Geotechnical Site Investigation
measured & derived geotechnical parameters
Part ONE

Common *in situ* tests

SPT (standard penetration test)

CPT (cone penetration test)

FVT (vane shear test)

DMT (dilatometer test)

PMT (pressuremeter test)

Permeability test

The Purpose

1. To fully understand the tasks we are carrying out

    Standard Penetration Test: it is a very boring job; it is so simple any one can do it.

    SPT should be carried out properly so that the result will approximately reflect the undrained shear strength of soil and soft-rocks.

2. To be aware of derived parameters used as engineering design parameters

    SPT –N needs to be corrected (N_{60} , $N_{1(60)}$) to obtain derived geotechnical design parameters

3. To appreciate the basic foundation engineering design methods

    ASD: allowable stress design

    LRFD: load & resistance factor design

ASD vs LRFD

Allowable Stress Design (ASD)

ASD: $R_n/FS \geq \sum Qi$

Resistance ≥ Effects of Loads

Limitations

- Does not adequately account for the variability of loads and resistance
- Does not embody a reasonable measure of strength
- Subjective selection of factor of safety

Load and Resistance Factor Design (LRFD)

LRFD: $R = \varphi R_n \geq \sum \eta_i \gamma_i Q_i = Q$

Limitations

- Require the availability of statistical data and probabilistic design algorithms
- Resistance factors vary with design methods
- Require the change in design procedure from ASD

Explanation

Where

R_n = nominal strength (e. g., ultimate bearing capacity)

$\sum Q_i$ = nominal load effect

FS = factor of safety

R_n = nominal resistance

φ = statistically-based resistance factor

η_i = load modifier to account for ductility, redundancy and operational importance

γ_i = statistically-based load factor

Q_i = load effect.

LRFD: *load & resistance factor design*

LRFD approach applies separate factors to account for uncertainties in loads and resistances based on the reliability theory.

Reliability-based design takes into account the statistical variability by using the mean and the standard deviation (or the coefficient of variation) of all loads and resistance parameters. Given a set of loads and resistance parameters the process can calculate the "probability of failure".

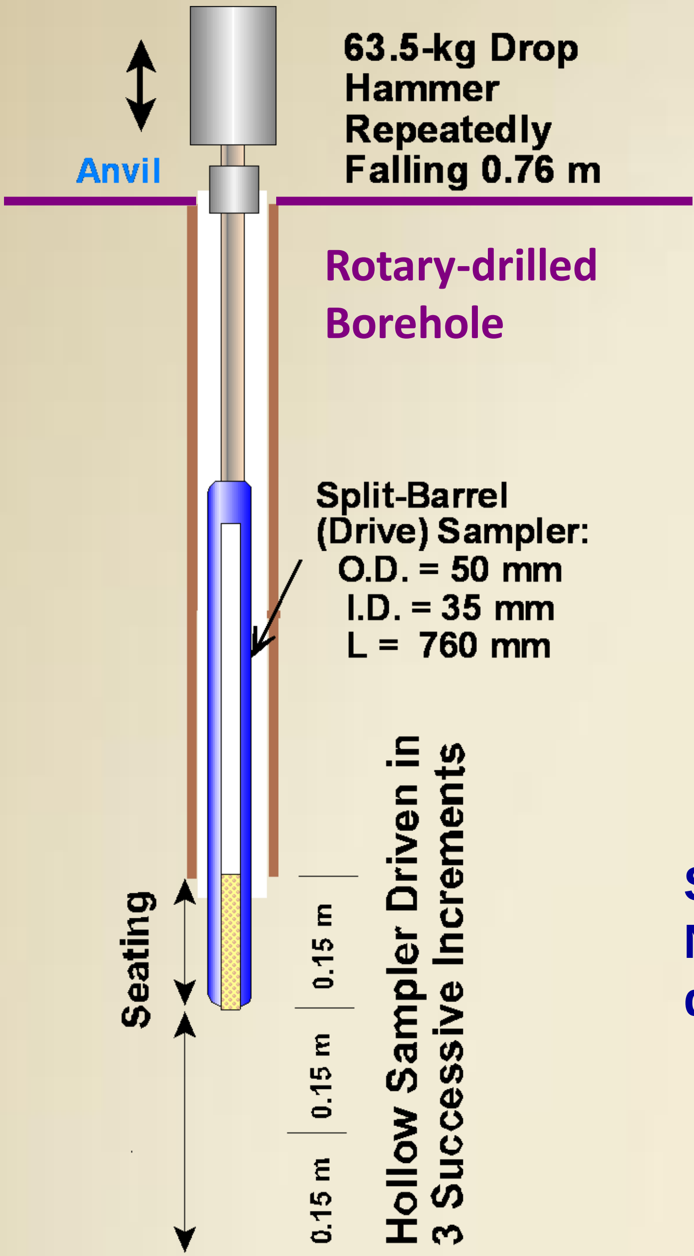
In the LRFD method, external loads are multiplied by load factors while the soil resistances are multiplied by resistance factors.

LRFD recognizes the difference in statistical variability among different loads by using different multipliers for different loads.

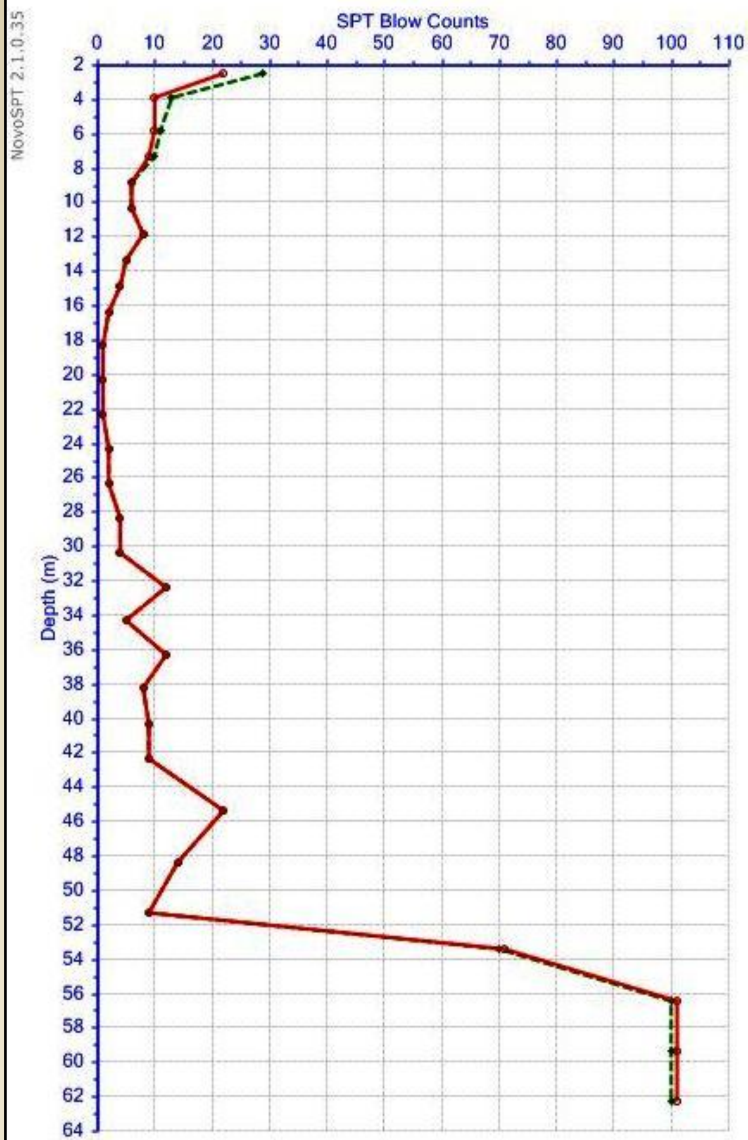
Load and resistance can be modeled by a normal or log normal probability density function based on its distribution characteristics.

$SPT\text{-}N$, N_{60}, $N_{1(60)}$, $N_{1(60)sc}$
&
derived parameters

Standard Penetration Test



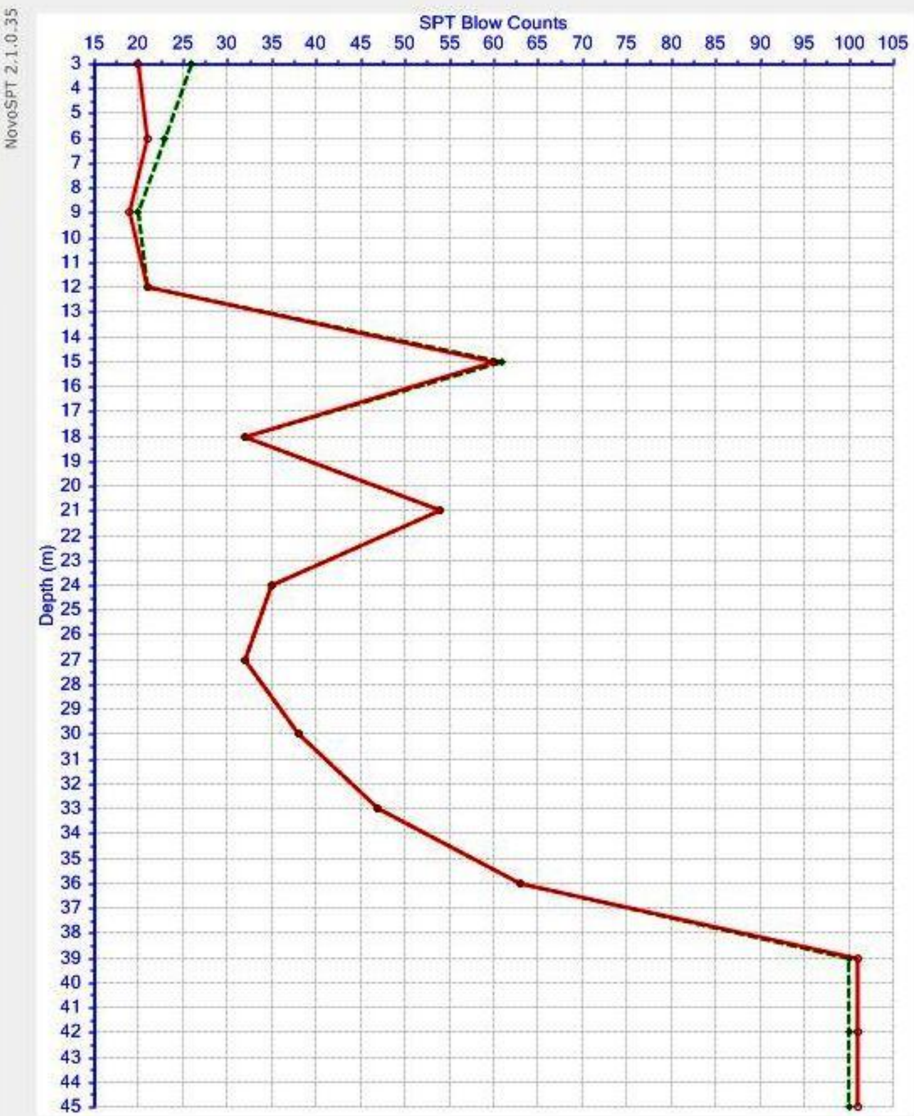
Standard Penetration Test (SPT)
N = measured Number of Blows to drive sampler 300 mm into soil.



Data

Depth (m)	SPT N	SPT N60=N.Ce.Cs.Cr.Cb
2.46	29	22
3.95	13	10
5.85	11	10
7.35	10	9
8.85	6	6
10.35	6	6
11.85	8	8
13.35	5	5
14.85	4	4
16.35	2	2
18.35	1	1
20.35	1	1
22.35	1	1
24.35	2	2
26.35	2	2
28.35	4	4
30.35	4	4
32.35	12	12
34.35	5	5
36.35	12	12
38.25	8	8
40.35	9	9
42.35	9	9
45.35	22	22
48.35	14	14
51.35	9	9
53.45	70	71
56.44	100	101
59.38	100	101
62.27	100	101

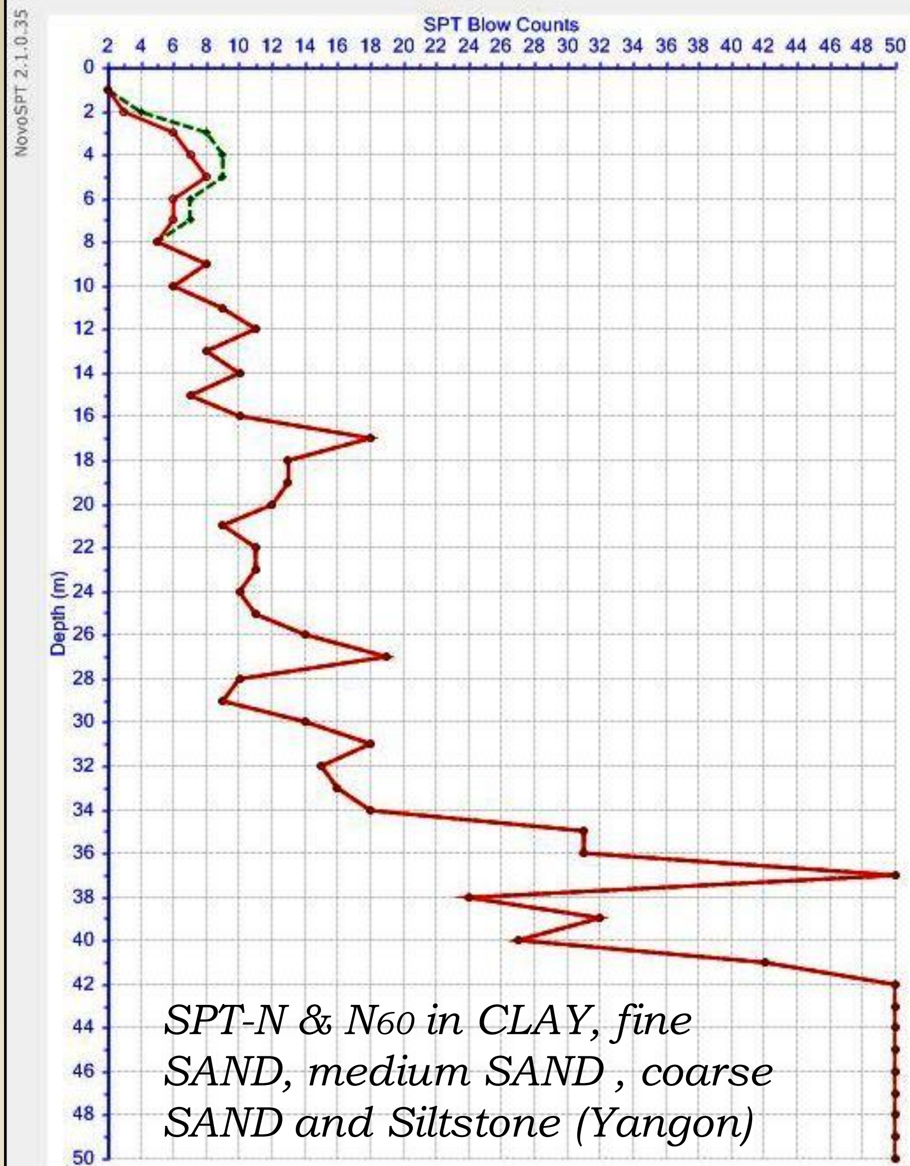
Typical SPT-N & N_{60} in FILL, Marine CLAY and OA



Data

Depth (m)	SPT N	SPT N60=N.Ce.Cs.Cr.Cb
3	26	20
6	23	21
9	20	19
12	21	21
15	61	60
18	32	32
21	54	54
24	35	35
27	32	32
30	38	38
33	47	47
36	63	63
39	100	101
42	100	101
45	100	101

SPT-N and N60 in reclamation area (Sand FILL and OA) at Changi East



SPT-N & N60 in CLAY, fine SAND, medium SAND , coarse SAND and Siltstone (Yangon)

Data

Depth (m)	SPT N	SPT N60=N.Ce.Cs.Cr.Cb
1	2	2
2	4	3
3	8	6
4	9	7
5	9	8
6	7	6
7	7	6
8	5	5
9	8	8
10	6	6
11	9	9
12	11	11
13	8	8
14	10	10
15	7	7
16	10	10
17	18	18
18	13	13
19	13	13
20	12	12
21	9	9
22	11	11
23	11	11
24	10	10
25	11	11
26	14	14
27	19	19
28	10	10
29	9	9
30	14	14
31	18	18
32	15	15
33	16	16
34	18	18
35	31	31
36	31	31
37	50	50
38	24	24
39	32	32

The meaning of SPT- *N* *v*alue

SPT- *N value in sandy soil indicates the friction angle in sandy soil layer*

SPT- *N value in clay soil indicates the stiffness the clay stratum*

Correlation between Fiction Angle (ϕ) & SPT-N Value

Hatakanda and Uchida Equation (1996)

$$\phi = 3.5 \text{ x } (N)^{0.5} + 22.3$$

where, ϕ = friction angle

N = SPT value

Note: *This equation ignores the particle size.*

Most tests are done on medium to coarse sands

Fine sands will have a lower friction angle.

