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Chapter 10
INTERPRETATION OF ENGINEERING PROPERTIES

10.1 GENERAL

10.1.1 Overview

This Chapter identifies methods for establishing soil and rock properties required for geotechnical design. These design properties are based on field explorations and laboratory tests described in Chapters 8 and 9, respectively. Site performance data should also be used if available, to help determine the final geotechnical properties to be used for design.

For most projects, the interpretation of engineering properties requires a sound understanding of the geology at the project site. This understanding includes the process that form the rock or soil layers at the site, the major strata that occur within the depth of interest and the geologic history of the site. The geologic history includes past glaciations, erosion and deposition from rivers and changes in groundwater elevation. By understanding the likely history of the deposits, particularly methods of formation and changes in environmental conditions, a basis for the interpretation of engineering properties can be developed.

Another fundamental requirement for interpreting engineering properties is understanding the development plans for the project (e.g., bridges, retaining walls, highway embankments, slope excavations). Each of these project components has different requirements for design soil or rock properties. The anticipated construction sequence is also an important consideration in the evaluation of design properties. In the absence of a clear understanding of the development plans, it is easy to waste effort on determining soil properties not required for design or determining properties that are inconsistent with the final loading mechanisms.

After the project geotechnical specialist has developed an understanding of the site geology and project development plans, systematic processes are used to interpret engineering properties from the results of field investigations, field testing and laboratory tests. These processes range from direct property determination (e.g., laboratory test results) to the use of empirical correlations based on field measurements or soil classification index tests. It is the project geotechnical specialist's responsibility to determine which parameters are critical to the project's design and, based on those parameters, determine an acceptable level of accuracy.

10.1.2 Role of Uncertainty in Property Interpretation

Natural geomaterials are particularly difficult to quantify because of the various factors that affect their behavior. For example, soils are affected by their initial stress state, direction of loading, composition, drainage conditions and loading rate; the engineering behavior of rock depends on the number and location of discontinuities, fractures, joints, fissures, cracks and planes of weakness. Usually, it is impossible to define all these characteristics through field explorations and laboratory testing; cost does not allow the detail to resolve all uncertainties. For this reason, an important component of geomaterial property determination is establishing reasonable property variations and the properties that represent the best estimate.

The process of determining “best estimates” and likely variations is often difficult. While additional testing may help quantify best estimates and likely ranges, often the amount of data collected is not sufficient to provide quantitative determinations of means and standard deviations. It is often necessary to use experience in combination with a sound understanding of the geologic history of the site to determine the design soil or rock properties. Experience from previous design and construction in similar geologic settings serves as a good basis for selecting design properties.

The tendency to be “conservative” in material property determination is not necessarily the best approach to property selection. There are two reasons for this.

- First, being conservative can result in unreasonably expensive designs. Load and resistance factors or factors of safety are used during design to account for uncertainties in load and capacity. Selecting lower bound soil properties when combined with required load and resistance factors or factors of safety can lead to excessive conservatism.

- Second, in some areas (e.g., seismic loading), use of conservative properties can result in unconservative loads during the design seismic event. For example, underestimation of the low-strain shear modulus (G_{max}) of the soil may lead to an unconservative estimate of peak ground motions.

Good practice during property selection is to provide a best estimate of the soil or rock property and to identify a reasonable variation of that property. This allows the project designer to evaluate the response under realistic conditions and to determine potential consequences if unexpected conditions occur.

10.1.3 <u>Role of Drainage in Property Interpretation</u>

A fundamental concept that must be recognized during property interpretation is the role of pore pressures and porewater drainage in the selection of material properties. This issue is primarily related to the behavior of soils, but it can also affect the behavior of the rock mass along fractures and joints. The drainage within soil controls the strength of the material and whether the material behaves in an undrained state, a drained state or somewhere in between. This difference will result in different material response. Drainage also determines the rate at which soil consolidation occurs. Similar concepts apply to the fractures and joints within rock.

Drainage occurs as water within soil voids, or sometimes fractures and joints in rock, responds to the load. The load can be the result of new construction or it can be due to natural mechanisms within the earth (e.g., earthquake). If the soil is fully saturated, the pore fluid initially carries the load, resulting in higher porewater pressures and lower effective stress and, hence, lower strength. Before this pressure dissipates, the soil is considered to be undrained. As drainage occurs, the porewater pressures decrease and strength increases. Volumetric change also accompanies drainage, leading to settlement of the soil. After the porewater pressures have fully dissipated, the soil or rock mass behaves in a drained state.

Whether the geomaterial behaves in an undrained or drained condition will depend on the permeability of the soil or rock mass and the rate of loading. For example, if loading is relatively slow and permeability is high, as might be the case for clean sandy gravel, excess porewater pressures may not develop and the soil responds in a drained condition. However, if permeability is low, the soil may behave in an undrained state for a significant period after the load is applied. The rate of loading relative to the permeability is also an important consideration. The same geomaterial that behaves in a drained condition during embankment loading may behave in an undrained state for seismic loads. In this case, there is a change in the relationship between rate of loading and permeability or drainage characteristics, resulting in a different material response.

These drainage considerations constitute a fundamental concept of geotechnical engineering analyses as discussed in Part III of the *Geotechnical Manual*, and they are fundamental to material property selection.

10.1.4 References

For further guidance on interpreting engineering properties, the project geotechnical specialist should review the following documents:

- Geotechnical Engineering Circular No. 5*, Evaluation of Soil and Rock Properties*, FWHA-IF-02-034, Sabatini et al.;
- *Manual on Subsurface Investigations — Geotechnical Site Characterization*, FHWA-NHI-01-031; and
- *Manual on Estimating Soil Properties for Foundation Design*, EPRI EL-6800.

10.2 INTERPRETATIONS OF SOIL PROPERTIES

10.2.1 Overview

After the lab and field data have been collected and reviewed, the process of final material property selection begins. At this stage, the project geotechnical specialist should have:

- summarized data obtained in the field and from laboratory test results,
- collected information about the geology of the area and the project development plans, and
- reviewed other projects in the area with similar soil conditions.

