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In-situ testing

by David Nash

Standard penetration tests

Other dynamic sounding methods

In-situ vane

Cone penetration tests (CPT, piezocone (CUPT))

Pressuremeter (Menard, Selfboring (SBPM))

Dilatometer

Geophysical (Crosshole, downhole, refraction, SASW)

reference: **In-situ testing in Geomechanics: the Main Tests** by Fernando Schnaid (pub 2009 Taylor & Francis)

Why are in-situ tests useful?

Testing can be carried out and interpreted quickly;

Appropriate tests can test a large volume of soil/rock;

They can be used as part of ground profiling;

They test the ground in its in-situ state, under in-situ stresses;

Tests can be carried out in materials that can't be sampled;

They avoid the difficulties of sample disturbance;

They may be used to measure some soil properties directly (E_0, G_0, K_0, c_v);

They may be correlated with laboratory tests to derive soil properties (c_u, ϕ);

They may be correlated with engineering behaviour at full scale.

But …

You do not have control over the boundary stresses and strains;

Insertion of the device may actually disturb the ground before the test is carried out.

Interpretation of in-situ tests

1. *Wholly empirical interpretation. No fundamental analysis is possible. Stress paths, strain levels, drainage conditions and rate of loading are either uncontrolled or inappropriate. (Examples: SPT)*
2. *Semi-analytical interpretation. Some relationships between parameters and measurements may be developed, but in reality interpretation is semi-empirical, either because both stress paths and strain levels vary widely within the mass of ground under test, or drainage is uncontrolled, or inappropriate shearing rates are used. (Examples: plate test, vane test, CPT.)*
3. *Analytical interpretation. Stress paths are controlled, and similar (although strain levels and drainage are not). (Example: self-boring pressuremeter.)*

"Site Investigation" by C.R.I. Clayton, M.C. Matthews and N.E. Simons

http://www.geotechnique.info/SI/SI%20Book%20Chapter%209.pdf

Standard Penetration Test (SPT)

Standard penetration test (SPT) equipment

65kg hammer free fall 0.76m

Count blows for each 0.075m

N value is sum of blows for increments 3,4,5,6

thus if blowcounts are: 3,2,5,4,6,5
N = 5+4+6+5 = 20

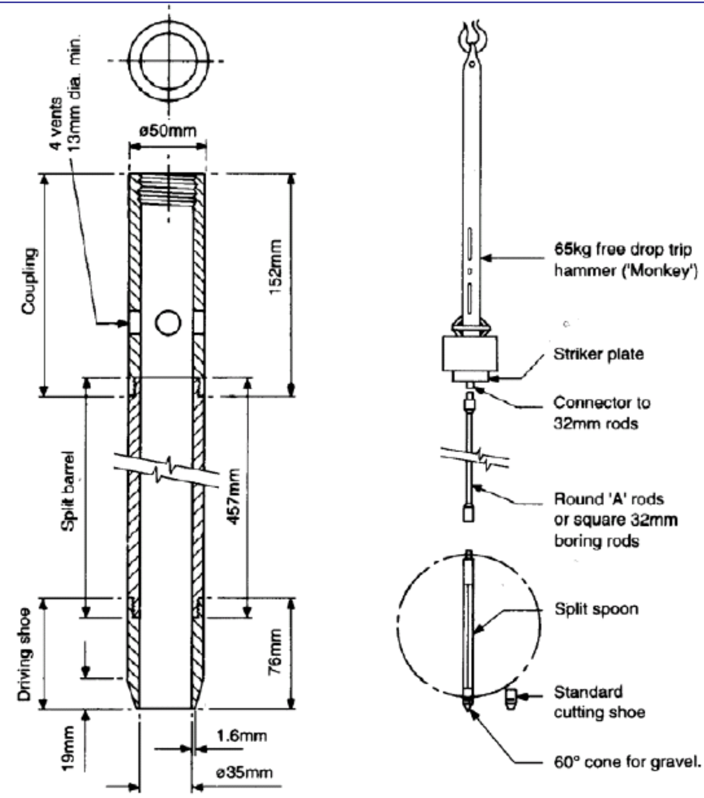

Fig. 7.5 Standard Penetration Test' equipment.

Correlation with density of granular materials and ϕ'

Many correlations published including:

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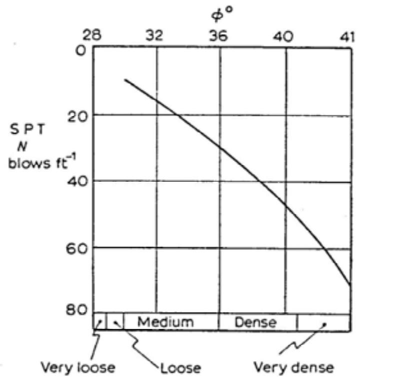
Table 7.5 Relationship between standard penetration, density and strength (*after* Terzaghi and Peck, 1967)

Soil	*Penetration Resistance N blows ft^{-1}*	*State*	*Relative Density percentage*	*Unconfined Compressive Strength kN m^{-2}*
Sand	0– 4	Very loose	0– 15	
	4–10	Loose	15– 35	
	10–30	Medium	35– 65	
	30–50	Dense	65– 85	
	>50	Very dense	85–100	
Clay	< 2	Very soft		< 25
	2– 4	Soft		25– 50
	4– 8	Medium		50–100
	8–15	Stiff		100–200
	15–30	Very stiff		200–400
	>30	Hard		>400



