This chapter is a short summary of "Geoteknik Mühendisliğinde Saha Deneyleri" (Erol and Çekinmez, 2014). Please refer to the book for more detailed information and references.

CHAPTER 1

SOIL CHARACTERIZATION

1.1. Introduction

In order to solve geotechnical problems first of all soil behavior must be modelled accurately. Since 1900's different researchers developed various numerical and theoretical methods to represent realistic soil behavior. Recently there are various methods that can precisely model even complex soil behavior of different soil types. Before using either the numerical or theoretical methods, soil parameters must be accurately determined to define the exact in-situ behavior of it. For this two main approaches are valid:

(i) Sampling and laboratory tests → parameters are calculated (direct approach)
(ii) In-situ tests → parameters are calculated by means of correlations (indirect approach)

Indeed, both approaches are mutually complementary approaches.

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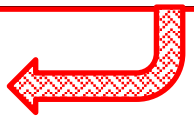
Why do we use indirect approach ?

Laboratoty tests are composed of two main group tests:

	Soil Classification Tests	**Strength and Compressibility Tests**
Examples:	Sieve Analysis, Hydrometer, Atterberg Limit Tests, Specific Gravity Test, Dry Density Test, Maximum and Minimum Void Ratio Tests, Proctor Test, etc.	Direct Shear Box Test, Triaxial Tests, Unconfined Compression Test, Consolidation (Oedometer) Test, Swell Tests, etc.
Sample Type:	Disturbed sample	Undisturbed sample
Time-Cost:	Fast and economic tests	Slow and expensive tests



- For cohesionless soils: it is difficult to obtain high quality undisturbed samples.
- For cohesive soils: obtaining sufficient amount of results from strength and compressibility tests takes too long time.

Thus, for both cohesionless and cohesive soils various empirical methods (correlations) were developed.
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How the correlation methods are developed?
For a site and specific soil type, laboratory tests and in-situ tests are performed. Using statistical methods (regression etc.) correlation is developed in order to get laboratory test results from the results of in-situ tests.
}

Be careful before using these correlations! Not every correlation can represent each soil type. Correlations developed for different soils under different in-situ conditions. For example a correlation developed for behavior of London Clay may not realistically represent Ankara Clay. Moreover, the standard deviations of these correlations are high which the accuracy of these is another controversial issue. Even for same soil type, different correlations calculate different values for the parameters. In addition, mostly used static and dynamic penetration tests apply higher strains during the test process than the strain levels that soil encounter during soil-structure interaction.

1.2. Standard Penetration Test (SPT)

1.2.1. Soil Classification

Clayton (1993) proposed a simple way to classify soil/rock depending on the SPT-N value.

Table 1.1. Soil and rock classification from SPT-N values (Clayton, 1993)

Soil Type		***SPT-N***	***Soil Classification***
Sand	$(N_1)_{60}$	0 – 3	Very loose
		3 – 8	Loose
		8 – 25	Medium dense
		25 – 42	Dense
		42 – 58	Very dense
Clay	N_{60}	0 – 4	Very soft
		4 – 8	Soft
		8 – 15	Medium Stiff
		15 – 30	Stiff
		30 – 60	Very stiff
		> 60	Hard
Weak rock	N_{60}	0 – 80	Very weak
		80 – 200	Weak
		> 200	Medium weak to very stiff

1.2.2. Cohesionless Soils

1.2.2.1. Relative Density (D_r)

Terzaghi and Peck (1967), proposed a general approach for the relation between SPT-N and D_r (Table 1.2).

Table 1.2. SPT-N - D_r relation (Terzaghi and Peck, 1967)

SPT-N Range	***Soil Classification***	***D_r (%)***
0 – 4	Very loose	0 – 15
4 – 10	Loose	15 – 35
10 – 30	Medium dense	35 – 65
30 – 50	Dense	65 – 85
> 50	Very dense	85 – 100

Coduto (2000) showed the difference between correlations proposed by Holtz and Gibbs (1979) and Bazaraa (1967) in Figure 1.1.

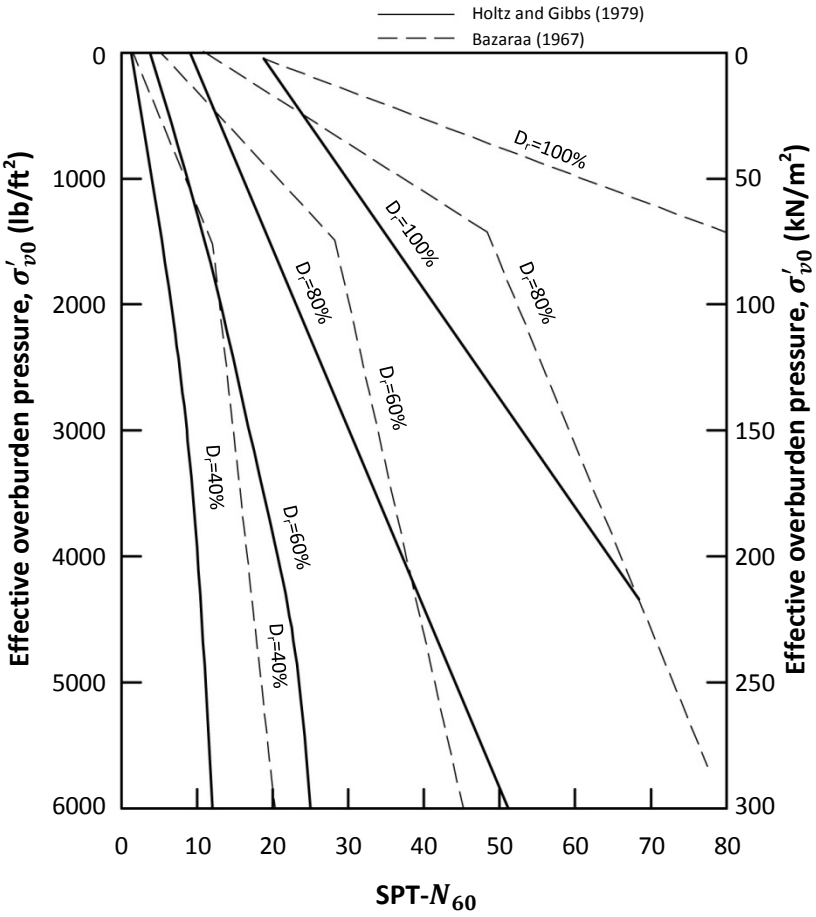


Figure 1.1. Comparison between correlations proposed by Bazaraa (1967) and Holtz and Gibbs (1979) (Coduto, 2000)

1.2.2.2. Angle of Shearing Resistance (ϕ')

Stroud (1988) revised the correlation which was previously proposed by Peck et al. (1974). In Figure 1.2 relation between SPT-$(N_1)_{60}$ - ϕ' is given depending on overconsolidation ratio (OCR). Here, NC: normally consolidated soils.

