USE OF CPT/CPTU FOR SULUTION OF PRACTICAL PROBLEMS

Indirect design method:

- Interprete CPT/CPTU results to arrive at soil design parameters
- Classical foundation analysis

Direct design method:

- Use CPT/CPTU results directly without intermediate step of soil parameters

DIRECT APPLICATIONS OF CPT/CPTU RESULTS

- **Correlations to SPT (standard penetration tests)**
- **Axial capacity of piles**
- Bearing capacity and settlement of shallow foundations
- **Ground improvement - quality control**
- *Liquefaction potential evaluation*

CPT/SPT CORRELATIONS

Depends on several factors:

- Energy level delivered to SPT - use N_{60}
- Grain size distribution (D_{50})
- Fines content (FC)
- Overburden stress + other factors

Comment:

Single most important factor influencing N value is energy delivered to SPT sampler, expressed as rod energy ratio. Energy ratio of 60% is generally accepted to represent average SPT energy. Results should be corrected to N_{60}.

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CPT/SPT CORRELATIONS

Depends on several factors:

- Energy level delivered to SPT - use N_{60}
- Grain size distribution (D_{50})
- Fines content (FC)
- Overburden stress + other factors

Correlations most used:

Robertson et al. 1983
Kulhawy and Mayne, 1990

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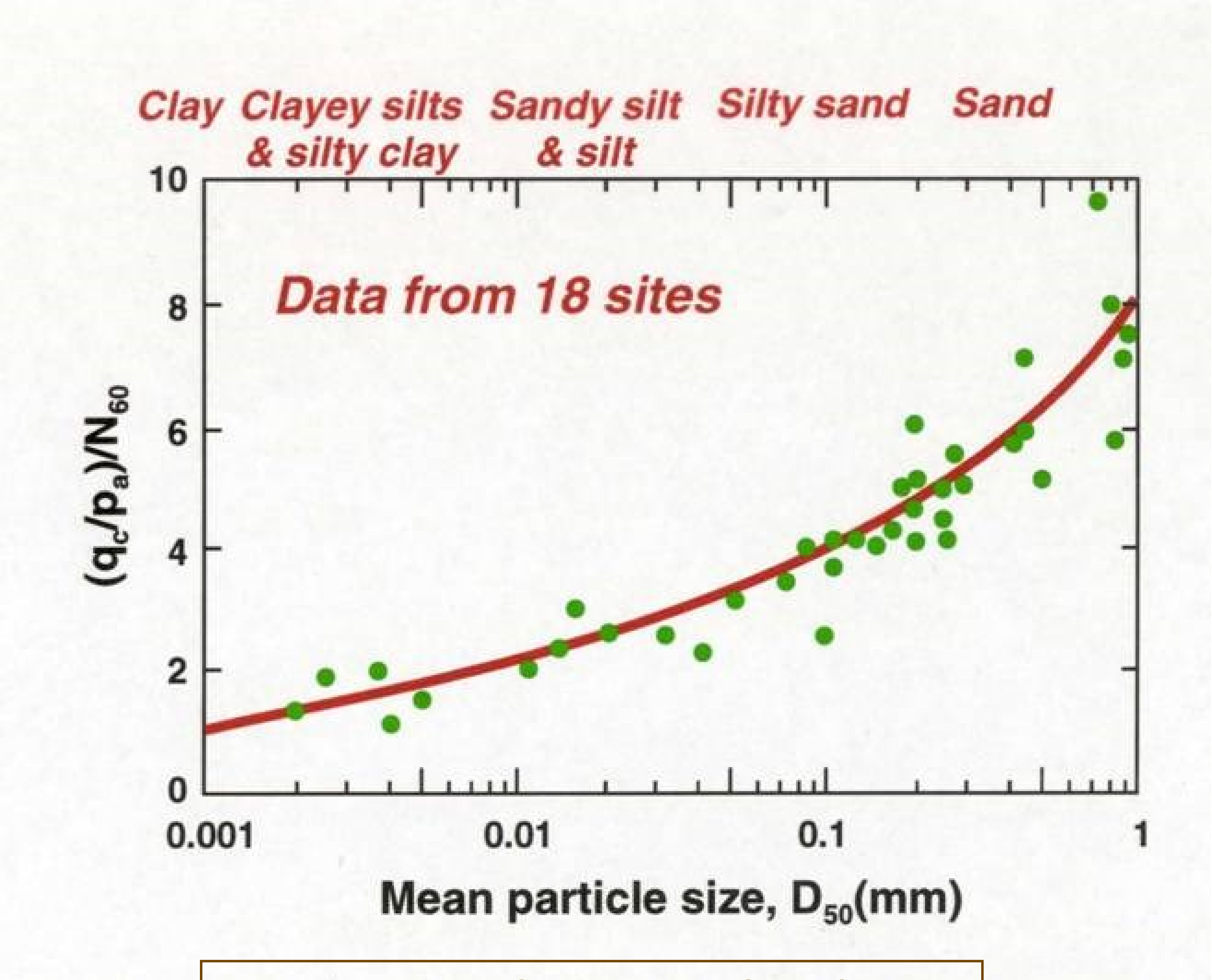
CPT/SPT CORRELATIONS



Robertson and Campanella (1983)

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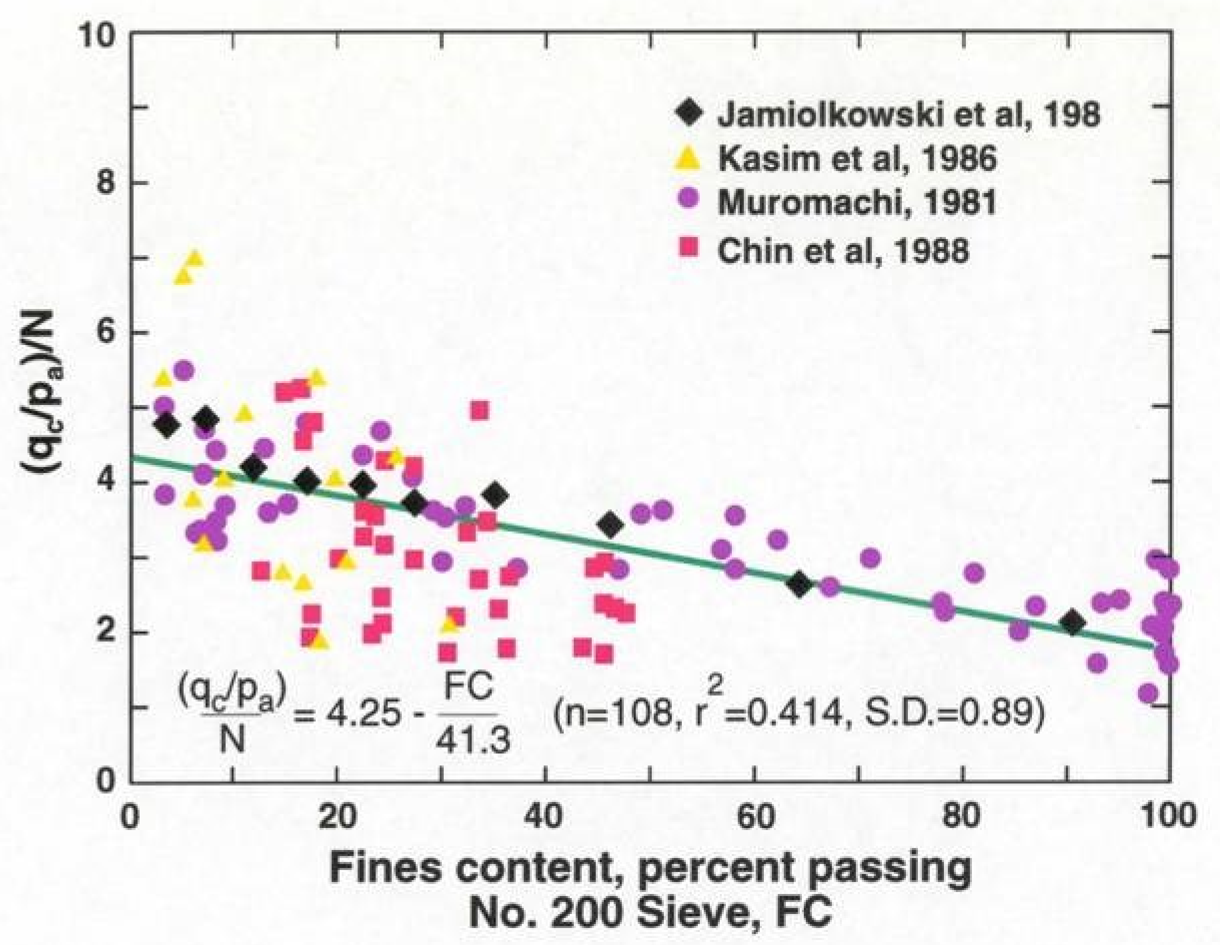
Pa = reference stress = 1 atm = 100 kPa

CPT/SPT CORRELATIONS
Effects of fines content



Mayne and Kulhavy (1990)

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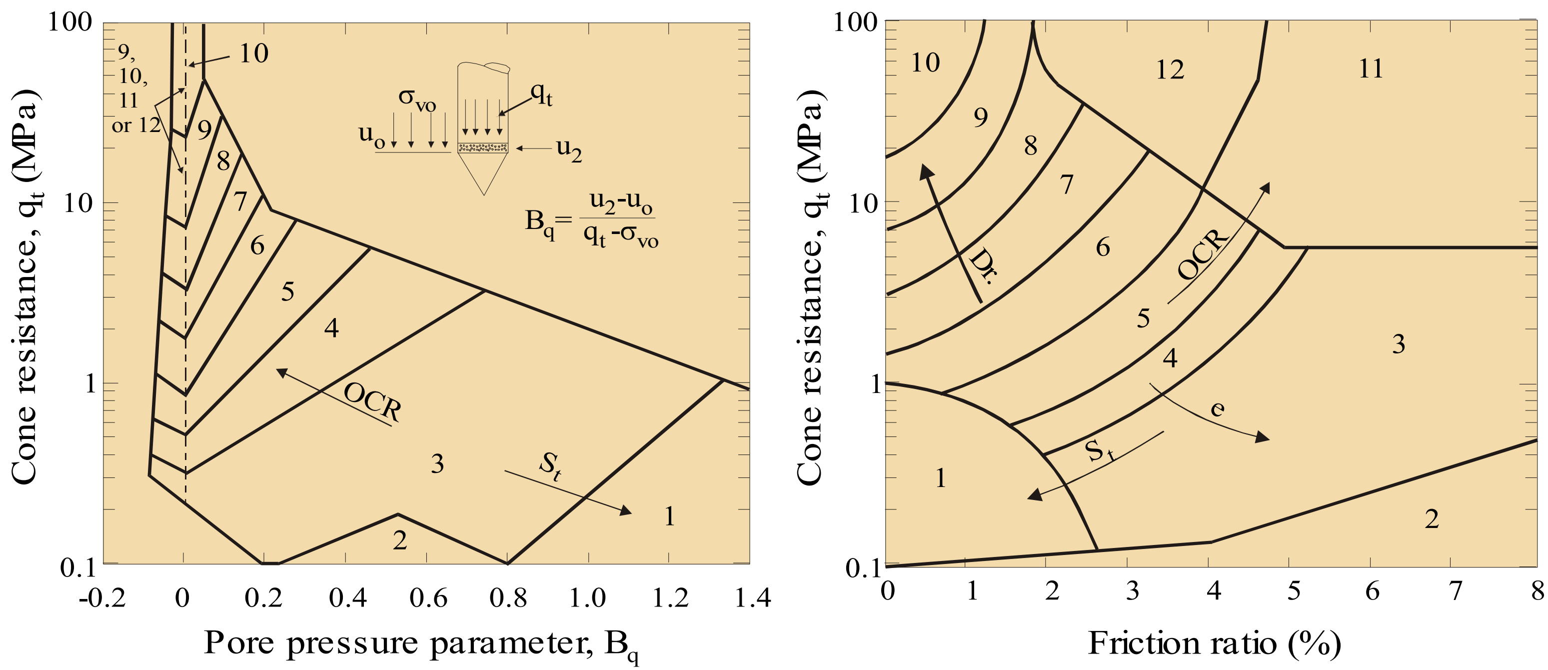
If no grain size data available- use Soil behaviour classification chart



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*

** Overconsolidated or cemented.*

Soil Behaviour Chart (Robertson et al, 1986)

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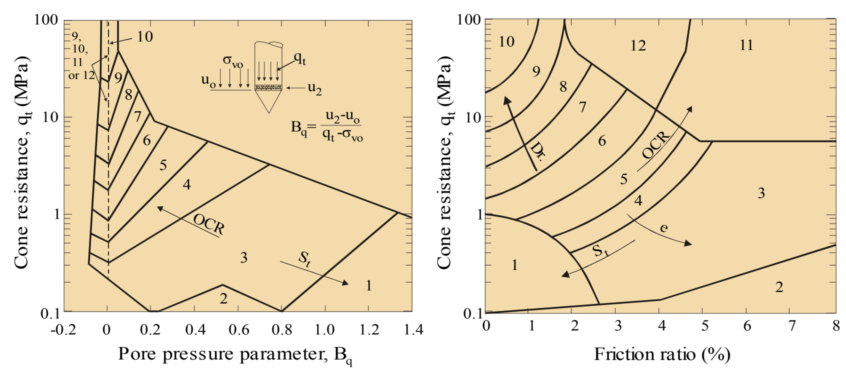
Robertson et al.,1986

SOIL CLASSIFICATIONS AND RATIOS

Zone	Soil behavior type	$(q_c/p_a)/N_{60}$
1	Sensitive fine grained	2
2	Organic material	1
3	clay	1
4	Silty clay to clay	1.5
5	clayey silt to silty clay	2
6	Sandy silt to clayey silt	2.5
7	Silty sand to sandy silt	3
8	Sand to silty sand	4
9	sand	5
10	Gravely sand to sand	6
11	Very stiff fine grained	1
12	Sand to clayey sand	2