Particular care must be exercised when empirical correlations are used to determine engineering properties. Properties estimated from correlations have greater variability than direct field measurements or measurements using laboratory performance data. The inherent variability of the soil and the uncertainties in determining engineering properties means that properties estimated from correlations to field measurements or laboratory test results usually are based on multiple measurements within each significant geologic unit (i.e., if the geologic unit is large enough to obtain multiple measurements). Checks should also be made with alternative methods whenever practical.

If the lab and field data are not consistent with each other or with previous experience, the variances should be evaluated, poor data eliminated and data trends identified. At this stage, it may be necessary to conduct additional field investigations or laboratory tests to try to resolve discrepancies, particularly if the discrepancies affect key elements of the project.

10.2.2 Engineering Properties from Field Measurements

10.2.2.1 Property Determinations

There are two methods for determining engineering design properties from field measurements:

1. Direct Property Determination. In this method, field measurements provide an estimate of the engineering property. The determination of undrained shear strength from in-situ vane shear tests is an example of this type of determination.

2. Indirect Property Determination. In this method, empirical correlations between the field measurement and engineering properties are used. The determination of strength from Standard Penetration Test (SPT) and Cone Penetration Test (CPT) measurements is an example of this type of determination.

Direct and indirect property determinations from the most commonly used field methods (e.g., SPT, CPT, VST, permeability) are summarized in the following Sections. See FHWA *Evaluation of Soil and Rock Properties* and *Subsurface Investigations — Geotechnical Site Characteristics* for a more detailed discussion property determinations.

10.2.2.2 Standard Penetration Test (SPT)

The most common field test measurement used within the geotechnical profession and by MDT is the SPT. As discussed in Chapter 8, this test provides a measurement of the resistance of the soil, in terms of hammer blow counts (e.g., N-value), when driving a split spoon sampler the final 12 in (300 mm) of the 18-in (450-mm) total. The N-values are used to estimate the consistency, strength and, in some cases, the compressibility of the soil.

10.2.2.2.1 Corrections to Standard Penetration Tests

A number of factors affect the SPT blowcount and need to be considered before using the N-values to estimate engineering properties. Correction factors are used to adjust the field measured N-value to a standard that is consistent with the test energies that were originally used to develop the empirical correlation. The factors range from the energy of the system to details of the testing equipment. The FHWA *Evaluation of Soil and Rock Properties* and FHWA *Subsurface Investigations — Geotechnical Site Characteristics* provide guidance in the determination of these factors.

Two correction factors are particularly important during the use of the SPT blowcount:

1. Energy Correction. The energy delivered during the SPT can vary over a relatively wide range, depending on the hammer system used to conduct the SPT. For this reason, most empirical correlations are based on SPT blowcounts normalized to 60% of the theoretical energy, referred to as the N_{60} value. The energy for a particular hammer system can be determined using the procedures in ASTM D 4633. Periodic recalibration of the hammer system is also necessary. SPTs conducted by the Geotechnical Section use an automatic hammer to provide more consistency in the blowcount measurement. Automatic hammers used on MDT drill rigs are periodically tested to verify actual hammer energies.

2. Overburden Stress. Because N-values of similar materials increase with increasing effective overburden stress, the corrected blowcount (N_{60}) for many recent empirical correlations has also been normalized to 1-atmosphere (100 kPa) effective overburden stress using overburden normalization schemes. The blowcount normalized to 1-atmosphere at 60% energy is referred to as the $(N_1)_{60}$ value. See the FHWA Manuals for equations that should be used to make the overburden correction.

The project geotechnical specialist should understand which blowcount is applicable when using SPT correlations to determine soil properties. Some published correlations are based on corrected values for both energy and confining pressure $(N_1)_{60}$, and some are based on blowcounts corrected for energy (N_{60}) only.

10.2.2.2.2 N-Value Correlations

Figures 10.2-A and 10.2-B provide typical correlations used to interpret the relative density and friction angle for granular soils and the consistency and undrained strength for cohesive soils. In these figures, the blowcount is corrected to 60% energy, but not to an overburden pressure of 1-atmosphere (100 kPa).

N_{60} Value (blows/ft or 305 mm)	Relative Density	Approximate ϕ'_{tc} (degrees)	
		Peck, Hanson and Thornburn	Meyerhof
0 to 4	very loose	< 28	< 30
4 to 10	loose	28 to 30	30 to 35
10 to 30	medium	30 to 36	35 to 40
30 to 50	dense	36 to 41	40 to 45
> 50	very dense	> 41	> 45

Figure 10.2-A — CORRELATION OF SPT N_{60} VALUE AND FRICTION ANGLE FOR GRANULAR SOILS

N Value (blows/ft (blows/305 mm))	Consistency	Approximate Undrained Strength (S_u) (tsf (kPa))
0 to 2	very soft	< 1/8 (13)
2 to 4	soft	1/8 to 1/4 (13 to 25)
4 to 8	medium	1/4 to 1/2 (25 to 50)
8 to 15	stiff	1/2 to 1 (50 to 100)
15 to 30	very stiff	1 to 2 (100 to 200)
> 30	hard	> 2 (200)

Figure 10.2-B — CORRELATION OF SPT N_{60} VALUE AND UNDRAINED STRENGTH (S_u) FOR COHESIVE SOILS

The project geotechnical specialist should apply engineering judgment when deciding whether to use the upper, average or lower values in Figures 10.2-A and 10.2-B. For example, the lower portion of the range in Figure 10.2-A tends to occur in cohesionless soil with a higher fines content; whereas, the upper portion of the range will occur in coarser material with less than 5% fines. The correlations shown for cohesive soil is commonly referenced; however, the relationship for cohesive soil is generally regarded as approximate, unless site-specific calibrations have been conducted with field vane shear results or undisturbed laboratory test data.

