

USE OF CPT/CPTU FOR SOLUTION OF PRACTICAL PROBLEMS

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Indirect design method:

- Interpret 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

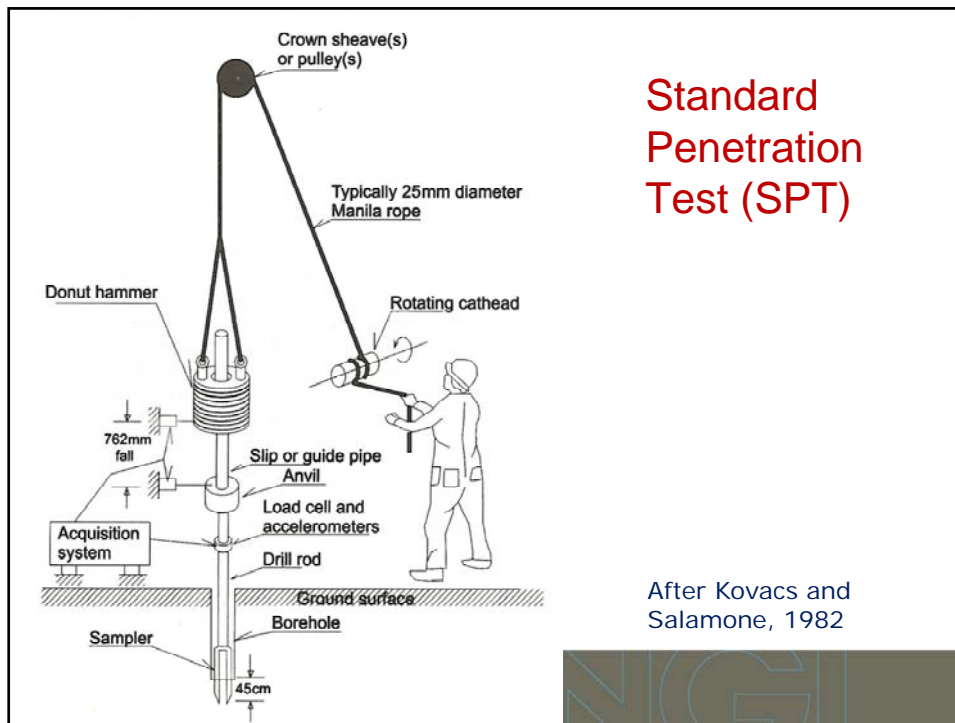
DIRECT APPLICATIONS OF CPT/CPTU RESULTS

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

SPT – Standard penetration test

- One of the most used in situ tests in the world
- The basis of the test consists of driving a sampler by dropping a hammer of 63,5 kg mass on to an anvil or drive head from a height of 760 mm.
- The number of blows (N) necessary to achieve a penetration of the sampler of 300 mm is the penetration resistance.

A disturbed sample of 38 mm diameter is obtained



Need for SPT/CPT correlations

- It is now recognized that the CPTU gives much more reliable and useful data compared to the SPT
- Still many engineers have a lot of experience with the SPT and asks for N – values when CPTs have been performed
- In many projects earlier investigations included SPT data and it is very useful to "convert" the N-values to q_c in order to compare the old and new data

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} .

NINGI

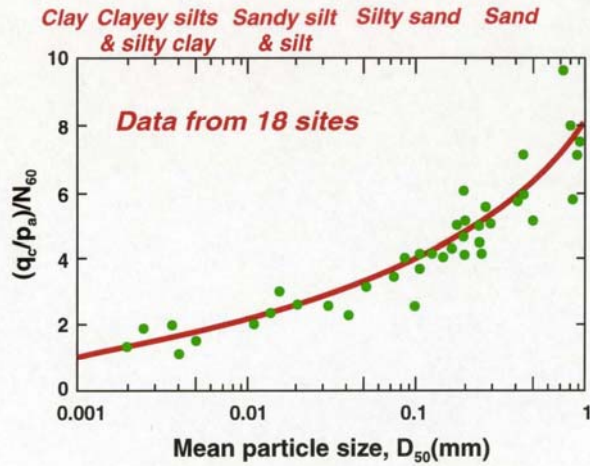
CPT/SPT CORRELATIONS

Correlations most used:

- Robertson et al. 1983
- Kulhawy and Mayne, 1990

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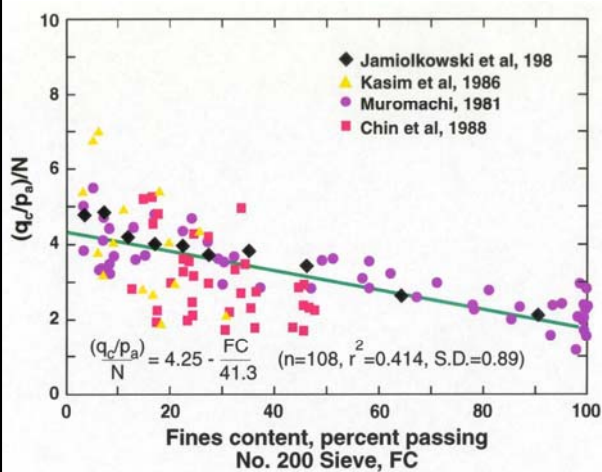
CPT/SPT CORRELATIONS



P_a = reference stress
= 1 atm = 100 kPa

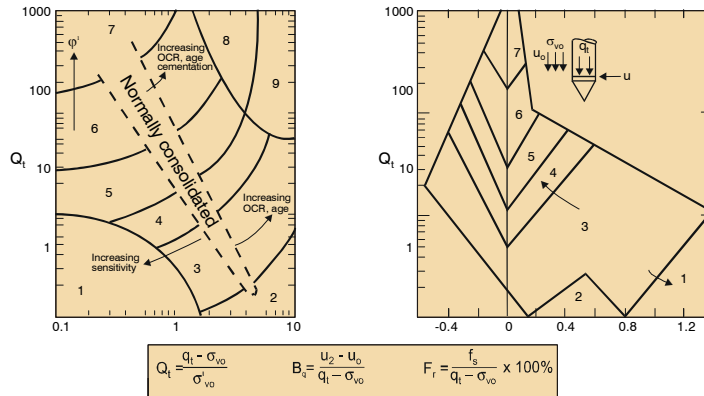
Robertson and Campanella (1983)

CPT/SPT CORRELATIONS Effects of fines content



Mayne and Kulhavy (1990)

Normalized soil behaviour classification chart



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

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

$$Q_t = \frac{q_t - \sigma_{v0}}{\sigma_{v0}'}, \quad F_r = \frac{f_s}{q_t - \sigma_{v0}}$$

