Chapter 7
GEOMECHANICS

Final

SCDOT GEOTECHNICAL DESIGN MANUAL

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CHAPTER 7

GEOMECHANICS

7.1 INTRODUCTION

This chapter presents the geotechnical design philosophy of SCDOT. This philosophy includes the approach to the geotechnical investigations of the project, and the correlations that link the field and laboratory work that precedes this chapter to the engineering analysis that is subsequent to this chapter. The approach to the geotechnical investigation of transportation projects entails the use of preliminary and final explorations and reports. The development of an understanding of the regional and local geological environment and the effect of seismicity on the project is required. The geotechnical approach provided in this chapter is not meant to be the only approach, but a representative approach of the thought process expected to be used on SCDOT projects. The geotechnical engineer-of-record shall develop a design approach that reflects both the requirements of this Manual as well as a good standard-of-practice. While there is some flexibility in the approach to the design process, the correlations provided in this chapter must be used unless written permission is obtained in advance. All requests for changes shall be forwarded to the PCS/GDS for review prior to approval. These correlations were adopted after a review of the geotechnical state of practice within the United States.

7.2 GEOTECHNICAL DESIGN APPROACH

Geotechnical engineering requires the use of science, art, and economics to perform analyses and designs that are suitable for the public use. The science of geotechnical engineering consists of using the appropriate theories to interpret field data, develop geologic profiles, select foundation types, perform analyses, develop designs, plans and specifications, construction monitoring, maintenance, etc.

The art of geotechnical engineering is far more esoteric and relies on the judgment and experience of the engineer. This is accomplished by knowing applicability and limitations of the geotechnical analytical theories and assessing the uncertainties associated with soil properties, design methodologies, and the resulting impact on structural performance. The engineer is required to evaluate the design or analysis and decide if it is "reasonable" and will it meet the performance expectations that have been established. Reasonableness is a subjective term that depends on the engineer's experience, both in design and construction. If the solution does not appear reasonable, the engineer should make the appropriate changes to develop a reasonable solution. In addition, the engineer should document why the first solution was not reasonable and why the second solution is reasonable. This documentation is an important part of the development of the design approach. If the solution appears reasonable, then design should proceed to the economics of geotechnical engineering.

The economics of geotechnical engineering assesses the effectiveness of the solution from a cost perspective. Sometimes geotechnical engineers get caught up in the science and art of geotechnical engineering and do not evaluate other non-geotechnical solutions that may be cost effective both in design and construction. For example, alternate alignments should be explored to avoid poor soils, decreasing vertical alignment to reduce surface loads, placing alternate designs on the plans to facilitate competitive bidding, etc. The science, art, and economics are not sequential facets of geotechnical engineering but are very often intermixed throughout the design process.

7.3 GEOTECHNICAL ENGINEERING QUALITY ASSURANCE

A formal internal geotechnical engineering quality assurance plan should be established for all phases of the geotechnical engineering process. The first-line geotechnical engineer is expected to perform analyses with due diligence and a self-prescribed set of checks and balances. The geotechnical quality control plan should include milestones in the project development where analysis, recommendations, etc. are reviewed by at least one other geotechnical engineer of equal experience or higher seniority. Formal documentation of the quality assurance process should be detectable upon review of geotechnical calculations, reports, etc. All engineering work shall be performed under the direct supervision of a Professional Engineer (P.E.) licensed by the South Carolina State Board of Registration for Professional Engineers and Surveyors in accordance with Chapter 22 of Title 40 of the 1976 Code of Laws of South Carolina, latest amendment.

7.4 DEVELOPMENT OF SUBSURFACE PROFILES

The SCDOT geotechnical design process indicated in Chapter 4, allows for a preliminary and a final geotechnical exploration program for all projects. The primary purpose of the preliminary exploration is to provide a first glance at the project, while the final exploration is to provide all of the necessary geotechnical information to complete the final design.

It is incumbent upon the geotechnical engineer to understand the geology of the project site and determine the potential effects of the geology on the project. The geotechnical engineer should also have knowledge of the regional geology that should be used in the development of the exploration program for the project. In addition to the geologic environment, the geotechnical engineer should be aware of the seismic environment (see Chapter 11 for geology and seismicity and Chapter 12 for site class discussions). The geotechnical engineer is also required to know and understand the impacts of the design earthquake event on the subsurface conditions at the project site (see Chapters 13 and 14 for the impacts and designs, respectively). The geologic formation and local seismicity may have a bearing on the selection of the foundation type and potential capacity. For example, for driven piles bearing in the Cooper Marl formation of the Charleston area, precast, prestressed concrete piles should penetrate the formation approximately 5 feet, with most of the capacity being developed by steel H-pile extensions attached below the prestressed pile, penetrating into the Marl.

The geotechnical engineer should develop a subsurface profile for both the preliminary and final geotechnical subsurface explorations. The subsurface profile developed should take into consideration the site variability as indicated in Section 7.5. The profile should account for all available data and is normally depicted along the longitudinal axis of the structure. However, in some cases, subsurface profiles transverse to the axis of the structure may be required to determine if a formation is varying (i.e. sloping bearing strata) along the transverse axis.

7.5 SITE VARIABILITY

Keeping in mind the geologic framework of the site, the geotechnical engineer should evaluate the site variability (SV). Site variation can be categorized as Low, Medium, or High. If a project site has a “High” site variability (SV), the extent of the “Site” should be subdivided to obtain smaller “Sites” with either Low or Medium variability. The use of a “High” site variability (SV) for geotechnical design shall only be allowed upon consultation with the PCS/GDS. The site variability (SV) determination may be based on judgment; however, justification for the selection of the site variability is required. Conversely, the determination of site variability may be based on the shear strength of the subsurface soils. The shear strength may be based on Standard Penetration Test (SPT), the Cone Penetration Test (CPT), or the results of other field or laboratory testing. Soil property (i.e. shear strength) selection for the determination of resistance factors and SV should be consistent with Chapter 9. If shear strengths are used to determine SV, then the Coefficient of Variation (COV) of the shear strengths shall be determined. The COV shall be used to determine the SV as indicated in Table 7-1.

Table 7-1, Site Variability Defined By Soil Shear Strength COV

Site Variability (SV)	COV
Low	< 25%
Medium	25% ≤ COV < 40%
High	≤ 40%

7.6 PRELIMINARY GEOTECHNICAL SUBSURFACE EXPLORATION

Prior to the commencement of the preliminary exploration, the geotechnical engineer shall visit the site and conduct a GeoScoping. The GeoScoping consists of the observation of the project site to identify areas that may impact the project from the geotechnical perspective. These areas may be selected for exploration during the preliminary exploration if the site is located within the existing SCDOT Right-of-Way (ROW). If the areas of concern are located outside of the existing SCDOT ROW, then the areas should be investigated during the final exploration. For projects conducted by SCDOT, the results of the GeoScoping shall be reported on the appropriate forms (see Appendix A). For consultant projects, the consultant shall use the form developed and approved by the consulting firm. The form shall be included as an appendix to the preliminary geotechnical report. An engineering professional with experience in observing and reviewing sites for potential geotechnical concerns shall be responsible for conducting the GeoScoping.

The preliminary exploration requirements are detailed in Chapter 4, while the contents of the preliminary geotechnical report are detailed in Chapter 21. The primary purpose of the preliminary exploration is to provide a first glance at the project. Typically the preliminary exploration will be short on project details. However, the most important details that will be known are what type of project is it (i.e. bridge replacement, new road, intersection improvement, etc.) and where the project is located. In many cases, the final alignment and structure locations may not be known. The primary purpose of this type of exploration is not to provide final designs, but to determine if there are any issues that could significantly affect the project. These issues should be identified and the potential impacts and consequences of these design issues evaluated. Design issues should be identified and documented for additional exploration during the final geotechnical exploration. If the project is located completely within the SCDOT ROW, then the entire exploration may be performed during the preliminary exploration phase of the project; however, the report prepared shall be a preliminary report that meets the requirements of Chapter 21.

7.7 FINAL GEOTECHNICAL SUBSURFACE EXPLORATION

The final geotechnical exploration shall conform to the requirements detailed in Chapter 4, while the contents of the final geotechnical report shall conform to the requirements detailed in Chapter 21. The final exploration shall be laid out to use the testing locations from the preliminary exploration to the greatest extent possible without compromising the results of the final exploration. The final exploration shall include those areas identified during the preliminary exploration or during the GeoScoping as requiring additional investigation. If these areas impact the performance of the project, these impacts shall be brought to the immediate attention of the Design/Program Manager. In addition, the geotechnical engineer shall also include recommended mitigation methods.

7.8 FIELD DATA CORRECTIONS AND NORMALIZATION

In-situ testing methods such as Standard Penetrometer Test (SPT), electronic Cone Penetrometer Test (CPT), electronic Piezocone Penetrometer Test with pore pressure readings (CPTu), and Flat Plate Dilatometer Test (DMT) may require corrections or adjustments prior using the results for soil property correlation or in design. These in-situ testing methods are described in Chapter 5. The SPT and CPT field data are the most commonly corrected or normalized to account for overburden pressure, energy, rod length, non-standard sampler configuration, borehole diameter, fines content, and the presence of thin very stiff layers. The data obtained from the DMT is corrected for the effects of the instrument operation on the results of the testing. All corrections for in-situ testing methods that are used in geotechnical design and analyses shall be documented in the geotechnical report. The following sections discuss corrections and adjustments in greater detail.

7.8.1 SPT Corrections

Many correlations exist that relate the corrected N-values to relative density (D_r), peak effective angle of internal friction (ϕ'), undrained shear strength (S_u), and other parameters; therefore it is incumbent upon the designer to understand the correlations being used and the requirements of the correlations for corrected N-values. Design methods are available for using N-values directly in the design of driven piles, embankments, spread footings, and drilled shafts. These corrections are especially important in liquefaction potential assessments (Chapter 13 – Geotechnical Seismic Hazards). Design calculations using SPT N-value correlations should be performed using corrected N-values, however, only the actual field SPT N_{meas}-values should be plotted on the soil logs and profiles depicting the results of SPT borings. Each of the corrections is discussed in greater detail in the following sections.

7.8.1.1 Energy Correction (C_E)

The type of hammer used to collect split-spoon samples must be noted on the boring logs. Typically correlations used between soil parameters and N-values are based on a hammer having an energy potential of 60 percent of the theoretical maximum. Typically a split-spoon sampler advanced with a manual safety hammer will have an approximate energy level of 60 percent (ER ≈ 60%). The energy ratio (ER) is the measured energy divided by the theoretical maximum (i.e. 140-pound hammer dropping 30 inches or 4,200 inch-pounds). The measured energy is determined as discussed in Chapter 5.

Split-spoon samples are also advanced with either an automatic hammer (ER ≈ 90%) or a donut hammer (ER ≈ 45%) **[<u>Reminder:</u> *The use of the donut hammer is not permitted*]**. The corrections for the donut hammer are provided for information only because some past projects were performed using the donut hammer. N-values obtained using either the automatic or the donut hammer will require correction prior to being used in engineering analysis. The energy correction factor (C_E) shall be determined using the following equation. Typical C_E values are provided in Table 7-2 for each hammer type. These correction factors should only be used when the actual hammer energy has not been previously measured.

$$\mathbf{C_E = \frac{ER}{60}} \qquad \text{Equation 7-1}$$

Where ER is the measured energy expressed as an integer (i.e. 90 percent energy is ER = 90).

Table 7-2, Energy Ratio by Hammer Type (C_E)

Hammer Type	Energy Ratio (ER) %	C_E
Automatic	80	1.33
Safety	60	1.00
Donut	45	0.75

7.8.1.2 Overburden Correction (C_N)

N_{meas}-values will increase with depth due to increasing overburden pressure. The overburden correction is used to standardize all N-values to a reference overburden pressure. The reference overburden pressure is 1 ton per square foot (tsf) (1 atmosphere). The overburden correction factor (C_N) (Cetin et al., 2004) is provided below.

$$\mathbf{C_N} = \left(\frac{\mathbf{1}}{\sigma'_{\mathbf{V}}} \right)^{\mathbf{0.5}} \leq \mathbf{1.6} \qquad \text{Equation 7-2}$$

7.8.1.3 Rod Length Correction (C_R)

N_{meas}-values measured in the field should be corrected for the length of the rod used to obtain the sample. The original N_{60}-value measurements were obtained using long rods (i.e. rod length greater than 33 feet); therefore, a correction to obtain “equivalent” N_{60}-values for short rod length (i.e. rod length less than 33 feet) is required. Typically the rod length will be the depth of the sample (d) plus an assumed 7 feet of stick up above the ground surface. The rod length correction factor (C_R) equation is provided below with typical values presented in Table 7-3 (McGregor and Duncan, 1998).

$$C_R = e^{-e^{(-0.11d-0.77)}} \qquad \text{Equation 7-3}$$

Table 7-3, Rod Length Correction (C_R)

Rod Length (feet)	C_R
< 13	0.75
13 – 20	0.85
20.1 – 33	0.95
> 33	1.00

7.8.1.4 Sampler Configuration Correction (C_S)

The sampler configuration correction factor (C_S) (Cetin et al., 2004) is used to account for samplers designed to be used with liners, but the liners are omitted during sampling. If the sampler is not designed for liners or if the correct size liner is used no correction is required (i.e. C_S = 1.0). When liners are omitted there is an increase to the inside diameter of the sampler; therefore, the friction between the soil and the sampler is reduced. The sampler configuration correction factor is presented in Table 7-4.

Table 7-4, Sampler Configuration Correction (C_S)

Sampler Configuration	C_S
Standard Sampler not designed for liners	1.0
Standard Sampler design for and used with liners	1.0
Standard Sampler designed for liners and used without liners:	
$N_{meas} \leq 10$	1.1
$11 \leq N_{meas} \leq 29$	$1 + N_{meas}/100$
$30 \leq N_{meas}$	1.3

7.8.1.5 Borehole Diameter Correction (C_B)

The borehole diameter affects the N_{meas}-value if the borehole diameter is greater than 4.5 inches. Large diameter boreholes allow for stress relaxation of the soil materials. This stress relaxation can be significant in sands, but have a negligible effect in cohesive soils. Therefore, for cohesive soils use C_B equal to 1.0. Listed in Table 7-5 are the borehole diameter correction factors (C_B) (McGregor and Duncan, 1998).

Table 7-5, Borehole Diameter Correction (C_B)

Borehole Diameter (inches)	C_B
2-1/2 – 4-1/2	1.0
6	1.05
8	1.15

7.8.1.6 Fines Content Correction (C_F)

The N_{meas}-value may require correction for fines content (FC). This correction is applied during liquefaction analysis (see Chapter 13). It should be noted that a different fines correction is required for determination of seismic soil settlement (Chapter 13). The fines content correction (C_F) (Cetin et al., 2004) is determined by the following equation.

$$\mathbf{C_F} = (\mathbf{1} + \mathbf{0.004FC}) + \mathbf{0.05}\left(\frac{\mathbf{FC}}{\mathbf{N^*_{1,60}}}\right) \qquad \text{Equation 7-4}$$

Where FC is the percent fines content expressed as an integer (i.e. 15 percent fines is FC =15). This fines content correction factor is limited to fines contents between 5 percent and 35 percent (5% ≤ FC ≤ 35%). For fines content less than 5 percent use FC = 0 and for fines content greater than 35 percent use FC = 35. $N^*_{1,60}$ is defined in the following section.

7.8.1.7 Corrected N-values

As indicated previously the N-values measured in the field (N_{meas}) require corrections or adjustments prior to being used for the selection of design parameters or in direct design methods. The N-value requirements of the correlations or the direct design methods should be well understood and known to the engineer. Corrections typically applied to the N_{meas} -values are listed in the following equations.

$$\mathbf{N_{60} = N_{meas} \cdot C_E}$$ Equation 7-5

$$\mathbf{N_{1,60} = N_{60} \cdot C_N}$$ Equation 7-6

$$\mathbf{N^*_{60} = N_{meas} \cdot C_E \cdot C_R \cdot C_S \cdot C_B}$$ Equation 7-7

$$\mathbf{N^*_{1,60} = N^*_{60} \cdot C_N}$$ Equation 7-8

$$\mathbf{N^*_{1,60,CS} = N^*_{1,60} \cdot C_F}$$ Equation 7-9

7.8.2 CPT Corrections

The CPT tip resistance (q_c) and sleeve resistance (f_s) require corrections to account for the effect of overburden on the tip and sleeve resistance. The tip resistance may also be corrected to account for thin stiff layers located between softer soil layers. These corrections are discussed in the following sections.

7.8.2.1 Effective Overburden Normalization

The measured CPT tip resistance (q_c) and sleeve resistance (f_s) are influenced by the effective overburden stress. This effect is accounted for by normalizing the measured resistances to a standard overburden stress of 1 tsf (1 atm). The normalized CPT tip resistance ($q_{c,1}$) and sleeve resistance ($f_{s,1}$), are computed as indicated by the following equations.

$$\mathbf{q_{c,1} = C_q q_c}$$ Equation 7-10

$$\mathbf{f_{s,1} = C_q f_s}$$ Equation 7-11

Where,

q_c = Measured CPT tip resistance. Units of MPa (1 MPa ≅ 10.442 tsf)

f_s = Measured CPT sleeve resistance. Units of MPa (1 MPa ≅ 10.442 tsf)

C_q = Overburden normalization factor is the same for q_c and f_s as indicated in Equation 7-12.

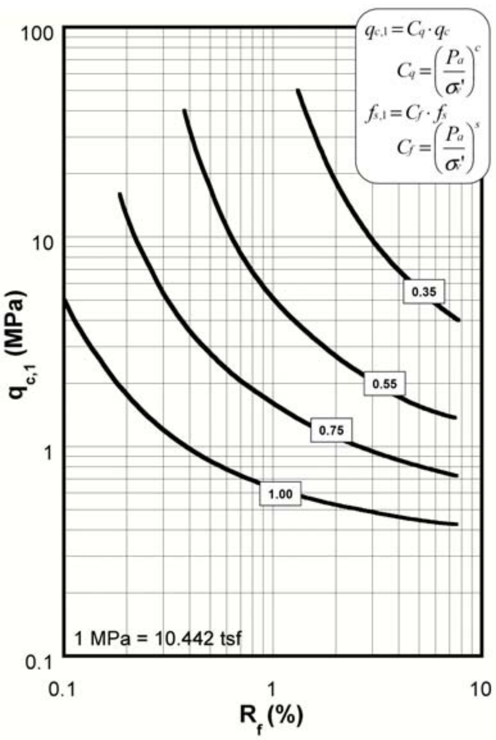
$$\mathbf{C_q = \left(\frac{P_a}{\sigma'_v}\right)^c \leq 1.7}$$ Equation 7-12

Where,

σ'_v = Effective overburden stress in units of tsf at the time that the CPT testing was performed. Future variations in water table or surcharges should not be included in the calculations.

P_a = Atmospheric pressure, taken as 1 tsf (1 atm)

c = Normalization exponent that can be determined from Figure 7-1.



$$c = f_1 \left(\frac{R_f}{f_3} \right)^{f_2}$$ Equation 7-13

q_c = measured CPT tip resistance
R_f = friction ratio (f_s/q_c) in percent
f_s = measured CPT friction resistance

$$f_1 = 0.78(q_c)^{-0.33}$$

$$f_2 = \left(0.32(q_c)^{-0.35}\right) - 0.49$$

$$f_3 = abs\left[\log(10 + q_c)\right]^{1.21}$$

Iterative Calculation Method:

1. Select an initial estimate of the normalization exponent (c) using measured q_c using Figure 7-1(A) or Equation 7-13.
2. Normalize tip using Equation 7-11. (Note: R_f will not change when tip and friction sleeve are normalized equivalently)
3. Compute revised normalization exponent using Equation 7-13.
4. Iterate procedure until acceptable convergence.

