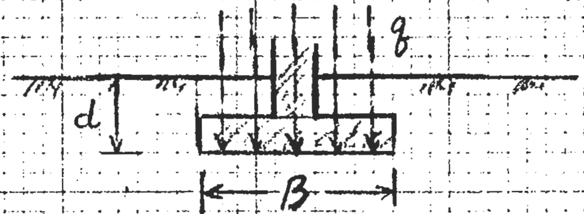
III-4 SHALLOW FOUNDATIONS ON SAND: BEARING CAPACITY

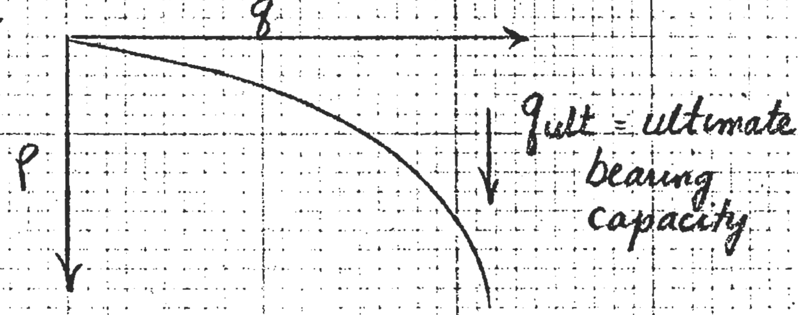
Page No.

1. INTRODUCTION

1.1 Definitions

– "Shallow" Fdn = footing or mat with $d/B \le 1$





1.2 Design Criteria

(1) Adequate safety $q_{allowable} = \dfrac{q_{ult}}{F}$ ← Terzaghi Eqn. modified

F = Factor of Safety } ≈ 3 Bldg.

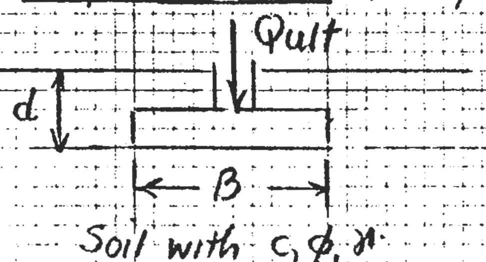
(2) allowable settlement

- Criteria Part III-5 · Usually governs foundations on sand
  - Emphasis on prediction methods

2. ULTIMATE BEARING CAPACITY OF SOIL: THEORY

(Also see Vesic, 1973: JSMFD, ASCE, Vol. 99, SM1)

2.1 Physical Model (Strip Footing)



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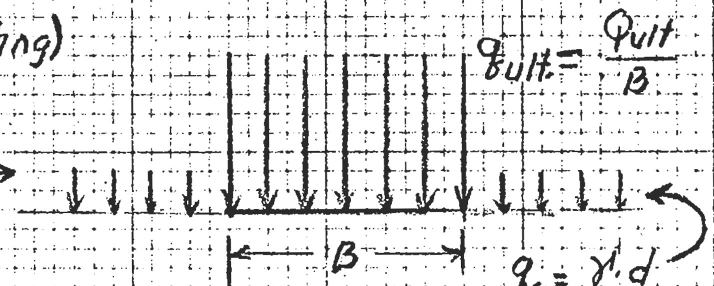
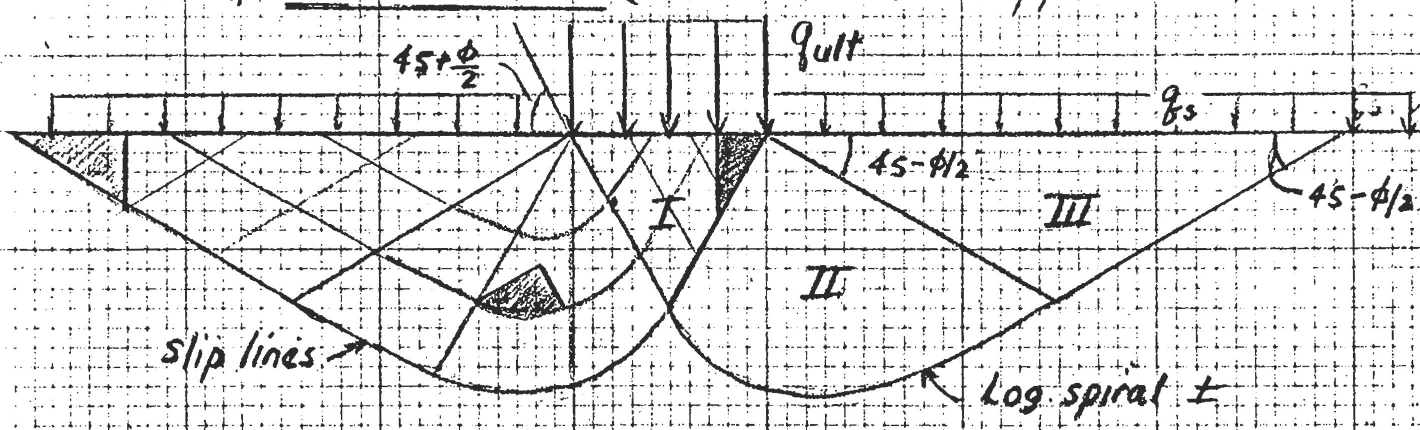


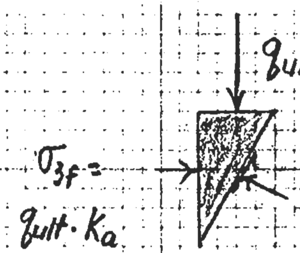
Neglect Strength of soil above base.

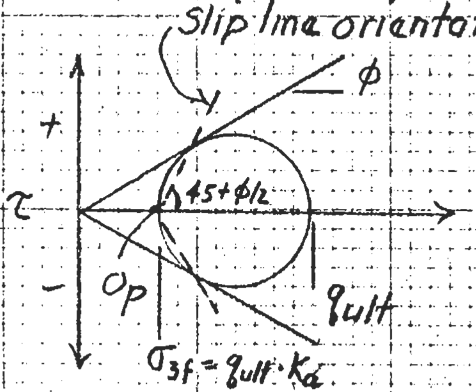
2.2 Shear Zones (Frictionless Base : approximate)



Three Plastic Zones (States of failure drawn for $c=0$)

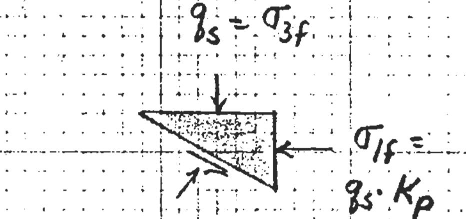
I Rankine Active

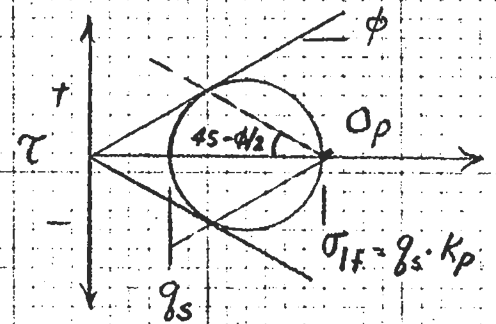




III Rankine Passive

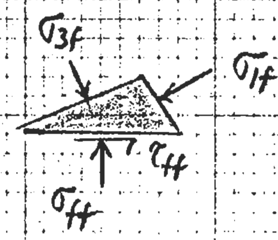
NOTE: Differant scales for Mohr circles

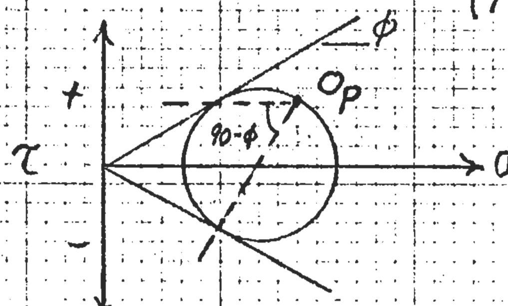




II Prandtl Radial

(For horizontal slip line)





2.3 Resultant Solution: Strip Footing (Incompressible Soil)

$$q_{ult.} = c N_c + \tfrac{1}{2} \gamma B N_\gamma + q_s N_q \quad \text{with } q_s = \gamma \cdot d$$

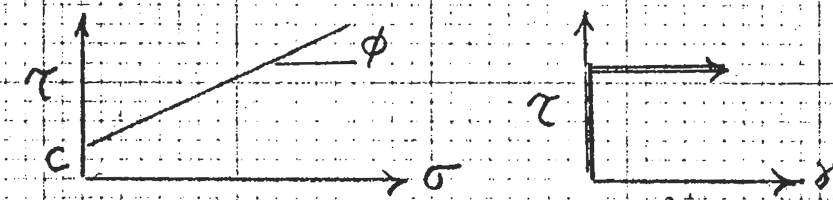
Cohesion   soil weight   surcharge

where N_c, N_γ, N_q = bearing capacity factors $= f(\phi)$

For later in term $\left\{ \text{DRAINED SHEAR} = f(\sigma') \rightarrow c', \phi', \gamma'; \ \text{UNDRAINED SHEAR} = f(\sigma) \rightarrow c, \phi, \gamma \right\}$

2.4 Values of B.C. Factors (Vesic (1973) for details)

(1) Theory of plasticity for rigid perfectly plastic soil → N_c & N_q (Solved for $\gamma = 0$)



For smooth base where $N_\phi = \dfrac{1+\sin\phi}{1-\sin\phi} = \tan^2(45+\frac{\phi}{2})$

$$\boxed{N_q = N_\phi \, e^{\pi \tan\phi}} \qquad \boxed{N_c = \cot\phi \, (N_q - 1)}$$

{ NOTE: $N_\phi = (\sigma_1/\sigma_3)_f$ for $c=0$ }

(2) Value of N_γ controversial since rigorous theoretical solution not available; and comparison of predicted vs. model footing test results inconclusive due to effects of: a) σ' level & σ_2 on value of ϕ' of sands
b) soil compressibility ($\Delta V \neq 0$)

Vesic (1973) recommends Caquot & Kérisel (1953) →

$$\boxed{N_\gamma \approx 2 \tan\phi \, (N_q + 1)}$$

(3) See Table III 4-1 (p 5) for tabulated results (these differ from Fig. 14.13 of L & W).