Zone refers to Soil Behaviour type diagram



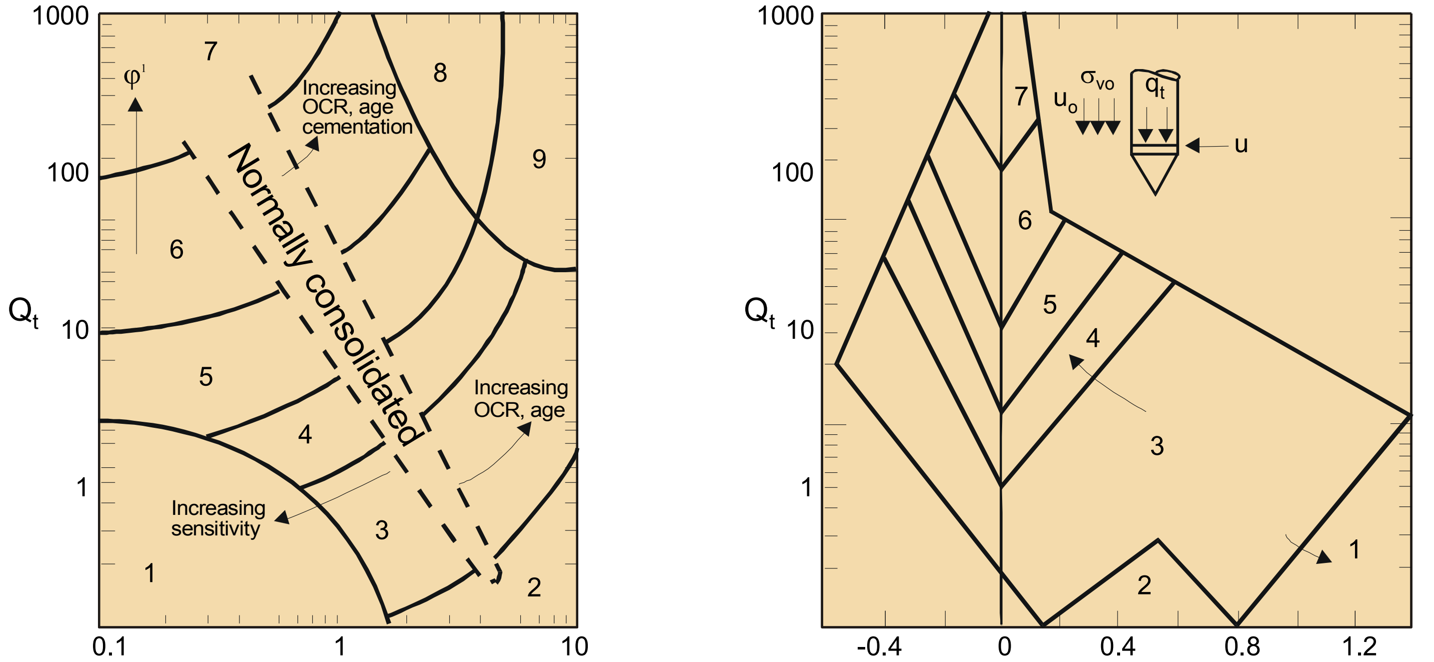
Zone: Soil Behaviour Type:

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*

** Overconsolidated or cemented.*

Soil Behaviour Chart (Robertson et al, 1986)

Normalized soil behaviour classification chart



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

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

Robertson,1990

CPT/SPT CORRELATIONS

In lack of soil grain size data, use Robertson (1990) soil classification chart to define soil behaviour type index:

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

$$Qt = \frac{q_t - \sigma_{v0}}{\sigma_{v0}'}, \quad Fr = \frac{f_s}{\sigma_{v0}'}$$

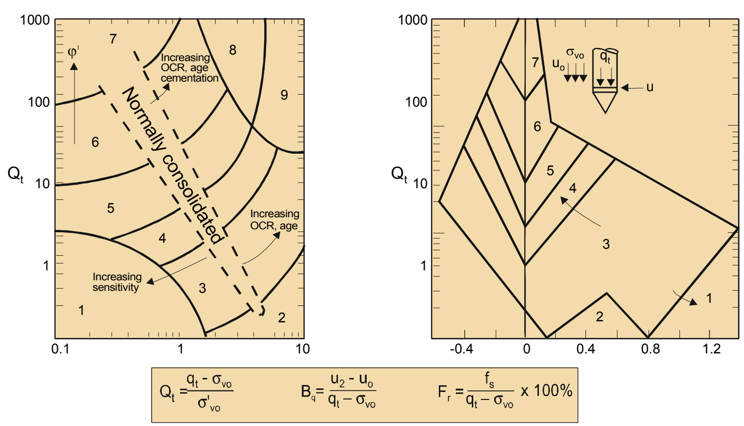
$$(q_c / p_a) / N_{60} = 8.5(1 - I_c / 4.6)$$

p_a = atm. Press. = 100 kPa

N_{60}: SPT value corresponding to energy ratio of 60%

BOUNDARIES OF SOIL BEHAVIOUR TYPE

Soil behaviour type Index I_c	Zone	Soil behaviour type
$I_c < 1.31$	7	Gravilly sand
$1.31 < I_c < 1.205$	6	Sands – clean sand to silty sand
$2.05 < I_c < 2.60$	5	Sand mixturees – silty sands to sandy silts
$2.60 < I_c < 2.95$	4	Silt mixtures – clayey silts to silty clay
$2.95 < I_c < 3.60$	3	Clays
$I_c < 3.06$	2	Organic soils - peat



Zone	Soil behaviour type
1.	Sensitive, fine grained
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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

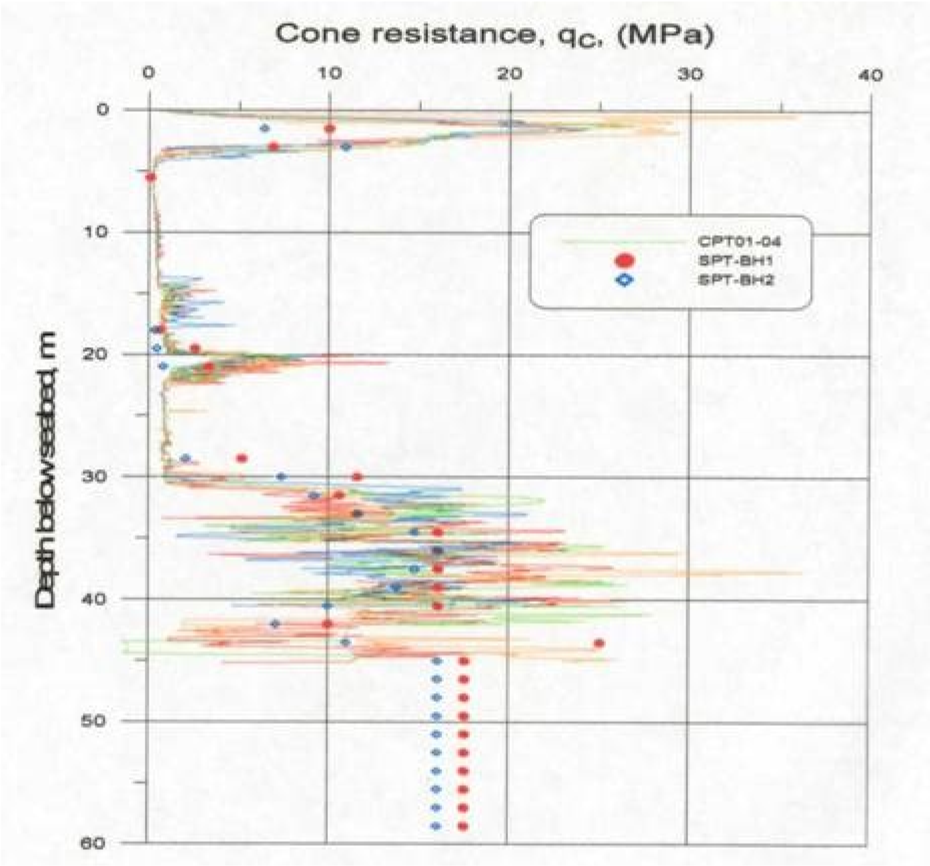
$$I_c = \left(\left(3.47 - \log Q_t\right)^2 + \left(\log F_r + 1.22\right)^2 \right)^{0.5}$$

Example CPT/SPT Correlations

Westport Warehose facility outside Kuala Lumpur

Soil investigation by Soils and Foundations Sdn.Bhd

A lot of old investigations with SPT



CPT/SPT correlations

- **If grain size distribution data are available**
  - *Use $(q_c/p_a)/N_{60}$ from Robertson et al.,1983 (Fig.6.1)(D_{50})*
  - *and/or $(q_c/p_a)/N$ from Fig. 6.3 (Fines content)*

- **If grain size distribution data are <u>not</u> available**
  - *Use soil behaviour index , I_C (= $f(Q_t,F_r)$)*

  $$(q_c/p_a)/N_{60} = 8.5(1 - I_C/4.6)$$

PILE BEARING CAPACITY

Several studies

- **Robertson et al., 1988; 8 cases**
- **Briaud, 1988; 78 pile load tests**
- **Tand and Funegård, 1989; 13 cases**
- **Sharp et al.,1988; 28 cases**
- **NGI, 1998**

All show CPT methods better than other methods

AXIAL PILE CAPACITY

$Q_{ult} = f_p A_s + q_p A_p$ (side friction plus tip resistance)

Bustamante and Gianeselli (1982)

$$f_p = q_c/\alpha$$

$$q_p = k_c \bullet q_{ca}$$

α and k_c empirical constants for different pile and soil types

Based on a very large number of case histories (197) in France tables have been made with α and k_c factors according to soil type and to type of pile

BEARING CAPACITY FACTORS, k_c

(BUSTAMANTE AND GIANESELLI, 1982)

Nature of soil	q_c (Mpa)	Factors k_c Group I	Group II
Soft clay and mud	< 1	0.4	0.5
Moderately compact clay	1 to 5	0.35	0.45
Silt and loose sand	≤ 5	0.4	0.5
Compact to stiff clay and compact silt	>5	0.45	0.55
Soft chalc	≤ 5	0.2	0.3
Moderately compact sand and gravel	5 to 12	0.4	0.5
Weathered to fragmented chalk	> 5	0.2	0.4
Compact to very compact sand and gravel	> 12	0.3	0.4

$$q_p = k_c \bullet q_{ca}$$

Group I: plain bored piles; mud bored piles; micro piles (grouted under low pressure); cased bored piles; hollow auger bored piles; piers; barrettes.

Group II: cast screwed piles; driven precast piles; prestressed tubular piles; driven cast piles; jacked metal piles; micropiles (small diameter piles grouted under high pressure with diameter < 250 med mer); driven grouted piles (low pressure grouting); driven metal piles; driven rammed piles; jacket concrete piles; high pressure grouted piles of large diameter.

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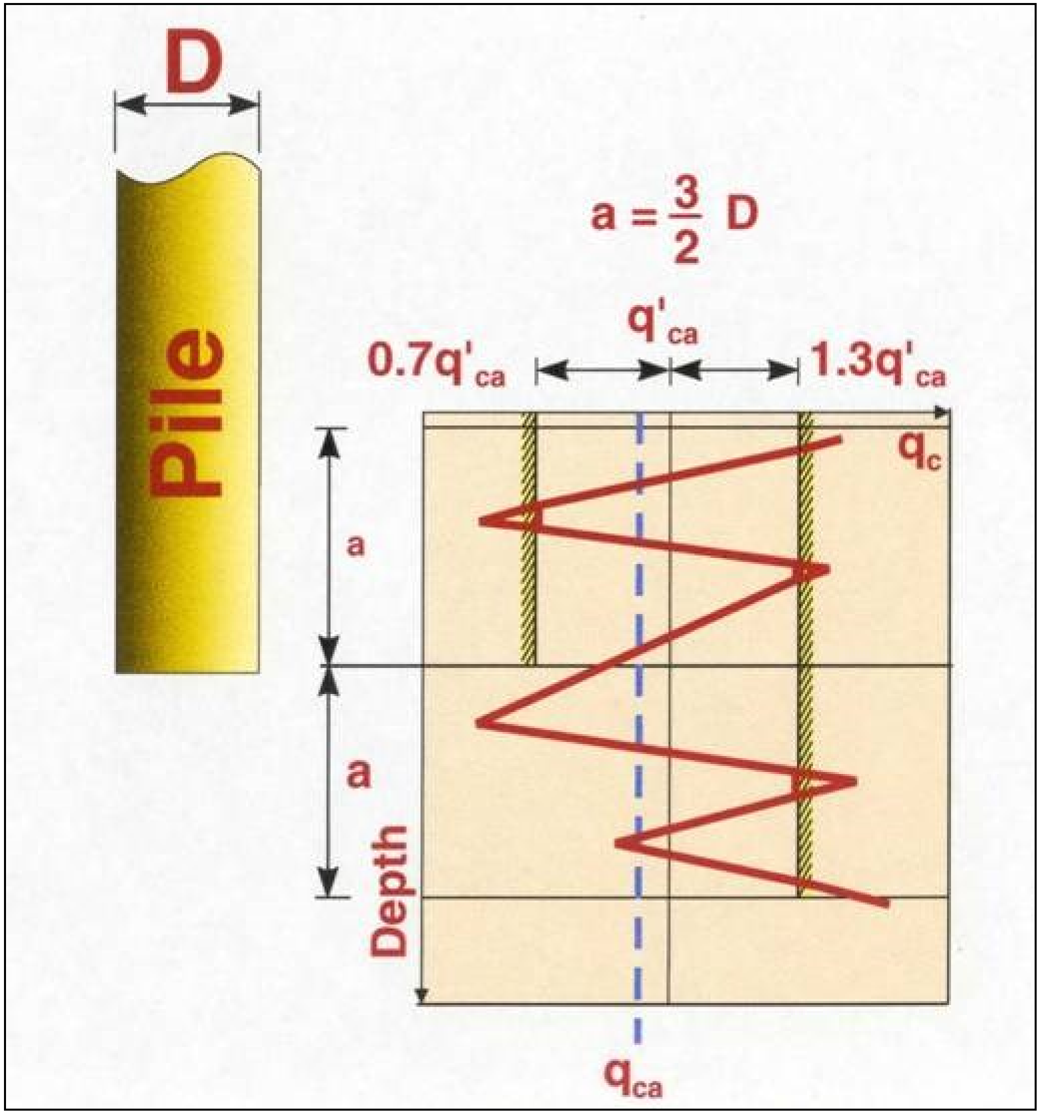
Computation of q_c for tip resistance



Pile end bearing is dependant on soil above and below pile tip. Need to evaluate average q_c to represent this influence area.

