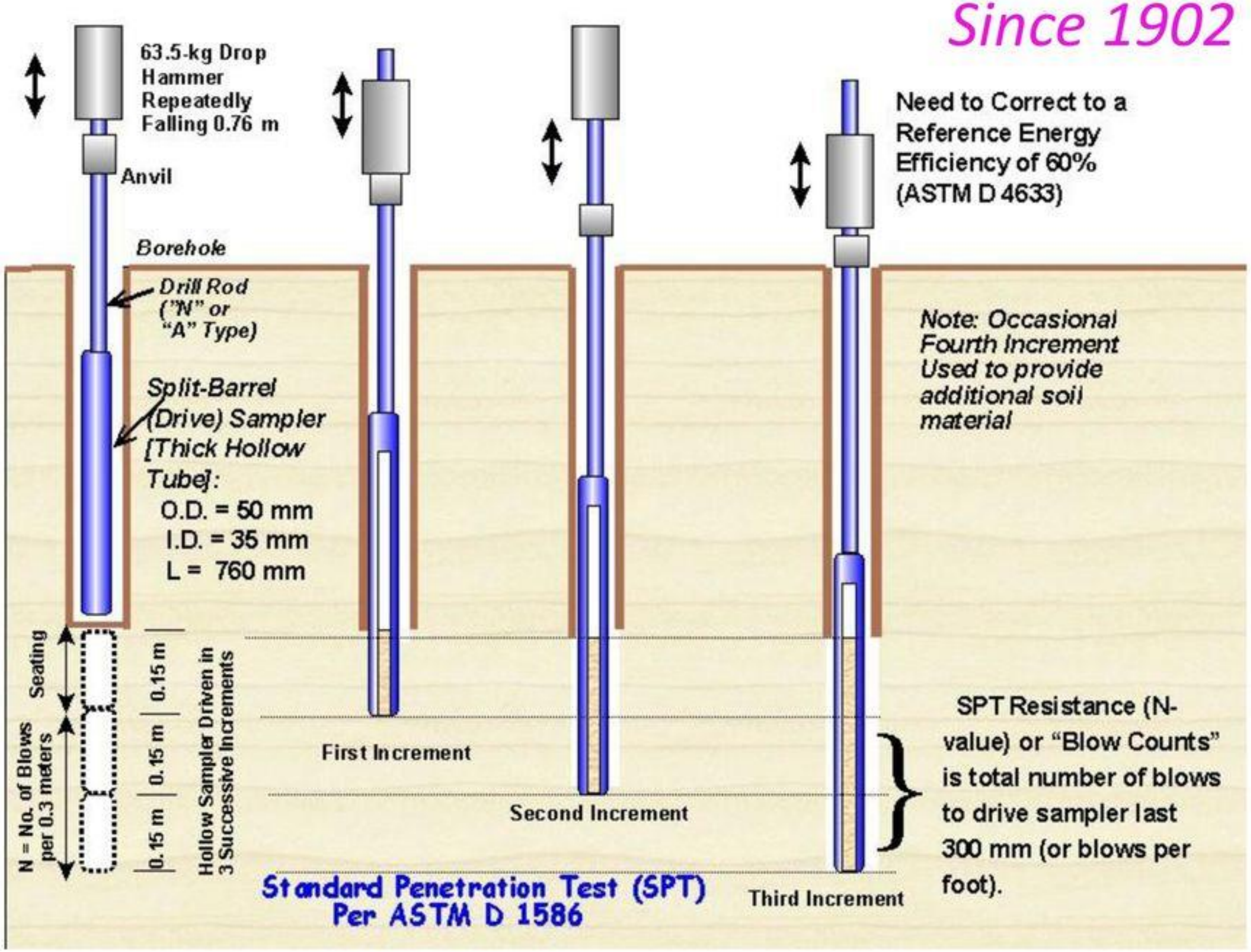
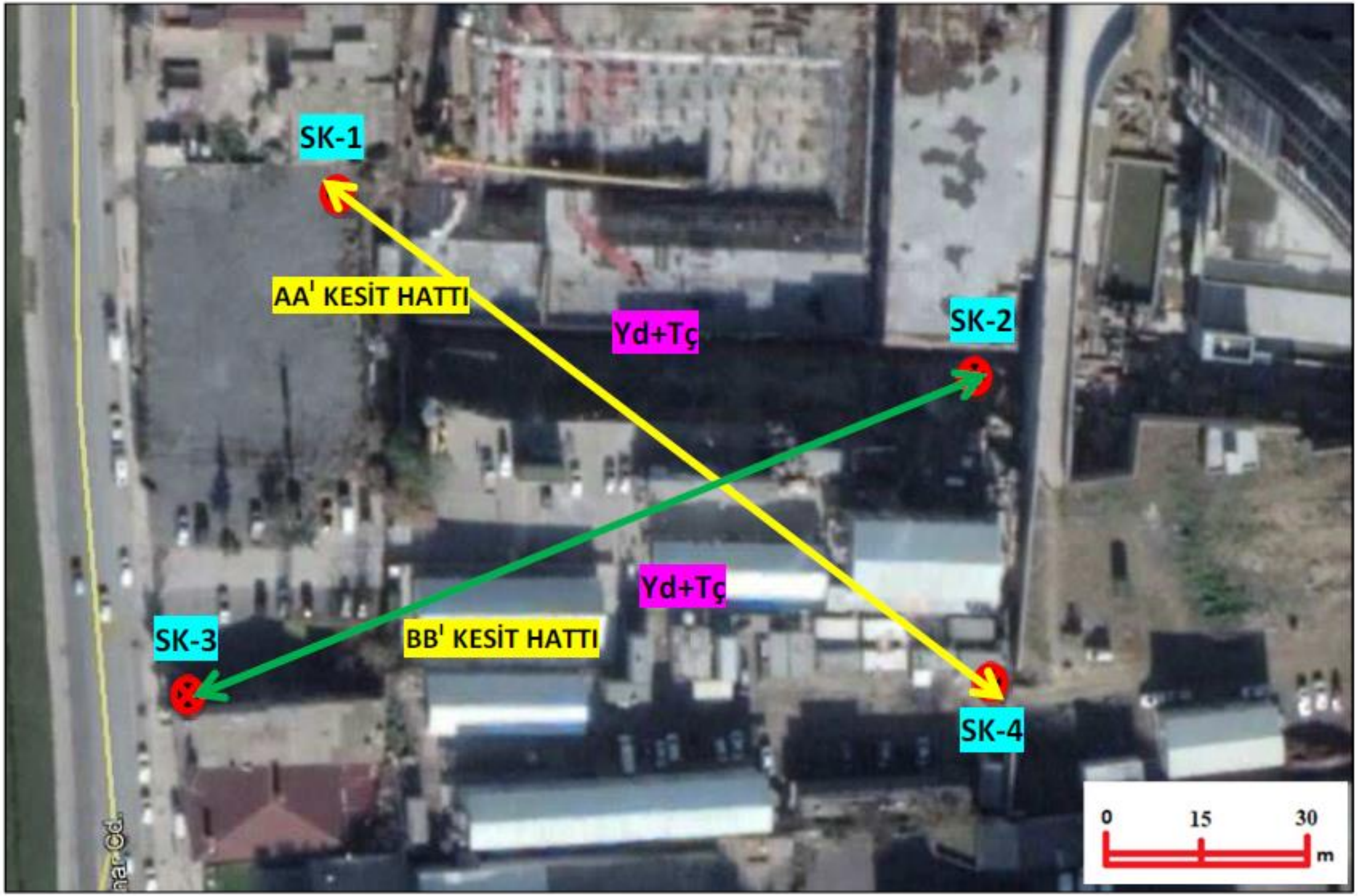
GEOTECHNICAL FIELD TEST CORRELATIONS

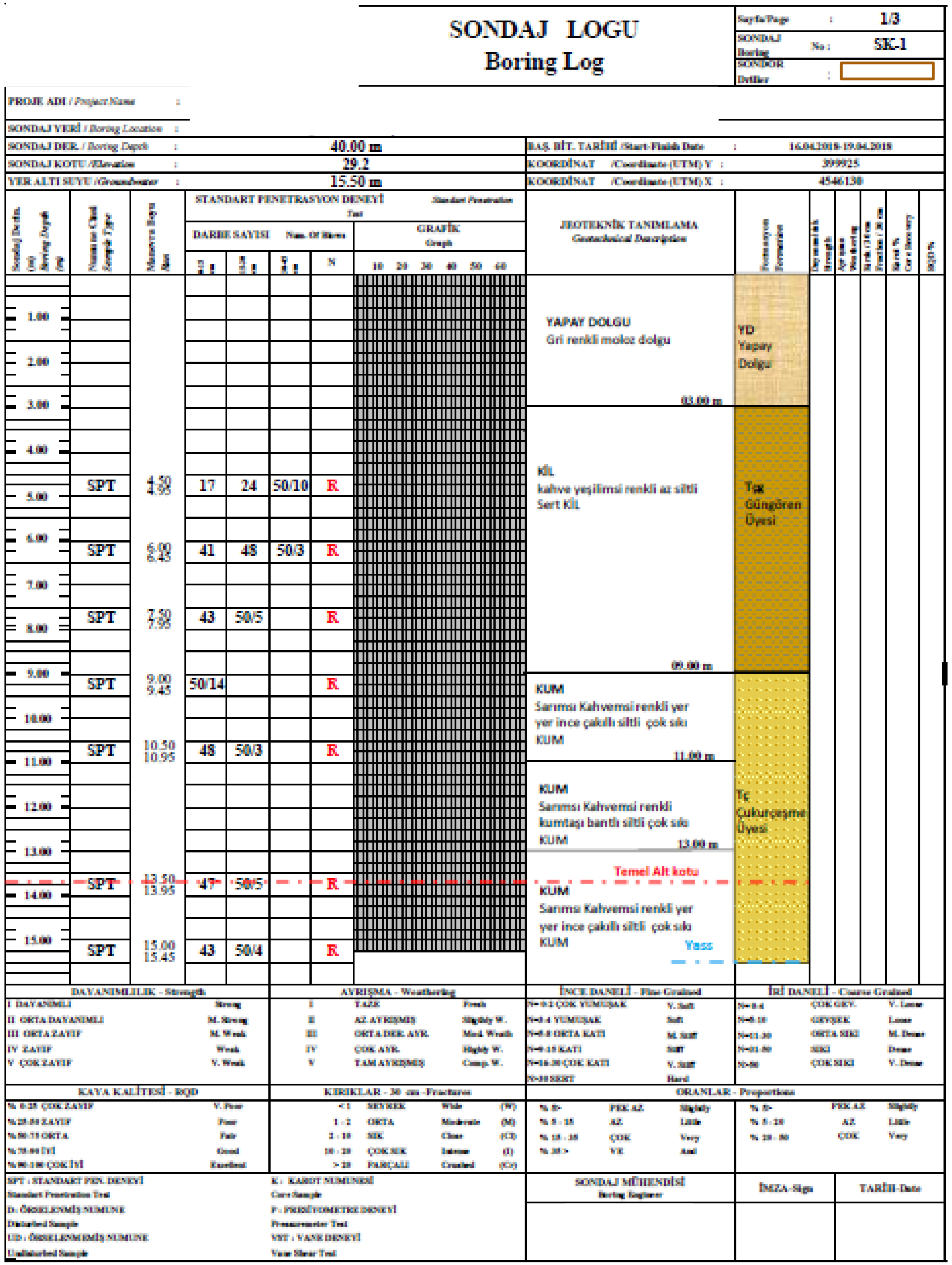
SPT – CPT – VANE SHEAR

Standard Penetration Test (SPT)

Since 1902




SK-1
AA' KESİT HATTI
Yd+Tç
SK-2
Yd+Tç
SK-3
BB' KESİT HATTI
SK-4
0
15
30
m



SONDAJ LOGU
Boring Log

Sayfa/Page	:	1/3
SONDAJ Boring	No :	SK-1
SONDÖR Driller	:	

PROJE ADI / *Project Name* :

SONDAJ YERİ / *Boring Location* :

SONDAJ DER. / *Boring Depth* :	40.00 m	BAŞ. BİT. TARİHİ /Start-Finish Date	:	16.04.2018-19.04.2018
SONDAJ KOTU /*Elevation* :	29.2	KOORDİNAT /Coordinate (UTM) Y	:	399925
YER ALTI SUYU /*Groundwater* :	15.50 m	KOORDİNAT /Coordinate (UTM) X	:	4546130

STANDART PENETRASYON DENEYİ *Standart Penetration Test*

Numune Cinsi *Sample Type*	Manevra Boyu	DARBE SAYISI Num. Of Blows 0-15 cm	15-30 cm	30-45 cm	N
SPT	4.50 4.95	17	24	50/10	R
SPT	6.00 6.45	41	48	50/3	R
SPT	7.50 7.95	43	50/5		R
SPT	9.00 9.45	50/14			R
SPT	10.50 10.95	48	50/3		R
SPT	13.50 13.95	47	50/5		R
SPT	15.00 15.45	43	50/4		R

JEOTEKNİK TANIMLAMA *Geotechnical Description*	Formasyon Formation
YAPAY DOLGU Gri renkli moloz dolgu 03.00 m	YD Yapay Dolgu
KİL kahve yeşilimsi renkli az siltli Sert KİL 09.00 m	Tgç Güngören Üyesi
KUM Sarımsı Kahvemsi renkli yer yer ince çakıllı siltli çok sıkı KUM 11.00 m	Tç Çukurçeşme Üyesi
KUM Sarımsı Kahvemsi renkli kumtaşı bantlı siltli çok sıkı KUM 13.00 m	
Temel Alt kotu KUM Sarımsı Kahvemsi renkli yer yer ince çakıllı siltli çok sıkı KUM Yass	

DAYANIMLILIK - Strength		AYRIŞMA - Weathering		
I DAYANIMLI	Strong	I	TAZE	Fresh
II ORTA DAYANIMLI	M. Strong	II	AZ AYRIŞMIŞ	Slightly W.
III ORTA ZAYIF	M. Weak	III	ORTA DER. AYR.	Mod. Weath
IV ZAYIF	Weak	IV	ÇOK AYR.	Highly W.
V ÇOK ZAYIF	V. Weak	V	TAM AYRIŞMIŞ	Comp. W.

İNCE DANELİ - Fine Grained		İRİ DANELİ - Coarse Grained		
N= 0-2 ÇOK YUMUŞAK	V. Soft	N= 0-4	ÇOK GEV.	V. Loose
N=3-4 YUMUŞAK	Soft	N=5-10	GEVŞEK	Loose
N=5-8 ORTA KATI	M. Stiff	N=11-30	ORTA SIKI	M. Dense
N=9-15 KATI	Stiff	N=31-50	SIKI	Dense
N=16-30 ÇOK KATI	V. Stiff	N>50	ÇOK SIKI	V. Dense
N>30 SERT	Hard			

KAYA KALİTESİ - RQD		KIRIKLAR - 30 cm -Fractures			
% 0-25 ÇOK ZAYIF	V. Poor	< 1	SEYREK	Wide	(W)
%25-50 ZAYIF	Poor	1 - 2	ORTA	Moderate	(M)
%50-75 ORTA	Fair	2 - 10	SIK	Close	(Cl)
%75-90 İYİ	Good	10 - 20	ÇOK SIK	Intense	(I)
%90-100 ÇOK İYİ	Excellent	> 20	PARÇALI	Crushed	(Cr)

ORANLAR - Proportions

% 5-	PEK AZ	Slightly	% 5-	PEK AZ	Slightly
% 5 - 15	AZ	Little	% 5 - 20	AZ	Little
% 15 - 35	ÇOK	Very	% 20 - 50	ÇOK	Very
% 35 >	VE	And			

SPT : STANDART PEN. DENEYİ
Standart Penetration Test
D : ÖRSELENMİŞ NUMUNE
Disturbed Sample
UD : ÖRSELENMEMİŞ NUMUNE
Undisturbed Sample

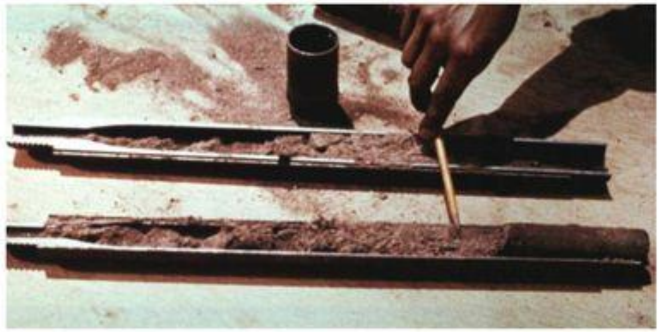
K : KAROT NUMUNESİ
Core Sample
P : PRESİYOMETRE DENEYİ
Pressuremeter Test
VST : VANE DENEYİ
Vane Shear Test

SONDAJ MÜHENDİSİ Boring Engineer	İMZA-Sign	TARİH-Date

Standard Penetration Test

Advantages

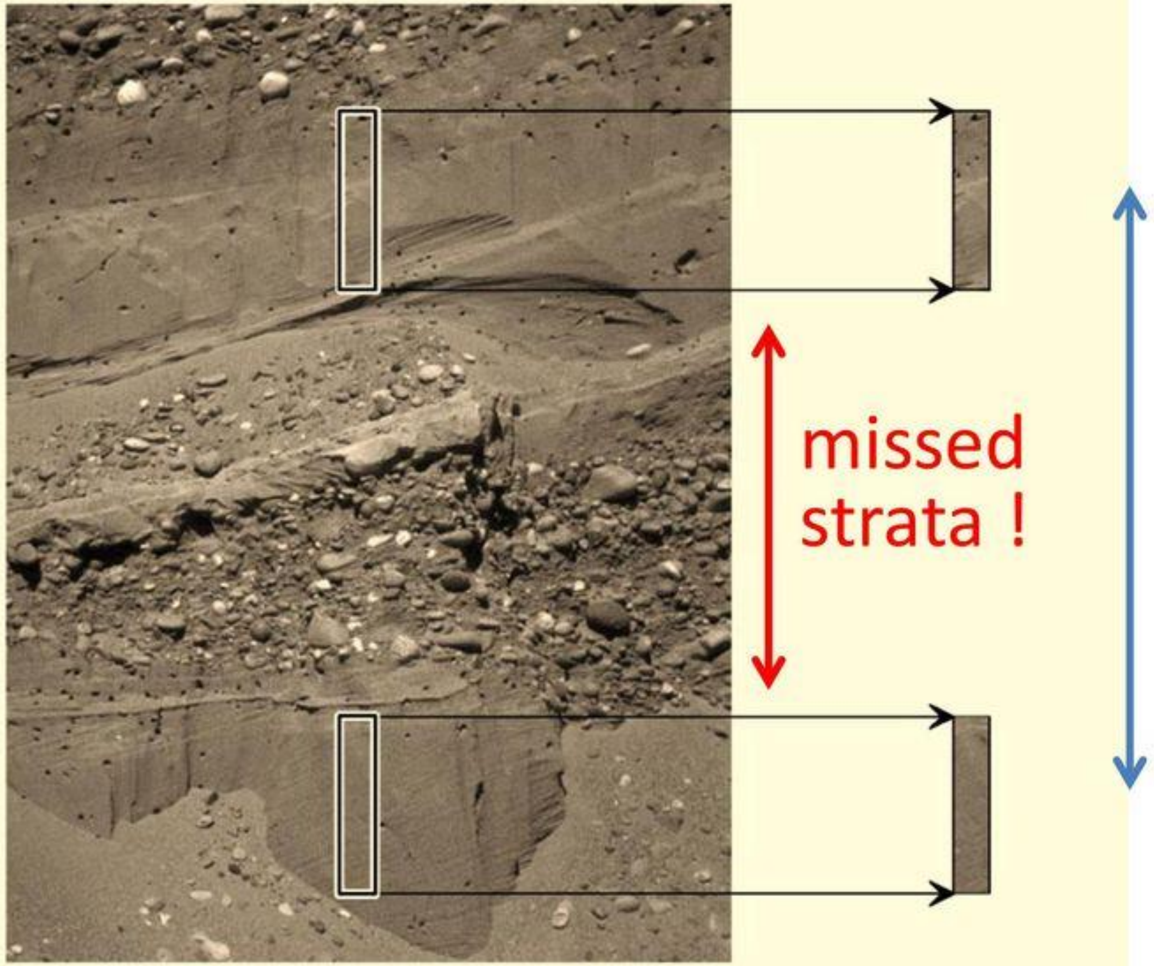
- Obtain Sample + Number
- Simple & rugged device at low cost
- Suitable in many soil types
- Can perform in weak rocks
- Available (worldwide)



