_{60} \) = the particle size for which 60% of the sample weight is finer.
- \( D_{30} \) = the particle size for which 30% of the sample weight is finer.
- \( D_{10} \) = the particle size for which 10% of the sample weight is finer.

A “well-graded” gravel must have a \( C_u \) value > 4, and “well-graded” sands must have a \( C_u \) value > 6. For “well-graded” sands and gravels, a \( C_c \) value from 1 to 3 is required. Sands and gravels not meeting these conditions are termed poorly graded.

6.5.1.1 Sieve Analysis

The sieve analysis is a method used to determine the grain size distribution of soils. The soil is passed through a series of woven wires with square openings of decreasing sizes. The percentage of sample retained on the different sieve sizes provides a basis for classifying the soil. The NYSDOT sieve analysis method is outlined in the Geotechnical Engineering Bureau’s Test Method for the Grain-Size Analysis of Granular Soil Materials (GTM-20). For the distribution of particle sizes smaller than No. 200 (0.075 mm), a sedimentation process utilizing a hydrometer is necessary. For material in which the particle sizes are larger than 4 in. (100 mm), the distribution is determined visually.
Figure 6-7 Gradation Test Procedure

6.5.1.1.1 Sieve Analysis of Topsoil

The sieve analysis is a method used to determine the grain size distribution of topsoil. Due to the inherent characteristics of topsoil (in particular the amount of fine material), the topsoil sieve analysis procedure contains annotations regarding moisture content determination and sieving material on the No. 200 sieve. The NYSDOT sieve analysis method is outlined in the Geotechnical Engineering Bureau’s *Test Method for the Grain-Size Analysis of Topsoil* (GTM-22).

6.5.1.2 Hydrometer

The Hydrometer analysis is used to determine the particle size distribution in a soil that is finer than a No. 200 sieve size (0.075 mm), which is the smallest standard size opening in the sieve analysis. The procedure is based on the sedimentation of soil grains in water. It is expressed by Stokes Law, which says the velocity of the soil sedimentation is based on the soil particles shape, size, weight, and viscosity of the water. Thus, the hydrometer analysis measures the change in specific gravity of a soil-water suspension as soil particles settle out over time. The NYSDOT hydrometer method is outlined in the Geotechnical Engineering Bureau’s *Test Method and Discussion for the Particle Size Analysis of Soils by Hydrometer Method* (GTM-13).

For additional references, see ASTM D422 - *Standard Test Method for Particle-Size Analysis of Soils* (AASHTO T88 - *Standard Method of Test for Particle Size Analysis of Soils*).
Scale B

Specific Gravity Hydrometer Readings

Bouyoucos Hydrometer Readings - g/L

Scale A

The specific gravity scale (S.P.G.) shall be calibrated to read 1.000 at 68°F (20°C) and it shall extend beyond the limits shown, so as to read from 0.995 to 1.038.

The grams per liter scale (g/L) shall be extended 5 g/L above zero (1,000 S.P.G.) and down to 60 g/L. The bulb shall be symmetrical above and below the middle diameter, and shall be blown into a mold to assure uniformity of product.

* The diameter of the stem may be varied to adjust the length of a scale specified but the stem shall be uniform in diameter from the top to bottom.

The accuracy of the scale shall be ±1 scale division distributed uniformly over the scale length.

Hydrometers equipped with Scale B shall be identified as No. 151H.

Hydrometers equipped with Scale A shall be identified as No. 152H.

Figure 6-8 Hydrometer (From AASHTO T-88-86)
6.5.2 Moisture Content

The moisture content \( (w) \) is defined as the ratio of the weight of water in a sample to the weight of solids. The sample is oven-dried and is considered as weight of dry soil. Organic soils can have the water content determined, but must be dried at a lower temperature to prevent degradation of the organic matter. The NYSDOT moisture content examination is outlined in the Geotechnical Engineering Bureau’s Soil Mechanics Laboratory Test Procedures (GTP-6).

For additional references, see ASTM D2216 - Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass (AASHTO T265 - Standard Method of Test for Laboratory Determination of Moisture Content of Soils).

6.5.3 Atterberg Limits

A fine-grained soil can exist in any of several states of consistency. The particular state of consistency of any particular soil depends primarily upon the amount of water present in the soil-water system. The behavior of the soil is therefore related directly to the amount of water which is present. In 1911, A. Atterberg defined the boundaries of four states of consistency in terms of limits. These limits and the zones between the limits are illustrated in Figure 6-9.
The liquid limit, plastic limit, and shrinkage limit are arbitrary boundaries, but the procedures for obtaining these values have been standardized over a period of years. The limits are extremely useful in correlating anticipated soil behavior with previous experience on soils in similar consistency states. Each limit represents a water content at which the soil changes from one state to another. The plasticity index represents the range of water contents through which the soil is in the plastic state. The plasticity index is therefore simply the water content at the liquid limit minus the water content at the plastic limit.

It is often possible, for certain soil types, to set up a semi-empirical relationship between the Atterberg limits and other engineering properties. Such a procedure can be very helpful because the limits are usually more easily determined than such properties as compressibility, permeability, or shearing strength.
The shrinkage limit can be useful in predicting the loss of volume which an embankment material may undergo when removed from a wet borrow and subsequently dried and rolled into a fill.

The NYSDOT Atterberg Limit analyses are outlined in the Geotechnical Engineering Bureau’s *Test Method for Liquid Limit, Plastic Limit, and Plasticity Index* (GTM-7).

### 6.5.3.1 Plastic Limit

The plastic limit (PL) is the moisture content at which a soil transitions from being in a semisolid state to a plastic state. For additional references, see ASTM D4318 - *Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils* (AASHTO T90 - *Standard Method of Test for Determining the Plastic Limit and Plasticity Index of Soils*).

### 6.5.3.2 Liquid Limit

The liquid limit (LL) is defined as the moisture content at which a soil transitions from a plastic state to a liquid state. For additional references, see ASTM D4318 - *Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils* (AASHTO T89 - *Standard Method of Test for Determining the Liquid Limit of Soils*).

### 6.5.3.3 Plasticity Index

The plasticity index (PI) is defined as the difference between the liquid limit and the plastic limit of a soil. The PI represents the range of moisture contents within which the soil behaves as a plastic solid.

<table>
<thead>
<tr>
<th>PI Range</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>Nonplastic</td>
</tr>
<tr>
<td>1 – 5</td>
<td>Slightly Plastic</td>
</tr>
<tr>
<td>5 – 10</td>
<td>Low Plasticity</td>
</tr>
<tr>
<td>10 – 20</td>
<td>Medium Plasticity</td>
</tr>
<tr>
<td>20 – 40</td>
<td>High Plasticity</td>
</tr>
<tr>
<td>&gt; 40</td>
<td>Very High Plasticity</td>
</tr>
</tbody>
</table>

*Table 6-5 Plasticity Categories*

### 6.5.3.4 Liquidity Index

The relative consistency of a cohesive soil sample in the natural state is expressed by the liquidity index (LI). The LI is calculated by scaling the natural water content of a soil sample to the limits (i.e. a ratio of difference between natural water content, plastic limit, and liquid limit). The liquidity index is a good indicator of geologic history and relative soil properties as shown in Figure 6-10.
The natural moisture content exceeding the liquid limit may or may not be indicative of a normally consolidated condition. In New York State, there are numerous areas where LL exceeds the natural moisture content by 5 to 10% but consolidation testing will show that the soil is preconsolidated.

6.5.3.5 Activity

The degree of plasticity related to the clay content is called the Activity of the soil. In most natural clays, the actual clay-sized material (smaller than 2 μm) comprise only a fraction of the total. Therefore, the degree to which the soil acts like a clay depends upon the character of the clay-sized material and its relative amount in the soil.

6.5.4 Specific Gravity of Soils

The specific gravity of soil, $G_s$, is defined as the ratio of the unit weight of a given material to the unit weight of water. The procedure is applicable only for soils composed of particles smaller than the No. 4 sieve (4.75 mm). The NYSDOT specific gravity analysis is outlined in the Geotechnical Engineering Bureau’s Test Method for the Determination of the Apparent Specific Gravity of Soils (GTM-14).

For additional references, see ASTM D854 - Standard Test Methods for Specific Gravity of Soil Solids by Water Pycnometer (AASHTO T100 - Standard Method of Test for Specific Gravity of Soils). If the soil contains particles larger than the No. 4 sieve (4.75 mm), see ASTM C127-Standard Test Method for Density, Relative Density (Specific Gravity), and Absorption of Coarse Aggregate.
6.5.5 Organic Content

Organic soils demonstrate very poor engineering characteristics, most notably low strength and high compressibility. In the field these soils can usually be identified by their dark color, musty odor and low unit weight. The most used laboratory test for design purposes is the Ignition Loss test, which measures how much of a dried sample’s mass burns off when placed in a muffle furnace. The results are presented as a percentage of the total solid mass.


6.6 LABORATORY TESTING FOR THE DETERMINATION OF ROCK INDEX PROPERTIES

6.6.1 Punch-Penetration Index Test

The punch-penetration index test evaluates the penetration resistance of the intact rock cores. The test consists of utilizing a standard conical indentor which is pressed into a rock sample that has been cast in a confining steel ring. The load and displacement of the indentor are recorded. The slope of the force-penetration curve indicates the excavatability of the rock (i.e., the energy needed for efficient chipping.). This is affected by the stiffness, brittleness, and porosity of the sample.

