BASICS OF DESIGN OF PILED FOUNDATIONS

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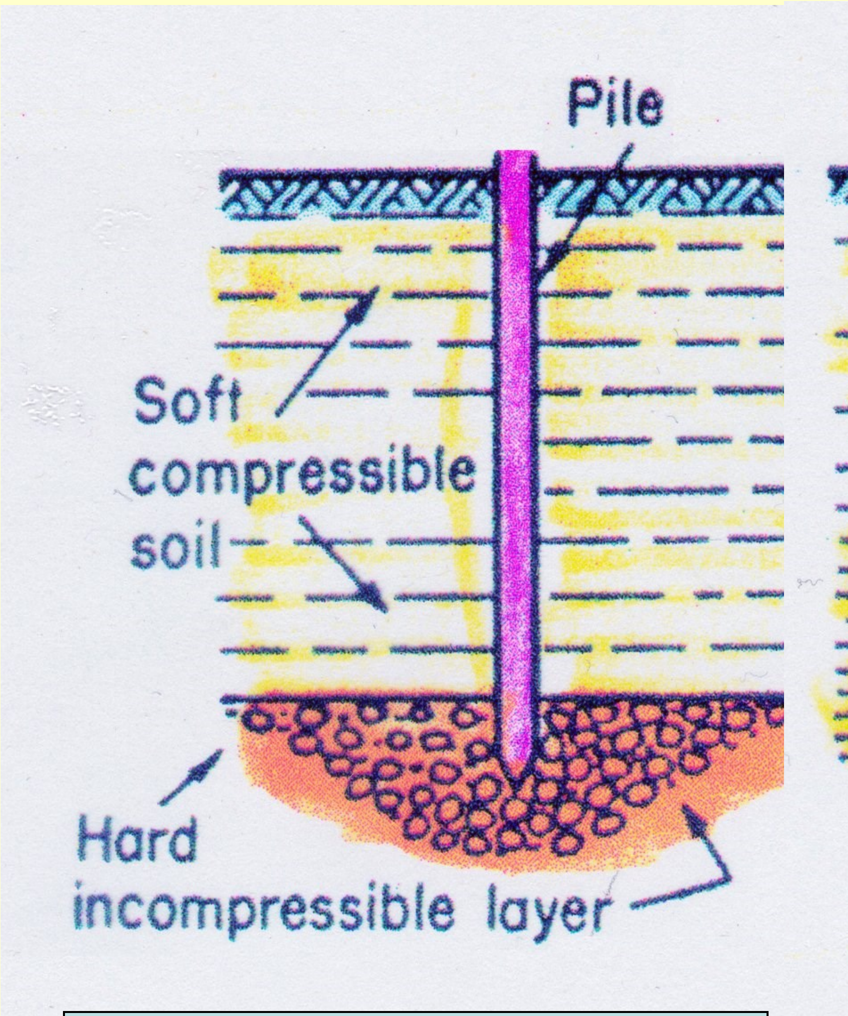
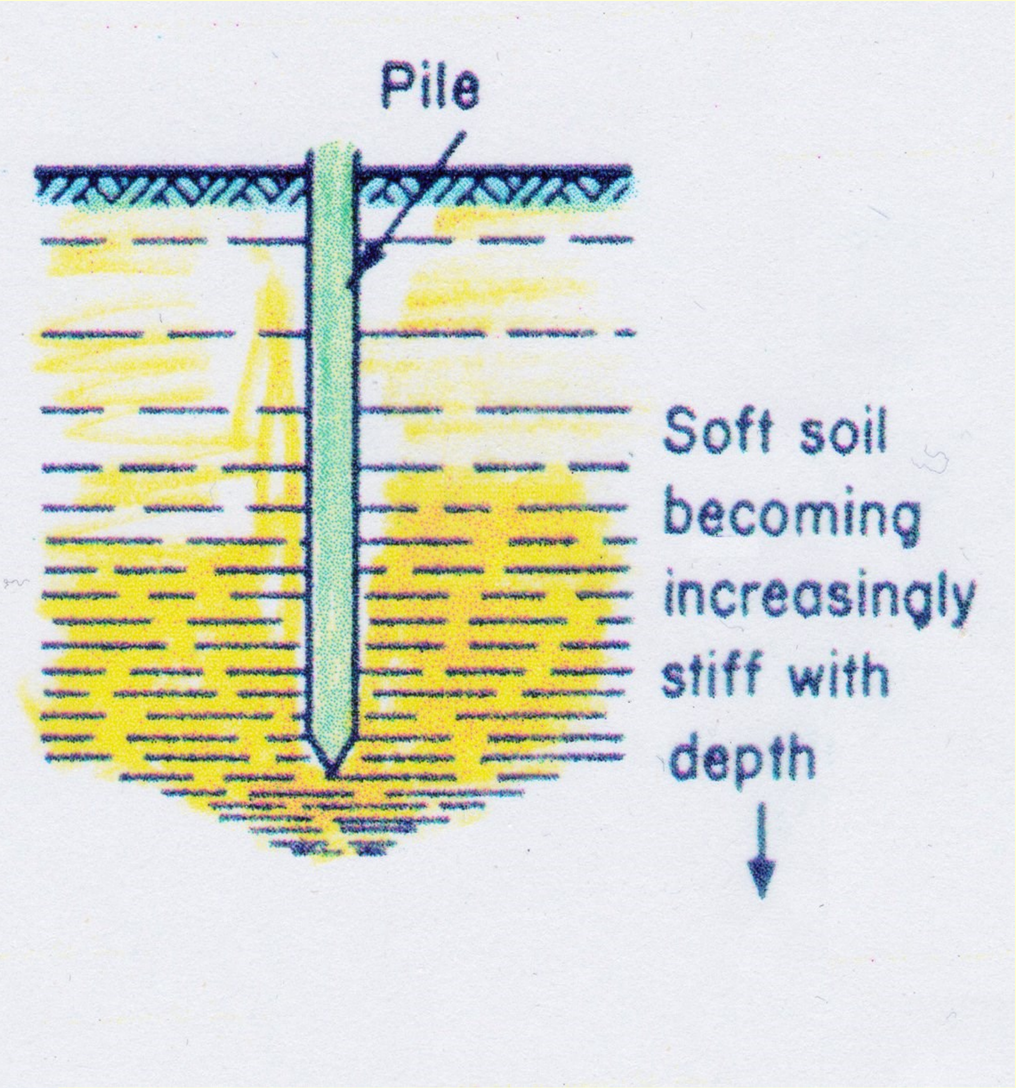
Background and Basic Principles

