Chapter 5 Engineering Properties of Soil and Rock

5.1 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 combination. Site performance data should also be used if available to help determine the final geotechnical properties for design. The geotechnical designer'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 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.

5.2 The Geologic Stratum as the Basis for Property Characterization

The development of soil and rock properties for geotechnical design purposes begins with developing/defining the geologic strata present at the site in question. Therefore, 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, stress history, and degree of disturbance, and generally has similarities throughout the stratum in terms of density, source material, stress history, hydrogeology, and macrostructure. The properties of each stratum shall be consistent with the stratum's geologic depositional and stress history, and macrostructure. Note that geologic units/formations identified in geologic maps may contain multiple geologic strata as defined in this GDM.

Once the geologic strata are defined, Engineering Stratagraphic Units (ESU's) are developed for the purpose of defining zones within the subsurface profile with similar properties for design. If there are multiple geologic strata as previously defined that have approximately the same engineering properties, multiple geologic strata may be grouped into a single ESU to simplify the design. However, soil and rock properties for design should not be averaged across multiple geologic strata except as noted later in this section, or unless averaging the properties results in an insignificant difference in the design outcome. If it is not clear that averaging the properties together will have an insignificant difference in the design outcome, the most conservative value of the property in question for the strata grouped together into one ESU should be used for design, or the strata should not be grouped together into one ESU.

The properties of a given geologic stratum at a project site may 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. It should also be recognized that some properties (e.g., undrained shear strength in normally consolidated clays) may vary as a 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 shall be developed taking this variation into account, which may result in multiple values of the property within the stratum and therefore multiple ESU's within the stratum.

Since ESU's are defined as zones of soil or rock with consistent engineering properties, properties of ESU's shall not be averaged together, except as noted in the following sentences. For design methods that require a very simplified stratigraphy be used, to create the simplified stratigraphy, a weighted average of the properties from each ESU based on the design ESU thickness should be used to estimate the properties of the simplified ESU for the design method in question. An example of this approach is provided in the AASHTO LRFD Bridge Design Specifications, Article C3.10.3.1, in particular Table 1 of that article. However, there is a significant risk that weaker materials, seams, layers, or structures (e.g., fractures, fissures, slickensides) within a stratum or ESU will dominate the performance of the geotechnical structure being designed, the design properties selected shall reflect the weakest aspects of the stratum or ESU rather than taking a weighted average.

5.3 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, changes in stresses, the presence of water, rate and direction of loading, and time 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 facility that is constructed. Normally consolidated clays can gain strength with increases in effective stress, and overconsolidated clays may lose strength with time when exposed in cuts, unloaded, or exposed to water. Some construction materials such as weak rock may lose strength due to weathering within the design life of the embankment. These long-term effects shall be considered when selecting properties to use for design.

5.4 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

The two most common in-situ test methods for use in soil are the Standard Penetration Test, (SPT) and the cone penetrometer test (CPT). Section 5.4 describes these tests as well as other in-situ tests. 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.

Back-analysis is used to tie the soil or rock properties to the quantifiable performance of the slope, embankment, wall, or foundation (see Section 5.7).

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.

5.5 In-Situ Field Testing

Standards and details regarding field tests such as the Standard Penetration Test (SPT), the Cone Penetrometer Test (CPT), the vane shear test, and other tests and their use provided in Sabatini et al. (2002) should be followed, except as specifically noted herein. Regarding Standard Penetration Tests (SPT), the N-values obtained in the field depend on the equipment used and the skill of the operator, and shall be corrected before they are used in design so that they are consistent with the design method and correlations being used. Many of the correlations developed to determine soil properties are based on N60-values.

SPT N values shall be corrected for hammer efficiency, if applicable to the design method or correlation being used, using the following relationship.

$$N_{60} = (ER/60\%)\ N \tag{5-1}$$

Where:

N	=	uncorrected SPT value (blows/ft)
N60	=	SPT blow count corrected for hammer efficiency (blows/ft)
ER	=	Hammer efficiency expressed as percent of theoretical free fall energy delivered by the hammer system actually used.

The following values for ER may be assumed if hammer specific data are not available:

ER	=	60% for conventional drop hammer using rope and cathead
ER	=	80% for automatic trip hammer

Hammer efficiency (ER) for specific hammer systems used in local practice may be used in lieu of the values provided. If used, specific hammer system efficiencies shall be developed in general accordance with ASTM D-4633 for dynamic analysis of driven piles or other accepted procedure. See Chapter 3 for additional information on ER, including specific measurements conducted for WSDOT drilling equipment.

Corrections for rod length, hole size, and use of a liner may also be made if appropriate. In general, these are only significant in unusual cases or where there is significant variation from standard procedures. These corrections may be significant for evaluation of liquefaction. Information on these additional corrections may be found in: "Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils"; Publication Number: MCEER-97-0022; T.L. Youd, I.M. Idriss (1997).

N-values are also affected by overburden pressure, and shall be corrected for that effect, if applicable to the design method or correlation being used. N values corrected for both overburden and the efficiency of the field procedures used shall be designated as N160. The overburden correction equation that should be used is:

$$N1_{60} = C_N N_{60} \quad (5\text{-}2)$$

Where:

C_N = [0.77 log10 (20/σ′v)], CN < 2.0 (5-3)

C_N = correction factor for overburden

N_{60} = N-value corrected for energy efficiency

σ'_v = vertical effective stress at the location of the SPT N-value (TSF)

In general, correlations between N-values and soil properties should only be used for cohesionless soils, and sand in particular. Caution should be used when using N-values obtained in gravelly soil. Gravel particles can plug the sampler, resulting in higher blow counts and estimates of friction angles than actually exist. Caution should also be used when using N-values to determine silt or clay parameters, due to the dynamic nature of the test and resulting rapid changes in pore pressures and disturbance within the deposit. Correlations of N-values with cohesive soil properties should generally be considered as preliminary. N-values can also be used for liquefaction analysis. See Chapter 6 for more information regarding the use of N-values for liquefaction analysis.

In general design practice, hydraulic conductivity is estimated based on grain size characteristics of the soil strata (see Highway Runoff Manual M 31-16, Section 4-5). In critical applications, the hydraulic conductivity may be determined through in-situ testing. A discussion of field measurement of permeability is presented in Sabatini et al. (2002) and Mayne et al. (2002), and ASTM D 4043 presents a guide for the selection of various field methods. If in-situ test methods are utilized to determine hydraulic conductivity, one or more of the following methods should be used:

- Well pumping tests
- Packer permeability tests
- Seepage Tests
- Slug tests
- Piezocone tests
- Flood tests or Pit Infiltration Tests (PIT) – applies mainly to infiltration facility design – see Section 4-5 of the Highway Runoff Manual (2004) M 31-16.

5.5.1 Well Pumping Tests

Pump tests can be used to provide an estimate of the overall hydraulic conductivity of a geologic formation, and since it is in essence a full scale test, it directly accounts for the layering and directionality of the hydraulic characteristics of the formation. The data provided can be used to determine the requirements for construction dewatering systems for excavations. However, pump tests can be quite expensive and can take a significant amount of time to complete. Furthermore, care must be exercised when conducting this type of test, especially if potentially contaminated zones are present that could be mobilized during pumping. This could also create problems with disposal of the pumped water. Impact to adjacent facilities, such as drinking wells and subsidence caused by dewatering, should be evaluated when planning this type of test. For this test, the method prescribed in ASTM D 4050 should be used. Analysis of the results of pumping tests requires experience and a thorough knowledge of the actual geologic conditions present at the test location. The time-drawdown response curves are unique to a particular geologic condition. Therefore, knowledge of the actual geologic conditions present at the test location is required in order to choose the correct analysis procedure, e.g., whether the aquifer is leaky, unconfined, or bounded, etc.

5.5.2 Packer Permeability Tests

Packer permeability tests can be used to measure the hydraulic conductivity of a specific soil or rock unit. The information obtained is used primarily in seepage studies. This test is conducted by inserting the packer units to the desired test location after the boring has been properly cleaned out. The packers are expanded to seal off the zone being tested, and water is injected into the borehole under constant pressure. Measurements of the flow rate are taken at regular time intervals. Upon completion of testing at a particular depth, the packers are lowered to a new test depth. Test depths should be determined from cores and geophysical logs of the borehole, prior to hydraulic conductivity testing. Note that if the packer test is performed in soil borings, casing must be installed. See Mayne et al. (2002) for additional information on this type of test.

5.5.3 Seepage Tests

Three types of seepage tests are commonly used: falling head, rising head and constant water level methods. In general, either the rising or falling level methods should be used if the hydraulic conductivity is low enough to permit accurate determination of the water level. In the falling head method, the borehole or piezometer is filled with water that is allowed to seep into the soil. The rate of drop of the water surface in the casing is monitored. The rising head method consists of bailing the water out of the borehole and observing the rate of rise until the change becomes negligible. The constant water level method is used if soil is too permeable to allow accurate measurement of the rising or falling water level. General guidance on these types of tests is provided in Mayne et al. (2002).

Boreholes (or in subsequently installed piezometers) in which seepage tests are to be performed should be drilled using only clear water as the drilling fluid. This precludes the formation of a mud cake on the walls of the hole or clogging of the soil pores with drilling mud. The tests can be performed intermittently as the borehole is advanced. In general, the rising head test is preferred because there is less chance of clogging soil pores with suspended sediment.

Data from seepage tests only reflect the hydraulic conditions near the borehole. In addition the actual area of seepage at the base of the borehole may not be accurately known. During the rising head test, there is the danger of the soil at the bottom of the borehole becoming loosened or “quick” if too great a gradient is imposed. However, seepage tests can be used in soils with lower hydraulic conductivities than is generally considered suitable for pumping tests and if large volumes of water do not require disposal. Also note that if the test is conducted inside the piezometer, the hydraulic conductivity measured from this could be influenced by the material placed inside the borehole around the screened pipe.

5.5.4 Slug Tests

These tests are easy to perform and can be performed in a borehole in which a screened pipe is installed. Two types of slug tests are commonly used, falling head and rising head. Falling head slug tests are conducted by lowering a solid object such as a weighted plastic cylinder into the borehole causing an instantaneous water level rise. As the water level gradually returns to static, the rate is recorded. A rising head slug test can then be performed by suddenly removing the slug, causing an instantaneous lowering of the water level. By monitoring the rate of rise or fall of the water level in the borehole, an estimate of the hydraulic conductivity can be determined. For this test, the method prescribed in ASTM D 4044 should be used. However slug tests are not very reliable and may underestimate hydraulic conductivity by one or two orders of magnitude, particularly if the test well has been inadequately developed prior to testing. The test data will not provide an indication of the accuracy of the computed value unless a pumping test is done in conjunction with the slug test. Because the slug tests are short duration, they reflect hydraulic properties of the soil immediately surrounding the well intake.