Correlation between Friction Angle (ϕ) SPT(N) Value *contd.*

<u>Hatakanda and Uchida Equation (1996)</u>

Modified

$$\phi = 3.5 \times (N)^{0.5} + 20 \qquad \text{fine sand}$$

$$\phi = 3.5 \times (N)^{0.5} + 21 \qquad \text{medium sand}$$

$$\phi = 3.5 \times (N)^{0.5} + 22 \qquad \text{coarse sand}$$

where, ϕ = friction angle
N = SPT value

Hatakanda, M. and Uchida, A., 1996: Empirical correlation between penetration resistance and effective friction angle of sandy soil. Soils and Foundations 36 (4): 1-9

$$\phi = 53.881 - 27.6034 .\ e^{-0.0147N}$$

Where,

N = average SPT value of strata (soil layer)

Peck, R. *et al.*, 1974. Foundation Engineering. John Wiley & Sons, New York

SPT vs. Coefficient of sub-grade reaction

SPT-N	8	10	15	20	30
k (kN/m^3)	2.67E-6	4.08E-6	7.38E-6	9.74E-6	1.45E-5

Johnson, S. M, and Kavanaugh, T. C., 1968. The Design of Foundation for Buildings. McGraw-Hill, New York.

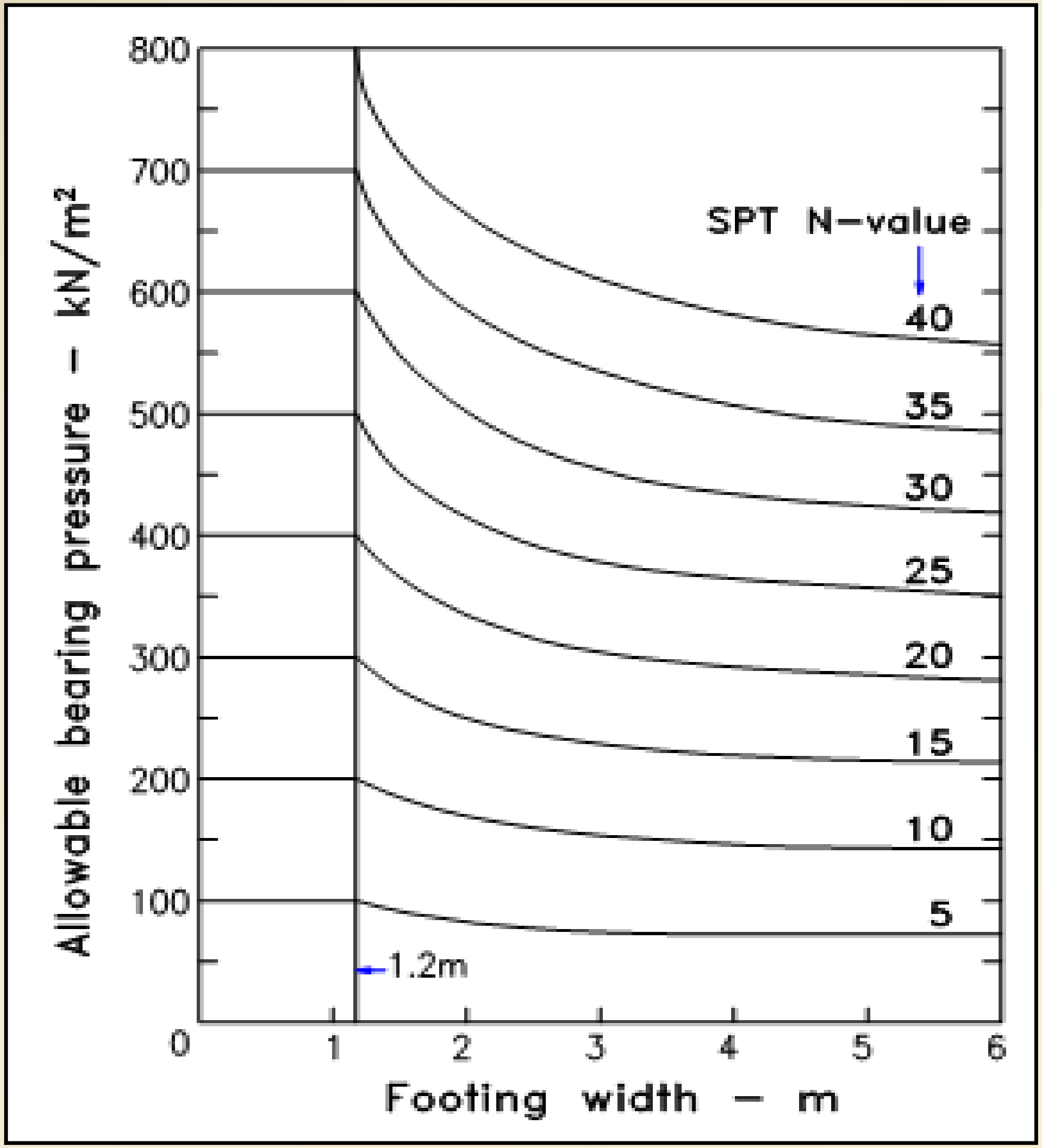
ASD: *allowable stress design based on SPT-N*

$Q_{allowable}$ = 1.5 N ksf (Meyerhoff, 1956),
1.0 N ksf (Terzaghi and Peck, 1967),
0.37 N ksf (Strounf and Butler, 1975), and
0.5 N ksf (Reese, Touma, and O'Neill, 1976)
(1 ksf = 47.88 kPa)

- All these empirical formulas for the allowable end bearing capacity were proposed by different researchers and practitioners assuming a factor of safety of 2.5.
- All uncertainty is embedded in the factor of safety (FS).

- These formula gears towards ASD, for it predicts the allowable soil and rock resistances using the SPT blow count (N) alone.
- Allowable stress design (ASD) treats each load on a structure with equal statistical variability.

Allowable bearing pressure for footing of settlement limited to 25 mm
(Bowles, 1982)



Rule of thumb methods to compute bearing capacity

Bearing capacity of FINE SAND:

Allowable bearing capacity (kPa) $= 9.6\ N_{average}$ *(not to exceed 380 kPa)*

$= 0.2\ N_{average}$ *(not to exceed 8 ksf)*

Procedure

Step 1. Find the average SPT-N value below the bottom of footing to a depth equal to width of the footing.

Step 2. If the soil within this range is fine sand, the above rule of thumb can be used.

Rule of thumb methods to compute bearing capacity *contd.*

Bearing capacity of Medium to Coarse SAND:

Allowable bearing capacity (kPa) = 9.6 $N_{average}$ *(not to exceed 575 kPa)*

= 0.2 $N_{average}$ *(not to exceed 12 ksf)*

Procedure

Step 1. Find the average SPT-N value below the bottom of footing to a depth equal to width of the footing.

Step 2. If the soil within this range is medium to coarse sand, the above rule of thumb can be used.

Note: if the average SPT-N value is < 10, soil should be compacted.

SPT-N corrections

Corrected SPT: N_{60} & $N_{1(60)}$

$$N_{60} = N_m \times C_E \times C_S \times C_B \times C_R$$

$$N_{1(60)} = C_N \times N_{60}$$

Where,

N_m = SPT measured in field

C_N = overburden correlation factor = $(P_a/\sigma')^{0.5}$

P_a = 100 kPa

σ' = effective stress of soil at point of measurement

C_E = energy correlation factor for SPT hammer, safety hammer(0.6 – 0.85); donut hammer (0.3-0.6); automatic hammer (0.8-1.0)

C_B = borehole diameter correction, 65 – 115 mm (1.0); 150 mm(1.05); 200 mm (1.15)

C_R = rod length correlation, <3m (0.75); 3 – 4m, 0.8, 4-6m, 0.85; 6-10m, 0.95; 10-30m, 1.0)(i.e., adjustment for weight of rods)

Cs = sampling method, standard sampler (1.0); sampler w/o liner (1.1-1.3)

Bearing capacity methods using N_{60}

Meyerhof, 1976 (based on 25mm settlement)

$$q_a = N_{60}.Kd/F1 \qquad B \le F4$$

$$q_a = N_{60}.Kd.(B+F3)/(B.F2) \qquad B > F4$$

where

$Kd = 1 + D_f/(3B) \le 1.33$,

F1 to F4 defined as SI units:

- F1=0.05 , F2=0.08 , F3=0.30, F4=1.20
- N_{60} = average SPT blow counts from 0.5B above to 2B below the foundation level.

Bearing capacity methods using N60 (*contd.)*

Burland and Burbidge, 1985 (based on 25 mm settlement)

$$q_a = 2540.N_{60}^{1.4}/(10^T.B^{0.75})$$

Where

N_{60} = average SPT blow counts to a depth of $B^{0.75}$ below footing

$T \sim 2.23$

Parry, 1977 (based on 25mm settlement)

The allowable bearing capacity for cohesionless soil

$$q_a = 30N_{60} \qquad D_f \leq B$$

Where

N_{60} = average SPT blow counts below 0.75B underneath the footing.

General Terzaghi Formula

The following Terzaghi equation is used for indirect estimation of bearing capacity of shallow footing on cohesionless soil.

$$q_{ult} = (qN_q)+(0.5\gamma BN_\gamma)$$

where:

q = the overburden stress at foundation level (D_f).

$N_q = e^{[\pi.\tan(\phi)]}\ [\tan(\pi/4+\phi/2)]^2$ Bowles 1996

$N_\gamma = 1.5(N_q-1).\tan(f)$ Brinch & Hansen 1970

ϕ = friction angle correlated by Hatanaka and Uchida (1996) equation, based on SPT at foundation level

$N_{1(60)}$

Peck, 1974

Allowable bearing capacity using $N_{1(60)}$

$$q_a = 10.6 N_{1(60)}$$

$N_{1(60)} = Cn.N_{60}$

Example computation using a SPT program (NovoSPT pro 2.1.035)
SPT data: Marina South

- ❑ A shallow foundation is placed on SAND
- ❑ The footing depth (Df) is 4.15 m below ground level (where sand layer starts)
- ❑ The footing width (B) is 1.0 to 3.0 m
- ❑ Shear Failure Safety Factor is 3.0

  Note: Safety factor is applied only to Terzaghi method. Others are based on 25 mm settlement.

Soil parameters

- ✓ ϕ (Hatanaka & Uchida, 1996) = 32.1
- ✓ N_q (Bowles, 1996) = 23.45
- ✓ N_γ (Brinch & Hansen, 1970) = 21.12
- ✓ N_{60} = 7; $N_{1(60)}$ = 8
- ✓ Effective stress at D_f (kPa) = 76.91

Bearing Capacity (kPa) results

for comparison

Equation	B=1m	B=1.5m	B=2m	B=2.5m	B=3m
Burland and Burbidge, 1985 (25mm settlement)	228	168	216	182	159
Bowles/Meyerhof, 1976 (25mm settlement)	259	195	201	200	208
Parry, 1977 (25mm settlement)	Df>B				
Terzaghi (Ultimate)	652	677	703	728	754

End bearing capacity of piles in sandy soil

$$q = c \times N \ (MN/m^2)$$

$$q = 20.88 \times c \times N \ (ksf)$$

q = end bearing capacity of the pile

Total end bearing = q x area $(\pi d^2/4)$

N = SPT-N value (per 30.48cm)

c = 0.45 for pure sand

c =0.35 for silty sand

Martin, R. E, Seli, J. J., Powell, G. W. , and Bertoulin, M. 1987. Concrete Pile Design in Tidewater Virginia. *ASCE Journal of Geotechnical Engineering 113(6):568-585.*

End bearing capacity in Clay (driven pile)

Skempton (1959)

$$q = 9 \,.\, C_u$$

q = end bearing capacity

C_u = cohesion of soil at tip of pile

Cf#

Martin et al., 1987

$$q = C \,.\, N \quad MN/m^2$$

$C = 0.20$ N = SPT value at pile tip

End bearing capacity in Clay (bored pile)

Shioi and Fukui (1982)

