WEEK 8

ISO 9001:2000 Certificate No: 404070

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ACTIVITY

Lecture (3 hours)

LEARNING OUTCOMES

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Week 8 : (3H) **Coverage : Field tests, Correlations of field data.**

Learning outcomes:

At the end of this lecture/week the students would be able to:

- **discuss different field tests to determine bearing capacity and shear strength of soils**
- **Discuss the geophysical methods of ground investigation**

OUTLINE of PRESENTATION

4.1 Introduction to *in situ* testing

4.2 Methods of *in situ* testing and analysis

4.3 Geophysical Methods of Ground Investigation

4.1 Introduction to *in situ* testing

Refers to the procedure of determining soil properties or other subsurface conditions at the actual surface or subsurface location.

***In situ* soil testing includes determination of shear strength, permeability, *in situ* density, plate bearing and settlement, lateral movement, and pore pressure measurement**

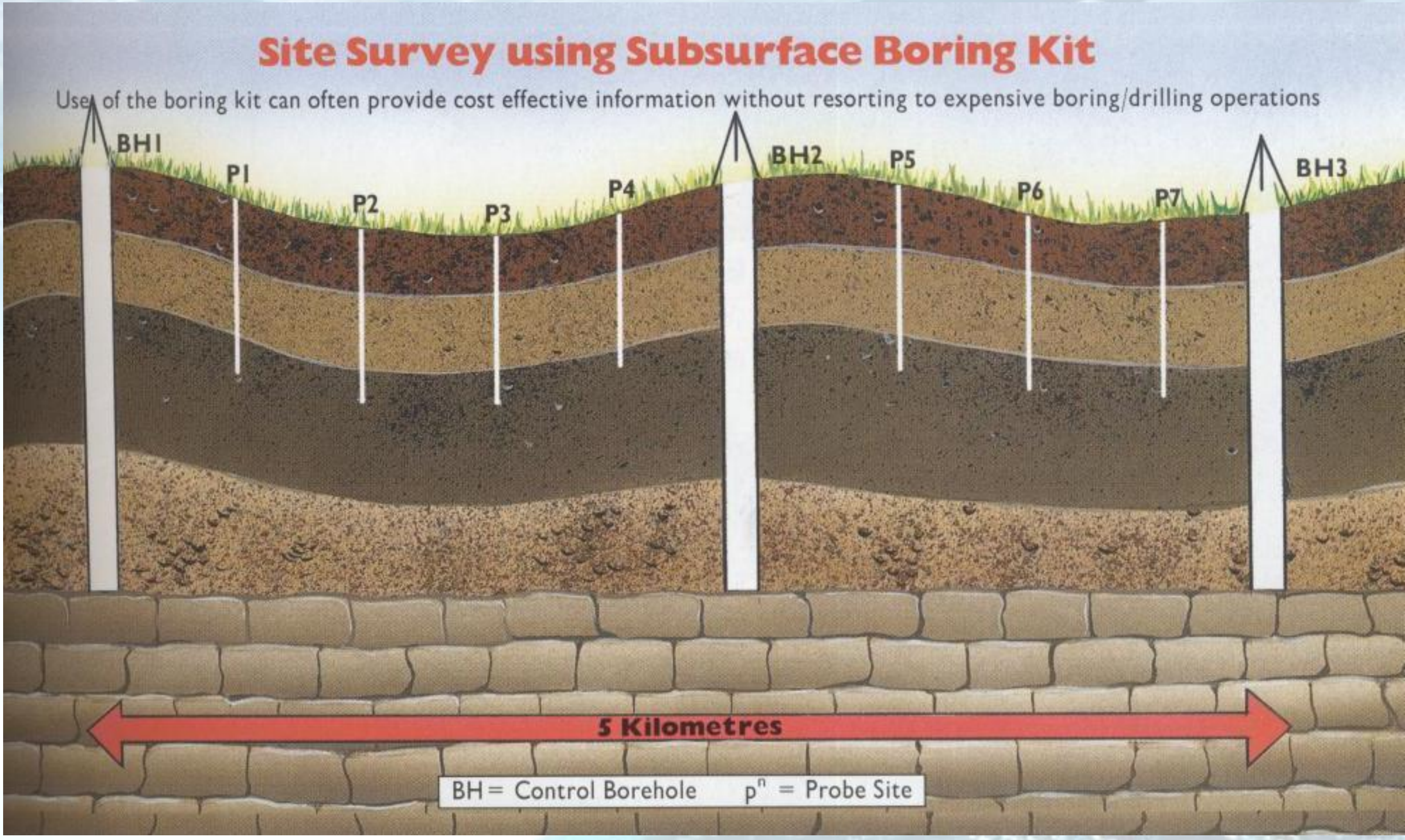
In situ shear strengths are determined where undisturbed samples for laboratory testing cannot be obtained or where it is desired to eliminate the need to obtain samples.

Introduction to *in situ* testing … cont.

Over the years several *in situ* testing devices have emerged to characterize the soil and to measure strength and deformation properties. The most popular devices are :

- **Vane shear test (VST)**
- **Standard penetration test (SPT)**
- **Cone penetrometer test (CPT)**
- **Pressuremeter test (PMT)**
- **Flat Plate dilatometer (DMT)**
- **JKR/Mackintosh Probe**

Boreholes and Probe points



4.2 Methods of *in situ* Testing

4.2.1 Vane Shear Test (VST)

VST is simple, inexpensive, quick to perform. The vane is pushed, usually from the bottom of a borehole to the desired depth. A torque is applied at a rate of 6^o per minute by a torque head device located above the soil surface and attached to the shear vane rod.

Errors in the measurement of the torque include excessive friction, variable rotation and calibration. The VST cannot be used for coarse grained soils and very stiff clays

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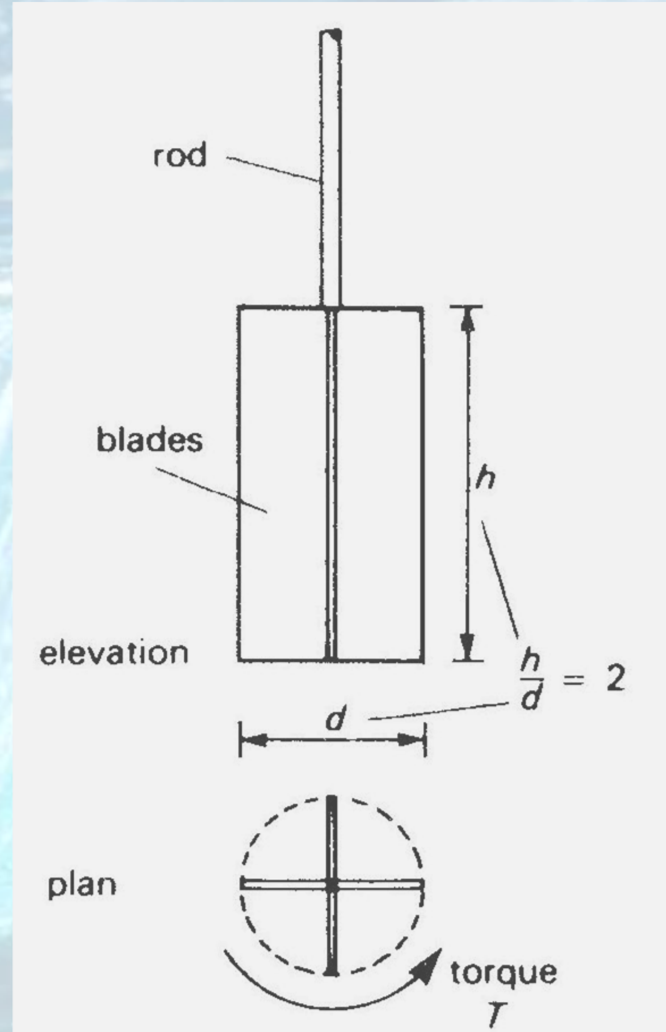
Lab. Vane Shear Apparatus



$$T = c_u \left\{ \pi h d \text{ x } \frac{d}{2} + 2\pi \frac{d^2}{4} \text{ x } \frac{d}{3} \right\}$$

