A) STANDARD PENETRATION TEST (SPT):

5. Remove the SPT sampler; remove and save the soil sample.
6. Drill the boring to the depth of the next test and repeat steps 2 through 6 as required.

Thus, N values may be obtained at intervals no closer than 18 in (450 mm).

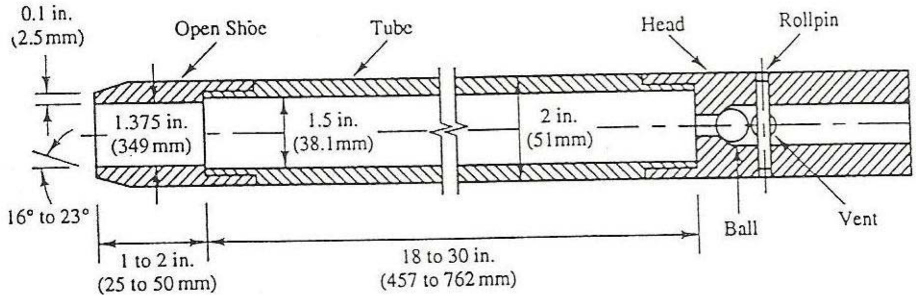


Figure 1. The SPT sampler (Adopted from ASTM D1586: Copyright ASTM, reprinted with permission)

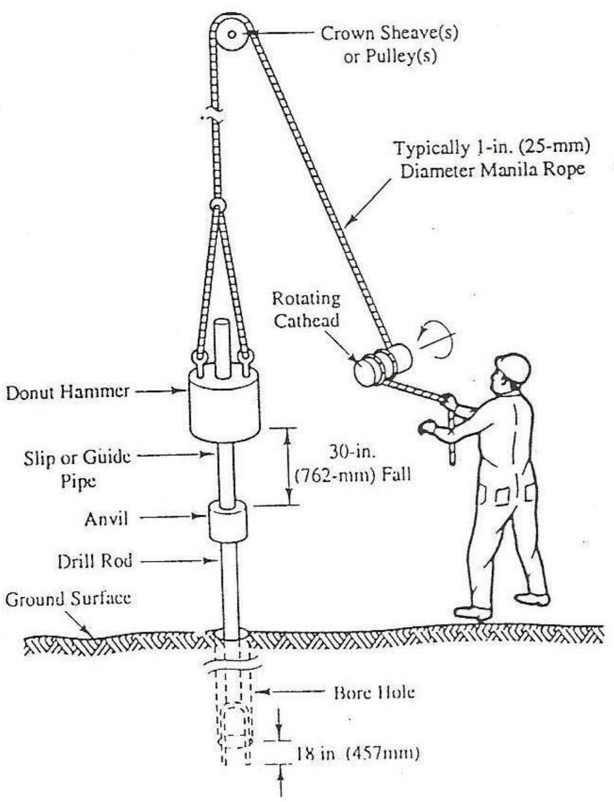


Figure 2. The SPT Sampler in place in the boring with hammer, rope and cathead in place (Adapted from Korvaes et al., 1981)

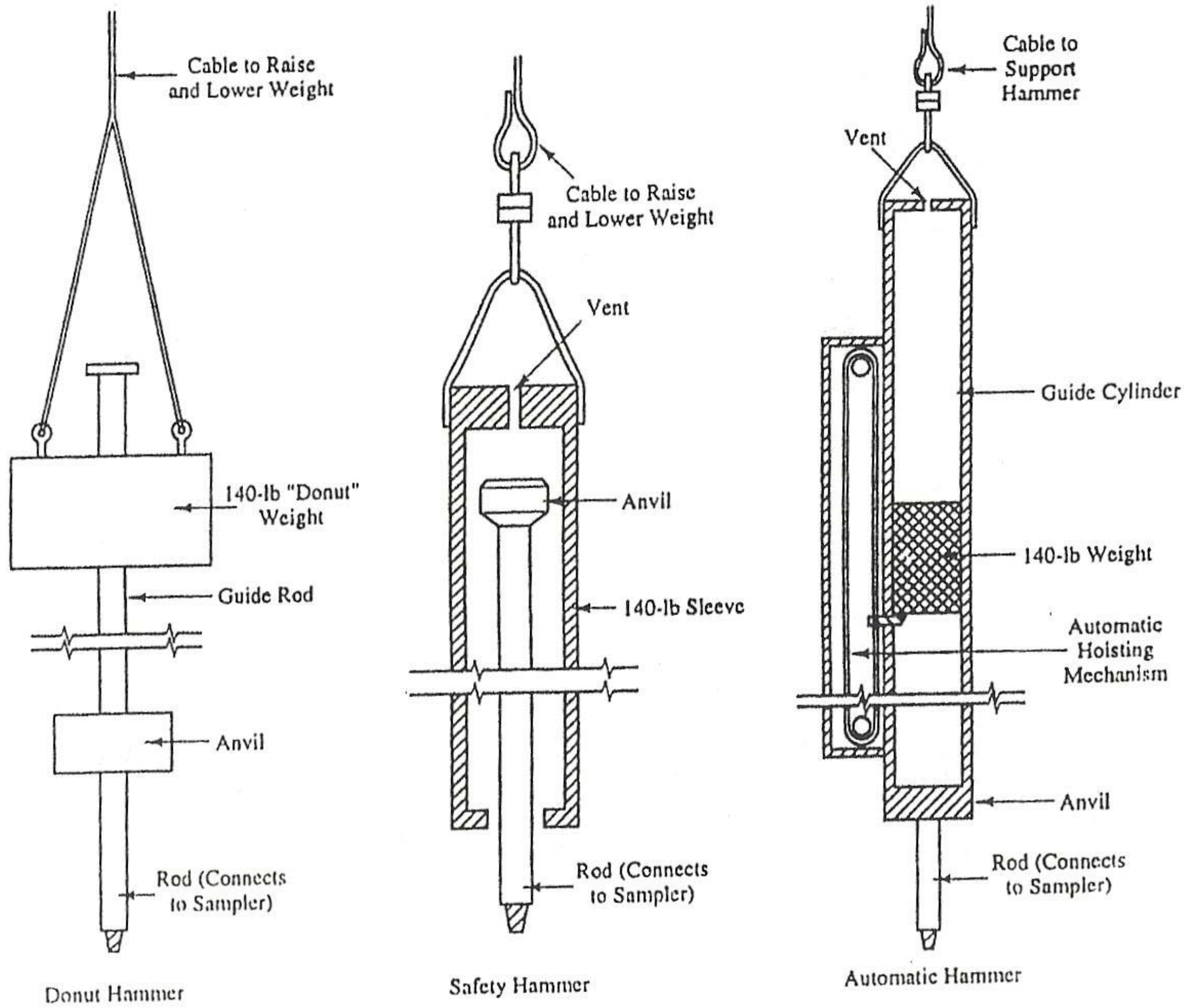


Figure 3. Types of SPT Hammers

Table 1. SPT Hammer Efficiencies

Country	Hammer Type	Hammer Release Mechanism	Hammer Efficiency E_m
Argentina	Donut	Cathead	0.45
Brazil	Pin Weight	Hand Dropped	0.72
China	Automatic	Trip	0.60
	Donut	Hand dropped	0.55
	Donut	Cathead	0.50
Colombia	Donut	Cathead	0.50
Japan	Donut	Tombi trigger	0.78 - 0.85
	Donut	Cathead 2 turns + special release	0.65 - 0.67
UK	Automatic	Trip	0.73
USA	Safety	2 turns on cathead	0.55 - 0.60
	Donut	2 turns on cathead	0.45
Venezuela	Donut	Cathead	0.43

Adapted from Clayton (1990)

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).

Hatanaka and Uchida (1996); $\phi' = \sqrt{20N} + 20^{\circ}$

$$\phi' = \sqrt{12N_{45}} + 20^{\circ}$$

A lower bound for the above equation is given as;

$$\phi' = \sqrt{12N_{45}} + 15^{\circ}$$

Table 3. Empirical Coefficients for BS 8002 ϕ' equation

A – **Angularity**[1)]	A (degrees)
Rounded	0
Sub-angular	2
Angular	4
B – **Grading of Soil**[2)]	B (degrees)
Uniform	0
Moderate grading	2
Well graded	4
C – **N'**[3)] (blows 300 mm)	C (degrees)
< 10	0
20	2
30	6
40	9

[1)] Angularity is estimated from visual description of soil.
[2)] Grading can be determined from grading curve by use of: Uniformity coefficient $=D_{60}/D_{10}$
Where D_{10} and D_{60} are particle sizes such that in the sample, 10% of the material is finer than D_{10} and 60% is finer than D_{60}.

Grading	Uniformity Coefficient
Uniform	< 2
Moderate grading	2 to 6
Well graded	> 6

A step-graded soil should be treated as uniform or moderately graded soil according to the grading of the finer fraction.
[3)] N' from results of standard penetration test modified where necessary for overburden pressure.
Intermediate values of A, B and C by interpolation.