5.4.5 Piezocone Tests

Details of the equipment and methodology used to conduct the piezocone test are provided in Sabatini et al. (2002). Piezocone data can be useful to estimate the hydraulic conductivity of silts and clays from interpretation of the coefficient of horizontal consolidation, ch, obtained from the piezocone measurements. The procedure involves pushing the cone to the desired depth, followed by recording pore pressures while the cone is held stationary. The test is usually run until 50 percent of the excess pore pressure has dissipated (t50). This requires knowledge of the initial in situ pore pressure at the test location. Dissipation tests are generally effective in silts and clays where large excess pore pressures are generated during insertion of the cone. Hydraulic conductivity can be estimated using various correlations with t50 and coefficient of horizontal consolidation (ch), (see Lunne et al. (1997), and Sabatini et al. (2002)). Estimation of hydraulic conductivity from CPT tests is subject to a large amount of uncertainty, and should be used only as a preliminary estimate of permeability.

5.5.6 Flood Tests

Flood tests or pilot infiltration tests are not always feasible, and in general are only used where unusual site conditions are encountered that are poorly modeled by correlation to soil gradation characteristics, and there is plenty of water available to conduct the test. The key to the success of this type of test is the estimate of the hydraulic gradient during the test, recognizing that the test hydraulic gradient could be much higher than the hydraulic gradient that is likely in service for the facility being designed. For more information, see the Highway Runoff Manual (2004).

5.6 Laboratory Testing of Soil and Rock

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 behave 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.

5.6.1 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 Chapter 3 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. Carefully handle samples during extrusion of samples; samples must be extruded properly and supported upon their exit from the tube.
3. Avoid long-term storage of soil samples in Shelby tubes.
4. Properly number and identify samples.
5. Store samples in properly controlled environments.
6. Visually examine and identify soil samples after removal of smear from the sample surface.
7. Use pocket penetrometer or miniature vane only for an indication of strength.
8. Carefully select “representative” specimens for testing.
9. Have a sufficient number of samples to select from.
10. Always consult the field logs for proper selection of specimens.
11. Recognize disturbances caused by sampling, the presence of cuttings, drilling mud or other foreign matter and avoid during selection of specimens.
12. Do not depend solely on the visual identification of soils for classification.
13. Always perform organic content tests when classifying soils as peat or organic. Visual classifications of organic soils may be very misleading.
14. Do not dry soils in overheated or underheated ovens.
15. 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.
16. Performance of Atterberg limits requires carefully adjusted drop height of the Liquid Limit machine and proper rolling of Plastic Limit specimens.
17. Do not use tap water for tests where distilled water is specified.
18. Properly cure stabilization test specimens.

19. Never assume that all samples are saturated as received.
20. Saturation must be performed using properly staged back pressures.
21. Use properly fitted o-rings, membranes, etc. in triaxial or permeability tests.
22. Evenly trim the ends and sides of undisturbed samples.
23. Be careful to identify slickensides and natural fissures. Report slickensides and natural fissures.
24. Also do not mistakenly identify failures due to slickensides as shear failures.
25. Do not use unconfined compression test results (stress-strain curves) to determine elastic modulus values.
26. Incremental loading of consolidation tests should only be performed after the completion of each primary stage.
27. Use proper loading rate for strength tests.
28. Do not guesstimate e-log p curves from accelerated, incomplete consolidation tests.
29. Avoid "Reconstructing" soil specimens, disturbed by sampling or handling, for undisturbed testing.
30. Correctly label laboratory test specimens.
31. Do not take shortcut: such as using non-standard equipment or non-standard test procedures.
32. Periodically calibrate all testing equipment and maintain calibration records.
33. Always test a sufficient number of samples to obtain representative results in variable material.

5.6.2 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 tests should be selected to provide the desired and necessary data as economically as possible. Specific geotechnical information requirements are provided in the GDM chapters that address design of specific types of geotechnical features. 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
- Input property data needed for specific design procedures

Details regarding specific types of laboratory tests and their use are provided in Sabatini et al. (2002). Specifics regarding what is required in a laboratory testing plan is provided in Section 2.4.

5.7 Back-Analysis Based on Known Performance or Failure

Back-analysis to determine engineering properties of soil or rock is most often used with geotechnical failures. When failures occur, back analysis can be used to model the conditions, and loads which resulted in failure. Back-analysis can also be used in some situations where failure has not occurred but the geotechnical performance can be quantified (e.g., deformations). Back-analysis is a quantitative approach to adjust soil or rock properties to match measureable site performance.

To successfully carry out this approach, it is important to define the site geometry and stratigraphy, geologic history of the subsurface strata to be encountered, loading conditions, ground water conditions, and measurable soil properties. Since there are typically a number of variables to consider in most back-analyses (e.g., soil shear strength and unit weight of each stratum/ESU, the stratigraphy itself, the groundwater regime, the failure or deformation mechanism, the amount of deformation that has occurred, the location of the failure surface, the loading that occurred at the time the observed behavior occurred, etc.), all of the variables need to be defined before conducting the back-analysis so that the parameter of interest can be determined in a meaningful way.

Transient loading such as construction equipment live load shall not be included in the back-analysis, unless the transient load clearly caused failure (i.e., slope failed while equipment was on slope). If transient loads are included in the back-analysis, the rate of loading and its effect on the soil properties shall be addressed in the analysis.

To that end, the parameters used for the back-analysis shall be determined in a way that is consistent with the requirements provided in this manual. The back-analysis is then used to adjust the parameter of interest so that predicted behavior is consistent with the observed behavior. The observed behavior must be measurable in some way so that consistency between the observed and predicted behavior is quantifiably recognizable. If the behavior/performance is not quantifiable, then back-analysis will not be meaningful for determining or verifying design parameters.

If a back-analysis is to be conducted, the considerations and recommendations provided by Duncan and Stark (1992) shall be used. While the Duncan and Stark paper was written with regard to application to back-analysis of slope failures, the principles provided are generally applicable to other back-analysis situations.

5.7.1 Back-Analysis of Slopes

With landslides or slope failures, if the factor of safety for the slope is to be used as the performance measurement, a slope factor of safety of 1.0 shall be used, and shall accurately model the failure surface geometry and failure mechanism (Turner and Schuster 1996). It is important to determine or estimate the conditions that initiated the slope failure to successfully back-analyze the slope failure. See Stark, et al. (2011) for the principles that should be used to conduct slope failure back-analyses and a detailed example.

For first time slides, and slides in which the total historical deformation is relatively small, it shall be recognized that the shear strength estimated from the back-analysis is the mobilized shear strength at time of failure, not necessarily the residual shear strength, as the full development of residual strength conditions depends on the amount of deformation that has occurred along the slide failure surface (Hussain et al. 2010, Stark et al. 2011). In first time slides, the back-calculated shear strength is likely to be closer to the fully softened shear strength than the residual shear strength. Laboratory shear strength testing to measure the residual shear strength of the deposit should also be conducted and used in combination with the back-analyzed parameters for design purposes.

5.7.2 Back-Analysis of Soil Settlement Resulting from Changes in Loading

For embankment settlement, the performance measurement to be used is typically the magnitude of settlement measured, the rate at which the settlement occurred, or both. Pore pressure changes that occurred during embankment placement may also be used to help assess the rate of strength gain in soft compressible soils. If the embankment is reinforced with geosynthetic, strain in the geosynthetic should also be measured and used for back-analysis purposes. 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 be determined if the magnitudes of the loads and structural details are known. As with slope stability analysis, the stratigraphy of the subsurface soil must be adequately known, including the history of the groundwater level at the site.

5.7.3 Back-Analysis of Foundations

Essentially, use of foundation load tests to measure foundation bearing resistance and deflection characteristics is a form of back-analysis, when such data is used to estimate soil properties, enabling the prediction of foundation performance in adjacent areas where the same soil or rock strata are encountered, but the thickness of the strata/ ESU's are different.

5.7.4 Use of Numerical Modeling for Back-Analysis

Numerical models typically have many degrees of freedom, and high quality input data is usually required to use such a complex tool for this purpose. If numerical models are used, they shall have gone through a calibration process for a similar situation. Approval by the WSDOT State Geotechnical Engineer is required for use of numerical modeling techniques for the purpose of back-analysis to estimate soil or rock properties. Approval will be based on the adequacy of the numerical model calibration, how well the performance to be modeled is defined and quantified, and how well the variables/input parameters in the model are defined and measured such that a unique value of the parameter of interest can be accurately estimated.

5.8 Engineering Properties of Soil

5.8.1 Laboratory Index Property Testing

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. For soils with greater than 10 percent passing the No. 200 sieve, a decision will need to be made regarding the full soil gradation curve and whether a hydrometer test in addition to sieve testing of the coarser particles (AASHTO T88) is necessary, or if a coarse gradation is sufficient (AASHTO T27). The full gradation range (AASHTO T88) will be needed in the following situations:

- Lateral load analysis of deep foundations using strain wedge theory
- Liquefaction analysis
- Infiltration design, or other analyses that require the determination of hydraulic conductivities
- Other analyses that require a d_{10} size, coefficient of uniformity, etc.

Classification using the coarse sieving only (AASHTO T27) may be adequate for design of MSE walls, general earthwork, footing foundations, gravity walls, and noise walls. These end use needs shall be considered when planning the laboratory investigation for a project.

5.8.2 Laboratory Performance Testing

Laboratory performance testing is mainly used to estimate strength, compressibility, and permeability characteristics of soil and rock. For rock, the focus of laboratory performance testing is typically on the shear strength of the intact rock, or on the shear strength of specific discontinuities (i.e., joint/seam) within the rock mass. See Section 5.9 for additional discussion on rock properties. 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. There are a variety of shear strength tests that can be conducted, and the specific type of test selected depends on the specific application. See Sabatini et al. (2002) for specific guidance on the types of shear strength tests needed for various applications, as well as the chapters in the GDM that cover specific geotechnical design topics.

Disturbed soil shear strength testing is less commonly performed, and is primarily used as supplementary information when performing back-analysis of existing slopes, or for fill material and construction quality assurance when a minimum shear strength is required. It is difficult to obtain very accurate shear strength values of soils in natural deposits through shear strength testing of disturbed (remolded) specimens, since the in-situ density and soil structure is quite difficult to accurately recreate, especially considering the specific in-situ density may not be known. The accuracy of this technique in this case must be recognized when interpreting the results. However, for estimating the shear strength of compacted backfill, more accurate results can be obtained, since the soil placement method, as well as the in-situ density and moisture content, can be recreated in the laboratory with some degree of confidence. The key in the latter case is the specimen size allowed by the testing device, as in many cases, compacted fills have a significant percentage of gravel sized particles, requiring fairly large test specimens <u>and test apparatus</u> (i.e., minimum 3 to 4 inch diameter, or narrowest dimension specimens of 3 to 4 inches).