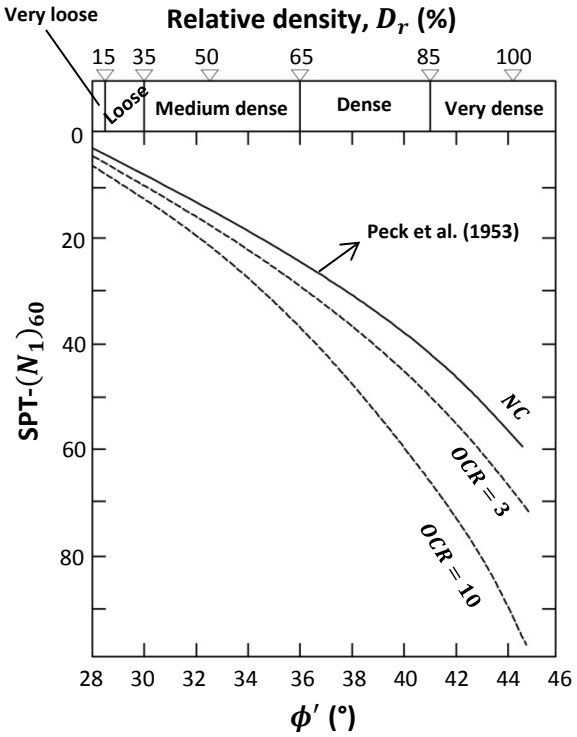


Figure 1.2. Relation between SPT-$(N_1)_{60}$ - ϕ' - OCR (Stroud, 1988)

Bowles (1996) proposed relation between SPT-$(N_1)_{70}$ – relative density (D_r) – angle of shearing resistance (ϕ') – saturated unit weight (γ_s) as given in Table 1.3. Relation between relative density and ϕ' values can also be represented by the following analytic equation. Here, D_r is in decimal.

$$\phi'(°) = 28° + 15°D_r(\mp 2°)$$

Table 1.3. Relation between SPT-$(N_1)_{70}$ - D_r - ϕ' - γ_s (Bowles, 1996)

Definition		*Very loose*	*Loose*	*Medium dense*	*Dense*	*Very dense*
D_r		0	0.15	0.35	0.65	0.85
$(N_1)_{70}$	**Fine**	1 – 2	3 – 6	7 – 15	16 – 30	-
	Medium	2 – 3	4 – 7	8 – 20	21 – 40	> 40
	Coarse	3 – 6	5 – 9	10 – 25	26 – 45	> 45
ϕ'	**Fine**	26 – 28	28 – 30	30 – 34	33 – 38	< 50
	Medium	27 – 28	30 – 32	32 – 36	36 – 42	
	Coarse	28 – 30	30 – 34	33 – 40	40 – 50	
$\gamma_s(kN/m^3)$		11 – 16	14 – 18	17 – 20	17 – 22	20 – 23

Schmertmann (1975) proposed relation between σ'_{v0} - SPT-N - ϕ' as shown in Figure 1.3.

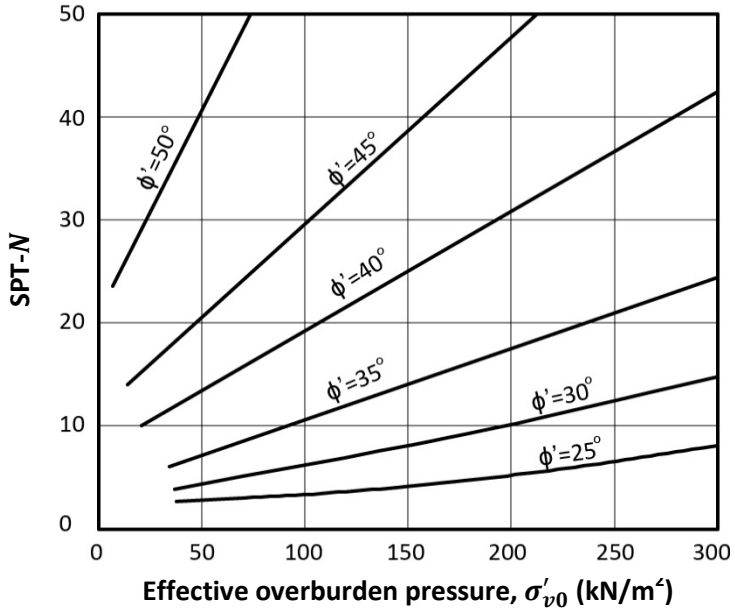


Figure 1.3. Relation between σ'_{v0} - SPT-N - ϕ' (Schmertmann, 1975)

1.2.2.3. Deformation Modulus (E_s)

Stroud (1988) stated that relation between SPT-N – E_s depends on the load level (q_{net}/q_{ult}) applied on the soil. By using a wide database, Stroud (1988) proposed relation between SPT-N – E_s – (q_{net}/q_{ult}) for both cohesion and cohesionless soils (Figure 1.4). Here, q_{net}: net foundation pressure and q_{ult}: net ultimate bearing capacity.

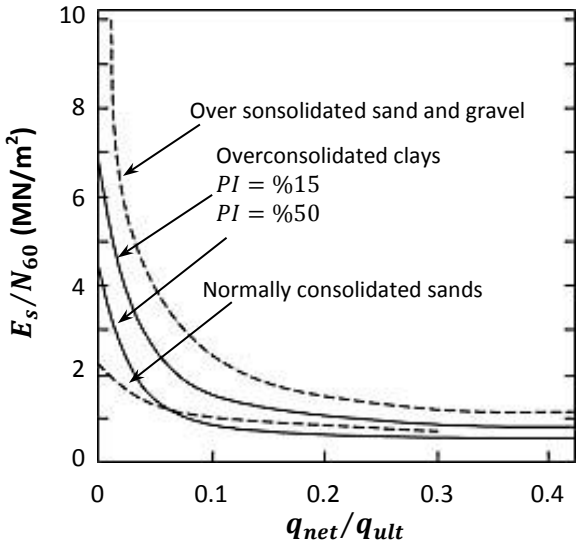


Figure 1.4. Relation between E_s - SPT-N – (q_{net}/q_{ult}) (Stroud, 1988)

Kulhawy and Mayne (1990) proposed correlations for sandy soils as shown in the following equation set.

$$E_s\ (kN/m^2) = \begin{Bmatrix} Silty, clayey\ sands: & 500N_{60} \\ Clean\ sands: & 1000N_{60} \\ Over\ consolidated\ clean\ sands: & 1500N_{60} \end{Bmatrix}$$

Bowles (1996) summarized the various correlations developed for different soil types as given in Table 1.4. Correlations listed in Table 1.4 are given for normally consolidated sands and must be multiplied by $(OCR)^{0.5}$ for overconsolidated sands.