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

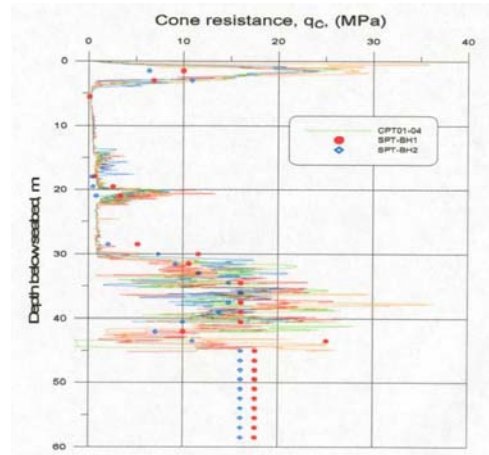
$p_a = \text{atm. Press.} = 100 \text{ kPa}$

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

Example CPT/SPT Correlations

Westport
Warehouse
facility outside
Kuala Lumpur

Soil
investigation by
Soils and
Foundations
Sdn.Bhd



A lot of old investigations with SPT

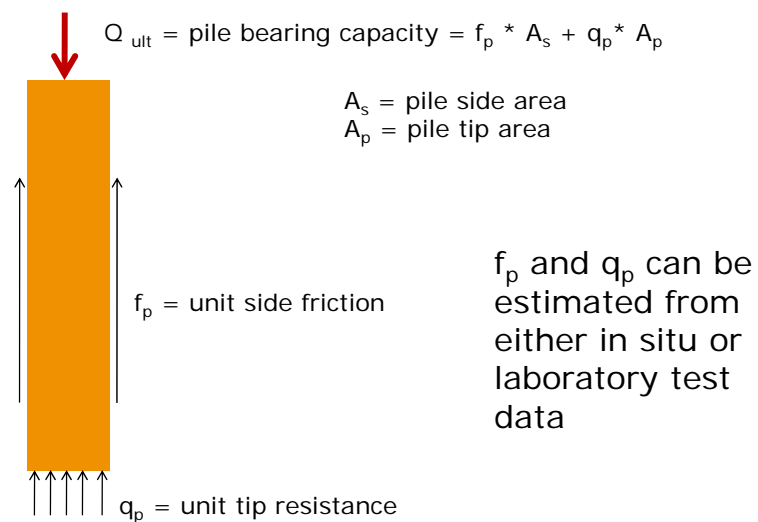
CPT/SPT correlations

- **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 not available**
Use soil behaviour index , $I_c (= f(Q_t, F_t))$
$$(q_c/p_a)/N_{60} = 8.5(1 - I_c/4.6)$$

Pile bearing capacity

- Earliest application of CPT data
- Complicated by large variety of pile types and installation procedures
- Most design techniques based on empirical methods

PILE BEARING CAPACITY



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 laboratory based methods.

French method by Bustamante and Gianceselli (1982) proved to be most consistent

AXIAL PILE CAPACITY

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

Bustamante and Gianceselli (1982)

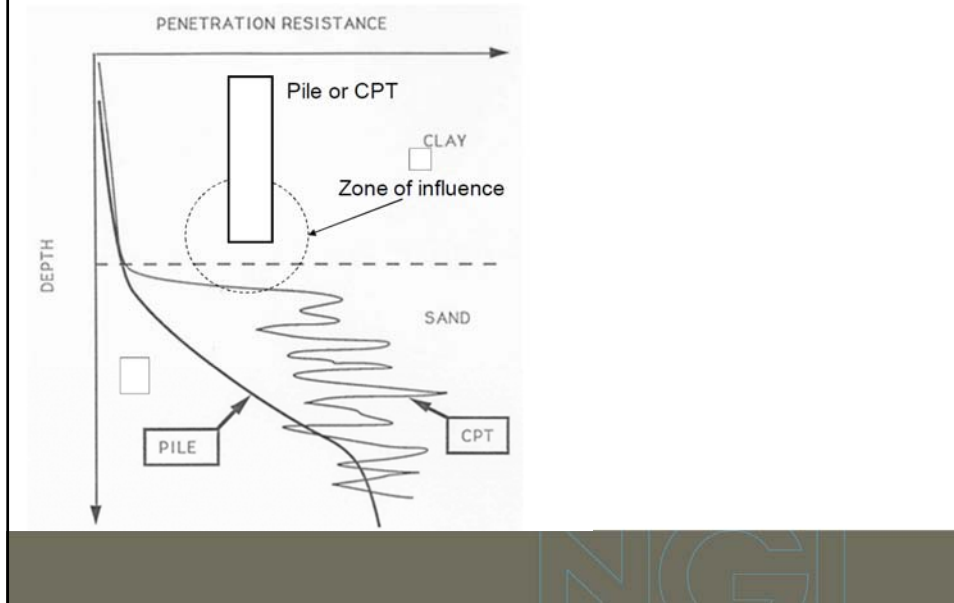
$$f_p = q_c / \alpha$$

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

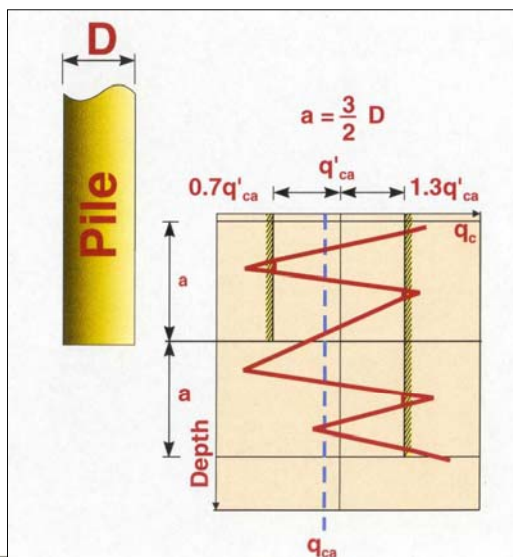
α and k_c empirical constants for different pile and soil types

Based on a 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

Scale effect for unit end bearing



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)

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 chalk	≤ 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

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

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 driven; grouted driven metal piles; driven rammed piles; jacket concrete piles; high pressure grouted piles of large diameter.