Bustamante and Gianesseli(1982)

FRICTION COEFFICIENT, α

(BUSTAMANTE AND GIANESELLI, 1982)

Nature of soil	q_c (Mpa)	Category Coefficients, α			
		I		II	
		A	B	A	B
Soft clay and mud	< 1	30	90	90	30
Moderately compact clay	1 to 5	40	80	40	80
Silt and loose sand	≤ 5	60	150	60	120
Compact to stiff clay and compact clay	> 5	60	120	60	120
Soft chalk	≤ 5	100	120	100	120
Moderately compact sand and gravel	5 to 12	100	200	100	200
Weathered to fragmented chalk	> 5	60	80	60	80
Compact to very compact sand and gravel	< 12	150	300	150	200

$$f_p = q_c/\alpha$$

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FRICTION COEFFICIENT, α
(BUSTAMANTE AND GIANESELLI, 1982) Ctd.

Nature of soil	q_c (Mpa)	Category					
		Maximum limit of f_p (Mpa)					
		I		II		III	
		A	B	A	B	A	B
Soft clay and mud	< 1	0.015	0.015	0.015	0.015	0.035	
Moderately compact clay	1 to 5	0.035 (0.08)	0.35 (0.08)	0.035 (0.08)	0.035	0.08	0.12 ≤
Silt and loose sand	≤ 5	0.035	0.035	0.035	0.035	0.08	-
Compact to stiff clay and compact clay	> 5	0.035 (0.08)	0.035 (0.08)	0.035 (0.08)	0.035	0.08	0.20 ≤
Soft chalk	≤ 5	0.035	0.035	0.035	0.035	0.08	-
Moderately compact sand and gravel	5 to 12	0.08 (0.12)	0.035 (0.08)	0.035 (0.12)	0.08	0.12	0.20 ≤

$$f_p = q_c/\alpha$$

NGI

FRICTION COEFFICIENT, α
(BUSTAMANTE AND GIANESCELLI, 1982) Ctd.

Nature of soil	q_c (Mpa)	Category					
		Maximum limit of f_p (Mpa)					
		I		II		III	
		A	B	A	B	A	B
Weathered to fragment chalk	> 5	0.12 (0.15)	0.08 (0.12)	0.12 (0.15)	0.12	0.15	0.20 ≤
Compact to very compact sand and gravel	> 12	0.12 (0.15)	0.08 (0.12)	0.12 (0.15)	0.12	0.15	0.20 ≤

Category: IA: plain bored piles; hollow auger bored piles; micropiles (grouted under low pressure); cast screwed piles; piers; barrettes. IB: cased bored piles; driven cast piles. IIA: driven precast piles; prestressed tubular piles; jacket concrete piles. IIB: driven metal piles; jacked metal piles. IIIA: driven grouted piles; driven rammed piles. IIIB: high pressure grouted piles of large diameter > 250 mm; micropiles (grouted under high pressure).

Note: Maximum limit unit skin friction, f_p: bracket values apply careful execution and minimum disturbance of soil due to construction.

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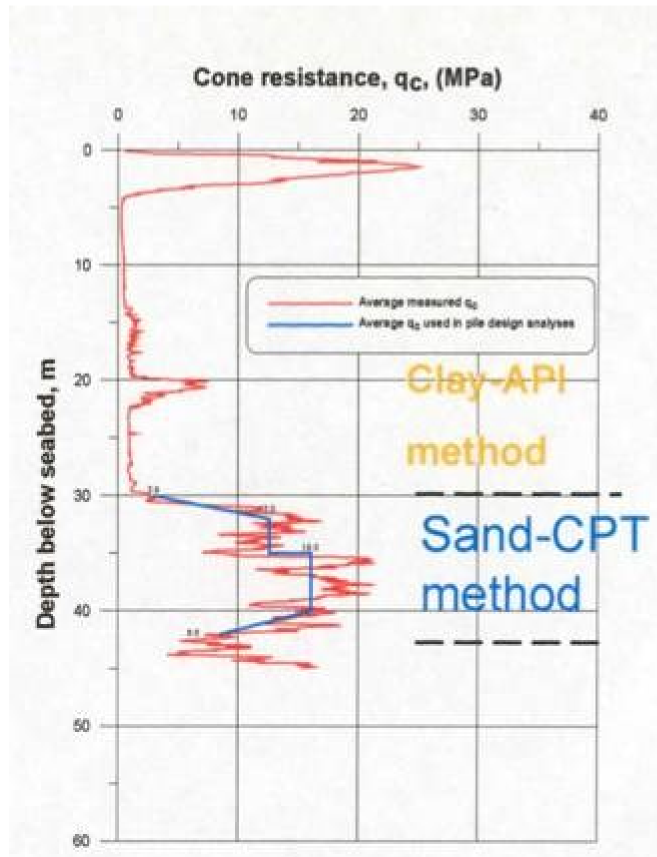
Pile Capacity from CPT

Example from Westport, Kuala Lumpur

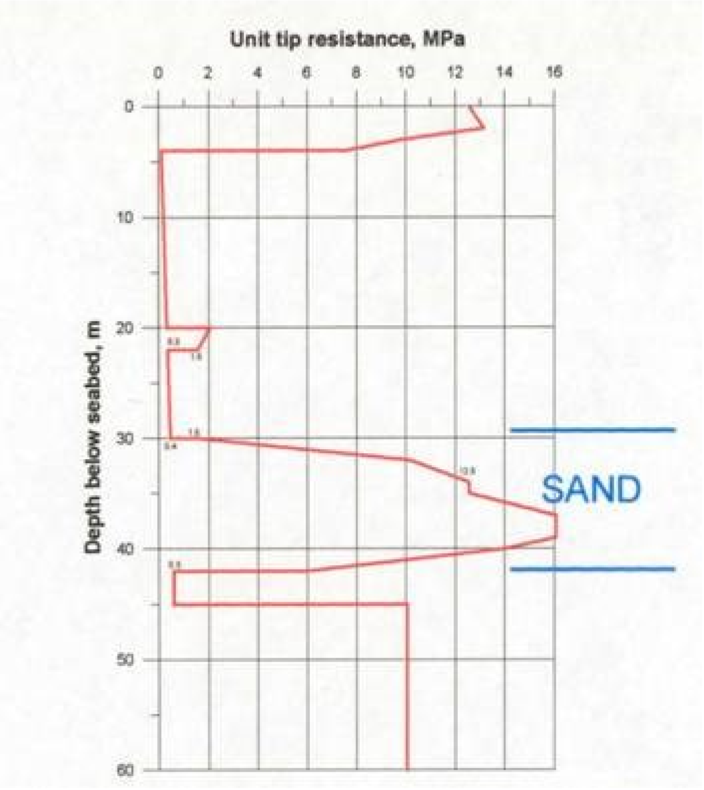
Cone resistance in sand for pile bearing capacity calculation



Pile Capacity from CPTU

Example from Westport Kuala Lumpur

Pile tip resistance in sand by CPT method



Pile bearing capacity from CPTU data

- **It is recommended to use several methods and to adopt the lowest value for evaluation of pile bearing capacity**
  - *Bustamante and Gianeselly(1982) (French method)*
  - *de Ruiter and Beeringen (1979) (European method)*
  - *Imperial College Method (1996)(mainly sand)*
  - *Almeida et al (1996) (clay only--- uses q_t)*

- **If local experience exist, may use only method that has shown to give the best prediction**

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Ground improvement - quality control

Purpose of deep compaction is often to fulfill one of the following:

- Increase bearing capacity (i.e. shear strength)
- Reduce settlements (i.e.increase modulus)
- Increase resistance to liquefaction (i.e. density)

- Cone resistance in cohesionless soils is governed by factors including soil density, in situ stresses, stress history and soil compressiblity
- Changes in cone resistance can therefore be used to document effectiveness of compaction

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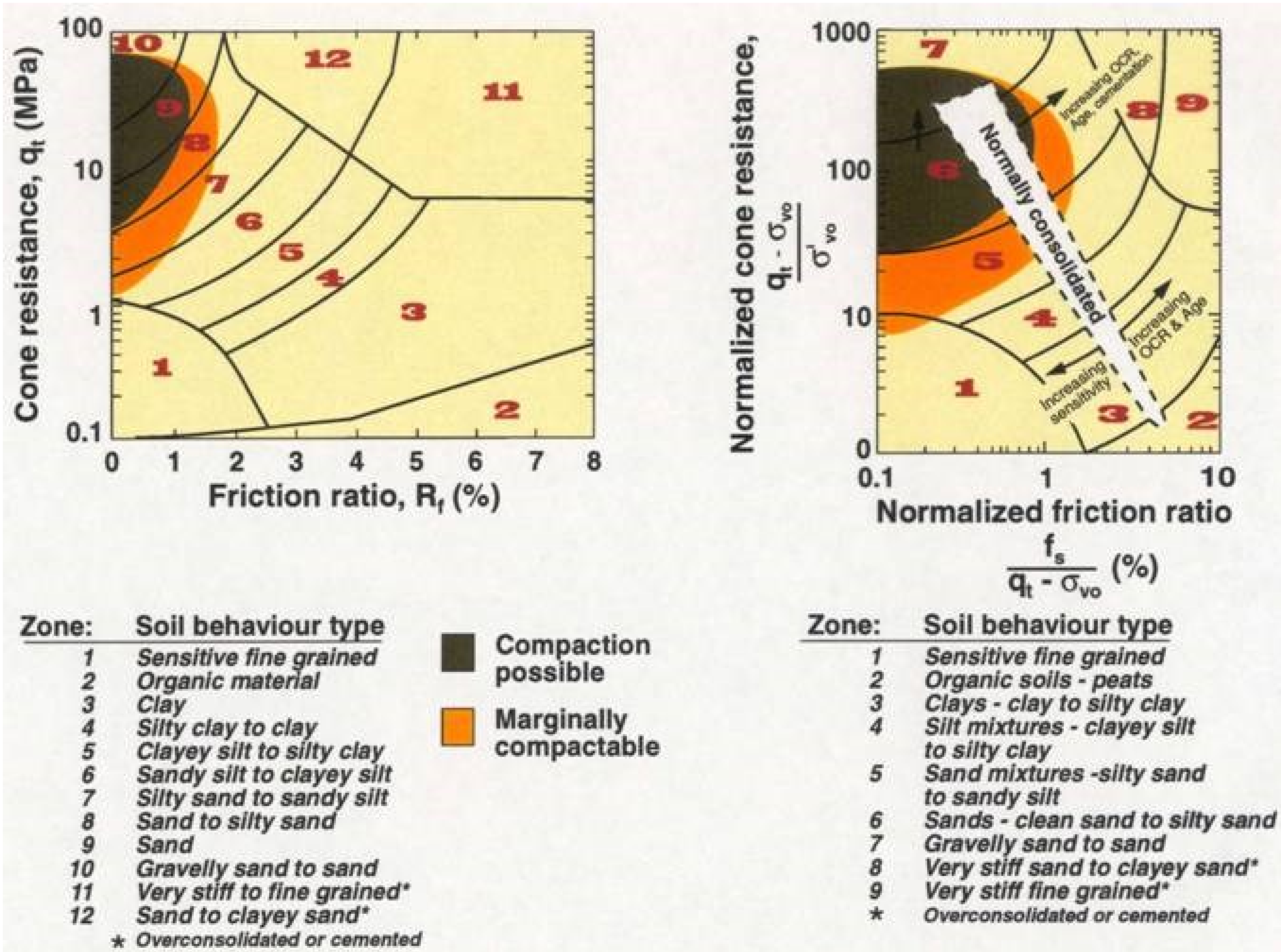
Deep compaction

- vibrocompaction
- vibro-replacement
- dynamic compaction
- compaction piles
- deep blasting

