Cone Penetration Test
The Use of Soil Behavior Types and Correlation of Geotechnical Parameters

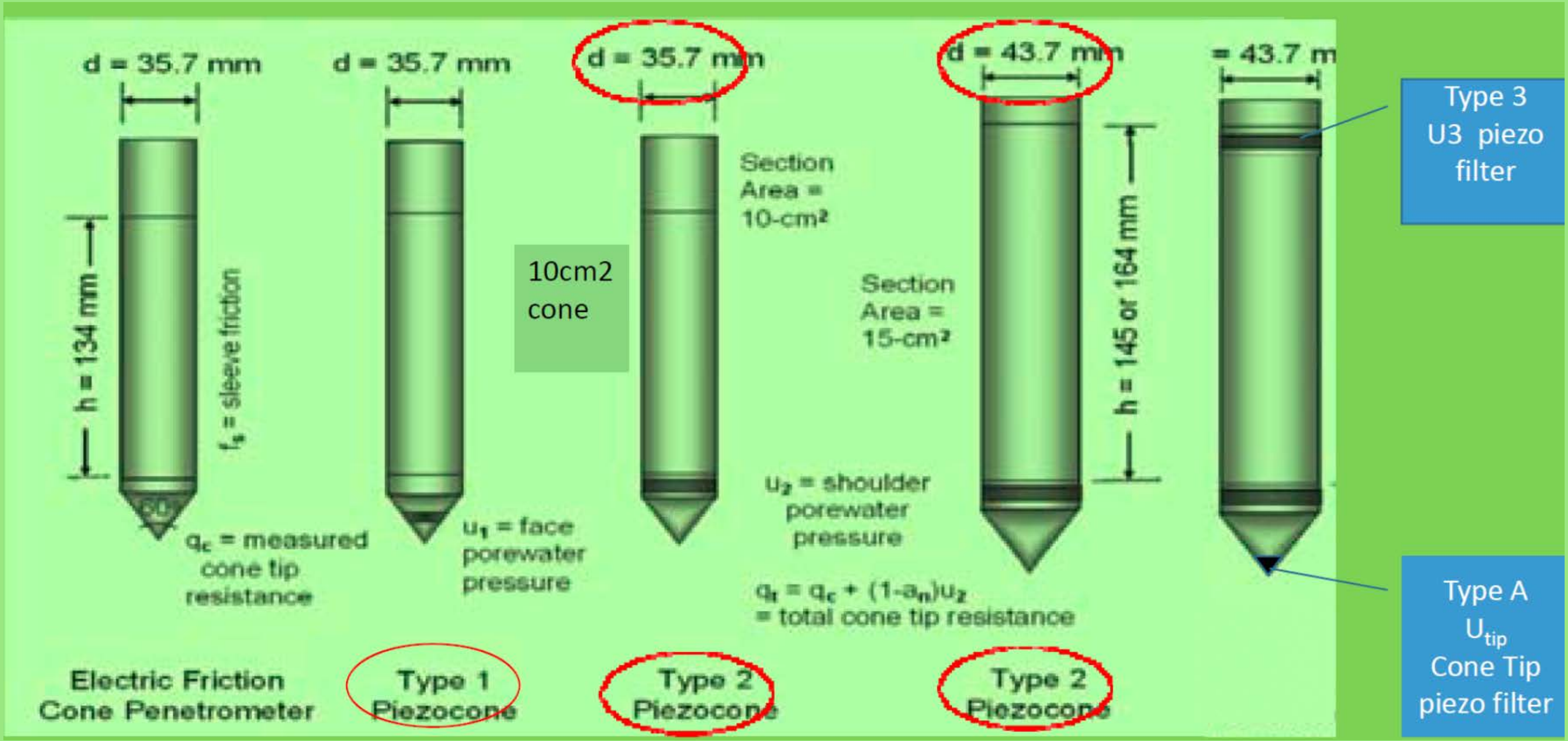
Ir. Raymond S. M. Chan of (Bachy Soletanche Group Ltd)

MSc (Const. Mgt), MSc (App. Geo), MHKIE, RPE (Civil), AS (GIFW), MASCE, MCIOB, FGS

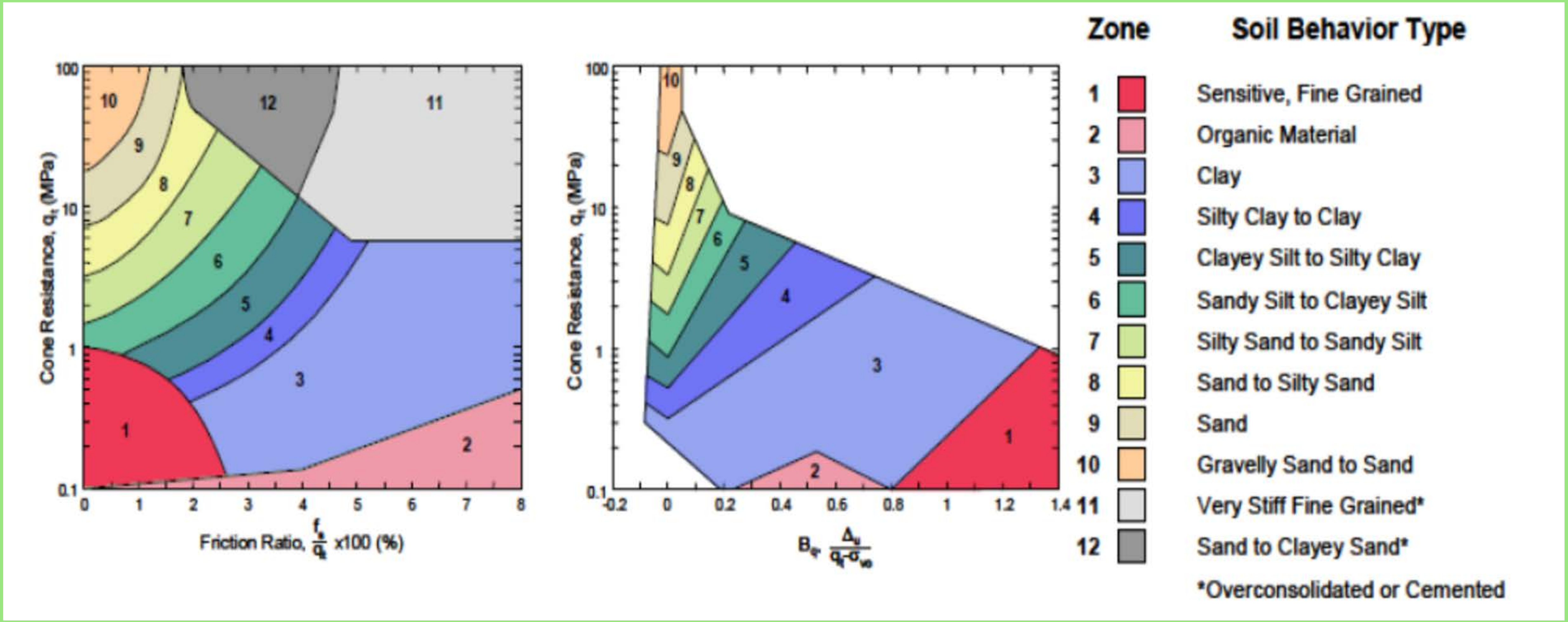
Content

Soil Classification with SBT and SBTn
Correlation of Soil Parameters and CPT
Undrained Shear Strength Test
Standard Penetration Test
Dissipation Tests for Consolidation Characteristics
Application of CPT
Deep Compaction
Deep Cement Mixing
Challenges for Adopting CPT in Hong Kong

Dimensions and Measurements Taken by 10 cm2 and 15 cm2 Piezo Cone Penetrometers



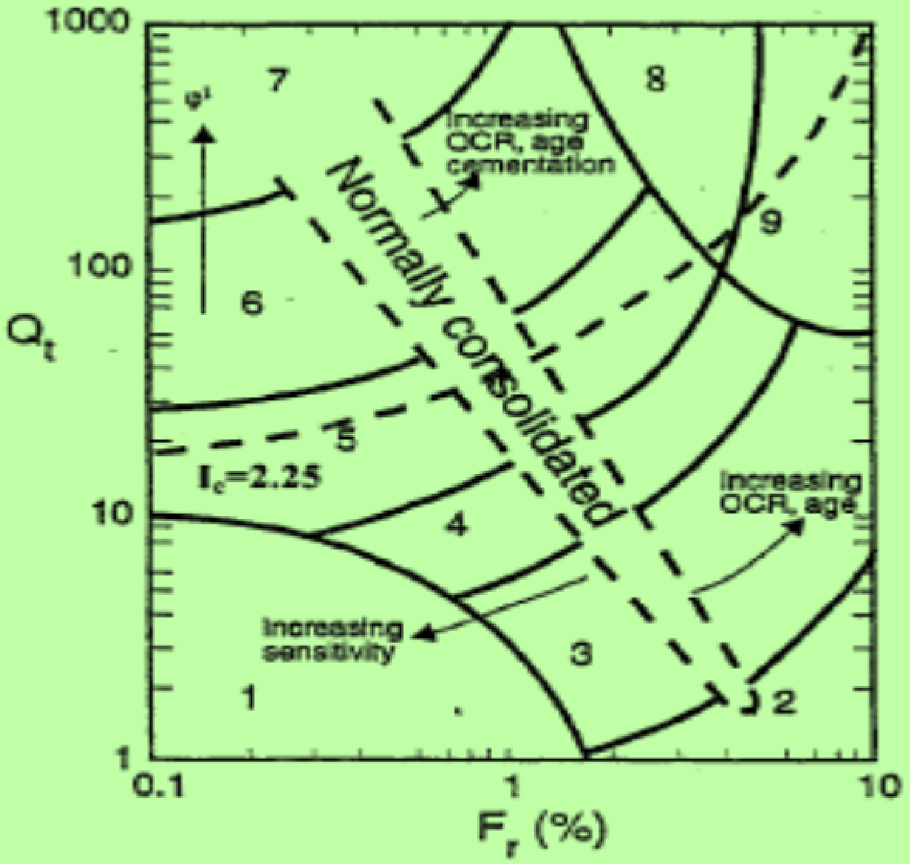
CPTu Soil Behaviour Type (SBT) Chart by Robertson et al 1986)

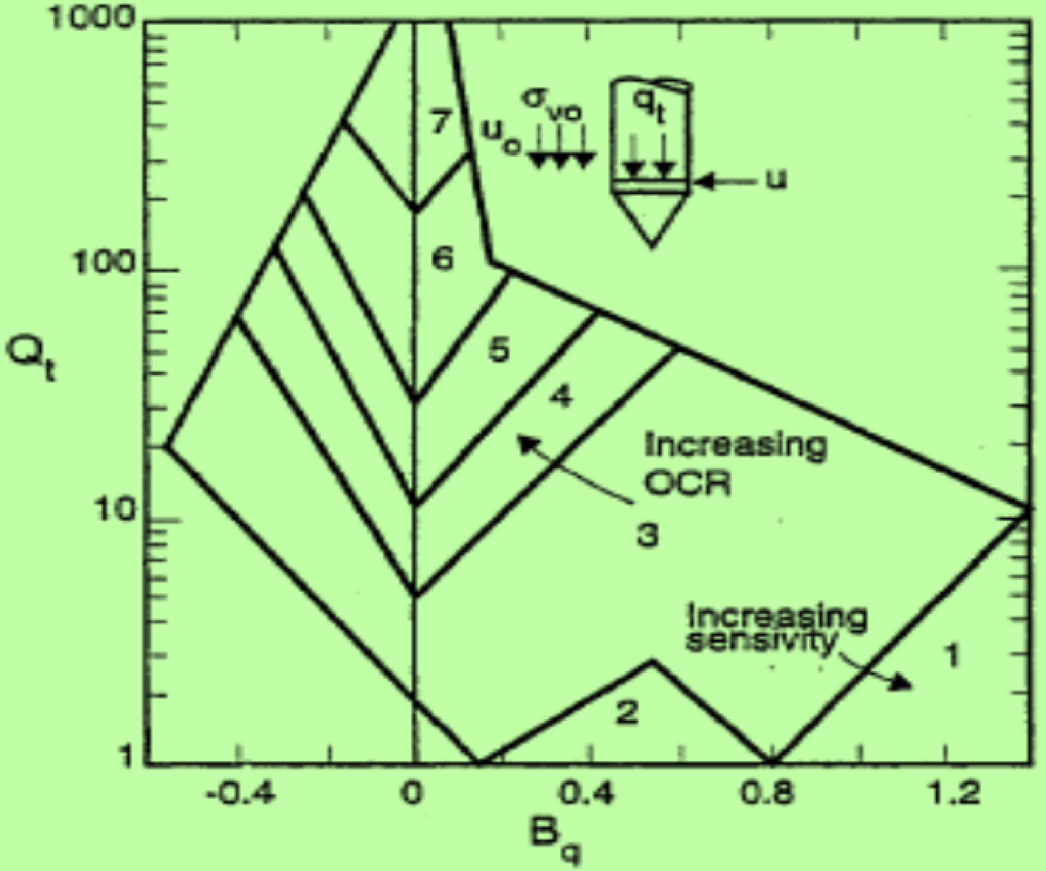


Soil Classification with Soil Bahavour Type (SBT)

- Soil Behavior Type (STB) Robertson 1986
- The Soil Behavior Type (SBT) provides a guide to mechanical characteristics like strength, stiffness and compressibility of soil.
- It is different from the physical characteristics like grain size distribution and Atterberg limits that classified by Soil Classification Unit System (SCUC) and the traditional particle size distribution classification soil from Geoguide 3.
- In general, this chart is appropriate for CPT at depth of not greater than 20m.

Normalized Soil Behavior Type (Robertson 1990)





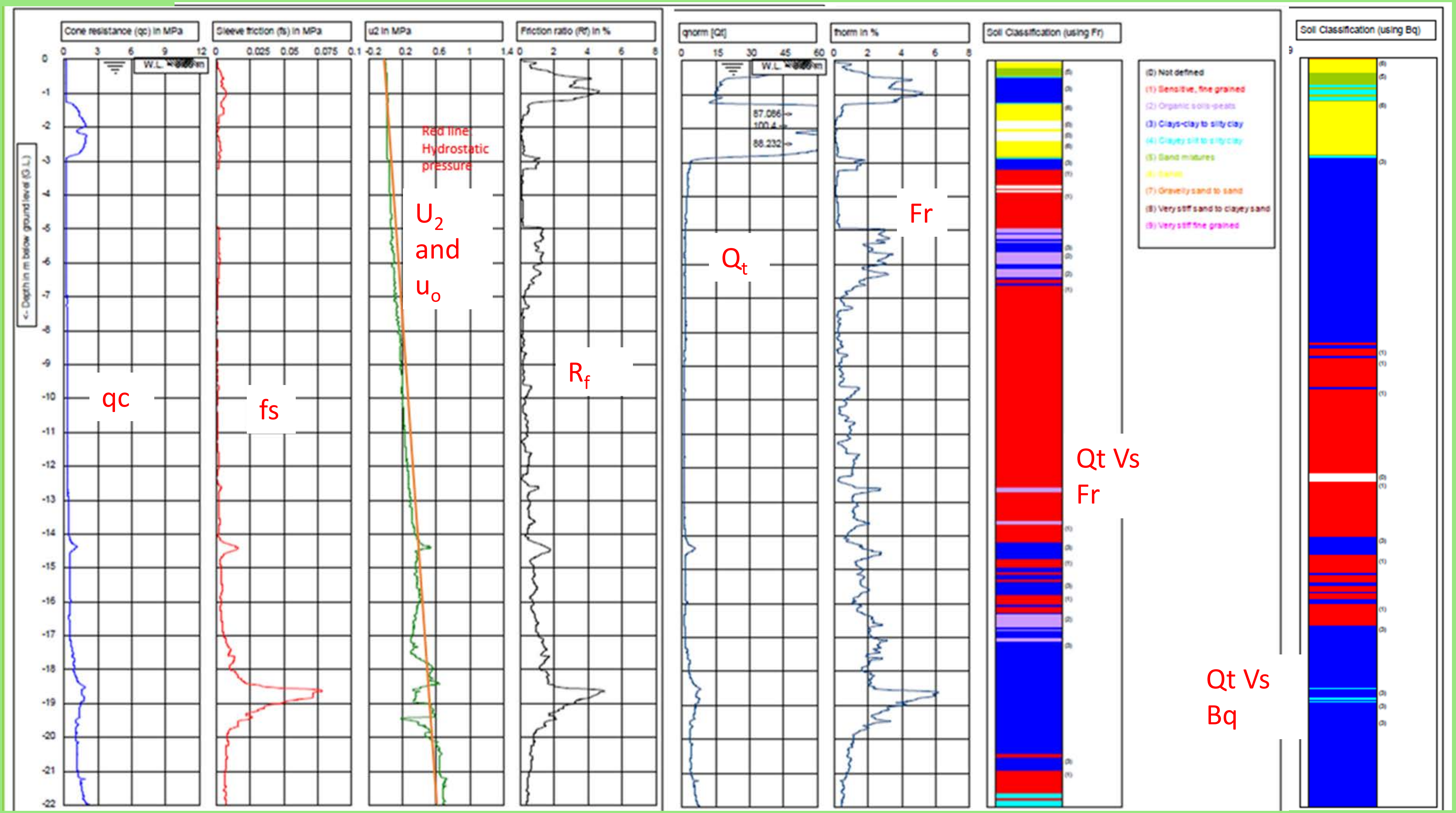
$$Q_t = \frac{q_t - \sigma_{vo}}{\sigma'_{vo}} \qquad B_q = \frac{u_2 - u_o}{q_t - \sigma_{vo}} \qquad F_r = \frac{f_s}{q_t - \sigma_{vo}} \times 100\%$$

Zone	Soil behaviour type
1.	Sensitive, fine grained;
2.	Organic soils-peats;
3.	Clays-clay to silty clay;

Zone	Soil behaviour type
4.	Silt mixtures clayey silt to silty clay
5.	Sand mixtures; silty sand to sand silty
6.	Sands; clean sands to silty sands

Zone	Soil behaviour type
7.	Gravelly sand to sand;
8.	Very stiff sand to clayey sand
9.	Very stiff fine grained

Comparison of the STB Classification by Q_t-F_r plot and Q_t-B_q Plots



Contours of SBT Index, Ic on CPT Normalized SBT Qt –Fr Chart

$$I_c = ((3.47 - \log Q_t)^2 + (\log F_r + 1.22)^2)^{0.5}$$

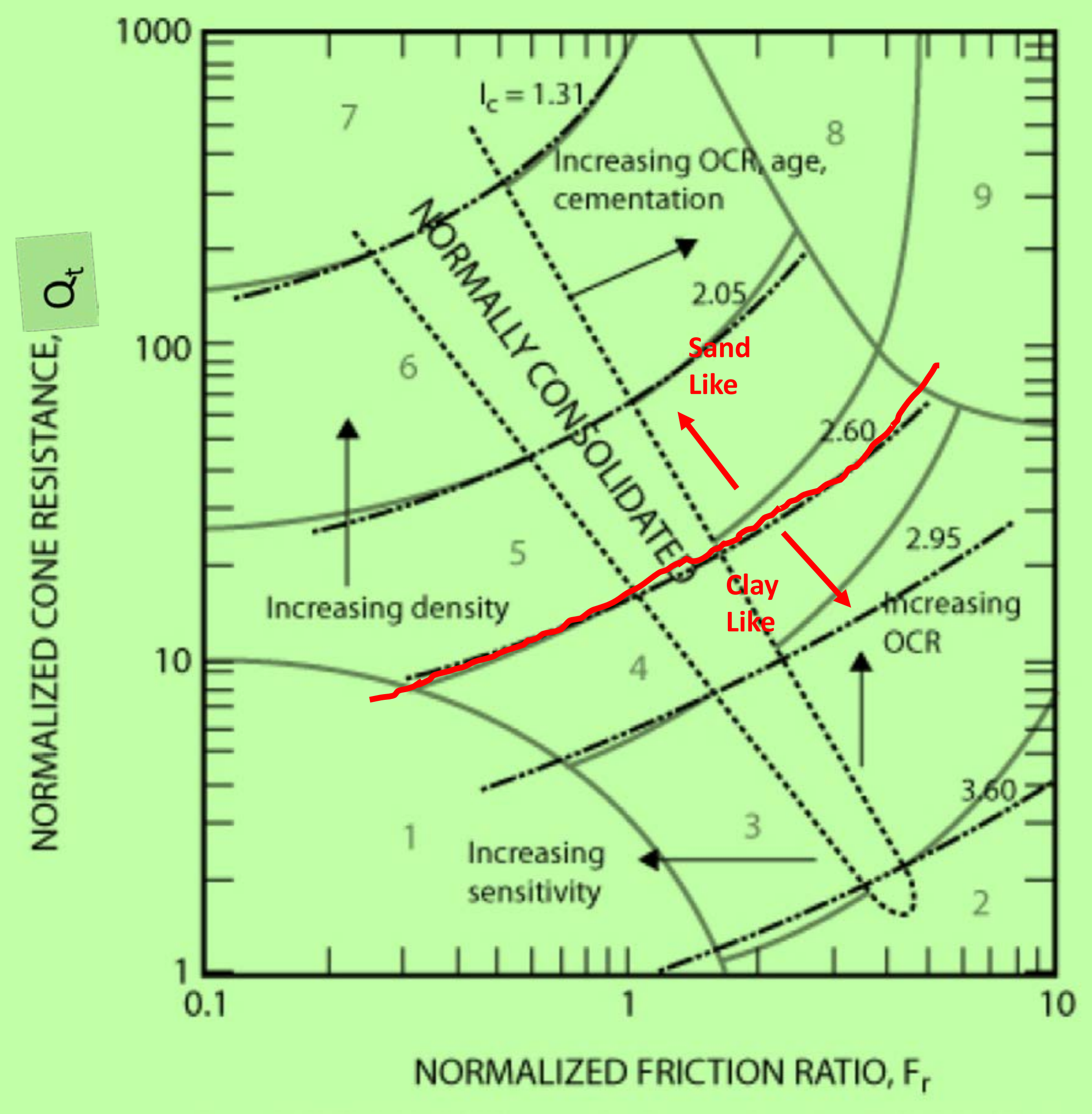
where:

Q_t = normalized cone penetration resistance (dimensionless)
= $(q_t - \sigma_{vo})/\sigma'_{vo}$

F_r = normalized friction ratio, in %
= $(f_s/(q_t - \sigma_{vo}))$ x 100%

Zone	Soil Behavior Type	I_c
1	*Sensitive, fine grained*	N/A
2	*Organic soils – clay*	> 3.6
3	*Clays – silty clay to clay*	2.95 – 3.6
4	*Silt mixtures – clayey silt to silty clay*	2.60 – 2.95
5	*Sand mixtures – silty sand to sandy silt*	2.05 – 2.6
6	*Sands – clean sand to silty sand*	1.31 – 2.05
7	*Gravelly sand to dense sand*	< 1.31
8	*Very stiff sand to clayey sand**	N/A
9	*Very stiff, fine grained**	N/A

* *Heavily overconsolidated or cemented*



Updated Normalization of STBn with I_c, Q_{tn}, and n Iteration

$$I_c = \sqrt{(3.47 - \log Q_{tn})^2 + (\log F + 1.22)^2}$$

$$Q_{tn} = \left(\frac{q_t - \sigma_{v0}}{\sigma_{atm}}\right)\left(\frac{\sigma_{atm}}{\sigma'_{v0}}\right)^n$$

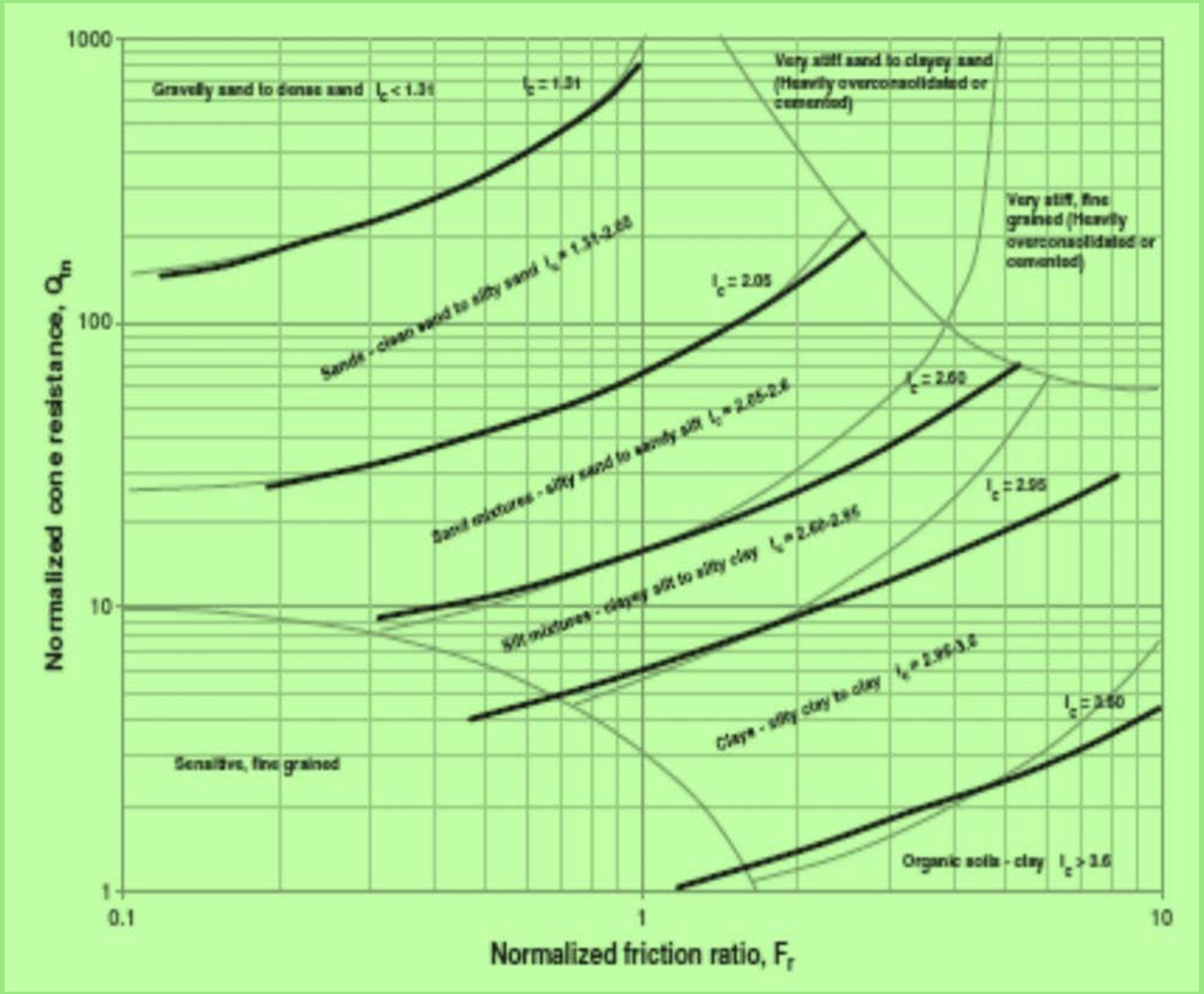
$$n = 0.381 \cdot I_c + 0.05 \cdot \left(\frac{\sigma'_{v0}}{\sigma_{atm}}\right) - 0.15$$

Where σ_{atm} (or Pa in some textbooks) is the atmospheric pressure = 100KPa = 1 bar

n is the Stress Exponent

=1 for clay, =0.5 for sand, =0.70 for silt

If n=1, $Q_{tn} = Q_{t1} = Q_t$ and it comes back to the same equation for Q_t again for clay



Iteration Procedures

$$Q_{tn} = \frac{(q_t - \sigma_{vo})/\sigma_{atm}}{(\sigma'_{vo}/\sigma_{atm})^n}$$

Step 3, put n to calculate Qtn again.

Step 4, Ic from new Q_{tn} at least 3th iteration or more, the I_C and n will be convergent to a practicable value.

Step 1, Use n=1, *Qtn=Qt, Calculate* Ic.

$$n = 0.381 \cdot I_c + 0.05\left(\frac{\sigma'_{vo}}{\sigma_{atm}}\right) - 0.15$$

$$n \leq 1.0$$

Step 2, put the calculate Ic to calculate n again.

$$I_c = \sqrt{[3.47 - logQ_{tn}]^2 + [1.22 + logF_r]^2}$$

Correlation of Undrained Shear Strength and CPT

$$s_u = \frac{q_t - \sigma_v}{N_{kt}}$$

(Kulhawy & Mayne, 1990)

- Typically Nkt ~10 to 18, Averagely with 14.
- Nkt tends to increase with increasing plasticity
- Decrease with increasing soil sensitivity.
- It is applicable for SBTn in Zone 1, 2, 3, 4 and 9

N_{kt} is in range below and the values are normally used. However, the use

- CPT value in fissured clays is restrained (Meisina, 2013).
- Soft clay: Nkt = 14±4
- Overconsolidated clay: Nkt = 17±5
- Fissured clay: Nkt = 10±30

Lunne et al., 1997 showed that N_{kt} varies with Bq, where N_{kt} decreases and Bq increases, when Bq ~ 1.0 (i.e.. sensitive clay), Nkt can be as low as 6.

Approach II Area of Chek Lap Kok Airport

Where N_k should be read as N_{kt}

Summary of design parameters (Greiner-Maunsell, 1991a)

Typical Index Properties and Recommended Design Parameters	Upper Soft Clay	Stiff Clay	Firm-to-Stiff Clay	Lower Sand
Unit Weight (Mg/m^3)	1.45	1.90	1.85	2.00
Void Ratio, e_o	2.00	(2)	1.03	0.65
Maximum Past Pressure, P_p (3)	$P_p = 4.5 + 7z$	N.A.	$P_p = 55 + 15z$	N.A.
Compression Index, C_c	1.20	(2)	0.42	N.A.
Recompression Index, C_{cr}	0.10	(2)	0.085	0.03
Coeff. of Consolidation, c_v (m^2/year)	1.3	(2)	2.2	N.A.
Coeff. of Reconsolidation, c_{vr} (m^2/year)	20	(2)	15	N.A.
Undrained Shear Strength $N_k = q_{net}/S_u$ (kN/m^2)	$N_k = 23.5$	$N_k = 21.25$ (1)	$N_k = 17$	N.A.
Secondary Compression C_α	N.A.	0.3%	1.5%	N.A.

Notes:(1) A material factor of 0.8 was applied to the Stiff Clay for a conservative solution i.e. $N_k = 17 \times 0.8 = 21.25$.

(2) Data extracted from Figure 5.12 for each area.

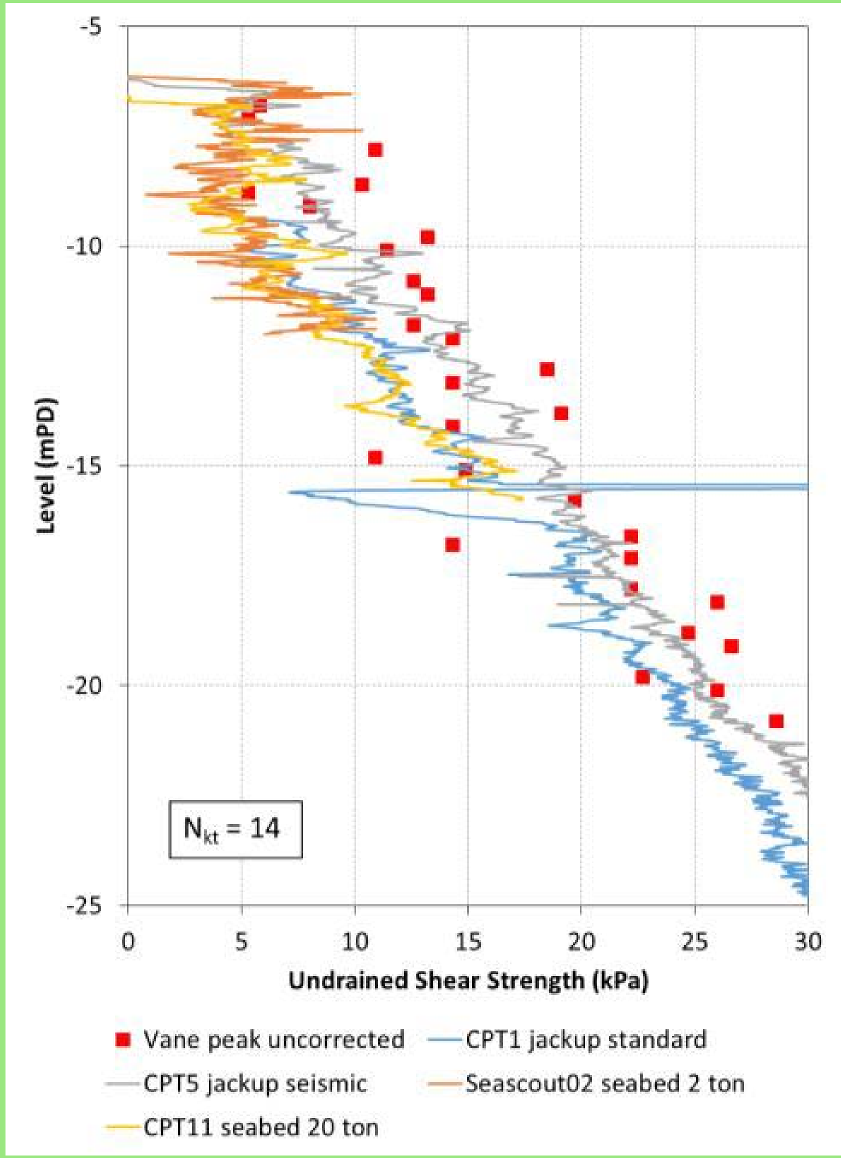
(3) "z" is depth below the seabed.

Plots of Vane Shear Results and CPT at the Site in Lantau

- N_{kt} is ranged between 9 and 18.
- The mean of N_{kt} is 14 is adopted for the site

It was found that the deviation is much at shallow depth from seabed -3 to -10mPD. It may be due to the weight of the 20 ton CPT seabed unit that disturbed the soil strength.

Conclusion: To be more conservative for other calculation, take N_{kt} to be 16.

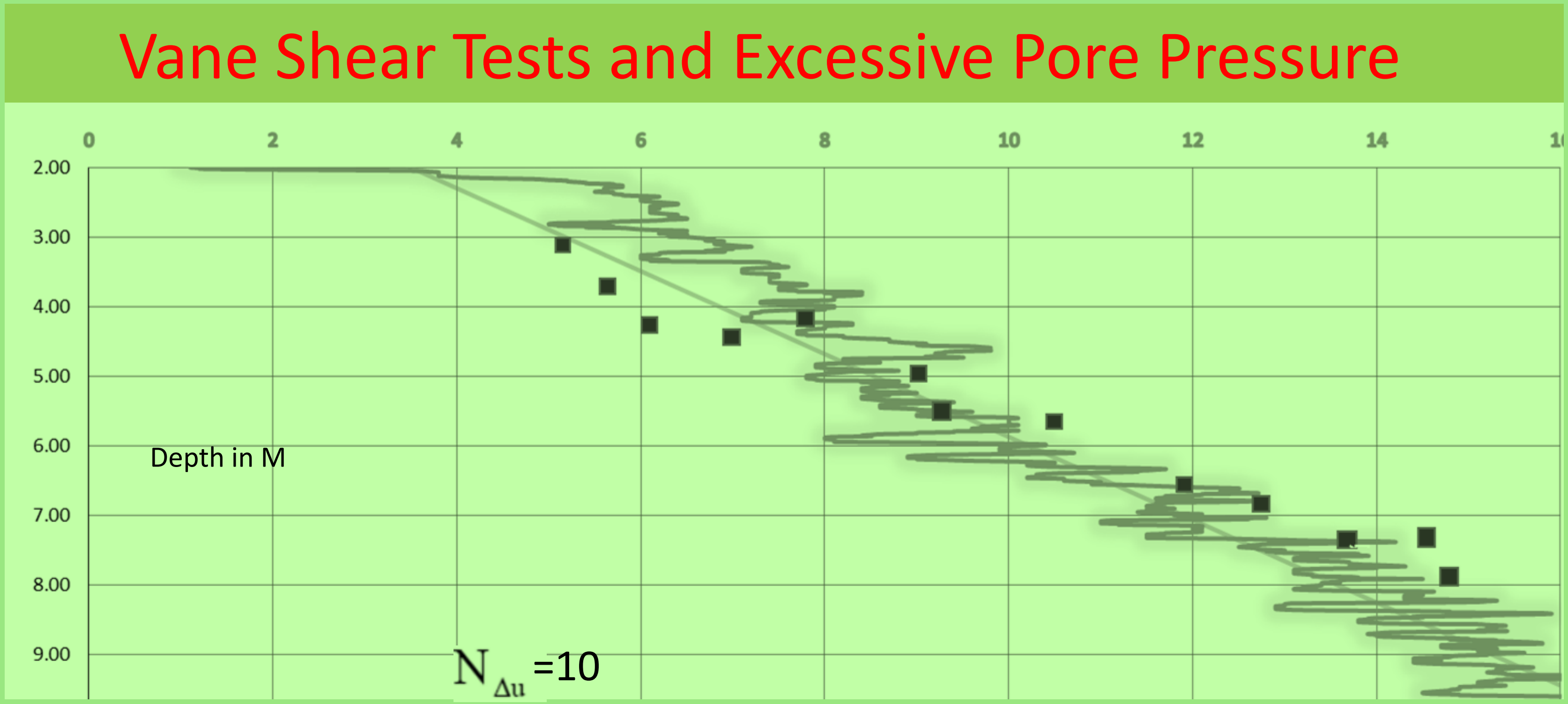


Vane Shear Test and Excess Pore Pressure from CPT

As qc may not be measured in accuracy in very soft clay at shallow and intermittent depth, the N_{kt} values will not be applicable. The following equation should be used:

$$s_u = \frac{\Delta u}{N_{\Delta u}}$$

$$N_{\Delta u} = B_q N_{kt}$$



$N_{\Delta u}$ should be ranged between 4 and 10, but the field tests at Site B in Hong Kong found that most of the results are ranged between 8 and 13, and the average value is around 10.

Note - No clogging of cone filter.
– Cone should be fully is saturated without air bubble.
– It is seldom to be used in Hong Kong in very soft clay strata.

