Cone Penetration Test Design Parameter Correlations

This module provides Cone Penetration Test (CPT) semi-empirical correlations for geotechnical design parameters. Appropriate usage of the CPT and correlations for design depends on several factors that are discussed in this module. A brief overview of CPTs is also included.

Overview of CPT

The CPT is a fast and reliable in-situ penetration testing method used to assess subsurface stratigraphy, and interpret engineering properties of soils such as density, undrained shear strength, and effective friction angle. The CPT involves pushing an instrumented electronic penetrometer into soil and soft ground and recording multiple measurements continuously with depth. Per the ASTM D-5778 Field Test Procedure, measurements of tip resistance (q_c), sleeve friction (f_s), and porewater pressure (u) are obtained with depth.

The electronic friction cone (ECPT) is the standard type of cone penetrometer. The axial load is measured over the cone tip area, giving the tip resistance (q_c) and resistance over a side area giving the sleeve friction (f_s). With the addition of porous filters and transducers, the penetration porewater pressures (u) in saturated soils can be measured, thus termed a piezocone penetration test (CPT_u). The seismic piezocone ($SCPT_u$) contains geophones to permit the profiling of shear wave velocity measurements. Table 1 summarizes various types of CPTs commonly available.

Table 1: Common Types of Cone Penetration Tests

Type of CPT	q_c	f_s	u	q_t	V_s	Applications
Electric Friction Cone (ECPT)	x	x				Fill placement, Natural sands, Soils above the groundwater table
Piezocone Penetration Test (CPT_u)	x	x	x	x		All soil types. Requires u_2 for correction of q_c to q_t
Seismic Piezocone Test ($SCPT_u$)	x	x	x	x	x	Provides fundamental soil stiffness with depth

q_c = measured point stress or cone tip resistance
f_s = measured sleeve friction
u = penetration porewater pressure (u_1 at face; u_2 at shoulder)
q_t = total cone resistance
V_s = shear wave velocity.

Below is a summary of advantages and disadvantages of the CPT.

Advantages:

- Fast and continuous profiling
- Repeatable and reliable data (not operator-dependent)
- Economical and productive
- Strong theoretical basis for interpretation
- No spoils generated

Disadvantages:

- Skilled operators required
- Limited soil samples (for laboratory tests)
- Potentially limited exploration depth (due to the pushing limitations of the equipment and/or presence of boulder/cobble/gravel/cemented layers)

Geotechnical Investigation using CPT

Selection of appropriate geotechnical investigation methods should be in accordance with the *Geotechnical Investigation*s module. When the CPT is selected to supplement soil borings:

- Project-specific calibration and verification of CPT correlations for geotechnical design parameters is required. The calibration boring should be located within 10 feet of the CPT sounding.
- Planning of more CPTs with a few conventional borings is an efficient investigation strategy.
- Consider the variation of reliability of CPT correlations for geotechnical design parameters.

CPT Applicability and Correlation Reliability

The CPT provides continuous, repeatable, and more reliable data (not operator-dependent) and in turn better soil stratigraphic profiles and soil characteristics in terms of soil behavior compared to conventional borings. However, the correlations developed to estimate geotechnical parameters have evolved over years and have varying reliability and applicability for various soil types.

The perceived applicability and reliability of the CPT is discussed in “Guide to Cone Penetration Testing or Geotechnical Engineering” by P.K. Robertson and is summarized for two main soil types, coarse (granular behavior) and fine (cohesive behavior) grained soils in Table 2 below.