(A) CPT Normalization Exponent Curves	**(B) Iterative Calculation Method of Normalization Exponent (*c*)**

Figure 7-1, Normalization of CPT Overburden Exponent (*c*) (Moss et al., 2006)

7.8.2.2 Thin Layer Correction

When the measured CPT tip resistance (q_c) is obtained in a thin layer of stiff soils that is embedded between softer surrounding soils, the measured tip resistance (q_c) will be reduced due to the effects of the underlying softer soils. This case commonly occurs in fluvial environments where granular soils are interbedded between layers of cohesive soils. Granular soils that are affected by this reduction in tip resistance (q_c) are typically sand layers that are less than 5 feet thick. The CPT tip resistance for this special case that is normalized and corrected for the thin layer ($q_{c,1,thin}$) and is computed as indicated in the following equation.

$$\mathbf{q_{c,1,thin} = C_{thin}(q_{c,1})}$$ Equation 7-14

Where,

$q_{c,1}$ = Measured CPT tip resistance. Units of MPa (1 MPa ≅ 10.442 tsf)

C_{thin} = Thin layer correction factor. The C_{thin} is determined from Figure 7-2 (See recommended bold red lines) based on the ratio of uncorrected q_c values for layers B and A (q_{cB}/q_{cA}) and the thickness of the thin layer (*h*). The value for C_{thin} should be limited to $C_{thin} \leq 1.8$ for thin layer thickness, h < 5 feet (1200 mm). A value of C_{thin} = 1.0 should be used for granular soil layers with a thickness, $h \geq 5$ feet (1200 mm). These corrections apply to a 10 cm^2 cone (diameter, d=35.7mm).

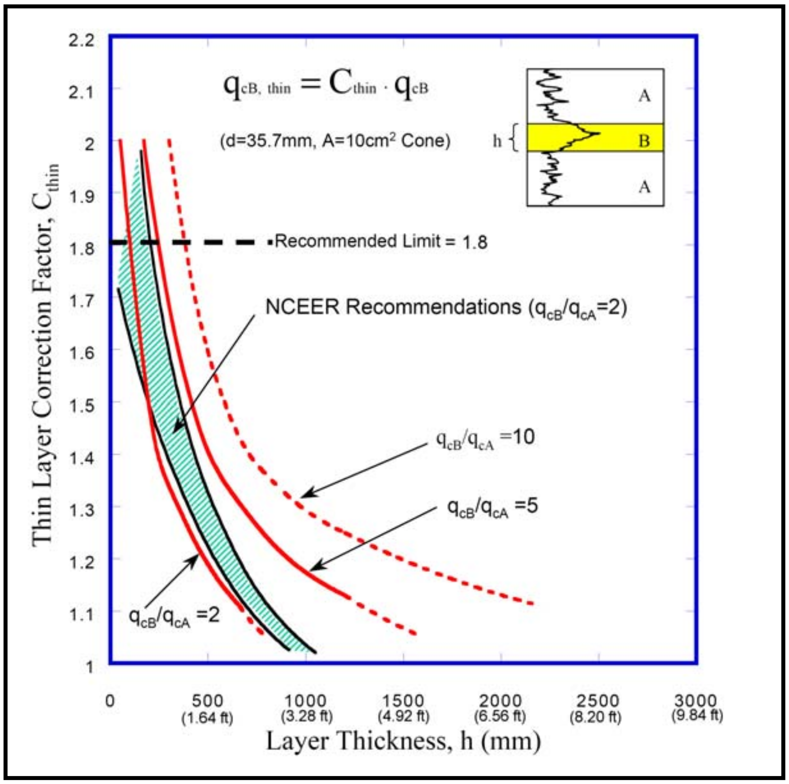


Figure 7-2, CPT Thin Layer Correction (C_{Thin})
(Moss et al., 2006)

In lieu of using Figure 7-2 the following equation may be used to compute the C_{thin}.

$$\mathbf{C_{thin} = A(304.878\,h)^{B} \leq 1.800 \quad for\ \ h < 5\ feet\ and\ \left(\frac{q_{cB}}{q_{cA}}\right) \leq 5} \qquad \text{Equation 7-15}$$

Where,

h = layer thickness in feet

$$A = 3.744\left({q_{cB}}/{q_{cA}}\right)^{0.491}$$

$$B = -0.050\ln\left({q_{cB}}/{q_{cA}}\right) - 0.204$$

$$\left({q_{cB}}/{q_{cA}}\right) = \text{Stiffness Ratio}$$

7.8.2.3 Correlating CPT Tip Resistance To SPT N-Values

Since some design methodologies have only been developed for SPT blow counts, the CPT tip resistance is sometimes correlated to SPT blow counts. It is recommended that the normalized cone tip resistance, $q_{c,1}$, or the normalized cone tip resistance adjusted for the effects of "fines", $q_{c,1,mod}$, be normalized and corrected as indicated in Chapter 13 first and then correlated to normalized SPT values $N_{1,60}$ or $N_{1,60,cs}$. The following correlation by Jefferies and Davies (1993) should be used to correlate the CPT tip resistance to the SPT blow count.

$$\mathbf{N_{1,60}} = \frac{\mathbf{q_{c,1}}}{\mathbf{8.5}\left(\mathbf{1} - \frac{\mathbf{I_c}}{\mathbf{4.75}}\right)}$$ Equation 7-16

$$\mathbf{N_{1,60,cs}} = \frac{\mathbf{q_{c,1,mod}}}{\mathbf{8.5}\left(\mathbf{1} - \frac{\mathbf{I_c}}{\mathbf{4.75}}\right)}$$ Equation 7-17

Where,

$q_{c,1}$ = Normalized CPT cone tip resistance Units of tsf. See Section 7.8.2.1.
$q_{c,1,mod}$ = Normalized CPT cone tip resistance adjusted for "fines" Units of tsf. See Chapter 13.
I_c = Soil behavior type.

The soil behavior type, I_c, is computed using normalized tip resistance (Q_T), normalized sleeve friction (F_R), and normalized pore pressure (B_q). The following equations should be used.

$$\mathbf{Q_T} = \frac{\mathbf{q_{c,1}} - \sigma_{\mathbf{v}}}{\sigma'_{\mathbf{v}}}$$ Equation 7-18

$$\mathbf{F_R} = \frac{\mathbf{f_{s,1}}}{\left(\mathbf{q_{c,1}} - \sigma_{\mathbf{v}}\right)} \times \mathbf{100}$$ Equation 7-19

$$\mathbf{B_q} = \frac{\left(\mathbf{U_2} - \mathbf{U_0}\right)}{\left(\mathbf{q_t} - \sigma_{\mathbf{v}}\right)}$$ Equation 7-20

Where,

$q_{c,1}$ = Where q_c is the normalized CPT cone tip resistance, units of tsf.
$f_{s,1}$ = Where f_s is the normalized CPT cone tip resistance, units of tsf.
σ'_v = Effective overburden pressure, units of tsf
σ_v = Total overburden pressure, units of tsf
U_2 = Pore pressure measurement located on the tip shoulder, unit of tsf
U_0 = Hydrostatic water pressure, units of tsf

The soil behavior type, I_c, is computed using the following equation.

$$I_c = \sqrt{\left[3 - \log\left(Q_T\left(1 - B_q\right)\right)\right]^2 + \left[1.5 + \left(1.3\log\left(F_R\right)\right)\right]^2}$$ Equation 7-21

The soil behavior type, I_c, can be generally correlated to a soil classification as indicated in Table 7-6.

Table 7-6, Soil CPT Index (I_c) and Soil Classification

CPT Index (I_c)	Soil Classification
I_c <1.25	Gravelly Sands
$1.25 \le I_c < 1.90$	Sands – Clean Sand to Silty Sand
$1.90 \le I_c < 2.54$	Sand Mixtures – Silty Sand to Sandy Silt
$2.54 \le I_c < 2.82$	Silt Mixture – Clayey Silt to Silty Clay
$2.82 \le I_c < 3.22$	Clays

7.8.3 Dilatometer Corrections

The data A, B, and C pressure readings from the dilatometer require correction to account for the effects of the physical composition of the instrument (i.e. the stiffness of the membrane, new membranes are stiffer than used membranes). The horizontal stress index (K_D) shall be reported for all DMT results. The DMT corrections and computations for the horizontal stress index (K_D) shall be computed in accordance with FHWA-SA-91-044, *The Flat Dilatometer Test*, publication dated February 1992.

7.9 SOIL LOADING CONDITIONS AND SOIL SHEAR STRENGTH SELECTION

Geotechnical engineering as presented in this Manual has a statistical (LRFD) and performance-base design components that require selection of appropriate soil properties in order to design within an appropriate margin of safety consistent with Chapter 9 and also to predict as reasonable as possible the geotechnical performance required in Chapter 10. The selection of soil shear strengths by the geotechnical engineer requires that the designer have a good understanding of the loading conditions and soil behavior, high quality soil sampling and testing, and local geotechnical experience with the various geologic formations. This section provides guidance in the selection of shear strengths for cohesive soils (i.e. clays) and cohesionless soils (i.e. sands and nonplastic silts) for use in geotechnical design. The selection of shear strength parameters for rock is covered in the Section 7.14.

For an in-depth review of the topics addressed in this Section, see Sabatini et al. (2002) and Duncan and Wright (2005).

Geotechnical load resisting analyses that are typically performed in the design of transportation facilities are bearing capacity of a shallow foundation, axial (tension and compression) load carrying capacity of deep foundations (drilled shafts and piles), lateral carrying capacity of deep foundations, stability analyses of hillside slopes and constructed embankments, sliding resistance of earth retaining structures, and passive soil capacity resistance. Each of these analyses can have various loading conditions that are associated with the limit state (Strength, Service, Extreme Event) under evaluation.

Soil shear strength is not a unique property and must be determined based on the anticipated soil response for the loading condition being evaluated. This requires the following three-step evaluation process:

- **<u>Evaluate the Soil Loading</u>**: The soil loading should be investigated based on the soil loading rate, the direction of loading, and the boundary conditions for the limit state (Strength, Service, Extreme Event) being evaluated.

- **<u>Evaluate Soil Response</u>**: The soil response should be evaluated based on pore pressure build-up (Δu), the soil's state of stress, volumetric soil changes during shearing, and the anticipated magnitude of soil deformation or strain for the soil loading being applied.

- **<u>Evaluate Appropriate Soil Strength Determination Method:</u>** This consists of determining the most appropriate soil testing method that best models the loading condition and the soil response for determination of soil shear strength design parameters. Also included in this step is the review of the results for reasonableness based on available correlations and regional experience.

The three-step evaluation process is discussed in detail in the following Sections.

7.9.1 <u>Soil Loading</u>

The soil loading can be evaluated with respect to loading rate, direction of loading, and boundary conditions. The loading rate primarily affects the soils response with respect to pore water pressure build-up (Δu). When the loading rate either increases or decreases the pore water pressure ($\Delta u \neq 0$), the loading is referred to as short-term loading. Conversely, if the loading rate does not affect the pore water pressure ($\Delta u = 0$), the loading is referred to as a long-term loading.

Short-term loadings typically occur during construction such as when earth-moving equipment place large soil loads within a relatively short amount of time. The actual construction equipment (cranes, dump trucks, compaction equipment, etc.) should also be considered during the evaluation the construction loadings. Construction loadings are typically evaluated under the Strength limit state. Earthquakes or impacts (vessel or vehicle collisions) that can apply a significant amount of loading on the soil within a short amount of time are also referred to as short-term loadings. Because of the relative transient and infrequent nature of earthquake and impact loadings, geotechnical design for these types of loadings are performed under the Extreme Event limit states.

Long-term loadings are typically the result of static driving loads placed on the soils when performing limit state equilibrium analyses such as those that occur with embankments, retaining walls, or foundation that have been in place for a sufficient length of time that the pore water pressures have dissipated. These types of loadings are typically evaluated under the Strength and Service limit states.

The direction of loading is directly related to the critical failure surface and it's angle of incidence with respect to the soil element under evaluation. This becomes important when analyzing the soil shear strength with respect to a base of a retaining wall sliding over the foundation or during the analysis of soil stability where the failure surface intersects the soil at various angles within the soil mass. The shear strength is also affected by plane strain loading condition as is typically observed under structures such as continuous wall footings. Plane strain loading occurs when the strain in the direction of intermediate principal stress is zero.

Soil loading boundary conditions result from the soil-structure interaction between the loads imposed by the structure and the soil. The loadings and soil response are interdependent based on the stress-strain characteristics of the structure and the soil. Boundary conditions also include the frictional interface response between the structure and the soil. These boundary conditions can be very complex and affect the magnitude of the soil loadings, magnitude of the soil resistance, the distribution of the soil loading (rigid or flexible foundation), and the direction of the loading.

7.9.2 Soil Response

The soil response is influenced significantly by the soils pore water pressure response (Δu) resulting from the rate of loading as the soils attempt to reach a state of equilibrium. The undrained condition is a soil response that occurs when there is either an increase (+) in pore water pressure ($\Delta u > 0$) or a decrease (-) in pore water pressure ($\Delta u < 0$) within the soil during soil loading. The drained condition is a soil response that occurs when there is no change in pore water pressure ($\Delta u = 0$) as a result of the soil loading.

The pore water pressure response (Δu) that allows water to move in or out of the soil over time is dependent on the soil drainage characteristics and the drainage path. The time for drainage to occur can be estimated by using Terzaghi's theory of one-dimensional consolidation where the time required to reach 99% of the equilibrium volume change, t_{99}, is determined by the following equation.

$$\boldsymbol{t}_{99} = 4\frac{\boldsymbol{D}^2}{\boldsymbol{C_v}}$$

Equation 7-22

Where,

D	=	Longest distance that water must travel to flow out of the soil mass
C_v	=	Coefficient of vertical consolidation (units length squared per unit of time)

Typical drainage times for various types of soil deposits based on Equation 7-22 are provided in Figure 7-3. It can readily be seen that cohesionless soils (sands) drain within minutes to hours while cohesive soils (clays) drain within months to years. Silty soils can drain within hours to days. Even though a soil formation may behave in an undrained condition at the beginning of the load application with excess pore water pressures ($\Delta u \neq 0$), with sufficient time to allow for pore pressure dissipation, the soils will reach a drained condition where static loads are in equilibrium and there is no excess pore water pressure ($\Delta u = 0$). Because soil layers may have different drainage characteristics and drainage paths within a soil profile, soil layers may be at various stages of drainage with some soil layers responding in an undrained condition while other layers respond in a drained condition.

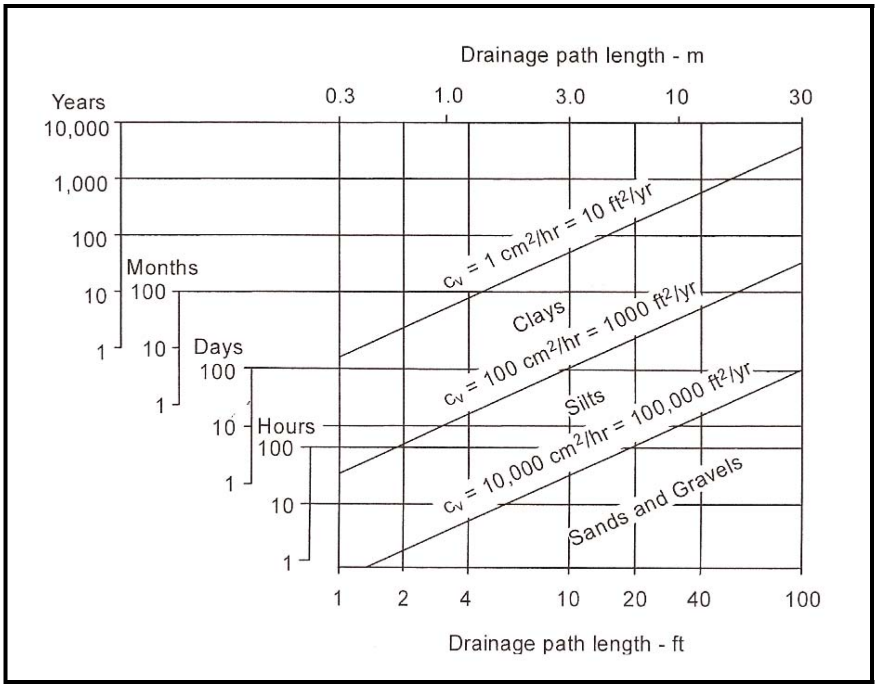


Figure 7-3, Drainage Time Required (Duncan and Wright, 2005)

There are various soil models that are used to characterize soil shear strength. The simplest and most commonly used soil shear strength model is the Mohr-Coulomb soil failure criteria. More sophisticated soil shear strength models such as critical state soil mechanics and numerical models (finite element constitutive soil models) exist and are to be used when simpler models such as the Mohr-Coulomb soil failure criteria cannot accurately predict the soil response.

When undrained conditions exist ($\Delta u \neq 0$), total stress parameters are used to evaluate soil shear strength. Total stress is characterized by using total shear strength parameters (c, ϕ) and total stress, σ_{vo}, (total unit weights). The basic Mohr-Coulomb soil failure criteria for total stress shear strength (τ), also referred to as the undrained shear strength (S_u), is shown in the following equation.

$$\tau = \mathbf{c} + \sigma_{\mathbf{vo}} \tan \phi \qquad \text{Equation 7-23}$$

Where,

- c = Total soil cohesion.
- σ_{vo} = Total vertical overburden pressure. Total unit weights (γ_T) are used.
- ϕ = Total internal soil friction angle. The total internal soil friction angle for cohesive soils is typically assumed to equal zero (ϕ = 0). Total internal soil friction angle (ϕ) for a cohesionless soil is typically less than the effective internal soil friction angle (ϕ').

When drained conditions exist ($\Delta u = 0$), effective stress parameters are used to evaluate soil shear strength. Effective stress is characterized by using effective shear strength parameters (c', ϕ') and effective stress, σ'_{vo}, (effective unit weights). The basic Mohr-Coulomb soil failure criteria for effective stress shear strength (τ') is shown in the following equation.

$$\tau' = \boldsymbol{c}' + \sigma'_{vo} \tan\phi' \qquad \text{Equation 7-24}$$

Where,

c' = Effective soil cohesion. The effective cohesion for cohesive and cohesionless soils is typically assumed to equal zero ($c' = 0$).

σ'_{vo} = Effective vertical overburden pressure. Effective unit weights ($\gamma' = \gamma_T - \gamma_w$) are used.

ϕ' = Effective internal soil friction angle. The effective internal soil friction angle (ϕ') for a cohesionless soil is typically greater than the total internal soil friction angle (ϕ).

Another factor that affects soil response of cohesive soils is the in-situ stress state. The stress state is defined by either total (σ_{vo}) or effective (σ'_{vo}) vertical stress, total (σ_{ho}) or effective (σ'_{ho}) horizontal stress, and the effective preconsolidation stress (σ'_p or p'_c). The effective preconsolidation stress is the largest state of stress that the soil has experienced. The state of stress is often quantified by the overconsolidation ratio (*OCR*) as indicated by the following equation.

$$\mathbf{OCR} = \frac{\boldsymbol{\sigma}'_{\mathbf{p}}}{\boldsymbol{\sigma}'_{\mathbf{vo}}} \qquad \text{Equation 7-25}$$

Cohesive soils are often defined by the following in-situ state of stress:

- **Normally Consolidated (NC; *OCR* = 1):** If the effective overburden stress (σ'_{vo}) is approximately equal to the effective preconsolidation stress (σ'_p).