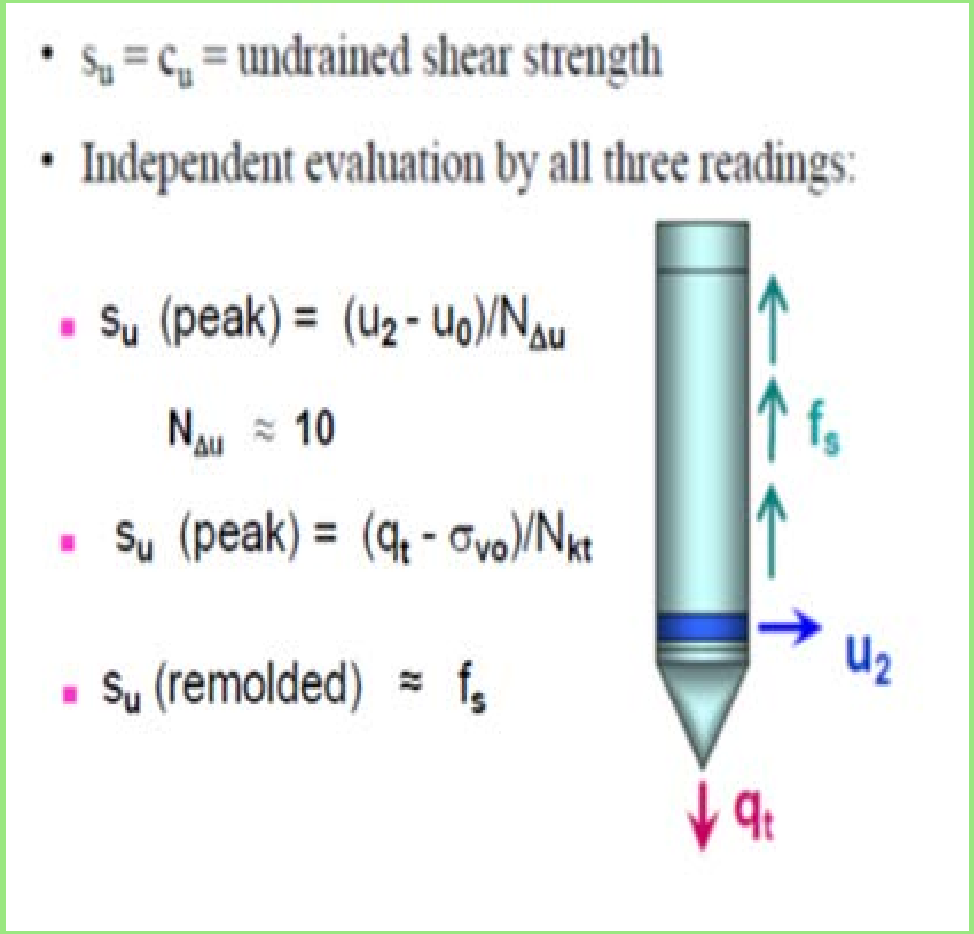
Peak and Remolded Undrained Shear Strengths

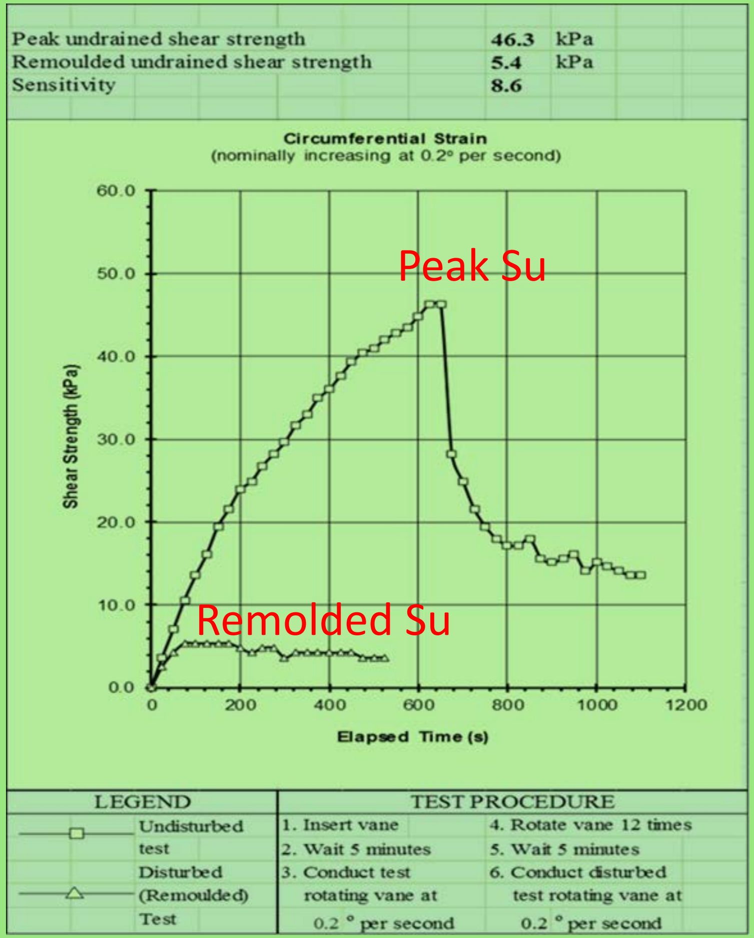
Apart from the CPT could be derived to evaluating the peak strength of the clay, the equation expressed the measured sleeve friction resistance (fs) that can be considered as a remolded shear strength of clays (Gorman, et al. 1975):

$f_s \approx s_u$ (remolded)

This can serve as a lower bound in assessing the s_u profile.

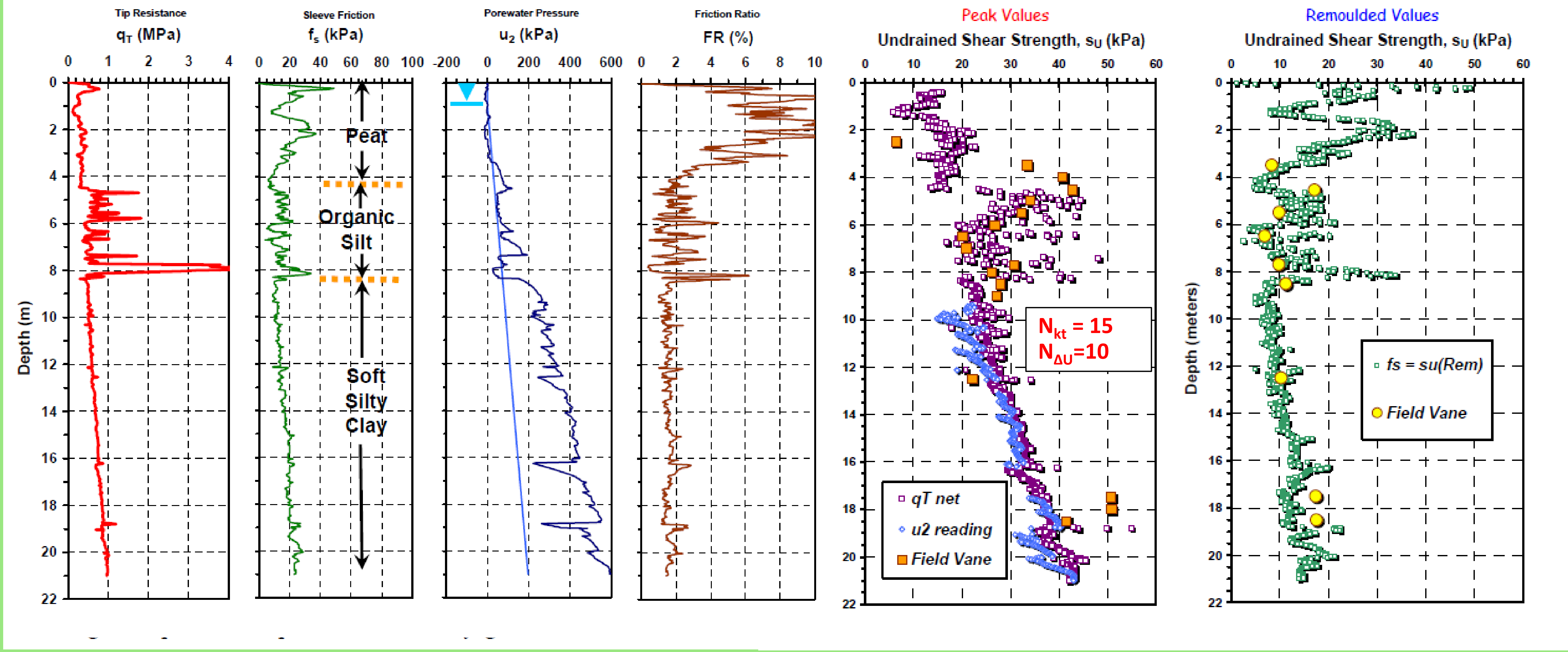
It is applicable for SBTn in Zone 1, 2, 3,4 and 9





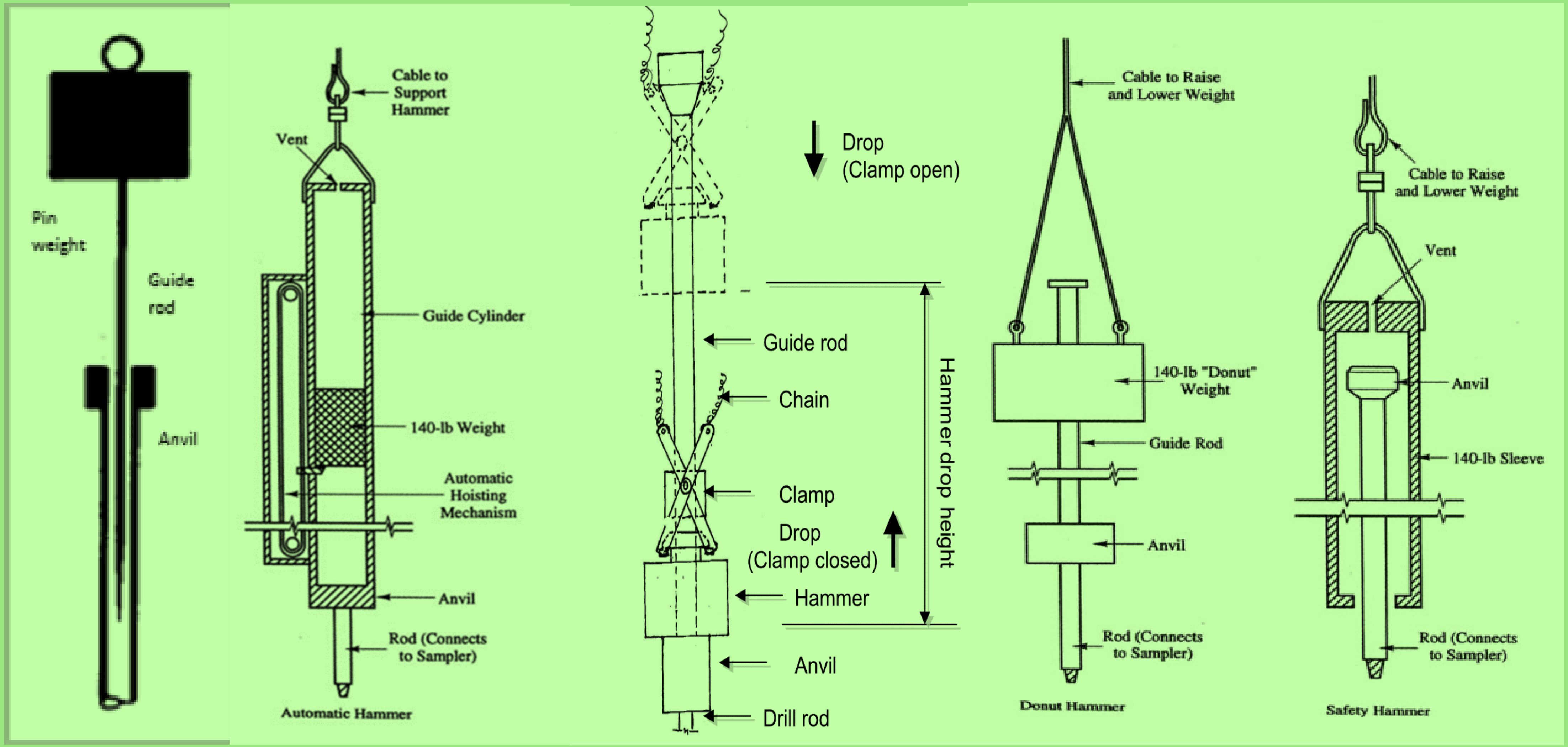
Field Vane Shear Test

Typical Field Test Results with Interpretation of Peak Su and Su (Remoulded)

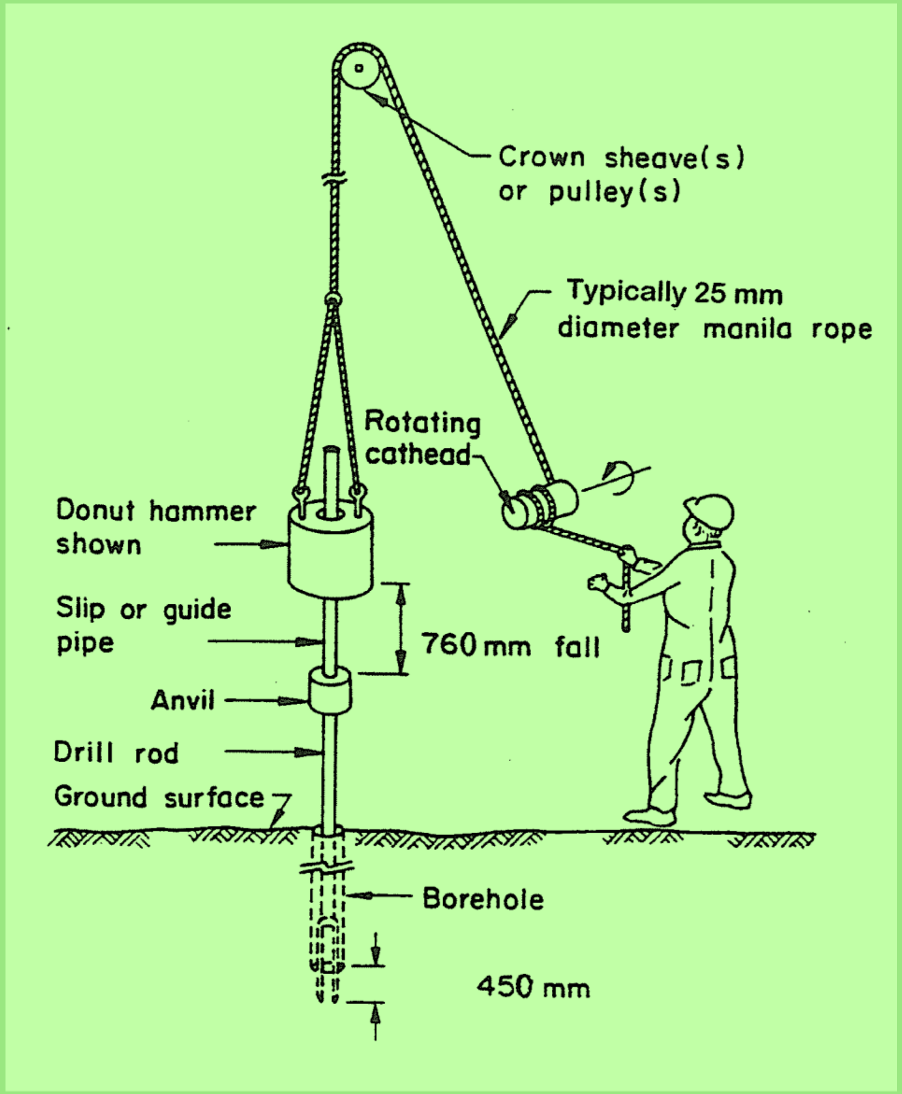


SPT and CPT Correlation

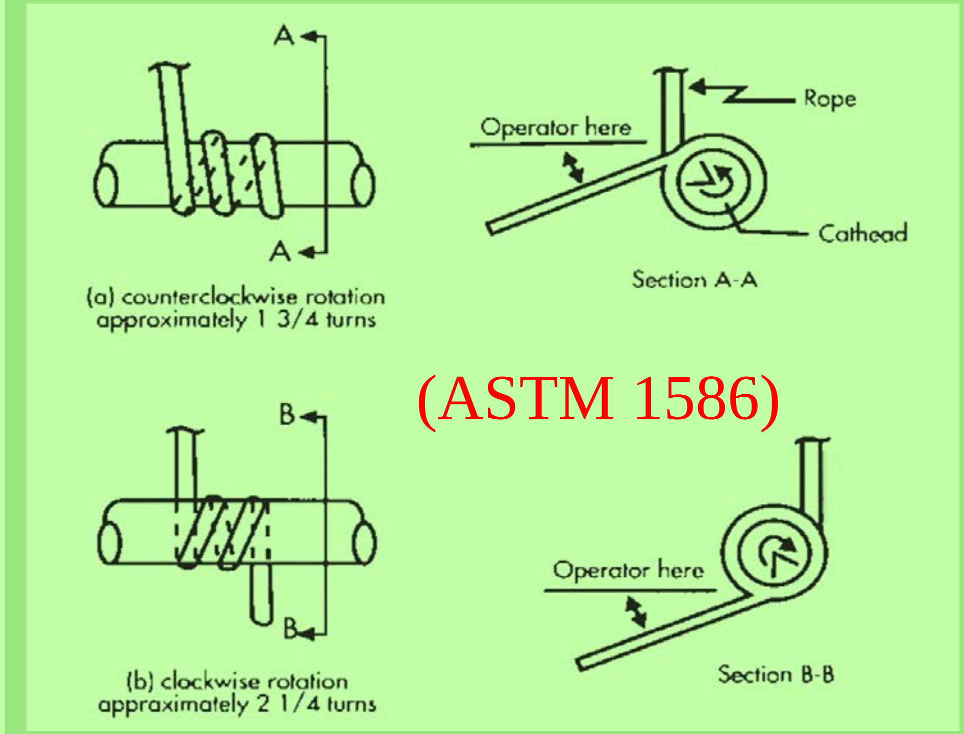
Different Types of SPT Hammers



- The rope and cathead Donut hammer is driven by manual release with rotating drum.
- The rope and cathead system for auto trip release hammer had occasionally been used in Hong Kong since early Nineties.
- It has been experienced that different skills for personnel will have different efficiencies.



Turns on Cathead



Hammer – release – country

- Donut – free fall (Tombi) – Japan
- Donut – rope and pulley – Japan
- Safety – rope and pulley – USA
- Donut – free fall (Trip) – Europe, China, Australia
- Donut – rope and pulley – China
- Donut – rope and pulley – USA

Question: Will the number of turns be different for a tall or little guy? Height of working platform in steps?

	Initial State Parameter				Strength Parameter			Deformation Characteristic *			Flow Characteristic	
Reliability for CPT Data												
Soil Type	Dr	Ψ	Ko	OCR	S_t	Su	φ'	E, G*	M	Go*	k	C_h
Coarse-gained (Sand)	2-3	2-3	5	5			2-3	2-3	2-3	2-3	3-4	3-4
Fine -grained (Clay)			2	1	2	1-2	4	2-4	2-3	2-4	2-3	2-3
Reliability for SPT Data												
Soil Type	Dr	Ψ	Ko	OCR	S_t	Su	φ'	E, G*	M	Go*	k	C_h
Coarse-gained (Sand)	3-4	4		5			3-4	4-5		4-5		
Fine -grained (Clay)		5	5	4	5	3-4	5	4-5	5	4-5	5	5

1=high; 2=high to moderate; 3=moderate; 4=moderate to low; 5=low; Blank = no applicability

* Improved by SCPT

CPT and SPT CORRELATION

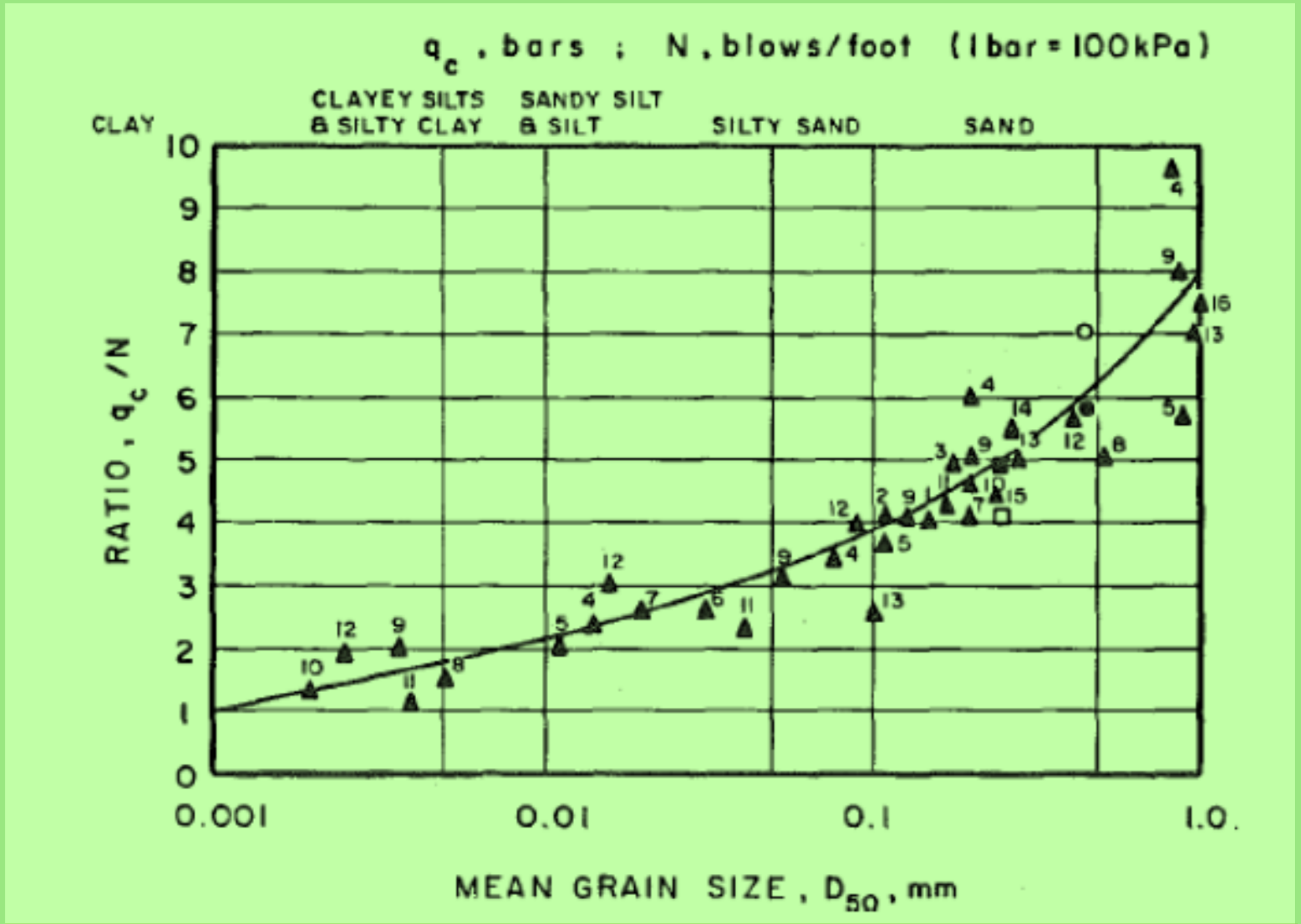
Several factors:

- **Energy level delivered to SPT with N_{60} is being used**
- **Grain size distribution (D_{50})**
- **Fines content (FC)**
- **Overburden stress and other factors**

Single most important factor influencing N value is energy delivered to SPT sampler that is expressed as rod energy ratio.
Energy ratio of 60% is generally accepted to represent average SPT energy, and the results should be corrected to N60.