Table 1.4. Correlations between SPT-N_{55} – E_s (Bowles, 1996)

Soil Type	***E_s (kN/m^2)***
NC Sands	$500(N_{55}+15)$ $7000\sqrt{N_{55}}$ $6000N_{55}$ $(15000-22000)\ln N_{55}$ $(2600-2900)N_{55}$
Saturated sands	$250(N_{55}+15)$
Gravelly sands	$1200(N_{55}+6)$ $600(N_{55}+6)$ (For $N_{55} \leq 15$) $2000+600(N_{55}+6)$ (For $N_{55} > 15$)
Clayey sands	$320(N_{55}+15)$
Silts, sandy silts, clayey silts	$300(N_{55}+6)$

FHWA (2002a) proposed the correlations listed in Table 1.5.

Table 1.5. Correlations between SPT-N – E_s (FHWA, 2002a)

Soil Type	***E_s (kN/m^2)***
Silt, sandy silt, low cohesive mix soils	$400(N_1)_{60}$
Clean fine-medium sand and low silty sands	$700(N_1)_{60}$
Coarse sand and gravelly sands	$1000(N_1)_{60}$
Sandy gravels	$1200(N_1)_{60}$

1.2.3. Cohesive Soils

1.2.3.1. Undrained Shear Strength (c_u)

Kulhawy and Mayne (1990) stated that relation between (c_u/p_a) and SPT-N given in Table 1.6 can approximately be represented by $(c_u/p_a) = 0.06\ N$.

Table 1.6. Relation between SPT-N – c_u (Terzaghi and Peck, 1967)

SPT-N	*Consistency*	*Approximate c_u/p_a ratio*
0 – 2	Very soft	< 1/8
2 – 4	Soft	1/8 – 1/4
4 – 8	Medium stiff	1/4 – 1/2
8 – 15	Stiff	1/2 – 1
15 – 30	Very stiff	1 – 2
> 30	Hard	> 2

p_a: atmospheric pressure ≅ 100 kN/m^2

Database comprised of 1200 SPT-N values from 42 different sites in U.K. by Stroud (1974). In this study, unconsolidated undrained (UU) triaxial tests were performed on high quality undisturbed samples in order to obtained actual undrained shear strength values of clayey soils. As a result Stroud (1974) reported that for clayey soils with plasticity index (PI) in range of 25 – 60 and SPT-N values in range of 10 – 60 blow/30 cm, undrained shear strength values obtained from UU tests are in range of 25 – 500 kN/m^2. Stroud (1974) proposed a correlation between f_1 and PI as shown in Figure 1.5 where f_1 is a factor depending on plasticity index and,

$$c_u(kN/m^2) = f_1 * N_{60}$$

The above equation can also be approximately represented by the following equation set.

$$c_u(kN/m^2) = \begin{Bmatrix} PI < 20: & (6-7)\,N_{60} \\ 20 < PI < 30: & (4-5)\,N_{60} \\ PI > 30: & 4.2\,N_{60} \end{Bmatrix}$$

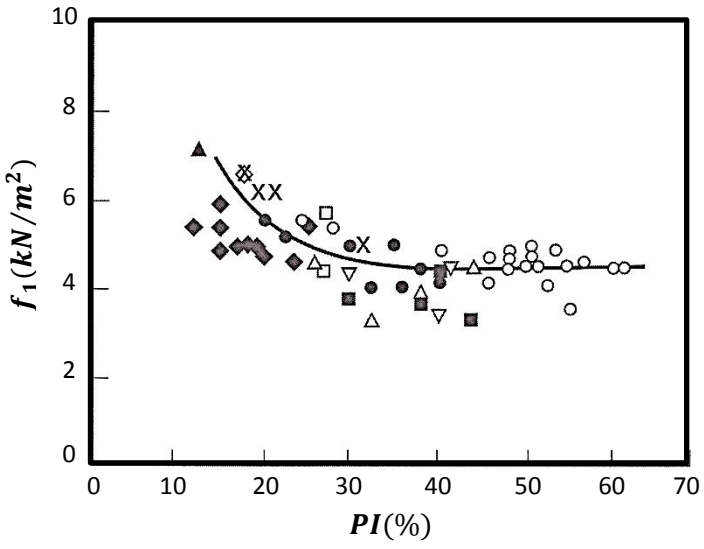


Figure 1.5. Relation between SPT-N_{60} - c_u - PI (Stroud, 1974)

1.2.3.2. Oedometric Deformation Modulus (M)

Based on the previously stated study, Stroud (1974) proposed a correlation between f_2 and PI for the relation between coefficient of volumetric compression (m_v) and SPT-N value as shown in Figure 1.6. Here, f_2 is a factor depending on plasticity index and

$$M(kN/m^2) = \frac{1}{m_v} = f_2 * N_{60}$$

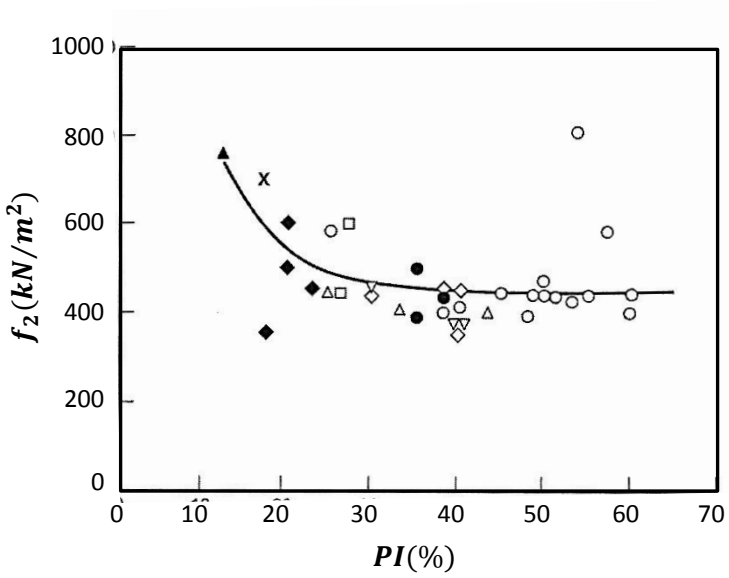


Figure 1.6. Relation between SPT-N_{60} - m_v - PI (Stroud, 1974)

1.3. Cone Penetration Test (CPT)

1.3.1. Soil Classification

Begemann (1965) proposed a soil classification chart depending on the sleeve friction (f_s) and cone resistance (q_c) as shown in Figure 1.7.