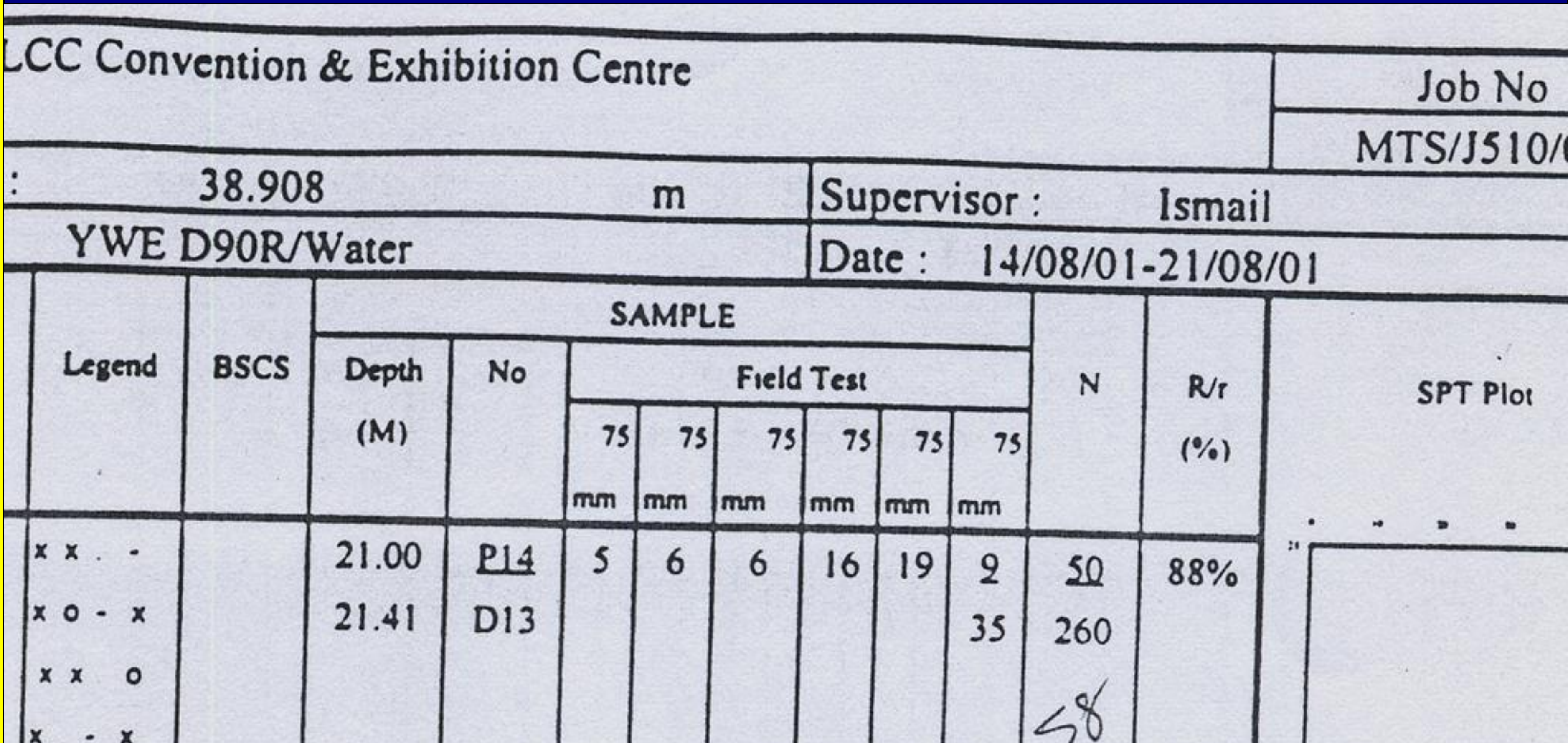
IN SITU TESTING & ANALYSIS

4.2.2 Standard Penetration Test (SPT)

The SPT is performed by driving a standard spoon sampler into the ground by blows from a drop hammer of mass 63.5 kg, free falling from 760 mm

IN SITU TESTING & ANALYSIS

SPT- data Recording



LCC Convention & Exhibition Centre

Job No MTS/J510/

: 38.908 m

Supervisor : Ismail

YWE D90R/Water

Date : 14/08/01-21/08/01

Legend	BSCS	Depth (M)	No	SAMPLE Field Test 75 mm	75 mm	75 mm	75 mm	75 mm	75 mm	N	R/r (%)	SPT Plot
x x . -		21.00	P14	5	6	6	16	19	9	50	88%	
x o - x		21.41	D13						35	260		
x x o												
x - x										58		

Note how the SPT value and recovery ratios are recorded !!

Typical Borelog

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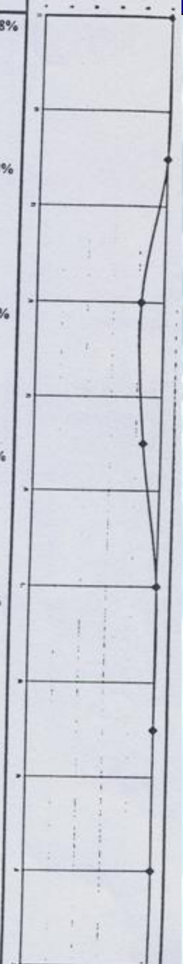
PROJECT :	Soil Investigation Works For Proposed KLCC Convention & Exhibition Centre	Job No MTS/J510/01
Borehole No. : BH-12	Reduced Level : 38.908 m	Supervisor : Ismail
Sheet No. 3 of 8	Type of Drill : YWE D90R/Water	Date : 14/08/01-21/08/01

Depth (M)	DESCRIPTION OF SOIL CONSISTENCY, COLOUR RELATIVE DENSITY, GRAIN SIZE, TEXTURE ETC	Legend	BSCS	Sample Depth (M)	Sample No	Field Test 75 mm	75 mm	75 mm	75 mm	75 mm	75 mm	N	R/r (%)
21.00, 22.00	Hard pale brown mottled with pale grey clayey SILT with a little of gravels and sand			21.00 21.41	P14 D13	5	6	6	16	19	9 35	50 260	88%
23.00, 24.00	Hard dark grey clayey SILT with a little of gravels			22.50 22.94	P15 D14	9	7	9	12	15	14 65	50 290	68%
25.00	Hard pale brown mottled with medium grey clayey SILT with traces of gravels			24.00 24.45	P16 D15	7	9	10	10	10	11	41	31%
26.00, 27.00	Hard reddish pale brown mottled with medium grey clayey SILT with traces of gravels			25.50 25.95	P17 D16	5	9	14	11	9	9	43	62%
28.00	Hard dark brown mottled with medium grey and red clayey SILT with some fine sand and gravels			27.00 27.305	P18 D17	6	20	20	23	2 5		50 155	56%
29.00, 30.00	Hard reddish medium brown clayey SILT with some fine sand and traces of gravels			28.50 28.805	P19 D18	13	12	16	28	6 5		50 155	66%
31.00	No recovery			30.00 30.375	P20	7	11	15	17	18 75		50 225	0%

SPT Plot



NOTES

P - Standard Penetration Test (SPT)
D - Disturbed sample
N - SPT Result
R/r - Recovery ratio
VS - Vane Shear Test
USS - Ultimate Shear Strength
SS - Shear Sensitivity
UD - 50 mm dia undisturbed sample
P - 50mm dia undisturbed piston sample
M - Mazier Sample
W - Water sample
C - Core sample (Rock)
RQD - Rock Quality Designation (%)
WL - Water level
BSCS - British Standard Classification System

LEGEND: CLAY, SILT, SAND, GRAVEL, PEAT

REMARKS

CONSISTENCY / RELATIVE DENSITY

Cohesive Soil (N)		Non Cohesive Soil (N)	
0 - 2	Very Soft	0 - 4	Very Loose
2 - 4	Soft	4 - 10	Loose
4 - 8	Medium Stiff	10 - 30	Medium Dense
8 - 15	Stiff	30 - 50	Dense
15 - 30	Very Stiff	50 >	Very Dense
30 >	Hard		

Logged by : Ismail (Supervisor)

Checked by : Ubaidillah (Geologist/T. Officer)

Standard Penetration Test

A dynamic test carried out in boreholes during site investigations. Terzaghi and Peck, 1967 provided a chart to determine the allowable bearing capacity. However, the N-values obtained from SI need to be corrected.

Correction due to the presence of water table (Terzaghi and Peck, 1948) :

$$N_{corr.} = 15 + \frac{1}{2}(N - 15)$$

Standard Penetration Test ….. cont.

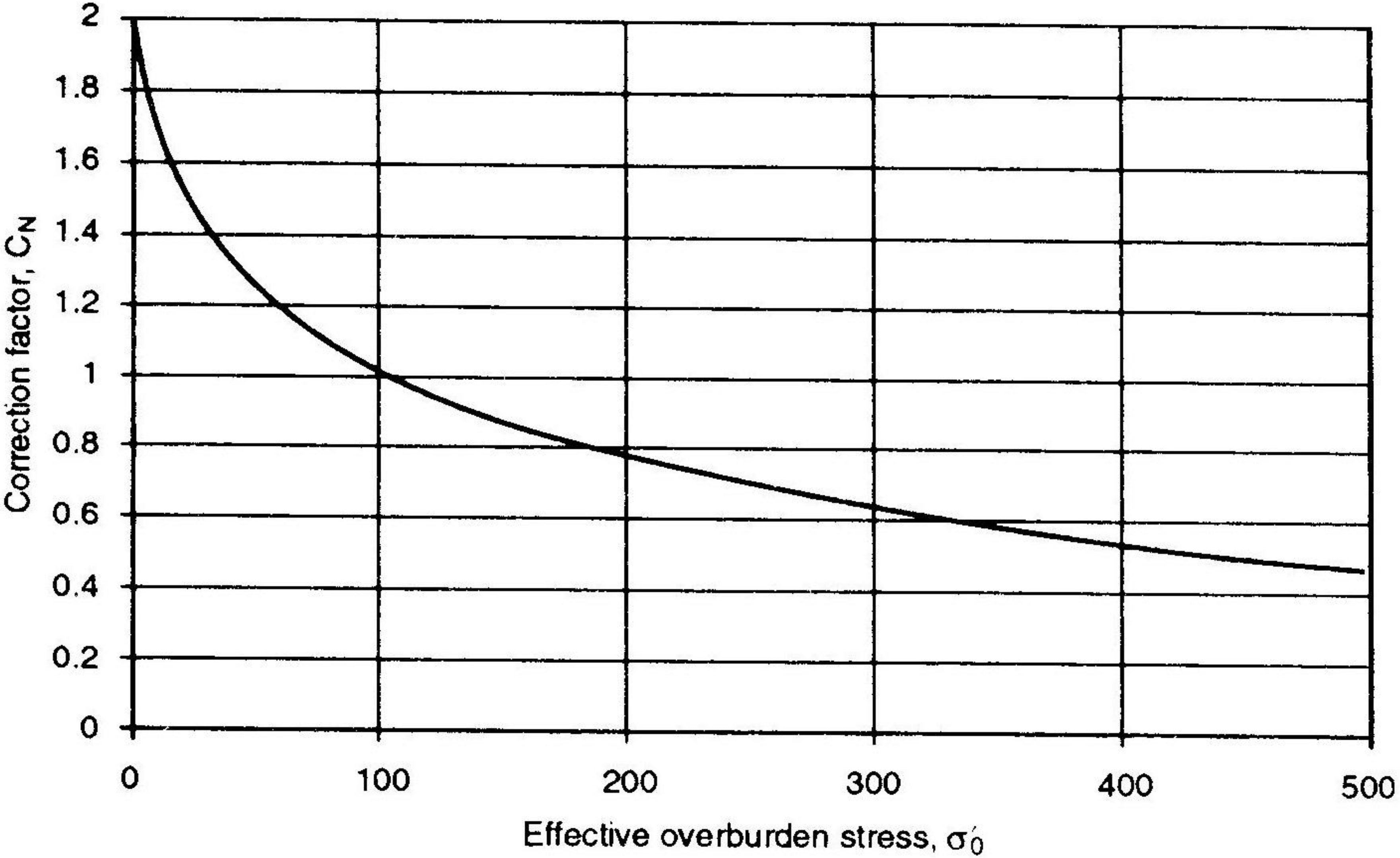
Correction with respect to effective overburden stress:

$$N_{corr.} = C_N \ N$$

$$where \quad C_N = a \ correction \ factor$$

Amongst a number of correction proposals is the chart given by Peck, Hanson and Thornburn (1974) in which :

$$C_N = 0.77 \log (2000/\sigma_o')$$



Correlation of SPT N-values for overburden stress (Peck, Hanson and Thornburn, 1974)