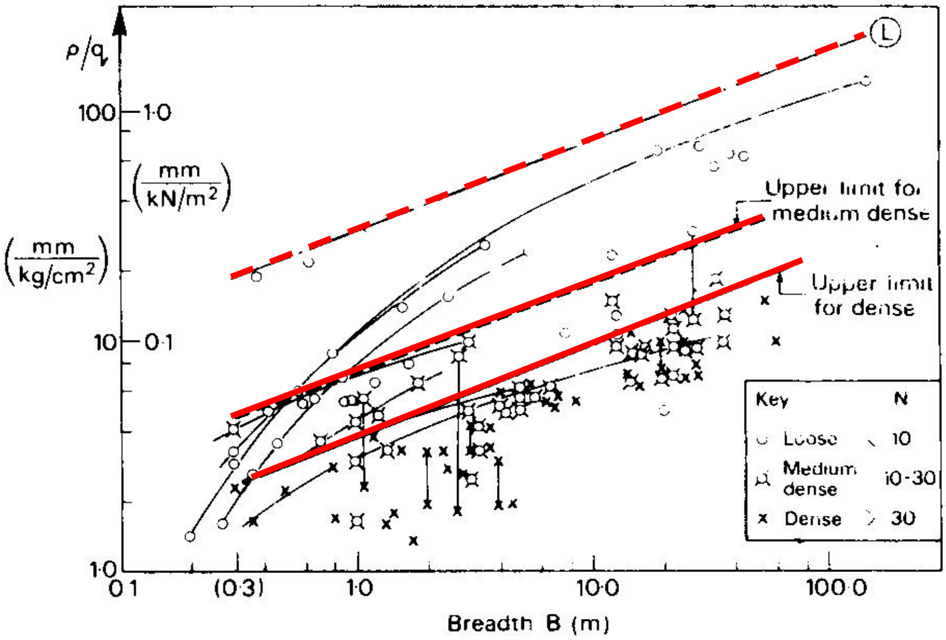
(b) ϕ–N relationship for Standard Penetration Test (after Peck *et al.* 1953)

Figure 7.21 Standard Penetration Test.

Relative density $R_d = (e_{max} - e)/(e_{max} - e_{min})$ x100%

$$R_D = \frac{\gamma_{d\,max}}{\gamma_d} \times \frac{\gamma_d - \gamma_{d\,min}}{\gamma_{d\,max} - \gamma_{d\,min}} \times 100\%$$

Observed settlement of footings on granular soils



Observed settlement of footings on sand of various relative densities.

Burland Broms and de Mello 1977

Estimation of settlement of footings on granular soils using SPT results

Various relationships have been suggested between the settlement of foundations on granular soil and SPT N-values.

One good example is that of Burland and Burbidge (1985), who correlated observation of settlements with SPT data and proposed:

Immediate settlement: $\rho_i = f_s.f_l\left(q' - \frac{2}{3}\sigma'_{v0}\right)B^{0.7}I_c \quad where\ I_c = \frac{1.71}{N^{1.4}}$

Long-term settlement: $\rho_t = f_t.\rho_i$

f_s = correction factor for shape L/B
f_l = correction factor for thickness of sand layer
f_t = correction factor for creep time
q' = applied effective bearing pressure
σ'_{v0} = previous average effective stress at foundation level
B = breadth of foundation
N = average SPT N-value in the layer below of thickness B
(corrected if below the water table but not adjusted for overburden stress)

see Burland and Burbidge (1995) *Settlement of Foundations on Sand and Gravel*. Proc ICE, Part 1 vol 78 pp1325-81

Burland and Burbidge (1985) compressibility I_c

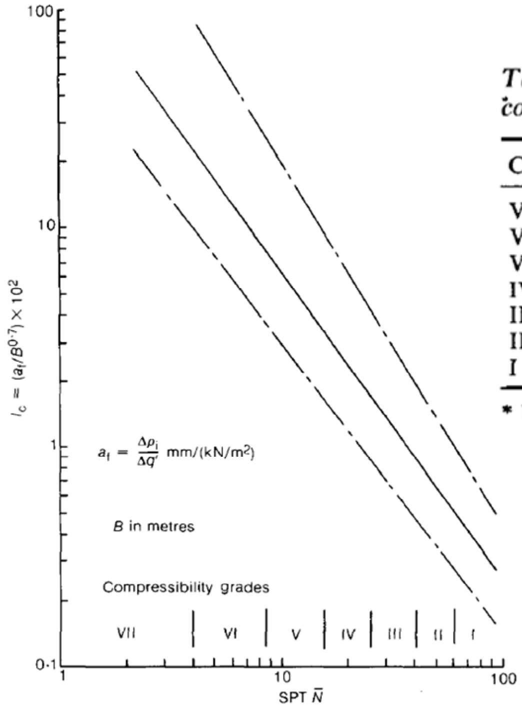


Fig. 1. Relationship between compressibility (I_c) and mean SPT blow count ($\bar{N}$) over depth of influence. Chain dotted lines show upper and lower limits (see Figs 22 and 23)

