Monday-2

In-situ Testing

Soil Characterization

Cone Penetration Test

(CPT & CPTu)

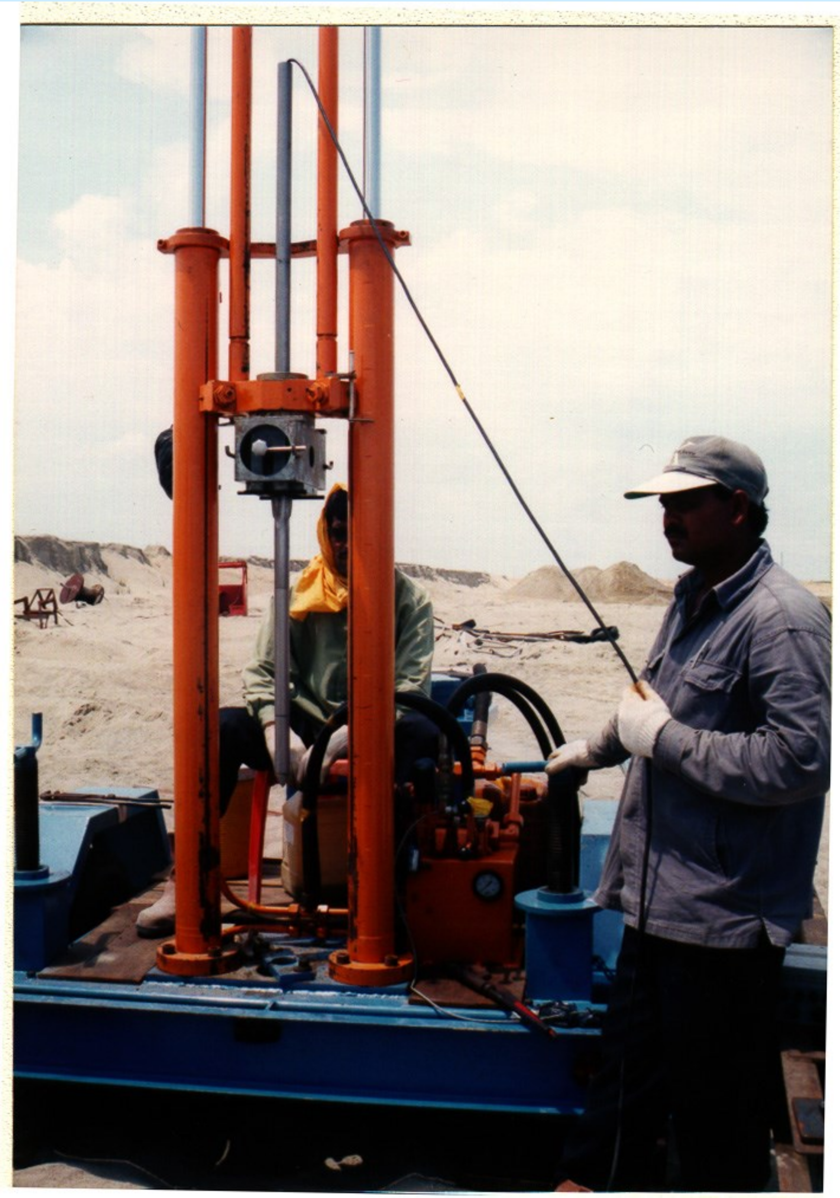


Fig. 1

Fig. 2

Cone Penetration Test

Table 1 Parameters available from available *in situ* tests according to ground conditions

Test type	Parameters required							
	K_0	φ'	c_u	σ_c	E'/G	E_u	G_{max}	k
SPT		G	C	R	G	C	G	
CPT		G	C		G			
Marchetti dilatometer	G,C				G			
Borehole pressuremeter			C		G,R	C		
Plate loading test			C		G,R	C		
Field vane			C				G,C,R	
Seismic field geophysics								
Se1fboring pressuremeter	G,C	G	C		G,C			
Falling/rising head test								G
Constant head test								C
Packer test								R

G = granular, C = cohesive, R = rock.

(Clayton *et al*, 1995)

TERMINOLOGY FOR CPTU AND WHAT WE MEASURE

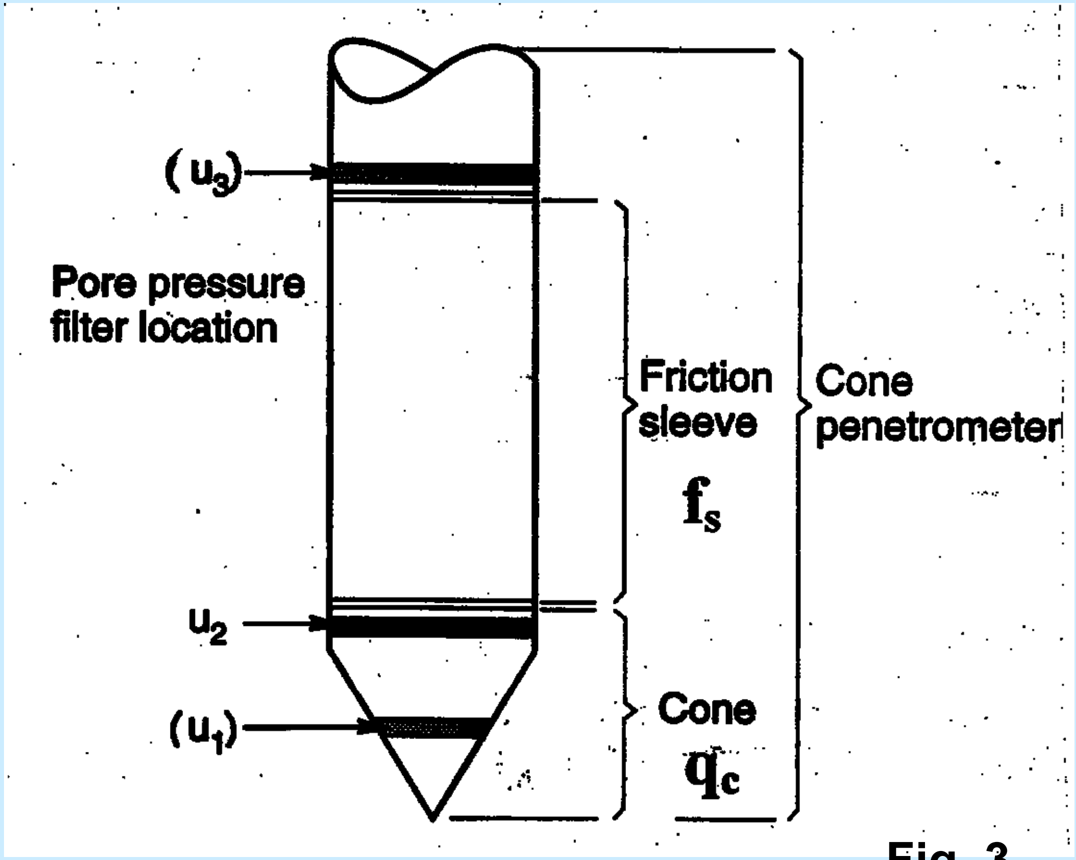


Fig. 3

In addition frequently measure inclination, i

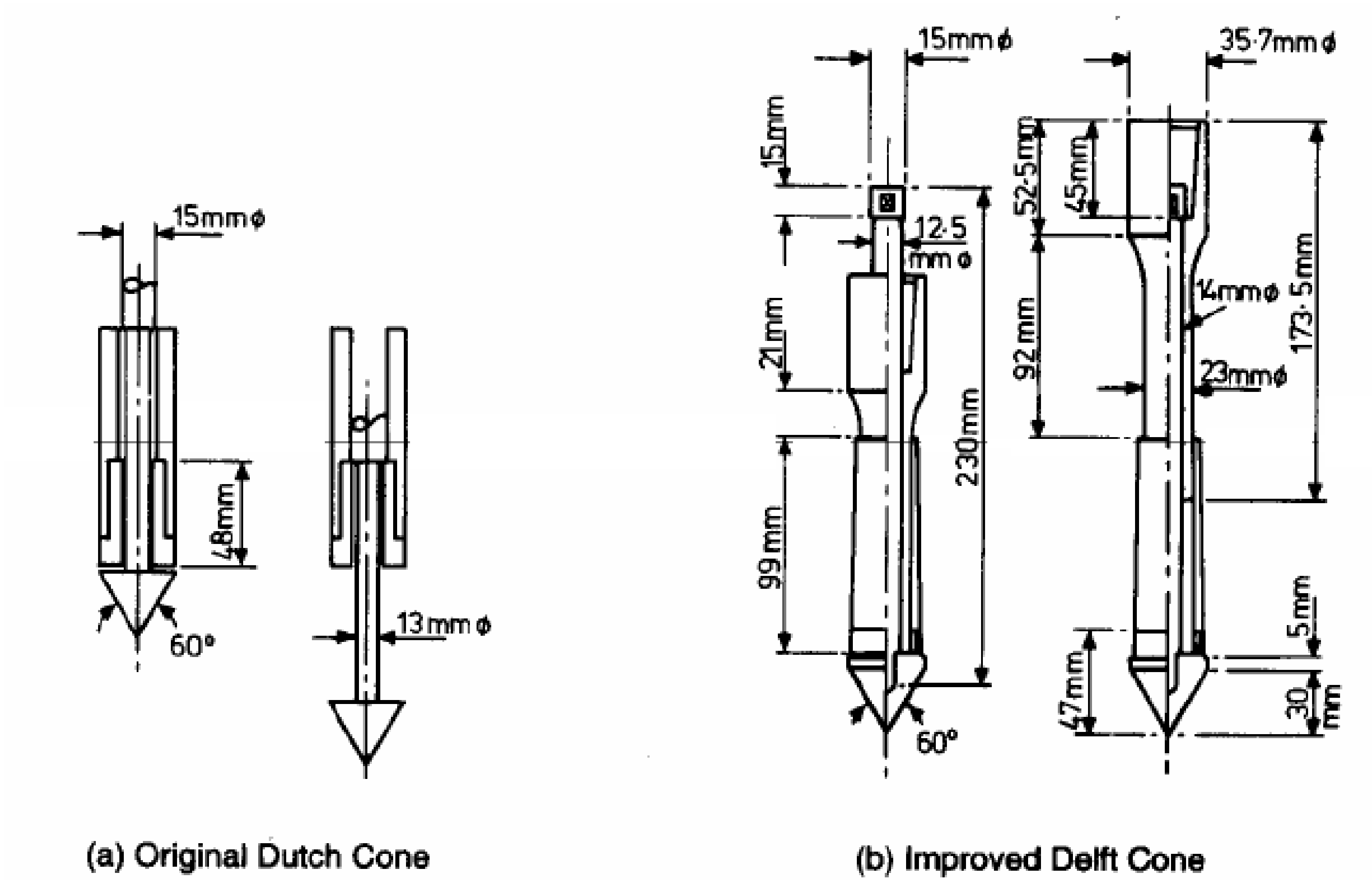


Fig. 4 Original dutch cone and improved mechanical Delft cone (Lousberg *et al.* 1974).

CPT rigs

Fig. 6
Geomil rig

Fig. 7

Geotech simple rig

Cone Penetration Test

- A standard cone penetrometer usually consists of a 60° cone, with a base area of 10 cm^2. During the test, the cone is pushed into the soil at a steady rate of typically 2 cm/s using hydraulic pressure.

- Typically, the cone point resistance q_c, and the unit shaft friction f_s are measured either mechanically or electrically.

Measured CPTU parameters

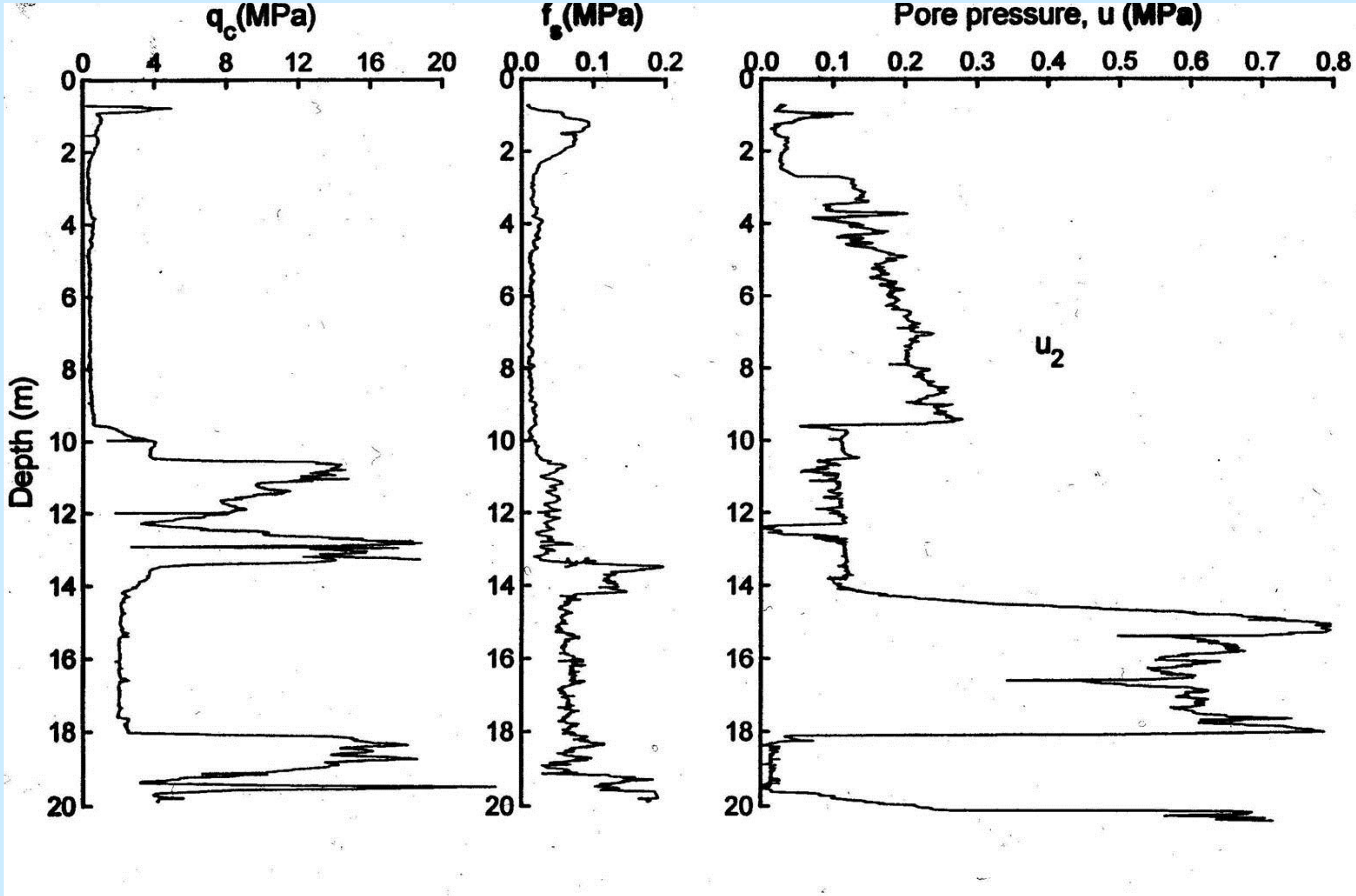


Fig. 8

CPTU profile from Holland

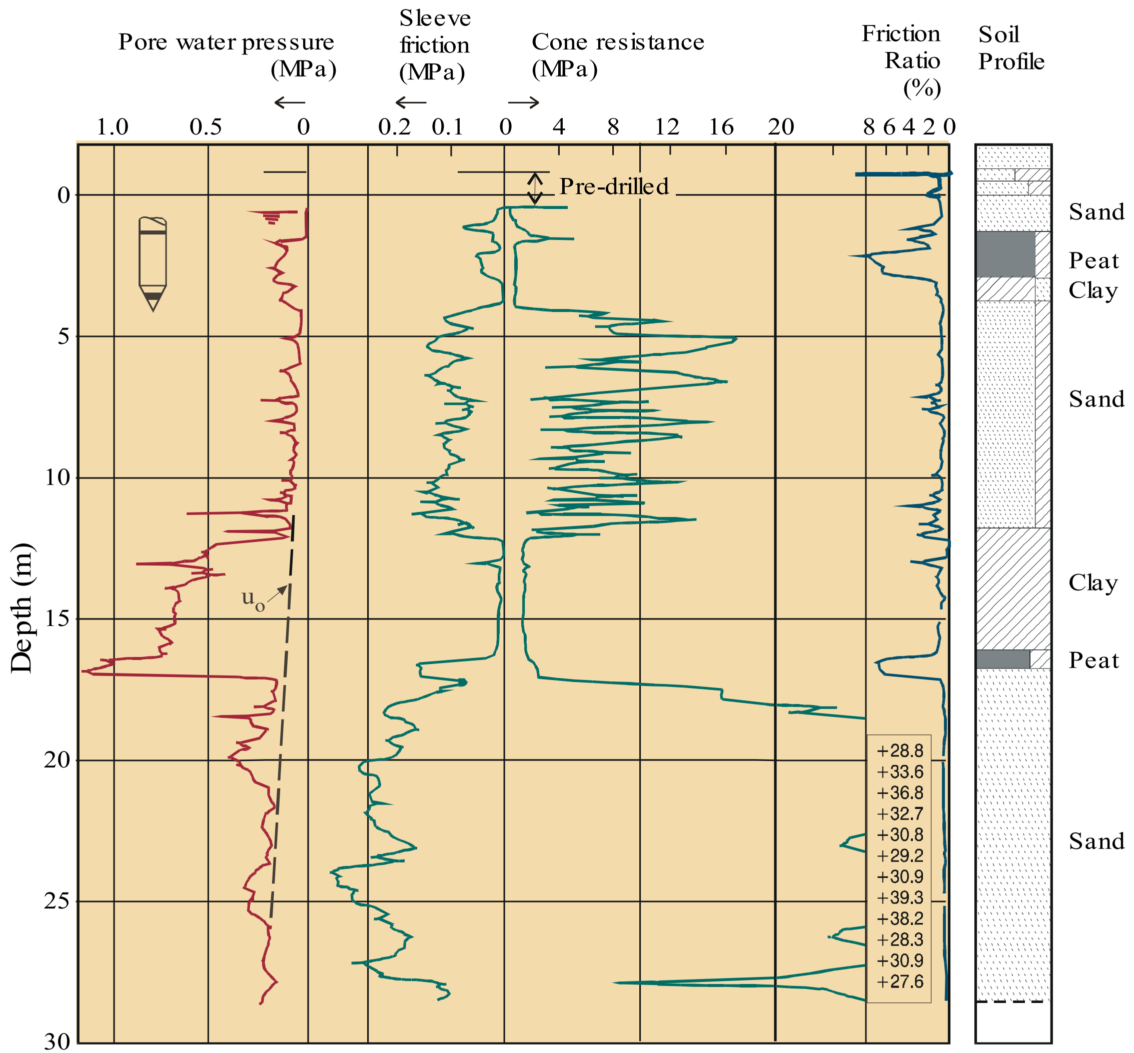


Fig. 9

CPTU profile in sand

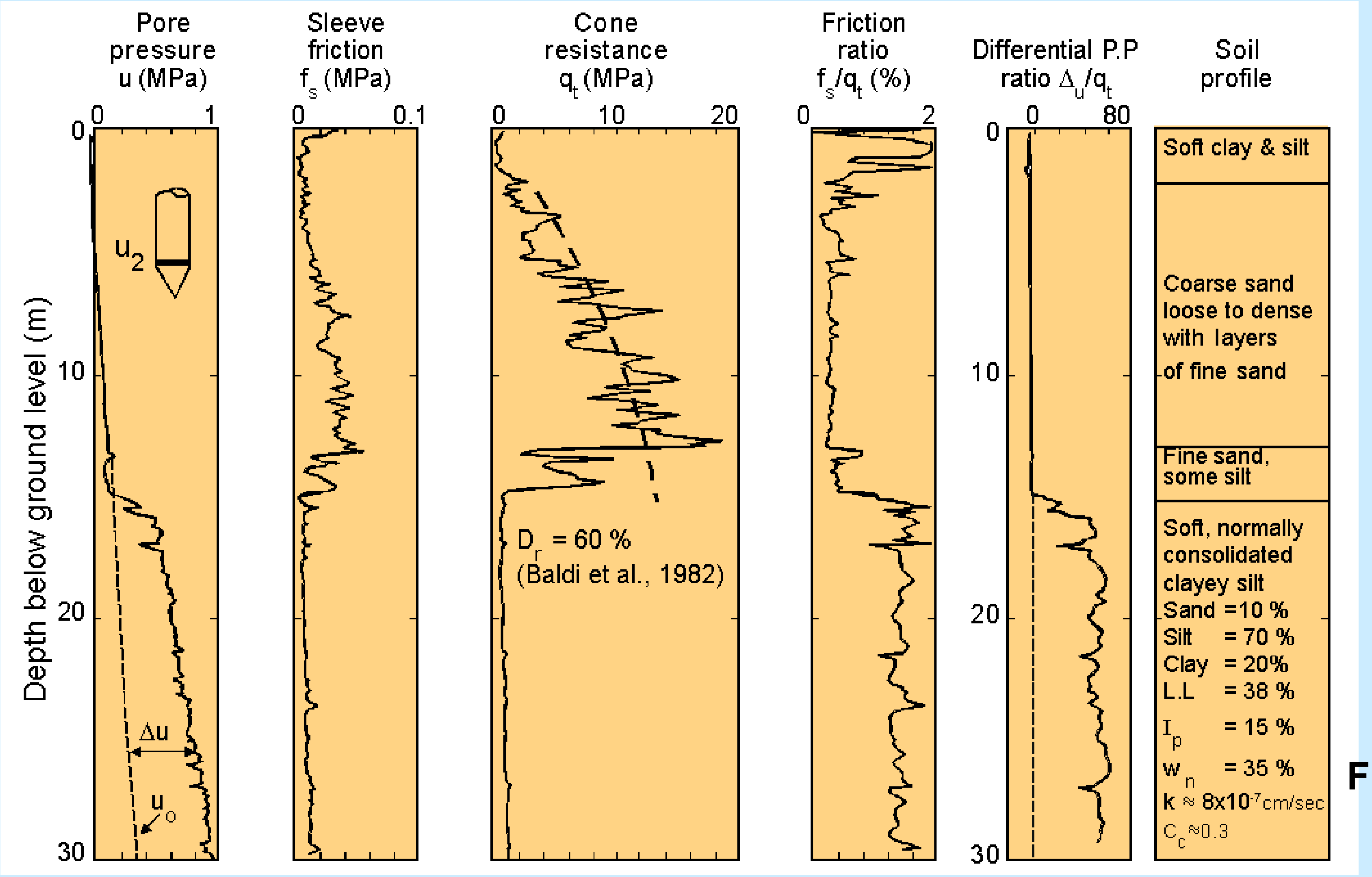
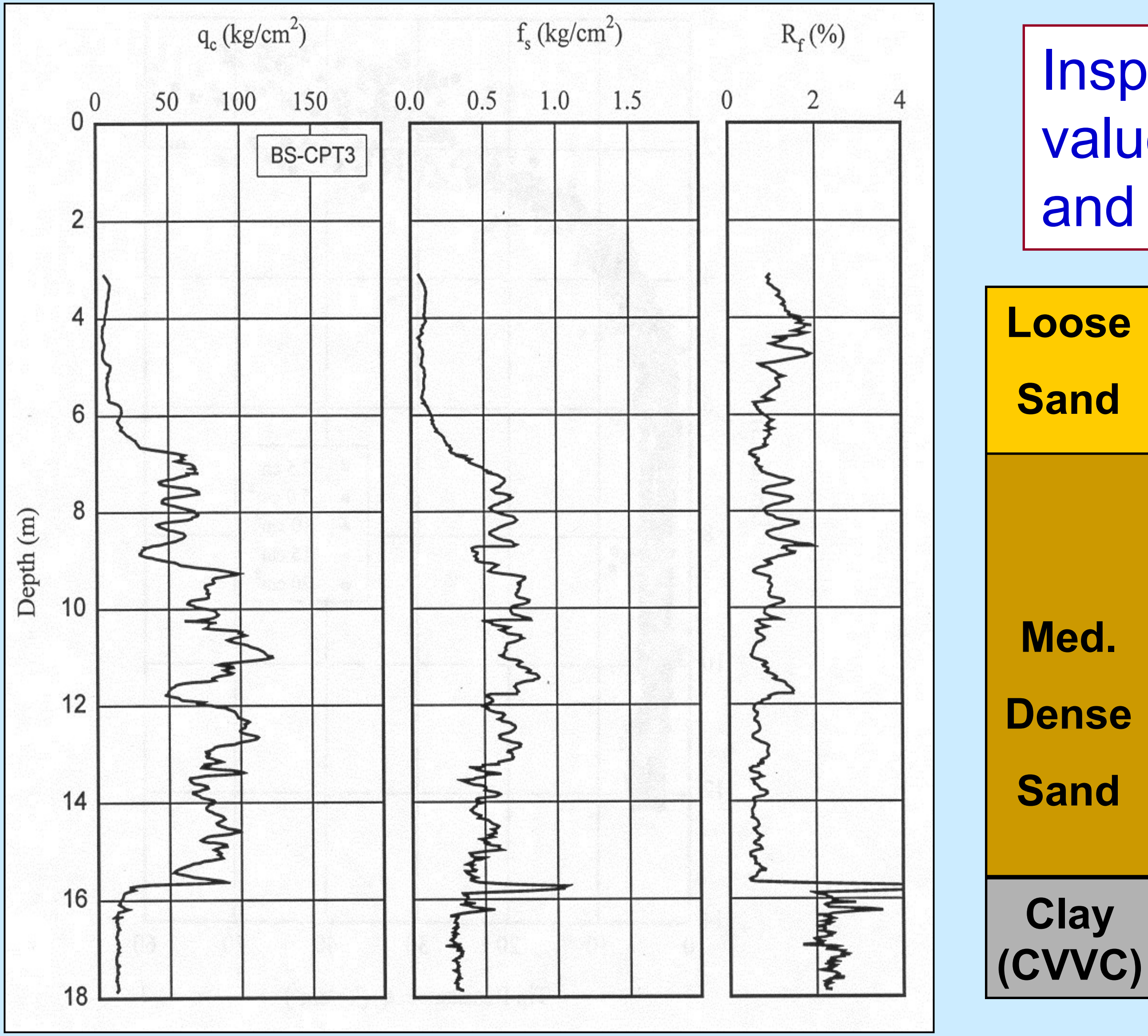


Fig. 10

McDonald's Farm, Vancouver, Canada

Example CPT in Western Massachusetts

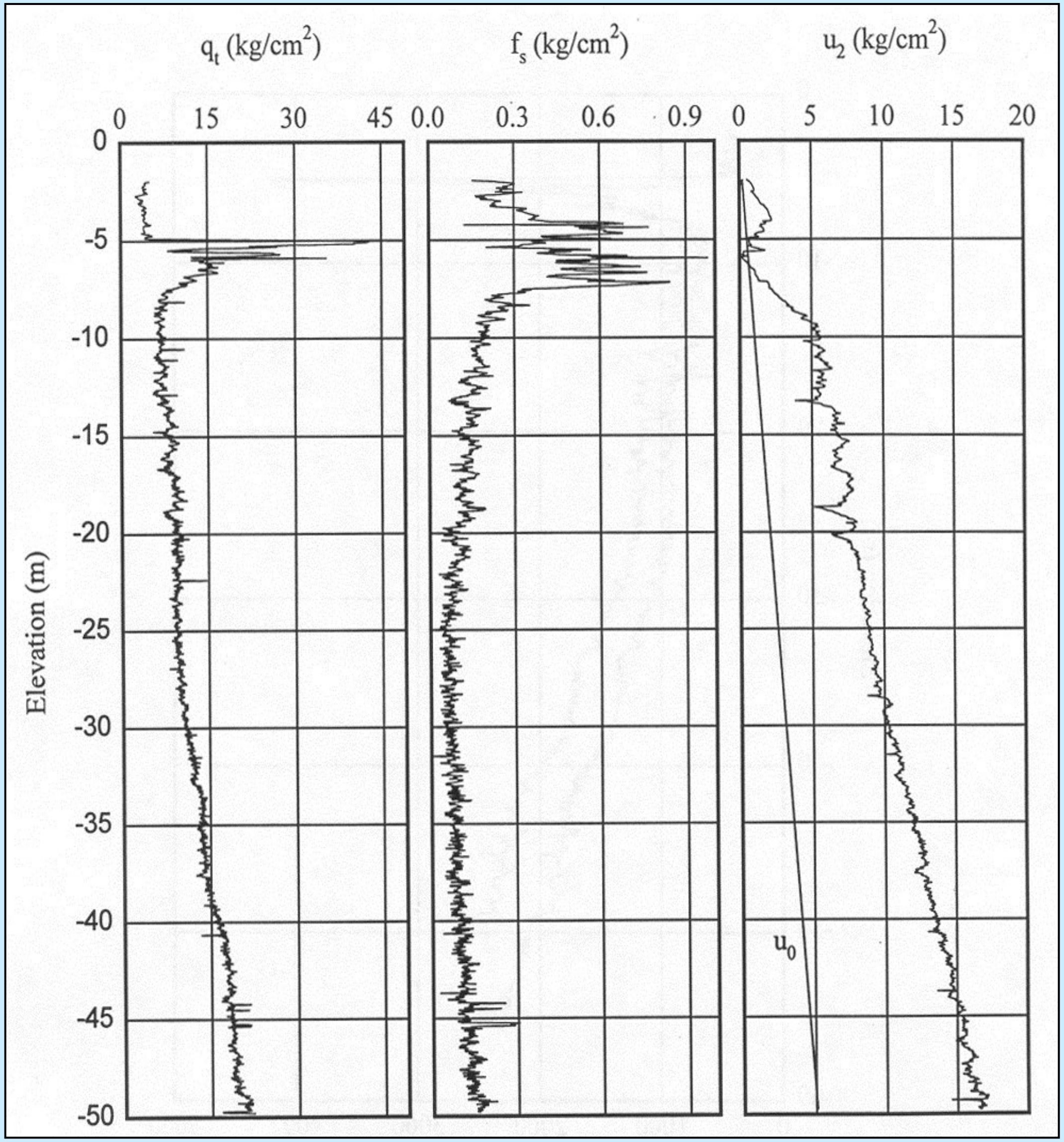


Inspect relative values of q_c, f_s and R_f

UNITS:
1 ksc
≈ 100 kPa
≈ 0.1 MPa
≈ 2000 psf
≈ 1 tsf

Fig. 11

Example CPTU in Eastern Massachusetts



Boston Blue Clay

Stiff Clay Crust

Uniform Soft Clay

SPT N = WOR (i.e., = 0)

Linear increase in q_t and u_2 with depth

High u_2 relative to u_0

Fig. 12

Derived CPTU parameters

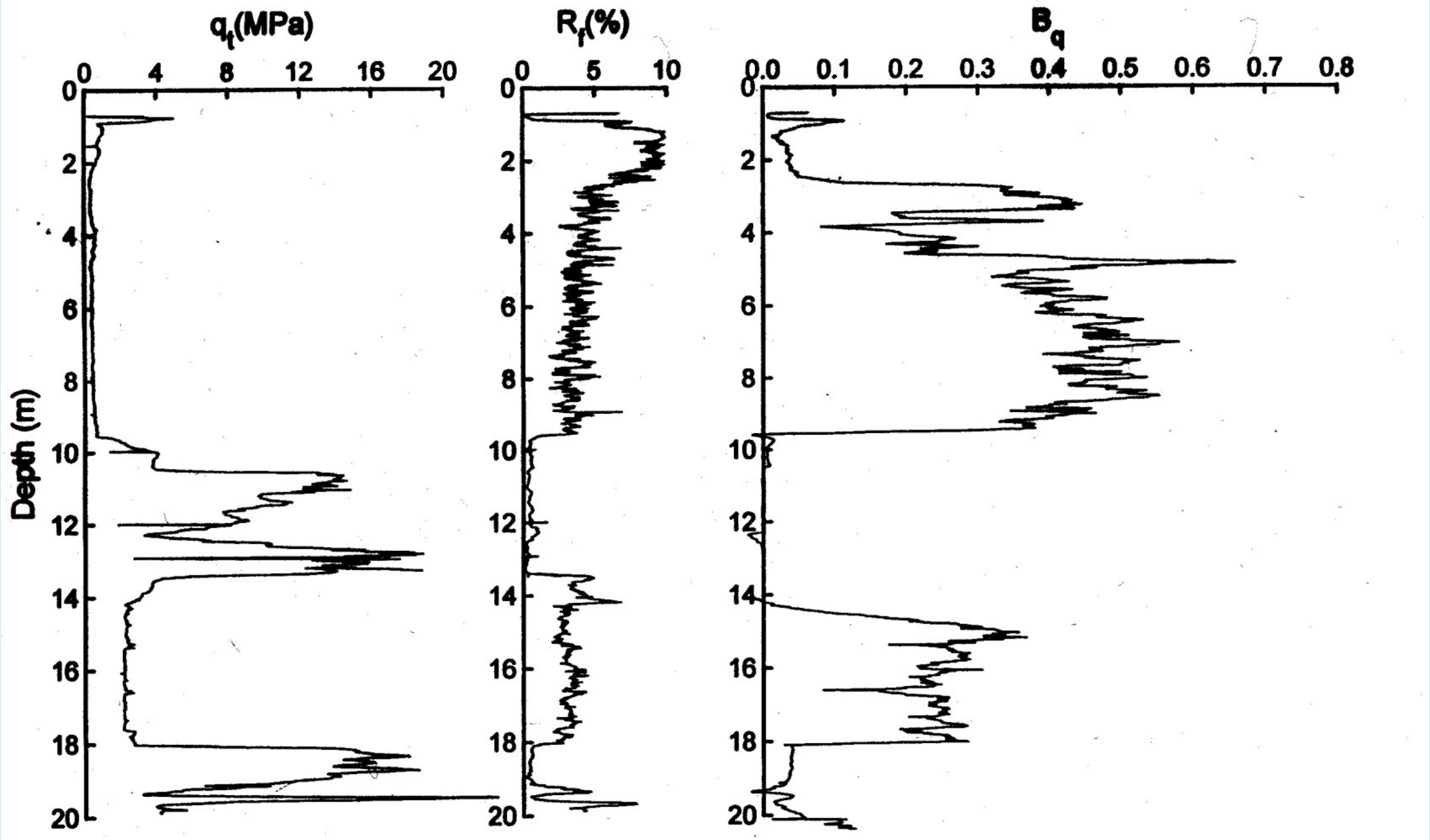


Fig. 14 Example: Derived CPTU Parameters

Measured Data and Calculated Variables

1. Measured Data

most common = q_c, f_s, and u_2

2. Calculated Variables

(for u_2 measurement):

Corrected tip resistance: $q_t = q_c + u_2(1\text{-}a)$

Excess pore pressure $Du = u_2 - u_0$

Friction Ratio: $R_f = f_s/q_c$

Normalized net tip resistance: $Q_c = (q_c - s_{vo})/s'_{vo}$

Normalized sleeve resistance: $F_r = f_s/(q_c - s_{vo})$

Pore Pressure Parameter: $B_q = (u_2 - u_o)/(q_t - s_{vo})$

Normalized Excess Pore Pressure: $U = (u_2 - u_o)/s'_{vo}$

Normalized Corrected Tip Resistance: $Q_t = (q_t - s_{vo})/s'_{vo}$

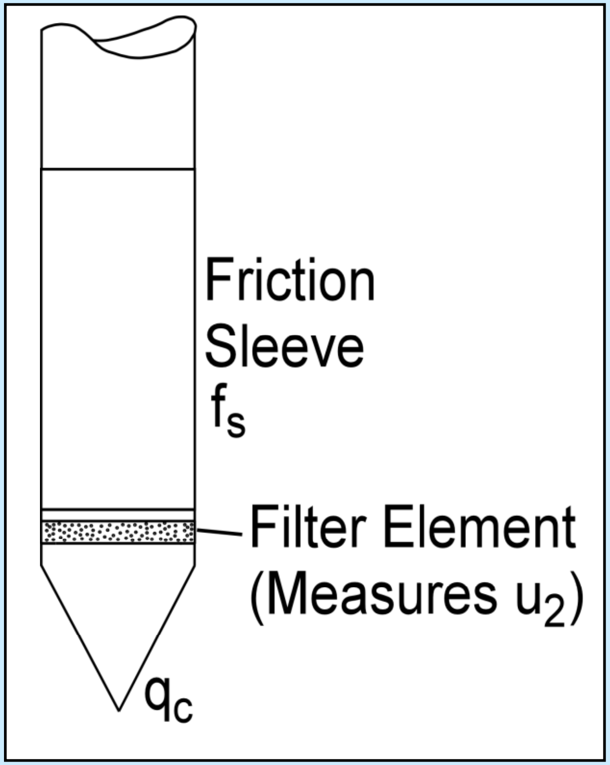


Fig. 15

Interpretation of Shear Strength from CPT

- **The use of q_c for the evaluation of undrained shear strength of clay uses the following:**

$$s_u = \frac{q_c - \sigma_{vo}}{N_k}$$

where N_k is an empirical cone bearing factor typically between 10 and 15 for normally consolidated clay and 15 and 20 for overconsolidated clay.