Various other correlations have been developed and are often cited in publications or used on a local basis. For example, the following correlations have been developed:

- SPT N-value versus preconsolidation pressure and overconsolidation ratio of cohesive soils (Mayne, P.W. and Kemper, J.B., "Profiling OCR in Stiff Clays by CPT and SPT," *Geotechnical Testing Journal*, ASTM, Vol. 11, No. 2, June 1988);

- Young's modulus and constrained modulus of cohesionless soils (*Design Manual 7.01 Soil Mechanics*, Department of Defense, UFC 3-220-10N, June 2005; Callanan, J.F. and Kulhawy, F.H., *Evaluation of Procedures for Predicting Foundation Uplift Movement*, Electric Power Research Institute, EPRI EL-4107, August 1985; *Manual on Estimating Soil Properties for Foundation Design*, EPRI EL-6800, August 1990); and

- Shear wave velocity and shear modulus (Kramer, S. L., *Geotechnical Earthquake Engineering*, Prentice-Hall, 1996).

10.2.2.2.3 Other Considerations

The following are several considerations that apply to the use of blowcount relations to determine the consistency, strength and compressibility of soils:

1. Appropriate Correlations. Only use soil property correlation if the correlation is well established. This requires comparing the prediction from the correlation to other direct measurements (e.g., undrained strength from SPT N-value to in-situ vane shear data); to high quality, measured laboratory performance data; or to back-analysis from full-scale performance of geotechnical elements affected by the geologic formation in question.

2. SPT Blowcount Estimates. Use extreme care when applying the SPT blowcount to estimate soil shear strength in soils with coarse gravel, cobbles or boulders. Large gravels, cobbles or boulders will often cause the SPT blowcounts to be unrealistically high. A larger sized SPT hammer and sampler (e.g., the Dames and Moore sampler having an inside diameter of 2.5 in (63 mm)) or a Becker hammer can be used in granular materials.

3. Uncertainties. Consider the uncertainties in the property measurement when selecting specific values for use in engineering design.

10.2.2.3 Cone Penetration Test (CPT)

The MDT Geotechnical Section uses 10 t (9,000 kg) electric cone penetrometer test (CPT) equipment to characterize soil conditions at some project sites, particularly where thick deposits of soft soil occur. The CPT provides semi-continuous readings of tip resistance (q_c) and sleeve friction (f_s) as a function of penetration depth. Pore pressures can also be recorded during the sounding using porewater filters, referred to as a piezocone test (i.e., CPTu). A horizontal geophone in the CPT can also be used to record mechanically induced shear waves from the surface, referred to as a seismic cone test. Results of CPTs are used to identify the following:

- layering and soil type at the sounding location,
- density and strength of cohesionless soil,
- consistency and undrained strength of cohesive soil, and
- permeability and shear wave velocity data.

10.2.2.3.1 Corrections to Cone Penetration Tests

The CPT is more standardized than the SPT in terms of the geometry of the test device, the rate of CPT penetration and collection of resistance data. Specifically, electronic transducers measure the resistance to penetration, data are recorded electronically and the results are processed through relatively well-accepted interpretation methods. This means that fewer corrections are needed before using the CPT results for interpreting engineering properties.

The primary correction to the CPT deals with the effects of water pressure on the force recorded by the CPT system. Most CPT vendors provide software that allows information collected in the field to be processed so that corrected tip stresses are provided as part of the processed field data.

Regular calibrations should be performed following manufacturer's recommendations to ensure that the force and pressure transducers are providing accurate data.

10.2.2.3.2 CPT Correlations

The corrected CPT results are processed to provide a plot of cone sleeve friction (f_s) and cone end resistance (q_t) as a function of depth. The friction ratio (FR), which is defined as the ratio of sleeve friction to end resistance ((f_s/q_t) x 100) should also be obtained. The FR is normally less than 1% in clean sands and greater than 4% in clays and silts of low to medium sensitivity. In highly sensitive clays, FR may be 1% or less.

A number of correlations have been developed to estimate soil properties from CPT results, as briefly summarized below. Also, see FHWA *Evaluation of Soil and Rock Properties* and FHWA *Subsurface Investigations — Geotechnical Site Characteristics* for more detailed discussions of these correlations:

1. Soil Type and Layering. An important and useful result of the CPT test is the plot of cone end resistance (q_t) versus depth. This plot gives the project geotechnical specialist a preliminary indication of the location of hard and soft layers within the sounding profile. Data from the corrected cone tip resistance and friction ratio can be used to estimate the soil type. The FHWA *Evaluation of Soil and Rock Properties* and FHWA *Subsurface Investigations — Geotechnical Site Characteristics* provide figures that may be used to determine correlation values. Alternative methods of interpreting soil type are also available (i.e., the Eslami-Fellenius method).

2. Strength and Consistency Information. Similar to the SPT, correlations have been developed between CPT field measurements and estimates of soil strength. These correlations take one of two forms – conversion to an equivalent SPT value or direct correlations. Of the two approaches, the latter is preferred and more commonly used as the confidence in direct correlations has improved. Consider the following:

    a. Direct Correlations. Various direct correlations have been developed between the cone end resistance and the friction angle of cohesionless soils and the undrained strength of cohesive soils. The FHWA *Evaluation of Soil and Rock Properties* and FHWA *Subsurface Investigations — Geotechnical Site*

Characteristics provide examples of the relationship for cohesionless and cohesive soils.

b. Cross-Checking. It is often valuable to perform a cross-check on strength or deformability properties estimated by the CPT results by obtaining the N-value and then using the relationships discussed in Section 10.2.2.2 for SPT. Figure 10.2-C shows a typical correlation between the normalized cone end resistance and the SPT N-value. Note that the correlation is normalized to 1-atmosphere (100 kPA). Also, note that there is considerable scatter in the data. This scatter suggests that the best approach is to develop a site-specific correlation by conducting a limited number of high quality SPTs along with the CPT soundings.

3. Pore Pressure Dissipation Results. CPT equipment that includes porewater pressure measurements during the sounding (referred to as a CPTu or a piezocone test) can be used to estimate approximate coefficient of consolidation of the fine-grained saturated soil. These pseudo time-rate consolidation measurements are obtained during the CPTu sounding by stopping the vertical advance of the CPTu probe. This determination is made during the CPT sounding by stopping the advance of the CPTu system and monitoring the dissipation of excess porewater pressures as a function of time.