(4) Some typical values

		0*			
ϕ°	=	0	30	35	40
N_c	=	5.14	30.1	46.1	75.3
N_q	=	1.00	18.4	33.3	64.2
N_γ	=	0	22.4	48.0	109
N_q/N_γ	=		0.8	0.7	0.6

* For undrained shear of saturated soil, $\phi = 0$ & $c = s_u$; $N_c = \pi + 2$

CCL 12/94

CCL 10/95

1.361-1.366 Part III-4

p5

Table 4.—Bearing Capacity Factors

ϕ (1)	N_c (2)	N_q (3)	N_γ (4)	N_q/N_c (5)	$\tan\phi$ (6)
0	5.14	1.00	0.00	0.20	0.00
1	5.38	1.09	0.07	0.20	0.02
2	5.63	1.20	0.15	0.21	0.03
3	5.90	1.31	0.24	0.22	0.05
4	6.19	1.43	0.34	0.23	0.07
5	6.49	1.57	0.45	0.24	0.09
6	6.81	1.72	0.57	0.25	0.11
7	7.16	1.88	0.71	0.26	0.12
8	7.53	2.06	0.86	0.27	0.14
9	7.92	2.25	1.03	0.28	0.16
10	8.35	2.47	1.22	0.30	0.18
11	8.80	2.71	1.44	0.31	0.19
12	9.28	2.97	1.69	0.32	0.21
13	9.81	3.26	1.97	0.33	0.23
14	10.37	3.59	2.29	0.35	0.25
15	10.98	3.94	2.65	0.36	0.27
16	11.63	4.34	3.06	0.37	0.29
17	12.34	4.77	3.53	0.39	0.31
18	13.10	5.26	4.07	0.40	0.32
19	13.93	5.80	4.68	0.42	0.34
20	14.83	6.40	5.39	0.43	0.36
21	15.82	7.07	6.20	0.45	0.38
22	16.88	7.82	7.13	0.46	0.40
23	18.05	8.66	8.20	0.48	0.42
24	19.32	9.60	9.44	0.50	0.45
25	20.72	10.66	10.88	0.51	0.47
26	22.25	11.85	12.54	0.53	0.49
27	23.94	13.20	14.47	0.55	0.51
28	25.80	14.72	16.72	0.57	0.53
29	27.86	16.44	19.34	0.59	0.55
30	30.14	18.40	22.40	0.61	0.58
31	32.67	20.63	25.99	0.63	0.60
32	35.49	23.18	30.22	0.65	0.62
33	38.64	26.09	35.19	0.68	0.65
34	42.16	29.44	41.06	0.70	0.67
35	46.12	33.30	48.03	0.72	0.70
36	50.59	37.75	56.31	0.75	0.73
37	55.63	42.92	66.19	0.77	0.75
38	61.35	48.93	78.03	0.80	0.78
39	67.87	55.96	92.25	0.82	0.81
40	75.31	64.20	109.41	0.85	0.84
41	83.86	73.90	130.22	0.88	0.87
42	93.71	85.38	155.55	0.91	0.90

Table 4.—*Continued*

(1)	(2)	(3)	(4)	(5)	(6)
43	105.11	99.02	186.54	0.94	0.93
44	118.37	115.31	224.64	0.97	0.97
45	133.88	134.88	271.76	1.01	1.00
46	152.10	158.51	330.35	1.04	1.04
47	173.64	187.21	403.67	1.08	1.07
48	199.26	222.31	496.01	1.12	1.11
49	229.93	265.51	613.16	1.15	1.15
50	266.89	319.07	762.89	1.20	1.19

Vesic, A.S. (1973). "Analysis of Ultimate Loads of Shallow Foundations". J. Soil Mech. & Fdn. Div., ASCE, Vol. 99, SM2, 45-73

$$N_\phi = \frac{1+\sin\phi}{1-\sin\phi} = \tan^2(45+\phi/2)$$

$$N_q = N_\phi \, e^{\pi \tan\phi}$$

$$N_c = \cot\phi \, (N_q - 1)$$

$$N_\gamma \approx 2\tan\phi \, (N_q + 1)$$

$$S_q = 1 + \tan\phi\left(\frac{B}{L}\right)$$

$$S_c = 1 + \frac{N_q}{N_c}\left(\frac{B}{L}\right)$$

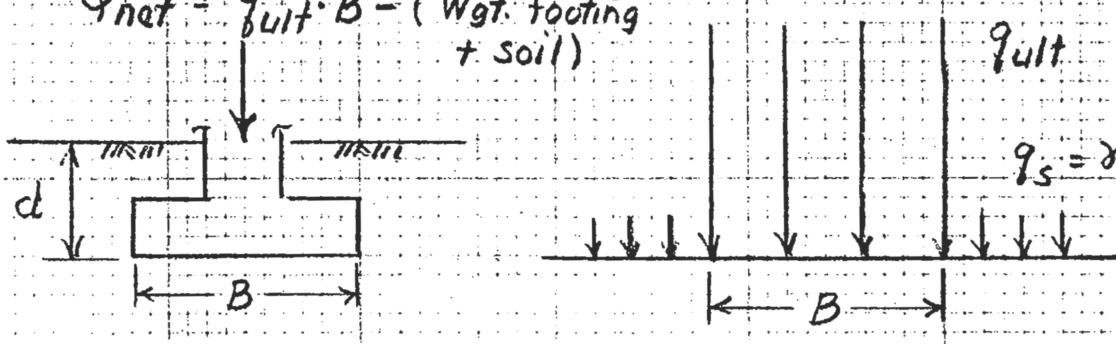
$$S_\gamma = 1 - 0.4\left(\frac{B}{L}\right)$$

Table III 4-1 Bearing Capacity Factors

2.5 Illustration of Results: Strip Footing on Dry Sand

$d/B \le 1$ & $c = 0$

$Q_{net} = q_{ult} \cdot B - $ (Wgt. footing + soil)



$$q_{ult} = \tfrac{1}{2}\gamma' B N_\gamma + \gamma' d N_q$$

$$= \tfrac{1}{2}\gamma' B N_\gamma \left(1 + \frac{2d}{B}\frac{N_q}{N_\gamma}\right) \quad \text{for constant } \gamma'$$

q_{ult} ← {Equals q_{ult} (ksf) for $\gamma' = 100$ pcf & $B = 10$ ft

$\phi°$	d=0	d=B
30	$11\gamma' B$	$+18\gamma' B = 29\gamma' B$
35	$24\gamma' B$	$+33\gamma' B = 57\gamma' B$
40	$55\gamma' B$	$+64\gamma' B = 119\gamma' B$

} Approx. DOUBLING per $\Delta\phi = 5°$!!

Linear increase*

For constant γ' & taking $N_q/N_\gamma \approx 1$ →

$$q_{ult}\ \frac{d>0}{d=0} = 1 + \frac{2d}{B} = 2 \text{ for } d/B = \tfrac{1}{2}$$
$$= 3 \text{ '' '' } = 1$$

Summary & Conclusions

1) Solution treats soil above footing as having weight only; i.e. NO STRENGTH (Hence ϕ' of soil above footing is not relevant)

2) ϕ', B and d/B all are VERY IMPORTANT

3) Should account for differing γ' above/below footing

* Actually not true since increasing B → increasing σ' level → decreasing ϕ'_p

2.6 Effect of Soil Compressibility (Function of D_r)

(1) See attached Figs. 1 & 2 from Vesic (1973) on p8

- General Shear (high D_r) → well defined rupture surfaces and q_{ult} ($D_r > 70\% \pm$)
- Local Shear (medium D_r) ⇒ rupture surfaces beneath footing but not outside & q_{ult} not so clear
- Punching Shear (low D_r) → poorly defined q_{ult} with large settlements & don't mobilized shear in Zones II & III ($D_r < 35\% \pm$)

(2) Empirical approaches used in practice { Use corrected ϕ'_c that is < peak $\phi' = \phi'_p$ }

* • T & P (1967) - "Loose" sand use $\tan\phi'_c = \frac{2}{3}\tan\phi'_p$

- PHT (1974) - Attached Fig. 19.5 (p8) plots N_γ & N_q vs ϕ'.
  N_γ / N_γ (theory) = 0.7→0.9 with increasing ϕ'. $N_q \approx N_q$ (theoretical)
- Vasic (1973) - Use $\tan\phi'_c = R.F. \tan\phi_p$

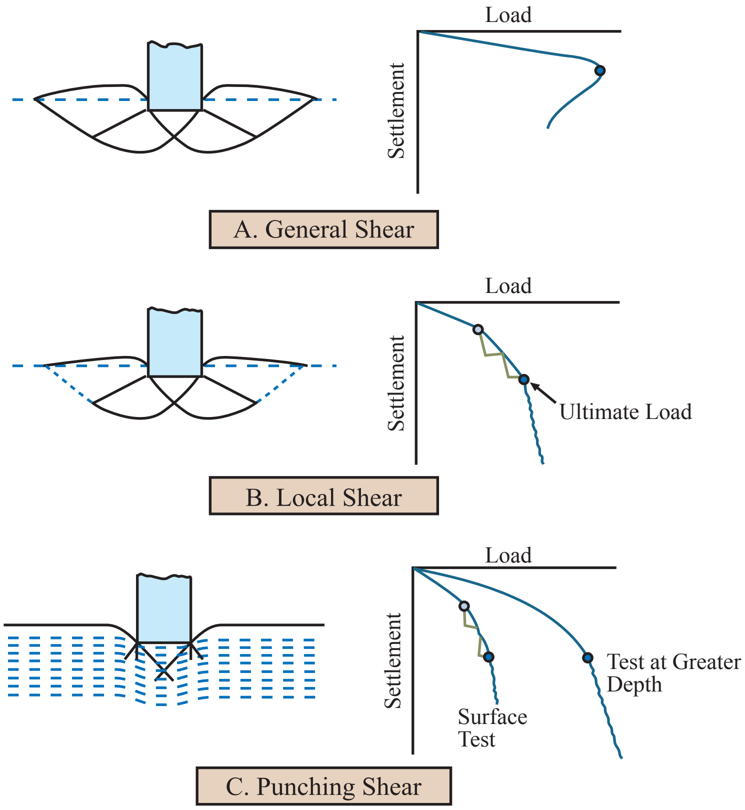
$$R.F. = 0.67 + D_r - 0.75 D_r^2 \quad (\text{for } D_r \le 0.67)$$

(3) Illustration

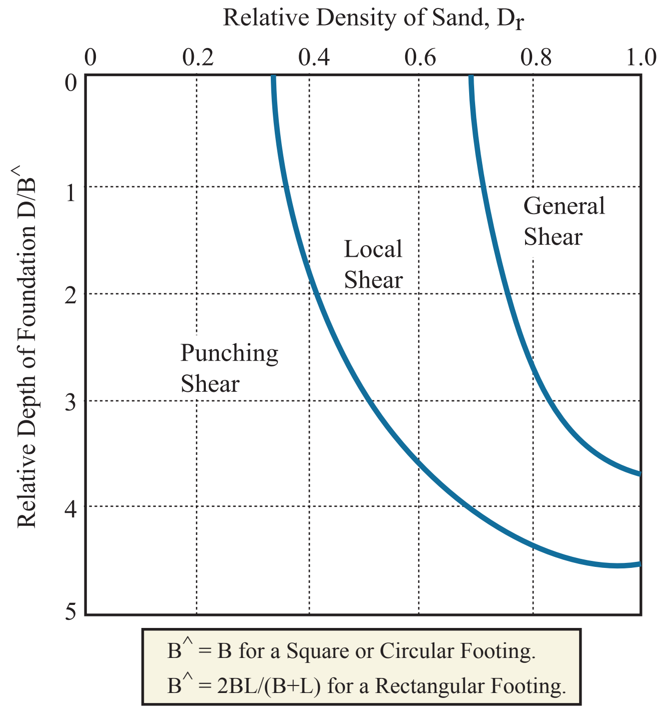
		Theory		* T & P (1967)			Vesic (1973)			PHT (1974)
D_r (%)	ϕ'_p	N_γ	N_q	ϕ'_c	N_γ	N_q	ϕ'_c	N_γ	N_q	N_γ
30	31	26.0	20.6	21.8	6.9	7.65	28.5	18.0	15.5	18
50	33	35.2	26.1	23.4	8.7	9.0	32.5	32.6	24.6	26
					↑ × 1/4	↑ × 1/3	CCL recommends for best estimate			

∴ very low

* Deleted because not in Terzaghi et al. (1996)

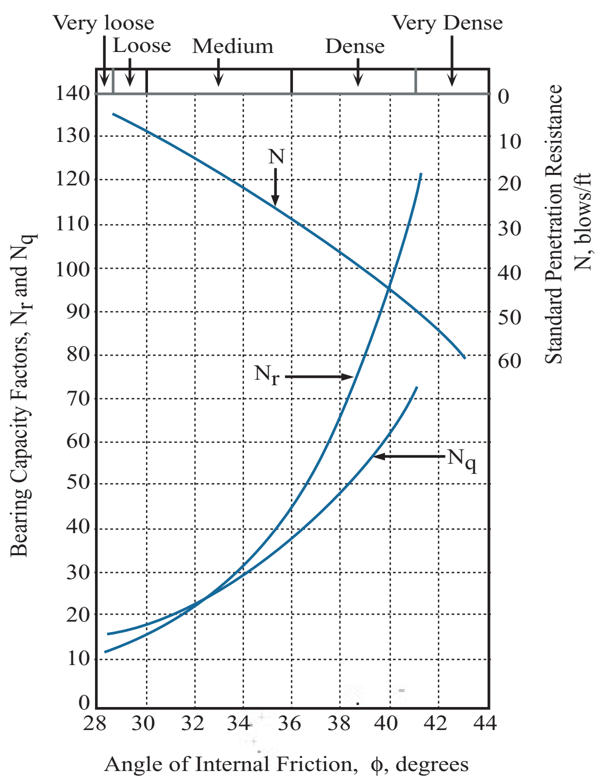


Modes of Bearing Capacity Failure (76)



Modes of Failure of Model Footings in Chattahoochee Sand (20,76)

Adapted from Vesic (1973) JSMFD, ASCE, 99(SMI)



Curves showing the relationship between bearing-capacity factors and Φ, as determined by theory, and rough empirical relationship between bearing capacity factors or Φ and values of standard penetration resistance *N*.