Because the punch-penetration test actually penetrates the rock, it provides the capability to reveal some important rock excavatability features that other tests may fail to illustrate. One important feature which the punch-penetration test has been successful in identifying is rock toughness, which is defined as the resistance of rock to effective chip formation under the cutting action of disc cutters (i.e. rock absorbing a high degree of energy prior to chip formation).

6.6.2 Cerchar Abrasivity Index (CAI) Test

The Cerchar Abrasivity Test was introduced in the 1970’s by the Centre d’Etudes et recherches des Charbonages (CERCHAR) de France for abrasivity testing in coal bearing rocks. The abrasivity of rock is a factor with considerable influence on the wear of tools. Wear is a result of material consumption and excavation speed, and is an important indicator of rock excavation. Wear depends on

- the machinery being used for excavation (i.e. the devices and all tools that have contact with the rock), vs.
- the rock and the geological conditions.

To evaluate the abrasivity of the intact rock cores, the sample is fixed in a holder with the fresh surface facing upward. A conical, 90° hardened steel pin, fastened in a 15 lb. (7.5 kg) head, is set on the fresh surface and drawn 0.4 in. (1 cm) across it in 1 second. This procedure is repeated for a total of five pins. The tips of the pins are examined under a reticular microscope and two
perpendicular diameters of the resulting wear flat are recorded for each pin.

The Cerchar Abrasivity Index (CAI) is calculated by:

**Equation 6-3**

\[
CAI = 0.0254 \sum_{i=1}^{10} d_i
\]

Where:

\( d_i \) = pin diameter (micro in.)

### 6.6.3 Thin Section Petrographic Analysis

Thin section petrographic analysis is used to evaluate the mineralogy of the intact rock cores. In petrography, the mineral content and the textural relationships within the rock are described in detail. A detailed analysis of minerals, by optical mineralogy in a thin section, is performed via a petrographic microscope. The micro-texture and structure reveal the origin of the rock.

In a petrographic analysis, a slice of rock is affixed to a microscope slide and then ground so thin that light can be transmitted through mineral grains. The extreme thinness of the section enables the various minerals to be distinguished according to their behavior in transmitted light. Polarizing filters within the petrological microscope produce crossed polarized light in which the crystals affect the light path to produce characteristic interference colors. This feature, together with other properties such as refractive index, crystal shape, and texture, enable almost all minerals and rock types to be identified.

### 6.7 LABORATORY TESTING FOR THE DETERMINATION OF SOIL PERFORMANCE PROPERTIES

Specific properties that are measured to characterize behavior for design and constructability assessments (e.g., shear strength, compressibility, hydraulic conductivity, etc.) are known as performance properties.

#### 6.7.1 Strength Tests

The shear strength is the internal resistance per unit area that the soil can handle before failure and is expressed as a stress. There are two components of shear strength; the cohesive element (expressed as the cohesion, \( c \), in units of force/unit area) and the frictional element (expressed as the angle of internal friction, \( \phi \)). These parameters are expressed in the form of total stress \( (c, \phi) \) or effective stress \( (c', \phi') \). The total stress on any subsurface element is produced by the overburden pressure plus any applied loads. The effective stress equals the total stress minus the pore water pressure. The common methods of ascertaining these parameters in the laboratory are discussed below. All of these tests are normally performed on undisturbed samples, but may also be performed on remolded samples.
6.7.1.1 Unconfined Compression Test

The unconfined compression test is a quick method of determining the value of undrained cohesion ($c_u$) for clay soils. The test involves a clay specimen with no confining pressure and an axial load being applied to observe the axial strains corresponding to various stress levels. The stress at failure is referred to as the unconfined compression strength. The $c_u$ is taken as one-half the unconfined compressive strength, $q_u$. See ASTM D2166 - Standard Test Method for Unconfined Compressive Strength of Cohesive Soil (AASHTO T208 - Standard Method of Test for Unconfined Compressive Strength of Cohesive Soil).

6.7.1.2 Triaxial Compression Test

The triaxial compression test is a more sophisticated testing procedure for determining the shear strength of a soil. The test involves a soil specimen subjected to an axial load until failure while also being subjected to confining pressure that approximates the in-situ stress conditions. There are three types of triaxial tests which are described below.

6.7.1.2.1 Unconsolidated-Undrained (UU) or Q (Quick) Test

In unconsolidated-undrained tests, the specimen is not permitted to change its initial water content before or during shear. The results are total stress parameters. This test is used primarily in the calculation of immediate embankment stability during quick-loading conditions. The NYSDOT Unconsolidated-Undrained procedure is outlined in the Geotechnical Engineering Bureau’s Soil Mechanics Laboratory Test Procedures (GTP-6).

For additional references, see ASTM D2850 - Standard Test Method for Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils (AASHTO T296 - Standard Method of Test for Unconsolidated, Undrained Compressive Strength of Cohesive Soils in Triaxial Compression).

6.7.1.2.2 Consolidated-Undrained (CU) or R (Regular) Test

The consolidated-undrained test is the most common type of triaxial test. This test allows the soil specimen to be consolidated under a confining pressure prior to shear. After the pore water pressure is dissipated, the drainage line will be closed and the specimen will be subjected to shear. Several tests on similar specimens with varying confining pressures may have to be made to determine the shear strength parameters. The NYSDOT Consolidated-Undrained procedure is outlined in the Geotechnical Engineering Bureau’s Soil Mechanics Laboratory Test Procedures (GTP-6).

For additional references, see ASTM D4767 - Standard Test Method for Consolidated Undrained Triaxial Compression Test for Cohesive Soils (AASHTO T297 - Standard Method of Test for Consolidated, Undrained Triaxial Compression Test on Cohesive Soils).
6.7.1.2.3 Consolidated-Drained (CD) or S (Slow) Test

The consolidated-drained test is similar to the consolidated-undrained test except that drainage is permitted during shear and the rate of shear is very slow. Thus, the buildup of excess pore pressure is prevented. Again, several tests on similar specimens will be conducted to determine the shear strength parameters. This test is used to determine parameters for calculating long-term stability of embankments. The NYSDOT Consolidated-Drained procedure is outlined in the Geotechnical Engineering Bureau’s Soil Mechanics Laboratory Test Procedures (GTP-6).

For additional references, see ASTM WK3821 - New Test Method for Consolidated Drained Triaxial Compression Test for Soils.

6.7.1.3 Direct Shear

The direct shear test is the oldest and simplest form of shear test. A soil sample is placed in a metal shear box and undergoes a horizontal force. The soil fails by shearing along a plane when the force is applied. The test can be performed either in stress-controlled or strain-controlled. In addition the test is typically performed as consolidated-drained test on cohesionless soils.


6.7.2 Consolidation Test

The amount of settlement induced by the placement of load bearing elements on the ground surface or the construction of earthen embankments will affect the performance of the structure. The amount of settlement is a function of the increase in pore water pressure caused by the loading and the reduction of this pressure over time. The reduction in pore pressure and the rate of the reduction are a function of the permeability of the in-situ soil. All soils undergo elastic compression and primary and secondary consolidation. Sandy (coarse-grained) soils tend to be relatively permeable and will therefore, undergo settlement much faster. The amount of elastic compression settlement can vary depending on the soil type; however, the time for this settlement to occur is relatively quick and will normally occur during construction. Clayey (fine-grained) soils have a much lower permeability and will, therefore, take longer to settle. Clayey soils undergo elastic compression during the initial stages of loading (i.e. the soil particles rearrange due to the loading). After elastic compression, clayey soils enter primary consolidation. Saturated clayey soils have a lower coefficient of permeability, thus the excess pore water pressure generated by loading will require a longer period of time to dissipate. Therefore in saturated clays, the amount and rate of settlement is of great importance in construction. For example, an embankment may settle until a bump or step exists between the approach and bridge abutment. The calculation of settlement involves many factors, including the magnitude of the load, the effect of the load at the depth at which compressible soils exist, the water table, and characteristics of the soil itself. Consolidation testing is performed to ascertain the nature of these characteristics.
The most often used method of consolidation testing is the one-dimensional test. The consolidation test unit consists of a consolidometer (oedometer) and a loading device as shown in Figure 6-11. The soil sample is placed between two porous stones, which permit drainage. Because strain in the horizontal direction is prevented, the vertical strain is equal to the volumetric strain.

The test is generally performed on a specimen of clay that is 0.75 or 1.0 in. (19 or 25 mm) in thickness and 2.5 in. (64 mm) in diameter. The 2.5 in. ring is the most common size ring because the specimen can be trimmed from a 3 in. (76 mm) thin wall tube sample.

The load applied to the specimen is doubled (Load Increment Ratio (LIR) equal to unity) and readings of vertical deformation versus time are obtained during each load increment. The information obtained from the test are:

- Compressibility of the soil for one-dimensional loading as defined by the compression curve, (vertical strain, $\varepsilon_v$, or void ratio, $e$, plotted versus log consolidation stress),
- Maximum previous stress $P_p$,
- Coefficient of vertical consolidation, $c_v$,
- Coefficient of secondary compression, $c_\alpha$.

These parameters are used to predict the rate of primary settlement and the amount of secondary consolidation.

The NYSDOT one-dimensional consolidation procedure is outlined in the Geotechnical Engineering Bureau’s *Soil Mechanics Laboratory Test Procedures* (GTP-6).