The effect of water table may be taken into account by applying the following correction :

$$C_w = \frac{1}{2}\left\{1 + \frac{D_w}{D + B}\right\}$$

where D_w = depth of water table below surface

D = founding depth below surface

B = footing breadth

Thus $q_a = C_w \, q_{TP}$

Yields conservative values with settlement < 25 mm

For wide footings and rafts the limiting values may be raised to 50 mm.

Meyerhof (1965) suggested that the q_{TP} values could be increased by 50% and that no correction should be made for the water table since the effect would be incorporated in the measured N-values. He proposed the following simple relationships :

For B < 1.25 m : $$q_a = \frac{s_L N}{1.9}$$

For rafts : $$q_a = \frac{s_L N}{2.84}$$

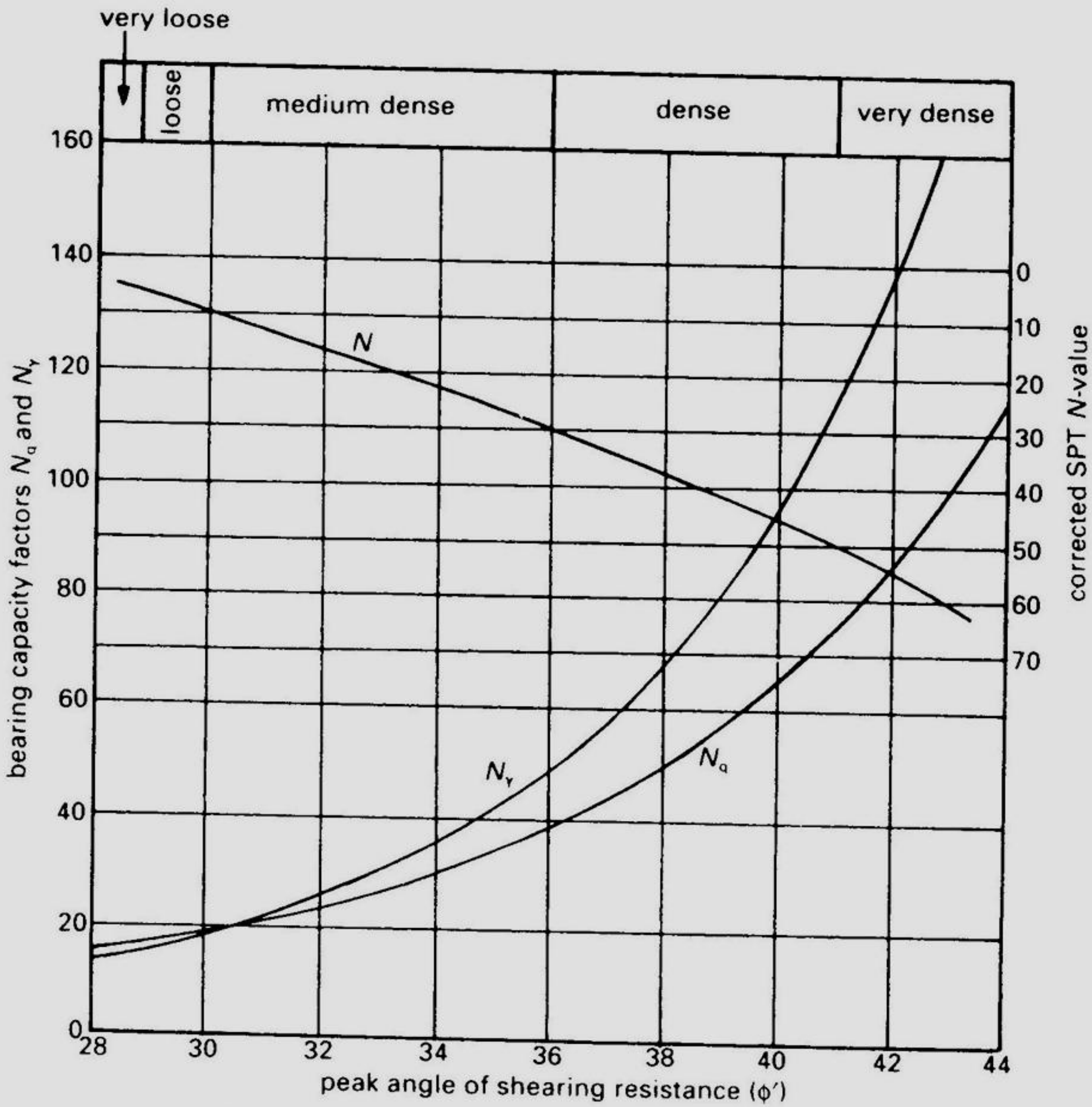
For B > 1.25 m : $$q_a = \frac{s_L N}{2.84}\left[\frac{B+0.33}{B}\right]^2$$

where s_L **= permitted settlement limit**

N **= average N-value between z = D and z = D + B***

B **= breadth of footing**

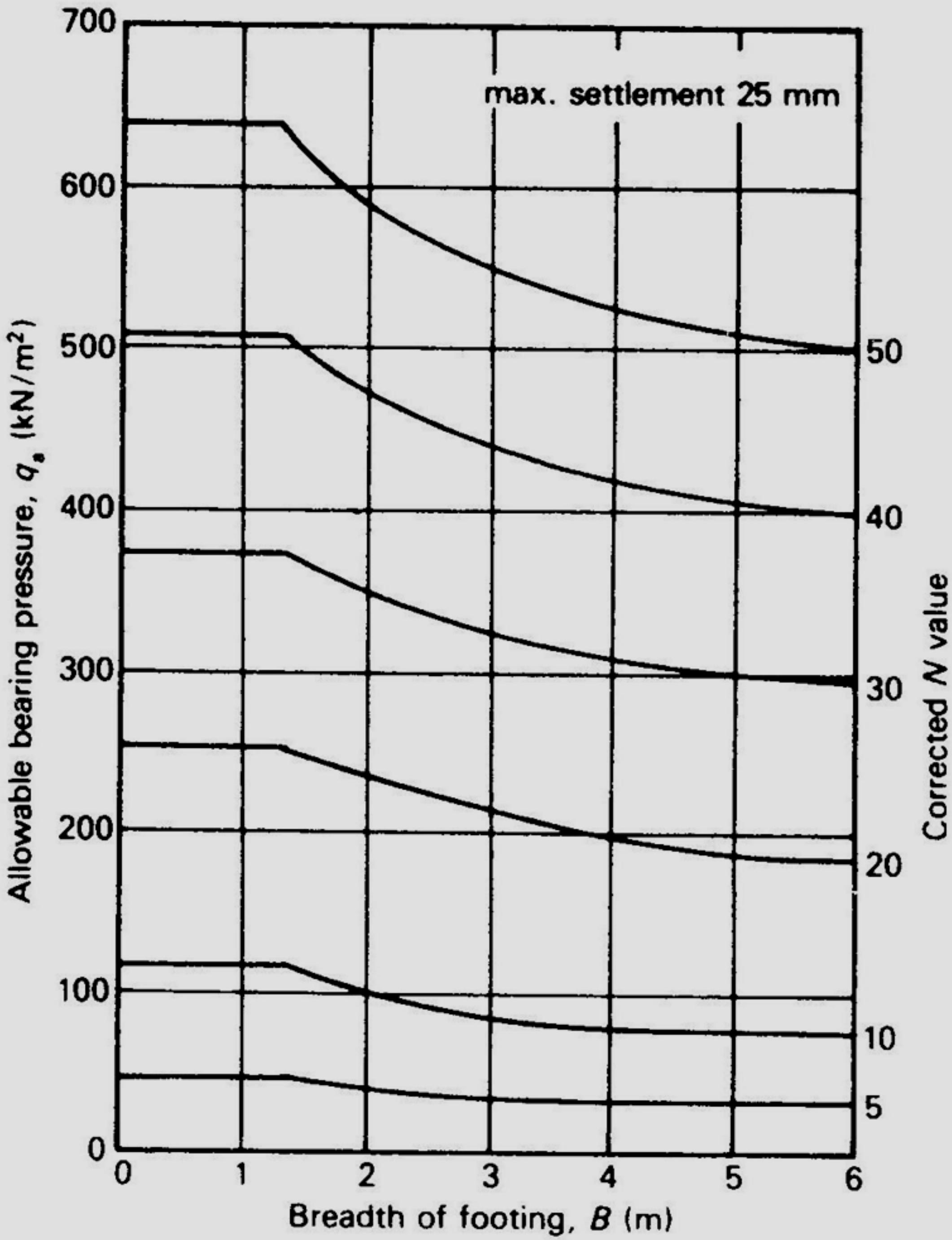
Relationship between *N*-value and *φ'*, N_q and N_γ



Notes: 1. φ'/N-value relationship after Peck, *et al.* (1974)
2. N_q/φ' and N_γ/φ' relationship from Table 11.2

Peck *et al.* proposed approximate relationships between the N-value and the peak angle of shearing resistance and also the bearing capacity factors.