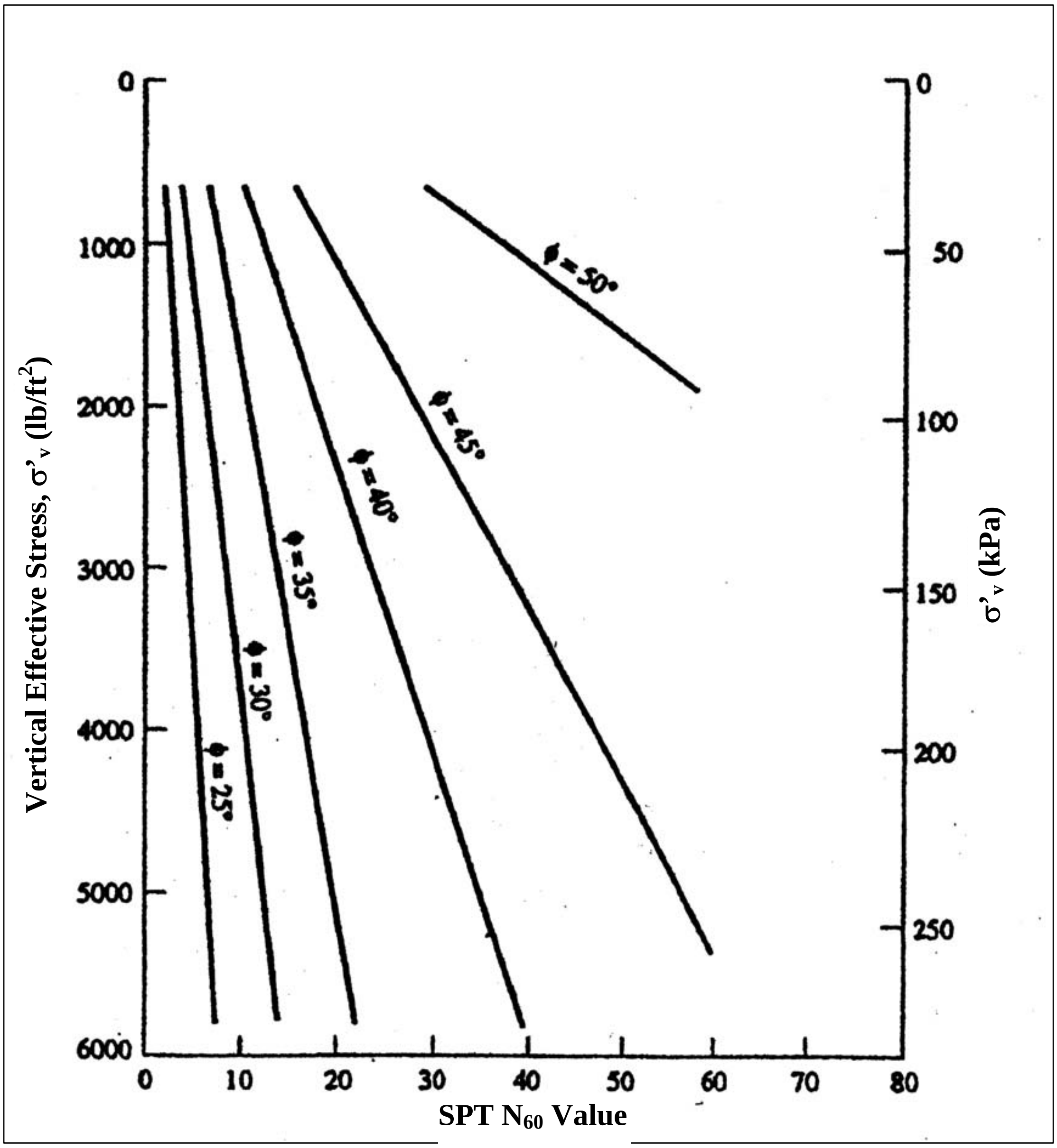


Figure 4. Empirical Correlation between N_{60} and ϕ for uncemented sands (Adapted from DeMello, 1971)

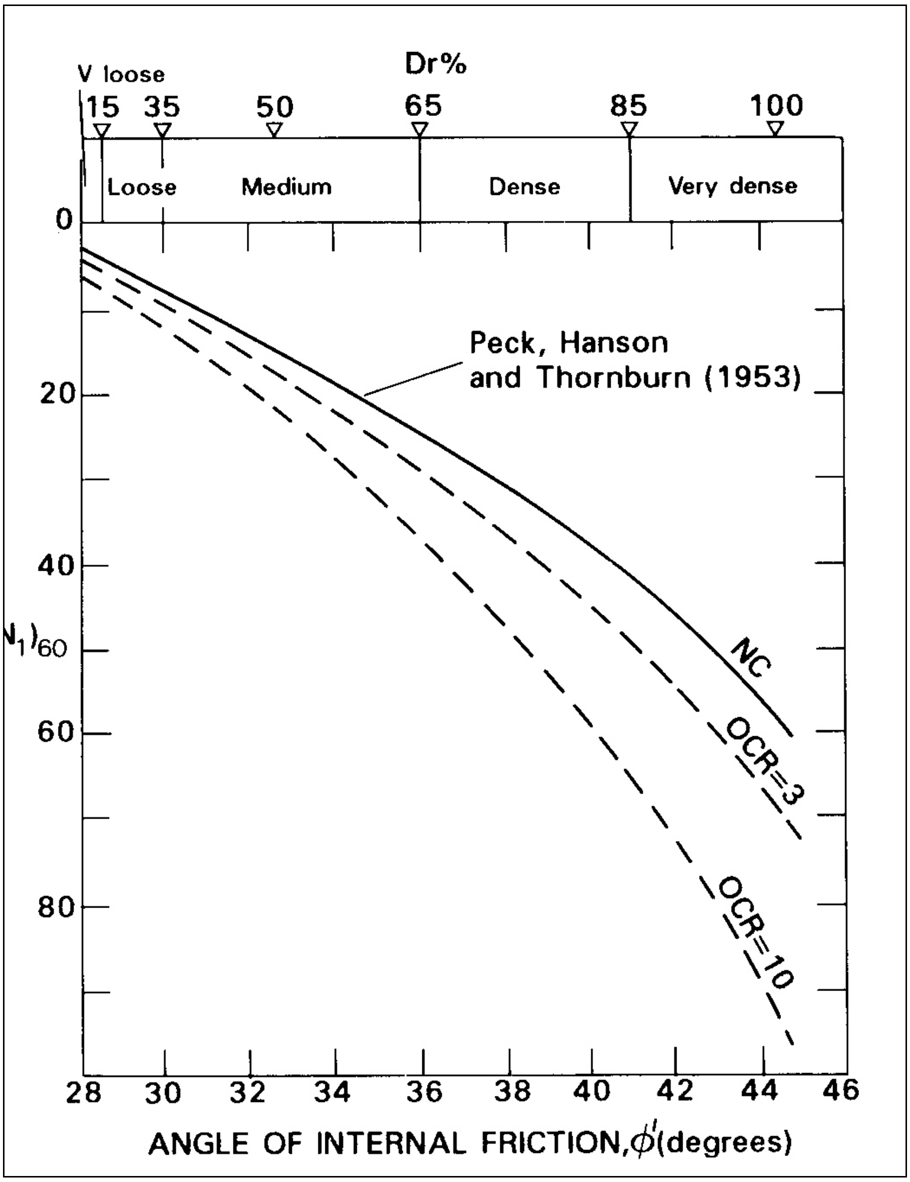


Figure 5. Effect of Overconsolidation Ratio on the Relationship between $(N_1)_{60}$ and Angle of Friction ϕ'