- **Overconsolidated (OC; *OCR* > 1):** If the effective overburden stress (σ'_{vo}) is less than the effective preconsolidation stress (σ'_p)

- **Underconsolidated (UC; *OCR* < 1):** If the effective overburden stress (σ'_{vo}) is greater than the effective preconsolidation stress (σ'_p)

Volumetric change (δ_v) during shearing can significantly affect the shear strength behavior of the soils. When the soil response is a decrease ($-\delta_v$) in volume during soil shearing the soils are termed to have ***contractive*** behavior. Loose sands and soft clays typically have contractive behavior. When the soil response is an increase ($+\delta_v$) in volume during soil shearing these soils are termed to have ***dilative*** behavior. Overconsolidated clays and medium-dense sands typically have dilative behavior. Soils that do not exhibit volumetric change during shearing (δ_v = 0) are termed to have ***steady state*** behavior.

For typical cohesive or cohesionless soils it has been observed that the soil shear stress (τ) varies as the soil strains or deforms during soil shearing. Selection of the appropriate soil shear strength to be used in design must be compatible with the deformation or strain that the soil will exhibit under the loading. This is best illustrated in Figure 7-4 where the drained stress-strain behavior of two stress-strain curves, each curve representing a different effective consolidation stress (σ'_{v1} and σ'_{v2}), are shown. On the left of Figure 7-4 is a shear stress vs. shear strain plot (τ-γ_s plot). Because there is a well-defined peak shear stress (τ_{max}) in the plots this would be indicative of dilative soil behavior of either dense sand or overconsolidated clay. The maximum shear stress (τ_{max}) is termed the ***peak shear strength*** ($\tau_{Peak} = \tau_{max}$). In overconsolidated clay soils, as the maximum shear stress (τ_{max}) is exceeded, post-peak strain softening occurs until a ***fully-softened strength*** (τ_{NC}) is reached. The fully-softened strength is a post-peak strain softening strength that is considered to be the shear strength that is equivalent to peak shear strength of the same soil in normally consolidated (NC) stress state ($\tau_{Peak} \approx \tau_{NC}$). For very large shearing strains in soils (cohesive or cohesionless) the shear stress value is reduced further to a residual shear strength (τ_r). The Mohr-Coulomb effective shear strength envelopes for peak shear strength ($\tau_{Peak} = \tau_{max}$), fully-softened shear strength ($\tau_{Peak} \approx \tau_{NC}$), and residual shear strength (τ_r) are illustrated on the right side of Figure 7-4.

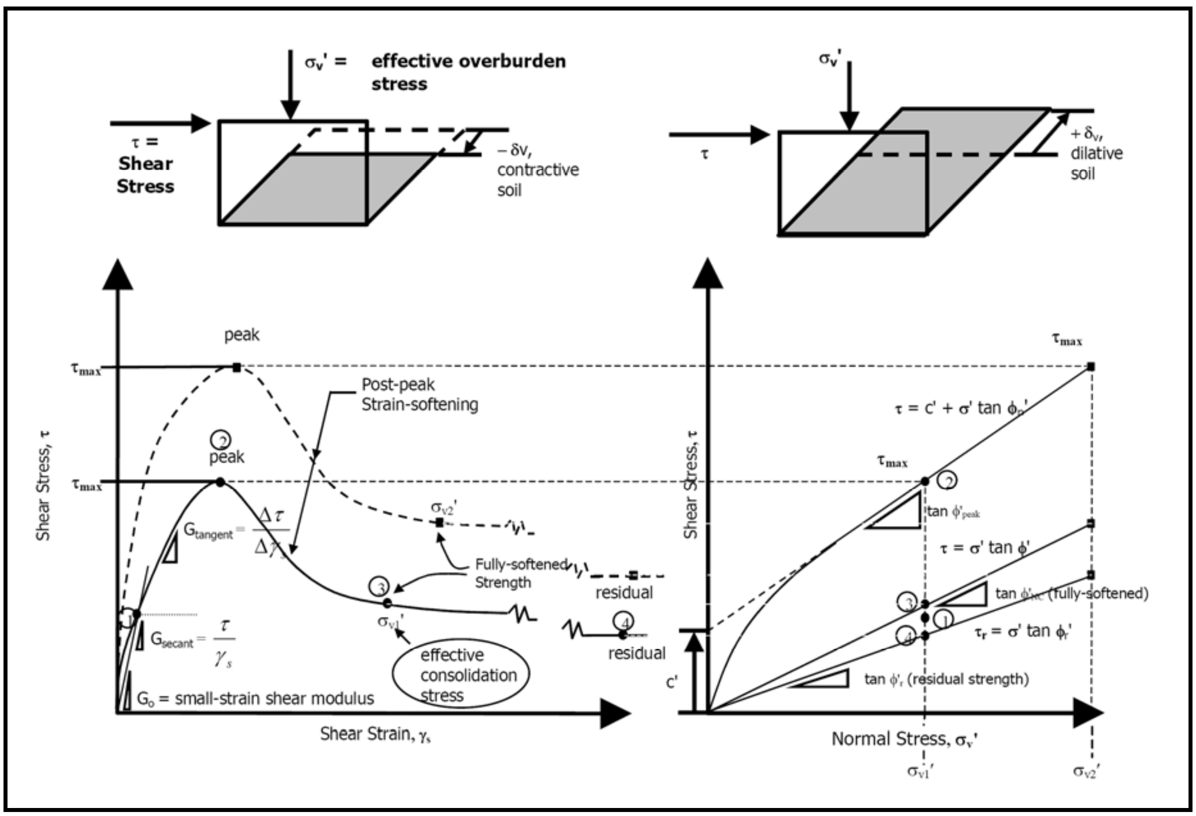


Figure 7-4, Drained Stress-Strain Behavior (Sabatini et al., 2002)

The soil behavior of typical cohesionless soils can be further illustrated by comparing the stress-strain behavior of granular soils with various densities as shown in Figure 7-5. Medium and dense sands typically reach a peak shear strength ($\tau_{Peak} = \tau_{max}$) value and then decrease to a residual shear strength value at large displacements. The volume of medium and dense sands initially decreases (contractive behavior) and then increases as the soil grains dilate (dilative behavior) with shear displacement until it reaches a point of almost constant volume (steady state behavior). The shear stress in loose sands increases with shear displacement to a maximum value and then remains constant. The volume of loose sands gradually decreases (contractive behavior) until it reaches a point of almost constant volume (steady state behavior).

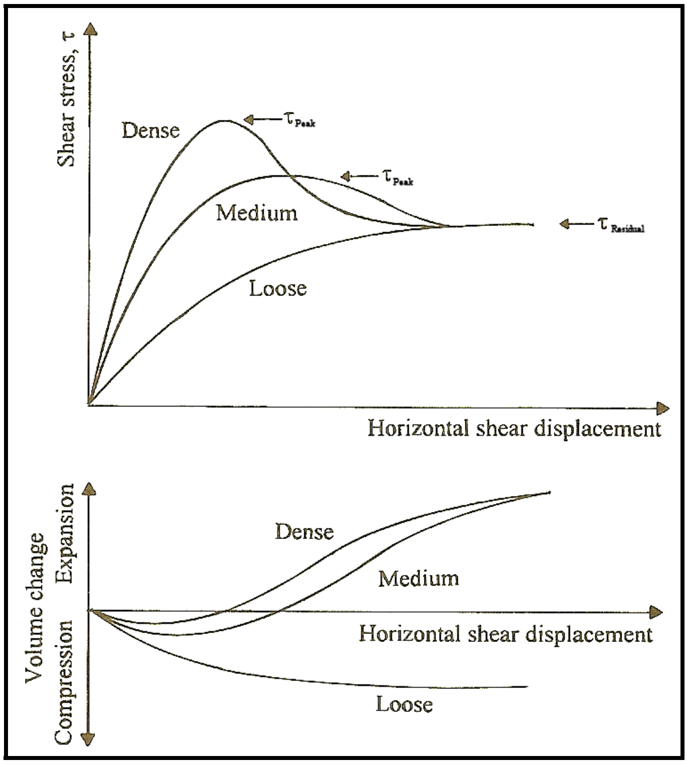


Figure 7-5, Shear Strength Sands (Direct Shear-Test) (Das, 1997)

The soil behavior of typical cohesive soils can be further illustrated by comparing the stress-strain behavior of normally consolidated clays (*OCR* = 1) with the stress-strain behavior of overconsolidated clays (*OCR* > 1) for consolidated drained and undrained Triaxial tests in Figures, 7-6 and 7-7, respectively. The stress-strain behavior for overconsolidated clays (*OCR* > 1) indicates that they are subject to strain softening, similar to medium-dense sands shown in Figure 7-5, and that normally consolidated clays (*OCR* = 1) increases in strength, similar to loose sands also shown in Figure 7-5. Overconsolidated (drained or undrained) clays typically reach peak shear strength ($\tau_{Peak} = \tau_{max}$) and then decrease to a fully-softened strength that is approximately equal to the peak shear strength of a normally consolidated clay ($\tau_{Peak} \approx \tau_{NC}$). The volume change of overconsolidated clays in a drained test is very similar to the volume change in medium-dense sand; the volume initially decreases (contractive behavior) and then increases (dilative behavior). The pore pressures in an undrained test of overconsolidated clays initially increase slightly and then become negative as the soil begins to expand or dilate. The shear stress (drained or undrained test) of a normally consolidated (OCR = 1) clay increases with shear displacement to a maximum value ($\tau_{Peak} = \tau_{NC}$). The volume of normally consolidated clays in a drained test gradually decreases (contractive behavior) as it reaches a point of almost constant volume (steady state behavior). The pore pressure in an undrained test of normally consolidated clay increases until failure and remains positive for the entire test.

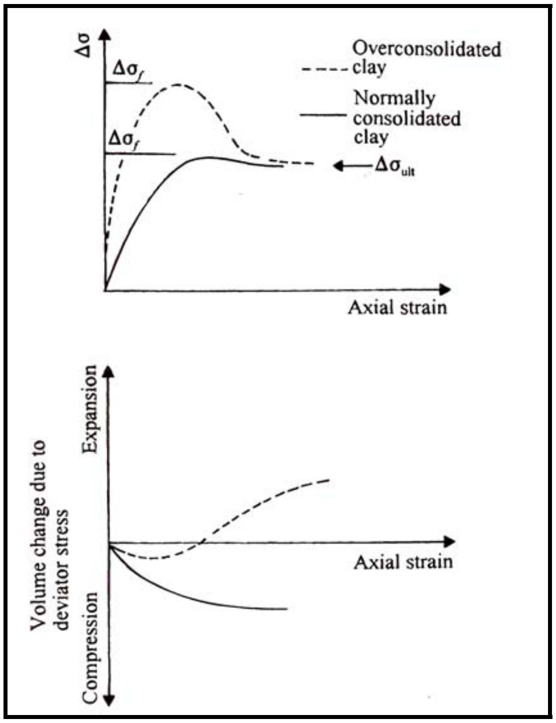


Figure 7-6, Shear Strength of Clay Consolidated Drained Triaxial
(Das, 1997)

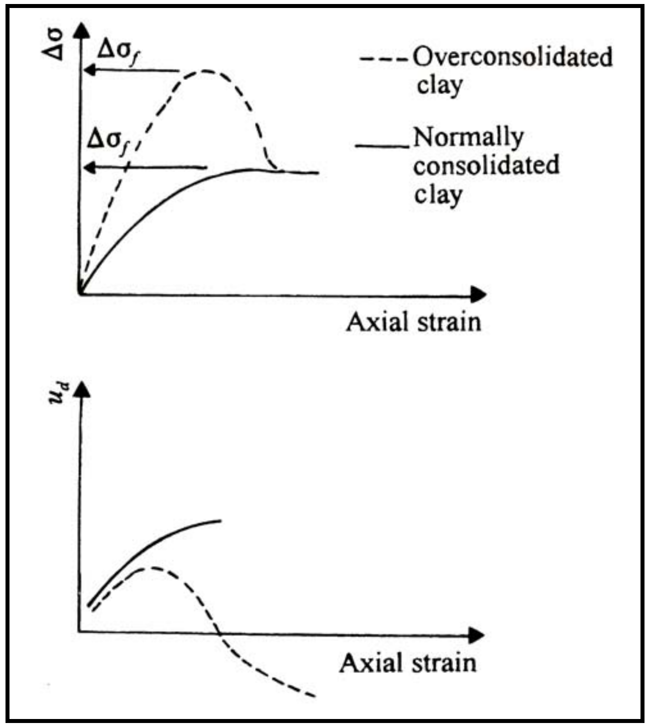


Figure 7-7, Shear Strength of Clay Consolidated Undrained Triaxial
(Das, 1997)

Selection of soil shear strengths should be made based on laboratory testing and soil strain level anticipated from analyses. Table 7-7 provides a summary of published stress-strain behavior from Holtz and Kovacs (1981), Terzaghi, Peck, and Mesri (1996), and Duncan and Wright (2005) for various soils types. This table is provided for "general" guidance in the selection of shear strengths and soil strain level anticipated from equilibrium analyses.

Table 7-7, Soil Shear Strength Selection Based on Strain Level

Cohesive Soils (Undrained)	**Strain Level [1]**		
	±2% Strains	**10–15% Strains**	**Large Strains >15%**
Clay (*OCR*=1)	$\tau_{Peak} = \tau_{NC}$	$\tau_{Peak} = \tau_{NC}$	$\tau_{Peak} = \tau_{NC}$
Clay (*OCR*>1)	τ_{Peak}	$\approx \tau_{NC}$	τ_r
Cohesionless Soils (Drained)	**Strain Level [1]**		
	±5% Strains	**15–20% Strains**	**Large Strains >20%**
Med. To Dense Sand	τ_{Peak}	τ_r	τ_r
Non-Liquefying Loose Sands	τ_{Peak}	τ_{Peak}	τ_r

Shear Strength Nomenclature:

τ_{Peak} = Peak Soil Shear Strength τ_{NC} = Normally Consolidated Soil Shear Strength

τ_r = Residual Soil Shear Strength

[1] Strain levels indicated are generalizations and are dependent on the stress-strain characteristics of the soil and should be verified by laboratory testing.

7.9.3 Soil Strength Testing

Once the soil loading and soil response has been evaluated the next step is to select the method of evaluating the soil shear strength. The shear strength can be evaluated by one of the following methods:

- Soil shear strength determined by geotechnical laboratory testing
- Soil shear strength correlations with in-situ field testing results
- Soil shear strength correlations based on index parameters

The laboratory testing should be selected based on shear strength testing method and the testing parameters best suited to model the loading condition and the soil response. Shear strength laboratory testing methods are described in Chapter 5. A summary of the design parameters that should be used in selection of the appropriate testing method and procedure is provided below:

- **Total or Effective Stress:** Selection of soil shear strength parameters based on total or effective stress state (drained or undrained). Guidance for typical geotechnical analyses for each limit state (Strength, Service, and Extreme Event) being analyzed is provided for bridge foundations in Table 7-8 and for earth retaining structures and embankments in Table 7-9. Total and effective shear strength determination guidelines for laboratory and in-situ testing are provided in Sections 7.10 and 7.11, respectively.

- **Soils Shear Strength:** Soil shear strength parameters (τ_{Peak} or τ_r) selection should be based on strain level anticipated from equilibrium analyses. See Table 7-7 for guidance. Seismic soil shear strengths used to design for the Extreme Event I limit state are discussed in Chapter 12.

- **Loading Direction:** The shearing direction should be compatible with how the soil is being loaded or unloaded and the angle of incidence with respect to soil normal stress. Figure 7-8 illustrates test methods that would be appropriate for shear modes for embankment instability shear surface. Figure 7-9 provides undrained strength (UU Triaxial) of typical clays and shales as a function of stress orientation.

Table 7-8, Bridge Foundation Soil Parameters

Limit State		Strength		Service	Extreme Event			
Load Combinations		Strength I, II, III, IV, V		Service I	Extreme Event I			
Seismic Event		N/A			FEE & SEE			
Loading Condition		Static			During Earthquake Shaking		Post-Earthquake	
Soil Shear Strength Stress State		Total	Effective	Effective	Total [1]	Drained	Total [1]	Drained
Shallow Foundation Design	Soil Bearing Resistance	√	√	---	√	√	√	---
	Sliding Frictional Resistance	√	√	---	√	√	√	---
	Sliding Passive Resistance	√	√	---	√	√	√	---
	Structural Capacity	√	√	---	√	√	√	---
	Lateral Displacement	---	---	√	√	√	√	---
	Vertical Settlement	---	---	∇	∇	∇	∇	∇
	Overall Stability	---	---	√	√	√	√	---
Deep Foundation Design	Axial Capacity	√	•	---	---	√	√	---
	Structural Capacity	√	√	---	---	√	√	---
	Lateral Displacements	---	---	√	√	√	√	---
	Vertical Settlement	---	---	∇	∇	∇	∇	∇

(1) Residual soil shear strengths of liquefied soils must include effects of strain softening due to liquefaction.

<u>Soil Stress State Legend:</u>

√ Indicates that soil stress state indicated requires analysis

--- Indicates that soil stress state does not require analysis

• Indicates that soil stress state may need to be evaluated depending on method of analysis

∇ Indicates that soil stress state transitions from undrained to drained (i.e. consolidation)

Table 7-9, Earth Retaining Structures & Embankment Soil Parameters

Limit State		Strength		Service		Extreme Event			
Load Combinations		Strength I, II, III, IV, V		Service I		Extreme Event I			
Seismic Event		N/A				FEE & SEE			
Loading Condition		Static				During Earthquake Shaking		Post-Earthquake	
Soil Shear Strength Stress State		Total	Effective	Total	Effective	Total [1]	Effective	Total [1]	Effective
Earth Retaining Structure Design	Soil Bearing Resistance	√	√	---	---	√	√	---	√
	Sliding Frictional Resistance	√	√	---	---	√	√	---	√
	Sliding Passive Resistance	√	√	---	---	√	√	---	√
	Structural Capacity	√	√	---	---	√	√	---	√
	Lateral Load Analysis (Lateral Displacements)	---	---	√	√	√	√	---	√
	Settlement	---	---	∇	∇	∇	∇	∇	∇
	Global Stability	---	---	√	√	√	√	---	√
Embankment Design	Soil Bearing Resistance	√	√	---	---	√	√	---	√
	Lateral Spread	√	√	---	---	√	√	---	√
	Lateral Squeeze	√	√	---	---	√	√	---	√
	Lateral Displacements	---	---	√	√	√	√	---	√
	Vertical Settlement	---	---	∇	∇	∇	∇	∇	∇
	Global Stability	---	---	√	√	√	√	---	√

[1] Residual soil shear strengths of liquefied soils must include effects of strain softening due to liquefaction

Soil Stress State Legend:

√ Indicates that soil stress state indicated requires analysis

--- Indicates that soil stress state does not require analysis

• Indicates that soil stress state may need to be evaluated depending on method of analysis

∇ Indicates that soil stress state transitions from undrained to drained (i.e. consolidation)

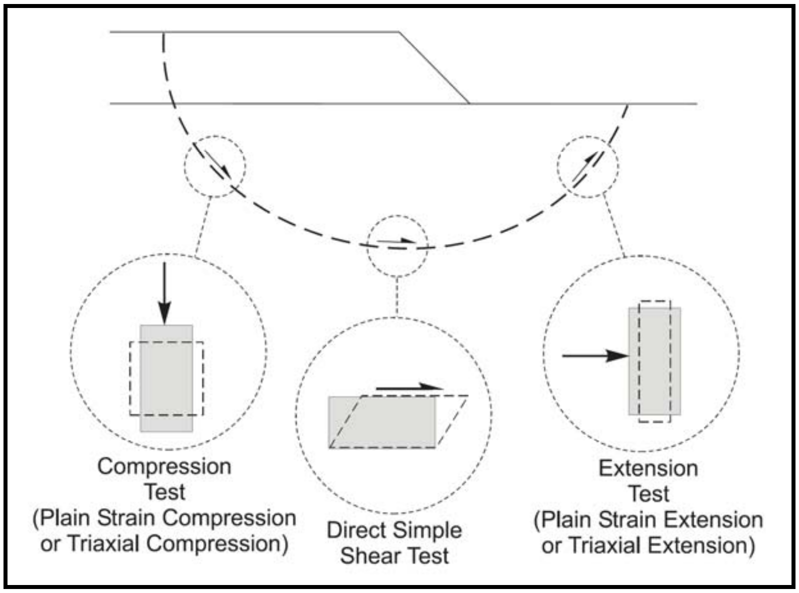


Figure 7-8, Shear Modes for Embankment Stability Shear Failure Surface (Sabatini, 2005)