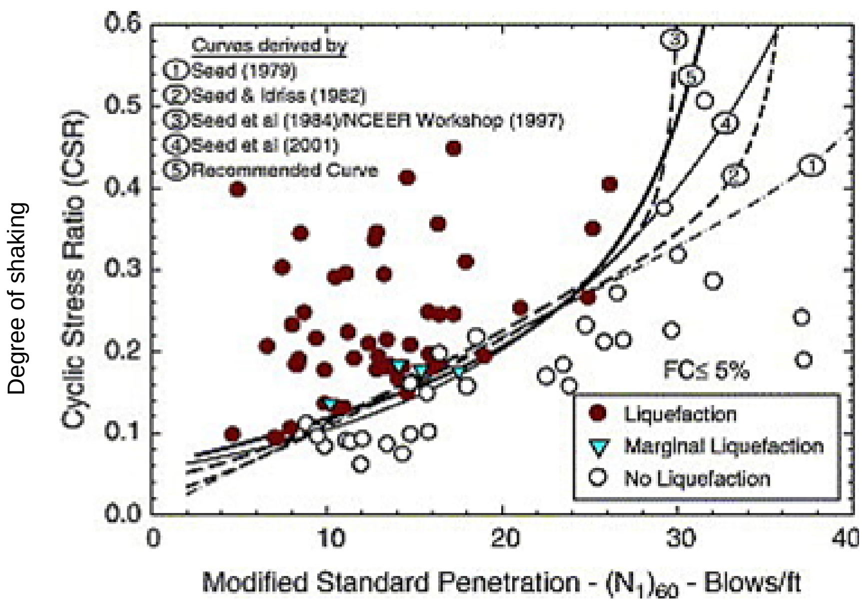
Table 1. Classification of compressibility of normally consolidated sands and gravels with SPT blow count

Compressibility grade	No. of blows N*	Interval
VII	<4	3
VI	4–8	5
V	9–15	7
IV	16–25	10
III	26–40	15
II	41–60	20
I	>60	

* Uncorrected for overburden pressure.

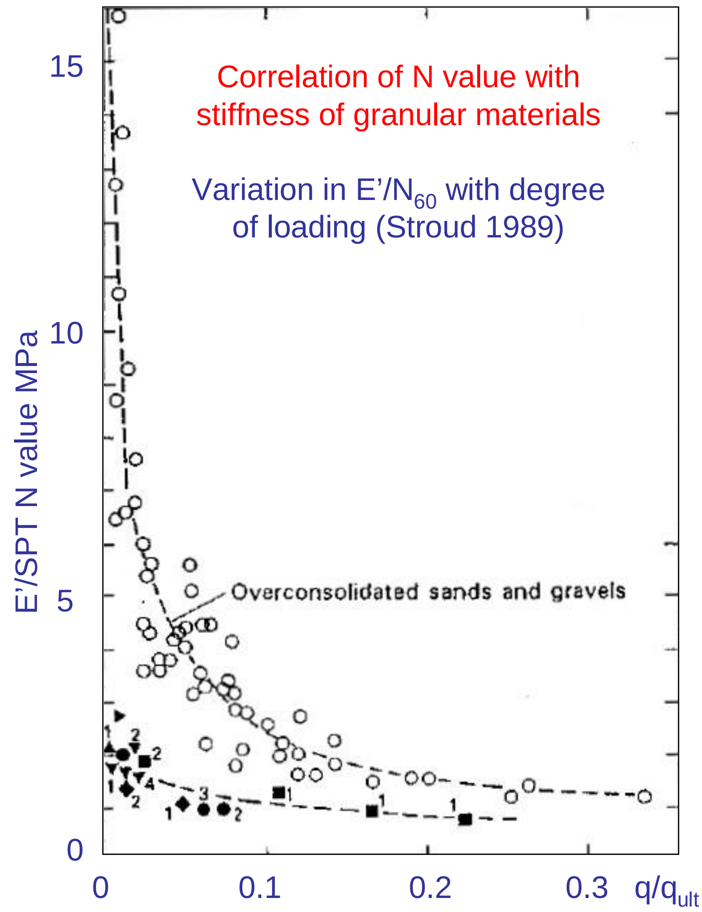
$$\text{where } I_c = \frac{1.71}{N^{1.4}}$$

Use in assessment of liquefaction

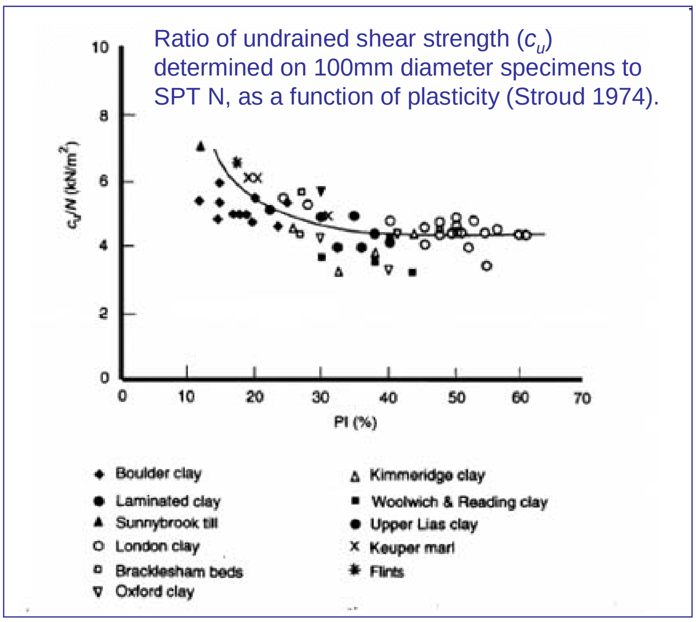


Case histories for sites underlain by clean sands showing the correlation of SPT with observations of liquefaction for different degrees of shaking and the recommended curve for M=7½ and at 100 kPa stress level.