For additional references, see ASTM D2435 - *Standard Test Methods for One-Dimensional Consolidation Properties of Soils Using Incremental Loading* (AASHTO T216 - *Standard Method of Test for One-Dimensional Consolidation Properties of Soils*).
6.7.3 Shrinkage and Swell

Certain soil types (highly plastic) have a large potential for volumetric change depending on the moisture content of the soil. These soils can shrink with decreasing moisture or swell with increasing moisture. Shrinkage can cause soil to pull away from structure thus reducing the bearing area or causing settlement of the structure beyond that predicted by settlement analysis. Swelling of the soil can cause an extra load to be applied to the structure that was not accounted for in design. Therefore, the potential for shrinkage and swelling should be determined for soils that have high plasticity.

6.7.3.1 Shrinkage

These tests are performed to determine the limits of a soil’s tendency to lose volume during decreases in moisture content. The shrinkage limit (SL) is presented as a percentage in moisture content, at which the volume of the soil mass ceases to change. See ASTM D427 - Test Method for Shrinkage Factors of Soils by the Mercury Method (AASHTO T92 - Standard Method of Test for Determining the Shrinkage Factors of Soils).
6.7.3.2 Swell

There are certain types of soils that can swell, particularly clay in the montmorillonite (a very soft phyllosilicate group of minerals that typically form in microscopic crystals, forming a clay) family. Swelling occurs when the moisture is allowed to increase causing the clay soil to increase in volume. There are a number of reasons for this to occur: the elastic rebound of the soil grains, the attraction of the clay mineral for water, the electrical repulsion of the clay particles and their adsorbed cations from one another, or the expansion of the air trapped in the soil voids. In the montmorillonite family, adsorption and repulsion predominate and this can cause swelling. Testing for swelling is difficult, but can be done. It is recommended that these soils not be used for roadway construction. The swell potential can be estimated from the test methods shown in ASTM D4546 - Standard Test Methods for One-Dimensional Swell or Settlement Potential of Cohesive Soils (AASHTO T258 - Standard Method of Test for Determining Expansive Soils).

6.7.4 Permeability

Permeability, also known as hydraulic conductivity, has the same units as velocity and is generally expressed in ft/min or m/sec. Coefficient of permeability is dependent on void ratio, grain-size distribution, pore-size distribution, roughness of mineral particles, fluid viscosity, and degree of saturation. There are three standard laboratory test procedures for determining the coefficient of permeability of soil: constant and falling head tests and flexible wall tests.

6.7.4.1 Laboratory Tests

6.7.4.1.1 Constant Head Test

In the constant head test, water is poured into a sample of soil, and the difference of head between the inlet and outlet remains constant during the testing. After the flow of water becomes constant, water that is collected in a flask is measured in quantity over a time period. This test is more suitable for coarse-grained soils that have a higher coefficient of permeability. See ASTM D2434 - Standard Test Method for Permeability of Granular Soils (Constant Head) (AASHTO T215 - Standard Method of Test for Permeability of Granular Soils (Constant Head)).

6.7.4.1.2 Falling Head Test

The falling head test uses a similar procedure to the constant head test, but the head is not kept constant. The permeability is measured by the decrease in head over a specified time. This test is more appropriate for fine-grained soils. Tests shall be performed in accordance with ASTM D5856 - Standard Test Method for Measurement of Hydraulic Conductivity of Porous Material Using a Rigid-Wall, Compaction-Mold Permeameter.
6.7.4.1.3 Flexible Wall Permeability

For fine-grained soils, tests performed using a triaxial cell are generally preferred. In-situ conditions can be modeled by application of an appropriate confining pressure. The sample can be saturated using back pressuring techniques. Water is then allowed to flow through the sample and measurements are taken until steady-state conditions occur. Tests shall be performed in accordance with ASTM D 5084 - *Standard Test Methods for Measurement of Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Permeameter*.

6.7.4.2 Particle Analysis

Saturated permeability is an essential parameter in the design of infiltration facilities. See NYSDOT GDM Chapter 20 for infiltration facility design.

The saturated permeability of a cohesionless granular material can be derived from the porosity and specific surface of the solids. The specific surface of a particle is defined as the total surface area contained in a unit volume of particle solids. The surface of a particle is its boundary, completely enclosing it. The surface area of a particle is a function of its volume for a given shape, if particle shape stays the same.

6.7.4.2.1 Specific Surface Analysis

The specific surface can be derived from grain-size analysis and a determination of the shape factor for each grain-size interval. A shape factor is determined by comparing the shape of the grains in that interval against a series of photos shown in Figure 6-12. These photos depict grains of standard shapes, and the corresponding shape factor is indicated beside each photo. Observed grain shapes frequently will lie between those in the standard photos, in which case the shape factor should be interpolated to best describe the observed shape. Also, the grains observed in a given interval may often have varying shapes. Shape variation usually follows size variation within the interval, so that the proportions can be visually estimated and a final weighted average shape factor can be recorded to satisfactorily represent the material contained in the interval.

The NYSDOT specific surface analysis is outlined in the Geotechnical Engineering Bureau’s *Test Procedure for Specific Surface Analysis* (GTP-5).
f=1.0  Glass beads.

f=1.15  Ottawa calibration sand.

f=1.25  Dredged sand from vicinity of Staten Island.

f=1.35  Cow Bay sand.

f=1.45  2Q sand (used in field density tests).

f=1.6  Crushed limestone.

f=1.8  Crushed Pyrex glass.

Figure 6-12 Particle Angularity (Shape) Factors
6.8 LABORATORY TESTING FOR THE DETERMINATION OF ROCK PERFORMANCE PROPERTIES

6.8.1 Resonant Frequency Test

Frequencies at which the response amplitude is a relative maximum are known as the system's resonant frequencies. At these frequencies, even small, periodic driving forces can produce large amplitude oscillations.

To investigate the fundamental frequencies, intact rock samples are struck with an impactor and the specimen response is measured by a lightweight accelerometer. Testing is performed in accordance with ASTM C215 – Standard Test Method for Fundamental Transverse, Longitudinal, and Torsional Resonant Frequencies of Concrete Specimens. The output of the accelerometer is recorded and the fundamental frequencies of the intact rock samples are determined from the waveforms recorded. The fundamental frequencies are directly related to the elastic properties of the rock, including the shear wave velocities.

6.8.2 Uniaxial Compressive Strength (UCS) Test

Compressive strength is the capacity of a material to withstand axially directed compressive forces. The most common measure of compressive strength is the uniaxial compressive strength (a.k.a. unconfined compressive strength).

The unconfined compressive strength is measured in accordance with the procedures given in ASTM D7012 - Standard Test Method for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying States of Stress and Temperature, utilizing a specimen with the length to diameter ratio between 2:1 to 2.5:1. There are high requirements on the flatness of the end-surfaces in order to obtain an even load distribution. The specimens are loaded axially up to failure or any other prescribed level whereby the specimen is deformed and the axial and the radial deformation can be measured using special equipment.

The unconfined compressive strength is calculated by:

**Equation 6-4**

\[ \sigma_c = \frac{F}{A} \]

Where:
- \( \sigma_c \) = Unconfined Compressive Strength (psi)
- F = Maximum Failure Load (lbs.)
- A = Cross sectional area of the core sample (in\(^2\))
6.8.3 Indirect (Brazilian) Tensile Strength Test

Although rocks are much weaker in tension than in compression or shear, tensile failure also plays an important role in some engineering activities (e.g. drilling, cutting and blasting of rocks). Tensile behavior of different rock formations can vary considerably, and neglecting such a parameter may overestimate the efficiency of the formation.

A laboratory technique to measure the tensile strength of rocks is the indirect tensile tests. A cylindrical specimen is loaded diametrically across the circular cross section. The loading causes a tensile deformation perpendicular to the loading direction, which yields a tensile failure. By registering the ultimate load and by knowing the dimensions of the specimen, the indirect tensile strength of the material can be computed. The indirect tensile strength is measured in accordance with the procedures given in ASTM D3867 - Standard Test Method for Splitting Tensile Strength of Intact Rock Core Specimens, utilizing a specimen with a thickness to diameter ratio between 0.2 to 0.75.

The indirect tensile strength is calculated by:

**Equation 6-5**

\[
\sigma_t = \frac{2F}{\pi LD}
\]

Where:
- \(\sigma_t\) = Indirect Tensile Strength (psi)
- \(D\) = Diameter of the core sample (in.)
- \(F\) = Maximum Failure Load (lbs.)
- \(L\) = Length of core sample (in.)

6.8.4 Point Load Test

The point load test was developed as a small, hand-portable test apparatus to provide an index for the strength classification of hard rocks in the field. Basically, the test method relies on the principle of inducing tensile stress into the rock by the application of a compressive force.

Point load test is carried out on core rock specimens to obtain the point load strength index and unconfined compressive strength. A correction is applied to account for the specimen size and shape, and the unconfined compressive strength is obtained from a correlation equation.