$$q = C \, . \, N \quad MN/m^2$$

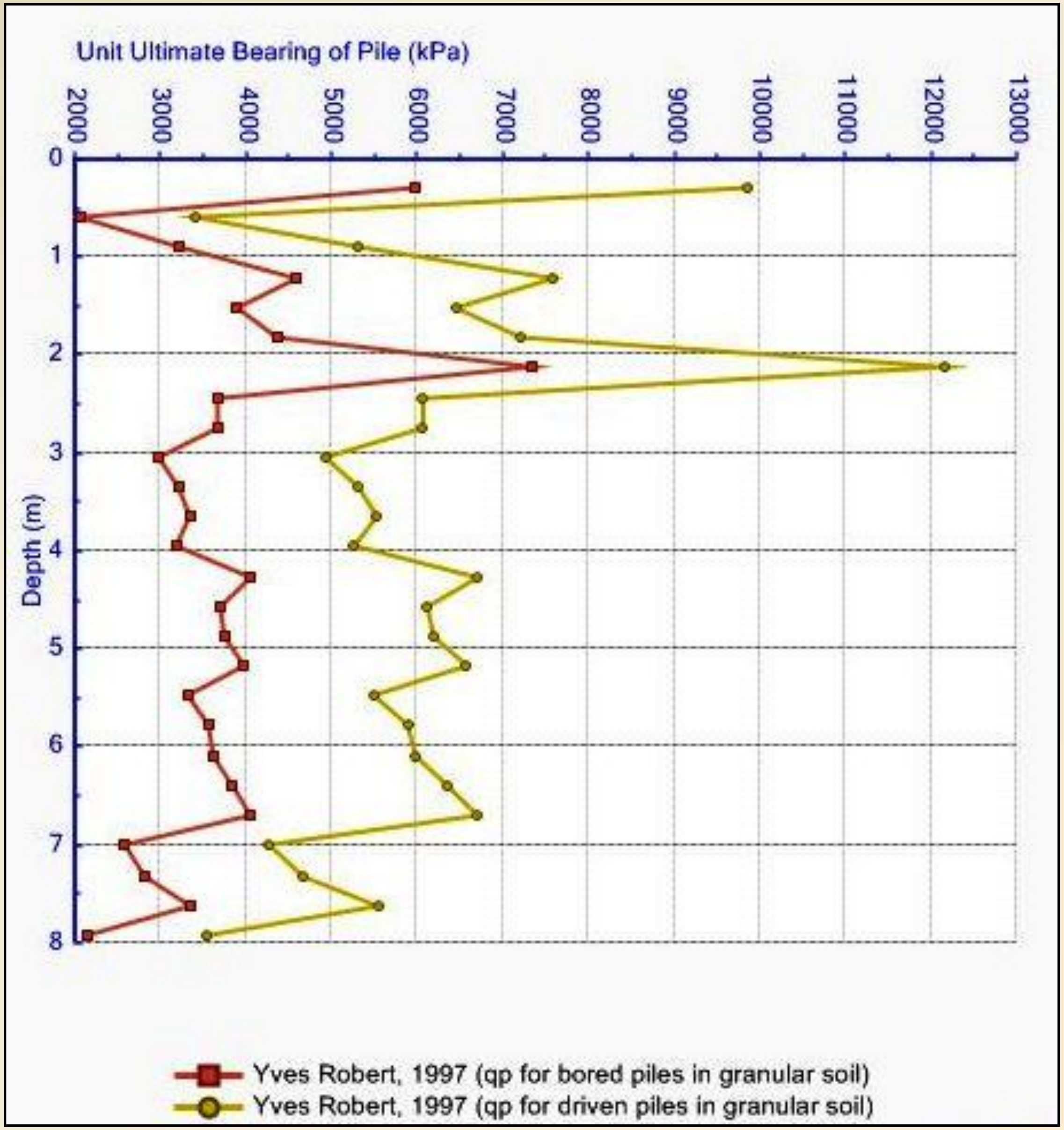
q = end bearing capacity

$C = 0.15$

N = SPT value at pile tip

Unit Ultimate Bearing Capacity of piles

Example using NovoSPT

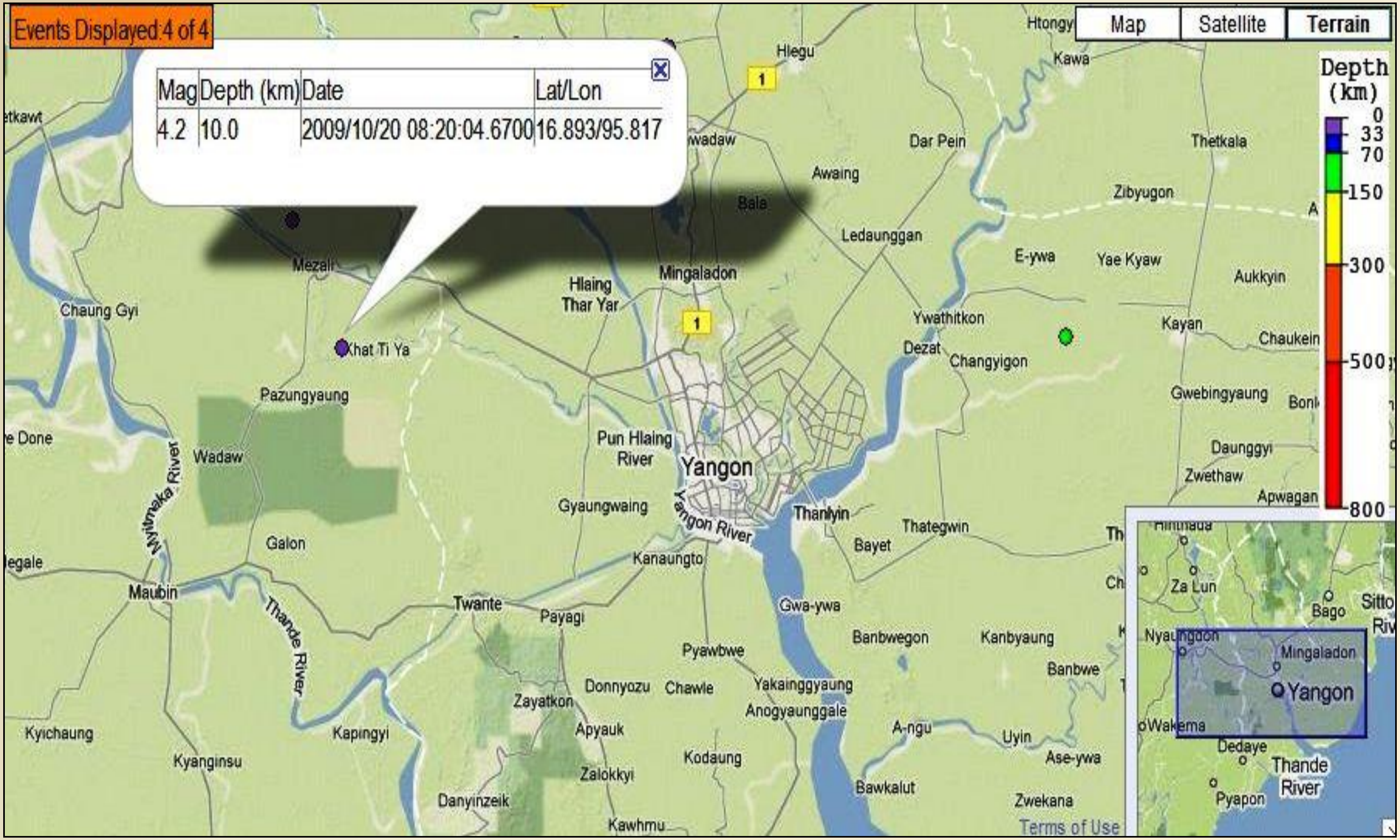


Liquefaction

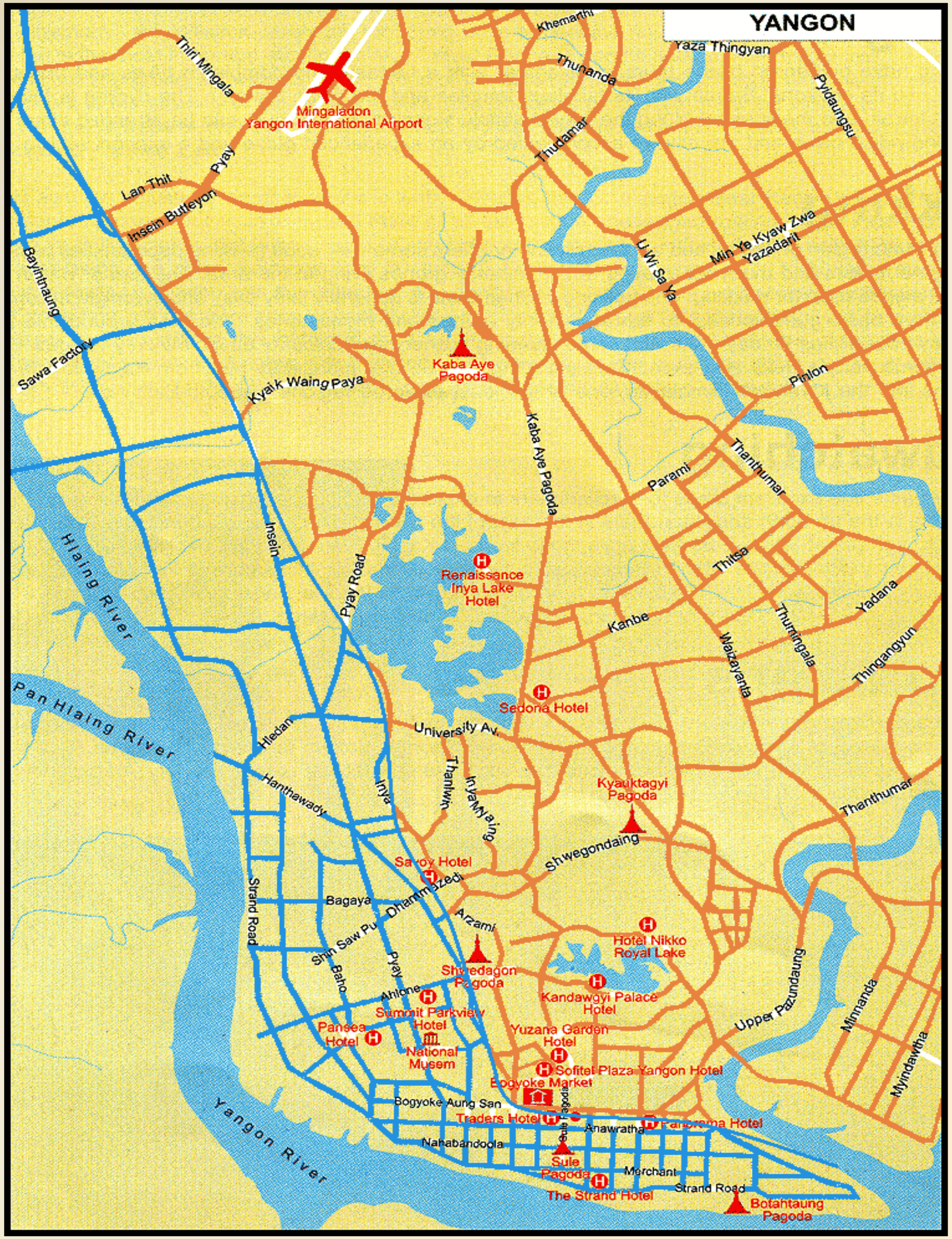
Sandy and silty soils have tendency to lose strength and turn into a liquid-like state during earthquakes.

This is due to increase in pore pressure in the soil caused by seismic waves.

Iris EQ web browser data

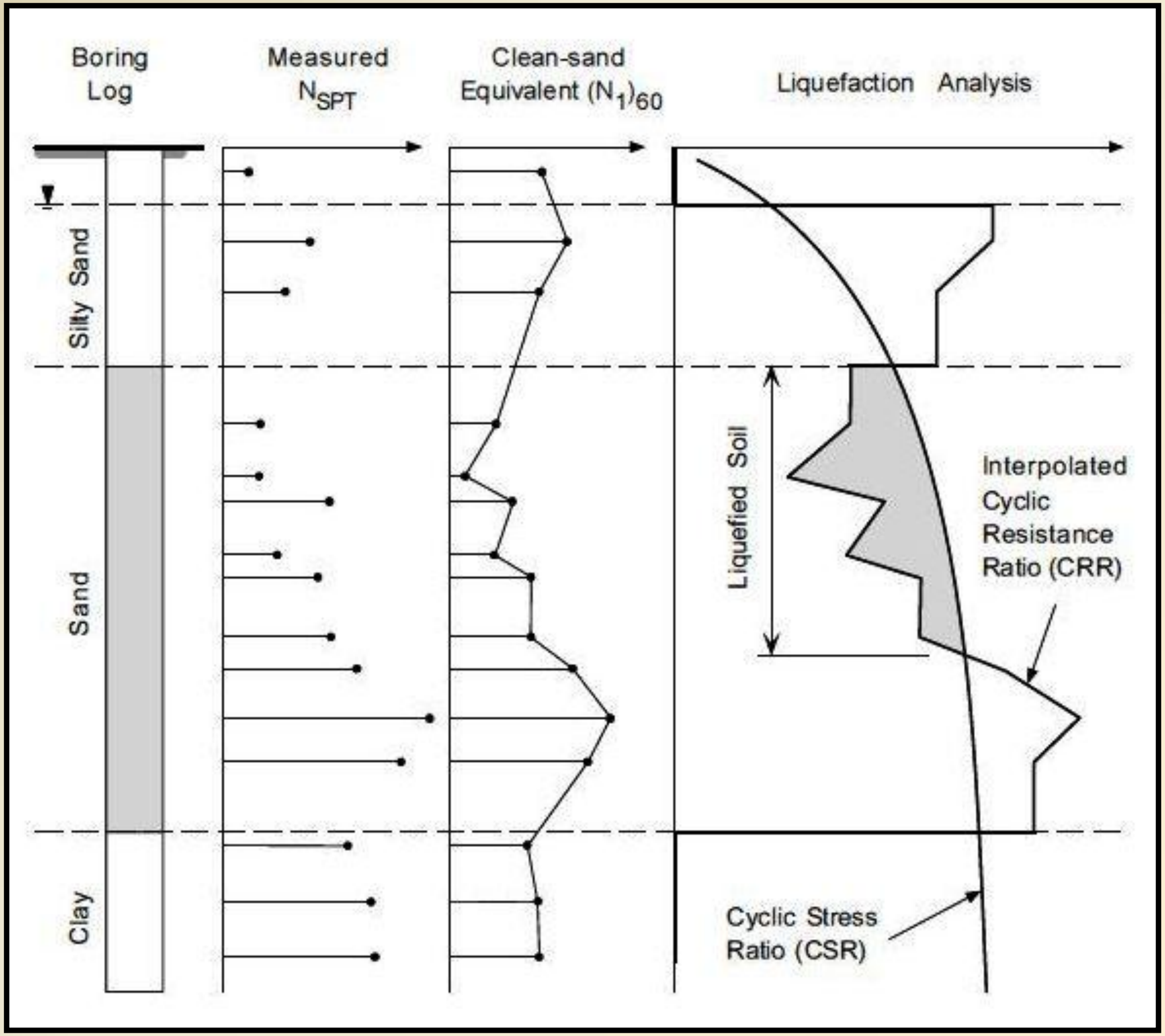


Yangon west Unconsolidated Sediments



Site ONE: liquefaction analysis for foundation

LQF Analysis by SPT-N



CSR (cyclic stress ratio) or SSR (seismic stress ratio)

$$CSR = \frac{\tau_{ave}}{\sigma'_{vo}} = 0.65 \frac{a_{max}}{g} \frac{\sigma_{vo}}{\sigma'_{vo}} r_d$$

where:

CSR	=	Cyclic stress ratio.
a_{max}	=	Soil surface of acceleration.
g	=	Acceleration due to gravity.
σ'_{vo}	=	Initial effective vertical stress at depth z.
σ_{vo}	=	Total vertical stress at depth z.
r_d	=	Dimensionless parameter that accounts for the stress reduction due to soil column deformability.

CRR (cyclic resistance ratio): soil resistance to liquefaction

A general rule is that any soil that has an SPT value higher than 30 will not liquefy.

For clean sand with less than 5% fines,

$CRR_{7.5}$= $1/[34-(N_1)_{60}]+(N_1)_{60}/135+50/[10\text{x}(N_1)_{60}+45]^2 - 2/200$

$CRR_{7.5}$ = soil resistance to liquefaction for an earthquake with a magnitude of 7.5 Richter

Note: correlation factor is needed for other magnitudes

$N_{1,60cs}$

(NCEER 1997, Youd et al. 2001)

$$N_{1,60cs} = \alpha + \beta N_{1,60}$$

where:

$$\alpha = \begin{cases} 0 & \text{for } FC \leq 5\% \\ \exp[1.76 - (190/FC^2)] & \text{for } 5\% < FC \leq 35\% \\ 5.0 & \text{for } FC > 35\% \end{cases}$$

$$\beta = \begin{cases} 1.0 & \text{for } FC \leq 5\% \\ [0.99 - (FC^{1.5}/1000)] & \text{for } 5\% < FC \leq 35\% \\ 1.2 & \text{for } FC > 35\% \end{cases}$$

Summary of liquefaction Analysis

(site classification for seismic site response)

Hledan Kamayut site (site category E & F)	SPT-N1(60) (average)	Dr, %	V s m/sec	Vs (30) (Vs1sc)	Thickness, m	LQF zone* (0.3g, M7.5)
HLD BH-02	6	< 50	189	160 (138)	13.0	10.0 – 23.0 m
HLD BH-06	5	< 45	178	150 (132)	17.0	8.0 – 25.0 m

Bo Soon Pat site (site category E)	SPT-N1(60) (average)	Dr, %	V s m/sec	Vs (30) (Vs1sc)	Thickness, m	LQF zone* (0.3g, M7.5)
BSP BH-04	11	>50	230	185 (160)	20	8-28
BSP BH-08	10	> 55	213	220 (157)	5	8.5-13.5

* below ground level

Note: amplification is greater in lower velocity

expected Site Period ≈ 1.0 s

$(Vs1)cs = 87.7\ [N1(60)cs]^{0.253}$

(after Andrus et al., 2003)