The FHWA *Evaluation of Soil and Rock Properties* suggests the calculation of c_h be based on a degree of consolidation of 50% or greater. The coefficient of consolidation determined in this manner is for lateral dissipation. In the vertical direction, the coefficient of consolidation is roughly half the horizontal for isotropic soils and can be 0.1 times the horizontal in highly stratified soils, as in varved clays. Additional discussion on this topic can be found in FHWA *Evaluation of Soil and Rock Properties* and FHWA *Subsurface Investigations — Geotechnical Site Characteristics*.

4. Other Correlations. Various other relationships have been developed between results of the cone penetrometer test and engineering soil properties. Examples include the following:

- preconsolidation pressure and overconsolidation ratio in cohesive soils (Mayne, "CPT Indexing of In-Situ OCR in Clays," *Use of In-Situ Tests in Geotechnical Engineering* (GSP 6), ASCE, 1986; Mayne, P.W. and Holtz, R.D., "Profiling Stress History from Piezocone Soundings," *Soils and Foundations*, Vol. 28, No. 1, March 1988; Sabatini et al., *Evaluation of Soil and Rock Properties*);
- constrained modulus of cohesive and cohesionless soils (*Manual on Estimating Soil Properties for Foundation Design*, EPRI EL-6800, August 1990); and
- shear wave velocity and shear modulus (Kramer, S. L., *Geotechnical Earthquake Engineering*, Prentice-Hall, 1996).

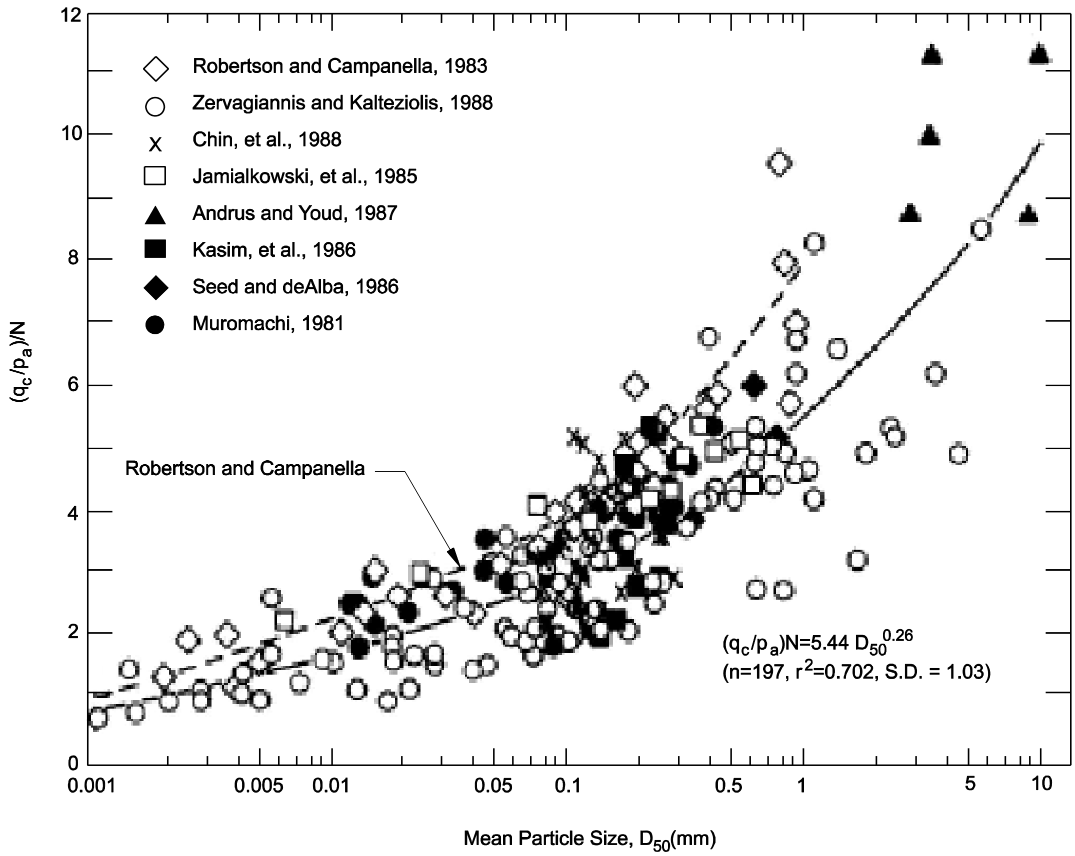

Figure 10.2-C — RECOMMENDED VARIATION OF Q_C/N WITH GRAIN SIZE (EPRI, 1990)

10.2.2.4 Vane Shear Test (VST)

Field measurement of strength can be obtained in cohesive soils using a field vane shear device. This device involves twisting a vane comprised of four blades in the soil. The height to diameter ratio of the conventional vane is 2. The geometry and torque required to twist the vane can be used to estimate three parameters:

- undrained shear strength ($s_{u,VST}$),
- remolded undrained shear strength ($s_{r,VST}$), and
- sensitivity ($S_{t,VST}$).

The subscript VST is added to each parameter to note that the parameter was obtained using vane shear test data. Procedures for correcting and interpreting VST results are summarized in the following Sections.

10.2.2.4.1 Strength Determination and Correction of Vane Shear Test Results

Equations in the FHWA *Evaluation of Soil and Rock Properties* can be used to calculate undrained shear strength of the soil. The remolded strength is achieved in the same manner as the peak strength, except the torque reading is taken during rotation of the vane following ten rapid turns. The torque associated with rod friction should be recorded prior to the remolded test, and then subtracted from the maximum torque recorded for calculations of $S_{r,VST}$. The sensitivity of the soil from vane shear tests is expressed as the peak strength to the remolded strength as shown in Equation 10.2-1:

$$S_t = \frac{S_{u,\,VST}}{S_{r,\,VST}} \qquad \text{(Equation 10.2-1)}$$

The S_u value from the VST should be corrected for strain rate during testing and soil anisotropy. The most common correction was developed by Bjerrum and is shown in Figure 10.2-D.