The point load strength is measured in accordance with the procedures given in ASTM D5731 - Standard Test Method for Determination of Point Load Strength Index of Rock and Application of Rock Strength Classifications, utilizing a specimen with a core diameter between 1 in. to 3 in.
The point load index strength is calculated by:

**Equation 6-6**

\[ I_s = \frac{F}{D_e^2} \]

Where:
- \( I_s \) = Point Load Index (psi)
- \( F \) = Failure Load (lbs.)
- \( D_e \) = Distance between platen tips (in.)
- \( D_e^2 = D^2 = 4A/\pi \) = for diametral core tests without penetration or, 
  = for axial, block and lump test
- \( A = WD = \) minimum cross sectional area of a plane through the platen contact points

**6.9 LABORATORY TESTING FOR THE DETERMINATION OF SOIL LONG-TERM PERMANENCE PROPERTIES**

Specific properties that are measured to ensure quality and predict performance (e.g. soundness, compaction, etc.) are known as long-term permanence properties.

**6.9.1 Compaction Tests**

The most effective means of ensuring the strength and minimizing the settlement potential of a fill is through adequate compaction. This can be checked by determining the soil density achieved during construction against laboratory testing of representative samples of the same soil compacted under controlled conditions. The filed density is determined by sand cone or volumeter apparatus. In order to evaluate the field density, it must be compared to a control density. See NYSDOT GDM Section 6.11.2 Compacted Soils.

There are two types of tests that can determine the optimum moisture content and maximum dry density of a soil; the Standard Proctor and the Modified Proctor. The results of the tests are used to determine appropriate methods of field compaction and to provide a standard by which to judge the acceptability of field compaction. The NYSDOT compaction procedure for determining the in-place density and moisture of earthwork is outlined in the Geotechnical Engineering Bureau's *Test Method for Earthwork Compaction Control by Sand Cone or Volumeter Apparatus* (GTM-9). This method may be used in conjunction with either the Standard (AASHTO T-99) or Modified Proctor (AASHTO T-180) Density Test for determining the percent of Maximum Density.

The results of the compaction tests are typically plotted as dry density versus moisture content. Tests have shown that moisture content has a great influence on the degree of compaction achieved by a given type of soil. In addition to moisture content, there are other important factors that affect compaction. The soil type has a great influence because of its various classifications,
such as grain size distribution, shape of the soil grains, specific gravity of soil solids, and amount and type of clay mineral present. The compaction energy (i.e. “effort”) also has an affect because it too has various conditions, such as number of blows, number of layers, weight of hammer, and height of the drop.

6.9.1.1 Standard Proctor

This test method uses a 5-1/2-pound rammer dropped from a height of 12 inches. The sample is compacted in the compaction cylinder (Figure 6-14) in three layers.

For additional references, see

- ASTM D698 - Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort (12,400 ft-lb/ft³ (600 kN-m/m³)),
- AASHTO T99 - Standard Method of Test for Moisture-Density Relations of Soils Using a 2.5-kg (5.5-lb) Rammer and a 305-mm (12-in.) Drop, and
Figure 6-13 Sand Cone Assembly
Figure 6-14 Compaction Cylinder Assembly
6.9.1.2 Modified Proctor

This test method uses a 10-pound rammer dropped from a height of 18 inches. The sample is compacted in the compaction cylinder (Figure 6-14) in five layers. The NYSDOT compaction procedure for determining the in-place density and moisture of earthwork is outlined in the Geotechnical Engineering Bureau’s Test Method for Earthwork Compaction Control by Sand Cone or Volumeter Apparatus (GTM-9). This method may be used in conjunction with either the Standard (AASHTO T-99) or Modified Proctor (AASHTO T-180) Density Test for determining the percent of Maximum Density.

See ASTM D1557 - Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lbf/ft3(2,700 kN·m/m3)), (AASHTO T180 - Standard Method of Test for Moisture-Density Relations of Soils Using a 4.54-kg (10-lb) Rammer and a 457-mm (18-in.) Drop), and the Geotechnical Engineering Bureau’s Test Method for Earthwork Compaction Control by Nuclear Gauge (GTM-10). Geotechnical Engineering Manual (GEM-10) Safety Procedure for Use with Nuclear Moisture-Density Gauges describes the operational safety procedures to be followed when using Nuclear Moisture – Density Gauges under the control of the Geotechnical Engineering Bureau.

6.9.2 Relative Density Test

The relative density tests are most commonly used for granular or unstructured soils. It is used to indicate the in-situ denseness or looseness of the granular soil. In comparison, Proctor tests often do not produce a well-defined moisture-density curve for cohesionless, free-draining soils. Therefore, relative density is expressed in terms of maximum and minimum possible dry unit weights and can be used to measure compaction in the field.

6.9.2.1 Maximum Index Density

In this test, soil is placed in a mold of known volume with a 2-psi surcharge load applied to it. The mold is then vertically vibrated at a specified frequency for a specified time. At the end of the vibrating period, the maximum index density can be calculated using the weight of the sand and the volume of the sand. See ASTM D4253 - Standard Test Methods for Maximum Index Density and Unit Weight of Soils Using a Vibratory Table.

6.9.2.2 Minimum Index Density

The test procedure requires sand being loosely poured into a mold at a designated height. The minimum index density can be calculated using the weight of the sand required to fill the mold and the volume of the mold. See ASTM D4254 - Standard Test Methods for Minimum Index Density and Unit Weight of Soils and Calculation of Relative Density.
6.9.3 Electro-Chemical Test

Electro-chemical tests provide quantitative information related to the aggressiveness of the subsurface environment, the surface water environment, and the potential for deterioration of foundation materials. Electro-chemical testing includes pH, resistivity, sulfate, and chloride contents. The electro-chemical tests should be performed on soil samples. In addition, surface water should also be tested in coastal regions where the potential intrusion of brackish (salty water) water may occur in tidal streams.

6.9.3.1 pH Testing

pH testing is used to determine the acidity or alkalinity of the subsurface or surface water environments. Acidic or alkaline environments have the potential for aggressively corroding structures placed within these environments. Soil samples collected during the normal course of a subsurface exploration should be used for pH testing. The NYSDOT pH analysis is outlined in the Geotechnical Engineering Bureau’s Test Method for the Determination of pH of Water or Soil by pH Meter (GTM-24).


6.9.3.2 Resistivity Testing

Resistivity testing is used to determine the electric conduction potential of the subsurface environment. The ability of soil to conduct electricity can have a significant impact on the corrosion of steel piling. If a soil has a high potential for conducting electricity, then sacrificial anodes may be required on the structure. The NYSDOT resistivity analysis is in accordance with AASHTO T288 – Standard Method of Test for Determining Minimum Laboratory Soil Resistivity.


6.9.3.3 Chloride Testing

Most salts are active participants in the corrosion process. Chlorides, sulphates and sulfides have been identified as being chief agents in promoting corrosion.

Subsurface soils and surface water should be tested for chloride if the presence of sea or brackish water is suspected. The NYSDOT chloride analysis is in accordance with Method A of AASHTO T291 – Standard Method of Test for Determining Water-Soluble Chloride Ion Content in Soil.
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For additional references, see ASTM D512 – Standard Test Methods for Chloride Ion in Water.

6.9.3.4 Sulfate Testing

As stated previously, chlorides, sulphates and sulfides have been identified as being chief agents in promoting corrosion. Subsurface soils and surface water should be tested for sulfate content. The NYSDOT sulfate analysis is in accordance with Method A (gravimetric) or Method B (turbidmetric) of AASHTO T290 – Standard Method of Test for Determining Water-Soluble Sulfate Ion Content in Soil.

For additional references, see ASTM D516 – Standard Test Methods for Sulfate Ion in Water.

6.9.3.5 Sulfide Testing

As stated previously, chlorides, sulphates and sulfides have been identified as being chief agents in promoting corrosion. Subsurface soils and surface water should be tested for sulfide content. The NYSDOT sulfide analysis is outlined in the Materials Bureau’s Test Method 711-12C.

6.9.4 Durability

A soundness test determines a granular materials resistance to disintegration by weathering and, in particular, freeze-thaw cycles. Aggregates that are durable (resistant to weathering) are less likely to degrade in the field and cause premature failure. A soundness test involves an initial sieve analysis to determine the distribution of particle sizes. The test will repeatedly submerge an aggregate sample in a magnesium sulfate solution. This process causes salt crystals to form in the aggregate’s water permeable pores. The formation of these crystals creates internal forces that apply pressure on aggregate pores and tend to break the aggregate. After the specified number of submerging and drying repetitions, the aggregate is again sieved to determine the percent loss of material. The formation of the salt crystals is intended to mimic the formation of ice crystals in the field therefore predicting an aggregate’s freeze-thaw performance.

6.9.4.1 Magnesium Sulfate Soundness Test

The durability of a granular material is determined by its resistance to disintegration under the action of repeated wetting and drying using a saturated solution of magnesium sulfate. The NYSDOT soundness analysis is outlined in the Geotechnical Engineering Bureau’s Test Method for Magnesium Sulfate Soundness of Granular Materials (GTM-21).
6.10 LABORATORY TESTING SUMMARY

Summarizing laboratory data enables the Departmental Geotechnical Engineer to interpret and implement design parameters into the geotechnical design. During the data collection and summary process, the Departmental Geotechnical Engineer should continuously evaluate the results of laboratory tests with respect to the initial subsurface model, understanding of the site, and assumed material behavior to determine the need to further explore particular areas.

Presentation of laboratory test results provides:

- An understanding of material behavior – As stated previously, the purpose of laboratory testing is to determine how the soil or rock will perform under the imposed conditions. In-situ stresses, the presence of water, rate and direction of loading can all affect the behavior of soils. Analysis of site samples with respect to how conditions will change over the life of the project provides an understanding of the existing subsurface conditions. With this understanding, the Departmental Geotechnical Engineer may proceed with an appropriate design.