Comments on liquefaction analysis

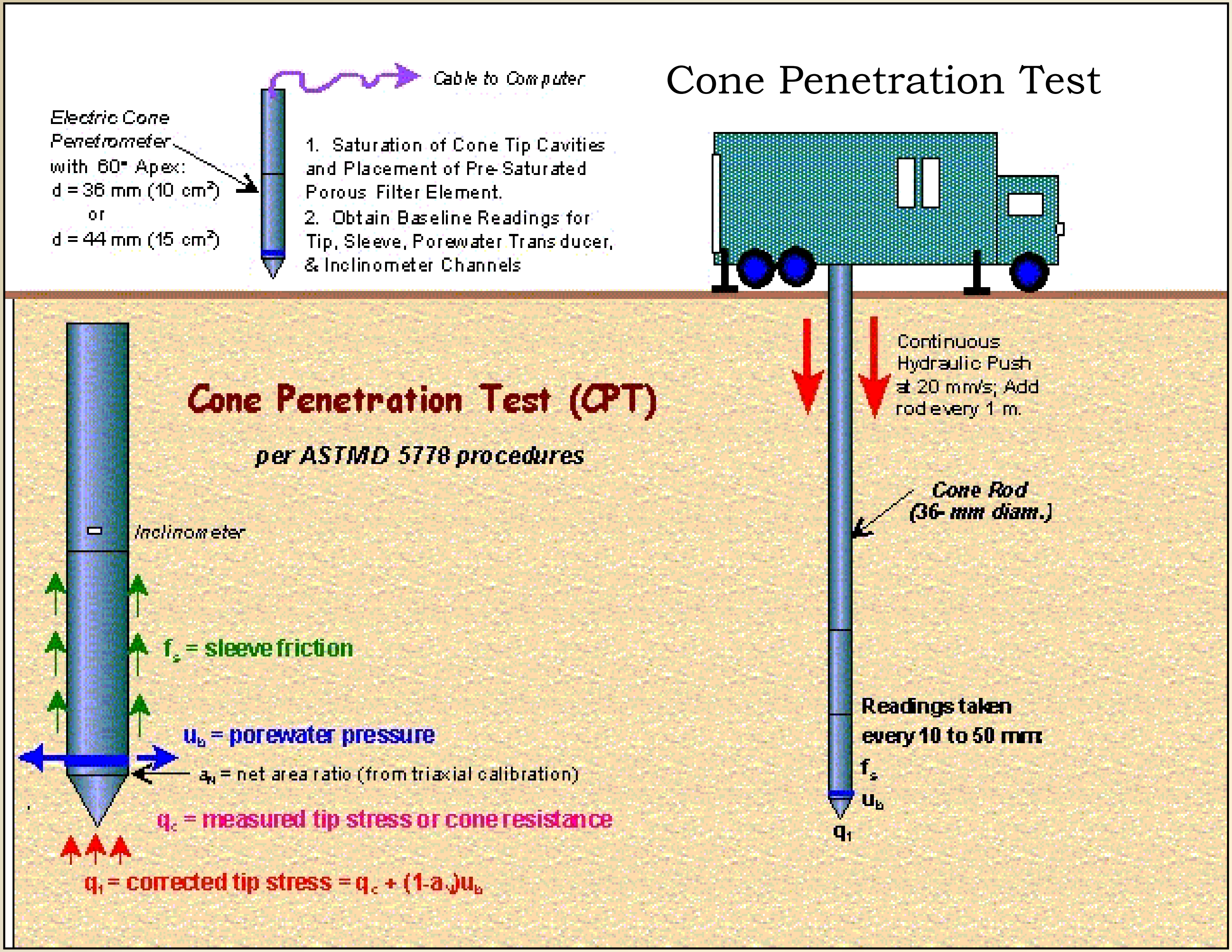
Summary of soil index and post liquefaction parameters

Site		Soil Index			Post LQF Parameters			Site Category
Hledan Kamayut		N1(60)cs	σ', kPa	Dr,%	Shear Strain, % (Max.)	Settlement, cm (Max.)	LDI, cm (Max.)	E & F
Depth, m 8.0 – 25.0	0.2g	5-15	110-280	< 50	50	55	450	
	0.3g				>50	105	1000	
Bo Soon Pat Downtown		N1(60)cs	σ', kPa	Dr,%	Shear Strain % (Max.)	Settlement, cm (Max.)	LDI, cm (Max.)	E
Depth, m 8.0 – 28.0	0.2g	10-18	100-300	35-55	50	20	110	
	0.3g				50	55	450	

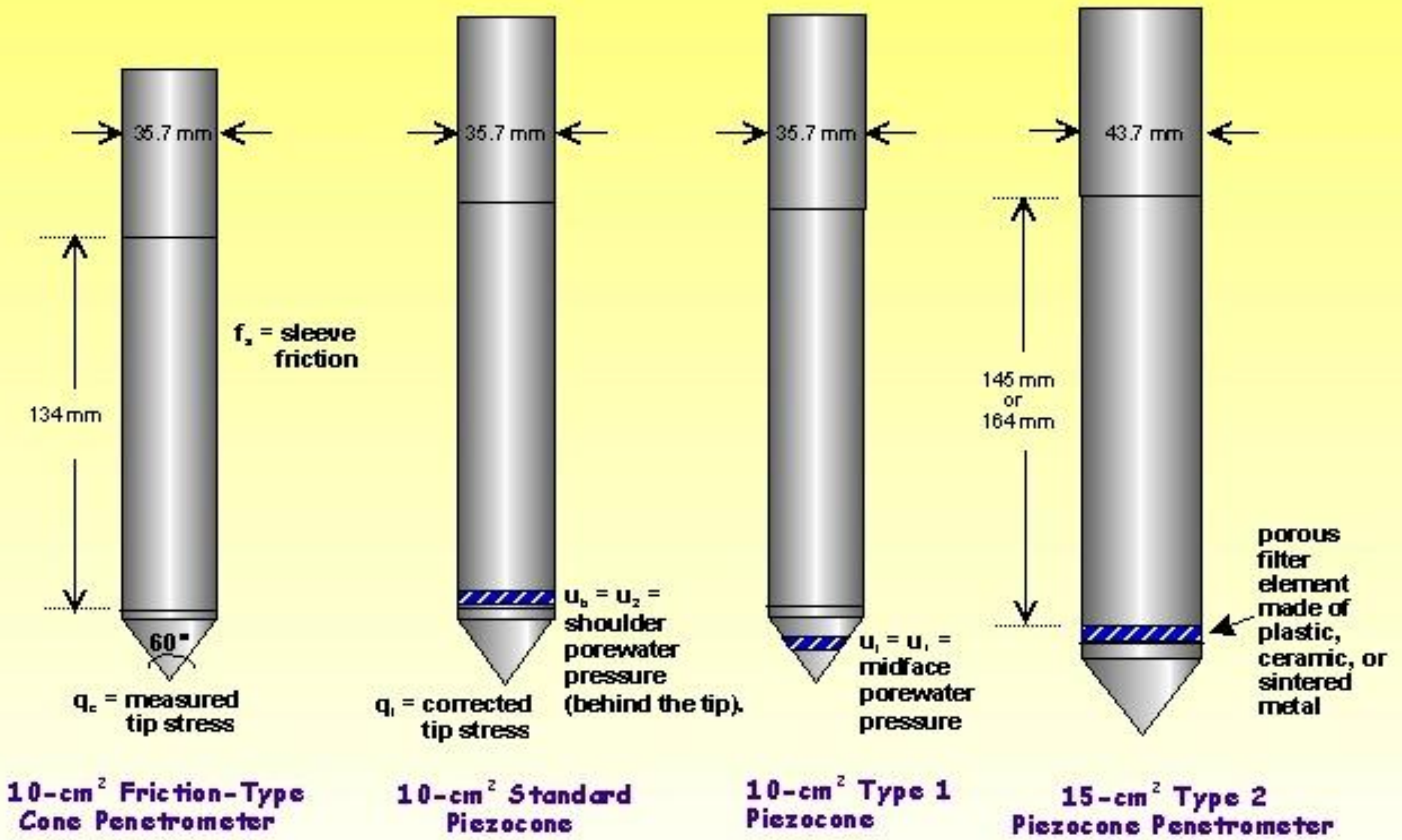
Thickness of penetrated surface layer is about 6.0m (after Obermeier *et al.*, 2005) at both sites at 0.3g.

CPT: *cone penetration test*

Cone Penetration Test



piezocones

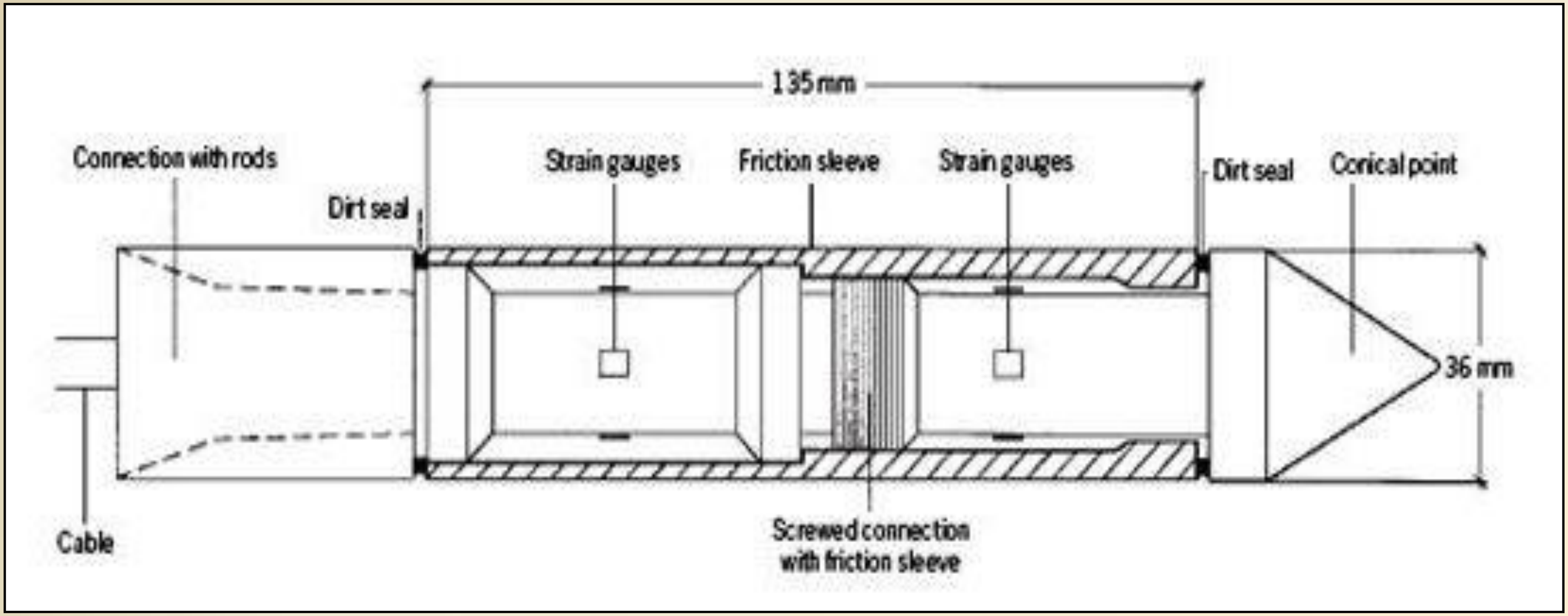

35.7 mm
35.7 mm
35.7 mm
43.7 mm
134 mm
f_s = sleeve friction
60°
q_c = measured tip stress
$u_b = u_2$ = shoulder porewater pressure (behind the tip).
q_t = corrected tip stress
$u_1 = u_t$ = midface porewater pressure
145 mm or 164 mm
porous filter element made of plastic, ceramic, or sintered metal
10-cm² Friction-Type Cone Penetrometer
10-cm² Standard Piezocone
10-cm² Type 1 Piezocone
15-cm² Type 2 Piezocone Penetrometer