Relationship between N-value and allowable bearing pressure (after Terzaghi and Peck, 1967)



The breadth of the footing and the N-value are used as entry data and the allowable bearing capacity (q_{TP}) is read off the left vertical axis.

IN SITU TESTING & ANALYSIS

SPT- Pile Analysis

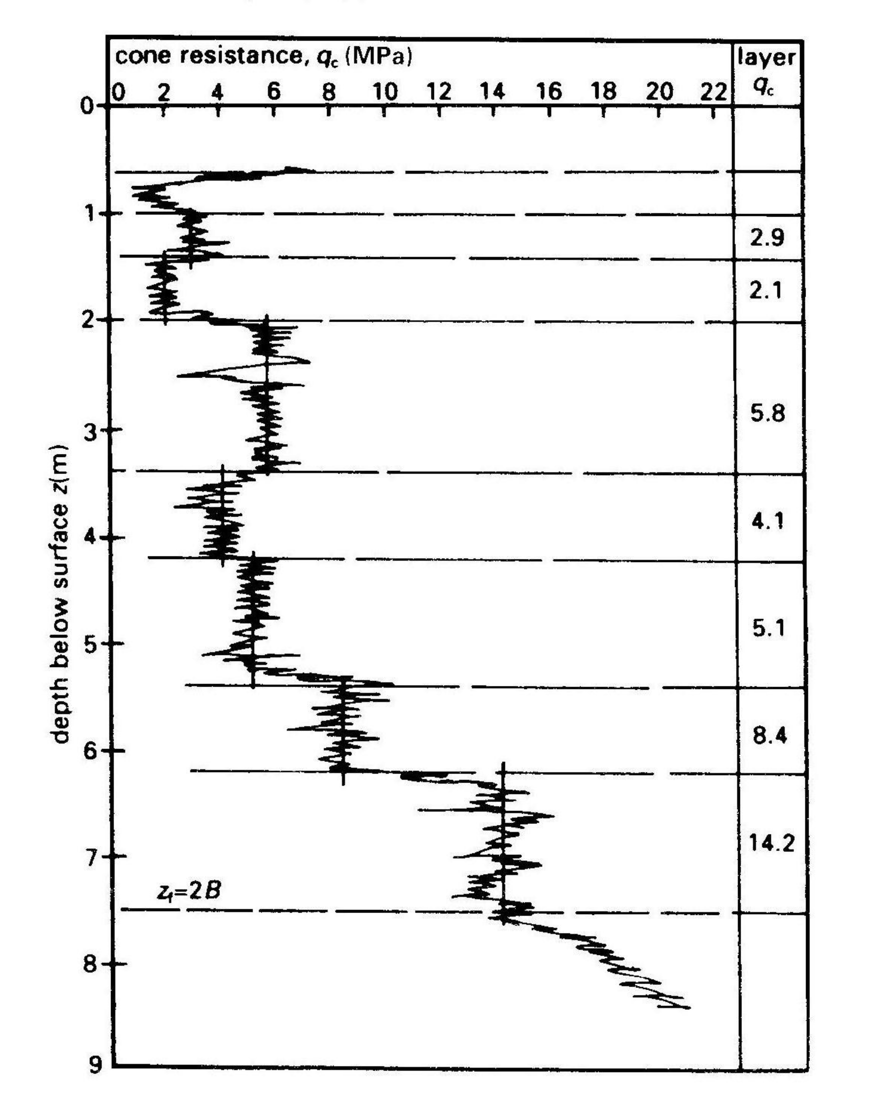
Pile type	Soil type	Ultimate base resistance q_b (kPa)	Ultimate shaft resistance f_s (kPa)
Driven	Gravelly sand and sand	$40(L/B)N \leq 400\ N$	$2N_{av}$
Driven	Sand silt and silt (ML)	$30(L/B)N \leq 300\ N$	$2N_{av}$
Bored	Gravels and sands	$13(L/B)N \leq 130\ N$	N_{av}
Bored	Sandy silt and silt (ML)	$10(L/B)N \leq 100\ N$	N_{av}

4.2.3 Cone Penetration test

Cone Penetration Equipment


Cone Penetrometer

CONE PENETRATION TEST ACCESSORIES & TEST RESULTS



Cone Penetration Test

The cone is pushed into the soil at a rate of 20 mm/s and the cone resistance (q_c) measured as the maximum force recorded during penetration divided by the end area. Although originally developed for the design of piles, the cone penetrometer has been used to estimate the bearing capacity and settlement of foundations.

A compressibility coefficient was suggested by de Beer and Martens (1951):

$$C = \frac{1.5\, q_c}{\sigma_o'}$$

where q_c = cone resistance (MPa)

σ_o' = effective overburden pressure (MPa)

Cone Penetration Test … cont.

The settlement s_i at the centre of a layer of thickness H is given by :

$$s_i = \frac{H}{C} \log\left(\frac{\sigma_o' + \Delta q}{\sigma_o'}\right)$$

where $\Delta q =$ Increase in stress at the centre of the layer due to a foundation pressure q

C = the expression in the previous slide.

Cone Penetration Test … cont.

However the above method is considered to overestimate the value of s_i. A rapid method was suggested by Meyerhof (1974).

$$s_i = \frac{q_n B}{2\bar{q}_c}$$

where

$q_n =$ net applied loading = $q - \sigma_o'$

$\bar{q}_c =$ average cone resistance over a depth below the footing equal to the breadth B

Cone Penetration Test … cont.

- *Schmertmann's method (1970)*
- *Schmertmann et al. 's method (1978)*

Probably the most thorough and reliable method for computing immediate settlement from CPT results.

$$s_i = C_1 C_2 q_{net} \sum \frac{I_z \Delta z}{E}$$

where : C_1 = 1 - 0.5(σ_o'/q_n) ; for $q_n \leq \sigma_o$' , C_1 = 0.5

C_2 = 1 + 0.2 log(10t) ; t is time in years

= 1.0 for immediately after construction case

I_z = vertical strain influence factor

Δz = thickness of sub layers

E = stiffness modulus = 2.5q_c for square foundation (L/B=1.0)

= 3.5q_c for long foundation (L/B>10)

General conclusion on CPT

CPT is quick to perform with fewer performance error compared with SPT. It can provide continuous records of soil conditions. However it cannot be used in dense, coarse-grained soil and mixed soils containing boulders, cobbles, clays and silt. The cone tip is prone to damage from contact with dense objects.

General conclusion … *cont.*

Other CPT variants are :

- ***Piezocone (uCPT or CPTu)*** **is a cone penetrometer that has porous elements inserted into the cone or sleeve to allow for pore water measurements.**
- ***Seismic cone (SCPT)*** **– geophones are installed inside the cone to record seismic waves resulting from hammer blows on the soil surface which produce surface disturbance. The recorded data are analysed to give damping characteristics and soil strength parameters.**

General conclusion … *cont.*

- *Vision cone (VisCPT or VisCPTu)* have miniature cameras installed in the CPT probe that provide continuous images of the soil adjacent to the cone. Through image processing, the soil texture can be inferred. The VisCPTu can also be used to detect liquefiable soils.

CPT/SPT relationship

Amongst the proposed relationships are :

$$q_c = 0.4N \quad (MPa)$$

Meyerhof (1956)

$$q_c = 0.25N \quad (MPa)$$

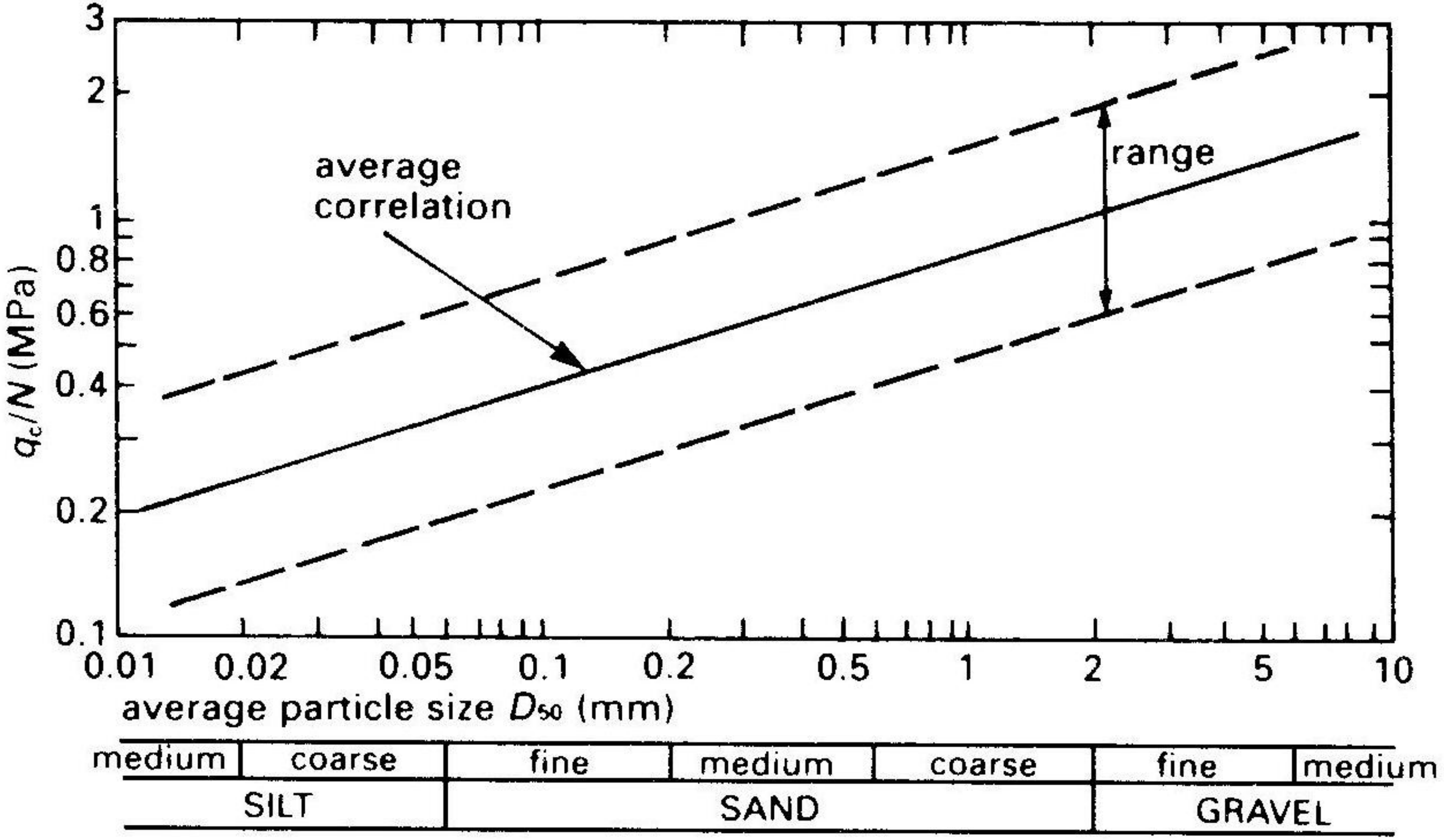
Meigh and Nixon (1961) for silty fine sand