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

$$f_p = q_c / \alpha$$

f_p should not exceed given maximum values

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)	0.08	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 ≤

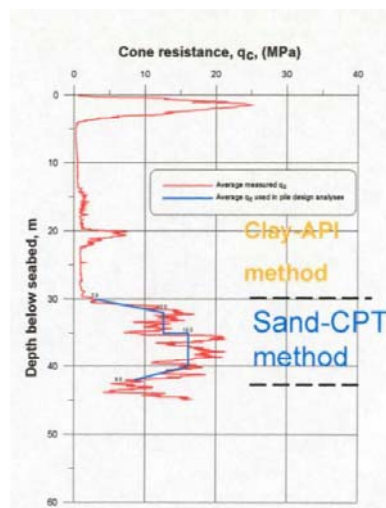
$$f_p = q_c / \alpha$$

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

Pile Capacity from CPT

Example from Westport, Kuala Lumpur

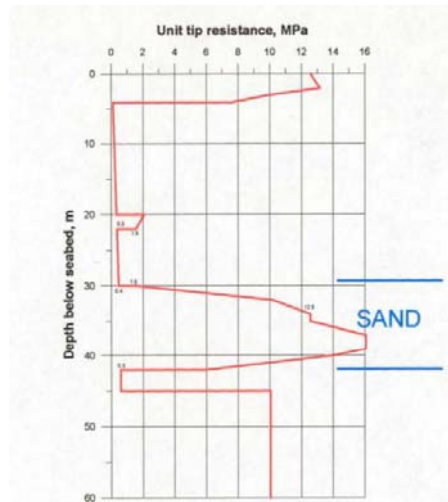
Cone resistance in sand for pile bearing capacity calculation



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 a conservative value for evaluation of pile bearing capacity**

Bustamante and Ganeselly(1982) (French method)

de Ruitter and Beeringen (1979) (European method)

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

Pile bearing capacity from CPTU data

Recent methods for offshore applications

- Imperial College Method (1996)
- NGI (2005)
- Fugro (2005)
- UWA (2005)

Included in Commentary of API 2011.

Comprehensive comparison published at ISFOG

conference in Perth (2005), all accepted by API

Other key references: Schneider et al. (2008) and

Knudsen et al. (2012)

Fugro -05 Formulation of shaft friction τ_f

$$\tau_f = 0.08 q_c \left(\frac{\sigma'_{v0}}{p_{ref}} \right)^{0.05} \left(\frac{h}{R^*} \right)^{-0.90} \quad (\text{compression loading for } h/R^* \geq 4)$$

$$\tau_f = 0.08 q_c \left(\frac{\sigma'_{v0}}{p_{ref}} \right)^{0.05} (4)^{-0.90} \left(\frac{h}{4R^*} \right) \quad (\text{compression loading for } h/R^* \leq 4)$$

$$\tau_f = 0.045 q_c \left(\frac{\sigma'_{v0}}{p_{ref}} \right)^{0.15} \left[\max \left(\frac{h}{R^*}, 4 \right) \right]^{-0.85} \quad (\text{tension loading})$$

Note: τ_f =local ultimate shaft friction; δ_f =interface friction angle; p_{ref} =100 kPa=reference stress; L =pile length; z =element depth; h =height above pile tip; R^* =equivalent pile radius= $(R^2 - R_i^2)^{0.5}$ where R_i =internal pile radius= $D_i/2$. For noncircular piles, an equivalent circular area is used to assess R^* and D . $\Delta\sigma'_{rd} \approx 4G\Delta y/D$ =change in radial stress during pile loading; $G \approx 185q_{c1N}^{-0.7}$ =operational level of shear modulus; see Eq. (8b) for calculation of q_{c1N} ; $\Delta y \approx 2R_a \approx 0.02$ mm=radial displacement during pile loading; and R_a =average roughness of the pile.

From Schneider et al. (2008)

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From Schneider et al. (2008)

ICP -05 Formulation of shaft friction τ_f

$$\tau_f = a \left[0.029 b q_c \left(\frac{\sigma'_{v0}}{p_{ref}} \right)^{0.13} \left[\max \left(\frac{h}{R^*}, 8 \right) \right]^{-0.38} + \Delta \sigma'_{rd} \right] \tan \delta_f$$

$a=0.9$ for open-ended piles in tension and 1.0 for all other cases

$b=0.8$ for piles in tension and 1.0 for piles in compression

Note: τ_f =local ultimate shaft friction; δ_f =interface friction angle; $p_{ref}=100$ kPa=reference stress; L =pile length; z =element depth; h =height above pile tip; R^* =equivalent pile radius= $(R^2 - R_i^2)^{0.5}$ where R_i =internal pile radius= $D_i/2$. For noncircular piles, an equivalent circular area is used to assess R^* and D . $\Delta \sigma'_{rd} \approx 4G\Delta y/D$ =change in radial stress during pile loading; $G \approx 185q_c q_{c1N}^{-0.7}$ =operational level of shear modulus; see Eq. (8b) for calculation of q_{c1N} ; $\Delta y \approx 2R_a \approx 0.02$ mm=radial displacement during pile loading; and R_a =average roughness of the pile.

From Schneider et al. (2008)

NGI-05 Formulation of shaft friction τ_f

$$\tau_f = z/L p_{ref} F_{D_r} F_{sig} F_{tip} F_{load} F_{mat} \geq \tau_{min}$$

$$F_{D_r} = 2.1(D_r - 0.1)^{1.7}$$

$$D_r = 0.4 \ln(q_{c1N}/22)$$

$$F_{sig} = (\sigma'_{v0}/p_{ref})^{0.25}$$

$F_{tip} = 1.0$ for driven open ended and 1.6 for ended

$F_{load} = 1.0$ for tension and 1.3 for compressi

$F_{mat} = 1.0$ for steel and 1.2 for concrete

$$\tau_{min} = 0.1 \sigma'_{v0}$$

Nominal relative density, may be > 1

$$q_{c1N} = (q_c/p_{ref})(p_{ref}/\sigma'_{v0})^{0.5}$$

Note: τ_f =local ultimate shaft friction; δ_f =interface friction angle; $p_{ref}=100$ kPa=reference stress; L =pile length; z =element depth; h =height above pile tip; R^* =equivalent pile radius= $(R^2 - R_i^2)^{0.5}$ where R_i =internal pile radius= $D_i/2$. For noncircular piles, an equivalent circular area is used to assess R^* and D . $\Delta \sigma'_{rd} \approx 4G\Delta y/D$ =change in radial stress during pile loading; $G \approx 185q_c q_{c1N}^{-0.7}$ =operational level of shear modulus; see Eq. (8b) for calculation of q_{c1N} ; $\Delta y \approx 2R_a \approx 0.02$ mm=radial displacement during pile loading; and R_a =average roughness of the pile.