Table 2: Perceived Applicability of CPT for Deriving Geotechnical Parameters (P.K. Robertson 2018)

Soil Type	D_r	Ψ	K_o	OCR	S_t	s_u	ϕ'	E, G*	M	G_0*	k	c_h
Coarse-grained (sand)	2-3	2-3	5	5	N/A	N/A	2-3	2-3	2-3	2-3	3-4	3-4
Fine-grained (clay)	N/A	N/A	2	1	2	1-2	4	2-4	2-3	2-4	2-3	2-3

Reliability: 1=high, 2=high to moderate, 3=moderate, 4=moderate to low, 5=low
*Reliability improved with Seismic Piezocone Test ($SCPT_u$)

Where:

D_r	Relative density	ϕ'	Peak effective friction angle
Ψ	State Parameter	E, G	Young's and Shear moduli
K_0	In-situ stress ratio	M	1-D Compressibility
OCR	Over consolidation ratio	G_0	Small strain shear moduli
S_t	Sensitivity	k	Permeability
s_u	Undrained shear strength	c_h	Coefficient of consolidation

This module presents only correlations for more commonly used strength parameters with high to moderate reliability levels and preliminary consolidation settlement related parameters. Parameters with low correlation reliability levels should not be used for final design.

CPT Correlation with Geotechnical Parameters

CPT correlations presented in this section are based on semi-empirical correlations using in-situ, laboratory and chamber test results, and the inversion of theoretical equations with some modification.

Below are CPT parameters used as inputs for correlations and/or determination of soil behavior type. Interpretation, correction, and normalization of the CPT raw data are required as defined below to use the correlations presented in this module. Software programs that directly produce correlated geotechnical parameters can be used but should be consistent with the equations recommended in this module.

CPT Parameters and Definition:

1. Cone resistance, $q_c = Q_c/A_c$
   Where, $Q_c$is the force acting on the cone and A_c is the projected area of the cone.

2. Corrected cone resistance, $q_t = q_c + u_2(1 - a_{net})$
   Where, u_2 is the measured pore water pressure at base of sleeve, just behind the cone and a_{net} is the net area ratio defined by Campanella and Robertson (1988) and determined from laboratory calibration with a typical value between 0.7 and 0.85. In the absence of u_2 such as in sandy soil, $q_c = q_t$.

3. Friction ratio, $R_f(\%) = \left(\frac{f_s}{q_t}\right) 100$

   Where, f_s is the sleeve friction resistance

4. Normalized cone resistance, $Q_{t1} = (q_t - \sigma_{v0})/\sigma'_{v0}$
   Where, σ'_{v0} is in-situ effective vertical overburden stress and σ_{v0} is in-situ total vertical overburden stress.

5. Normalized friction ratio, $F_r = \left(\frac{f_s}{q_t - \sigma_{v0}}\right) 100\ \%$

6. Normalized cone resistance, Q_{tn}

   $$Q_{tn} = \left(\frac{q_t - \sigma_{vo}}{Pa}\right)\left(\frac{Pa}{\sigma'_{vo}}\right)^n$$

   Where, Pa is atmosphere pressure (=101.3 Kpa/1.06 tsf) and n is stress exponent that is typically taken as 1.0 in clay soil and loose sands and less than 0.5 in dense sand.