- An establishment of baseline site conditions – Baselines are engineering interpretations or assumptions about geotechnical conditions that can affect the design of a project feature or its constructability. A baseline clearly defines the specific geotechnical condition that needs to be considered as the basis for developing a geotechnical design. When establishing a baseline, it must be recognized that subsurface conditions are inherently variable, and that variability can translate to design and construction risk. The baseline, however, must be analyzed thoroughly to be as clear and concise as possible, conveying what was assumed about the conditions being baselined.

- Quality of analysis – In order for methods of analyses to produce quality, reliable results, quality design parameters are required. As mentioned above, analysis of site samples with respect to how conditions will change over the life of the project provides an understanding of the existing subsurface conditions. With this understanding, the Departmental Geotechnical Engineer may assign appropriate design parameters to utilize in the method of analysis.

- An effective ability to troubleshoot during construction – If problems occur during construction, laboratory test results of the materials involved can provide valuable insight into what is causing the problem and how to incorporate a corrective fix. Being readily available, the laboratory results may be reviewed by the Departmental Geotechnical Engineer in conjunction with the site assessment to minimize delays and, ultimately, project costs.

- Documentation of design rationale – Design rationale is a record of decisions made during the design process, along with the associated reasoning. The design rationale is justification for the geotechnical design, which includes documentation of other alternatives explored, evaluation of trade-offs, and reasoning behind the design decisions for the preferred solution.
6.10.1 Undisturbed Data Summary

It is very important that information obtained on undisturbed samples be summarized both in tabular and graphical form. These summaries form the basis for the design and, when properly organized and presented, may be useful on future projects.

- Plasticity Chart
- Correlations:
  - Compression Ratio (CR) vs. Natural Moisture Content.
  - Recompression Ratio (RR) vs. Natural Moisture Content.
  - Coefficient of Vertical Consolidation ($C_v$) vs. Log Pressure.
- Summary Sheet (to be prepared for each undisturbed hole) is to portray elevation (depth not preferred) and include the following information:
  - Stress History (Past Pressure, $P_p$).
  - Strength Data (undrained shear strength, $S_u$).
  - Moisture Contents (from tube profiles and all testing performed).
  - Atterberg Limits.
- Graphs – should show all plotted points, not just smooth curves.
- Legends – When the results of more than one undisturbed hole are shown on a single graph, a legend should be used to differentiate the data.
- Identifiers – All data presentations (graphical or tabular) should include:
  - Title of the project.
  - File or project number.
  - Date of work.
  - Scale (if appropriate).
Figure 6-15 Example of Data Summary
Figure 6-16 Example of Data Summary
Figure 6-17 Example of Data Summary
6.10.2 The SHANSEP Procedure

SHANSEP stands for Stress History And Normalized Soil Engineering Properties. It is a procedure which uses the stress history (the profile of overconsolidation ratio (OCR) through the deposit) and normalized soil properties as a framework within which clay behavior can be incorporated into engineering design.

1. Evaluate the clay for uniformity over the area of interest.
2. If the deposit is found to be uniform, evaluate its stress history:
   a. Select a laboratory stress system to simulate ranges of expected in-situ stress systems.
   b. Select representative samples for testing.
   c. Consolidate samples under a confining stress ($\sigma_{vm}$) two to three times greater than the estimated preconsolidation pressures to “normally consolidate” the sample and overcome effects of sample disturbance.
   d. Allow samples to rebound from the laboratory maximum past pressure $\sigma_{vm}$ to desired OCR (Figure 6-18).
   e. Perform undrained shear tests on samples reconsolidated to desire OCR. The types of shear test used in this program are selected to model the various modes of failure (and therefore the failure surface orientations) appropriate to the field situation.

![Figure 6-18 SHANSEP Rebound](image-url)
3. The measured undrained shear strengths \( (s_u) \) are divided by their respective effective vertical consolidation stress \( (\sigma_{vc}) \) values to obtain a normalized plot of \( s_u/\sigma_{vc} \) as a function of the laboratory OCR = \( \sigma_{vm}/\sigma_{vc} \) (Figure 6-19). A normalized strength vs. OCR plot for various clays is shown in Figure 6-20.

![Figure 6-19 SHANSEP Normalized Strength Plot](image)

4. Apply the stress parameters to the stress history of Step 2 to give values of \( s_u \) for use in stability analyses. This is done at any elevation by multiplying the in-situ \( \sigma_{vo} \) or \( P_o \) by the laboratory \( s_u/\sigma_{vc} \) value as appropriate to the in-situ OCR at the elevation to give the corresponding \( s_u \) value.

The application of SHANSEP for a particular mode of failure is demonstrated in Figure 6-20 where (a) shows the stress history data used to obtain the average stress history profile of (b), (c) shows the \( s_u/\sigma_{vc} \) values obtained from laboratory tests as a function of OCR, (d) tabulates the computation of \( s_u \) and (e) shows the resulting of \( s_u \) profile.
Figure 6-20 Application of SHANSEP to Undrained Stability Analysis
EFFECT of PRECOMPRESSION on UNDRAINED SHEAR STRENGTH

Strength Ratio: \( \frac{(S_u/\sigma_v)_{OC}}{(S_u/\sigma_v)_{NC}} = OCR^m \)

Figure 6-21 SHANSEP Normalized Plot for Various Clays
The most involved part of the SHANSEP procedure is where the normalized strength parameters used in design are derived from a laboratory test program. For a more detailed discussion of SHANSEP, refer to Ladd and Fout (1974).

6.11 ENGINEERING CHARACTERISTICS OF SOIL

Laboratory index property testing is mainly used to classify soils, though in some cases, they can also be used with correlations to estimate specific soil design properties. Index tests include soil gradation and plasticity indices previously discussed in this chapter.

Laboratory performance testing is mainly used to estimate strength, compressibility, and permeability characteristics of soil. For soil, shear strength may be determined on either undisturbed specimens of finer grained soil (undisturbed specimens of granular soils are very difficult, if not impossible, to get), or disturbed or remolded specimens of fine or coarse grained soil as previously discussed in this chapter.
6.11.1 Organic Soils

Moisture content is a critical parameter for organic soils. The higher the organic content of a soil, the higher its moisture content can be and the less acceptable the material will be under Department facilities. The following figure illustrates nomenclature, moisture content, and general organic content for a variety of organic soils.

- Zone A:
  Fine sands and silts underlying organic deposits usually are loose and sometimes contain small amounts of organics that will darken the soil color. There soils will consolidate rapidly and will have adequate shear strength to support embankment weight, if left in place. Since these soils often appear to be “unsuitable” when excavated, moisture content tests on relatively undisturbed samples will be important for determining lower limit of excavation in field.
• Zone B:
  Soils may possibly be stabilized, depending on rate of consolidation, available shear strength, thickness of organic deposit, height of embankment, surcharge time before paving, and allowable post-construction settlement. Soils in Zone B at lower moisture contents (with exception of sensitive clays) usually have more favorable engineering properties. Strength and consolidation tests should be conducted on undisturbed samples when possible.

• Zone C:
  Soils are unsuitable for embankment foundation unless deposit is thin or has been precompressed by existing overburden. Otherwise, remove by excavation or displacement.

*Figure 6-23 Organic Material Percent by Volume of Total Sample*
### 6.11.2 Compacted Soils