Studies by employing the standard donut type hammer with a rope and cathead system:

1. Meyerhof(1956)
2. Meigh and Nixon
3. Rodin (1981)
4. De Alencar Velloso(1959)
5. Schmertmann(1970)
6. Sutherland(1974)
7. Thornburn & Macvicar (1974)
8. Campanella et al. (1979)
9. Nixon(1982)
10. Kruizinga(1982)
11. Douglas(1982)
12. Muromachi & Kobayashi (1982)
13. Goel(1982)
14. Ishihara & Koga(1981)
15. Laing(1983)
16. Mitchell (1983)



The Relation between q_c/N and Mean Grain Size from the Previous Studies

Effect of Fine Content and q_c

(Mayne and Kulhawy 1990)

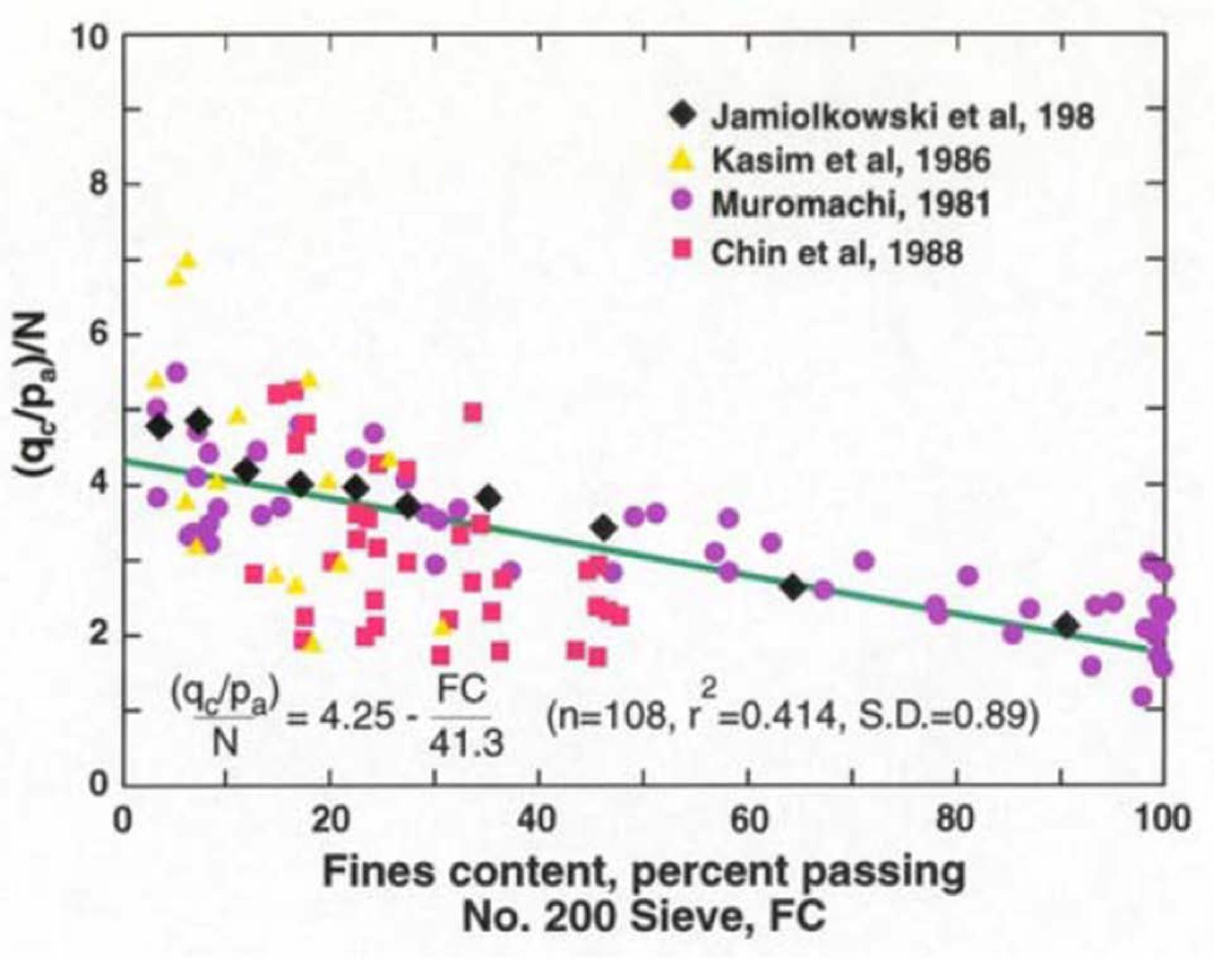
$$\frac{(q_c/P_a)}{N} = 4.25 - FC/41.3$$

Where Pa is the atmospheric pressure= 1Bar= 100KPa

Fine content is % of soil in weight passing through the Sieve No. 200 is equivalent to 0.074 mm (I.e. 74 microns)

Question:

For the correlation graph, what is the type of the SPT hammer being used ? What is the Energy Efficiency for the SPT Hammers?



SPT Correction Factors for N_{60}

N (correction) = N (measured) X ER / E60

$$N_{60} = \frac{ER\ N\ C_B C_S C_R}{60}$$

(From Skempton, 1986)

Where ER=Efficiency of the free-fall hammer energy (Ranged between 40 and 85 in the equation by ignoring the % in the equation)

SPT Correction Factor for Field Operation

Factor	Equipment Variables	Value 值
Borehole diameter factor, C_B	2.5 - 4.5 in (65 - 115 mm)	1.00
钻孔直径校正(CB):	6 in (150 mm)	1.05
	8 in (200 mm)	1.15
Sampling method factor, C_S	Standard sampler	1.00
采样器校正(Cs):	Sampler without liner (Not recommended 不建议)	1.20
Rod length factor, C_R	10 - 13 ft (3 - 4 m)	0.75
杆长校正(CR):	13 - 20 ft (4 - 6 m)	0.85
	20 - 30 ft (6 - 10 m)	0.95
	> 30 ft (> 10 m)	1.00

Adapted from Skempton (1986).

Summary for Energy Efficiency for Trip Hammer in Hong Kong

GEO Technical Note: TN 2/97, 1997

Hammer Fixity	ER_r	Energy Loss As Compared to Case 1
Case 1	43%	N/A
Case 2	36%	16%
Case 3	29%	33%

Notes : (1) ER_r is the calculated rod energy ratio.
(2) Case 1 - Hammer tightly-fitted at top and base.
(3) Case 2 - Hammer tightly-fitted at top, loosely-fitted at base.
(4) Case 3 - Hammer loosely-fitted at top, tightly-fitted at base.

Summary for SPT Hammers in Different Regions

	North America	South America	Middle East	United Kingdom	Japan	Hong Kong
Borehole Diameter (mm)		South America	100 to 150	152 to 375	65 to 110	89 to 140
Hammer Mechanism	Slip-rope, safety	Slip-rope	Automatic trip	Automatic trip	Slip-rope	Automatic trip
Average Rod Energy Efficiency(%)	45 to 60	45 to 50		73	65	?

1) GEO Technical Note: TN 2/97, 1997 with ER ranged from 16% to 33%. The results was reviewed for some errors for further study.

2) Doctorate Thesis by YANG Wenwei in 2006 , HKU (106 measurements with ER ranged from 33% to 80%, mean » 60%)

3) Philip Chung 2018, Hong Kong Geotechnical Conference 2018 by Geotechnical Division (The mean of the ER measured is 68%).

4) CEDD Contract No: GE/2019/16 – Ground Investigation for New Territories East, ER testing in progress.

CPT and SPT Correlation

Corrections mostly from Robertsen et al, 1983 or Kulhavy and Mayne, 1990

- **If grain size distribution data are available, use the Figure (qc/pa)/N_{60} Vs D50 from Robertson et al.,1983 or Figure (qc/pa)/N Vs Fines Content from Kulhawy et, al, 1990**

- **If grain size distribution data are not available, use soil behavior index, use the following equation from Jeffries and Davies 1993:**

$$(q_c/P_a)/N_{60} = 8.5\,(1 - I_C / 4.6)$$ **wh***ere Ic =* $I_c = ((3.47 - \log Q_t)^2 + (\log F_r + 1.22)^2)^{0.5}$

Q_t = Qt = normalized cone penetration resistance (dimensionless)
= $(q_t - \sigma_{vo})/\sigma'_{vo}$
F_r = normalized friction ratio in %
= $(fs/(qt - \sigma_{vo}))$ x 100%
P_a = atm. Press. = 100 kPa
N_{60} = SPT value corresponding to energy ratio of 60%

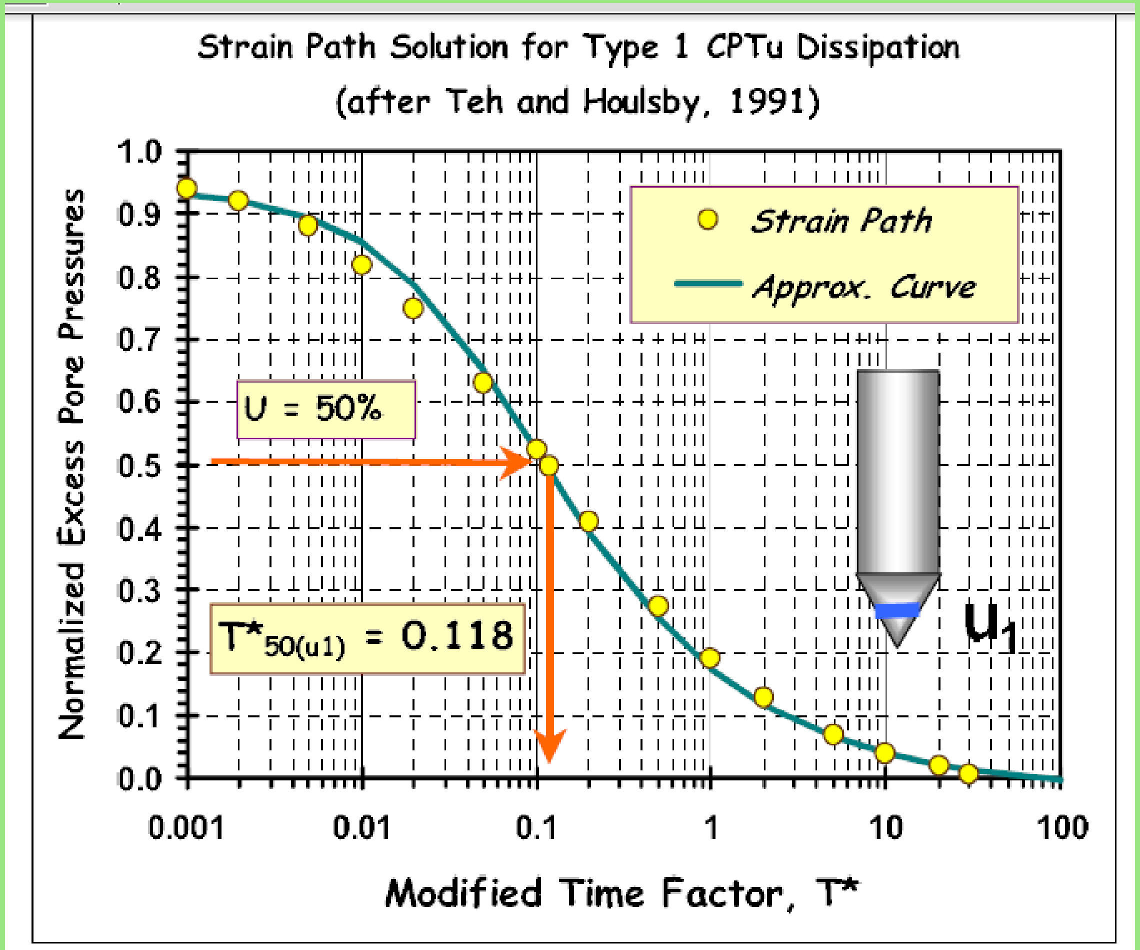
Note: As N_{60} for the above equation (obtained by correlations of different parameters) is based on the correctness of the ER (Energy Efficiency) for different types of SPT hammers, it should be reviewed or amended particularly for the auto-trip hammer adopted in Hong Kong.

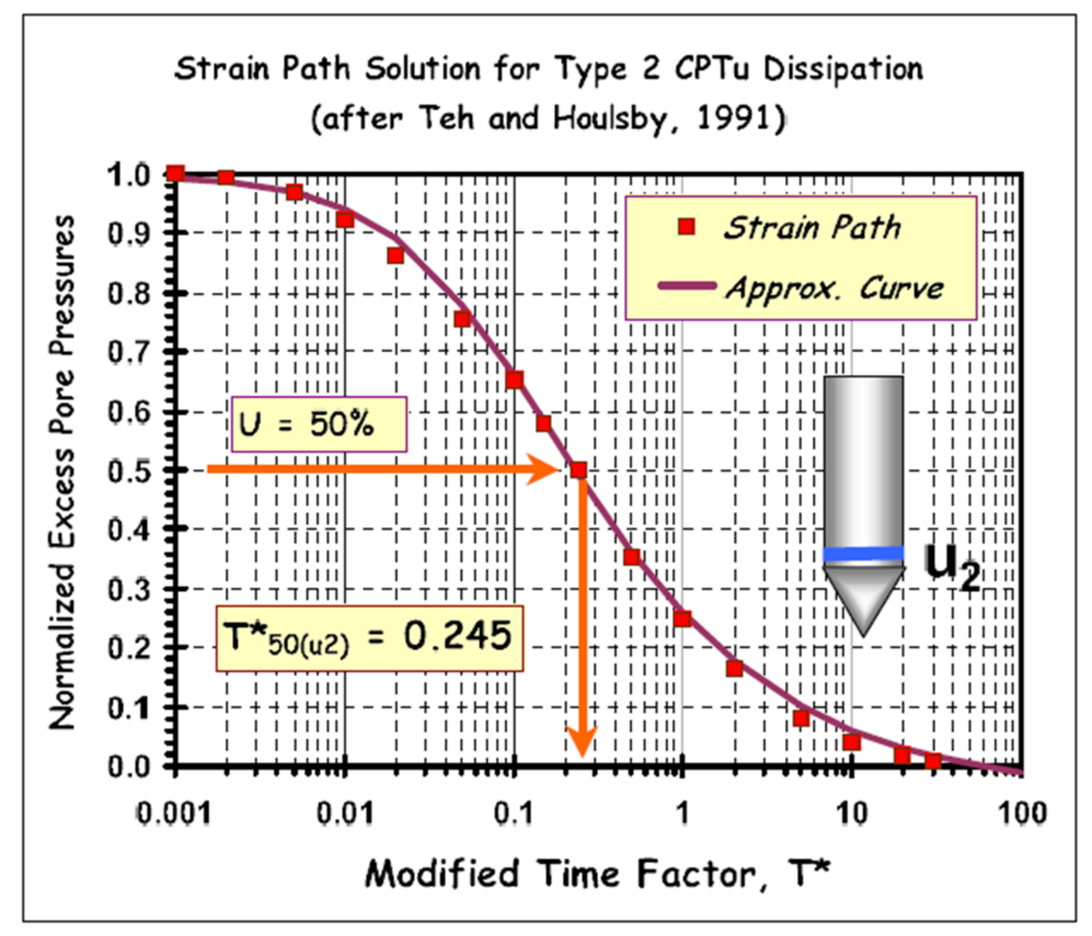
Correlation of SPT and CPT Values in Hong Kong

- SPT is most commonly used in Hong Kong.
- It has never been calibrated with SPT for N_{60}. since 1997 in Hong Kong.
- The previous works by GEO for ER found that the auto-trip hammer for SPT was max 43%, and recently calibrated SPT value was around 68%. It was concluded that parts of the energy dissipated due to some unknown factors.
- The ER is believed that the ER should be higher, and GEO still performs further study with review for this.
- The further studies are aimed at improving the equations for correlation of SPT N_{60} , $N_{60(1)}$.
- The N_{60} correlated from CPT equation should only be used as the approximate values in absence of more reliable data, and the values should be compared and corrected with some local data.

Dissipation Test and Consolidation Characteristics

Monotonic Dissipation Curve

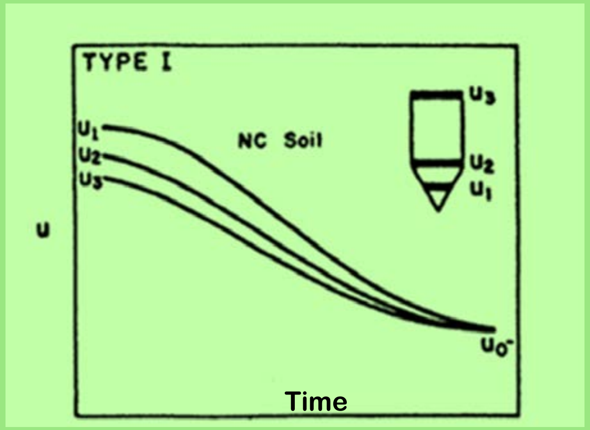




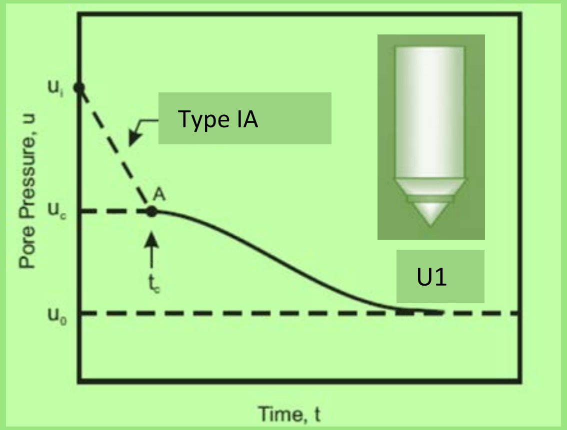
Horizontal Coefficient of Consolidation Ch $=T^*_{50} * r2 * (I_R)^{0.5}/t_{50}$

Diameter of CPT Cone = R = 1.785 cm for 10cm2 cone, and R=2.2 cm for 15cm2 cone

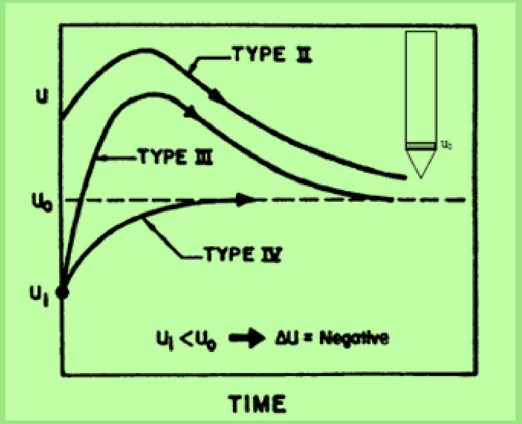
Rigidity Index = Ir = Shear Modulus / Undrained Shear Strength = G/Su



Normally to Lightly Overconsolidated Soil for U1 and U3 filters, and Heavily Consolidated soil for U1 filter



Unloading type of dissipation for U1 filter in overconsolidated soil



Pore Pressure Dissipation for Shoulder Piezocone Elements in Heavily Overconsolidated Clays (Sully and Campanella, 1994)

Cone Filter Type	Dissipation Response Type	Types of Dissipation Behavior	Soil Type
U1, U2 and U3	I	Monotonic	Normally to lightly consolidated soil
U1	I	Monotonic	Lightly to heavily consolidated soil
U1	IA	Monotonic	Normally to heavily consolidated soil
U2	II U > Uo	Dilative	Moderately to heavily consolidated, fissured soil or dense sand
U2	III U < Uo or Δu is negative	Dilative	Moderately to heavily consolidated, fissured soil or dense sand
U2	IV U < Uo and no peak	Dilative (Treated as inverse of monotonic Type I for C_h calculation)	Moderately to heavily consolidated, fissured soil or dense sand

Calculation of Coefficient of Horizontal Consolidation (C_h) from Dilatory (Non-standard) Type II and III Dissipation Curves

Authors	Theories and Methods Adopted	Interpretation Methods
Burns and Mayne (1998)	The method combines Cavity Expansion and Critical State Soil Mechanics theories	The solution process requires a computer program and iteration to obtain a good fit of the measured dissipation curve. During the fitting process both the horizontal coefficient of consolidation (Ch) and the rigidity index (Ir) are varied, which may be problematic and lacking a physical basis.
Sully et al. (1999)	Corrected curves with existing methods of interpretation (Teh and Houlsby 1991) based on the combination of strain path method with the large strain finite element analysis to evaluate C_h	Method 1: Logarithm of time plot is adopted. Shift the origin of time to that point where the measured pore pressure is a maximum. Piezocone dissipation tests was developed by Teh and Houlsby
		Method 2: Fit a square root of time plot to the post-maximum pore pressure dissipation curve in order to back-extrapolate the value of the initial pore pressure.
J.C. Chai et al (2004)	Based on the results of numerical analysis, an empirical equation is proposed	Use t_{50m} is calculated from the corrected t_{50} (The time corresponding to 50% dissipation of the measured maximum excess pore pressure) and the t_{umax} (Time for the measured excess pore pressure to reach its maximum value). The t_{50m} is time is then used in the standard interpretation (Teh and Housby 1991) of the value of C_h.