- **The estimate of s_u based on q_c is very crude particularly for electric cone where the hydraulic pressure that often exerted behind a cone tip could not be accounted for.**

Stratigraphic Profiling

Key Signatures to look for in measured data, e.g.:

1. Shape and magnitude of q_t profile – e.g., high in dense sand, low in soft clay

2. Shape of u profile and magnitude, especially relative to equilibrium pore pressure profile – e.g., high in soft clay, Du = 0 in medium density sand

3. Magnitude of R_f relative to that of q_t – e.g., if high and coupled with low q_t = soft clay.

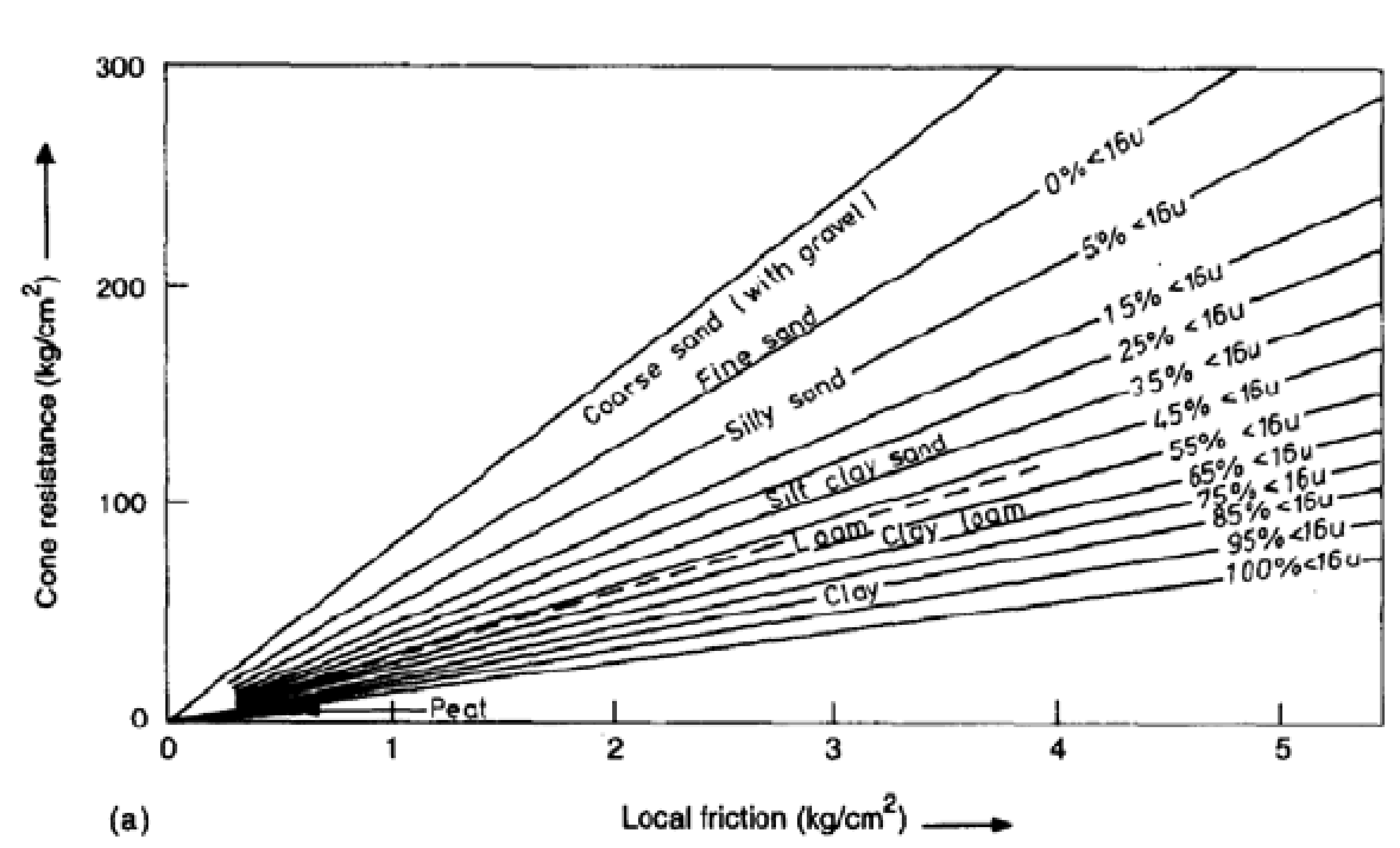
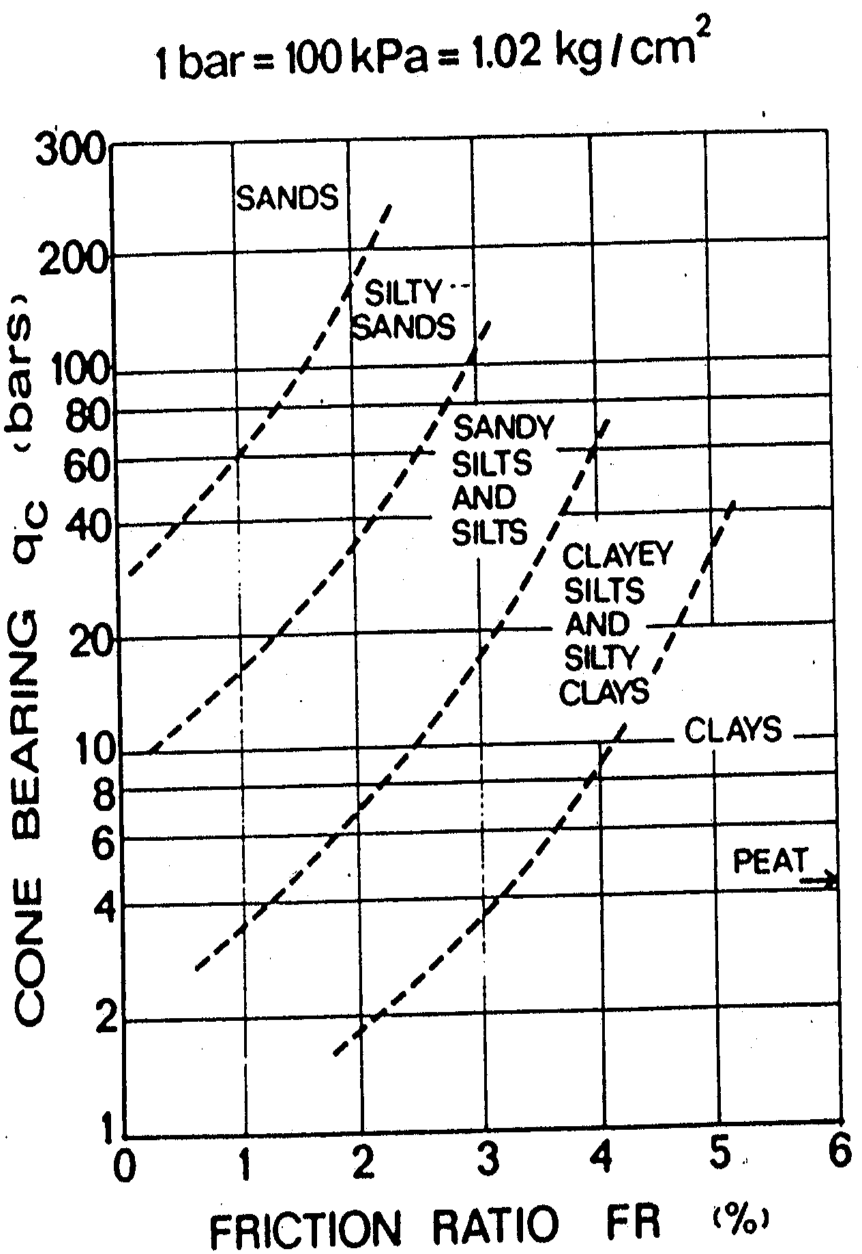


Fig. 16 (a) relationship between soil type, cone resistance and local friction (Begemann 1956)



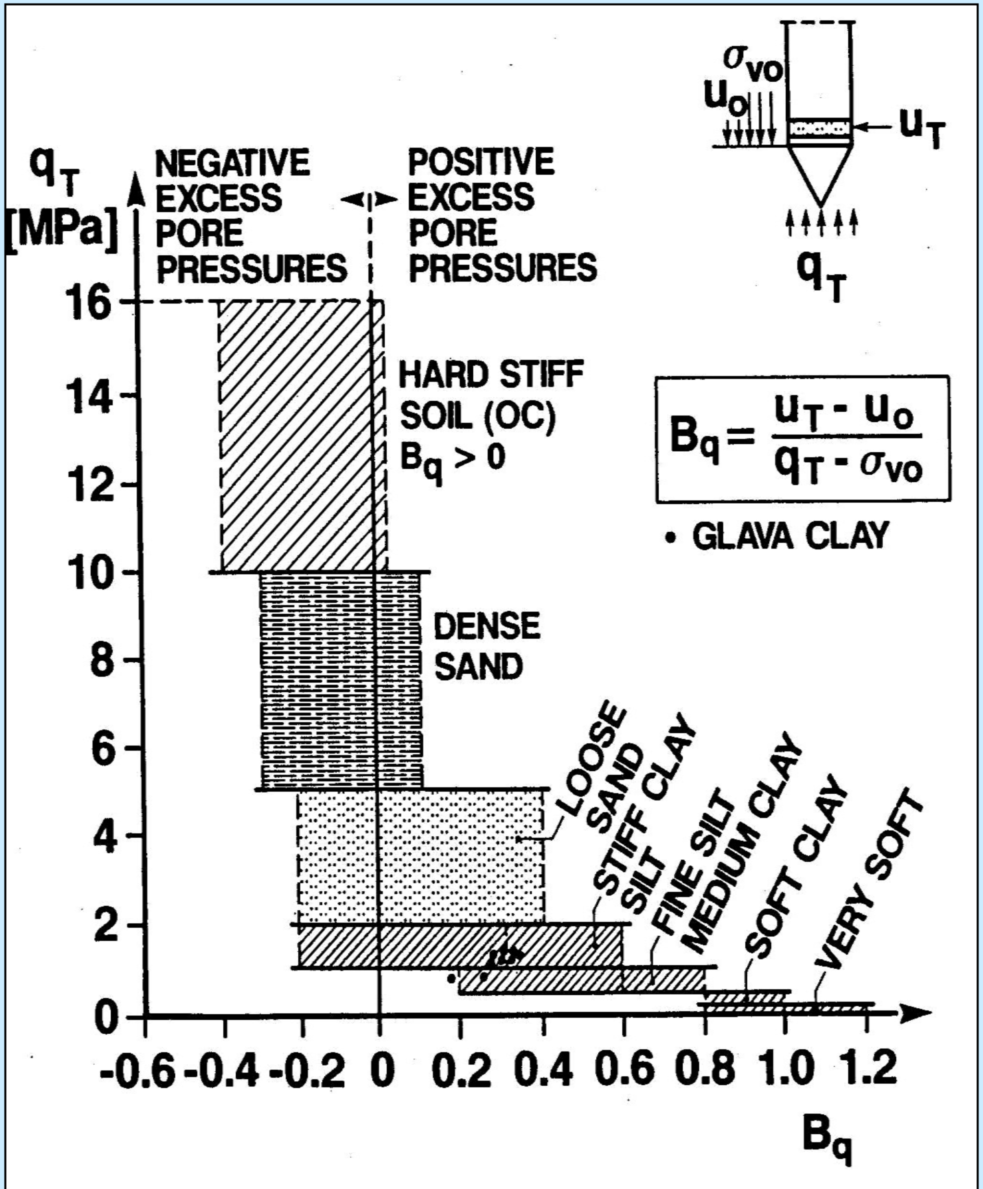
Soil Classification based on CPT (2)

Chart for Soil Classification

After Robertson & Campanella (1983)

Fig. 17

Pore Pressure (via B_q) for soil Classification

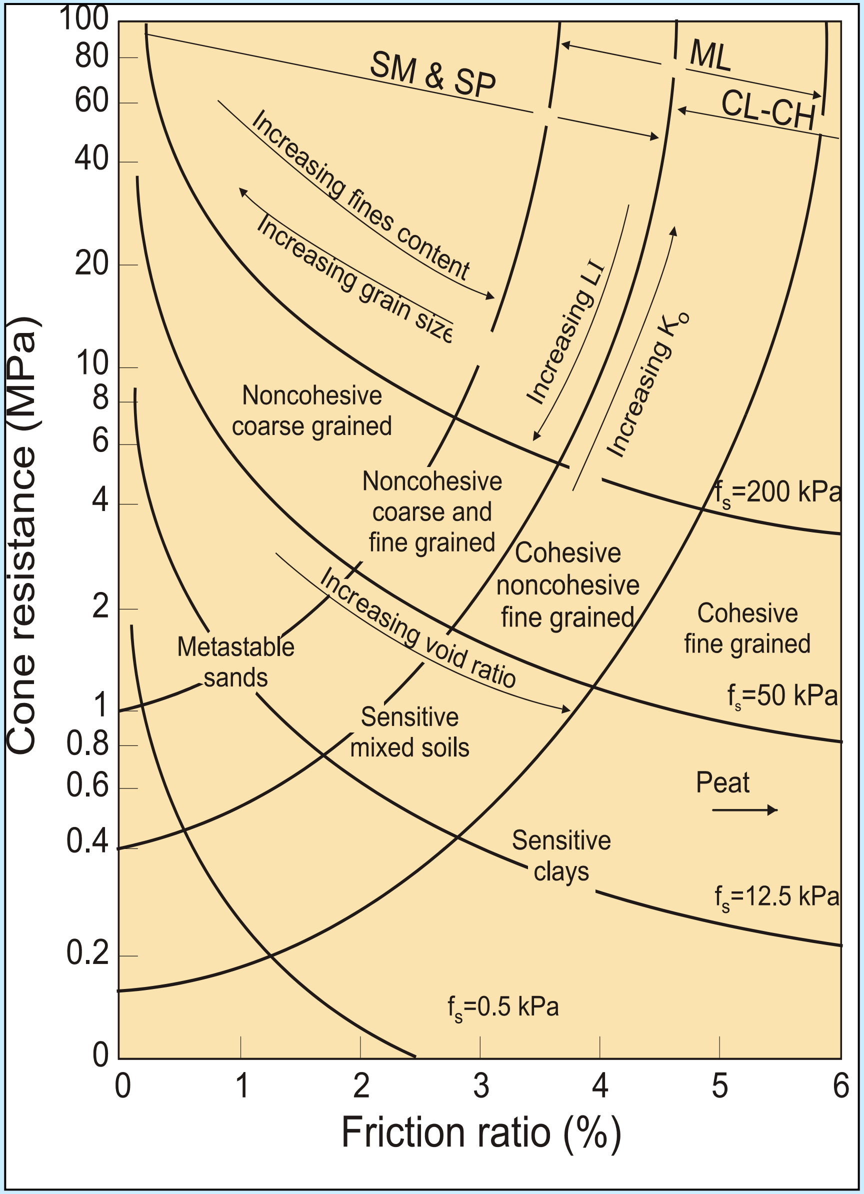


Note: measured u is function of location – chart is for u_2 position. Hence, negative pore pressures can occur.

[Janbu and Senneset,1984]

Fig. 18

CPT Soil Classification/Behavior Chart

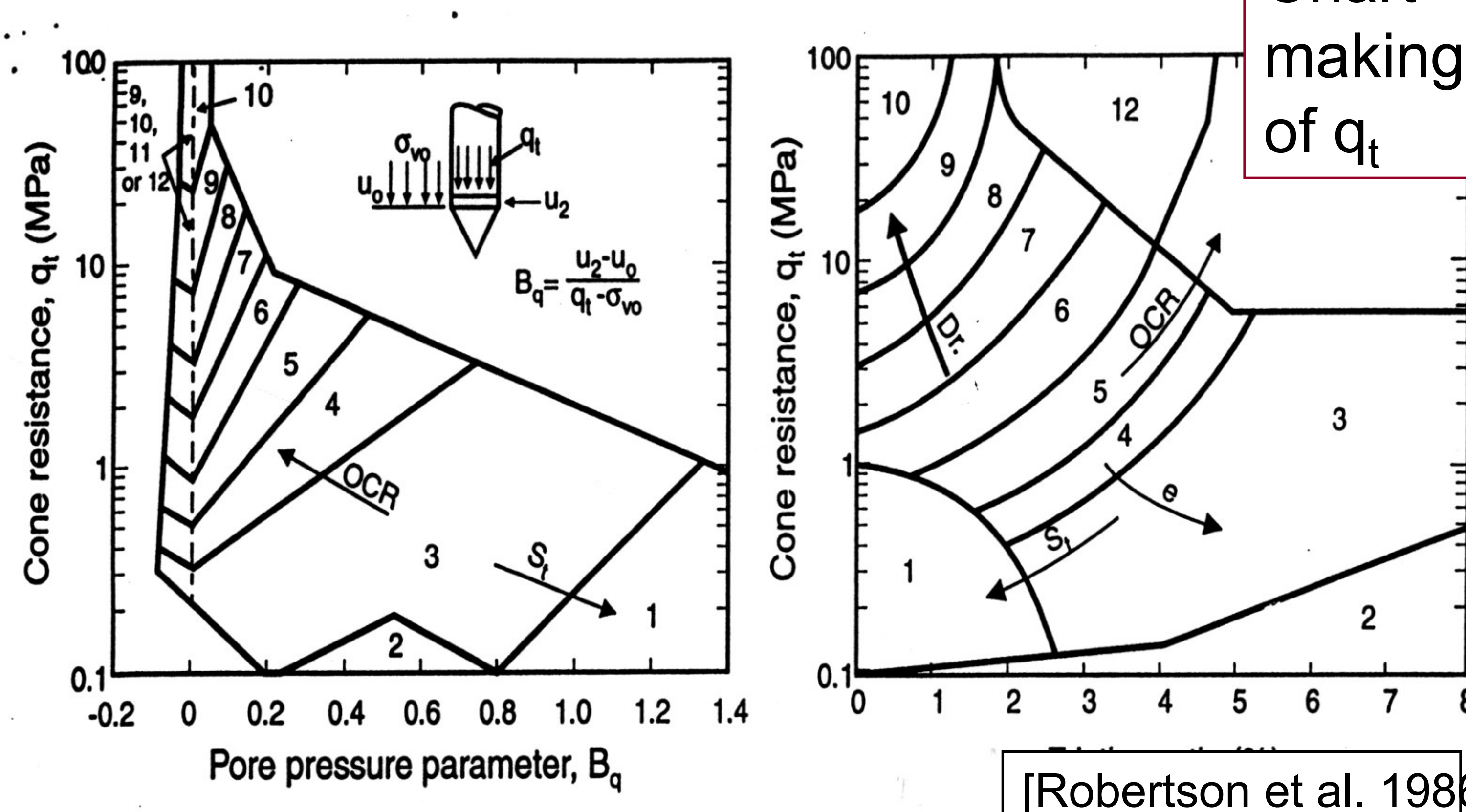


Based on q_c and f_s from CPT

[Fig. 19
Douglas and Olsen 1981]

Soil Behavior Type Classification Chart

Chart making use of q_t



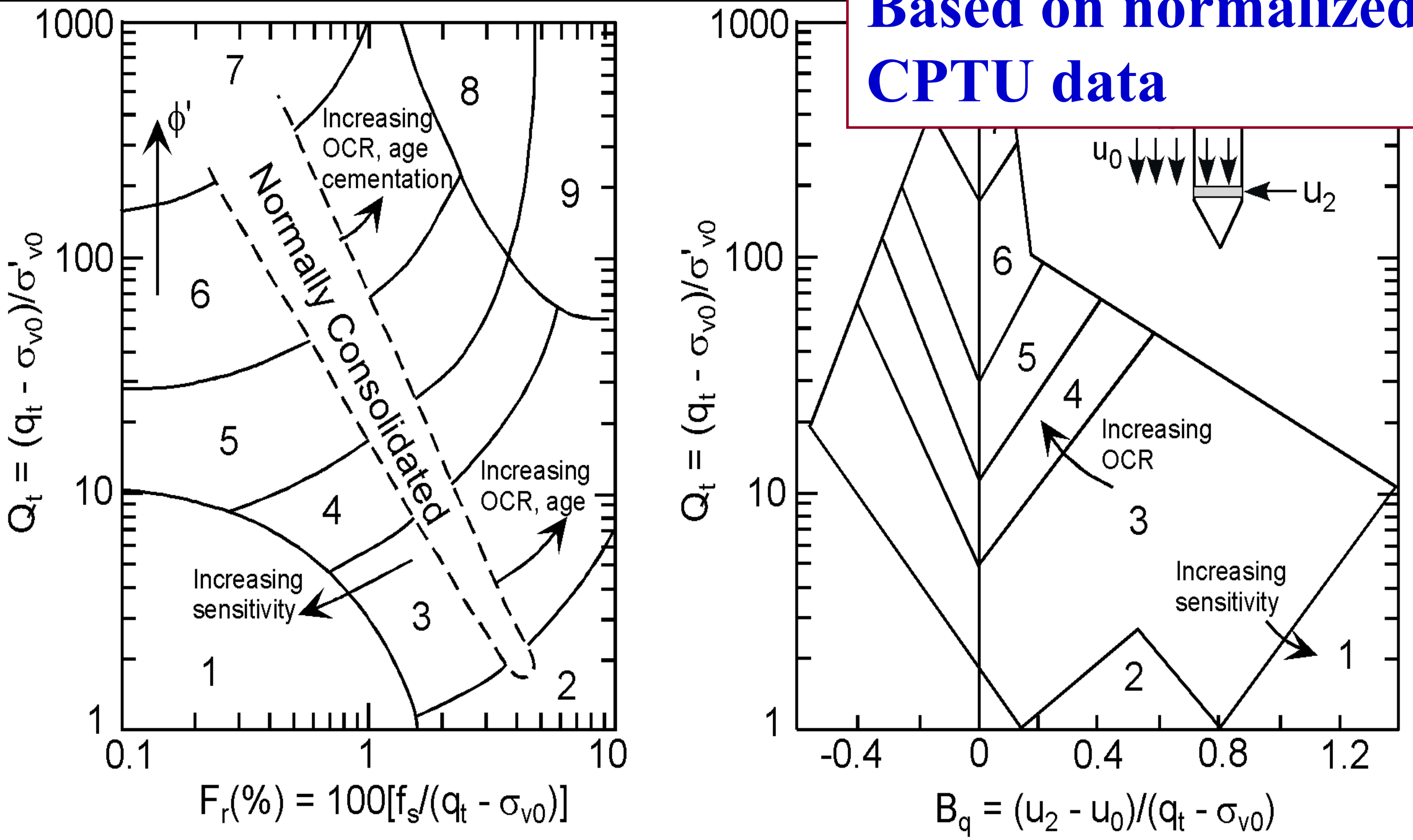
[Robertson et al. 1986]

Zone:	Soil Behaviour Type:
1.	Sensitive fine grained
2.	Organic material
3.	Clay
4.	Silty clay to clay
5.	Clayey silt to silty clay
6.	Sandy silt to clayey silt
7.	Silty sand to sandy silt
8.	Sand to silty sand
9.	Sand
10.	Gravelly sand to sand
11.	Very stiff fine grained*
12.	Sand to clayey sand*

Fig. 20

Soil Behavior : Classification Chart

Based on normalized CPTU data



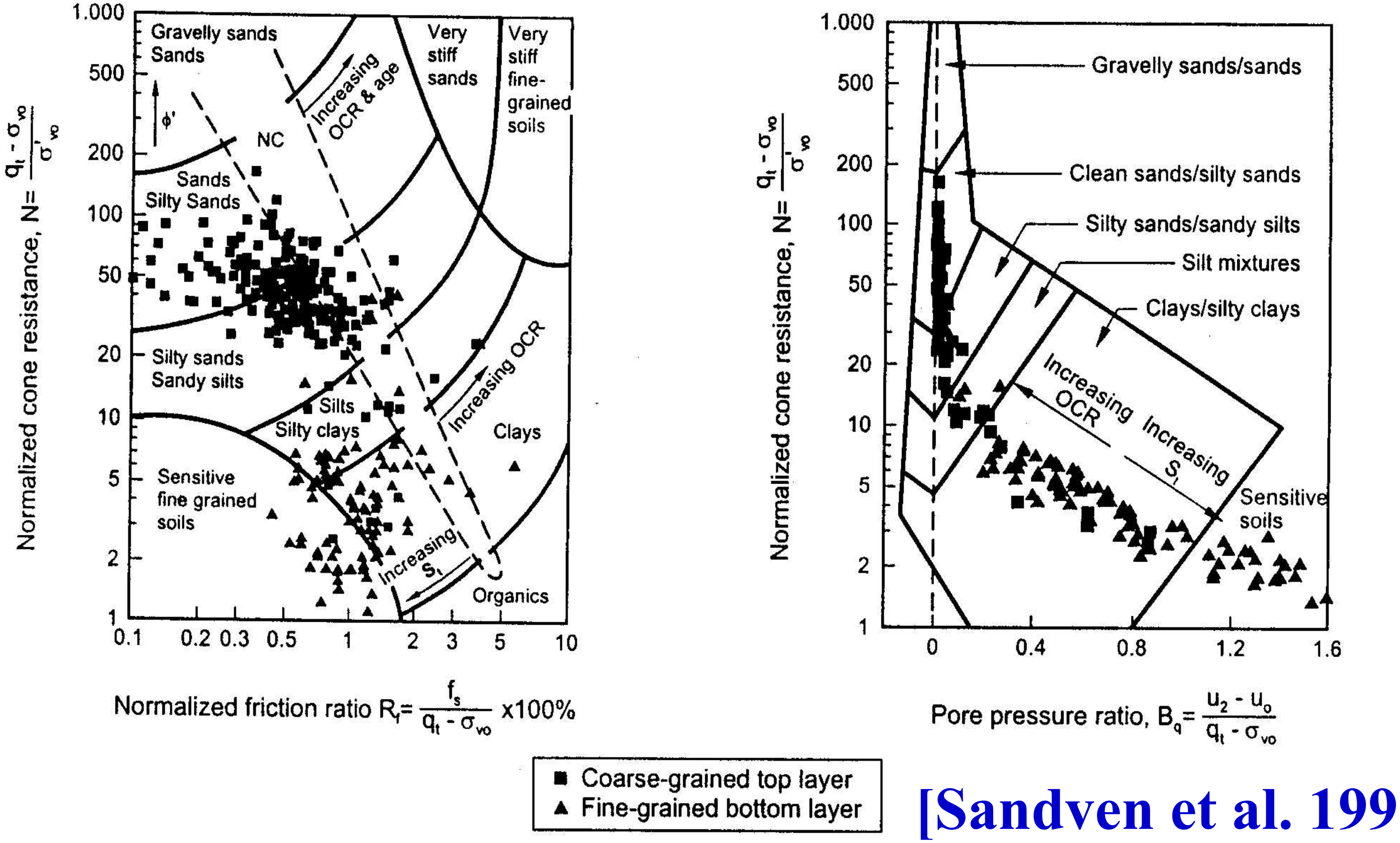
[Robertson 1990]

Soil Behavior Type by Zone Number

1. Sensitive, fine grained
2. Organic soils-peats
3. Clays-clay to silty clay
4. Silt mixtures clayey silt to silty clay
5. Sand mixtures; silty sand to sand silty
6. Sands; clean sands to silty sands
7. Gravelly sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

Fig. 21

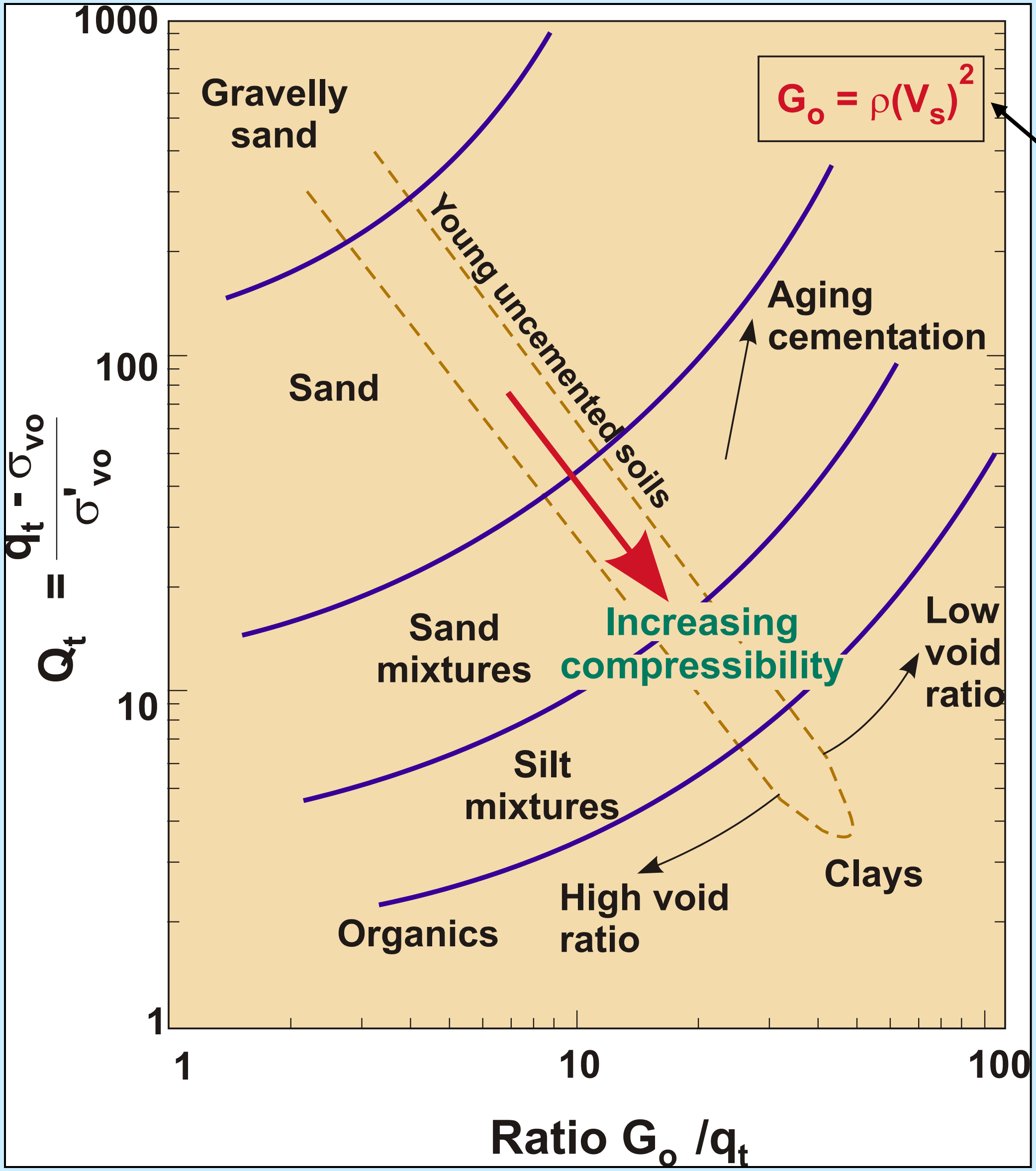
CPTU Soil Classification – Oslo Airport



[Sandven et al. 1998]

Fig. 22

Soil Classification/Behavior Chart using G_{max}



- $G_0 = G_{max}$
- V_s direct measure from seismic CPTU
- r_t must be estimated

[Robertson et al. 1995]

Fig. 23

CPT/SPT CORRELATIONS

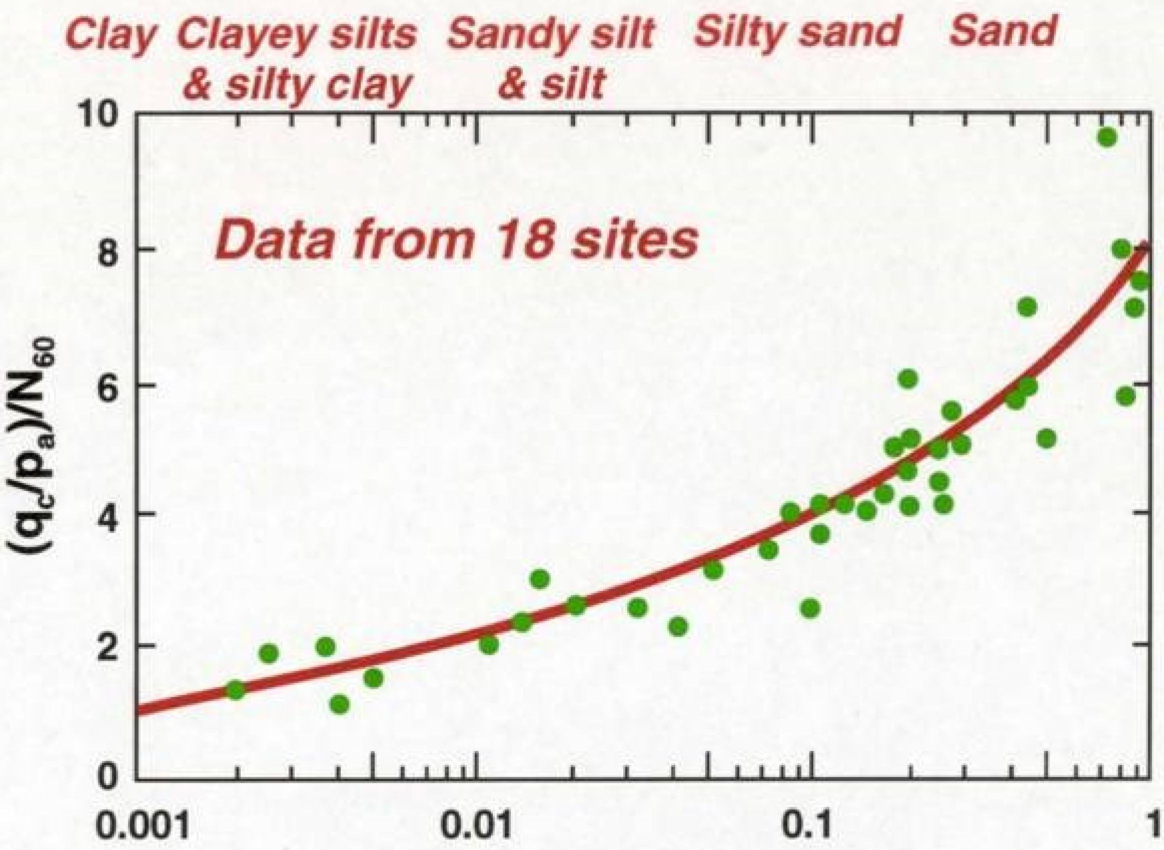


Fig. 24 Mean particle size, D_{50}(mm)

Robertson and Campanella (1983)

Pa = reference stress = 1 atm = 100 kPa

CPT/SPT CORRELATIONS
Effects of fines content

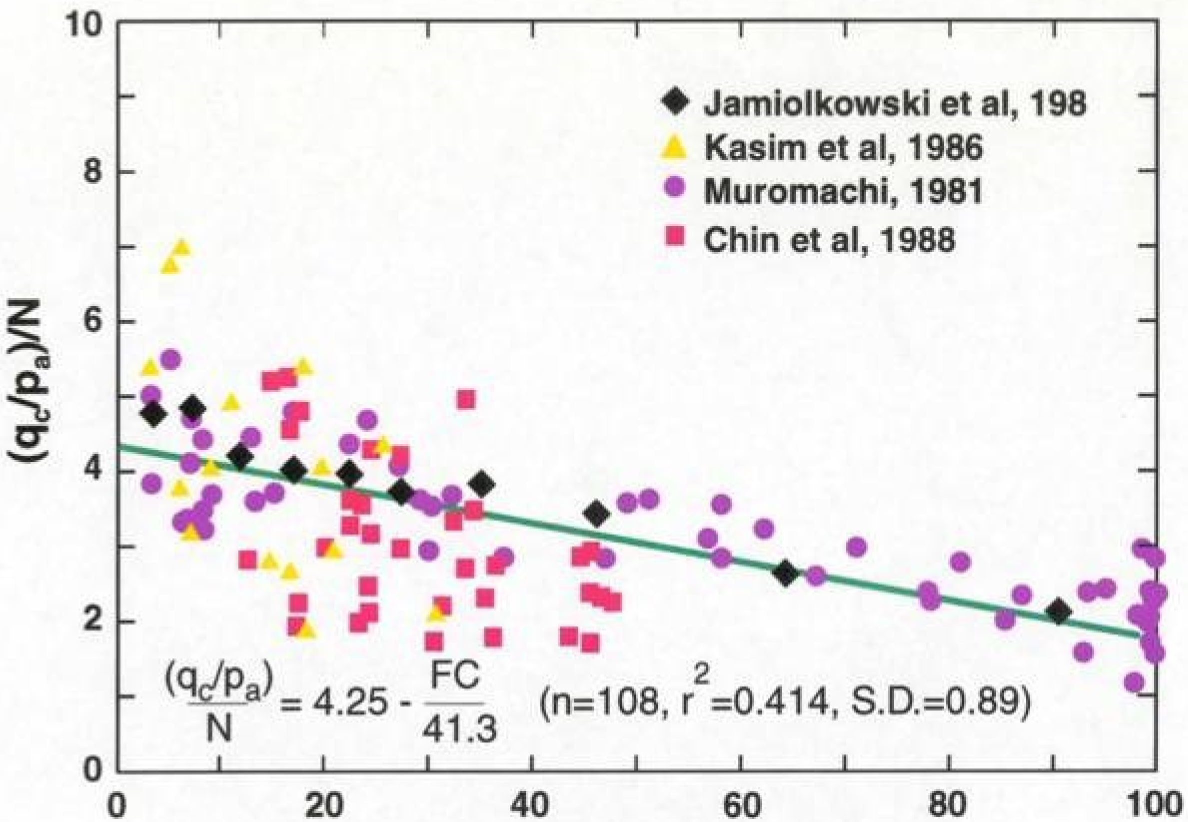


Fig. 25 Fines content, percent passing No. 200 Sieve, FC

Mayne and Kulhawy (1990)

Soil Classification from CPT/CPTU data

Methodology:

1. Quantify observations used to identify soil stratigraphy.

2. Empirically based, i.e., measured CPT/CPTU data are correlated with known soil profiles.

3. Early charts relied on direct use of reduced data, e.g., q_c or q_t and f_s or R_f.

4. Later charts make use of normalized parameters to account for increasing overburden stress with depth, e.g., Q_t, B_q.

Recommendations: CPT/CPTU based Soil Identification/Classification

- Use all information available, e.g., q_c or q_t, f_s, u, F_r, B_q

- Shape and magnitude of q_t profile gives indication on whether you are in uniform clay layer, sand layer, etc.

- Pore pressure profile readily indicates a drained condition (e.g., sand with Du = 0) or undrained (e.g., clay with Du > 0)

- Use q_t - R_f - B_q and/or Q_t-F_r-B_q diagrams to identify soil type. Accumulate local experience to create/modify diagrams.