Disadvantages

- Obtain Sample + Number
- Energy inefficiency problems
- Discontinuous - only taken every 5 feet (1.5 m)
- Disturbed sample (index tests only)
- Crude number for analysis
- Not applicable in soft clays and silts
- High variability and uncertainty

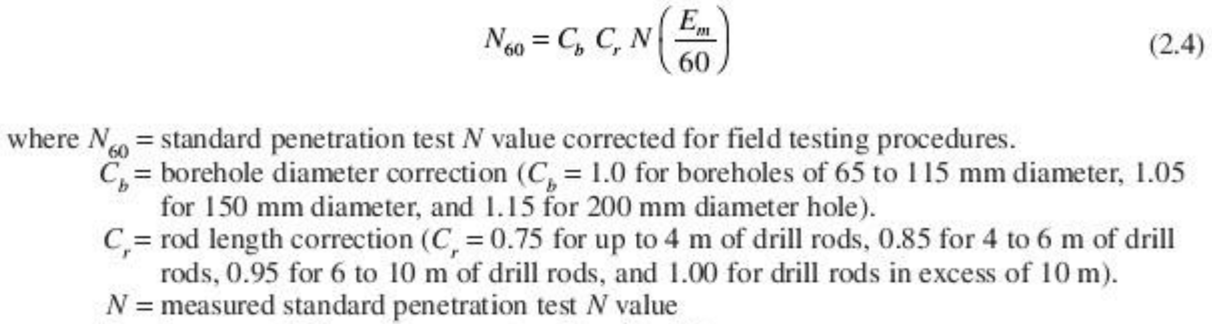
Disadvantage of SPT (Idriss & Boulanger 2008)



SPT Interval
of 5 feet
(1.5 m)

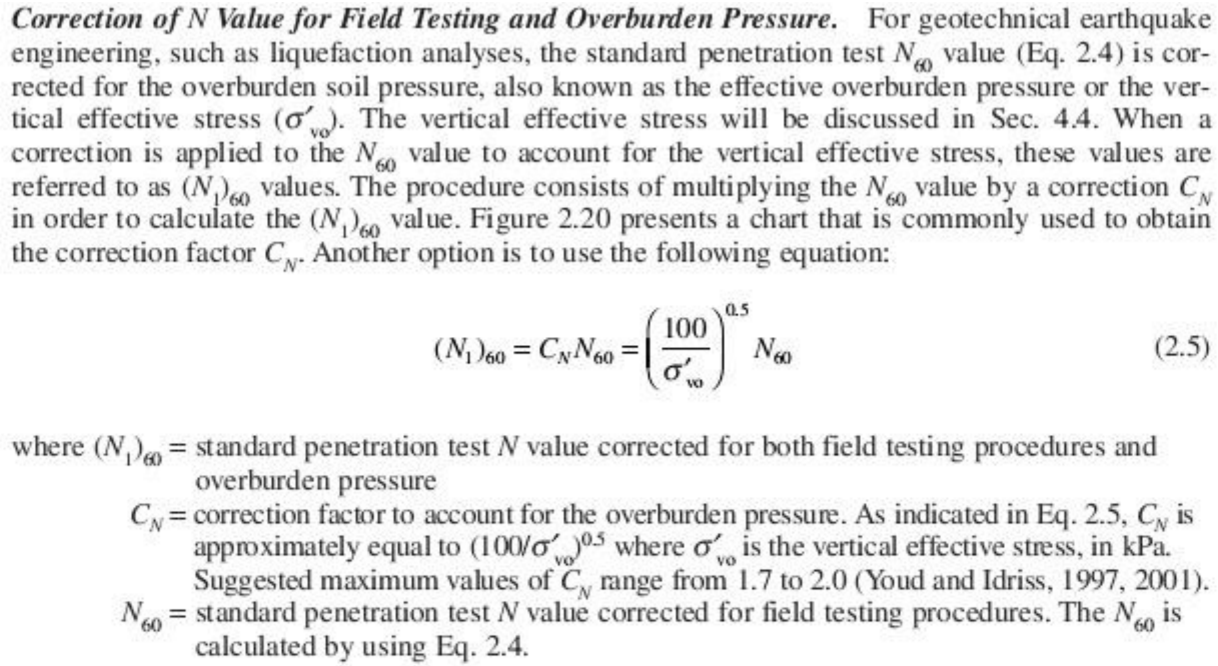
Figure 57. An excavation face showing the interlayering of sand, gravelly sand, and sandy gravel in an alluvial deposit and the portion of the deposit that would be observed via SPT samples at 1.5-m intervals.

SPT-N Corrections



$$N_{60} = C_b\ C_r\ N\left(\frac{E_m}{60}\right) \qquad (2.4)$$

where N_{60} = standard penetration test N value corrected for field testing procedures.
C_b = borehole diameter correction (C_b = 1.0 for boreholes of 65 to 115 mm diameter, 1.05 for 150 mm diameter, and 1.15 for 200 mm diameter hole).
C_r = rod length correction (C_r = 0.75 for up to 4 m of drill rods, 0.85 for 4 to 6 m of drill rods, 0.95 for 6 to 10 m of drill rods, and 1.00 for drill rods in excess of 10 m).
N = measured standard penetration test N value
E_m = hammer efficiency in percent, as described later



Correction of N Value for Field Testing and Overburden Pressure. For geotechnical earthquake engineering, such as liquefaction analyses, the standard penetration test N_{60} value (Eq. 2.4) is corrected for the overburden soil pressure, also known as the effective overburden pressure or the vertical effective stress (σ'_{vo}). The vertical effective stress will be discussed in Sec. 4.4. When a correction is applied to the N_{60} value to account for the vertical effective stress, these values are referred to as $(N_1)_{60}$ values. The procedure consists of multiplying the N_{60} value by a correction C_N in order to calculate the $(N_1)_{60}$ value. Figure 2.20 presents a chart that is commonly used to obtain the correction factor C_N. Another option is to use the following equation:

$$(N_1)_{60} = C_N N_{60} = \left(\frac{100}{\sigma'_{vo}}\right)^{0.5} N_{60} \qquad (2.5)$$

where $(N_1)_{60}$ = standard penetration test N value corrected for both field testing procedures and overburden pressure
C_N = correction factor to account for the overburden pressure. As indicated in Eq. 2.5, C_N is approximately equal to $(100/\sigma'_{vo})^{0.5}$ where σ'_{vo} is the vertical effective stress, in kPa. Suggested maximum values of C_N range from 1.7 to 2.0 (Youd and Idriss, 1997, 2001).
N_{60} = standard penetration test N value corrected for field testing procedures. The N_{60} is calculated by using Eq. 2.4.

DILATANCY CORRECTION

- Terzaghi and Peck (1967) recommended the following correction-

$$\boxed{\mathrm{N_c} = 15 + \frac{1}{2}\left(\mathrm{N_R} - 15\right)}$$

Where;

N_c - Corrected Penetration Number

N_R - Recorded Value

- If N_R<=15 ; $N_c = N_R$

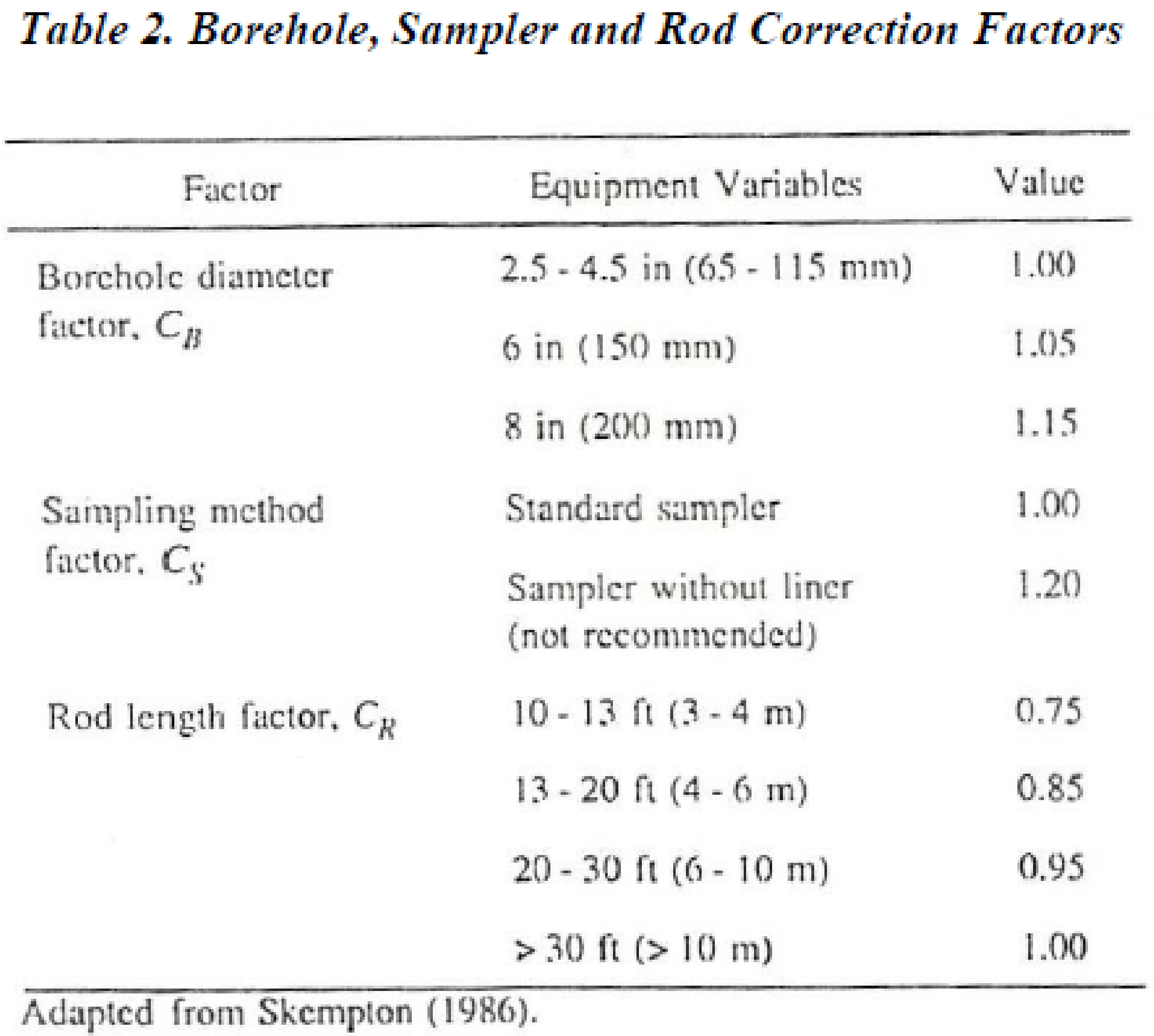


Table 2. Borehole, Sampler and Rod Correction Factors

Factor	Equipment Variables	Value
Borehole diameter factor, C_B	2.5 - 4.5 in (65 - 115 mm)	1.00
	6 in (150 mm)	1.05
	8 in (200 mm)	1.15
Sampling method factor, C_S	Standard sampler	1.00
	Sampler without liner (not recommended)	1.20
Rod length factor, C_R	10 - 13 ft (3 - 4 m)	0.75
	13 - 20 ft (4 - 6 m)	0.85
	20 - 30 ft (6 - 10 m)	0.95
	> 30 ft (> 10 m)	1.00

Adapted from Skempton (1986).

SPT-N Correlations

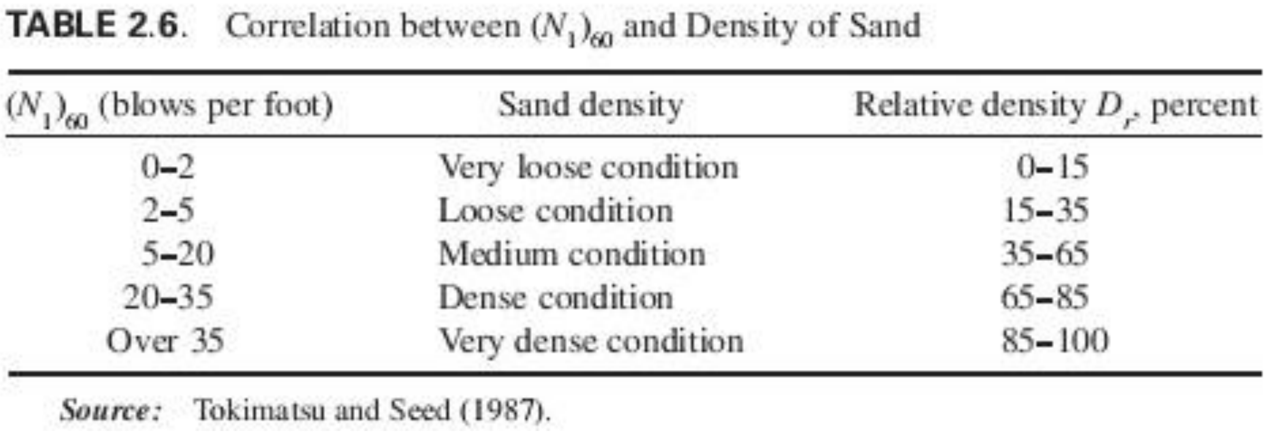


TABLE 2.6. Correlation between $(N_1)_{60}$ and Density of Sand

$(N_1)_{60}$ (blows per foot)	Sand density	Relative density D_r, percent
0–2	Very loose condition	0–15
2–5	Loose condition	15–35
5–20	Medium condition	35–65
20–35	Dense condition	65–85
Over 35	Very dense condition	85–100

Source: Tokimatsu and Seed (1987).

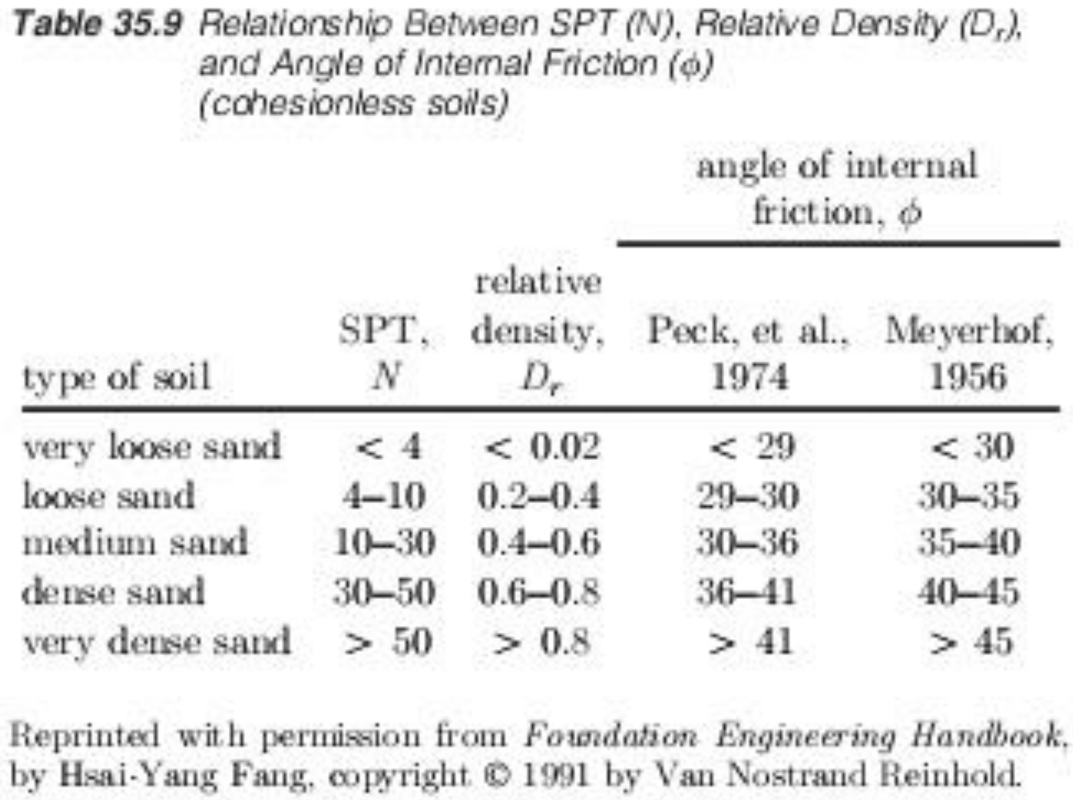


Table 35.9 *Relationship Between SPT (N), Relative Density (D_r), and Angle of Internal Friction (ϕ) (cohesionless soils)*

			angle of internal friction, ϕ	
type of soil	SPT, N	relative density, D_r	Peck, et al., 1974	Meyerhof, 1956
very loose sand	< 4	< 0.02	< 29	< 30
loose sand	4–10	0.2–0.4	29–30	30–35
medium sand	10–30	0.4–0.6	30–36	35–40
dense sand	30–50	0.6–0.8	36–41	40–45
very dense sand	> 50	> 0.8	> 41	> 45

Reprinted with permission from *Foundation Engineering Handbook*, by Hsai-Yang Fang, copyright © 1991 by Van Nostrand Reinhold.