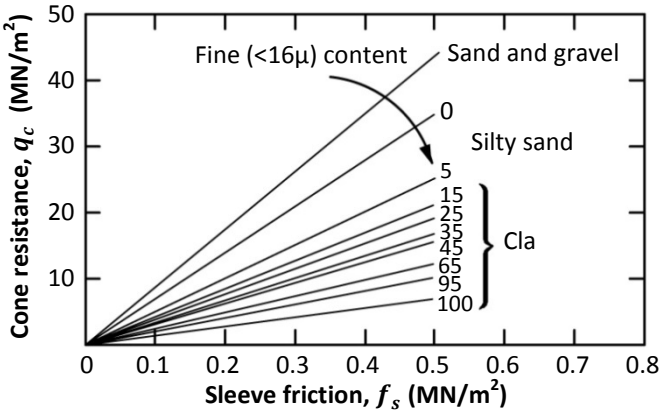
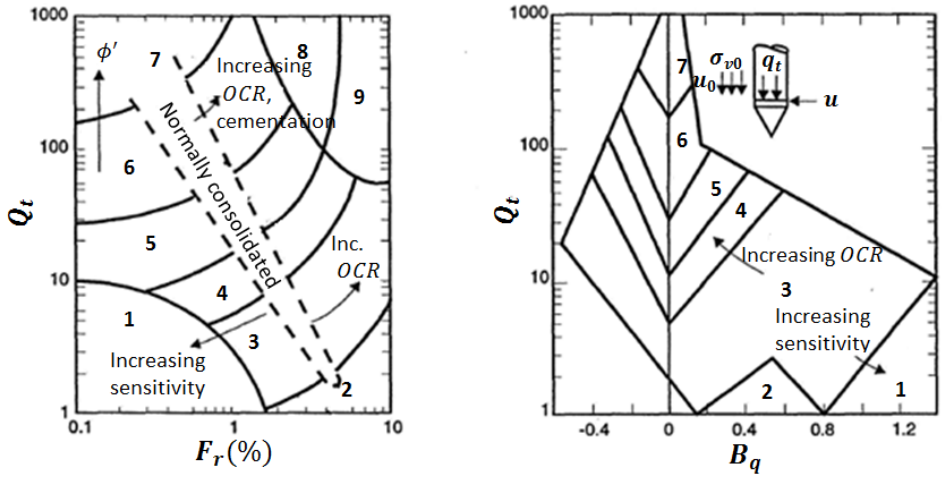


Figure 1.7. Soil classification (Begemann, 1965)

Robertson (1990) stated that similar with SPT-N values also cone resistance values would increase with depth in same type of soil. Hence, cone resistance values must be corrected for overburden pressure. Based on this approach Robertson (1990) proposed a soil classification chart as shown in Figure 1.8. In this chart, for CPT and CPTU tests the left-hand side and right-hand side charts must be used, respectively.



Zone	*Soil Behavior Type*				
1.	Sensitive, fine grained	4.	Clayey silt – silty clay	7.	Gravelly sand – sand
2.	Organic soil – peat	5.	Silty sand – sandy silt	8.	Very stiff sand – clayey sand*
3.	Clay – silty clay	6.	Clean sand – silty sand	9.	Very stiff fine grained*

***Over consolidated or cemented**

Figure 1.8. Soil behavior classification (Robertson, 1990)

In Figure 1.8; Q_t is normalized cone resistance, F_r is normalized friction ratio, B_q is pore pressure ratio and,

$$Q_t = (q_t - \sigma_{v0})/\sigma'_{v0}$$

and

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

Where; q_t: corrected total cone resistance, σ_{v0} and σ'_{v0}: total and effective overburden pressure at depth of soil investigated, respectively, a: area ratio (A_n/A_c).

$$F_r(\%) = [f_s/(q_t - \sigma_{v0})] * 100$$

$$B_q = \Delta u/q_n = (u_2 - u_0)/(q_t - \sigma_{v0})$$

{Please read pages 85 – 90 in the book for details.}

Robertson (1990) also defined a soil behavior index (I_m) in order to define the soil behavior analytically by using the following equation:

$$I_m = [(3.47 - \log Q_t)^2 + (\log F_r + 1.22)^2]^{0.5}$$

In Table 1.7, I_m values for different soil behaviors defined in Figure 1.8 are given.

Table 1.8. Ranges of I_m for different types of soil behavior (Robertson, 1990)

Zone	*Type of Soil Behavior*	I_m
1	Sensitive , fine grained	-
2	Organic soil – peat	> 3.6
3	Silty clay – clay	2.95 – 3.6
4	Clayey silt – silty clay	2.60 – 2.95
5	Silty sand – sandy silt	2.05 – 2.6
6	Clean sand – silty sand	1.31 – 2.05
7	Gravelly sand – dense sand	< 1.31
8	Very stiff sand – clayey sand*	-
9	Very stiff fine grained*	-

*Over consolidated or cemented

1.3.2. Cohesive Soils

1.3.2.1. Undrained Shear Strength (c_u)

Based on the bearing capacity theory, during cone penetration test bearing capacity is fully mobilized and relation between bearing capacity and undrained shear strength according to total stress approach is:

$$q_c = c_u N_k + \sigma_{v0}$$

Where, N_k: cone factor.

Fugro (2004) proposed the following values for N_k:

For normally consolidated clays, $N_k = 15 - 16$
For over consolidated clays, $N_k = 18 - 19$

1.3.2.2. Oedometric Deformation Modulus (M)

Sanglerat (1972) proposed a correlation coefficient (α_m) in Table 1.9 for the relation between coefficient of volumetric compression (m_v) and CPT-q_c value as shown in the following equation:

$$M = 1/m_v = \alpha_m \times q_c$$

Table 1.9. α_m coefficients (Sanglerat, 1972)

Range of q_c values	α_m values	Soil Type
$q_c < 0.7\ MN/m^2$ $0.7\ MN/m^2 < q_c < 2.0\ MN/m^2$ $q_c > 2.0\ MN/m^2$	$3 < \alpha_m < 8$ $2 < \alpha_m < 5$ $1 < \alpha_m < 2.5$	Low plasticity clay (CL)
$q_c > 2\ MN/m^2$ $q_c < 2\ MN/m^2$	$3 < \alpha_m < 6$ $1 < \alpha_m < 3$	Low plasticity silt (ML)
$q_c < 2\ MN/m^2$	$2 < \alpha_m < 6$	High plasticity silt and clay (MH ve CH)
$q_c < 1.2\ MN/m^2$	$2 < \alpha_m < 8$	Organic silt (OL)
$q_c < 0.7\ MN/m^2$ $50 < w < 100$ $100 < w < 200$ $w > 200$	 $1.5 < \alpha_m < 4$ $1 < \alpha_m < 1.5$ $0.4 < \alpha_m < 1$	Peat and organic clay (Pt, OH)

Where; w: natural water content (%).

1.3.3. Cohesionless Soils

1.3.3.1. Relative Density (D_r)

Jamiolkowski et al. (1985) proposed a correlation between D_r - q_c as shown in Figure 1.9 by using the results of calibration chamber test on various types of sand.