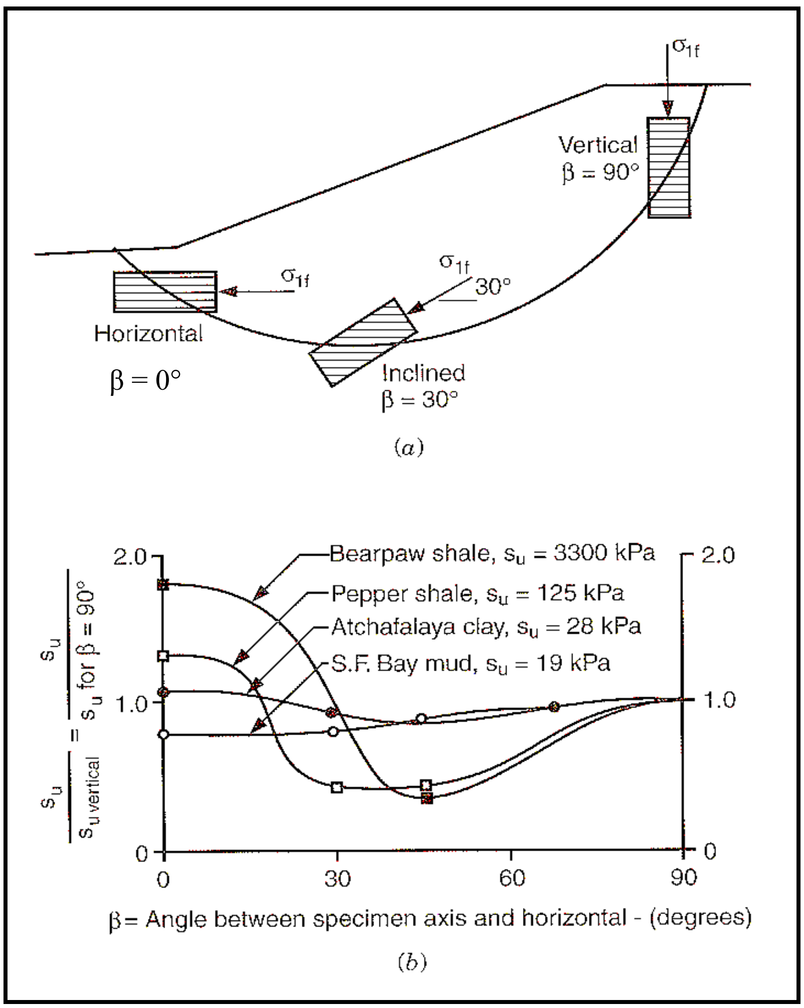


Figure 7-9, τ of Clays and Shales as Function of Failure Orientation (modified from Duncan and Wright, 2005)

The undrained and drained shear strengths of soils can be obtained from laboratory testing. The laboratory testing procedures are described in Chapter 5. A summary of laboratory testing methods suitable for determining the undrained and drained shear strengths of cohesive and cohesionless soils is provided in Table 7-10.

Table 7-10, Laboratory Testing Soil Shear Strength Determination

Laboratory Testing Method	Undrained Shear Strength				Drained Shear Strength			
	Cohesive		Cohesionless		Cohesive		Cohesionless	
	τ_{Peak}	τ_r	τ_{Peak}	τ_r	τ'_{Peak}	τ'_r	τ'_{Peak}	τ'_r
Unconfined Compression (UC) Test	√	√	---	---	---	---	---	---
Unconsolidated Undrained (UU) Test	√	√	---	---	---	---	---	---
Consolidated Drained (CD) Test	---	---	---	---	---	---	√	√
Consolidated Undrained (CU) Test with Pore Pressure Measurements	√	√	√	√	√	√	√	√
Direct Shear (DS) Test	---	---	---	---	---	---	√	√

√ - Indicates laboratory method provides indicated shear strength
--- - N/A
Definitions:
τ_{Peak} = Peak Undrained Shear Strength
τ_r = Residual Undrained Shear Strength
τ'_{Peak} = Peak Drained Shear Strength
τ'_r = Residual Drained Shear Strength

In-situ testing methods (Section 5.3) such as Standard Penetrometer Test (SPT), electronic Cone Penetrometer Test (CPT), electronic Piezocone Penetrometer Test (CPTu – CPT with pore pressure readings), Flat Plate Dilatometer Test (DMT), and Vane Shear Test (VST), can be used to evaluate soil shear strength parameters by the use of empirical/semi-empirical correlations. Even though the torvane (TV) or the pocket penetrometer (PP) are soil field testing methods, their use is restricted to only qualitative evaluation of relative shear strength during field visual classification of soil stratification. The major drawback to the use of in-situ field testing methods to obtain soil shear strength parameters is that the empirical/semi-empirical correlations are based on a limited soil database that is typically material or soil formation specific and therefore the reliability of these correlations must be verified for each project site until sufficient substantiated regional experience is available. Poor correlation between in-situ testing results and soil shear strength parameters may also be due to the poor repeatability of the in-situ testing methods. The electronic Cone Penetrometer Test (CPT) has been shown to be more repeatable while the Standard Penetration Test (SPT) has been shown to be highly variable. Another source of variability is the sensitivity of the test method to different soil types with different soil consistency (very soft to hard cohesive soils) or density (very loose to very dense cohesionless soils). In-situ penetration testing values correspond to the peak of the stress-strain shear strength curve as indicated in Figure 7-10. Since deformations induced from penetration tests are close to the initial stress state, correlations have been developed for the soil modulus.

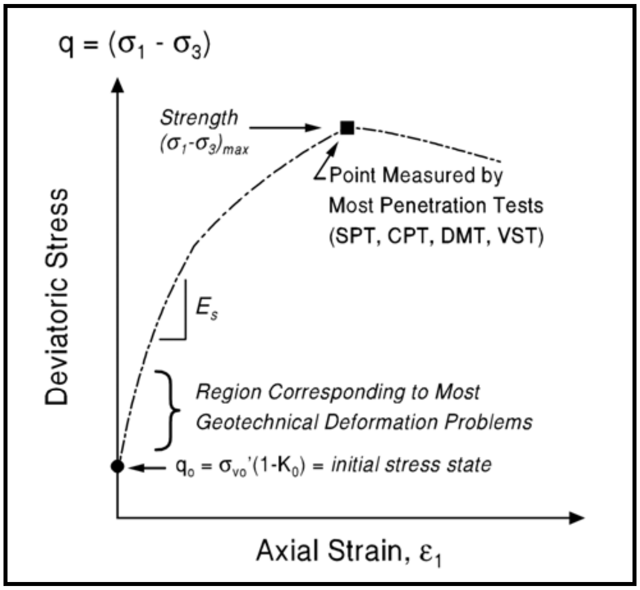


Figure 7-10, Shear Strength Measured by In-Situ Testing (Sabatini, 2005)

A summary of in-situ testing methods suitable for determining the undrained and drained shear strengths of cohesive and cohesionless soils is provided in Table 7-11. The suitability of in-situ testing methods to provide soil shear strength parameters is provided in Table 7-12.

Table 7-11, In-Situ Testing - Soil Shear Strength Determination

In-Situ Testing Method	Undrained Shear Strength				Drained Shear Strength			
	Cohesive		Cohesionless		Cohesive		Cohesionless	
	τ_{Peak}	τ_r	τ_{Peak}	τ_r	τ'_{Peak}	τ'_r	τ'_{Peak}	τ'_r
Standard Penetrometer Test (SPT)	√	---	---	---	---	---	√	---
Cone Penetrometer Test (CPT) or Piezocone with pore pressure measurements (CPTu)	√	√	---	---	---	---	√	---
Flat Plate Dilatometer Test (DMT)	√	---	---	---	---	---	√	---
Vane Shear Test (VST)	√	√	---	---	---	---	---	---

√ - Indicates in-situ method provides indicated shear strength
--- - N/A

Definitions:

τ_{Peak} = Peak Undrained Shear Strength
τ_r = Residual Undrained Shear Strength
τ'_{Peak} = Peak Drained Shear Strength
τ'_r = Residual Drained Shear Strength

Table 7-12, Soil Suitability of In-Situ Testing Methods (Modified from Canadian Geotechnical Manual (1982) and Holtz and Kovacs (1981))

In-Situ Test Method	Suitable Soils [(1)]	Unsuitable Soils	Correlated Properties	Remarks
Standard Penetrometer Test (SPT)	Sand, Clay, Residual Soils	Gravel	Sand and residual soil effective peak internal friction angle, clay undrained peak shear strength, soil modulus.	SPT repeatability is highly variable. Disturbed samples. Very variable S_u correlations are available for clays.
Cone Penetrometer Test (CPT) or Piezocone with pore pressure measurements (CPTu)	Sand, Silt, Clay, Residual Soil	Gravel	Sand, silt, and residual soil effective peak internal friction angle, clay and residual soil undrained peak shear strength, soil modulus.	Continuous evaluation of soil properties. CPT is very repeatable. No samples recovered.
Flat Plate Dilatometer Test (DMT)	Sand, Clay, and Residual Soil	Gravel	Sand, silt, and residual soil effective peak internal friction angle, clay and undrained peak shear strength, overconsolidation ratio, at-rest pressure coefficient, soil modulus.	Unreliable results may occur with very dense sand, cemented sand, and gravel. No samples recovered.
Vane Shear Test (VST)	Clay	Sand, Residual Soil, and Gravel	Clay undrained peak shear strength.	May overestimate shear strength. Very soft clays need to be corrected. Unreliable results may occur with fissured clays, varved clays, highly plastic clays, sand, residual soil, and gravel. VST repeatability may be variable with rate of rotation. No samples recovered.

[(1)] The suitability of testing Piedmont residual soils should be based on Mayne et al. (2000). Residual soils frequently have a dual USCS description of SM-ML and behave as both cohesive soils and cohesionless soils because the Piedmont residuum soil is close to the opening size of the U.S. No. 200 Sieve (0.075 mm).

Shear strength of cohesive and cohesionless soils can also be estimated based on effective overburden stress (σ'_{vo}), effective preconsolidation stress (σ'_p or p'_c), the overconsolidation ratio (OCR), and index properties such as grain size distribution (Fines Content – FC), moisture content (*w*), and Atterberg Limits (LL, PI). Index properties are described in Chapter 5. Unless indicated otherwise, these correlations are used only for preliminary analyses or for evaluating accuracy of laboratory or in-situ shear strength results.

7.10 TOTAL STRESS

Total stress is the force per unit area carried by both the soil grains and the water located in the pores between the soil grains. The total stress state uses undrained soil shear strengths ($\Delta u \neq 0$) and is typically used to resist short-term loadings (i.e. construction loading, earthquake loadings, etc.). The Mohr-Coulomb undrained shear strength equation ($\tau = S_u$) is defined as follows:

$$\tau = \mathbf{c} + \sigma_{\mathbf{v}} \, \mathbf{tan}\, \phi \qquad \text{Equation 7-26}$$

The deviator compression stress at failure ($\Delta\sigma_f$) for unconfined compression tests ($\sigma_3 = 0$) on clays is equal to the unconfined compression strength ($\sigma_1 = q_u = c$). The deviator compression stress at failure ($\Delta\sigma_f$) for undrained triaxial testing (unconsolidated or consolidated) is equal to the total major principal stress (σ_1) minus the total minor principal stress (σ_3) (see Figure 7-11).

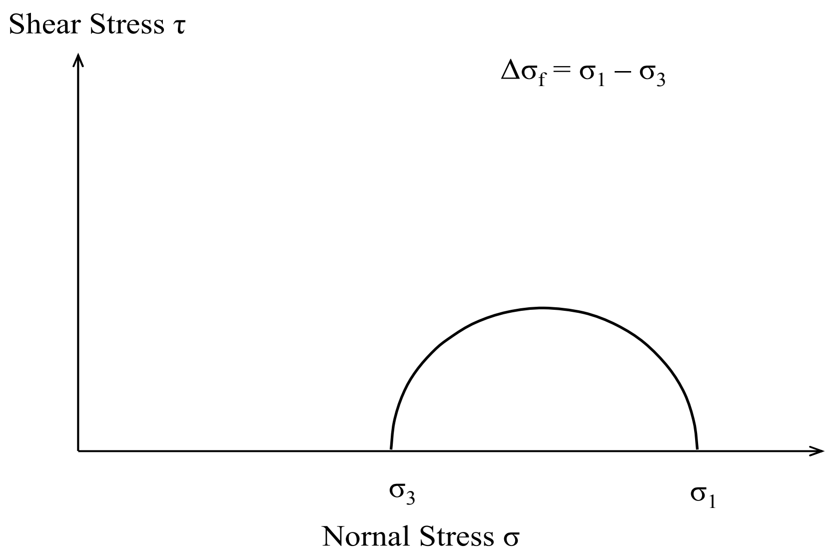


Figure 7-11, Total Principal Stresses

7.10.1 Cohesionless Soils

Undrained shear strengths of cohesionless soils (i.e. sand, low plasticity silts and residual soils) should be used when the rate of loading is so fast that the soil does not have sufficient time to drain such as in the case of rapid draw-down, cyclic loadings, earthquake loadings, and impact loadings. Geotechnical analyses for these types of loadings should use undrained shear strength parameters based on total stress analyses. The peak undrained shear strength in saturated cohesionless soils (τ_{Peak}) is also referred to in literature as the yield shear strength (τ_{yield}). The undrained peak shear strength (τ_{yield}) and the undrained residual shear strength (τ_r) of saturated cohesionless soils can be measured by conducting a consolidated undrained (CU) triaxial compression tests.

The peak undrained shear strength of cohesionless soils may also be determined by correlations developed for in-situ testing such as Standard Penetrometer Test (SPT) or Cone Penetrometer Test (CPT) as indicated in Chapter 5. As stated previously, in Section 7.9.3, the biggest drawback to the use of in-situ field testing methods to obtain undrained shear strengths of cohesionless soils is that the empirical correlations are based on a soil database that is material or soil formation specific and therefore the reliability of these correlations must be verified for each project site by substantiated regional experience or by conducting laboratory testing and calibrating the in-situ testing results.

Correlations have been proposed by Olson and Stark (2003) that relate yield strength ratio ($\tau_{yield}/\sigma'_{vo}$) to normalized SPT blowcount ($N^*_{1,60}$) and normalized CPT tip resistance ($q_{c,1}$). Where τ_{yield}, is the undrained peak shear strength of saturated cohesionless soils and σ'_{vo} is effective overburden pressure. Olson and Stark (2003) used case histories of static loading-induced failures and deformation-induced flow failures to assess the yield strength ratio ($\tau_{yield}/\sigma'_{vo}$).

The Olson and Stark (2003) relationship between yield shear strength ratio ($\tau_{yield}/\sigma'_{vo}$) and the normalized SPT blowcount ($N^*_{1,60}$) is provided in Figure 7-12. The average trend line for Figure 7-12 can be computed using the following equation.

$$\left(\frac{\tau_{\mathbf{yield}}}{\sigma'_{\mathbf{vo}}}\right) = \mathbf{0.205} + \mathbf{0.0075}\left(\mathbf{N}^*_{\mathbf{1,60}}\right) \pm \mathbf{0.04} \qquad \text{Equation 7-27}$$

Where,

$N^*_{1,60} \leq 12$ blow per foot

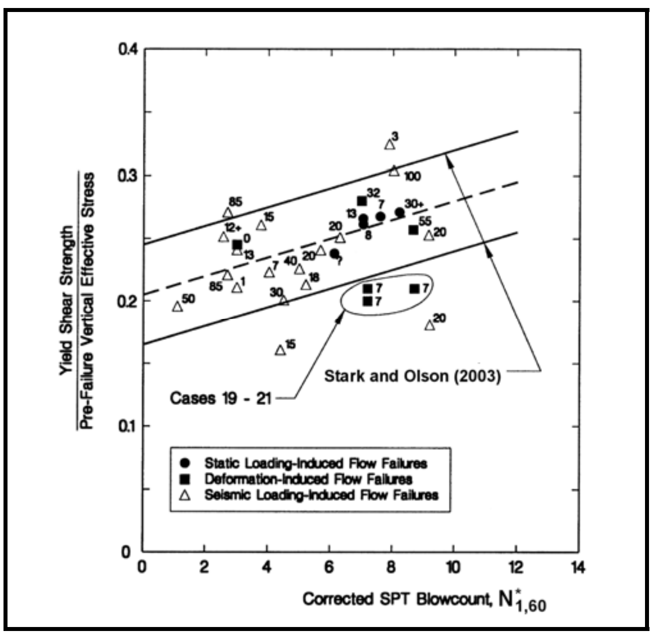


Figure 7-12, Yield Shear Strength Ratio - SPT Blowcount Relationship (Olson, 2001, Olson and Stark, 2003)

The Olson and Stark (2003) relationship between yield shear strength ratio ($\tau_{yield}/\sigma'_{vo}$) and the normalized CPT tip resistance ($q_{c,1}$) is provided Figure 7-13. The average trend line for Figure 7-13 can be computed using the following equation.

$$\left(\frac{\tau_{yield}}{\sigma'_{vo}}\right) = 0.205 + 0.0143\left(q_{c,1}\right) \pm 0.04 \qquad \text{Equation 7-28}$$

Where,

$q_{c,1} \leq 6.5$ MPa ≈ 68 tons per square foot (tsf)

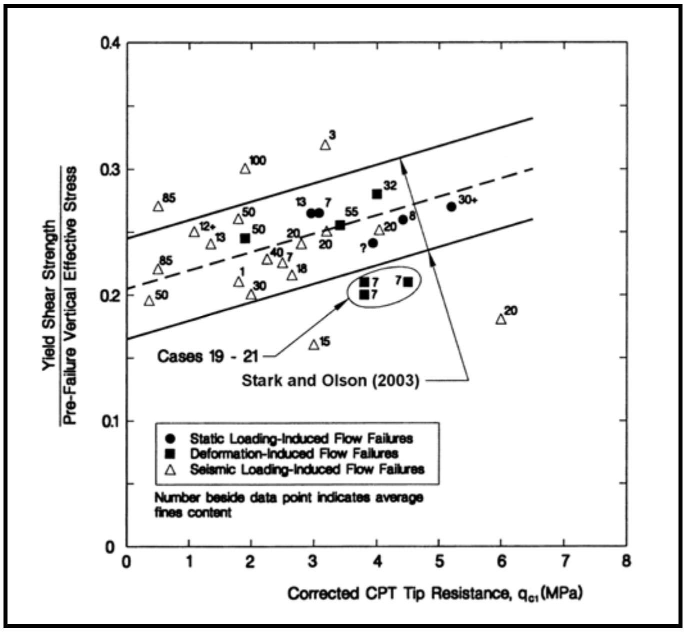


Figure 7-13, Yield Shear Strength Ratio - CPT Tip Resistance Relationship (Olson, 2001, Olson and Stark, 2003)

Undrained residual shear strength ratio of liquefied soils (τ_{rl}/σ'_{vo}) as proposed by Olson and Stark (2002, 2003) are presented in Chapter 12.