Adapted from Peck, Hanson & Thornburn (1974)

2.7 Shape Factors (from Vesic, 1973) - Empirical factors from model footing tests

- $q_{ult} = S_c\, c N_c + S_\gamma \frac{1}{2} \gamma B N_\gamma + S_q \gamma d N_q$

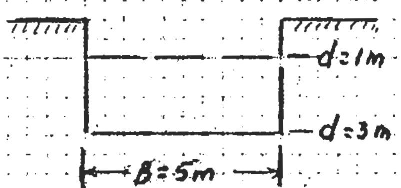
$$S_c = 1 + \left(\frac{N_q}{N_c}\right)\left(\frac{B}{L}\right) \qquad S_\gamma = 1 - 0.4\left(\frac{B}{L}\right) \qquad S_q = 1 + \tan\phi\left(\frac{B}{L}\right)$$

For $c = 0$ $\quad q_{ult} = \left(1 - 0.4\frac{B}{L}\right)\frac{1}{2}\gamma B N_\gamma + \left(1 + \tan\phi\,\frac{B}{L}\right)\gamma d N_q$

Decrease $\rightarrow 0.6$ for $B = L$ $\qquad$ Increase $\rightarrow 1 + \tan\phi$ for $B = L$ $\quad \phi = 35 \pm 5^\circ \rightarrow \times 1.7 \pm 0.13$

- Example

$\phi = 35^\circ$, $\gamma = 1.75$ TCM

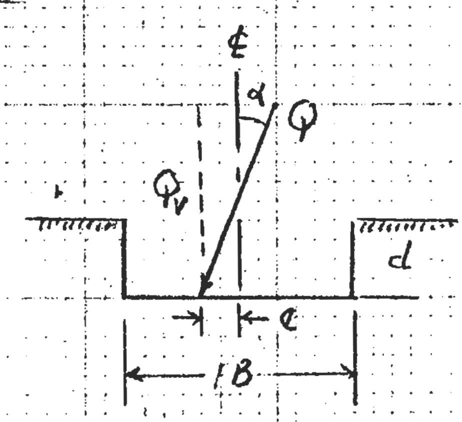


d/B	Shape	Component N_γ	Component N_q	q_{ult} (TSM)	
0.2	Strip	210	58	268	
	Square	126	99	225	× 0.84
0.6	Strip	210	175	385	
	Square	126	297	423	× 1.1

2.8 Inclined - Eccentric Loadings (Strip)

- L & W (1969) Eq. 14.13 from Meyerhof (1953)

$$q_{ult}(V) = \frac{Q_v}{B} = \left(1 - \frac{2e}{B}\right)^2\left(1 - \frac{\alpha}{\phi}\right)^2 \frac{1}{2}\gamma B N_\gamma + \left(1 - \frac{2e}{B}\right)\left(1 - \frac{\alpha}{90}\right)^2 \gamma d N_q$$



- Example from 2.7 for $d = 3m$, $\alpha = 10^\circ$ & $e/B = 0.1$

$$\left.\begin{array}{l} N_\gamma \text{ component}: (0.64)(0.51) = 0.32^5 \times 210 \rightarrow 68 \\ N_q \quad '' \quad : (0.8)(0.79) = 0.63 \times 175 \rightarrow 110 \end{array}\right\} q_{ult}(V) = 178 \quad \times 0.46!$$

3. ESTIMATION OF q_{ult} IN PRACTICE (Footings on Sand)*

3.1 Unit Weights (γ)

(1) Actual measurements

- Test pits with in situ tests (balloon, nuclear, etc)
- Tube sampling - unless special procedures, disturbance $\rightarrow \Delta\gamma$ (Loose sand densifies & vice versa)

(2) Estimate from soil type & D_r: Some examples are:

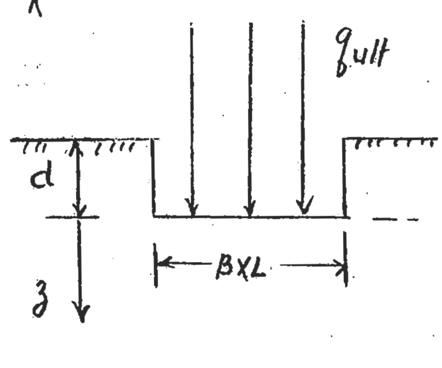
- L&W Table 3.2
- NAVFAC DM-7.1 (5/82) Fig 7 p7.1-149 (see Sheet E)
- But how estimate D_r? see Section 3.3

(3) How important is error in γ?

$e = 0.6 \pm 0.15$ Natural SANDS
- For typical dry sand, $\gamma_d = 105 \pm 10$ pcf (16.5 ± 1.6 kN/m³)
- " " submerged ", $\gamma_b = 65 \pm 5$ pcf (10 ± 0.8 kN/m³)

∴ Error should be < 10%

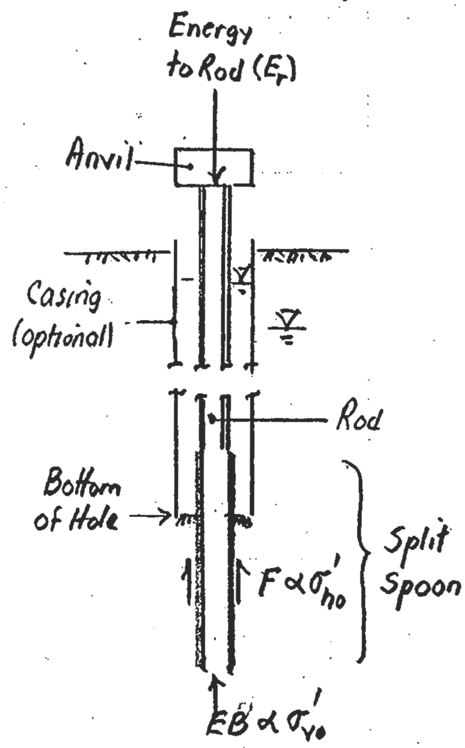
*



- Need values of γ both above and below level of footing
- Need to estimate average ϕ' over $z = 0$ to B

3.2 Standard Penetration Test (SPT)

1) Test Procedures (ASTM D1586-84) Also see Sheet A



a) Advance bore hole (washing or hollow stem auger)
- Dia ≈ $2\frac{1}{2}$-$4\frac{1}{2}$" (≤ 115mm) • Must keep higher water level in borehole

b) Energy applied via 140lb hammer falling 30" ($w \cdot h$ = 350 ft·lb), but actual energy is variable

See Sheet A, Fig. A-4 & Fig 4

c) Rod dia. = 2 ± 3/8" (50±10mm) with wgt ≈ 7±3 kg/m

d) Split spoon = 2.0" OD × $1\frac{3}{8}$"-$1\frac{1}{2}$" ID × ℓ = 2 ± $\frac{1}{2}$ ft

See Sheet A & important note wrt liner/no liner

e) Record blow counts

			↓ Increasing embedment
0-6"	6		
6-12"	8	} = N (blows/12 in.)	
12-18"	11		

f) Penetration resistance due to end bearing and exterior/interior friction. For granular soils, $EB \propto \sigma'_{vo}$ & exterior $F \propto \sigma'_{ho}$. ∴ N increases with depth for homogeneous granular deposit

2) Factors Affecting N (Other than depth & soil characteristics) See Sheet B

a) Actual energy (E_r) applied to top of rod = Energy Ratio (ER) × 350 ft·lb

- ER = Velocity Efficiency × Dynamic Efficiency (Mainly weight of anvil)
  - Velocity Efficiency: Method used release hammer { automated / rope on cathead }
  - } See Sheet B Table 6, Fig. 2-17 & Table 5, p12
- ER varies from ≈ 45% for typical US practice with donut hammer & 2 rope turns to ≈ 80% for Japanese practice with Tombi trigger release
- Recommended standard reference uses ER = 60%

  ∴ $N_{60} \approx N \cdot \frac{ER}{60}$

b) Other factors include rod length, oversize ID of split spoon and oversize boring diameter à la Sheet B

3) <u>Recommended standardized N_{60}</u> : See Sheet B, Tables 6 & 7 & Table 5

$$N_{60} = C_{ER} \cdot C_{RL} \cdot C_S \cdot C_B \times \text{Measured } N$$

C_B: Oversize bore hole (= 1.0 → 1.15)

C_S: US split spoon w/o liner (= 1.2)

C_{RL}: length of rod < 10m (= 1.0 → 0.75 at $l \approx$ 3-4m)

C_{ER}: actual energy ratio compared to 60% $= \frac{ER}{60}$ (= 1.3+ at ER = 80%, typical Japan Tombi; = 0.75 at ER = 45%, typical US donut)

Table 5 of Skempton (1986) Geot. 36(3), 425-447 $\quad ER = VE \cdot DE$

	Release			Hammer system			ER (%)
	Type	Cathead	VE (%)	Hammer	Anvil weight: kg	DE	
Waterways Experiment Station	Trip	—	100	Vicksburg	0	0·83	83
Japan	Tombi	—	100	Donut	2	0·78	78
Japan	Slip-rope (2 turns)	Small	83	Donut	2	0·78	65
USA	Slip-rope (2 turns)	Large	70	Safety	2·5	0·79	55
UK	Slip-rope (1 turn)	Small	85	Old standard	3	0·71	60
USA	Slip-rope (2 turns)	Large	70	Donut	≈12	0·64	45
UK	Trip	—	100	Pilcon	19	0·60	60

VE = Velocity Efficiency $\quad$ DE = Dynamic Efficiency $\quad$ ER = Energy Ratio

Some examples of reported N for $N_{60} = 20$ (l > 10m, dia ≤ 4.5")

(1) USA, donut with 12kg anvil, 2 rope turns on large cathead

- Old data w/ old sampler (ID = 35 mm) $\quad N = 20/\left(\frac{45}{60}\right) = 26.7 \approx 27$
- New sampler without liner $\quad N = 20/\left(\frac{45}{60} \times 1.2\right) = 22.2 \approx 22$

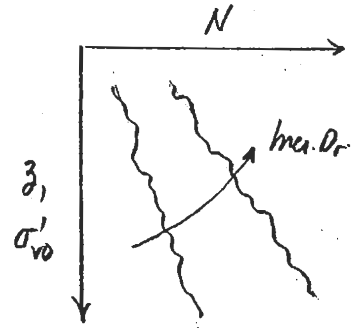
(2) Japan, donut with 2kg anvil

- Tompi release $\quad N = 20/\left(\frac{78}{60}\right) = 15.4 \approx 15$
- 2 rope turns, small cathead $\quad N = 20/\left(\frac{65}{60}\right) = 18.5$