Typically, a disturbed sample of the granular backfill material (or native material in the case of obtaining supplementary information for back-analysis of existing slopes) is sieved to remove particles that are too large for the testing device and test standard, and is compacted into a mold to simulate the final density and moisture condition of the material. The specimens may or may not be saturated after compacting them and placing them in the shear testing device, depending on the condition that is to be simulated. In general, a drained test is conducted, or if it is saturated, the pore pressure during shearing can be measured (possible for triaxial testing; generally not possible for direct shear testing) to obtained drained shear strength parameters. Otherwise, the test is run slow enough to be assured that the specimen is fully drained during shearing (note that estimating the testing rate to assure drainage can be difficult). Multiple specimens using at least three confining pressures should be tested to obtain a shear strength envelope. See Sabatini et al. (2002) for additional details.

Tests to evaluate compressibility or permeability of existing subsurface deposits must be conducted on undisturbed specimens, and the less disturbance the better. See Sabatini et al. (2002) for additional requirements regarding these and other types of laboratory performance tests that should be followed.

The hydraulic conductivity of a soil is influenced by the particle size and gradation, the void ratio, mineral composition, and soil fabric. In general the hydraulic conductivity, or permeability, increases with increasing grain size; however, the size and shape of the voids also <u>have</u> a significant influence. The smaller the voids <u>are</u>, the lower the permeability. Mineral composition and soil fabric have little effect on the permeability of gravel, sand, and non-plastic silt, but are important for plastic silts and clays. Therefore, relationships between particle size and permeability are available for coarse-grained materials, some of which are presented in the Correlations subsection (Section 5.6.2). In general, for clays, the lower the ion exchange capacity of the soil, the higher the permeability. Likewise, the more flocculated (open) the structure, the higher the permeability.

The methods commonly used to determine the hydraulic conductivity in the laboratory include, the constant head test, the falling head test, and direct or indirect methods during a consolidation test. The laboratory tests for determining the hydraulic conductivity are generally considered quite unreliable. Even with considerable attention to test procedures and equipment design, tests may only provide values within an order of magnitude of actual conditions. Some of the factors for this are:

- The soil in-situ is generally stratified and this is difficult to duplicate in the laboratory.
- The horizontal value of k is usually needed, but testing is usually done on tube samples with vertical values obtained.

- In sand, the horizontal and vertical values of k are significantly different, often on the order of k_h = 10 to 100k_v.
- The small size of laboratory samples leads to boundary condition effects.
- Saturated steady-state soil conditions are used for testing, but partially saturated soil water flow often exists in the field.
- On low permeability soils, the time necessary to complete the tests causes evaporation and equipment leaks to be significant factors.
- The hydraulic gradient in the laboratory is often 5 or more to reduce testing time, whereas in the field it is more likely in the range of 0.1 to 2.

The hydraulic conductivity is expected to vary across the site; however, it is important to differentiate errors from actual field variations. When determining the hydraulic conductivity, the field and laboratory values should be tabulated along with the other known data such as sample location, soil type, grain-size distribution, Atterberg limits, water content, stress conditions, gradients, and test methods. Once this table is constructed, it will be much easier to group like soil types and k values to delineate distinct areas within the site, and eliminate potentially erroneous data.

5.8.3 Correlations to Estimate Engineering Properties of Soil

Correlations that relate in-situ index test results such as the SPT or CPT or laboratory soil index testing may be used in lieu of or in conjunction with performance laboratory testing and back-analysis of site performance data to estimate input parameters for the design of the geotechnical elements of a project. Since properties estimated from correlations tend to have greater variability than measurement using laboratory performance data (see Phoon et al., 1995), properties estimated from correlation to in-situ field index testing or laboratory index testing should be based on multiple measurements within each geologic unit (if the geologic unit is large enough to obtain multiple measurements). A minimum of 3 to 5 measurements should be obtained from each geologic unit as the basis for estimating design properties.

The drained friction angle of granular deposits estimated from SPT measurements shall be determined based on the correlation provided in Table 5-1.

$N1_{60}$ from SPT (blows/ft)	φ (o)
<4	25-30
4	27-32
10	30-35
30	35-40
50	38-43

Correlation of SPT N values to drained friction angle of granular soils (modified after Bowles, 1977 as reported in AASHTO 2012)
Table 5-1

The correlation used is modified after Bowles (1977). The correlation of Peck, Hanson and Thornburn (1974) falls within the ranges specified. Experience should be used to select specific values within the ranges. In general, finer materials, materials with significant silt-sized material, and materials in which the particles are rounded to sub-rounded will fall in the lower portion of the range. Coarser materials with less then 5% fines, and materials in which the particles are sub-angular to angular will fall in the upper portion of the range.

Care should be exercised when using other correlations of SPT results to soil parameters. Some published correlations are based on corrected values (N160) and some are based on uncorrected values (N). The designer shall ascertain the basis of the correlation and use either N160 or N as appropriate. Care shall also be exercised when using SPT blow counts to estimate soil shear strength for soils with gravel, cobbles, or boulders. Gravels, cobbles, or boulders could cause the SPT blow counts to be unrealistically high.

Correlations for other soil properties (other than as specifically addressed above for the soil friction angle) as provided in Sabatini et al. (2002) may be used if the correlation is widely accepted and if the accuracy of the correlation is known. However, such correlations shall not be extrapolated to estimate properties beyond the range of the empirical data used to establish the correlation. Care shall also be exercised when using correlations near the extremities of the empirical basis for the correlations, and the resulting additional uncertainty in the estimated properties shall be addressed in the design in which those properties are used. Local geologic formation-specific correlations may be used if well established by: (1) data comparing the prediction from the correlation to measured high quality laboratory performance data, or (2) back-analysis from full-scale performance of geotechnical elements affected by the geologic formation in question.

Regarding soil hydraulic conductivity, the correlations provided in the Highway Runoff Manual, should be used.

5.9 Engineering Properties of Rock

Engineering properties of rock are controlled by the discontinuities within the rock mass and the properties of the intact rock. Therefore, engineering properties for rock must account for the properties of the intact rock 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 requisite engineering properties.

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. Common engineering properties typically obtained from laboratory tests include specific gravity, point load strength, compressive strength, tensile strength, shear strength, modulus, and slake durability. Rock mass properties are determined by visual examination and description of discontinuities within the rock mass following the suggested methodology of the International Society of Rock Mechanics (ISRM 1978), and how these discontinuities will affect the behavior of the rock mass when subjected to the proposed construction and loading.

Point load tests should be calibrated to unconfined compression strength test results on the same rock type . Point load tests shall not be used for weak to extremely rock (R0, R1, and R2 rock) with uniaxial compressive strength less than 3600 psi (25 MPa).

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) using the Rock Mechanics Rating (RMR) system 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 shear 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 t through comparison to actual rock slope stability data, WSDOT 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 fractured rock masses in which the stability of the rock slope, or rock surrounding the foundation is not structurally controlled. See Chapter 12 for additional requirements regarding the assessment of rock mass properties.

Some design methods were specifically developed using the older Hoek and Brown (1988) RMR method, such as the design of spread footings on rock in the AASHTO LRFD Bridge Design manual (specifically Article 10.6.3.2). In such cases, the older Hoek and Brown method shall be used until such time that the design procedure has been updated to use the newer GSI index method.

5.10 Determination and Use of Soil Cohesion

Soil cohesion is defined as shear strength resulting from inter-particle attraction effect that is independent of normal stress but varies considerably with water content and rate of loading (Bowles 1979).

The use of cohesion due to inter-particle attraction, such as occurs in clays and clayey silts, for design shall be considered cautiously for long-term design and in general shall not be fully relied upon for long-term loading, unless local experience indicates that a particular value of cohesion in a given geologic unit can be relied upon (note: evidence of that local experience, such as results from previous back-analyses that demonstrate good long-term performance can be reliably achieved, shall be included in the calculation package). If cohesion is used in such cases, it shall be a conservative lower bound value. It is especially important to not rely upon cohesive shear strength if displacement in the soil has occurred in the past or potentially could occur in the future, in fractured or fissured soil, or if moisture content changes over time could occur. In these cases, a drained cohesion value near zero shall be used. For short-term applications, such as in temporary cuts or walls, or during seismic loading, some soil cohesion may be considered for use in design, provided that potential displacement and water content changes are adequately controlled or taken into account. To justify the use of cohesion where structures (e.g., anchored walls) are used to restrain or prevent soil deformation, a deformation analysis of the restraining system shall be conducted to demonstrate that the deformation will be adequately controlled.

Apparent cohesion is defined as the cohesion that results from surface tension due to moisture in unsaturated, but not dry, soils, primarily in sands and non-plastic silts. Apparent cohesion shall not be relied upon for the design of permanent works. For temporary works, apparent cohesion may only be used if the moisture content of the soils can be preserved or controlled and the magnitude of the apparent cohesion is conservatively assessed.

For sands and gravels with 10% fines or less by weight, cohesion shall not be relied upon for both short-term and long-term design situations, as in most cases, most of the cohesion that may be present is apparent cohesion, which is not a reliable source of shear strength.

5.11 Final Selection of Design Values

5.11.1 Overview

After the field and laboratory testing is completed, the geotechnical designer shall review the quality and consistency of the data, and shall 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 geotechnical designer generally has several sources of data consisting of that obtained in the field, laboratory test results, and correlations from index testing. In addition, the geotechnical designer may have results of back- analyses, or 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 shall 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 Section 5.1, 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, stress history, and degree of disturbance, 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, from historical experience with the subsurface conditions at or near the site, and from the results of back- analyses shall be compared to determine the engineering properties for the various geologic units encountered throughout the site. If soil/rock data from nearby sites in the same or similar geologic unit are considered, site specific test data shall have priority in the selection of design parameters relative to non-site specific historical data for the geologic unit in question at the site.

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. The validity and reliability of the data and its usefulness in selecting final design parameters shall be evaluated.

After a review of data reliability, a review of the variability of the selected parameters shall 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.

5.11.2 Data Reliability and Variability

Inconsistencies in data shall 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 (e.g., test results that are potentially invalid due to sample disturbance, affected by recording errors, affected by procedural errors, etc.).

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 of 30 or less. If much higher blow counts are recorded and the Standard Penetration tests were performed correctly, 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. If the reason for the difference between the new site specific test data and the historical data from nearby sites is not clear, then the site specific test data shall be given priority with regard to final selection of design parameters.

3) **Performance Comparison** – Assess results with respect to historic performance of structures at the site or within similar soils as described in Setion 5.7. Compare the results from the back-analyses to the properties determined from field and lab testing for the project site. The newly collected data should be correlated with the

parameters determined from observation of measurable 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, and disturbance resulting from the sampling and test specimen preparation. It may be found that certain tests consistently provide high or low values compared to anticipated results.