Idriss and Boulanger (2006)



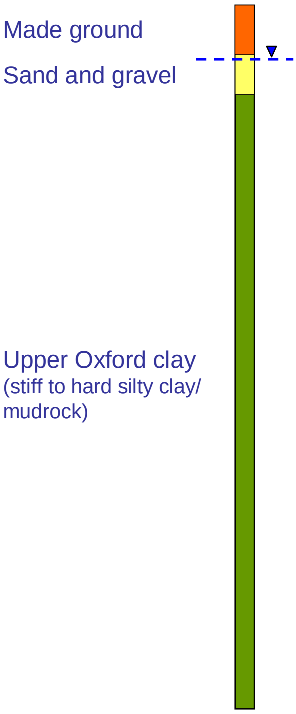
Correlation with undrained strength of clay

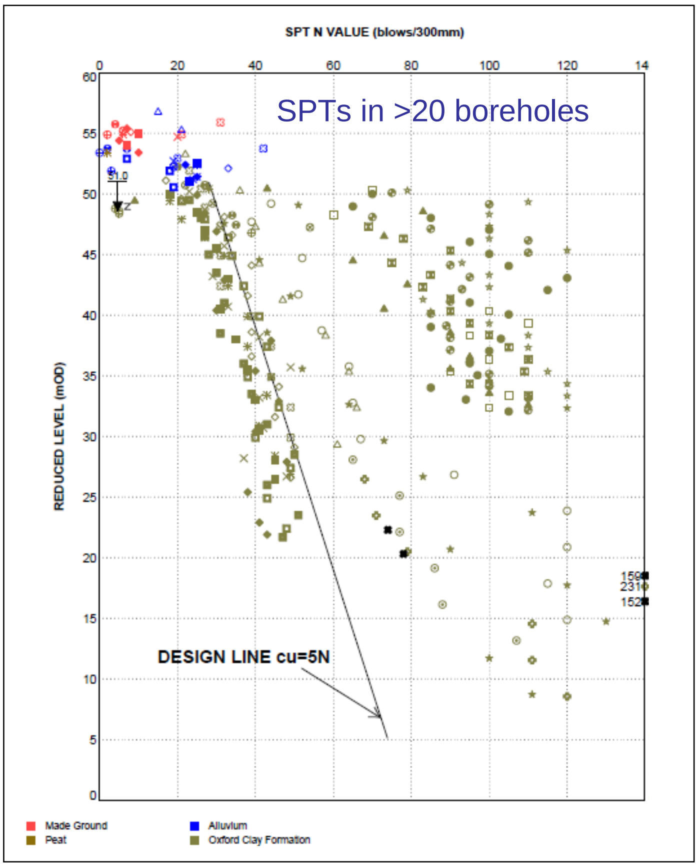


typically $c_u = 5N_{60}$

Stroud, M.A. (1989) "The Standard Penetration Test-its Application and Interpretation". Institution of Civil Engineers Conference on Penetration Testing, Birmingham, United Kingdom. Thomas Telford, London, pp. 29-49

Site investigation in Oxford





Standard penetration test (SPT) equipment
Difficulties in interpretation

Non-standard equipment (particularly hammer, rods)

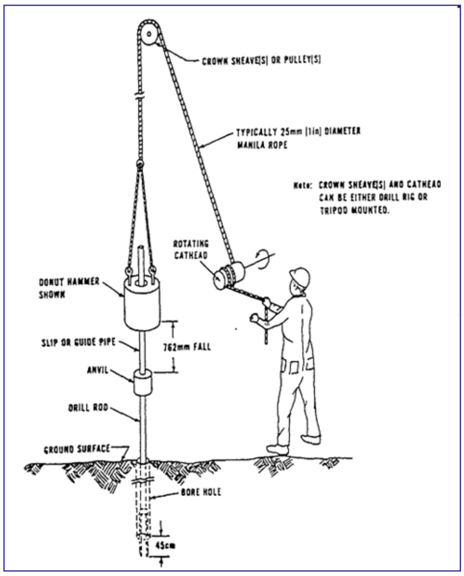
Non-standard technique (poor tightening of screwed rods) reducing energy at depth

Disturbance of soil below borehole (eg boiling)

Effect of overburden stress / depth

Borehole diameter

Correction for loss of energy



To overcome this, Seed et al (1985) proposed correcting all SPT values to that for an SPT system in which 60% of the theoretical free fall energy is imparted to the rods - ie. to N_{60}. This equates to the USA safety hammer with cathead and winch (rope and pulley) drop system on which many correlations have been based.

This is done by

- $N_{60} = N_m.ER_m/60$
- where N_m = SPT 'N' value for the method used in the investigation

  ER_m = rod energy ratio for method used in the investigation

The UK Pilcon trip hammer has ER = 60%, so no correction is necessary when that hammer is in use.

Correction for overburden stress

Skempton (1986) reviewed the available information and proposed a method where the SPT value is corrected for the energy of the hammer, and to an overburden pressure of 100kPa (= 1bar) by

$(N_1)_{60} = C_N.N_{60}$ where $N_{60} = N_m.ER_m/60$

C_N = correction factor

$= 2/(1 + \sigma'v)$ for normally consolidated fine sand ($\sigma'v$ in bars)