7.10.2 Cohesive Soils

The undrained shear strength (τ) of cohesive soils (i.e. clay, highly plastic silts and residual soils) can be determined using unconfined compression (UC) tests, unconsolidated undrained (UU) triaxial tests, or consolidated undrained (CU) triaxial tests of undisturbed samples. Typically the total internal friction angle is negligible and assumed equal to zero ($\phi = 0$) and the Mohr-Coulomb shear strength equation for the undrained shear strength (τ) of cohesive soils can be expressed as indicated by the following equation.

$$\tau = c = \frac{\Delta\sigma_f}{2} \qquad \text{Equation 7-29}$$

The undrained shear strength of cohesive soils may also be determined by in-situ testing such as Standard Penetrometer Test (SPT), Cone Penetrometer Test (CPT), Flat Plate Dilatometer Test (DMT), or Vane Shear Test (VST) as described in Chapter 5. As stated previously, in Section 7.9.3, the biggest drawback to the use of in-situ field testing methods to obtain undrained shear strengths of cohesive soils is that the empirical correlations are based on a soil database that is material or soil formation specific and therefore the reliability of these correlations must be verified for each project site by substantiated regional experience or by conducting laboratory testing and calibrating the in-situ testing results.

The Standard Penetration Test (SPT) can provide highly variable results in cohesive soils as indicated in Table 7-10. However, the following correlations may be used if laboratory undrained shear strengths are correlated to the corrected N_{60} value obtained from the Standard Penetration Test. Peak undrained shear strength (τ), in units of ksf, for cohesive soils (McGregor and Duncan, 1986) can be computed for low plasticity clays using Equation 7-30 and medium to high plasticity clays using Equation 7-31. Plasticity is defined in Chapter 6.

$$\tau = \mathbf{c} = \mathbf{0.075\,N_{60}}$$ Equation 7-30

$$\tau = \mathbf{c} = \mathbf{0.15\,N_{60}}$$ Equation 7-31

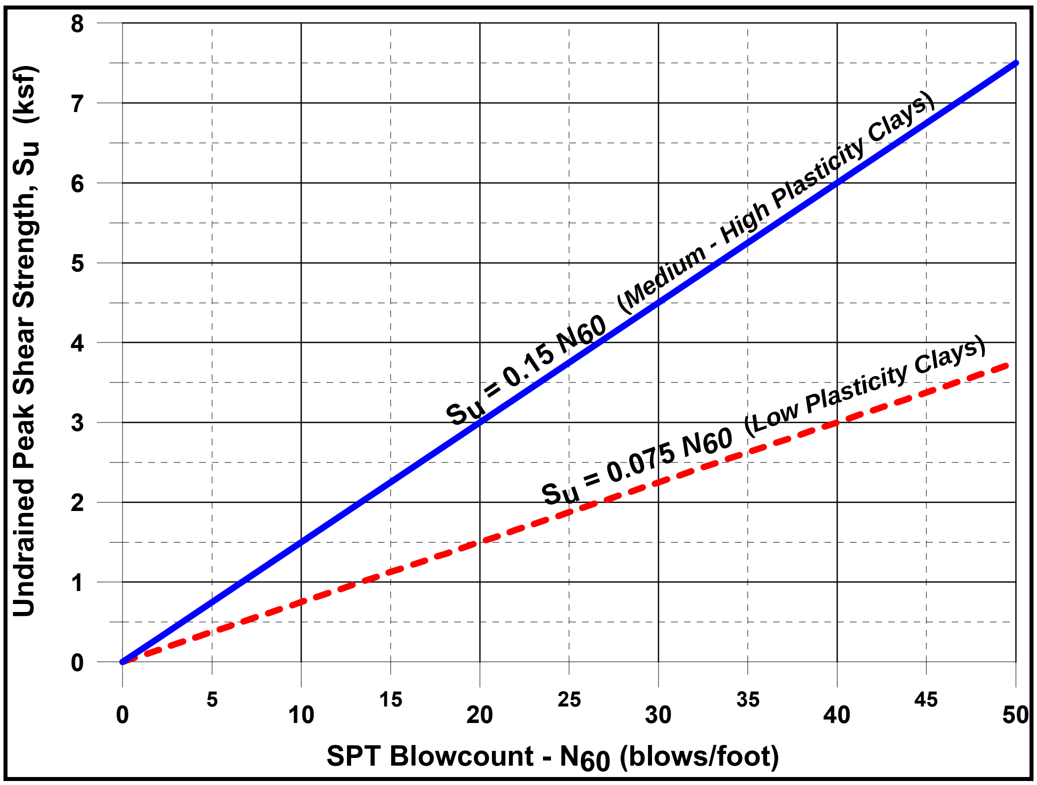


Figure 7-14, Undrained Shear Strength – SPT Relationship (McGregor and Duncan, 1986)

The peak undrained shear strength (τ) of cohesive soils can also be obtained from the Cone Penetrometer Test (CPT) (Sabatini, 2005) as indicated by the following equation.

$$\tau = \mathbf{c} = \frac{\mathbf{q_c} - \sigma_{\mathbf{vo}}}{\mathbf{N_k^*}}$$ Equation 7-32

Where,

q_c = CPT tip resistance (measured, uncorrected)
σ_{vo} = total overburden pressure at test depth
N^*_k = cone factor.

The cone factor has been found to be approximately equal to 14 ± 5. Because of the large variation in N^*_k, CPT testing results shall be correlated with soil borings and laboratory testing to back-calculate the cone factor for the specific soil types under evaluation.

The Flat Plate Dilatometer Test (DMT) results should be corrected and correlated to undrained shear based on the FHWA Publication FHWA-SA-91-044, *The Flat Dilatometer Test*.

The peak undrained shear strength (τ) of cohesive soils can also be obtained from the Vane Shear Test (VST) (Aas et al., 1986) can be used as indicated by the following equation.

$$\tau = \mu \mathbf{S_{vane}}$$ Equation 7-33

Where,

μ = Vane correction factor (see Figure 7-15)
S_{vane} = VST field measured undrained shear strength. The S_{vane} interpretation results should be based on ASTM STP1014 (1988).

The VST field measured undrained shear strength, S_{vane}, should be computed based on the following equation.

$$S_{vane} = \frac{6T}{(7\pi D^3)} \quad for\ \frac{H}{D} = 2$$ Equation 7-34

Where,

T = VST torque resistance
D = Diameter of field vane
H = Height of field vane

The vane correction factor (μ) is determined from the Aas et al. (1986) relationship shown in Figure 7-15. The vane correction factor (μ) is computed by entering the top chart with PI and (S_{vane}/σ'_{vo}) to establish whether the clay is within the normally consolidated (NC) range between the limits "young" and "aged", or overconsolidated (OC). The lower chart is used by entering the (S_{vane}/σ'_{vo}) and selecting the vane correction factor (μ) for the appropriate NC or OC curves. A maximum vane correction factor (μ) of 1.0 is recommended by Aas, et. al (1986).

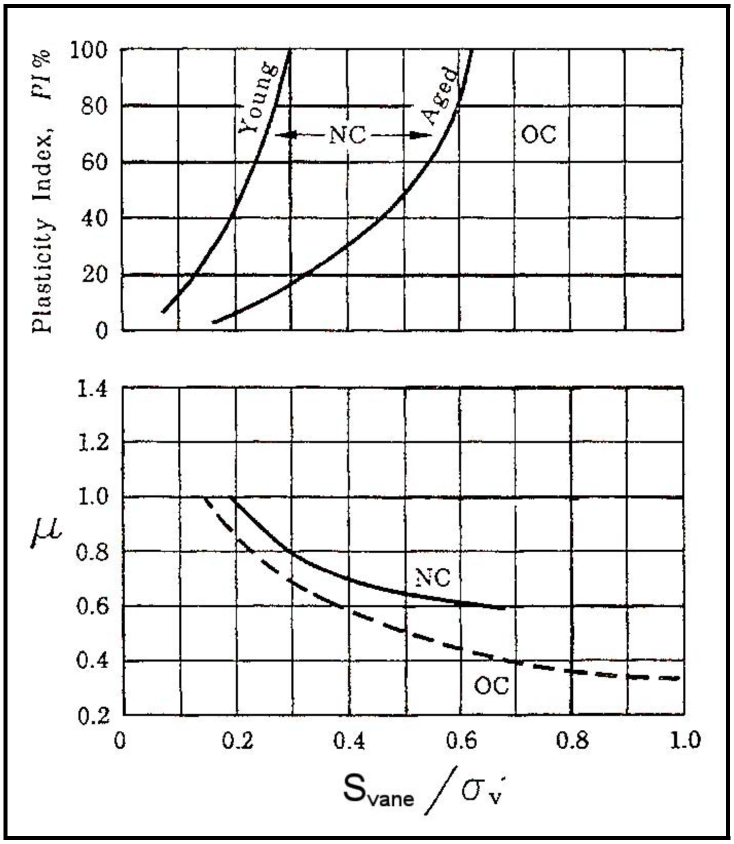


Figure 7-15, Vane Shear Correction Factor (Aas, et. al., 1986)

Empirical correlations based on SHANSHEP laboratory testing results can be used for preliminary designs and to evaluate the peak undrained shear strength (S_u) obtained from laboratory testing or in-situ testing. This method is only applicable to clays without sensitive structure where undrained shear strength increases proportionally with the effective overburden pressure (σ'_{vo}). The SHANSHEP laboratory test results of Ladd et al. (1977) revealed trends in undrained shear strength ratio (S_u / σ'_v) as a function of overconsolidation ratio as indicated in Figure 7-16.

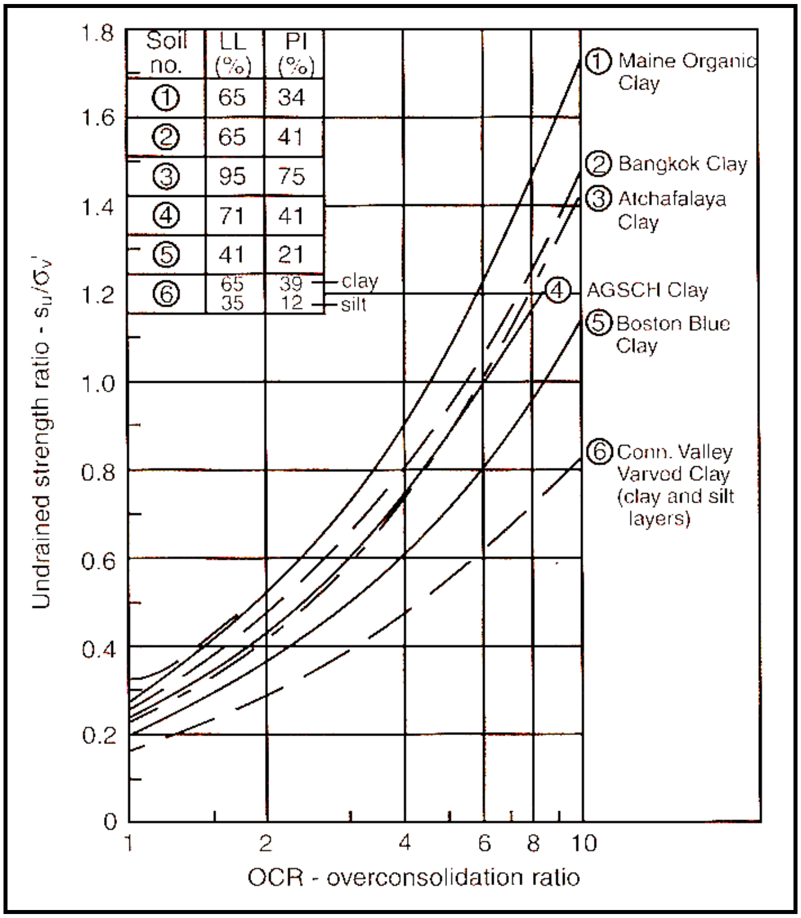


Figure 7-16, Undrained Shear Strength Ratio and OCR Relationship (Ladd et al., 1977)

The average peak undrained shear strengths (τ) shown in Figure 7-16 can be approximated by an empirical formula developed by Jamiolkowski et al. (1985) as indicated by the following equation.

$$\tau = \left(\mathbf{0.23}(\mathbf{OCR})^{\mathbf{0.8}}\right)\sigma'_{\mathbf{vo}}$$ Equation 7-35

Where,

τ	=	undrained shear strength (tsf)
OCR	=	overconsolidation ratio
σ'_{vo}	=	effective overburden pressure at test depth (tsf)

The undrained shear strength (τ) can be compared to the remolded shear strength (τ_R) (residual undrained shear strength, τ_r) to determine the sensitivity (S_t) of cohesive soils. Sensitivity is the measure of the breakdown and loss of interparticle attractive forces and bonds within cohesive soils. Typically in dispersed cohesive soils the loss is relatively small, but in highly flocculated structures the loss in strength can be large. Sensitivity is determined using the following equation.

$$\mathbf{S_t} = \frac{\tau}{\tau_{\mathbf{R}}}$$ Equation 7-36

The description of sensitivity is defined in the following table.

Table 7-13, Sensitivity of Cohesive Soils (Modified from Spangler and Handy, 1982)

Sensitivity	Descriptive Term
< 1	Insensitive
1 - 2	Slightly Sensitive
3 - 4	Medium Sensitive
5 - 8	Sensitive
9 - 16	Very Sensitive
17 - 32	Slightly Quick
33 - 64	Medium Quick
>64	Quick

The remolded shear strength of cohesive soils (τ_R) can be determined from remolded triaxial specimens or from in-situ testing methods (electro-piezocone or field vane). Triaxial specimens should have the same moisture content as the undisturbed sample as well as the same degree of saturation and confining pressure. Further sensitivity can be related to the liquidity index using the following figure.

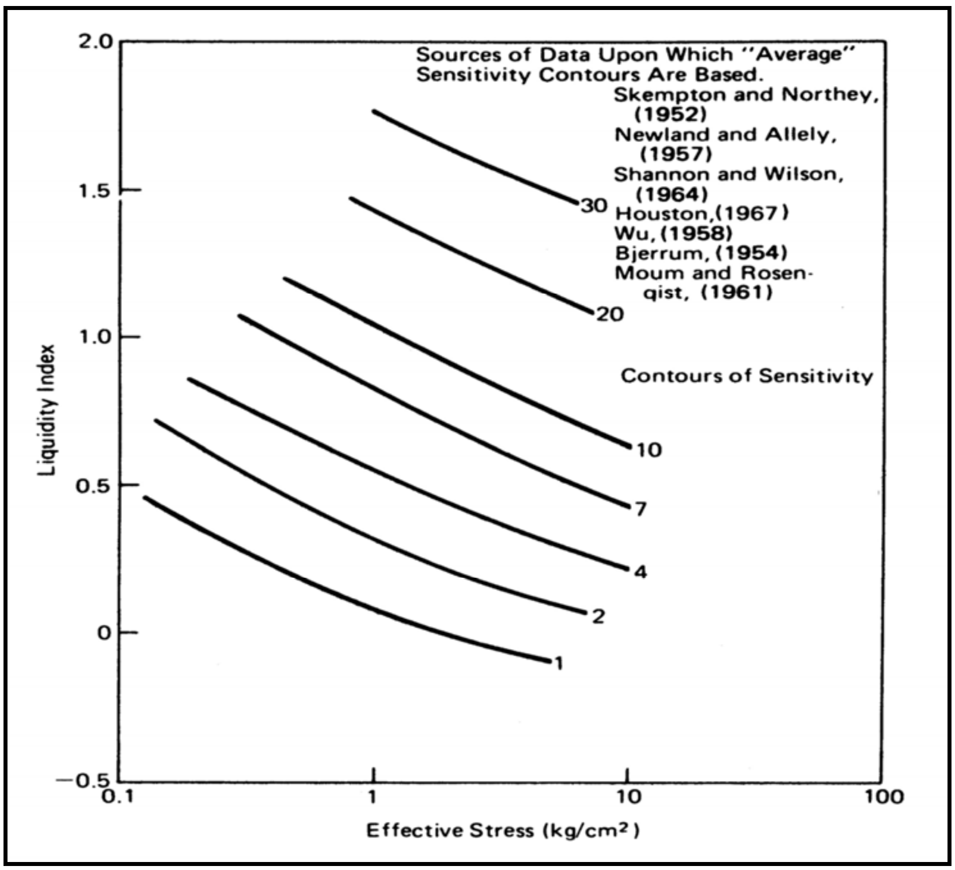


Figure 7-17, Sensitivity based on Liquidity Index and σ'_{vo} (Mitchell, 1993)

The Liquidity Index (LI) can also be related to remolded shear strength (τ_R = c_{ur} = S_{ur}) as indicated in the following.

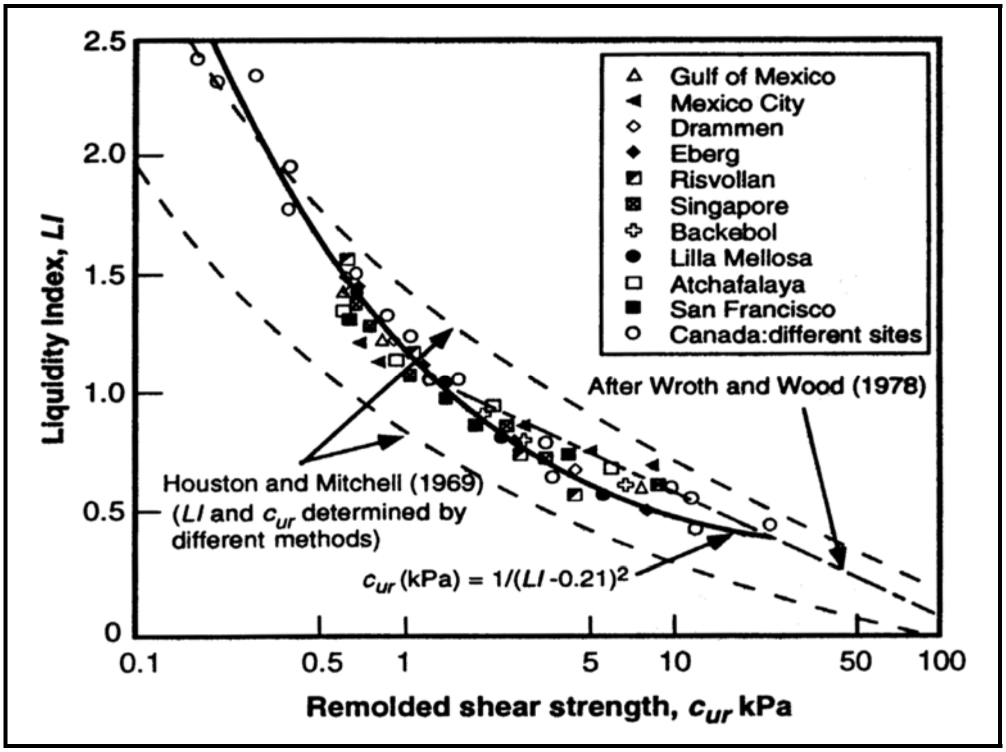


Figure 7-18, Remolded Shear Strength vs Liquidity Index (Mitchell, 1993)

Where,

1 kPa = 0.0209 ksf

The Liquidity Index (LI) is the relationship between natural moisture content, Plastic Limit (PL), and the Liquid Limit (LL). The LI is a measure of the relative softness of a cohesive soil as indicated by the closeness of the natural moisture content to the liquid limit. The LI can be determined by the following equation.

$$\mathbf{LI} = \frac{(\mathbf{w} - \mathbf{PL})}{(\mathbf{LL} - \mathbf{PL})} \qquad \text{Equation 7-37}$$

Where,

w = natural moisture content
LL = Liquid Limit
PL = Plastic Limit

The undrained residual shear strength of cohesive soils (S_t < 2) can be estimated for preliminary design and to evaluate the undrained residual shear strength (τ_r = S_{ur}) obtained from laboratory testing or in-situ testing. The undrained residual shear strength (τ_r = S_{ur}) can be estimated by reducing peak undrained shear strength (τ) by a residual shear strength loss factor (λ) as indicated in the following equation.

$$\tau_r = \lambda \tau \qquad \text{Equation 7-38}$$

The residual shear strength loss factor (λ) typically ranges from 0.50 to 0.67 depending on the type of clay soil. The residual shear strength loss factors (λ) recommended in Table 7-14 are based on the results of a pile soil set-up factor study prepared by Rauche et al. (1996)

Table 7-14, Residual Shear Strength Loss Factor (λ)

Soil Type		Residual Shear Strength Loss Factor (λ)
USCS	Description	
Low Plasticity Clay	CL-ML	0.57
Medium to High Plasticity Clay	CL & CH	0.50

7.10.3 ϕ-c Soils

The undrained shear strength of soils that have both ϕ and c components should be determined in the laboratory using the appropriate testing methods. However, if the samples for this type of testing have not been obtained (e.g. during the preliminary exploration), then the soil should be treated as if the soil were either completely cohesive or cohesionless. For soils that are difficult to determine the approximate classification, the undrained shear strength parameters for both cohesive and cohesionless soils should be determined and the more conservative design should be used.