Brisbane November 28, 2008

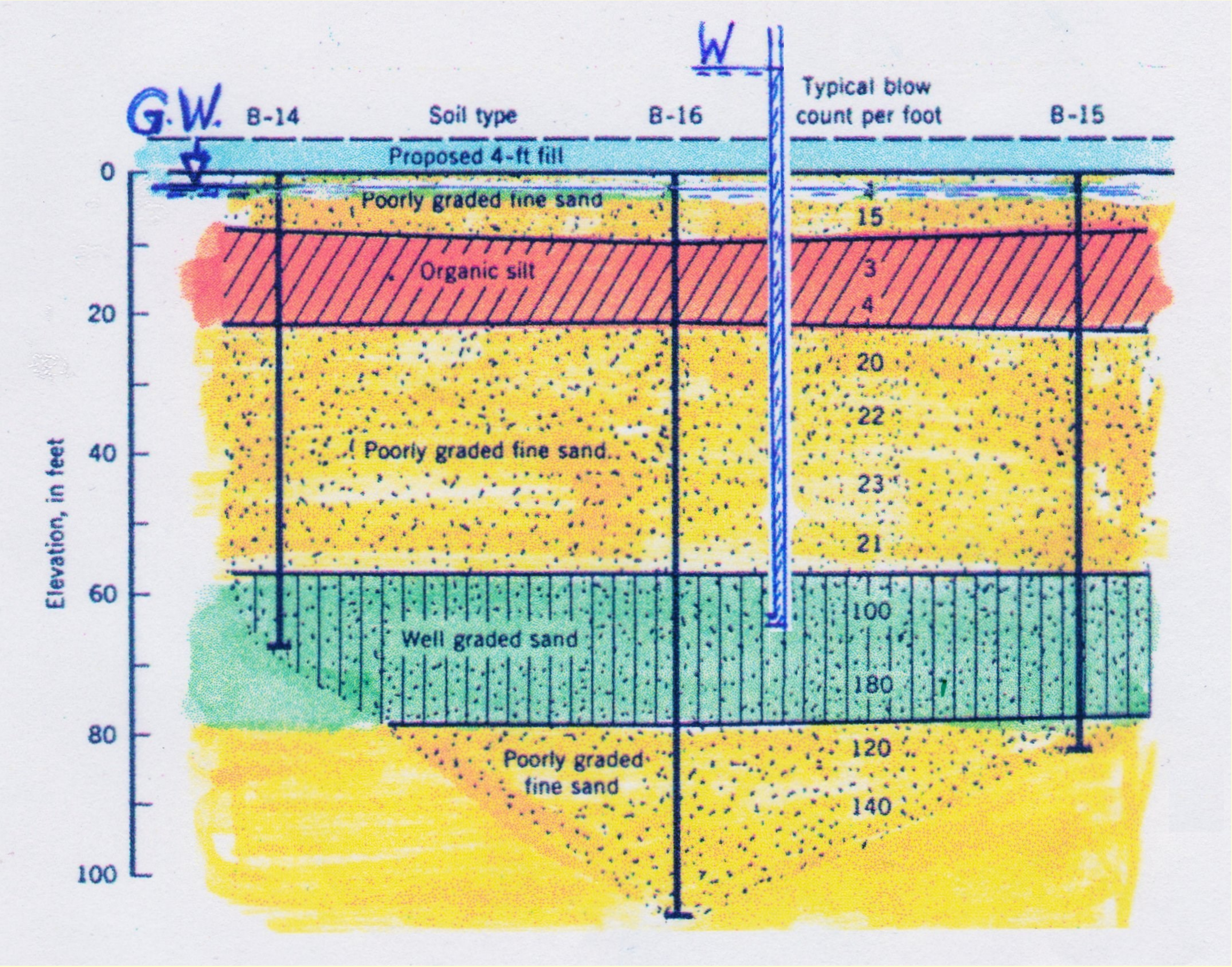
The Textbooks — They come no better



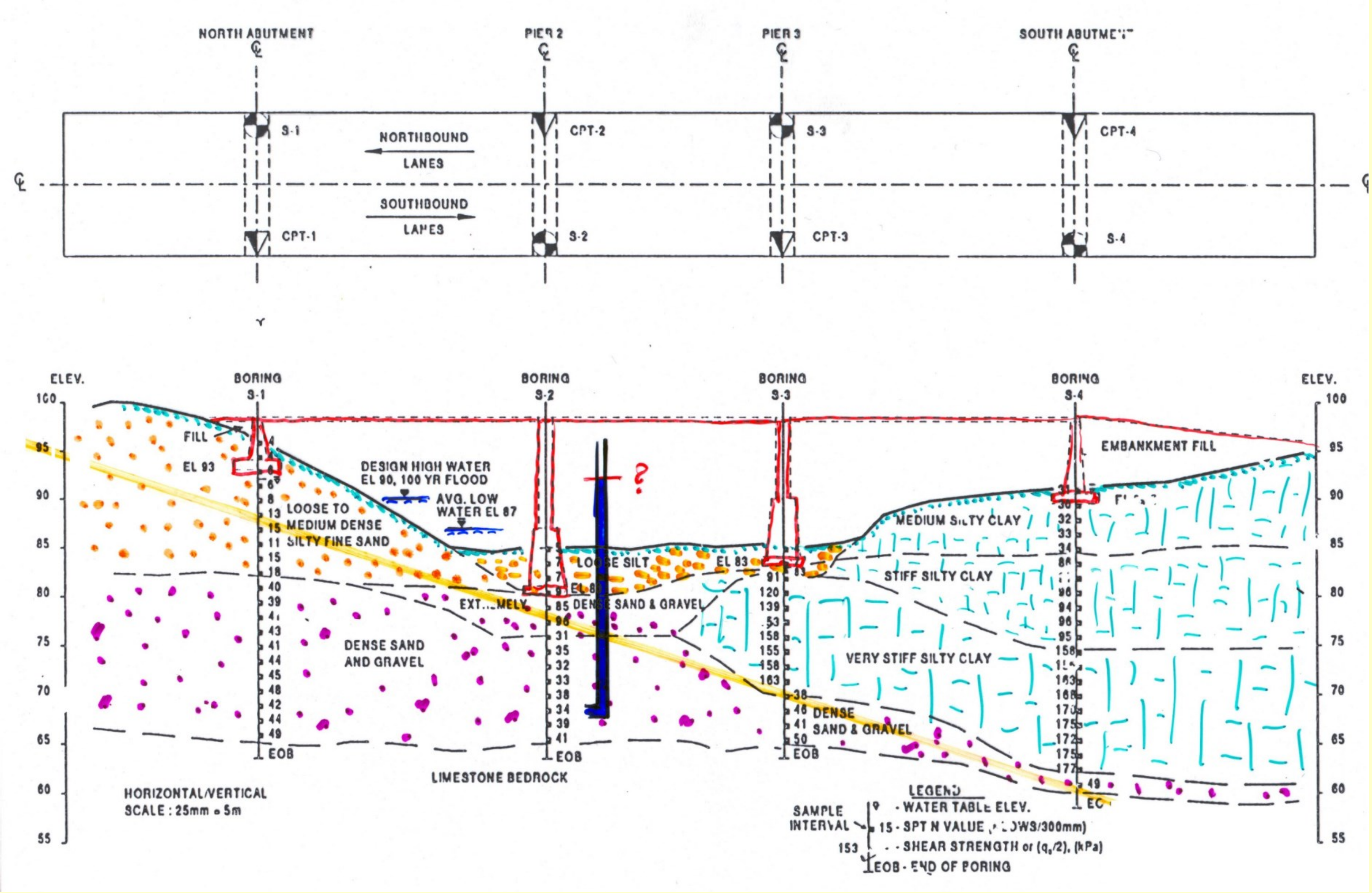


This is one of the few showing more than one soil layer

The Reality — With a bit of needed "W" add-on



The Reality — Getting closer, at least



Movement, Settlement, and Creep

Movement occurs as a result of an increase of stress, but the term should be reserved to deformation due to *increase of total stress*. Movement is the result of a transfer of stress to the soil (the movement occurs as necessary to build up the resistance to the load), and when the involved, or influenced, soil volume successively increases as the stress increases. For example, when adding load increments to a pile or to a plate in a static loading test (where, erroneously, the term "settlement" is often used). As a term, movement is used when the involved, or influenced, soil volume increases as the load increases.

Settlement is volume reduction of the subsoil as a consequence of an *increase in effective stress*. It consists of the sum of "elastic" compression and deformation due to consolidation. The elastic compression is the compression of the soil grains (soil skeleton) and of any free gas present in the voids. The elastic compression is often called "immediate settlement". It occurs quickly and is normally small (the elastic compression is not associated with expulsion of water). The deformation due to consolidation, on the other hand, is volume change due to the compression of the soil structure associated with an expulsion of water**—**consolidation. In the process, the imposed stress, initially carried by the pore water, is transferred to the soil structure. Consolidation occurs quickly in coarse-grained soils, but slowly in fine-grained soils. As a term, settlement is used when the involved, or influenced, soil volume stays constant as the effective stress increases.

Creep is compression occurring *without an increase of effective stress*. Creep is usually small, but may in some soils add significantly to the compression of the soil skeleton and, thus, to the total deformation of the soil. It is then acceptable to talk in terms of creep settlement.

When the deformation is due to a combined effect of load transfer, increase of effective stress, and creep during long-term conditions, the term "settlement" is normally used.