7. Soil Behavior Type Index, $I_c = [(3.47 - logQ_{tn})^2 + (logF_r + 1.22)^2\ ]^{0.5}$

8. Stress exponent, n

   $$n = 0.381(I_c) + 0.05\left(\sigma'_{v0}/P_a\right) - 0.15$$

9. Normalized net pore pressure ratio, B_q.

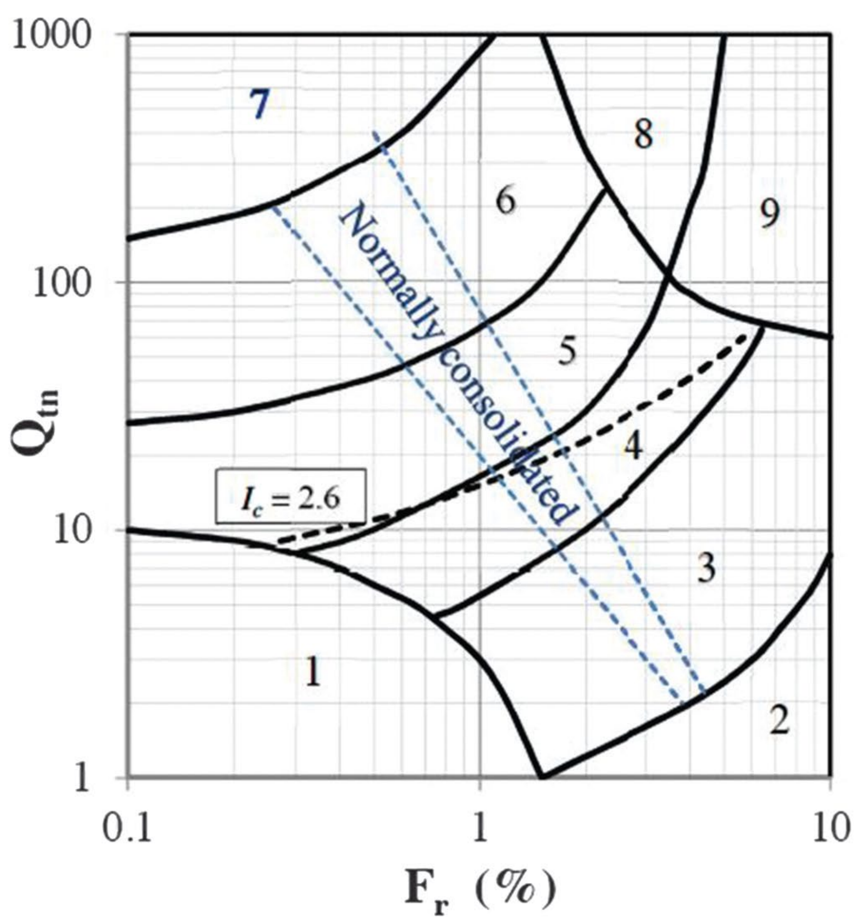
   $B_q = \frac{u_2 - u_0}{q_n} = \frac{\Delta u}{q_n}$

   Where u_0 is steady state water pressure and $q_n = q_t - \sigma_{vo}$

10. Soil Behavior Type (SBTn) and Chart by Robertson (1990)

I_c Range	SBTn Zone	Common SBTn Description
---	1	Sensitive fine-grained
I_c > 3.60	2	Clay – organic soil
2.95 < I_c< 3.60	3	Clays – clay to silty clay
2.60 < I_c< 2.95	4	Silt mixtures – clayey silt & silty clay
2.05 < I_c< 2.60	5	Sand mixtures – silty sand to sandy silt
1.31 < I_c< 2.05	6	Sands – clean sands to silty sands
I_c < 1.31	7	Dense sand to gravelly sand
---	8	Stiff sand to clayey sand*
---	9	Stiff fine-grained*

* Heavily overconsolidated or cemented



Normalized Sleeve Ratio (F_r) versus Normalized Cone Resistance (Q_{tn})
Robertson (1990)

Equivalent Standard Penetration Test (SPT) Blow Counts, N_{60} and $N_{1(60)}$

Use the following correlations to convert CPT data to SPT Data.

$$\frac{\left(\frac{q_t}{P_a}\right)}{N_{60}} = \frac{(Q_{tn})}{N_{1(60)}} = 10^{(1.1268-0.2817 I_c)}$$ (Robertson, 2012)

Effective Friction Angle, ϕ' (Coarse-Grained Soil, SBTn = 5, 6, 7, and 8)

Use the following correlations to convert CPT data to effective friction angle for applicable soil types.

- $\tan \phi' = \frac{1}{2.68}\left[log\left(\frac{q_c}{\sigma'_{v0}}\right) + 0.29\right]$ for uncemented, unaged, moderately compressible, predominately quartz sand (Robertson and Campanella, 1983a)
- $\phi' = 17.6 + 11 log Q_{tn}$ for clean, rounded, uncemented quartz sand (Kulhawy and Mayne, 1990)
- $\phi' = \phi'_{cv} + 15.84 log Q_{(tn,cs)} - 26.88$ (Robertson, 2010)

  Where,

  ϕ'_{cv} is dependent on mineralogy (typically 33 degrees for quartz sand to 40 degrees for felspathic sand)

  $Q_{(tn,cs)} = K_c Q_{tn}$.

  K_c = 1.0 for $I_c \leq 1.64$

  $K_c = -0.403 I_c^4 + 5.581 I_c^3 - 21.63 I_c^2 + 33.75 I_c - 17.88$ for $I_c > 1.64$.