SPT-N Correlations

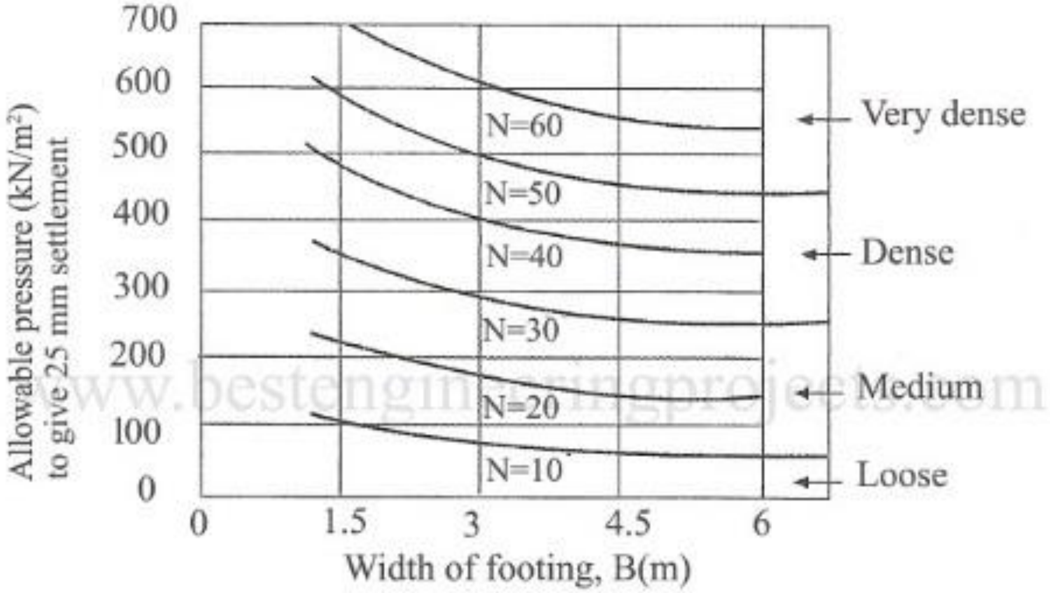


Fig 1 Correlation of Allowable Bearing Pressure to Give 25 mm Settlement to SPT ‘N’ Value after Terzaghi and Peak (1948)

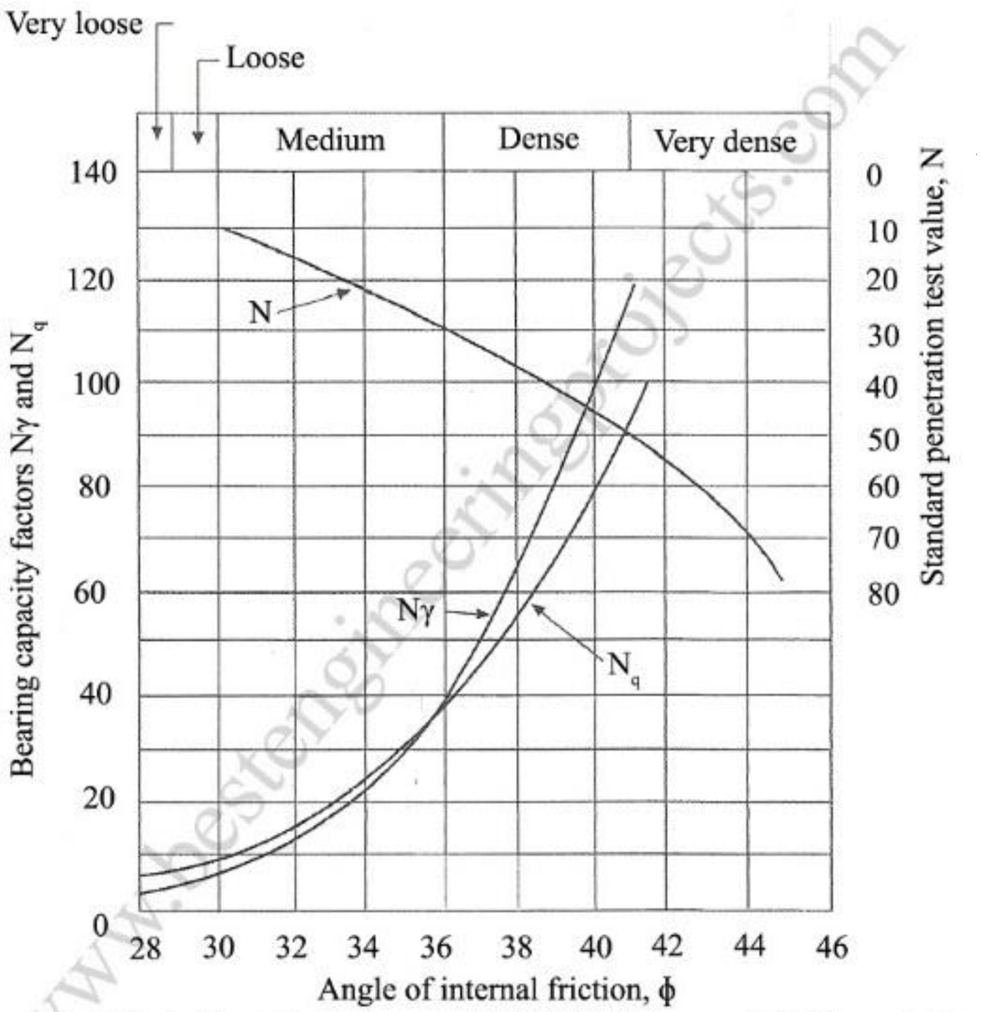


Fig 2 Relationship between SPT ‘N’ Value and ϕ, N_q and $N\gamma$ after peak, Hanson and Thornburn (1974)

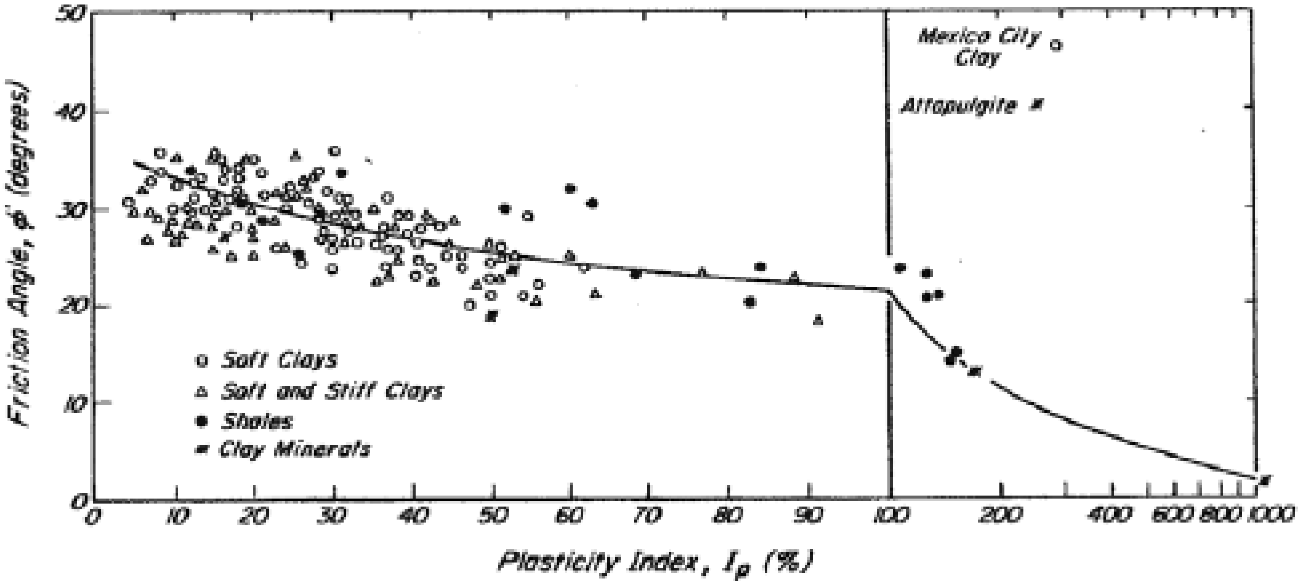
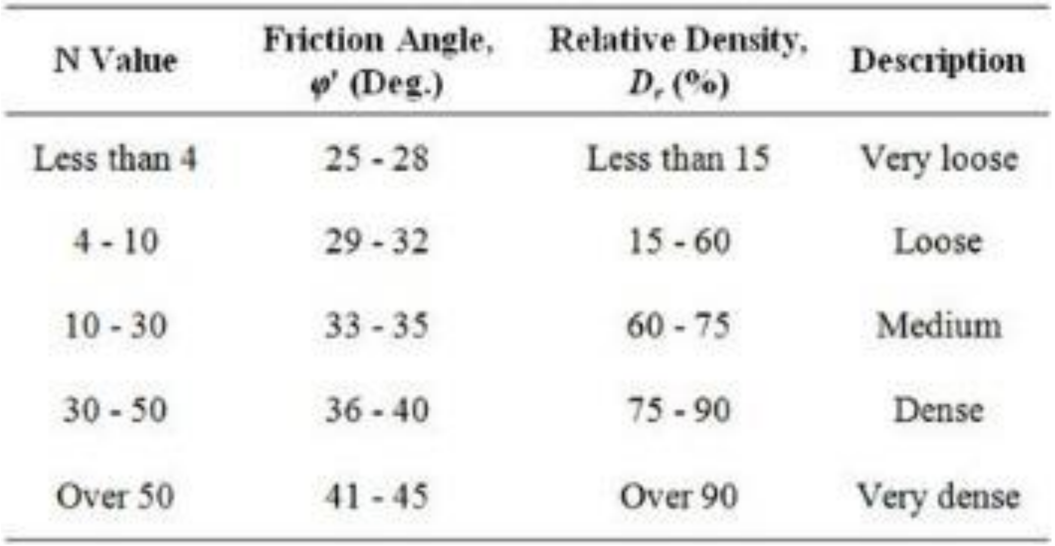
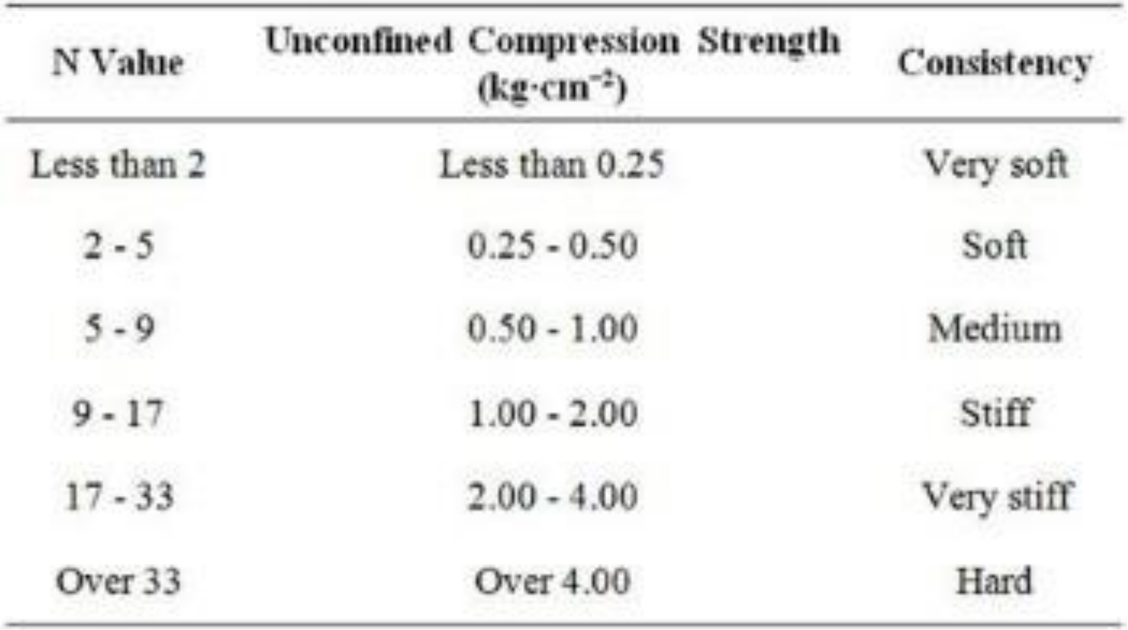


Figure 7. Values of friction angle ϕ' for clays of various compositions as reflected in plasticity index (Terzaghi, Peck and Mesri, 1996)

SPT-N Correlations



N Value	Friction Angle, φ' (Deg.)	Relative Density, D_r (%)	Description
Less than 4	25 - 28	Less than 15	Very loose
4 - 10	29 - 32	15 - 60	Loose
10 - 30	33 - 35	60 - 75	Medium
30 - 50	36 - 40	75 - 90	Dense
Over 50	41 - 45	Over 90	Very dense



N Value	Unconfined Compression Strength $(kg \cdot cm^{-2})$	Consistency
Less than 2	Less than 0.25	Very soft
2 - 5	0.25 - 0.50	Soft
5 - 9	0.50 - 1.00	Medium
9 - 17	1.00 - 2.00	Stiff
17 - 33	2.00 - 4.00	Very stiff
Over 33	Over 4.00	Hard

Correlation between N and Relative Density Dr

- correlation between N_{60} and Relative Density of Granular Soil

$$D_r(\%) = \left[\frac{N_{60}\left(0.23 + \frac{0.06}{D_{50}}\right)^{1.7}}{9}\left(\frac{1}{\frac{\sigma'_o}{p_a}}\right)\right]^{0.5}(100)$$

General

$$D_r = \left\{\frac{N_{60}}{\left[17 + 24\left(\frac{\sigma'_o}{p_a}\right)\right]}\right\}^{0.5}$$

For Clean sand only

where

D_r = relative density

σ'_o = effective overburden pressure

D_{50} = sieve size through which 50% of the soil will pass (mm)

p_a = atmospheric pressure ($\approx$ 100 kN/m^2,

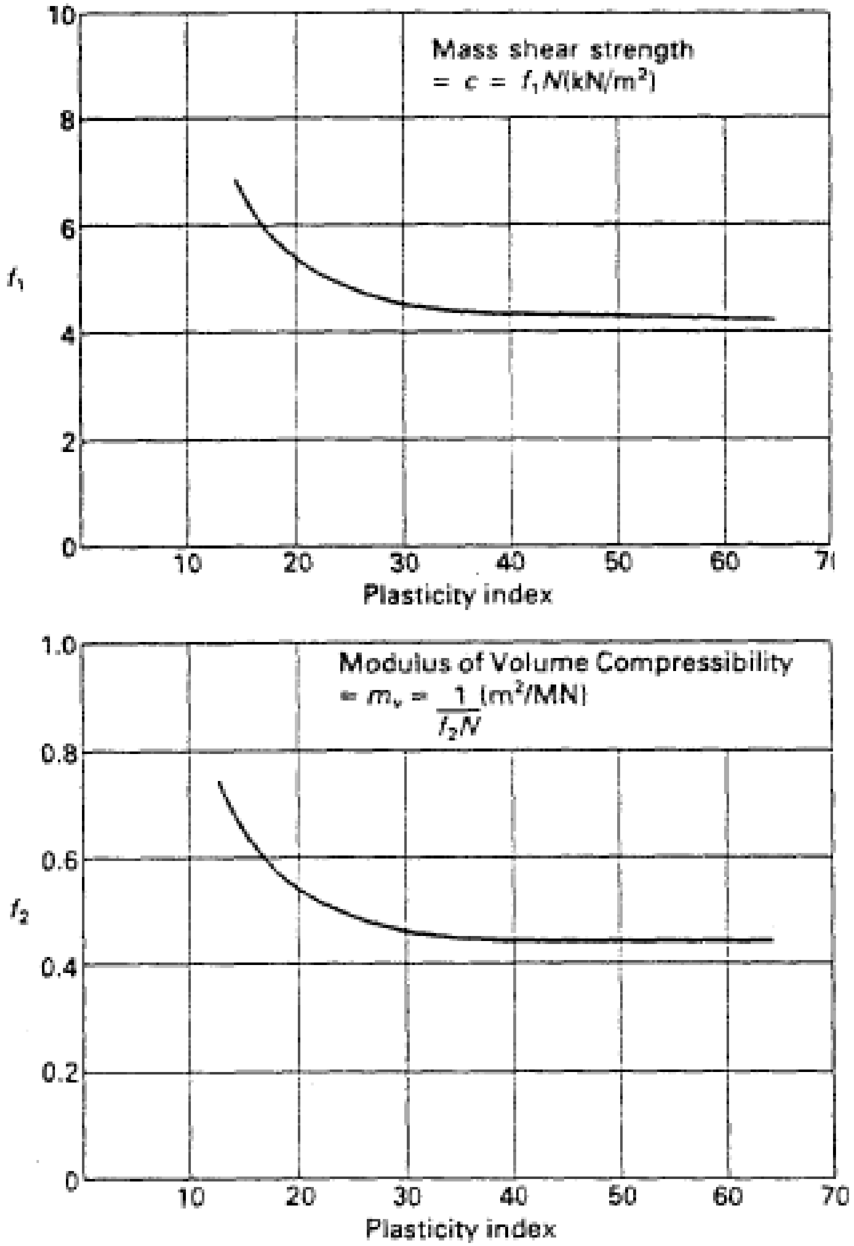


Figure 8. Relationship between Mass Shear Strength, Modulus of Volume Compressibility, Plasticity Index, and SPT-N values (after Stroud, 1975)

Table 4. Stroud (1989) recommendation for c_u ($c_u = f_1 * N_{60}$)

Soil Type	f_1 (kN/m^2)
Overconsolidated clays IP = 50% IP = 15%	 4.5 5.5
Insensitive weak rocks $N_{60} < 200$	5.0