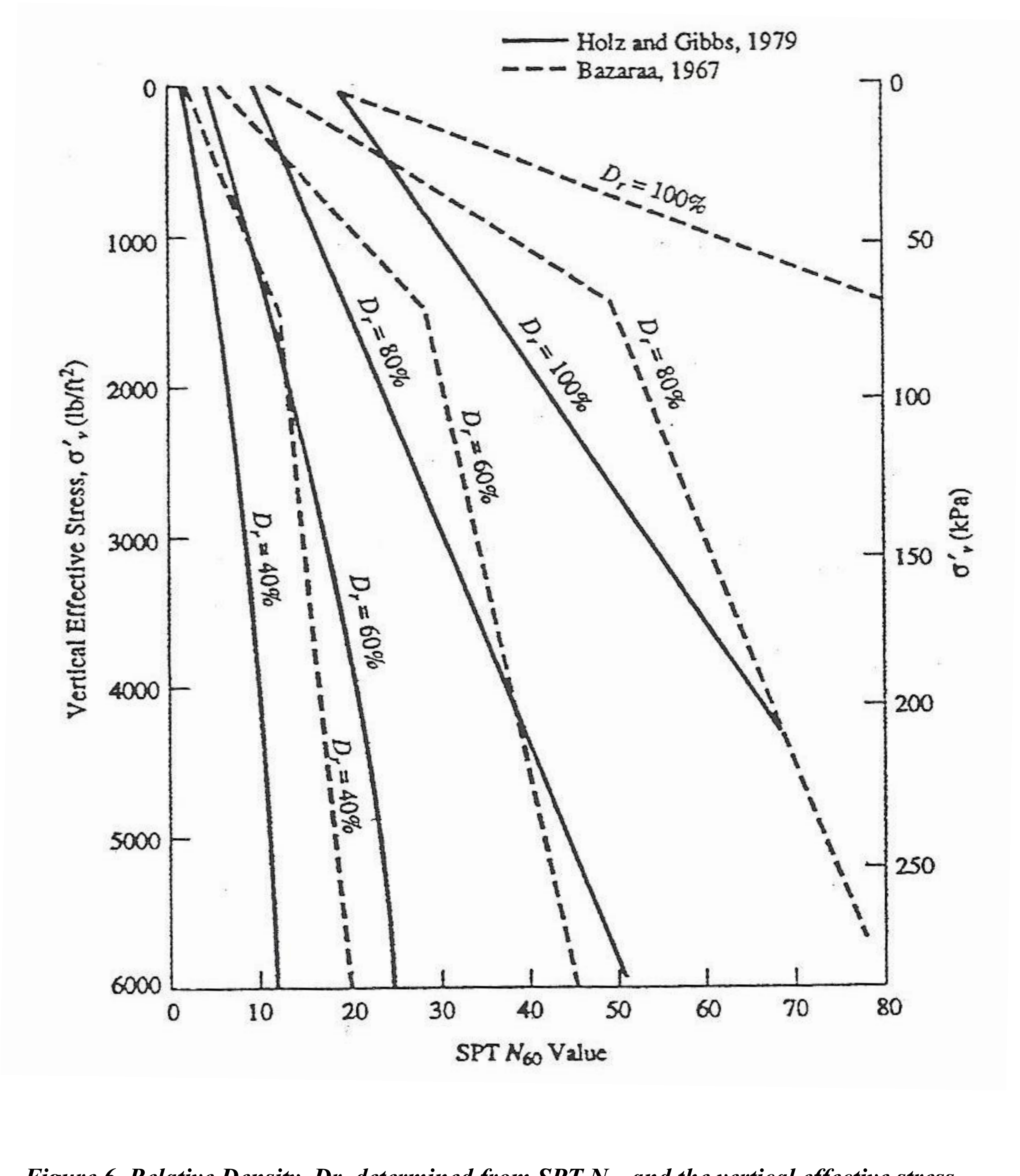


Figure 6. Relative Density, Dr, determined from SPT N_{60} and the vertical effective stress, σ_v', at the test location (Adapted from USBR, 1974; Bazaraa, 1967)

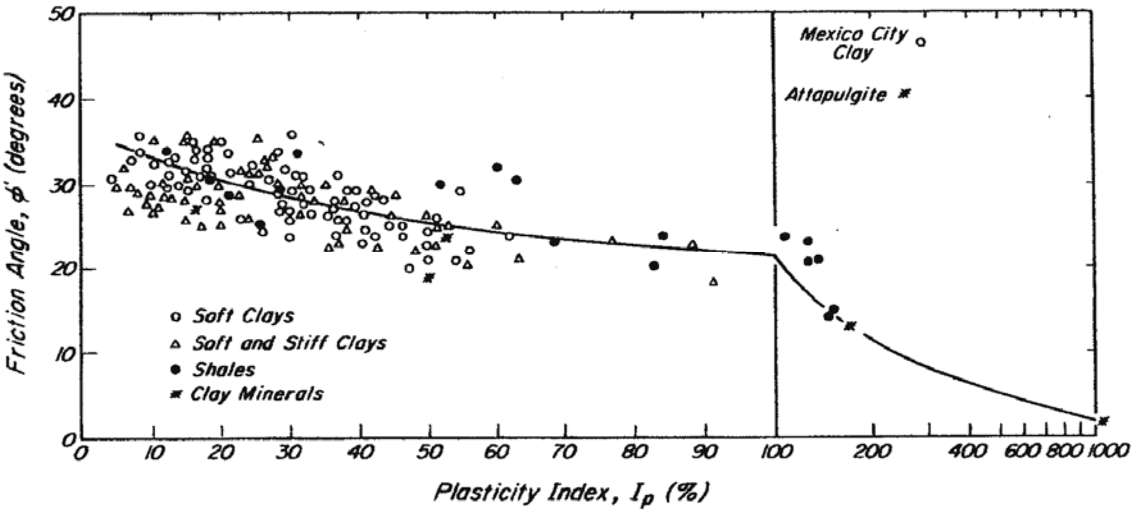
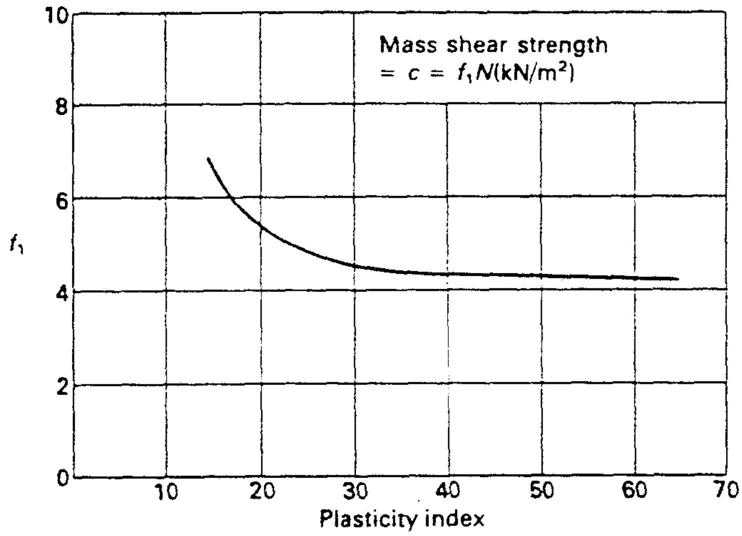


Figure 7. Values of friction angle ϕ' for clays of various compositions as reflected in plasticity index (Terzaghi, Peck and Mesri, 1996)



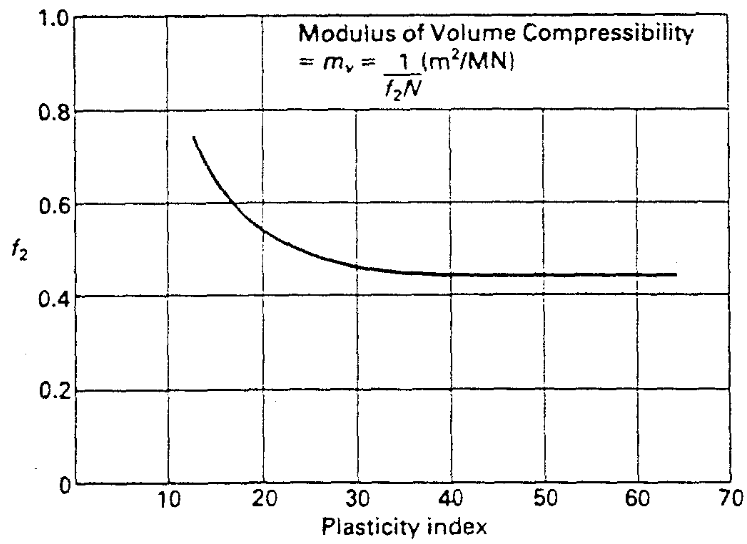


Figure 8. Relationship between Mass Shear Strength, Modulus of Volume Compressibility, Plasticity Index, and SPT-N values (after Stroud, 1975)