- Convert coarse-grained CPT data to $N_{1(60)}$ using the correlation presented in the previous section and then correlate $N_{1(60)}$ to effective friction angle using the correlations presented in the Soil Correlations Module for the appropriate soil type. This method provides flexibility in evaluating various types of coarse-grained soils (SC, SM, SP, GC, etc.).

For reference, a comparison of friction angle values derived from CPT and SPT data from a Caltrans project is shown in the Appendix (references 4 and 5).

Undrained Shear Strength, S_u (Fine-Grained Soil, SBTn = 1, 2, 3, and 4)

Use the following correlations to convert CPT data to undrained shear strength.

$\left(\frac{S_u}{\sigma'_{v0}}\right) = \left(\frac{q_t-\sigma_{v0}}{\sigma'_{v0}}\right)\left(\frac{1}{N_{kt}}\right) = \frac{Q_{t1}}{N_{kt}}$ (Robertson, 2009)

$S_u = \frac{q_t-\sigma_v}{N_{kt}}$ (Robertson, 2009)

$N_{kt} = 10.5 + 7logF_r$ (Robertson, 2012)

Note that N_{kt} varies from 10 to 20 with a typical value of 14. N_{kt} less than 14 is generally not recommended for use unless a project specific calibration is done with laboratory tested undrained shear strength values.

For reference, a comparison of undrained shear strength values derived from CPT data, and laboratory and field testing from a Caltrans project is shown in the Appendix (references 4 and 5).

Shear Wave Velocity, V_s

Although the direct measurement of shear-wave velocity such as from $SCPT_u$ is always preferred over the CPT correlation, the correlations can be used for seismic analysis of small and low-risk projects when direct measurements are not available. For shear wave velocity correlated to CPT, which is typically used for seismic acceleration response analysis, refer to the *Design Acceleration Response Spectrum (ARS)* module.

Drained Young's Modulus, E' (Coarse-Grained Soil, SBTn = 5, 6, 7, and 8)

Use the following correlation for Young's modulus for uncemented silica-based coarse-grained soil with $I_c < 2.60$.

$E' = 0.015[10^{0.55I_c+1.68}](q_t - \sigma_{v0})$ (Robertson, 2009)

Above equation is derived based on the following correlations and assumptions:

- a loading level $\left(\frac{q}{q_{ult}}\right)$ of 0.2 to 0.3 (average factor of safety of about 4 in terms of bearing capacity)
- $G_0 = \left(\frac{\gamma}{g}\right)V_s^2 = \alpha_G(q_t - \sigma_{v0})$
- $\alpha_G = \left(\frac{\rho}{P_a}\right)\alpha_{vs} = 0.0188[10^{0.55I_c+1.68}]$ with average unit weight, $\gamma = 18\ kN/m^3$ (ρ = 1.84).

- $E' = 2(1+\upsilon)G = \sim 2.5G$ with Poisson's ratio, υ ranging from 0.1 to 0.3 (typical for most soils under drained conditions)
- $\frac{G}{G_0} = 1 - f\left(\frac{q}{q_{ult}}\right)^g = \sim 0.3\ to\ 0.38$ and $G = 0.8G_0$ with f = 1 and g = 0.3 for uncemented soils and loading level from 0.2 to 0.3

For a loading level outside the range of 0.2 to 0.3, use the following relationship.

$$E' = 0.047\left[1 - \left({}^{q}/_{q_{ult}}\right)^{0.3}\right]\left[10^{(0.55I_c+1.68)}\right](q_t - \sigma_{v0})$$

Constrained Modulus, *M*

Use the following correlation for constrained modulus.

$M = \alpha_M(q_t - \sigma_{v0})$ (Robertson, 2009)

Where, α_M is the constrained modulus cone factor, derived as:

when $I_c > 2.2$,

$\alpha_M = Q_{tn}$ when $Q_{tn} < 14$
$\alpha_M = 14$ when $Q_{tn} \geq 14$

when $I_c < 2.2$,

$\alpha_M = 0.03\left[10^{(0.55I_c+1.68)}\right]$

Compression Index, *Cc* or C_r (Fine-Grained Soil, SBTn = 2, 3, and 4)

Use the following correlation for compression or recompression indices ($I_c > 2.2$). The compression index from undrained cone penetration will be approximate and should be used for preliminary analysis only. Additional laboratory consolidation tests should be performed to calibrate and verify the correlation for use in final design.