From Schneider et al. (2008)

UWA-05 Formulation of shaft friction τ_f

$$\tau_f = \frac{f_t}{f_c} \left[0.03 q_c A_{rs,eff}^{0.3} \left[\max \left(\frac{h}{D}, 2 \right)^{-0.5} \right] + \Delta \sigma'_{rd} \right] \tan \delta_f$$

$$A_{rs,eff} \text{ (effective area ratio)} = 1 - \text{IFR} (D_i / D)^2$$

$$A_r \text{ (area ratio)} = 1 - (D_i / D)^2$$

IFR (= $\Delta L_p / \Delta z$). When IFR is not measured, average IFR = $\min[1, (D_i(\text{m}) / 1.5)^{0.2}]$

f_t / f_c = ratio of tension to compression capacity (equal to 1 for compression and 0.75 in tension)

Note: τ_f = local ultimate shaft friction; δ_f = interface friction angle; p_{ref} = 100 kPa = reference stress; L = pile length; z = element depth; h = height above pile tip; $R^* = \text{equivalent pile radius} = (R^2 - R_i^2)^{0.5}$ where R_i = internal pile radius = $D_i / 2$. For noncircular piles, an equivalent circular area is used to assess R^* and D . $\Delta \sigma'_{rd} \approx 4G \Delta y / D$ = change in radial stress during pile loading; $G \approx 185 q_{c1N}^{-0.17}$ = operational level of shear modulus; see Eq. (8b) for calculation of q_{c1N} ; $\Delta y \approx 2R_p \approx 0.02$ mm = radial displacement during pile loading; and R_p = average roughness of the pile.

From Schneider et al. (2008)

CPT based formulations for tip resistance

Methods	End condition	Design equations
Fugro-05	Closed and open ended	$q_{t0.1} / q_{c,avg} = 8.5 (p_{ref} / q_{c,avg})^{0.5} A_r^{0.25}$
ICP-05	Closed ended Open ended	$q_{t0.1} / q_{c,avg} = \text{maximum} [1 - 0.5 \log(D / D_{CPT}), 0.3]$ if $D_i \geq 2.0(D_p - 0.3)$ or $D_i \geq 0.083 q_{c,avg} / p_{ref} D_{CPT}$, D_i in meters, then the pile is "unplugged," and $q_{t0.1} / q_{c,avg} = A_r$ if not, the pile is "plugged," and $q_{t0.1} / q_{c,avg} = \text{maximum} [0.5 - 0.25 \log(D / D_{CPT}), 0.15, A_r]$
NGI-05	Closed ended	$q_{t0.1} / q_{c,tip} = F_{D_i} = 0.8 / (1 + D_i^2)$
NGI-05	Open ended	$q_{t0.1} = \text{minimum} [\text{plugged } q_{t0.1}, \text{unplugged } q_{t0.1}]$ the plugged $q_{t0.1}$ value is calculated as: $q_{t0.1} / q_{c,tip} = F_{D_i} = 0.7 / (1 + 3D_i^2)$ the unplugged $q_{t0.1}$ is calculated as: $q_{t0.1} = q_{b,amr} A_r + q_{b,plug} (1 - A_r)$; $q_{b,amr} = q_{c,tip}$ and $q_{b,plug} = 12 \tau_{f,avg} L / (\pi D_i)$ $\tau_{f,avg}$: averaged external skin friction (Clausen et al. 2005); L : pile embedment depth
UWA-05	Closed and open ended	$q_{t0.1} / q_{c,avg} = 0.15 + 0.45 A_{rs,eff}$

Notes: D : pile outer diameter; D_i : pile inner diameter; p_{ref} = 100 kPa; D_{CPT} = 0.036 m; A_r (area ratio) = $1 - (D_i / D)^2$; $A_{rs,eff}$ (effective area ratio) = $1 - \text{FFR} (D_i / D)^2$; FFR: IFR (= $\Delta L_p / \Delta z$) averaged over last 3D of the pile penetration; $q_{c,avg} = q_c$ averaged $\pm 1.5D$ over pile tip level for Fugro-05 and ICP-05 methods; $q_{c,avg} = q_c$ averaged using Dutch averaging technique for UWA-05 method; $D_i = 0.4 \ln(q_{c1N} / 22)$; as decimal, nominal relative density which may be greater than 1.0 for NGI method.

From Schneider et al. (2008)

Pile bearing capacity using the 4 CPT methods

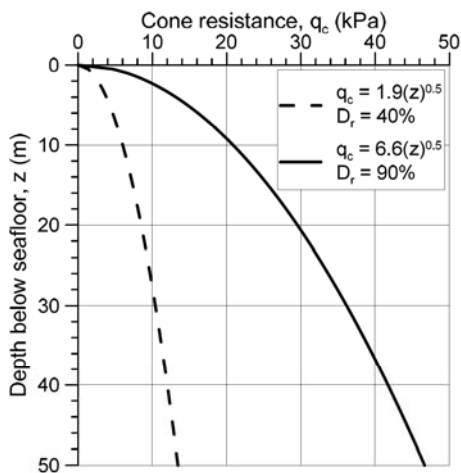
Table 9. Performance of Design Methods for Entire 77 Pile Database

Method	Entire database		
	μ_{gR}	Median	$\sigma_{\ln R}$
API-00	0.76	0.78	0.60
LCPC-82 (French method)	1.23	1.27	0.40
EF-97	1.40	1.36	0.42
Fugro-05	1.04	1.11	0.35
ICP-05	0.91	0.92	0.27
NGI-05	1.01	0.96	0.31
UWA-05	0.92	0.90	0.25

μ_{gR}
Geometric mean
 $\sigma_{\ln R}$
Standard deviation of $\ln(Q_c/Q_m)$

From Schneider et al. (2008)

Comparison of 4 CPT pile bearing capacity methods

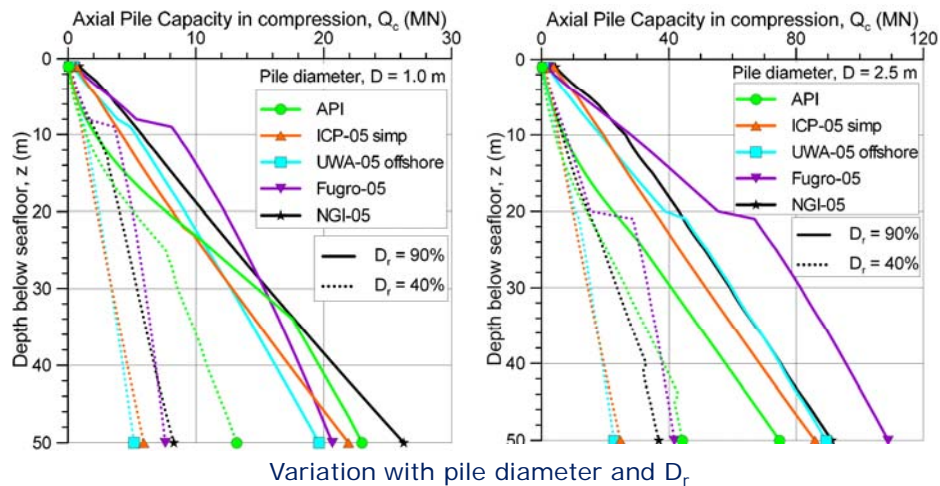


Plugged open-ended steel piles, driven

Figure 1: Idealised sand profiles for parametric study

From paper by: Knudsen, Langford, Lacasse and Aas, 2012

Pile bearing capacity using the 4 CPT methods



From Knudsen et al. (2012)