GEOTECH AB

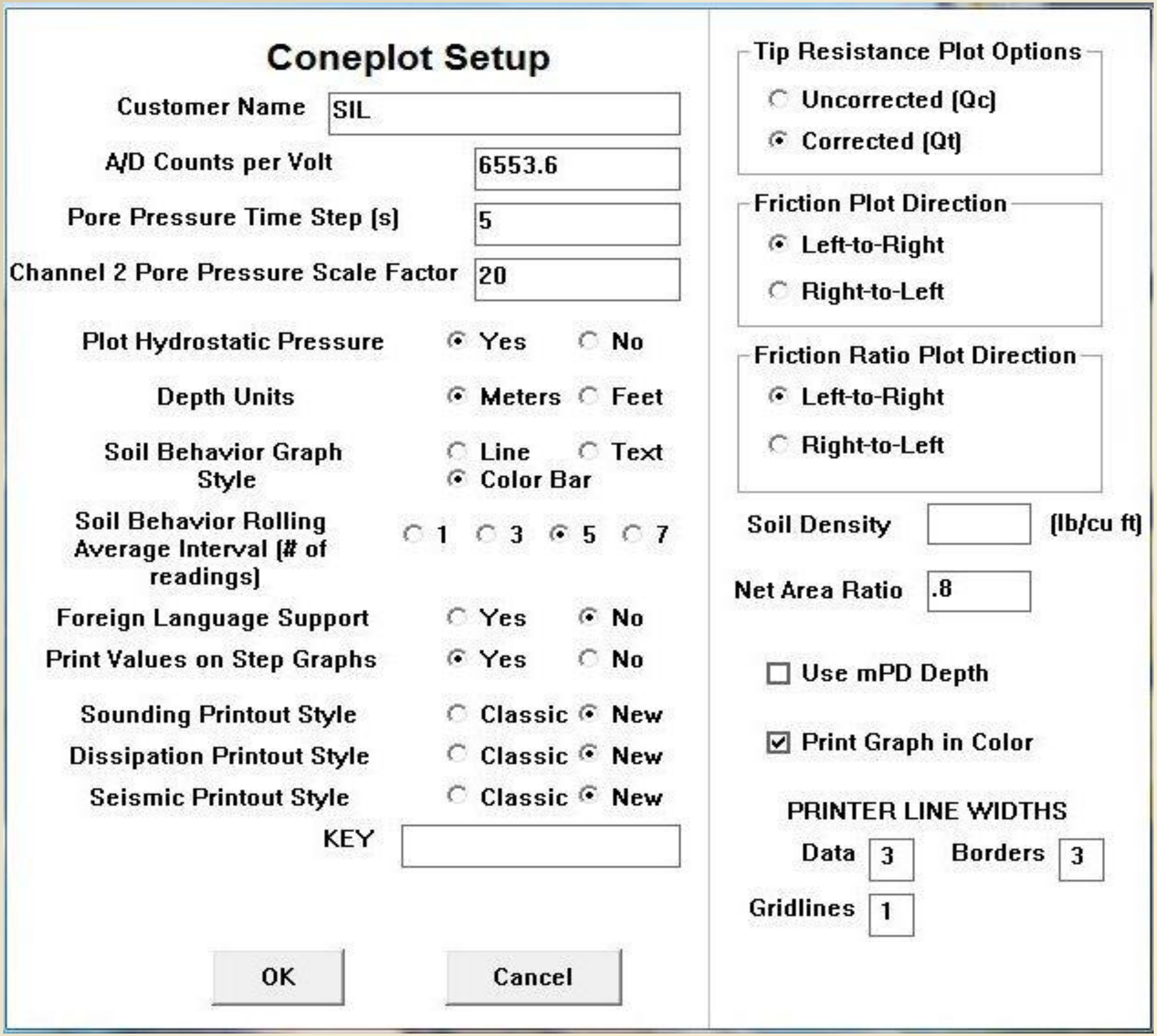
TECHNICAL SPECIFICATIONS

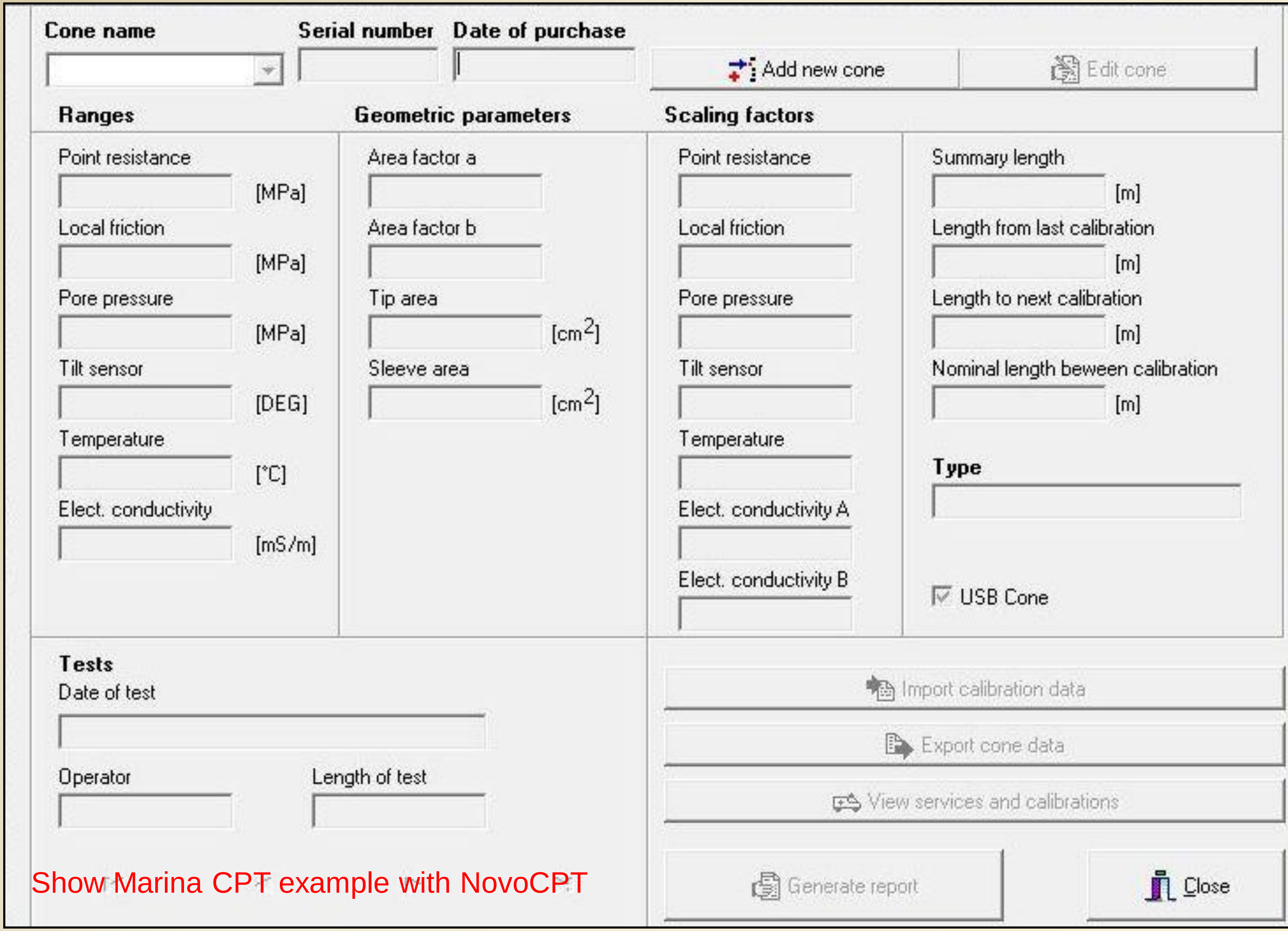
	Nova	Classic
Point resistance (qc)	20, 50, 100 MPa	10, 50, 100 MPa
-Accuracy	< 0.2% FS	< 0.4% FS
-Resolution	< 0.0025% FS	< 0.08% FS
-Net area factor, cone	0,82	0,58
Sleeve friction (fs)	0.5 and 1 MPa	0.5 MPa
-Accuracy	< 0.2% FS	< 0.4% FS
-Resolution	< 0.0025% FS	< 0.08% FS
-Net area factor, sleeve friction	0.0	0.014
Dynamic por pressure (u)	1, 1.5 and 5 MPa	2.5 MPa
-Accuracy	< 0.4% FS	< 0.5% FS
-Resolution	< 0.0025% FS	< 0.08% FS
Tilt sensor	0-40 deg.	0-40 deg.
Weigth	~ 1.25 kg	~ 2,1 kg
Length	230 mm	470 mm

Details of a piezocone



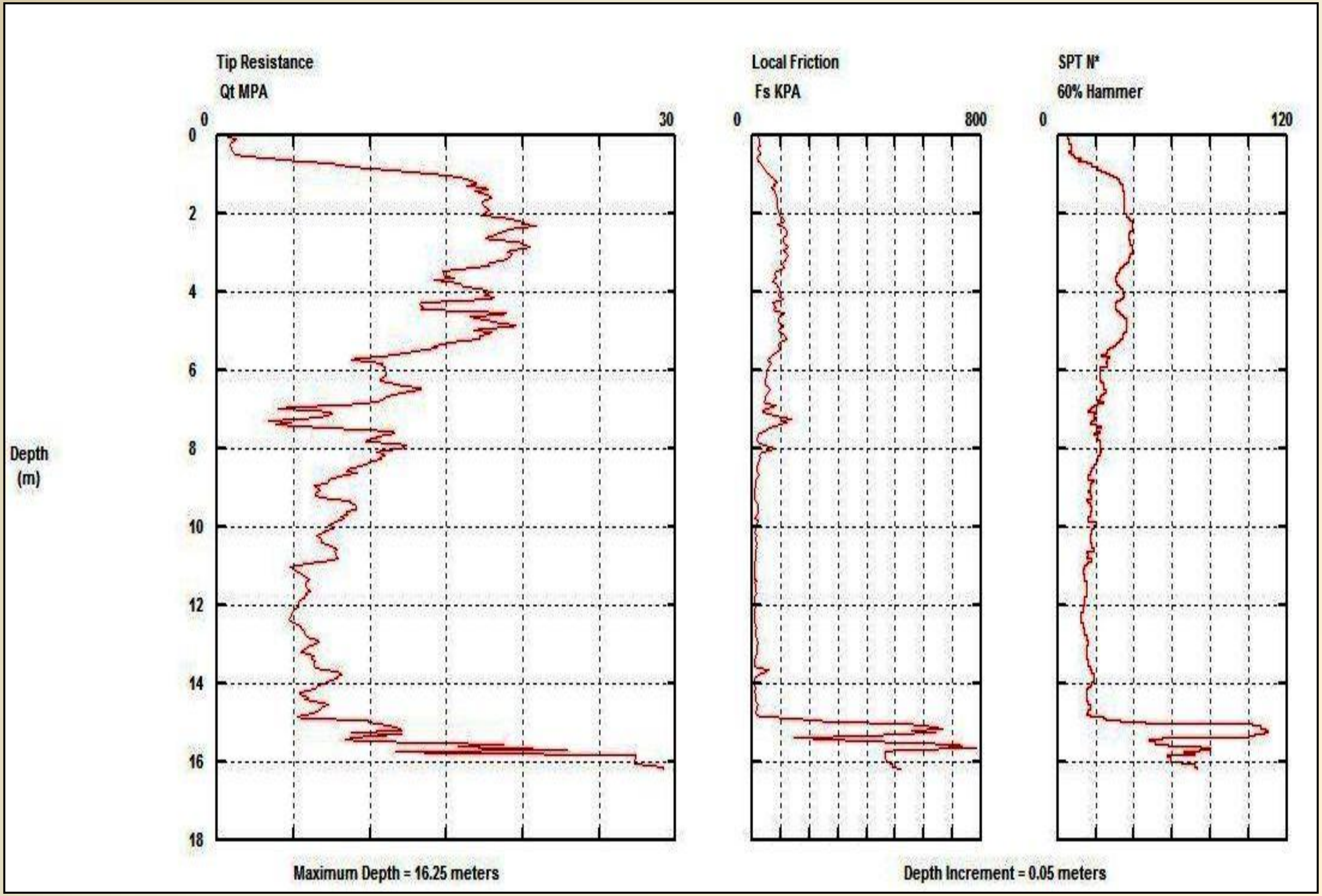
CPT rig set up for operation


Coneplot Setup
Customer Name
SIL
A/D Counts per Volt
6553.6
Pore Pressure Time Step (s)
5
Channel 2 Pore Pressure Scale Factor
20
Plot Hydrostatic Pressure
Yes
No
Depth Units
Meters
Feet
Soil Behavior Graph Style
Line
Text
Color Bar
Soil Behavior Rolling Average Interval (# of readings)
1
3
5
7
Foreign Language Support
Yes
No
Print Values on Step Graphs
Yes
No
Sounding Printout Style
Classic
New
Dissipation Printout Style
Classic
New
Seismic Printout Style
Classic
New
KEY
OK
Cancel
Tip Resistance Plot Options
Uncorrected (Qc)
Corrected (Qt)
Friction Plot Direction
Left-to-Right
Right-to-Left
Friction Ratio Plot Direction
Left-to-Right
Right-to-Left
Soil Density
(lb/cu ft)
Net Area Ratio
.8
Use mPD Depth
Print Graph in Color
PRINTER LINE WIDTHS
Data
3
Borders
3
Gridlines
1

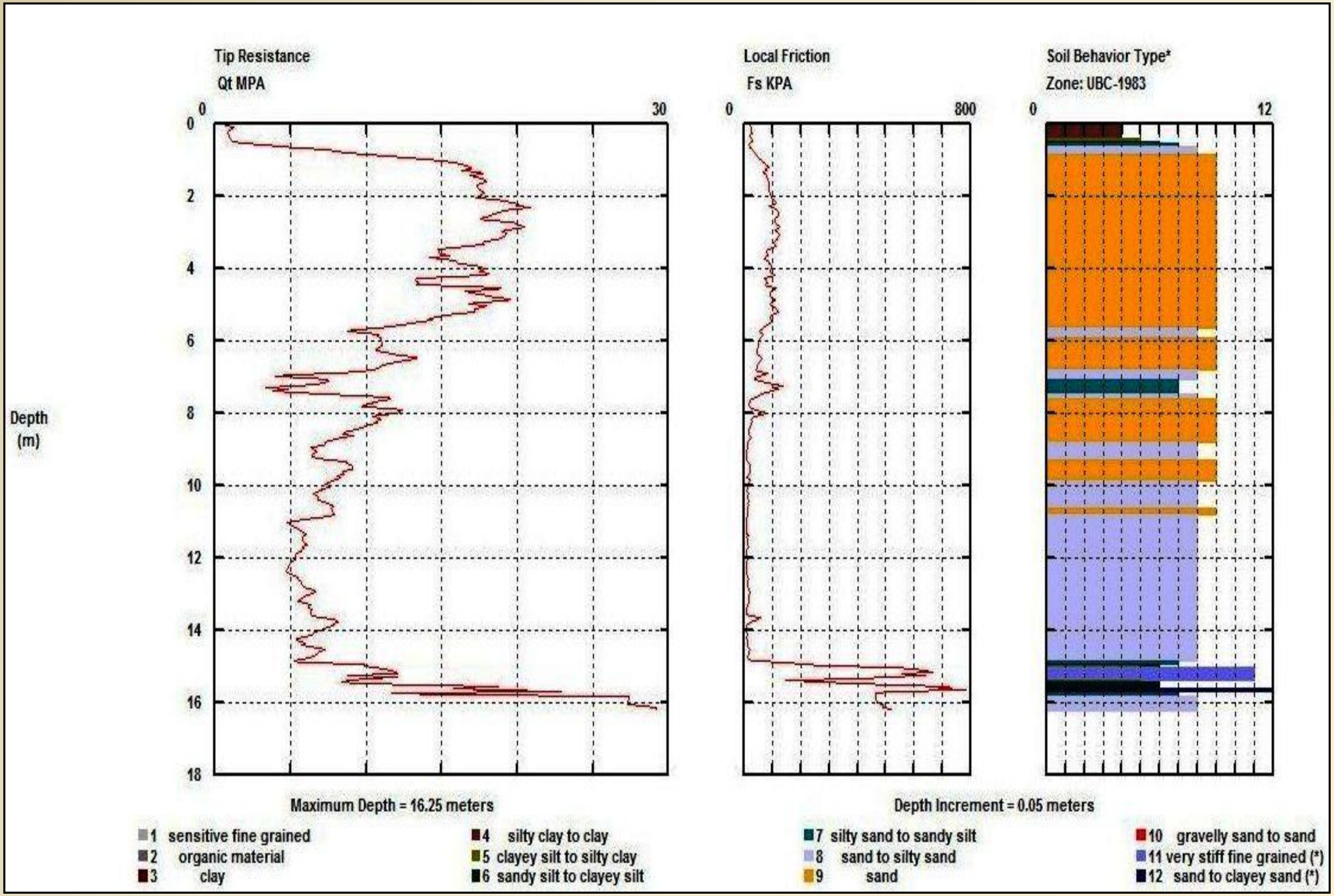


Show Marina CPT example with NovoCPT

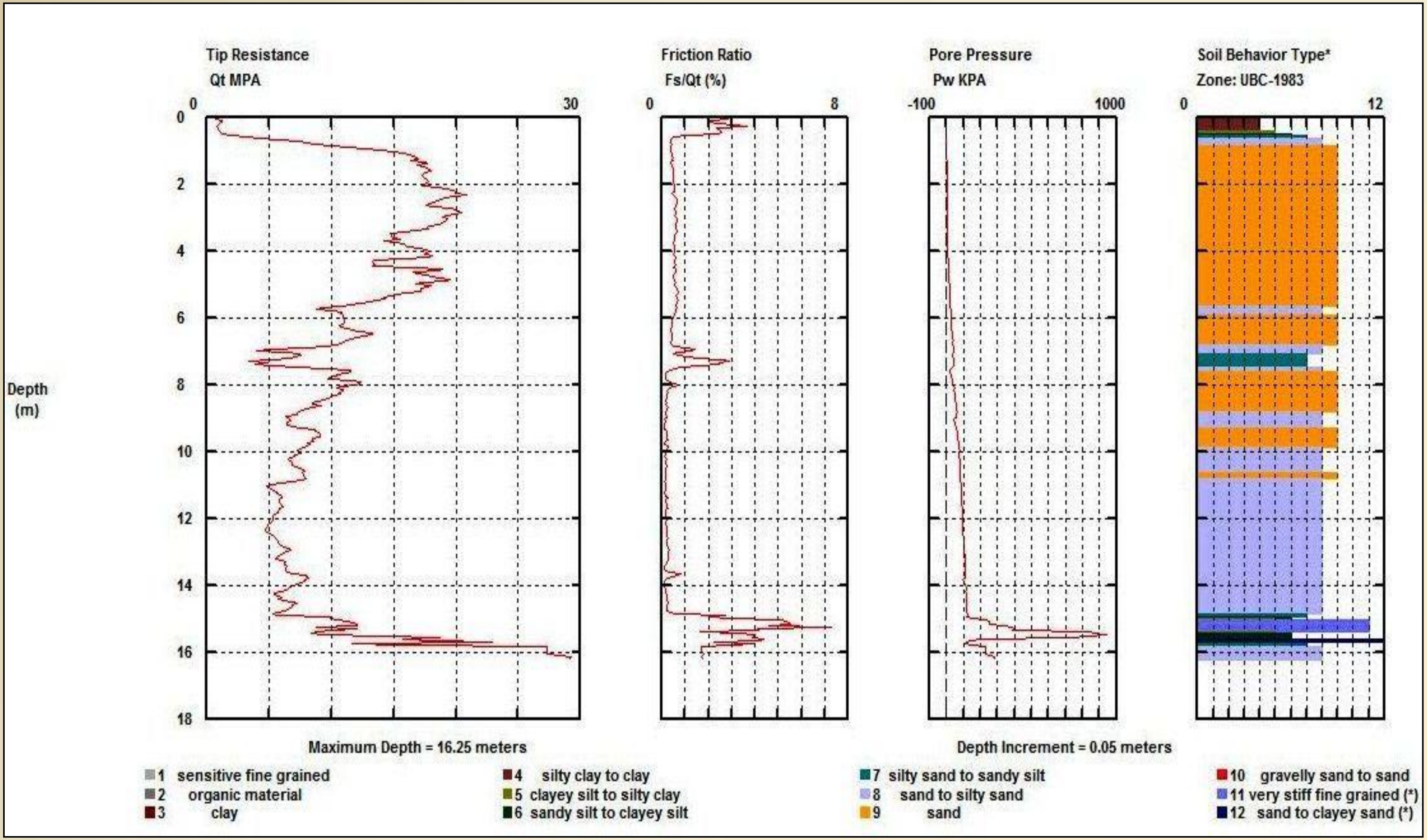
Measured & derived geotechnical parameters