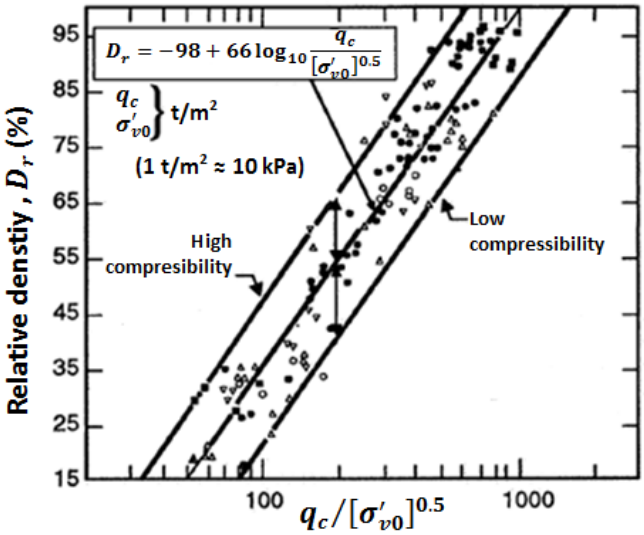


Figure 1.9. Relation between q_c - σ'_{v0} - D_r (Jamiolkowski et al., 1985)

1.3.3.2. Angle of Shearing Resistance (ϕ')

Robertson and Campanella (1983) performed calibration chamber and drained triaxial tests on uncemented and medium compressible clean silica sands and proposed a correlation between effective overburden stress (σ'_{v0}), cone resistance (q_c) and ϕ' as shown in Figure 1.10.

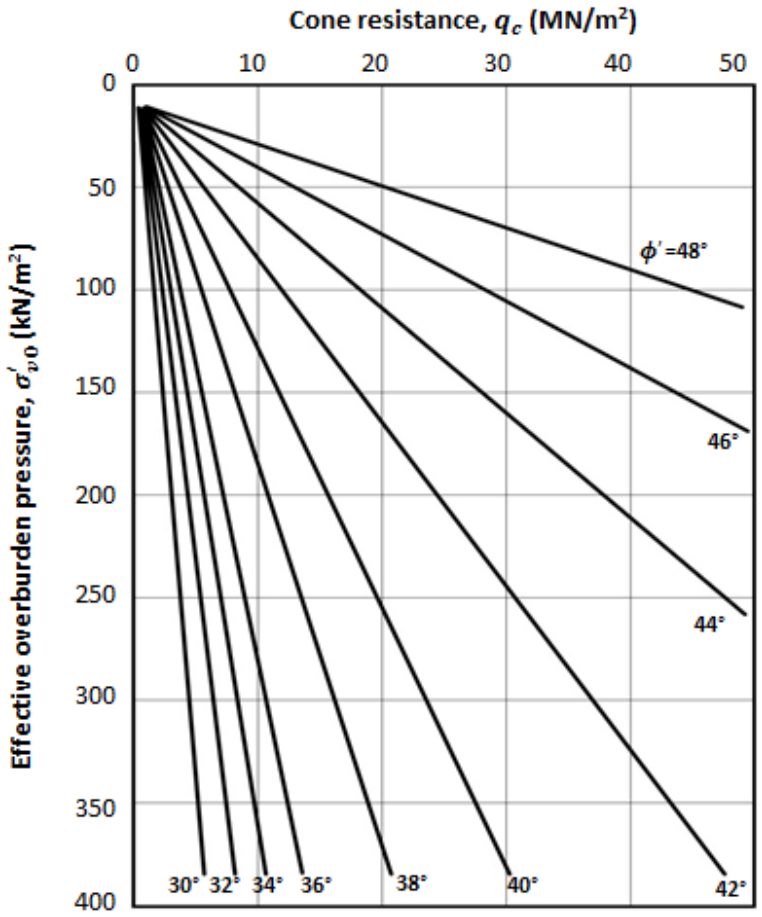


Figure 1.10. Relation between q_c - ϕ'- σ'_{v0} (Robertson and Campanella, 1983)

1.3.3.3. Deformation Modulus (E_s)

Deformation modulus of cohesionless soils depends on relative density of soil, over consolidation ratio of soil and effective overburden pressure on the soil.

Robertson and Campanella (1983) proposed a correlation between cone resistance and deformation modulus for two different strain levels: E_{25} and E_{50} depending on the effective overburden pressure as shown in Figure 1.11.

By using results calibration chamber test, Lunne and Christophersen (1983) proposed a correlation between M_0 and q_c as shown in Figure 1.12. The correlation can also represented by the following equation sets:

NC Sands,

$$\begin{aligned} q_c &< 10\ MN/m^2 \quad & M_0 &= 4q_c \\ 10\ MN/m^2 < q_c &< 50\ MN/m^2 \quad & M_0 &= 2q_c + 20\ (MN/m^2) \\ q_c &> 50\ MN/m^2 \quad & M_0 &= 120\ MN/m^2 \end{aligned}$$

OC Sands,

$$\begin{aligned} q_c &< 50\ MN/m^2 \quad & M_0 &= 5q_c \\ q_c &> 50\ MN/m^2 \quad & M_0 &= 250q_c \end{aligned}$$

Lunne and Christophersen (1983) also stated that if there is a stress increase at an amount of $\Delta\sigma_v$ than for range of stress within $(\sigma'_{v0}) - (\sigma'_{v0} + \Delta\sigma_v)$ a correlation proposed by Janbu (1963) is valid for M:

$$M = M_0[(\sigma'_{v0} + 0.5\Delta\sigma_v)/\sigma'_{v0}]^{0.5}$$

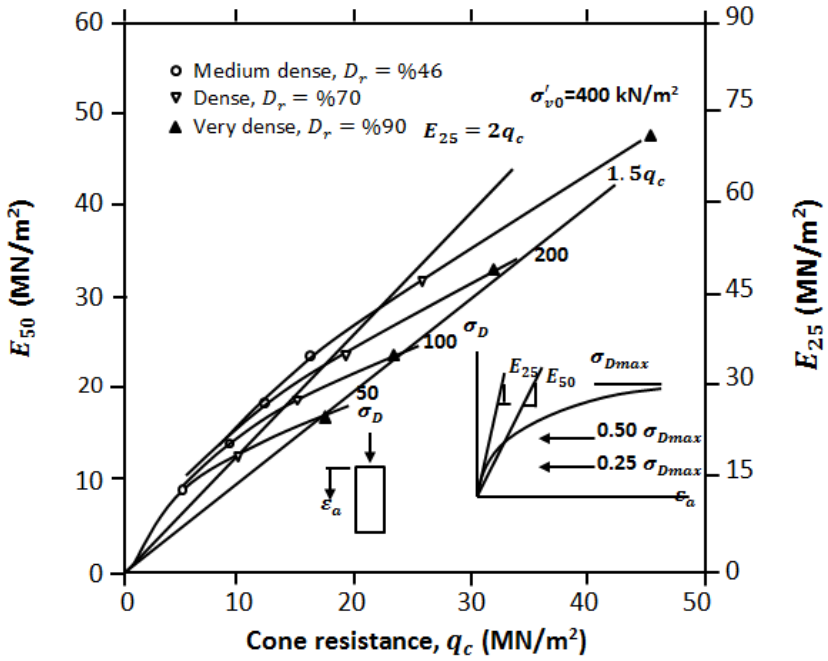


Figure 1.11. Relation between E_{25} - q_c ve E_{50} - q_c (Robertson and Campanella, 1983)