- Short dissipation tests can help in identifying soil type

- Measurements using other sensors (e.g., V_s) can enhance soil identification

CPTU Derived Soil Engineering Parameters for CLAY

1. Key Aspects of Clay Soil Behavior

2. Important engineering design parameters

3. Background and application of CPTU correlations for estimation of design parameters

4. Applied to Case Studies in follow-on lecture.

Basic Soil Behavior - CLAY

1-D Consolidation

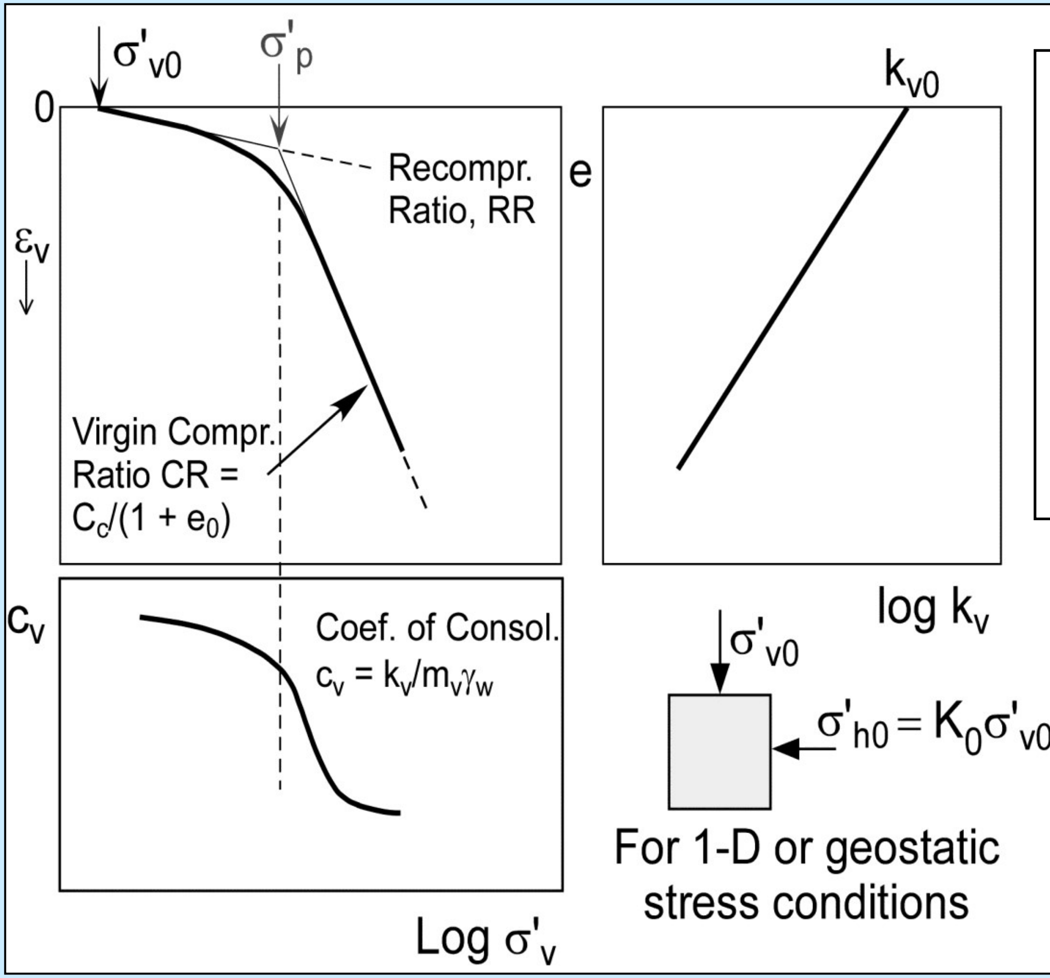


Fig. 26

Key Aspects:

1. Compressibility (RR and CR)
2. Yield stress (s'_p)
3. Coefficient of consolidation (c_v)
4. Hydraulic conductivity (k_v)
5. Horizontal stress (s'_{h0} or K_0)

Most Important Parameter:

Yield stress = $s'_{vy} \equiv s'_p \equiv p'_c$

Also known as:

- **Preconsolidation stress**
- **Maximum past pressure**

Basic Soil Behavior - CLAY

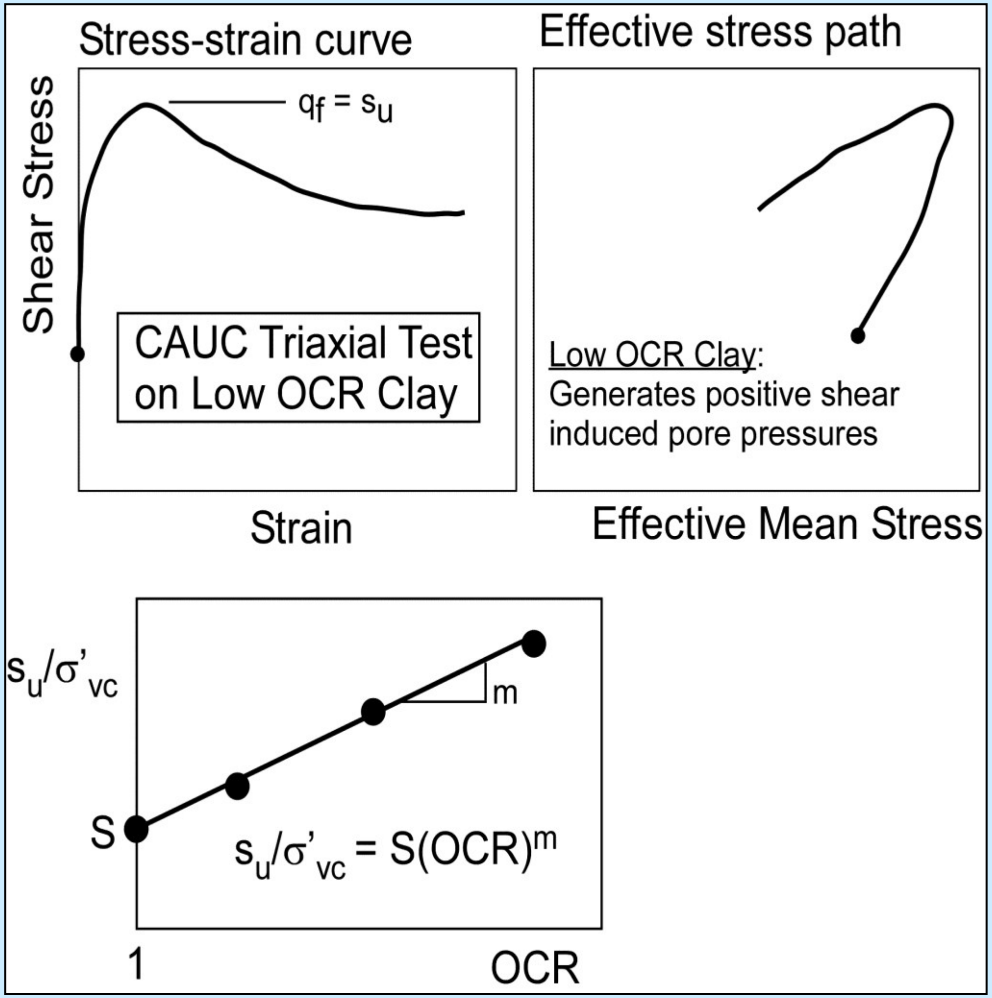


Fig. 27

Undrained Shear Strength

Key Aspects:

1. **Shear induced pore pressures**
2. **Effect of OCR**
3. **Anisotropy**
4. **Rate effects**

Most Important Parameter: Undrained shear strength = s_u

General Aspects of CPTU Testing in Clay

1. **Penetration is generally undrained and therefore excess pore pressures will be generated.**

2. **Cone resistance and sleeve friction (if relevant) should be corrected using the measured pore pressures.**

3. **The measured pore pressures can also be used directly for interpretation in terms of soil design parameters.**

Interpretation of CPTU data in clay

1. State Parameters = In situ state of stress and stress history

2. Strength parameters

3. Deformation characteristics

4. Flow and consolidation characteristics

5. In situ pore pressure

In Situ State Parameters

1. Soil Unit weight: g_w for computation of in situ vertical effective stress (s'_{v0})

2. Stress history
   s'_p and $OCR = s'_p / s'_{v0}$

3. In situ horizontal effective stress
   $s'_{h0} = K_0 s'_{v0}$

Estimation of Soil Unit Weight

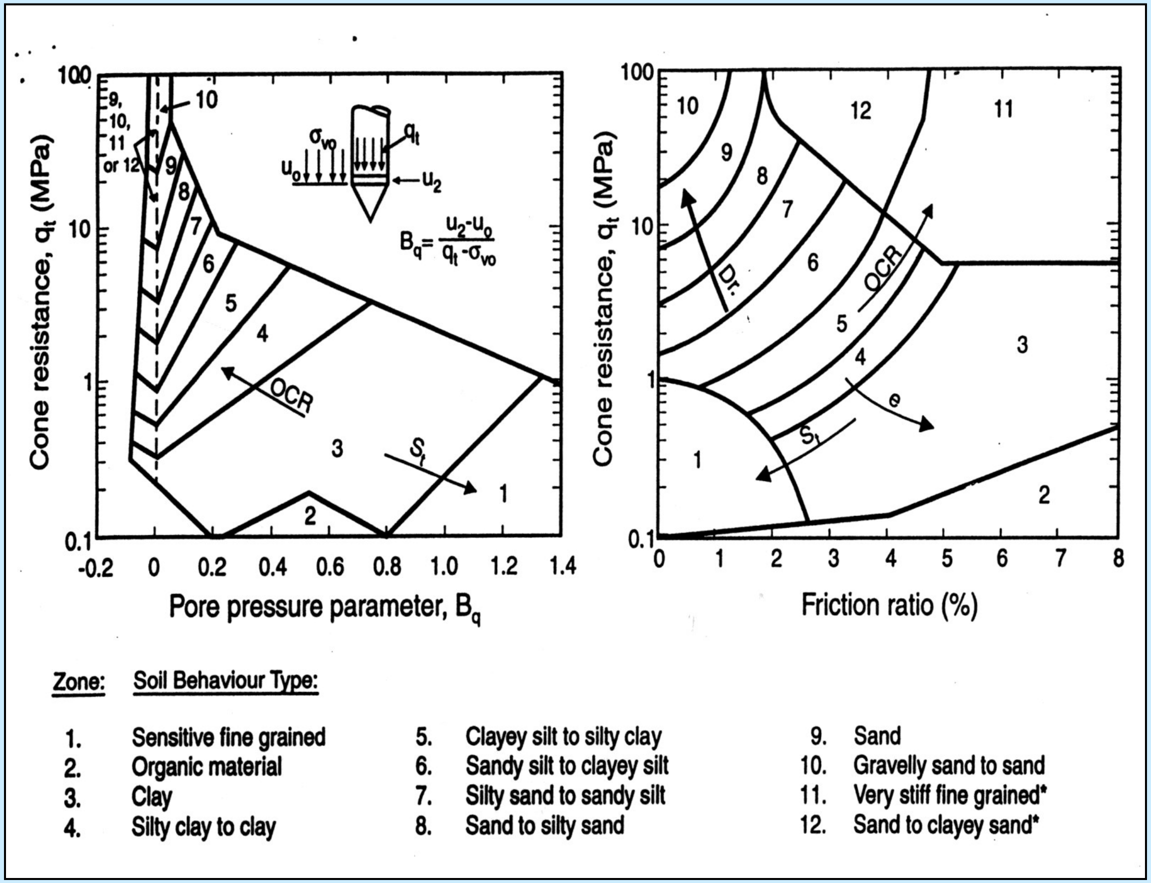


Fig. 28

[Robertson et al. 1986]

Zone	Approximate Unit Weight (kN/m^3)
1	17.5
2	12.5
3	17.5
4	18.0
5	18.0
6	18.0
7	18.5
8	19.0
9	19.5
10	20.0
11	20.5
12	19.0

Note: 1 $kN/m^3 \approx 6.36$ pcf

Stress History: OCR = s'_p/s'_{v0}

Estimation of Stress History (OCR or s'_p) can be based on:

- **Direct correlation with CPTU data**

- **Pore pressure differential via dual element piezocone**

- **Indirect correlation via undrained shear strength**

CPTU Stress History Correlations

Wroth (1984), Mayne(1991) and others proposed theoretical basis (cavity expansion; critical state soil mechanics) for the following potential correlations between CPTU data and s'$_p$ or OCR:

σ'_p = = f(Δu_1 or Δu_2)
σ'_p = f(q_t - σ_{v0})
σ'_p = f(q_t - u_2)

OCR = f(B_q= Δu_2/(q_t - σ_{vo}))
OCR = f(Q_t = (q_t - σ_{vo})/ σ'_{v0}))
OCR = f((q_t - u_2)/ σ'_{v0})

Most Common:

$s'_p = k(q_t - s_{v0})$
or
$OCR = k[(q_t - s_{v0})/s'_{v0}]$

CPTU Stress History Correlations

Comprehensive study initially by Chen and Mayne (1996) with later updates (e.g., Mayne 2005):

$\sigma'_p = 0.47(\Delta u_1) = 0.53(\Delta u_2)$

$\sigma'_p = 0.33(q_t - \sigma_{v0})$

$\sigma'_p = 0.60(q_t - u_2)$ ← Most common

<u>Note</u>: values listed above are from best fit regressions; there is a sizable range in all values, e.g., k ranges from 0.2 to 0.5 for $\sigma'_p = k(q_t - \sigma_{v0})$

Importance of Sample Quality– Boston Blue Clay

Used 4 sampling methods

1. **Poor**: SPT sampler

2. **Fair**: Standard 76 mm thin walled tube sampler (with free or fixed piston)

3. **Good**: Fixed piston sampler in mudded borehole using modified 76 mm diameter thin walled tube

4. **Best**: Sherbrooke Block Sampler

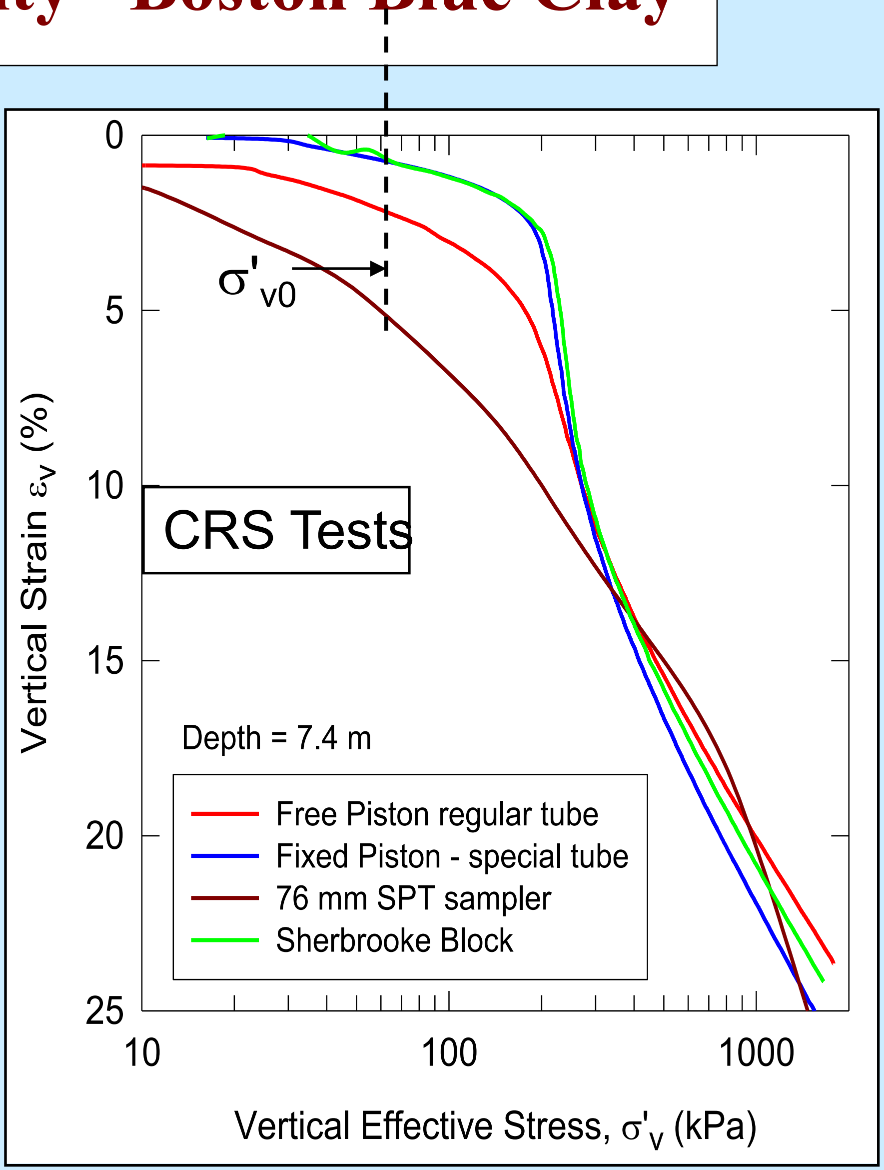
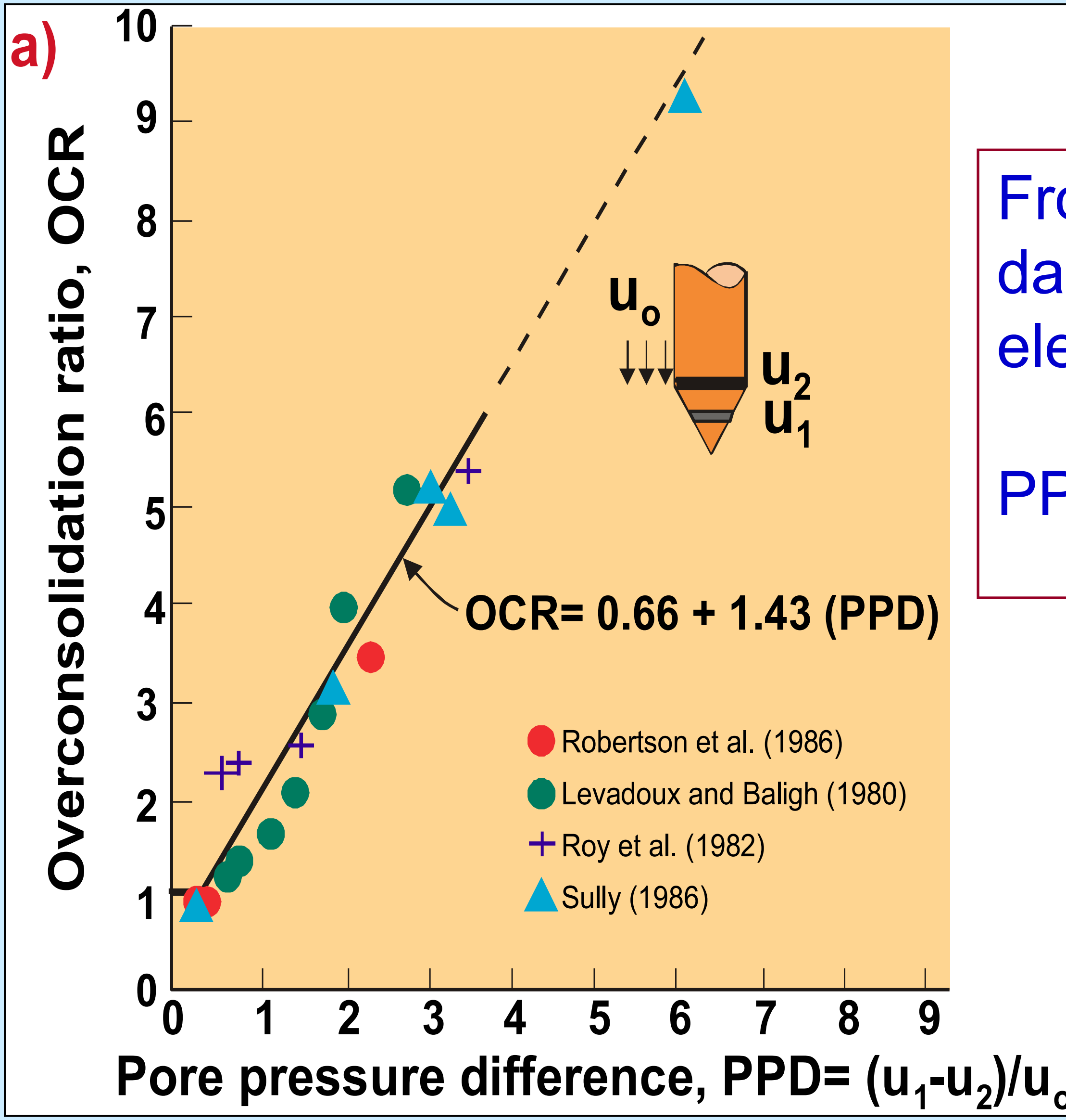


Fig. 29

CPTU Stress History Correlations



From pore pressure data using dual element piezocone

$PPD = (u_1 - u_2)/u_0$

[Sully et al.,1988]

Fig. 30

In Situ Horizontal Effective Stress

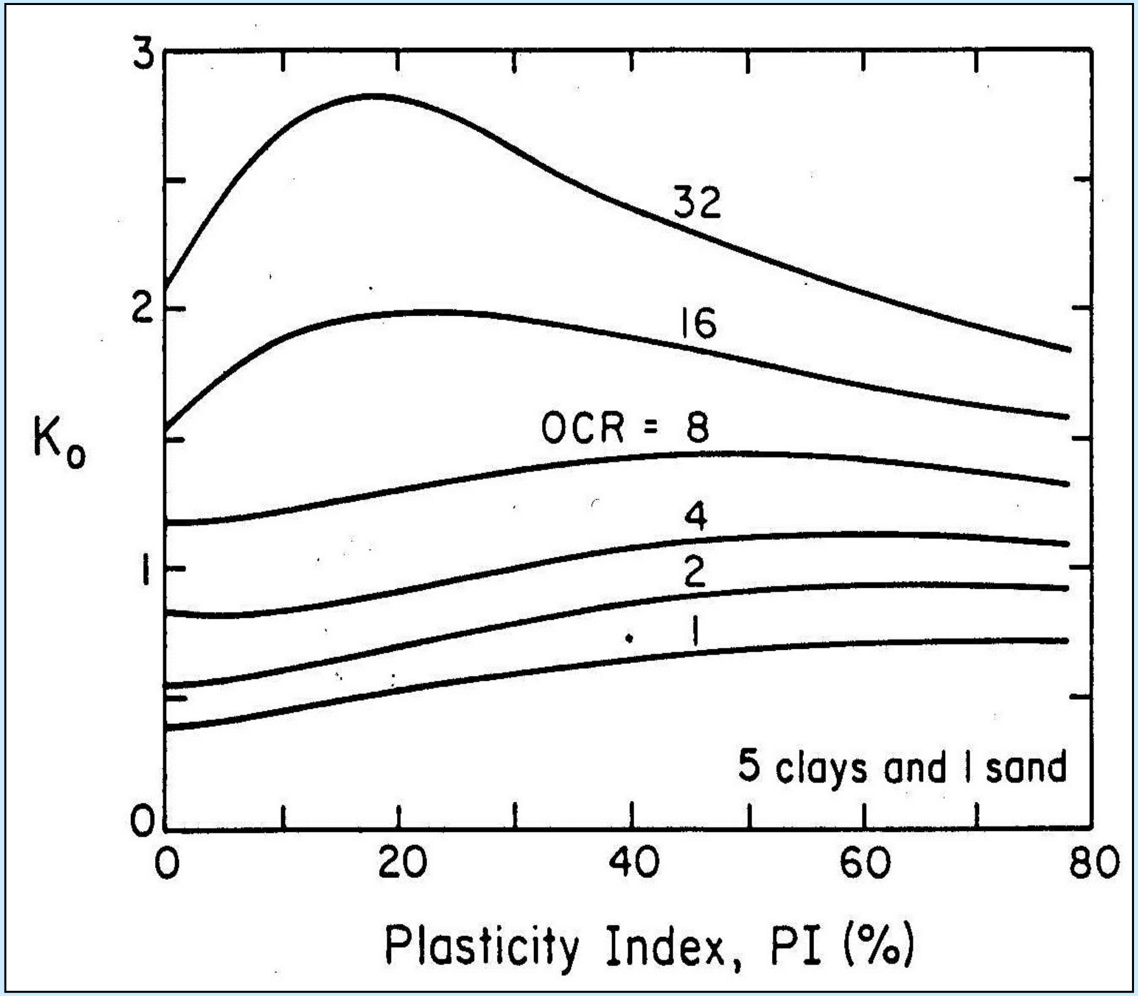
There are currently no reliable methods for determining the in situ horizontal effective stress, $s'_{h0} = K_0(s'_{v0})$ from CPTU data

For approximate (preliminary) estimates consider correlations based on:

- **OCR via CPTU correlations for OCR or s_u**
- **Measured pore pressure difference**

K_0-OCR-PI Relationship

[Brooker and Ireland 1965]

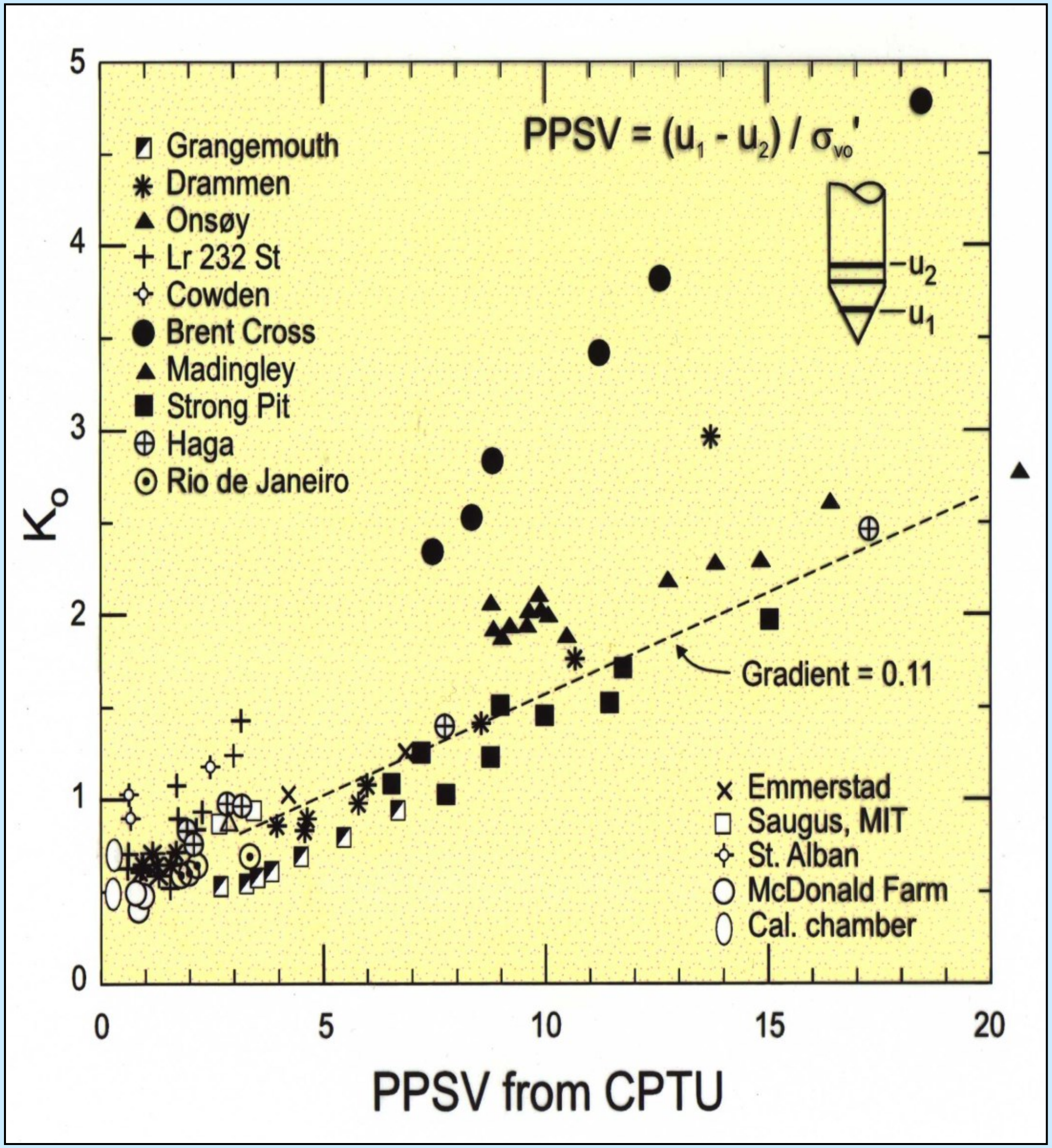


Need values for Plasticity Index (PI) and OCR.

Determine OCR from 1) CPTU correlations or via 2) undrained shear strength correlation (next slide)

Fig. 31

Estimate K_0 from Dual Element Piezocone



Difference between u_1 and u_2 increases with increasing OCR → K_0 also increases with increasing OCR, hence positive correlation between $(u_1 – u_2)/s'_{v0}$ and K_0.

[Sully and Campanella 1991]

Fig. 32

Undrained Shear Strength from CPTU Data

$$s_u = q_{net}/N_{kt} = (q_t - \sigma_{v0})/N_{kt}$$

Most Common

$$s_u = \Delta u/N_{\Delta u} = (u_2 - u_0)/N_{\Delta u}$$

Often used

$$s_u = q_e/N_{ke} = (q_t - u_2)/N_{ke}$$

Seldom used

Need empirical correlation factors N_{kt}, N_{Du}, or N_{ke} factors as correlated to a specific measure of undrained shear strength, e.g., s_u(CAUC) or s_u(ave)

Deformation Parameters

1. Constrained Modulus – for 1-D compression, M
2. Undrained Young's Modulus, E_u
3. Small strain shear modulus, G_{max}

Two approaches for use of CPT/CPTU data to estimate deformation parameters:

1. Indirect methods that require an estimate of another parameter such as undrained shear strength s_u.

2. Direct methods that relate cone resistance directly to modulus.

Example of Direct Correlation between CPTU and G_{max}

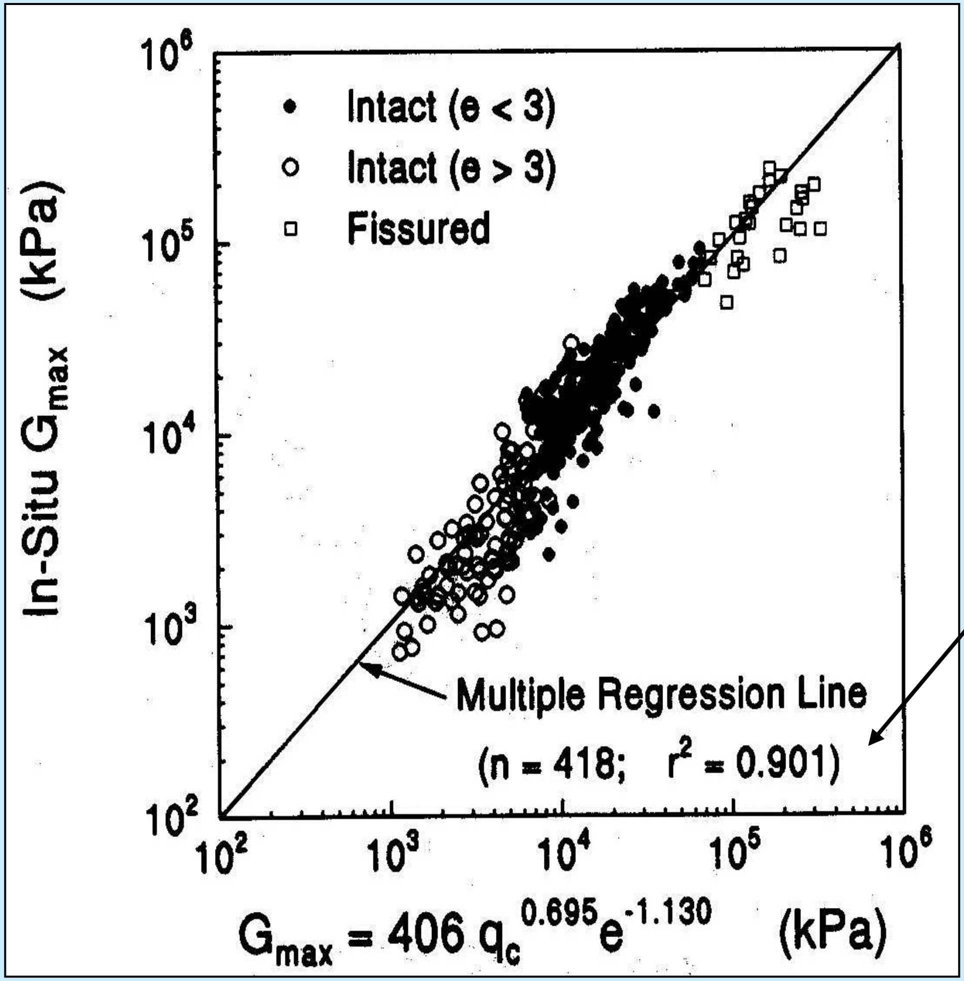


Fig. 33

Mayne and Rix (1993)

Estimation of small strain shear modulus G_{max} for clays from CPT q_c data + estimate e.

<u>Note</u>: G_{max} is anisotropic + in the context of CPT/CPTU testing, better to measure directly down hole with seismic cone (= G_{vh})

Consolidation and Hydraulic Conductivity

Measurement: dissipation of penetration pore pressures during pause in penetration. Can be u_1 or u_2. Ideally measure until Du = 0 but time depends on c_h and k_h.

Derived Soil Properties:

1. Coefficient of Consolidation, c_h

2. Hydraulic Conductivity (= permeability), k_h

Since the dissipation is radial, c_h and k_h are derived. Some clays can have highly anisotropic consolidation and flow parameters (e.g., varved clays) – need to use published anisotropy ratios to estimate k_v and c_v.

Theory for CPTU derived c_h and k_h

c_h

Terzaghi Theory: $c_v = (TH^2)/t$

Torstensson (1975, 1977) suggested use time at 50% dissipation and for CPTU geometry thus,

$c_h = (T_{50}/t_{50})r^2$

Hence for 10 cm^2 cone, $c_h = 0.00153/t_{50}$ [m^2/s]

k_h

Terzaghi Theory: $k_h = c_h g_w m_h$

Determine c_h from dissipation test + need estimate m_h = coefficient of volume change, which can be correlated to q_c or q_t

Recommendations - CPTU Derived Soil Engineering Parameters for CLAY

1. Do not eliminate sampling and laboratory testing
2. Verify reliability of results and that undrained conditions prevail
3. With increasing experience modify correlations for local conditions

Good CPTU Interpretation methods exist for:

- Soil Unit Weight (g_w)
- Stress History: OCR or s'_p
- Undrained Shear Strength for s_u(CAUC) and s_u(ave)
- Small strain shear modulus (G_{max})
- Coefficient of Consolidation (c_h)

Approximate estimates can be made from CPTU data for:

1. In Situ horizontal effective stress (s'_{h0} or K_0)
2. Remolded undrained shear strength (s_{ur}) or Sensitivity (S_t)
3. Hydraulic Conductivity (k_h)

Recommendations: CPT/CPTU based Soil Identification/Classification

- Use all information available, e.g., q_c or q_t, f_s, u, F_r, B_q

- Shape and magnitude of q_t profile gives indication on whether you are in uniform clay layer, sand layer, etc.

- Pore pressure profile readily indicates a drained condition (e.g., sand with $\Delta u = 0$) or undrained (e.g., clay with $\Delta u > 0$)

- Use q_t - R_f - B_q and/or Q_t-F_r-B_q diagrams to identify soil type. Accumulate local experience to create/modify diagrams.

- Short dissipation tests can help in identifying soil type

- Measurements using other sensors (e.g., V_s) can enhance soil identification

OCR from Piezocone Test

For clays with $S_t \leq 8$, and OCR ≤ 8, the correlation between B_q and OCR can be approximated as follows according to Chang (1991a):

$$OCR = \frac{2.3\,\mathrm{B_q}}{3.7\,\mathrm{B}_q - 1}$$

The equation provides consistent but slightly conservative estimates of OCRs for both the Singapore and the Malaysian marine clays, according to Chang (1991a).

Coefficient of consolidation from piezocone dissipation test

(2) Teh and Housby (1991)

Define modified Time Factor as

$$T^* = \frac{c_h t}{a^2 \sqrt{I_r}}$$

where c_h is the coefficient of consolidation from radial drainage, t is the time elapsed, and a is the radius of the cone, and I_r is rigidity index.