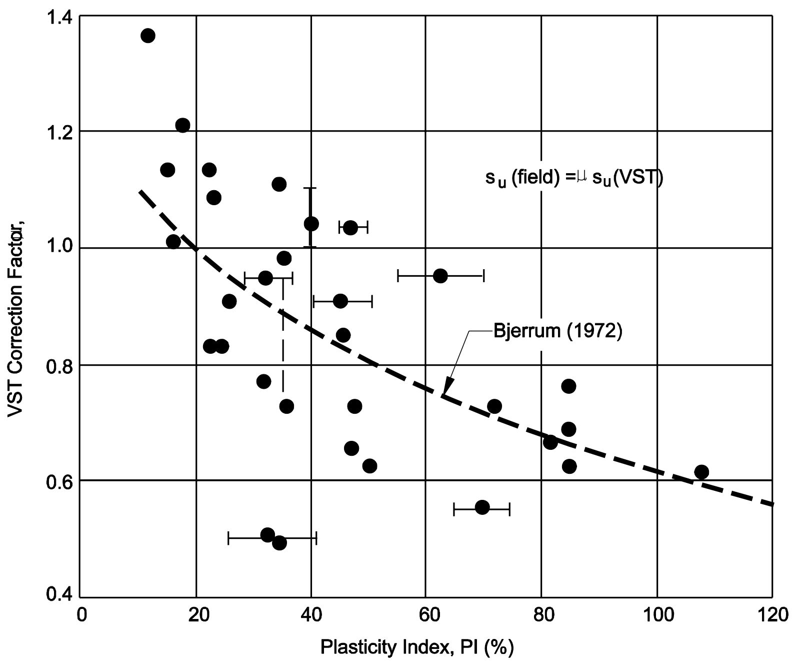

Figure 10.2-D — FIELD VST CORRECTION FACTOR

10.2.2.4.2 VST Constraints

The corrected VST provides a direct measurement of the undrained strength of the soil. The boundary conditions of the test are constrained by the geometry of the plane, which makes the test more suitable for measurement of strength in some soil types than others. For example, the test can provide misleading strength values if the soil is highly anisotropic.

The assumption is that there is no drainage across the failure surface during rotation. This limits the application to soils that have low permeability. If the soils are very stiff or contain coarser grained material (i.e., gravel), insertion of the vane system and the deformations along the failure surface may preclude effective measurement of strength.

10.2.3 Field Measurements of Permeability in Soil

Several field methods are available for obtaining information about the permeability of the soil. These methods include interpretation of CPT porewater dissipation data, and performance of slug tests and groundwater pump tests. Methods for estimating the coefficient of consolidation and the permeability from CPT dissipation tests are discussed in Section 10.2.2.3. Other methods include conducting slug tests and groundwater pump tests, which are discussed in FHWA *Subsurface Investigations — Geotechnical Site Characteristics*.

10.2.4 Engineering Properties from Laboratory Tests

The results of laboratory tests described in Chapter 9 provide the second basis for determining soil properties necessary for the geotechnical design. As discussed in Chapter 9, laboratory tests can provide information regarding the classification and the engineering properties of the soil. In most cases, engineering properties are determined directly from laboratory tests. However, as discussed in Section 10.2.5, engineering properties can also be determined indirectly by empirical correlations. The following Sections focus on the direct determination of engineering properties from laboratory tests.

10.2.4.1 Influence of Soil Disturbance on Property Measurement

The accuracy of the engineering property determination in the laboratory is significantly affected by the quality of the sample that is tested. Chapter 8 discusses handling and care of samples in the field; Chapter 9 describes issues related to sample handling, storage and selection in the laboratory. However, even when appropriate field and laboratory methods are used, sample disturbance can still occur, which is not always obvious. Key tasks after receiving the laboratory test information is, therefore, to review the results to confirm that engineering properties are reasonable.

The project geotechnical specialist can use various methods to judge the quality of the test results:

- The results of strength tests can be compared to estimates of strength parameters from SPT and CPT correlations, and from correlations based on classification properties; see Section 10.2.5.
- The form of the test data can also provide an indication of disturbance. The most obvious example is the shape of a consolidation test curve. Samples that are disturbed will have a flatter change in the e-log p' curve than results from testing undisturbed samples. Similar observations can be made from the shapes of the stress-strain curve or p'-q' plots from triaxial strength testing.
- The methods used in the field and any notes recorded on the boring log indicating difficulties with drilling or sampling for the test specimen.

Engineering judgment goes into determining whether the test results are representative of in-situ conditions. The effects of disturbance on soil property determination will potentially have a significant effect on foundation design studies and, therefore, a critical review of the laboratory results is a very important step in the design process.

10.2.4.2 Strength Test Corrections

Most laboratory properties are interpreted for direct use during design. However, the triaxial test may need to be corrected to account for differences in stress paths between the lab test and the field loading condition. The stress path effect is most evident for cohesive soils. Various studies have shown that the stress path during a conventional unconsolidated undrained (UU) triaxial test or consolidated, isotropic undrained (CIUC) triaxial test results in a different strength than other paths of loading (e.g., direct simple shear (DSS)). Further guidance is provided in FHWA *Evaluation of Soil and Rock Properties*.

During geotechnical design, the project geotechnical specialist must make a decision on what stress path is representative of the loading condition and how this relates to the laboratory and field tests that have been conducted on soils samples from the site. For laboratory strength tests, this may mean that a correction to the measured test result is necessary. For example, the results of a CIUC test may need adjusting to reflect the DSS result if the latter is more representative of the actual stress path.

Many studies have been conducted to evaluate these strength differences. The plot in Figure 10.2-E illustrates a composite of the correction factors that have been developed from a number of different studies. Most geotechnical testing laboratories do not have the capability of conducting this type of test. In this situation, the relative comparisons shown in Figure 10.2-E can be used to make an approximate adjustment to more conventional triaxial test results.