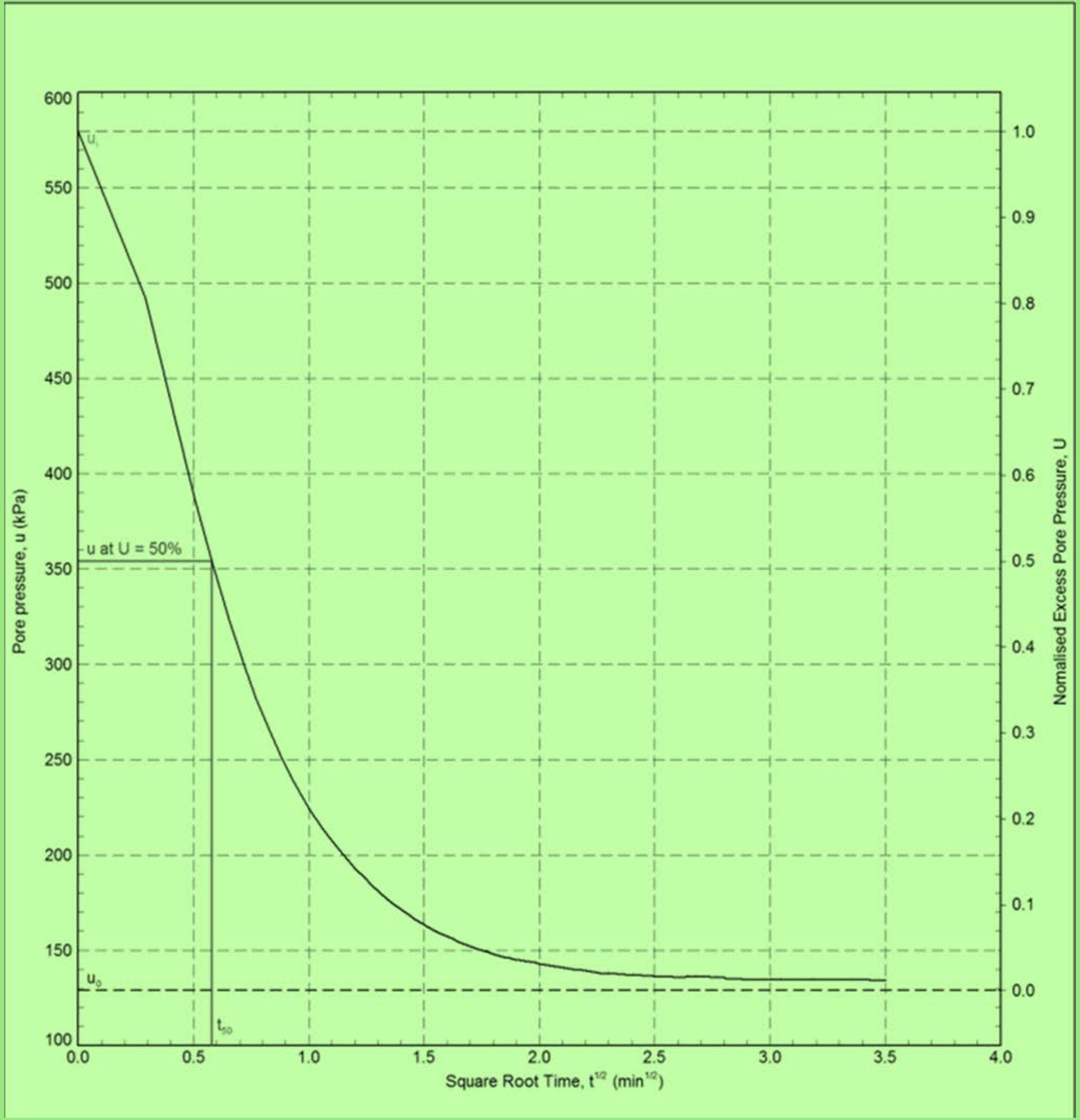
Typical Calculation for C_h for Monotonic (Standard) Dissipation

U_0=128.76KPa
U_i=579.6KPa
Dissipated pressure at 50%
= (128.76+579.6)/
T_{50}*= 0.245 for U_2 filter cone 2=354.18KPa
r=1.785cm for 10cm2 cone
$(t_{50})^{0.5}$=0.584
t_{50}=0.34min
I_R=150 (Assumed)

$$c_h = \frac{(T_{50}*) r^2 \sqrt{I_R}}{t_{50}}$$

C_h=0.245x1.785^{2}x(150)$^{0.5}$/0.34
= 28.12 cm^2/min
= 1.47x10^3 m2/yr

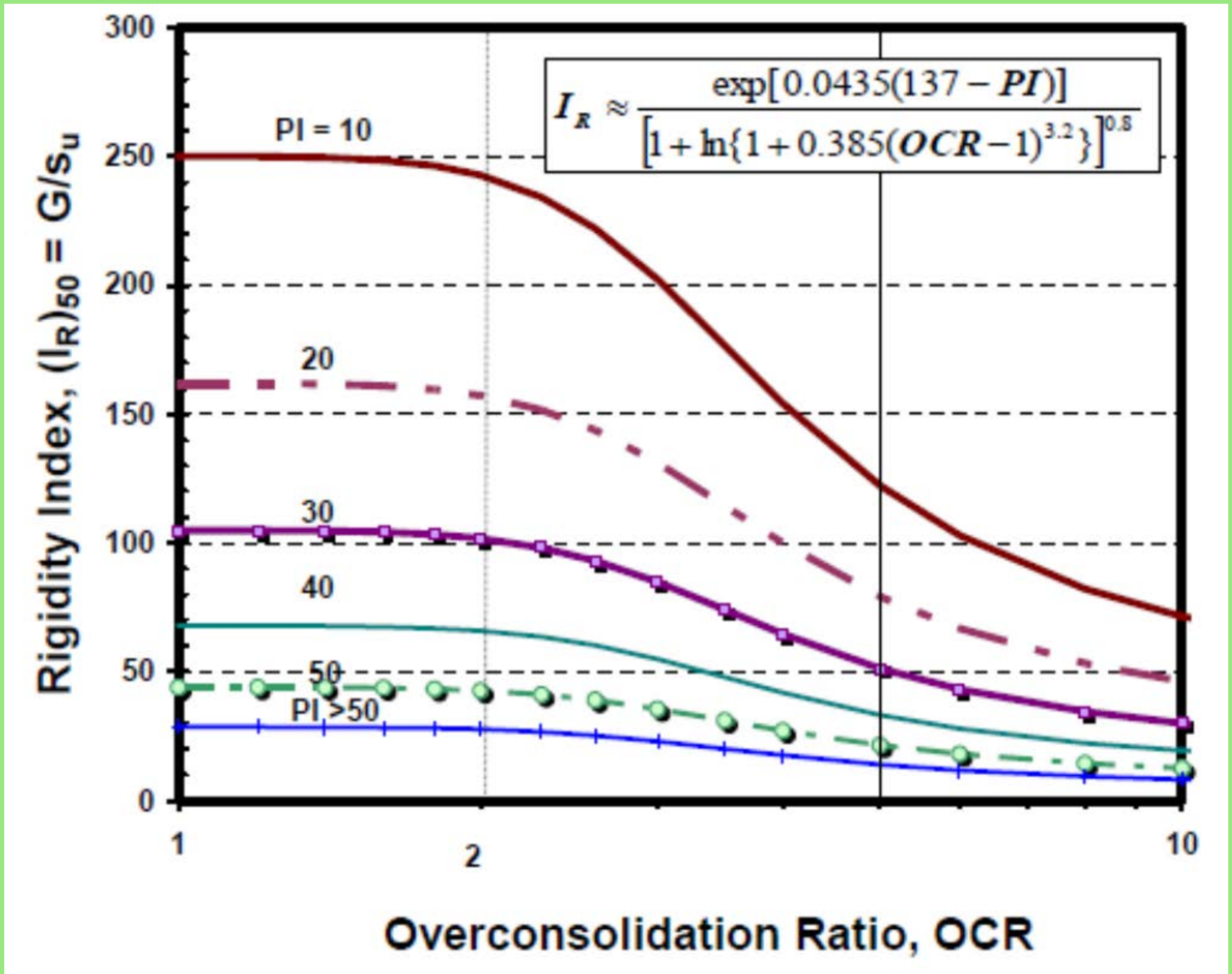
Note: Use the smaller cone, 10cm2 cone, will give you shorter time of t_{50} as compared with the 15cm2 cone.



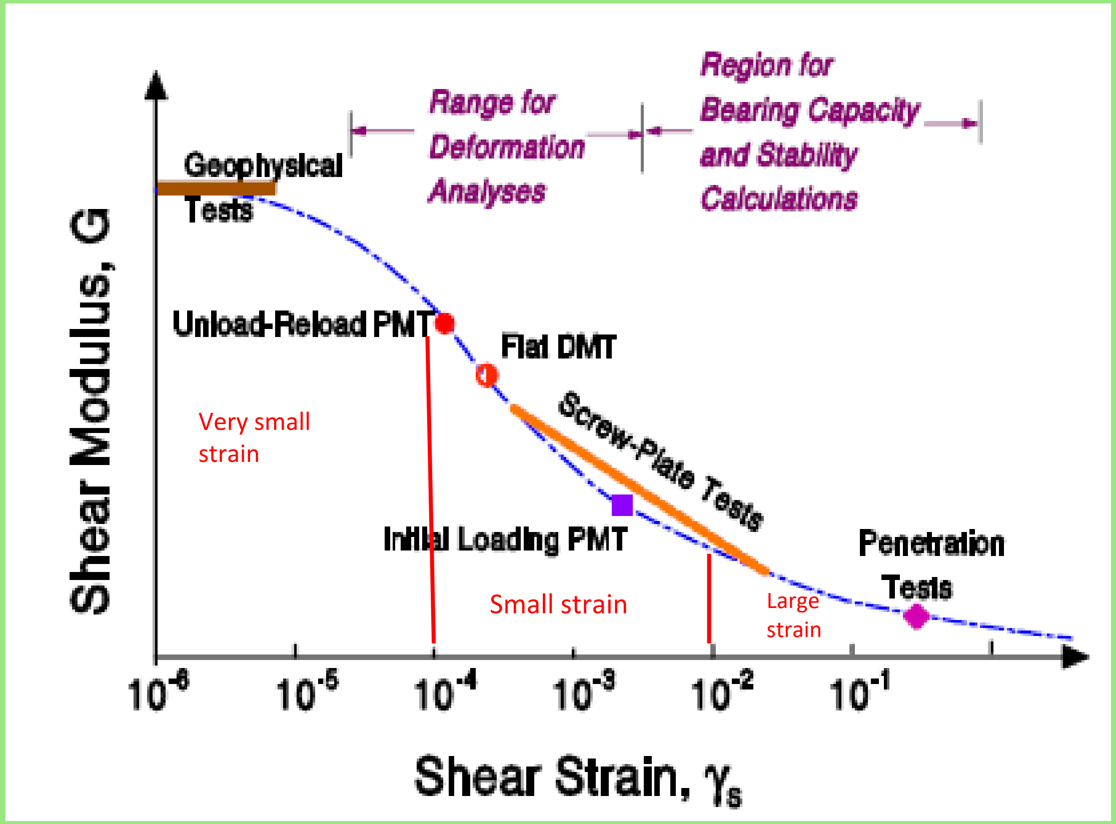
Selection of An Appropriate Shear Modulus is a primary challenge

Shear Modulus (G) is function of strain level, aging effects, various other factors (Wroth et al. 1979, and Schnaid et al. 1997).

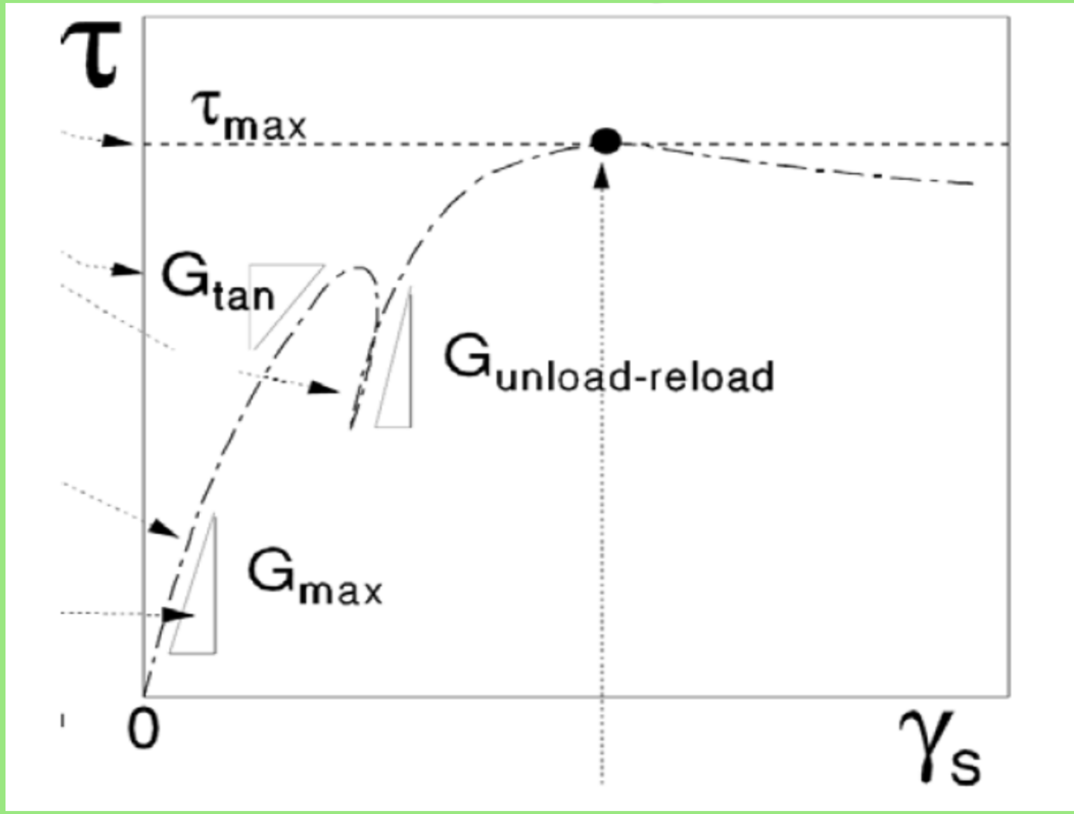
Researchers suggest that use of G_{50} (i.e. 50% of the mobilised strength that represents the average response of the engaged soil volume (Konrad & Law 1987, Schnaid et al. 1997). The G_{50} is appropriate for I_R since it most likely represents an average response of the soil around an advancing cone.



Evaluation of Rigidity Index from Plasticity Index and OCR (after Keaveny & Mitchell, 1986).



Reduction of Shear Modulus Vs Shear Strain



Shear Modulus Based On Stress Strain Response

The initial shear modulus, G_{max} (G_o), typically represents the tangent modulus at low strains (< 0.01%), while a secant modulus is used for larger strain levels and G decreases with increasing strain level (Houlsby & Wroth 1991, Mayne 2007).

Selection of Ir for Medium to Very Highly Plastic Soil

The range of Ir for the very highly plastic soils are between 20 to 40, and 500 for non plastic soil. The max ratio of $Ir^{0.5}$ is around 5 times It is less the half of one order in magnitude for calculating the C_h.

It is often considered as acceptable that accuracy in the estimate of the coefficient of consolidation varies within one order of magnitude (Robertson 2015).