$$q_c = 1.2N \quad (MPa)$$

Meigh and Nixon (1961) for coarse gravel

$$q_c/N \text{ vs Av. grain size } D_{50} \quad (MPa)$$

Burland and Burbidge (1985) – **see NEXT SLIDE**



Relationship between CPT and SPT (after Burland and Burbigde, 1985)

4.2.4 Pressuremeter test

Read-out device, hydraulic probe and lines conforming with the ASTM Standard Method for Pressuremeter Testing in soils D 4719-87.

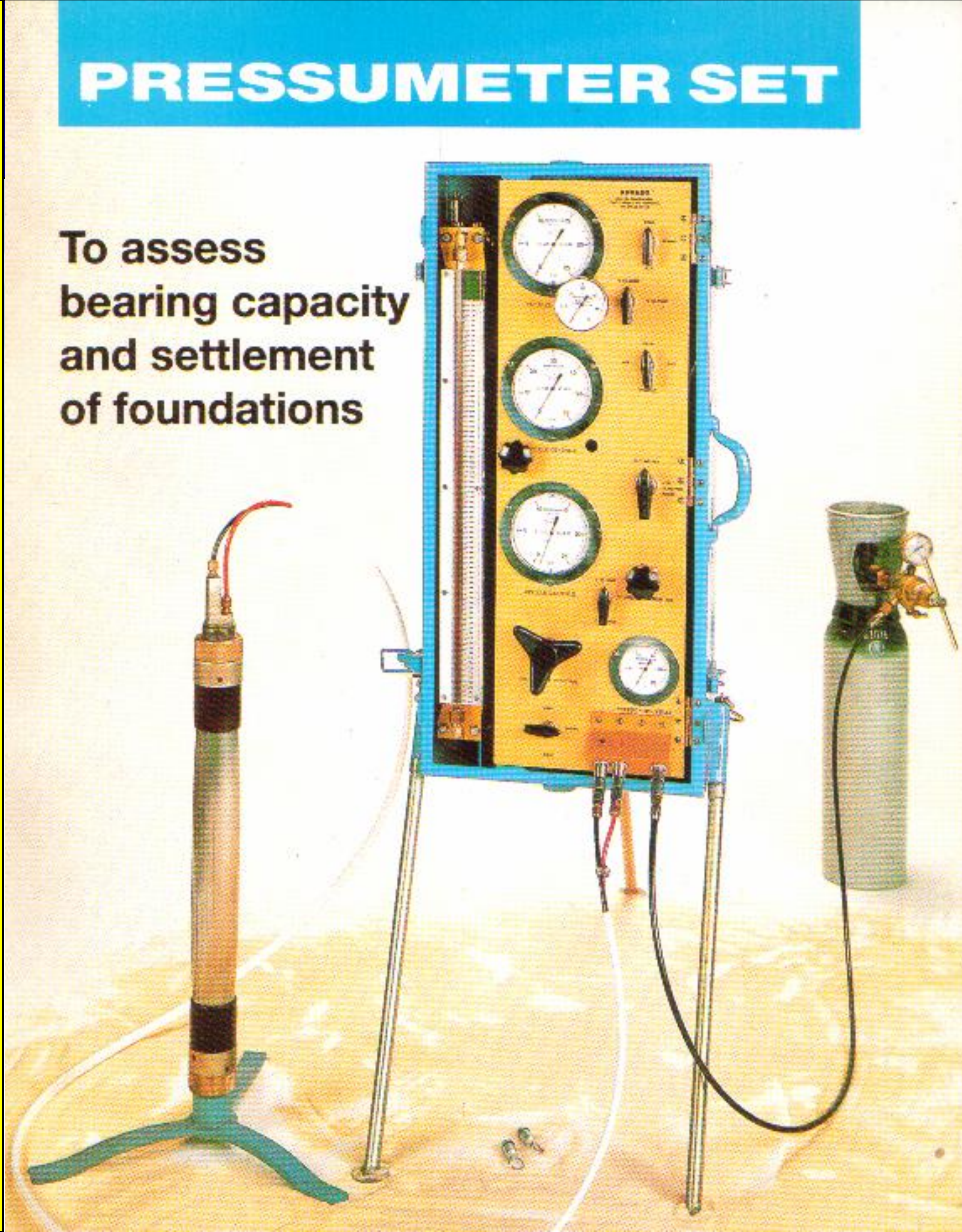
TEST DESCRIPTION

A pressuremeter test in an in-situ stress controlled loading test performed on the wall of a borehole using a cylindrical probe which can expand radially.

From the test readings a stress-strain curve can be obtained which yields :

- the pressuremeter modulus,
- the creep pressure,
- the limit pressure

once the volume and pressure calibrations have been performed.



Menard Pressuremeter

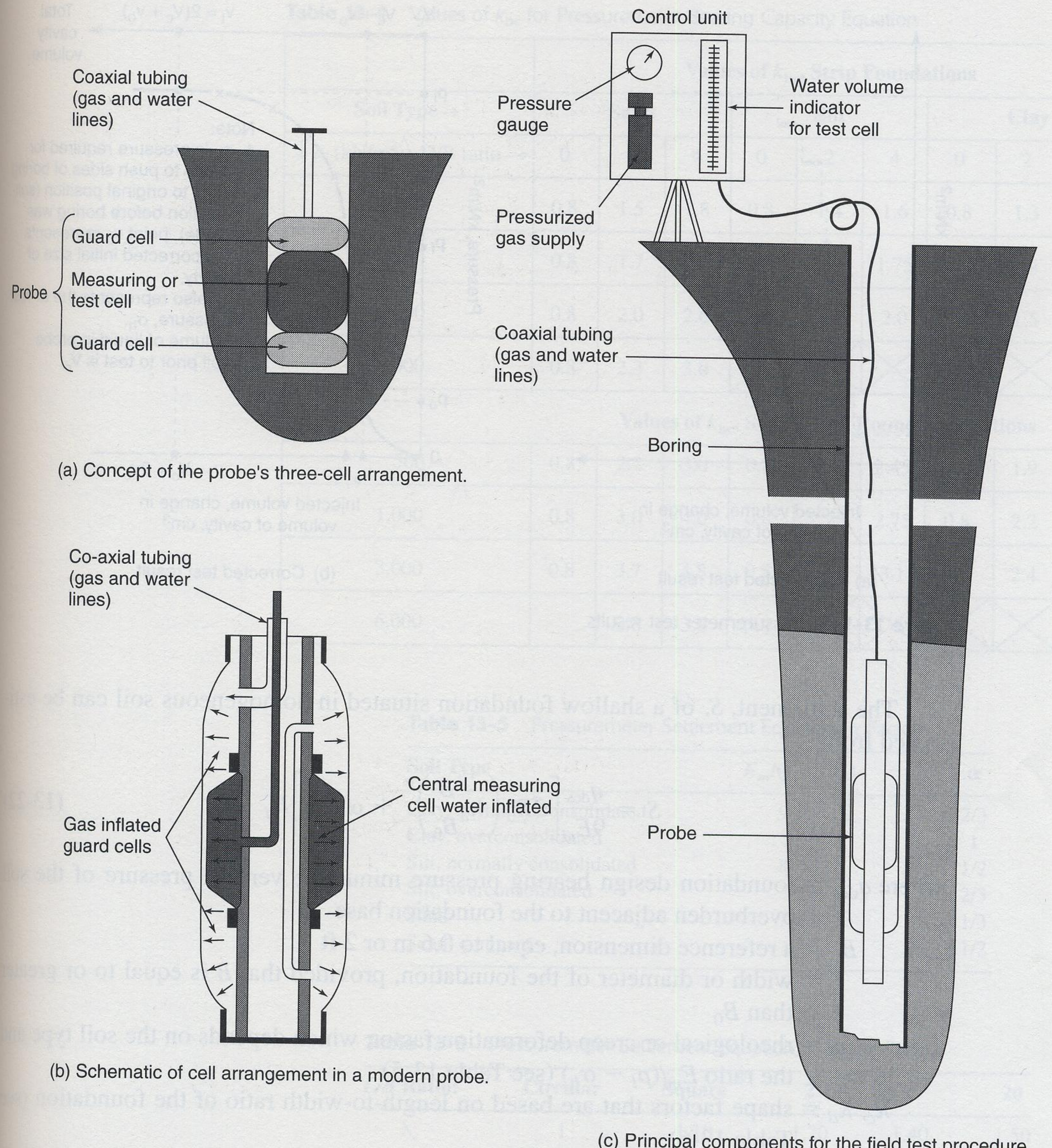


Figure 13-16 Menard-type pressuremeter: Components and Operating Principles.