$C_{c/r} = 2.3(1+e_0)/(Q_{t1})^2$ when $Q_{t1} < 14$ (Robertson, 2012)
$C_{c/r} = 2.3(1+e_0)/(14Q_{t1})$ when $Q_{t1} \geq 14$ (Robertson, 2012)

The above equations were derived from the following relationships between 1-D consolidation modulus and CPT-Constrained Modulus relation presented previously.

$$M = \alpha_M(q_t - \sigma_{v0}) = \frac{1}{m_v} = \delta\sigma_v/\delta\varepsilon = 2.3(1+e_0)\sigma'_{v0}/C_{c/r}$$

Pre-Consolidation Stress and Over-Consolidation ratio, OCR (Fine-Grained Soil, SBTn = 1, 2, 3, and 4)

Use the following correlations for OCR and pre-consolidation stress, σ'_p. Although the reliability level of this correlation is defined as high (Table 2), additional laboratory consolidation tests are recommended to verify its use for final design.

$\sigma'_p = k(q_t - \sigma_{v0})$ (Robertson, 2009)

$OCR = \frac{\sigma'_p}{\sigma'_{v0}} = k\left(\frac{(q_t - \sigma_{v0})}{\sigma'_{vo}}\right) = kQ_{t1}$ (Robertson, 2009)

Where, the pre-consolidation cone factor, k varies from 0.2 to 0.5 with an average value of 0.33 (Robertson, 2009). The higher values are recommended for aged, heavily over-consolidated clays. Robertson (2012) also proposed the following relationship for k.

$$k = \left[\frac{Q_{t1}^{0.2}}{0.25(10.5 + 7\log{(F_r)}}\right]^{1.25}$$

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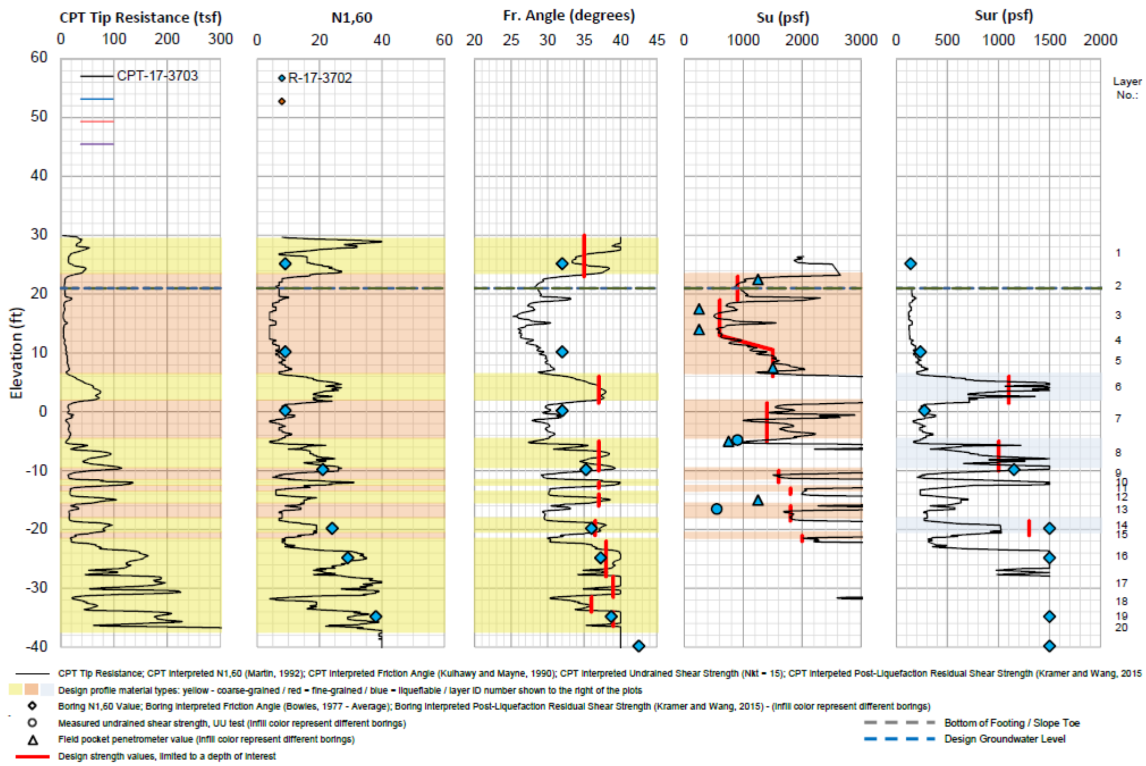
APPENDIX