The result of these five 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 as determined through the five step process described above, 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 geotechnical designer 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 geotechnical designer 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 geotechnical designer, it may be possible to establish upper and lower bounds along with the average for a given property.

5.11.3 Time Dependent Considerations

Properties of soil and rock can change over time (see Section 5.3). Examples of time dependent changes include, but are not limited to, the following:

- Material degradation due to weathering, moisture changes, etc.,
- Changes in properties such shear strength due to deformation,
- Changes resulting from short or long-term stress changes (e.g., removal of load due to excavation causing rebound)

When selecting soil and rock properties for design, the potential for these changes to occur during the life of the facility shall be addressed in the final selection of soil and rock properties. For example, if conducting a back analysis of a slope failure, especially if it is a first time slope failure, the back-analysis will determine the mobilized shear strength at the time the failure initiated and therefore may result in a value that is greater than the residual shear strength measured in laboratory testing or determined from correlations. The back-analyzed shear strength may therefore be greater than the shear strength along the post failure shear surface as well as the long-term shear strength that could occur in the future. In such cases, the shear strength that is representative of the long-term condition, i.e., the residual shear strength determined from the laboratory tests and correlations, should be selected for design.

5.11.4 Final Property Selection

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 shall be assessed. If the impact of this uncertainty is likely to be significant, parametric analyses shall be conducted, or more data could be obtained to help reduce the uncertainty. Since the sources of data that could be considered may include measured laboratory data, field test data, performance data (i.e., from back-analyses), 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 based on experience, 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 Chapter 8, 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 a significant 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. The consequence of failure should also bear on the determination of a design parameter. 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 geotechnical designer will 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, consideration for potentially using an average property value is not relevant, and property selection should be based on the considerations discussed previously, which in most cases the property values shall be selected conservatively to be consistent with past practice.

The process and examples to make the final determination of properties to be used for design provided by Sabatini et al. (2002) shall be followed, subject to the specific requirements in the GDM. Local experience with certain engineered and naturally occurring geologic units encountered in the state of Washington is summarized in Sections 5.12 and 5.13. The final selection of design properties for the engineered and naturally occurring geologic units described in these two GDM sections shall be consistent with the experience cited in these two GDM sections.

The documentation required to justify the selection of design parameters is specified in Section 23.3.2.

5.11.5 Development of the Subsurface Profile

While Section 5.8 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 are 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 ESU's exhibiting distinct engineering characteristics. The end product is the subsurface profile, a two dimensional or, if necessary, a three 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 stratigraphy commonly is not linear or continuous between borings. Field descriptions and engineering properties will aid in the comparisons.
3) Group, or possibly split up, the subsurface geologic strata based on engineering properties to create ESU's.
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.

5.12 Selection of Design Properties for Engineered Materials

This section provides guidelines for the selection of properties that are commonly used on WSDOT projects such as engineered fills. The engineering properties are based primarily on gradation and compaction requirements, with consideration of the geologic source of the fill material typical for the specific project location. For materials such as common borrow where the gradation specification is fairly broad, a wider range of properties will need to be considered.

Common Borrow – Per the WSDOT *Standard Specifications*, common borrow may be virtually any soil or aggregate either naturally occurring or processed which is substantially free of organics or other deleterious material, and is non-plastic. The specification allows for the use of more plastic common borrow when approved by the engineer. On WSDOT projects this material will generally be placed at 90 percent (Method B) or 95 percent (Method C) of Standard Proctor compaction. Because of the variability of the materials that may be used as common borrow, the estimation of an internal friction angle and unit weight should be based on the actual material used. A range of values for the different material properties is given in Table 5-2. Lower range values should be used for finer grained materials compacted to Method B specifications. In general during design, the specific source of borrow is not known. Therefore, it is not prudent to select a design friction angle that is near or above the upper end of the range unless the geotechnical designer has specific knowledge of the source(s) likely to be used, or unless quality assurance shear strength testing is conducted during construction. Depending on location, common borrow will may have a fines content sufficient to be moisture sensitive. This moisture sensitivity may affect the design property selection if it is likely that placement conditions are likely to be marginal due to the timing of construction.

Select Borrow – The requirements for select borrow ensure that the mixture will be granular and contain at least a minimal amount of gravel-size material. The materials are likely to be poorly graded sand and contain enough fines to be moderately moisture sensitive (the specification allows up to 10 percent fines). Select Borrow is not an all weather material. Triaxial or direct shear strength testing on material that meets Select Borrow gradation requirements indicates that drained friction angles of 38 to 45 degrees are likely when the soil is well compacted. Even in it loosest state, shear strength testing of relatively clean sands meeting Select Borrow requirements has indicated values of 30 to 35 degrees. However, these values are highly dependent on the geologic source of the material. Surficial deposits that particles which have been minimally transported/reworked (i.e. colluvium, some glacial deposits) can have more subangular to angular soil particles and hence, high shear strength values. Windblown, beach, or alluvial sands that have been rounded through significant transport could have significantly lower shear strength values. Left-overs from processed materials (e.g., scalpings) could also have relative low friction angles depending on the uniformity of the material and the degree of rounding in the soil particles. A range of values for shear strength and unit weight based on previous experience for well compacted Select Borrow is provided in Table 5-2. In general, during design the specific source of borrow is not known. Therefore, it is not prudent to select a design friction angle that is near or above the upper end of the range unless the geotechnical designer has specific knowledge of the source(s) likely to be used or unless quality assurance shear strength testing is conducted during construction. Select Borrow with significant fines content may sometimes be modeled as having a temporary or apparent cohesion value from 50 to 200 psf, subject to the requirements for the use of cohesion as specified in Section 5.10. If a cohesion value is used, the friction angle should be reduced so as not to increase the overall strength of the material. For long-term analysis, all the borrow material should be modeled with no cohesive strength.

Gravel Borrow – The gravel borrow specification should ensure a reasonably well graded sand and gravel mix. Because the fines content is under 7 percent, the material is only slightly moisture sensitive. However, in very wet conditions, material with lower fines content should be used. Larger diameter triaxial shear strength testing performed on well graded mixtures of gravel with sand that meet the Gravel Borrow specification indicate that very high internal angles of friction are possible, approaching 50 degrees, and that shear strength values less than 40 degrees are not likely. However, lower shear strength values are possible for Gravel Borrow from naturally occurring materials obtained from non-glacially derived sources such as wind blown or alluvial deposits. In many cases, processed materials are used for Gravel Borrow, and in general, this processed material has been crushed, resulting in rather angular particles and very high soil friction angles. Its unit weight can approach that of concrete if very well graded. A range of values for shear strength and unit weight based on previous experience is provided in Table 5-2. In general during design the specific source of borrow is not known. Therefore, it is not prudent to select a design friction angle that is near or above the upper end of the range unless the geotechnical designer has specific knowledge of the source(s) likely to be used or unless quality assurance shear strength testing is conducted during construction.

Gravel Backfill for Walls – Gravel backfill for walls is a free draining material that is generally used to facilitate drainage behind retaining walls. This material has similarities to Gravel Borrow, but generally contains fewer fines and is freer draining. Gravel backfill for Walls is likely to be a processed material and if crushed is likely to have a very high soil friction angle. A likely range of material properties is provided in Table 5-2.

Material	WSDOT Standard Specification	Soil Type (USCS classification)	φ (degrees)	Cohesion (psf)	Total Unit Weight (pcf)
Common Borrow	9-03.14(3)	ML, SM, GM	30 to 34	0	115 to 130
Select Borrow	9-03.14(2)	GP, GP-GM, SP, SP-SM	34 to 38	0	120 to 135
Gravel Borrow	9-03.14(1)	GW, GW-GM, SW, SW-SM	36 to 40	0	130 to 145
Gravel Backfill for Walls	9-03.12(2)	GW, GP, SW, SP	36 to 40	0	125 to 135

Presumptive Design Property Ranges for Compacted Borrow and Other WSDOT *Standard Specification* Materials
Table 5-2

Rock Embankment – Embankment material is considered rock embankment if 25 percent of the material is over 4 inches in diameter. Compactive effort is based on a method specification. Because of the nature of the material, compaction testing is generally not feasible. The specification allows for a broad range of material and properties such that the internal friction angle and unit weight can vary considerably based on the amount and type of rock in the fill. Rock excavated from cuts consisting of siltstone, sandstone and claystone may break down during the compaction process, resulting in less coarse material. Also, if the rock is weak, failure may occur through the rock fragments rather than around them. In these types of materials, the strength parameters may resemble those of earth embankments. For existing embankments, the soft rock may continue to weather with time, if the embankment materials continue to become wet. For embankments constructed of sound rock, the strength parameters may be much higher. For compacted earth embankments with sound rock, internal friction angles of up to 45 degrees may be reasonable. Unit weights for rock embankments generally range from 130 to 140 pcf.

Quarry Spalls and Rip Rap – Quarry spalls, light loose rip rap and heavy loose rip rap created from shot rock are often used as fill material below the water table or in shear keys in slope stability and landslide mitigation applications. WSDOT Standard Specification Section 9-13 provides minimum requirements for degradation and specific gravity for these materials. Therefore sound rock must be used for these applications. For design purposes, typical values of 105 to 120 pcf for the unit weight (this considers the large amount of void space due to the coarse open gradation of this type of material) and internal angles of friction of about 40 to 45 degrees should be used.

Wood Fiber – Wood fiber fills have been used by WSDOT for over 30 years in fill heights up to about 40 feet. The wood fiber has generally been used as light-weight fill material over soft soil to improve embankment stability. Wood fiber has also been used in emergency repair because rain and wet weather does not affect the placement and compaction of the embankment. Only fresh wood fiber should be used to prolong the life of the fill, and the maximum particle size should be 6 inches or less. The wood fiber is generally compacted in lifts of about 12 inches with two passes of a track dozer. Presumptive design values of 50 pcf for unit weight and an internal angle of friction of about 40 degrees may be used for the design of the wood fiber fills (Allen et al., 1993).

To mitigate the effects of leachate, the amount of water entering the wood should be minimized. Generally topsoil caps of about 2 feet in thickness are used. The pavement section should be a minimum of 2 feet (a thicker section may be needed depending on the depth of wood fiber fill). Wood fiber fill will experience creep settlement for several years and some pavement distress should be expected during that period. Additional information on the properties and durability of wood fiber fill is provided in Kilian and Ferry (1993).

Geofoam – Geofoam has been used as lightweight fill on WSDOT projects since 1995. Geofoam ranges in unit weight from about 1 to 2 pcf. Geofoam constructed from expanded polystyrene (EPS) is manufactured according to ASTM standards for minimum density (ASTM C 303), compressive strength (ASTM D 1621) and water absorption (ASTM C 272). Type I and II are generally used in highway applications. Bales of recycled industrial polystyrene waste are also available. These bales have been used to construct temporary haul roads over soft soil. However, these bales should not be used in permanent applications.