CPT is found to be best method to monitor and document effect of deep compaction

Important to consider time effect

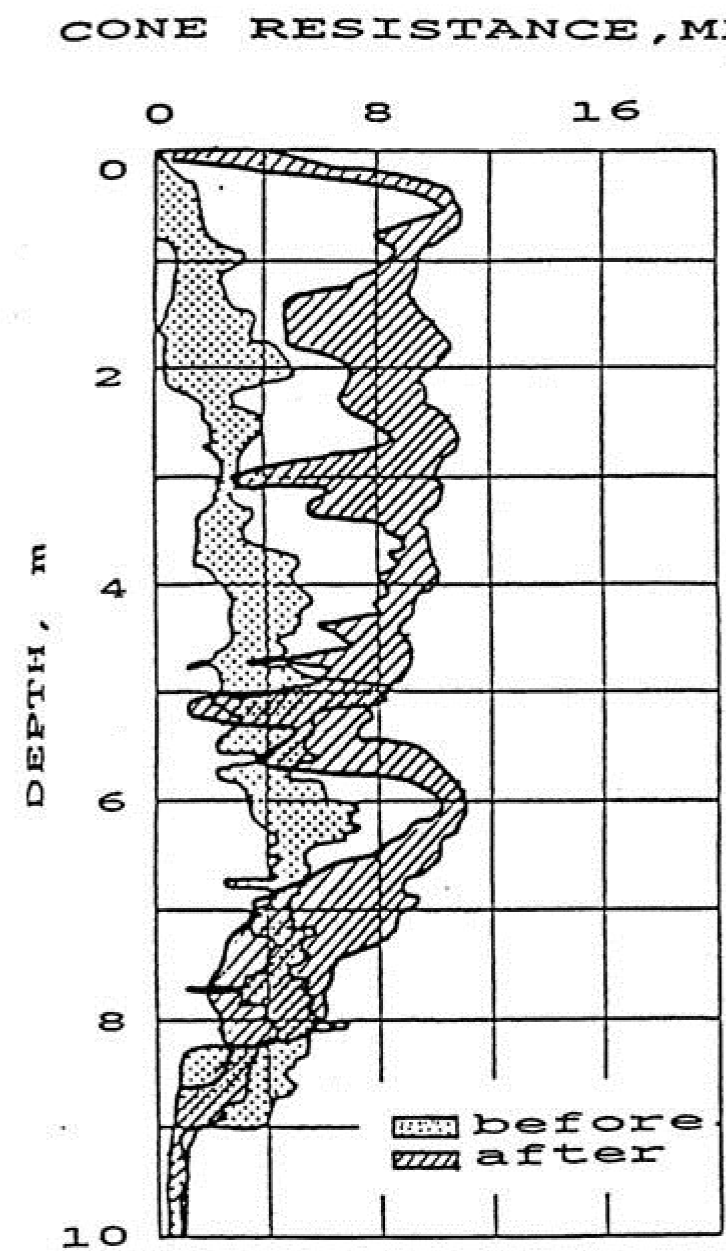
Suitability of soil for vibrocompaction



Massarsch(1994)

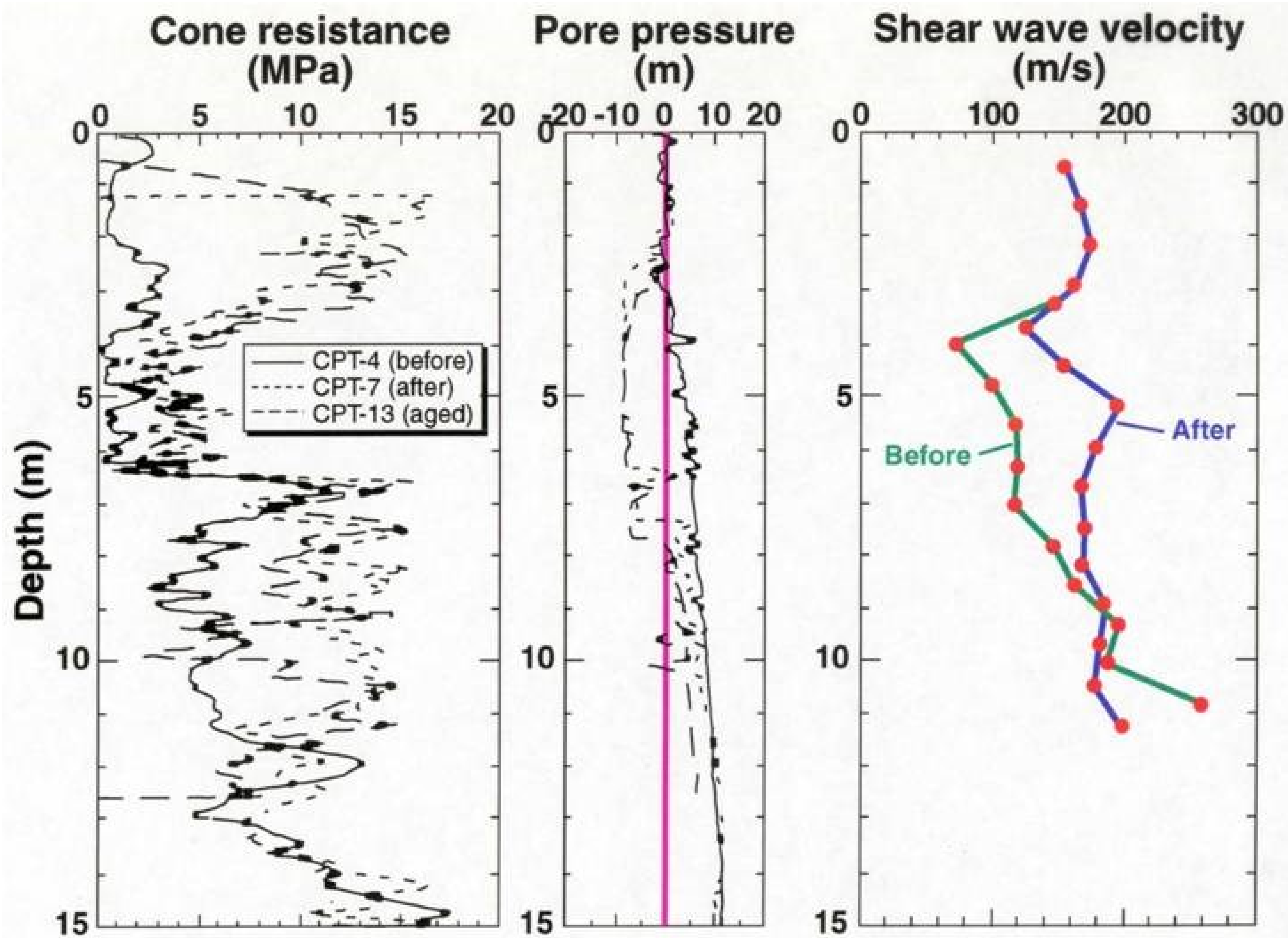
Compaction control

Range of cone penetration test values before and after compaction and surface compaction with vibrating plate



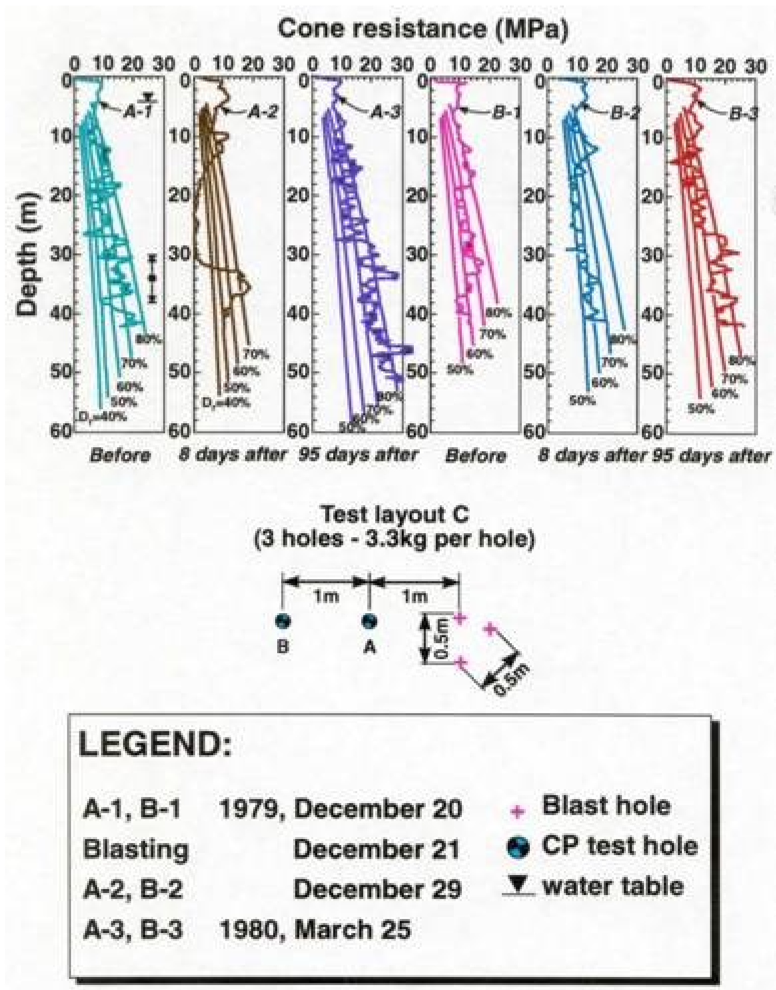
Lindberg and Massarsch(1991)

Influence of time on penetration resistance after dynamic compaction



From Woeller et al. (1995)

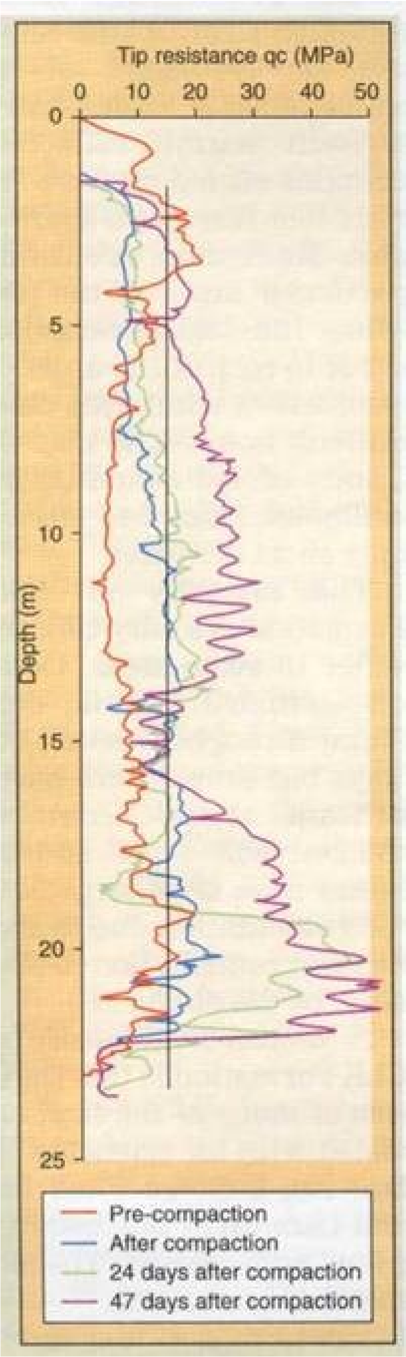
Compaction by blasting



Effect of time

From Mitchell and Solymar(1984)

The aging effects of sands



Effect of vibrocompaction at Chek Lap Kok airport in Hong Kong.

From Ng, Berner and Covil (1996)

Days after dynamic compaction 10 m silty sand (Schmertmann, 1991)

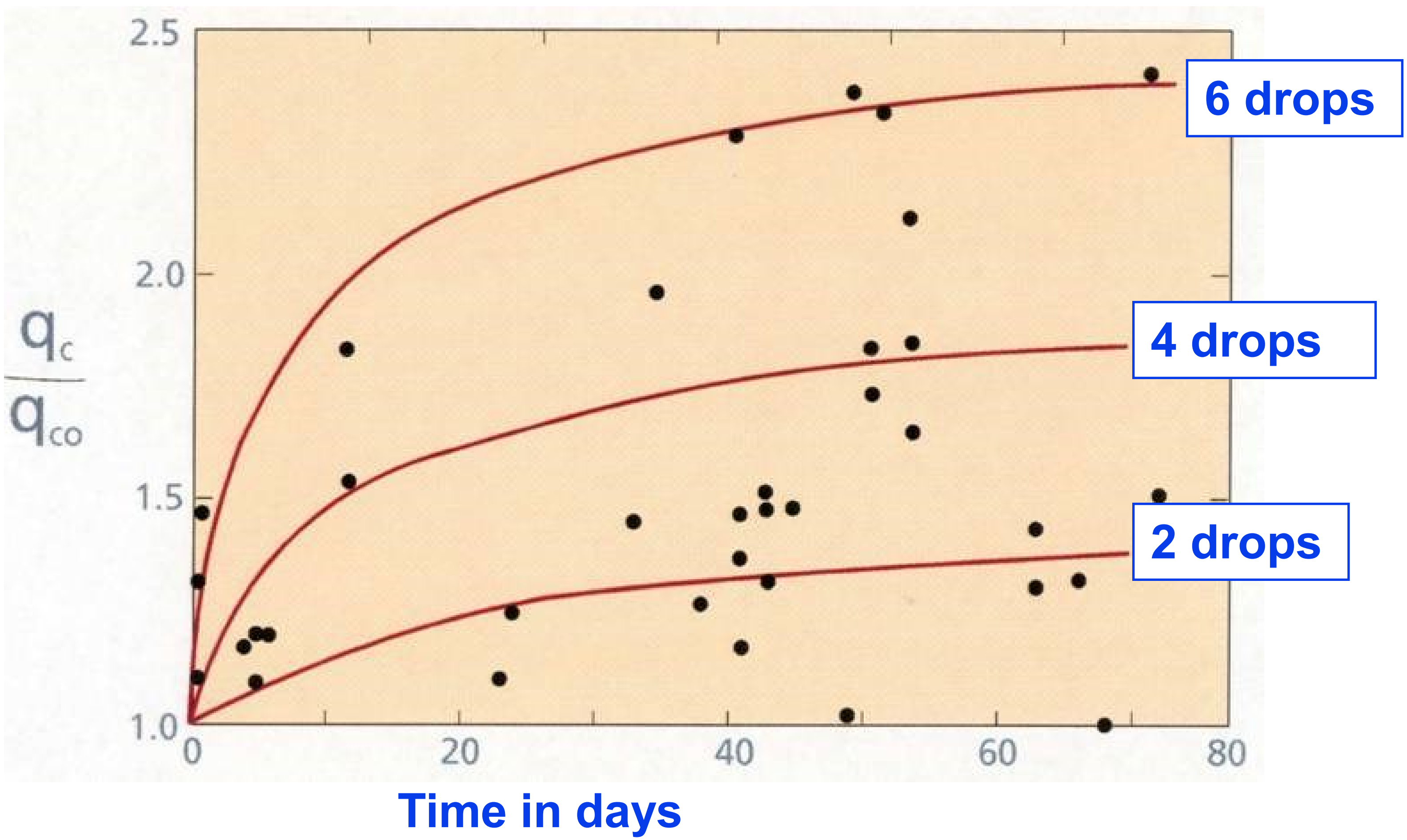


Diagram developed for correcting cone resistance measured just after compaction – large project in Florida