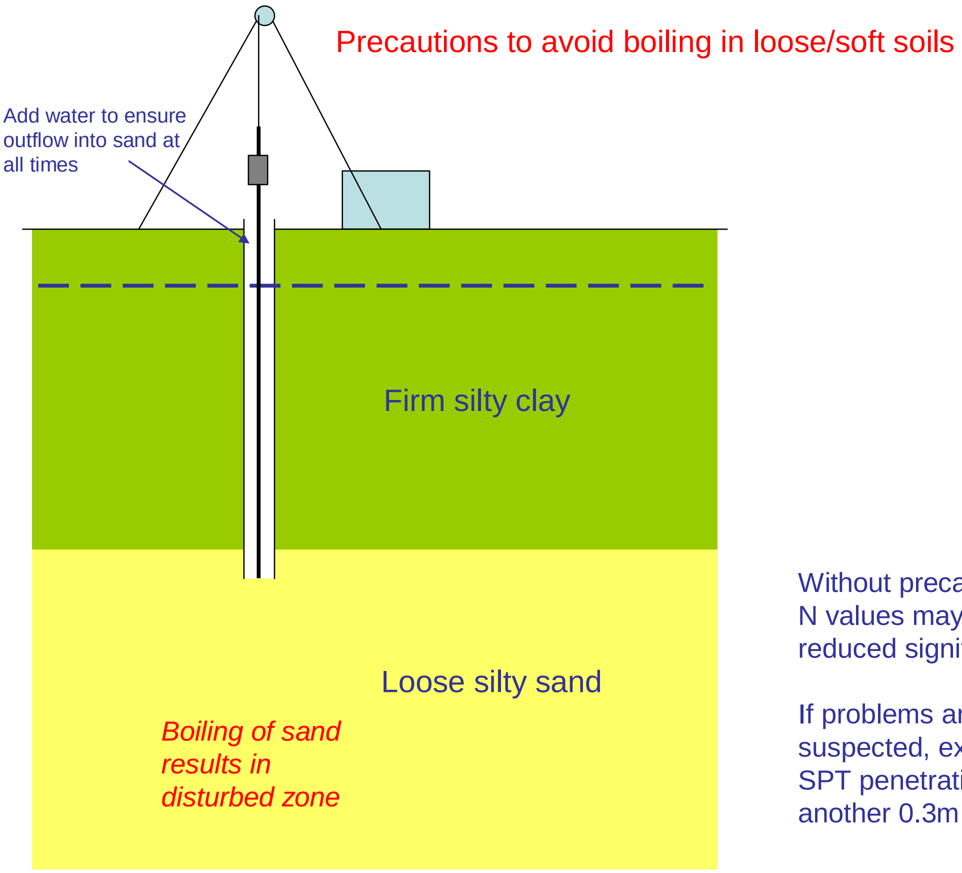
$= 3/(2 + \sigma'v)$ for normally consolidated dense coarse sand

$= 1.7/(0.7 + \sigma'v)$ for overconsolidated sand

Skempton (1986) then uses the $(N_1)_{60}$ value to estimate relative density from Table 2.5.

Table 2.5. Skempton method for estimating relative density from 'N' value

Relative Density %	Condition	$(N_1)_{60}$
0	Very loose	0
15	Loose	3
35	Medium dense	8
65	Dense	25
85	Very dense	42
100		58



Without precautions N values may be reduced significantly.

If problems are suspected, extend the SPT penetration by another 0.3m.

Conclusions

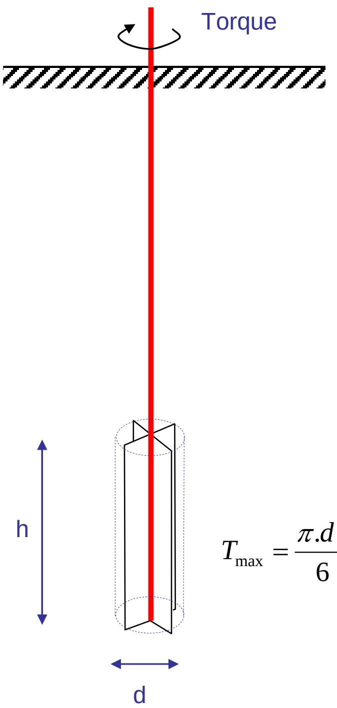
- The SPT is a widely used in-situ test.
- When carried out in accordance with the standard it gives a useful indication of relative density, stiffness and strength of soils.
- Test data should be corrected for non-standard equipment and procedures and for depth/overburden stress.
- Various correlations between N value and R_D, E′ and c_u and liquefaction have been developed. These should be calibrated against local experience.

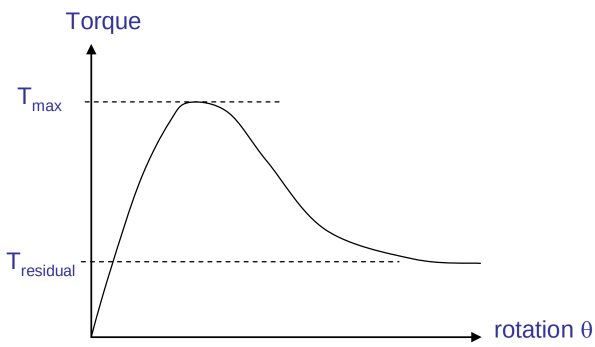
In-situ vane tests

by David Nash

In-situ vane test

Tests may be carried out at the base of a borehole or a penetration vane is used.





$$Sensitivity = \frac{T_{\max}}{T_{residual}}$$

$$T_{\max} = \frac{\pi.d^3}{6}\tau_h + \frac{\pi.d^2h}{2}\tau_v = \pi\frac{d^2}{6}(d+3h)c_u \quad \text{if } \tau_h = \tau_v = c_u$$