5.13 Properties of Predominant Geologic Units in Washington

This section contains a brief discussion of soil and rock types common to Washington state that have specific engineering properties that need consideration.

5.13.1 Loess

Loess is a windblown (eolian) soil consisting mostly of silt with minor amounts of sand and clay (Higgins et al., 1987). Due to its method of deposition, loess has an open (honeycomb) structure with very high void ratios. The clay component of loess plays a pivotal role because it acts as a binder (along with calcium carbonate in certain deposits) holding the structure together. However, upon wetting, either the water soluble calcium carbonate bonds dissolve or the large negative pore pressures within the clay that are holding the soil together are reduced and the soil can undergo shear failures and/or settlements.

Loess deposits encompass a large portion of southeastern Washington. Loess typically overlies portions of the Columbia River Basalt Group and is usually most pronounced at the tops of low hills and plateaus where erosion has been minimal (Joseph, 1990). Washington loess has been classified into four geologic units: Palouse Loess, Walla Walla Loess, Ritzville Loess, and Nez Perce Loess. However, these classifications hold little relevance to engineering behavior. For engineering purposes loess can generally be classified into three categories based on grain size: clayey loess, silty loess, and sandy loess (see Chapter 10).

Typical index and performance properties measured in loess are provided in Table 5-3, based on the research results provided in Report WA-RD 145.2 (Higgins and Fragazy, 1988). Density values typically increase from west to east across the state with corresponding increase in clay content. Higgins and Fragaszy observed that densities determined from Shelby tube samples in loess generally result in artificially high values due to disturbance of the open soil structure and subsequent densification. Studies of shear strength on loess have indicated that friction angles are usually fairly constant for a given deposit and are typically within the range of 27 to 29 degrees using CU tests. These studies have also indicated that cohesion values can be quite variable and depend on the degree of consolidation, moisture content and amount of clay binder. Research has shown that at low confining pressures, loess can lose all shear strength upon wetting.

Type of Loess	Liquid Limit	Plasticity Index	Dry Density (pcf)	Angle of Internal Friction (o)
Clayey	33 to 49	11 to 27	70 to 90, with maximum of up to 95 to 98 (generally increases with clay content)	27 to 29 from CU tests
Silty	14 to 32	0 to 11		
Sandy	Nonplastic	Nonplastic		

Typical Measured Properties For Loess Deposits in Washington State
Table 5-3

The possibility of wetting induced settlements shall be considered for any structure supported on loess by performing collapse tests. Collapse tests are usually performed as either single ring (ASTM D 5333) or double ring tests. Double ring tests have the advantage in that potential collapse can be estimated for any stress level. However, two identical samples must be obtained for testing. Single ring tests have the advantage in that they more closely simulate actual collapse conditions and thus give a more accurate estimate of collapse potential. However, collapse potential can only be estimated for a particular stress level, so care must be taken to choose an appropriate stress level for sample inundation during a test. When designing foundations in loess, it is important to consider long term conditions regarding possible changes in moisture content throughout the design life of the project. Proper drainage design is crucial to keeping as much water as possible from infiltrating into the soil around the structure. A possible mitigation technique could include overexcavation and recompaction to reduce or eliminate the potential for collapse settlement.

Loess typically has low values of permeability and infiltration rates. When designing stormwater management facilities in loess, detention ponds should generally be designed for very low infiltration rates.

Application of the properties of loess to cut slope stability is discussed in Chapter 10.

5.13.2 Peat/Organic Soils

Peats and organic soils are characterized by very low strength, very high compressibility (normally or slightly under-consolidated), low hydraulic conductivity, and having very important time-consolidation effects. Often associated with wetlands, ponds and near the margins of shallow lakes, these soils pose special challenges for the design of engineering transportation projects. Deep deposits (+100 feet in some cases) with very high water content, highly compressibility, low strength and local high groundwater conditions require careful consideration regarding settlement and stability of earth fill embankments, support for bridge foundations, and locating culverts.

The internal structure of peat, either fibrous or granular, affects its capacity for retaining and releasing water and influences its strength and performance. With natural water content often ranging from 200-600 percent (over 100 for organic silts and sands) and wet unit weight ranging from 70 to 90 pcf, it can experience considerable shrinkage (>50%) it dries. Rewetting usually cannot restore its original volume or moisture content. Under certain conditions, dried peat will oxidize and virtually disappear. Undisturbed sampling for laboratory testing is difficult. Field vane testing is frequently used to evaluate in place shear strength, though in very fibrous peats, reliable shear strength data is difficult to obtain even with the field vane shear test. Initial undisturbed values of 100 to 400 psf are not uncommon but remolded (residual) strengths can be 30 to 50 % less (Schmertmann, 1967). Vane shear strength, however, is a function of both vane size and peat moisture content. Usually, the lower the moisture of the peat and the greater its depth, the higher is its strength. Strength increases significantly when peat is consolidated, and peak strength only develops after large deformation has taken place. Due to the large amount of strain that can occur when embankment loads are placed on peats and organic soils, residual strengths may control the design.

Vertical settlement is also a major concern for constructing on organic soils. The amount of foundation settlement and the length of time for it to occur are usually estimated from conventional laboratory consolidation tests. Secondary compression can be quite large for peats and must always be evaluated when estimating long-term settlement. Based on experience in Washington State, compression index values based on vertical strain ($C_{c\varepsilon}$) typically range from 0.1 to 0.3 for organic silts and clays, and are generally above 0.3 to 0.4 for peats. The coefficient of secondary compression ($C_{\alpha\varepsilon}$) is typically equal to $0.05C_{c\varepsilon}$ to $0.06C_{c\varepsilon}$ for organic silts and peats, respectively.

5.13.3 Glacial Deposits

Till – Till is an unsorted and unstratified accumulation of glacial sediment deposited directly by glacial ice. Till is a heterogeneous mixture of different sized material with particle sizes ranging in size from clay to boulders. Although the matrix proportions of silt and clay vary from place to place, the matrix generally consists of silty sand or sandy silt (Troost and Booth, 2003). Tills in Washington are deposited by either continental glaciers or alpine glaciers. Many of the tills in Washington, especially those associated with continental glaciers, have been overridden by the advancing continental ice sheet and are highly over consolidated, but not all tills have been consolidated by glacial ice. Tills deposited by alpine glaciers are most commonly found in and along the valley margins of the Olympic Mountains and Cascade Range, and are commonly not over consolidated.

Glacial till is often found near the surface in the Puget Sound Lowland area. The Puget Sound Lowland is a north-south trending trough bordered by the Cascade Mountains to the east and the Olympic Mountains to the west. The most recent glaciation, the Vashon Stade of the Fraser Glaciation occupied the Puget Sound region between roughly 18,000 to 13,000 years ago. Glacial till deposited by this glaciation extends as far south as the Olympia area.

Till that has been glacially overridden generally has very high unit weights and very high soil strength even when predominantly fine grained. Because if its inherent strength and density, it provides good bearing resistance, has very small strain under applied loads, and exhibits good stand up times even in very steep slopes. Typicall properties for glacially overridden tills range from 40 to 45 degrees for internal friction angle with cohesion values of 100 to 1,000 psf. Unit weights used for design are typically in the range of 130 to 140 pcf for glacially overridden till. The cohesion component of the shear strength can typically be relied upon due to the relatively high fines content of this geologic unit combined with its heavily overconsolidated nature and locked in stress history. Furthermore, very steep, high exposures of till in the Puget Sound region have demonstrated long-term stability that cannot be explained without the presence of significant soil cohesion, verifying the reliability of this soil cohesion. However, where these till units are exposed, the upper 2 to 5 feet is often weathered and is typically medium dense to dense. The glacial till generally grades to dense to very dense below the weathered zone. This upper weathered zone, when located on steep slopes, has often been the source of slope instability and debris flows during wet weather. Glacial till that is exposed as a result of excavation, slope instability, or other removal of overlying material will degrade and lose strength with weathering. If the till unit is capped with a younger deposit and had been previously weathered, weathered till zones can be present at depth as well.

The dense nature of glacially overriden till tends to make excavation and pile installation difficult. It is not uncommon to have to rip till with a dozer or utilize large excavation equipment. Permeability in till is relatively low because of the fines content and the density. However, localized pockets and seams of sand with higher permeability that may also be water bearing are occasional encountered in till units. These localized pockets and seams may contribute slope stability problems.

Till that has not been glacially overridden and over consolidated should be treated as normally consolidated materials consistent with the till's grain size distribution. Accordingly, tills that tend to be finer grained will exhibit lower strength and higher strain than tills which are skewed toward the coarser fraction.

Wet weather construction in till is often difficult because of the relatively high fines content of till soils. When the moisture content is more than a few percent above the optimum moisture content and the till is disturbed or unconfined, till soils become muddy and unstable, and operation of equipment on these soils can become difficult. Within till cobble and boulder-sized material can be encountered at any time. Boulders in till deposits can range from a foot or two in diameter to tens of feet. In some areas cobble, boulders, and cobble/boulder mixtures can be nested together, making excavation very difficult.

Outwash – Outwash is a general term for sorted sediment that has been transported and deposited by glacial meltwater, usually in a braided stream environment. Typically, the sediment becomes finer grained with increasing distance from the glacier terminus.

Outwash tends to be more coarse grained and cleaner (fewer fines) than till. When it has been overridden by advancing ice, its strength properties are similar to till, but the cohesion is much lower due to a lack of fines, causing this material to have greater difficulty standing without raveling in a vertical cut, and in general can more easily cave in open excavations or drilled holes. Typically, the shear strength of glacially overridden (advance) outwash ranges from 40 to 45 degrees, with near zero cohesion for clean deposits. Since it contains less fines, it is more likely to have relatively high permeability and be water bearing. In very clean deposits, non-displacement type piles (e.g., H-piles) can "run" despite the very dense nature of the material.

Outwash that has not been glacially overridden may be indistinguishable from alluvial deposits. When normally consolidated outwash is encountered it exhibits strengths, densities, and other physical properties that are consistent with alluvium, with friction angles generally less than 40 degrees and little or no cohesion.

Within outwash, cobble and boulder-sized material can be encountered at any time. Boulders in outwash deposits can range from a foot or two in diameter to tens of feet. In some areas cobbles, boulders, and cobble/boulder mixtures can be nested together, making excavation very difficult.

Glacial Marine Drift (GMD) – Drift is a collective term used to describe all types of glacial sedimentary deposits, regardless of the size or amount of sorting. The term includes all sediment that is transported by a glacier, whether it is deposited directly by a glacier or indirectly by running water that originates from a glacier. In the Pacific Northwest, practitioners have commonly referred to fine-grained glacial sediments deposited in marine water as Glacial Marine Drift, or sometimes just Marine Drift.