4) Effect of Depth and OCR (At constant D_r)

a) General

- Increasing z → increasing σ' → increasing EB & F
- " OCR → " σ'_{ho} → " F
- at constant OCR $\dfrac{N}{D_r^2} \approx a + b\,\sigma'_{vo}$



b) Sources of data

- Calibration chambers (bin tests)
  - USBR & Gibbs & Holtz (1957) Coarse & silty fine sand
  - WES: Marcuson & Bieganousky (1977) Coarse, medium & fine sand
- Field data
  - Peck & Bazaraa (1969) U of I PhD thesis, dense coarse sands
  - Skempton (1986) Added data from Japan

c) Objective: Develop a relationship to obtain a corrected N at a reference overburden stress (Most use $\sigma'_{vo} = 1\ TSF \approx 1\ kg/cm^2 \approx 100\ kPa$)
∴ $N_1 = C_N \cdot N$ See Sheet C for equations & comparisons

d) CCL recommendation to obtain $N_1 = C_N N$

- For $\sigma'_{vo} > 1$ atm
  Liao & Whitman (1986)
  $$C_N = \sqrt{1/\sigma'_{vo}(TSF)} = 10/\sqrt{\sigma'_{vo}\,(kPa)}$$
  (Simple to remember and plots in middle, but gives C_N too high at $\sigma'_{vo} < 1$ atm)
- For $\sigma'_{vo} < 1$ atm
  Skempton (1986)
  $$C_N = \frac{2}{1 + \sigma'_{vo}(TSF)} = \frac{2}{1 + 0.01\,\sigma'_{vo}(kPa)}$$
  (Fairly simple and plots in middle)
- Values

σ'_{vo} (TSF) =	0.25	0.5	1.0	1.5	2.0	3.0
C_N =	1.60	1.33	1.0	0.82-	0.71	0.58-

3.3 Estimation of D_r From SPT N Data

1) Historical Perspective

a) Proposed correlations (modified for this summary); N_1 = corrected N at σ'_{vo} = 1 atm

(1) Peck & Bazaraa (1969) JSMFD 95(SM3): Field data on dense, coarse (OC?) sands

- $D_r = \sqrt{\frac{N_1}{85}}$ à la Skempton (1986) "Old" US practice → very low ER (high N_1)

(2) Holtz & Gibbs (1979) JGED 105(3) :: Mean from lab tests on coarse & f. silty sand
(1969) JSMFD 95(SM3)

- $D_r \approx \sqrt{\frac{N}{16 + 23\sigma'_{vo}(TSF)}} \rightarrow \sqrt{\frac{N_1}{39}}$ Probably high ER (low N_1)

(3) Marcuson & Bieganouski (1977) JGED 103(GT6,11): Mean from lab tests on four v. coarse to fine sands

- $D_r(\%) = 12.2 + 0.75\sqrt{222N + 2311 - 711(OCR) - 736\,\sigma'_{vo}(TSF) - 50\,C_u^2}$

- For OCR = 1, Skempton developed: $D_r = \sqrt{\frac{N_1}{52 \rightarrow 33}}$ Very high ER (low N_1)
  - 52: D_{50} = 2 mm
  - 33: D_{50} = 0.25 mm

b) Comparison of correlations
(also see p15)

	Predicted D_r (%)		
N_1	P&B ('69)	H&G ('79)	M&B ('77)
10	34	51	44 → 55 C → F sand
30	59	88	76 → 95 C → F sand
	FIELD	LAB	

c) Why so different (Largely from Skempton 1986)?

(1) Large differences in Energy Ratio (ER), e.g. $(N_1)_{60} \doteq 0.75 N_1$ for P&B ('69) with ER = 45%
vs $(N_1)_{60} \approx 1.1 N_1$ for M&B ('77)

(2) Lab testing on freshly deposited sands vs. field data on "aged" deposits, plus also maybe OC (both → increased N_1 at same D_r)

(3) At same D_r, increasing grain size (D_{50}) → increasing N_1

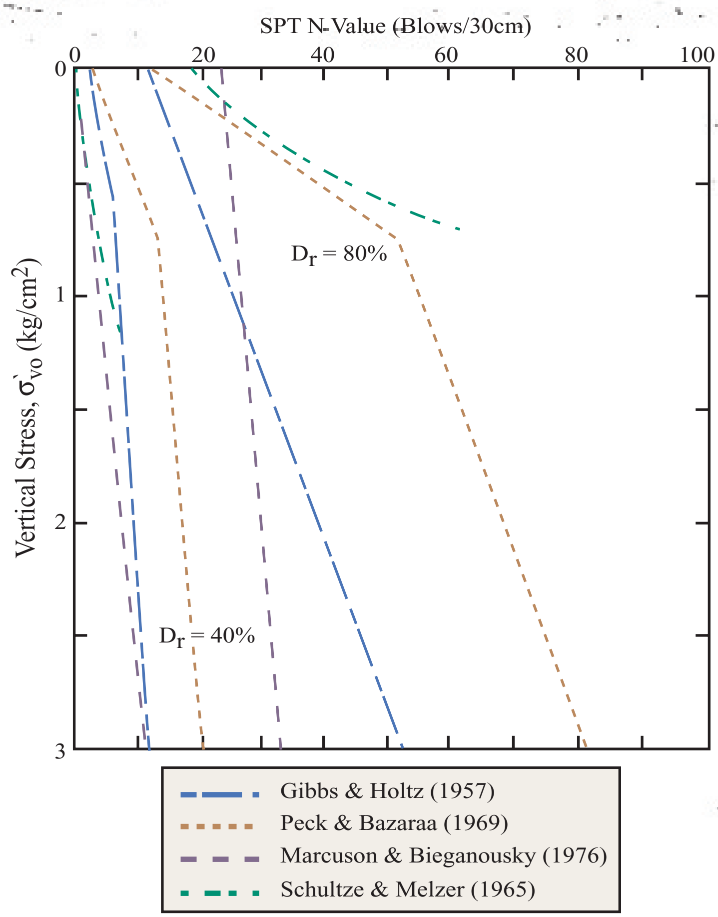


Fig. 47. Empirical correlations between standard penetration resistance and relative density for cohesionless soils.

Adapted from Ladd et al. (1977) 9th ICFMFE

Illustration of Differences in N-D_r Correlations

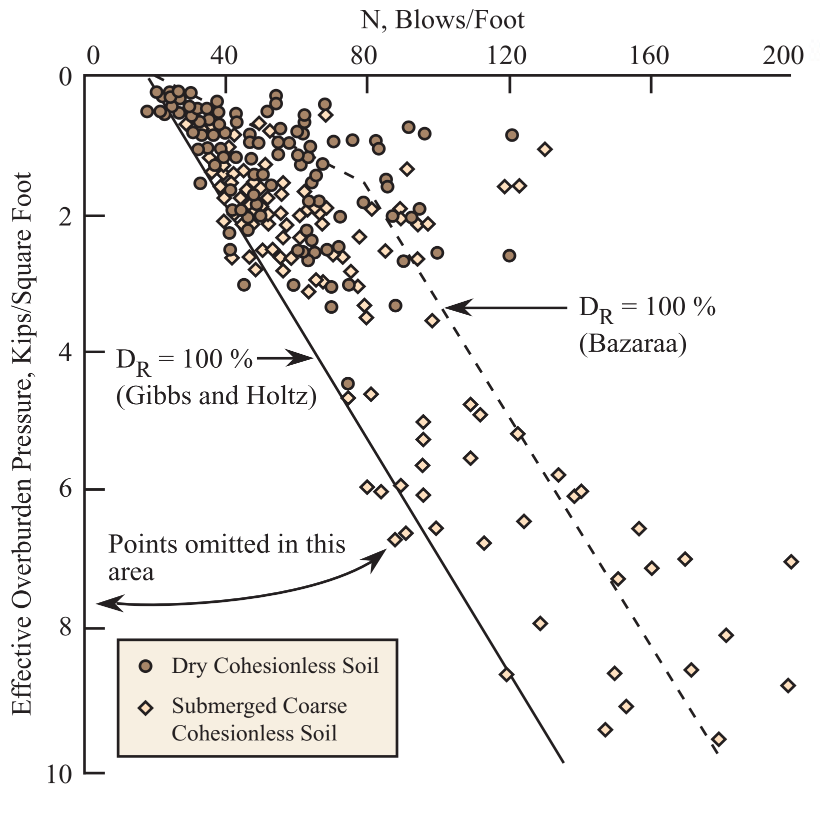
- Fig 47 compares 4 correlations (③ = lab tests on one fine sand)
- Fig 30 compares 2 correlations with field data on very dense sands

NOTE: Bazaraa = Peck & Bazaraa (1969) that used $\sigma'_{vo} = 0.75$ TSF as the reference stress.

$D_r = \sqrt{N_B/80}$ where

$N_B = \dfrac{4N}{(1+4\sigma'_{vo})}$ for $\sigma'_{vo} \leq 0.75$ TSF

$N_B = \dfrac{4N}{(3.25+\sigma'_{vo})}$ for $\sigma'_{vo} \geq 0.75$ TSF



Results of Standard Penetration Tests for Very Dense Sands

Adapted from Peck & Bazarra (1969)

2) Results from Skempton (1986)

- He evaluated above historical data plus Terzaghi & Peck (1948=1967) plus field data from Japan in terms of $(N_1)_{60}$, i.e. accounted for variations in energy ratio and N corrected to $\sigma'_{vo} = 1\,atm$
- Sheet D summarizes Skempton's results, which led to conclusions in 1c
- Note that T&P correlation can be closely fitted by $D_r = \sqrt{\frac{(N_1)_{60}}{60}}$

3) CCL Recommended Correlations

a) Natural sand deposits $D_r = \sqrt{\frac{(N_1)_{60}}{\begin{matrix}55 \text{ fine sand} \\ 65 \text{ coarse sand}\end{matrix}}}$ $D_{50} \lessgtr 0.5mm$

Note: May overestimate D_r for high OCR sands

b) New fill deposits $D_r = \sqrt{\frac{(N_1)_{60}}{55 + 25 \log D_{50}(mm)}}$

c) See Sections 3.2-3 & 4 for procedures to estimate N_{60} & $(N_1)_{60}$ respectively. For typical US practice using 2 rope turns on large cathead

$$\left.\begin{array}{ll} \cdot \text{ Donut hammer, standard sampler}: & N_{60} = 0.75N \\ \cdot \text{ " " , no liner } (\phi = 38mm): & N_{60} = 0.9N \\ \cdot \text{ Safety hammer, " " "} : & N_{60} = 1.1N \end{array}\right\} \begin{array}{l} \text{For } \ell > 10m\ (30') \\ \text{See Sheet B, Table 7} \\ \text{for } \ell < 10m \end{array}$$

3.4 Estimation of ϕ' From Lab Testing

1) On "undisturbed" samples: 2 problems
   - Change in density during sampling à la Section 3.1-1
   - Very difficult & expensive to set up test specimens

2) On reconstituted samples: 2 problems
   - Potential error in estimating D_r (Section 3.2), plus need e_{max} - e_{min}
   - Preparation technique to simulate natural sand structure

3) Conclusion: On very important jobs, consider use of in-situ freezing and sampling → lab testing

3.5 Estimation of ϕ' From D_r and Sand Type

1) Very indirect method, e.g., estimating D_r (usually from N data) and then estimating ϕ' vs. D_r as function of sand type (USCS)

2) Sheet E contains two ϕ'_p vs D_r correlations — Sands with $D_r = 75\%$
   - DM-7.1 probably is rather conservative → $\phi' = 36 \pm 1°$
   - Schmertmann (1978) probably is upper limit → $\phi' = 40 \pm 2°$

3) CCL also would use Bolton (1986), although this approach requires an estimate of ϕ'_{cs} (his Table 1 → $\phi'_{cs} = 34° \pm 2°$ SD for mostly uniform fine to coarse sands)

3.6 Estimation of ϕ' From SPT N Data

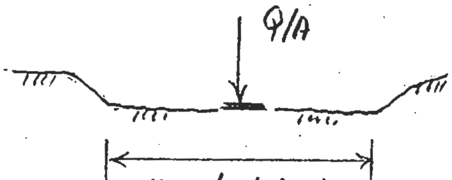
1) Fig. 1 in Sheet F presents two early correlations. Note significant difference in ϕ' at same $N \geq 10$. When using this chart, CCL recommends correcting the measured N to $\sigma'_{vo} \approx 1$ atm, ie, use N_1 (or even $(N_1)_{60}$).
   - For SPT in fine and silty sands, Meyerhof (1956) recommended reduced $N' = N + 0.5(N - 15)$ for $N > 15$ (due to partial drainage of dilatant sand, but only if below the water table)

2) Fig. 2 in Sheet F presents correlations between ϕ' and $(N_1)_{60}$ from TPM's ('96) book: based on "various proposals ... (PHT '53, De Mello '71, Schmertmann '75 & Stroud '88)". Further comments are:

- Underestimates ϕ' for calcareous sands (due to particle crushing)
- Overestimates ϕ' for OC sands (due to increased K_0 → more side friction)
- Agrees reasonably well with CPT correlations (See Section 3.8) that used a different data base

3.7 Estimation of ϕ' From Plate Load Tests (PLT)

1) PLT procedure à la ASTM D1198



- Dia. = 6-30 in
- Maintain each load until $d\rho/dt \le 0.001''/\text{min}$ for $\Delta t = 3$ min

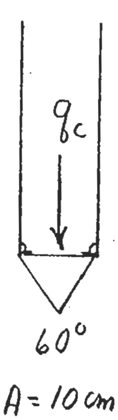
2) $q_{ult} = (0.6)\,\frac{1}{2}\,\gamma B N_\gamma$

∴ Estimate ϕ' from measured N_γ.