Typical size 150mm x 75mm

Standard rotation rate 6 degrees/minute

Geotechnical profile in soft clay at Drammen, Norway

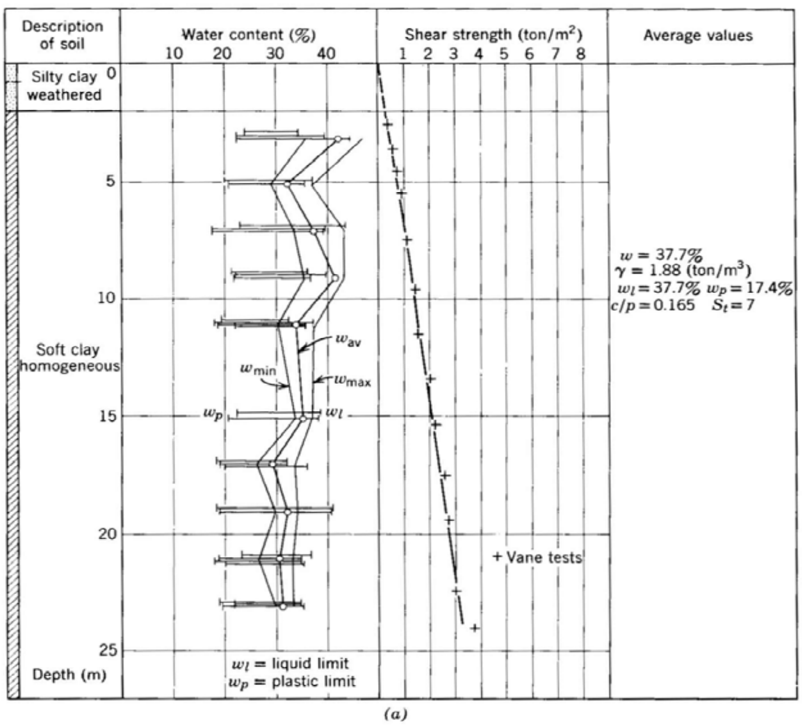


Fig. 7.7 Norwegian marine clay. (a) Results of a boring in Drammen. (b) Results of a boring at Manglerud in Oslo. (From Bjerrum, 1954.)