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 compressibility
- Changes in cone resistance can therefore be used to document effectiveness of compaction

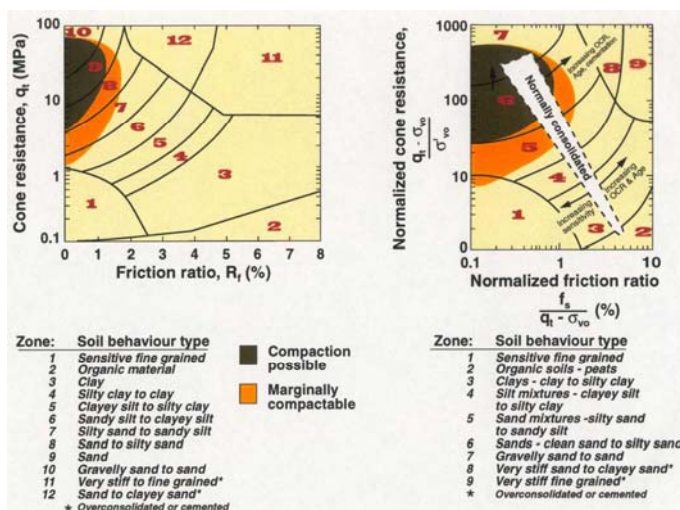
Deep compaction

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

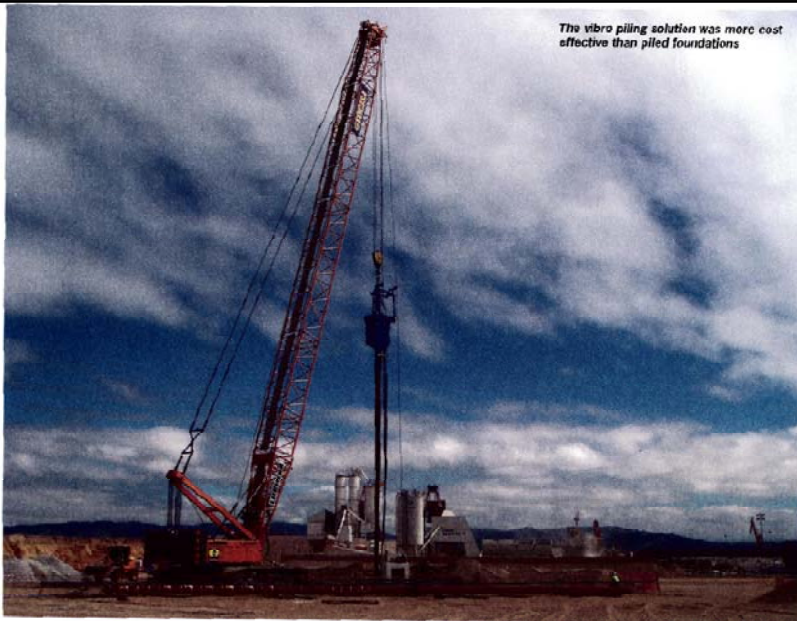
CPT is found to be best method to monitor and document effect of deep compaction

Important to consider time effect

Suitability of soil for vibrocompaction



Massarsch(1994)

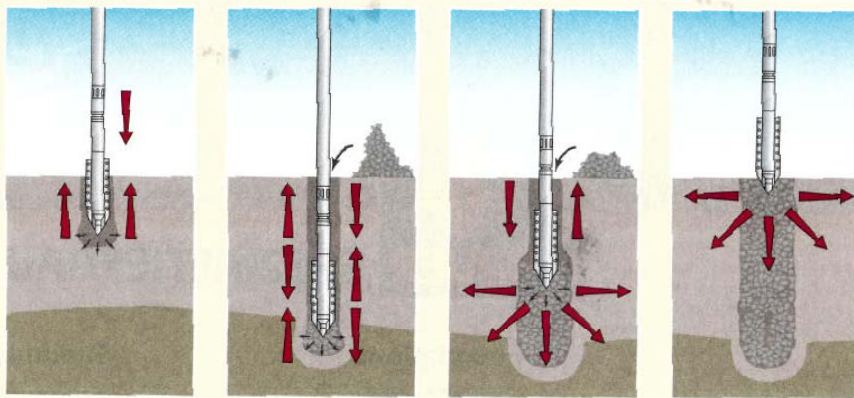


The vibro piling solution was more cost effective than piled foundations

Example vibropiling

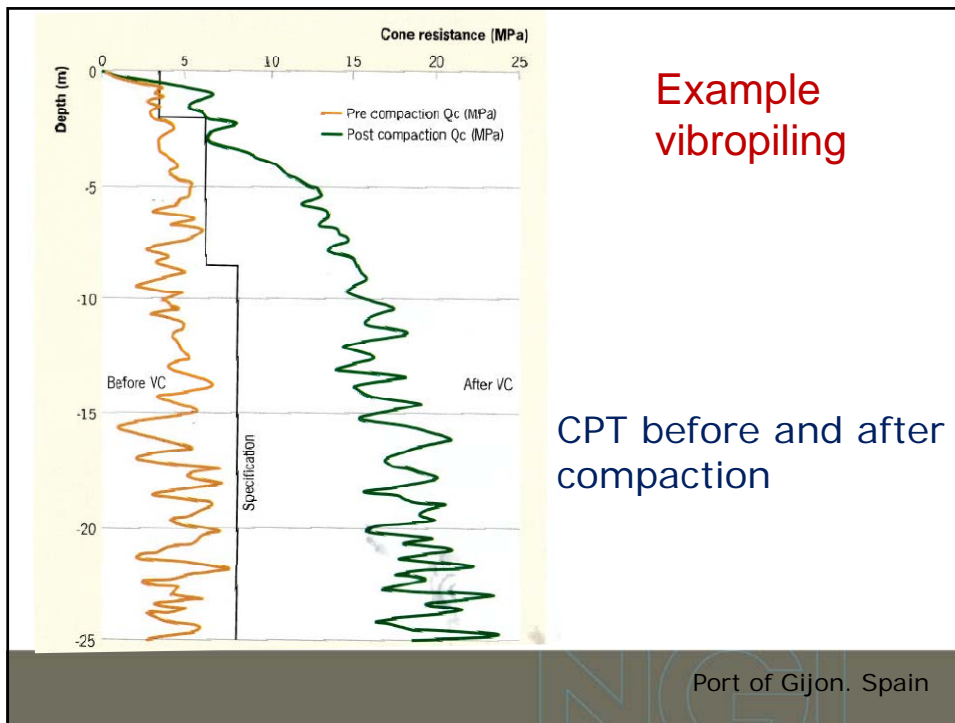
Port of Gijon. Spain

Example vibropiling



The vibroflot is penetrated to the required design depth before being slowly extracted, with pauses at regular intervals to ensure satisfactory levels of compaction are achieved at each depth. The vibroflot is withdrawn to the surface where a zone of compacted ground is formed around the insertion point.