Robertson et al. (2010) proposed the following equation for sands under axial strain at a level of 0.1%:

$$E_s = \alpha_E(q_t - \sigma_{v0})$$

and

$$\alpha_E = 0.015\left[10^{(0.55I_m + 1.68)}\right]$$

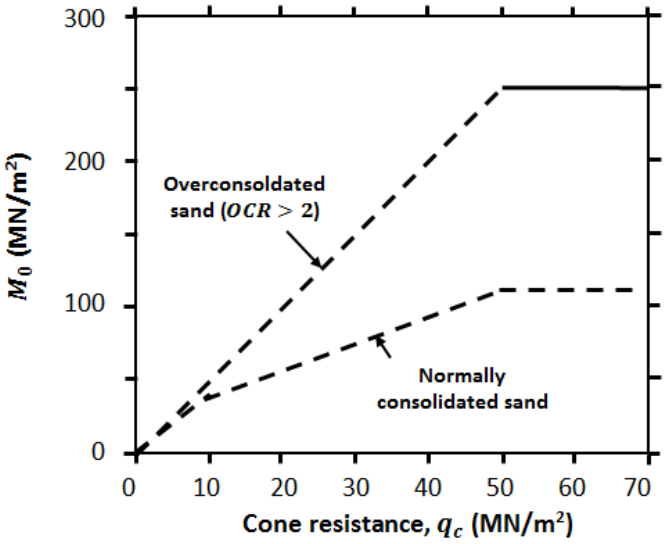


Figure 1.12. Relation between M_0 - q_c (Lunne and Christoffersen, 1983)

1.4. Pressuremeter Test (PMT)

1.4.1. Soil Classification

Briaud (1992) stated that for the determination of soil whether cohesive or cohesionless ratio of E_M/P_{LN} is an indicator:

$$\text{Cohesive soils:} \quad (E_M/P_{LN}) > 12$$
$$\text{Cohesionless soils:} \quad 7 < (E_M/P_{LN}) < 12$$

Briaud (1992) proposed the criteria given in Table 1.10 for the determination of soil type and compressibility/consistency conditions depending on the limit pressure (P_L) values. Moreover, Biraud (1992) stated that the net limit pressure (P_{LN}) and Menard's modulus (E_M) values are in the ranges given in Table 1.11 for different soil types.

Table 1.11. Soil classification depending on P_L values (Briaud, 1992)

Soil		***P_L (kN/m²)***	***SPT-N***	****c_u (kN/m²)***
Sand	Loose	0 – 500	0 – 10	
	Medium dense	500 – 1500	10 – 30	
	Dense	1500 – 2500	30 – 50	
	Very dense	> 2500	> 50	
Clay	Soft	0 – 200		0 – 25
	Medium stiff	200 – 400		25 – 50
	Stiff	400 – 800		50 – 100
	Very stiff	800 – 1600		100 – 200
	Hard	> 1600		> 200

*c_u : undrained shear strength

Table 1.12. P_{LN} and E_M ranges for various soil types (Briaud, 1992)

CLAY					
Soil Type	***Soft***	***Medium Stiff***	***Stiff***	***Very stiff***	***Hard***
P_{LN} (kN/m^2)	0 – 200	200 – 400	400 – 800	800 – 1600	> 1600
E_M (MN/m^2)	0 – 2.5	2.5 –5.0	5.0 – 12.0	12.0 – 25.0	> 25.0

SAND				
Soil Type	***Loose***	***Medium dense***	***Dense***	***Very dense***
P_{LN} (kN/m^2)	0 – 500	500 – 1500	1500 – 2500	> 2500
E_M (MN/m^2)	0 – 3.5	3.5 – 12.0	12.0 – 22.5	> 22.5

1.4.2. Undrained Shear Strength (c_u)

Baguelin et al. (1978) proposed a correlation between c_u and P_{LN} as shown in Figure 1.13. Same correlation can also be represented by the following equation.

$$\frac{c_u}{p_a} = 0.21\left[\frac{P_{LN}}{p_a}\right]^{0.75}$$

Where; p_a is atmospheric pressure (≈100 kN/m^2). The accuracy of this correlation is low as seen in Figure 1.13.

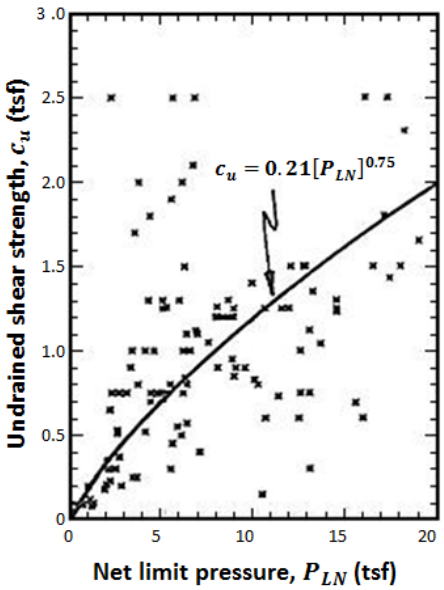


Figure 1.13. Relation between c_u - P_{LN} (p_a= 1 tsf ≈ 100 (kN/m^2)) (Briaud, 1992)

Another correlation is proposed by Baguelin et al. (1978) as shown in Figure 1.14.

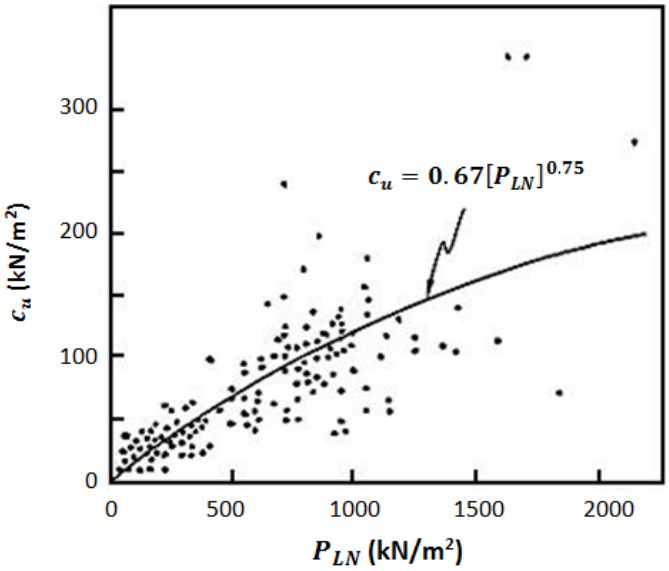


Figure 1.14. Relation between c_u - P_{LN} (Baguelin et al., 1978)

1.4.3. Oedometric Deformation Modulus (M)

Amar et al. (1991) stated that oedometric deformation modulus ($M = 1/m_v$) can be calculated by using the pressuremeter test data and the following equation.

$$M = E_M/\alpha_M$$

Where; α_M: Menard's factor (Table 1.13).