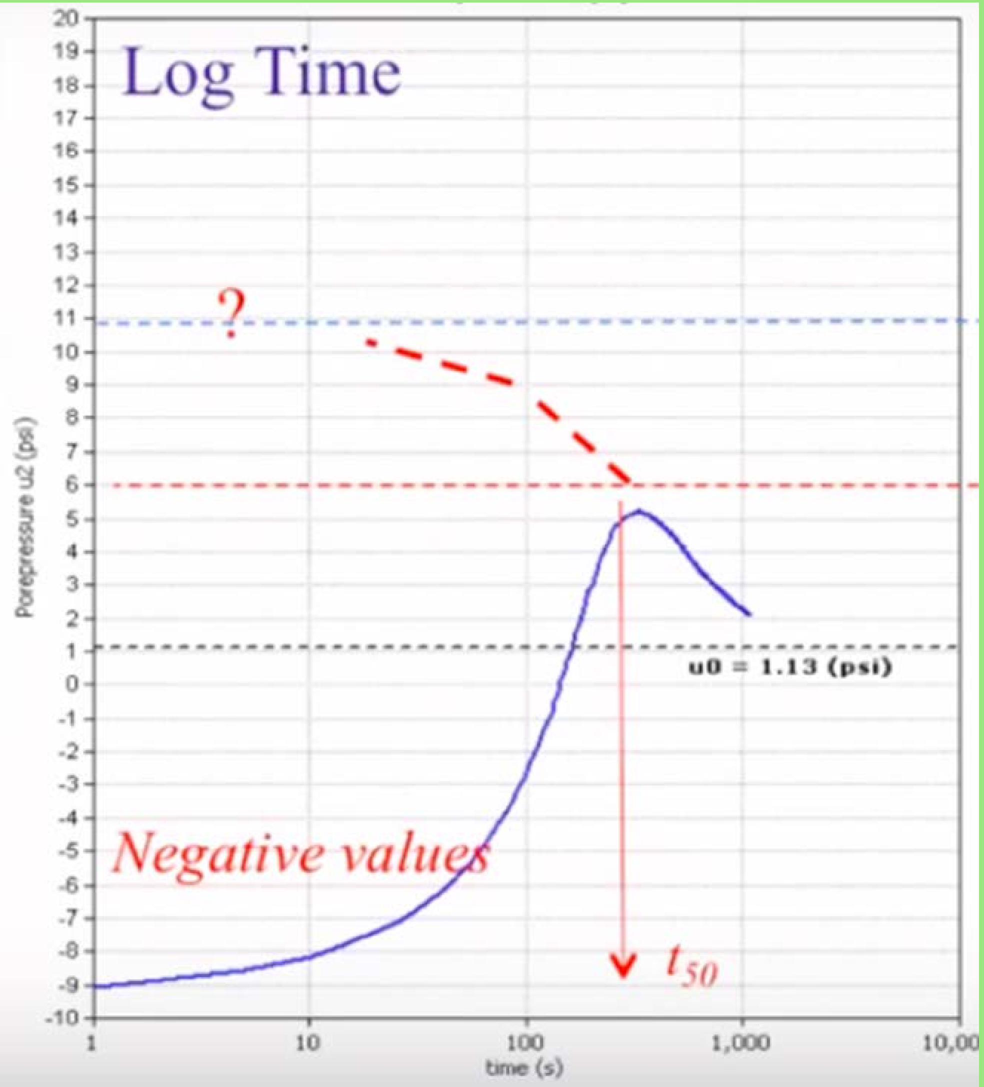
Soil Type	Plasticity Index	PI	OCR	Ir	$(Ir)^{0.5}$	Max Fold
Sand to Silty Sand	Non Plastic to Low	<10		500	**22.36**	5
Silty Sand to Silt	Medium Plastic	10*	1	250	**15.81**	3.54
Silty Clay to Clay	Highly Plastic	30	1	110	10.49	
Clay	Very Highly Plastic	50	1	40	6.32	
Sand to Silty Sand	Non Plastic to Low	<10				
Silty Sand to Silt	Medium Plastic	10	10	8	8.94	
Silty Clay to Clay	Highly Plastic	30	10	30	5.48	
Clay	Very Highly Plastic	50	10	20	**4.47**	
Remark	*** It is generally taken for 7 as Low Plastic but it is suggested to be 10 by Robertson**					

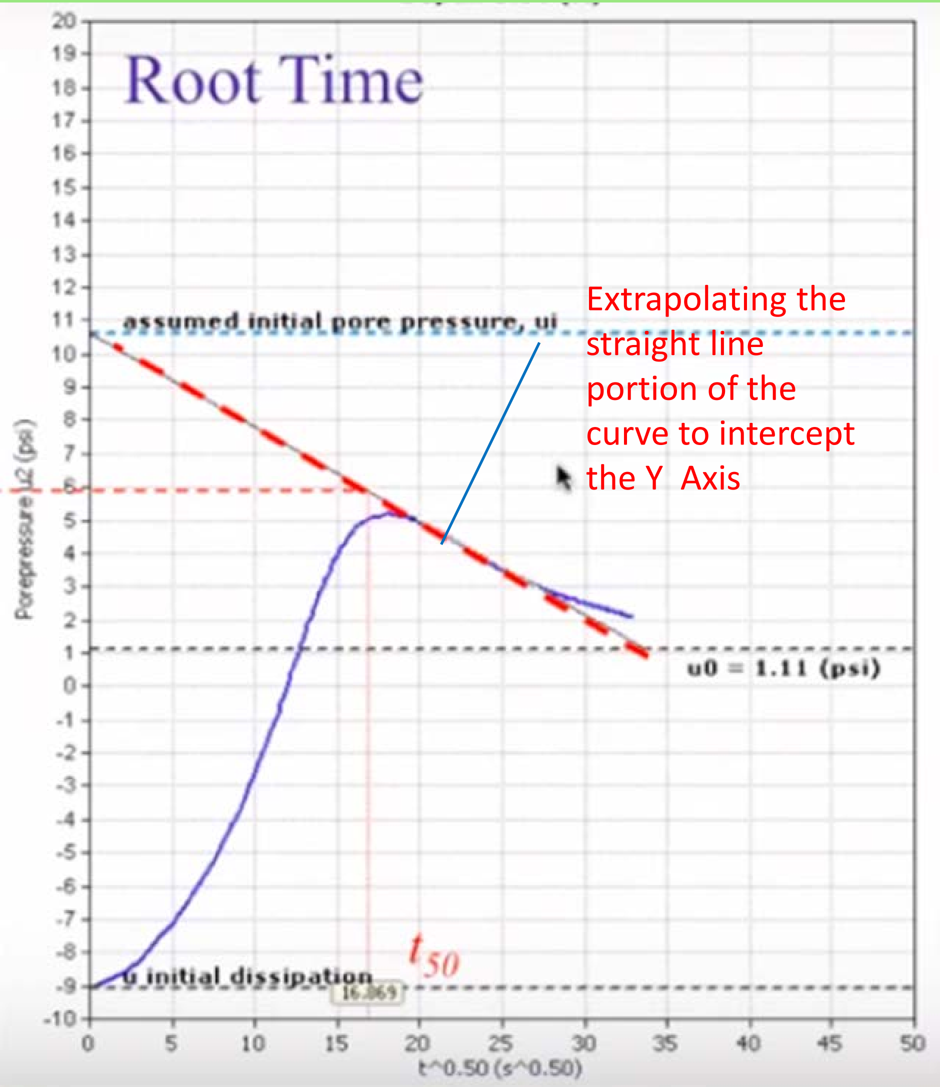
Plasticity index (%)	Soil type	Degree of plasticity	Degree of cohesiveness
0	Sand	Non-plastic	Non-cohesive
<7 (10 by Robertson)	Silt	Low plastic	Partly cohesive
7-17	Silt clay	Medium plastic	Cohesive
>17	Clay	High plastic	cohesive

Dilatory Dissipation Curve

Two Plots for Typical Dilatory Dissipation used U_2 Cone Filter Element.

The Square Root Time Plot is more applicable than the Log Time Plot.



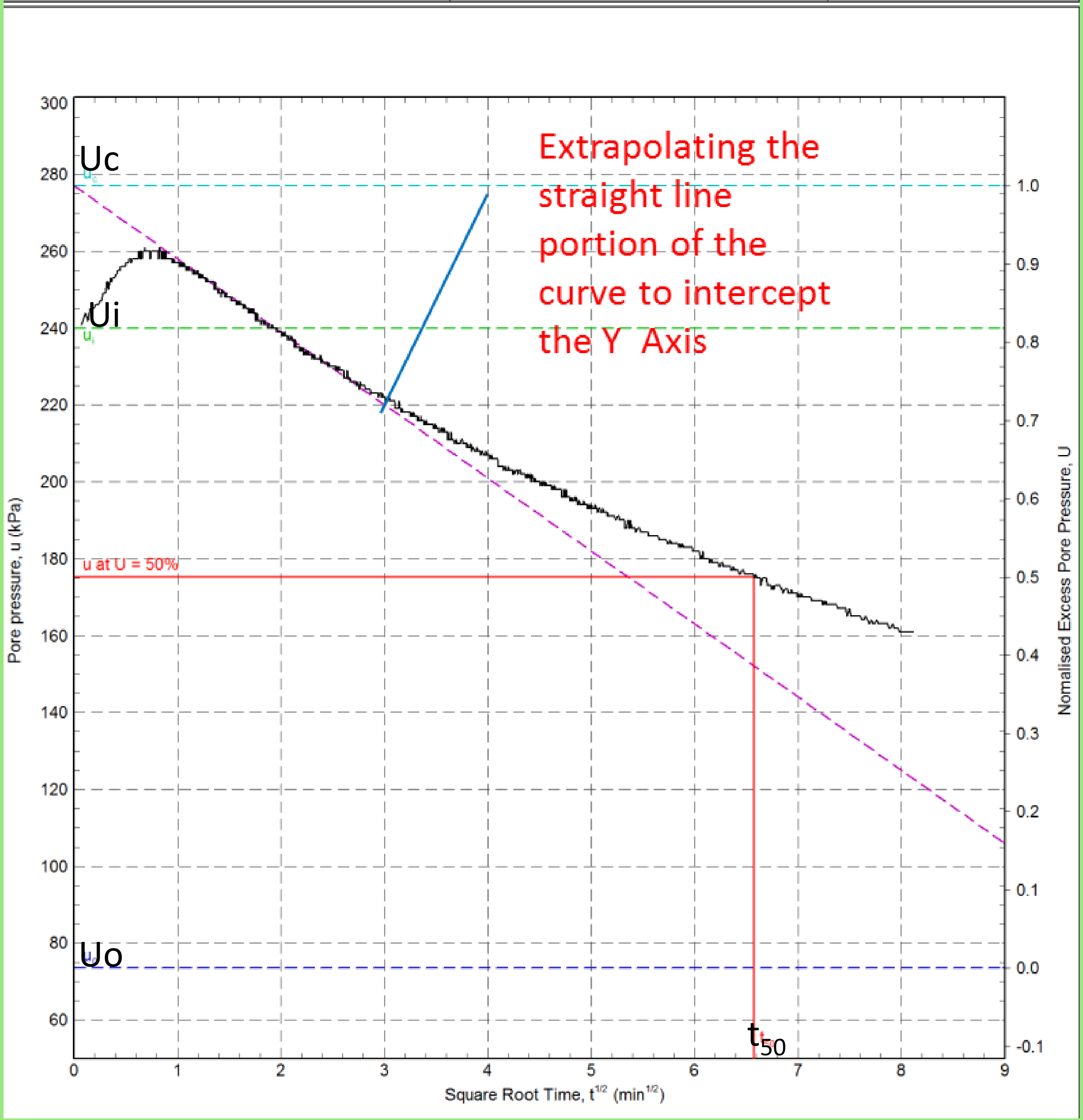


Typical Calculation for C_h for Dilatory Dissipation

U_0=73.48KPa
U_i=240KPa
Uc= 277KPa
Dissipated pressure at 50%
= (277+73.48)/2=175.24KPa
For 10 cm^2 cone with filter at shoulder
T_{50}*= 0.245
r=1.785cm
t_{50}= 43.27min
I_R=120 (Assumed)

$$c_h = \frac{(T_{50}*)r^2\sqrt{I_R}}{t_{50}}$$

C_h=0.245x1.785^{2}x(120)$^{0.5}$/43.27
= 0.198 cm^2/min
= 10.3 m2/yr



Proposed C_h Calculation for Dilatory (Non-standard) Dissipation by Chai et al. 2012

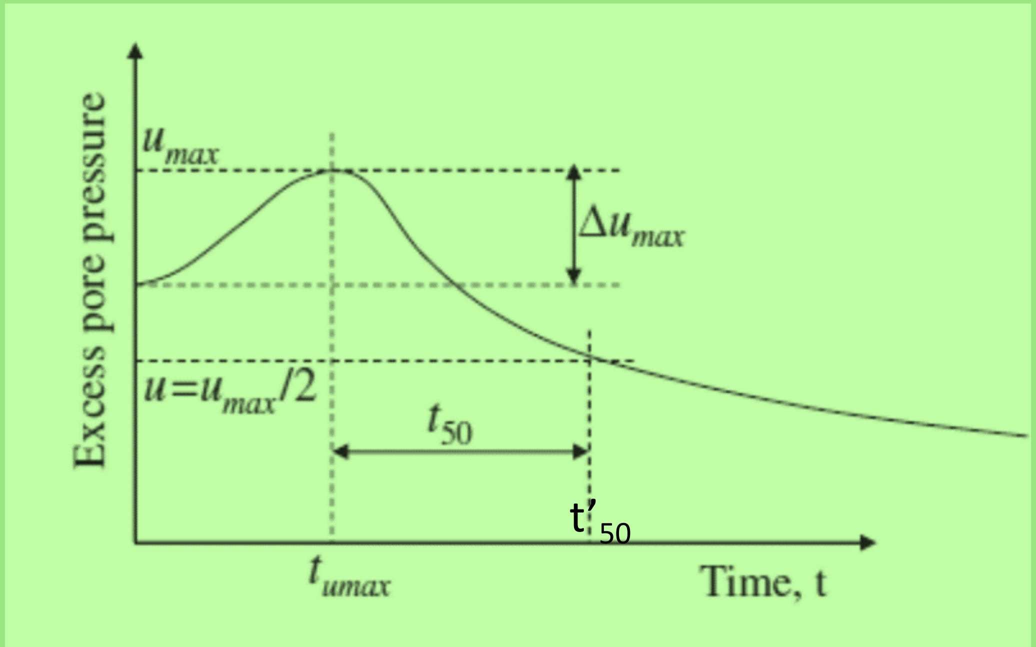
The equation proposed by Chai et al. (2012a) for evaluating t_{50m} is as follows:

$$t_{50m} = \frac{t_{50}}{1+18.5\left(\frac{t_{u\max}}{t_{50}}\right)^{0.67}\left(\frac{I_r}{200}\right)^{0.3}}$$

where t_{50m} is corrected time for 50% excess pore pressure dissipation, and t_{50} is time difference between the maximum and 50% of the maximum excess pore pressure. The t_{umax} is time for the measured excess pore pressure to reach its maximum value.

The corrected time is defined as t_{50m}, and then this value of t_{50m} is used in the equation proposed by Teh and Houlsby (1991) for standard dissipation curves to directly calculate the value of C_h. Then the C_h value can be calculated with the following equation

$$C_h = \frac{T^* \, r^2 \, (I_r)^{0.5}}{t_{50m}}$$

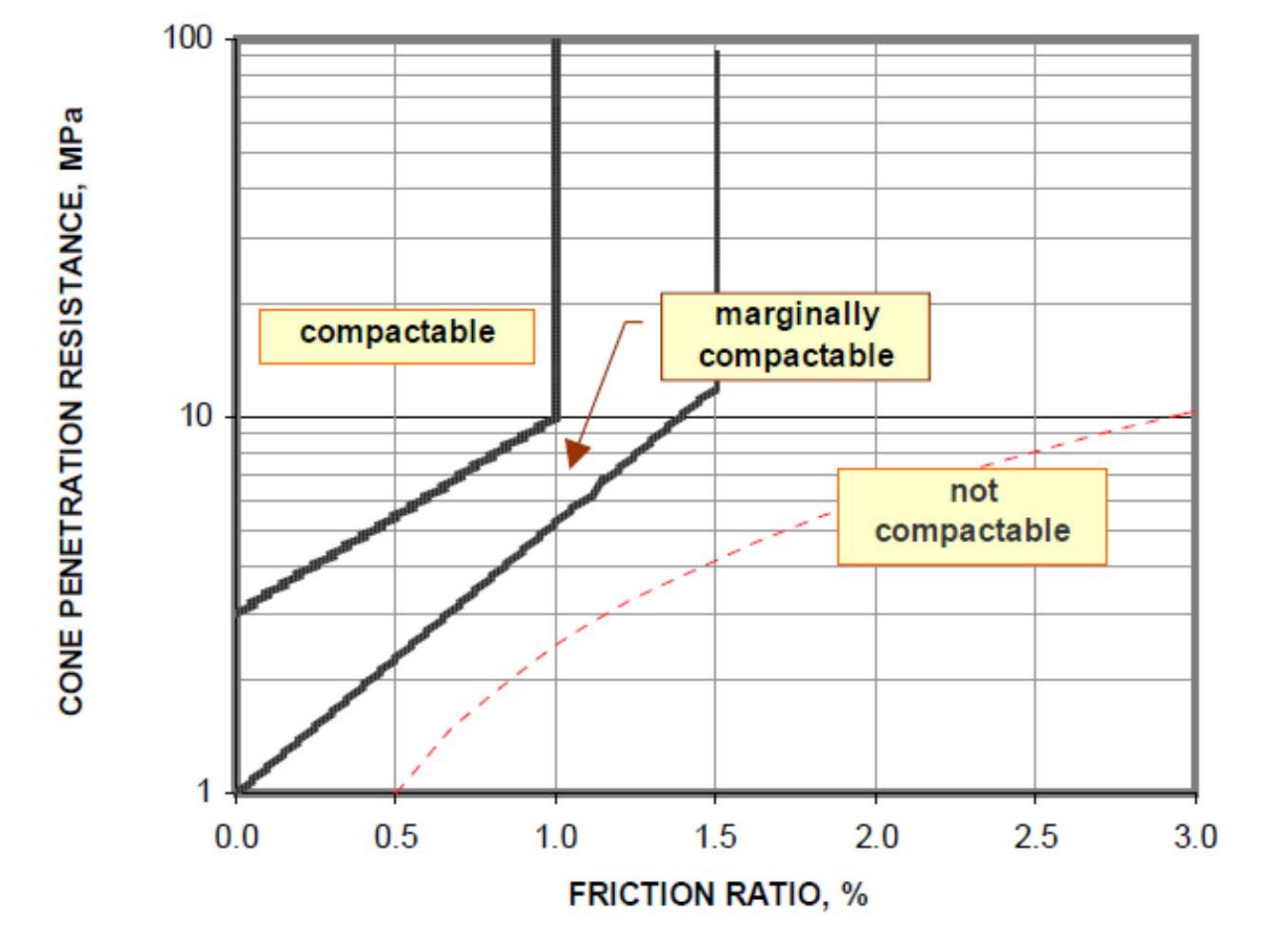


For the u_2 filter, the equation becomes

$$C_h = \frac{0.245 \, r^2 \, (I_r)^{0.5}}{t_{50m}}$$

Application of CPT on Deep Compaction

Soil Classification for Deep Compaction Based on CPT Data (After Massarsch 1991)