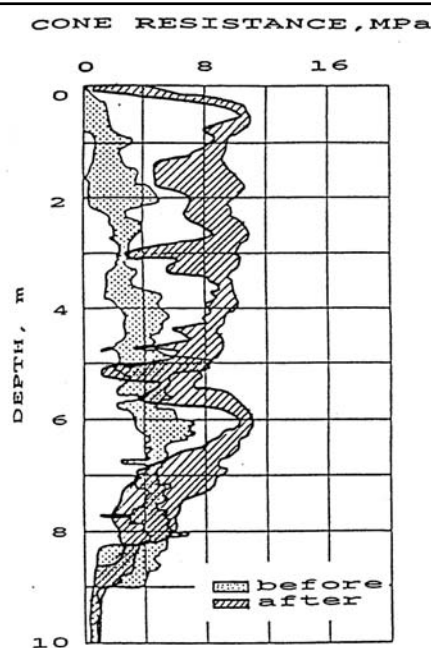
Port of Gijon. Spain



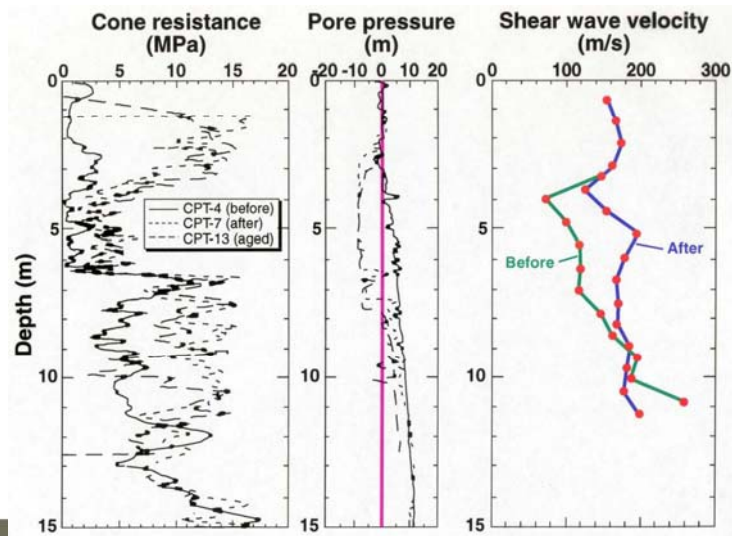
Compaction control

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

Lindberg and Massarsch(1991)



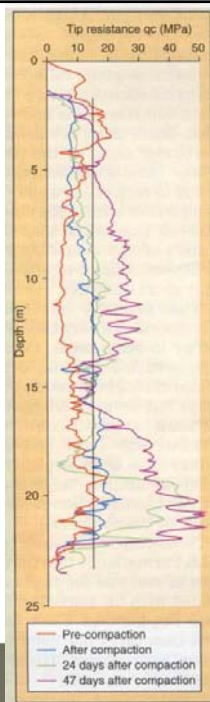
Effect penetration resistance after dynamic compaction



From Woeller et al. (1995)

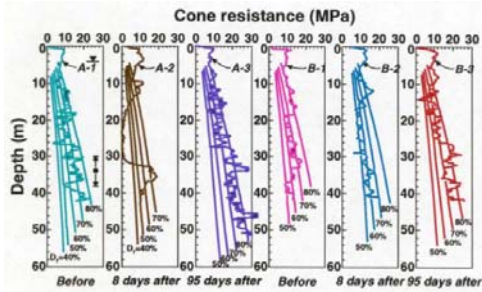
The aging effects of sands

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



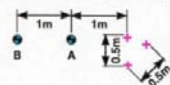
From Ng, Berner and Covil (1996)

Compaction by blasting



Effect of time

Test layout C
(3 holes - 3.3kg per hole)



LEGEND:

A-1, B-1 1979, December 20 + Blast hole
 Blasting December 21 ● CP test hole
 A-2, B-2 December 29 ▼ water table
 A-3, B-3 1980, March 25

From Mitchell and Solymar(1984)

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

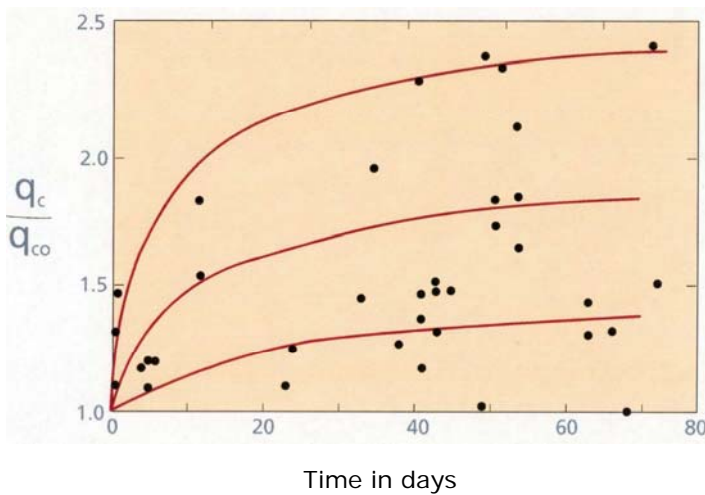


Diagram developed for correcting cone resistance measured just after compaction – large project in Florida

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
- Results of CPTs before and after compaction may be used to evaluate increase in D_r

CPTU data can be used to evaluate if compaction will be efficient or not (ref. soil behaviour chart)

Liquefaction resistance

- Major concern for structures constructed with or on sand and sandy silt.
- Cyclic loads from : earthquakes, wave loading, machine foundations and other
- Effect is to increase pore pressure causing loss of effective stress and soil strength

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

CPT/CPTU ideal because of its repeatability, 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

SCPTU is seismic cone: can measure shear wave velocity, v_s

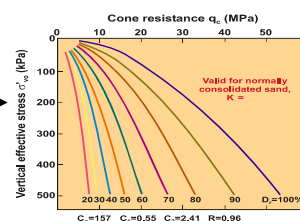
Liquefaction evaluation based on CPT/CPTU results

Different approaches :

1. a) Estimate D_r from q_c, σ_{v0}', 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_{v0}')