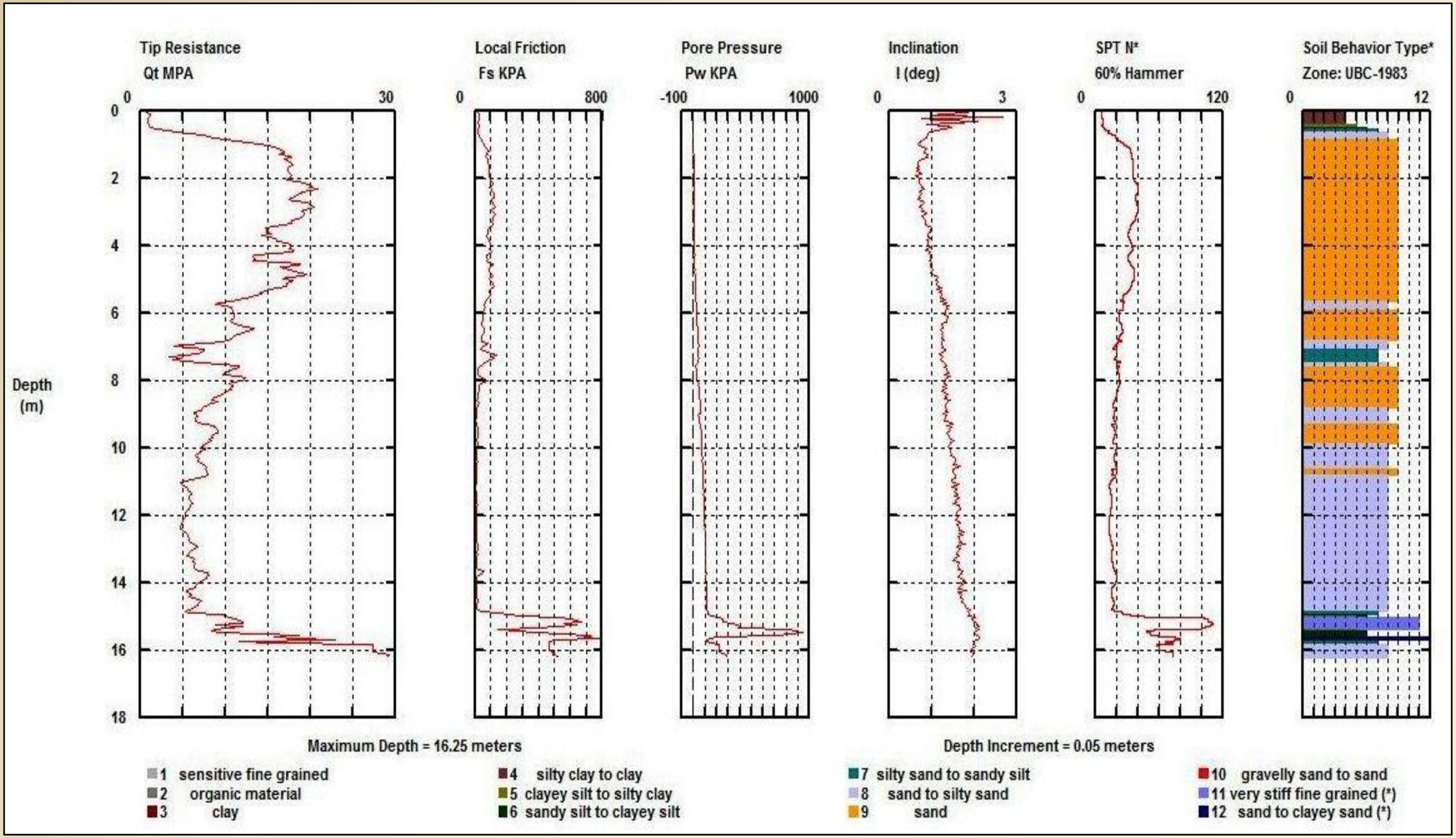
Measured parameters with soil interpretation



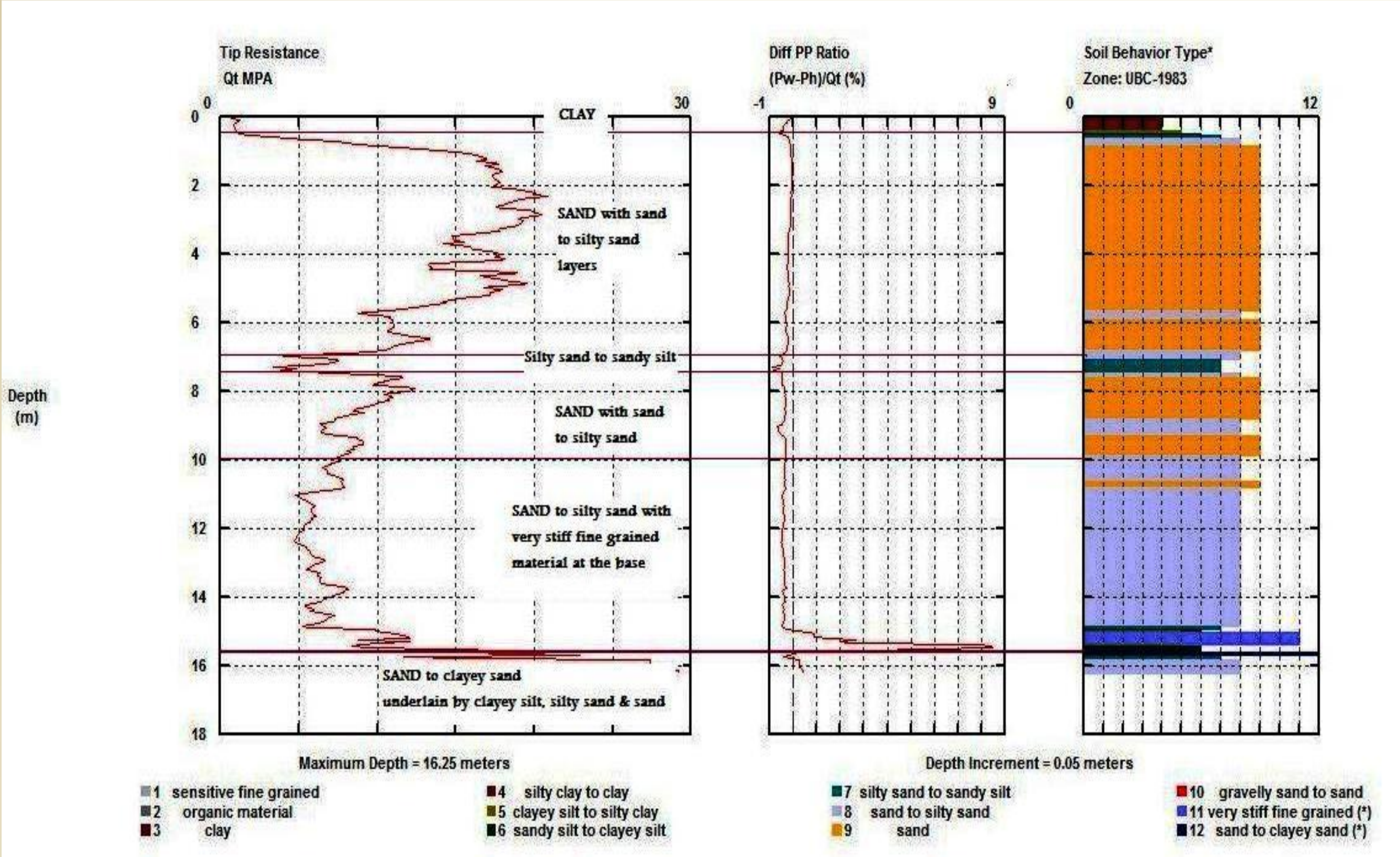
CPT Profiles



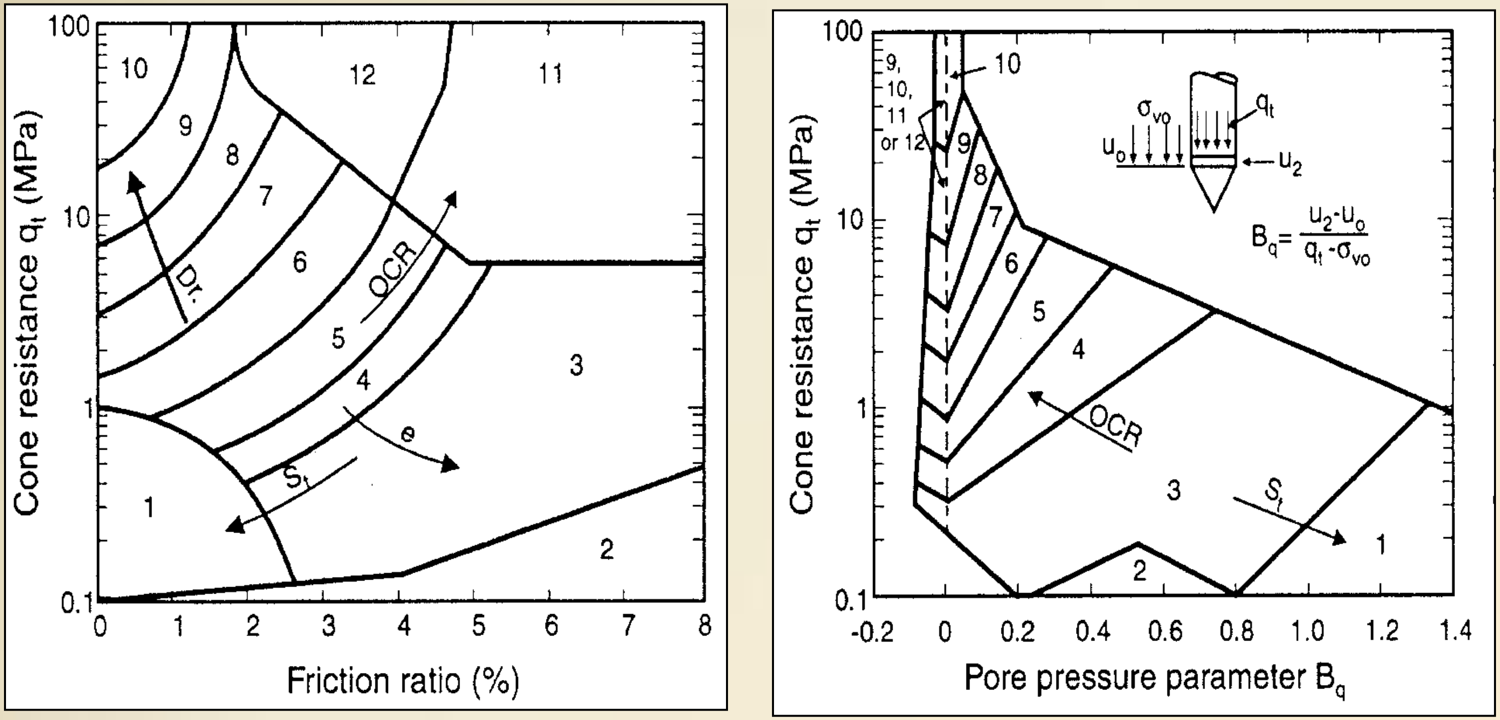
CPT Profiles – basic parameters



Detail interpretation



CPT Soil Behavioral Classification

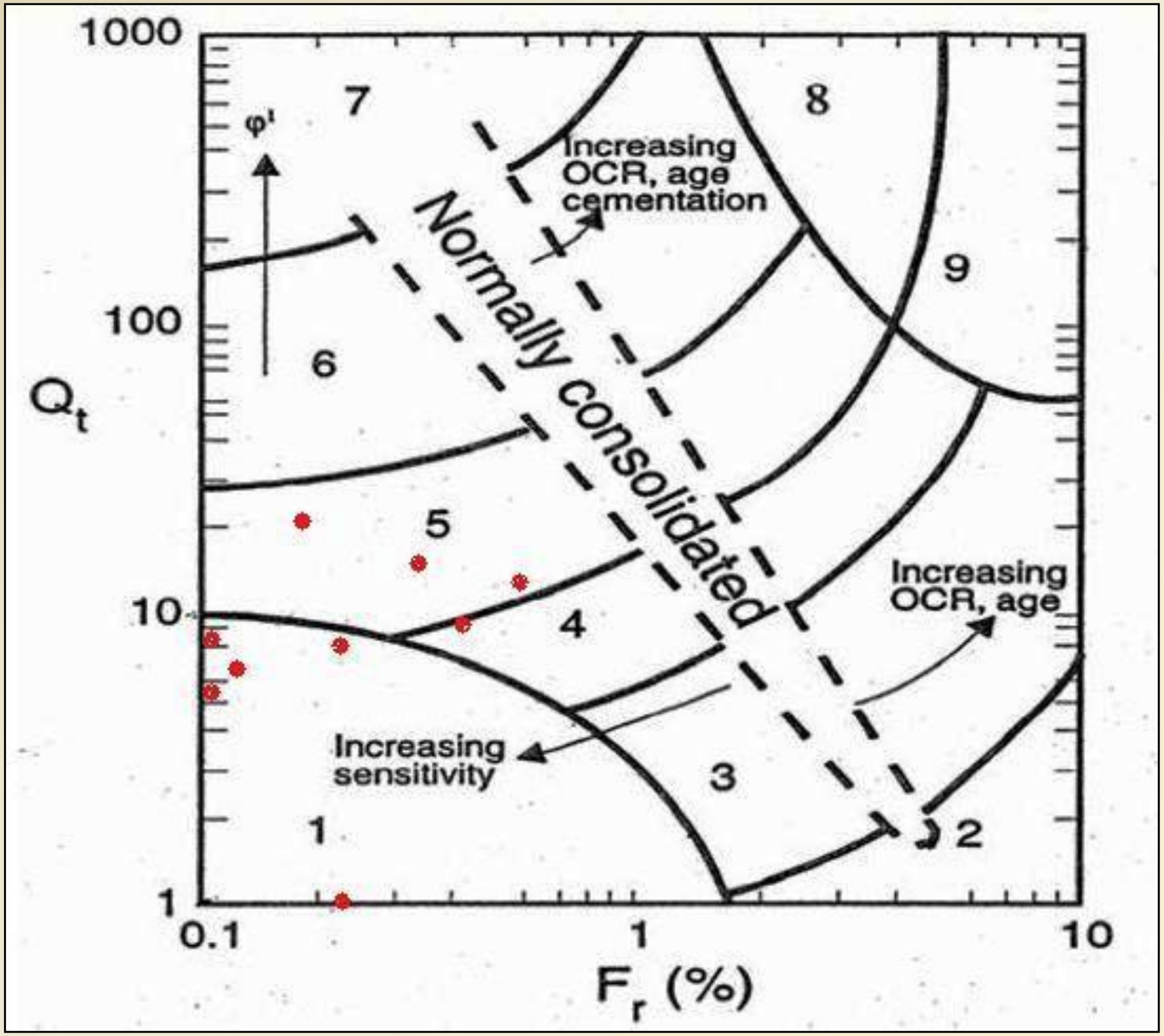


Soil Behavior Type (Robertson et al., 1986; Robertson & Campanella, 1988)