Field Vane Test

Field Vane Test (FVT)

1. In situ test developed to measure undrained shear strength (s_u) of fine-grained soils
2. Calibrated against back analysis of embankment failures, i.e., stability problems
3. Widely used as a frame of reference for other in situ tests and laboratory tests for interpretation of s_u

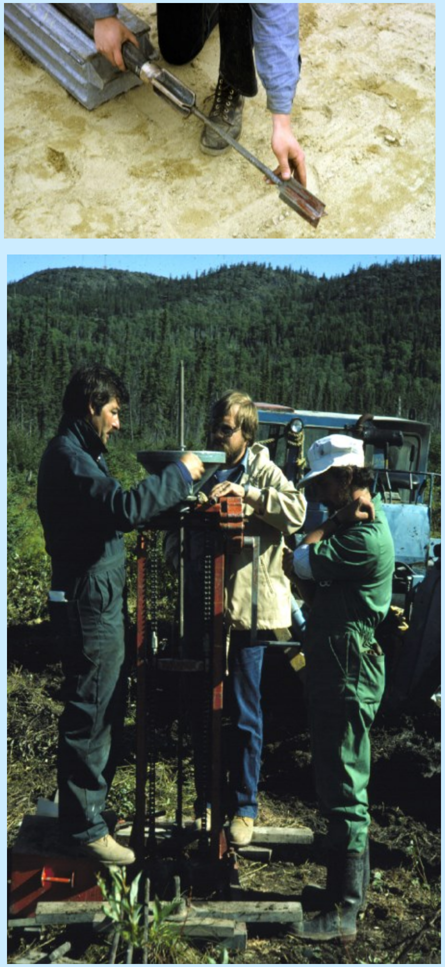


Fig. 1

FVT - Equipment and Mechanics

1. Push thin bladed vane into soil, rotate and measure torque

2. Usual geometry: rectangular with 4 blades, sized to match expected strength of soil, H/D = 2

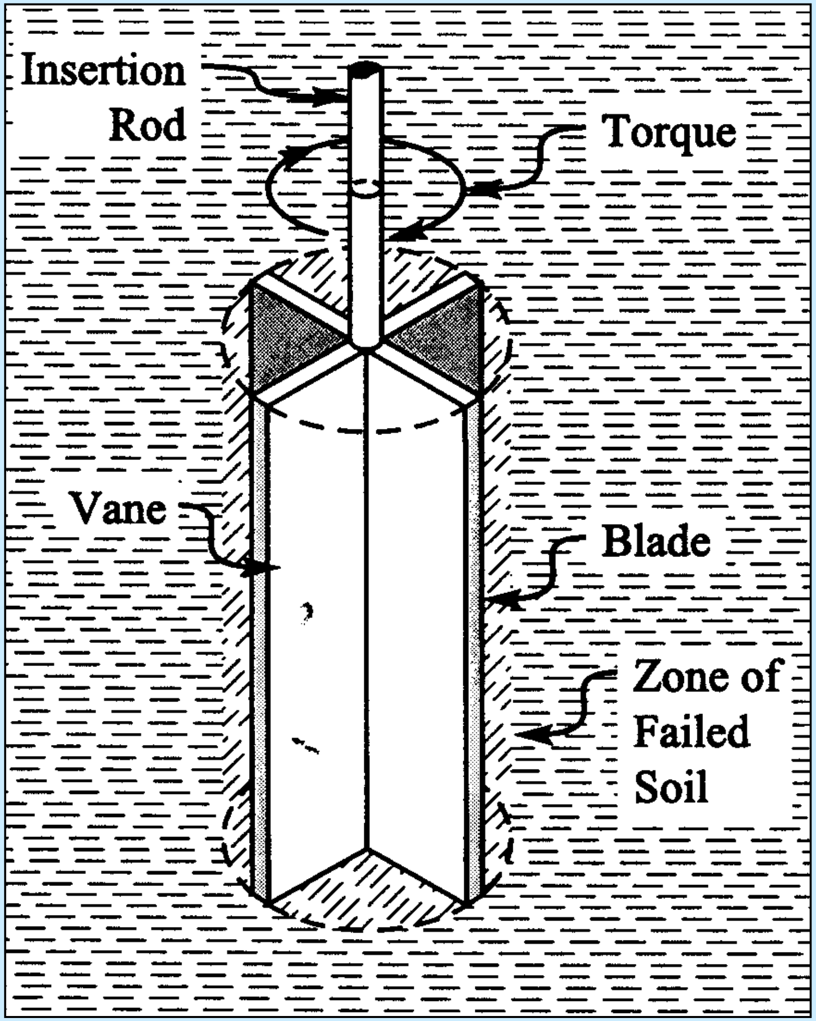


Fig. 2

Nilcon Vane Borer

Fig. 3

Fig. 4

GeoMil Electric Vane Tester

- Computer control and data acquisition
- 0.1 to 20 degrees per second
- real time plotting of torque vs rotation

Fig. 5

Fig. 6

Pictures from GeoMil

Geonor Vane

Acker Drill Co. Vane

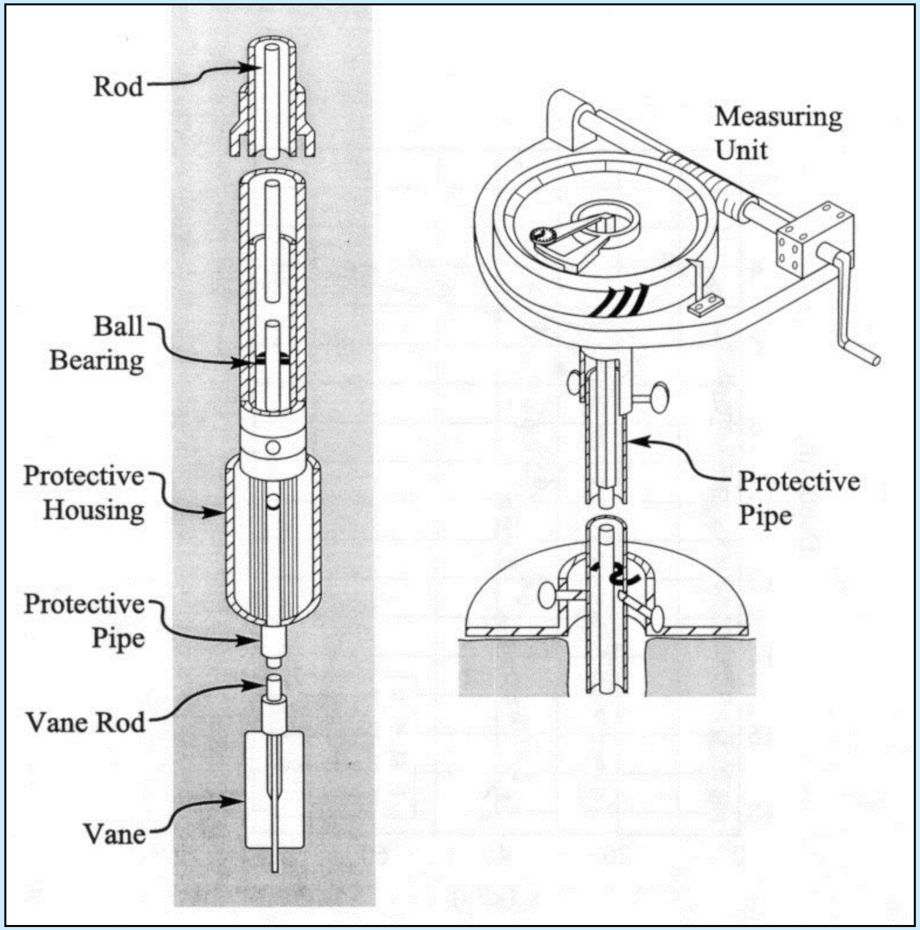


Fig. 7

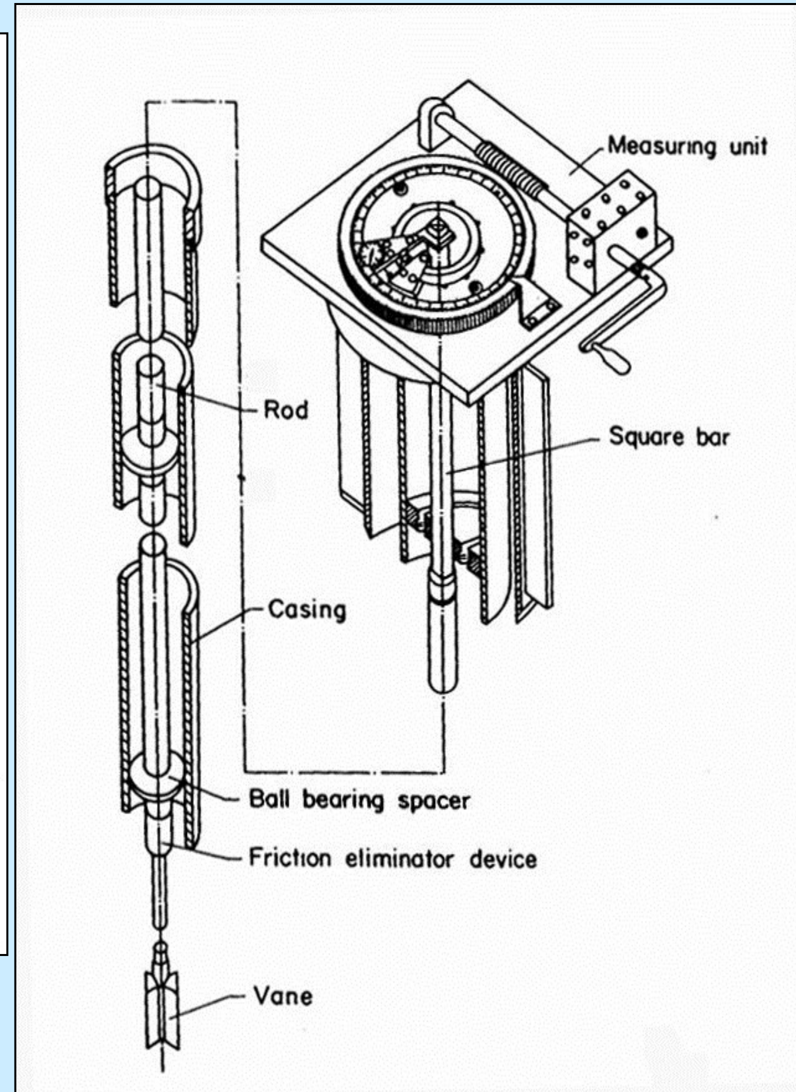


Fig. 8

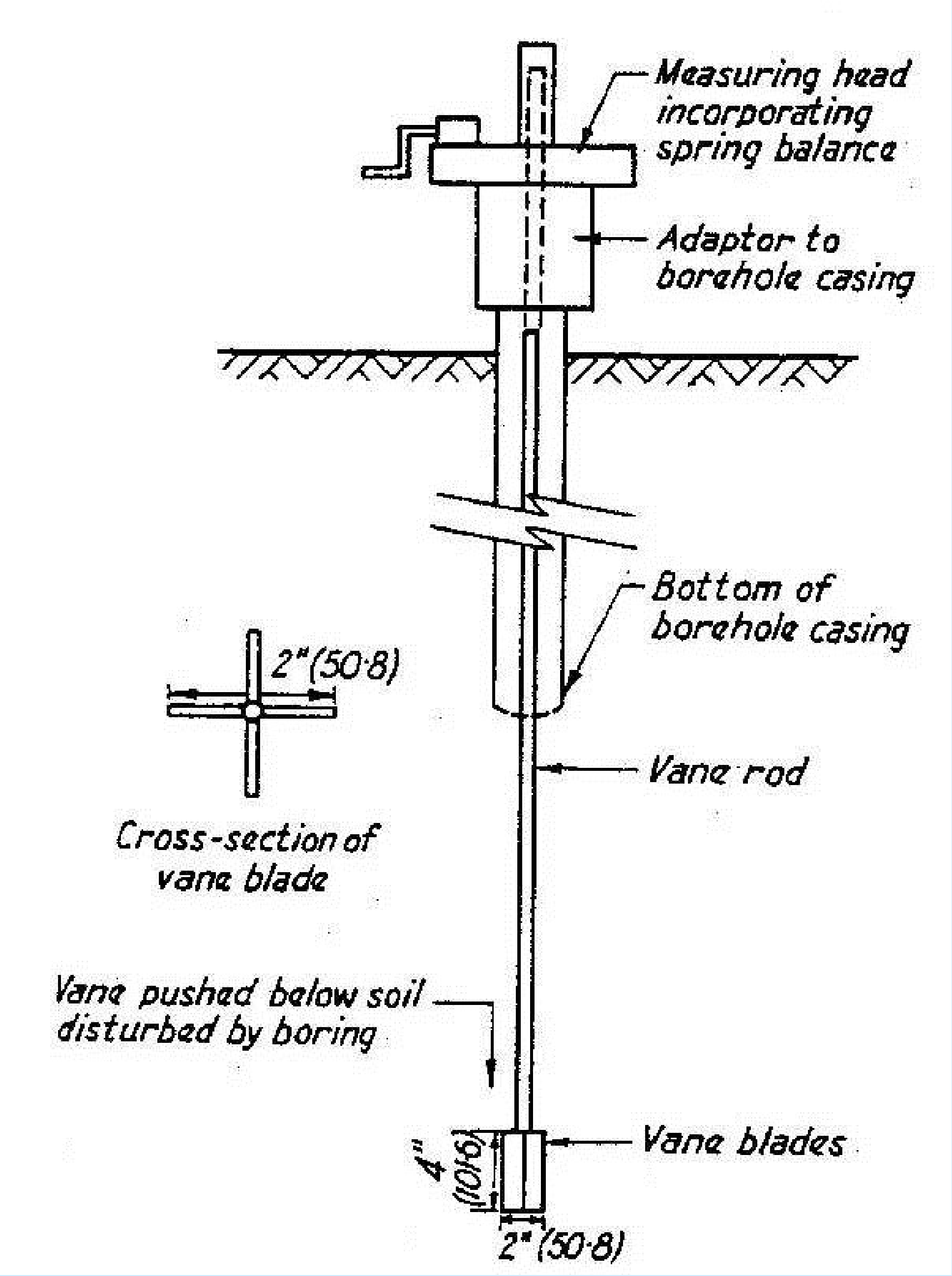
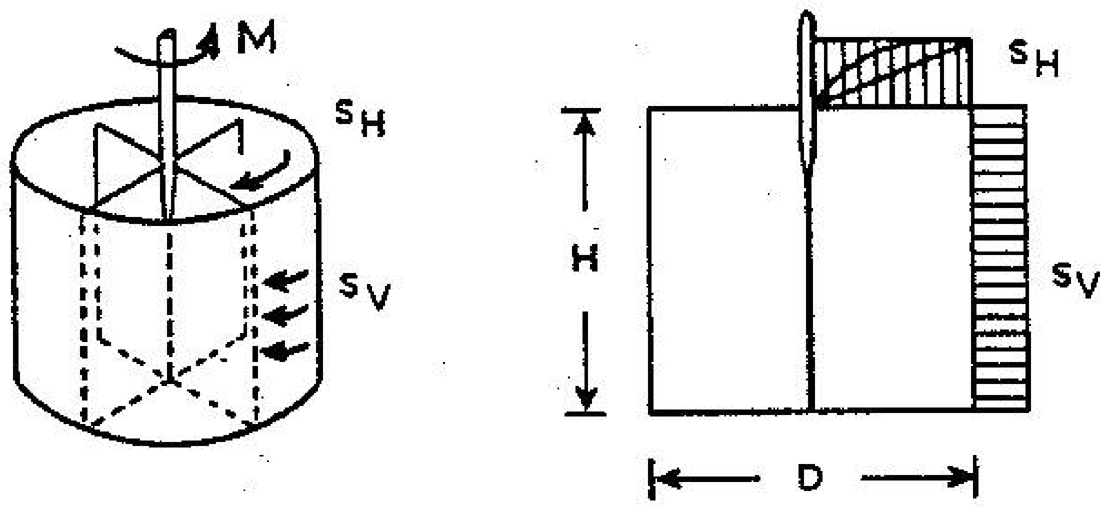

Measuring head
incorporating
spring balance
Adaptor to
borehole casing
Bottom of
borehole casing
2" (50·8)
Cross-section of
vane blade
Vane rod
Vane pushed below soil
disturbed by boring
Vane blades
4" (101·6)
2" (50·8)

Fig. 10



INTERPRETATION:

$$M = \pi D H \frac{D}{2}\, s_V + 2\frac{\pi D^2}{4}\,\frac{D}{3}\cdot s_H$$

$$\left[\frac{2}{\pi D^2 H}\right] M = s_V + s_H\left[\frac{1}{3}\,D/H\right]$$

In-situ vane strength

Fig. 11

Table 1 USA specifications for vane blades **(Clayton *et al*, 1995)**

Casing size	Vane diameter (mm)	Vane height (mm)	Blade thickness (mm)	Diameter of vane rod (mm)
AX	38.1	76.2	1.6	12.7
BX	50.8	101.6	1.6	12.7
NX	63.5	127.0	3.2	12.7
4in. (101.6mm)	92.1	184.1	3.2	12.7

Table 2 UK specifications for vane blades **(Clayton *et al*, 1995)**

Undrained shear strength (kPa)	Vane diameter (mm)	Vane height (mm)	Rod diameter (mm)
<50	75	150	<13
50—75	50	100	<13

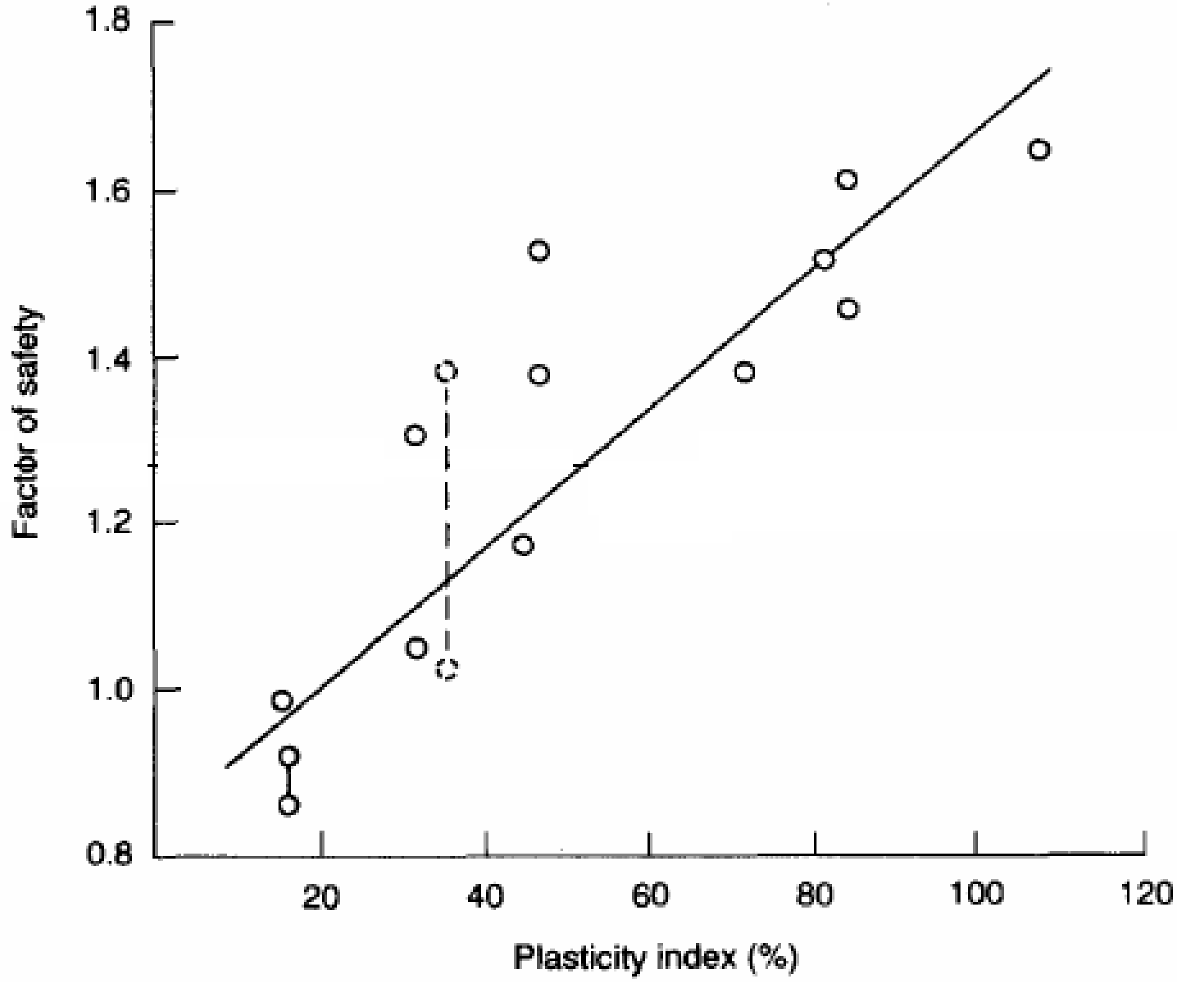


Fig. 15 Calculated factors of safety for failed embankments, based on (Bjerrum 1972).

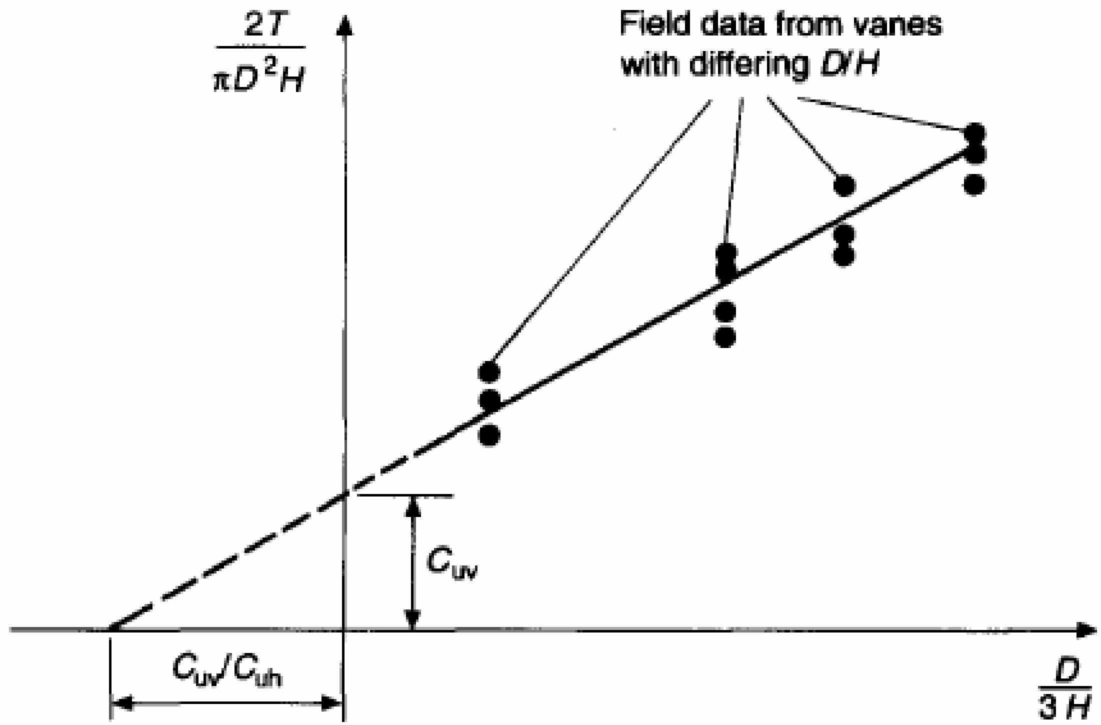


Fig. 16 Method of determining undrained strength anisotropy (Aas 1965, 1967).

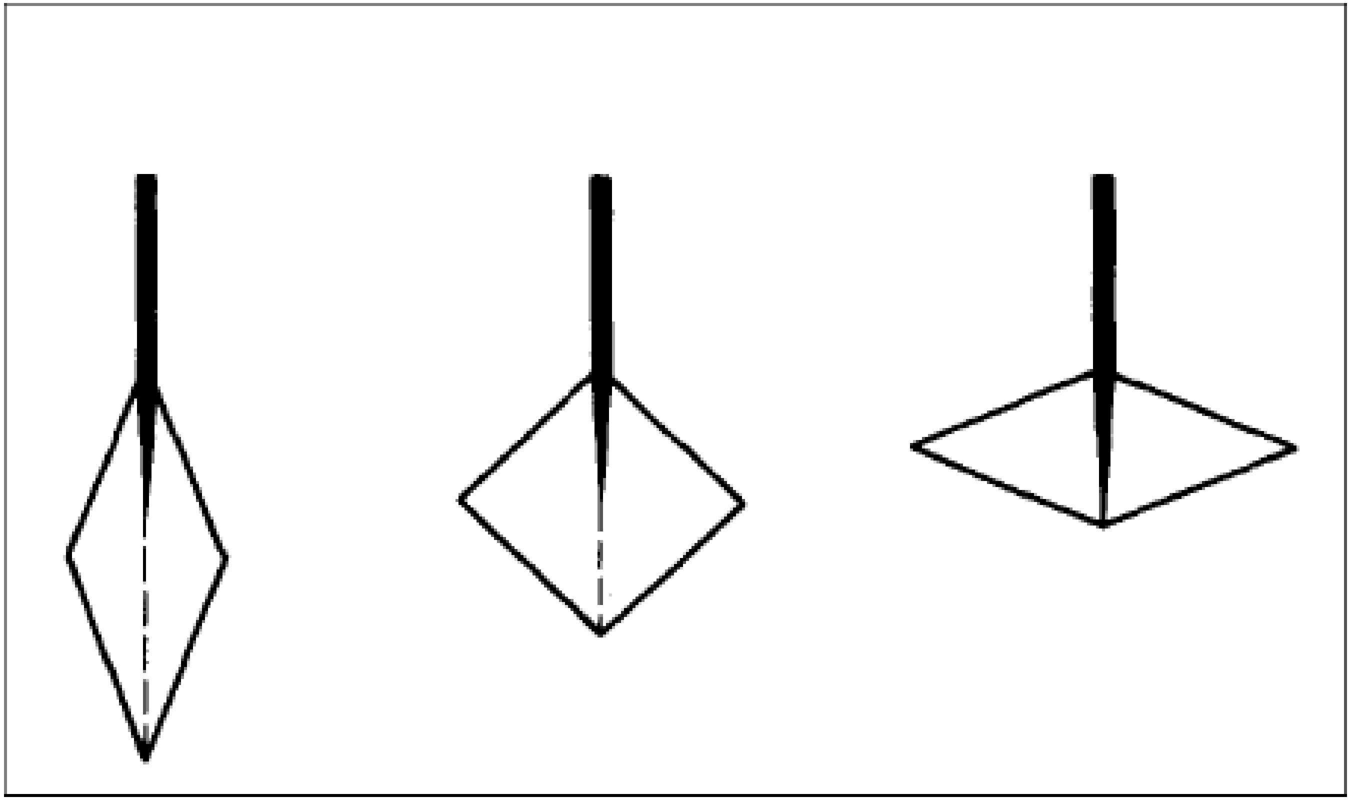


Fig. 17 Diamond shear vanes.
(Clayton *et al*, 1995)

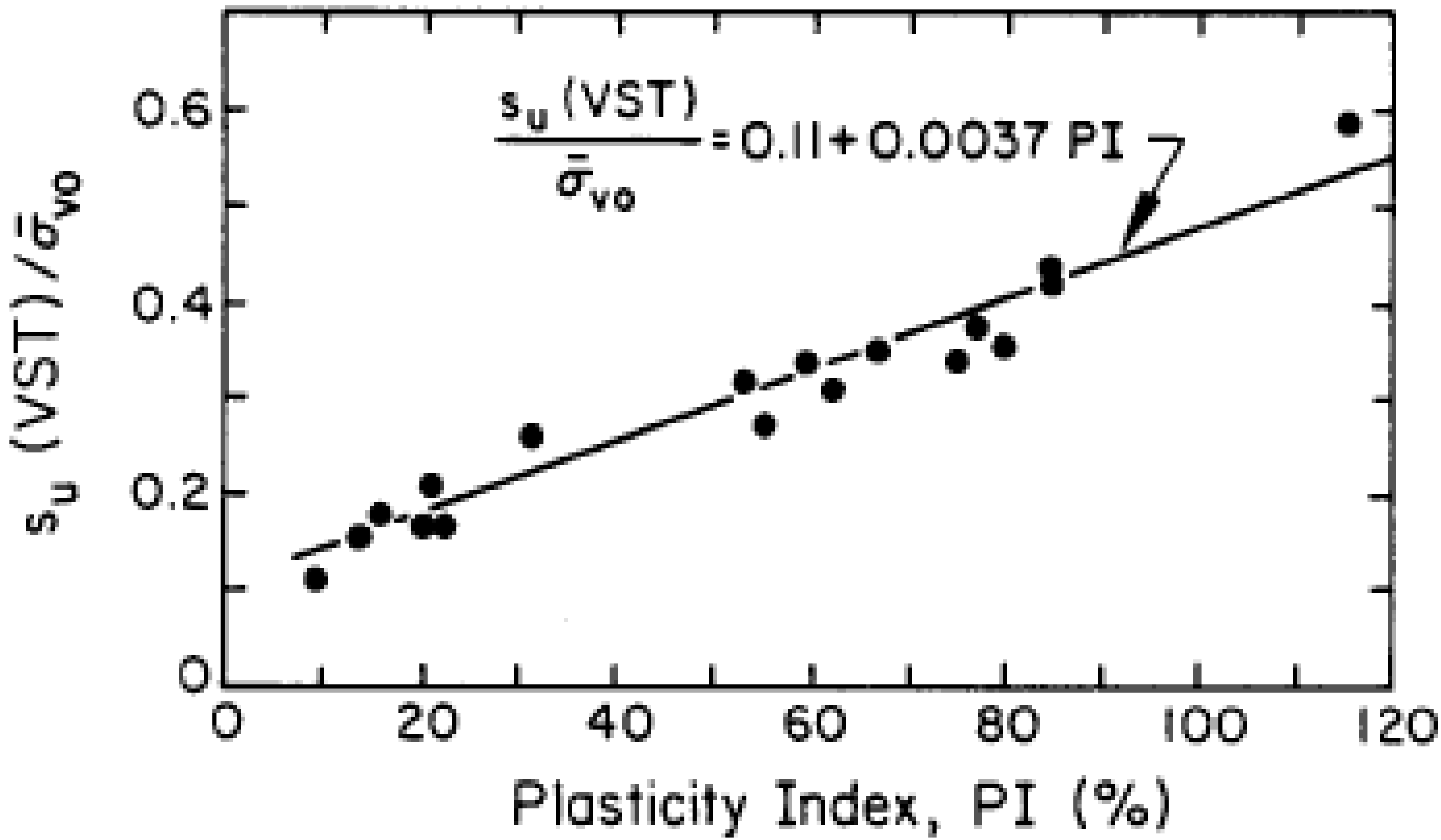


Fig. 18 $s_u(VST)/\bar{\sigma}_{vo}$ versus PI for NC Clays

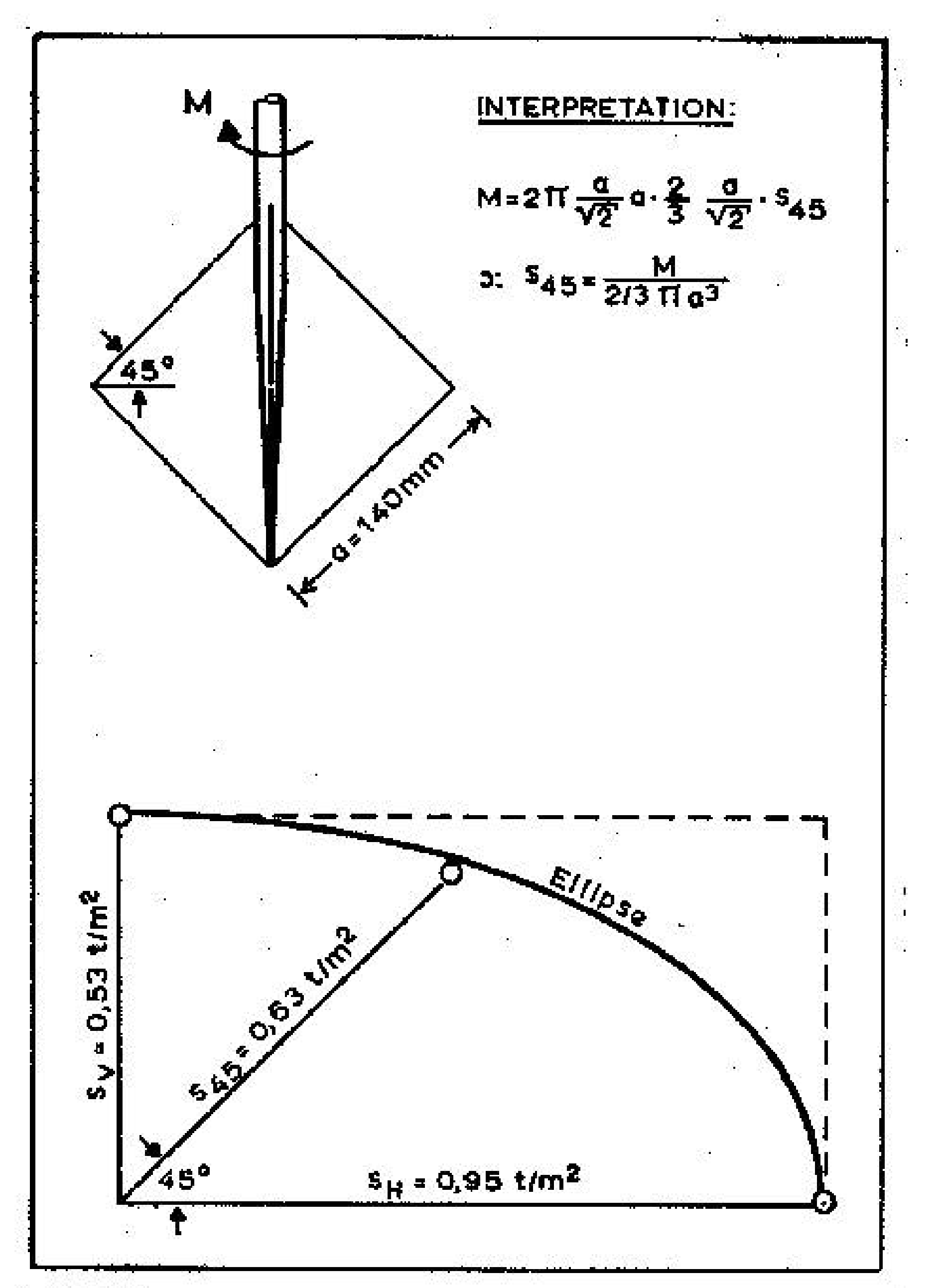


Fig. 19

An-isotropic vane strength

FVT – Test Variables

1. Installation
2. Consolidation Time
3. Shear Rate
4. Progressive Failure
5. Vane size
6. Vane Shape

FVT – Test Procedure

1. ASTM D2573 "*Standard Test Method for Field Vane Shear Test in Cohesive Soil*"
2. Rectangular vane w/ H/D = 2
3. Test at ≥ 5 diameters from base of borehole
4. Wait time after insertion? → 1 to 5 min
5. Rotate ≤ 0.1°/s = 6°/min, t_f ~ 2 – 5 min
6. After failure rotate ~ 10 times to measure s_{ur}
7. Test interval ≥ 2 ft

FVT Standards and Guidelines

Examples of some differences (after Lunne 2006)

Table 3

Parameters	ASTM[1]	BS[2]	NGF[3]	SGF[4]	CEN[5]
Vane blade diameter (mm)	38.1 / 50.8 63.5 / 92.1	50 / 75	55 / 65	40 – 100	40 – 100
Thickness of blade (mm)	1.6 / 3.0	??	2.0	0.8 – 3.0 / avg. ≤ 2.0	0.8 – 3.0
Procedure depth of insertion	5x hole dia.	3x hole dia.	0.5 m below shoe	5x hole dia.	5x hole dia. or 0.5 m
Rate of rotation	6°/min	6-12°/min	12°/min	not specified	6 - 12°/min
Time to failure	2 to 5 min	5 min	1 to 3 min	2 to 4 min	not specified
s_{ur} - min # revolutions	5 - 10	not given	25	20	≥ 10
Delay time	< 5 min	-	< 5 min?	2 - 5 min	2 – 5 min
Interval between tests	> 0.76 m	0.5 m	0.5 - 1.0 m?	> 0.5 m	≥ 0.5 m

FVT – Installation Disturbance

1. Depends on vane dimensions and soil properties
2. Use Perimeter Ratio $\alpha = 4e/\pi D$
3. Want low α, therefore D or ↓ e
4. Typical commercial

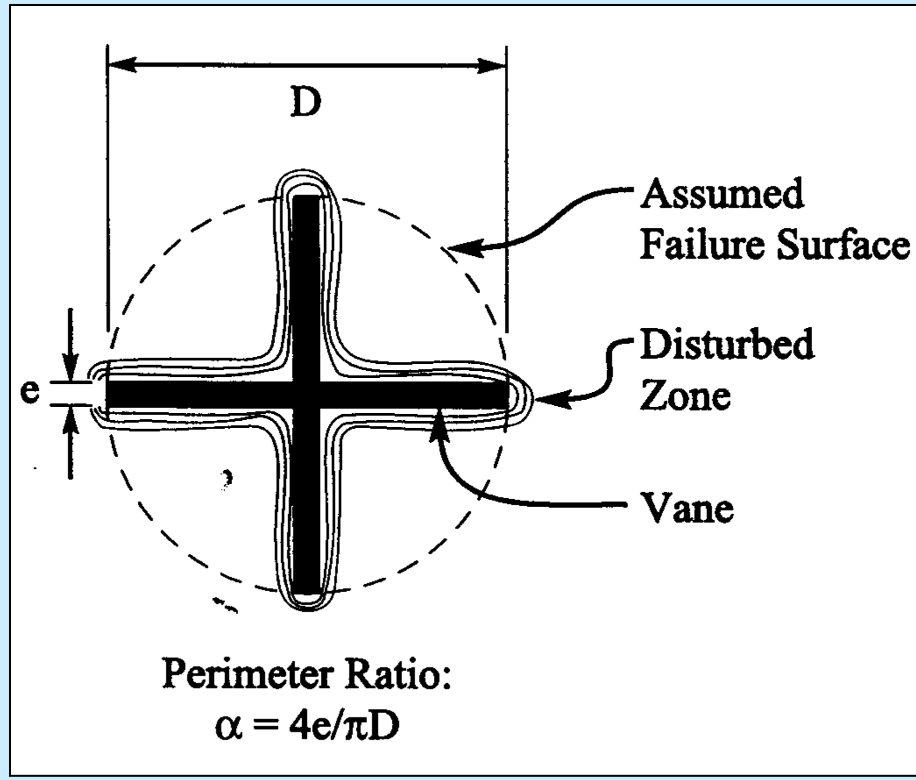


Fig. 21

FVT – Consolidation Time

1. Generate excess pore pressures during deployment – depends on OCR
2. What to do?
3. Usually 1 to 5 min after installation

FVT – Rate of Shearing

1. Strain rate effects
2. $V = r\omega$
3. Therefore must consider r and ω
4. Effect is function of soil type

FVT – Calculations

$$T = s_u(\pi DH)(D/2) + 2s_u(\pi D^2/4)(D/a)$$

where

T = torque

s_u = undrained shear strength

D = diameter of vane

H = height of vane

a = shape factor

Contribution of top and bottom surfa
is relatively minor

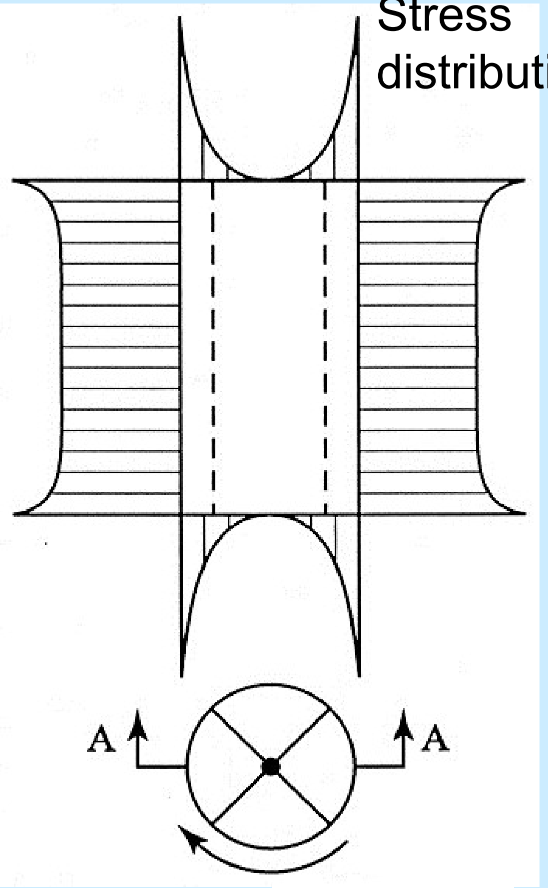


Fig. 22

FVT – Calculations (cont)

Typically use H/D = 2 and assume a = 3, therefore

$$\boxed{s_u = 6T/7\pi D^3}$$

FVT – Remolded Strength

1. Measure remolded shear strength = s_{ur}
2. Compute sensitivity S_t as

$$\boxed{s_t = s_u / s_{ur}}$$

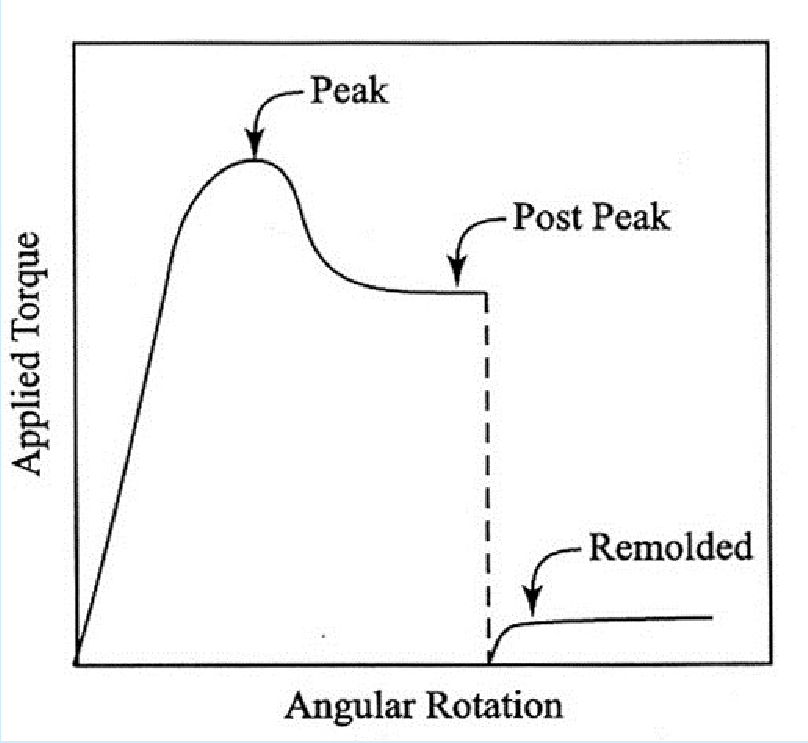


Fig. 23

- Remains the best in situ geotechnical tool to measure S_t

Example Field Vane profiles at UMass Amherst National Geotechnical Experimentation Site$_2$