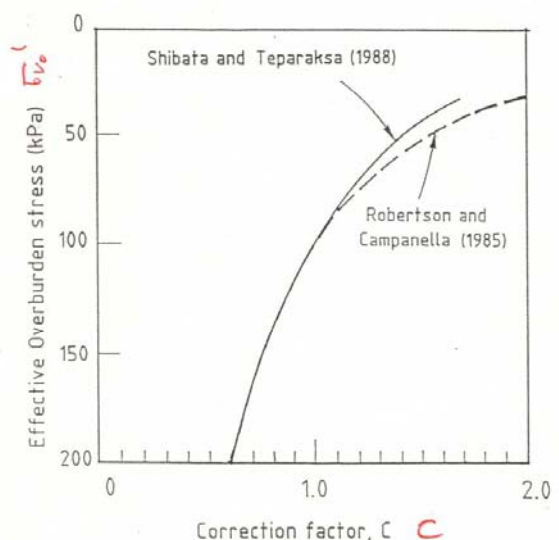
2. Estimate directly from CPT/CPTU results using empirical methods developed in North America and Japan



Liquefaction potential directly from CPT/CPTU results

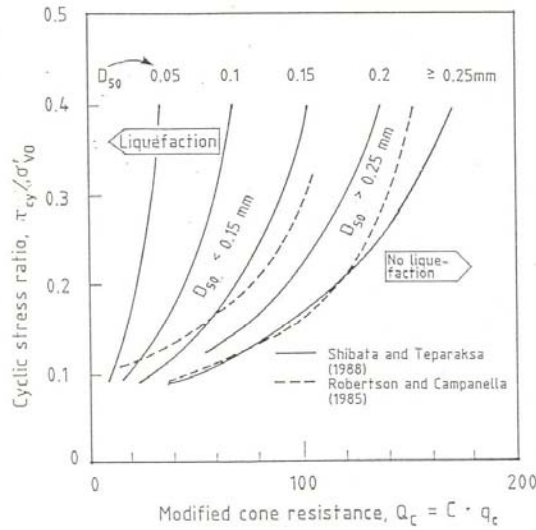
1. Correct q_c for overburden stress effect $Q_c = C \cdot q_c$
2. Estimate average cyclic stress ratio (due to wave loading or earthquake or other source) τ_{cy} / σ_{vo}'
3. Establish D_{50} by grain size analysis on obtained samples - or estimate from CPT/CPTU results using soil classification charts
4. Check liquefaction by τ_{cy} / σ_{vo}' , Q_c , D_{50} diagram

Liquefaction potential directly from CPT/CPTU results



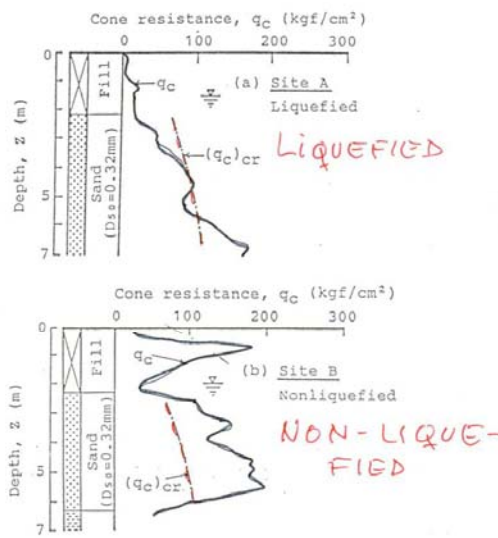
Correction factor for cone resistance to predict liquefaction potential of sand (from Shibata and Teparaksa, 1988)

Liquefaction potential directly from CPT/CPTU results

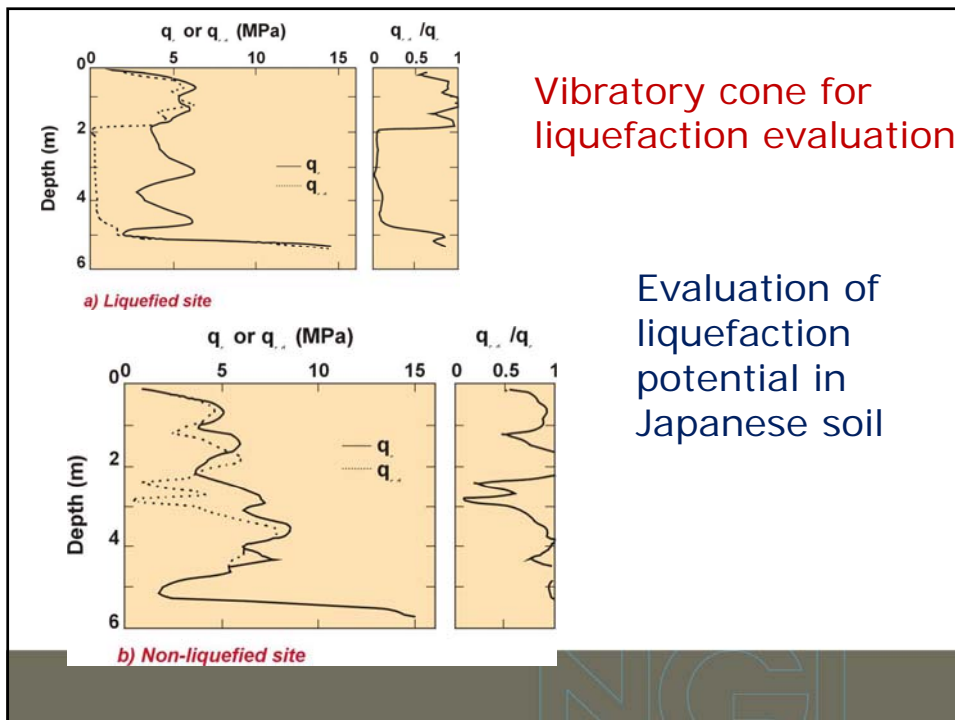
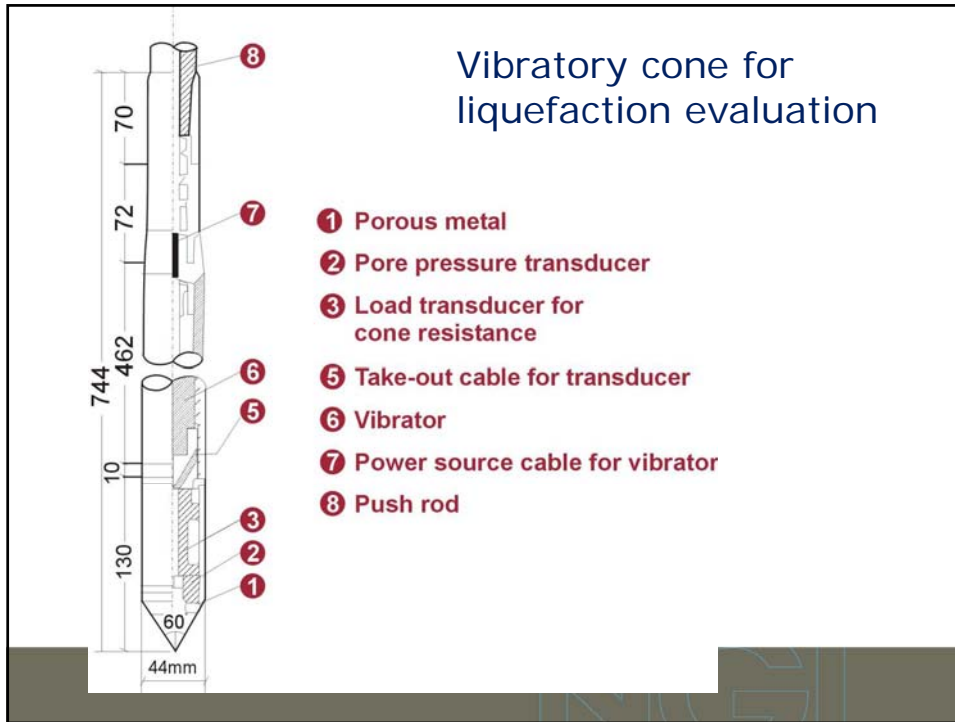