So, with this food for thought, on to the Fundamental Principles

Determining the effective stress is the key to geotechnical analysis

- *The not-so-good method:*

$$\Delta\sigma' = \gamma'\Delta h$$

γ' = buoyant unit weight

$$\sigma'_z = \sum(\gamma'\Delta h)$$

$$\gamma' = \gamma_t - \gamma_w(1-i)$$

It is much better to determine, separately, the total stress and the pore pressure. The effective stress is then the total stress minus the pore pressure.

$$\sigma_z = \sum(\gamma \Delta h)$$

$$\sigma'_z = \sigma - u$$

Determining pore pressure

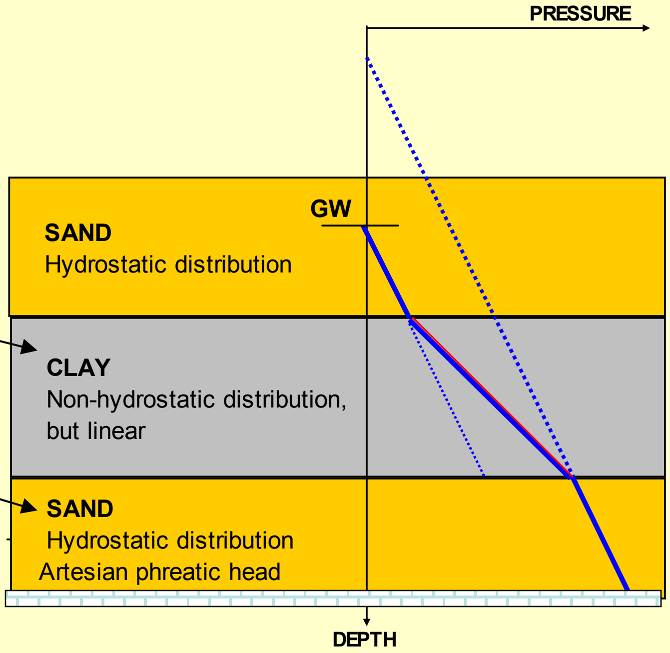
$$u = \gamma_w h$$

The height of the column of water (**h**; the head representing the water pressure) is usually not the distance to the ground surface nor, even, the distance to the groundwater table. For this reason, the height is usually referred to as the "*phreatic height*" or the "*piezometric height*" to separate it from the depth below the groundwater table or depth below the ground surface.

The pore pressure distribution is determined by applying the facts that

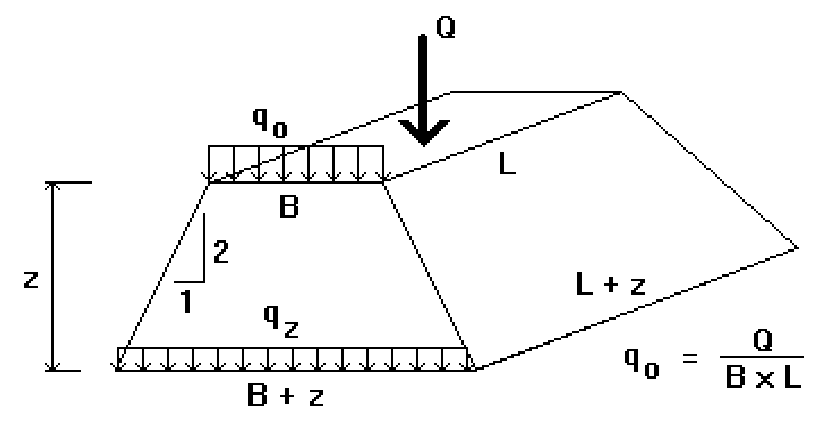
(1) in stationary conditions, the pore pressure distribution can be assumed to be linear in each individual soil layer

(2) in **pervious** soil layers that are "sandwiched" between less pervious layers, the pore pressure is **hydrostatic** (that is, the vertical gradient is unity)