Table 1.13. Menard's α_M factor (Briaud, 1992)

Soil Type	***Peat***		***Clay***		***Silt***		***Sand***		***Sand and Gravel***	
	E_M/P_{LN}	α_M	E_M/P_{LN}	α_M	E_M/P_{LN}	α_M	E_M/P_{LN}	α_M	E_M/P_{LN}	α_M
Overconsolidated	For all values	1	> 16	1	> 14	2/3	> 12	1/2	> 10	1/3
Normally consolidated		1	9 – 16	2/3	8 – 14	1/2	7 - 12	1/3	6 – 10	1/4
Weathered and/or remoulded		1	7 – 9	1/2		1/2		1/3		1/4
Rock	Very fractured		Others						Low fractured or weathered	
	$\alpha_M = 1/3$		$\alpha_M = 1/2$						$\alpha_M = 2/3$	

1.5. Other Correlations

1.5.1. Effective Angle of Shearing Resistance (ϕ') for Cohesive Soils

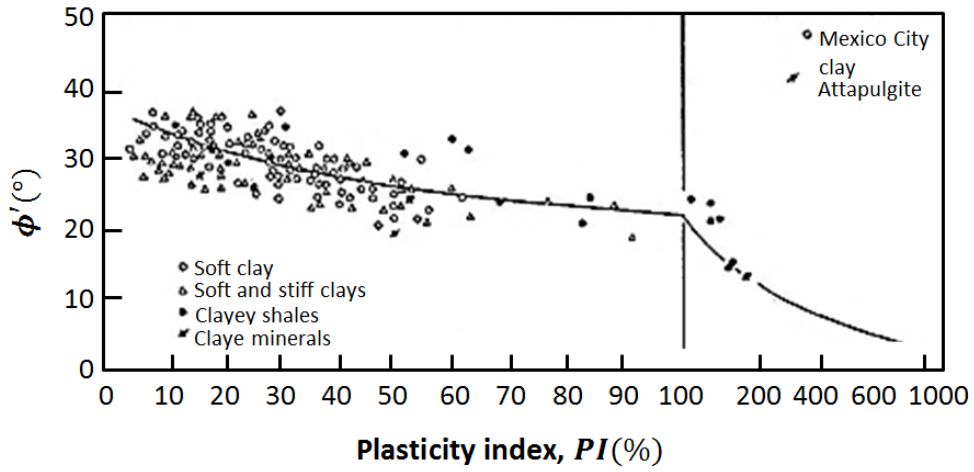


Figure 1.15. Relation between plasticity index - ϕ' (Terzaghi, Peck and Mesri, 1996)

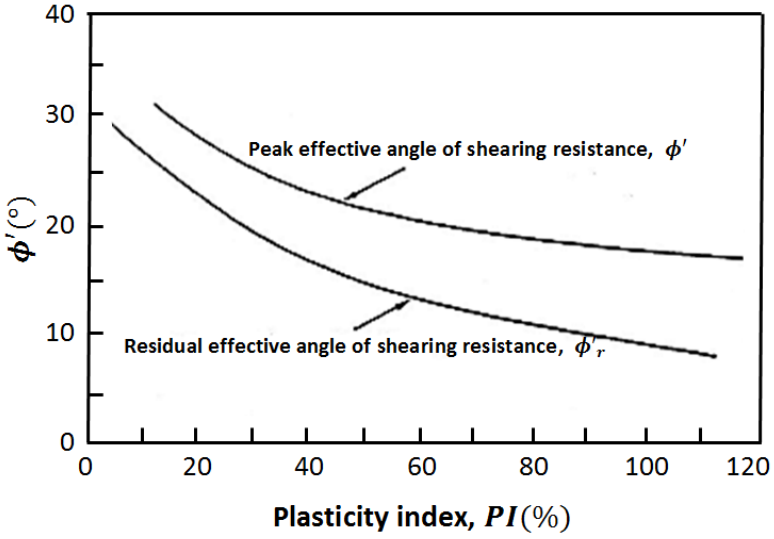


Figure 1.16. Relation between plasticity index - ϕ' (Gibson, 1953)

1.5.2. Effective Cohesion (c') for Cohesive Soils

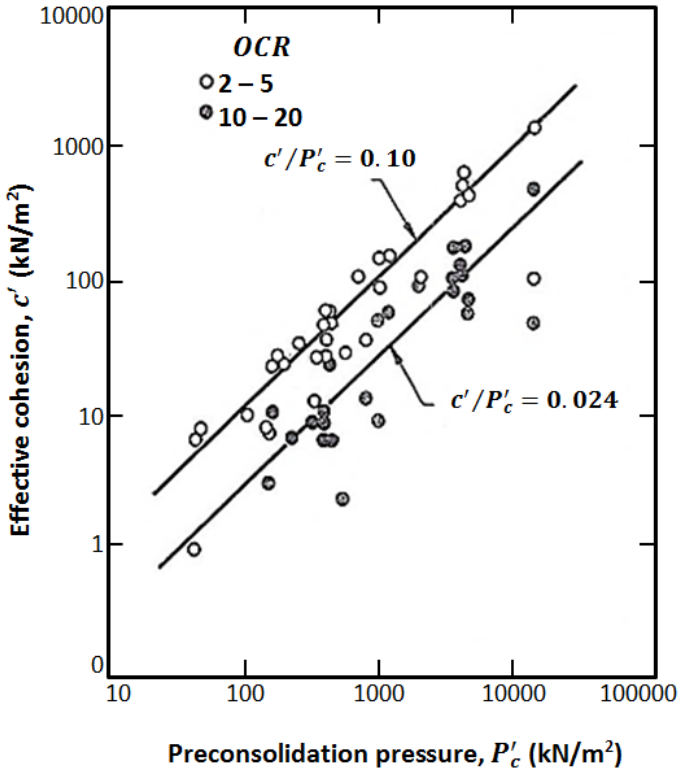


Figure 1.17. Relation between effective cohesion and preconsolidation pressure (Mesri and Abdel-Gaffar, 1993)

1.5.3. Consolidation Parameters for Cohesive Soils

Skempton (1944) and Terzaghi and Peck (1948) proposed a correlation between LL and C_c:

$$C_c = \alpha' \, (LL - 10)$$

Where; α' = 0.007 (Skempton, 1944) or α' = 0.009 (Terzaghi and Peck, 1948).