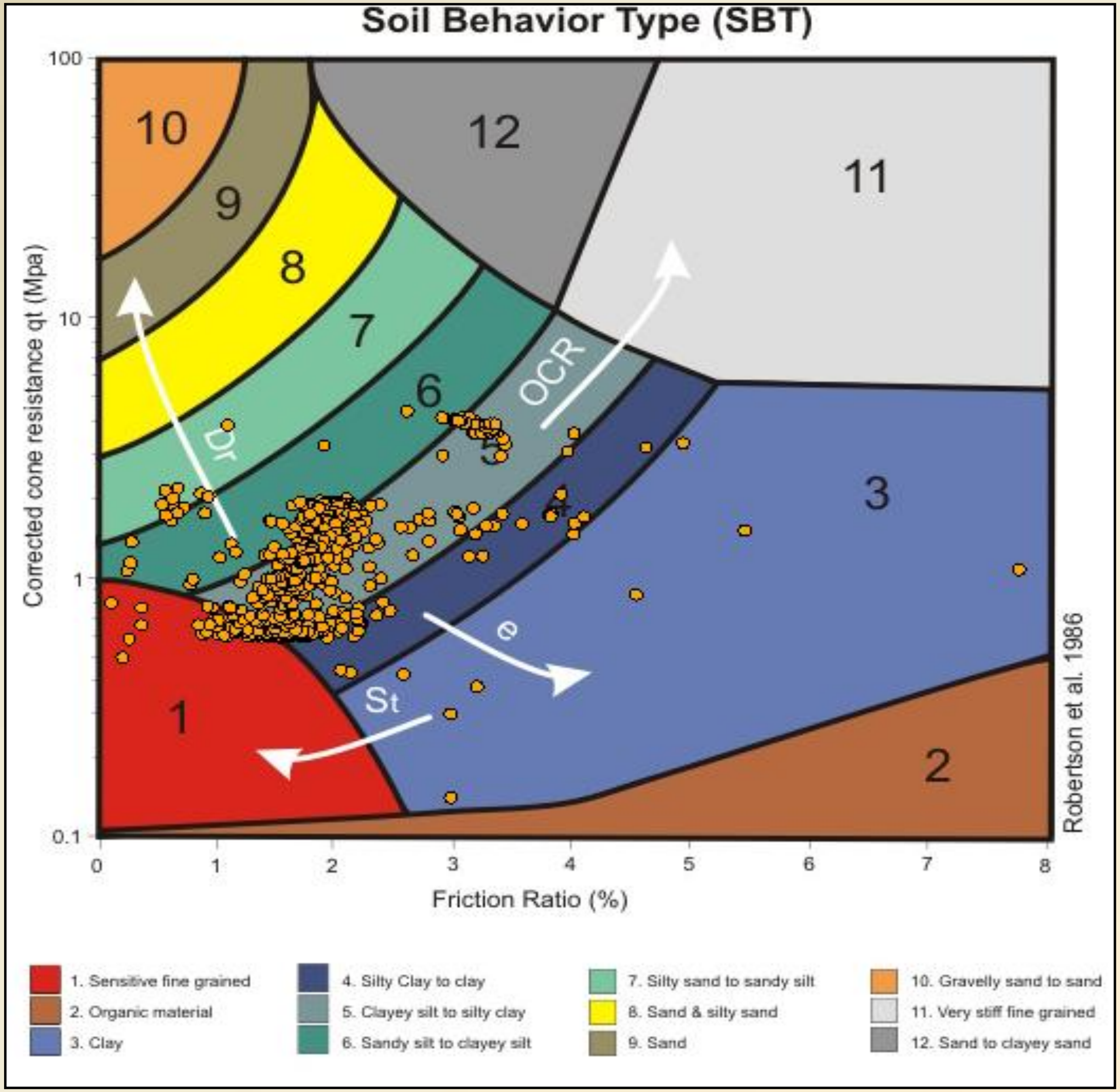
1 – Sensitive fine grained	**5 – Clayey silt to silty clay**	**9 – sand**
2 – Organic material	**6 – Sandy silt to silty sand**	**10 – Gravelly sand to sand**
3 – Clay	**7 – Silty sand to sandy silt**	**11 – Very stiff fine grained***
4 – Silty clay to clay	**8 – Sand to silty sand**	**12 – Sand to clayey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

****Note: Overconsolidated or cemented***

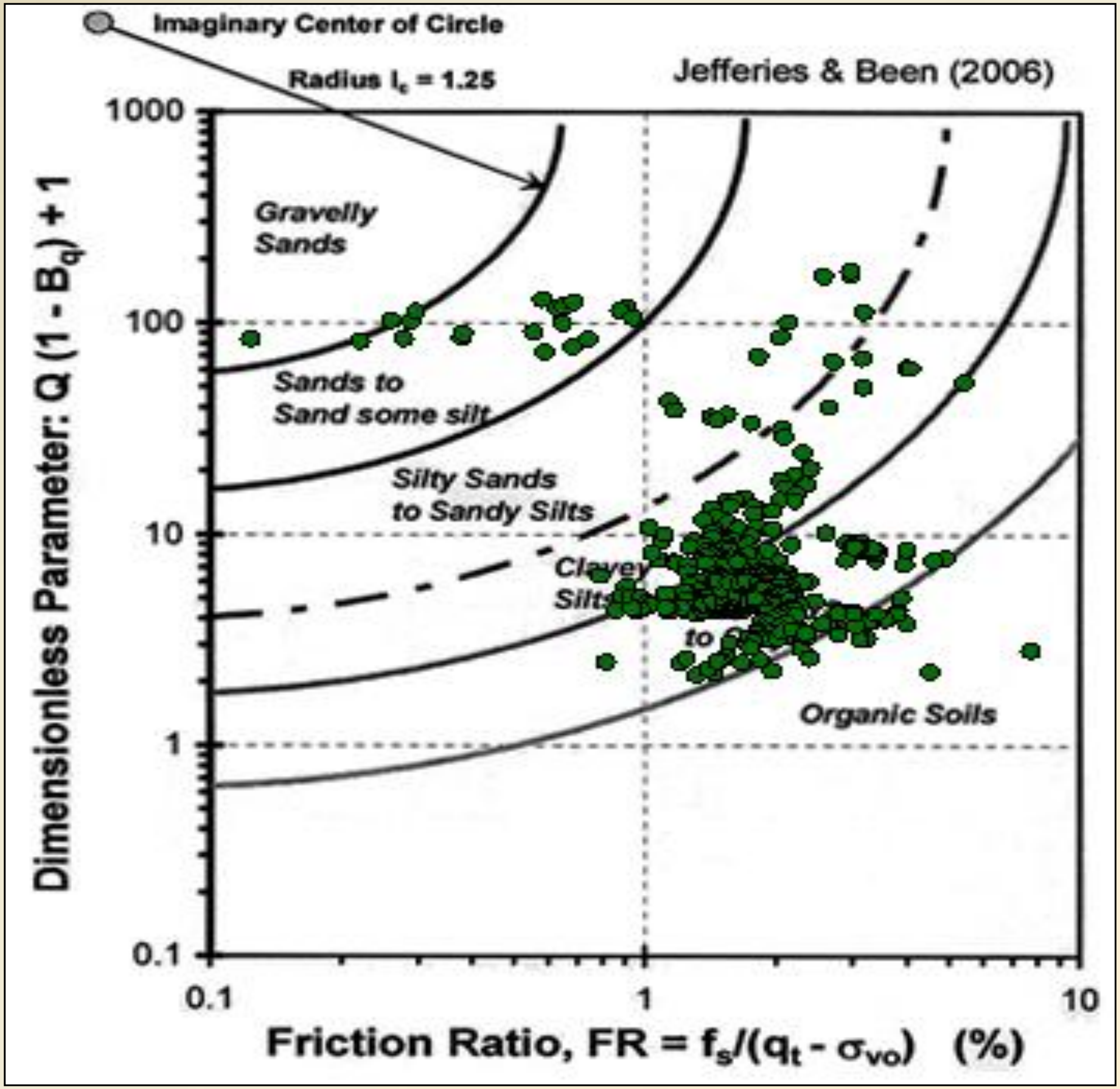
Soil interpretation based on Q_t and F_r