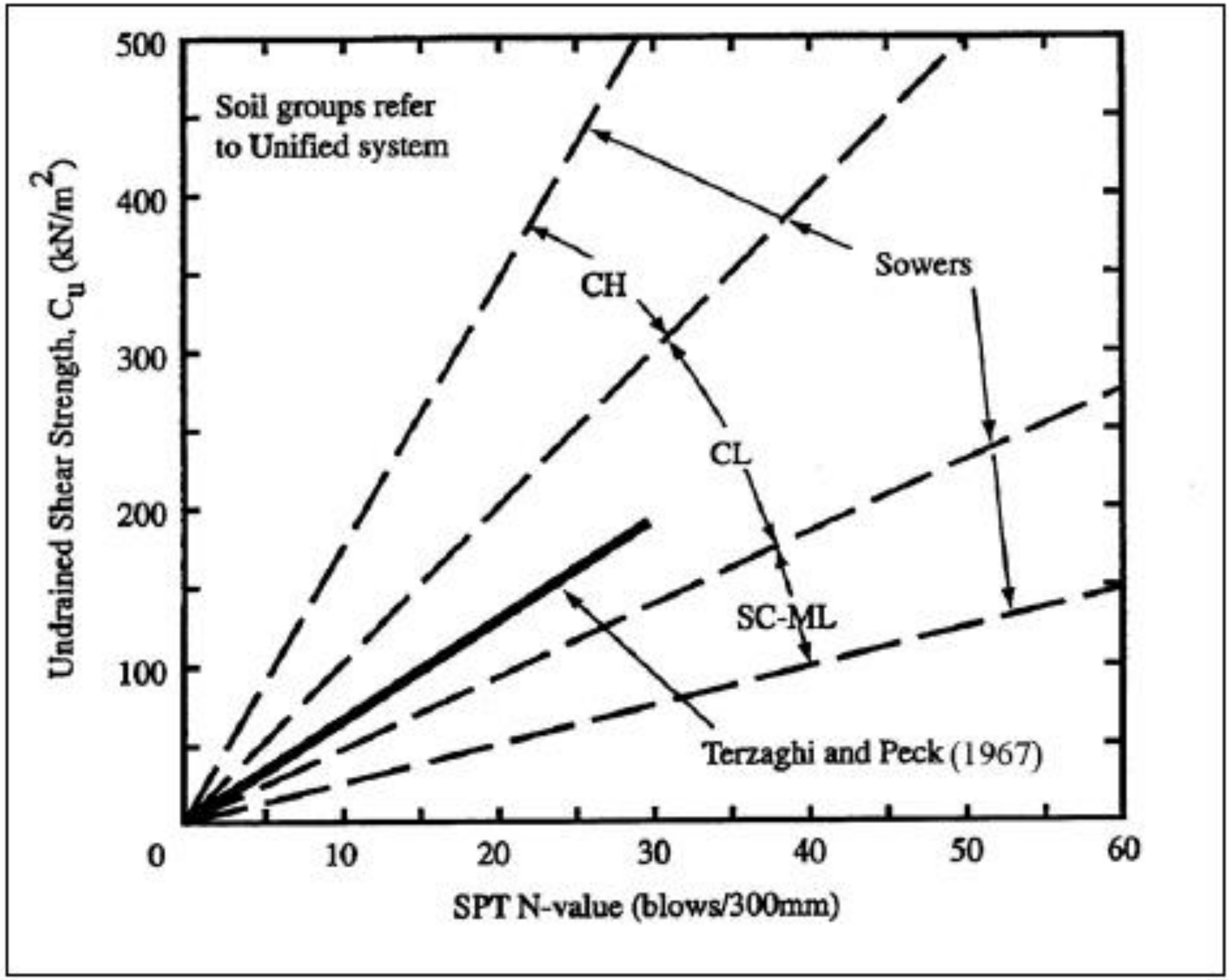


Figure 9. Approximate Correlation between Undrained Shear Strength and SPT-N values (After Sowers, 1979)

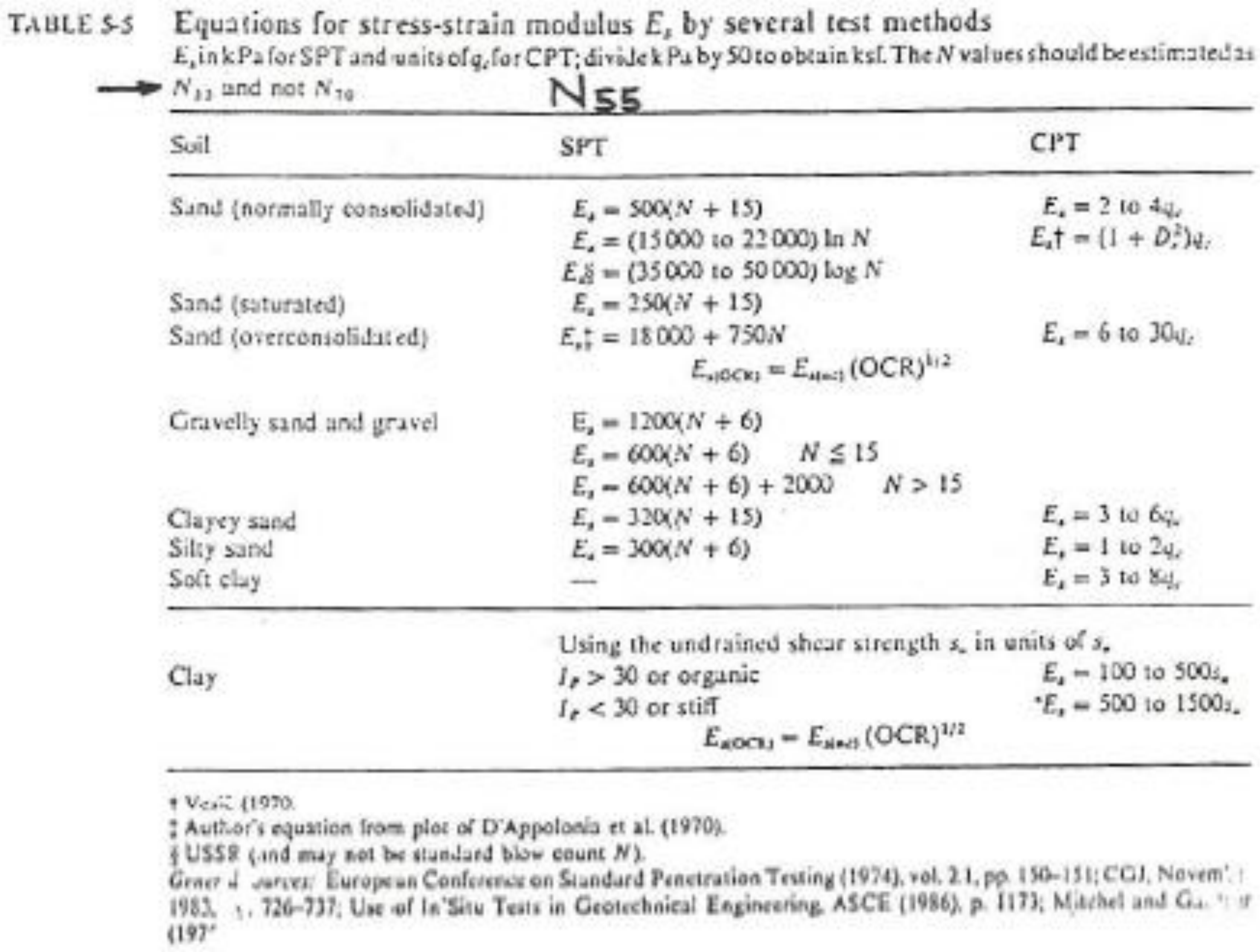


TABLE 5-5 Equations for stress-strain modulus E_s by several test methods

E_s in kPa for SPT and units of q_c for CPT; divide kPa by 50 to obtain ksf. The N values should be estimated as N_{55} and not N_{70}

N_{55}

Soil	SPT	CPT
Sand (normally consolidated)	$E_s = 500(N + 15)$ $E_s = (15\,000 \text{ to } 22\,000) \ln N$ $E_s§ = (35\,000 \text{ to } 50\,000) \log N$	$E_s = 2 \text{ to } 4q_c$ $E_s† = (1 + D_r^2)q_c$
Sand (saturated)	$E_s = 250(N + 15)$	
Sand (overconsolidated)	$E_s‡ = 18\,000 + 750N$ $E_{s(OCR)} = E_{s(nc)}(\text{OCR})^{1/2}$	$E_s = 6 \text{ to } 30q_c$
Gravelly sand and gravel	$E_s = 1200(N + 6)$ $E_s = 600(N + 6) \quad N \le 15$ $E_s = 600(N + 6) + 2000 \quad N > 15$	
Clayey sand	$E_s = 320(N + 15)$	$E_s = 3 \text{ to } 6q_c$
Silty sand	$E_s = 300(N + 6)$	$E_s = 1 \text{ to } 2q_c$
Soft clay	—	$E_s = 3 \text{ to } 8q_c$
	Using the undrained shear strength s_u in units of s_u	
Clay	$I_P > 30$ or organic	$E_s = 100 \text{ to } 500s_u$
	$I_P < 30$ or stiff	$^*E_s = 500 \text{ to } 1500s_u$
	$E_{s(OCR)} = E_{s(nc)}(\text{OCR})^{1/2}$	

† Vesić (1970).

‡ Author's equation from plot of D'Appolonia et al. (1970).

§ USSR (and may not be standard blow count N).

General sources: European Conference on Standard Penetration Testing (1974), vol. 2.1, pp. 150–151; CGJ, Novem 1983, , 726–737; Use of In'Situ Tests in Geotechnical Engineering, ASCE (1986), p. 1173; Mitchel and G (197

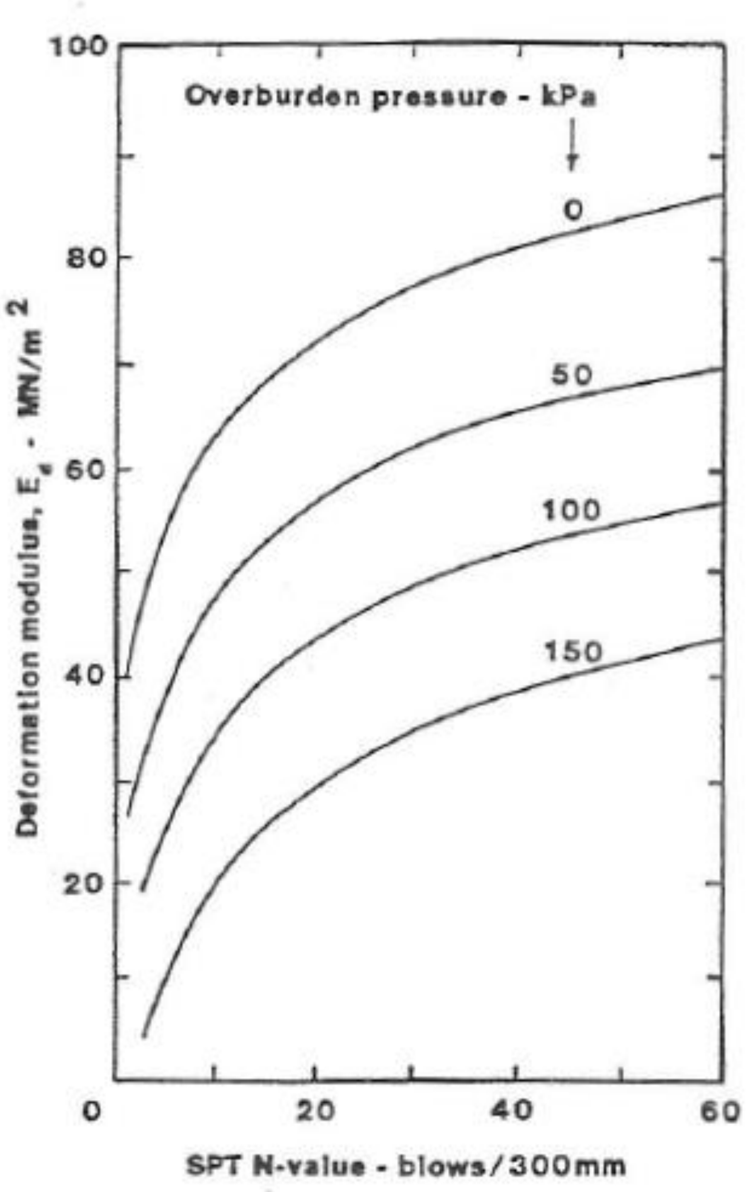


Figure 10. Correleation between deformation modulus, E_d and SPT N-value for granular soils (after Menzenbach, 1967)

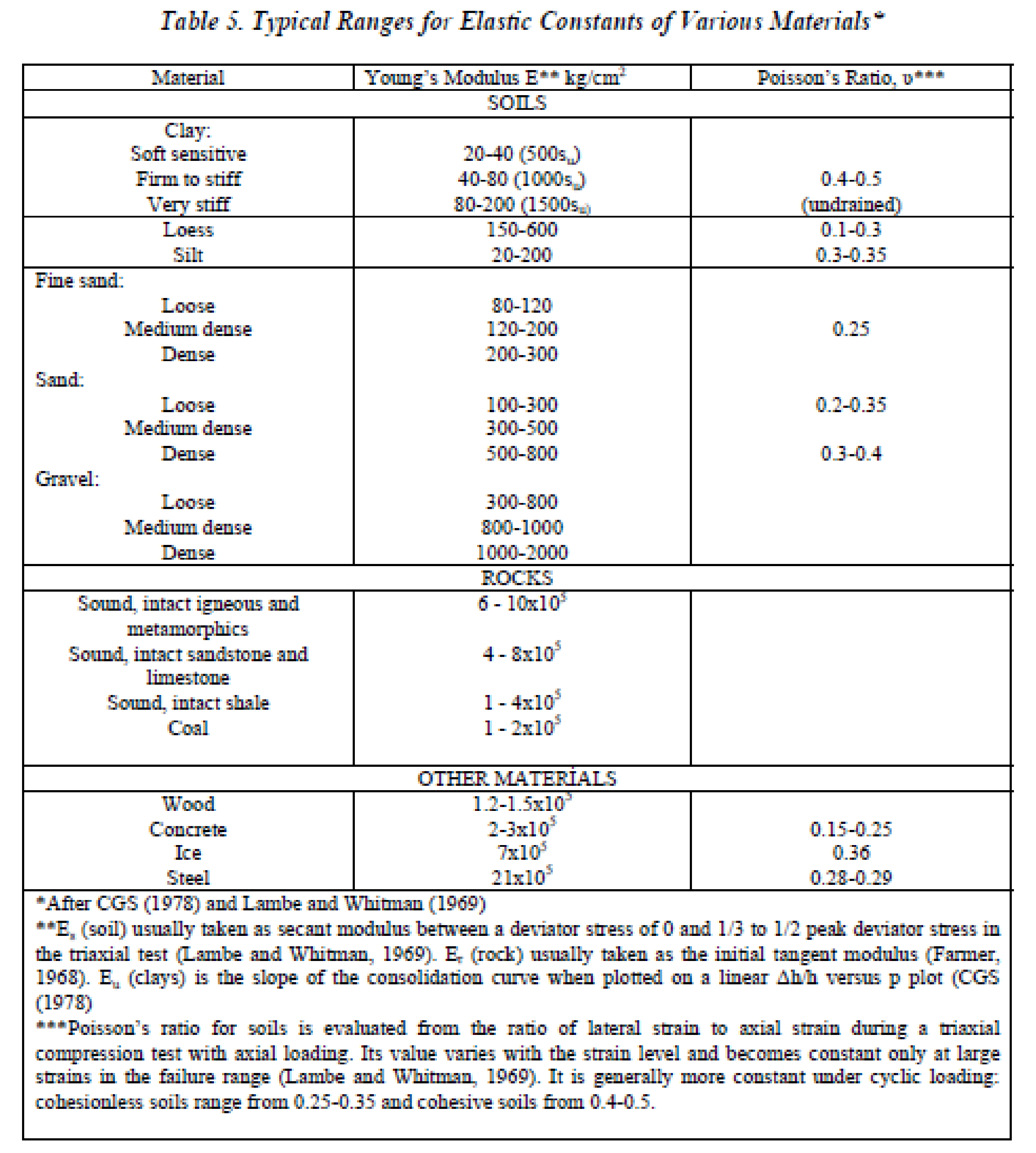


*Table 5. Typical Ranges for Elastic Constants of Various Materials**

Material	Young's Modulus E** kg/cm^2	Poisson's Ratio, υ***
SOILS		
Clay:		
Soft sensitive	20-40 (500s_u)	
Firm to stiff	40-80 (1000s_u)	0.4-0.5
Very stiff	80-200 (1500s_u)	(undrained)
Loess	150-600	0.1-0.3
Silt	20-200	0.3-0.35
Fine sand:		
Loose	80-120	
Medium dense	120-200	0.25
Dense	200-300	
Sand:		
Loose	100-300	0.2-0.35
Medium dense	300-500	
Dense	500-800	0.3-0.4
Gravel:		
Loose	300-800	
Medium dense	800-1000	
Dense	1000-2000	
ROCKS		
Sound, intact igneous and metamorphics	6 - 10x10^5	
Sound, intact sandstone and limestone	4 - 8x10^5	
Sound, intact shale	1 - 4x10^5	
Coal	1 - 2x10^5	
OTHER MATERIALS		
Wood	1.2-1.5x10^5	
Concrete	2-3x10^5	0.15-0.25
Ice	7x10^5	0.36
Steel	21x10^5	0.28-0.29

*After CGS (1978) and Lambe and Whitman (1969)

**E_s (soil) usually taken as secant modulus between a deviator stress of 0 and 1/3 to 1/2 peak deviator stress in the triaxial test (Lambe and Whitman, 1969). E_r (rock) usually taken as the initial tangent modulus (Farmer, 1968). E_u (clays) is the slope of the consolidation curve when plotted on a linear Δh/h versus p plot (CGS (1978)

***Poisson's ratio for soils is evaluated from the ratio of lateral strain to axial strain during a triaxial compression test with axial loading. Its value varies with the strain level and becomes constant only at large strains in the failure range (Lambe and Whitman, 1969). It is generally more constant under cyclic loading: cohesionless soils range from 0.25-0.35 and cohesive soils from 0.4-0.5.