Distribution of stress below a a small load area

The 2:1 method

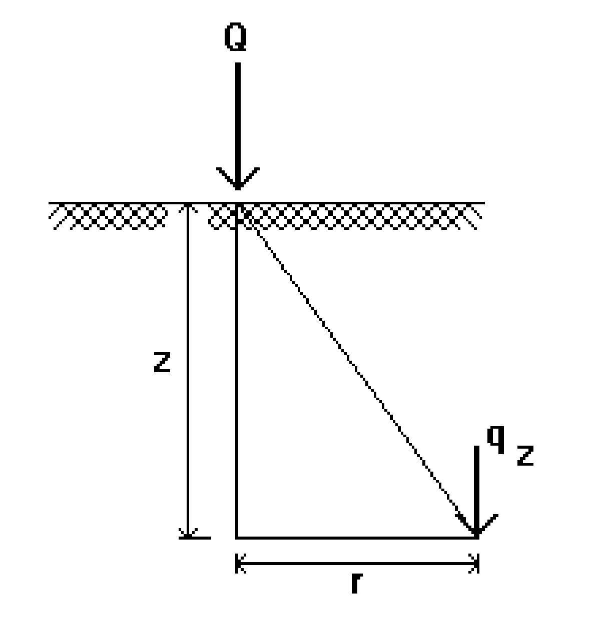


$$q_z = \quad q_0 \times \frac{B \times L}{(B+z)\times(L+z)}$$

The 2:1-method can only be used for distributions directly under the center of the footprint of the loaded area. It cannot be used to combine (add) stresses from adjacent load areas unless they all have the same center. it is then only applicable under the area with the smallest footprint.

The Boussinesq Method

Derived from calculation of stress from a point load on the surface of an elastic medium



$$q_z = Q \frac{3z^3}{2\pi\,(r^2 + z^2)^{5/2}}$$

Newmark's method for stress from a loaded area

Newmark (1935) integrated the Boussinesq equation over a finite area and obtained a relation for the stress under the **corner of a uniformly loaded rectangular** area, for example, a footing

(1) $$q_z = q_0 \times I = \frac{A \times B + C}{4\pi}$$

$$A = \frac{2mn\sqrt{m^2 + n^2 + 1}}{m^2 + n^2 + 1 + m^2 n^2}$$

$$B = \frac{m^2 + n^2 + 2}{m^2 + n^2 + 1}$$

$$C = \arctan\left[\frac{2mn\sqrt{m^2 + n^2 + 1}}{m^2 + n^2 + 1 - m^2 n^2}\right]$$

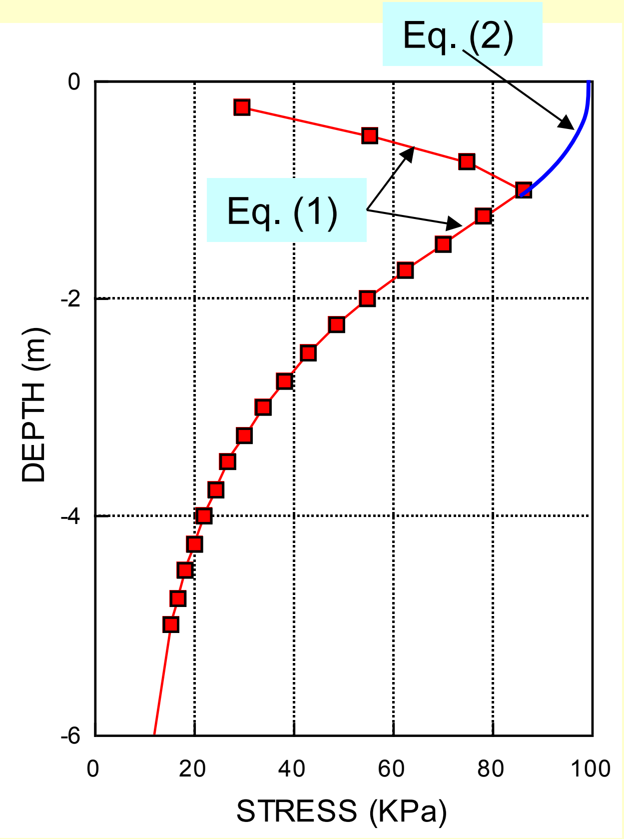
m = x/z
n = y/z
x = length of the loaded area
y = width of the loaded area
z = depth to the point under the corner where the stress is calculated