Menard Pressuremeter

4.2.5 Flat Plate Dilatometer (DMT)

Consists of tapered blade 95 mm wide and 15 mm thick and 240 mm long. On the flat face the dilatometer is a flexible steel membrane 60 mm in diameter that when inflated pushes the soil laterally. Tests are normally conducted every 200 mm. Results from the test have been related to undrained shear strength, lateral earth pressures, overconsolidation ratios and elastic modulus.

Simple and quick to conduct. Provides reasonable estimates horizontal stress and is less costly than the pressuremeter test.

4.2.6 JKR/Mackintosh Probe

- **Can be used to determine the thickness of unsuitable material to be removed and also for preliminary design of embankments**
- **Limited to about 15 m**
- **Record no. of blows/ft. then correlate to established chart to determine bearing capacity of soil**

Comparison between JKR Probe, Mackintosh Probe & SPT

Type of Penetrometer	Con		Weight of Hammer (kg)	Height of Fall (mm)	Energy per Unit Area N·m/m^2
	Diameter (mm)	Area (mm^2)			
JKR Probe	25	491	5	280	27972
Mackintosh Probe	27.9	611	4.5	300	21675
SPT	50	1963	65	760	246874

Comparison of Energy between JKR Probe, Mackintosh Probe & SPT

Ratio of Energy of SPT to JKR Probe = $\frac{246874}{27972} = 8.8$

Ratio of Energy of SPT to Mackintosh Probe = $\frac{246874}{21675} = 11.4$

If the cone base diameter of Mackintosh probe is 25 mm, then

Ratio of Energy of SPT to Mackintosh Probe = $\frac{246874}{26979} = 9.2$

Correlation between JKR Probe, Mackintosh Probe & SPT

(i) JKR Probe's correlation with SPT-N value

$$\text{SPT} - \text{N value} = \frac{\text{JKR value}}{8.8}$$

(ii) Mackintosh Probe's correlation with SPT-N value

$$\text{SPT} - \text{N value} = \frac{\text{Mackintosh value}}{11.4}$$

(iii) JKR Probe's correlation with Mackintosh Probe value

$$26\ \text{JKR value} = 20\ \text{Mackintosh Probe value}$$

SPT/JKR or Mackintosh Probe relationship

N (Blows/ ft)	Consistency	Unconfined Compressive strength (Ton/Sq Ft)	Unconfined Compressive strength (kPa)	JKR or Mackintosh Probe (Blows/ft)
2	Very soft	0.00 – 0.25	0.0 – 25	0 – 10
2 – 4	Soft	0.25 – 0.50	25 - 50	10 – 20
4 - 8	Medium (firm)	0.50 – 1.00	50 – 100	20 – 40
8 - 15-	Stiff	1.00 – 2.00	100 – 200	40 – 70
15 - 30	Very stiff	2.00 – 4.00	200 – 400	70 – 100
Over 30	Hard	4.00	400	100

Relationship between SPT, Mackintosh/ JKR probe and unconfined compressive strength of clay.

SPT/JKR or Mackintosh Probe relationship

N (Blows/ ft)	Relative density	Allowable soil pressure (Ton/Sq Ft)	**Allowable soil pressure (kPa)**	JKR or Mackintosh Probe (Blows/ft)
0 – 4	**Very loose**	**Not suitable**	**Not suitable**	**0 - 10**
4 - 10	**Loose**	**0.0 – 0.8**	**0 – 80**	**10 - 30**
10 - 30	**Medium**	**0.8 – 2.8**	**80 – 280**	**30 - 80**
30 - 50	**Dense**	**2.8 – 4.7**	**280 – 470**	**80 - 110**
Over 50	**Very dense**	**4.7**	**470**	**110**

Note : 1 Ton/sq ft = 100 kN/m^2

Relationship between SPT, Mackintosh/ JKR probe and allowable soil pressure of sand.

4.3 Geophysical Methods for Ground Investigation

Involve the techniques of determining underground materials by measuring some physical property of the material and, through some correlations, using the obtained values for identifications. Most methods determine conditions over a sizable distance. The methods do not actually measure engineering properties.

Several types can be utilised, namely:

- Seismic refraction method
- Electrical resistivity method
- Ground-penetrating radar

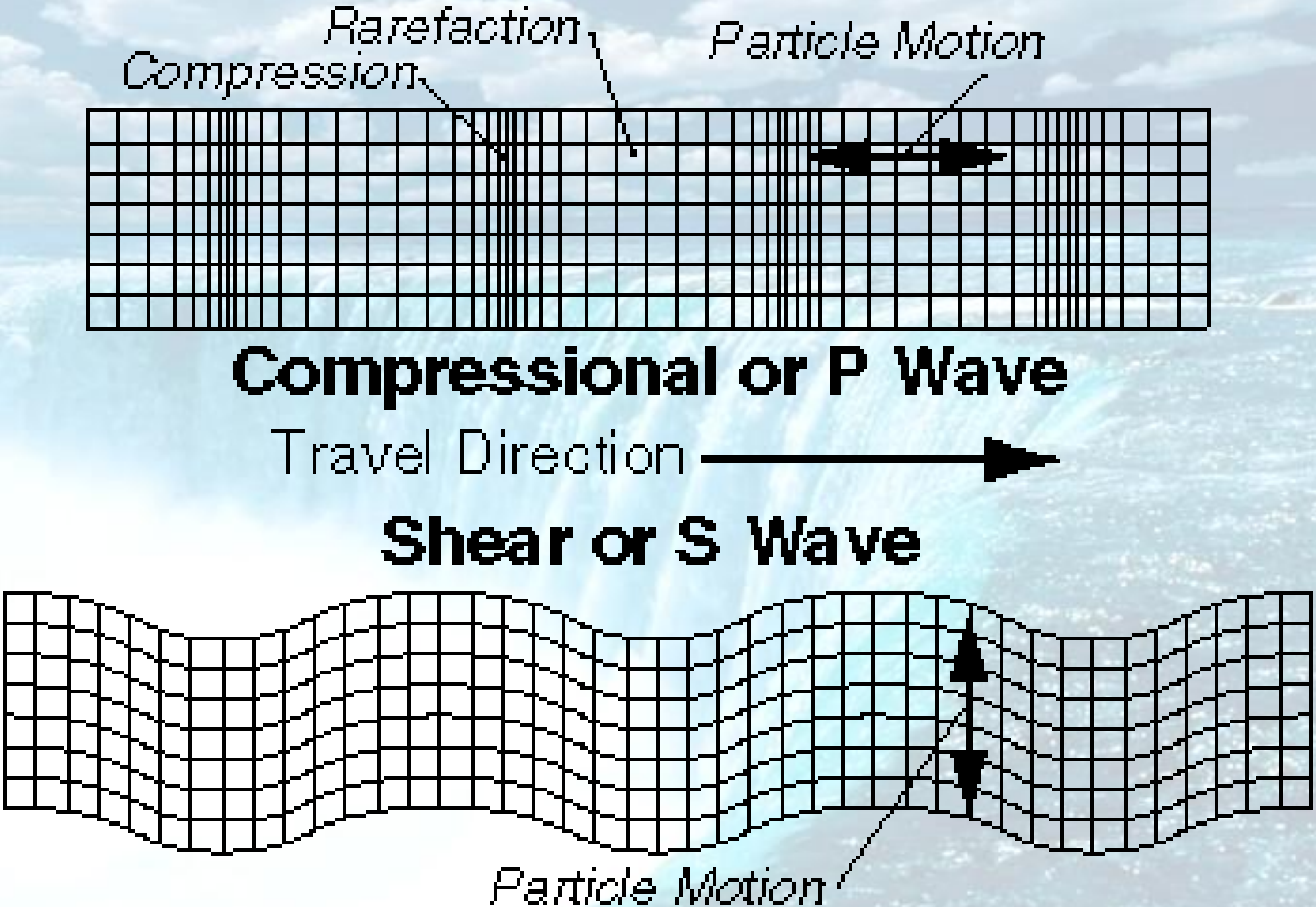
4.3.1 Seismic Refraction Method

Based on the seismic waves travelling through the surrounding soil and rock at speeds relating to the density and bonding characteristics of the material.

The velocity of the seismic waves passing through subsurface soil or rock materials is determined, and the magnitude of the velocity is then utilised to identify the material.

Seismic Refraction Method

- The mechanical properties of the rocks through which the seismic waves travel quickly organize the waves into two types.
- Compressional waves, also known as primary or P-waves, travel fastest, at speeds between 1.5 and 8 kilometers per second in the Earth's crust.
- Shear waves, also known as secondary or S-waves, travel more slowly, usually at 60% to 70% of the speed of P-waves.


Rarefaction
Compression
Particle Motion
Compressional or P Wave
Travel Direction
Shear or S Wave
Particle Motion

Representative Seismic values

Soil – unconsolidated material	m/sec
Most unconsolidated materials	Below 900
Soil – normal - hard-packed	250 – 450 450 – 600
Water	1500
Loose sand – above water table - below water table	250 – 600 450 – 1200
Loose mixed sand and gravel, wet	450 – 1100
Loose gravel, wet	450 – 900
Hard clay	600 - 1200

Representative Seismic values

Rock – consolidated material	m/sec
Most hard rocks	Above 2400
Shale – soft - hard	1200 - 2100 1800 - 3000
Sandstone – soft - hard	1500 – 2100 1800 - 3000
Limestone – weathered - hard	1200? 2400 - 5500
Basalt	2400 - 4000
Granite and unweathered gneiss	3000 - 6000
Compacted glacial tills, hardpan, cemented gravels	1200 -2100
Frozen soil	1200 - 2100
Pure ice	3000 - 3700

4.3.2 Electrical Resistivity Method

Resistivity is a property possessed by all materials.