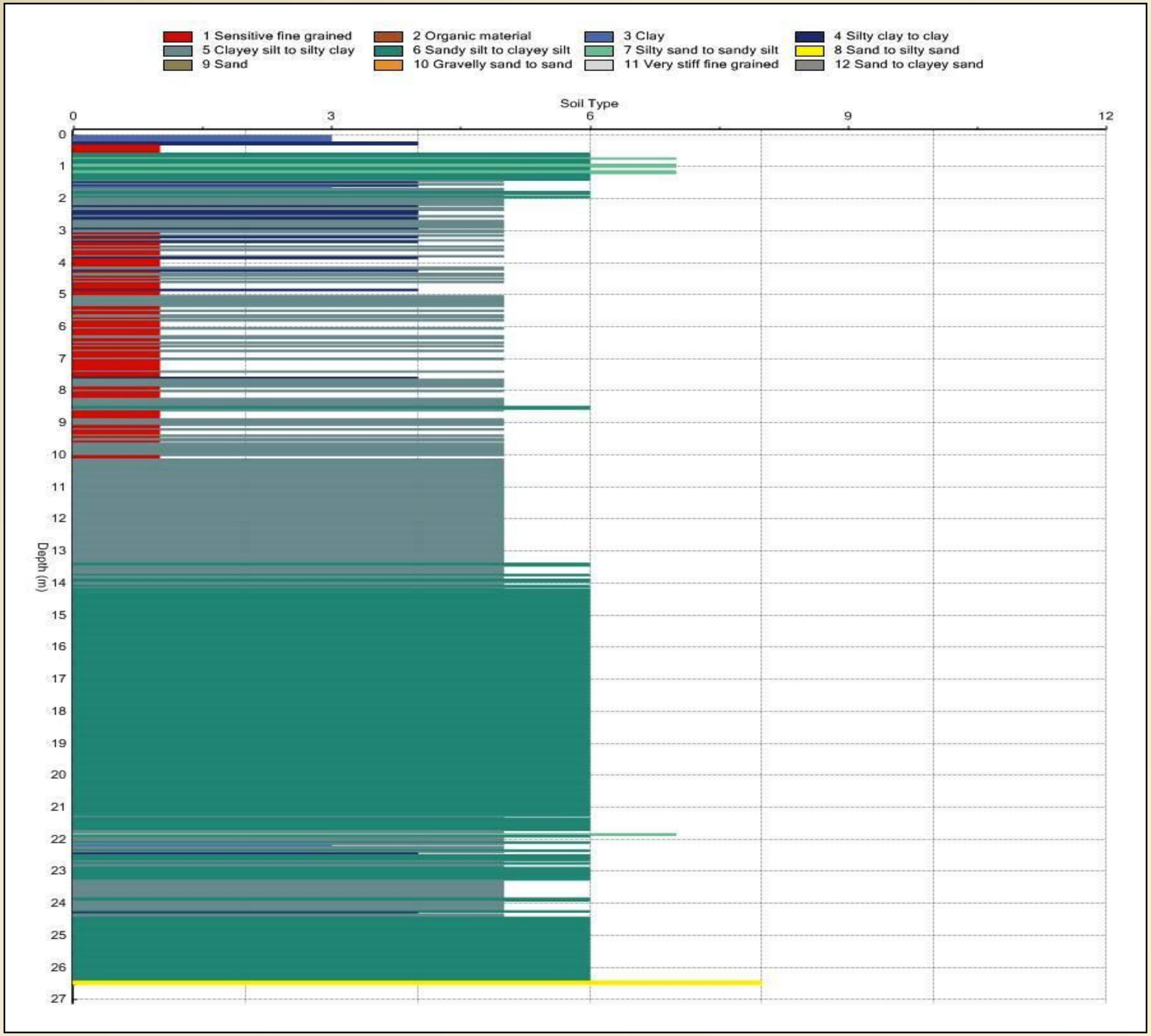
SBT
at Marina
South



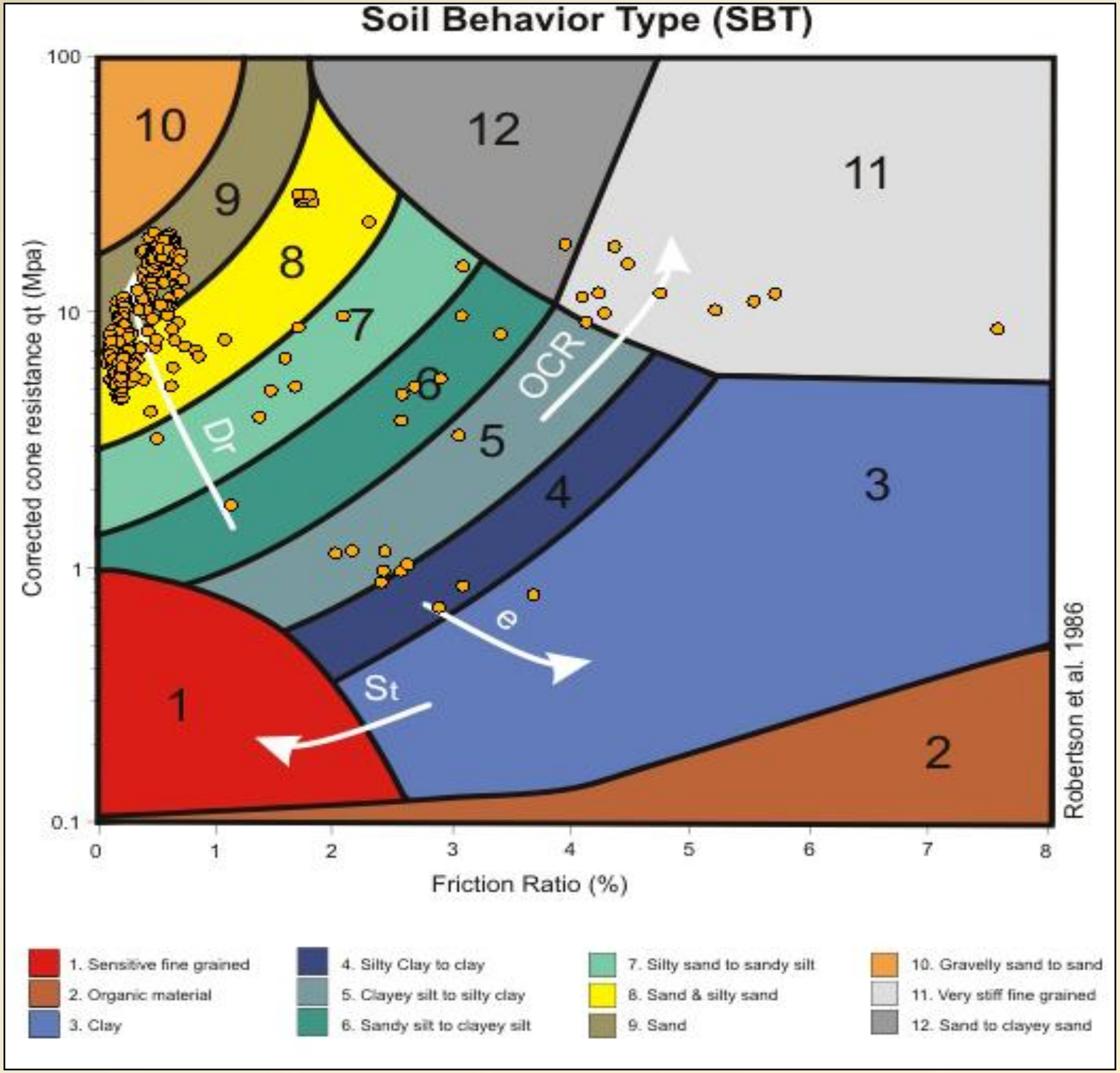
SBT Marina south



Marina south soil profile

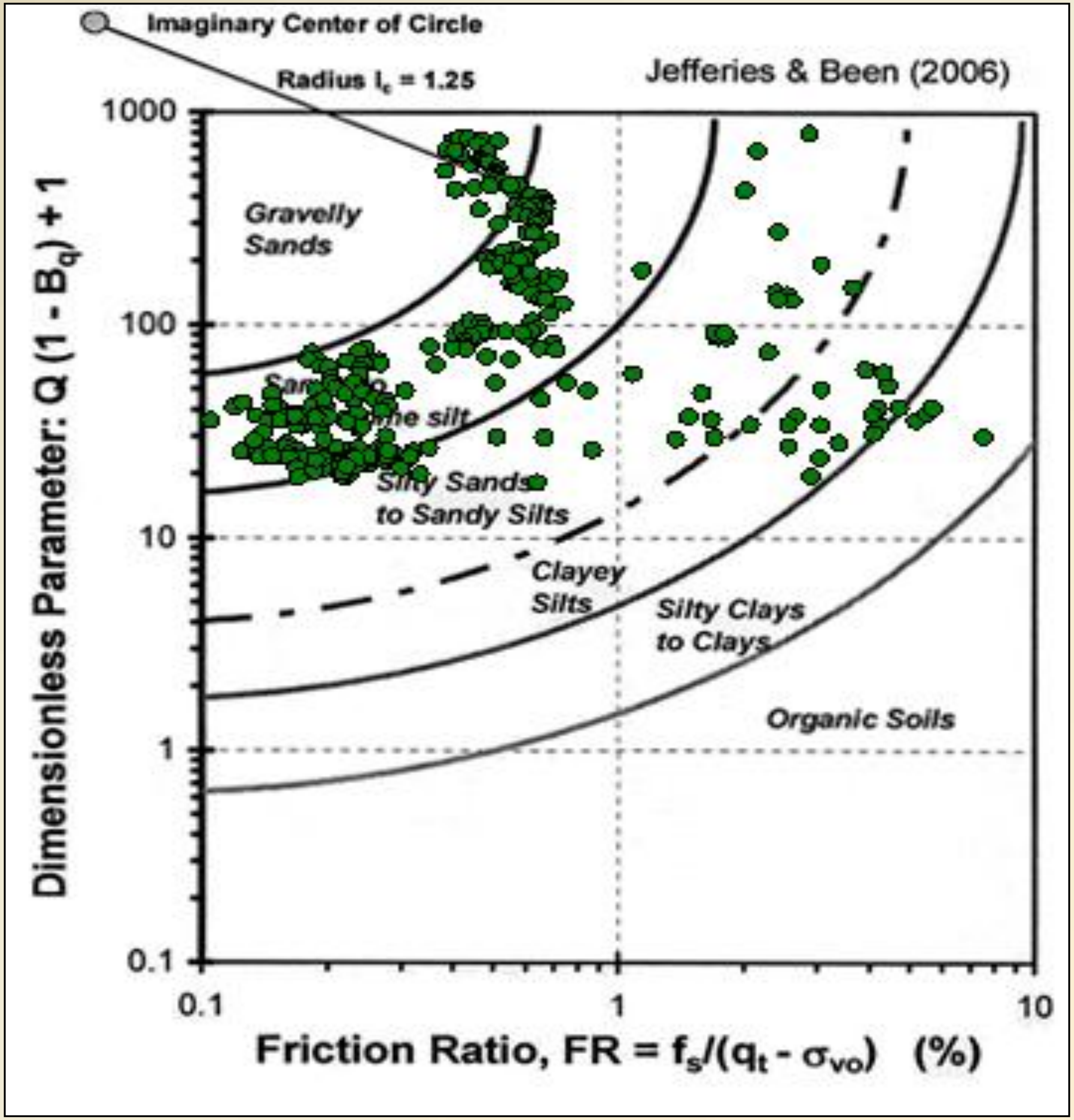


Changi East



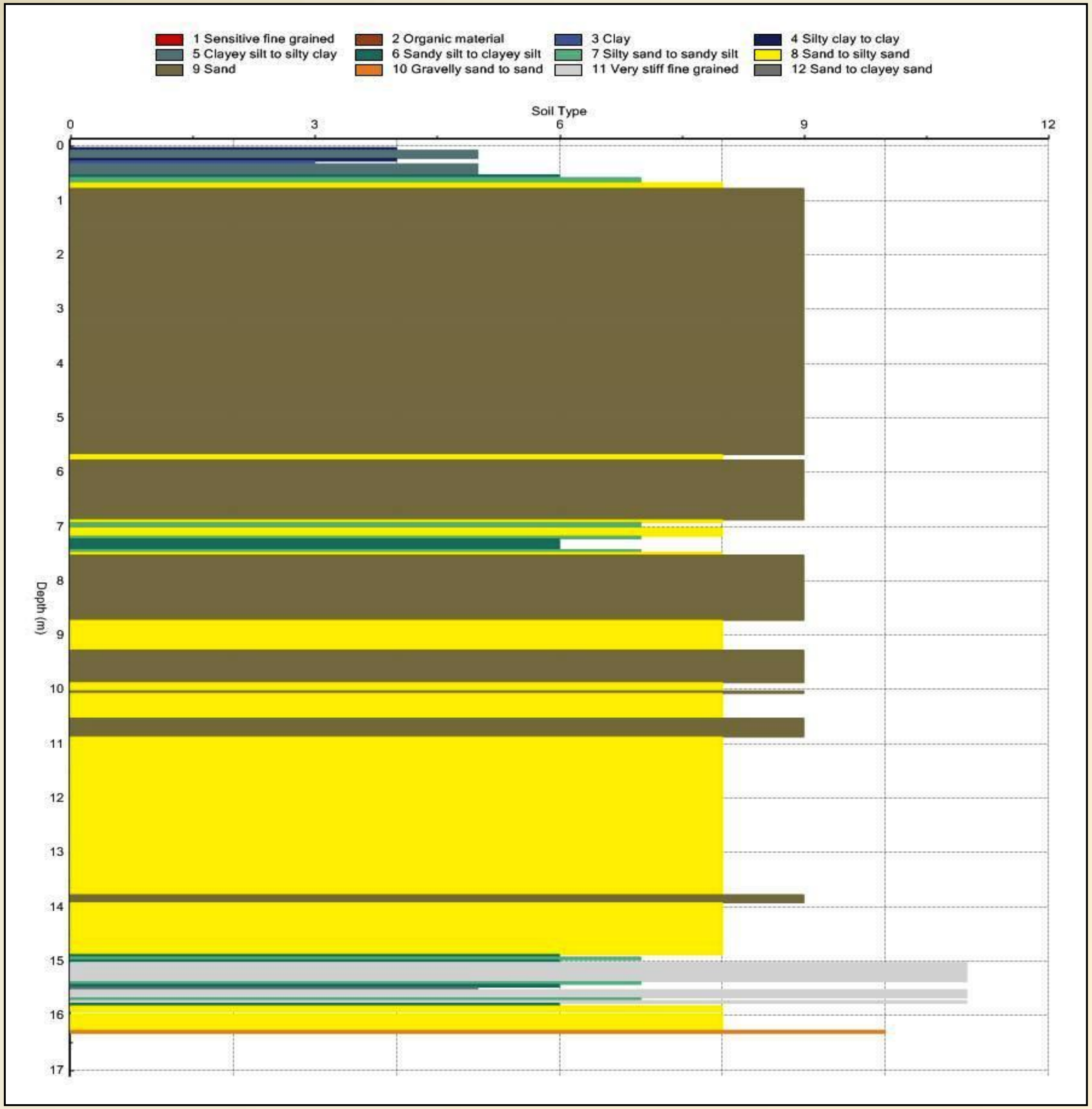
Soil Behavior Type (SBT)
Corrected cone resistance qt (Mpa)
Friction Ratio (%)
Robertson et al. 1986
10
9
8
7
6
5
4
3
2
1
11
12
Dr
OCR
e
St
1. Sensitive fine grained
2. Organic material
3. Clay
4. Silty Clay to clay
5. Clayey silt to silty clay
6. Sandy silt to clayey silt
7. Silty sand to sandy silt
8. Sand & silty sand
9. Sand
10. Gravelly sand to sand
11. Very stiff fine grained
12. Sand to clayey sand

Changi east



Imaginary Center of Circle
Radius I_c = 1.25
Jefferies & Been (2006)
Gravelly Sands
Silty Sands to Sandy Silts
Clayey Silts
Silty Clays to Clays
Organic Soils
Dimensionless Parameter: $Q(1 - B_q) + 1$
Friction Ratio, $FR = f_s/(q_t - \sigma_{vo})$ (%)

Changi East soil profile



CPT measured parameters

qc	MPa	Cone resistance	Measured parameter
fs	MPa	Local friction	Measured parameter
Rf	%	Friction ratio	fs / qc * 100%
I	°	Inclination	Measured parameter
$u_{(1,2,3)}$	MPa	Dynamic pore pressure	Measured parameter
u_0	MPa	Equilibrium pore pressure	ρwater * (depth-water level)
u / qc	-	Dynamic pore pressure ratio	u2 / qc
qt	MPa	Corrected cone resistance	$qc + (1-\alpha s) * u_2$ $\alpha s \approx 0.81$
Δu	MPa	Excess pore pressure	$u_2 - u_0$
qe	MPa	Effective cone resistance	$qc - u_2$
σv;z	kPa	Total vertical stress	Σ γdry + Σ γwet
σv;z'	kPa	Effective vertical stress	$\sigma v;z - u_0$
qn	kPa	Net cone resistance	qt - σv;z
Bq	-	Pore pressure ratio	Δu / qn
qnorm	-	Normalised cone resistance	qn / σv;z'
fnorm	%	Normalised local friction	fs / qn * 100%

CPT derived parameters

Dr	%	Relative density	1/C2 * LN(qc/(C0*σv;z')^C1) Consolidated: C0≈157, C1≈0.55, C2≈2.41 Over-consolidated: C0≈181, C1≈0.55, C2≈2.61
φ	°	Internal friction angle	ARCTAN(a + b * LN(qc/σv;z')) a≈0.105, b≈0.16
Su	kPa	Undrained shear strength	(qc - σv;z) / Nk Nk(min)≈12, Nk(max)≈20
Ic		Soil behaviour type index	sqrt((a-log qnorm) + (log fnorm+b)) a≈3.47, b≈1.22
N60		Equivalent SPT N60 value	(qc/pa) / (8.5*(1-Ic/4.6)) pa≈100
Qst	kN	Total friction	Measured parameter
Qt	kN	Total force	Measured parameter

Correlated Soil Properties (derived parameters) from CPT data

Equivalent SPT, N60:
Jefferies and Davis 1993
Permeability coefficient , K:
Robertson et al. 1986

Shear strength , Su / Cu
Overconsolidation ratio OCR:
Powel et al. 1998
Lunne et al. 1989
Mayne 2005

Clay undrained Young's modulus Es:
Duncan and Buchihmami 1976
Clay at-rest earth pressure Ko:
Kulhawy and Mayne 1990

Sand relative density Dr:
Jamiolkowski et al. 1985
Baldi et al. 1986
Tatsuoka 1990

Sand, Young's modulus , Es:
Bellotti et al. 1989

Sand at-rest earth pressure, Ko:
Kulhawy and Mayne 1990

Sand internal friction angle , ϕ:
Kulhawy and Mayne 1990
Hatanaka and Uchida 1996
Robertson and Campanella 1983
Sunneset et al. 1989
Mayne 2005

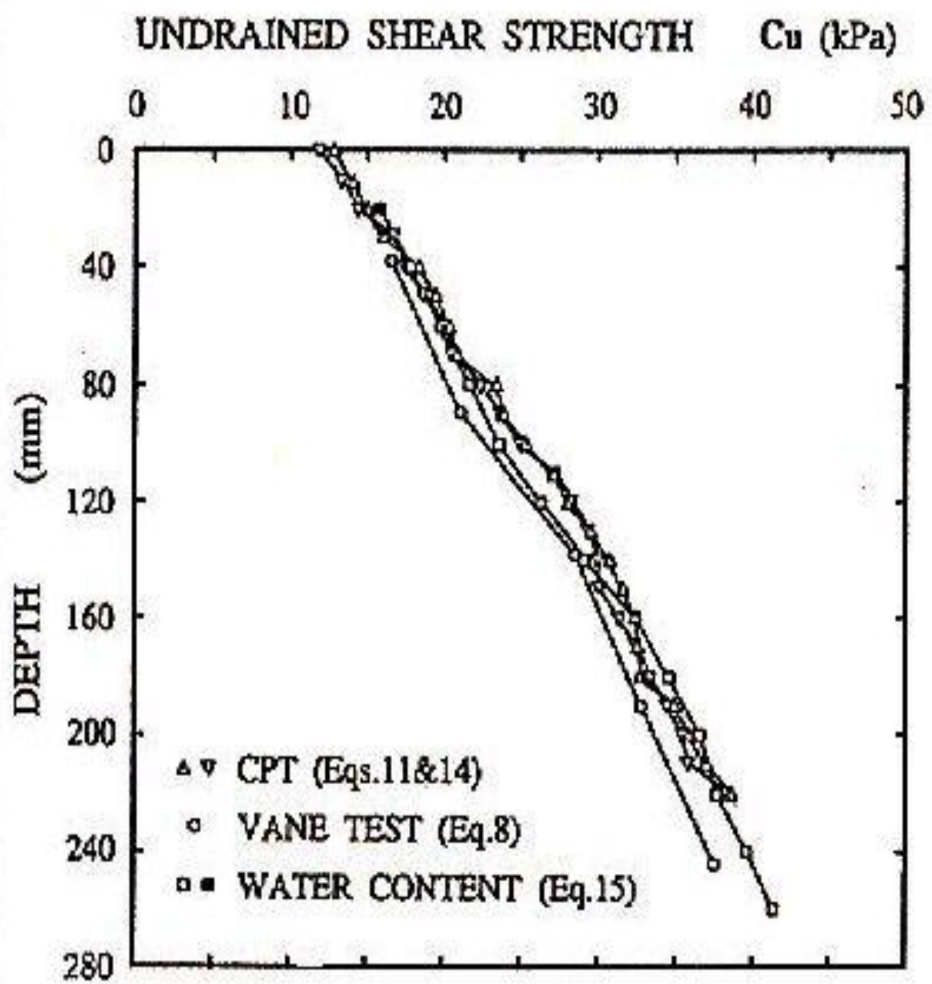
Unit weight:
Robertson et al. 1986
Fines content , Fc:
Robertson and Fear 1995 ($FC=1.75*I_C^3-3.7$)
Constrained modulus , M:
Robertson 2009
Soil behaviour type index , Ic:
Robertson 1990

Correlation of N_{60} and q_t

$$N_{60} = (q_t/p_a)/[8.5(1-I_c/4.6)]$$

Jefferies, M. G. and Davies, M. P., (1993), “Use of CPTu to estimate equivalent SPT N60”, *ASTM Geotechnical Testing Journal*, Vol. 16, No. 4

Comparison of derived parameters based on CPT, FVT and Water Content



CPT

$$c_u = \frac{q_c^* - 0.737\,\sigma_{vo}}{12.5}$$

$$q_c = q_c^*\{1.0 + 0.10 \log(\frac{v}{20})\}$$

q_c^* : cone resistance at a relative penetration rate v^*

VANE

$$c_u = \frac{s_u}{1.07}$$

s_u : measured undrained shear strength

WATER CONTENT (empirical relation)

$$\log(c_u) = 3.804 - .101\,w$$

(after Tani and Craig, 1995)

Shallow Foundation, Settlement & Pile Capacity examples using NovoCPT

SPT-CPT Correlations

Soil type	**Mean grain size (*D 50*), mm**	***Qc /N***
Clay	0.001	1.0
Silty Clay	0.005	1.7
Clayey Silt	0.01	2.1
Sandy Clay	0.05	3.0
Silty Sand	0.01	4.0
Sand	0.5 1.0	5.7 7.0

Q_c = CPT value in bars (1 bar = 100 kPa)

Robertson et al. (1983)

Important references

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Robertson, P. K., Campanella, R. G., Gillespie, D. and Grieg, J. (1986), “Use of piezometers cone data”. *Proceedings of the ASCE Specialty Conference In Situ ’86: Use of In Situ Tests in Geotechnical Engineering*, Blacksburg, VA

Kulhawy, F. H., and Mayne, P. W., (1990), “Manual for estimating soil properties for foundation design.”, *Report EL -6800,* EPRI, Palo Alto, CA.

Lunne, T., Robertson, P. K. and Powell, J. J. M. 1996. Cone Penetration Testing In Geotechnical Practice

Mayne, Paul W. 2005. Engineering Design Using the Cone Penetration Test

Thank you all for your patience
&
deeply appreciate EC of MGSS for their devotion and kind effort in propagation of knowledge in engineering geology & geotechnical engineering

WISHING YOU ALL THE BEST IN WHATEVER YOU DO!
19 SEPTEMBER 2010
SINGAPORE