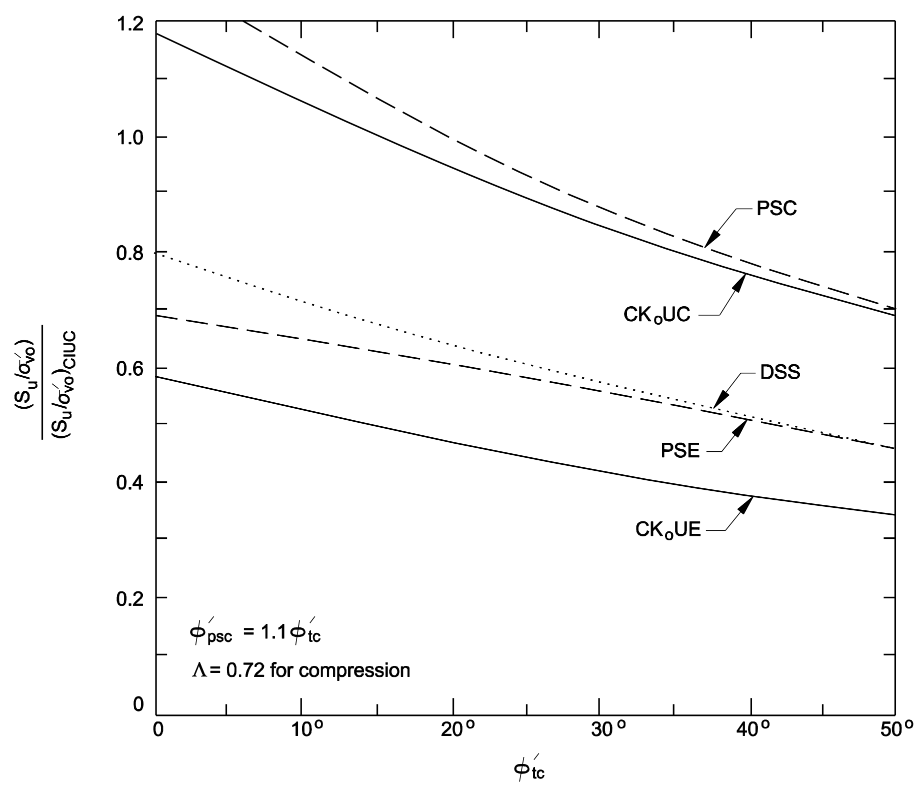

Figure 10.2-E — NORMALIZED STRENGTH RATIO FOR MAJOR LABORATORY SHEAR TEST TYPES

10.2.5 Engineering Properties from Empirical Correlations

Various empirical correlations between engineering properties and soil classification properties have been developed. These empirical correlations are often based on statistical regression analyses between the engineering or stress-state property and the index test result. Consequently, considerable care must be taken when using these correlations, as significant differences can occur between the actual and predicted properties for a specific soil type. As long as this limitation is recognized, the empirical correlations provide a good basis for checking or estimating engineering properties, or for verifying the reasonableness of field and laboratory test results.

The following list of engineering properties has been correlated to soil classification index properties:

- In-Situ Stress State:
  - + effective preconsolidation pressure versus liquidity index and sensitivity in cohesive soils,
  - + overconsolidation ratio from liquidity index in cohesive soils, and
  - + effective horizontal stress (K_o) from plasticity index and liquid limit in cohesive soils.
- Strength:
  - + effective friction angle in cohesionless soils from relative density,
  - + effective friction angle in clays from plasticity index, and
  - + vane shear strength from plasticity index and liquidity index for undisturbed and remolded cohesive soils.
- Deformability:
  - + compression and recompression indices as a function of liquid limit and plasticity index,
  - + coefficient of consolidation versus liquid limit, and
  - + coefficient of secondary compression versus moisture content.
- Coefficient of permeability as a function of particle size and relative density.

Many of the above relationships are summarized in the *Manual on Estimating Soil Properties for Foundation Design* (EPRI, 1990), FHWA *Evaluation of Soil and Rock Properties* and FHWA *Subsurface Investigations — Geotechnical Site Characteristics*.

10.3 INTERPRETATION OF ROCK PROPERTIES

Engineering properties of rock are generally controlled by the discontinuities within the rock mass and not the properties of the intact material. Therefore, engineering properties for rock must account for the properties of the rock mass as a whole, specifically considering joint spacing, fractures, degree of weathering, direction of dip for slip planes, type of infilling and groundwater conditions. A combination of laboratory testing of small samples, empirical analysis and field observations should be employed to determine the engineering properties of rock masses, with greater emphasis placed on visual observations and qualitative descriptions of the rock mass than the process used to determine engineering properties of soil.

10.3.1 Rock Property Characterization

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. Engineering properties obtained from laboratory tests include:

- specific gravity;
- unit weight;
- elastic properties (e.g., ultrasonic velocity, modulus, Poisson's ratio);
- compressive strength; and
- tensile strength.

Rock mass properties are determined by visual examination of discontinuities within the rock mass and how these discontinuities will affect the behavior or the rock mass when subject to proposed construction. The following Sections provide a brief overview of the relationship between rock mass classification, intact rock property determination and rock properties used for design.

10.3.1.1 Rock Mass Classification

The classification of rock mass is a difficult process, usually requiring the expertise of an experienced geologist to recognize and appropriately characterize the rock mass. Various classification systems have been developed. The two most widely used methods include the Rock Mass Rating (RMR) system and the Geological Strength Index (GSI) system. These methods combine the effects of intact rock compressive strength, Rock Quality Designation (RQD), spacing of discontinuities, ground water conditions and orientation of discontinuities.

Additional information about these rock classification systems are provided in the FHWA *Evaluation of Soil and Rock Properties*, FHWA *Subsurface Investigations — Geotechnical Site Characteristics* and the FHWA *Rock Slopes: Reference Manual*. ASTM D 5878 also summarizes the guidelines for these rock classification systems.

10.3.1.2 Intact Rock Property Determination

The intact properties of rock can be determined by either field or laboratory tests, as discussed in Chapters 8 and 9 and summarized below:

1. <u>In-Situ Tests</u>. Available in-situ tests include various types of borehole dilatometers. These tests provide a measurement of the rock modulus as measured in a borehole. The results from these tests also provide information about the rock mass, depending on the number of borehole tests. The larger zone of rock tested by the in-situ test captures more of the characteristics of the rock mass. Specialized tests can also be conducted in the field to measure components of the rock mass characteristics (e.g., interface strength along rock joints). However, the cost of these tests precludes use on most typical projects. Even with in-situ test results that theoretically include more macro rock mass considerations, the project geotechnical specialist must determine whether the test adequately captures the rock mass characteristics.