In addition to sand and fine-grained materials, glacial marine drift contains variable amounts of clastic debris from melting icebergs, floating ice, and gravity currents. Most commonly glacial marine drift consists of poorly graded granular material within a clayey matrix. Composition varies from gravelly, silty sand with a trace of clay to silty sand and silty clay with varying percentages of sand and gravel. Because of the marine environment, it can contain shell and wood fragments, and occasional cobbles and boulders.

In and around Bellingham, the glacial marine drift typically consists of unsorted, unstratified silt and clay with varying amounts of sand, gravel, cobbles and occasional boulders, with small percentages of shells and wood. It is typically found at the surface or below Holocene age deposits. The upper portion of this unit, sometimes to about 15 feet of depth, can be quite stiff as a result of desiccation or partial ice contact in upland areas. This stiffer desiccated zone typically grades from medium stiff to very soft with depth. The entire glaciomarine drift profile can be stiff when only a thin section of the drift mantles bedrock at shallow depths. Conversely, the entire profile is typically soft in the Blaine area and can be soft when in low, perennially saturated areas. This geologic unit can be very thick (150 feet or more).

The properties of this unit are extremely variable, varying as a function of location, depth, loading history, saturation and other factors. The soft to medium stiff glaciomarine drift typically has very low shear strength, very low permeability and high compressibility. Based on vane shear and laboratory testing of this unit, the soft portion of this unit below the stiff crust typically has undrained shear strengths of approximately 500 to 1000 psf, and can be as low as 200 to 300 psf. The upper stiff crust is typically stronger, and may be capable of supporting lightly loaded footing supported structures. Atterberg limits testing will typically classify the softer material as a low plasticity clay; although, it can range to high plasticity. Consolidation parameters are variable, with the compression index (CC) in the range of 0.06 to over 0.2. Time rates of consolidation can also be quite variable.

Wet weather construction in glaciomarine drift is very difficult because of the relatively high clay content of these soils. When the moisture content of these soils is more than a few percent above the optimum moisture content, they become muddy and unstable, and operation of equipment can become very difficult. Localized sandy and gravelly layers in the drift can be saturated and are capable of producing significant amounts of water in cuts.

Glaciolacustrine – Glaciolacustrine deposits form in glacial meltwater lakes that may occur during both advancing and recessional glacial episodes. Glaciolacustrine deposits are commonly stratified and tend to be fine grained, typically consisting of silt and clay and often with sand laminae. Glaciolacustrine deposits accumulated during glacial advances may be overridden by the ice, causing the deposits to be highly overconsolidated and typically very stiff to hard. An example of glacially overridden undisturbed laminated silt/clay deposits is provided in Figure 5-1. When not glacially overridden, such as during the last glacial recessional period, glaciolacustrine deposits may behave similarly to other normally consolidated lacustrine deposits.

Example of Glacially Overridden Laminated Clay Exposed in Highway Excavation on Beacon Hill Near The Intersection SR-5 and SR-90
Figure 5-1

Fine-grained, glacially overridden deposits are widespread in the Puget Sound region, and have been encountered on projects in the Seattle area in the vicinity of SR-5, SR-90, SR-99, SR-405, and SR-520. These fine-grained deposits may be glaciolacustrine in origin associated with one of the more than six continental glaciations that have inundated the region during the Pleistocene. In the Seattle area, the most recent (Vashon Stade) of these advance glaciolacustrine units are named the Lawton Clay. This deposit can be more than 150 feet thick in the Seattle area (Troost and Booth, 2003). Additionally, fine-grained bedded units may be associated with interglacial periods (i.e., Olympia Beds) that may be somewhat similar in initial appearance to glaciolacustrine deposits. The widespread presence in the Seattle area of both glacial and interglacial, fine-grained, overconsolidated deposits has led many geotechnical practitioners to refer to any such deposit as “Seattle clay”, often irrespective of its age or origin. Collectively, these fine-grained overconsolidated deposits are often a primary material affecting engineering design in the Seattle area.

Extensive disturbance of these fine-grained, overconsolidated deposits is commonly observed, evidenced by fracturing and slickensides. A slickenside is a condition in which relative movement has occurred along the fracture, and is discernible by its shiny and commonly striated fracture surface. More extreme disturbance may involve disoriented/transported blocks within a matrix of intensely sheared and fractured silt and clay.

There are a variety of causes that may lead to post-depositional disturbance of these glaciolacustrine deposits. Vertical stresses and subsequent dewatering and consolidation through ice loading can induce fracturing, sometimes producing predictable fracture sets/networks. Lateral stresses induced by ice movement/flow can cause considerable deformation, shearing and translational movements (sometimes termed “shoving”) within the underlying sediments, a process referred to as glaciotectonics (e.g., Figure 5-2). Following deglaciation, stress relief associated with unloading, isostacy, exhumation, and erosion can induce further fracturing within the sediments. Another post-depositional disturbance mechanism causing extensive fracturing and disturbance of these deposits is landsliding on exposed slopes that occurred between glacial episodes and following the last glaciation. Figure 5-3 shows a tilted laminated clay block that was overridden and smeared by a subsequent glacial advance. Figure 5-4 shows a deep (approximately 40 feet) test pit exposing layers of weathered clay, water-bearing gravel, and unweathered clay, illustrating the highly variable structure and depositional environment that can occur in these reconsolidated landslide deposits. These reconsolidated landslide deposits, in particular, can become highly unstable when exposed in excavations or natural slopes. Ground motions and crustal deformation induced by regionally active tectonic processes are another source of disturbance to these deposits.

Figure 5-2(a)

Exposure Near the East End of Sr-520 Illustrating Fractured and Sheared Structure Within Glacially Overridden Clay Deposit Believed to be Due to Glaciotectonics (a) Overview of Exposure, (b) Close-Up Showing Clay Structure
Figure 5-2(b)

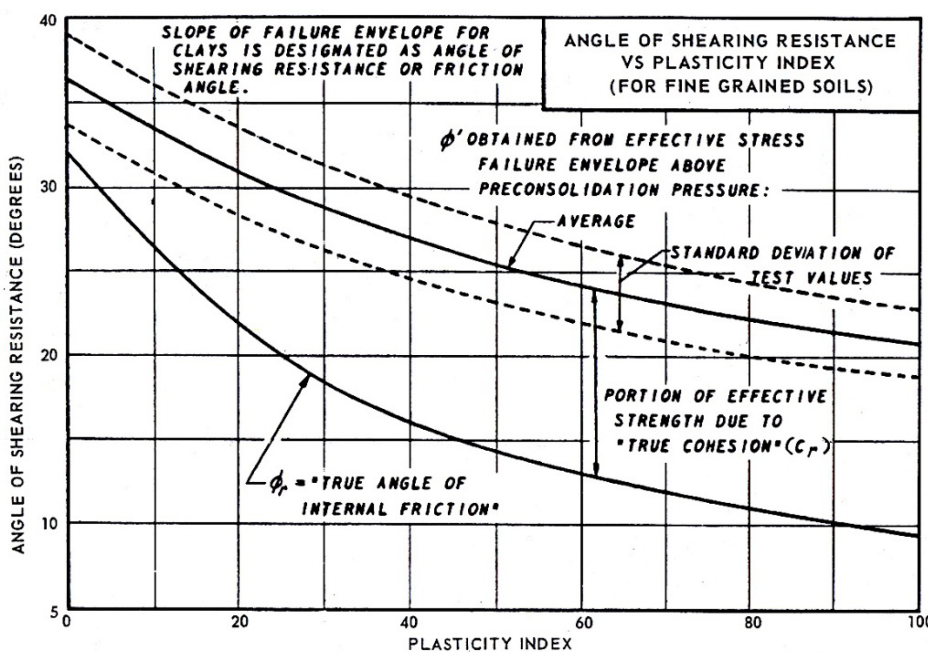
Example on Beacon Hill of Highly Disturbed Glacially Overconsolidated Clay Associated with a Paleolandslide Deposit; Note Near-Vertical Orientation of Laminae/Bedding Within the Landslide Block
Figure 5-3

Test Pit on Beacon Hill Showing Depositional Sequence Within a Glacially Overconsolidated Clay, Paleolandslide Deposit
Figure 5-4

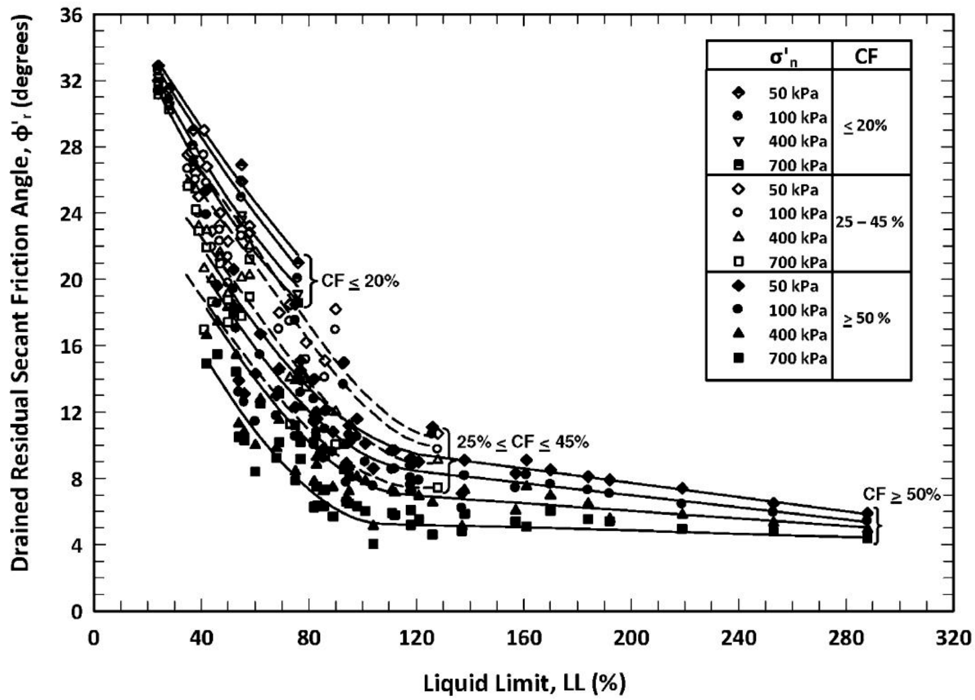
One of the most important geotechnical characteristics of these fine-grained overconsolidated deposits is that they generally have high in situ lateral stresses. Relaxation of these locked in stresses have created significant slope stability problems in both open and shored excavations. As excavations are completed, these deposits experience a lateral elastic rebound, which leads to their internal weakening. The failure mechanism is thought to consist of shear movement and/or tensional opening along pre-existing fractures. Depending on the extent of disturbance, failure surfaces/zones may need to shear along existing fractures and through intact clay blocks to fully develop. Linkage of fractures and subsequent hydrostatic pressure buildup within them can then further displace larger blocks/masses. With movement comes a drastic reduction in shear strength (often to a residual state) within these larger blocks/masses, which then lead to progressive slope failures. Such instability occurred in the downtown Seattle area when cuts were made within these deposits to construct Interstate 5 and Interstate 90. Fine, water-bearing sand laminae within the silts and clays often further exacerbate instability in exposures, not only in open cuts, but also in the form of caving in relatively small diameter shaft excavations.