- A lacustrine Varved clay deposit with an upper desiccated crust

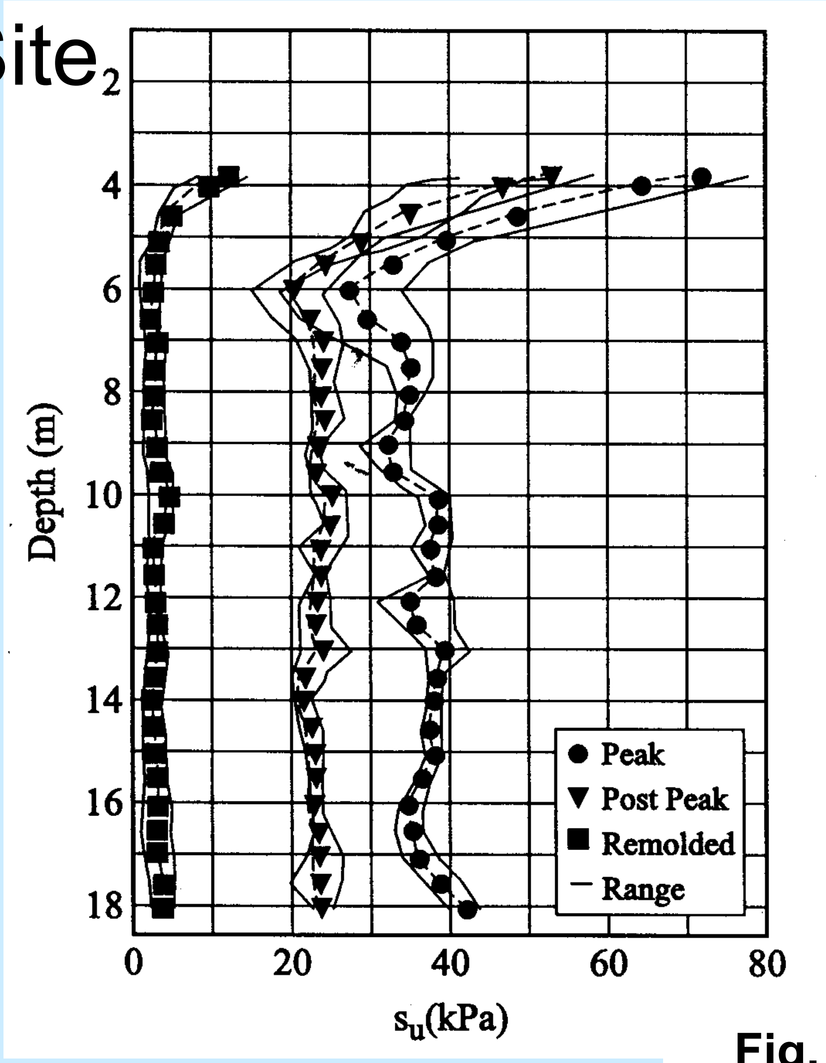


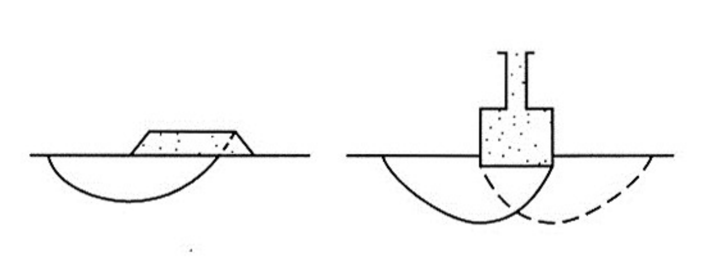
Fig. 24

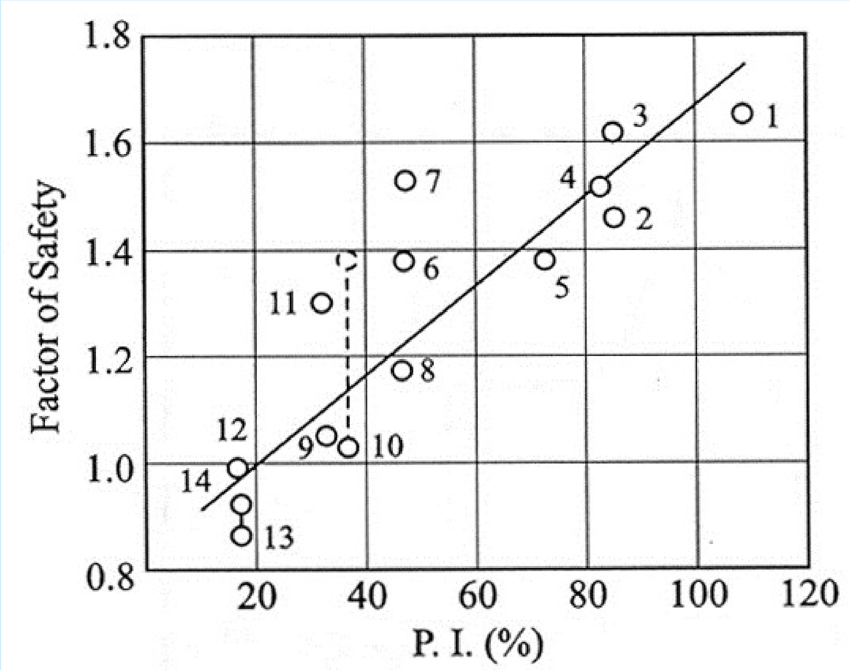
FVT – Correction Factors

Bjerrum (1972) suggested s_u(FVT) needs to be corrected for stability analysis

$$s_u = \mu s_{u(FVT)}$$

where μ = 1/FS based on stability of embankments. To compensate for disturbance, strain rate, anisotropy and progressive failure

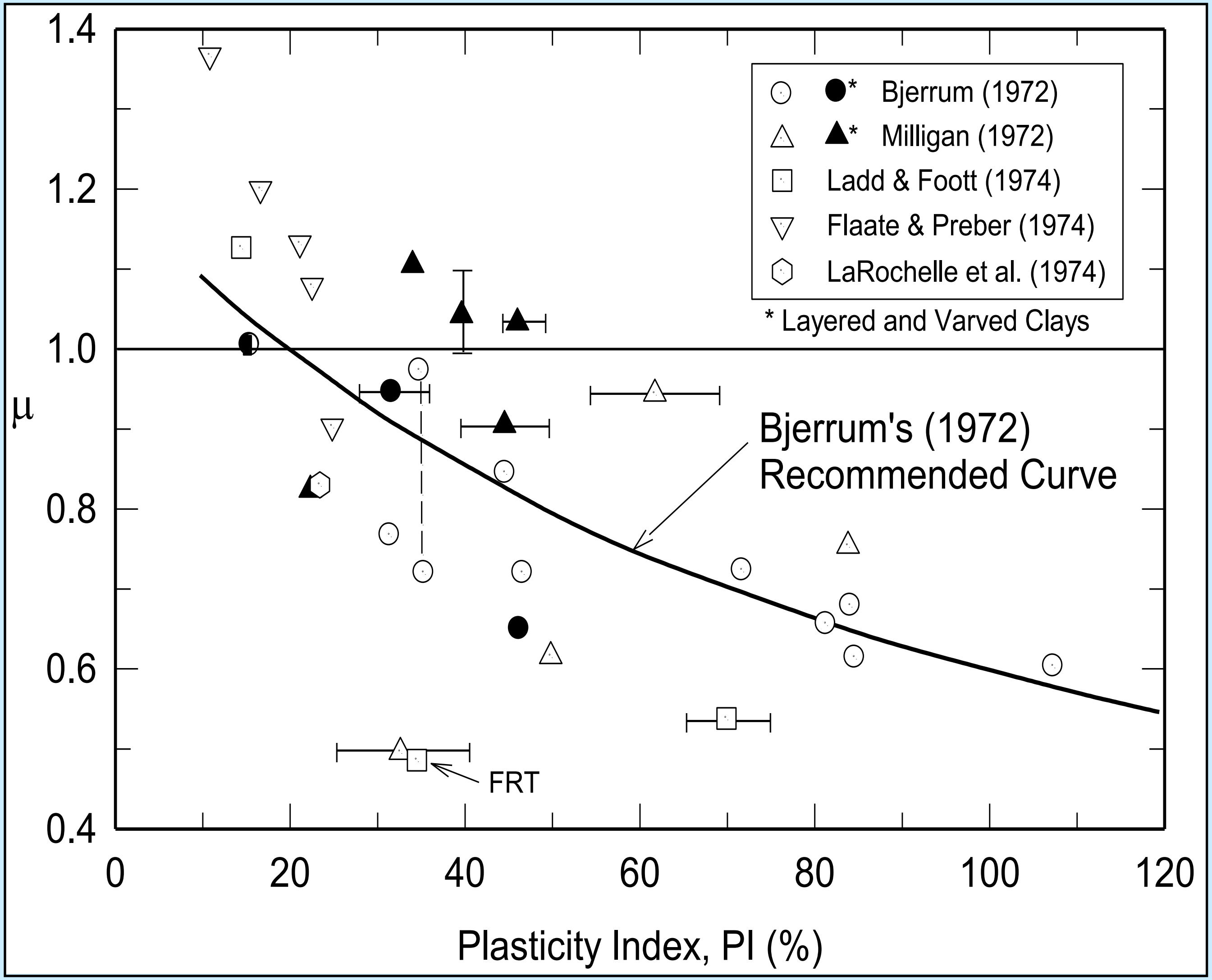




[after Bjerrum 1972]

Fig. 25

Embankment failures → $s_u(\text{ave}) = \mu s_u(\text{FV})$



[from Ladd and DeGroot 2003]

Fig. 26

FVT – Recommendations

1. Rectangular vane with constant cross section, H/D = 2
2. Calibrated torque head, gear driven
3. Insert slowly and begin test within 1 min.
4. Peak, post-peak, & remolded strength
5. Report geometry of vane used + gear system
6. Use Bjerrum’s correction factor for stability problems only

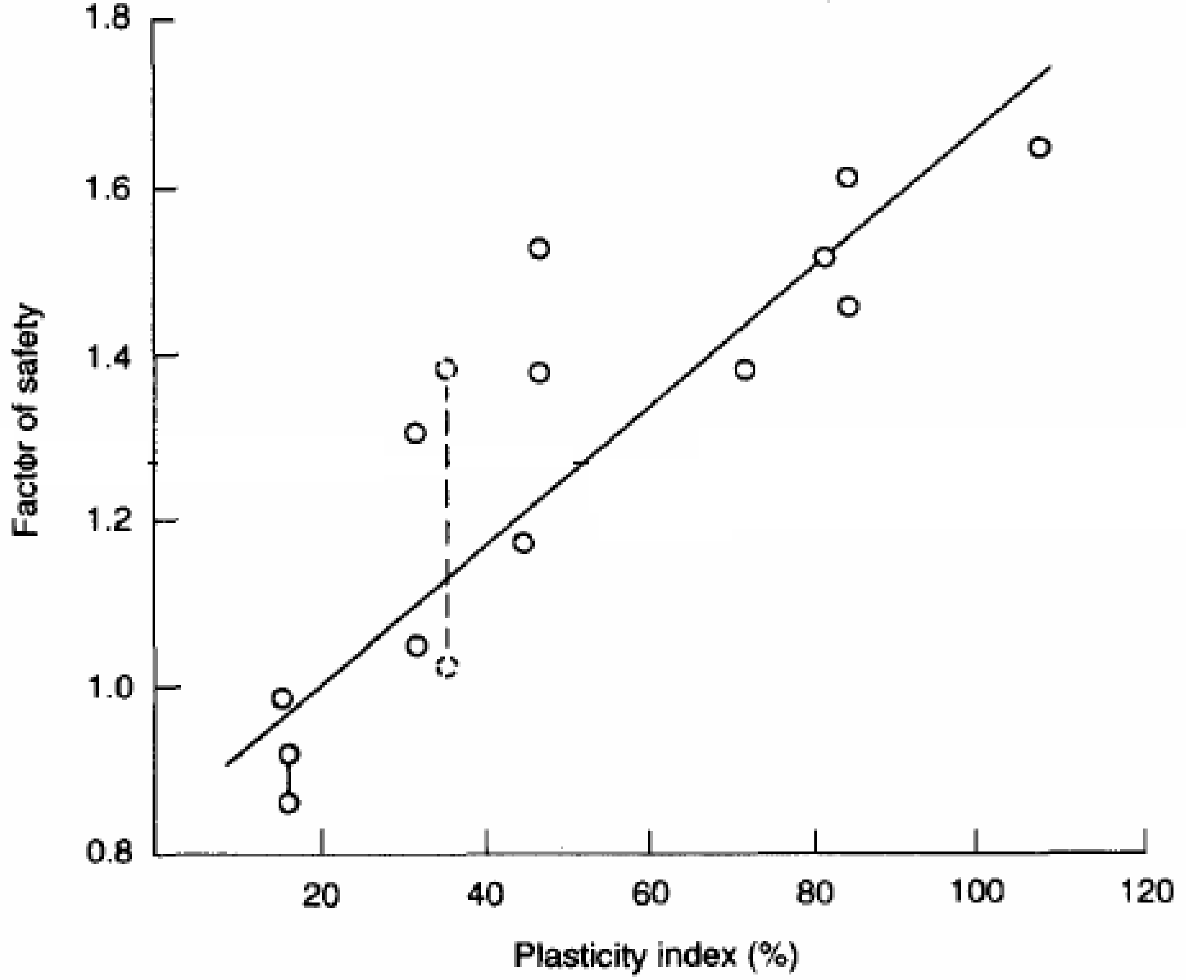


Fig. 9.16 Calculated factors of safety for failed embankments, based on (Bjerrum 1972).

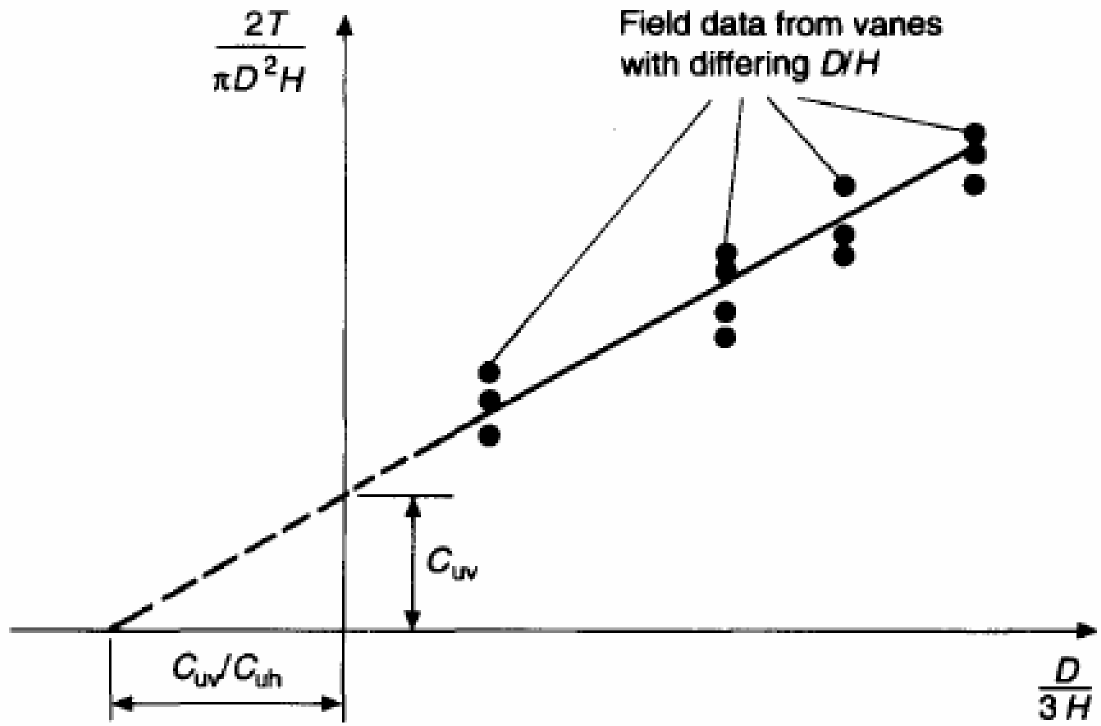


Fig. 9.17 Method of determining undrained strength anisotropy (Aas 1965, 1967).

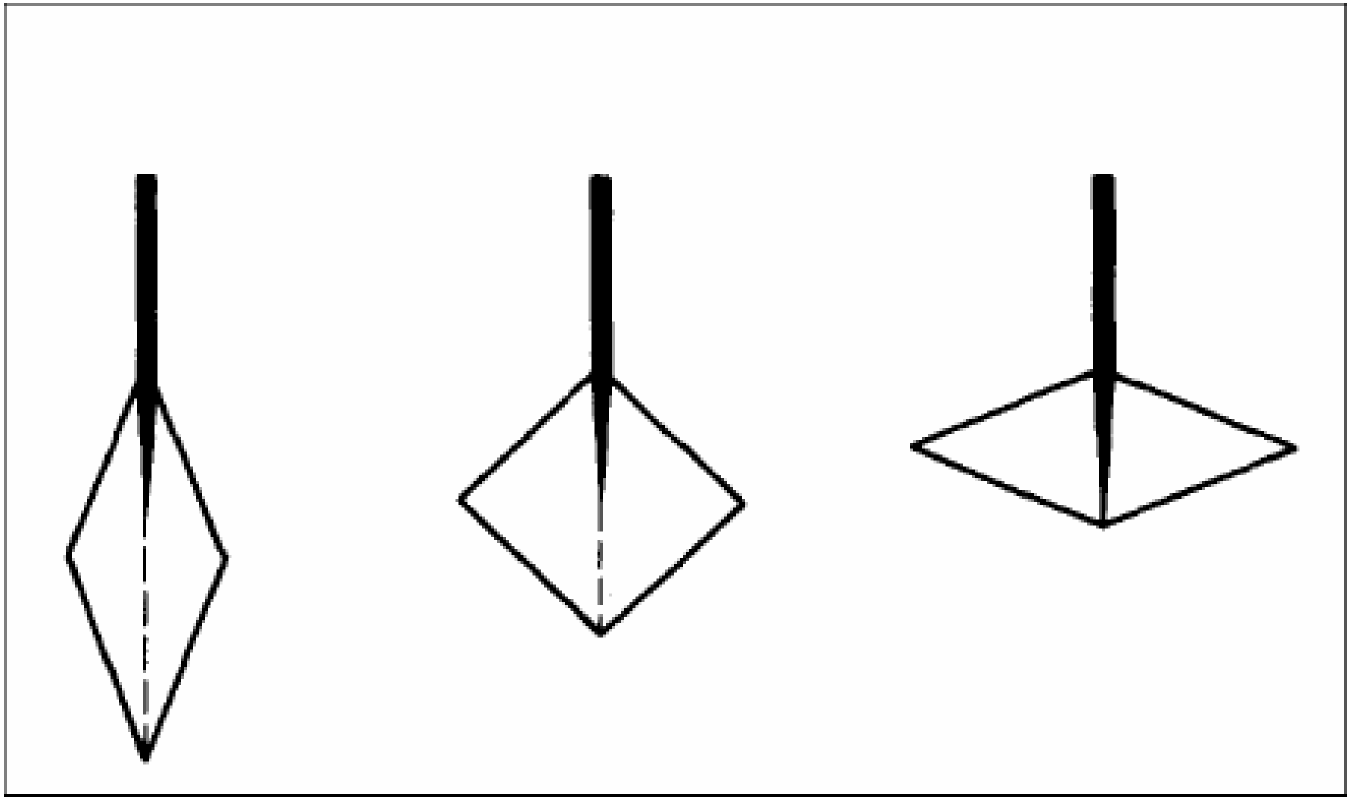


Fig. 9.18 Diamond shear vanes.

(Clayton *et al*, 1995)

FLAT PLATE DILATOMETER TEST (DMT)

Course in Brisbane 2 and 3 July 07

THE FLAT DILATOMETER

APPLICATIONS to GEOTECHNICAL DESIGN (DMT and SDMT)

With input from
Geotechnical Group, Dipartimento DISAT
Marchetti S., Monaco P, Totani G.
University of L'Aquila, Italy

DILATOMETER

- **Method was developed by Silvano Marchetti in Italy in 1970**
- **Established in profession after basic paper by Marchetti (1980)**
- **Initially introduced in Europe and North America**
- **Now used in 40 countries**

Marchetti, 2007

KEY REFERENCES

STANDARDS

Eurocode 7 (1997). Geotechnical design - Part 3: Design assisted by field testing, Section 9: Flat dilatometer test (DMT).

ASTM D 6635-01 (2001). "Standard Test Method for Performing the Flat Plate Dilatometer ".

MANUALS

→ ISSMGE TC16 (2001) DMT in Soil Investigations.

→ Short Course NOTES on Test Execution (Bali, 2001)

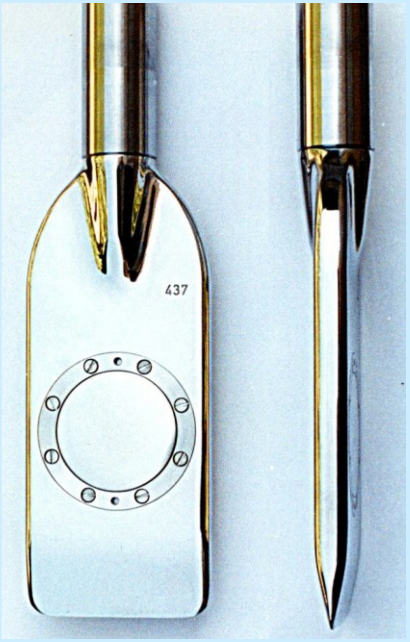
SDMT

→ Marchetti D. Experience with SDMT in various soil types (Taipei ISC 3, 2008)

DMT on the INTERNET

Bibliographic site <www.marchetti-dmt.it> download papers

GENERAL LAYOUT of DMT



Blade 95 mm wide, 15 mm thick, membrane dia., 60 mm

Fig. 1

Reading unit

Fig. 2

Push force provided by penetrometer or drill rig or other equipment

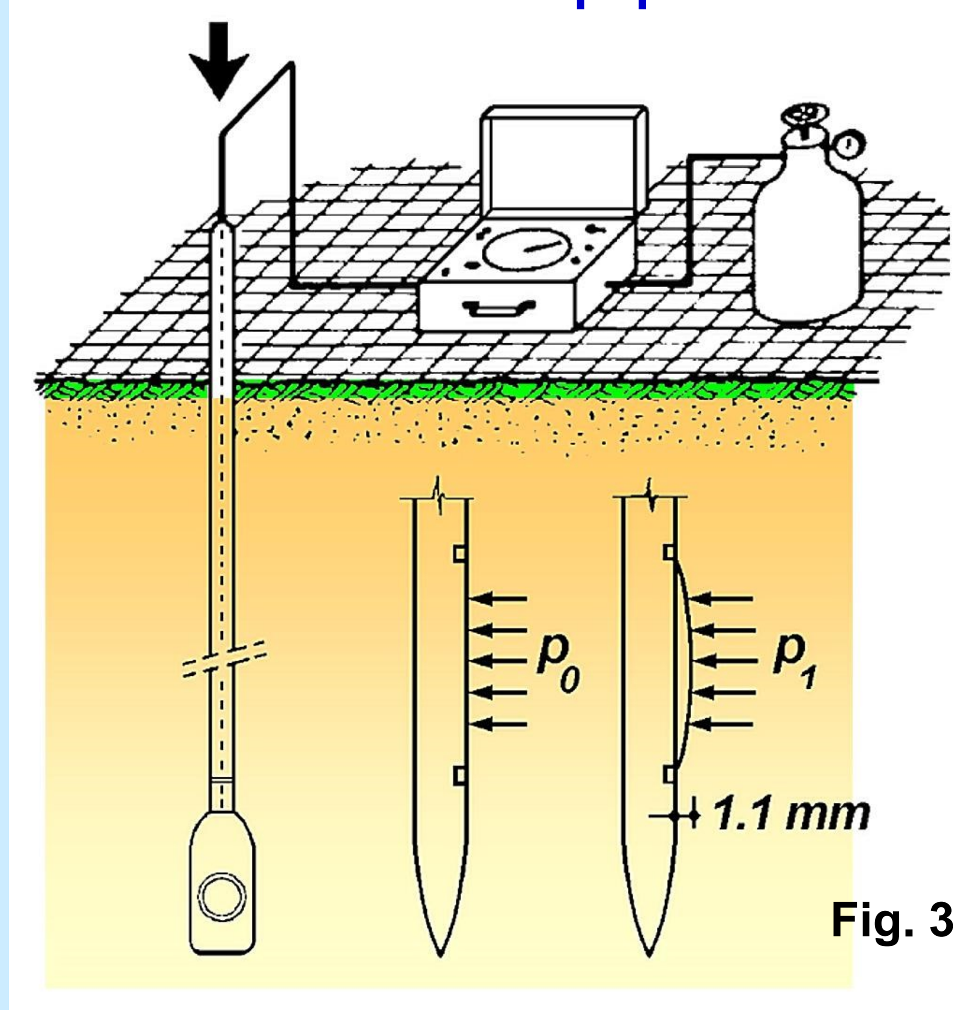
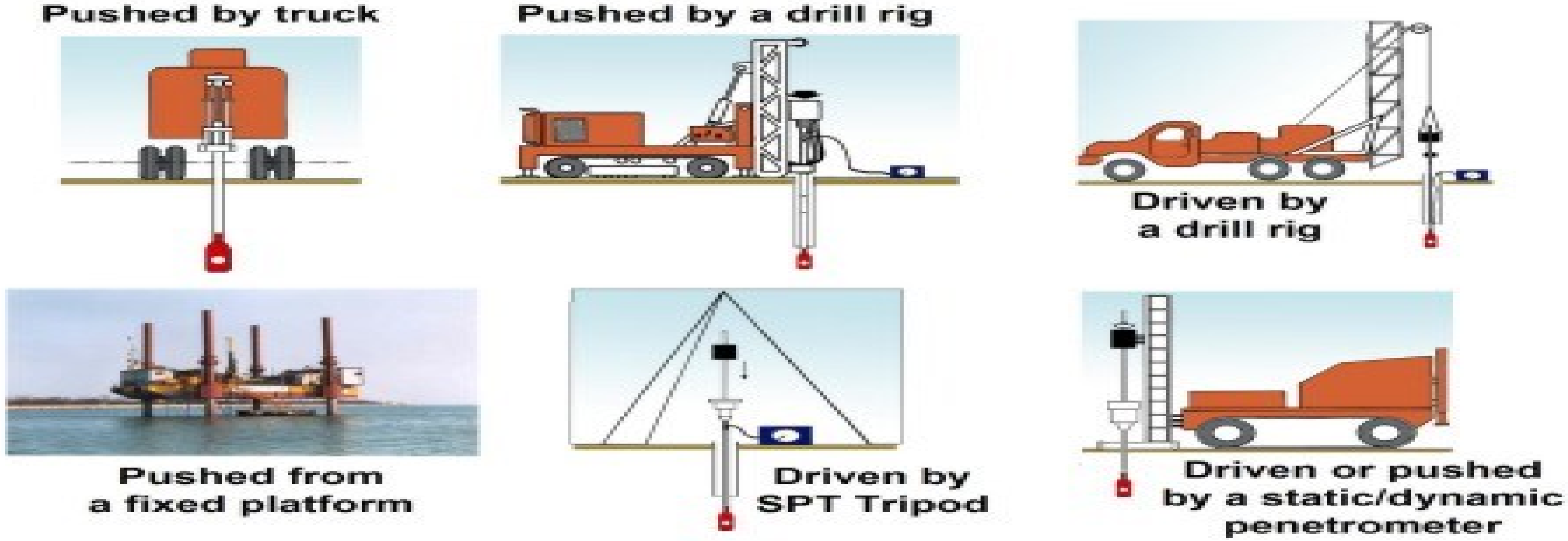


Fig. 3

WAYS OF INSERTING THE BLADE



"TORPEDO" INSERTION METHOD

Fig. 4

INSERTION of the BLADE

DMT USING A PENETROMETER

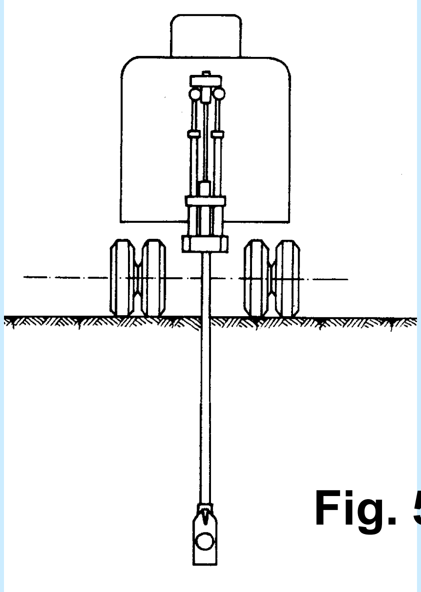
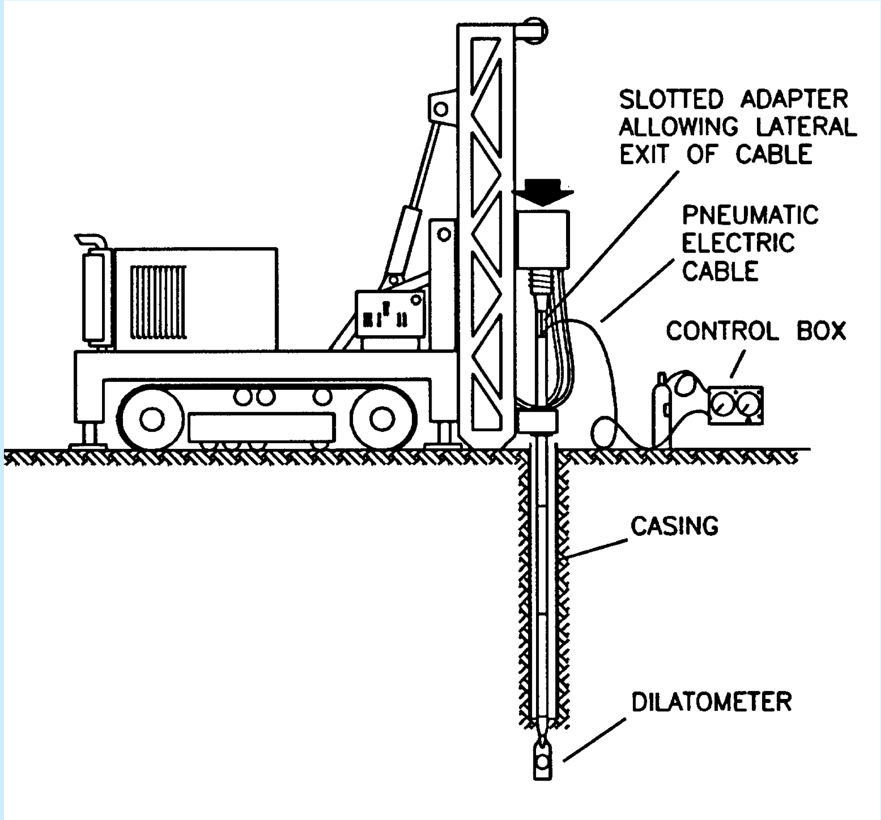


Fig. 5

DMT USING A DRILL RIG



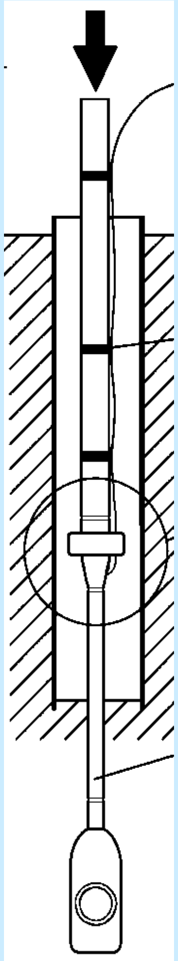


Fig. 6

BLADE WORKING PRINCIPLE

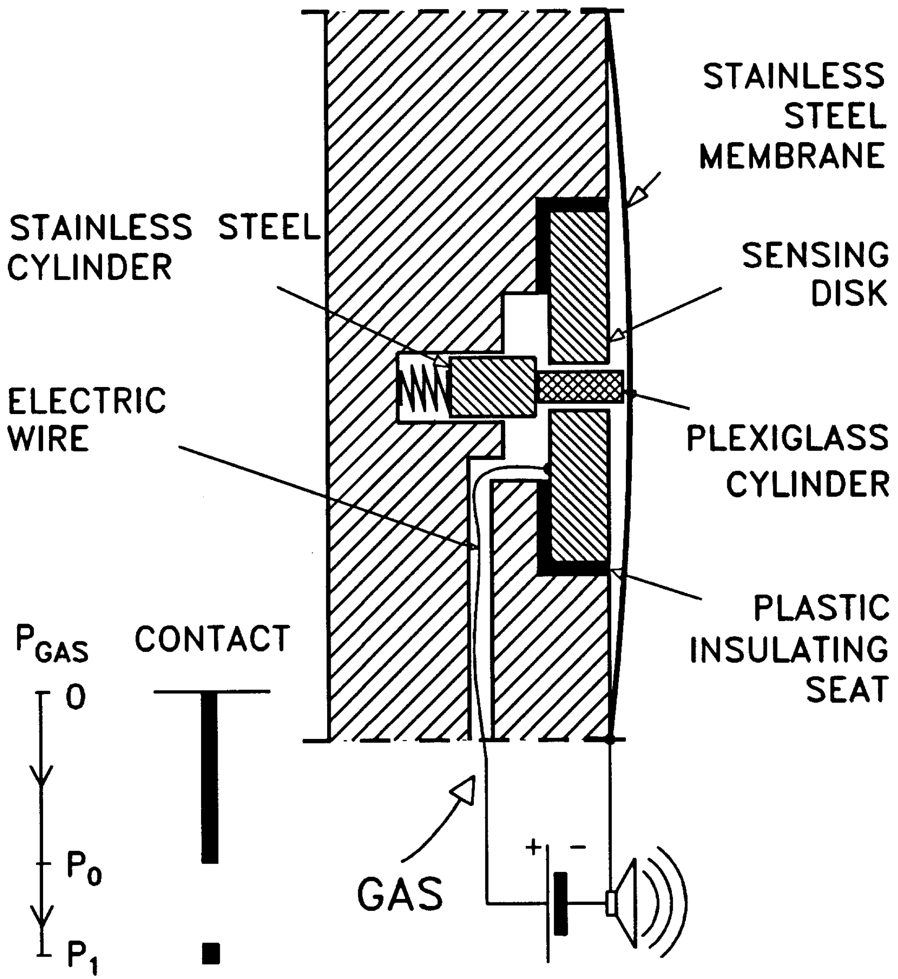


Fig. 7

In essence:

Is a mechanical switch (on-off)

Only mechanical parts – no electronics

Displacement 1.10 mm fixed by construction

Operator cannot regulate

BASIC (ASCE 1980) REDUCTION FORMULAE

Table 1

A, B → p_o and p_1

p_0 and p_1	**p_0**	Corrected First Reading	$p_0 = 1.05(A - Z_M + \Delta A) - 0.05(B - Z_M - \Delta B)$
	p_1	Corrected Second Reading	$p_1 = B - Z_M - \Delta B$

A = Lift off pressure

B = Pressure to expand 1.1 mm

ΔA and ΔB = Membrane correction factors

Z_M = gage zero offset (when vented to atmospheric pressure)

BASIC (ASCE 1980) REDUCTION FORMULAE

Table 2 p_o, p_1 → Id, Kd, Ed

p_0 and p_1	p_0	Corrected First Reading	$p_0 = 1.05(A - Z_M + \Delta A) - 0.05(B - Z_M - \Delta B)$
	p_1	Corrected Second Reading	$p_1 = B - Z_M - \Delta B$
Inter-mediate parameters	I_D	Material Index	$I_D = (p_1 - p_0) / (p_0 - u_0)$
	K_D	Horizontal Stress Index	$K_D = (p_0 - u_0) / \sigma'_{vo}$
	E_D	Dilatometer Modulus	$E_D = 34.7 (p_1 - p_0)$

SOILS that can be TESTED by DMT

- SAND, SILT, CLAY But can cross through GRAVEL layers ≈ 0.5 m

- Clays : Cu = 2-4 kPa to Cu = 10 bar (MARLS)

- Moduli : up to 400 MPa

- Not just soft soils. LIMIT is push capacity (blade 25 tons). Trucks 20 ton DMT fast & easily in hard soils.