3) Remarks: Is very expensive and must test soil at representative depths

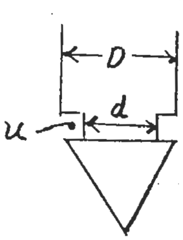
3.8 Correlations for Cone Penetration Tests (CPT)

1) CPT procedure à la ASTM D 3441-94 (For Electric CPT)



- Penetrate at 1-2 cm/sec
- Internal load cell measures q_c

Note: Really should measure $q_t = q_c + u(1-a)$



where $a = \dfrac{d^2}{D^2}$ ($a \approx 0.5$ to 0.85, but usually 0.7 ± 0.05)

2) Relationship between CPT q_c and SPT N

- See Sheet G, Fig. 11.58 → q_c(bars)/N ≈ 4 to 8 for fine to coarse sand.

3) Estimation of Dr (Sheet G; mostly using data set leading to Sheet H)

- Fig. 4 shows that sand compressibility affects $D_r = f(q_c \,\&\, \sigma'_{v0})$, i.e., higher compressibility → lower q_c at same $D_r \,\&\, \sigma'_{v0}$

- Fig. 5 presents $D_r = f(q_c \,\&\, \sigma'_{v0})$ for NC sands of moderate compressibility. Authors suggest using σ'_{h0} for OC sands; they also state that Fig. 5 is "approximate and should be used as a guide" due to unknown sand compressibility at high σ' levels around cone tip

4) Estimation of ϕ'

a) Sheet H presents evaluation of data from several series of tests in calibration (bin) chambers

- Fig. 6 - Compares q_c/σ'_{v0} vs $\tan\phi'$ from theories and experimental data.
- Fig 7 - proposed correlation between q_c vs σ'_{v0} and ϕ', where $\phi' = \phi'$ for TC with $\sigma'_c = \sigma'_{3f}$ = in situ σ'_{h0} for NC quartz sands, Note linear q_c vs σ'_{v0} which is surprising to CCL

b) Sheet I presents correlation in TPM ('96) that presumably used same data set as for Sheet H, but now plotted vs q_{c1} assuming $q_{c1}(kPa) = 10\, q_c / \sqrt{\sigma'_{v0}\, kPa}$ (i.e., same eqn. used to get $N_1 = C_N N$ à la p13)

- CCL added data scaled from Fig. 7 (Sheet H) at σ'_{v0} = 1 bar ≡ 100 kPa
- CCL also added eqn. for $\phi' = f(q_{c1})$, which may be valid only for q_c data obtained at σ'_{v0} near 1 atm

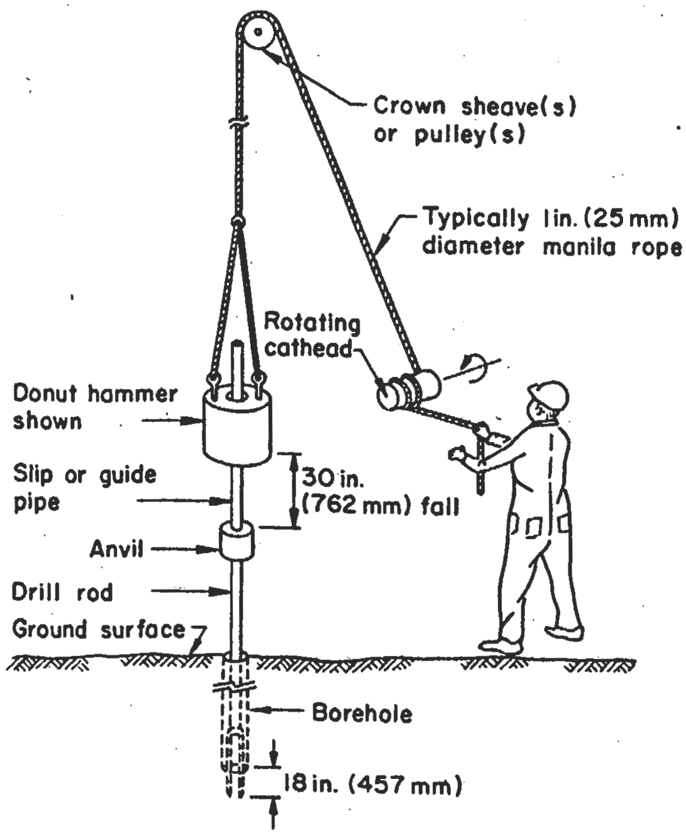


Figure A-4. Equipment Used to Perform the SPT

Source: Kovacs, et al.*

Skempton (1986)

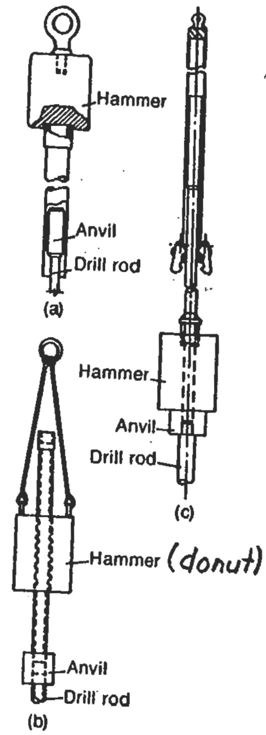
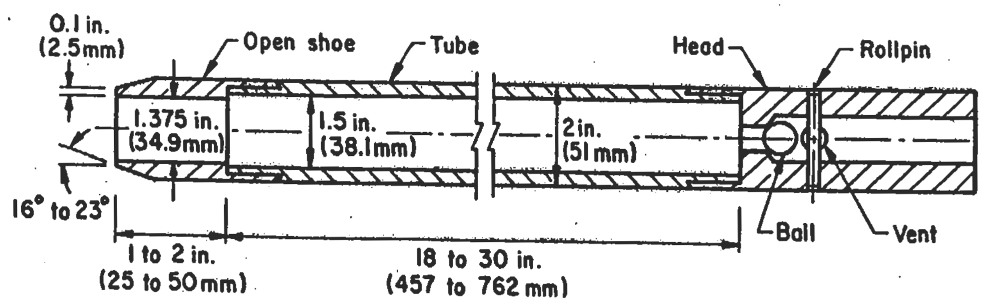


Fig. 4. SPT hammers: (a) old standard; (b) donut; (c) trip

* 1981 report to Nat. Bur. of Standards

From Kulhawy & Mayne (1990) Cornell report to EPRI



ASTM D 1586-84 Note that ID = 1.5" (38mm) enables use of a thin liner to end up with an ID = 1 3/8" (35mm), which is the original dia. & considered an international standard. However, liner is seldom used in the US!

Information on SPT Equipment

A

A. Energy Efficiency

1) Energy delivered to rod stem, E_r = ER × ($w \cdot h$ = 140 lb × $\frac{30}{12}$ ft = 350 ft·lb)

2) Factors affecting Energy Ratio (ER) = Velocity Effic. (VE) × Dynamic Effic. (DE)

a) VE mainly affected by release mechanism: (see Fig. 2-17 & Table 6)

- Automated (trigger/trip) → VE ≈ 1.0
- Rope around cathead → VE ≈ 0.7 - 0.85 for 2 turns

b) DE affected by weight of anvil: inc. wgt (2→20 kg) → dec. DE (0.8→0.6)

3) ER = 60% accepted as best reference. ∴ $N_{60} \approx N\,(ER/60)$

B. Other Factors (Table 7 & Fig. 5)

1) Rod length < 10 m → higher N 2) No liner → lower N 3) Large boring dia. → lower N (granular)

C) N_{60} Eq. $N_{60} = C_{ER} \cdot C_{RL} \cdot C_S \cdot C_B \cdot$ measured N

Table 6. Summary of rod energy ratios (Skempton 1986)

	Hammer	Release	ER_r: %	$ER_r/60$ = C_{ER}
Japan	Donut	Tombi	78	1·3
	Donut	2 turns of rope	65	1·1
China	Pilcon type	Trip	60	1·0
	Donut	Manual	55	0·9
USA	Safety	2 turns of rope	55	0·9
	Donut	2 turns of rope	45	0·75
UK	Pilcon, Dando,	Trip	60	1·0
	old standard	2 turns of rope	50	0·8

Table 7. Approximate corrections to measured N values (Skempton 1986)

Rod length:	>10 m	1·0	C_{RL}
(Fig. 5)	6–10 m	0·95	
	4–6 m	0·85	
	3–4 m	0·75	
Standard sampler		1·0	C_S
US sampler without liners		1·2	
Borehole diameter:	65–115 mm	1·0	C_B
	150 mm	1·05	
	200 mm	1·15	

Kulhawy & Mayne (1990)

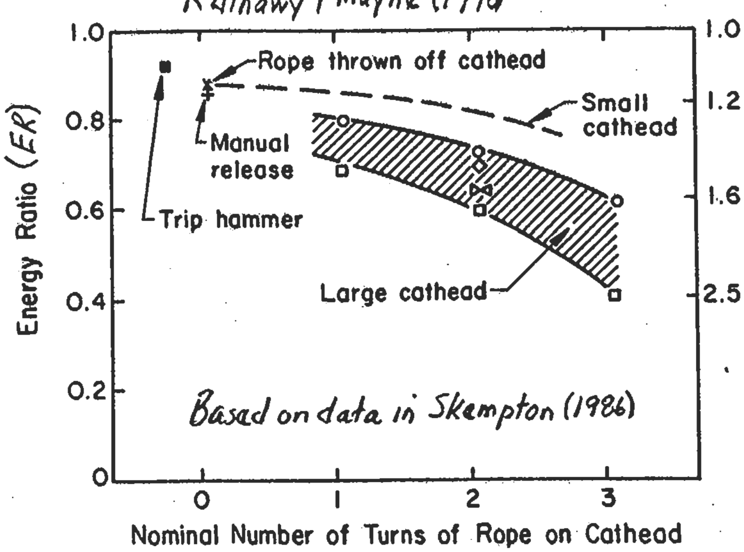


Figure 2-17. Energy Ratio Variations

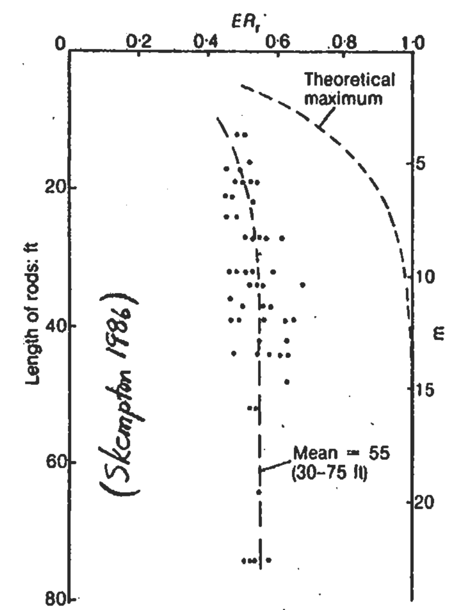


Fig. 5. Effect of rod length for a safety hammer with two-turn slip-rope (after Schmertmann & Palacios, 1979)