__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

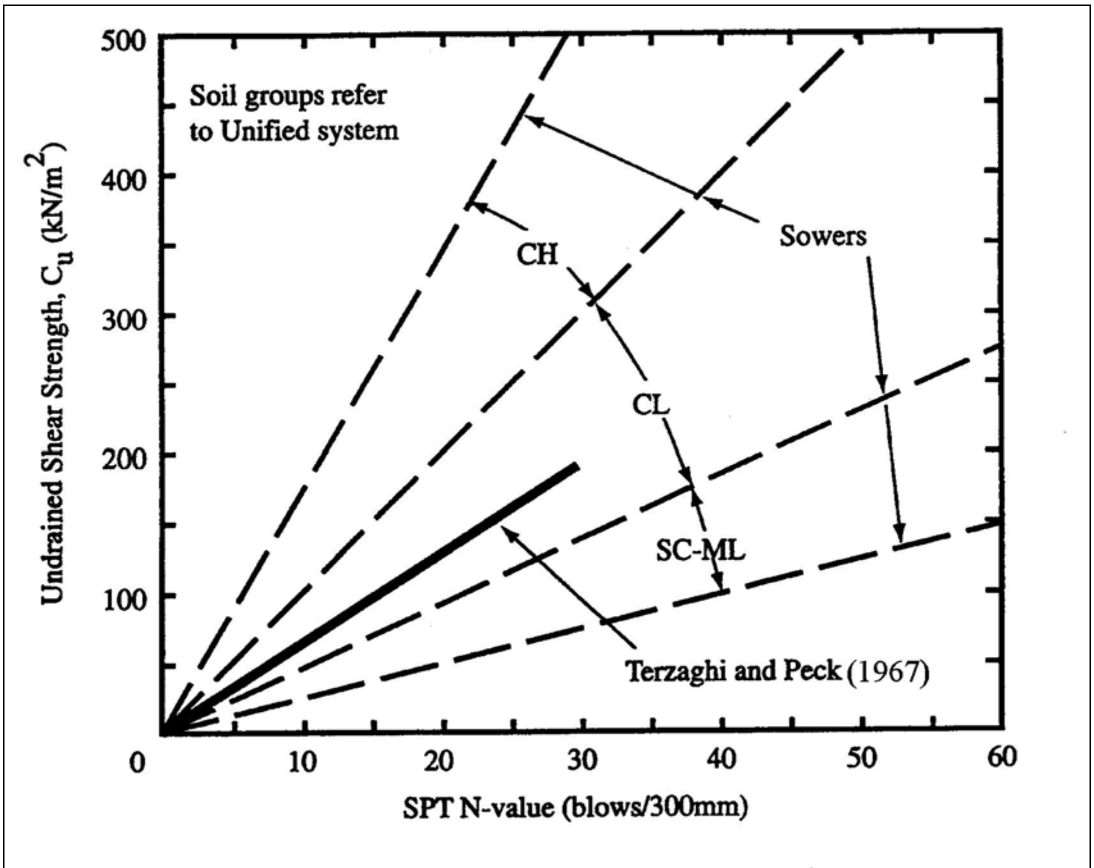


__Figure 9. Approximate Correlation between Undrained Shear Strength and SPT-N values (After Sowers, 1979)__

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

† 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, November 1983, p. 726–737; Use of In-Situ Tests in Geotechnical Engineering, ASCE (1986), p. 1173; Mitchel and Gardner (197[illegible]

substantial D/B ratios. This means that one may not obtain very good estimates of E_s at depths beyond the critical depth (usually taken as some fraction of D/B) of the cone unless the depth via overburden pressure is somehow included in the equation for E_s. This might be done using a new variable C_3' from 1 to 100 as follows:

$$C_3 = \left(\frac{C_3' + p_o'}{p_c'}\right)^n \qquad \text{or} \qquad C_3 = C_3' + \log p_o'$$

where p_o' = effective overburden pressure as previously defined and n = exponent of value ranging from 0.4 to 0.7.

5-9 SIZE EFFECTS ON SETTLEMENTS AND BEARING CAPACITY

A major problem in foundation design is to proportion the footings and/or contact pressure so that settlements between adjacent footings are nearly equal. Figure 5-9 illustrates the problem (and why plate load tests have little real value). It is evident that if the depth of influence is $H = 5B$, a 0.3 m square plate has an influence depth of $5 \times 0.3 = 1.5$ m where a 2 m prototype would have a depth of

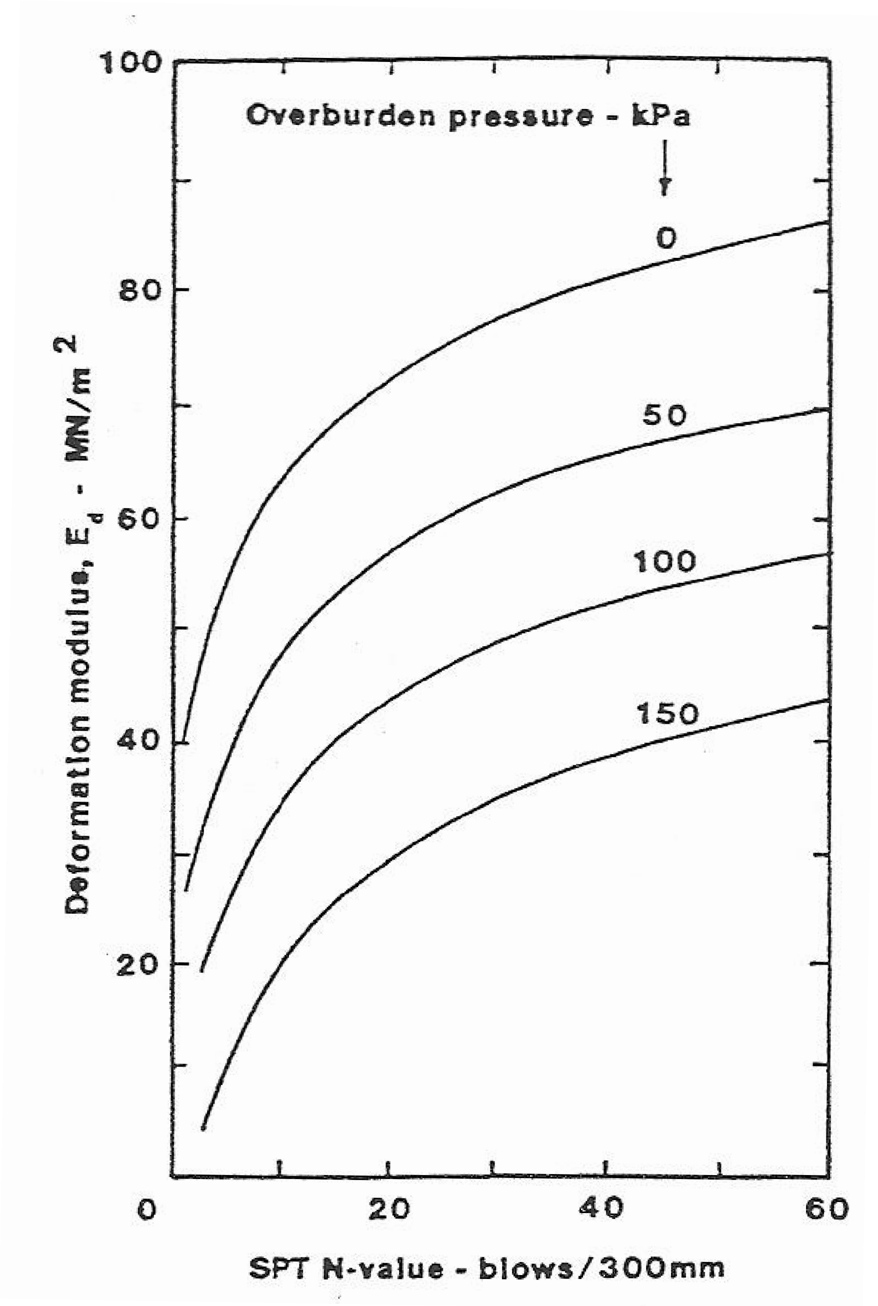


Figure 10. Correleation between deformation modulus, E_d and SPT N-value for granular soils (after Menzenbach, 1967)

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 MATERİALS		
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.

Table 6. Typical Values of Small-Strain Shear Modulus (AASHTO, 1996)

Soil Type	Small-strain shear modulus, G_o (kPa)
Soft clays	2,750 to 13,750
Firm clays	6,900 to 34,500
Silty sands	27,600 to 138,000
Dense sands and gravels	69,000 to 345,000

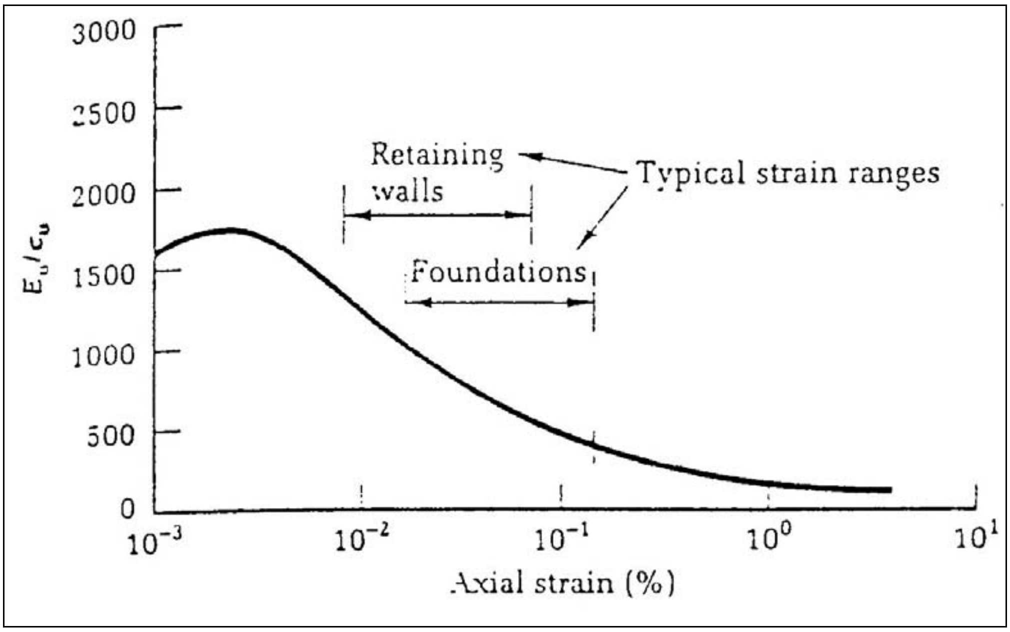


Figure 11. Relationship between E_u / c_u and Axial Strain (after Jardine et al., 1985)