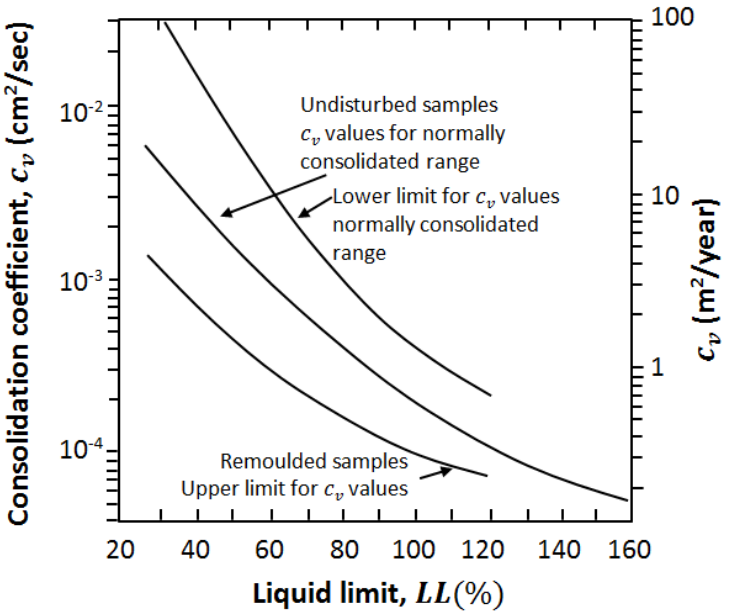
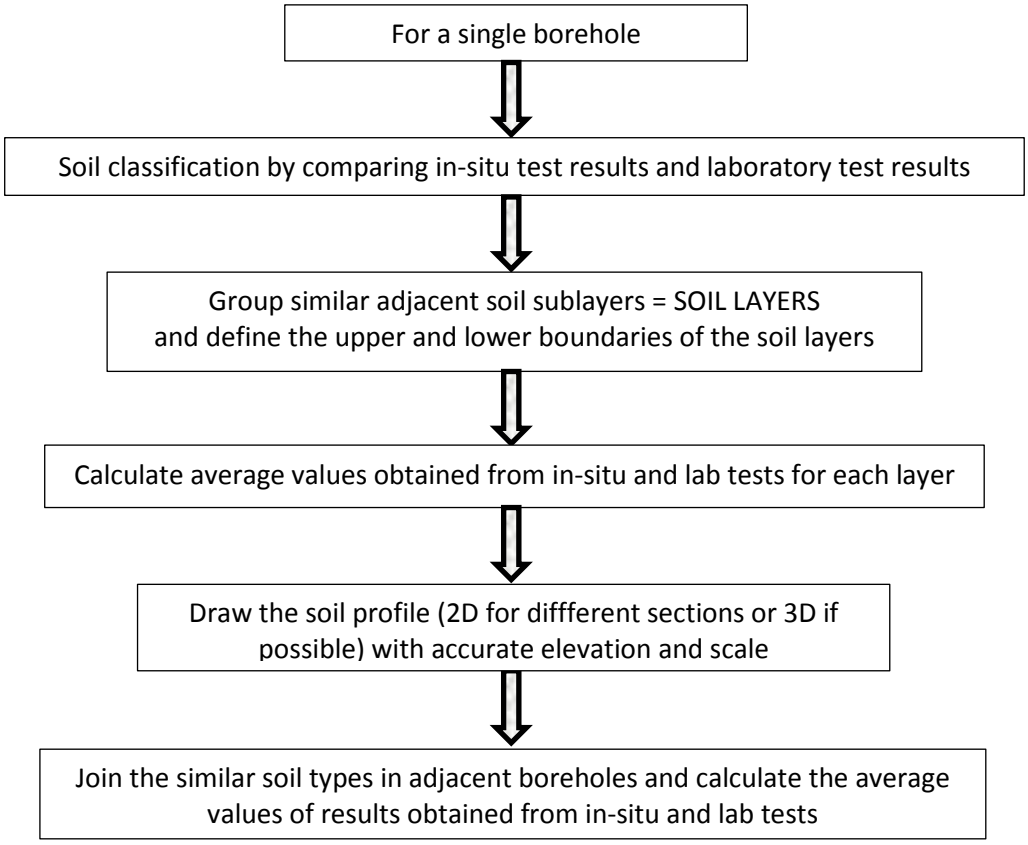
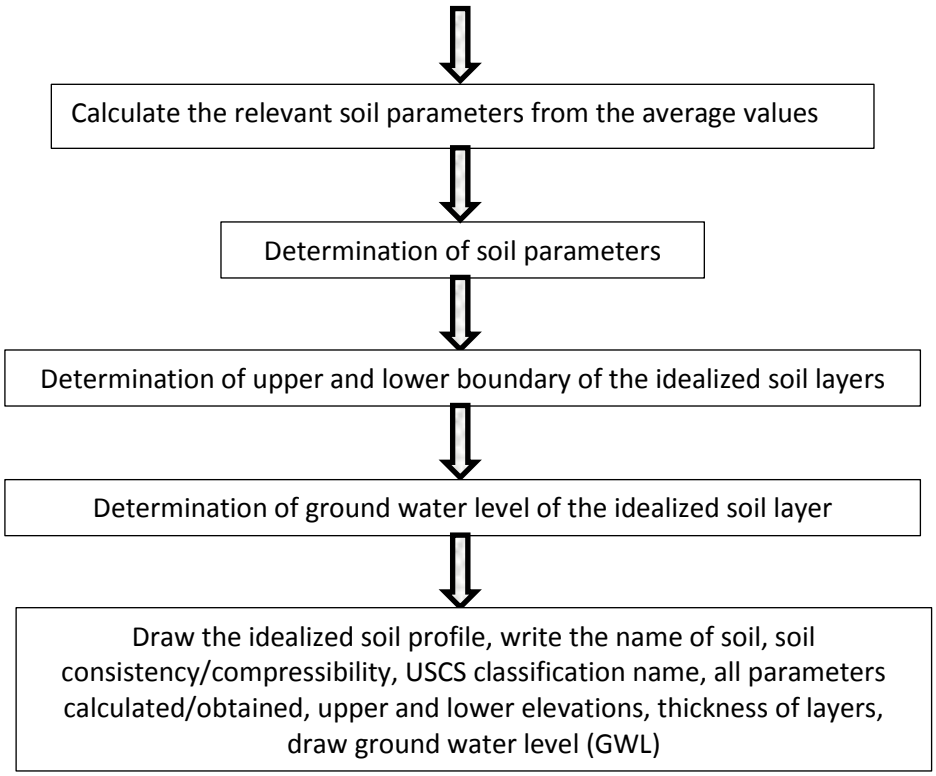


Figure 1.18. Relation between correlation coefficient and liquid limit (US Navy, 1982)

1.6. Idealized Soil Profile




Calculate the relevant soil parameters from the average values
Determination of soil parameters
Determination of upper and lower boundary of the idealized soil layers
Determination of ground water level of the idealized soil layer
Draw the idealized soil profile, write the name of soil, soil consistency/compressibility, USCS classification name, all parameters calculated/obtained, upper and lower elevations, thickness of layers, draw ground water level (GWL)