Factors Affecting SPT N Values (Skempton 1986)

A. Equations for C_N : $N_1 = C_N N$ For reference $\sigma'_{vo} \approx 1$ atm

1) Peck, Hanson & Thornburn (1974) Book : Evaluation of Peck & Bazaraa (1969) field data

$$C_N = 0.77 \log\left(\frac{20}{\sigma'_{vo}(TSF)}\right)$$

2) Seed (1976) ASCE Report : Evaluation of Gibbs & Holtz (1957) lab data.

$$C_N = 1 - 1.25 \log \sigma'_{vo} (TSF)$$

3) Seed (1979) JGE 105(2) : Evaluation of Marcuson et al. (1977) lab data

C_N = 2 lines for $D_r = 50 \pm 10\%$ & $D_r = 70 \pm 10\%$ (no equation)

4) Liao & Whitman (1986) JGE 112(3) : Evaluation of prior correlations

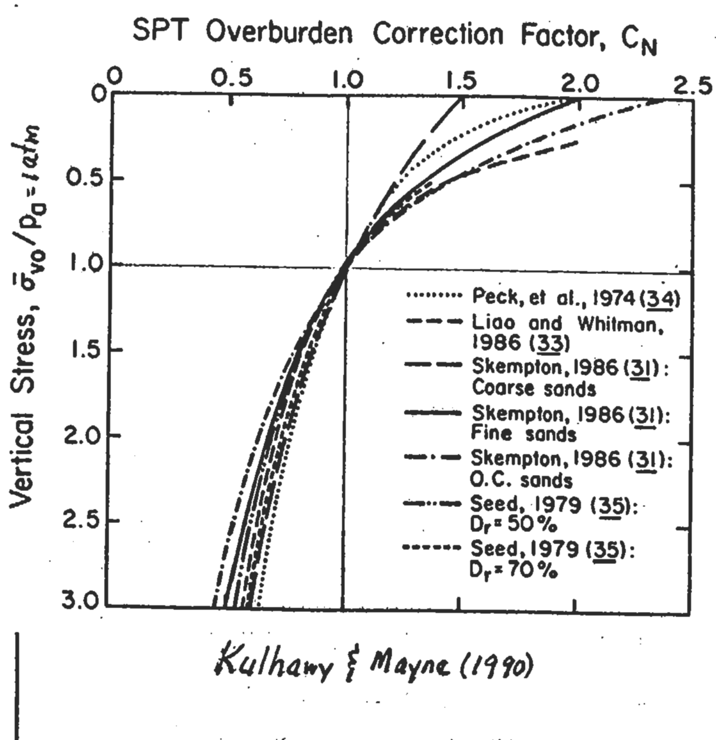
$$C_N = \sqrt{1/\sigma'_{vo}} \ (TSF, kg/cm^2)$$

5) Skempton (1986) : Evaluation of lab and field data

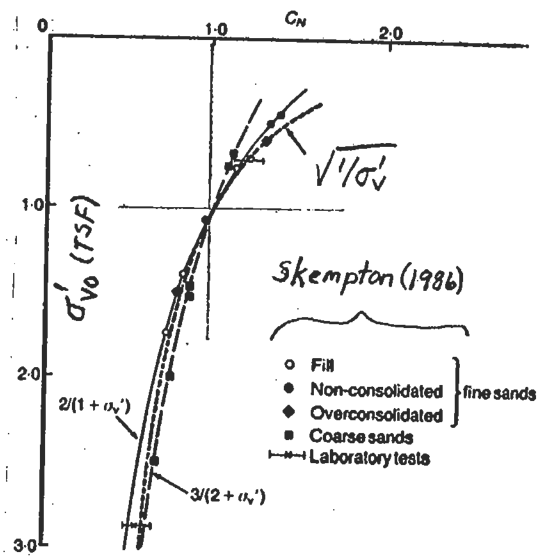
$$C_N = \frac{2}{1+\sigma'_{vo}(TSF)} \qquad C_N = \frac{3}{2+\sigma'_{vo}(TSF)}$$

"Fine sands of medium D_r"   " dense coarse sands when NC"

B. Comparisons



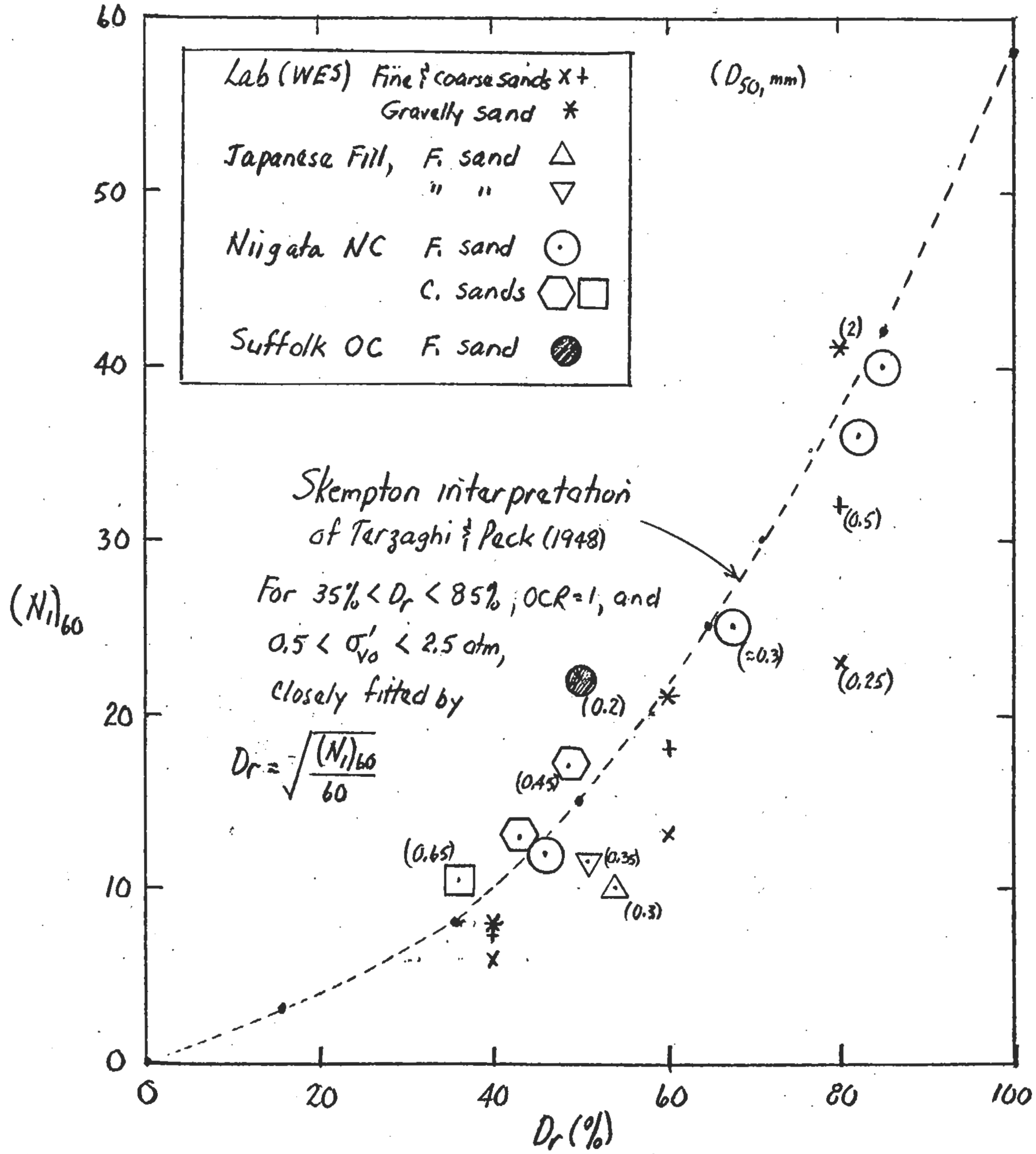
Kulhawy & Mayne (1990)



Liao & Whitman (1987)
Geot. 37(3)

C

From Skempton (1986: Geot. 36(3)) $(N_1)_{60}$ = measured N corrected to Energy Ratio = 60% and σ'_{vo} = 1 atm ≈ 1 TSF ≈ 1 kgf/cm² ≈ 100 kPa



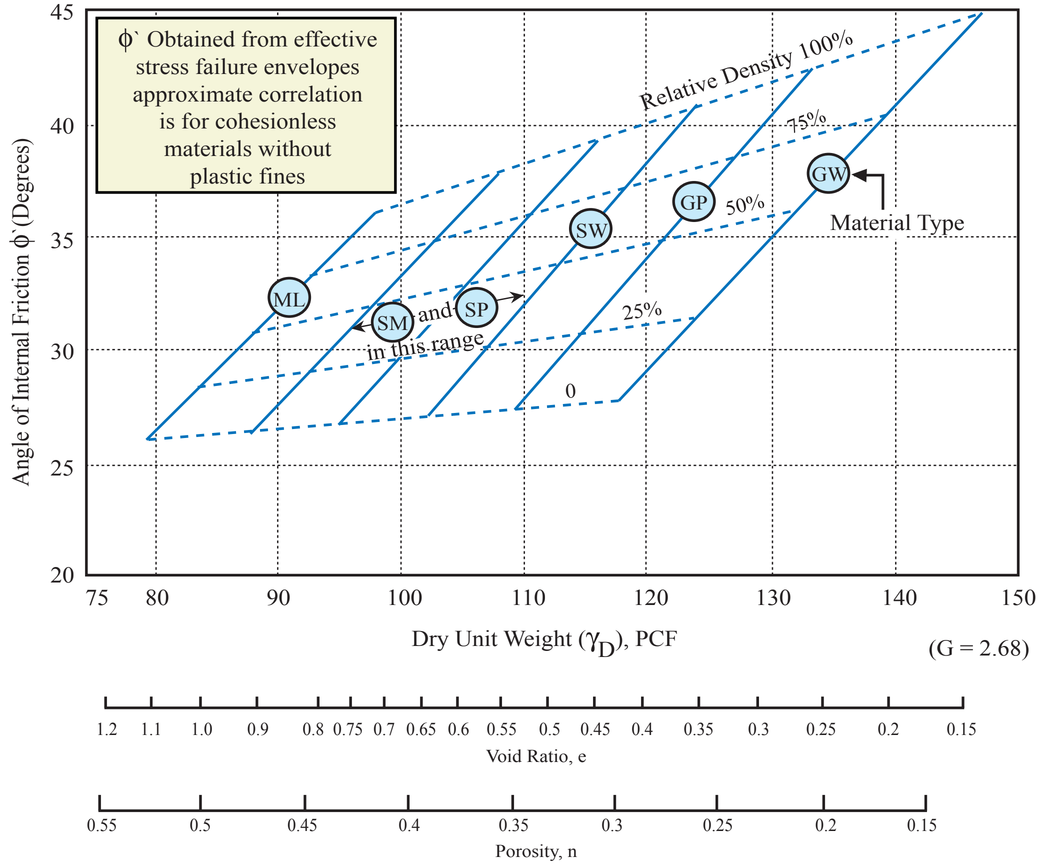
NOTE: For same D_r, $(N_1)_{60}$ increases with:

1) Increasing mean grain size, D_{50}
2) Aging. Therefore higher for natural deposits than for recent fills and lab testing programs
3) Overconsolidation ratio, OCR

D

CCL 10/10/95 | 1.361-1.366 Part III-4

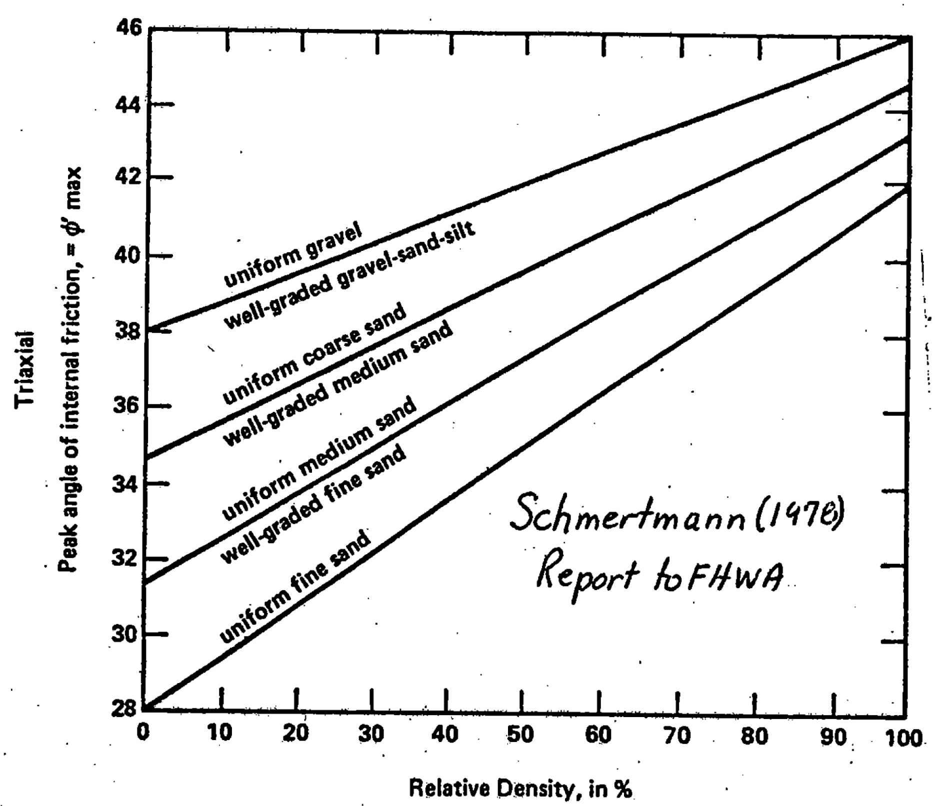
Angle of Internal Friction Vs Density (For Coarse Grained Soils)



Correlations Of Strength Characteristics For Granular Soils

Adapted from NAVFAC DM-7.1 (5/82) p 7.1 - 149

Chart for the approximate evaluation of the peak angle of internal friction after the relative density has been evaluated. Modified from: Burmister, Donald M., "The Importance and Practical Use of Relative Density in Soil Mechanics," ASTM Proc., Vol. 48, 1948



E

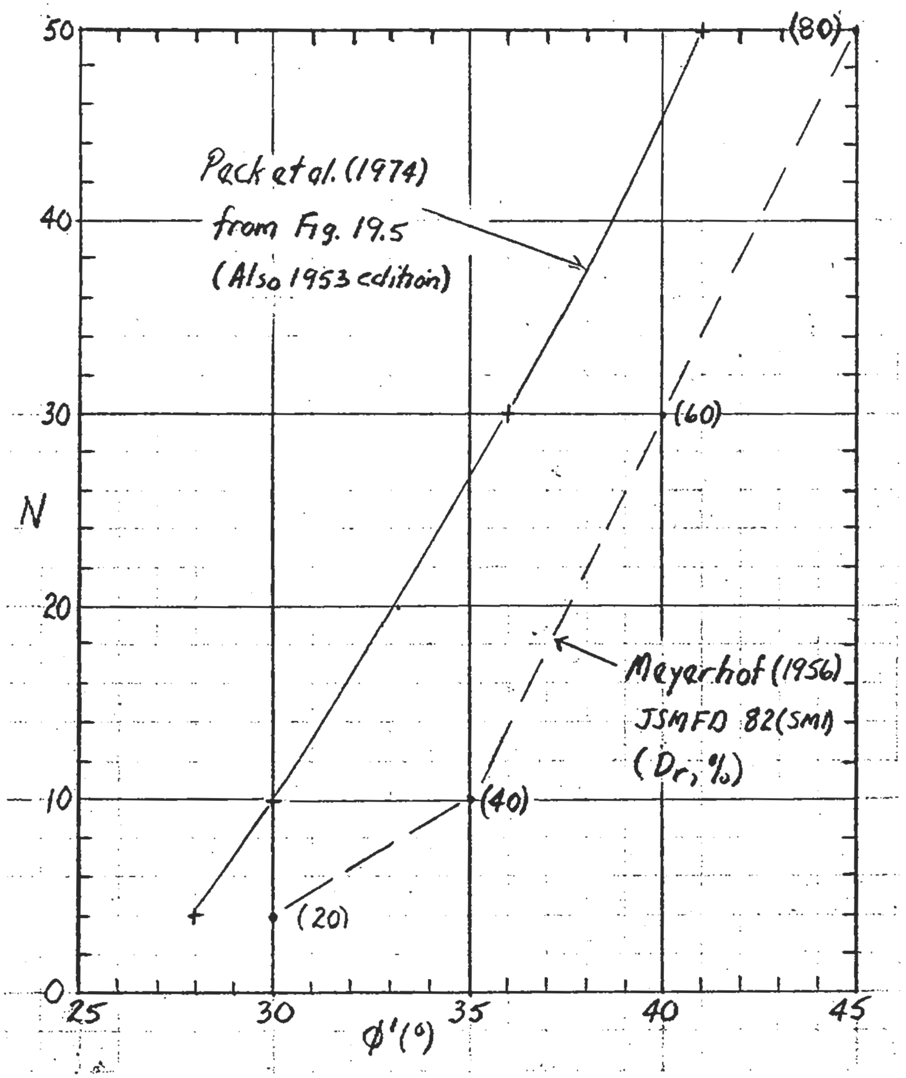


Fig. 1 "Early" Correlations

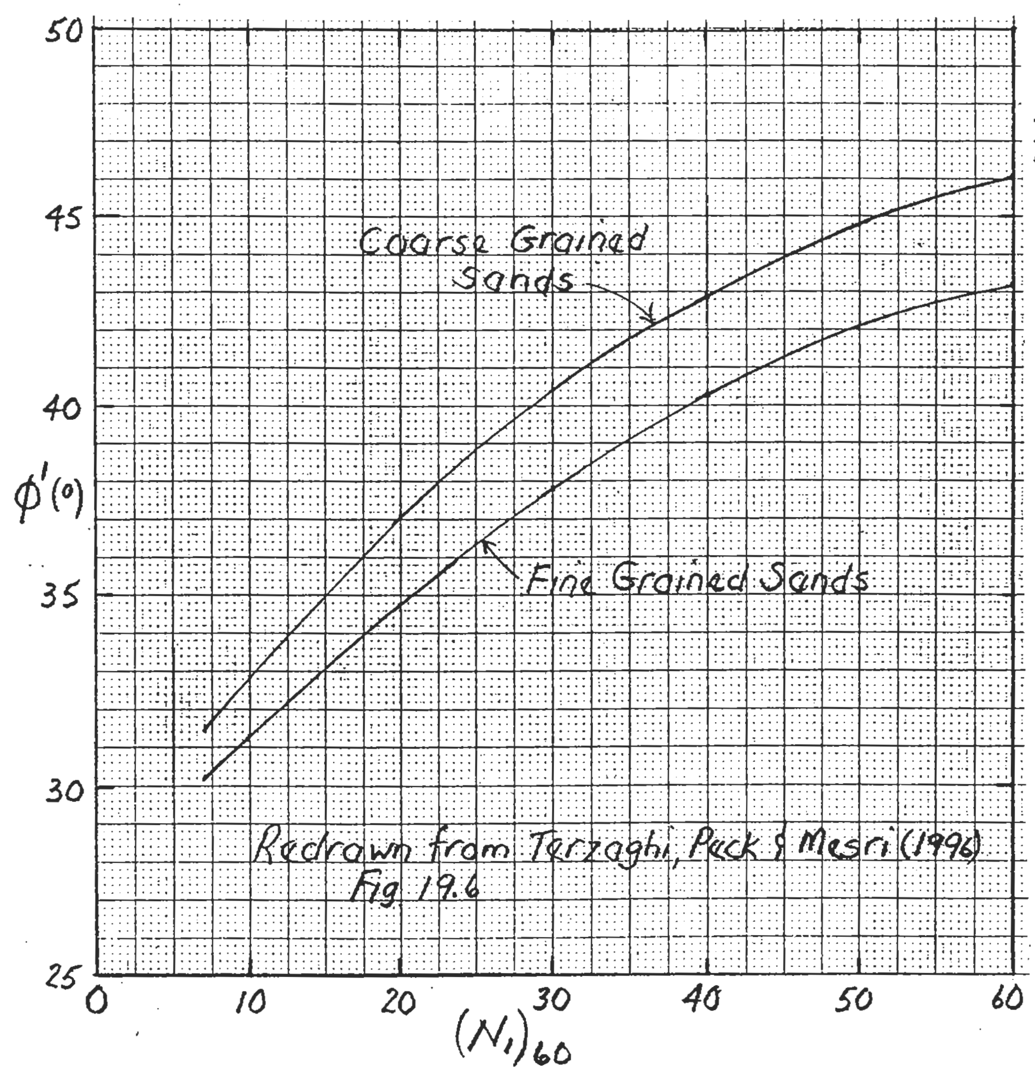


Fig. 2 Recent Correlation

Friction Angle from SPT Blowcount Empirical Correlations

F

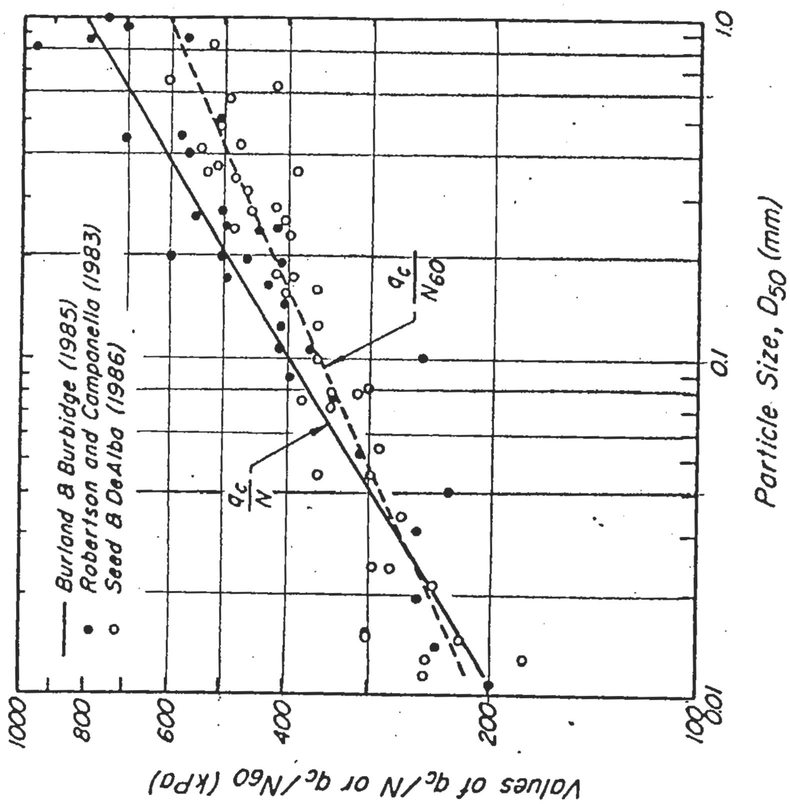


Figure 11.15 Relation between cone penetration resistance q_c and standard penetration N or N_{60} values of sands as related to the median particle size D_{50} of the sands.