Based on considerable experience, the long-term design of project geotechnical elements affected by these fine-grained overconsolidated deposits should be based on residual strength parameters. However, exceptions to this are provided in the paragraphs that follow.

For these deposits, the relationship between the residual friction angle and the plasticity index as reported in NAVFAC DM7 generally works well for estimating the residual shear strength (see Figure 5-5). The Stark and Hussain (2013) correlations for residual strength (see Figure 5-6) also work well for these deposits. In practice, shear strength values that have been estimated based on back-analysis of landslides and cut slope failures in this region are in the range of 13 to 17 degrees.



Correlation Between Residual Shear Strength of Overconsolidated Clays and Plasticity Index (After NAVFAC, 1971)
Figure 5-5



Correlation Between Residual Shear Strength of Overconsolidated Clays and Plasticity Index, Clay Fraction Cf, and Effective Normal Stress (After Stark and Hussain 2013)
Figure 5-6

Correlations with index soil properties such as the plasticity index, such as shown in Figure 5-5, or such as provided in Stark and Hussain (2013) in Figure 5-6, can be used to estimate the residual shear strength of soil. Laboratory tests on the site specific soils should be conducted, if possible, to measure the residual friction angle. When laboratory shear strength tests are conducted to determine the residual friction angle, high displacement tests such as the ring shear test should be used.

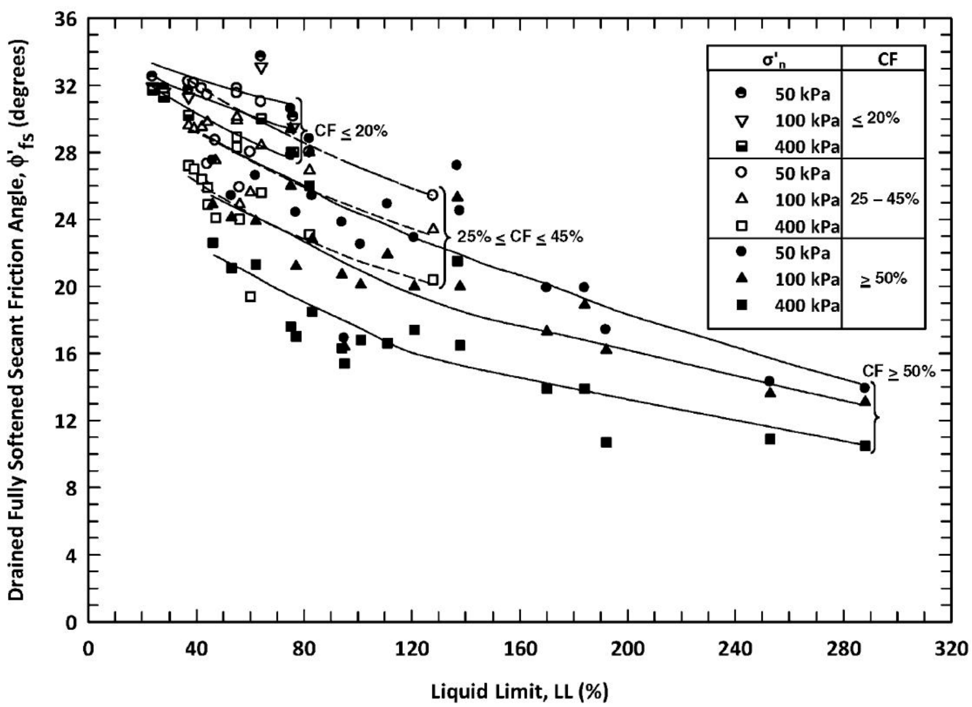
Designing for residual shear strength of the clay is a reasonable and safe approach in these fine-grained glacially consolidated soils, and is the default approach in post-depositionally disturbed deposits of fine-grained glacially consolidated soil, though there may be limited cases where a slightly higher shear strength could be used for design. For example, the glacially overridden clay deposits described earlier (e.g., figures 5-2 through 5-4) have been broken up enough to warrant the use of residual shear strength in most cases. If more detailed investigation is conducted (e.g., through back-analysis of previous slope failures or marginally stable slopes at the site in question, extensive laboratory shear strength testing, other possible testing or evaluation techniques, and consideration of site geological history of the strata in question) and demonstrates the shear strength of the existing deposit is greater than its residual value, higher design shear strengths may be justified, provided that any potential future deformation of the clay strata is prevented. In no case, however, in these glaciolacustrine deposits that have been post-depositionally disturbed due to phenomenon such as landsliding, glacial shoving, and shearing due to fault activity, shall a shear strength greater than the fully softened shear strength be used for design, even if future deformation of the clay deposit can be fully restrained. This applies to both temporary and permanent designs.

Note that the fully softened friction angle for clays is defined in Mesri and Shahien (2003) as:

"The fully softened strength envelope (often defined for stiff clays and shales by peak strength of reconstituted normally consolidated specimens) …."

In essence, this fully softened shear strength reflects the strength of an overconsolidated clay that has been disturbed, but the "plate-shaped" clay particles have not been fully aligned. This is in contrast to the situation in which a clay has been sufficiently sheared to reach a state of residual strength, such as along a landslide failure surface or along slickensides, in which all the clay particles have been aligned, producing the lowest possible shear strength.

Stark and Hussain (2013) provide recommended correlations to estimate the fully softened shear strength (see Figure 5-7) that should be used to estimate the fully softened shear strength, if laboratory site specific shear strength test data are not available. Alternatively, laboratory testing could be conducted to establish the fully softened shear strength. Guidelines regarding the type of laboratory testing required are provided in Stark, et al. (2005), and additional considerations for laboratory testing are provided in Stark and Hussain (2013).



Correlation Between Fully Softened Shear Strength of Overconsolidated Clays and Plasticity Index (After Stark and Hussain 2013)
Figure 5-7

Intact deposits of glacially overridden clays and clayey silts (i.e., those not subjected to the geologic disturbance processes described previously) may be designed for shear strengths approaching their peak values provided that (1) the clay has not been subject to deformation resulting from previous construction or erosion that caused unloading of the clay, or (2) the clay is deep enough to not be affected and will not be subject to unloading and deformation in the planned construction. Structures (e.g., tieback walls) designed to restrain the clay to prevent deformation may be used in combination with shear strengths near their peak values if previous construction that could potentially have caused removal/unloading of the clay has not occurred prior to the construction of the restraining structure. Otherwise, residual shear strength should be used for design within the clay. Intact glacially overridden clay that is deep enough below the final ground surface to not be affected by potential unloading may be designed for shear strength near its peak value.

As with most fine grained soils, wet weather construction in overconsolidated silt/clay is generally difficult. When the moisture content of these soils is more than a few percent above the optimum moisture content, they become muddy and unstable, and operation of equipment on these soils can become difficult.

Groundwater modeling of these glacially overridden clays can be very complex. Where below the groundwater surface, these clays may visually appear moist or dry. However, even with that appearance these clays can be saturated. Because they are fine grained and highly compact, water generally does not freely flow from these soils. More freely flowing ground water may be present in these deposits in localized or thin sand or gravel seams (e.g., Figure 5-4), between laminations in the clay, and within fissures in the clay, whereas the intact portions of the clay appear to be moist. The water within these fissures and sand or gravel seams is often hydraulically connected, having a similar effect with regard to stresses and stability as occurs in fractured rock masses that contain water. Due to the nature of the clay and the tendency of the clay surfaces within boreholes to become smeared during drilling, standard standpipe piezometers may take a very long time to stabilize adequately to get accurate water level readings – electrical piezometers, such as vibrating wire, should be used to get more accurate water level readings within a reasonable period of time.

Even though this geologic deposit is generally fine-grained, due to the highly overconsolidated nature of this deposit, settlement can generally be considered elastic in nature, and settlement, for the most part, occurs as the load is applied. This makes placement of spread footings on this deposit feasible if designed for relatively low bearing stress, and provided the footing is not placed on a slope that could allow an overall stability failure due to the footing load (see Chapter 8).

For additional discussion on geotechnical characterization and design in glacially overconsolidated clays, see Mesri and Shahien (2003) and Stark, et al. (2005).

5.13.4 Colluvium and Talus

Colluvium is a general term used to describe soil and rock material that has been transported through rainwash, sheetwash and downslope creep that collect on or at the base of slopes. Colluvium is typified by poorly sorted mixtures of soil and rock particles ranging in size from clay to large boulders. Talus is a gravitationally derived deposit that forms downslope of steep rock slopes, comprised of a generally loose assemblage of coarse, angular rock fragments of varied size and shape. Talus is commonly collectively referred with the term colluvium.

Colluvium is a very common deposit, encompassing upwards of 90 percent of the ground surface in mountainous areas. Colluvial deposits are typically shallow (less than about 25 to 30 feet thick), with thickness increasing towards the base of slopes. Colluvium commonly directly overlies bedrock on unglaciated slopes and intermixes with alluvial material in stream bottoms.

Subsurface investigations in colluvium using drilling equipment are often complicated by because of the heterogeneity of the deposit and possible presence of cobbles boulders. In addition, site access and safety issues also can pose problems. Test pits and trenches offer alternatives to conventional drilling that may provide better results. Subsurface investigations in talus can be especially difficult. Engineering properties of talus are extremely difficult to determine in the laboratory or in situ. A useful method for determining shear strength properties in both colluvium and talus is to analyze an existing slope failure. For talus, this may be the only way to estimate shear strength parameters. Talus deposits can be highly compressible because of the presence of large void spaces. Colluvial and talus slopes are generally marginally stable. In fact, talus slopes are usually inclined at the angle of repose of the constituent material. Cut slopes in colluvium often result in steepened slopes beyond the angle of repose, resulting in instability. Slope instability is often manifested by individual rocks dislodging from the slope face and rolling downslope. While the slope remains steeper than the angle of repose, a continuous and progressive failure will occur.