Ground improvement - quality control

For large projects:

- Develop experience with increase in cone resistance with time after compaction took place.
- Use this experience to make criteria for acceptance or rejection based on CPT/CPTUs carried out just after compaction took place

- Where resistance to liquefaction is major issue, measurement of shear wave velocity will provide additional data
- CPTU data can be used to evaluate if compaction will be efficient or not (ref. soil behaviour chart)

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Liquefaction resistance

- Major concern for structures constructed with or on sand and sandy silt.
- Cyclic loads from : earthquakes, wave loading, machine foundations and other

- To evaluate potential for soil liquefaction important to determine soil stratigraphy and *in situ* soil state

- CPT/CPTU ideal because of its repeatablity, reliability, continuous data and cost effectiveness

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Evaluation of liquefaction potential

- **CPT/CPTU provide valuable data**
  - ***detect even thin sand layers that could liquefy***
  - ***pore pressure data tells us about groundwater conditions and additional information to estimate grain size and fines content (together w/sleeve friction)***
  - ***cone resistance gives input to in situ state of sandy soils***
- **SCPTU can give valuable additional data**
  - ***soil type***
  - ***state of soil in situ***

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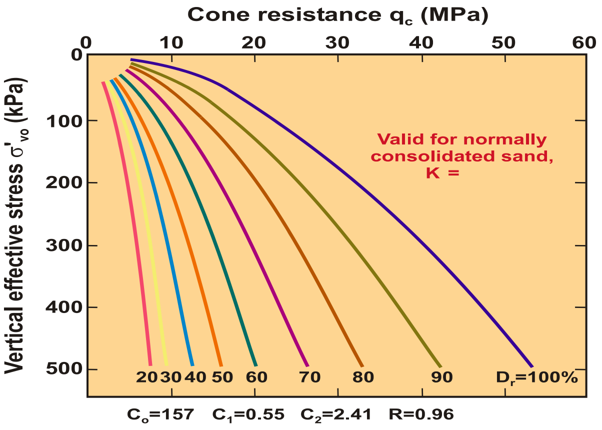
Liquefaction control from CPT/CPTU

<u>Different approaches :</u>

1. a) Estimate D_r from q_c, σ_{vo}', D_r relationship



b) Perform cyclic triaxial and/or direct simple shear tests in laboratory on samples reconstituted to estimated D_r and relevant cyclic stress level (τ_{cy}/ σ_{vo}')

2. Estimate directly from CPT/CPTU results using empirical methods developed in North America and Japan

Liquefaction potential directly from CPT/CPTU results

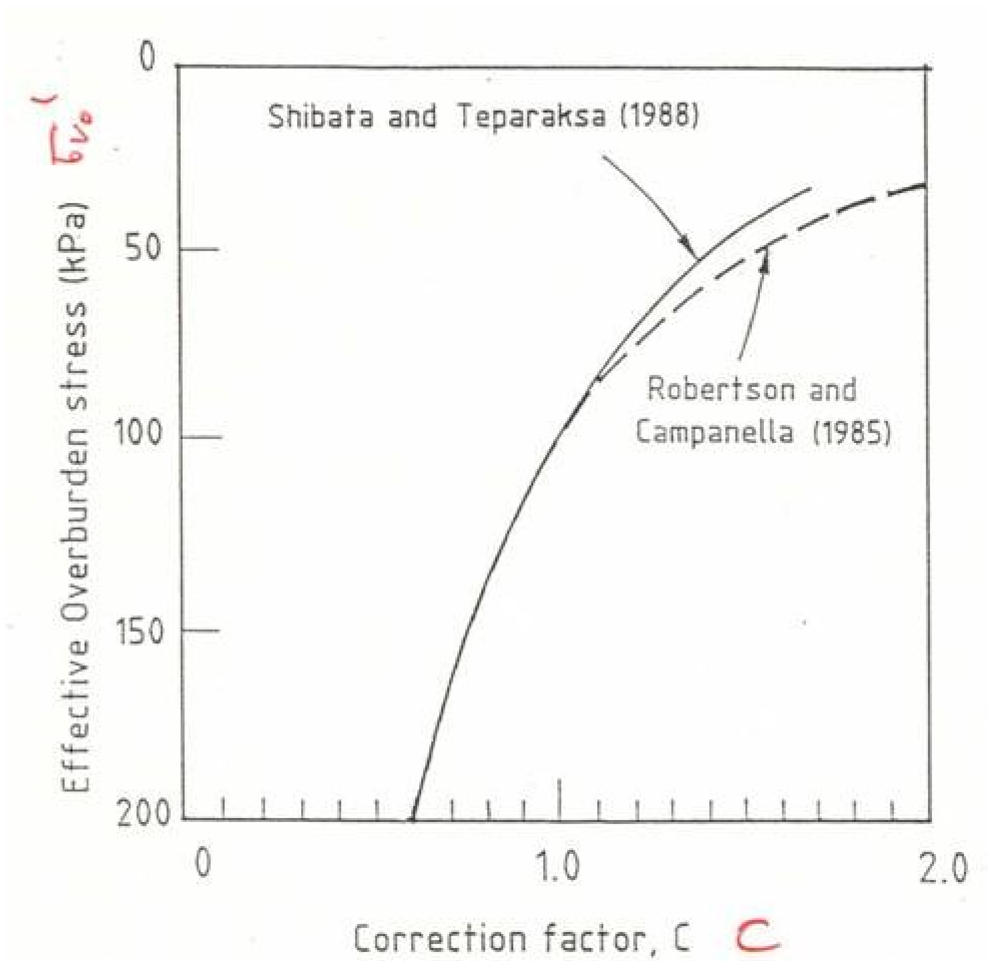
1. Correct q_c for overburden stress effect
$Q_c = C^*q_c$

2. Estimate average cyclic stress ratio (due to wave loading or earthquake or other source) τ_{cy}/σ_{vo}'

3. Establish D_{50} by grain size analysis on obtained sample -or estimate from CPT/CPTU results using soil classification charts

4. Check liquefaction by τ_{cy}/σ_{vo}', Q_c, D_{50} diagram

Liquefaction potential directly from CPT/CPTU results



Correction factor for cone resistance to predict liquefaction potential of sand (from Shibata and Teparaksa, 1988)

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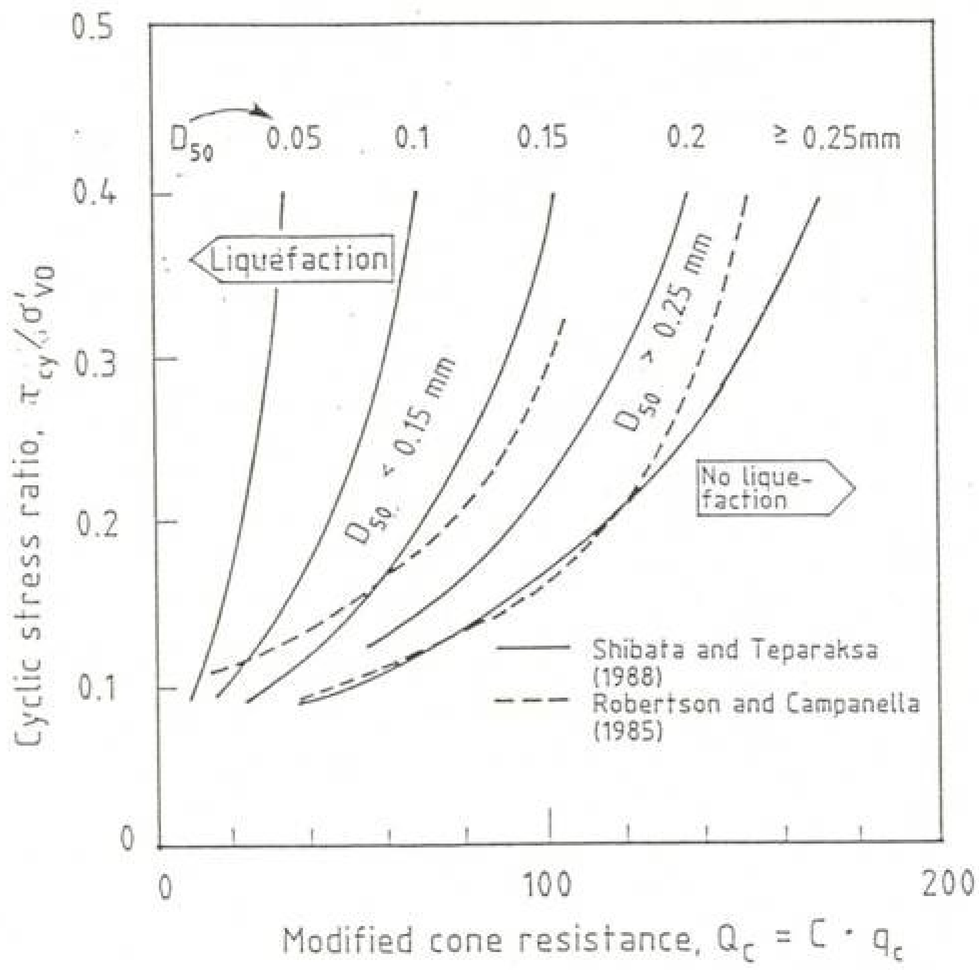
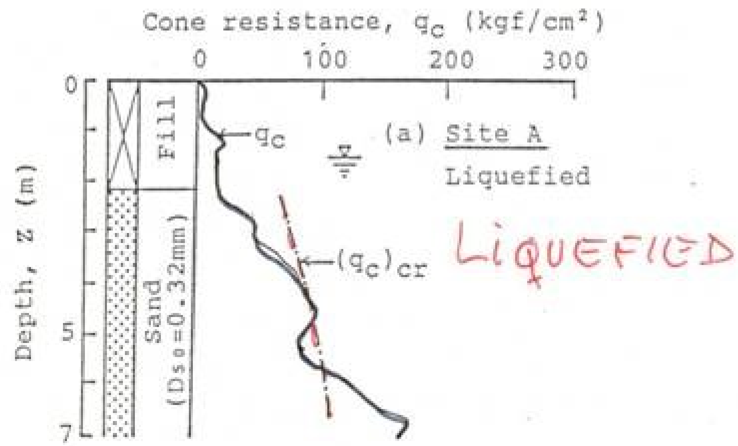
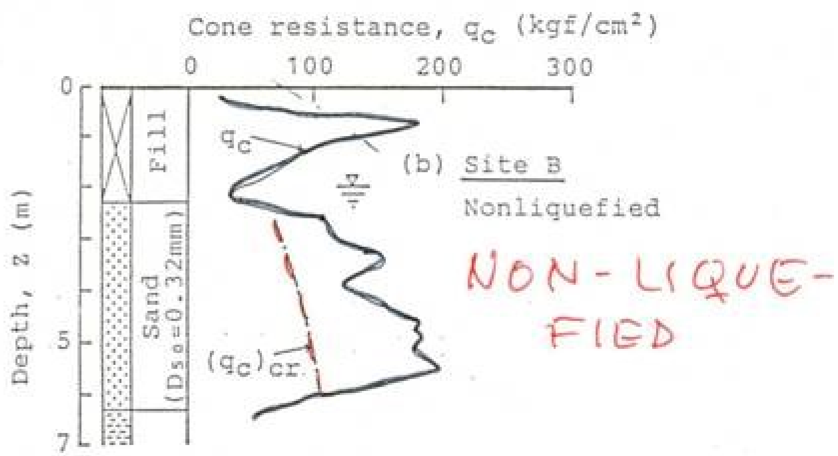
Liquefaction potential directly from CPT/CPTU results



Liquefaction potential from cone resistance (after Shibata and Teparaksa, 1988)

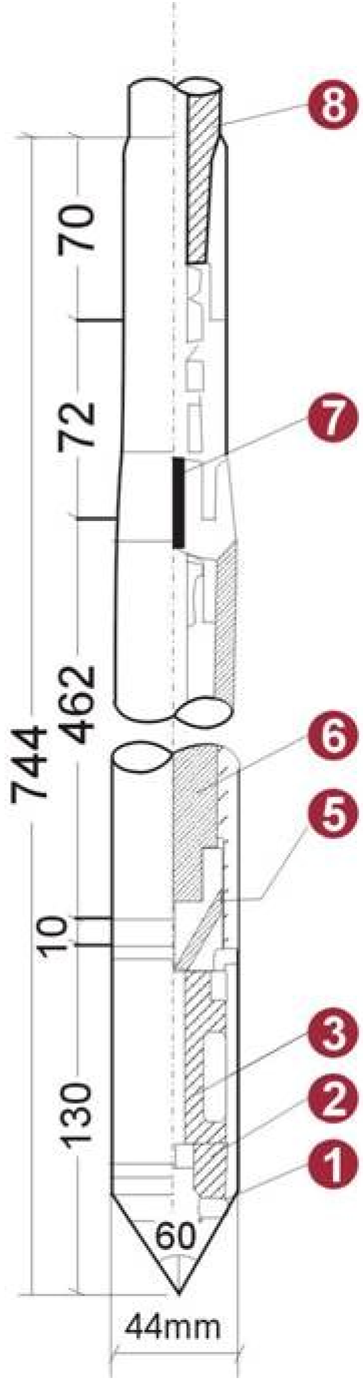
Liquefaction potential directly from CPT/CPTU results