7.10.4 Maximum Allowable Total Soil Shear Strengths

SCDOT has established maximum allowable peak (c, ϕ) and residual (c_r, ϕ_r) undrained soil shear strength design parameters shown in Table 7-15, for use in design. These soil shear strength design parameters may not be exceeded without laboratory testing and the express written permission of the PCS/GDS.

Table 7-15, Maximum Allowable Total Soil Shear Strengths

Soil Type		Peak		Residual	
		c (psf)	ϕ (degrees)	c_r (psf)	ϕ_r (degrees)
USCS	Description				
GW, GP, GM, GC	Stone and Gravel	0	34	0	18
SW	Coarse Grained Sand	0	17	0	7
SM, SP	Fine Grained Sand	0	17	0	7
SP	Uniform Rounded Sand	0	15	0	6
ML, MH, SC	Silt, Clayey Sand, Clayey Silt	1,500	15	1,200	6
SM-ML	Residual Soils	900	14	700	6
CL-ML	NC Clay (Low Plasticity)	1,500	0	900	0
CL, CH	NC Clay (Med-High Plasticity)	2,500	0	1250	0
CL-ML	OC Clay (Low Plasticity)	2,500	0	1400	0
CL, CH	OC Clay (Med-High Plasticity)	4,000	0	2000	0

7.11 EFFECTIVE STRESS

Effective stress is the force per unit area carried by the soil grains. The effective stress state uses drained soil shear strengths ($\Delta u = 0$). The Mohr-Coulomb drained shear strength equation is defined as follows.

$$\tau' = \mathbf{c}' + \sigma'_{\mathbf{v}}\ \mathbf{tan}\,\phi' \qquad \text{Equation 7-39}$$

The deviator compression stress at failure ($\Delta\sigma_f$) for undrained triaxial testing (consolidated) is equal to the total or effective major principal stress (σ_1) minus the total or effective minor principal stress (σ_3). The effective major and minor principal stresses are the total major and minor principal stresses minus the pore pressure at failure (u_f) (see Figure 7-19).

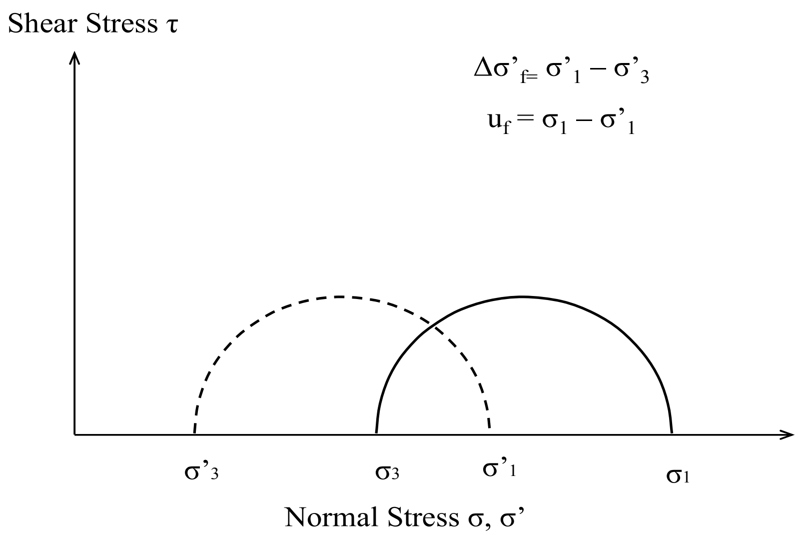


Figure 7-19, Effective Principal Stresses

7.11.1 <u>Cohesionless Soils</u>

Drained shear strengths of cohesionless soils (i.e. sand, low plasticity silts, and residual soils) should be used when there is relatively no change in pore water pressure ($\Delta u \approx 0$) as a result of soil loading. Cohesionless soils that are subjected to construction loads and static driving loads typically use peak or residual drained shear strengths due to the relatively rapid (minutes to hours) drainage characteristics of granular soils as indicated in Section 7.9.2. The peak or residual drained soil shear strength parameters can be obtained from consolidated drained (CD) triaxial tests, consolidated undrained (CU) triaxial tests with pore pressure measurements, or direct shear (DS) tests. Typically the effective cohesion (c') is negligible and assumed to be equal to zero ($c' = 0$) and the Mohr-Coulomb shear strength criteria for drained shear strength of cohesionless soils can then be expressed as indicated in the following equation.

$$\tau' = \sigma'_v\ \tan\phi' \qquad \text{Equation 7-40}$$

The peak drained shear strength of cohesionless soils may also be determined by in-situ testing methods such as the Standard Penetrometer Test (SPT), Cone Penetrometer Test (CPT), or Flat Plate Dilatometer Test (DMT). As stated previously, in Section 7.9.3, the biggest drawback to the use of in-situ field testing methods to obtain drained shear strengths of cohesionless soils is that the empirical correlations are based on a soil database that is material or soil formation specific and therefore the reliability of these correlations must be verified for each project site by either using substantiated regional experience or conducting laboratory testing and calibrating the in-situ testing results.

The effective peak friction angle, ϕ', of cohesionless soils can be obtained from Standard Penetrometer Test (SPT). Most SPT correlations were developed for clean sands and their use for micaceous sands/silts, silty soils, and gravelly soils may be may be unreliable as indicated below:

- SPT blow counts in micaceous sands or silts may be significantly reduced producing very conservative correlations.
- SPT blow counts in silty soils may produce highly variable results and may require verification by laboratory triaxial testing depending on a sensitivity analysis of the impact of the variability of results on the analyses and consequently the impact on the project.
- SPT blow counts in gravelly soils may overestimate the penetration resistance. Conservative selection of shear strength parameter or substantiated local experience should be used in lieu of laboratory testing.

The effective peak friction angle, ϕ', of cohesionless soils can be estimated using the relationship of Hatanaka and Uchida (1996) for corrected N-values ($N^*_{1,60}$) as indicated by Figure 7-20.

$$\phi' = \left[15.4 N^*_{1,60}\right]^{0.5} + 20^{\circ}$$ Equation 7-41

Where,
4 blows per foot $\leq N^*_{1,60} \leq$ 50 blows per foot

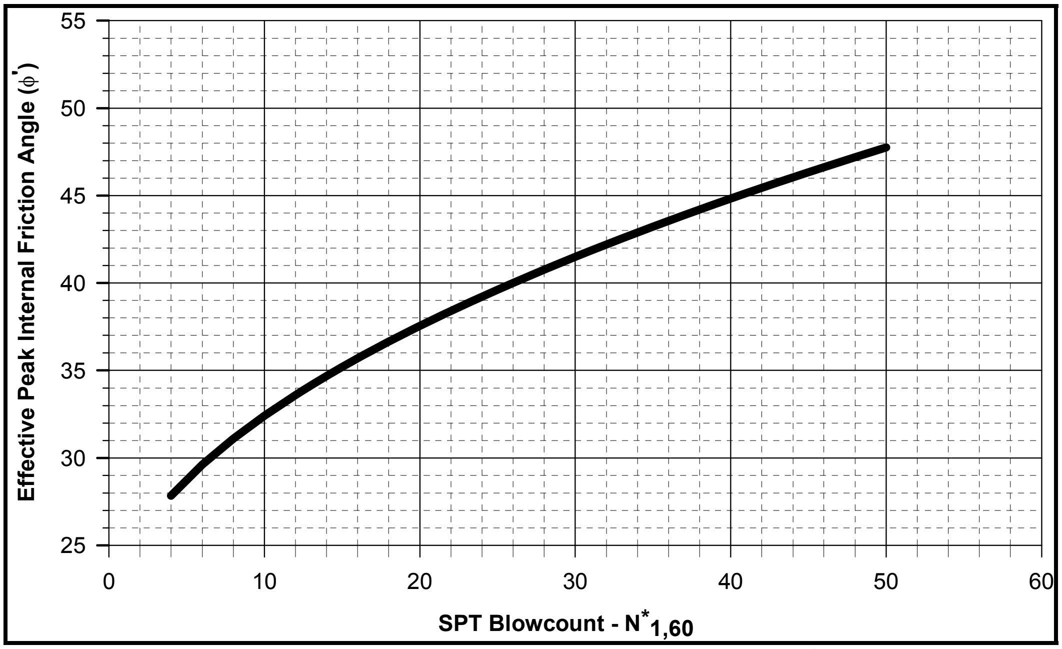


Figure 7-20, Effective Peak Friction Angle and SPT ($N^*_{1,60}$) Relationship (Based on Hatanaka and Uchida, 1996)

The effective friction angle, ϕ', of cohesionless soils can also be estimated by Cone Penetrometer Test (CPT) based on Robertson and Campanella (1983). This method requires the estimation of the effective overburden pressure (σ'_{vo}) and the cone tip resistance (q_c) measured, uncorrected using the relationship in Figure 7-21. This relationship may be approximated by the following equation.

$$\phi' = \tan^{-1}\left[0.1 + 0.38\log\left(\frac{q_c}{\sigma'_{vo}}\right)\right]$$ Equation 7-42

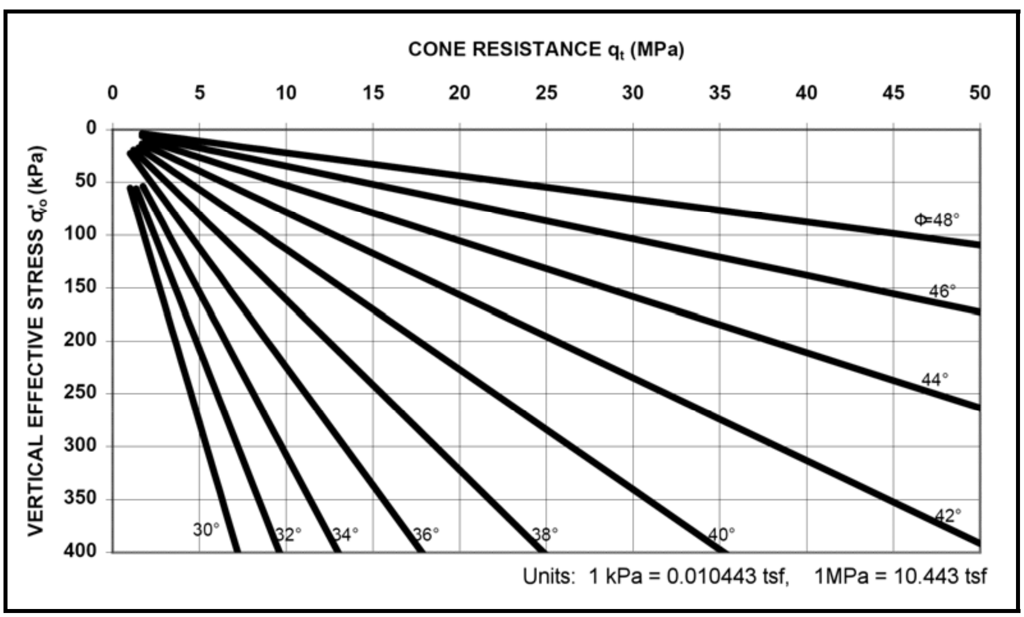


Figure 7-21, Effective Peak Friction Angle and CPT (q_c) Relationship (Robertson and Campanella, 1983)

The effective friction angle, ϕ', of cohesionless soils can also be estimated by Flat Plate Dilatometer Test (DMT) using the Robertson and Campanella (1991) relationship shown in Figure 7-22. This method requires the determination of the horizontal stress index (K_D) by the procedures outlined in FHWA-SA-91-044, *The Flat Plate Dilatometer*. The Robertson and Campanella (1991) relationship may be approximated by the following equation.

$$\phi' = 28^{\circ} + 14.6\log(K_D) - 2.1\log^2(K_D)$$ Equation 7-43

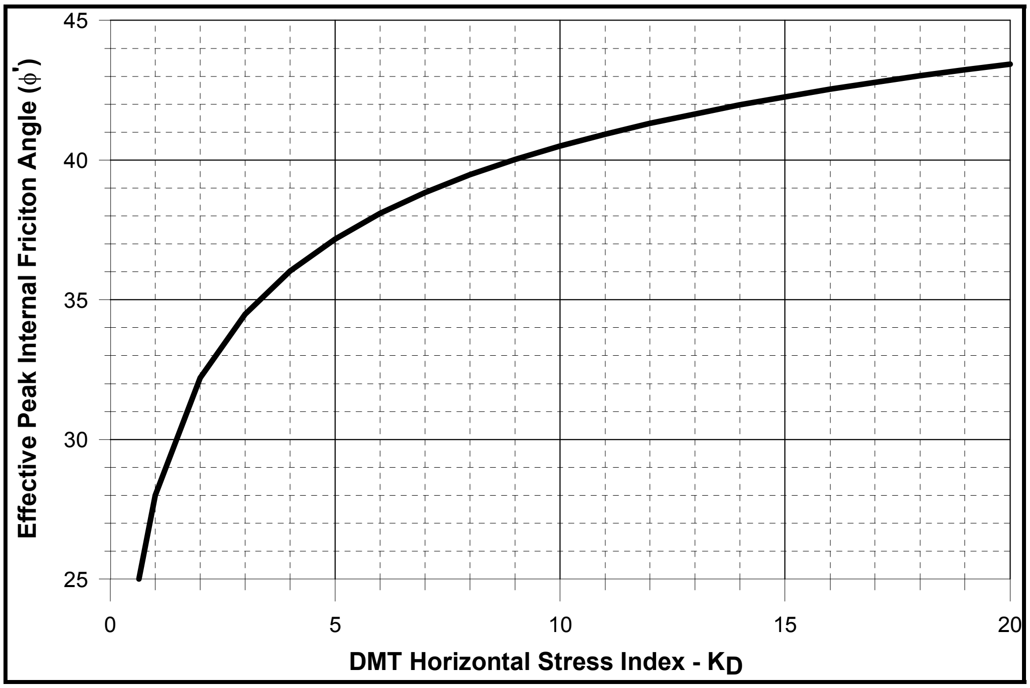


Figure 7-22, Effective Peak Friction Angle and DMT (K_D) Relationship (Robertson and Campanella, 1991)

7.11.2 Cohesive Soils

Drained shear strengths of cohesive soils (i.e. clay, high plasticity silts and residual soils) should be used when there is relatively no change in pore water pressure ($\Delta u \approx 0$) as a result of soil loading such as static driving loads. Geotechnical analyses for these types of loadings should use drained shear strength parameters based on effective stress analyses. The peak or residual drained soil shear strength parameters can be obtained from consolidated drained (CD) triaxial testing (this test is normally not performed because of the time requirements for testing), or consolidated undrained (CU) triaxial testing with pore pressure measurements. Typically for normally consolidated clays the effective cohesion (c') is negligible and is assumed to be equal to zero ($c' = 0$) and the Mohr-Coulomb shear strength equation for drained shear strength of cohesive soils can be expressed as indicated in the following equation.

$$\tau' = \sigma'_v \tan\phi'$$ Equation 7-44

Typically for overconsolidated clays the effective cohesion is greater than zero with the effective friction angle less than that determined for normally consolidated clays. When the preconsolidation pressure (σ'_p or p'_c) is exceeded the overconsolidated clay becomes normally consolidated (see Figure 7 -23).

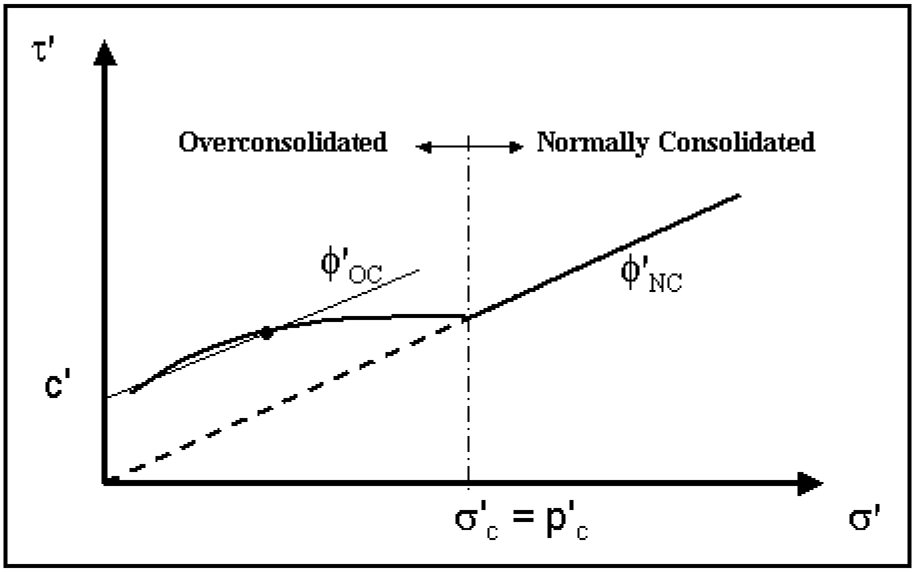


Figure 7-23, Overconsolidated Clay Failure Envelope (CUw/pp Triaxial Test)

The effective peak, fully softened, and residual drained shear strength of cohesive soils should not be evaluated using in-situ testing methods.

Correlations have been developed between drained shear strengths of cohesive soils and index parameters such as plasticity index (I_P or PI), liquid limit (LL), clay fraction (CF) and effective overburden pressure (σ'_{vo} = effective normal stress). Similarly to relationships developed for in-situ testing methods, these relationships for drained shear strengths of cohesive soils were developed based on a soil database that is typically material or soil formation specific and may require verification by laboratory triaxial testing depending on a sensitivity analysis of the impact of the variability of results on the analyses and consequently the impact on the project. These relationships should be used to evaluate the validity of laboratory testing results and to improve the relationship database for regional soil deposits by the SCDOT.