Friction Ratio Vs Fine Content from Suzuki et al. 1995

Limits of Application for Deep Vibro compaction Technique

From Keller,

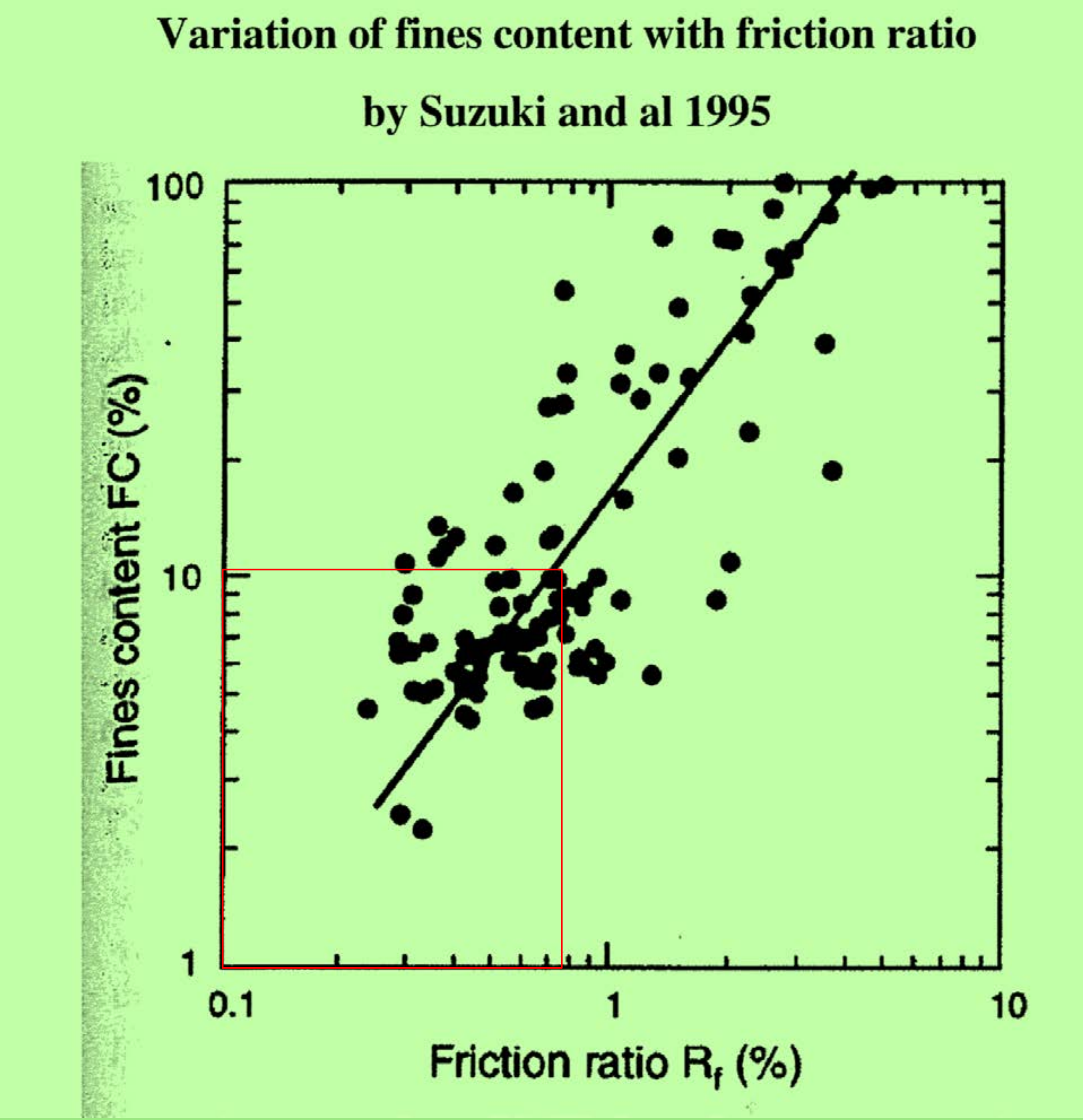
Fine Content: FC < 15%

Friction Ratio: Rf < 1 %

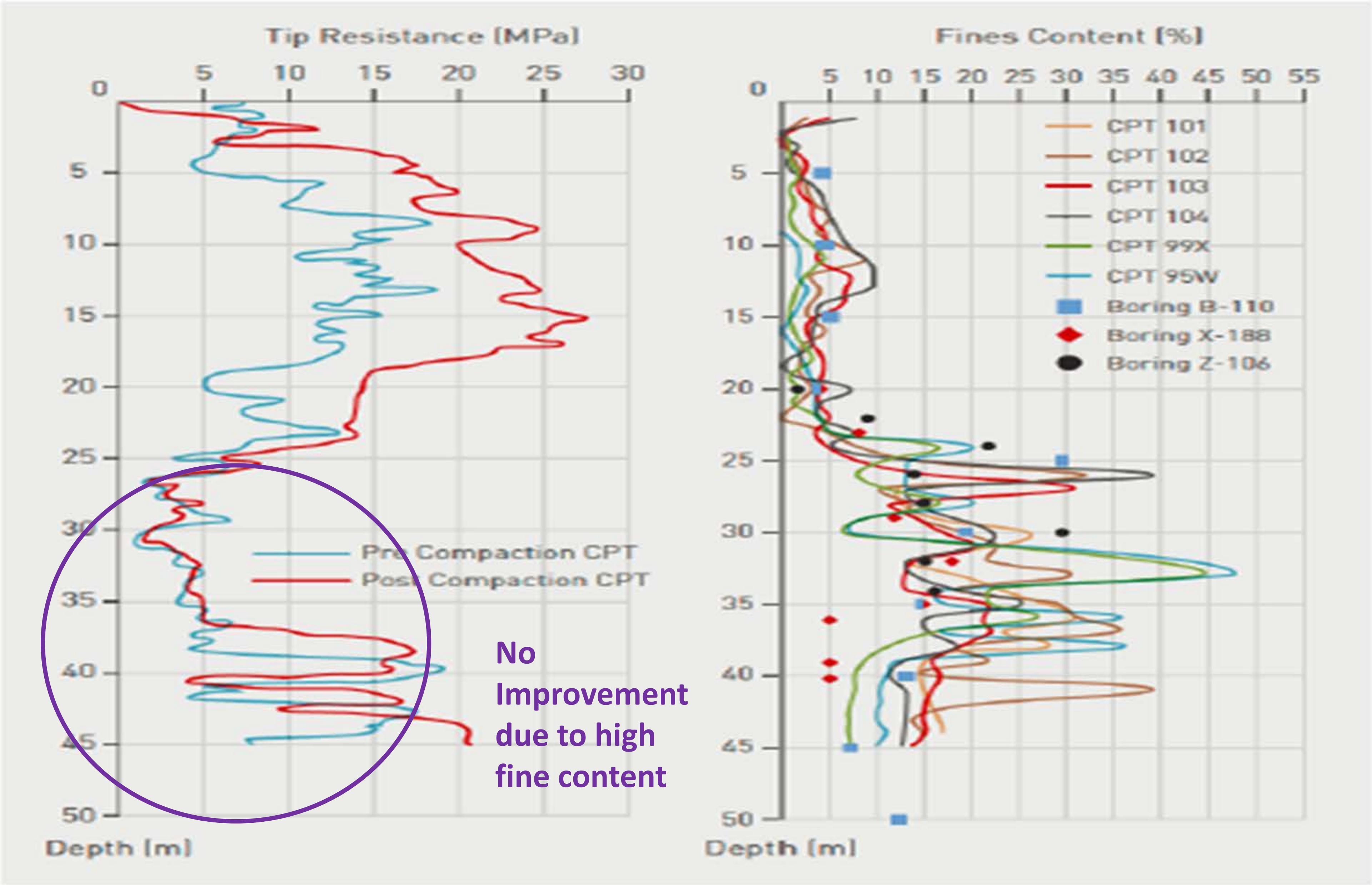
From Vibroflotation (Adopted by Bachy Soletanche Group in Hong Kong in 2005)

Fine Content: FC < 10%

Friction Ratio: Rf < 0.8 %



Typical Results of Pre and Post CPT of Vibro Compaction with Fines Content Assessment



Relative Density Use in Deep Compaction as Acceptance Criterion

$$D_r = \frac{e_{max} - e}{e_{max} - e_{min}} \times 100\%$$

$$= \frac{\gamma_{d\,max}}{\gamma_d} \times \frac{\gamma_d - \gamma_{d\,min}}{\gamma_{d\,max} - \gamma_{d\,min}} \times 100\% \qquad (3.1)$$

where

e_{min} = void ratio of soil in densest condition
e_{max} = void ratio of soil in loosest condition
e = in-place void ratio
$\gamma_{d\,max}$ = dry unit weight of soil in densest condition
$\gamma_{d\,min}$ = dry unit weight of soil in loosest condition
γ_d = in-place dry unit weight

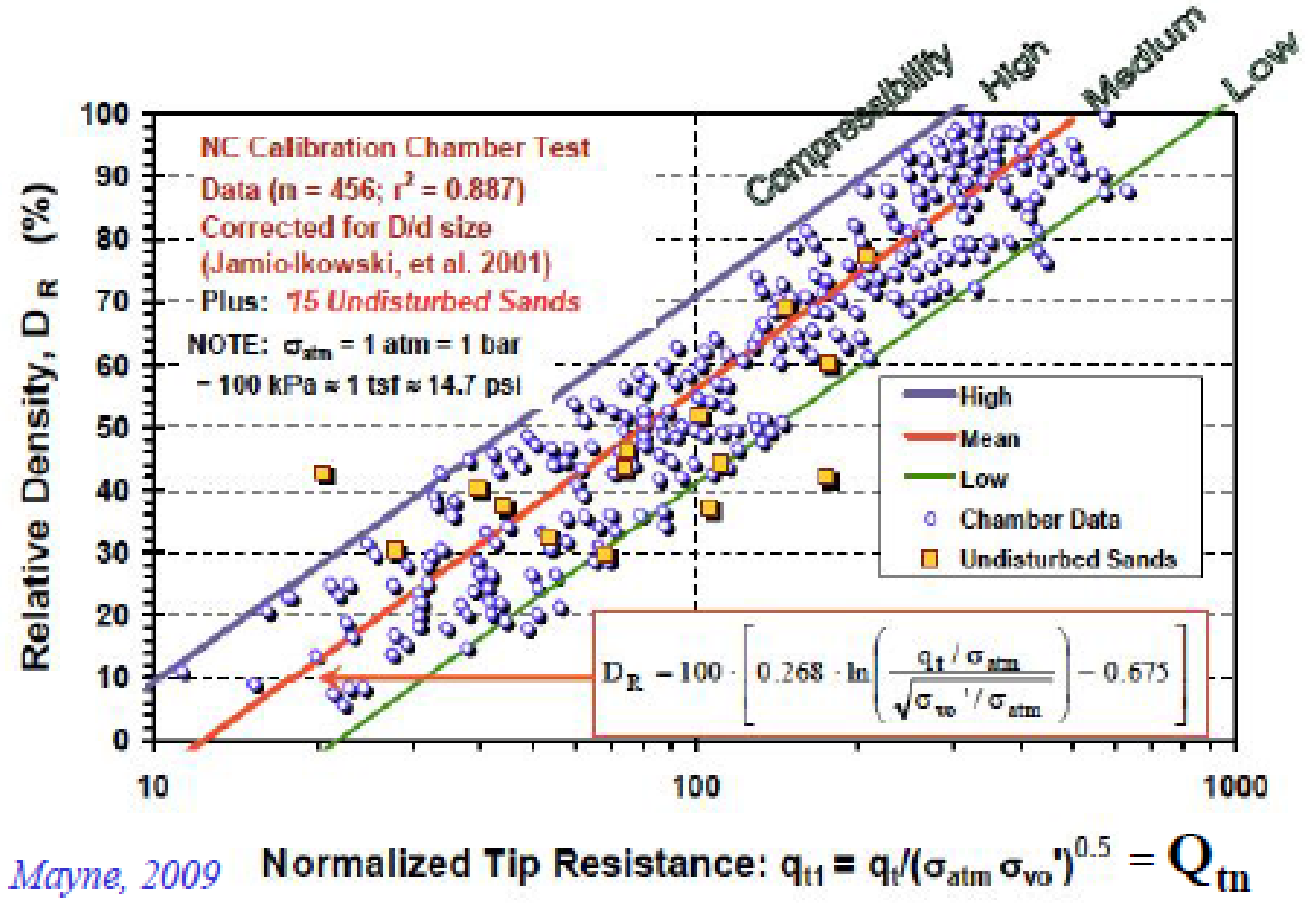
Relative Density (%)	Descriptive Term
0–15	Very loose
15–35	Loose
35–65	Medium
65–85	Dense
85–100	Very dense

Note:

- Sand compressibility is controlled by grain size, shape and mineralogy.
- The emin and emax are difficult to determine.
- Most relationship between Dr and CPT are based on calibration chamber (CC)test for clean sand.
- Research has shown that the stress strain and strength behaviors are too complicated to be represented only Dr. However, most of the professionals still use it as it has been adopting for long time, and it is simple to use.
- Angular sand is more compressible than round sand.
- Carbonate or high mica sand is more compressible than quartz sand.

Many of the correlation of the developed by CPT are based on the results of the laboratory conducted in calibration chamber with uniformly graded sands (Clean sand).

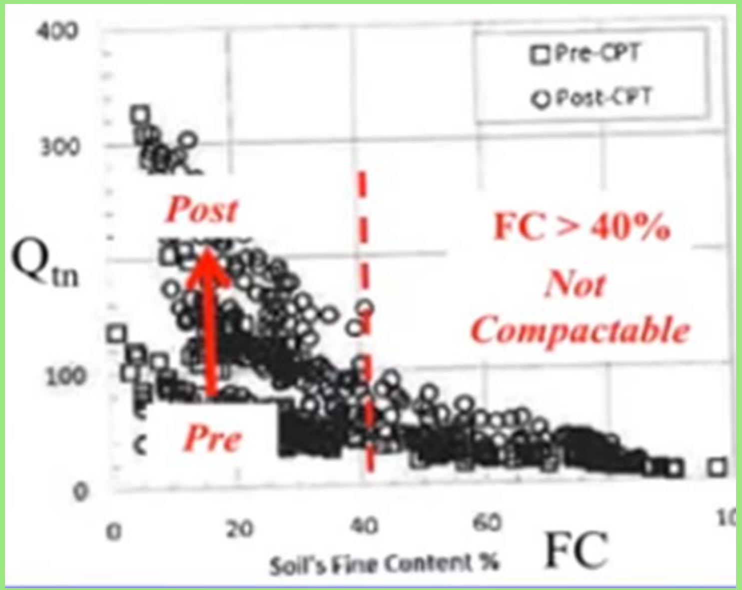
Since natural sand deposits are uniform, they may contain fines and varying degrees of aging. Therefore, this correlation should be considered to be approximate.

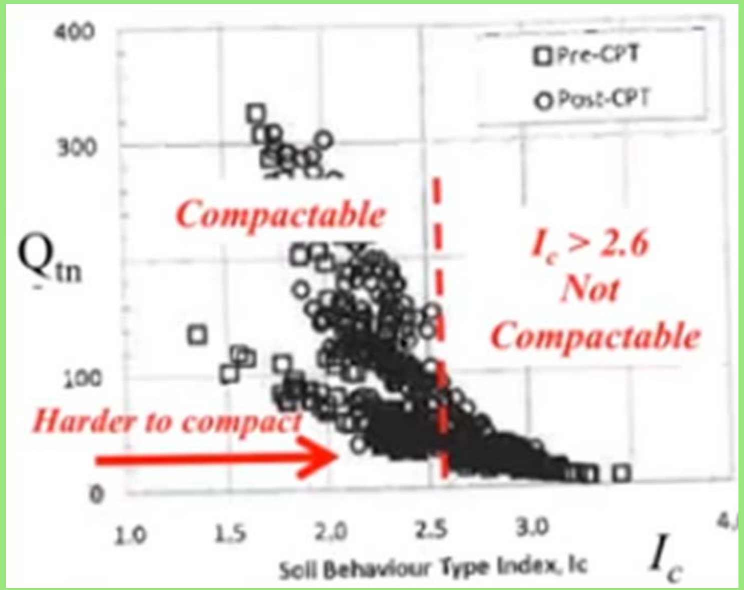


The Relative Density Equation is derived by Jamiolkowski et al (2001)

Compactability Related to Ic, FC and Fr

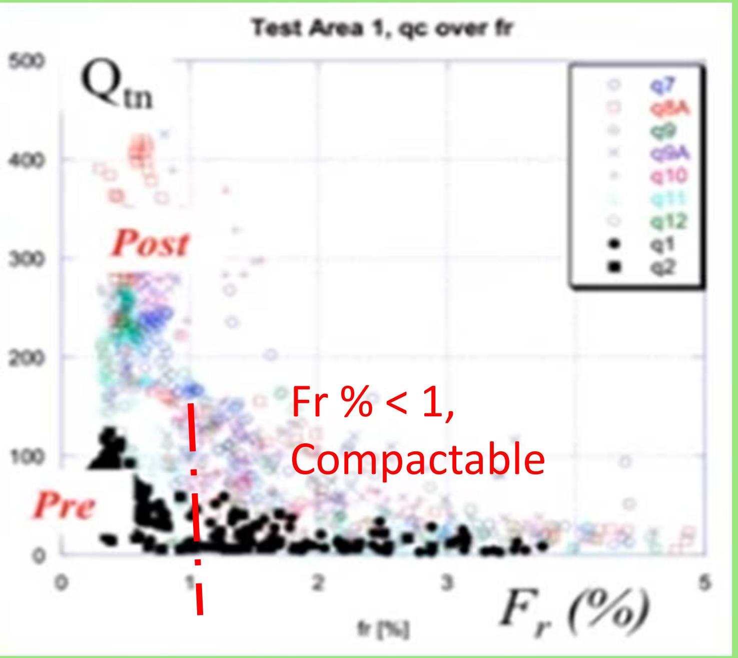
Graphs from
Kirsch and Kirsch (2010)

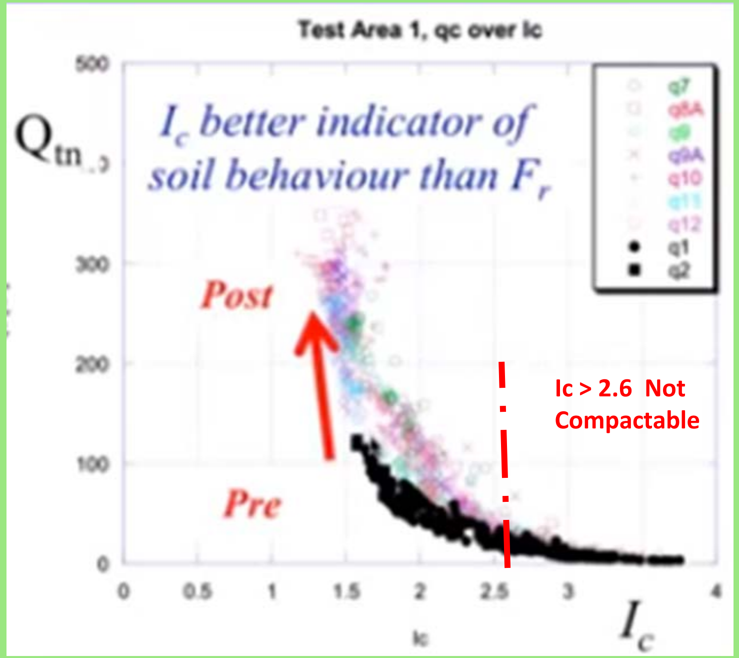




Graphs From Degon 2005

Sandy soils with fine content (> around 40%) and high CPT Ic (Ic > 2.6) are generally not or less compactable.





President Harbour in Gdynia Port

Sand fill and aged Holocence Sand with silt and mud inclusion.

Water table is 1m below the ground level.

Some parts of the superficial layers were hydraulic fill.

A dense Pleistocene sand with mud inclusion

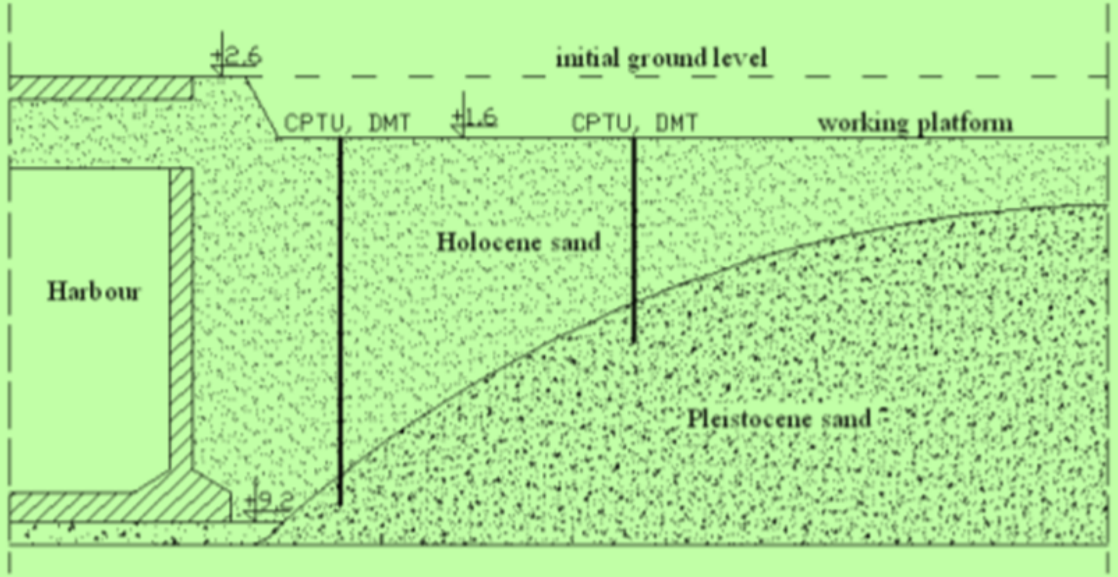


Fig. 1. Simplified cross section (not to scale), Bałachowski and Kozak [9]