Liquefaction potential from cone resistance (after Shibata and Teparaksa, 1988)

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)



CPTU and evaluation of liquefaction

Detailed methodology given by:

Robertson, P.K. and Wride, C.E. (1998)
Evaluating cyclic liquefaction potential using the cone penetration test.
Canadian Geotechnical Journal, 35(3): 442 – 459.

Robertson, P.K. (2010)
Evaluation of flow liquefaction and liquefied strength using the cone penetration test.
Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 136(6):842 – 853.

SUMMARY: 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

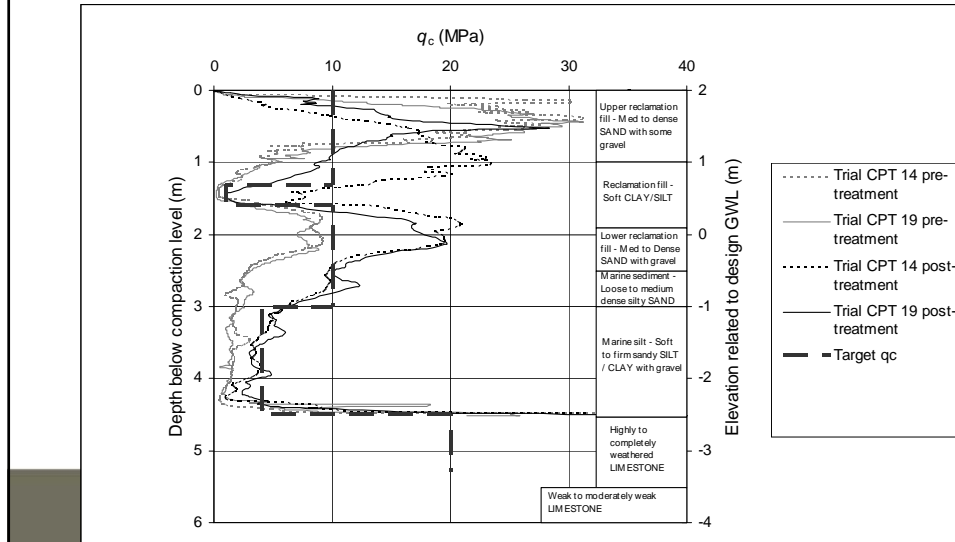
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Rapid impact compactor

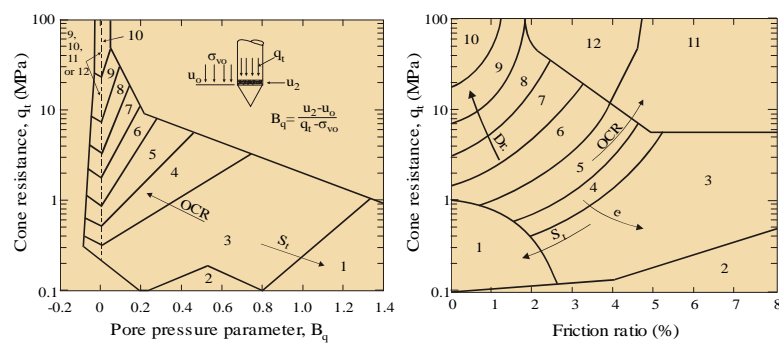


INGI

Effect of rapid impact compactor



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



Zone: Soil Behaviour Type:

- | | | |
|---------------------------|------------------------------|------------------------------|
| 1. Sensitive fine grained | 5. Clayey silt to silty clay | 9. Sand |
| 2. Organic material | 6. Sandy silt to clayey silt | 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* |

* Overconsolidated or cemented.

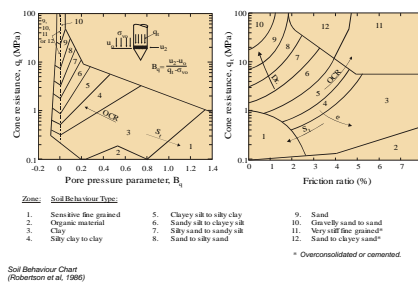
Soil Behaviour Chart
(Robertson et al, 1986)

Robertson et al., 1986

SOIL CLASSIFICATIONS AND RATIOS

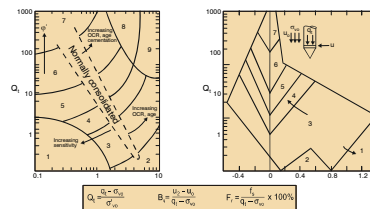
Zone	Soil behavior type	(q _c /p _a)/N ₆₀
1	Sensitive fine grained	2
2	Organic material	1
3	clay	1
4	Silty clay to clay	1.5
5	clayey silt to silty clay	2
6	Sandy silt to clayey silt	2.5
7	Silty sand to sandy silt	3
8	Sand to silty sand	4
9	sand	5
10	Gravelly sand to sand	6
11	Very stiff fine grained	1
12	Sand to clayey sand	2

Zone refers to Soil
Behaviour type diagram



BOUNDARIES OF SOIL BEHAVIOUR TYPE INDEX

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 mixturees – clayey silts to silty clay
2.95 < I _c < 3.60	3	Clays
I _c < 3.06	2	Organic soils - peat



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

Zone Soil behaviour type Zone Soil behaviour type Zone Soil behaviour type
 1. Sensitive fine grained 4. Silty sand to clayey silt to silty clay 7. Gravilly sand to sand
 2. Organic soils-peats 5. Sand mixturees, silty sand to sand silty 8. Very stiff sand to clayey sand
 3. Clayey clay to silty clay 6. Clean, clean sand to silty sand 9. Very stiff fine grained