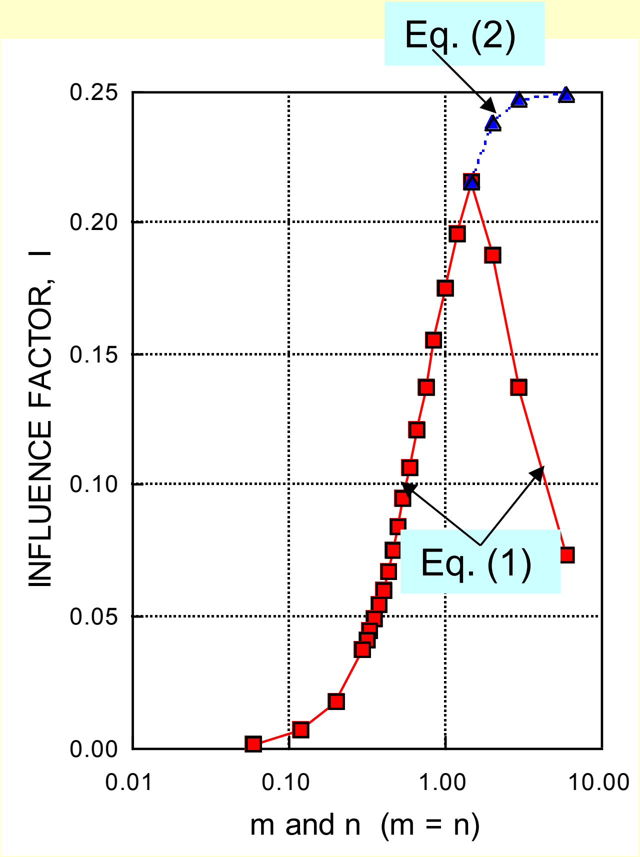
- Eq. 1 does not result in correct stress values near the ground surface. To represent the stress near the ground surface, Newmark's integration applies an additional equation:

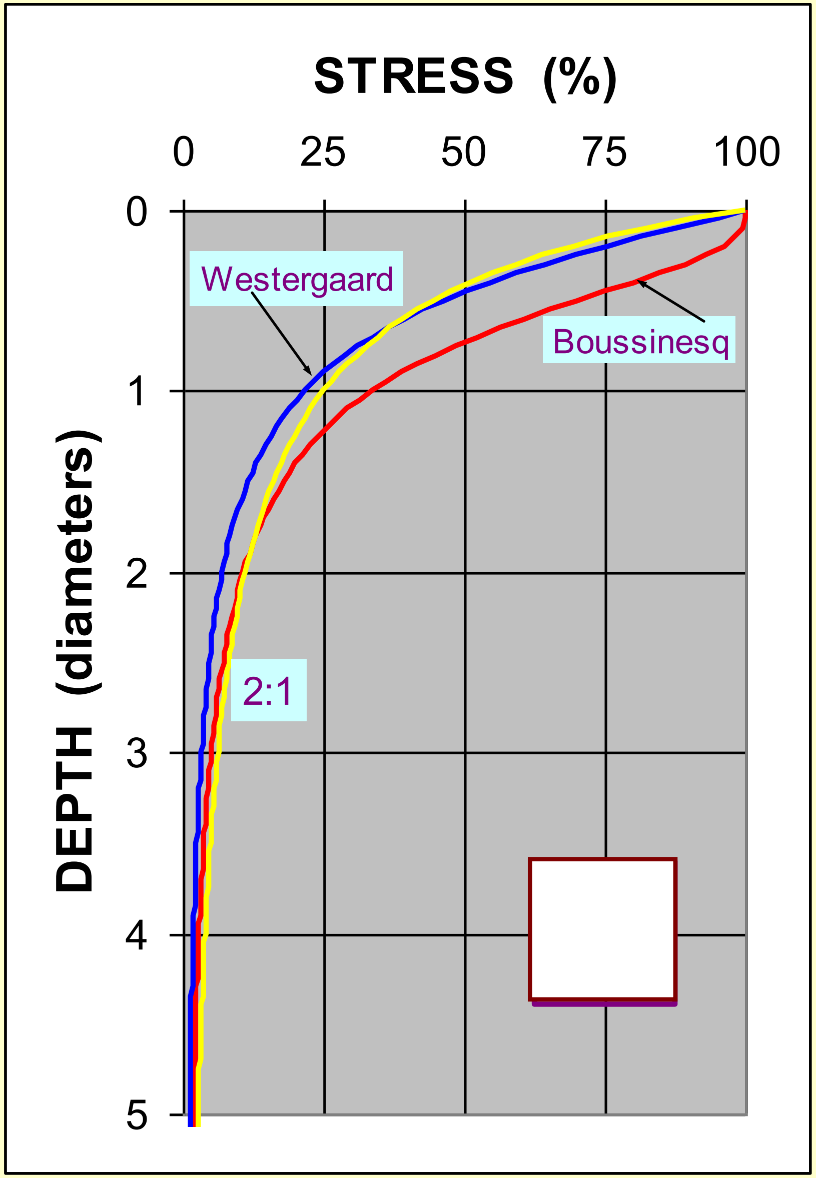
(2) $$q_z = q_0 \times I = \frac{A \times B + \pi - C}{4\pi}$$