<table>
<thead>
<tr>
<th>Group Symbol</th>
<th>Soil Type</th>
<th>Range of Maximum Dry Unit Weight, pcf</th>
<th>Range of Optimum Moisture Percent</th>
<th>Typical Value of Compression</th>
<th>Typical Strength Characteristics</th>
<th>Typical Coefficient of Permeability, ft/min.</th>
<th>Range of Subgrade Modulus, k 100 ksi/in²</th>
</tr>
</thead>
<tbody>
<tr>
<td>CW</td>
<td>Well graded clean gravel, gravel-sand mixtures.</td>
<td>125 - 135</td>
<td>8 - 18</td>
<td>0.3</td>
<td>9.6</td>
<td>30.79</td>
<td>40 - 80</td>
</tr>
<tr>
<td>GP</td>
<td>Poorly graded clean gravel, gravel-sand mix</td>
<td>115 - 125</td>
<td>14 - 11</td>
<td>0.4</td>
<td>0.9</td>
<td>30.74</td>
<td>10⁻¹</td>
</tr>
<tr>
<td>GM</td>
<td>Silty gravel, poorly graded gravel-sand-silt.</td>
<td>120 - 135</td>
<td>12 - 8</td>
<td>0.5</td>
<td>1.1</td>
<td>30.67</td>
<td>10⁻⁰</td>
</tr>
<tr>
<td>GC</td>
<td>Clayey gravel, poorly graded gravel-sand-clay.</td>
<td>115 - 130</td>
<td>14 - 9</td>
<td>0.7</td>
<td>1.8</td>
<td>30.60</td>
<td>10⁻⁰</td>
</tr>
<tr>
<td>SW</td>
<td>Well graded clean sand, gravelly sand.</td>
<td>110 - 120</td>
<td>16 - 9</td>
<td>0.6</td>
<td>1.2</td>
<td>30.79</td>
<td>10⁻⁰</td>
</tr>
<tr>
<td>SP</td>
<td>Poorly graded clean sand, semi-gravel mix.</td>
<td>100 - 120</td>
<td>21 - 12</td>
<td>0.8</td>
<td>1.4</td>
<td>30.74</td>
<td>10⁻⁰</td>
</tr>
<tr>
<td>SM</td>
<td>Silty sand, poorly graded sand-silt mix</td>
<td>110 - 125</td>
<td>16 - 11</td>
<td>0.8</td>
<td>1.6</td>
<td>30.67</td>
<td>10⁻⁰</td>
</tr>
<tr>
<td>DM-SC</td>
<td>Sandy-silt clay mix with slightly plastic fines.</td>
<td>110 - 130</td>
<td>15 - 11</td>
<td>1.0</td>
<td>1.4</td>
<td>30.66</td>
<td>10⁻⁰</td>
</tr>
<tr>
<td>SC</td>
<td>Clayey sand, poorly graded sand-clay-silt.</td>
<td>105 - 125</td>
<td>19 - 11</td>
<td>1.1</td>
<td>2.2</td>
<td>30.60</td>
<td>10⁻⁰</td>
</tr>
<tr>
<td>NL</td>
<td>Inorganic silts and clayey silts.</td>
<td>95 - 120</td>
<td>24 - 12</td>
<td>0.9</td>
<td>1.7</td>
<td>30.62</td>
<td>10⁻⁰</td>
</tr>
<tr>
<td>NL-CL</td>
<td>Mixture of inorganic silt and clay.</td>
<td>100 - 120</td>
<td>22 - 12</td>
<td>1.0</td>
<td>2.2</td>
<td>30.62</td>
<td>10⁻⁰</td>
</tr>
<tr>
<td>CL</td>
<td>Inorganic clays of low to medium plasticity.</td>
<td>95 - 120</td>
<td>24 - 12</td>
<td>1.3</td>
<td>2.5</td>
<td>30.62</td>
<td>10⁻⁰</td>
</tr>
<tr>
<td>GL</td>
<td>Organic silts and silt-clays, low plasticity.</td>
<td>80 - 100</td>
<td>33 - 21</td>
<td>1.1</td>
<td>2.2</td>
<td>30.62</td>
<td>10⁻⁰</td>
</tr>
<tr>
<td>MB</td>
<td>Inorganic clayey silts, elastic silts.</td>
<td>70 - 95</td>
<td>40 - 24</td>
<td>2.0</td>
<td>3.8</td>
<td>30.62</td>
<td>10⁻⁰</td>
</tr>
<tr>
<td>CH</td>
<td>Inorganic clays of high plasticity</td>
<td>75 - 105</td>
<td>36 - 19</td>
<td>2.6</td>
<td>3.9</td>
<td>30.62</td>
<td>10⁻⁰</td>
</tr>
<tr>
<td>DH</td>
<td>Organic clays and silt clays.</td>
<td>65 - 100</td>
<td>45 - 21</td>
<td>2.0</td>
<td>3.0</td>
<td>30.62</td>
<td>10⁻⁰</td>
</tr>
</tbody>
</table>

**Notes:**
1. All properties are for condition of "Standard Proctor" maximum density, except values of k and CSR which are for "modified Proctor" maximum density.
2. Typical strength characteristics are for effective strength envelopes and are obtained from USBR data.
3. Compression values are for vertical loading with complete lateral confinement.
4. (>) Indicates that typical property is greater than the value shown.
5. (---) Indicates insufficient data available for an estimate.

**Figure 6-24 Properties of Compacted Soils (NAVFAC DM-7.2)**
As stated in NYSDOT GDM Section 6.9.1 *Compaction Tests*, in order to evaluate the field density, it must be compared to a control density. Beginning in the late 1950’s and continuing through much of the 1960’s, a research program was advanced to develop a standard set of Moisture-Density Relationship graphs, or curves, for use as a quality assurance tool on DOT projects across New York State. These curves would supersede the necessity to develop a set of project-specific curves for every soil type encountered, thereby significantly reducing the time needed to determine the in-place density of earthwork materials to compare against specification requirements.

Soil compaction information was gathered from such sources as USDA Soil Conservation Service Soil Surveys, as well as existing moisture-density determinations developed for numerous individual projects ranging from Buffalo to Long Island. Next, Departmental Geotechnical Engineers and Technicians from several DOT Regions, or “Districts” as they were formerly known, conducted scores of additional tests on a variety of soil types.

All data was pooled, statistical analyses were applied, and a “best fit” method was graphically employed to generate the NYSDOT Compaction Control Curves. Figures 6-25 and 6-26 identifies Family 1 (Well-Graded) and Figure 6-27 identifies Family 2 (Sand).
Figure 6-25 Family 1 (Higher Range Density) Compaction Control Curves

LOCUS OF MAXIMUM DENSITY

FAMILY 1: To be used for well-graded soils which generally contain minor amounts of Silt and/or Clay. Also to be used for Silty or Clayey soils. (These soils may be of any color and may vary somewhat from the above descriptions.)

Location: Where encountered
CHAPTER 6
Engineering Properties of Soil and Rock

Figure 6-26 Family 1 (Lower Range Density) Compaction Control Curves
FAMILY 2: To be used for cohesionless soils. These soils may be of any color. They may range from clean sands to clean sands containing varying amounts of gravel which may be subrounded or rounded, and they include dredged sands. The soils may contain minor amounts of silt. They may be poorly-graded uniform or skip-graded.

Figure 6-27 Family 2 Compaction Control Curves
The existence of either a drained or an undrained condition in a soil depends on:

- The soil type (e.g. fine-grained or coarse-grained)
- Geological formation (fissures, sand layers in clays, etc.)
- Rate of loading – For a rate of loading associated with a normal construction activity, saturated coarse-grained soils (e.g. sands and gravels) experience drained conditions and saturated fine-grained soils (e.g. silts and clays) experience undrained conditions. If the rate of loading is fast enough (e.g. during an earthquake), coarse-grained soils can experience undrained loading, which may result in liquefaction.

An undrained condition occurs when the pore water is unable to drain out of the soil. In an undrained condition, the rate of loading is much quicker than the rate at which the pore water is able to drain out of the soil. As a result, most of the external loading is taken by the pore water, resulting in an increase in the pore water pressure. The tendency of soil to change volume is suppressed during undrained loading.

Figure 6-28 provides a relationship between the undrained shear strength and the moisture content based on compactive effort. It is important to ensure that compacted soils are protected against increases in moisture content because the strength of such soils will decrease, which may result in detrimental effects on the facilities they support.

![Figure 6-28 Undrained Shear Strength vs. Moisture Content for Bag Samples](image-url)
6.11.3 Permeability

As described in NYSDOT GDM Section 6.7.4 Permeability, the coefficient of permeability is dependent on void ratio, grain-size distribution, pore-size distribution, roughness of mineral particles, fluid viscosity, and degree of saturation. Tables 6-6 and 6-7 provide some typical values of permeability coefficients and Figures 6-29 and 6-30 provide values of permeability based on grain size.

![Table 6-6 Typical Values of Permeability Coefficients (After Hough)](image)

![Table 6-7 Range of Permeabilities of Natural Soils](image)
Figure 6-29 Permeability vs. Grain Size
Figure 6-30 Permeability vs. Grain Size
6.11.4 Drainage Capabilities of Soils

Drainable water moves through soils in direct proportion to the soil’s permeability (i.e. a low permeability results in the slow drainage of saturated soils). As described previously, the size and shape of soil pores and their connectivity determine soil permeability. The infiltration rate of a soil refers to how much water a type of soil can absorb over a specific time period. Infiltration rates are determined by soil permeability and surface conditions. Coarsely textured soils, including sand and gravel, generally have high soil permeabilities and high infiltration rates. Figure 6-31 provides some general drainage capabilities for assorted soil types.

Figure 6-31 Drainage Capabilities of Assorted Soil Types
6.12 ENGINEERING CHARACTERISTICS OF ROCK

Engineering properties of rock are generally controlled by the discontinuities within the rock mass and not the properties of the intact material. Therefore, engineering properties for rock must account for the properties of the intact pieces and for the properties of the rock mass as a whole, specifically considering the discontinuities within the rock mass. A combination of laboratory testing of small samples, empirical analysis, and field observations should be employed to determine the engineering properties of rock masses, with greater emphasis placed on visual observations and quantitative descriptions of the rock mass.

Rock properties can be divided into two categories: intact rock properties and rock mass properties. Intact rock properties are determined from laboratory tests on small samples typically obtained from coring, outcrops or exposures along existing cuts. Engineering properties typically obtained from laboratory tests include specific gravity, unit weight, ultrasonic velocity, compressive strength, tensile strength, and shear strength. Rock mass properties are determined by visual examination of discontinuities within the rock mass, and how these discontinuities will affect the behavior of the rock mass when subjected to the proposed construction.