The method for determining subsurface conditions utilizes the knowledge that in soil and rock materials, the resistance values differ sufficiently to permit that property to be used for identification purposes

Electrical Resistivity Method … cont.

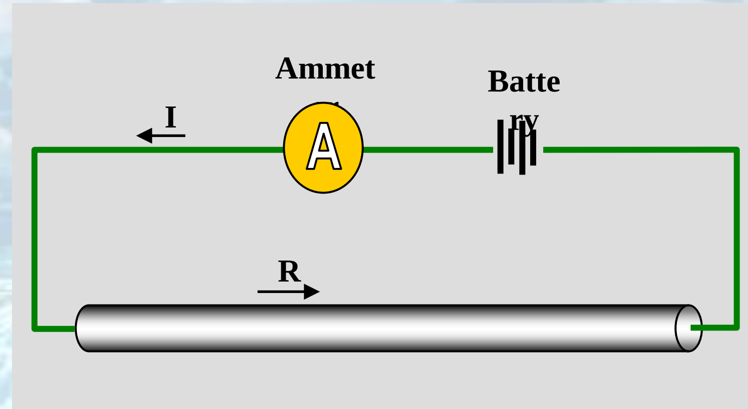
Two different field procedures are used :

Electrical profiling **used for establishing boundaries between different materials and has practical application in prospecting for sand and gravel deposits or ore deposits.**

Electrical sounding **used to provide information on the variation of subsurface conditions with depth and has practical application in indicating layered conditions and approximate thicknesses.**

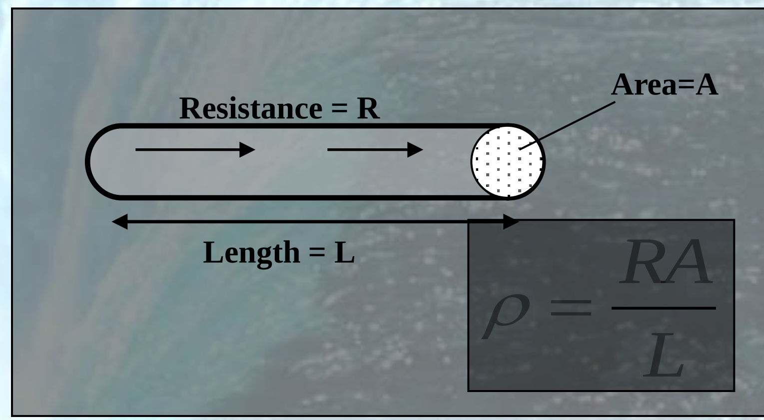
Concept on electrical resistivity

- Ohm's Law, V=IR, R=V/I
- The geometrically independent quantity is called *resistivity*.

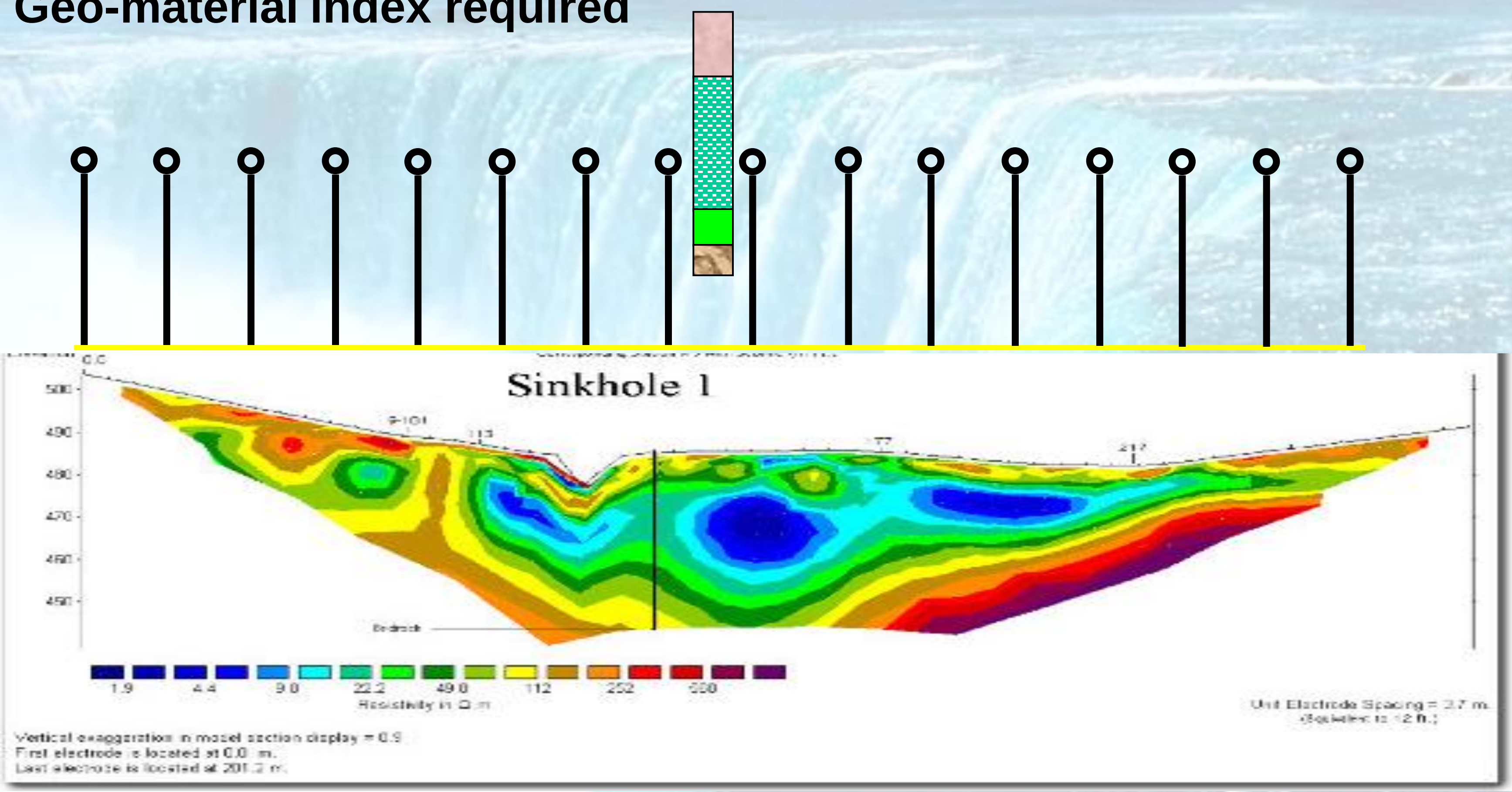


- **Resistivity is a fundamental parameter of a material and describes how easily a wire or the material can transmit an electrical current.**

- **<u>Resistance is a characteristic</u> of a particular path of an electrical current whereas <u>resistivity is a physical property</u> of a material.**



- **Electrical Resistivity Method**
- **Significant in investigating subsurface profile**
- **Image need to be interpreted**
- **Geo-material index required**