In normally consolidated clays (OCR = 1) the shear strength test will result in a peak effective friction angle (ϕ'). Terzaghi et al. (1996) proposed the relationship in Figure 7-24 between peak effective friction angle (ϕ') for normally consolidated clays and the plasticity index (I_P or PI). For plasticity indices above 60 percent, the peak effective friction angle (ϕ') should be determined from laboratory testing. The Terzaghi et al. (1996) relationship between peak effective friction angle (ϕ') for normally consolidated clays and the plasticity index (I_P or PI) may be estimated by the following equation.

$$\phi' = \mathbf{35.7^o - 0.28(PI) + 0.00145(PI)^2 \pm 8^\circ} \qquad \text{Equation 7-45}$$

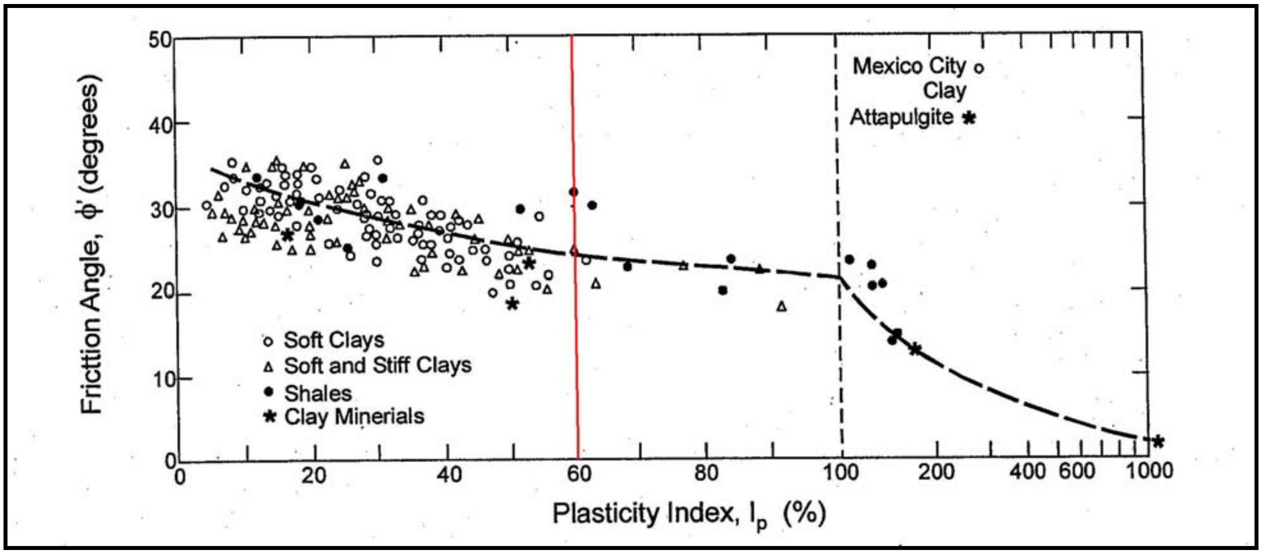


Figure 7-24, Plasticity Index versus Drained Friction Angle For NC Clays (Terzaghi, Peck, and Mesri, 1996)

As indicated earlier, overconsolidated clays reach a peak undrained and then experience shear strain softening to fully softened state. Stark and Eid (1997) proposed the relationship indicated in Figure 7-25 to estimate the fully softened or the peak normally consolidated (NC) effective friction angle (ϕ'). This correlation uses the Liquid Limit (LL), clay size fraction (CF %), and effective overburden pressure (σ'_{vo} = effective normal stress).

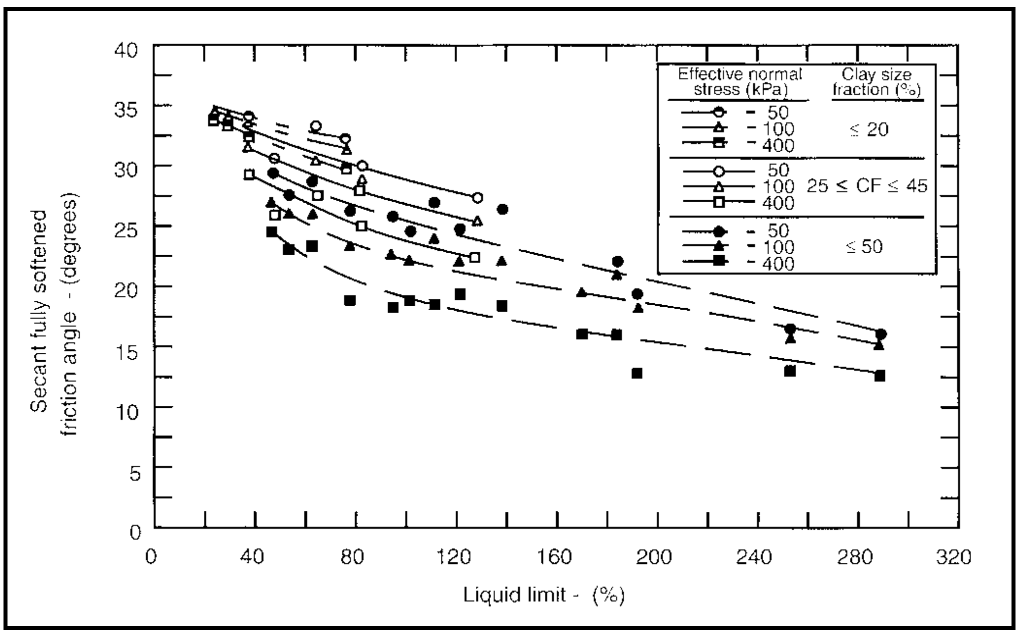


Figure 7-25, Fully Softened (NC) Friction Angle and Liquid Limit Relationship (Stark and Eid, 1997)

For either normally consolidated (OCR = 1) or overconsolidated (OCR > 1) the drained residual friction angle is the same. Stark and Eid (1994) proposed the relationship indicated in Figure 7-26 to estimate the effective residual friction angle (ϕ'_r). This correlation uses the Liquid Limit

(LL), clay size fraction (CF %), and effective overburden pressure (σ'_{vo} = effective normal stress).

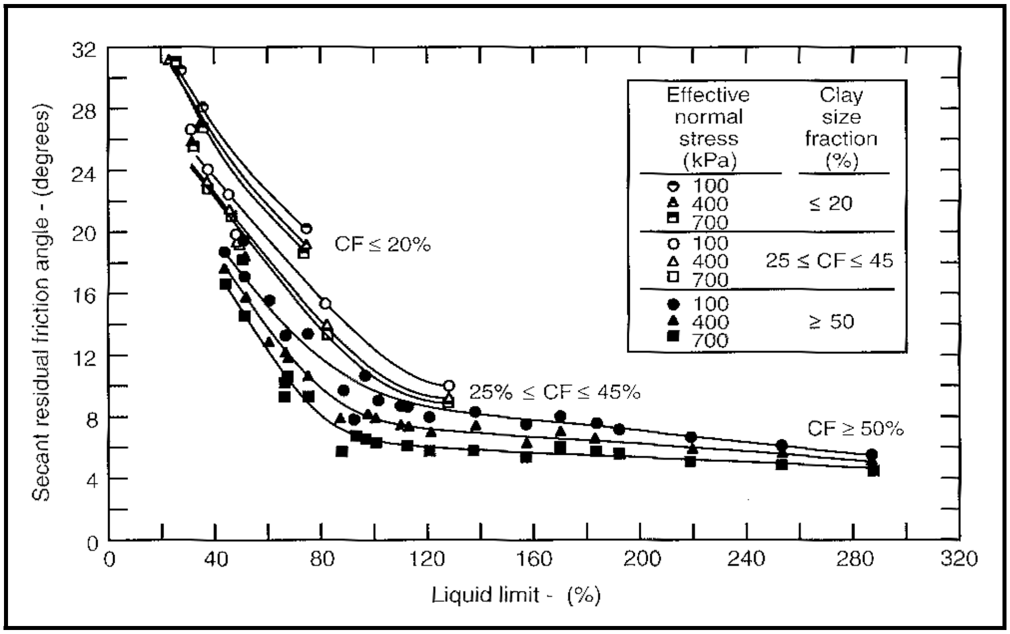


Figure 7-26, Drained Residual Friction Angle and Liquid Limit Relationship (Stark and Eid, 1994)

7.11.3 ϕ' – c' Soils

The drained shear strength of soils that have both ϕ' and c′ components should be determined in the laboratory using the appropriate testing methods. However, if the samples for this type of testing have not been obtained (e.g. during the preliminary exploration), then the soil should be treated as if the soil were either cohesive soils or cohesionless soils. For soils that are difficult to determine the approximate classification, the drained shear strength parameters for both cohesive and cohesionless should be determined and the more conservative design should be used.

7.11.4 Maximum Allowable Effective Soil Shear Strength

SCDOT has established maximum allowable effective soil shear strength design parameters (c', ϕ') shown in Table 7-16, for use in design. These soil shear strength design parameters (c', ϕ') may not be exceeded without laboratory testing and the written permission of the PCS/GDS.

Table 7-16, Maximum Allowable Effective Soil Shear Strengths

Soil Description		Peak [1]		Residual	
		c'	ϕ'	c'	ϕ'
USCS	**Description**	**(psf)**	**(degrees)**	**(psf)**	**(degrees)**
GW, GP, GM, GC	Stone and Gravel	0	40	0	34
SW	Coarse Grained Sand	0	38	0	32
SM, SP	Fine Grained Sand	0	36	0	30
SP	Uniform Rounded Sand	0	32	0	32
ML, MH, SC	Silt, Clayey Sand, Clayey Silt	0	30	0	27
SM-ML	Residual Soils	0	27	0	22
CL-ML	NC Clay (Low Plasticity)	0	35	0	31
CL, CH	NC Clay (Med-High Plasticity)	0	26	0	16
CL-ML	OC Clay (Low Plasticity)	0	34	0	31
CL, CH	OC Clay (Med-High Plasticity)	0	28	0	16

(1) The same maximum peak effective shear strength parameters shall be used for peak effective internal friction angle of normally consolidated cohesive soils and to the fully-softened internal friction angle of overconsolidated cohesive soils.

7.12 BORROW MATERIALS SOIL SHEAR STRENGTH SELECTION

This section pertains to the selection of soil shear strength design parameters for borrow materials used in embankments or behind retaining walls (other than MSE walls or reinforced slopes). Soil shear strength selection shall be based on the soil loading and soil response considerations presented in Section 7.9. The soil shear strength design parameters selected must be locally available, cost effective, and be achievable during construction. The selection of soil shear strength design parameters that require the importation of materials from outside of the general project area should be avoided. To this end, bulk samples will be obtained from existing fill embankments or from proposed cut areas tested as indicated in Chapter 4. The purpose of sampling and testing the existing fill is the assumption that similar fill materials will be available locally. The purpose of sampling and testing proposed cut areas is to determine the suitability of the material for use as fill. The selection of soil shear strength design required for borrow sources should take into consideration the construction borrow specifications as indicated in Section 7.12.1.

The procedure for selecting soil shear strength design parameters varies depending on the type of project as indicated below:

- **Design-Build Projects:** The selection of soil shear strength design parameters for borrow materials requires that the Contractor obtain soil shear strength parameters from all potential borrow pit sources. Evaluation of the soil shear strength design parameters requires that a composite bulk sample be obtained from the borrow source and have the following laboratory tests performed:

    - Moisture Density Relationship (Standard Proctor)
    - Grain Size Distribution with wash #200 Sieve
    - Moisture-Plasticity Relationship Determination (Atterberg Limits)
    - Natural Moisture Content

- Consolidated Undrained (CU) Triaxial Shear Test with pore pressure measurements (sample remolded to 95% of Standard Proctor with moisture -1 percent to +2 percent of optimum moisture content) to obtain drained and undrained shear strength parameters

- **Traditional Design-Bid-Build W/Existing Embankments:** This type of project can occur when existing roads are being improved by widening the existing road. An investigation of locally available materials should be made to confirm that the existing embankment soils are still locally available. If the existing embankment soils are available, the selection of soil shear strength design parameters for these type of projects will be based on using laboratory testing from composite bulk sample obtained from the existing embankment as required in Chapter 4 and appropriately select the drained and undrained soil shear strength design parameters for the borrow material. The plans and contract documents may specify the minimum required soil shear strength parameters for the borrow sources based on the existing embankment soils, if necessary. If the existing embankment soils are not locally available, the borrow material shear strength parameters will be determined as if the project were on a new alignment.

- **Traditional Design-Bid-Build On New Alignment:** This type of project requires the pre-selection of soil shear strength design parameters without performing any laboratory testing. The preliminary subsurface investigation may need to identify locally available soils (or borrow sources) and appropriately select soil shear strength design parameters for the borrow materials. Locally available soils can be investigated by using USDA Soil Survey maps as indicated in Section 7.12.2. The plans and contract documents may specify the minimum required soil shear strength parameters for the borrow sources, if necessary.

7.12.1 SCDOT Borrow Specifications

The 2007 SCDOT Standard Specifications For Highway Construction, Section 203, provides the requirements for borrow material. Embankment material must not have optimum moisture content greater than 25.0% as defined in accordance with SC-T-29. Acceptable soils for use in embankments and as subgrade vary by county indicated by the following two Groups.

- **Group A:** Includes the following counties: Abbeville, Anderson, Cherokee, Chester, Edgefield, Fairfield, Greenville, Greenwood, Lancaster, Laurens, McCormick, Newberry, Oconee, Pickens, Saluda, Spartanburg, Union, and York. Below the upper 5 feet of embankment, any soil that does not meet the description of muck may be used provided it is stable when compacted to the required density.

- **Group B:** Aiken, Allendale, Bamberg, Barnwell, Beaufort, Berkeley, Calhoun, Charleston, Chesterfield, Clarendon, Colleton, Darlington, Dillon, Dorchester, Florence, Georgetown, Hampton, Horry, Jasper, Kershaw, Lee, Lexington, Marion, Marlboro, Orangeburg, Richland, Sumter, and Williamsburg. The soil material below the upper 5 feet of embankment is soils that classify as A-1, A-2, A-3, A-4, A-5, and A-6.

Groups A and B are shown graphically on a South Carolina map in Figure 7-27.

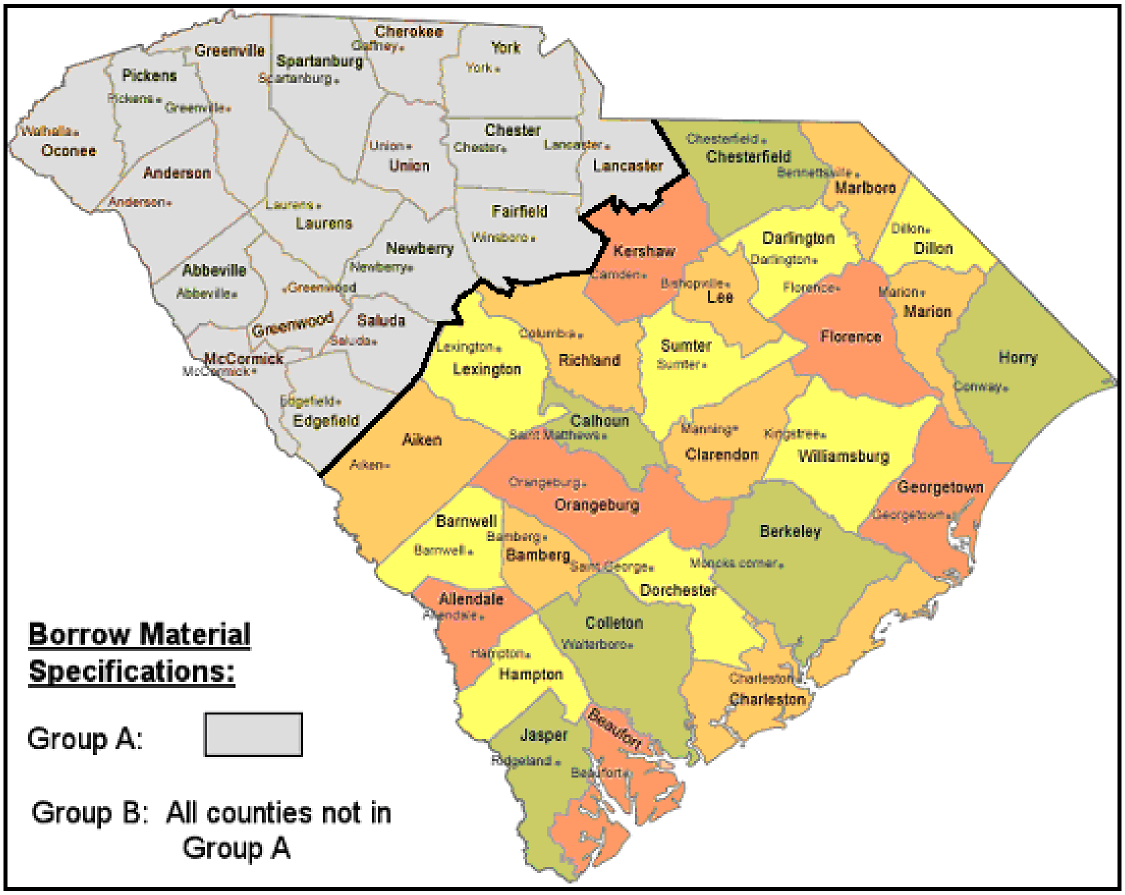


Figure 7-27, Borrow Material Specifications By County

A brief geologic description of the surface soils in Groups A and B are provided below and for more detail see Chapter 11.

- **Group A**: This group is located northwest of the “Fall Line” in the Blue Ridge and Piedmont physiographic geologic units. The Blue Ridge unit surface soils typically consist of residual soil profile consisting of clayey soils near the surface where weathering is more advanced, underlain by sandy silts and silty sands. There may be colluvial (old land-slide) material on the slopes. The Piedmont unit has a residual soil profile that typically consists of clayey soils near the surface, where soil weathering is more advanced, underlain by sandy silts and silty sands. The residual soil profile exists in areas not disturbed by erosion or the activities of man.

- **Group B:** This group is located south and east of the “Fall Line” in the Coastal Plain physiographic geologic unit. Sedimentary soils are found at the surface that consist of unconsolidated sand, clay, gravel, marl, cemented sands, and limestone.

7.12.2 USDA Soil Survey Maps

Locally available borrow sources can be researched by using the United States Department of Agriculture (USDA) Soil Survey Maps. A listing of USDA Soil Surveys that are available can be obtained by selecting "South Carolina" at http://soils.usda.gov/survey/printed_surveys/ and reviewing results by county.

Soil surveys can be obtained as either printed documents, CD-ROM, downloading online .pdf documents, or generated using USDA Web Soil Survey (WWS) Internet application.

The USDA Soil Survey Maps typically indicate Soil Map Units that are described based on USDA textural classification system. Recent USDA Soil Surveys manuscripts contain tables with equivalent material descriptions for the AASHTO soil classification system and the Unified Soil Classification system (USCS). When only the USDA textural classification is indicated in the maps, the geotechnical engineer will need to correlate the USDA textural classifications to the AASHTO soil classification system and USCS.

The USDA Web Soil Survey (WSS) Internet application can be accessed at: http://websoilsurvey.nrcs.usda.gov/app/. The USDA Web Soil Survey (WSS) is an online web application that can provide soil data and natural resource information produced by the National Cooperative Soil Survey. The web site is under constant development and being updated with new information. Soil survey maps and maps of Roadfill sources for project specific locations can be generated as shown in Figure 7-28 and Figures 7-29, respectively.

Figure 7-28, USDA Soil Map – Newberry County, South Carolina (USDA Web Soil Survey - WSS)

Figure 7-29, USDA Roadfill Source Map - Newberry County, South Carolina (USDA Web Soil Survey - WSS)

7.12.3 Compacted Soils Shear Strength Selection

Compacted soils are used to construct roadway embankments, bridge approaches, and backfill behind retaining walls. This Section does not govern the selection of backfill soil properties for MSE walls or reinforced slopes. The method of selecting soil shear strength parameters for compacted soils will be either:

- Measured using consolidated-undrained triaxial tests with pore pressure measurements.
- Conservatively selected based on drained soil shear strength parameters typically encountered in South Carolina soils.

The method to be used for selection will be dependent on the type of project as discussed in Section 7.12.

SCDOT experience with borrow materials typically found in Group A are Piedmont residual soils. These borrow materials are typically classified as micaceous clayey silts and micaceous sandy silts, clays, and silty soils in partially drained conditions. These soils may have USCS classifications of either ML or MH and typically have liquid limits (LL) greater than 30. Published laboratory shear strength testing results for Piedmont residual soils (Sabatini, 2002, Appendix A, page A-40) indicate an average effective friction angle of 35.2^{o} with a $\pm$1 standard deviation range of $29.9^{o} < \phi' < 40.5^{o}$. A conservative lower bound of 27.3^{o} is also indicated.