President Harbour in Gdynia Port

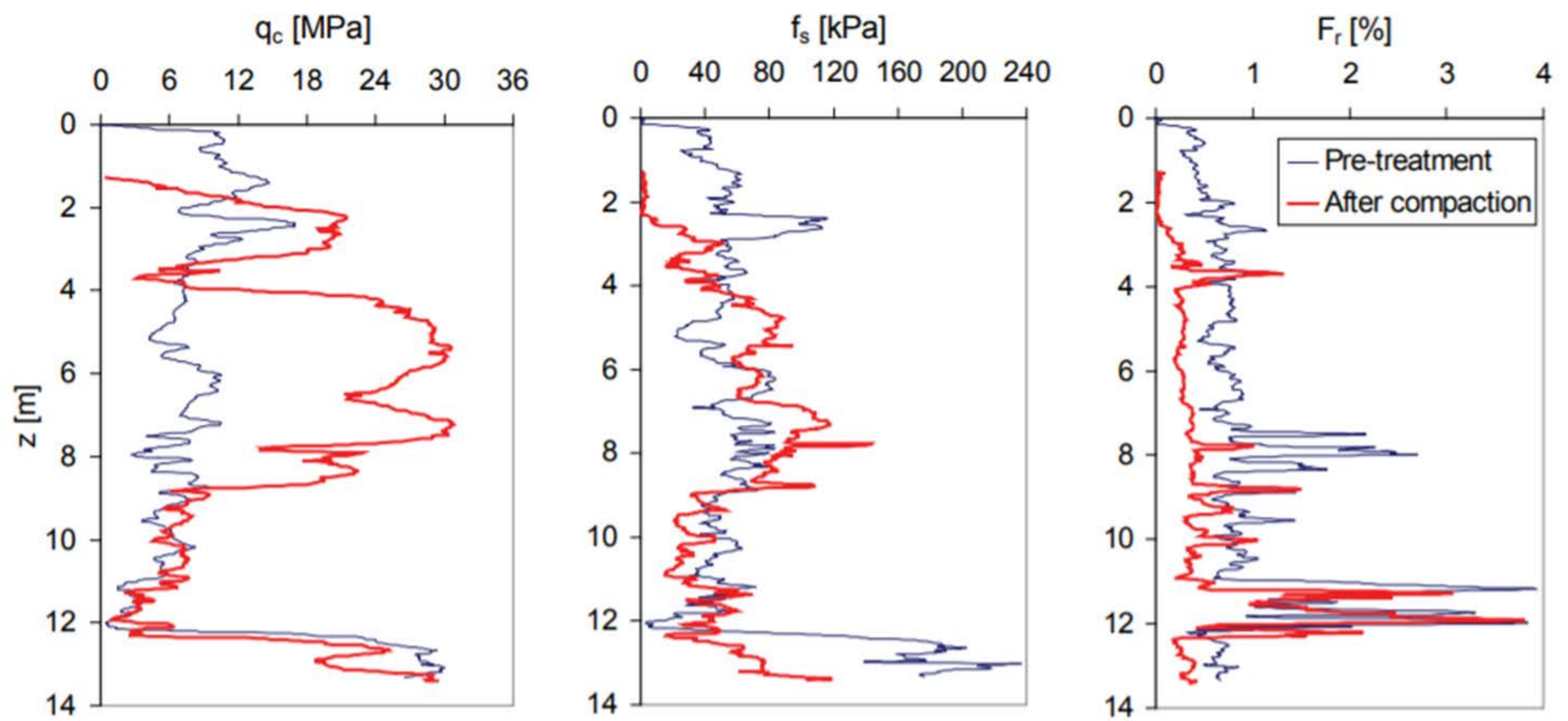
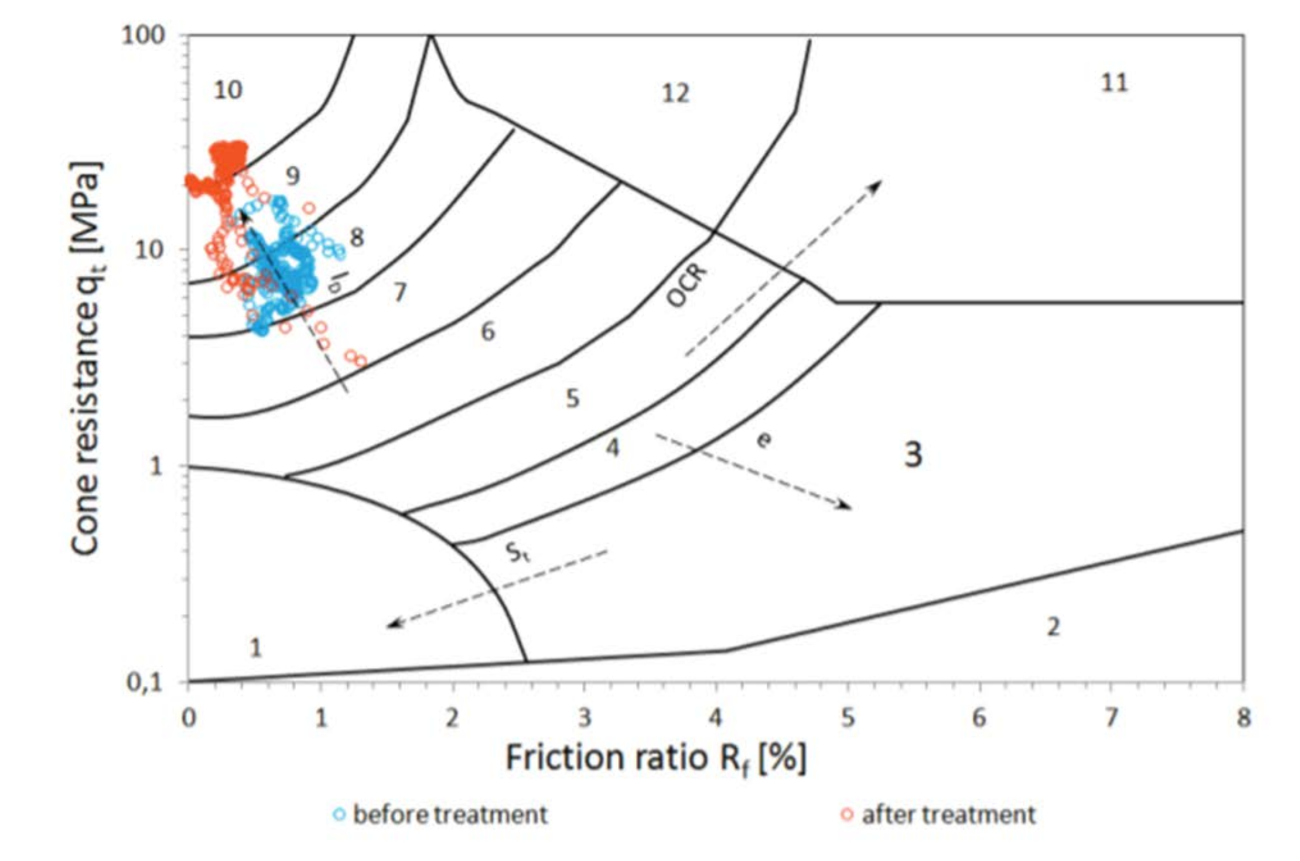


Fig. 2. Comparison of pre- and post-treatment CPTU results, Bałachowski and Kozak [9]

Analysis of soil type behavior using the classification charts and soil type behavior index Ic provides better, more comprehensive and normalized approach to the soil improvement.

Overall improvement factor based on Ic regardless of the soil nature and depth.

Despite it is shown in the SBT that the soil properties are changed from silty sand and sand mixtures to sands and gravelly sand. **the soil granulometry remains the same in deed after the deep compaction.**



Soil classification chart before and after treatment

The curved normalized cone resistance versus Ic was shifted to the right
The improvement ratio proves the increase in normalized cone resistance after the compaction.

Values of the improvement factor decrease with soil behavior type index, i.e. with fine content.

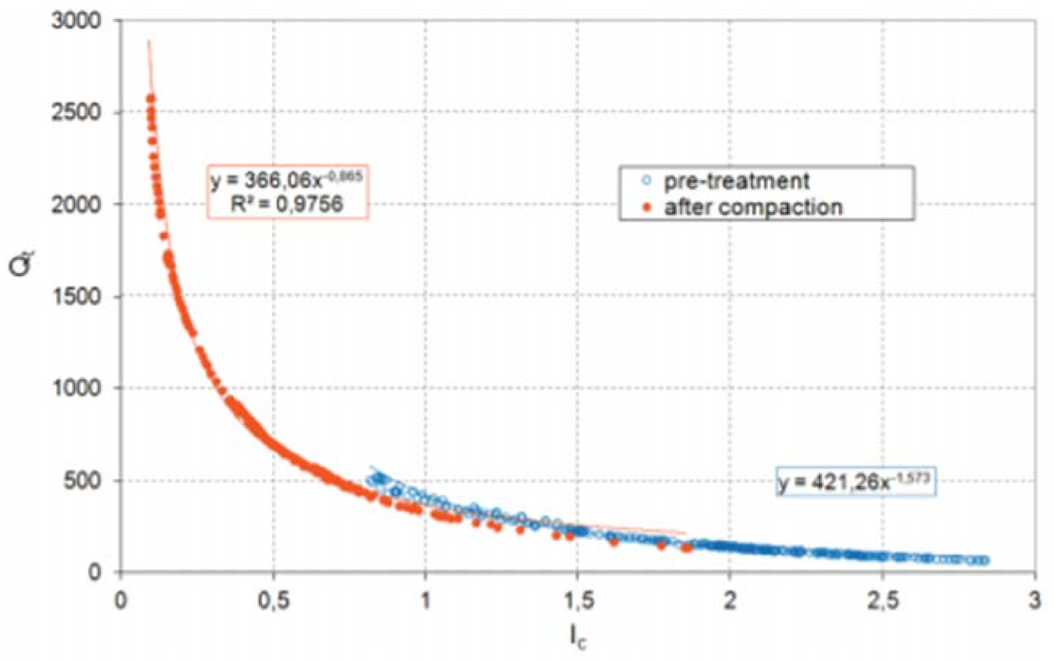


Fig. 6. Normalised cone resistance vs. soil behaviour type index

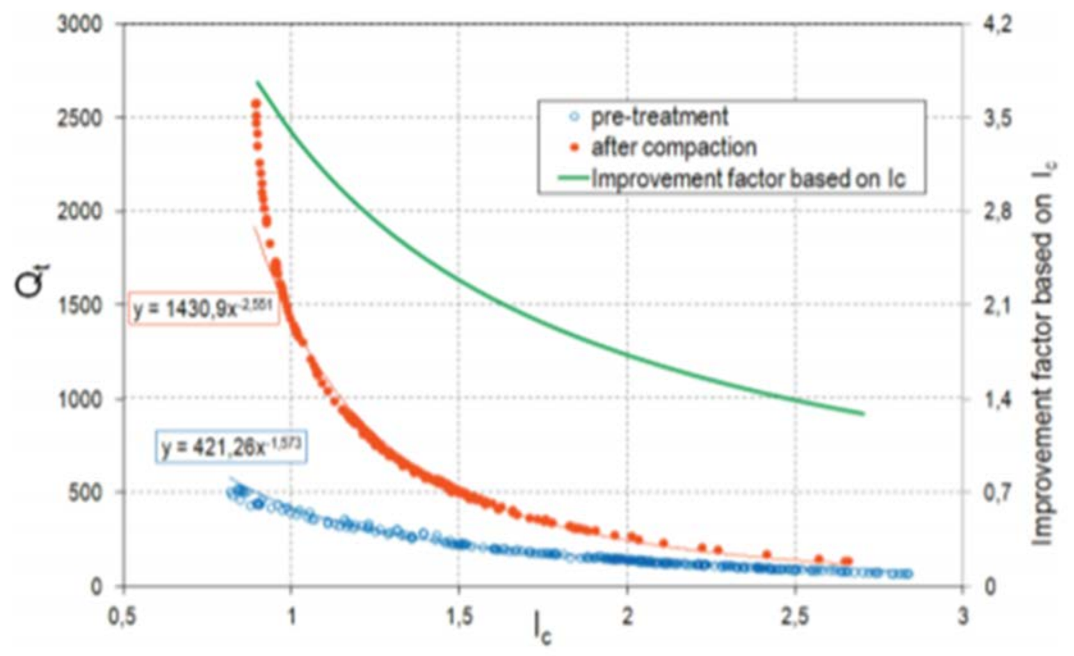


Fig. 7. Shifted normalised cone resistance and improvement factor based on soil behaviour type

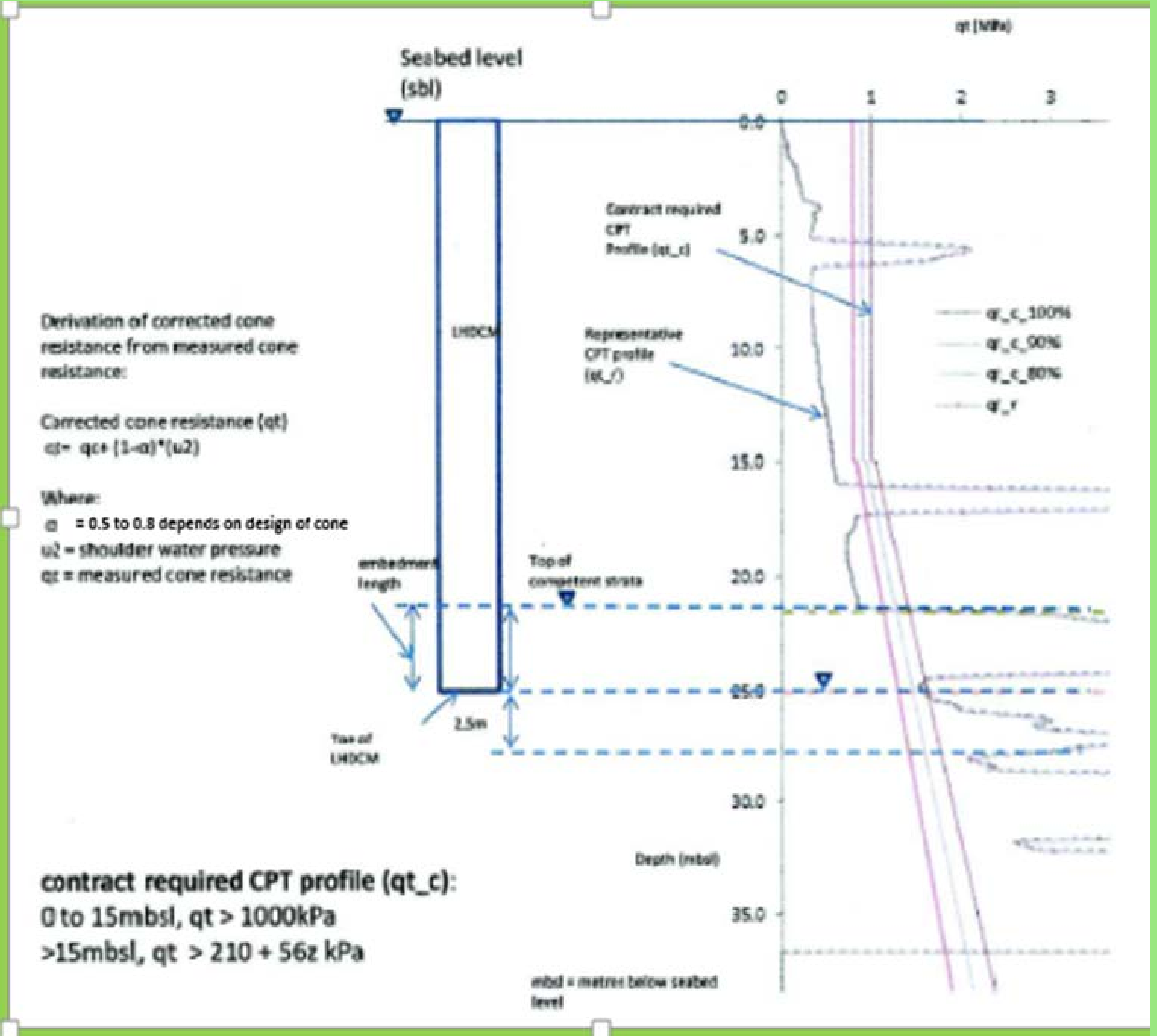
Application in Deep Cement Mixing

Application in Deep Cement Mixing in Third Runway

it is specified in the contract required CPT profile (q_c values), as shown as the red line profile at the figure, should be based on the following requirements:

1. For depth of the CPT shallower than 15m below seabed level (msbl), the corrected q_t should be greater than 1,000 KPa.
2. For depth of greater than 15mbl, the corrected q_t valve should be greater than 210+56z KPa, where z is the depth below the seabed level.

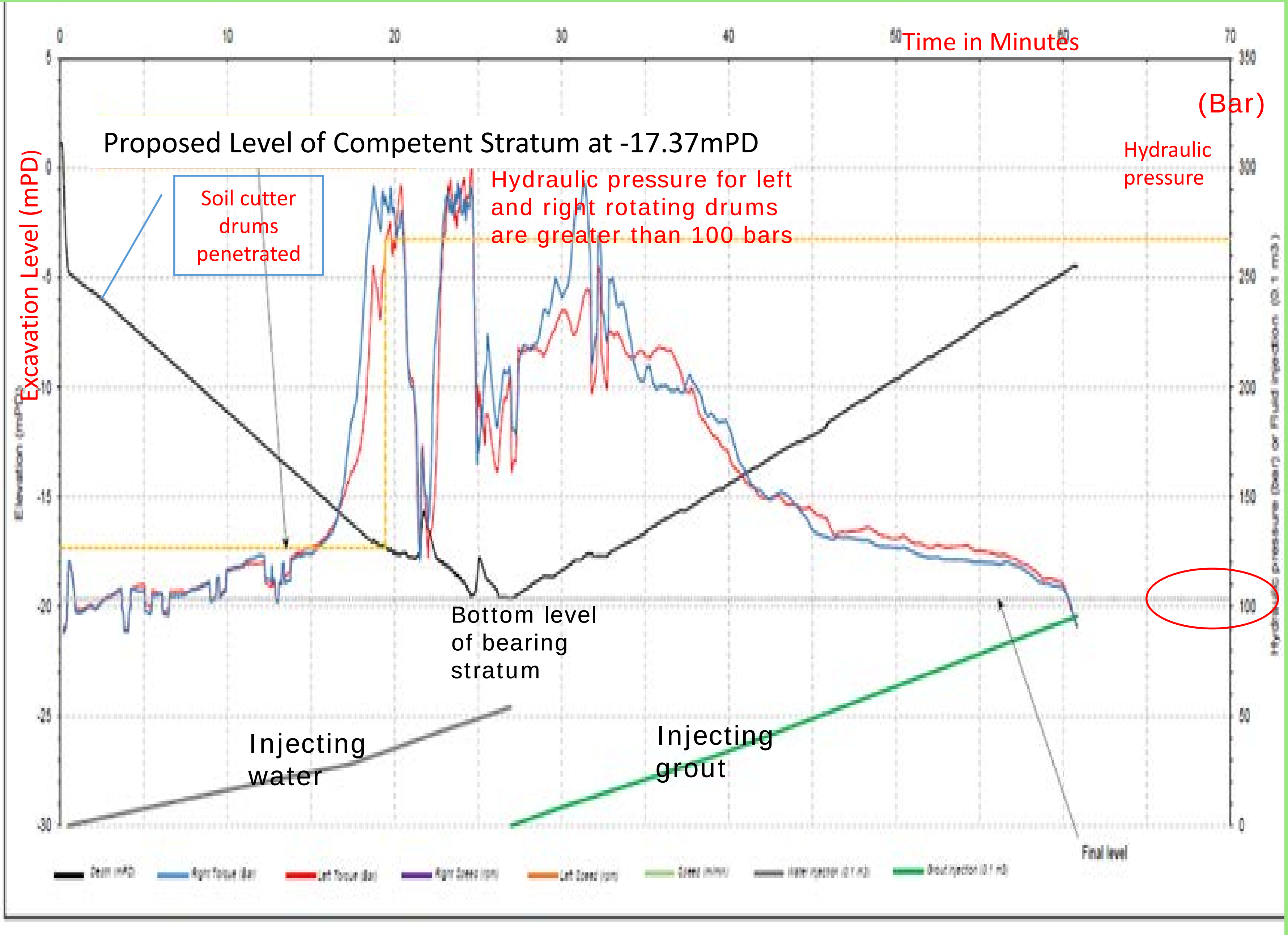
The raw data q_c values are transformed into q_t values, and then calculated with filtering, shortening, and smoothing methods to get the q_t_r values (Representative CPT profile). The trail for the potential top level of the competent stratum should be checked such that q_t_r values should be greater than 90% of the q_t_c vales and the q_t_r values should be greater than 80% of q_t_c along 2m below the potential top level. After that, the data should be further adjusted and assessed with several procedures specified in the Appendix of the contract (C3205) to determine the termination level of the DCM panel required.



Termination Depth Related with Soil Cutting Drum Pressure Calibrated with qc of CPT

The qc has been calibrated with torques and pressures at the left and right rotating soil cutting drums during trial with CPT penetration.

During deep mixing, the rotating drums are sustained torque from soil (i.e. pressure at the hydraulic pressure sensors inside the soil cutter drums). Once the pressures in the drums reach 100 bars, it is equivalent to the qc of 1MPa, and therefore, the penetration for the drums can be terminated.



Some Challenges for Adaptation of CPT in Hong Kong

1. Ground oftens too hard, and it contains corestones and boulders.
2. Excessive inclination during penetration.
3. Reach refusal in penetration.
4. CPT needs more experience and data analysis with too much expertise.
5. Not as common and simple as SPT in terms of cost and acceptance for adopt by engineers.
6. Technicians are generally not well trained for operation.
7. Equipment is comparatively expensive and need good maintenance.
8. Professionals and designers are limited to use CPT data in correlation with geotechnical parameters but seldom to use for direct approach in design like foundation etc.
9. Lack of systematic research and statistical data for adoption in design purpose in Hong Kong.
10. It is suggested that GEO could be a leading Governmental Department to provide guidebook as they did in publication of the foundation guidebook.

END

Thank You!