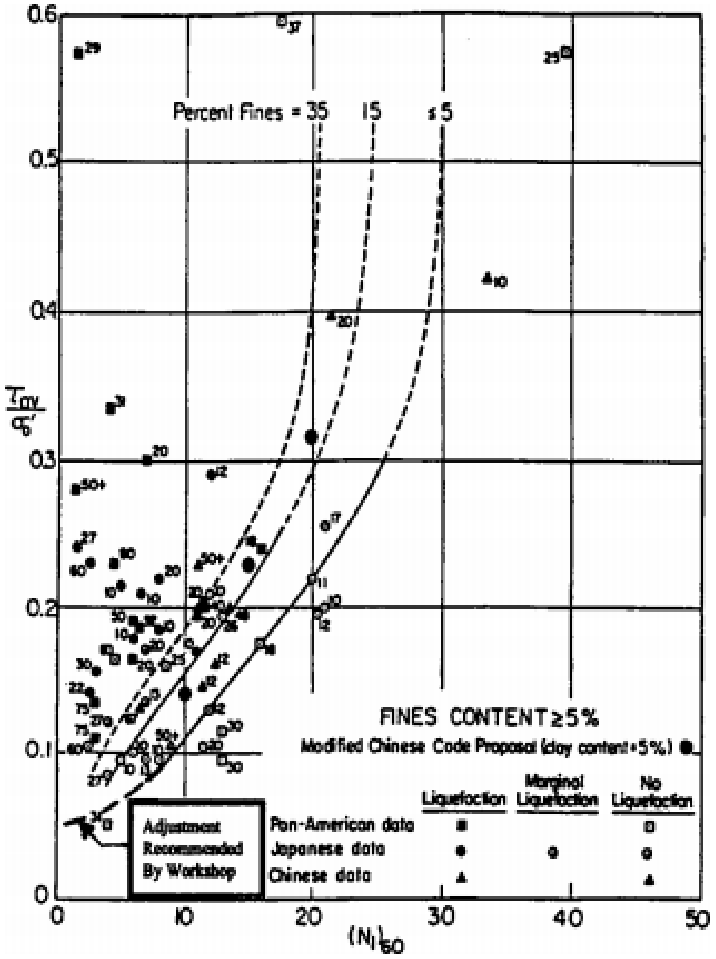


Fig. 1. Correlation between equivalent uniform cyclic stress ratio and standard penetration test $N_{1,60}$ value for events of magnitude $M \approx 7.5$ and for varying fines contents, with adjustment at low cyclic stress ratio as recommended by National Center for Earthquake Engineering Research working group (Seed et al. 1984)

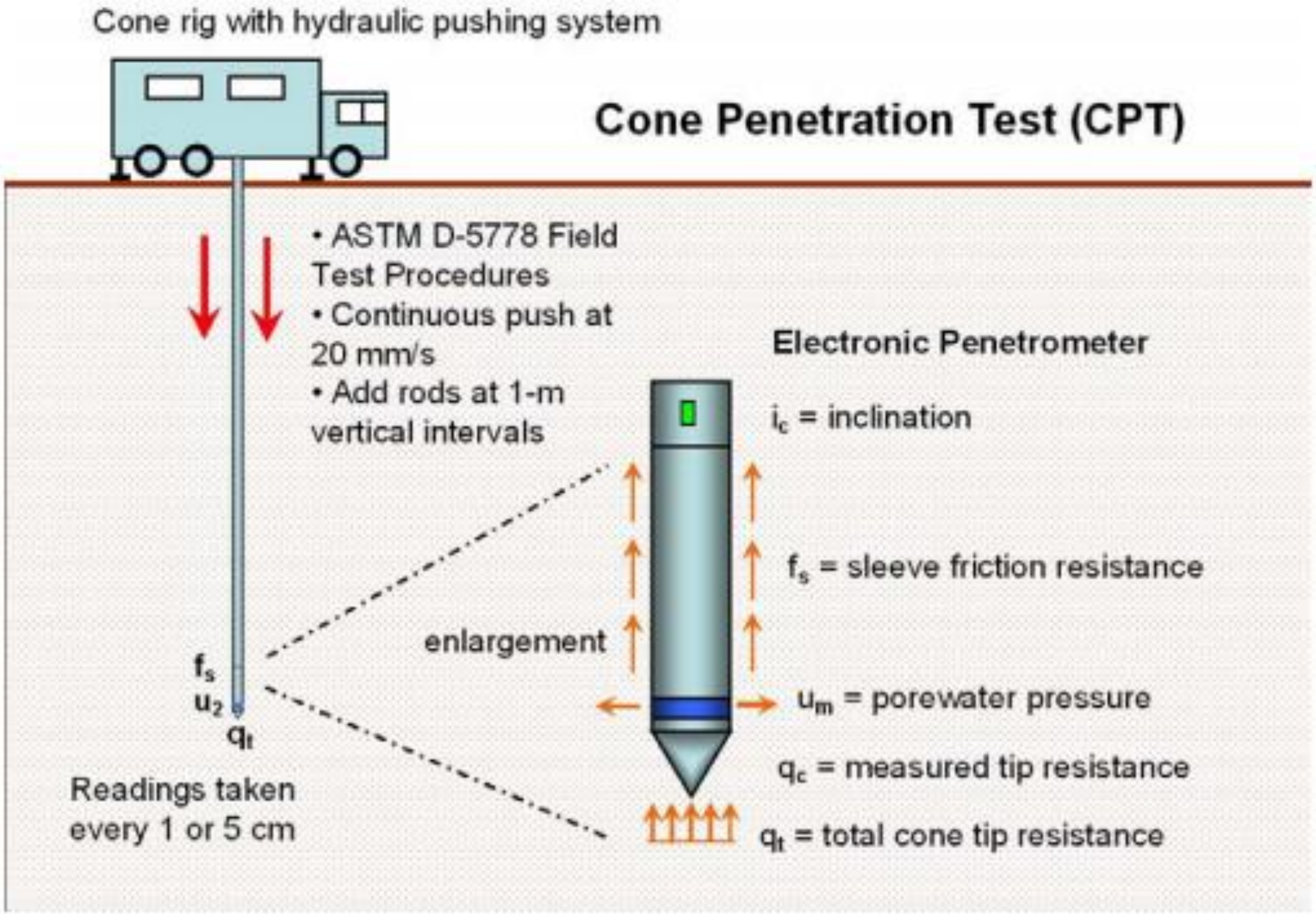


Figure 1. Overview of the Cone Penetration Test (CPT) Per ASTM D 5778 Procedures.

(a) (b) (c)

Figure 11. Selection of Penetrometers from: (a) van den Berg series, (b) Fugro series (left to right: 33-, 15-, 10-, 5-, and 1-cm^2 sizes), and (c) Georgia Tech collection (bottom to top): 5-cm^2 friction, four 10-cm^2 piezocones (type 2, type 1, type 2 seismic, dual-piezo-element), and 15-cm^2 triple-element type.

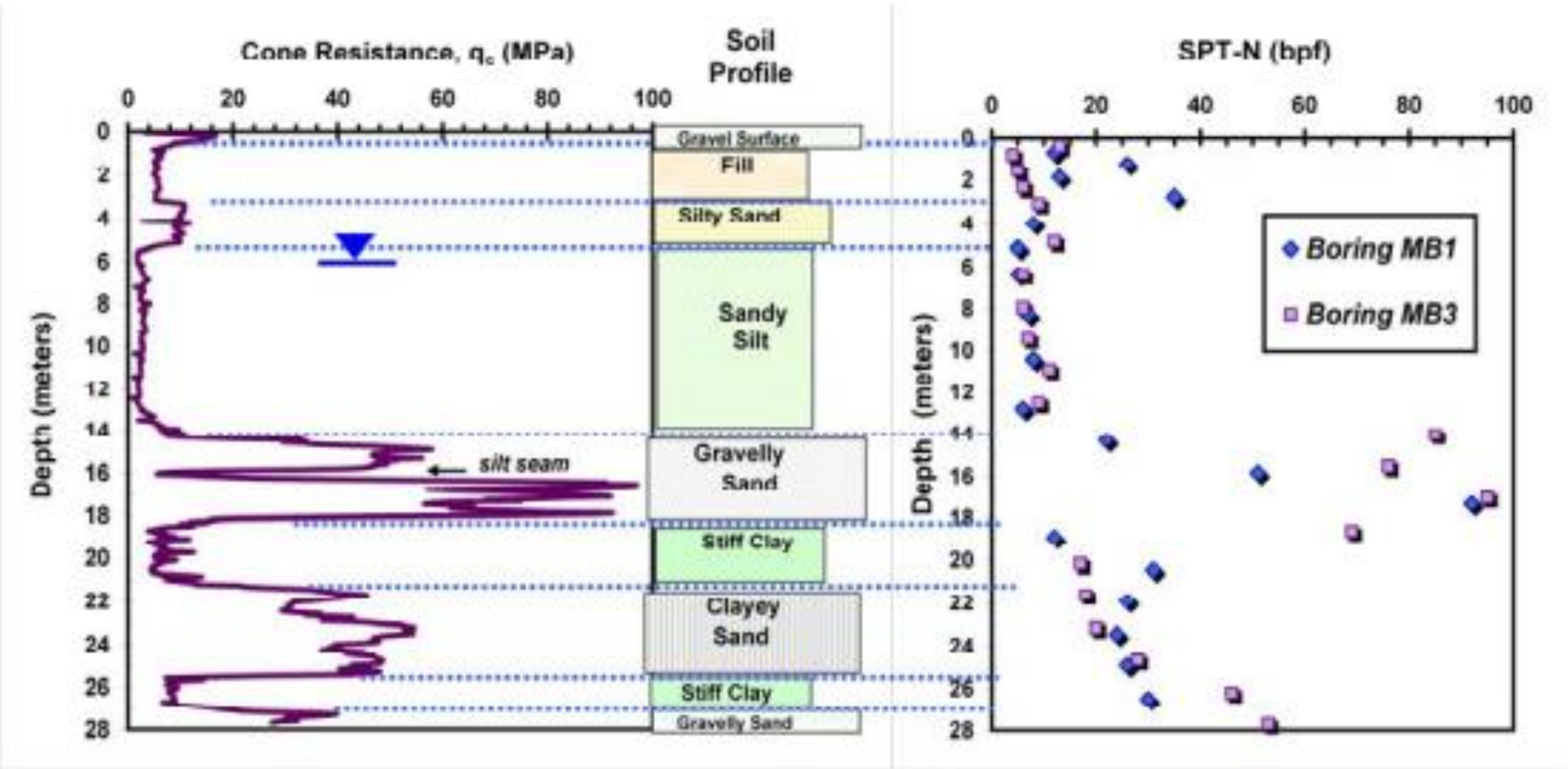
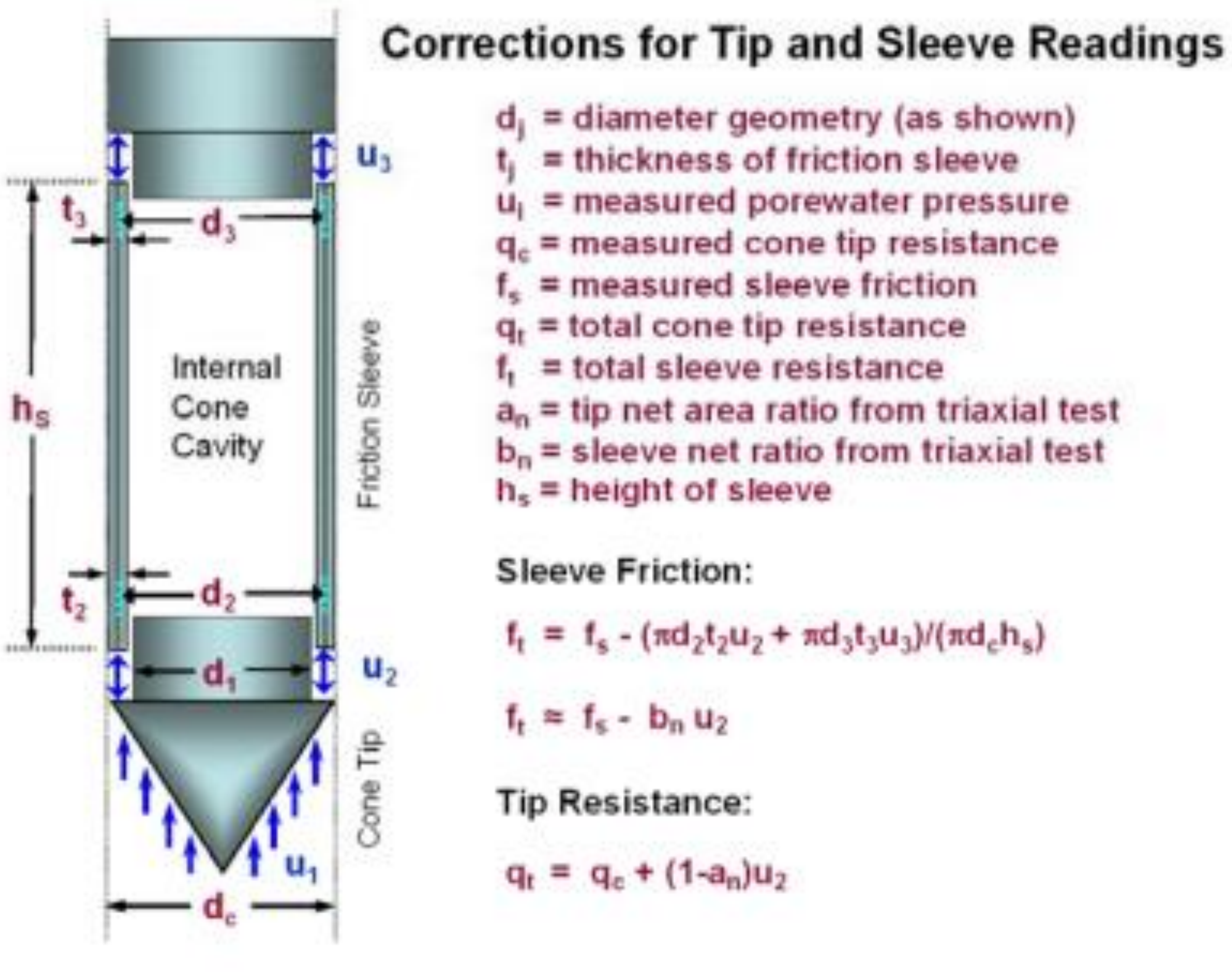


Figure 2. Companion Profile of CPT Cone Tip Resistance and Soil Boring Log with SPT-N values.



Corrections for Tip and Sleeve Readings

d_j = diameter geometry (as shown)
t_j = thickness of friction sleeve
u_i = measured porewater pressure
q_c = measured cone tip resistance
f_s = measured sleeve friction
q_t = total cone tip resistance
f_t = total sleeve resistance
a_n = tip net area ratio from triaxial test
b_n = sleeve net ratio from triaxial test
h_s = height of sleeve

Sleeve Friction:

$$f_t = f_s - (\pi d_2 t_2 u_2 + \pi d_3 t_3 u_3)/(\pi d_c h_s)$$

$$f_t \approx f_s - b_n u_2$$

Tip Resistance:

$$q_t = q_c + (1-a_n)u_2$$

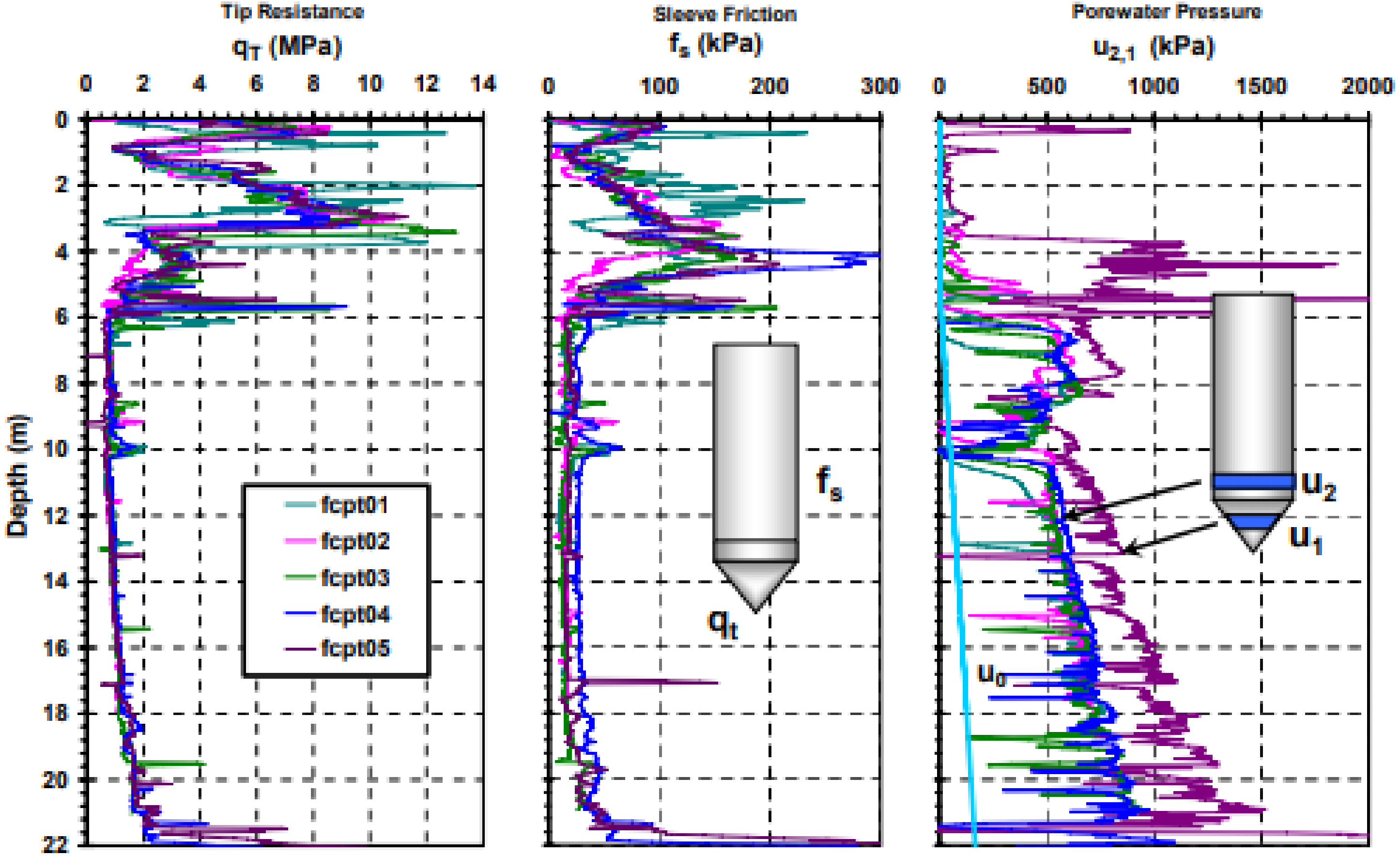


Figure 15. Series of Piezocone Penetration Tests at Northwestern University.