Construction in colluvium is usually difficult because of the typical heterogeneity of deposits and corresponding unfavorable characteristics such as particle size, strength variations and large void spaces. In addition, there is the possibility of long-term creep movement. Large settlements are also possible in talus. Foundations for structures in talus should extend through the deposit and bear on more competent material. Slope failures in colluvium are most often caused by infiltration of water from intense rainfall. Modifications to natural slopes in the form of cut slopes, construction of drainage ditches, and impropery channelized stormwater are ways that water can infiltrate into a colluvial soil and initiate a slope failure. Careful consideration must be given to the design of drainage facilities to prevent saturation of colluvial deposits.

5.13.5 Columbia River Sand

These sands are located in the Vancouver area, and both up and down river along the Columbia River west of the Cascades. These sands may have been deposited by backwaters from the glacial Lake Missoula catastrophic floods. The sands are poorly graded and range from loose to medium dense. The sand is susceptible to liquefaction if located below the water table. The sands do not provide a significant amount of frictional resistance for piles, and non-displacement piles may tend to run in these deposits. Based on the observed stability of slopes in this formation, soil friction angles of 28o to 32o should be expected.

5.13.6 Columbia Basin Basalts

The basalt flows that dominate the Columbia Basin were erupted into a structural and topographic low between the northern Rocky Mountains and the rising Cascade Range. During periods between the flows, erosion took place and tuffs, sandstones, and conglomerates were deposited on top of basalt flows (Thorsen, 1989). In some areas lake beds formed. The resulting drainage systems and lakes were responsible for the extensive layer of sediments between, interfingering with, and overlying the basalt flows. These interbedded sediments are generally thicker in areas peripheral to the flows, especially in and along the western margin of the basin. During the interludes between flows, deep saprolites formed on some flow surfaces. Present topographic relief on the basin has been provided largely by a series of east-west trending anticlinal folds, by the cutting of catastrophic glacial meltwater floods, and by the Columbia River system.

The most obvious evidence of bedrock slope failures in the basin is the presence of basalt talus slopes fringing the river canyons and abandoned channels. Such talus are generally standing at near the angle of repose.

Bedrock failures are most commonly in the form of very large slumps, slump flows, and translational landslides, controlled by weak interbeds or palagonite zones between flows. Most of these are ancient failures and occur in areas of regional tilting or are associated with anticlinal ridges. The final triggering, in many cases, appears to have been oversteepening of slopes or removal of toe support.

Along I-82, SR-12, and SR-410 on the western margin of the province and in a structural basin near Pasco, layers of weak sediments interfinger with basalt flows. Some of these sediments are compact enough to be considered siltstone or sandstone and are rich in montmorillionite. Slumps and translation failures are common in some places along planes sloping as little as 8 degrees. Most landslides are associated with pre-existing failure surfaces developed by folding and or ancient landslides. In the Spokane and Grande Ronde areas thick sections of sediments make up a major part of the landslide complexes.

5.13.7 Latah Formation

Much of Eastern Washington is underlain with thick sequences of basaltic flow rock. These flows spread out over a vast area that now comprises what is commonly known as the Columbia Plateau physiographic province (see Section 5.9.6). Consisting of extrusive volcanic rocks, they make up the Columbia River Basalt Group (Griggs, 1959). This geologic unit includes numerous basalt formations, each of which includes several individual flows that are commonly separated from one another by sedimentary lacustrine deposits (Smith et al., 1989). In the Spokane area, these sedimentary rock units are called the Latah Formation.

Most of the sedimentary layers between the basalt flows range from claystone to fine-grained sandstone in which very finely laminated siltstone is predominant. The fresh rock ranges in color from various shades of gray to almost white, tan and rust. Because of its generally poorly indurated state, the Latah rarely outcrops. It erodes rapidly and therefore is usually covered with colluvium or in steeper terrain, it is hidden under the rubble of overlying basaltic rocks.

The main engineering concern for the Latah Formation is its potential for rapid deterioration by softening and eroding when exposed to water and cyclic wetting and drying (Hosterman, 1969). The landslide potential of this geologic unit is also of great engineering concern. While its undisturbed state can often justify relatively high bearing resistance, foundation bearing surfaces need to be protected from precipitation and groundwater. Construction drainage is important and should be planned in advance of excavating. Bearing surface protection measures often include mud slabs or gravel blankets.

In the Spokane area, landslide deposits fringe many of the buttes (Thorsen, 1989). Disoriented blocks of basalt lie in a matrix of disturbed silts. The Latah Formation typically has low permeability. The basalt above it is often highly fractured, and joints commonly fill with water. Although this source of groundwater may be limited, when it is present, and the excavation extends through the Latah-basalt contact, the Latah will often erode (pipe) back under the basalt causing potential instability. The Latah is also susceptible to surface erosion if left exposed in steep cuts. Shotcrete is often used to provide a protective coating for excavation surfaces. Fiber-reinforced shotcrete and soil nailing are frequently used for temporary excavation shoring.

The Latah Formation has been the cause of a number of landslides in northeast Washington and in Idaho. Measured long-term shear strengths have been observed to be in the range of 14 to 17 degrees. It is especially critical to consider the long-term strength of this formation when cutting into this formation or adding load on this formation.

5.13.8 Coastal Range Siltstone/Claystone

The Coast Range, or Willapa Hills, are situated between the Olympic Mountains to the north and the Columbia River to the south. Thick sequences of Tertiary sedimentary and volcanic rocks are present. The rocks are not intensely deformed but have been subjected to compressional tectonism and have been somewhat folded and faulted (Lasmanis, 1991). The Willapa Hills have rounded topography, deep weathering profiles, and typically thick residual soil development. The interbedded sandstone and fine-grained sedimentary formations are encountered in highway cuts. The material from these cuts has been used in embankments. Some of the rock excavated from these cuts will slake when exposed to air and water and cause settlement of the embankment, instability and pavement distortion.

Locally thick clayey residual soils are present and extensive areas are underlain by sedimentary and volcanic rocks that are inherently weak. Tuffaceous siltstone and tilted sedimentary rocks with weak interbeds are common. The volcanic units are generally altered and or mechanically weak as a result of brecciation. Large and small-scale deep-seated and shallow landsliding are widespread geomorphic processes in this province. The dominant forms of landsliding are translational landslides, earthflows or slump-earthflows, and debris flows (Thorsen, 1989). Many of these are made up of both soil and bedrock. Reactivation of landslide in some areas can be traced to stream cutting along the toe of a slide.

5.13.9 Troutdale Formation

The Troutdale Formation consists of poorly to moderately consolidated and weakly lithified silt, sand and gravel deposited by the ancestral Columbia River. These deposits can be divided into two general parts; a lower gravel section containing cobbles, and upper section that contains volcanic glass sands. The formation is typically a terrestrial deposit found in and proximal to the present-day flood plain of the Columbia River and the Portland Basin. The granular components of the formation are typically well-rounded as a result of the depositional environment and are occasionally weakly cemented. Occasional boulders have been found in this formation. Excavation for drilled shafts and soldier piles in these soils can be very difficult because of the boulders and cemented sands.

Slope stability issues have been observed in the Troutdale Formation. Significant landslides have occurred in this unit in the Kelso area. Wet weather construction can be difficult if the soils have significant fines content. As described above, when the moisture content of soil with relatively high fines content rises a few percent above optimum, the soils become muddy and unstable. Permeability in this geologic unit varies based on the fines content or presence of lenses or layers of cemented and/or fine-grained material.

5.13.10 Marine Basalts - Crescent Formation

The Crescent Formation basalts were erupted close to the North American shoreline in a marine setting during Eocene time (Lasmanis, 1991). The formation consists mostly of thick submarine basalt flows, which commonly formed as pillow lavas. The Crescent Formation was deposited upon continentally derived marine sediments and is locally interbedded with sedimentary rocks. The Crescent Formation extends from the Willapa Hills area to the Olympic Peninsula. During the middle Eocene, the Crescent Formation was deformed during accretion to North America. The pillow basalts have extensive zones of palagonite and interstitial clay. Along the Olympic Peninsula the basalts are generally highly fractured and are often moderately weathered to decomposed.

The properties of the marine basalts are variable and depend on the amount of fracturing, mineralogy, alteration and weathering. Borrow from cut sections is generally suitable for use in embankments; however, it may not be suitable for use as riprap or quarry spalls because of degradation and slaking characteristics. All marine basalts should be tested for degradation before use as riprap or quarry spalls in permanent applications.

5.13.11 Mélange Rocks on Olympic Peninsula

During the middle Miocene, convergence of the Juan de Fuca plate with the North American plate accelerated to the point that sedimentary, volcanic, and metamorphic rocks along the west flank of the Olympics were broken, jumbled, and chaotically mixed to form a mélange (Thorsen, 1989). This formation is known as the Hoh rock assemblage. Hoh mélange rocks are exposed along 45 miles of the western coast. Successive accretionary packages of sediments within the core of the mountains are composed of folded and faulted Hoh and Ozette mélange rock. Typical of mélange mixtures, which have been broken, sheared and jumbled together by tectonic collision, the Hoh includes a wide range of rock types. Resistant sandstone and conglomerated sequences are extensively exposed in headlands and terraces along the Olympic coast. The mélange rocks may include pillow basalt, deep ocean clay and submarine fan deposits. Slopes in tilted sedimentary rocks that have been extensively altered and/or contain weak interbeds have been undercut by wave action in places along the Strait of Juan de Fuca. Slump flows or bedding plane block glides form along the interbeds.

Because of the variability of the mélange rocks and the potential for failure planes, caution should be used when designing cuts. A robust field exploration program is essential to determine the geometry and properties of the soil and rock layers.

5.14 Application of the Observational Method to Adjust Design Properties

The observational method as described by Peck (1969) and Wu (2008) may be used to adjust design parameters based on measured performance during construction. This approach may be used in the following ways:

- Planning during design that measurements will be taken and observations will be made during construction to verify the design assumptions used, or
- To address unexpected performance during construction.

The application of the observational method includes the following elements (Peck, 1969):

1. "Exploration sufficient to establish at least the general nature, pattern and properties of the deposits, but not necessarily in detail.
2. Assessment of the most probable conditions and the most unfavorable conceivable deviations from these conditions. In this assessment geology often plays a major role.
3. Establishment of the design based on a working hypothesis of behavior anticipated under the most probable conditions.
4. Selection of quantities to be observed as construction proceeds and calculation of their anticipated values on the basis of the working hypothesis.
5. Calculation of values of the same quantities under the most unfavorable conditions compatible with the available data concerning the subsurface conditions.
6. Selection in advance of a course of action or modification of design for every foreseeable significant deviation of the observational findings from those predicted on the basis of the working hypothesis.
7. Measurement of quantities to be observed and evaluation of actual conditions.
8. Modification of design to suit actual conditions."

If the observational method is to be used as part of the design process, the design shall meet the requirements of this manual, adjusting the design as needed during construction to be consistent with the performance observed.

5.15 References

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