REPRODUCIBILITY of DMT

NC clay Onsoy, Norway

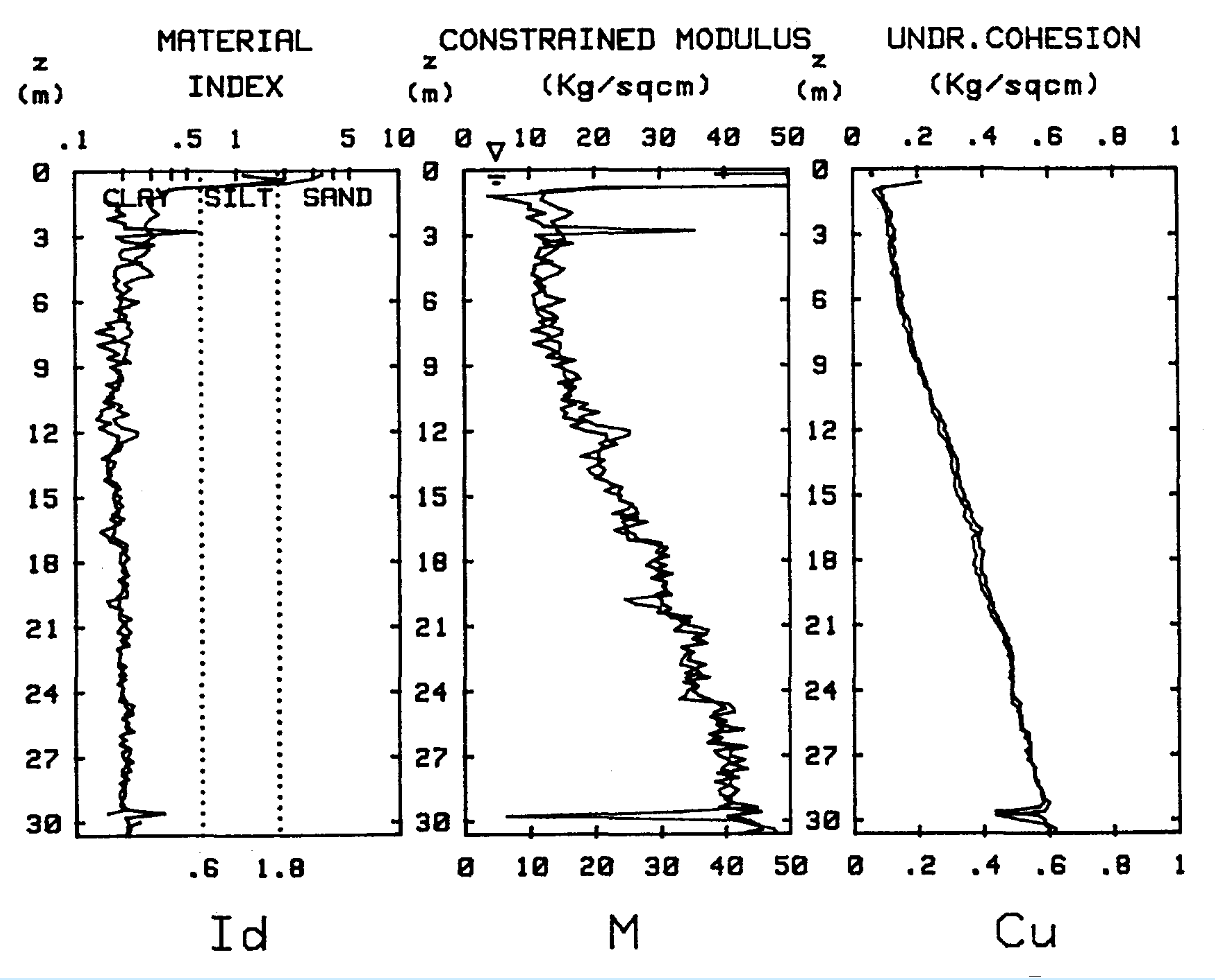


Fig. 8 Cestari (SGI), Lacasse (NGI), Lunne (NGI), Marchetti (Aq) (1980)

BASIC (ASCE 1980) REDUCTION FORMULAE

Table 3 A, B → Id, Kd, Ed → Soil parameters (M, Cu …)

p_0 and p_1	p_0	Corrected First Reading	$p_0 = 1.05(A - Z_M + \Delta A) - 0.05(B - Z_M - \Delta B)$
	p_1	Corrected Second Reading	$p_1 = B - Z_M - \Delta B$
Intermediate parameters	I_D	Material Index	$I_D = (p_1 - p_0) / (p_0 - u_0)$
	K_D	Horizontal Stress Index	$K_D = (p_0 - u_0) / \sigma'_{vo}$
	E_D	Dilatometer Modulus	$E_D = 34.7\,(p_1 - p_0)$
Interpreted parameters	K_0	Coeff. Earth Pressure in Situ	$K_{0,DMT} = (K_D / 1.5)^{0.47} - 0.6$
	OCR	Overconsolidation Ratio	$OCR_{DMT} = (0.5\,K_D)^{1.56}$
	Cu	Undrained Shear Strength	$Cu_{DMT} = 0.22\,\sigma'_{vo}\,(0.5\,K_D)^{1.25}$
	φ	Friction Angle	$\varphi_{safe,DMT} = 28 + 14.6 \log K_D - 2.1 \log^2 K_D$
	ch	Coefficient of Consolidation	$C_{h,DMTA} \approx 7\text{cm}^2 / T_{flex}$
	kh	Coefficient of permeability	$k_h = C_h \gamma_w / M_h$ $(M_h \approx K_0 M_{DMT})$
	γ	Unit Weight and Description	(see chart)
	M	Vertical Drained Constrained Modulus	$M_{DMT} = R_M E_D$ if $I_D \le 0.6$ $R_M = 0.14 + 2.36 \log K_D$ if $I_D \ge 3$ $R_M = 0.5 + 2 \log K_D$ if $0.6 < I_D < 3$ $R_M = R_{M,0} + (2.5 - R_{M,0}) \log K_D$ where $R_{M,0} = 0.14 + 0.15(I_D - 0.6)$ If $K_D > 10$ $R_M = 0.32 + 2.18 \log K_D$ If $R_M < 0.85$ set $R_M = 0.85$

Correction factor

Rm=f(Kd,Id)

- Distortion
- Horiz to Vert
- Drained-undrained

BASIC (ASCE 1980) REDUCTION FORMULAE

Table 4 A, B → Id, Kd, Ed → Soil parameters (M, Cu …)

p_0 and p_1	p_0	Corrected First Reading	$p_0 = 1.05(A - Z_M + \Delta A) - 0.05(B - Z_M - \Delta B)$
	p_1	Corrected Second Reading	$p_1 = B - Z_M - \Delta B$
Inter-mediate parameters	I_D	Material Index	$I_D = (p_1 - p_0) / (p_0 - u_0)$
	K_D	Horizontal Stress Index	$K_D = (p_0 - u_0) / \sigma'_{vo}$
	E_D	Dilatometer Modulus	$E_D = 34.7 (p_1 - p_0)$
Interpreted parameters	K_0	Coeff. Earth Pressure in Situ	$K_{0,DMT} = (K_D / 1.5)^{0.47} - 0.6$
	OCR	Overconsolidation Ratio	$OCR_{DMT} = (0.5 K_D)^{1.56}$
	C_u	Undrained Shear Strength	$C_{u,DMT} = 0.22 \sigma'_{vo} (0.5 K_D)^{1.25}$
	φ	Friction Angle	$\varphi_{safe,DMT} = 28 + 14.6 \log K_D - 2.1 \log^2 K_D$
	c_h	Coefficient of Consolidation	$C_{h,DMTA} \approx 7 cm^2 / T_{flex}$
	k_h	Coefficient of permeability	$k_h = C_h \gamma_w / M_h$ $(M_h \approx K_0 M_{DMT})$
	γ	Unit Weight and Description	(see chart)
	M	Vertical Drained Constrained Modulus	$M_{DMT} = R_M E_D$ if $I_D \leq 0.6$ $R_M = 0.14 + 2.36 \log K_D$ if $I_D \geq 3$ $R_M = 0.5 + 2 \log K_D$ if $0.6 < I_D < 3$ $R_M = R_{M,0} + (2.5 - R_{M,0}) \log K_D$ where $R_{M,0} = 0.14 + 0.15 (I_D - 0.6)$ If $K_D > 10$ $R_M = 0.32 + 2.18 \log K_D$ If $R_M < 0.85$ set $R_M = 0.85$
	U_0	Equilibrium pore pressure	$U_0 = p_2 \approx C - Z_M + \Delta A$

Correction factor

$R_m = f(K_d, I_d)$

- Distortion
- Horiz to Vert
- Drained -undrained

C = closing pressure

PRESENTATION of DMT RESULTS

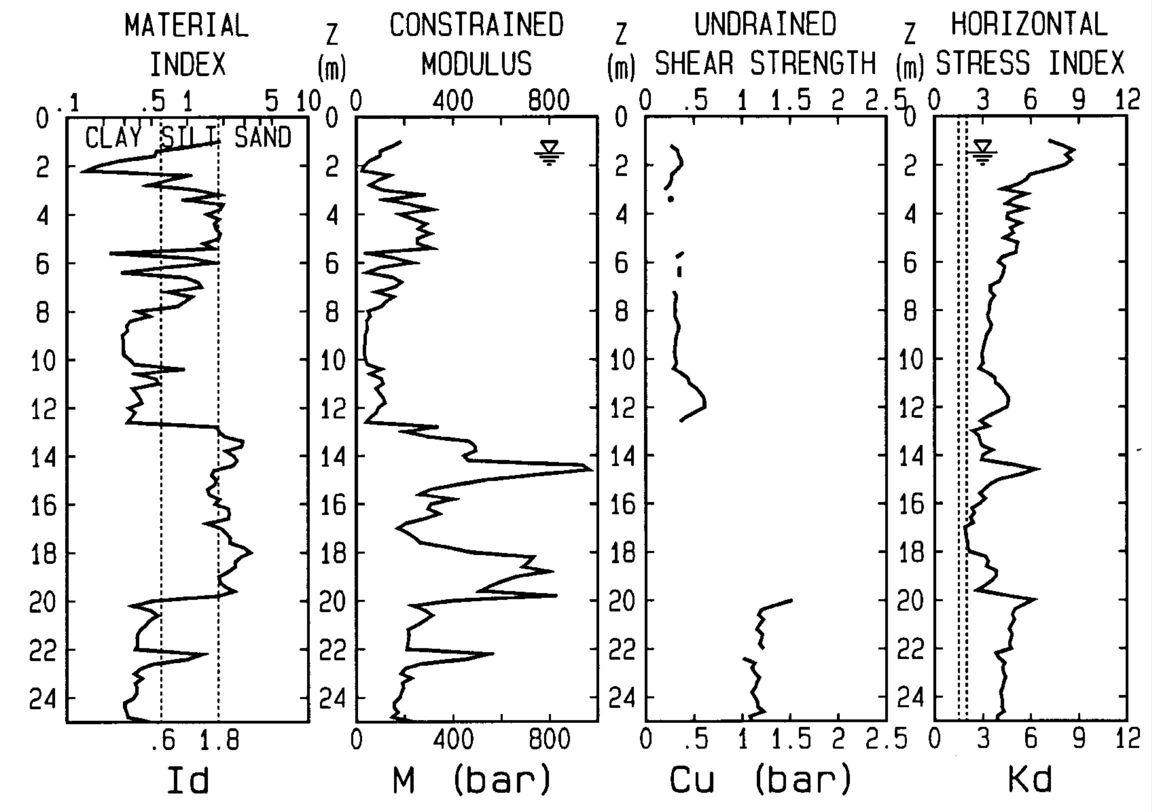


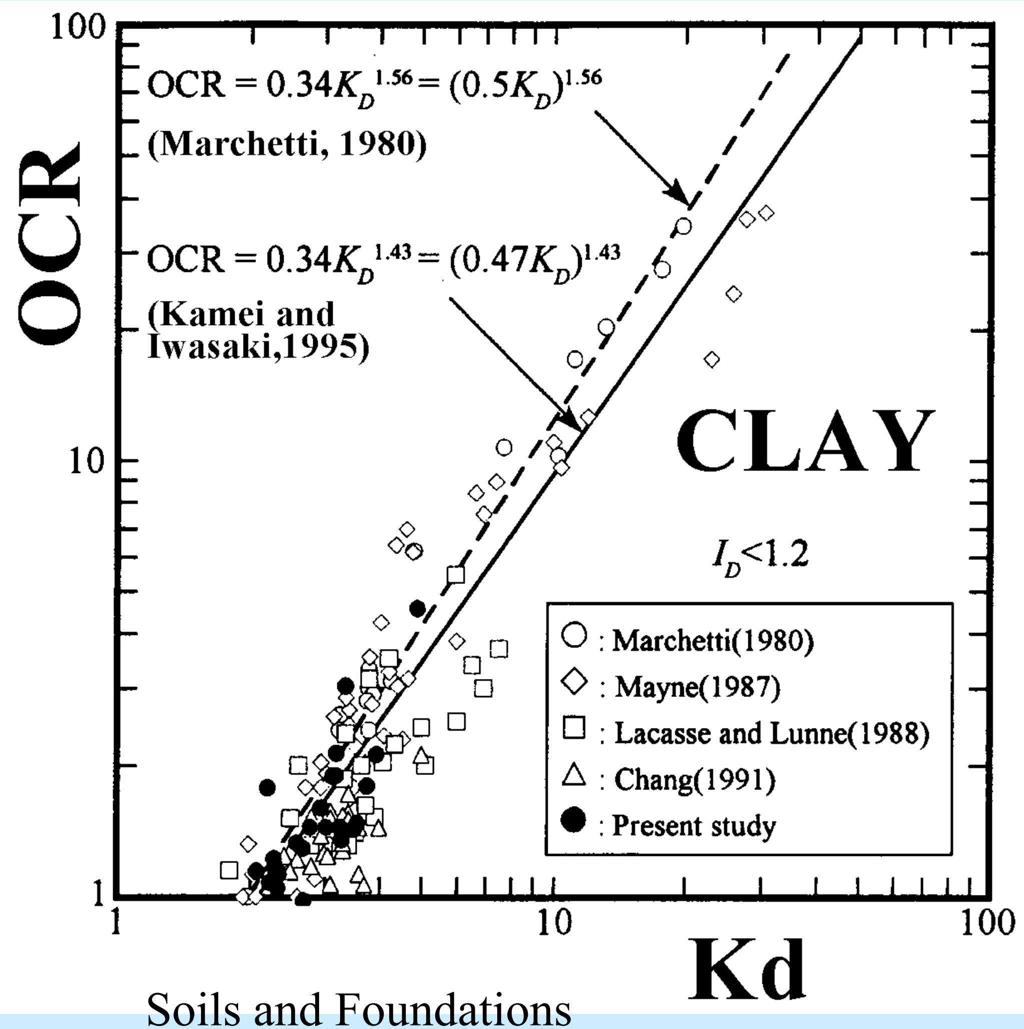
Fig. 9

HOW TO USE DMT RESULTS

- **M and Cu : common, usual way**
- **Id : soil type (sand, silt, clay)**
- **Kd <u>similar</u> shape OCR (useful to *understand* history of deposit).**

<u>NOTE</u> :
Kd = 2 →
OCR ≈ 1

K_d strongly related to OCR



Soils and Foundations
June 1995

K_d = Horizontal Stress Index
or
"Stress History Index"

K_d reflects stress history (overconsolidation,aging, cementation, prestraining ...)

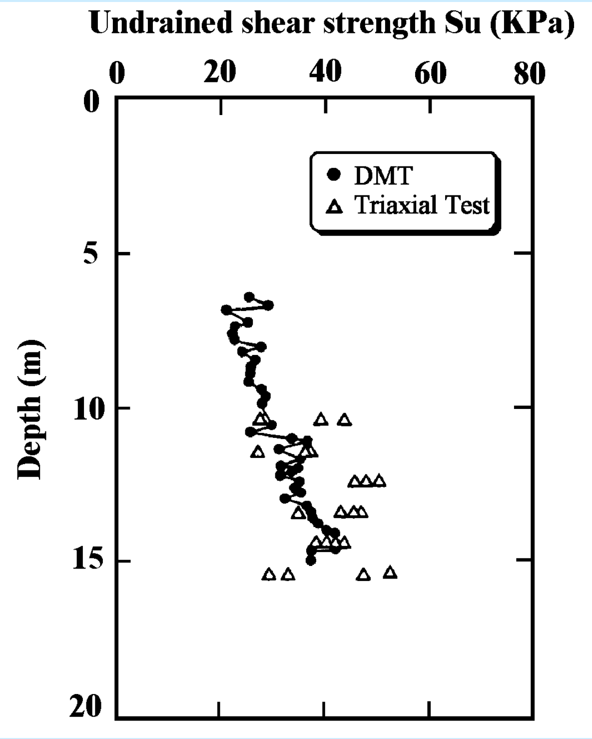
Fig. 10

c_u validated in many research national sites worldwide. Mostly good agreement

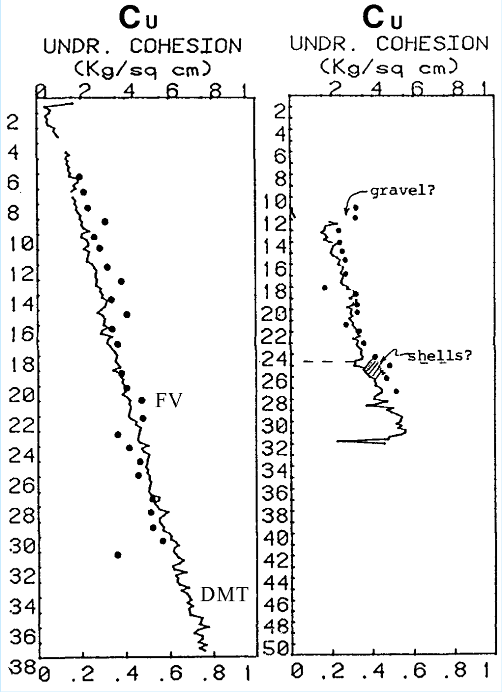
Tokyo Bay Clay

Skeena Ontario Canada

Bothkennar UK



Vs Field Vane



Cu

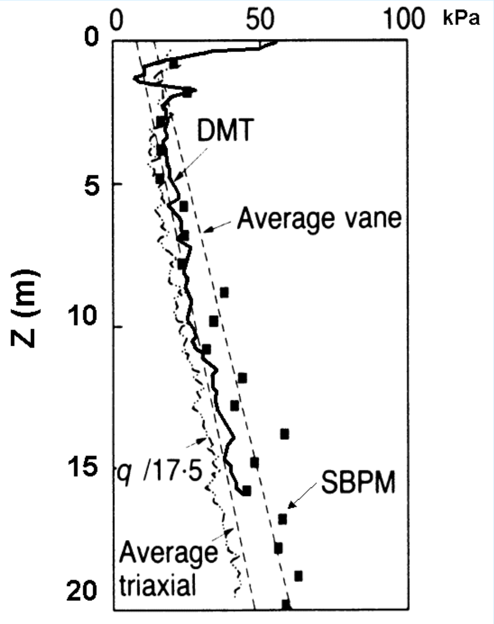


Fig. 11

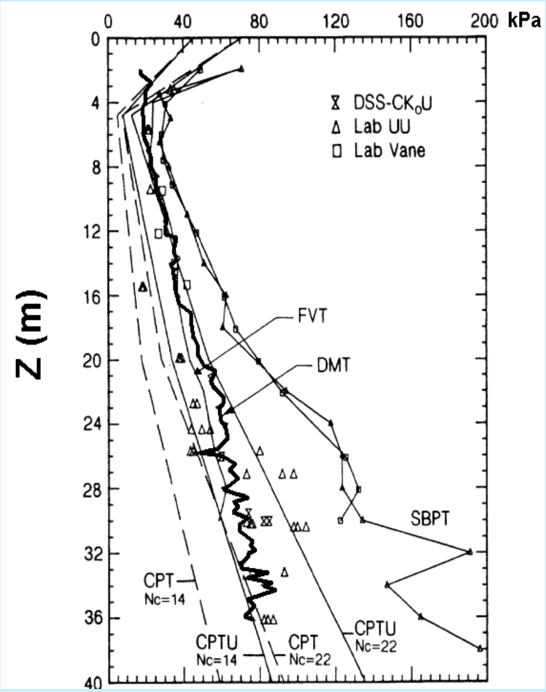
… continued Cu

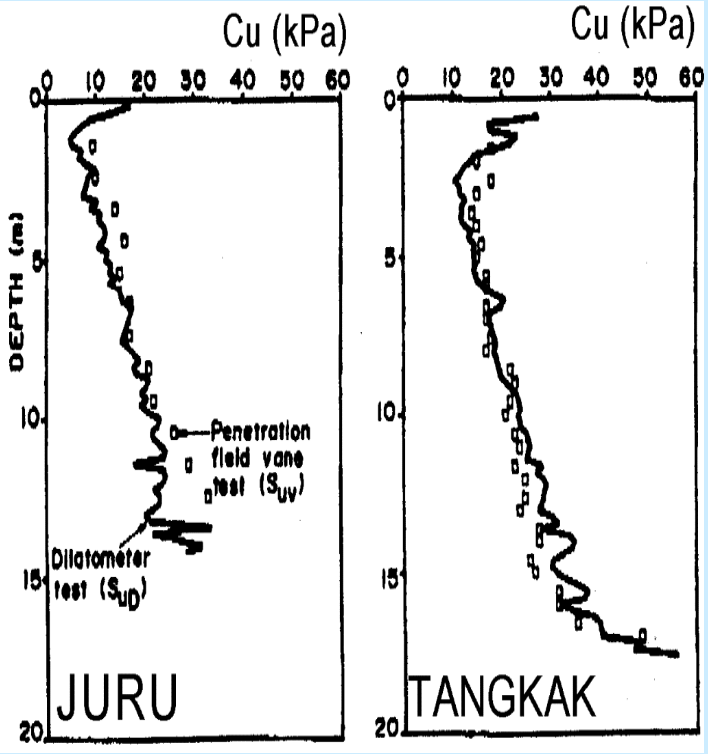
Fucino Italy

2 Malaysian Clays

Recife clay Brazil

Cu





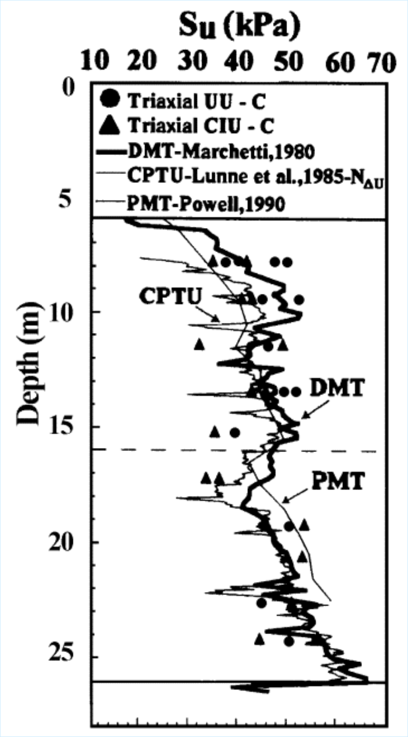
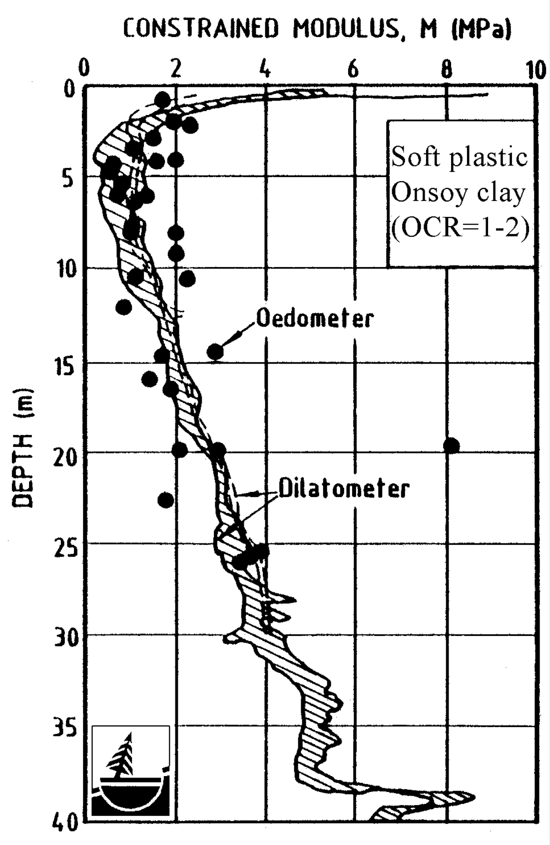


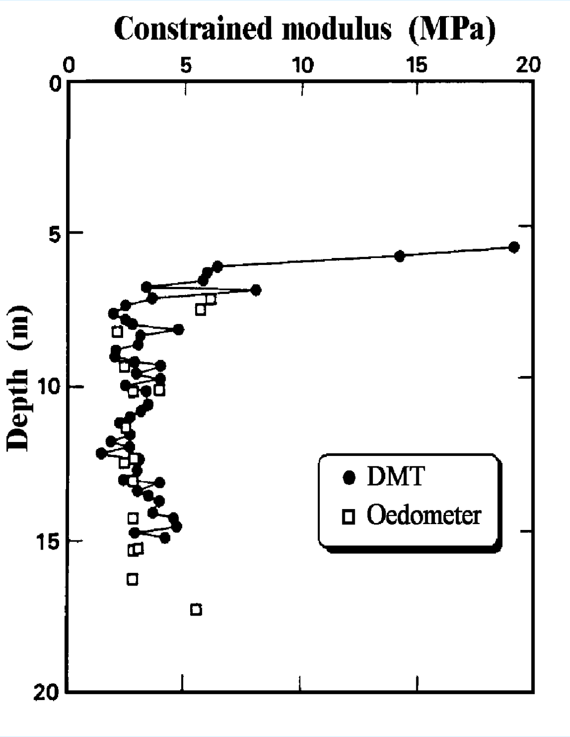
Fig. 12

M validations – similar good agreement

Onsoy Clay Norway



Tokyo Bay Clay



Bangkok Clay

M (kPa)

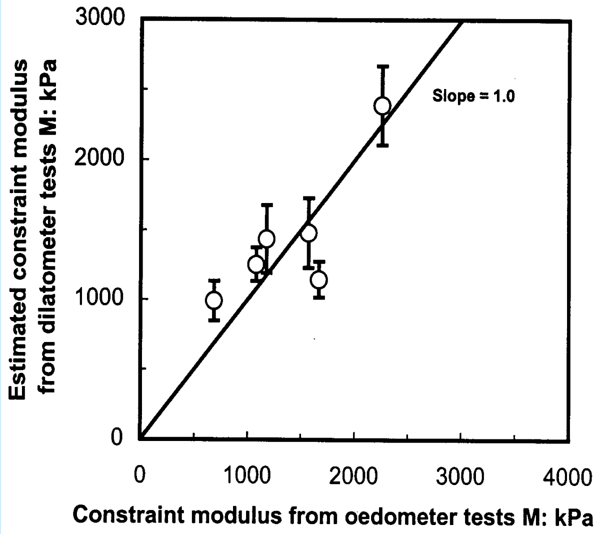
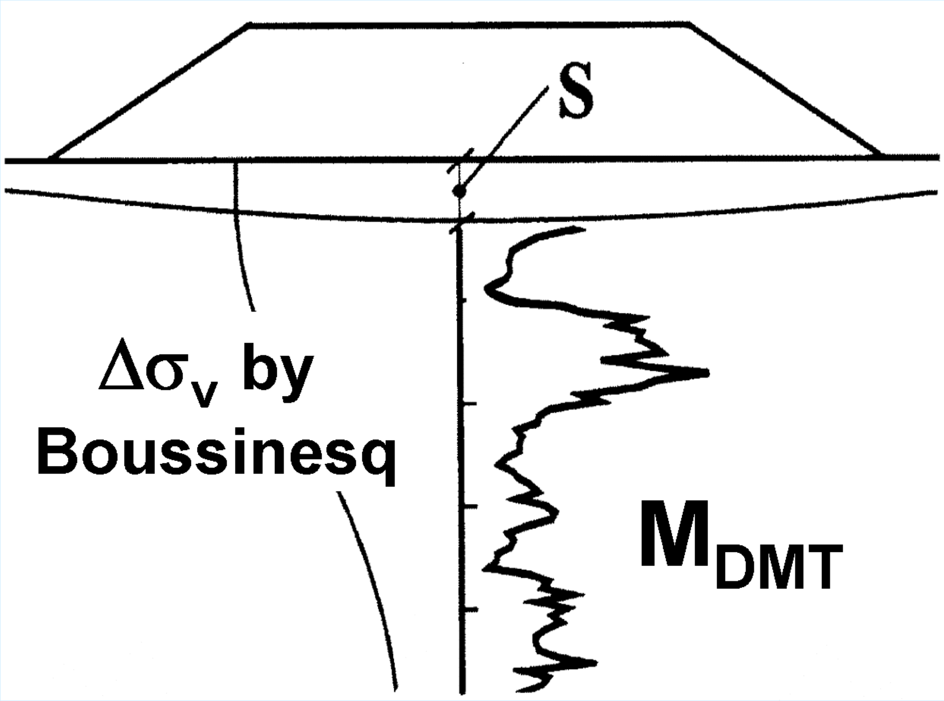


Fig. 13

APPLICATION N° 1 SETTLEMENTS



Generally used method

$$S = \sum \frac{\Delta\sigma_v}{M} \cdot \Delta Z$$

Fig. 14

DMT-calculated vs observed SETTLEMENTS

Table 5 **SCHMERTMANN, 1986 - 16 CASE-HISTORY**

Proc. In Situ '86 ASCE Spec. Conf. VIP, Blacksburg, p.303.

No	Location	Structure	Compressible soil	Settlement (mm) DMT	**	meas	Ratio DMT/ meas.
1	Tampa	Bridge pier	HOC Clay	*25	b,d	15	1.67
2	Jacksonville	Power Plant	Compacted sand	*15	b,o	14	1.07 (ave.3)
3	Lynn Haven	Factory	Peaty sd.	188	a	185	1.02
4	British Columbia	Test embankment	Peat org. sd.	2030	a	2850	0.71
5a b c	Fredricton " "	Surcharge 3' plate building	Sand Sand Quick cl. Silt	*11 *22 *78	a a a	15 28 35	0.73 0.79 2.23
6a b	Ontario "	Road embankment building	Peat Peat	*300 *262	a,o a,o	275 270	1.09 0.97
7	Miami	4' plate	Peat	93	b	71	1.31
8a b	Peterborough "	Apt. bldg Factory	Sd. & si.	*58 *20	a, o a, o	48 17	1.21 1.18
9	"	Water tank	Si. clay	*30	b,o	31	0.97
10a b	Linkoping "	2x3 m plate 1.1x1.3m plate	Si. sand Si. sand	*9 *4	a,o a,o	6.7 3	1.34 1.33
11	Sunne	House	Silt & sand	*10	b,o	8	1.25

Typical range of settlement prediction: 70% - 150%

DMT-CALCULATED vs OBSERVED. Ave : 1.18

Accuracy of Settlement predictions

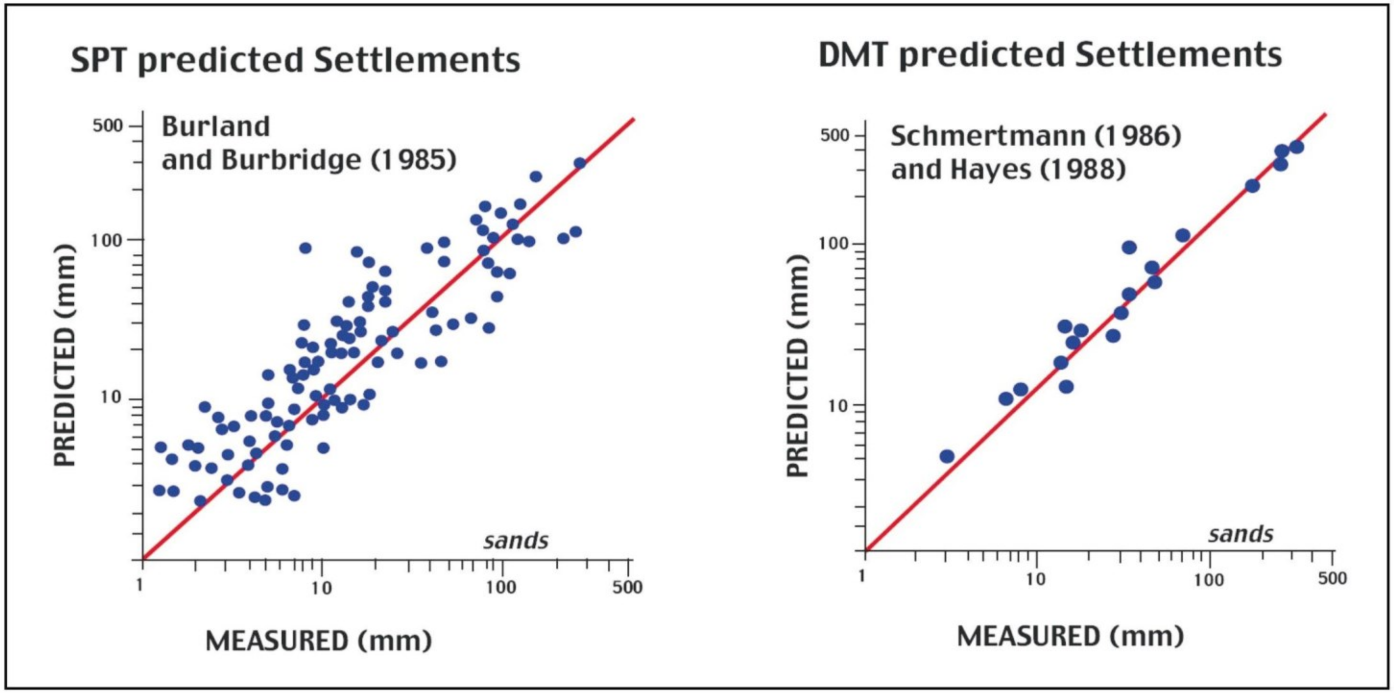


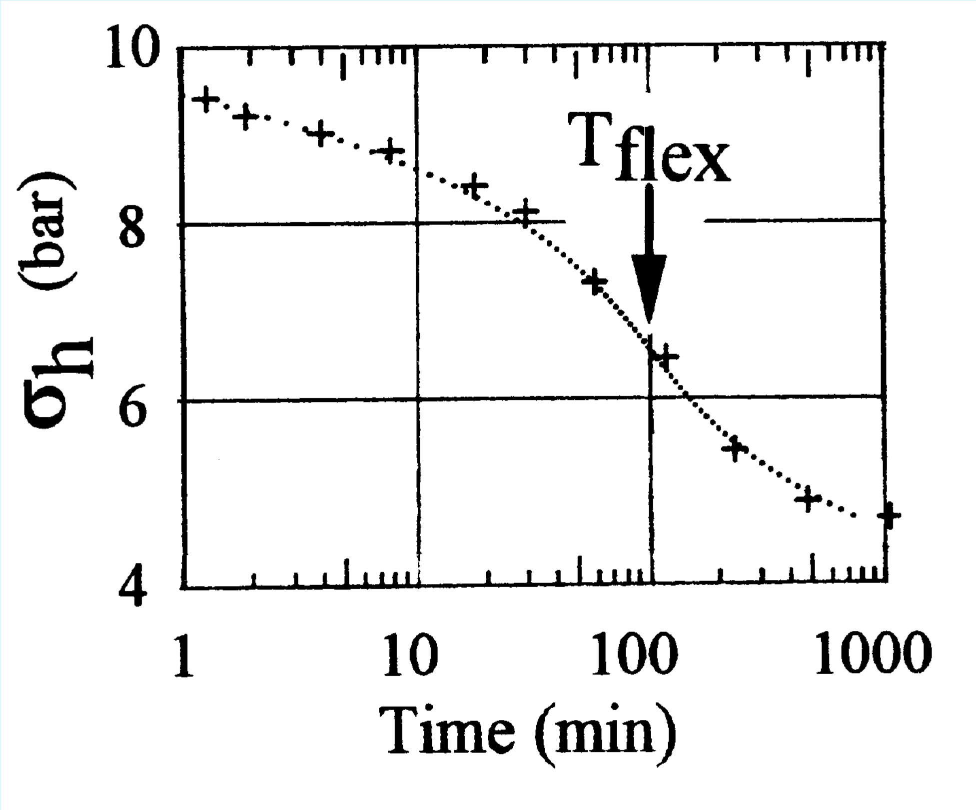
Fig. 15 Bullock & Failmezger (Porto 2004)

Possible reasons higher accuracy DMT :

1. **Availability of Stress History parameter Kd**
2. **Wedges deform soil << than cones**
3. **Modulus by *mini load test* relates better to modulus than penetr. resistance**

Coefficient of consolidation/permeability from Tflex

Fig. 16 **Stop Penetration and Monitor σh Decay**



$$C_h \cong \frac{7cm^2}{T_{flex}} \qquad k = \frac{C \cdot \gamma_w}{M}$$

DMT BEST APPLICATIONS

- M and Cu profiles
- Estimating settlements, deformation
- Monitoring soil improvement
- Recognize soil type
- Verify if a clay slope contains active/old slip surfaces

Useful information also on:

- OCR and Ko in clay
- Coefficient of consolidation/permeability
- P-y curves for laterally loaded piles
- Sand liquefiability
- Friction angle in sand
- (Some info OCR and Ko in sand)

Marchetti, 2007

Presentation of DMT Results

Material Index:

$$I_D = (p_1 - p_o)/(p_o - u_o)$$

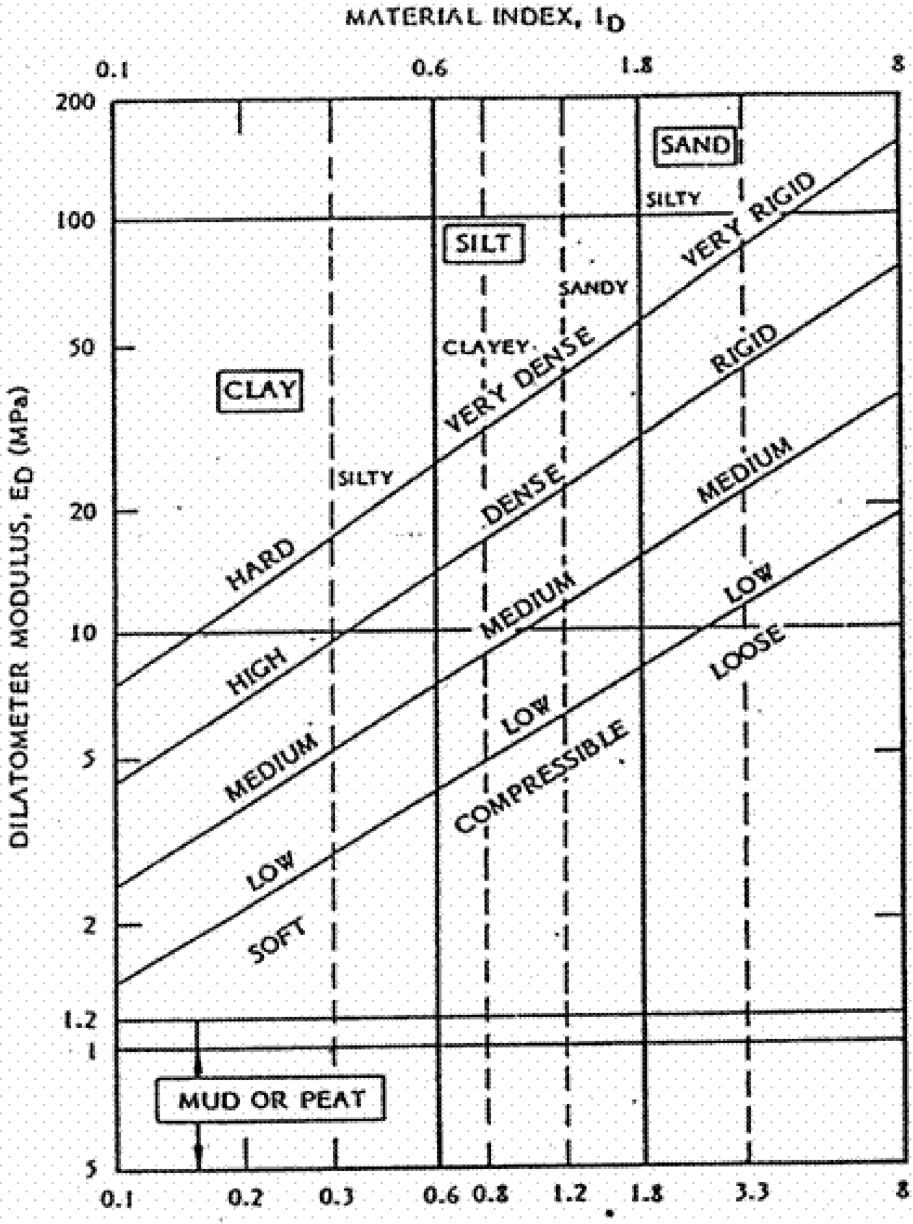
Horizontal Stress Index:

$$K_D = (p_o - u_o)/\sigma'_{vo}$$

Dilatometer Modulus:

$$E_D = 34.7\,(p_1 - p_o)$$

where p_o and p_1 are the measured pressures that correspond to lift-off and 1.10mm delfection of the membrane on the dilatometer, u_o is the in-situ pore pressure, σ'_{vo} is the effective overburden pressure.



Soil Classification based on DMT

Fig. 25 : Chart for Soil Identification Based on DMT (after Marchetti and Crapps, 1981)

Estimating Horizontal In-situ Stress from DMT

"Clay"

(1) Marchetti (1980) proposed a correlation between K_o and K_D for uncemented natural clays as follows:

$$K_o = (K_D / 1.5)0.47 - 0.6$$

(2) Lunne et al. (1990), however, suggested the following two correlations for "Young" clay ($s_u/\sigma'_{vo} \leq 0.8$) and "Old" clay ($s_u/\sigma'_{vo} \geq 0.8$), respectively, on the basis of high quality data from several research sites:

$K_o = 0.34\, K_D^{0.47}$ **for Young clays**

$K_o = 0.68\, K_D^{0.47}$ **for Old clays**

Estimating Undrained Shear Strength from DMT

- **Marchetti (1980) proposed the following correlation:**

$$s_u/\sigma'_{vo} = 0.22\,(K_D/2)^m$$

where m =1.25 based on investigations in Italy.

- **Chang (1988) found that Marchetti's correlation leads to overestimates of s_u for the Singapore marine clay, but underestimates of the s_u for the Singapore peaty clay.**

- **Bo et al. (2001), indicated that Marchetti's correlation with the exponent 1.25 replaced by 1.0 and 0.7, respectively, provided good estimates of s_u values that are comparable to those from the field vane tests for the upper marine clay and the lower marine clay of Singapore.**

Estimating OCR from DMT

(1) According to Marchetti (1980), K_D correlates strongly with the OCR of soil. For uncemented cohesive soil with simple stress history, the following correlation has been proposed:

$$OCR = (0.5\,K_D)^{1.56}$$

(2) Chang (1991) indicated that Marchett's (1980) equation tended to lead to an over-estimation of OCR for Singapore marine clay and suggested that the exponent 1.56 be replaced by 0.84 for the estimation of OCR in Singapore marine clay, as follows:

$$OCR = (0.5\,K_D)^{0.84}$$

Estimating OCR from DMT

(3) Lacasse and Lunne (1988) suggested the following correlations:

$$\mathbf{OCR = 0.225\, K_D^{\,m}}$$

where m ranged from 1.35 for highly plastic clays to 1.67 for clays of low plasticity.