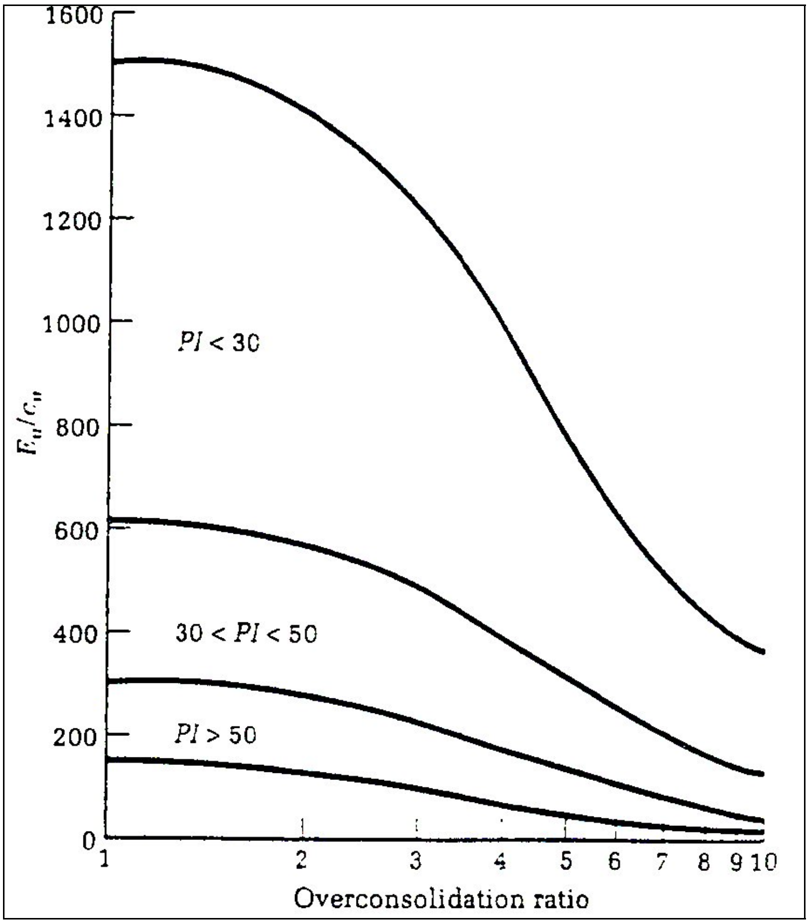


Figure 12. Relationship between E_u / c_u Ratio for Clays with Plasticity Index and Degree of Overconsolidation (after Jamiolkowski et al., 1979)

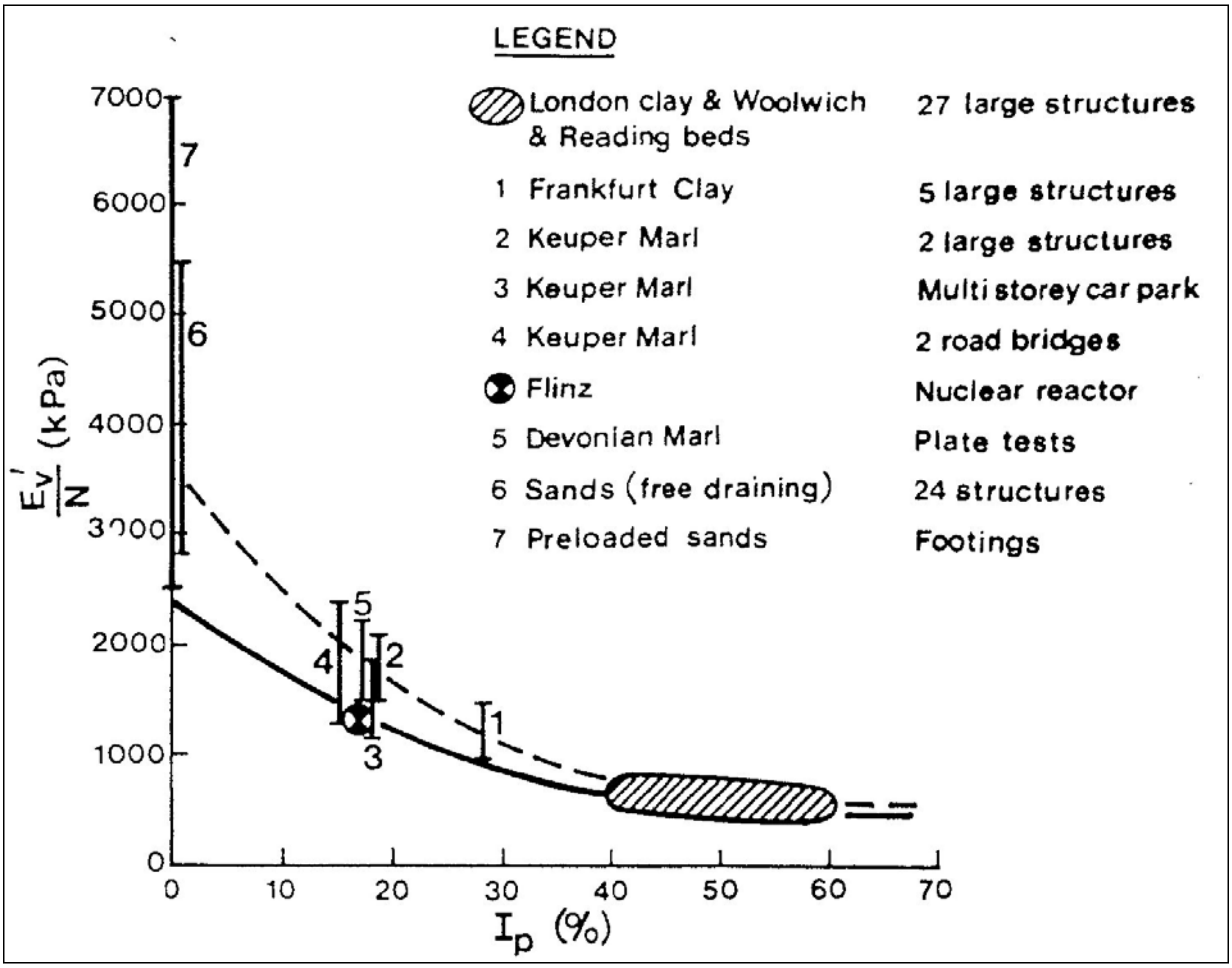


Figure 13. The Variation of E_v' / N with Plasticity Index (after Stroud, 1975)

Table 7. Skempton and Bjerrum (1957) Consolidation Settlement Correction Factors

Type of Clay	μ_g
Very sensitive clays (soft alluvial)	1.0-1.2
Normally consolidated clays	0.7-1.0
Overconsolidated clays (London clays)	0.5-0.7
Heavily overconsol. clays (Glacial Tills)	0.2-0.5

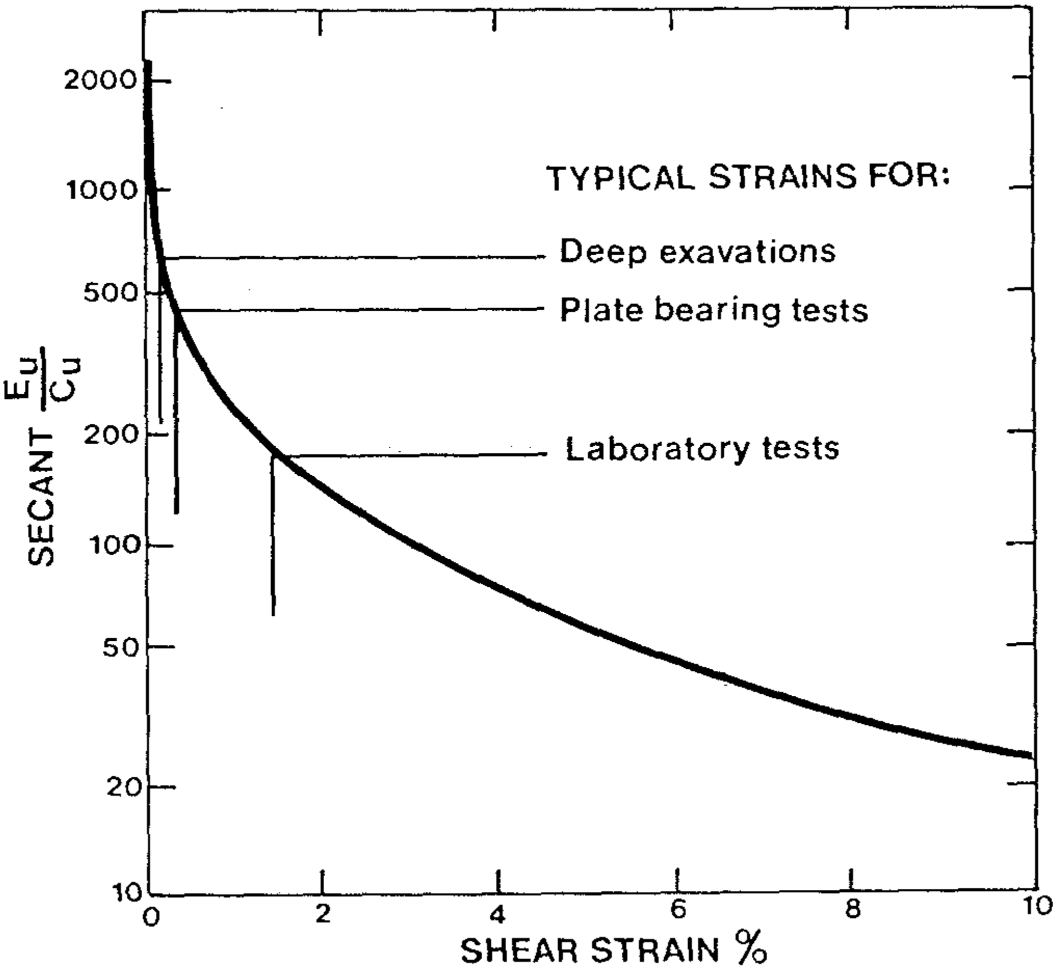


Figure 14. The Variation of Second Young's Modulus with Shear Strain, derived from the Mathematical Model for London Clay (Simpson, O'Riordan and Croft, 1979)