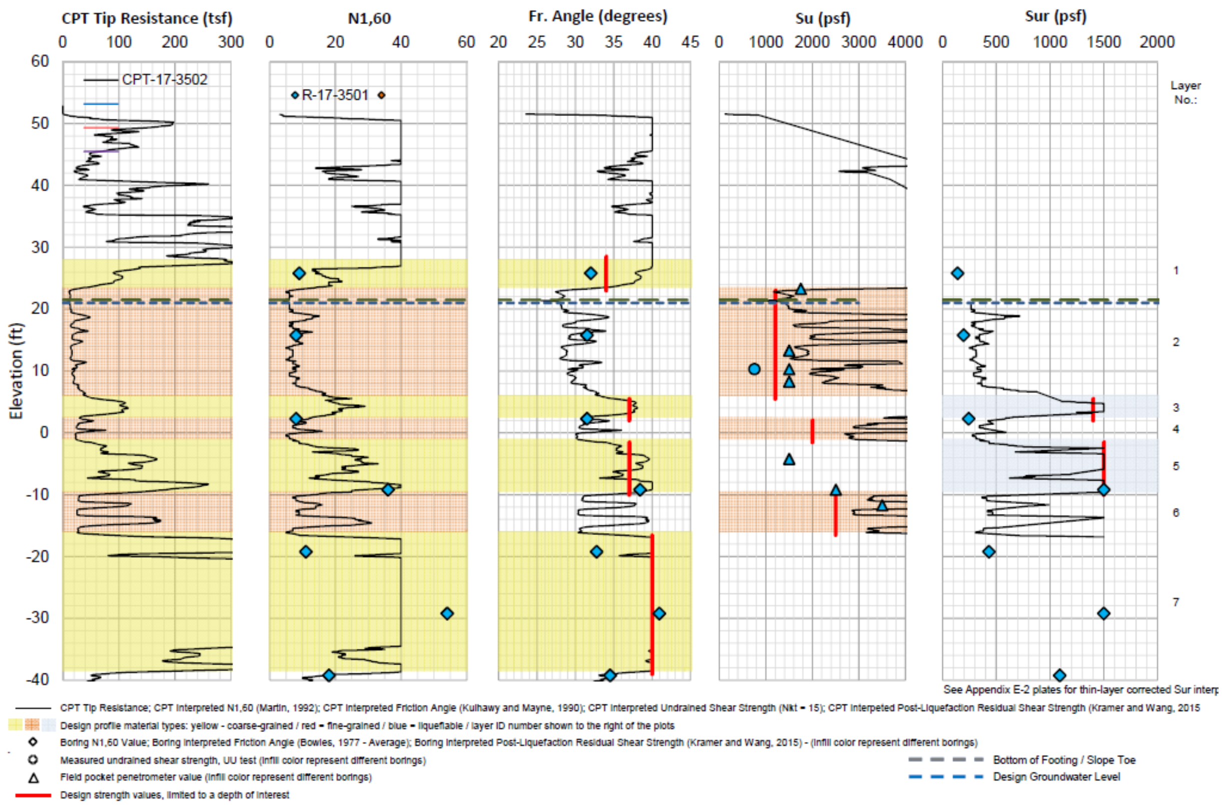
Comparison of Measured (Strength) Parameters and CPT Correlated Parameters