(4) Powell and Uglow (1988) found that Marchetti's (1980) correlation overpredicted the OCR for clay deposits younger than 70000 years and underpredicted the OCR for "Old" clays.

Constrained Modulus from DMT

(1) According to Marchetti (1980), the dilatometer modulus E_D is a measure of the stiffness of the soil after penetration of blade and E_D can be correlated with the drained constrained modulus $M = R_m E_D$ of the soil as follows:

$$R_m = 0.14 + 2.36 \log K_D, \quad \text{for } ID \leq 0.6$$
$$= R_{mo} + (2.5 - Rmo) \log K_D, \quad \text{for } 0.6 \leq ID \leq 3.0$$
$$= 0.5 + 2 \log KD, \quad \text{for } ID \geq 3.0$$

where $R_{mo} = 0.14 + 0.15\,(ID - 0.6)$ and $R_m \geq 0.85$.

(2) Schmertamann (1988) indicated that $E_D = E_{25}'$ (E at 25% of strength mobilization) for normally consolidated (N.C.) uncemented sand.

Coefficient of Consolidation from DMT - Basis

(1) The dilatometer dissipation test involves recording A-reading corresponding to lift-off of the membrane or C-reading that corresponds to the returning of the membrane to the lift-off position with time. The C-reading produces the p_2 pressure after correcting for membrane stiffness.

(2) Campanella and Roberston (1985) by using a research dilatometer and allowing the pressure to increase gradually from 0 to p_o and then to p_1, and then gradually reduced to p_2, observed that as shown in Fig. 5.6.

(3) For the test in sand, the closing pressure p_2 matches the initial in-situ pore pressure. For the test in clay, the p_2 pressure approximately equals to the measured pore pressure. It is, therefore, possible to deduce the coefficient of consolidation with respect to horizontal drainage, c_h from the DMT dissipation record.

Coefficient of Consolidation from DMT

Schmertmann's (Schmertmann, 1988) Method

(1) In this method, C-reading (or the p_2) is plotted against the $\sqrt{t}$ (t = time elapsed) and the time corresponding to 50% consolidation, t_{50} is determined, as illustrated in Fig. 5.7.

(2) Gupta and Davidson's (1986) procedure, developed for piezocone dissipation analysis, was modified and used for the interpretation of c_h, which is defined as c_h(DMTC). The procedure involves estimating rigidity index, E_u/s_u, and pore pressure parameter at failure, A_f, for the clay, and determining T_{50}, from the dissipation curves as shown in Fig. 5.8, for A_f = 0.9. An adjustment will be required if A_f is different from 0.9.

(3) By assuming $R^2 = 600\ mm^2$ for a test involving the standard Marchetti dilatometer, c_h (DMTC) can be calculated from:

$$c_h = 600\left(\frac{T_{50}}{t_{50}}\right)$$

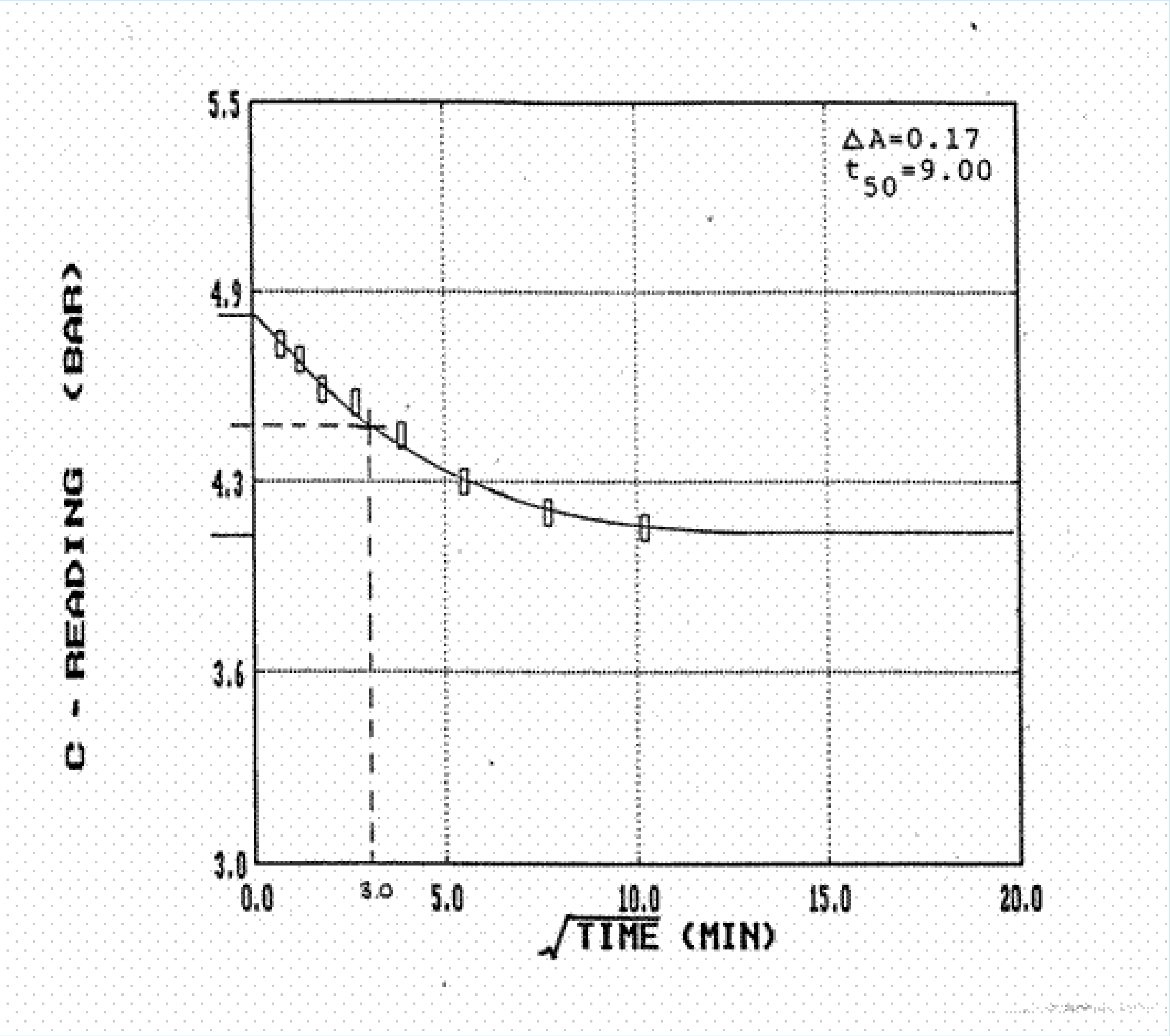


Fig. 26 : Typical Dissipation Curve from DMT C-Dissipation Test

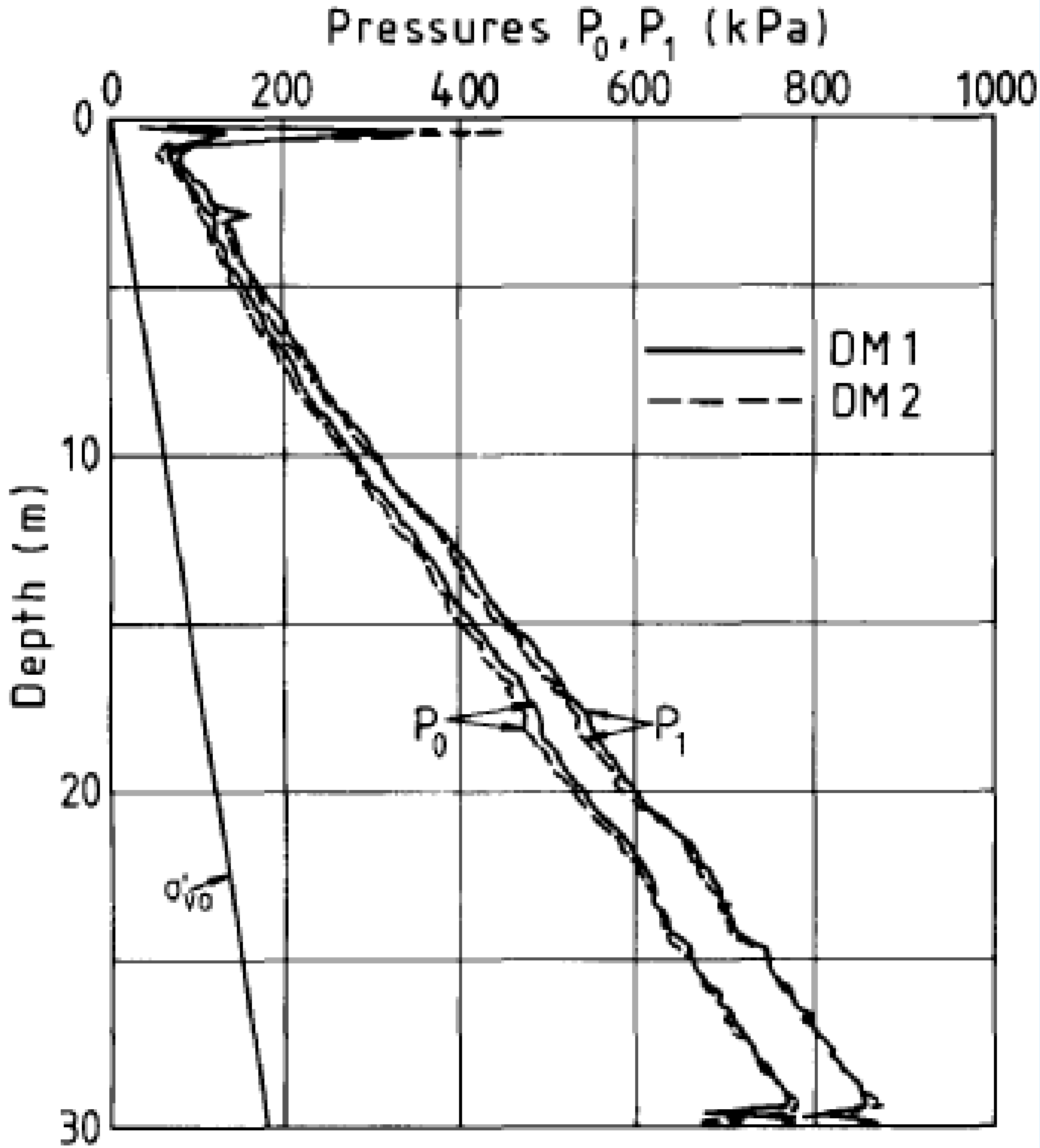


Fig. 27 Contact and expansion pressures in Onsøy. **(Suzanne & Tom, 1983)**

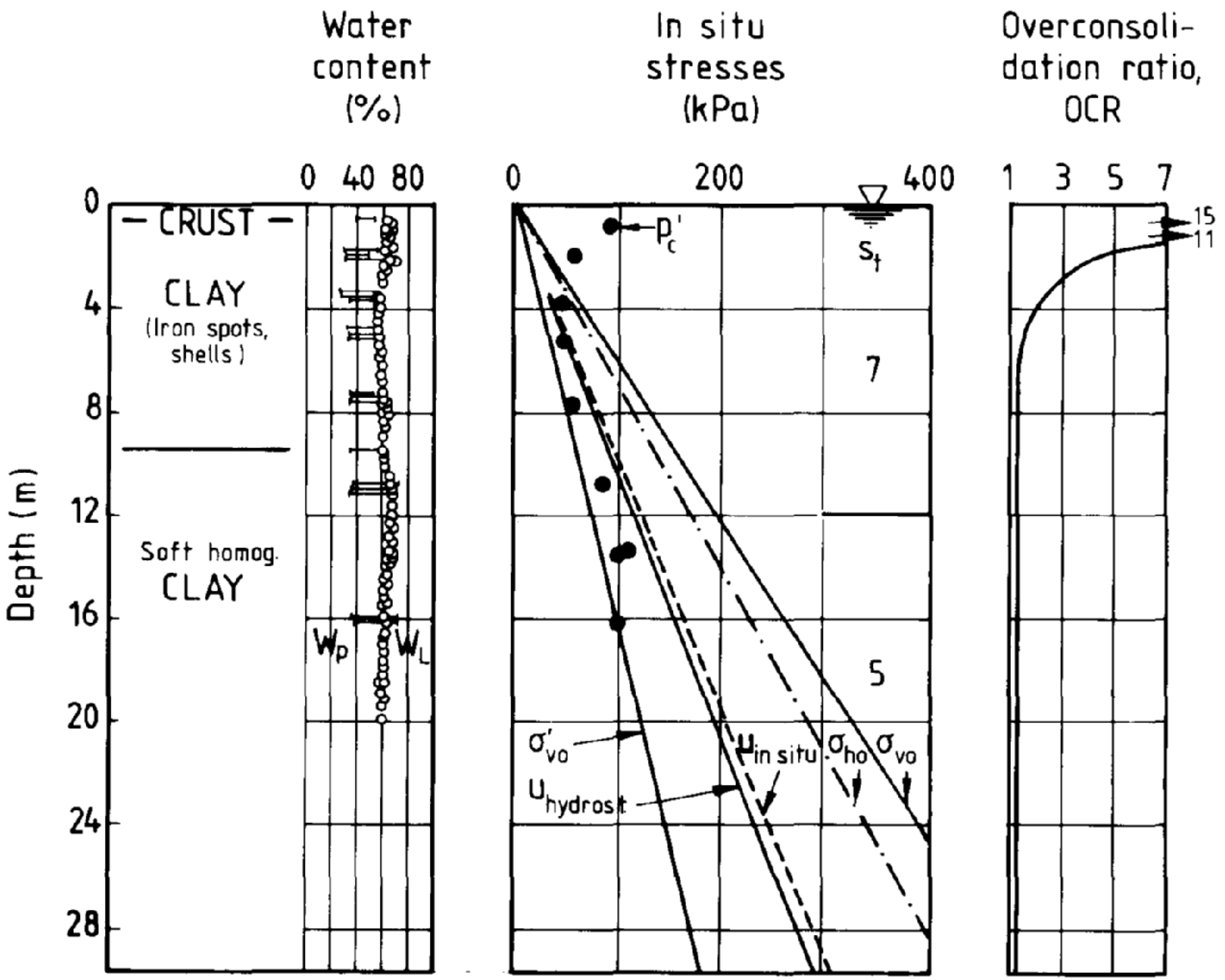


Fig. 28 Onsøy test site. **(Suzanne & Tom, 1983)**

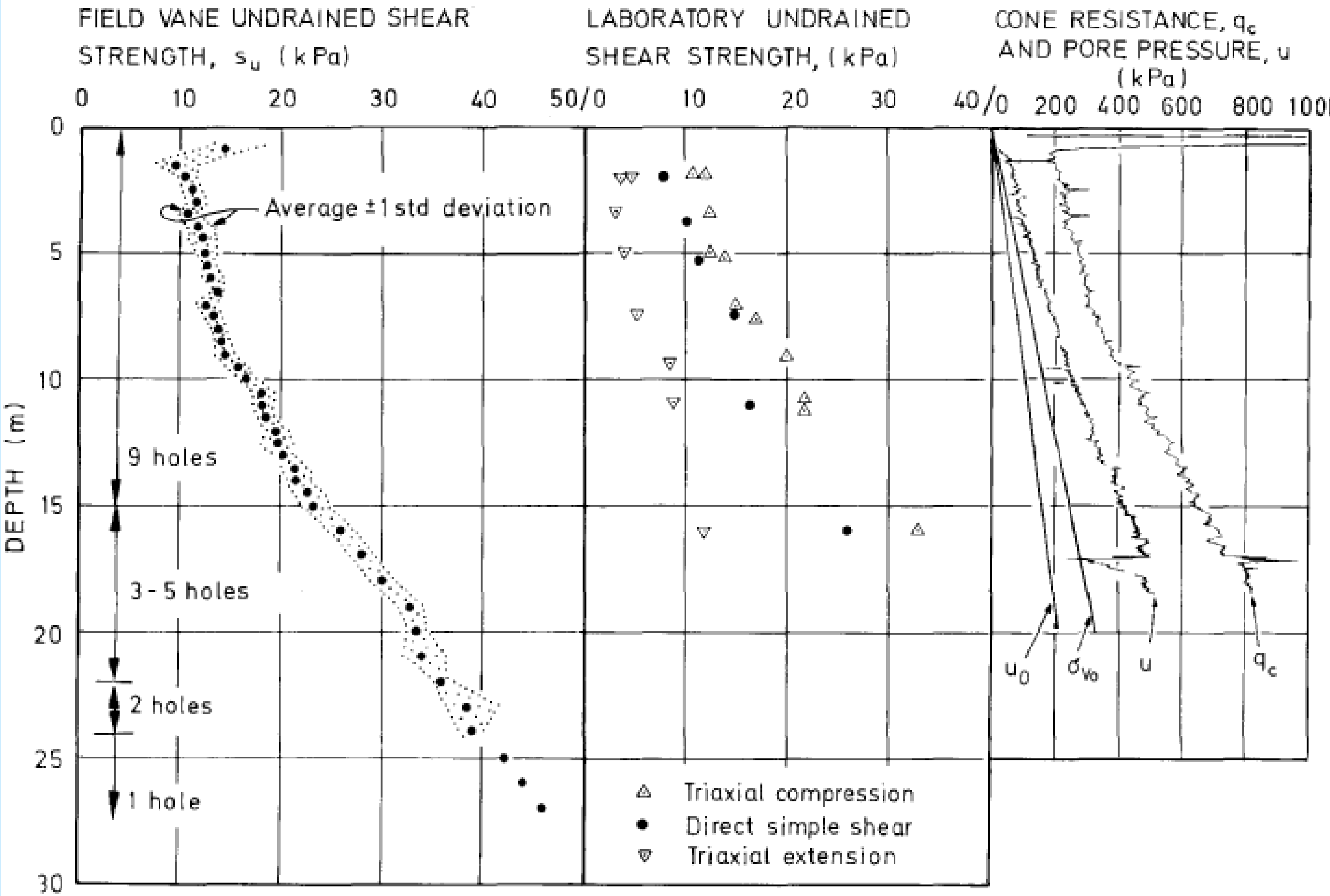


Fig. 29 Undrained shear strength in Onsøy. **(Suzanne & Tom, 1983)**

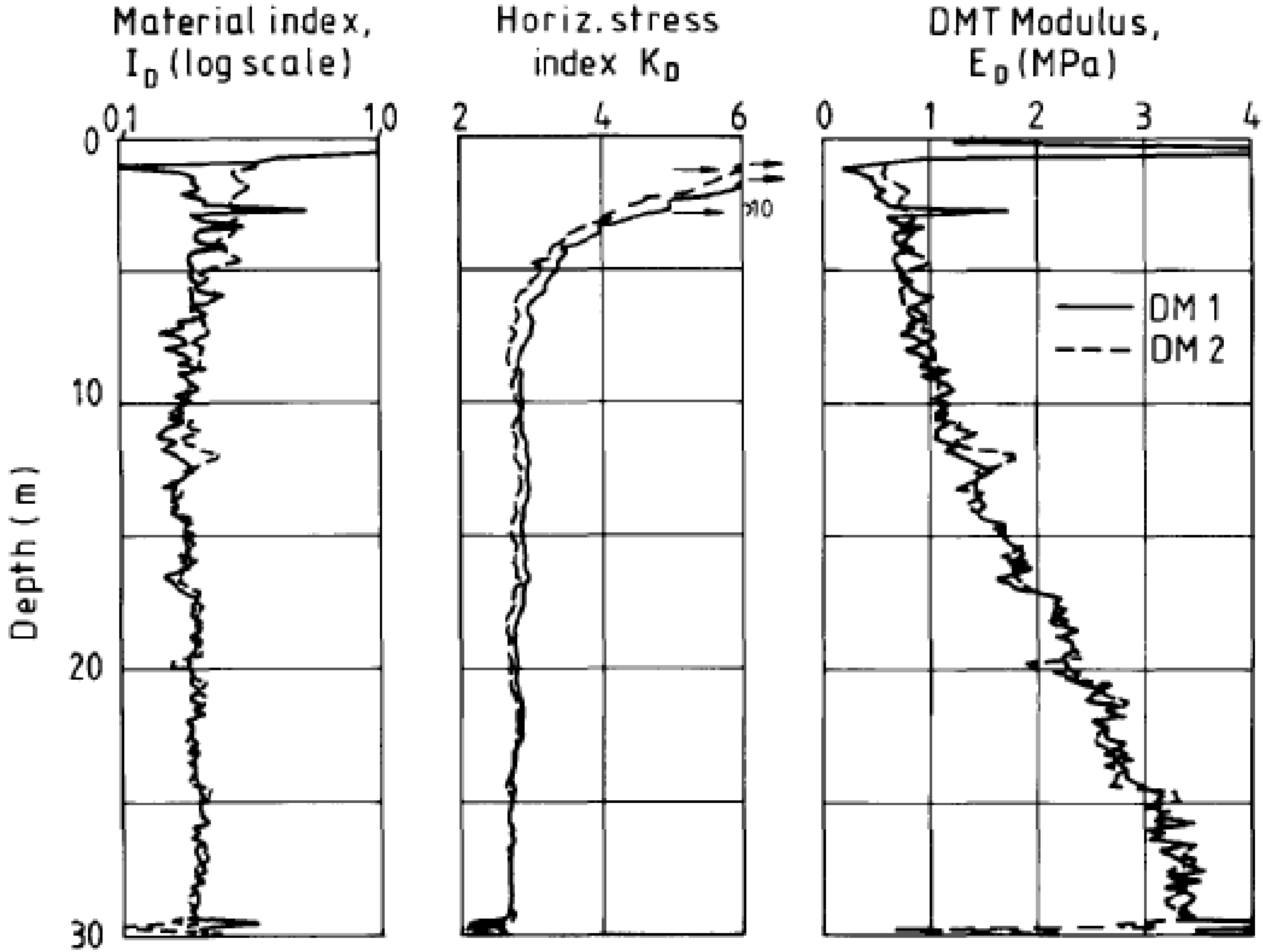


Fig. 30 Dilatometer parameters in Onsøy. **(Suzanne & Tom, 1983)**

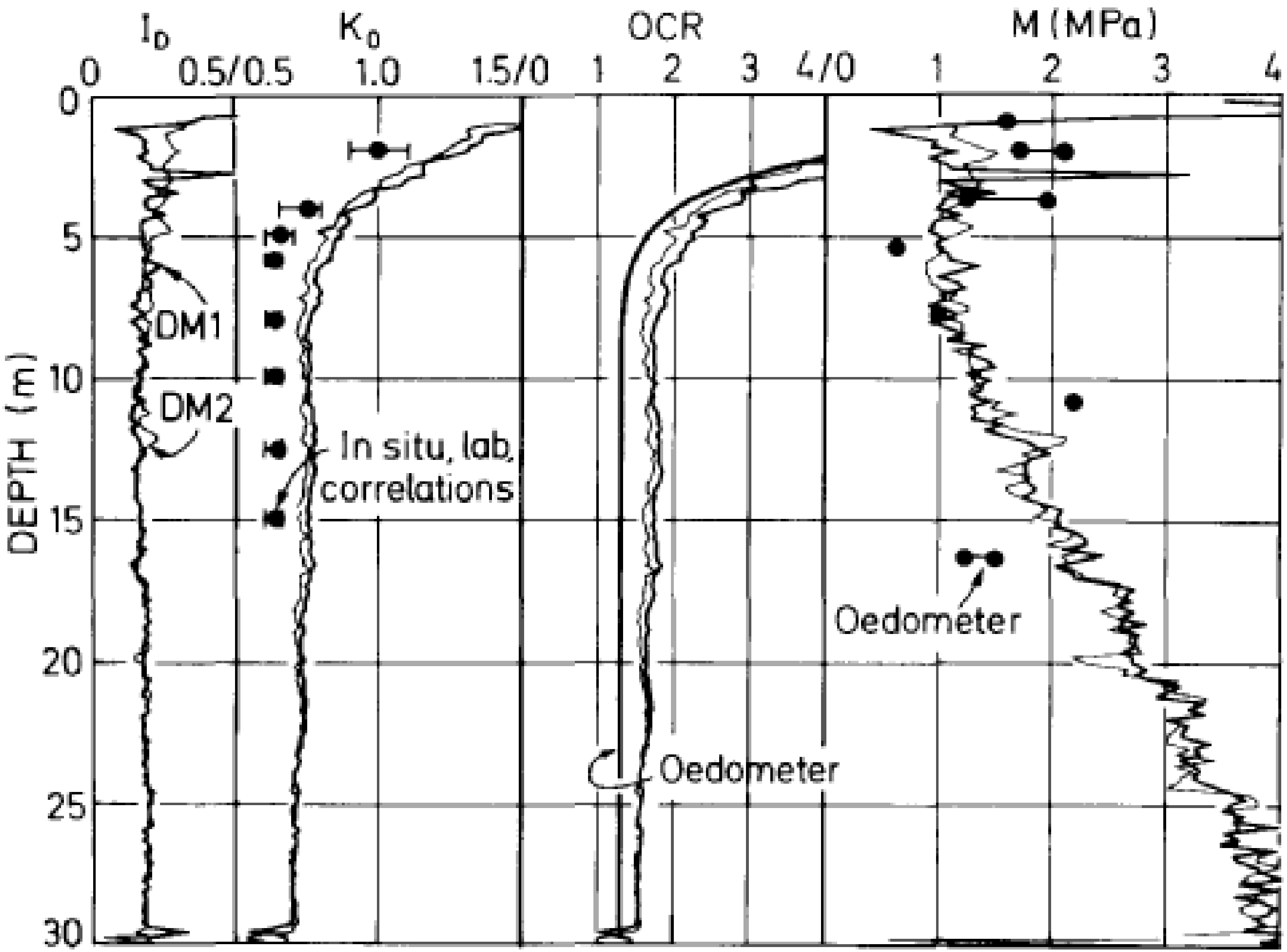


Fig. 31 Soil parameters derived from dilatometer tests in Onsøy.
(Suzanne & Tom, 1983)