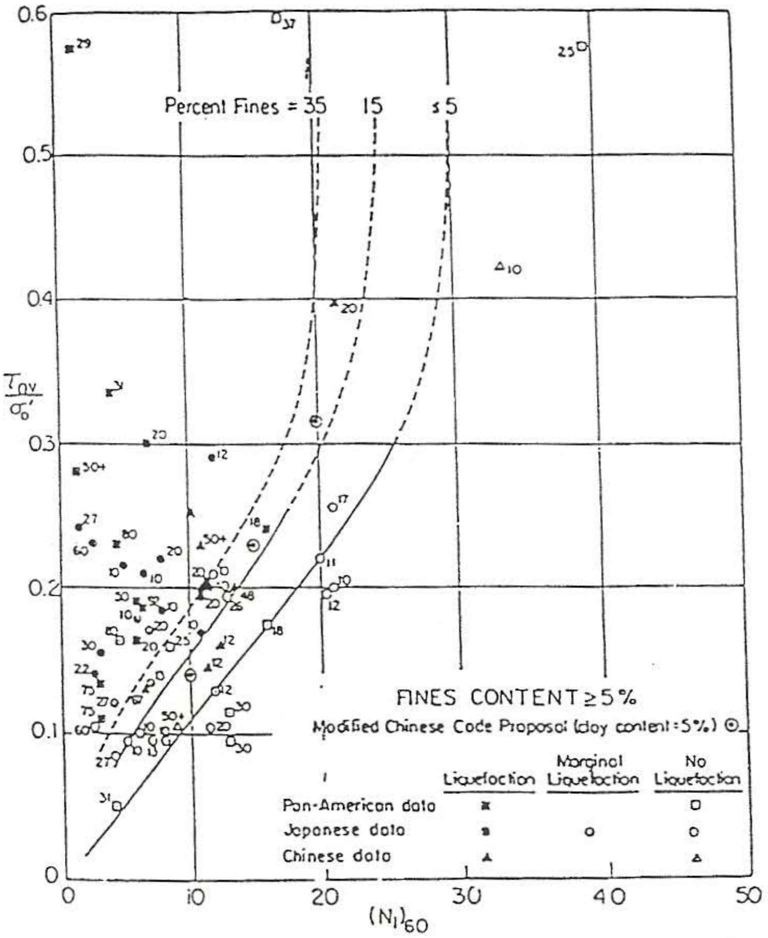


FIGURE 4-7 Relationships between stress ratio causing liquefaction and $(N_1)_{60}$ values for silty sands for magnitude 7.5 earthquakes. Boundary points specified by the Chinese Building Code are shown for comparison. Source: Seed et al. (1984).

$$\tau_{av}/\sigma_o' = 0.65\, a_{max}\left(\sigma_o\, r_d/\sigma_o' g\right)$$

a_{max}: max acceleration at ground surface

σ_o : total overburden stress

σ_o' : effective " "

r_d : stress reduction factor
(1 at ground level, 0.9 at 10m depth)

a_{max}: f (distance to epicenter, soil type, depth to bedrock etc.

Figure 15. Relationships between stress ratio causing liquefaction and $(N_1)_{60}$ values for silty sands for magnitude 7.5 eathquakes. Boundary points specified by the Chinese Building Code are shown for comparision. <u>Source:</u> Seed et al. (1984).

B) CONE PENETRATION TEST (CPT):

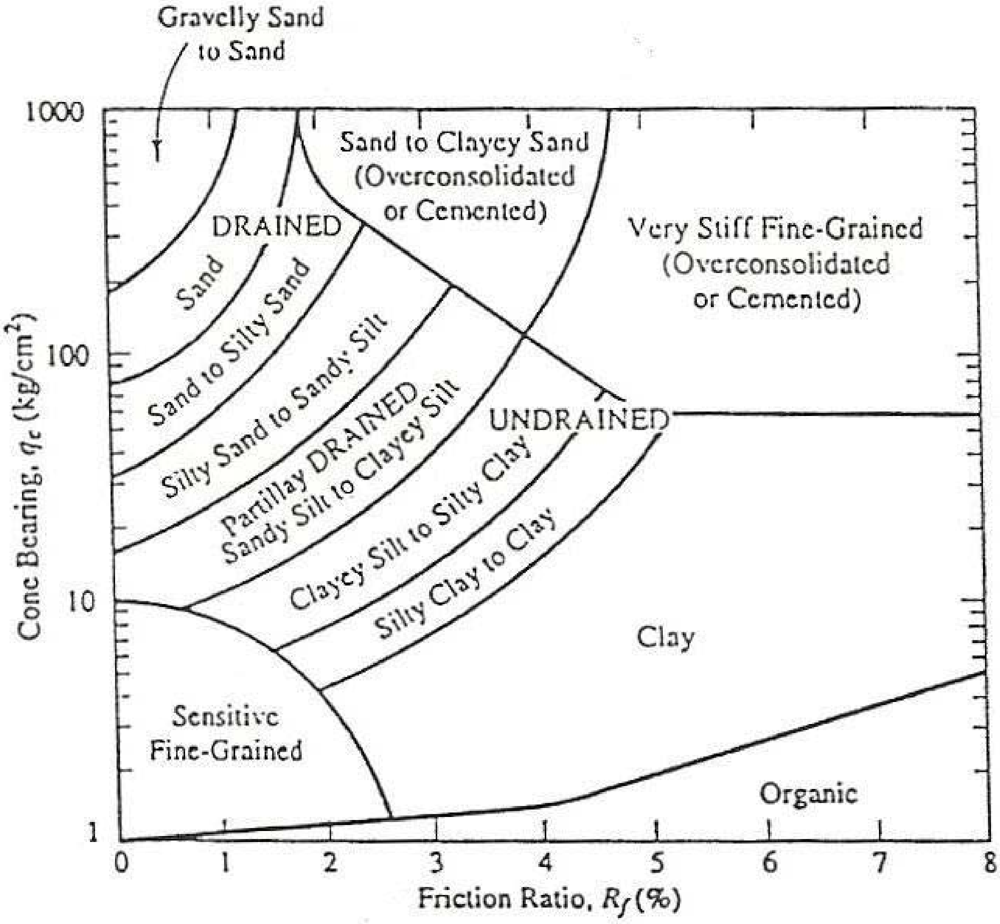


Figure 16. Classification of soil based on CPT test results (Adapted from Robertson and Campanella, 1983)

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

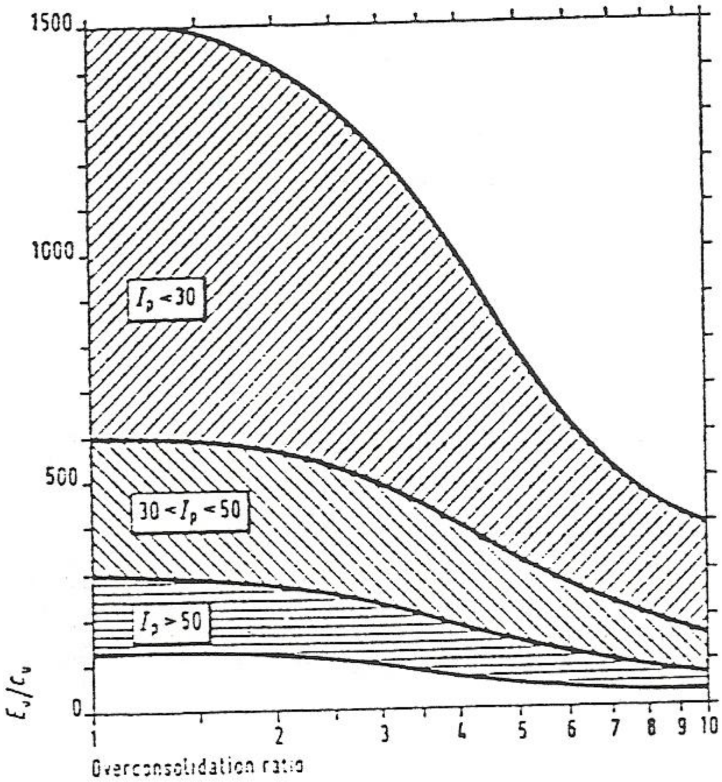


Figure 17. Ratio of undrained Young 's Modulus to shear strength against overconsolidation for clays (after Duncan and Buchignani, 1976)