EA No. 12-0H1004/Bridge No. 55-1119



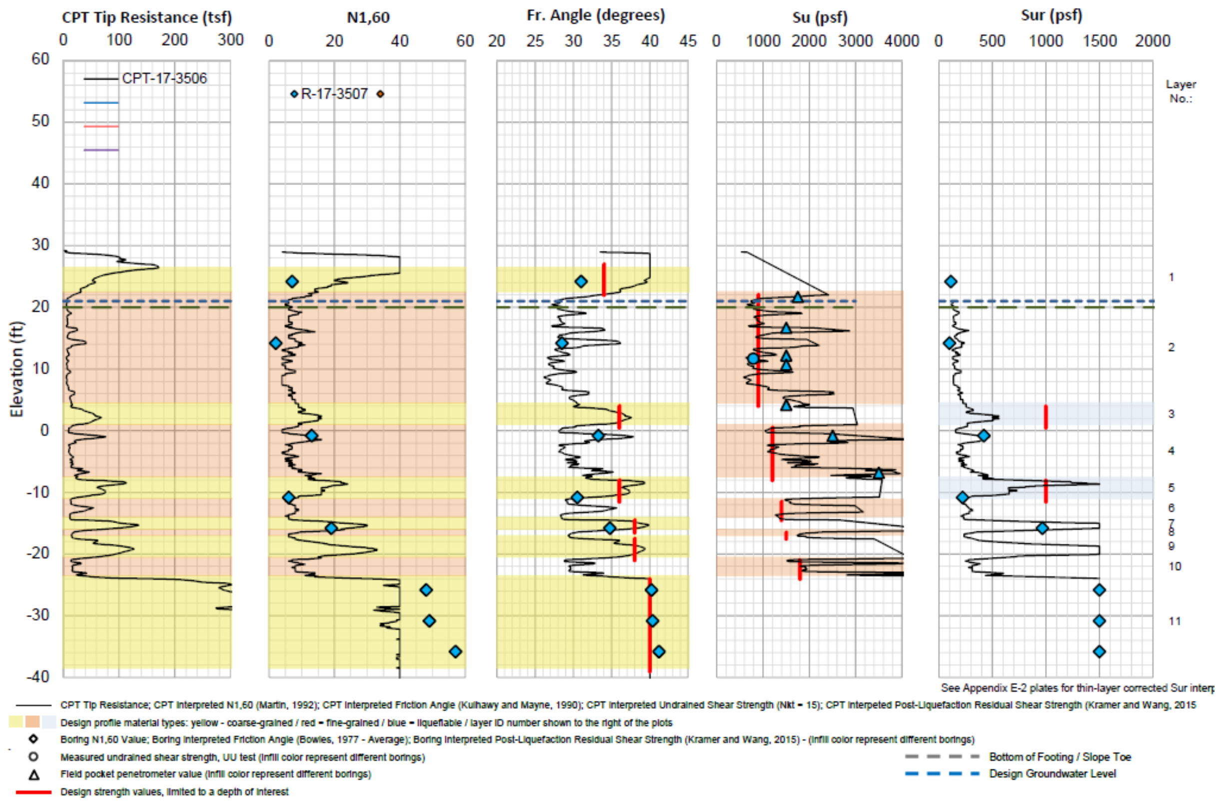
GENERALIZED SOIL PROFILE - Goldenwest St Overcrossing
CPT-17-3703, R-17-3702
I-405 Improvement Project
Orange County, California

EA No. 12-0H1004/Bridge No. 55-1130



GENERALIZED SOIL PROFILE - Bolsa Ave Overcrossing (LPILE Analysis)
CPT-17-3502, R-17-3501
I-405 Improvement Project
Orange County, California

EA No. 12-0H1004/Bridge No. 55-1130



GENERALIZED SOIL PROFILE - Bolsa Ave Overcrossing (LPILE Analysis)
CPT-17-3506, R-17-3507
I-405 Improvement Project
Orange County, California