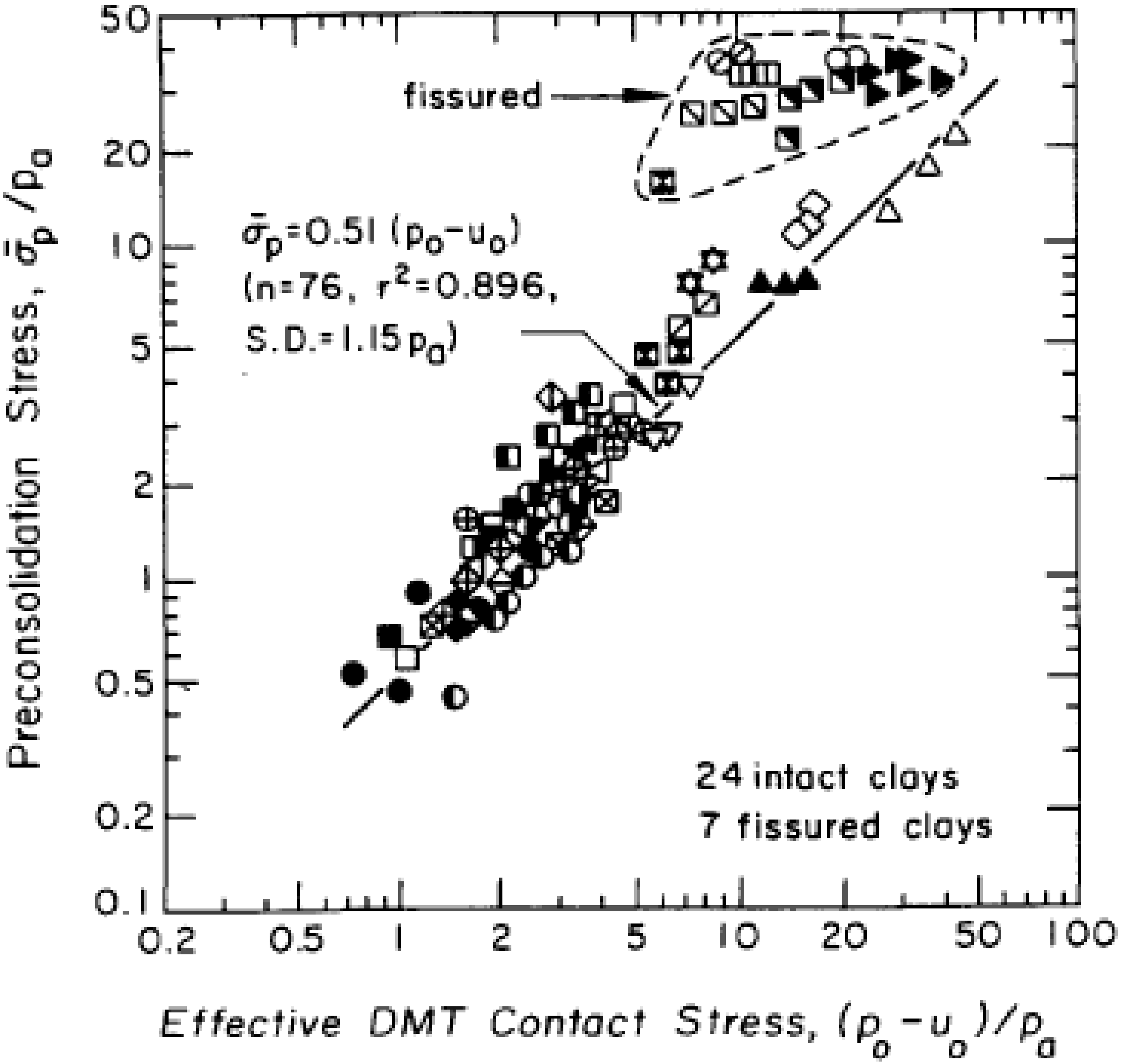


Fig. 32 $\bar{\sigma}_p$ Correlated with DMT p_o

Dilatometer Tests in Two Soft Marine Clays

Suzanne Lacasse and Tom Lunne, 1983

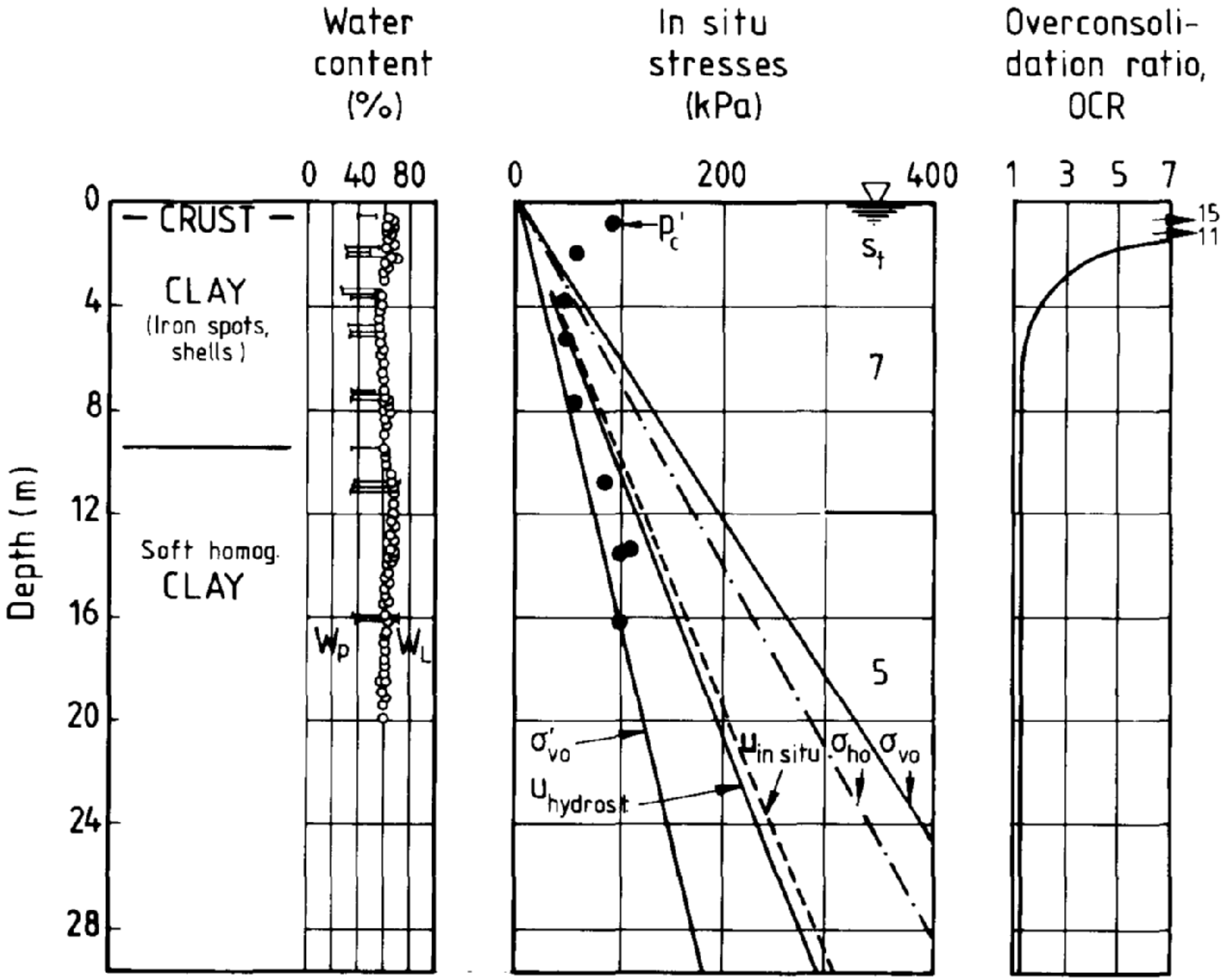


Fig. 1. Onsøy test site. **(Suzanne & Tom, 1983)**

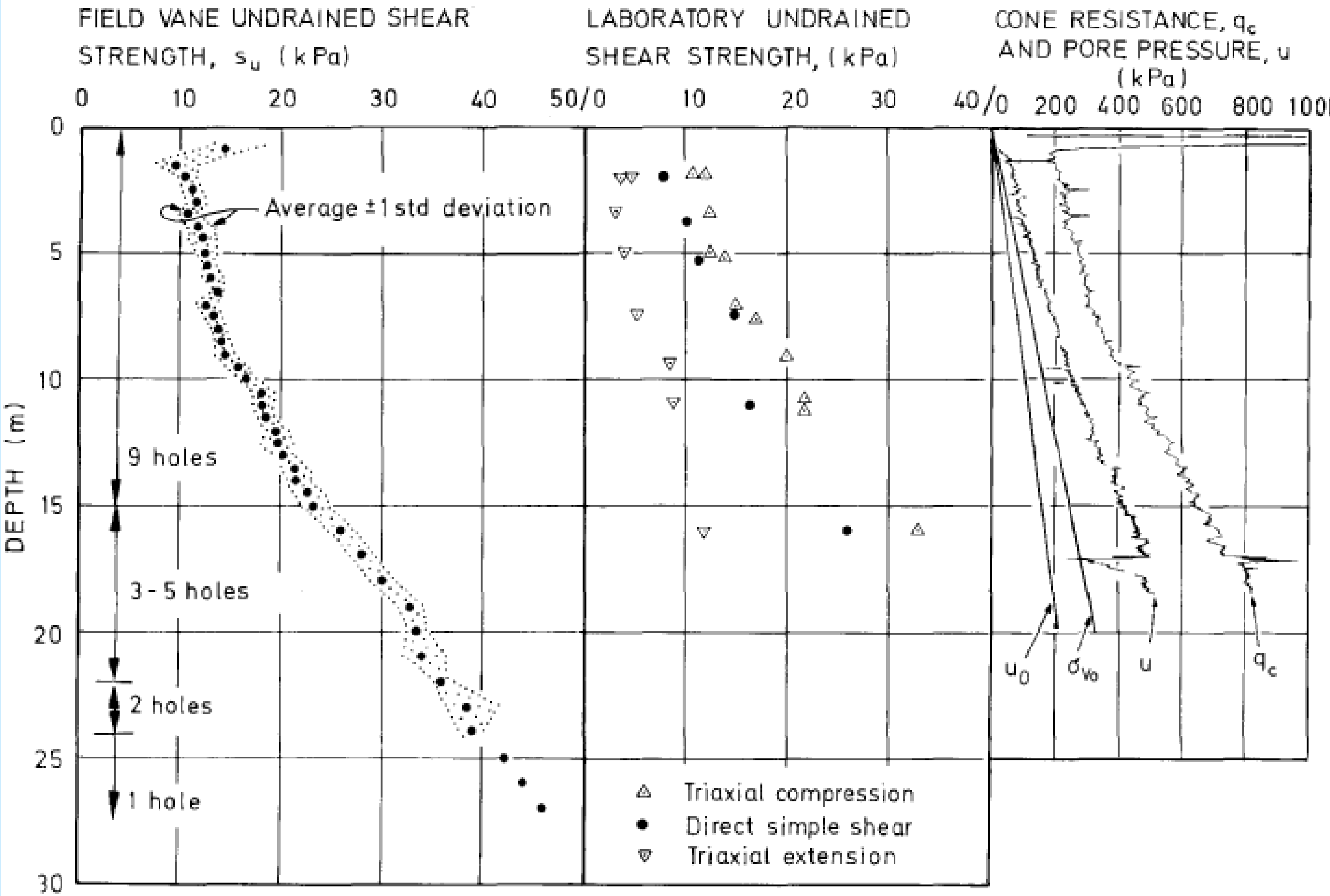


Fig. 2. Undrained shear strength in Onsøy. **(Suzanne & Tom, 1983)**

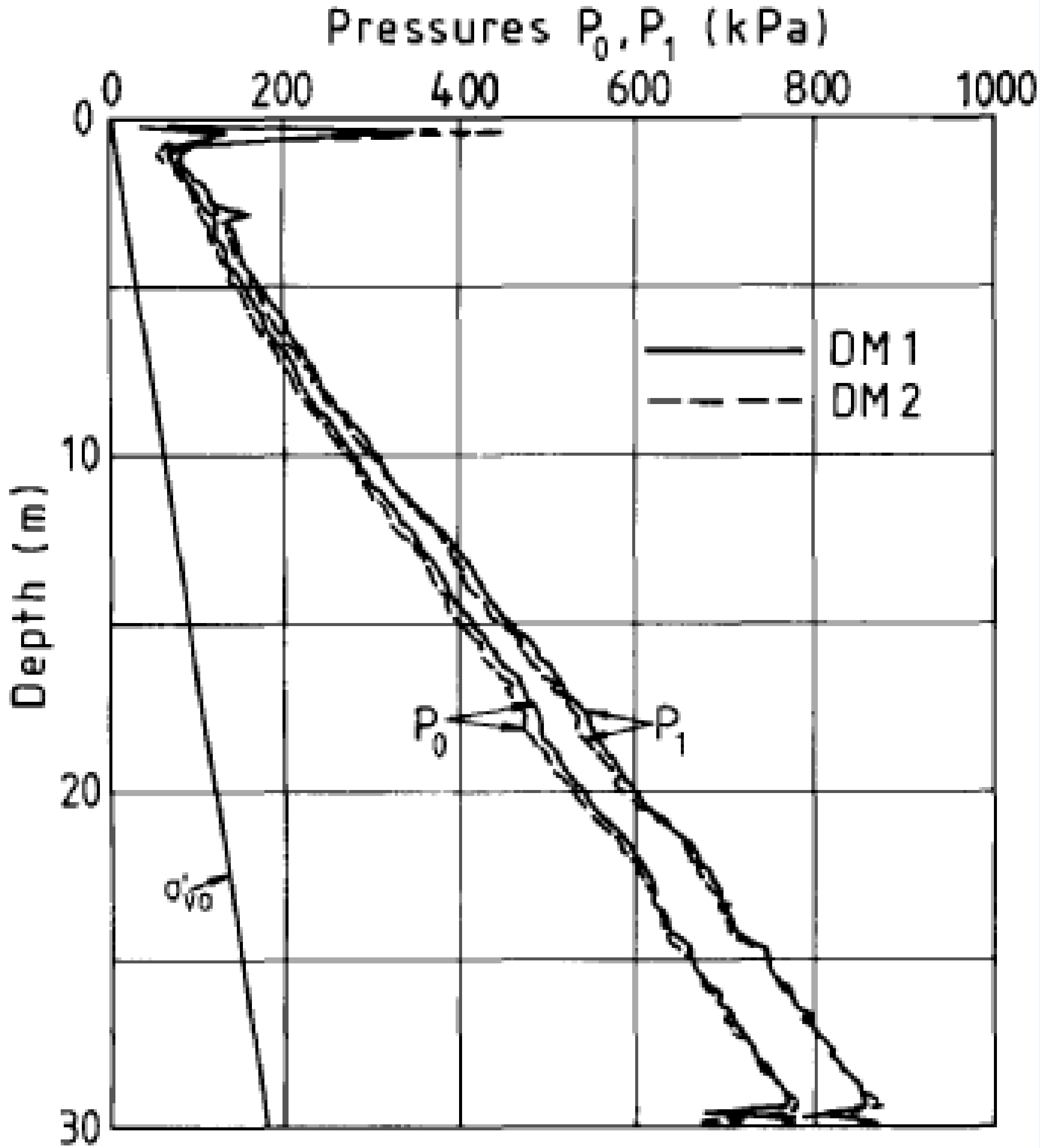


Fig. 3. Contact and expansion pressures in Onsøy. **(Suzanne & Tom, 1983)**

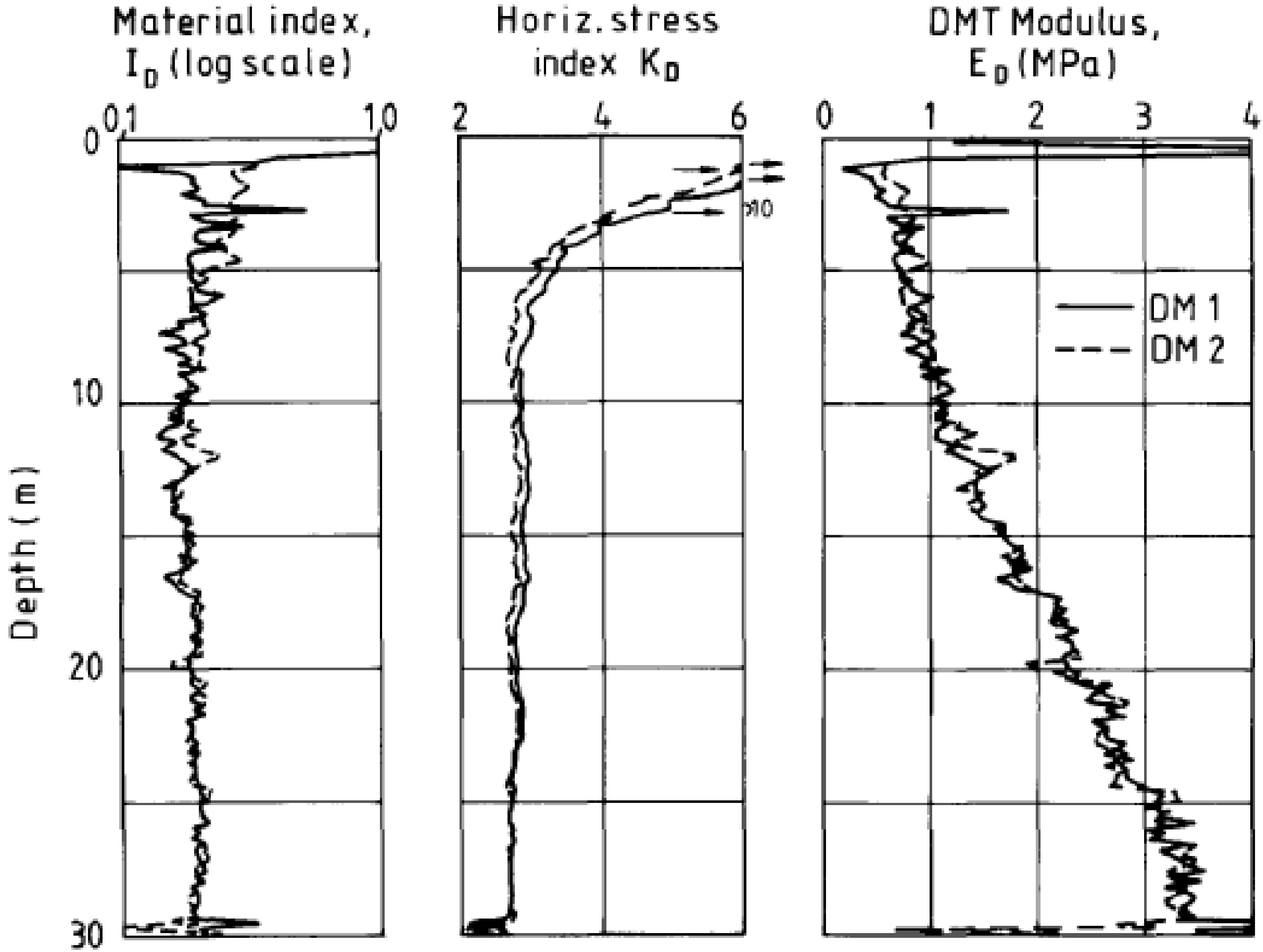


Fig. 4. Dilatometer parameters in Onsøy. **(Suzanne & Tom, 1983)**

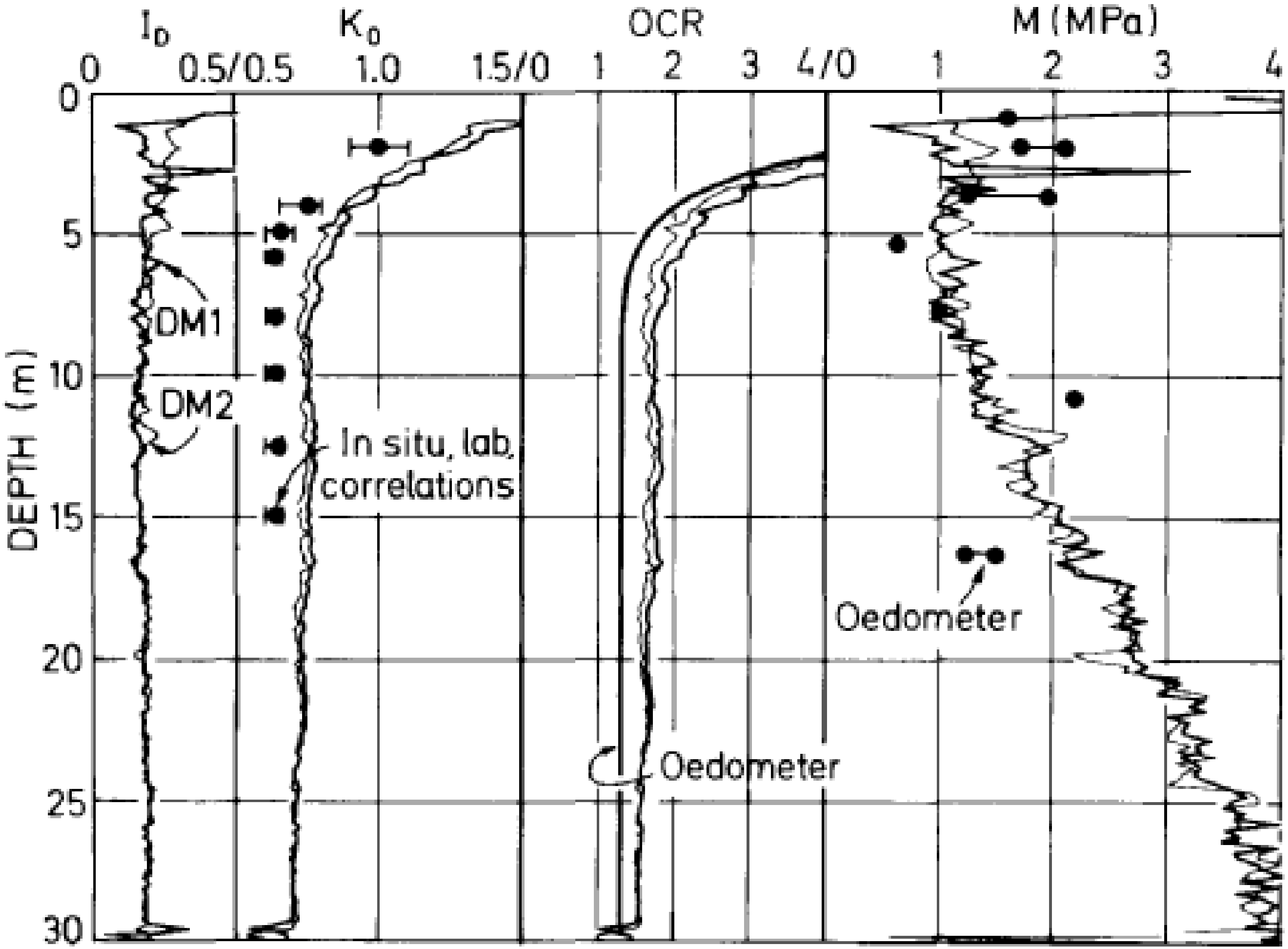


Fig. 5. Soil parameters derived from dilatometer tests in Onsøy.

(Suzanne & Tom, 1983)

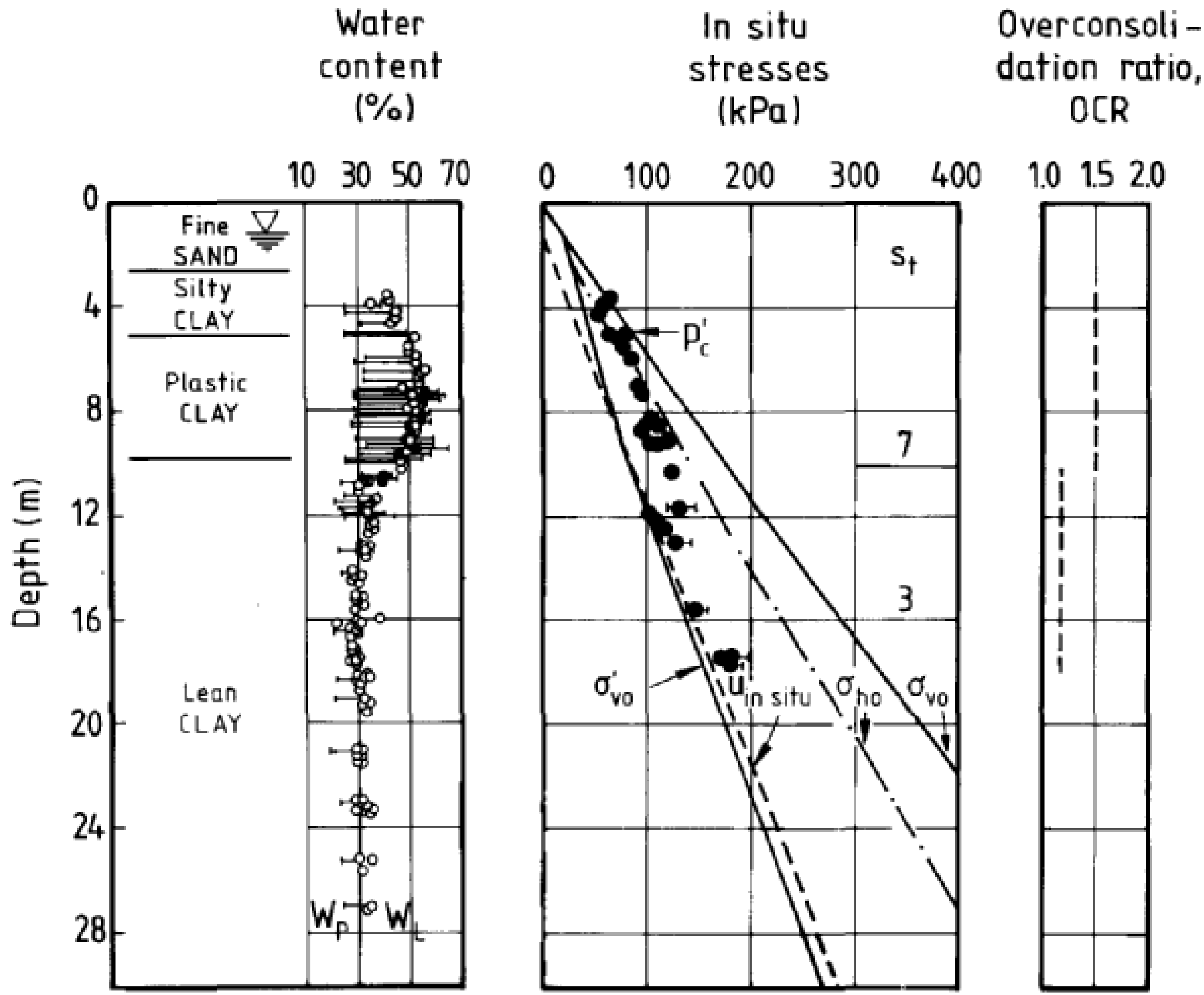


Fig. 6. Drammen test site. **(Suzanne & Tom, 1983)**

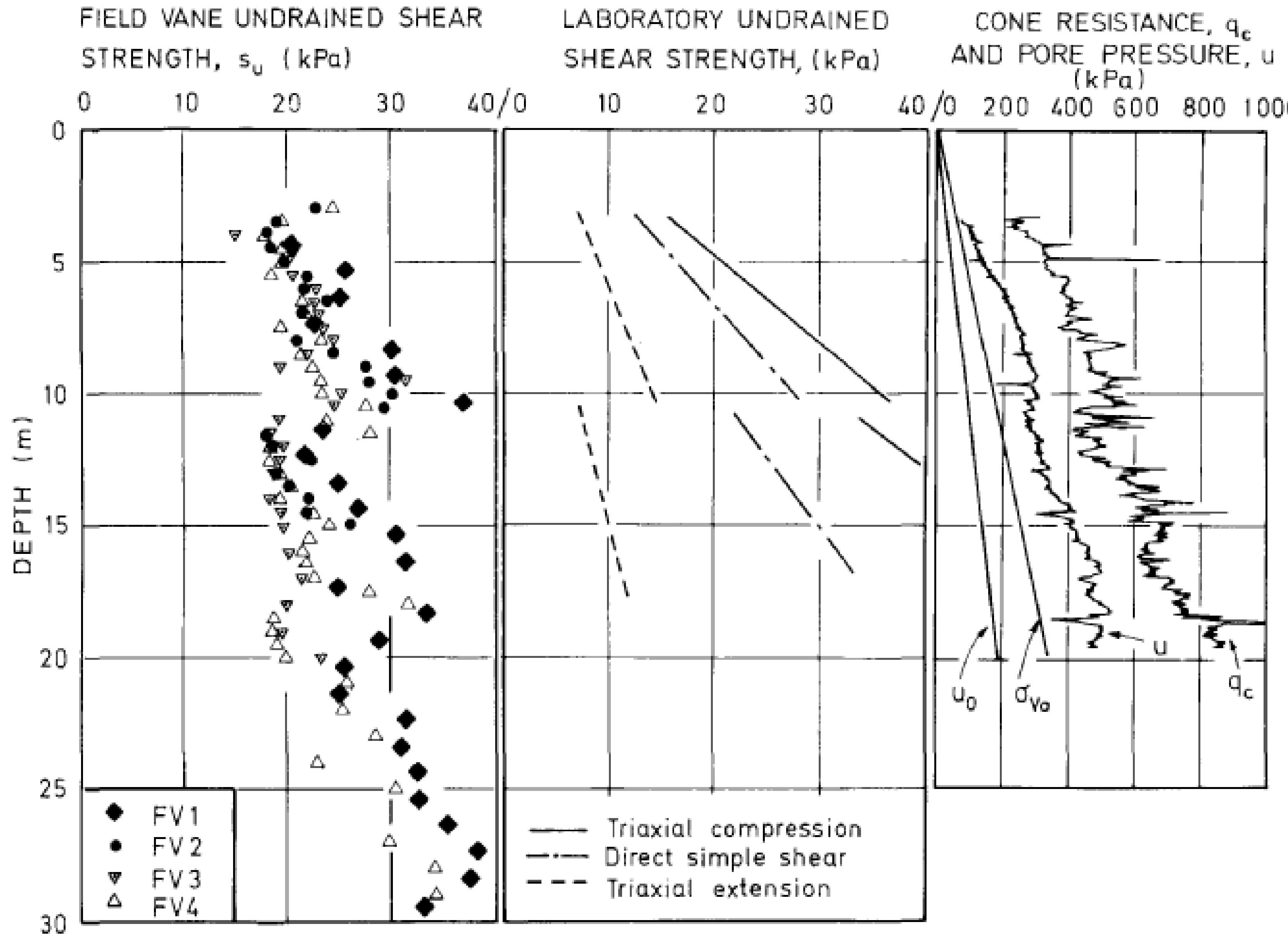


Fig. 7. Undrained shear strength in Drammen. **(Suzanne & Tom, 1983)**

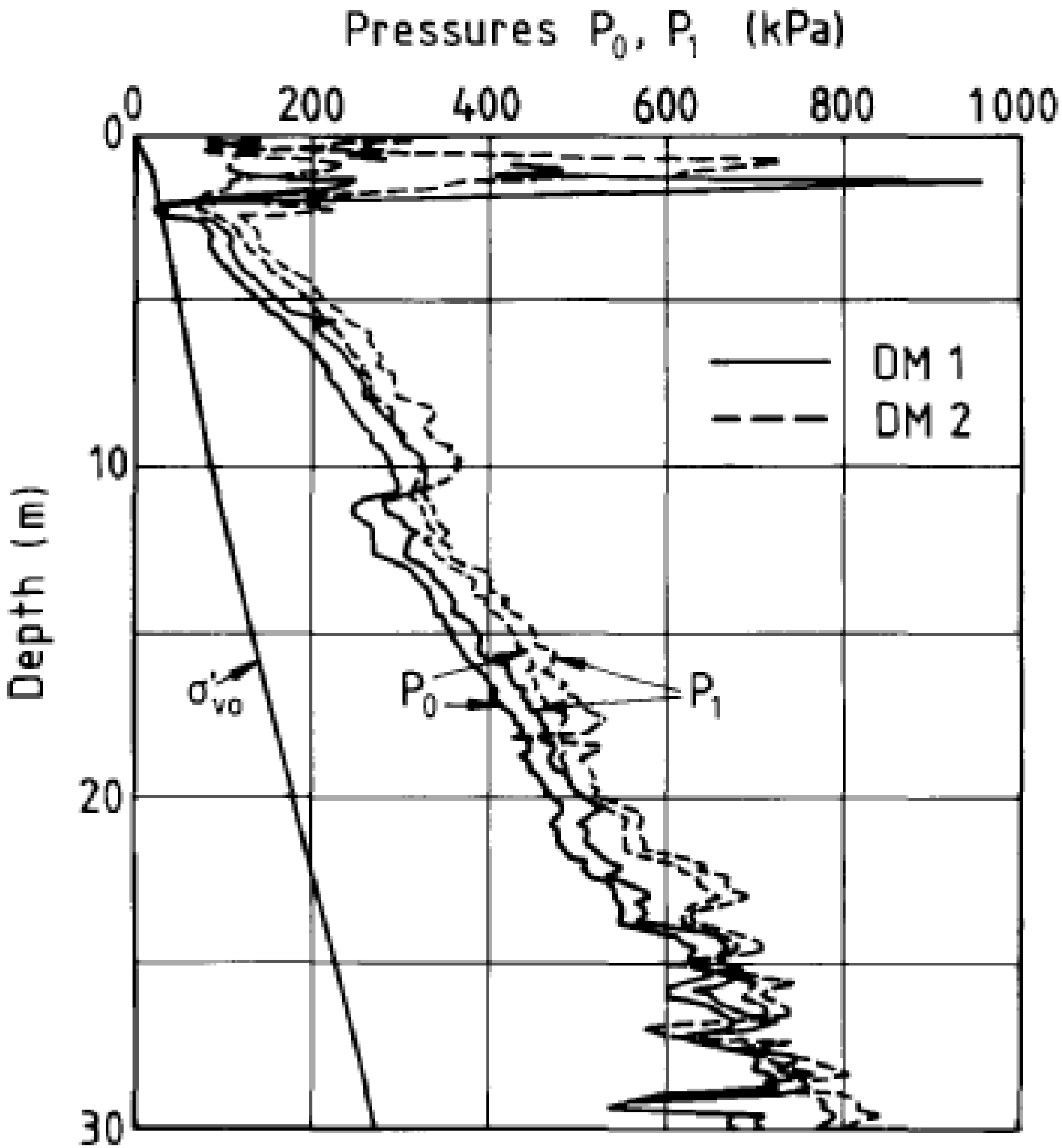


Fig. 8. Contact and expansion pressures in Drammen. **(Suzanne & Tom, 1983)**

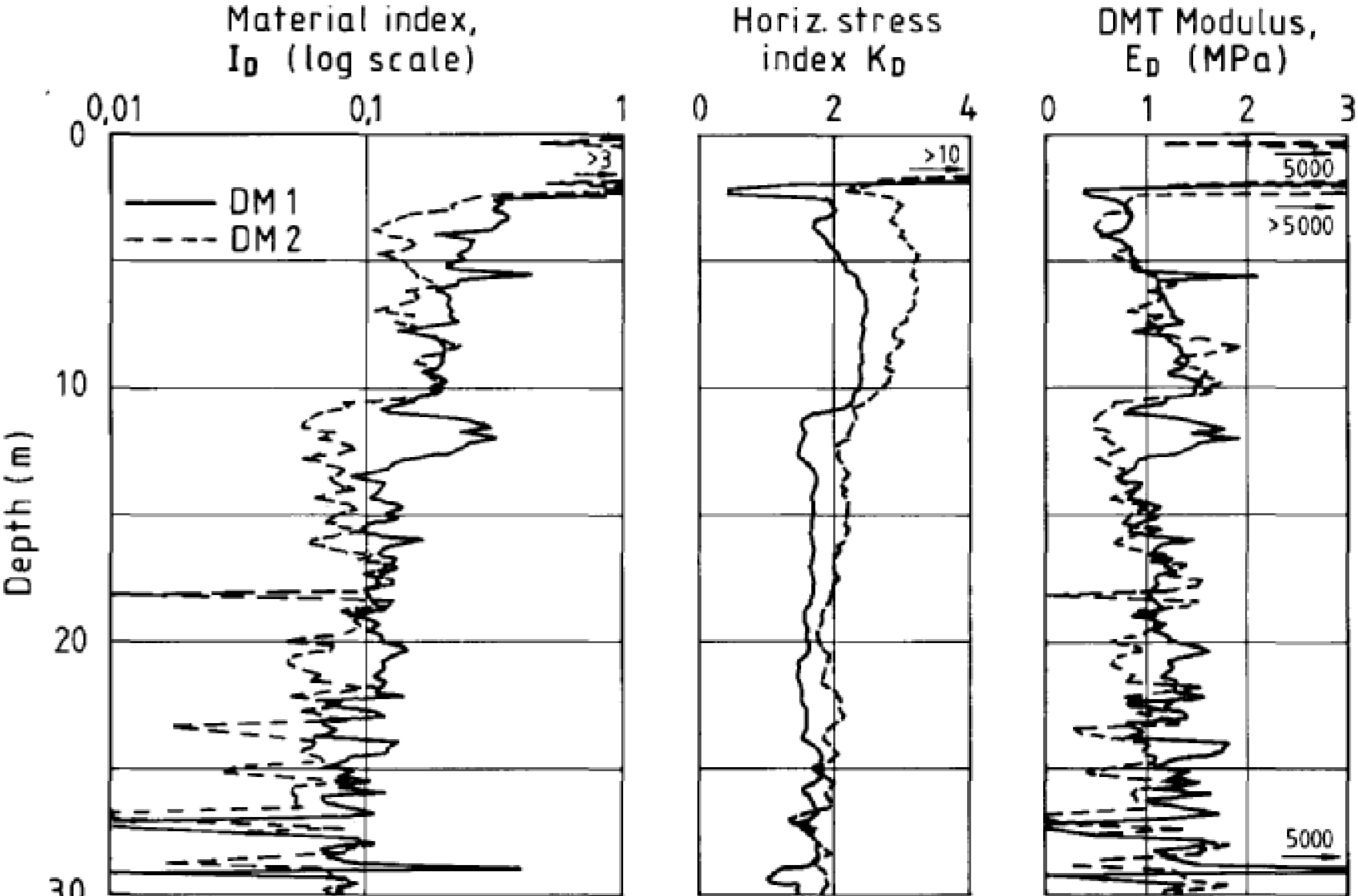


Fig. 9. Dilatometer parameters in Drammen. **(Suzanne & Tom, 1983)**

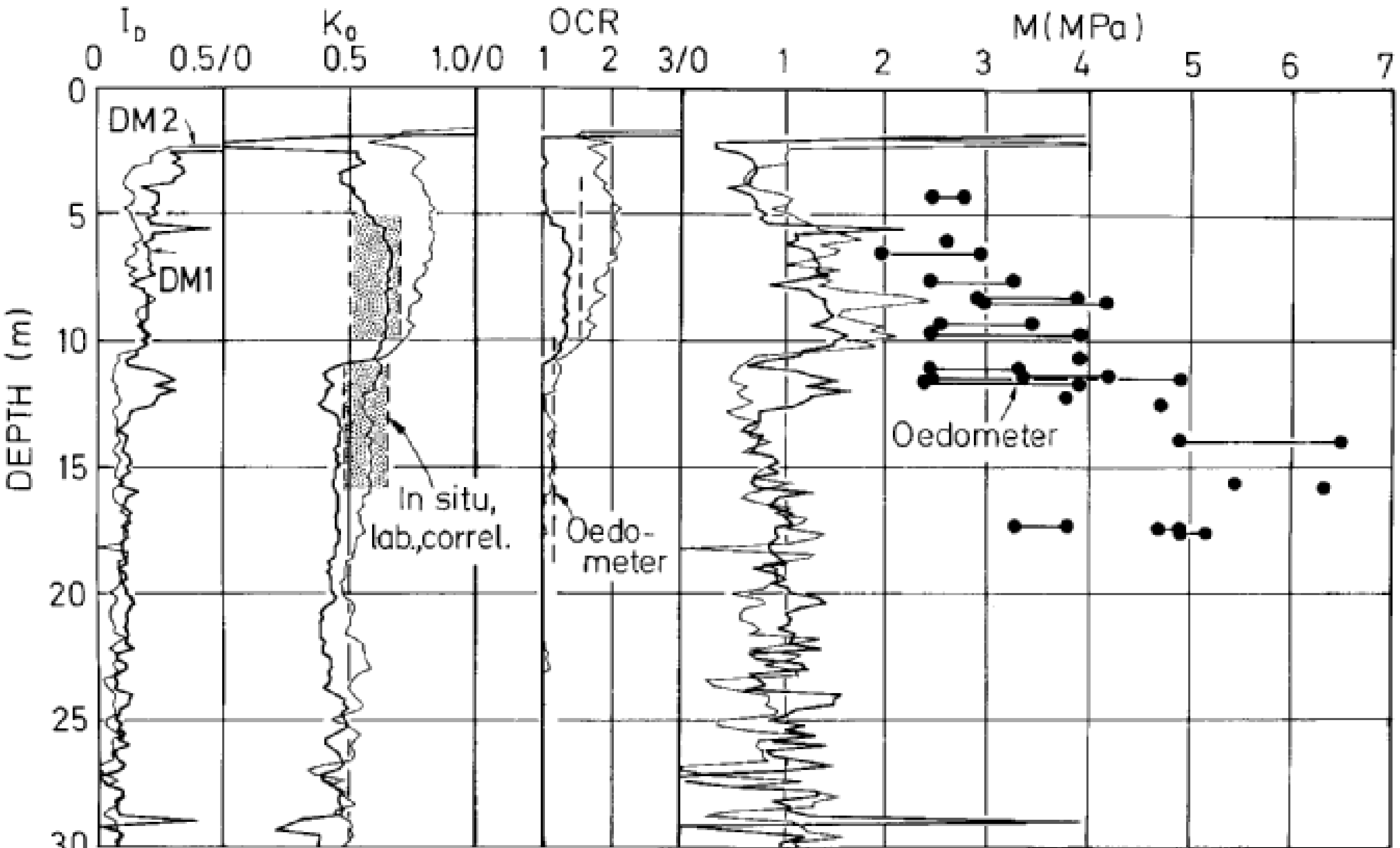


Fig. 10. Soil parameters derived from dilatometer tests in Drammen.

(Suzanne & Tom, 1983)

Table 1. Results of dilatometer tests in soft clay. **(Suzanne & Tom, 1983)**

Test site	Depth (m)	I_p (%)	K_o		OCR		M(MPa)	
			DMT	Reference	DMT	Reference	DMT	Reference
ONSØY	3	23	1.00	0.85	3.3–3.5	3.0	1.0–1.3	1.2–2.0
	11	36	0.75	0.65	1.7–1.8	1.3	1.3–1.5	2.1
DRAMMEN	6	28	0.6–0.8	0.5–0.7	1.3–2.1	1.5	1.1–1.8	2.6
	12	10	0.4–0.6	0.48–0.65	1.0–1.2	1.2	0.6–1.7	2.4–3.9

Section 2
Basic Soil Characterization

CONSISTENCY OF CLAY VERSUS N

N Value (blows/ft or 305 mm)	Consistency
0 to 2	Very soft
2 to 4	Soft
4 to 8	Medium
8 to 15	Stiff
15 to 30	Very stiff
> 30	Hard

Source: Terzaghi and Peck (27), p. 347.

CONSISTENCY INDEX OF CLAY VERSUS N and q_c

N Value (blows/ft or 305 mm)	Cone Tip Resistance, q_c/p_a	Consistency	Consistency Index
< 2	< 5	Very soft	< 0.5
2 to 8	5 to 15	Soft to medium	0.5 to 0.75
8 to 15	15 to 30	Stiff	0.75 to 1.0
15 to 30	30 to 60	Very stiff	1.0 to 1.5
> 30	> 60	Hard	> 1.5

Source: Szechy and Varga (43), p. 105.

Section 4 - Strength

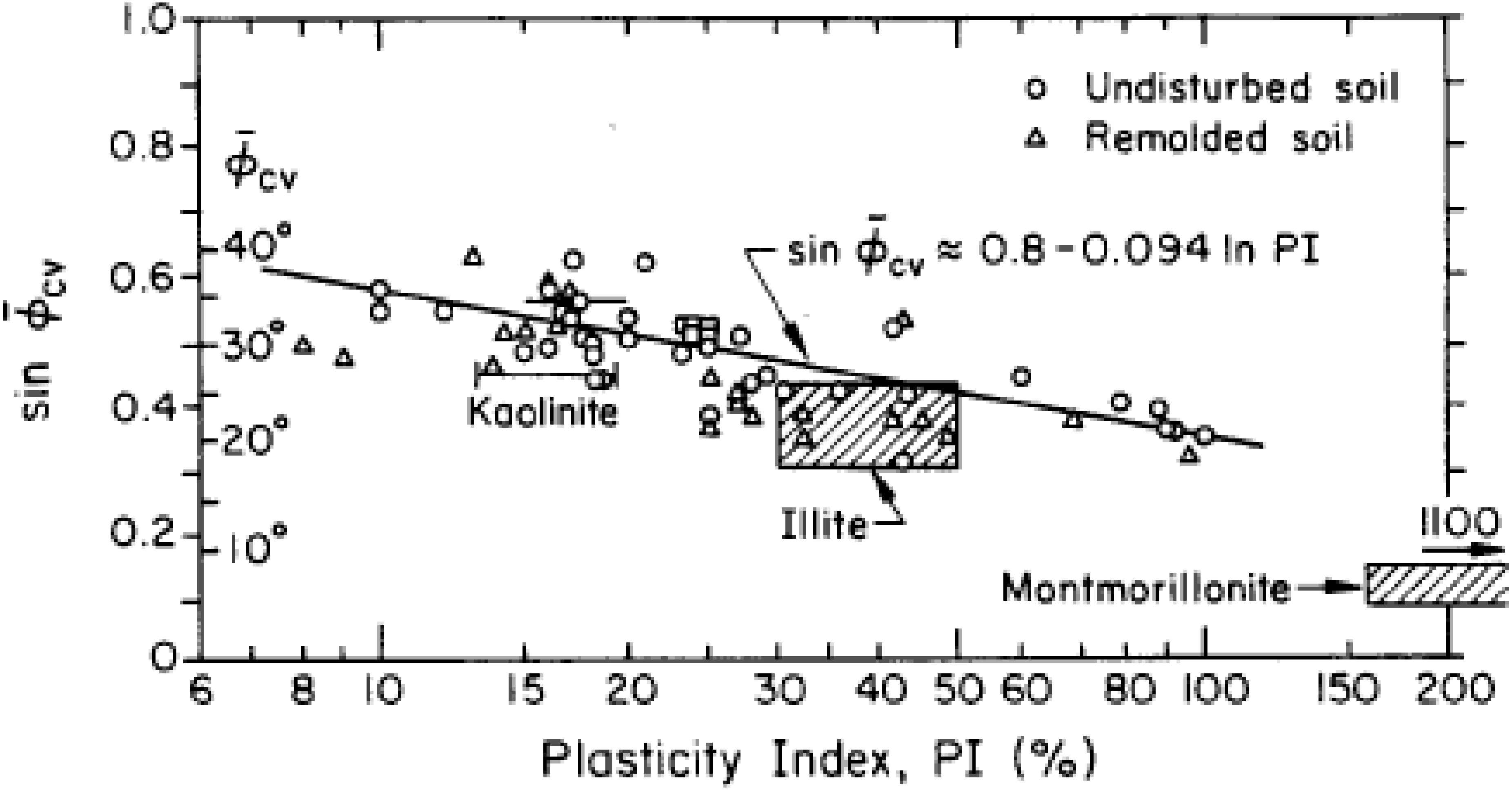


Figure 4-20. $\bar{\phi}_{cv}$ for NC Clays versus PI

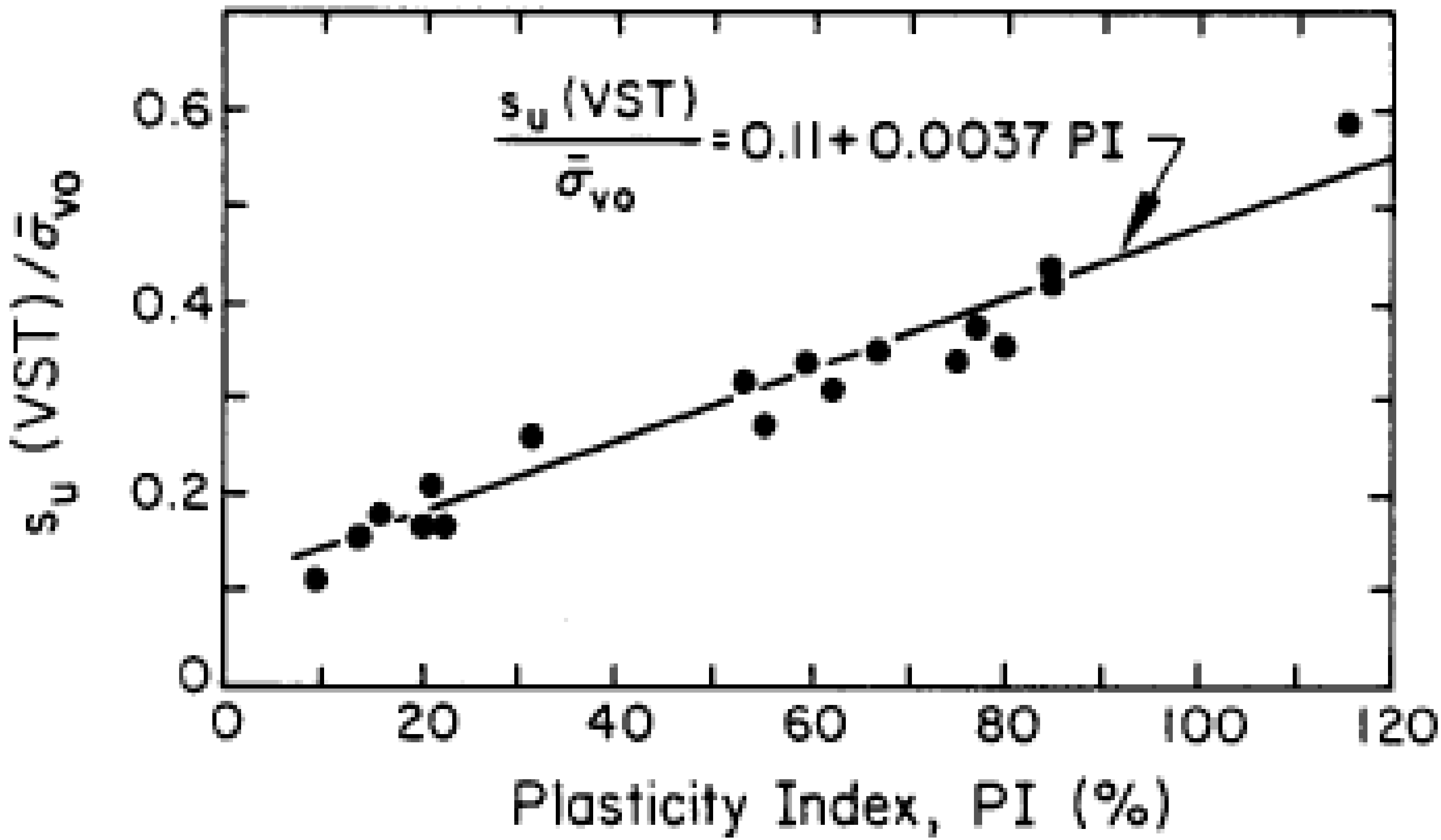


Figure 4-26. $s_u(VST)/\bar{\sigma}_{vo}$ versus PI for NC Clays

CLASSIFICATION OF SENSITIVITY

Clay Description	S_t
Insensitive	≈ 1
Slightly sensitive	1 to 2
Medium sensitive	2 to 4
Very sensitive	4 to 8
Slightly quick	8 to 16
Medium quick	16 to 32
Very quick	32 to 64
Extra quick	> 64

Source: Mitchell (22), p. 208.