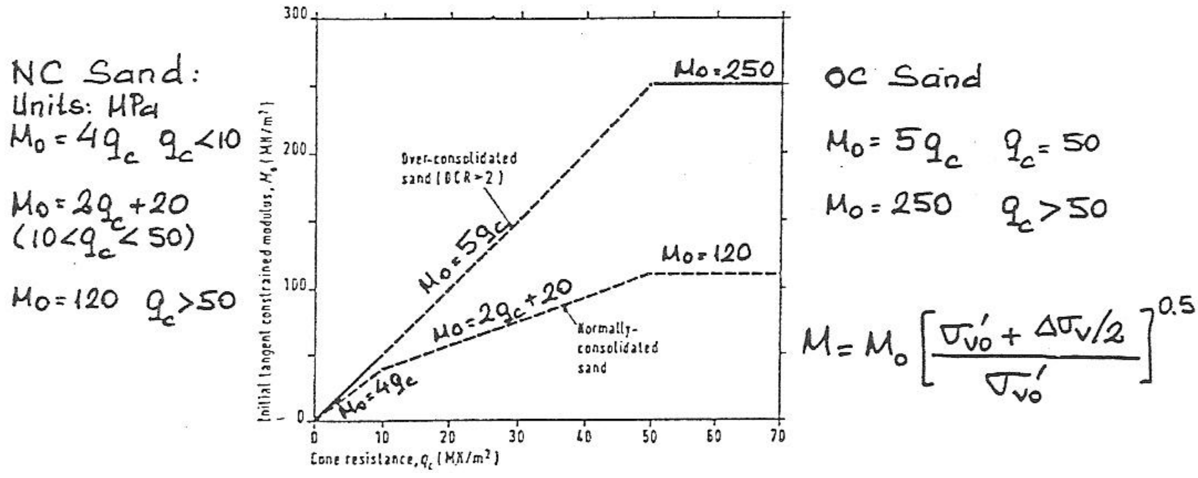


Figure 18. Initial tangent constrained modulus for normally-consolidated sands (after Lunne and Christoffersen, 1983)

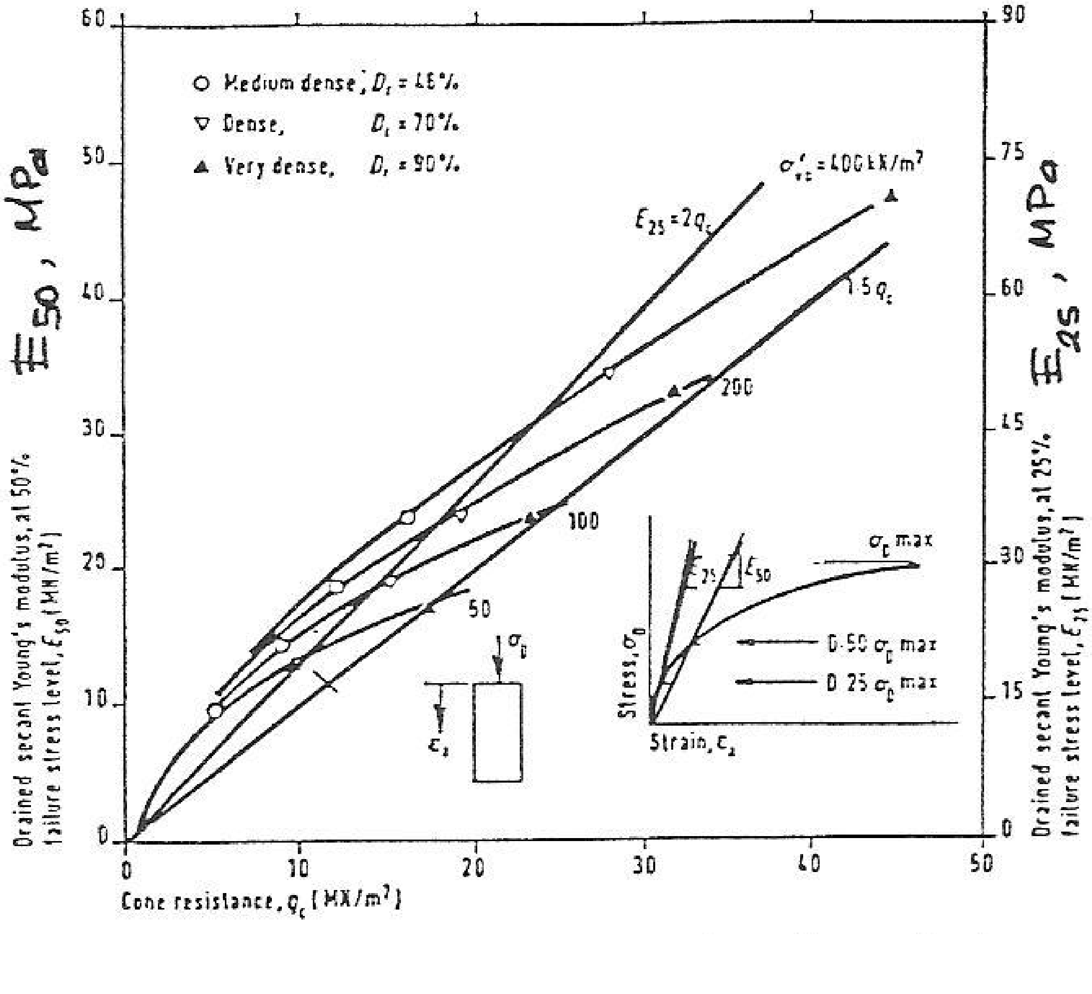


Figure 19. Secant Young 's Modulus values for uncemented, normally-consolidated quartz sands (after Robertson and Campanella, 1983 based on data from Baldi et. al., 1981)

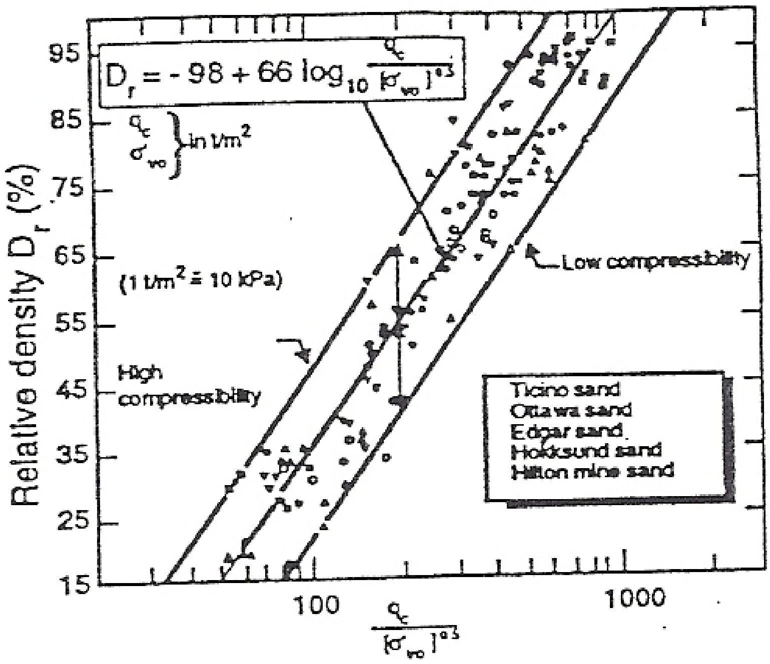


Figure 20. Influence of compressibility on NC, uncemented, unaged, predominantly quartz sands (after Jamiolkowski et al., 1985)

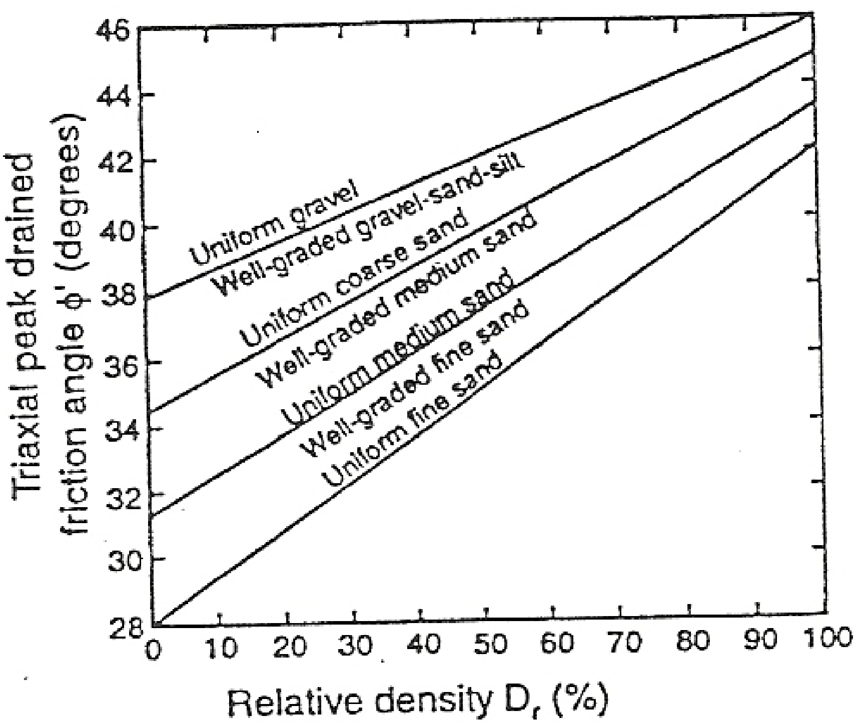


Figure 21. Relationship between Ø', and D_r suggested by Schmertmann (1978)