The methodology and related considerations provided by Sabatini, et al. (2002) should be used to assess the design properties for the intact rock and the rock mass as a whole. However, the portion of Sabatini, et al. (2002) that addresses the determination of fractured rock mass shear strength parameters (Hoek and Brown, 1988) is outdated. The original work by Hoek and Brown has been updated and is described in Hoek, et al. (2002). The updated method uses a Geological Strength Index (GSI) to characterize the rock mass for the purpose of estimating strength parameters, and has been developed based on re-examination of hundreds of tunnel and slope stability analyses in which both the 1988 and 2002 criteria were used and compared to field results. While the 1988 method has been more widely published in national (e.g., FHWA) design manuals than has the updated approach provided in Hoek, et al. (2002), considering that the original developers of the method have recognized the short-comings of the 1988 method and have reassessed it through comparison to actual rock slope stability data, NYSDOT considers the Hoek, et al. (2002) to be the most accurate methodology. Therefore the Hoek, et al. (2002) method should be used for fractured rock mass shear strength determination. Note that this method is only to be used for highly fractured rock masses in which the stability of the rock slope is not structurally controlled. See NYSDOT GDM Chapter 15 for additional requirements regarding the assessment of rock mass properties.
6.12.1 Unit Weight of Rock

Scaling of rock slopes includes the removal of loose overhangs, weathered pockets or unconnected rock from the slope. See NYSDOT GDM Chapter 15.

Scaling operations proceed either manually (with hand tools), or by a mechanical device designed to catch onto and pull loose rock and other debris from the slope. In addition, rock and debris which cannot be removed by manual or mechanical means but constitutes a potential problem may be removed by drilling and blasting.

The specification for scaling requires a Special Note for the conversion of weight to volume along with an estimated quantity of material to be removed. This Special Note contained sections for the Designer to fill-in with the unit weight of the site specific material provided by an Engineering Geologist. Table 6-8 provides the conversion factors for assorted rock types.
<table>
<thead>
<tr>
<th>UNIT WEIGHT (lb/ft²)</th>
<th>UNIT WEIGHT (kN/m³)</th>
<th>ROCK TYPE</th>
<th>CONVERSION FACTOR (English) (yd³/lb)</th>
<th>CONVERSION FACTOR (Metric) (m³/kg)</th>
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<tr>
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<td>29.85</td>
<td>BASALT</td>
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Table 6-8 Unit Weight of Assorted Rock Types
### 6.12.2 Average Weight of Various Materials Blasted

<table>
<thead>
<tr>
<th>MATERIAL</th>
<th>Specific Gravity</th>
<th>SOLID</th>
<th></th>
<th>BROKEN</th>
<th></th>
</tr>
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<tbody>
<tr>
<td></td>
<td></td>
<td>Pounds per Cu. Ft.</td>
<td>Cu. Ft. per Ton</td>
<td>Tons per Cu. Yd.</td>
<td>Pounds per Cu. Ft.</td>
</tr>
<tr>
<td>Basalt</td>
<td>2.8-3.0</td>
<td>190</td>
<td>10.5</td>
<td>2.57</td>
<td>125</td>
</tr>
<tr>
<td>Coal-Anthracite</td>
<td>1.3-1.8</td>
<td>100</td>
<td>20.0</td>
<td>1.35</td>
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</tr>
<tr>
<td>Coal-Bituminous</td>
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<tr>
<td>Diabase</td>
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<td>6.4</td>
<td>4.25</td>
<td>205</td>
</tr>
<tr>
<td>Marble</td>
<td>2.1-2.9</td>
<td>155</td>
<td>12.8</td>
<td>2.09</td>
<td>100</td>
</tr>
<tr>
<td>Mica-Schist</td>
<td>2.5-2.9</td>
<td>170</td>
<td>11.8</td>
<td>2.30</td>
<td>110</td>
</tr>
<tr>
<td>Porphyr</td>
<td>2.5-2.6</td>
<td>160</td>
<td>12.5</td>
<td>2.16</td>
<td>105</td>
</tr>
<tr>
<td>Quartzite</td>
<td>2.0-2.8</td>
<td>160</td>
<td>12.5</td>
<td>2.16</td>
<td>105</td>
</tr>
<tr>
<td>Salt-Rock</td>
<td>2.1-2.6</td>
<td>145</td>
<td>13.8</td>
<td>1.96</td>
<td>95</td>
</tr>
<tr>
<td>Sandstone</td>
<td>2.0-2.8</td>
<td>150</td>
<td>13.3</td>
<td>2.03</td>
<td>95</td>
</tr>
<tr>
<td>Shale</td>
<td>2.4-2.8</td>
<td>160</td>
<td>12.5</td>
<td>2.16</td>
<td>105</td>
</tr>
<tr>
<td>Silica Sand</td>
<td>2.2-2.8</td>
<td>160</td>
<td>12.5</td>
<td>2.16</td>
<td>105</td>
</tr>
<tr>
<td>Slate</td>
<td>2.5-2.8</td>
<td>170</td>
<td>11.6</td>
<td>2.30</td>
<td>110</td>
</tr>
<tr>
<td>Talc</td>
<td>2.6-2.8</td>
<td>165</td>
<td>12.2</td>
<td>2.23</td>
<td>110</td>
</tr>
<tr>
<td>Trap Rock</td>
<td>2.6-3.0</td>
<td>175</td>
<td>11.4</td>
<td>2.36</td>
<td>115</td>
</tr>
</tbody>
</table>

Table 6-9 Average Weights of Materials Before and After Blasting
6.12.3 Description of Rock Quality

See NYSDOT GDM Chapter 5 for a discussion on Rock Quality Designation.

<table>
<thead>
<tr>
<th>RQD (Rock Quality Designation)</th>
<th>Description of Rock Quality</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 – 25</td>
<td>Very Poor</td>
</tr>
<tr>
<td>25 – 50</td>
<td>Poor</td>
</tr>
<tr>
<td>50 – 75</td>
<td>Fair</td>
</tr>
<tr>
<td>75 – 90</td>
<td>Good</td>
</tr>
<tr>
<td>90 – 100</td>
<td>Excellent</td>
</tr>
</tbody>
</table>

Table 6-10 Description of Rock Quality

6.12.4 Allowable Bearing Pressure for Footings on Rock

<table>
<thead>
<tr>
<th>Material</th>
<th>RQD</th>
<th>Allowable Contact Pressure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Such igneous and sedimentary rock as crystalline bedrock, including</td>
<td>RQD = 75 to 100 %</td>
<td>120 tsf (11491 kPa)</td>
</tr>
<tr>
<td>granite, diorite, gneiss, traprock; and hard limestone, and dolomite,</td>
<td>RQD = 50 to 75 %</td>
<td>65 tsf (6224 kPa)</td>
</tr>
<tr>
<td>in sound condition:</td>
<td>RQD = 25 to 50 %</td>
<td>30 tsf (2873 kPa)</td>
</tr>
<tr>
<td></td>
<td>RQD = 0 to 25 %</td>
<td>10 tsf (958 kPa)</td>
</tr>
<tr>
<td>Such metamorphic rock as foliated rocks, such as schist or slate; and</td>
<td>RQD &gt; 50 %</td>
<td>40 tsf (3830 kPa)</td>
</tr>
<tr>
<td>bedded limestone, in sound condition:</td>
<td>RQD &lt; 50 %</td>
<td>10 tsf (958 kPa)</td>
</tr>
<tr>
<td>Sedimentary rocks, including hard shales and sandstones, in sound</td>
<td>RQD &gt; 50 %</td>
<td>25 tsf (2394 kPa)</td>
</tr>
<tr>
<td>condition:</td>
<td>RQD &lt; 50 %</td>
<td>10 tsf (958 kPa)</td>
</tr>
<tr>
<td>Soft or broken bedrock (excluding shale), and soft limestone:</td>
<td>RQD &gt; 50 %</td>
<td>12 tsf (1149 kPa)</td>
</tr>
<tr>
<td></td>
<td>RQD &lt; 50 %</td>
<td>8 tsf (766 kPa)</td>
</tr>
<tr>
<td>Soft shale:</td>
<td></td>
<td>4 tsf (383 kPa)</td>
</tr>
</tbody>
</table>

Table 6-11 Recommended Allowable Bearing Pressure for Footings on Rock
6.13 FINAL SELECTION OF DESIGN VALUES

6.13.1 Overview

After the field and laboratory testing is completed, the geotechnical designer should review the quality and consistency of the data, and should determine if the results are consistent with expectations. Once the lab and field data have been collected, the process of final material property selection begins. At this stage, the Departmental Geotechnical Engineer generally has several sources of data consisting of that obtained in the field, laboratory test results and correlations from index testing.

- Drill logs (casing blows, spoon blows, water table, ground elevation, artesian, soil breaks).
- Laboratory visuals (soil description, moisture content).
- Topographic sheets (contours, man-made features).
- Soil Maps (soil breaks, nature of topsoil).
- Air photos (swamps, surface soils, man-made features, old failures, drainage patterns, landforms).
- Well data (water elevation, artesian)
- Previous reports (soils in the area).
- Laboratory data (P.I., grain size, strengths, consolidation data, permeability).
- Field Inspections.

In addition, the Departmental Geotechnical Engineer may have experience based on other projects in the area or in similar soil/rock conditions. Therefore, if the results are not consistent with each other or previous experience, the reasons for the differences should be evaluated, poor data eliminated and trends in data identified. At this stage it may be necessary to conduct additional performance tests to try to resolve discrepancies.