Note that c_u increases linearly with depth and σ'_v

thus c_u/σ'_v is constant

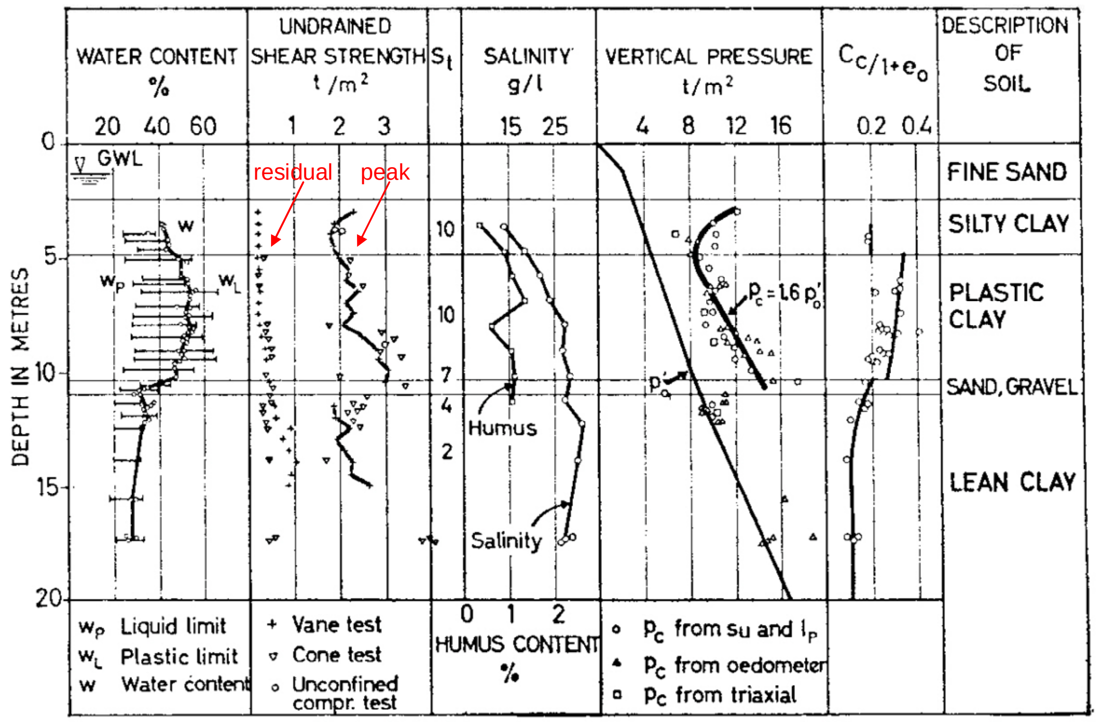


Fig. 19. Geotechnical profile typical of the considered area in Drammen

Geotechnical profile in soft clay at Tilbury, UK

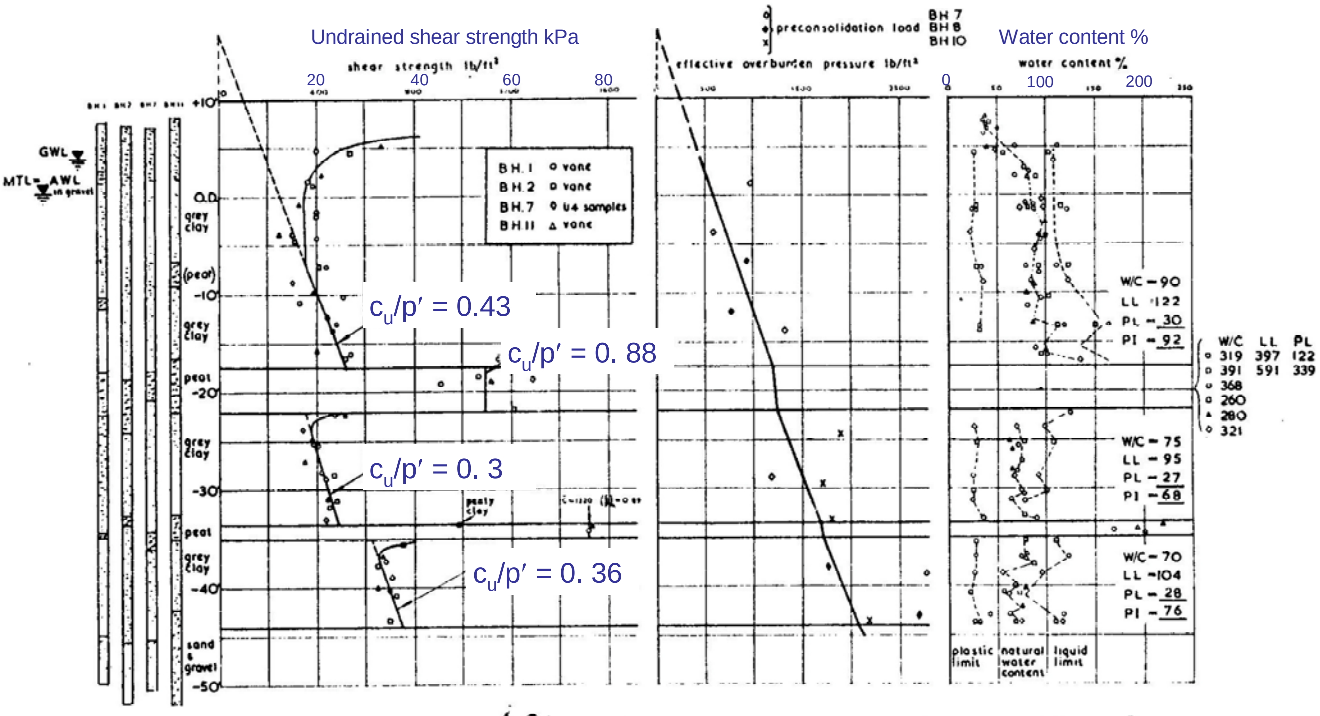


Fig. 4 Tilbury Marsh Borings
Sondages de marais à Tilbury

(Skempton and Henkel 1953)

Empirical relationship between c_u/σ'_v and plasticity index

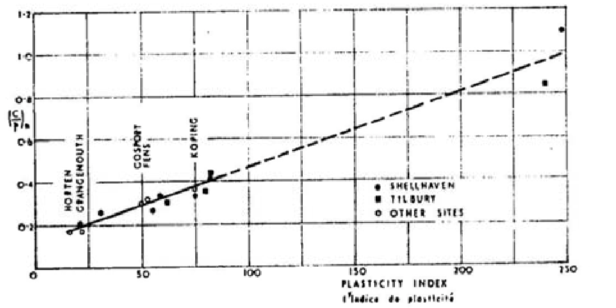
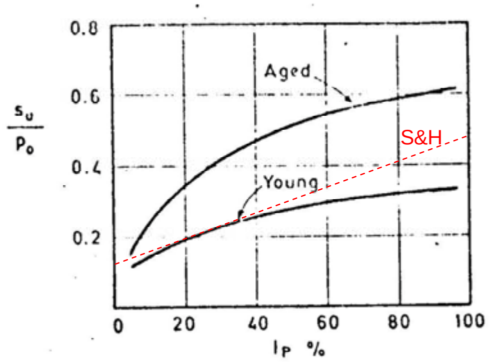


Fig. 8 Relation Between $\left(\frac{c}{p}\right)_u$ and Plastic Index

(Skempton and Henkel 1953)



(Bjerrum 1972)

$c_u/\sigma'_v = 0.11 + 0.0037 * PI$

Correction factor for vane strengths derived from back-analysis of embankment failures (Bjerrum 1972)

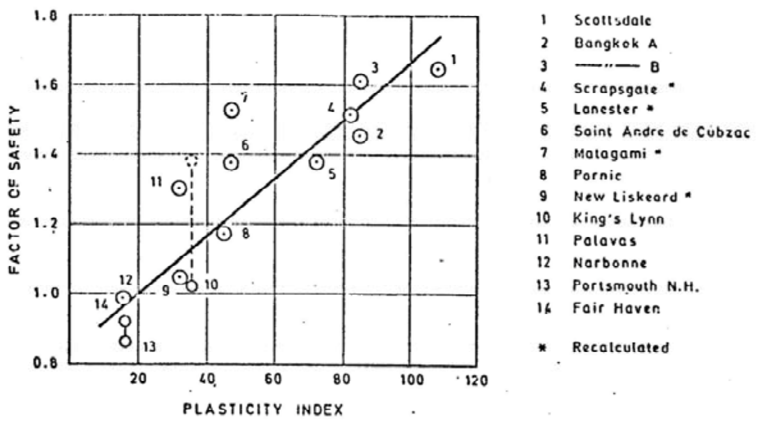
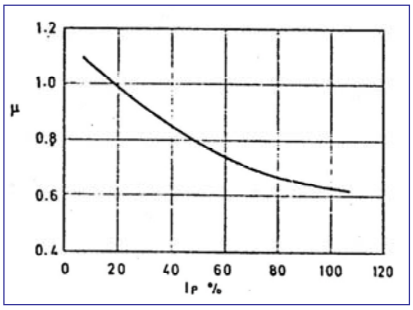