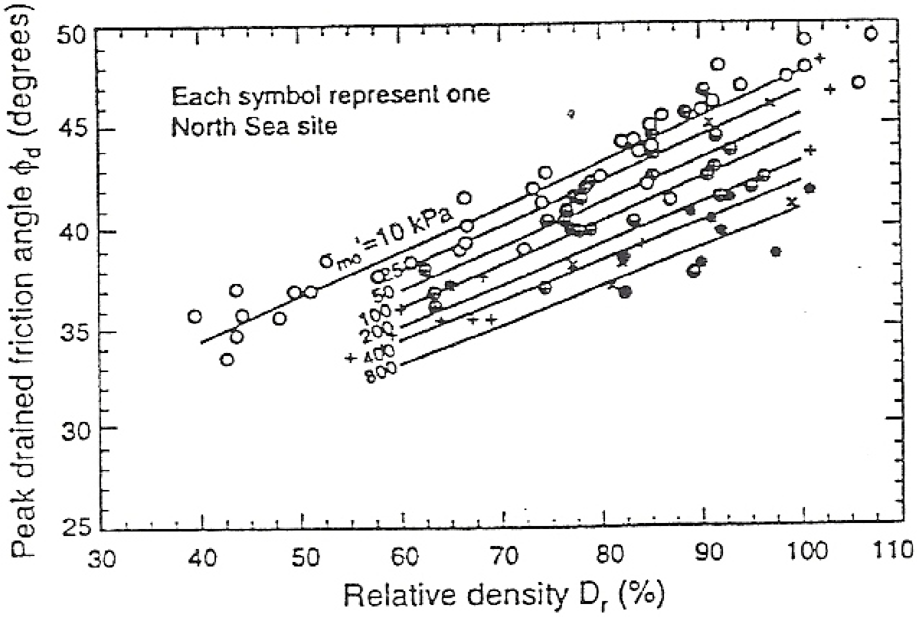


Figure 22. Correlations between Ø', D_r and σ'$_{mo}$ for line to medium, uniform silica sands (after Kleven et al., 1986)

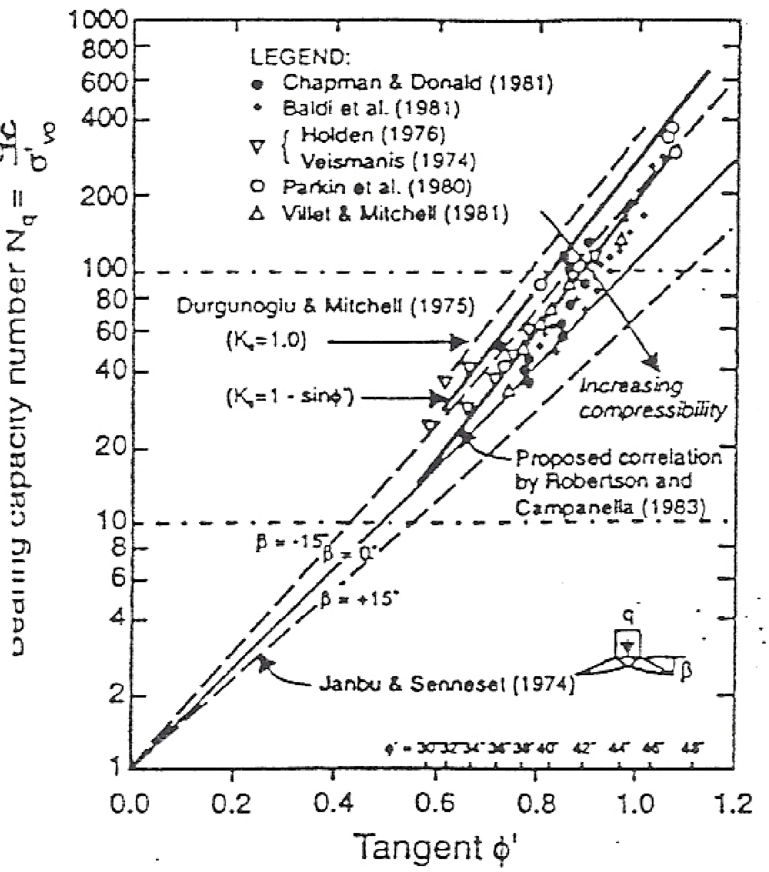


Figure 23. Relationship between bearing capacity number and friction angle form large calibration chamber tests (after Robertson and Campanella, 1983b)

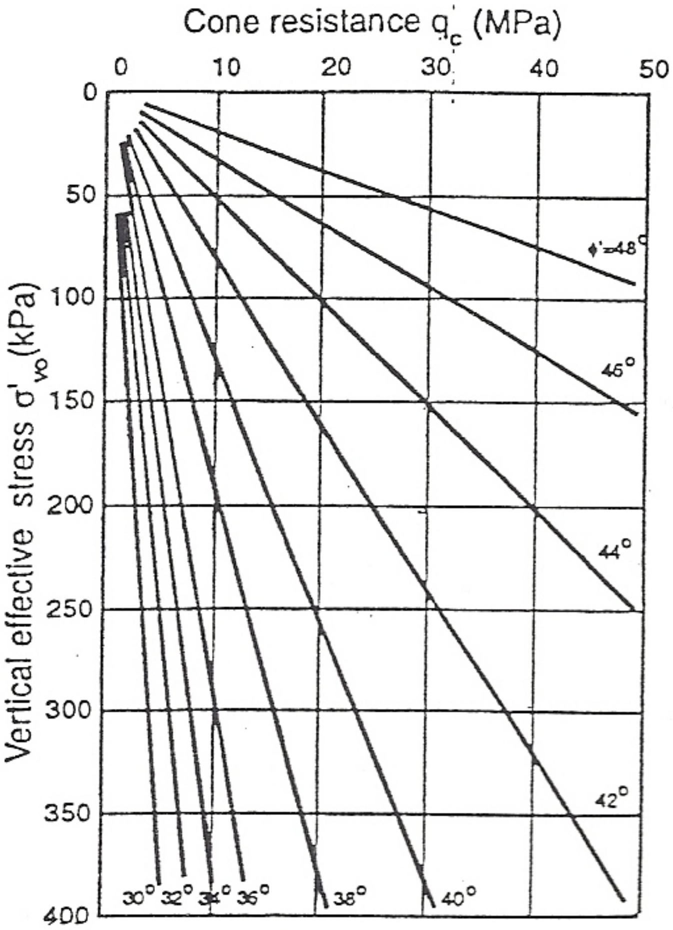


Figure 24. σ'_{vo}, q_c, $Ø'$ relationships (after Robertson and Campenella, 1983b)

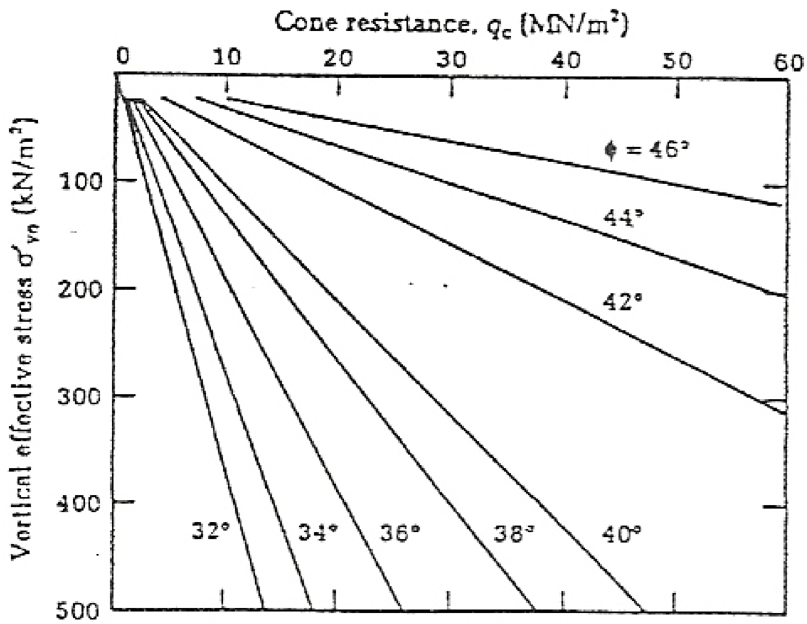


Figure 25.Relationship between angle of shearing resistance and cone resistance for an uncemented, normally consolidated quartz sand (after Durgunoğlu and Mitchell)

C) 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

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