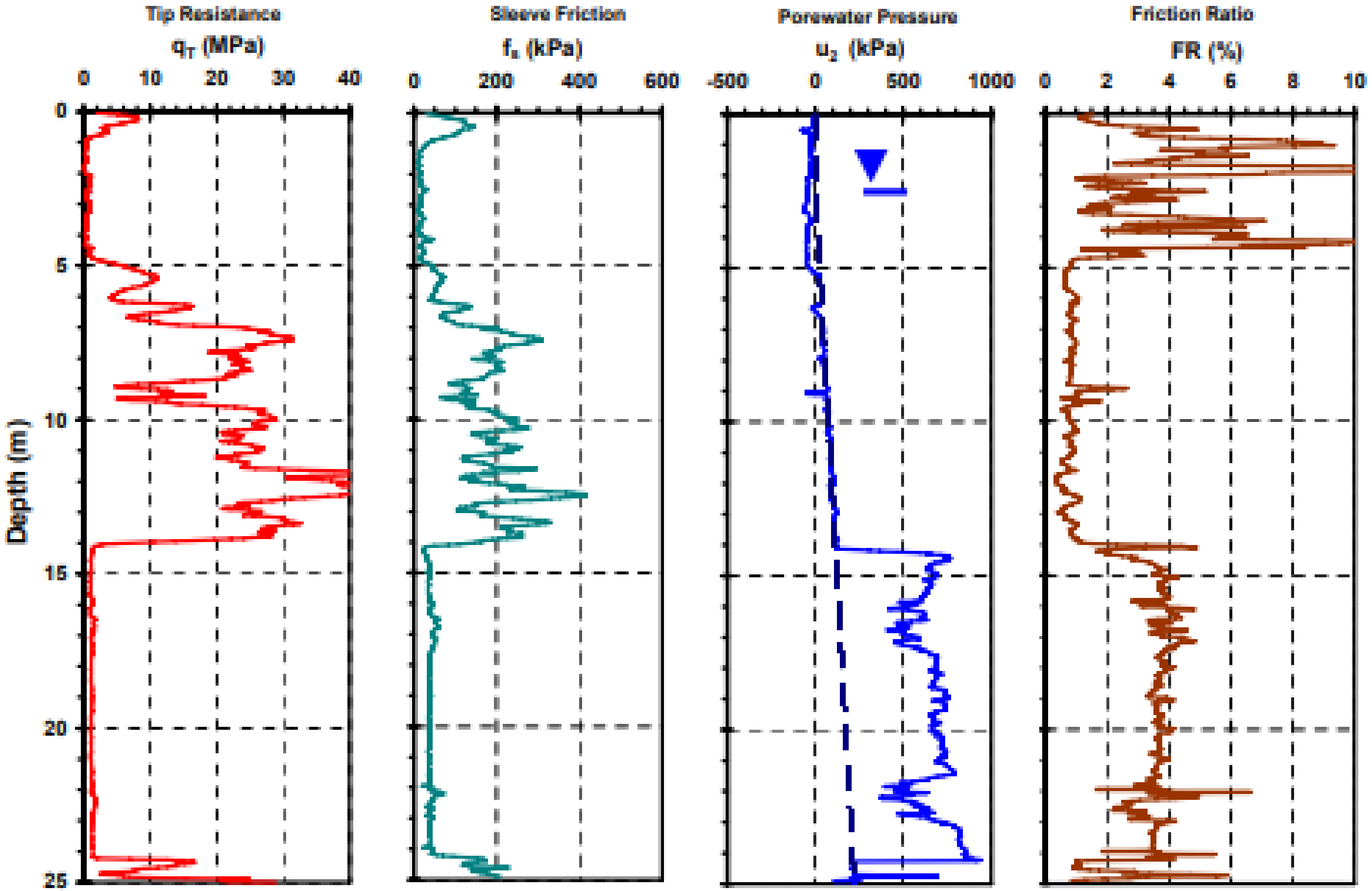
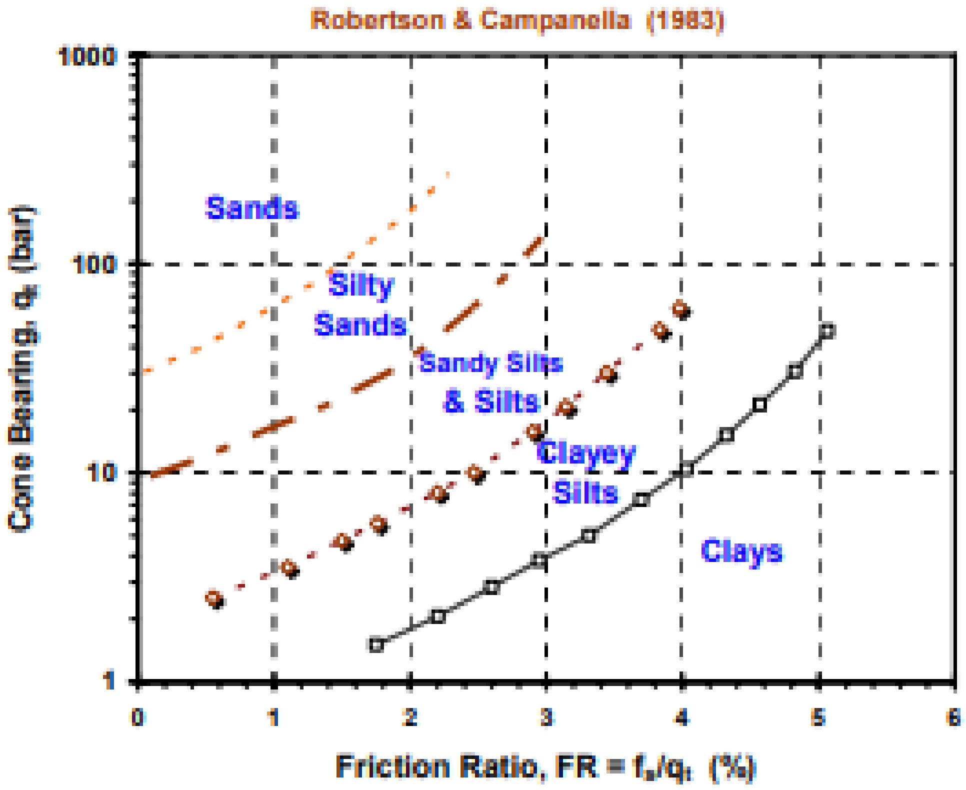


Figure 23. Presentation of CPTu Results, showing (a) Total Cone Tip Resistance, (b) Sleeve Friction, (c) Shoulder Porewater Pressures, and (d) Friction ratio (FR = $R_f = f_s/q_t$) with Depth in Steele, Missouri.

Soil Classification using CPT



Simplified CPT Soil Type Classification Chart (after Robertson & Campanella, 1983).

Soil Classification using CPT

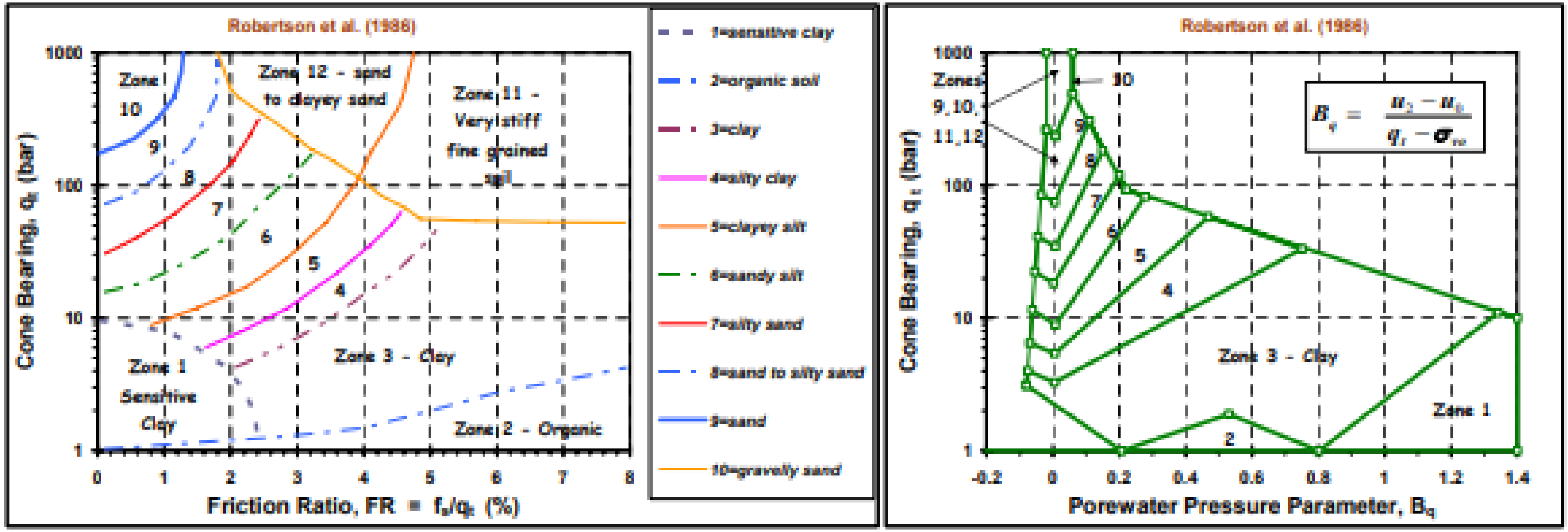
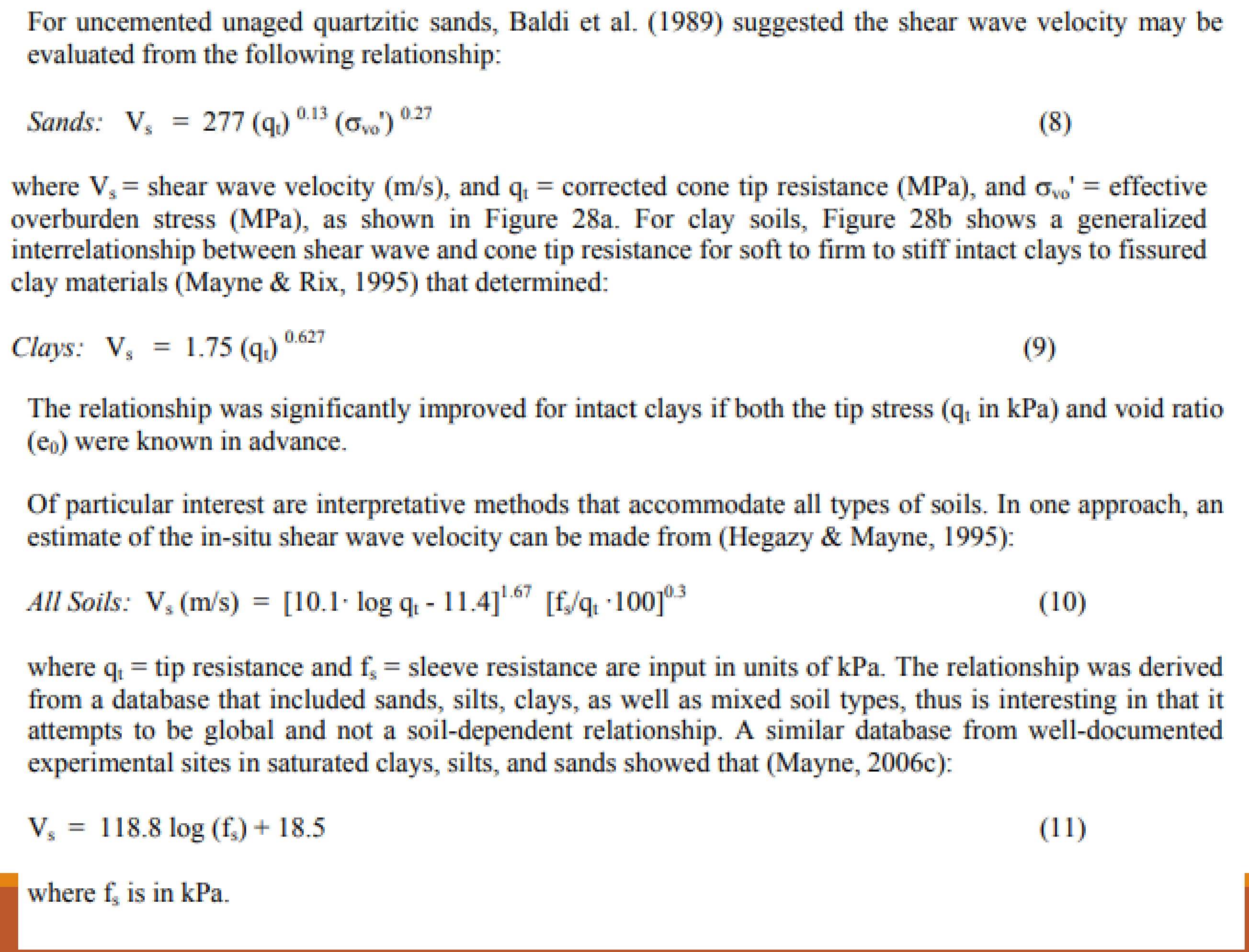


Figure 25. CPTu Soil Behavioral Type (SBT) for Layer Classification (after Robertson, et al. 1986).

Determination of Vs Using CPT



For uncemented unaged quartzitic sands, Baldi et al. (1989) suggested the shear wave velocity may be evaluated from the following relationship:

Sands: $V_s = 277 (q_t)^{0.13} (\sigma_{vo}')^{0.27}$ (8)

where V_s = shear wave velocity (m/s), and q_t = corrected cone tip resistance (MPa), and σ_{vo}' = effective overburden stress (MPa), as shown in Figure 28a. For clay soils, Figure 28b shows a generalized interrelationship between shear wave and cone tip resistance for soft to firm to stiff intact clays to fissured clay materials (Mayne & Rix, 1995) that determined:

Clays: $V_s = 1.75 (q_t)^{0.627}$ (9)

The relationship was significantly improved for intact clays if both the tip stress (q_t in kPa) and void ratio (e_0) were known in advance.

Of particular interest are interpretative methods that accommodate all types of soils. In one approach, an estimate of the in-situ shear wave velocity can be made from (Hegazy & Mayne, 1995):

All Soils: $V_s\ (m/s) = [10.1 \cdot \log q_t - 11.4]^{1.67}\ [f_s/q_t \cdot 100]^{0.3}$ (10)

where q_t = tip resistance and f_s = sleeve resistance are input in units of kPa. The relationship was derived from a database that included sands, silts, clays, as well as mixed soil types, thus is interesting in that it attempts to be global and not a soil-dependent relationship. A similar database from well-documented experimental sites in saturated clays, silts, and sands showed that (Mayne, 2006c):

$V_s = 118.8 \log (f_s) + 18.5$ (11)

where f_s is in kPa.

Determination of Preconsolidation Pressure Using CPT

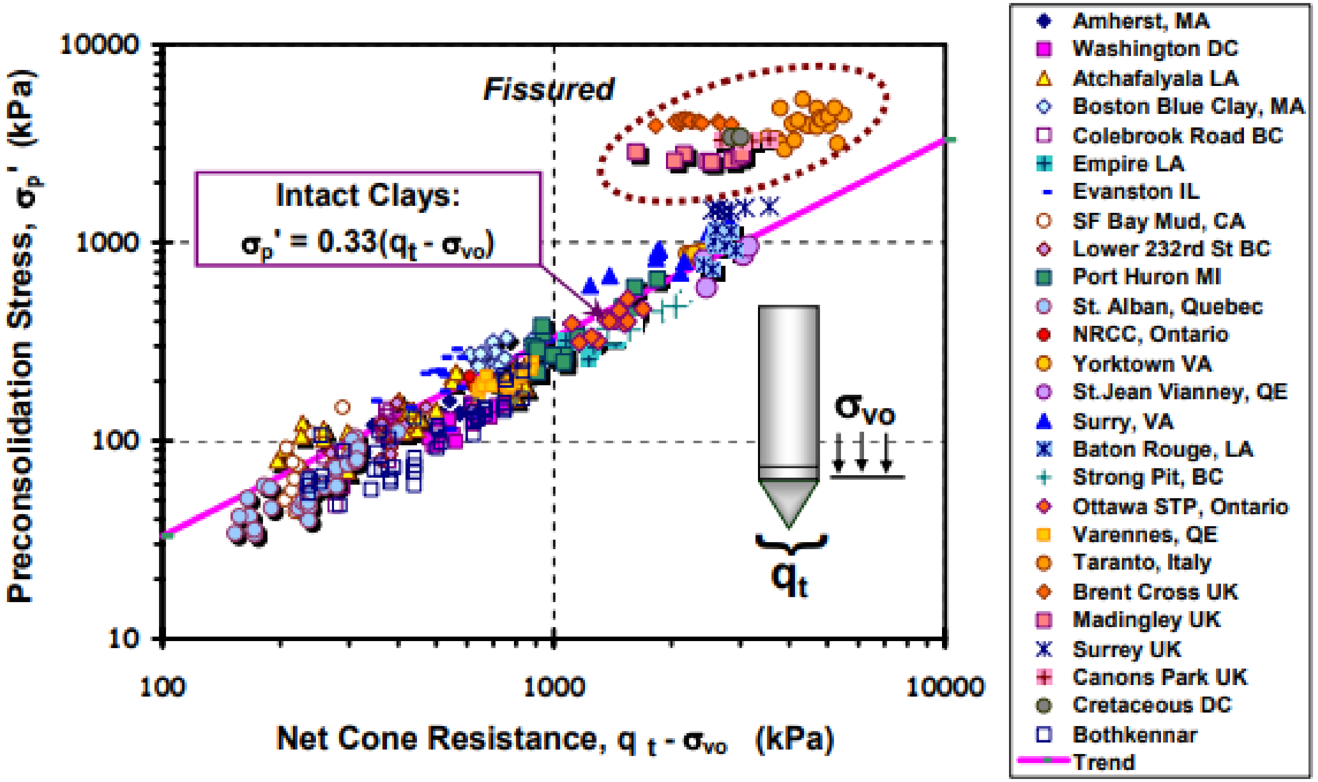
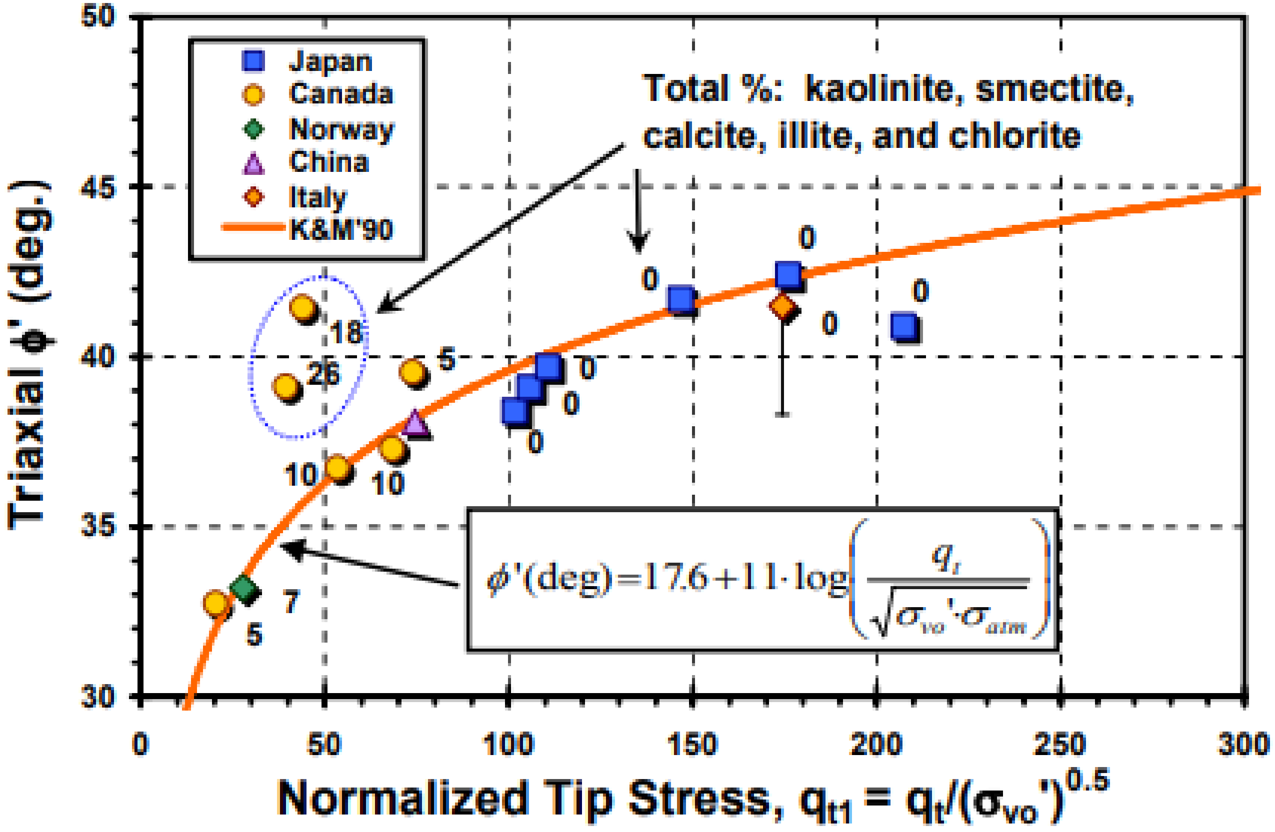


Figure 34. First-Order Relationship for Preconsolidation Stress from Net Cone Resistance in Clays.