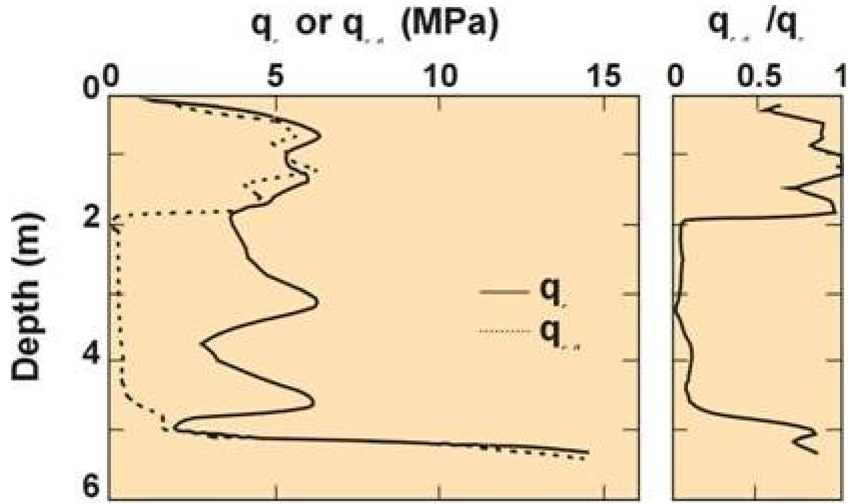
Comparison of q_c with estimated $(q_c)_{cr}$ value in 1983 Nihonkaichuba earthquake (from Shibata and Teparaksa, 1988)

Vibratory cone for liquefaction evaluation

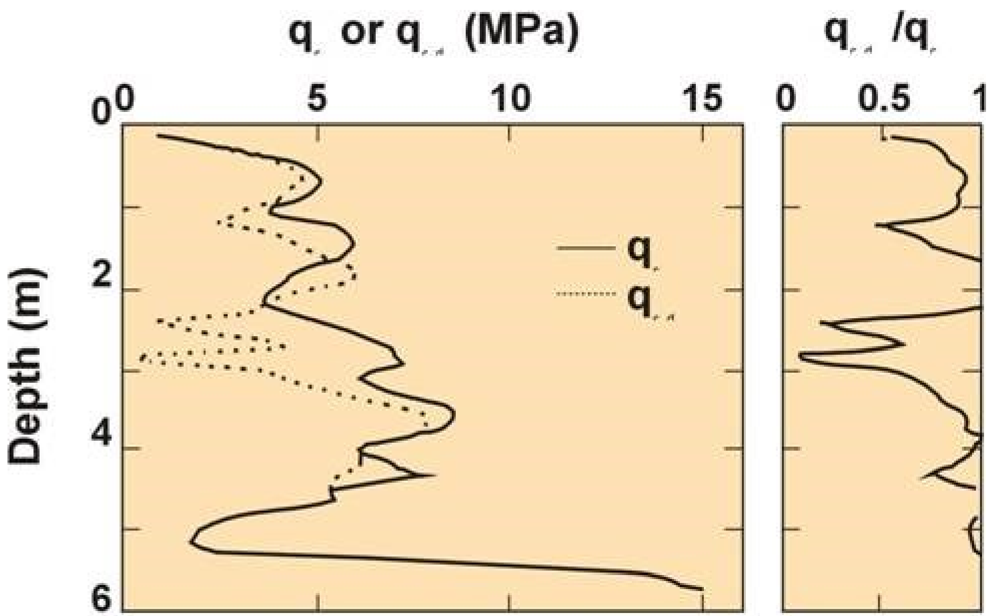


1. Porous metal
2. Pore pressure transducer
3. Load transducer for cone resistance
5. Take-out cable for transducer
6. Vibrator
7. Power source cable for vibrator
8. Push rod

Evalaution of liquefaction potential in Japanese soil



a) Liquefied site



b) Non-liquefied site

PERCEIVED APPLICABILITY OF THE CPT/CPTU FOR VARIOUS DIRECT DESIGN PROBLEMS

	Pile design	Bearing capacity	Settlement	Compaction control	Liquefaction
Sand	1-2	1-2	2-3	1-2	1-2
Clay	1-2	1-2	3-4	3-4	
Intermediate soils	1-2	2-3	3-4	2-3	

Reliability rating:

1=High

2=High to moderate

3=Moderate

4=Moderate to low

5=Low

Reserve overheads

Pile Design method
(after de Ruiter European CPT and Beringen, 1979)

<u>Clay :</u>

Unit skin friction, f_p, minimum of:

$-f_p = \alpha * s_u$

.where $\alpha = 1$ for NC clays ; 0.5 for OC clays

Unit tip resistance, q_p, minimum of :

$-q_p = N_c * s_u$ where $N_c = 9$ and $s_u = q_c/N$

$N_k = 15 - 20$

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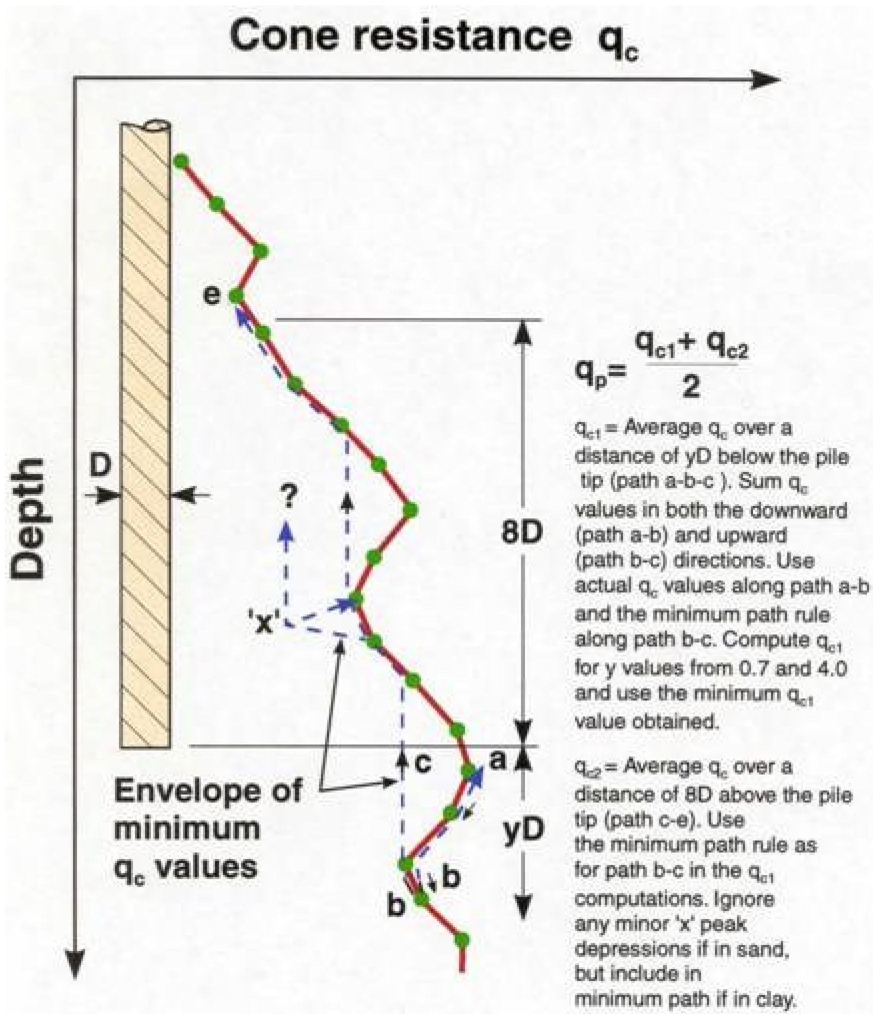
Computation of q_c for pile tip resistance : 'European method'



De Ruiter and Beeringen(1979)

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Pile Design method
(after de Ruiter European CPT and Beringen, 1979)

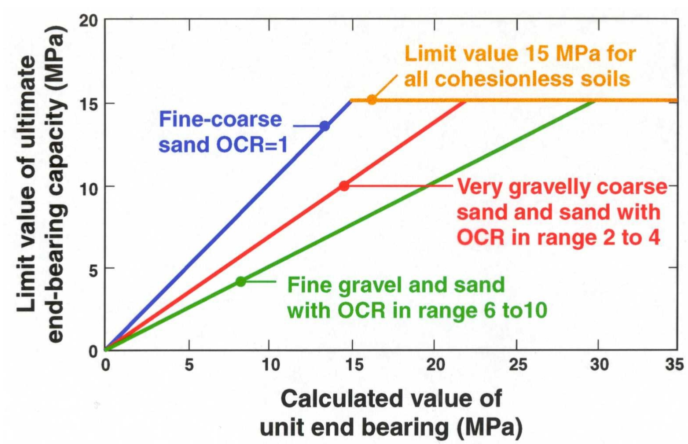
SAND:

Unit skin friction, f_p, minimum of :

- f_1 = 0.12 Mpa
- f_2 = CPT sleeve friction, f_s
- $f_3 = q_c/300$ (compression piles)
- $f_4 = q_c/400$ (tension piles)

Unit end bearing, q_p, minimum of :

- q_p from fig. 6.6



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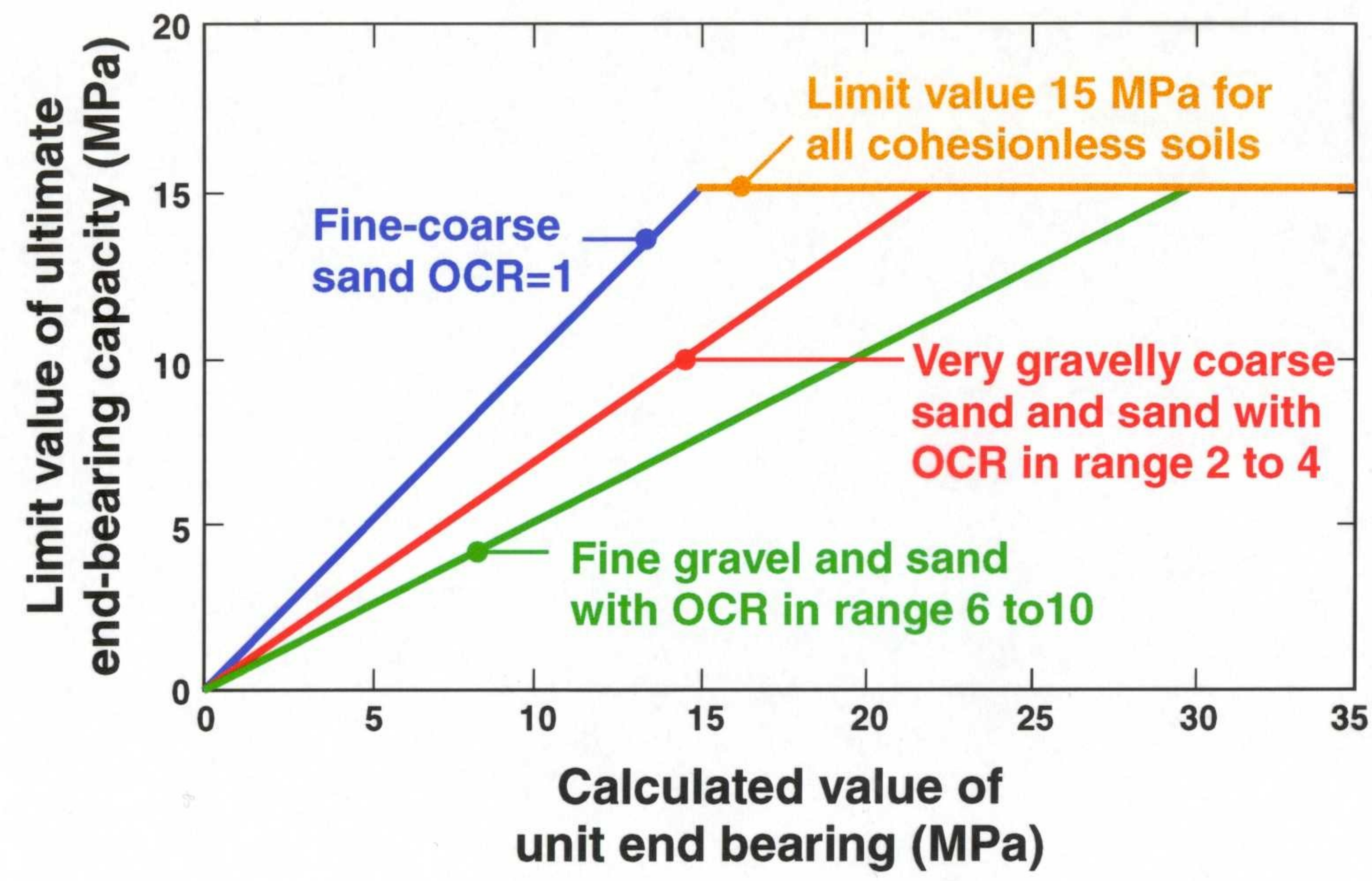
Limited values of pile tip resistance



De Ruiter and Beeringen (1979)