(Terzaghi, Peck & Mesri 1996)

Correlations Between SPT N Values and CPT Cone Resistance

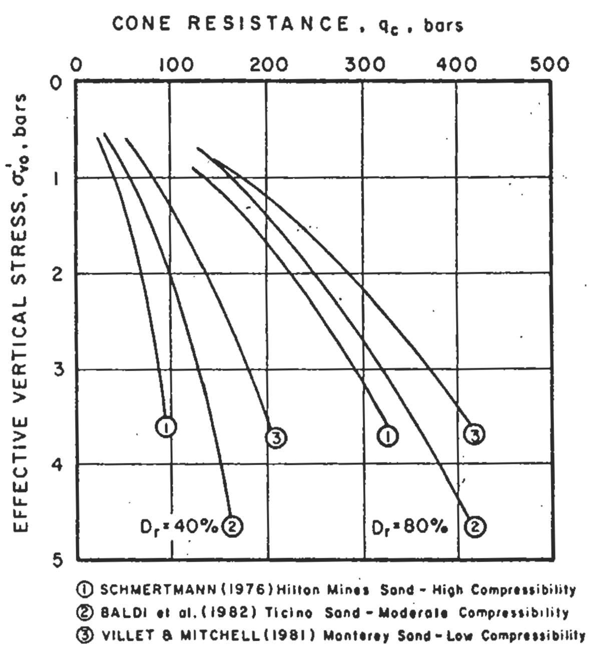


FIG. 4. Comparison of different relative density relationships.

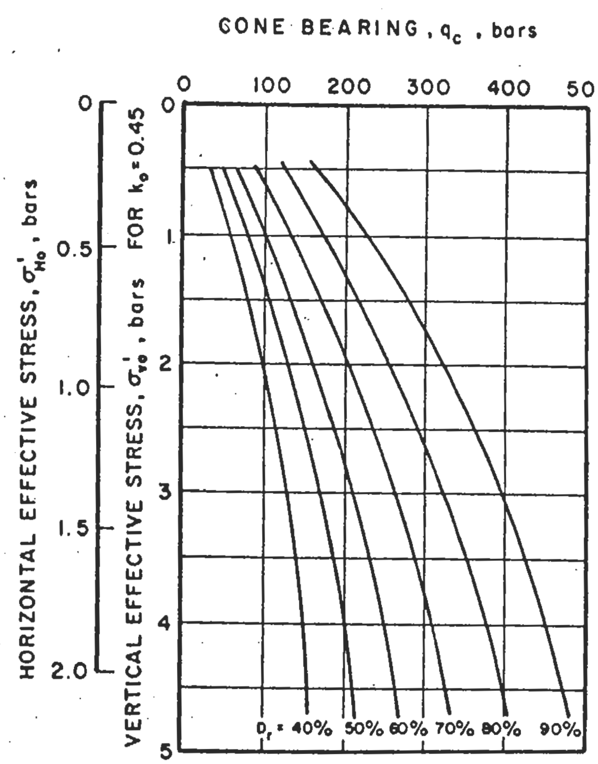


FIG. 5. Relative density relationship for uncemented and unaged quartz sands (after Baldi *et al.* 1982).

Robertson & Campanella (1983)
CGJ 20(4)

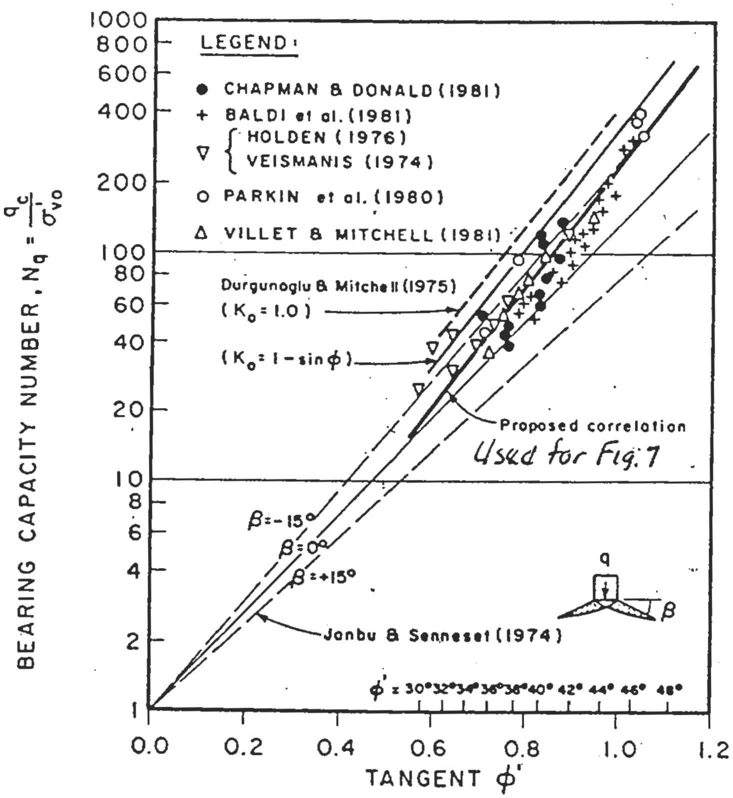


FIG. 6. Relationship between bearing capacity number and peak friction angle from large calibration chamber tests.

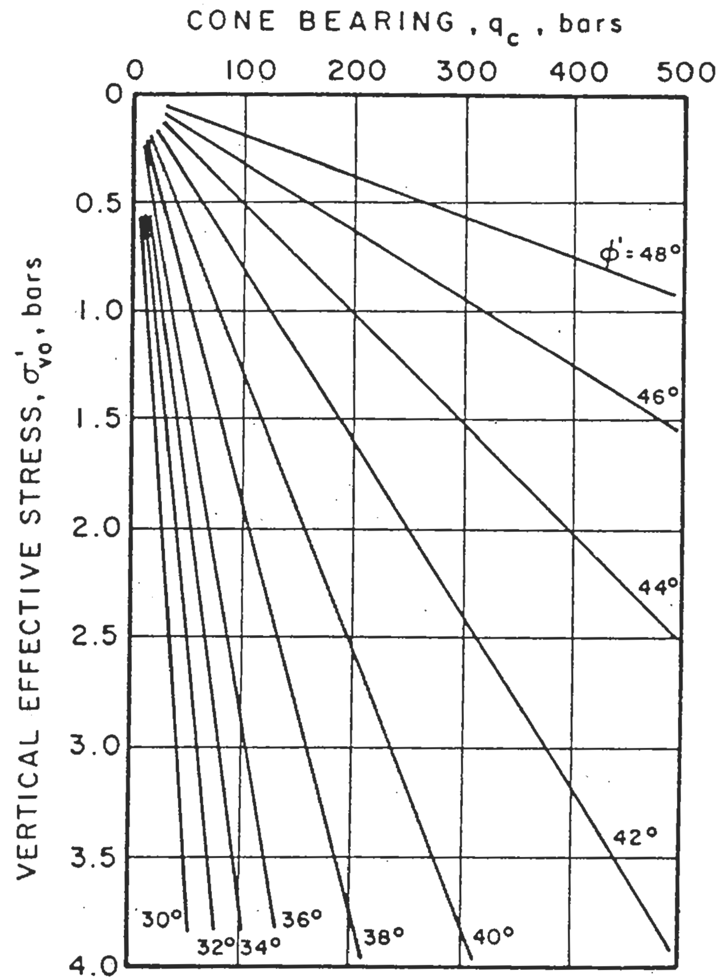


FIG. 7. Proposed correlations between cone bearing and peak friction angle for uncemented, quartz sands.

Robertson & Campanella (1983)
CGJ 20(4)

NOTE: $\phi' = \phi'_{TC}$ for $\sigma'_{3f} \approx$ In Situ σ'_{ho} ; also note linear q_c vs σ'_{vo} relationship

H

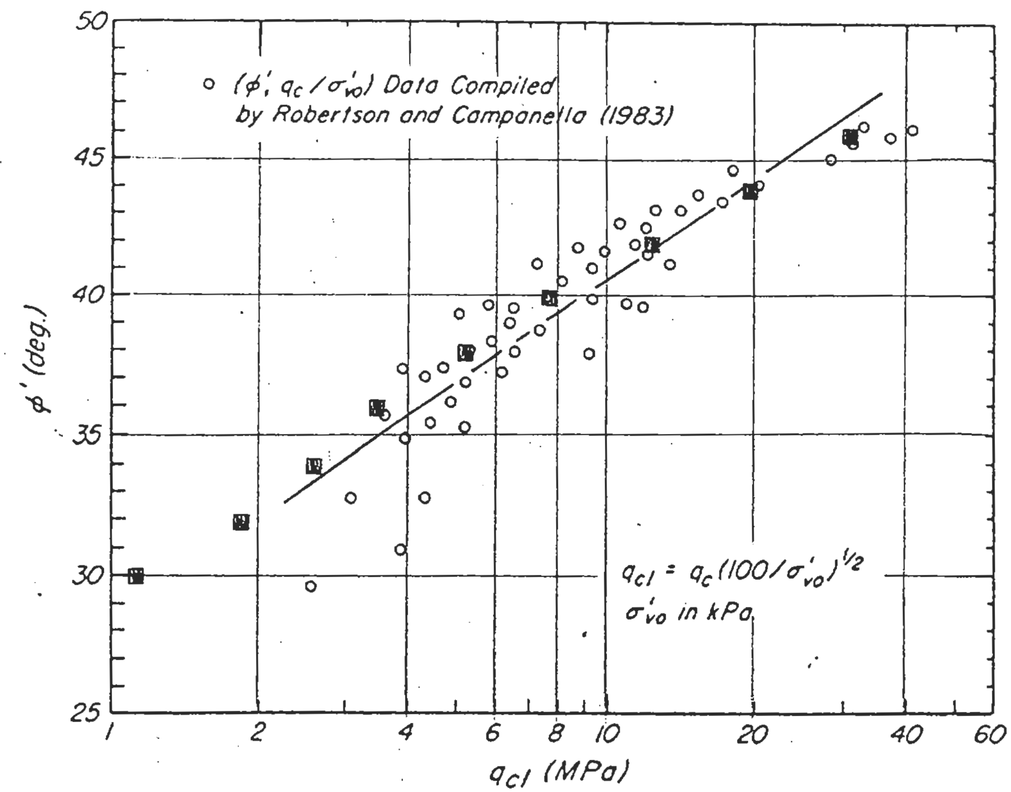


Figure 19.5 Empirical correlation between friction angle ϕ' of sands and normalized push cone tip penetration resistance. (Terzaghi, Peck & Mesri 1996)

■ Scaled from Fig. 7 (Sheet H) at $\sigma'_{vo} = 1\,bar$; therefore $q_c = q_{c1}$

Linear regression → $\phi' = 30.0\,(q_{c1}, MPa)^{0.130}$ ($n=9$, $r^2 = 0.986$)

Correlation line on Fig. 19.5 → $\phi' \approx 28.8\,(q_{c1}, MPa)^{0.145}$; however, Fig. 7 correlation shows $q_c \propto \sigma'_{vo}$, not $q_c \propto 1/\sqrt{\sigma'_{vo}}$ as assumed in Fig. 19.5