As stated in NYSDOT GDM Section 6.1 Overview, the focus of geotechnical design property assessment and final selection is on the individual geologic strata identified at the project site. A geologic stratum is characterized as having the same geologic depositional history and stress history, and generally has similarities throughout the stratum in its density, source material, stress history, and hydrogeology. All of the information that has been obtained up to this point including preliminary office and field reconnaissance, boring logs, CPT soundings etc., and laboratory data are used to determine soil and rock engineering properties of interest and develop a subsurface model of the site to be used for design. Data from different sources of field and lab tests, from site geological characterization of the site subsurface conditions, from visual observations obtained from the site reconnaissance, and from historical experience with the subsurface conditions at or near the site must be combined to determine the engineering properties for the various geologic units encountered throughout the site. However, soil and rock properties for design should not be averaged across multiple strata, since the focus of this property characterization is on the individual geologic stratum. Often, results from a single test (e.g. SPT N-values) may show significant scatter across a site for a given soil/rock unit. Perhaps data obtained from a particular soil unit for a specific property from two different tests (e.g. field vane shear tests and lab UU tests) do not agree. Techniques should be employed to determine the validity and reliability of the data and its usefulness in selecting final design parameters. After a
review of data reliability, a review of the variability of the selected parameters should be carried out. Variability can manifest itself in two ways: 1) the inherent in-situ variability of a particular parameter due to the variability of the soil unit itself, and 2) the variability associated with estimating the parameter from the various testing methods. From this step, final selection of design parameters can commence and, from there, completion of the subsurface profile.

6.13.2 Data Reliability and Variability

Inconsistencies in data should be examined to determine possible causes and assess any mitigation procedures that may be warranted to correct, exclude, or downplay the significance of any suspect data. The following procedures provide a step-by-step method for analyzing data and resolving inconsistencies as outlined by Sabatini, et al. (2002):

1) Data Validation: Assess the field and the laboratory test results to determine whether the reported test results are accurate and are recorded correctly for the appropriate material. For lab tests on undisturbed samples, consider the effects of sample disturbance on the quality of the data. For index tests (e.g. grain size, compaction), make sure that the sample accurately represents the in-situ condition. Disregard or downplay potentially questionable results.

2) Historical Comparison: Assess results with respect to anticipated results based on site and/or regional testing and geologic history. If the new results are inconsistent with other site or regional data, it will be necessary to assess whether the new data is anomalous or whether the new site conditions differ from those from which previous data was collected. For example, an alluvial deposit might be expected to consist of medium dense Silty SAND with SPT blow counts less than 30. If much higher blow counts are recorded, the reason could be the deposit is actually dense (and therefore higher friction angles can be assumed), or gravel may be present and is influencing the SPT data. Most likely it is the second case, and the engineering properties should probably be adjusted to account for this. But if consideration had not been given as to what to expect, values for properties might be used that could result in an unconservative design.

3) Performance Comparison: Assess results with respect to historic performance of structures at the site or within similar soils. Back analysis of previous landslides and retaining wall movement in the same geologic units under consideration, if available, should be performed to estimate shear strength parameters. Settlement data from existing embankments, if available, should be used to estimate compressibility and settlement rates. Results can be compared to the properties determined from field and lab testing for the project site. The newly collected data can be correlated with the parameters determined from observation of performance and the field and lab tests performed for the previous project.

4) Correlation Calibration: If feasible, develop site-specific correlations using the new field and lab data. Assess whether this correlation is within the range of variability typically associated with the correlation based on previous historic data used to develop the generic correlation.

5) Assess Influence of Test Complexity: Assess results from the perspective of the tests themselves. Some tests may be easy to run and calibrate, but provide data of a “general” nature while other tests are complex and subject to operator influence, yet provide
“specific” test results. When comparing results from different tests consider which tests have proven to give more accurate or reliable results in the past, or more accurately approximate anticipated actual field conditions. For example, results of field vane shear tests may be used to determine undrained shear strength for deep clays instead of laboratory UU tests because of the differences in stress states between the field and lab samples. It may be found that certain tests consistently provide high or low values compared to anticipated results.

The result of these steps is to determine whether or not the data obtained for the particular tests in question is valid. Where it is indicated that test results are invalid or questionable, the results should be downplayed or thrown out. If the test results are proven to be valid, the conclusion can be drawn that the soil unit itself and its corresponding engineering properties are variable (vertically, aerially, or both).

The next step is to determine the amount of variability that can be expected for a given engineering property in a particular geologic unit, and how that variability should influence the selection of the final design value. Sabatini, et al. (2002) list several techniques that can be used:

1) Experience: In some cases, the Departmental Geotechnical Engineer may have accumulated extensive experience in the region such that it is possible to accurately select an average, typical or design value for the selected property, as well as the appropriate variability for the property.
2) Statistics: If a Departmental Geotechnical Engineer has extensive experience in a region, or there has been extensive testing by others with published or available results, there may be sufficient data to formally establish the average value and the variability (mean and standard deviation) for the specific property. See Sabatini, et al. (2002) and Phoon, et al. (1995) for information on the variability associated with various engineering properties.
3) Establish Best-Case and Worst-Case Scenarios: Based on the experience of the Departmental Geotechnical Engineer, it may be possible to establish upper and lower bounds along with the average for a given property.

6.13.3 Final Property Selection

The final step is to incorporate the results of the previous section into the selection of design values for required design properties.

Recognizing the variability discussed in the previous section, depending on the amount of variability estimated or measured, the potential impact of that variability (or uncertainty) on the level of safety in the design should be assessed. If the impact of this uncertainty is likely to be significant, parametric analyses should be conducted, or more data could be obtained to help reduce the uncertainty. Since the sources of data that could be considered may include both measured laboratory data, field test data, performance data, and other previous experience with the geologic unit(s) in question, it will not be possible to statistically combine all this data together to determine the most likely property value. Engineering judgment, combined with
parametric analyses as needed, will be needed to make this final assessment and design property determination. At that point, a decision must be made as to whether the final design value selected should reflect the interpreted average value for the property, or a value that is somewhere between the most likely average value and the most conservative estimate of the property. However, the desire for design safety must be balanced with the cost effectiveness and constructability of the design. In some cases, being too conservative with the design could result in an un-constructible design (e.g., the use of very conservative design parameters could result in a pile foundation that must be driven deep into a very dense soil unit that in reality is too dense to penetrate with available equipment).

Note that, in NYSDOT GDM Chapter 11, where reliability theory was used to establish load and resistance factors, the factors were developed assuming that mean values for the design properties are used. However, even in those cases, design values that are more conservative than the mean may still be appropriate, especially if there is an unusual amount of uncertainty in the assessment of the design properties due, for example, to highly variable site conditions, lack of high quality data to assess property values, or due to widely divergent property values from the different methods used to assess properties within a given geologic unit. Depending on the availability of soil or rock property data and the variability of the geologic strata under consideration, it may not be possible to reliably estimate the average value of the properties needed for design. In such cases, the Departmental Geotechnical Engineer may have no choice but to use a more conservative selection of design parameters to mitigate the additional risks created by potential variability or the paucity of relevant data. Note that, for those resistance factors that were determined based on calibration by fitting to allowable stress design, this property selection issue is not relevant, and property selection should be based on the considerations discussed previously.

The process and examples to make the final determination of properties to be used for design provided by Sabatini, et al. (2002) should be followed.

### 6.13.4 Development of the Subsurface Profile

While NYSDOT GDM Section 6.9 *Laboratory Testing for Determination of Soil Long-Term Permanence Properties* generally follows a sequential order, it is important to understand that the selection of design values and production of a subsurface profile is more of an iterative process. The development of design property values should begin and end with the development of the subsurface profile. Test results and boring logs will likely be revisited several times as the data is developed and analyzed before the relation of the subsurface units to each other and their engineering properties are finalized.

The ultimate goal of a subsurface investigation is to develop a working model that depicts major subsurface layers exhibiting distinct engineering characteristics. The end product is the subsurface profile, a two dimensional depiction of the site stratigraphy. The following steps outline the creation of the subsurface profile:

1) Complete the field and lab work and incorporate the data into the preliminary logs.
2) Lay out the logs relative to their respective field locations and compare and match up the
different soil and rock units at adjacent boring locations, if possible. However, caution should be exercised when attempting to connect units in adjacent borings, as the geologic stratigraphy does not always fit into nice neat layers. Field descriptions and engineering properties will aid in the comparisons.

3) Group the subsurface units based on engineering properties.

4) Create cross sections by plotting borings at their respective elevations and positions horizontal to one another with appropriate scales. If appropriate, two cross sections should be developed that are at right angles to each other so that lateral trends in stratigraphy can be evaluated when a site contains both lateral and transverse extents (i.e. a building or large embankment).

5) Analyze the profile to see how it compares with expected results and knowledge of geologic (depositional) history. Have anomalies and unexpected results encountered during exploration and testing been adequately addressed during the process? Make sure that all of the subsurface features and properties pertinent to design have been addressed.

Subsurface sections used during the design phase will portray the soils in an interpretive manner. The sections should be chosen so as to present in the best possible manner the conditions described. A clear differentiation should always be maintained between factual and interpretive data. The cross section should always show the name of the person who made the interpretation and the date of the interpretation.

Where subsurface sections are to be included in contract documents, the information shown is limited to factual data such as the ground surface line and the logs of drill holes located in their actual position with respect to the ground surface line. Actual locations of features such as bedrock, water table, etc. are not shown by continuous lines but only where they are encountered in each hole.
Figure 6-32 General Subsurface Profile
6.14 REFERENCES


Sowers, G.F. and Hedges, C.S., *Dynamic Cone for Shallow In-Situ Penetration Testing*, Vane Shear and Cone Penetration Resistance Testing of In-Situ Soils, ASTM STP399, 1966