Settlements of shallow foundations on sand

Schmertmann (1970,1978)

$$s = C_1 * C_2 * \Delta p * \Sigma(I_z/E_s)\ \Delta z$$

C_1 = correction for depth of embedment

C_2 = creep (time) correction

Δp = net extra foundation stress

I_z = strain influence factor

E_s = Equivalent Young's modulus = $\alpha * q_c$

α = 2.5 square footing ; α = 3.5 long footing

Δz = thickness of sublayer

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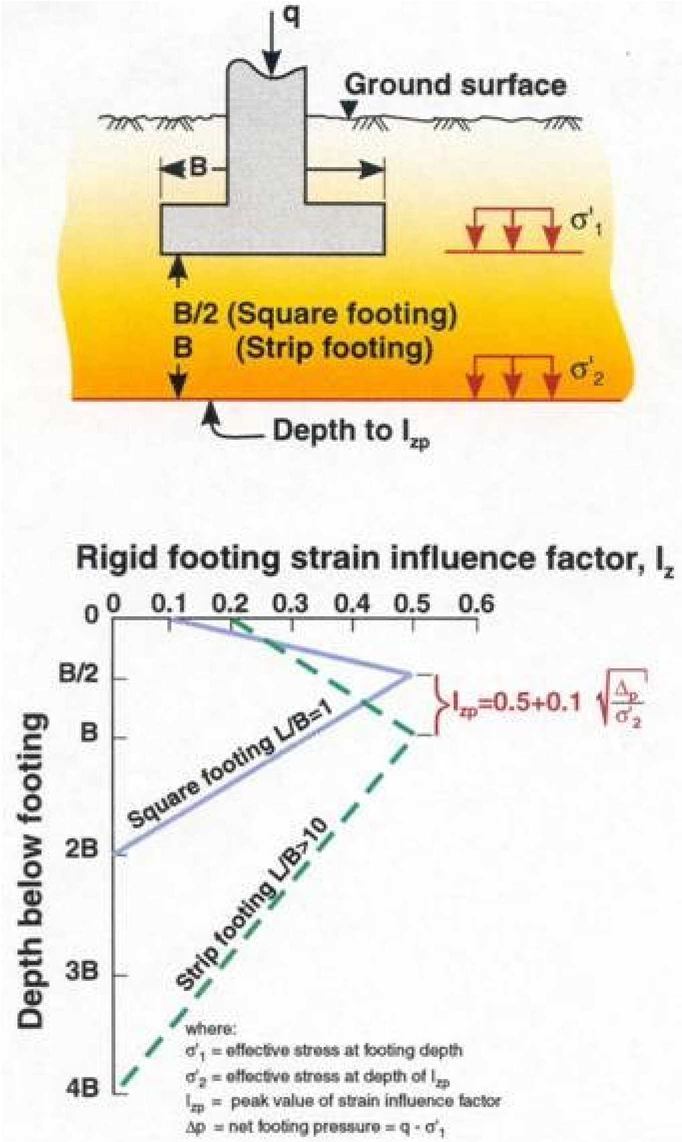
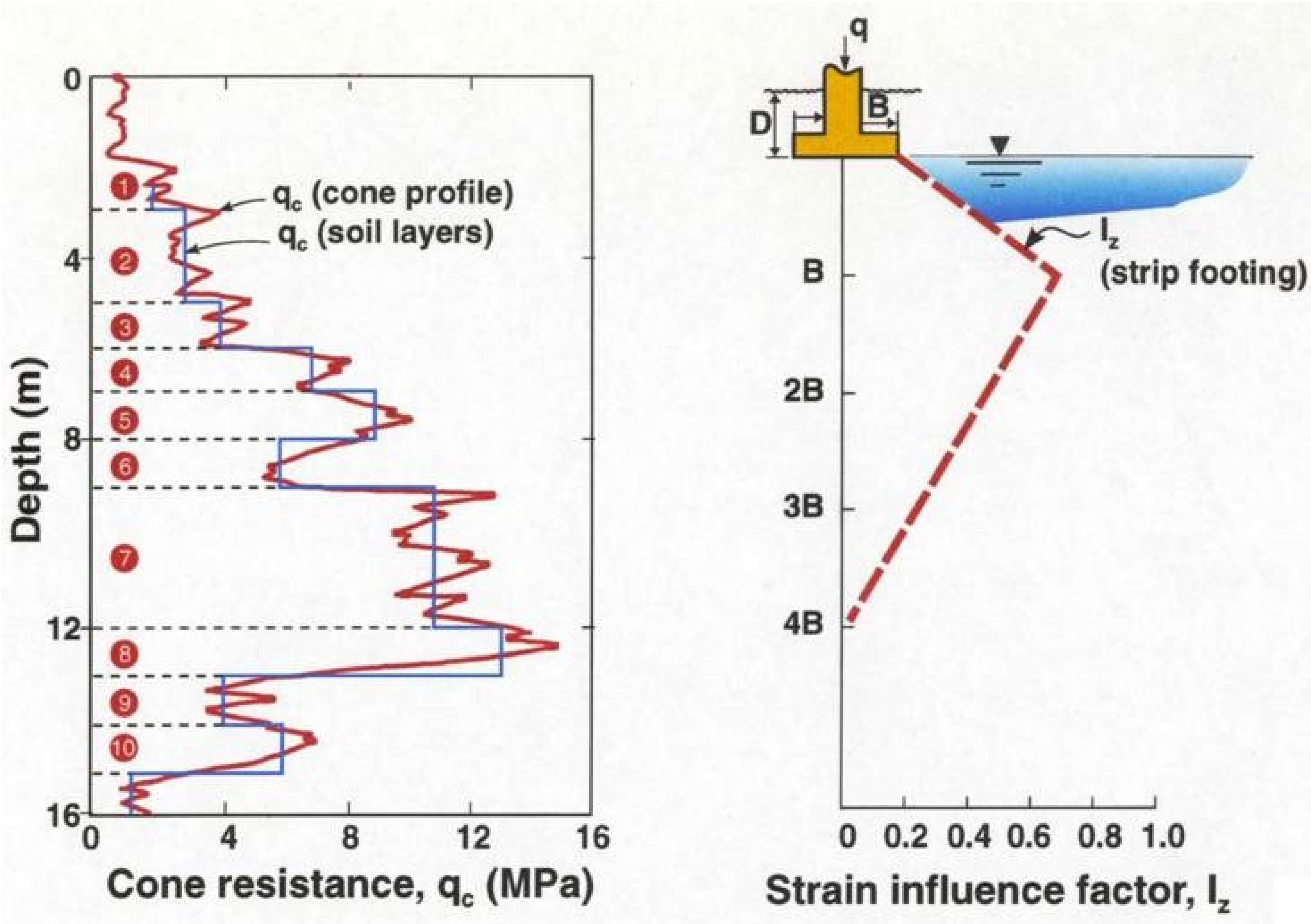
Strain influence method for footings on sand

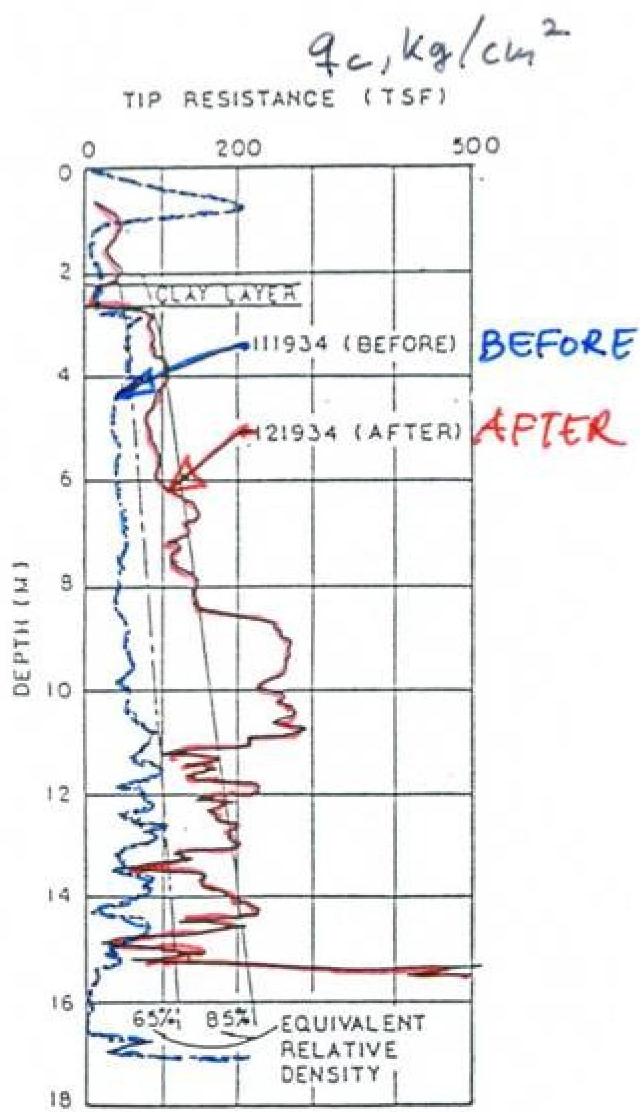


Schmertmann(1970)

Strain influence method for footings on sand (Schmertmann,1970)



Compaction control



Example of comparative before and after CPT logs with a near-surface clay layer

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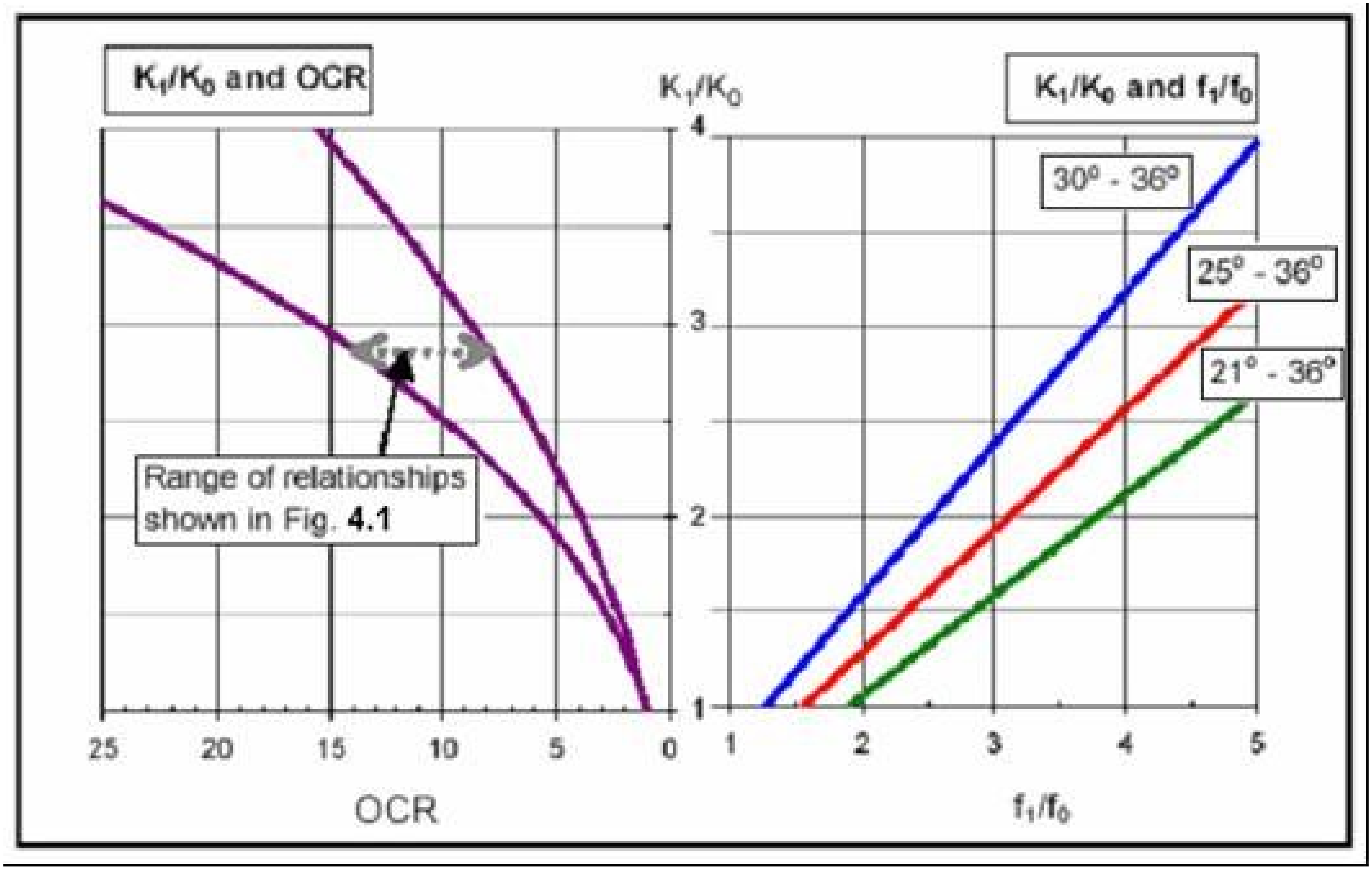
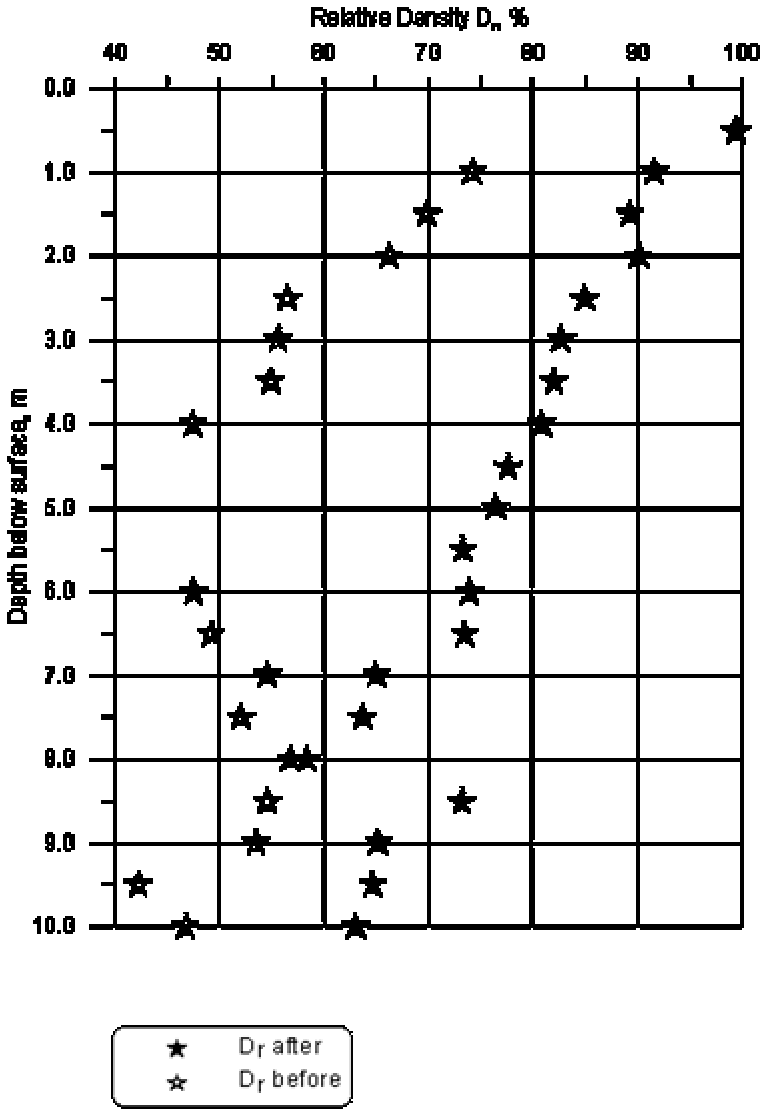
Chart for finding change in K_o and D_r