SCDOT experience with borrow materials typically found in Group B are Coastal Plain soils that are typically uniform fine sands that are sometimes difficult to compact and behave similar to silts. When these soils are encountered, caution should be used in selecting effective soil shear strength friction angles since values typically range from $28^{o} < \phi' < 32^{o}$.

7.12.4 Maximum Allowable Soil Shear Strengths Compacted Soils

Maximum acceptable effective soil shear strength parameters (***c'*, *ϕ'***) have been established in Table 7-18. Maximum total shear strength parameters for cohesive soils is 1,500 psf for CL-ML and 2,500 psf for CL and CH. Values outside of these ranges may only be used if the specific source of material is identified for the project and enough material is available for construction. The selection prior to or during design of a specific source of material is anticipated to occur only during design/build projects. A request for exceeding the stated maximums must be made in writing to the PCS/GDS. The PCS/GDS will indicate what testing is required prior to acceptance of exceeding the maximums. Upon receipt of the testing results, the PCS/GDS shall issue a letter to the project team indicating acceptance or rejection of the request for exceeding the range of acceptable range of soil shear strengths.

Table 7-17, Maximum Allowable Soil Shear Strengths For Compacted Soils

Soil Description		Effective	
USCS	Description	c' (psf)	ϕ' (degrees)
GW, GP, GM, GC	Stone and Gravel	0	38
SW	Coarse Grained Sand	0	36
SM, SP	Fine Grained Sand	0	34
SP	Uniform Rounded Sand	0	30
ML, MH, SC	Silt, Clayey Sand, Clayey Silt	50	28
SM-ML	Residual Soil	50	24
CL-ML	Clay (Low Plasticity)	50	32
CL, CH	Clay (Med-High Plasticity)	50	26

7.13 SOIL SETTLEMENT PARAMETERS

Settlements are caused by the introduction of loads (stresses) on to the subsurface soils located beneath a site. These settlements can be divided into two primary categories, elastic and time-dependent settlements (consolidation). Settlements (strains) are a function of the load (stress) placed on the subsurface soils. Elastic settlements typically predominate in the cohesionless soils while time-dependent settlements predominate in cohesive soils. Settlement parameters can be developed from high quality laboratory testing (triaxial shear for elastic parameters and consolidation testing for time-dependent parameters). However, for cohesionless soils, obtaining high quality samples for testing can be extremely difficult. Therefore, in-direct methods (correlations) of measuring the elastic parameters are used. Time-dependent settlement parameters correlations for cohesive soils also exist. These correlations should be used for either preliminary analyses or for evaluating the accuracy of laboratory consolidation testing.

7.13.1 Elastic Parameters

Elastic settlements are instantaneous and recoverable. These settlements are calculated using elastic theory. The determination of elastic settlements is provided in Chapter 17. In the determination of the elastic settlements the elastic modulus, E, (tangent or secant) and the Poisson's ratio, ν, are used. Since E and ν are both dependent of the laboratory testing method (unconfined, confined, undrained, drained), the overconsolidation ratio, water content, strain rate and sample disturbance, considerable engineering judgment is required to obtain reasonable values for use in design. Provided in Table 7-19 are elastic modulus correlations with $N^*_{1,60}$ values. Table 7-20 provides typical values of soil elastic modulus and Poisson's ratio for various soil types.

Table 7-18, Elastic Modulus Correlations For Soil (AASHTO, 2007)

Soil Type	Elastic Modulus, E_s (psi)
Silts, sandy silts, slightly cohesive mixtures	$56N^*_{1,60}$
Clean fine to medium sands and slightly silty sands	$97N^*_{1,60}$
Coarse sands	$139N^*_{1,60}$
Sandy gravels and gravels	$167N^*_{1,60}$

Table 7-19, Typical Elastic Modulus and Poisson Ratio Values For Soil (AASHTO, 2007)

Soil Type	Typical Elastic Modulus Values, E (ksi)	Poisson's Ratio, ν
Clay:		0.4 – 0.5 (Undrained)
Soft sensitive	0.347 – 2.08	
Medium stiff to stiff	2.08 – 6.94	
Very stiff	6.94 – 13.89	
Silt	0.278 – 2.78	0.3 – 0.35
Fine Sand:		0.25
Loose	1.11 – 1.67	
Medium dense	1.67 – 2.78	
Dense	2.78 – 4.17	
Sand:		
Loose	1.39 – 4.17	0.20 – 0.36
Medium dense	4.17 – 6.94	
Dense	6.94 – 11.11	0.30 – 0.40
Gravel:		
Loose	4.17 – 11.11	0.20 – 0.35
Medium dense	11.11 – 13.89	
Dense	13.89 – 27.78	0.30 – 0.40

7.13.2 Consolidation Parameters

Consolidation settlements involve the removal of water from the interstitial spaces between soil grains and the rearrangement of the soil grains. Typically, fine-grained soils (clays and silts) are considered to undergo consolidation settlements. However, sands and gravels may also undergo consolidation settlements. The consolidation settlements in sands and gravels occur very quickly, usually during construction, because of the relatively pervious nature of these materials. Fine-grained soils are typically more impervious and therefore will require more time to settle. Further these soil types may also undergo more settlement than sands and gravels because of the volume of water within these soils. To determine the amount of consolidation settlement that a soil will undergo, the following soil parameters are required: compression, recompression, and secondary compression indices, consolidation coefficient and the preconsolidation pressure. These parameters are normally determined from consolidation testing (see Chapter 5). However, for preliminary estimates and to verify the results of the consolidation testing the correlations listed in the following sections may be used. These correlations should not be used for final design, except where the geotechnical design engineer considers the results of the consolidation testing to be questionable. The engineer shall document the reason for the use of the correlations. In addition, all of the consolidation parameters shall be clearly provided in the geotechnical report.

7.13.2.1 Compression Index

The Compression Index (C_c) has been related to Atterberg Limits by Terzaghi and Peck (1967) and Wroth and Wood (1978). The Terzaghi and Peck formula (Equation 7-46) is limited to inorganic clays with sensitivity up to 4 and a LL less than 100. In addition, NAVFAC (1982) (Equations 7-47 and 7-48) also provides a correlation between C_c and e_o that is applicable to all clays.

$$\mathbf{C_c = 0.009(LL - 10)}$$ Equation 7-46

$$\mathbf{C_c = 0.5G_s\left(\frac{PI}{100}\right)}$$ Equation 7-47

$$\mathbf{C_c = 1.15(e_o - 0.35)}$$ Equation 7-48

Where,

- LL = Liquid Limit (%)
- PI = Plasticity Index (%)
- G_S = Specific gravity of the solids
- e_o = initial void ratio

The Compression Index may also be related to strain as indicated below.

$$\boldsymbol{C_{\varepsilon c} = \frac{C_c}{(1 + e_o)}}$$ Equation 7-49

7.13.2.2 Recompression Index

The Recompression Index (C_r) can be correlated to the C_c values. Ladd (1973) indicates the C_r value is approximately 10 to 20 percent of the C_c value. The Recompression Index may also be related to strain as indicated by the following equation.

$$\boldsymbol{C_{\varepsilon r} = \frac{C_r}{(1 + e_o)}}$$ Equation 7-50

7.13.2.3 Secondary Compression Index

Secondary compression occurs after the completion of elastic and primary consolidation settlements. Secondary compression settlement should be included in the estimate of total settlement for a given project. The amount of secondary compression settlement should be determined. The Secondary Compression Index (C_α) like the other consolidation settlement parameters is best determined from consolidation testing; however, correlations exist that may be used to provide a preliminary estimate of secondary compression settlement. In addition, these correlations may be used to verify the results of the consolidation testing. Provided in Figure 7-30 is a chart of C_α versus the natural moisture content of soil.

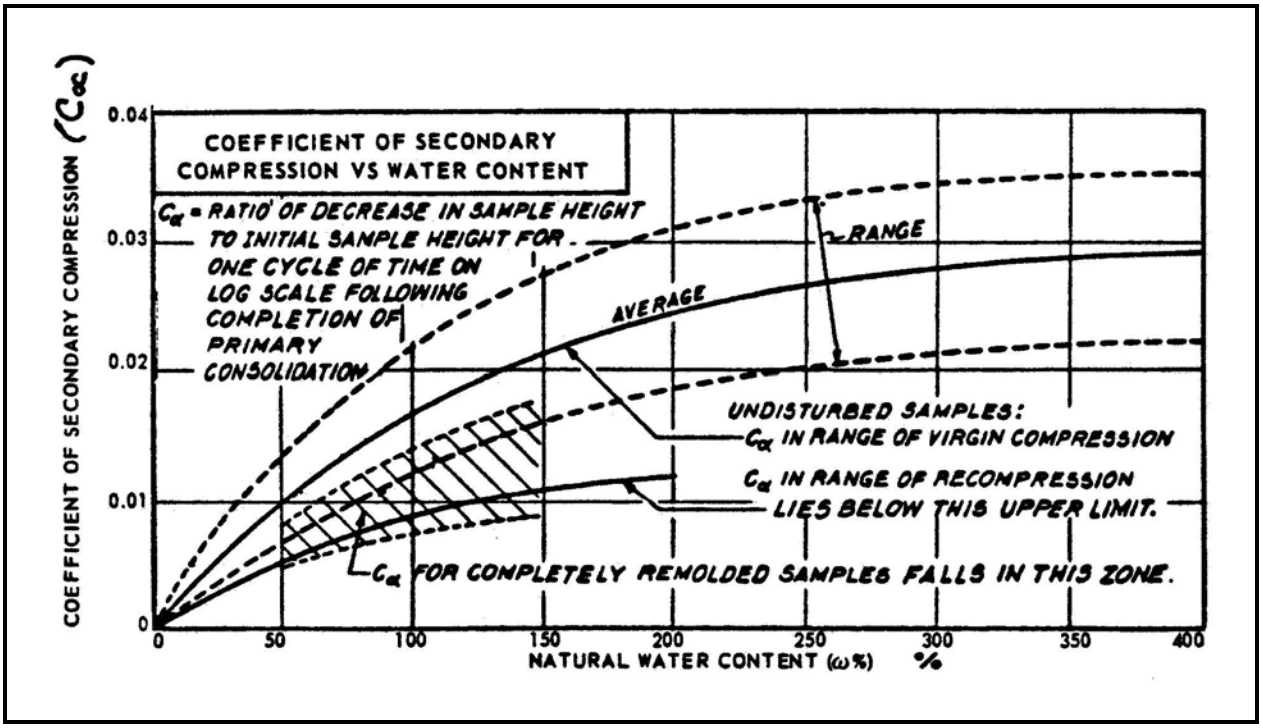


Figure 7-30, Secondary Compression Index Chart
NAVFAC DM-7.1, 1982

The Secondary Compression Index may also be related to strain as indicated below.

$$\mathbf{C}_{\varepsilon\alpha} = \frac{\mathbf{C}_\alpha}{(\mathbf{1} + \mathbf{e_o})}$$ Equation 7-51

7.13.2.4 Consolidation Coefficient

The preceding sections dealt with the amount of settlement that could be anticipated at a project location. This section will provide the methods to estimate the time for consolidation settlement. As indicated previously, elastic settlements are anticipated to occur relatively instantaneously (i.e. during construction) while consolidation settlements are anticipated to occur at some time after the structure has been completed. The rate of consolidation is directly related to the permeability of the soil. As with the consolidation parameters, the Consolidation Coefficient (C_v) should be determined from the results of consolidation testing. Correlations exist that may be used to provide a preliminary estimate of Consolidation Coefficient. In addition, these correlations may be used to verify the results of the consolidation testing. Provided in Figure 7-31 is a chart of C_v versus the Liquid Limit of soil.

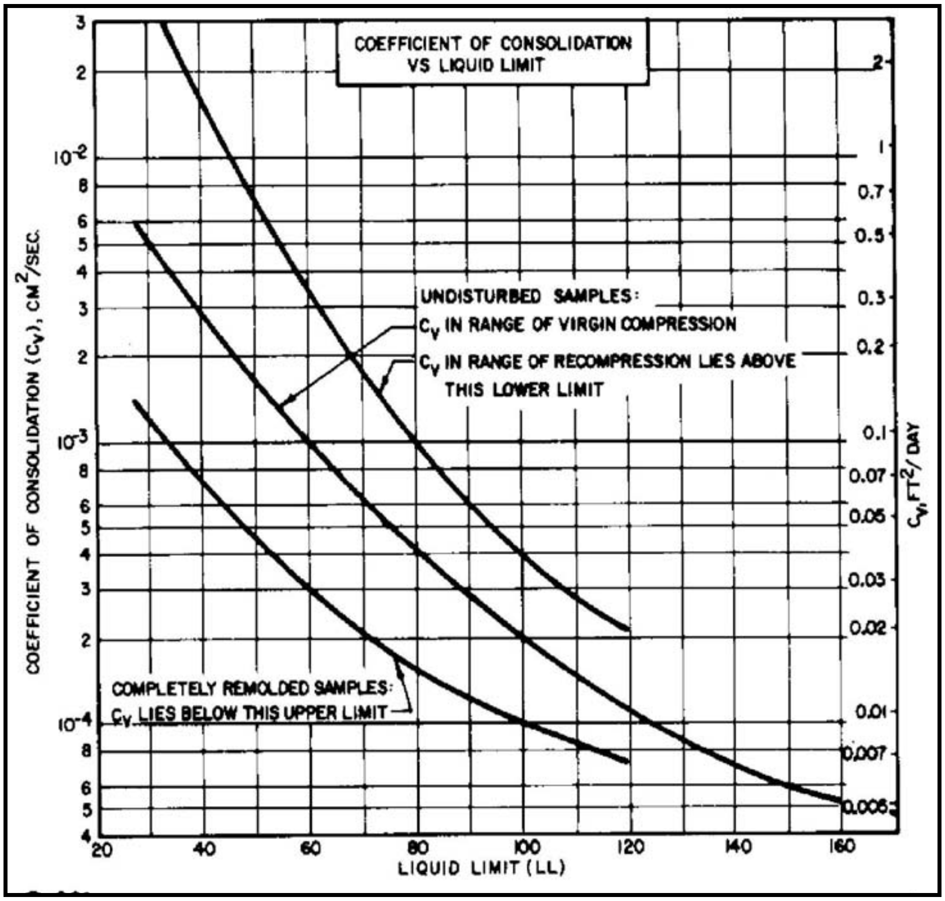


Figure 7-31, Consolidation Coefficient and Liquid Limit Relationship NAVFAC DM-7.1, 1982

7.13.2.5 Effective Preconsolidation Stress

The effective preconsolidation stress (σ'_p or p'_c) in soils is used to determine whether to use the Compression or Recompression Index. The effective preconsolidation stress (σ'_p) is the maximum past pressure that a soil has been exposed to since deposition. Similarly to the other consolidation parameters the σ'_p is best determined from consolidation testing. Correlations also exist; however, these correlations should only be used for either preliminary analyses or for evaluating the accuracy of laboratory consolidation testing. The effective preconsolidation stress (σ'_p or p'_c) can be correlated to total cohesion, c_u (NAVFAC DM-7.1, 1986). As with the other consolidation parameters the correlated σ'_p should be used for preliminary estimates only.

Equation 7-52

$$\sigma'_p = \frac{c_u}{(0.11 + 0.0037PI)}$$

The σ'_p can also be estimated from Cone Penetrometer Testing (CPT) using the following equations (Sabatini, 2002).

$$\sigma'_p = 0.33(q_c - \sigma_v) \qquad \text{Equation 7-53}$$

CPT Piezocone (face element):

$$\sigma'_p = 0.47(u_1 - u_o) \qquad \text{Equation 7-54}$$

CPT Piezocone (shoulder element):

$$\sigma'_p = 0.54(u_2 - u_o) \qquad \text{Equation 7-55}$$

7.14 ROCK PARAMETER DETERMINATION

While the shear strength of individual rock cores is obtained from unconfined axial compression testing, the shear strength of the entire rock mass should be used for design. Therefore, the shear strength and consolidation parameters should be developed using the RMR as defined in Chapter 6.

7.14.1 Shear Strength Parameters

The rock mass shear strength should be evaluated using the Hoek and Brown criteria (AASHTO, 2007). The shear strength of the rock mass is represented by a curved envelope that is a function of the unconfined (uniaxial) compressive strength of the intact rock, q_u, and two dimensionless factors. The rock mass shear strength, τ, (in ksf) is defined as indicated below.

$$\tau = (\cot\phi'_i - \cos\phi'_i)\, m \left(\frac{q_u}{8}\right) \qquad \text{Equation 7-56}$$

$$\phi'_i = \tan^{-1}\left\{4h\cos^2\left[30 + 0.33\sin^{-1}\left(h^{-1.5}\right)\right] - 1\right\}^{-0.5} \qquad \text{Equation 7-57}$$

$$h = 1 + \frac{\left[16\left(m\,\sigma'_n + s\,q_u\right)\right]}{3\,m^2 q_u} \qquad \text{Equation 7-58}$$

Where,

ϕ'_I = instantaneous friction angle of the rock mass (degrees)
q_u = average unconfined rock core compressive strength (ksf)
σ'_n = effective normal stress (ksf)
m and s from Table 7-21

Table 7-20, Constants m and s based on RMR (AASHTO, 2007)

Rock Quality	Constants	**Rock Type:** **A** = Carbonate rocks with well developed crystal cleavage – dolomite, limestone and marble **B** = Lithified argrillaceous rocks – mudstone, siltstone, shale and slate (normal to cleavage) **C** = Areanaceous rocks with strong crystals and poorly developed crystal cleavage – sandstone and quartzite **D** = Fine-grained polyminerallic igneous crystalline rocks – andesite, dolerite, diabase and rhyolite **E** = Coarse-grained polyminerallic igneous and metamorphic crystalline rocks – amphilboltie, gabbro, gneiss, granite, norite, and quartz-diorite				
		A	**B**	**C**	**D**	**E**
Intact rock samples RMR = 100	m s	7.00 1.00	10.00 1.00	15.00 1.00	17.00 1.00	25.00 1.00
Very good quality rock mass RMR = 85	m s	2.40 0.082	3.43 0.082	5.14 0.082	5.82 0.082	8.567 0.082
Good quality rock mass RMR = 65	m s	0.575 0.00293	0.821 0.00293	1.231 0.00293	1.395 0.00293	2.052 0.00293
Fair quality rock mass RMR = 44	m s	0.128 0.00009	0.183 0.00009	0.275 0.00009	0.311 0.00009	0.458 0.00009
Poor quality rock mass RMR = 23	m s	0.029 $3*10^{-6}$	0.041 $3*10^{-6}$	0.061 $3*10^{-6}$	0.069 $3*10^{-6}$	0.102 $3*10^{-6}$
Very poor quality rock mass RMR = 3	m s	0.007 $1*10^{-7}$	0.010 $1*10^{-7}$	0.015 $1*10^{-7}$	0.017 $1*10^{-7}$	0.025 $1*10^{-7}$

7.14.2 Elastic Parameters

Rocks will primarily undergo elastic settlements. The elastic settlements will be instantaneous and recoverable. These settlements are calculated using elastic theory. The determination of elastic settlements is provided in Chapter 17. In the determination of the elastic settlements, the elastic modulus, E, is required. The elastic modulus of a rock mass is the lesser of modulus determined from intact rock core testing or from the equations below (AASHTO, 2007).

$$E_m = 145\left(10^{\left(\frac{RMR-10}{40}\right)}\right) \qquad \text{Equation 7-59}$$

$$E_m = \left(\frac{E_m}{E_i}\right)E_i \qquad \text{Equation 7-60}$$

Where,

E_m = elastic modulus of rock mass (ksi)
E_i = elastic modulus of intact rock (ksi)
RMR = Adjusted Rock Mass Rating from Chapter 6

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