FIELD MEASUREMENT

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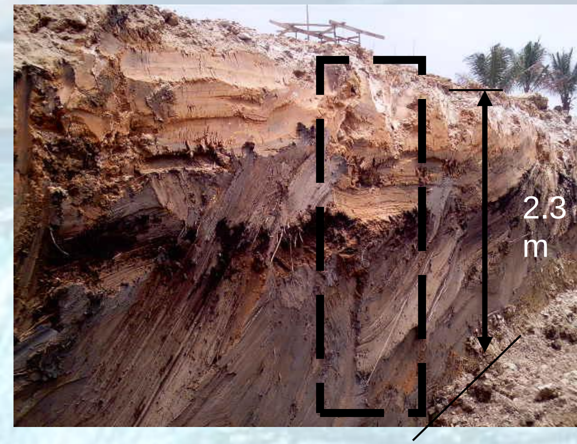
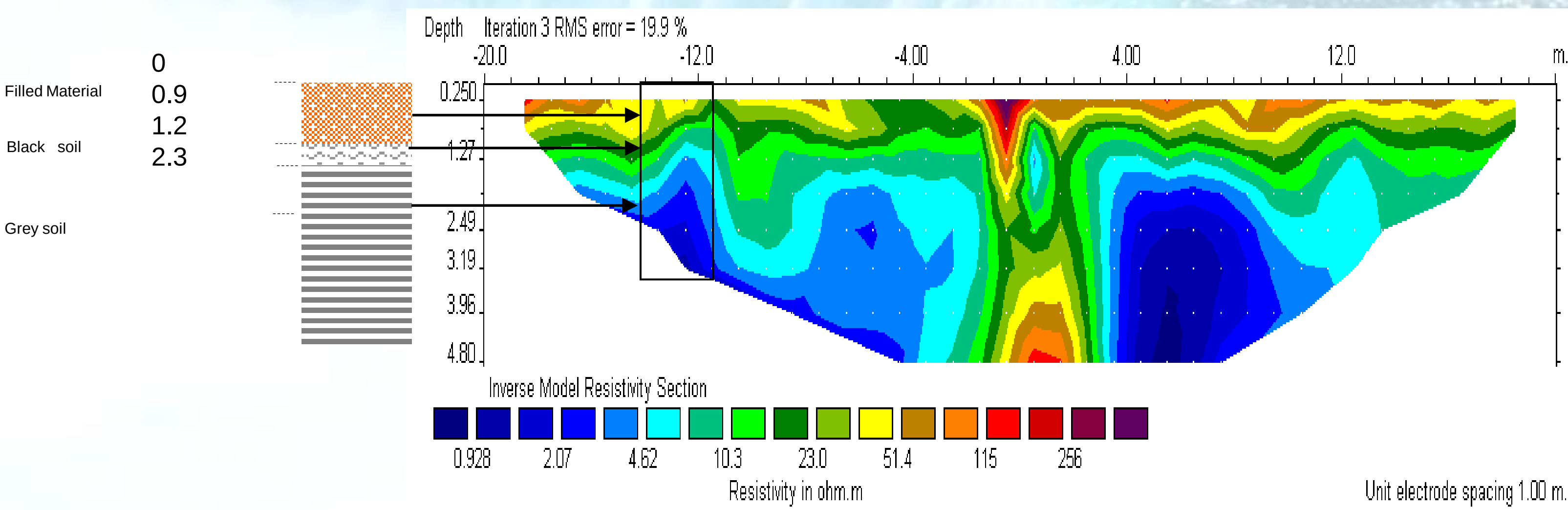
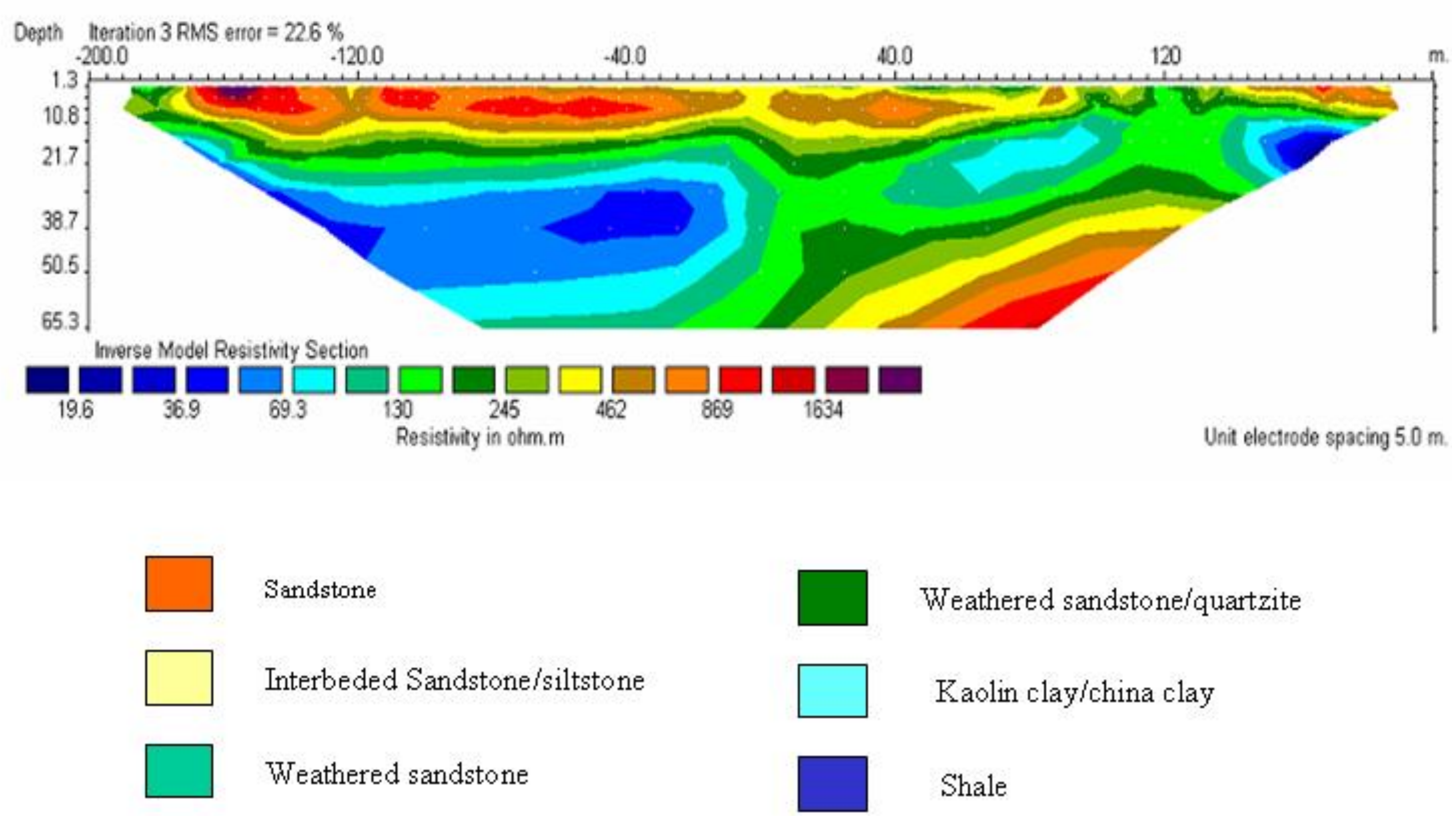
Electrical Resistivity on Marine Clay Deposit





JPS ground water survey for projek tanaman cili Diraja Kelantan

LRAPAM_1



Representative Resistivity values

Types of Materials	Resistivity (ohm-ft)
Wet-to-moist clayey soils	5 - 10
Wet-to-moist silty clay and silty soils	10 - 50
Wet-to-moist silty and sandy soils	50 - 500
Well-fractured to slightly fractured bedrock with moist soil filled cracks	500 - 1000
Sand and gravel with silt	1000
Slightly fractured bedrock with dry soil-filled cracks; sand and gravel with layers of silt	1000 - 8000
Massive bedded and hard bedrock; coarse dry sand and gravel deposits	8000 +

4.3.3 Ground-penetrating Radar

Also identified as ground-probing radar. Capable of defining the shallow zones of soil and rock materials that underlie an area.

The method relies on the penetration and reflection of high frequency radio waves.

4.1.5 Plate Bearing Test

Plate Bearing Test

Estimated immediate settlement of a foundation (Terzaghi and Peck, 1967) is given by the expression :

$$s_B = s_b \left\{ \frac{2B}{B+b} \right\}^2$$

where

s_b = settlement of a test plate of side dimension *b*

s_B = settlement of a foundation of side dimension *B* at the same intensity of loading.

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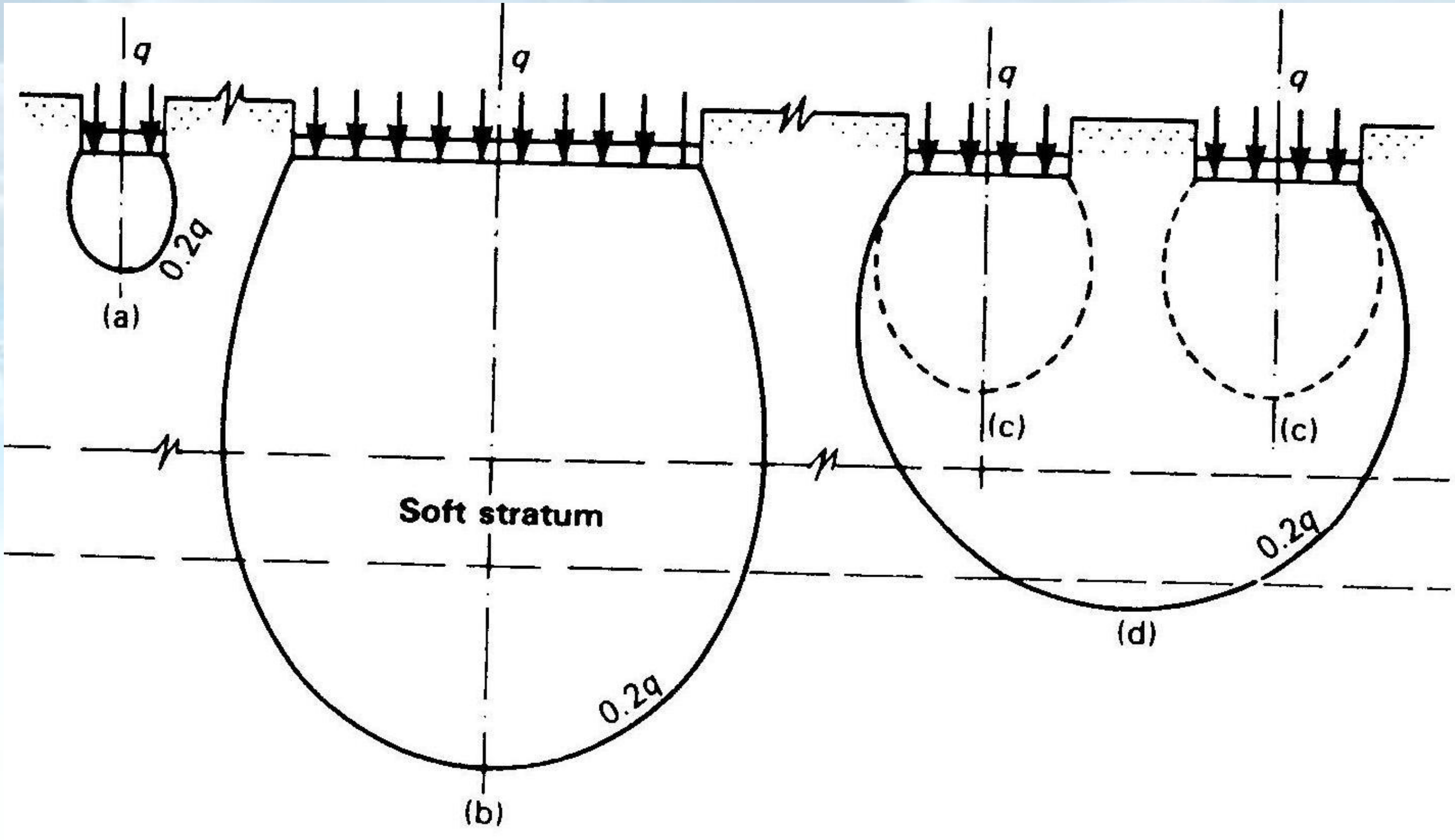
PBT - Application



Pressure bulbs indicating depth to which soil is significantly stressed – be mindful about misleading prediction based on PBT !!!