Relative density calculated according to Baldi et al (1986) using the mean effective stress calculated with the K_0 values in previous slide



Case: Changi airport (Massarsch and Fellenius, 2002)

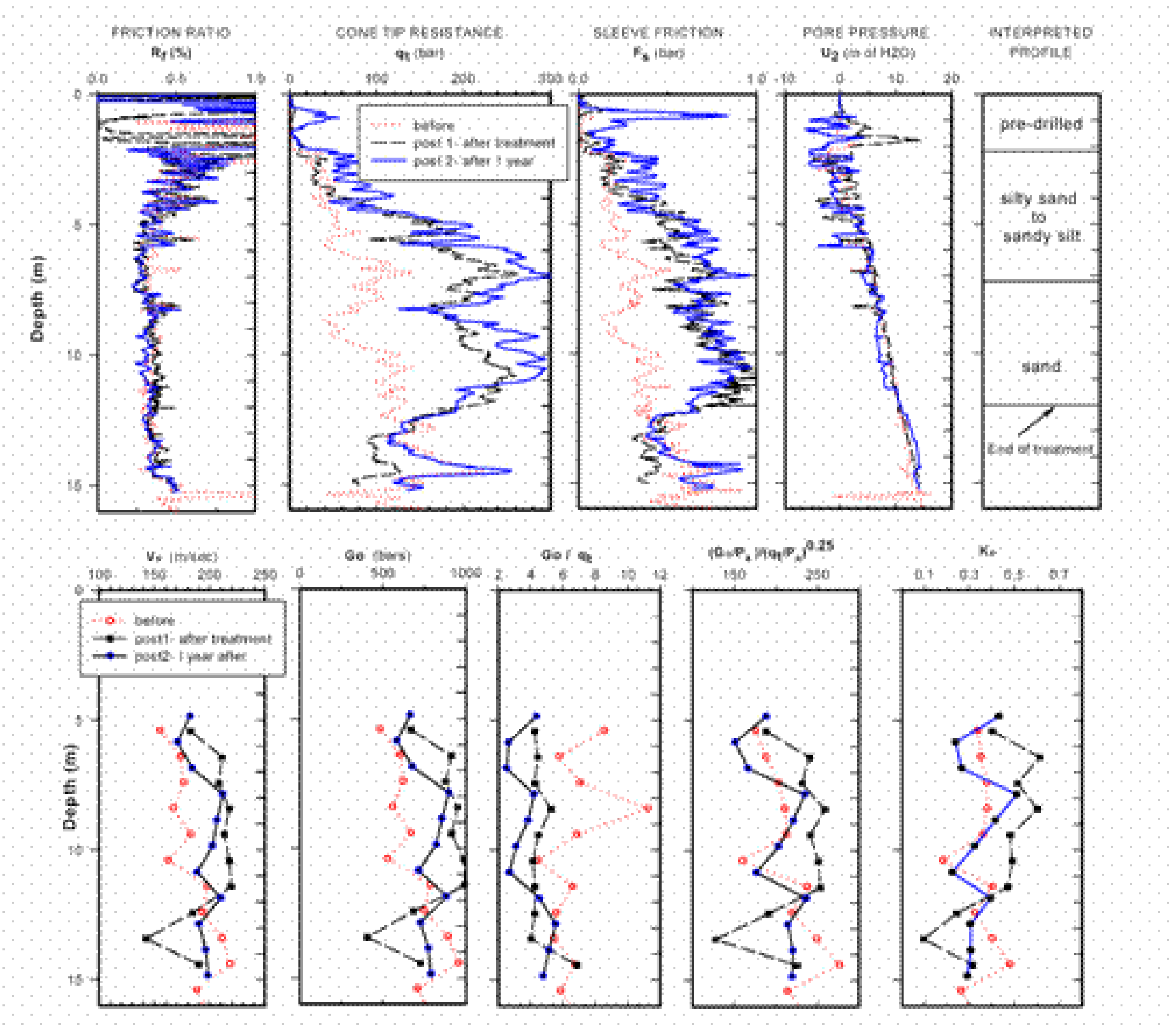


Figure 5.8 Seismic CPT results before and after compaction by vibro-compaction (after Howie et al. 2001)

$$K_o = f(\sigma_{vo}', G_o, q_t)$$

K_o of hydraulic fills and changes with compaction

Massarsch and Fellenius (2002) present a method for estimating the *change* in K_0 of a hydraulic fill before and after compaction. This simple method uses the sleeve friction measured during CPTUs and estimates of the respective internal friction angles with the following formula:

$$K_{01} / K_{00} = (f_{s1} \cdot \tan \phi'_0) / (f_{s0} \cdot \tan \phi'_1) \qquad \text{Eq. 4.1}$$

Where

K_{00}	=	coefficient of earth pressure at rest before compaction
K_{01}	=	coefficient of earth pressure at rest after compaction
ϕ'_0	=	internal angle of friction before compaction
ϕ'_1	=	internal angle of friction after compaction
f_{s0}	=	sleeve friction on cone before compaction
f_{s1}	=	sleeve friction on cone after compaction

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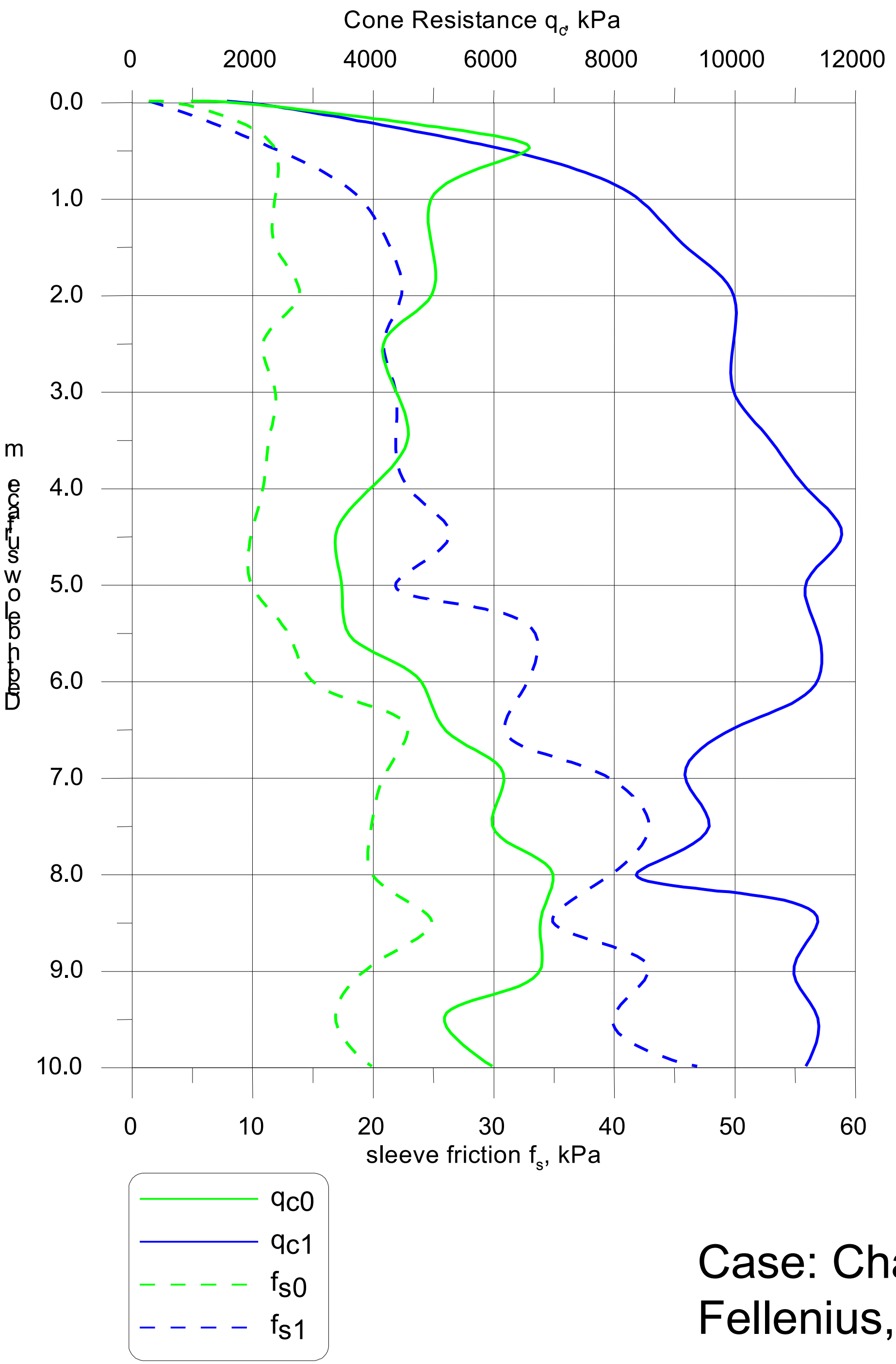
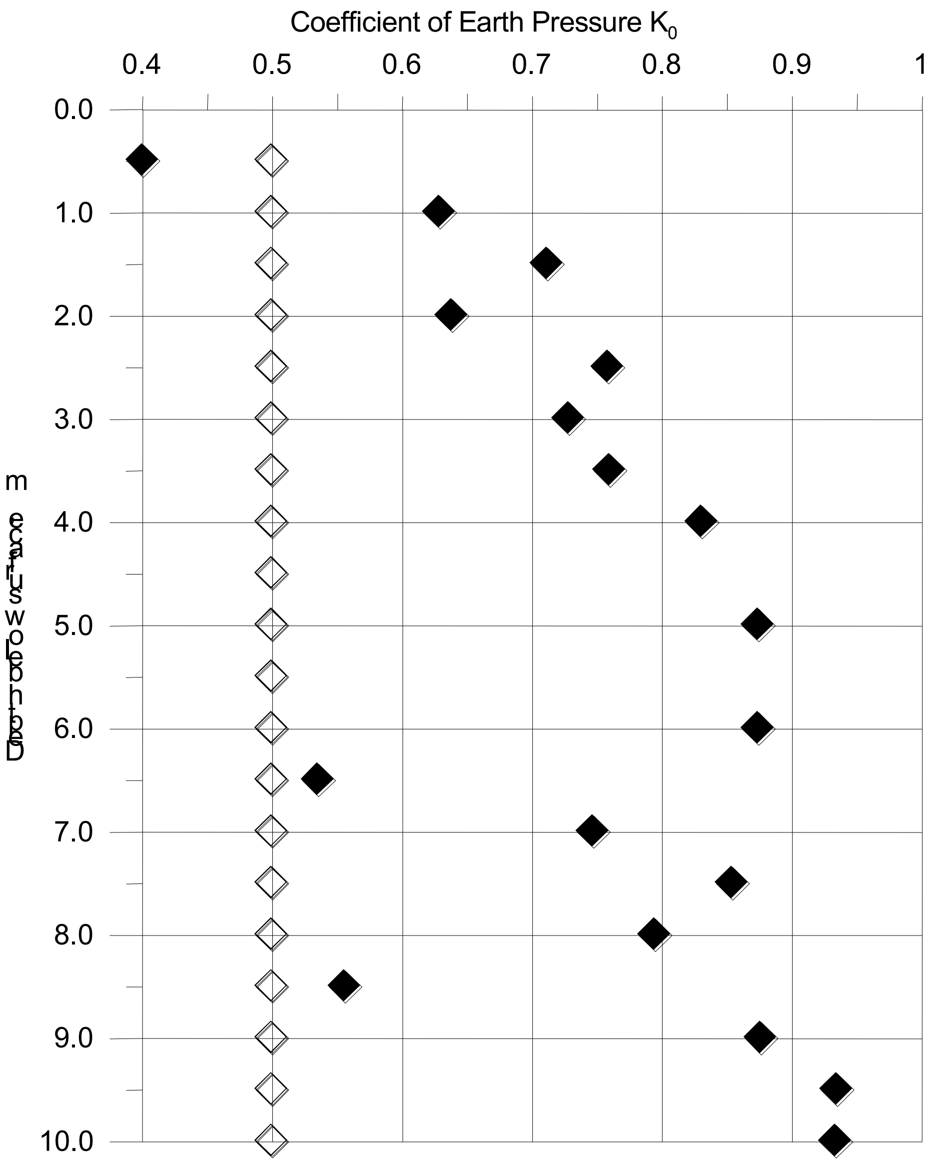
K_o of hydraulic fills and changes with compaction

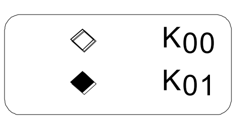
- Effect of compaction is to increase both density (or D_r) and in situ horizontal stress (or K_o)
- Massarsch and Fellenius(2002) have suggested approach for evaluating change in K_o due to compaction from CPT results



Cone resistance and sleeve friction before and after compaction

Case: Changi airport (Massarsch and Fellenius, 2002)





K_0 before and after compaction using friction angles of 30 and 36 degrees respectively

Case: Changi airport (Massarsch and Fellenius, 2002)

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