Determination of ϕ Using CPT



Determination of D_R Using CPT

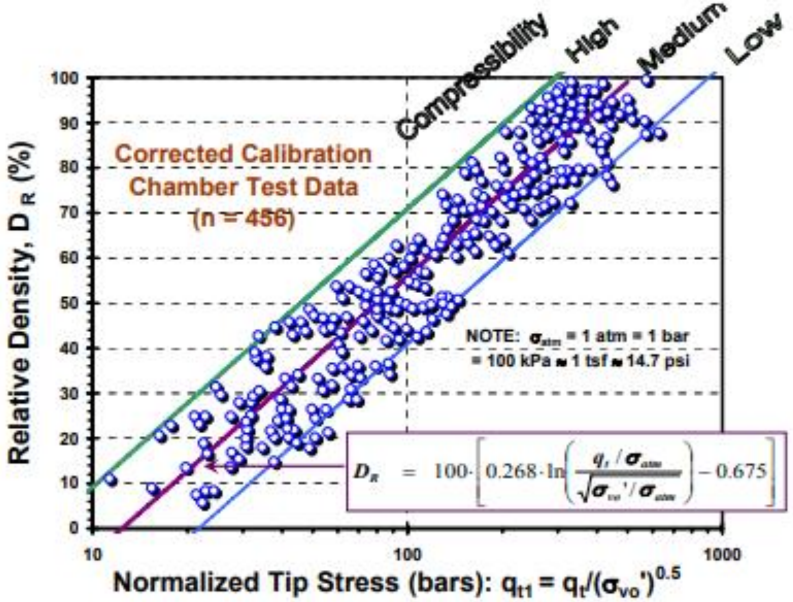
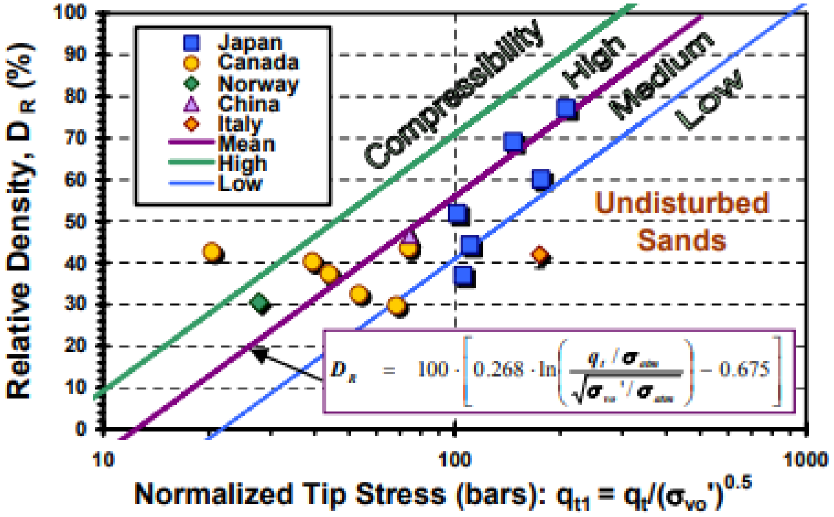
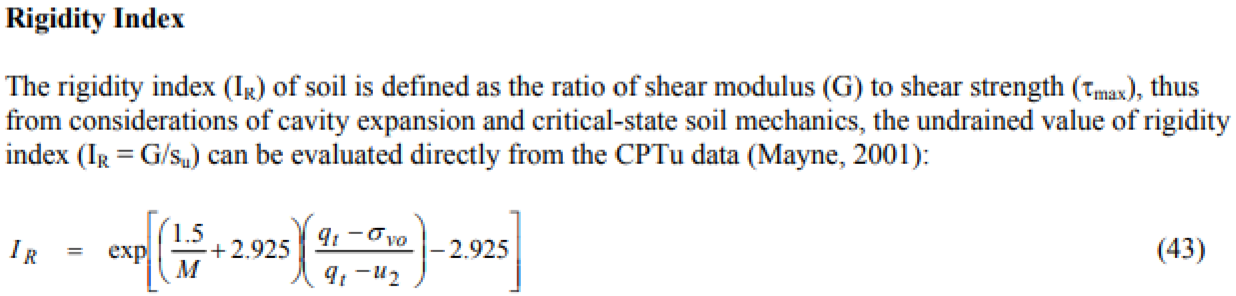


Figure 46. Relative Density Relationship with Normalized Tip Stress and Sand Compressibility from Corrected Chamber Test Results (after Jamiolkowski, et al. 2001).



Relative Density of Undisturbed (Frozen) Quartz Sands vs. Normalized Cone Tip Resistance.

Determination of I_R and Bearing Capacity Using CPT



Rigidity Index

The rigidity index (I_R) of soil is defined as the ratio of shear modulus (G) to shear strength (τ_{max}), thus from considerations of cavity expansion and critical-state soil mechanics, the undrained value of rigidity index ($I_R = G/s_u$) can be evaluated directly from the CPTu data (Mayne, 2001):

$$I_R \quad = \quad \exp\left[\left(\frac{1.5}{M}+2.925\right)\left(\frac{q_t-\sigma_{vo}}{q_t-u_2}\right)-2.925\right] \qquad (43)$$

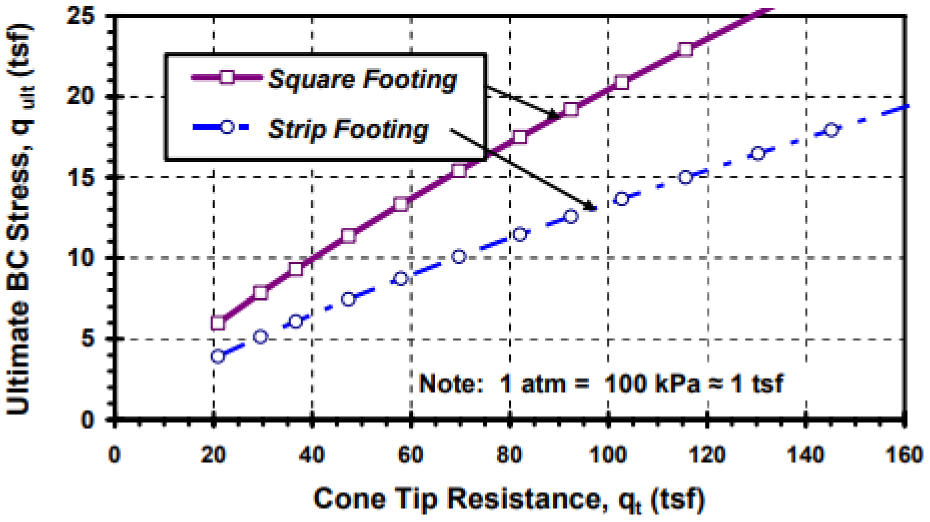


Figure 67. Direct Relationship for Ultimate Bearing Stress and CPT Measured Tip Stress in Sands (after Schmertmann, 1978).

Liquefaction Calculations with CPT

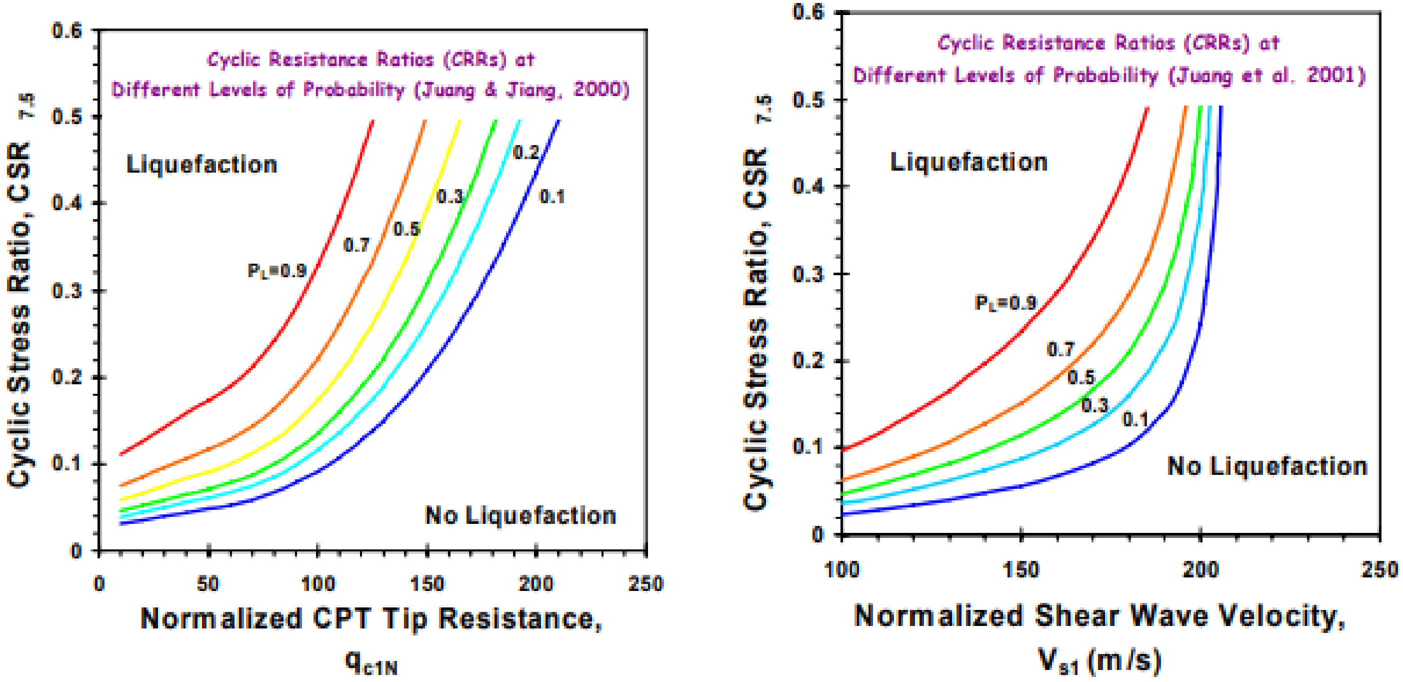


Figure 86. Probabilistic Cyclic Resistance Ratios (CRRs) for Clean Sands based on (a) Cone Tip Resistance and (b) Shear Wave Velocity (after Juang & Jiang, 2000).

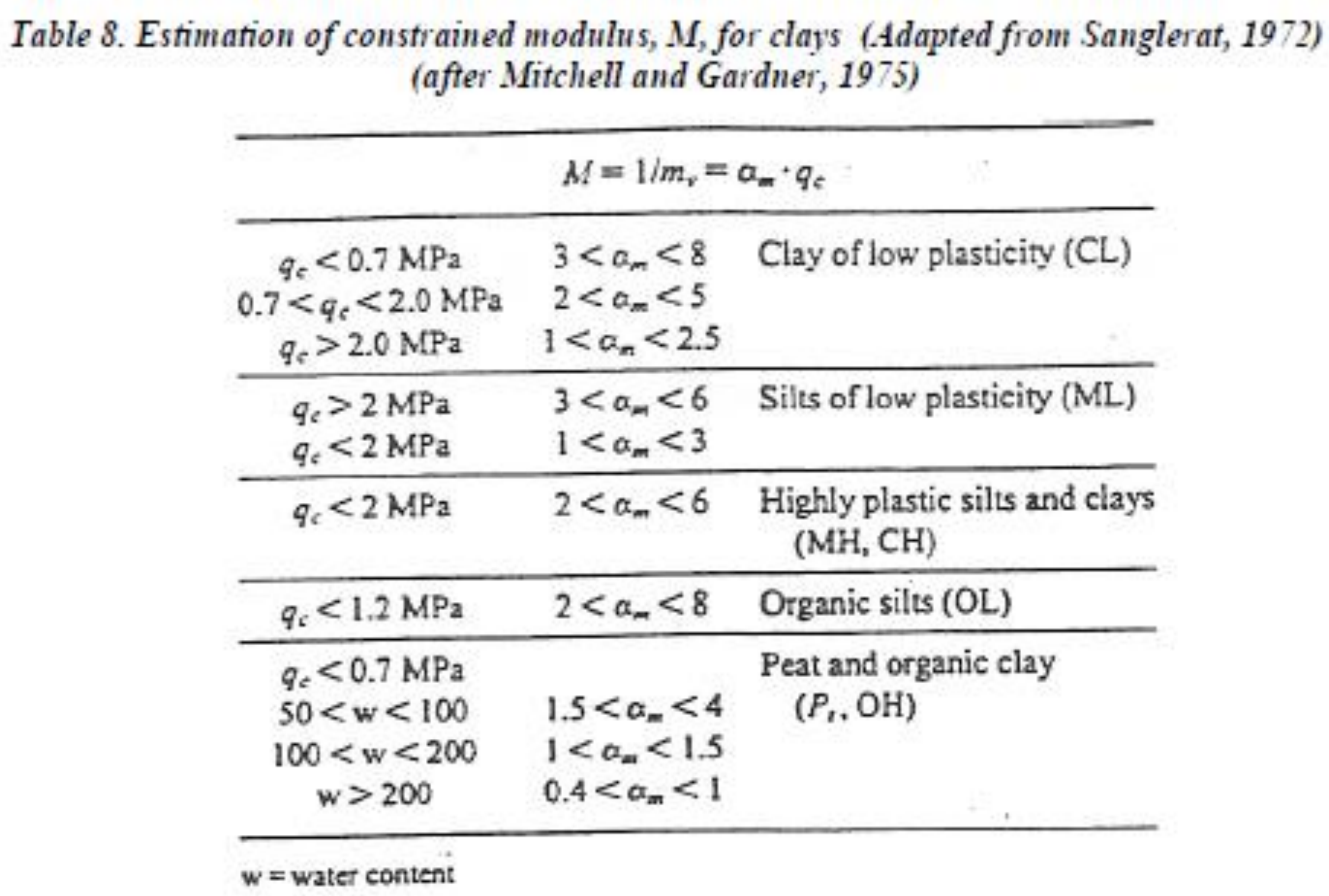
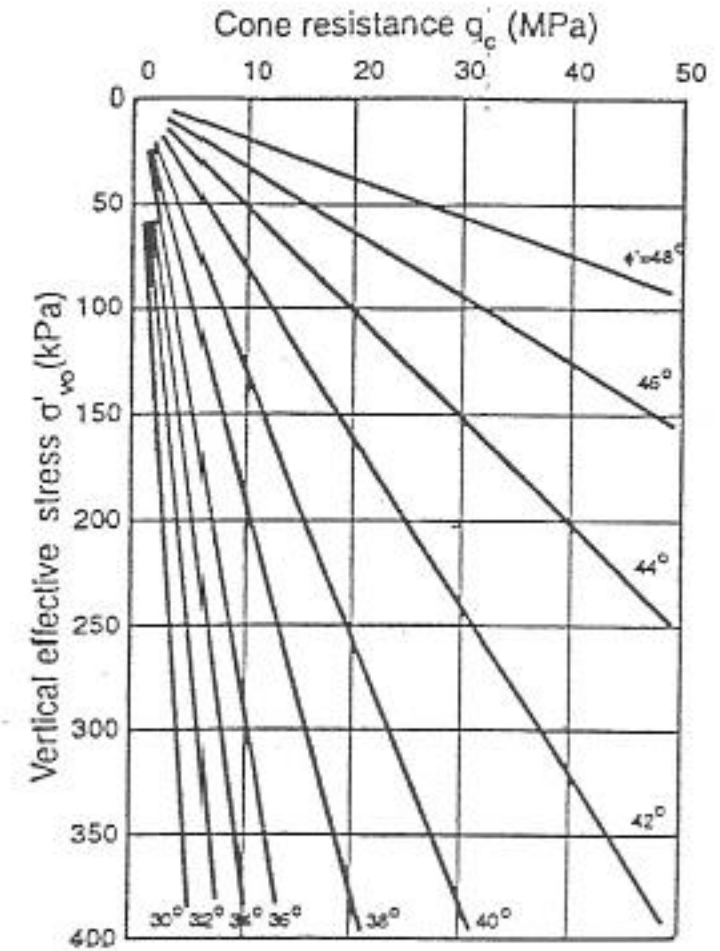