2. <u>Laboratory Tests</u>. Tests on intact samples of rock can be performed in the laboratory. These tests range from point load tests to triaxial compression tests. Tests are conducted on relatively small samples or cores; consequently, they do not consider the overall rock mass. Though laboratory tests do not adequately account for the overall properties of the rock mass, information from these tests, when modified according to rock mass classification information, provides a basis for engineering design.

3. <u>Published Results</u>. This involves the use of published tables showing the unconfined compressive strength of different rock types (e.g., *FHWA Subsurface Investigations — Geotechnical Site Characteristics*). These strength values can provide a useful starting point for the assessment of rock strength. However, large variations in strength can occur for intact rock samples within any specific rock type. This potential for variation must be considered when using this type of information.

10.3.2 <u>Rock Properties for Design</u>

In most cases, rock properties used for design must represent the rock mass rather than the intact strength of a rock sample. The two properties of primary interest for geotechnical design are the rock mass modulus and the rock mass strength.

The FHWA *Evaluation of Soil and Rock Properties* and FHWA *Subsurface Investigations — Geotechnical Site Characteristics* further discuss rock property characterization, including the determination of rock mass modulus and rock mass strength, alternative methods of determining strength and modulus using empirical correlations, results of intact rock testing and reference to in-situ borehole rock testing.

10.4 CHARACTERIZATION OF SITE FOR DESIGN

10.4.1 Development of Subsurface Profiles

Subsurface profiles should be developed from the field and laboratory test data. Development of these profiles is usually the first task performed by the project geotechnical specialist for design. Longitudinal profiles are typically developed along the roadway or bridge alignment, and a limited number of transverse profiles may be included for key locations (e.g., major bridge foundations, cut slopes, high embankments). These profiles provide an effective means of summarizing pertinent subsurface information and illustrating the relationship of the various investigation sites. The subsurface profiles, coupled with judgment and an understanding of the geologic setting, aid the project geotechnical specialist in interpreting subsurface conditions between the investigation sites.

In developing a two-dimensional subsurface profile, the profile line (typically, the roadway centerline) should be defined on the base plan and the relevant borings projected to this line. Use judgment in the selection of the borings because projection of the borings, even for short distances, may result in misleading representation of the subsurface conditions. The subsurface profile should be presented at a scale appropriate to the depth of the borings, frequency of the borings and soundings, and overall length of the cross-section. Sometimes, an exaggerated scale of 10H:1V or 1H:20V may be required.

Road and bridge designers sometimes request that the Geotechnical Section present a continuous subsurface profile that shows an interpretation of the location, extent and nature of subsurface formations or deposits between borings. Where rock or soil profiles vary significantly between boring locations, the value of these presentations becomes questionable. The project geotechnical specialist must be cautious in presenting these data. These presentations should include clear and simple caveats explaining that the profiles as presented cannot be fully relied upon. Lines separating strata should include “question mark” symbols wherever data are uncertain or where different conditions could be interpreted.

Where there is a requirement to provide highly reliable continuous subsurface profiles, the project geotechnical specialist should increase the frequency of borings and, possibly, supplement the information with and/or use geophysical methods to determine the continuity or heterogeneity of subsurface conditions.

10.4.2 Correlation of Field and Laboratory Information

The subsurface profile is intended to provide a best estimate of the material types, layering and groundwater location at the project site. These profiles are developed based on the field explorations and laboratory tests (e.g., SPT N-values, CPT soundings, visual soil descriptions, laboratory test results). Ideally, information collected by these different methods is consistent and the process of developing soil or rock properties for design is a straightforward task. Occasionally, this ideal situation actually occurs. However, more often than not, inconsistencies between descriptions or properties need to be reconciled.

The process of reconciling inconsistencies is not simple. It requires an appreciation for the geological processes that led to the development of the geologic conditions, and it requires an understanding of the methods used to characterize the properties of the materials. For both field and laboratory testing, the boundary conditions during the test (e.g., geometry, load paths, drainage conditions) will affect the characterization of properties.

Much of the reconciliation process results from experience. However, the following guidelines can help understand and often remove inconsistencies:

- Compare results of testing to results of work in similar site conditions to determine whether the inconsistency is unique to the site.
- Use empirical correlations to check the "reasonableness" of the information.
- Consider the anticipated loading conditions relative to the stress state during the in-situ or laboratory test to determine whether the inconsistency is an issue.
- Check calculations, including calibration factors, to confirm that the inconsistency is not a math error or equipment issue.
- Look at the bulk of the data to determine whether the inconsistency is an anomaly or is within the scatter normally associated with the material type.
- Evaluate whether a unique environmental condition (e.g., weather, artesian pressure) could have caused the anomaly.
- Discuss the results with a colleague.

Sometimes these efforts to reconcile differences are not successful. In this case, the project geotechnical specialist should further explore the affects of the inconsistency on the results of engineering analyses. If the inconsistency is "real," then it may be necessary to perform analyses with the inconsistency considered and not considered. Parametric studies are useful in this regard.

In most cases, the approach should not be to bind the soil or rock properties. Where discrepancies occur, the preferred approach is to analyze using the best rational estimate of soil or rock properties and, then, evaluate the potential consequences of using lower or higher parameters. With this information, it is then possible to meet with project personnel and make an informed decision on the approach that should be taken. The informed decision could be that further field and laboratory testing is required.

10.4.3 Approach for Handling Variability and Uncertainty

An important factor in the selection of design properties for geomaterials is the affects of variability and uncertainty in test results and soil/rock conditions. In some cases, it is possible to use statistics in the property evaluation and selection process. This approach is useful when the database of information is large enough to allow a formal statistical treatment. Few projects have a sufficient database to justify complex statistical evaluations. It is more common for projects that have a large database to simply work with or consider mean and standard deviation values.

Soil Slope and Slope Stability (Duncan and Wright, 2005) provides a formal discussion of reliability in geotechnical engineering, including the relationship between factors of safety and probabilistic measures (e.g., reliability used for engineering projects. This discussion includes tables that summarize the coefficient of variation for soil properties and in-situ tests. If suitable data are available, the probability of failure for a given condition can be calculated by following the procedures recommended in this document.