Fig. 2. Theoretical factor of safety at failure of embankments on soft clay, plotted against plasticity index of foundation clay.



$$c_{u\ corrected} = c_{ufv} \times \mu$$

Total stress ($\phi = 0$) analyses

Anisotropy of undrained shear strength of soft clay (Bjerrum 1972)

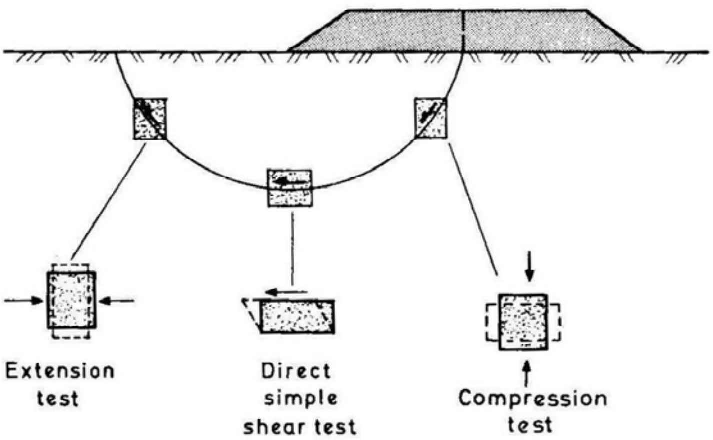


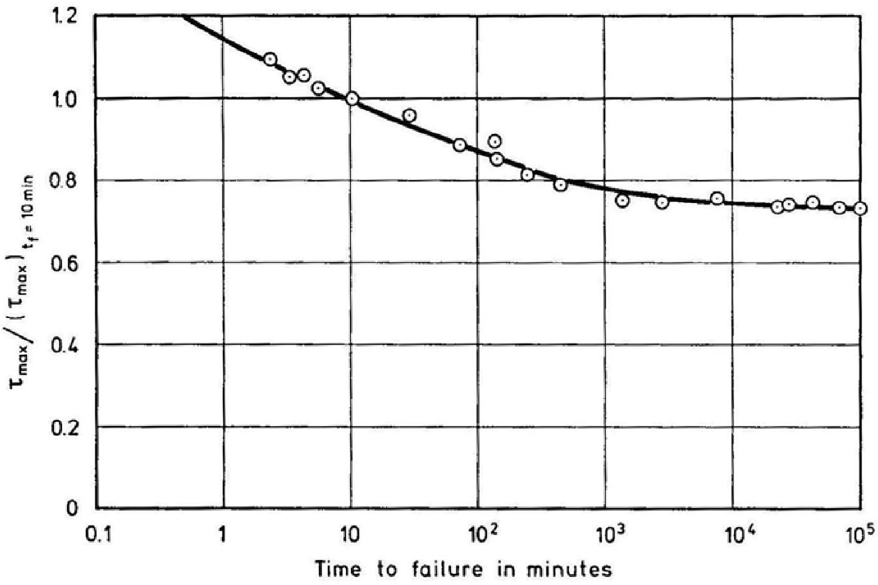
Table I. Undrained shear strength of Manglerud quick clay observed in different types of tests (Bjerrum and Kenney, 1968).

Test type		Average value of s_u/p
Vane tests in situ	Vertical	0.12
	45°	0.14
	Horizontal	0.18
Large in-situ shear box	Horizontal	0.24
	45° – down	0.30
	45° – up	0.08
Triaxial tests	Compression	0.29
	Extension	0.13
Simple direct-shear tests		0.18

Table II. Comparison between the results of compression and extension tests, direct simple shear tests and in-situ vane tests on soft clay (Bjerrum, 1972).

Type of soil	Index properties %				Triaxial tests τ/p_o		Simple shear test τ_h/p_o	Vane tests s_u/p_o	
	w	w_L	w_P	I_P	Compres.	Extension		Observed	Corrected for rate
Bangkok clay	140	150	65	85	0.70	0.40	0.41	0.59	0.47
Matagami clay	90	85	38	47	0.61	0.45	0.39	0.46	0.40
Drammen plastic clay	52	61	32	29	0.40	0.15	0.30	0.36	0.30
Vaterland clay	35	42	26	16	0.32	0.09	0.26	0.22	0.20
Studenterlunden	31	43	25	18	0.31	0.10	0.19	0.18	0.16
Drammen lean clay	30	33	22	11	0.34	0.09	0.22	0.24	0.21

Effect of rate of testing on undrained shear strength of soft clay (Bjerrum 1973)



Correlation between shear stress level and time to failure established from undrained triaxial compression tests on samples of plastic clay from Drammen (Berre and Bjerrum, 1973).

Uncertainties in what the vane measures

The in-situ vane strength is not identical to that measured in triaxial tests or that mobilised at failure in the field, for several reasons including:

Anisotropy of undrained shear strength

Rate of shear effects

In addition we cannot be certain that the shear stresses around the 'cylinder' are uniform at peak Torque.

Conclusions about the in-situ vane test

- A vane test may be carried out at the base of a borehole or as a penetration test.
- Developed in Sweden, it was the first test used to measure the in-situ undrained strength of soft clays, and is still widely used around the world.
- The ratio of the peak strength to the remoulded strength is a measure of sensitivity.
- The ratio of vane strength to effective overburden pressure c_u/σ'_v has been correlated with Plasticity Index and OCR.
- Corrections are often applied to take account of anisotropy and rate effects to arrive at a design strength profile.