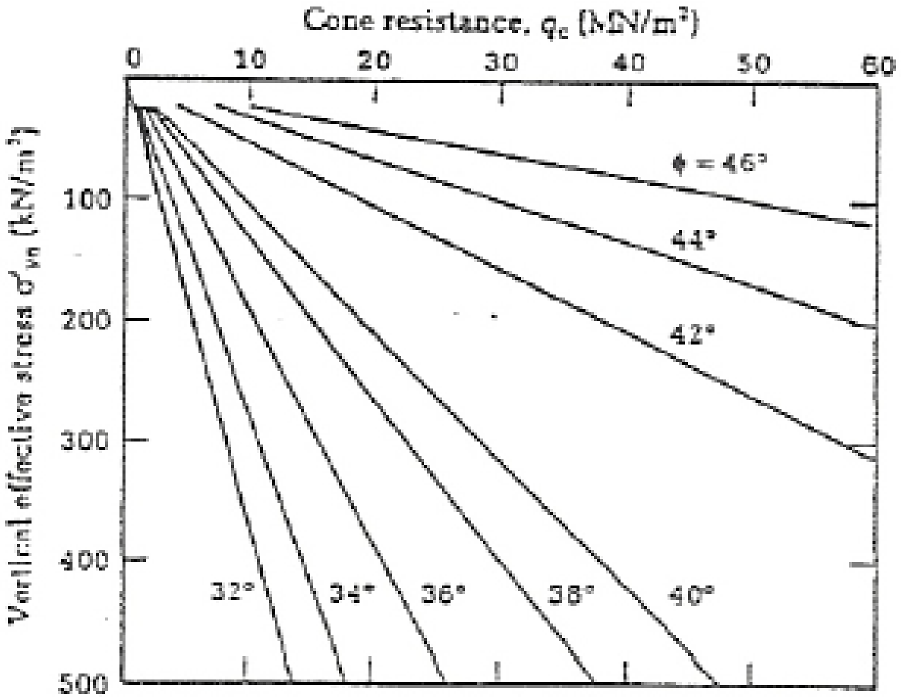
Table 8. Estimation of constrained modulus, M, for clays (Adapted from Sanglerat, 1972)
(after Mitchell and Gardner, 1975)

$M = 1/m_v = \alpha_m \cdot q_c$		
$q_c < 0.7$ MPa	$3 < \alpha_m < 8$	Clay of low plasticity (CL)
$0.7 < q_c < 2.0$ MPa	$2 < \alpha_m < 5$	
$q_c > 2.0$ MPa	$1 < \alpha_m < 2.5$	
$q_c > 2$ MPa	$3 < \alpha_m < 6$	Silts of low plasticity (ML)
$q_c < 2$ MPa	$1 < \alpha_m < 3$	
$q_c < 2$ MPa	$2 < \alpha_m < 6$	Highly plastic silts and clays (MH, CH)
$q_c < 1.2$ MPa	$2 < \alpha_m < 8$	Organic silts (OL)
$q_c < 0.7$ MPa		Peat and organic clay (P_t, OH)
$50 < w < 100$	$1.5 < \alpha_m < 4$	
$100 < w < 200$	$1 < \alpha_m < 1.5$	
$w > 200$	$0.4 < \alpha_m < 1$	

w = water content

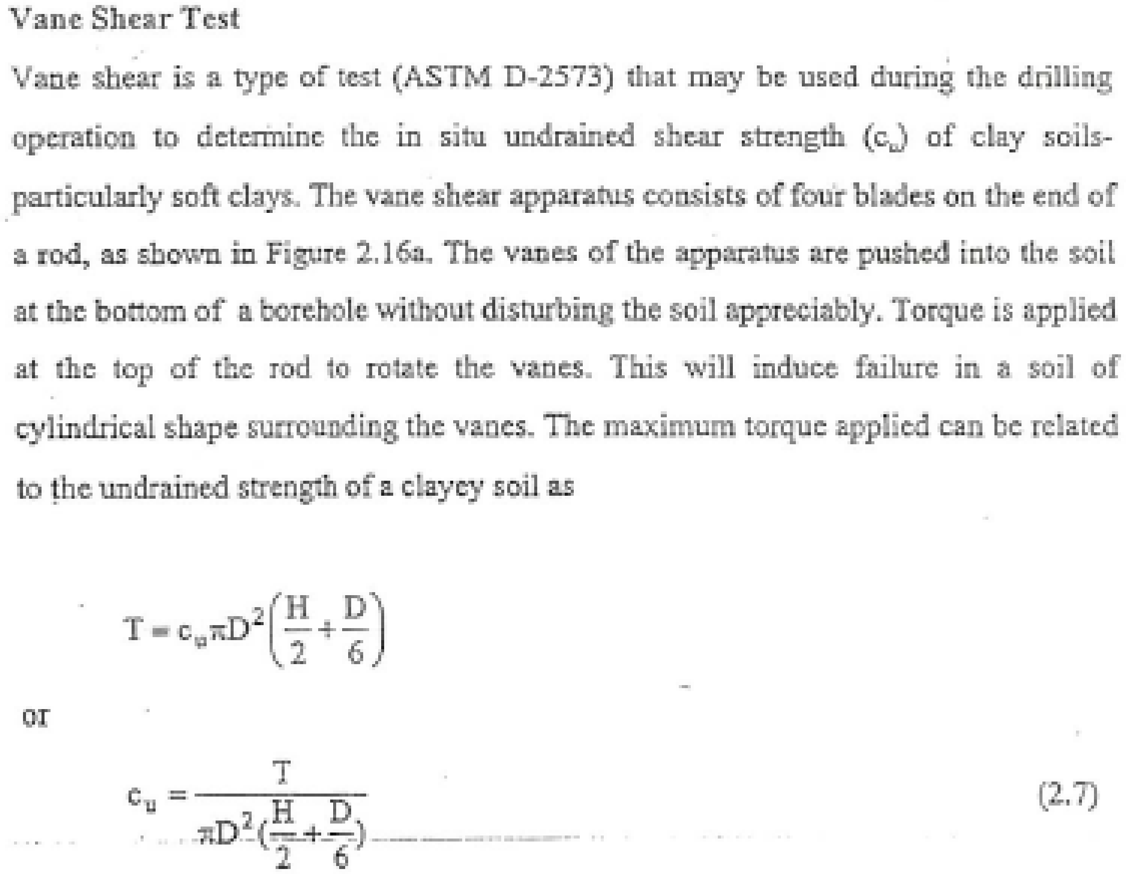


σ'_{vo}, q_c, Ø′ relationships (after Robertson and Campenella, 1983b)



gure 25.Relationship between angle of shearing resistance and cone resistance for an uncemented, normally consolidated quartz sand (after Durgunoğlu and Mitchell)

Vane Shear Test



Vane Shear Test

Vane shear is a type of test (ASTM D-2573) that may be used during the drilling operation to determine the in situ undrained shear strength (c_u) of clay soils-particularly soft clays. The vane shear apparatus consists of four blades on the end of a rod, as shown in Figure 2.16a. The vanes of the apparatus are pushed into the soil at the bottom of a borehole without disturbing the soil appreciably. Torque is applied at the top of the rod to rotate the vanes. This will induce failure in a soil of cylindrical shape surrounding the vanes. The maximum torque applied can be related to the undrained strength of a clayey soil as

$$T = c_u \pi D^2 \left(\frac{H}{2} + \frac{D}{6} \right)$$

or

$$c_u = \frac{T}{\pi D^2 (\frac{H}{2} + \frac{D}{6})} \qquad (2.7)$$

Compressibility Parameters

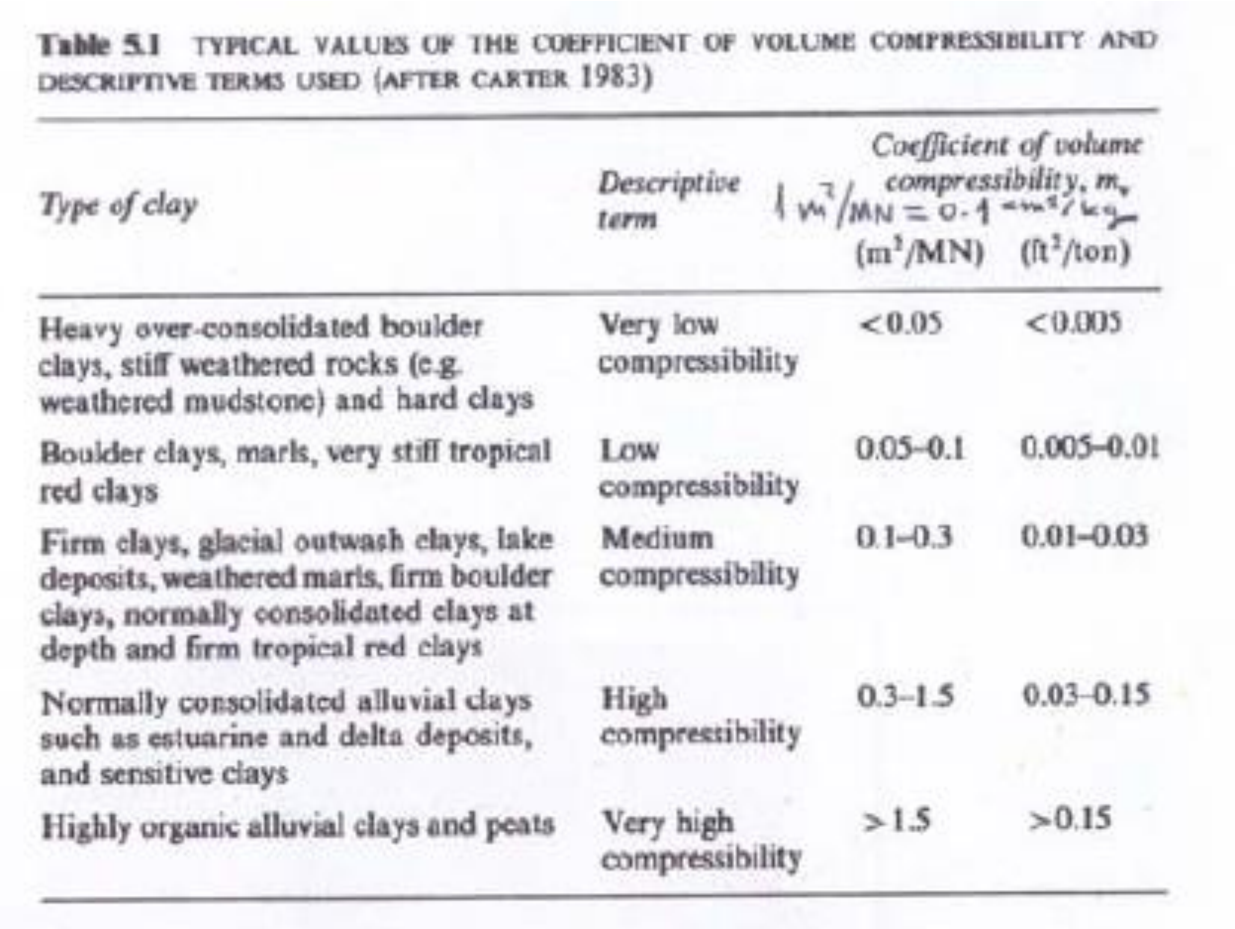


Table 5.1 TYPICAL VALUES OF THE COEFFICIENT OF VOLUME COMPRESSIBILITY AND DESCRIPTIVE TERMS USED (AFTER CARTER 1983)

Type of clay	*Descriptive term*	*Coefficient of volume compressibility, m_v* 1 m²/MN = 0.1 cm²/kg (m²/MN)	(ft²/ton)
Heavy over-consolidated boulder clays, stiff weathered rocks (e.g. weathered mudstone) and hard clays	Very low compressibility	<0.05	<0.005
Boulder clays, marls, very stiff tropical red clays	Low compressibility	0.05–0.1	0.005–0.01
Firm clays, glacial outwash clays, lake deposits, weathered marls, firm boulder clays, normally consolidated clays at depth and firm tropical red clays	Medium compressibility	0.1–0.3	0.01–0.03
Normally consolidated alluvial clays such as estuarine and delta deposits, and sensitive clays	High compressibility	0.3–1.5	0.03–0.15
Highly organic alluvial clays and peats	Very high compressibility	>1.5	>0.15

Compressibility Parameters

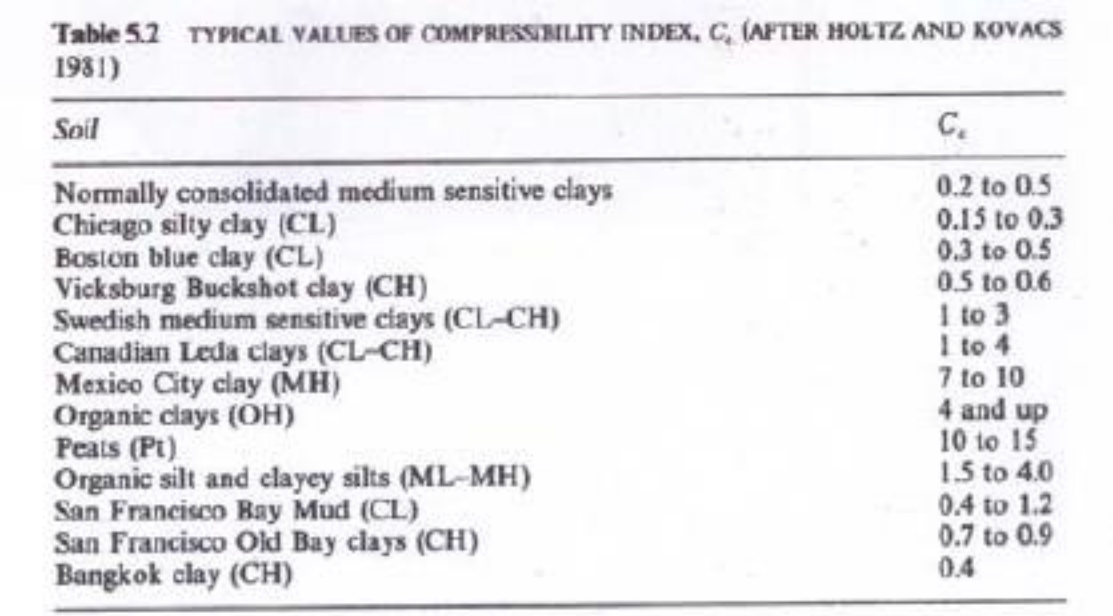


Table 5.2 TYPICAL VALUES OF COMPRESSIBILITY INDEX, C_c (AFTER HOLTZ AND KOVACS 1981)

Soil	C_c
Normally consolidated medium sensitive clays	0.2 to 0.5
Chicago silty clay (CL)	0.15 to 0.3
Boston blue clay (CL)	0.3 to 0.5
Vicksburg Buckshot clay (CH)	0.5 to 0.6
Swedish medium sensitive clays (CL–CH)	1 to 3
Canadian Leda clays (CL–CH)	1 to 4
Mexico City clay (MH)	7 to 10
Organic clays (OH)	4 and up
Peats (Pt)	10 to 15
Organic silt and clayey silts (ML–MH)	1.5 to 4.0
San Francisco Bay Mud (CL)	0.4 to 1.2
San Francisco Old Bay clays (CH)	0.7 to 0.9
Bangkok clay (CH)	0.4

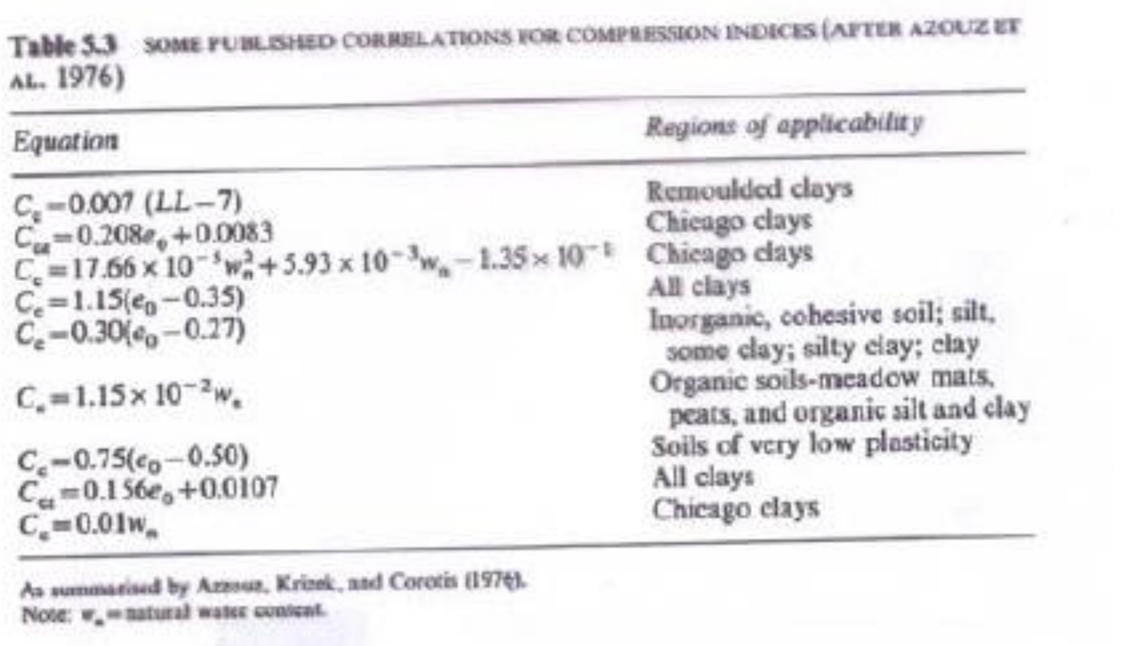
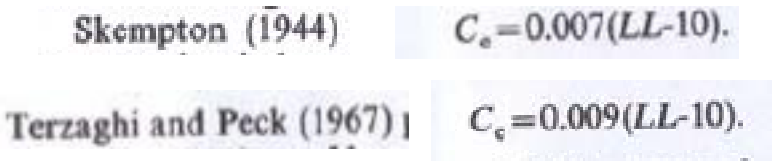


Table 5.3 SOME PUBLISHED CORRELATIONS FOR COMPRESSION INDICES (AFTER AZOUZ ET AL. 1976)

Equation	*Regions of applicability*
$C_c=0.007\ (LL-7)$	Remoulded clays
$C_{ce}=0.208e_0+0.0083$	Chicago clays
$C_c=17.66\times10^{-5}w_n^2+5.93\times10^{-3}w_n-1.35\times10^{-1}$	Chicago clays
$C_c=1.15(e_0-0.35)$	All clays
$C_c=0.30(e_0-0.27)$	Inorganic, cohesive soil; silt, some clay; silty clay; clay
$C_c=1.15\times10^{-2}w_n$	Organic soils-meadow mats, peats, and organic silt and clay
$C_c=0.75(e_0-0.50)$	Soils of very low plasticity
$C_{ce}=0.156e_0+0.0107$	All clays
$C_c=0.01w_n$	Chicago clays

As summarised by Azzouz, Krizek, and Corotis (1976).
Note: w_n = natural water content.



Skempton (1944) $C_c=0.007(LL\text{-}10)$.

Terzaghi and Peck (1967) $C_c=0.009(LL\text{-}10)$.