10.4.4 Selection of Design Properties for Engineered Fills

10.4.4.1 General

Properties of engineered fills warrant special mention relative to property selection, as these properties are not normally available to the project geotechnical specialist from the field investigation. Soil and rock used as engineered fills usually can be described in broad categories based on their gradation and compaction requirements, taking into consideration the typical geologic source of the fill material. These broad categories allow typical properties to be assigned to the fills. These properties can be useful when performing geotechnical design studies.

For materials where the gradation specification is broad (e.g., common borrow), consider a wider range of properties. In cases where the range of uncertainty has significant implication from construction costs or schedule, it is critical to narrow the range by sampling the borrow source and conducting appropriate field and laboratory tests.

10.4.4.2 Common Borrow

A common borrow material may consist of soil or aggregate either naturally occurring or processed that is substantially free of organics or other deleterious material and is generally non-plastic. However, in regions of Montana (primarily, eastern Montana) where granular material is relatively scarce, soils exhibiting plasticity are often used as common borrow. 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 property values for typical materials is provided in Figure 10.4-A. Lower range values should be used for compacted finer grained materials.

Often 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 project geotechnical specialist has specific knowledge of the source(s) likely to be used, or unless quality assurance shear strength testing is conducted during construction.

Some common borrow areas will have a high enough fines content to be moderately to highly moisture sensitive. This moisture sensitivity may affect the design property selection if placement conditions are likely to be marginal due to the timing of construction.

Material	Soil Type (USCS Classification)	φ (degrees)	Cohesion (psf) (kg/m^2)	Total Unit Weight (pcf) (kg/m^3)
Common Borrow	ML, SM, GM	28 to 32	0	115 to 130 (1840 to 2080)
Select Borrow	GP, GP-GM, SP, SP-SM	34 to 36	0	120 to 135 (1920 to 2160)
Gravel Borrow	GW, GW-GM, SW, SW-SM	36 to 40	0	130 to 145 (2080 to 2320)
Gravel Backfill for Walls	GW, GP, SW, SP	36 to 40	0	125 to 135 (2000 to 2160)

Figure 10.4-A — PRESUMPTIVE DESIGN PROPERTY RANGES FOR BORROW

10.4.4.3 Granular Embankment Fill

Specifications for granular embankment fill ensure that the mixture is granular and contains at least a minimal amount of gravel size material. Some granular embankment fill may be poorly graded sand and contain enough fines to be moderately moisture sensitive. Granular embankment fill is not an all-weather material.

Results of past triaxial or direct shear strength testing that meets granular embankment fill gradation requirements typically show that drained friction angles of 38° to 45° are likely when the soil is well compacted. Even in it loosest state, results of shear strength testing of relatively clean sands meeting granular embankment fill requirements range from values of 30° to 35°. However, these values are highly dependent on the geologic source of the material. Windblown or alluvial sands that have been rounded through significant transport could have significantly lower shear strength values. Leftovers 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 is provided in Figure 10.4-A.

As noted for common borrow, the specific source of borrow may not be known during design. Therefore, it is not prudent to select a design friction angle that is near or above the upper end of the range unless the project geotechnical specialist has specific knowledge of the source(s) likely to be used or unless quality assurance shear strength testing is conducted during construction.

10.4.4.4 Clean Granular Borrow

The clean granular borrow specification should ensure a reasonably well graded sand and gravel mix. Because the fines content is under 7% by definition, the material is only slightly moisture sensitive. However, if very wet conditions are anticipated because of the location or season, a lower fines content criterion may be desirable.

Larger diameter triaxial shear strength testing performed on well graded mixtures of gravel with sand indicate that very high internal angles of friction are possible (e.g., as high as 50°), and that shear strength values less than 40° are not likely. However, lower shear strength values may occur for clean granular borrow from naturally occurring materials obtained from non-glacially derived sources (e.g., wind blown, alluvial deposits).

In many cases, processed materials are used for clean granular borrow. In this case, this processed material is crushed, resulting in 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 is provided in Figure 10.4-A.

If the specific source of borrow is not known, it is generally not prudent to select a design friction angle that is near or above the upper end of the range unless the project geotechnical specialist has specific knowledge of the source(s) likely to be used or unless quality assurance shear strength testing is conducted during construction.

Gravel backfill behind walls should consist of free-draining material to facilitate drainage. This material has similarities to clean granular borrow, but generally contains fewer fines to facilitate the free-draining characteristics. Gravel backfill for walls is often a processed material and, if crushed, is likely to have a high soil friction angle. Typical ranges of material properties are provided in Figure 10.4-A. Where the selection of friction angle is critical to design, contract specifications should require that the assumed friction angle be confirmed with laboratory tests on the material to be used during construction.

10.4.4.5 Rock Embankments

Embankment material is considered rock embankment if 25% of the material is over 4 in (100 mm) in diameter. Compactive effort is generally based on a method specification. Because of the nature of the material, conventional compaction testing using a nuclear gauge and Proctor method is generally not feasible. The method specification allows for a broad range of material and properties so 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. In addition, if the rock is weak, failure may occur through the rock fragments rather than around them.

In these types of materials, the strength parameters will likely resemble those of earth embankments. For existing embankments, the soft rock may continue to weather with time if the embankment materials are continually wet or are exposed to numerous wet-dry cycles. Where these materials are anticipated for use as embankment materials, Micro-Deval tests should be conducted to determine the durability of the particles, and the results of these tests used to estimate a friction angle. As the material becomes softer, the frictional characteristics become more soil-like.

For sound rock embankments, the strength parameters may be much higher than those constructed of weak rock or intermediate geomaterials. For compacted earth embankments with sound rock, internal friction angles of up to 45° may be reasonable. Unit weights for rock embankments generally range from 130 pcf to 140 pcf (2080 kg/m^3 to 2240 kg/m^3).

10.4.4.6 Quarry Spall and Riprap

Quarry spalls, light loose riprap and heavy loose riprap 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. For design purposes, typical values of 120 pcf to 130 pcf (1920 kg/m^3 to 2080 kg/m^3) are used 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° are typically used for this type of materials.