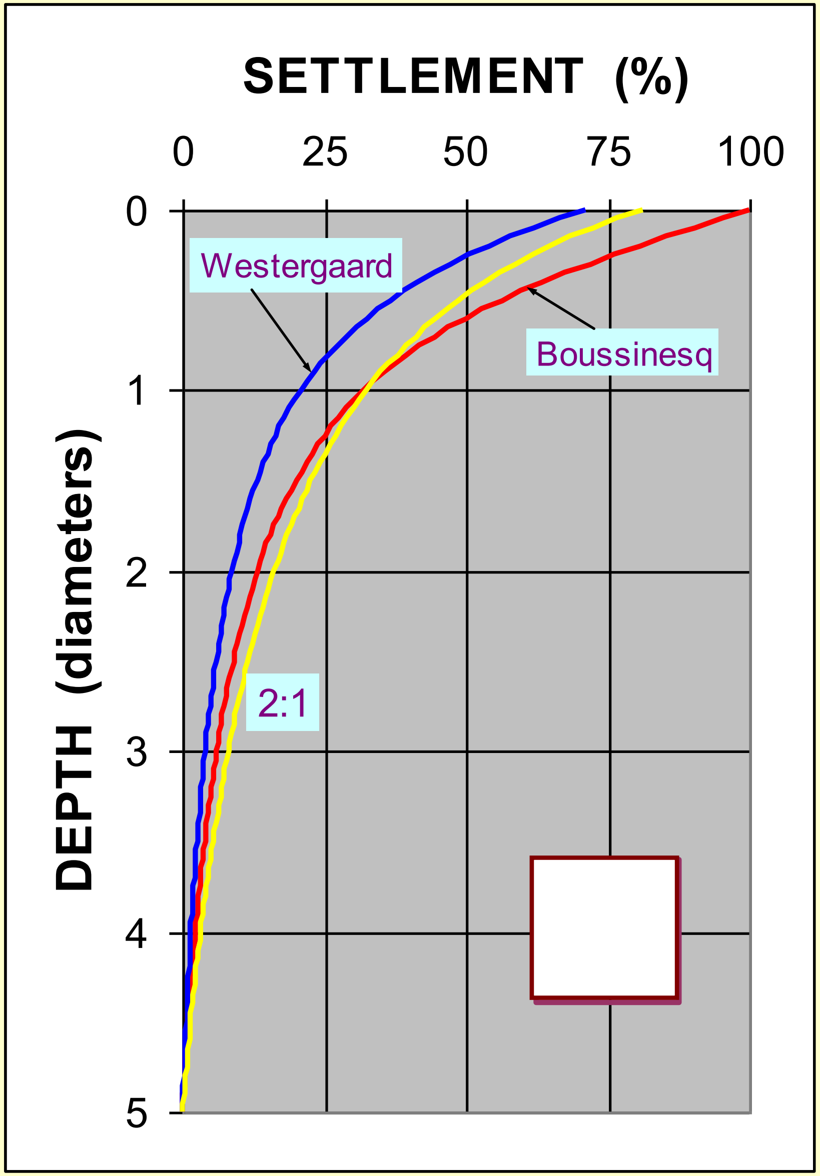
For where: $\mathbf{m^2 + n^2 + 1 \leq m^2 n^2}$

Stress distribution below the center of a square 3 m wide footing

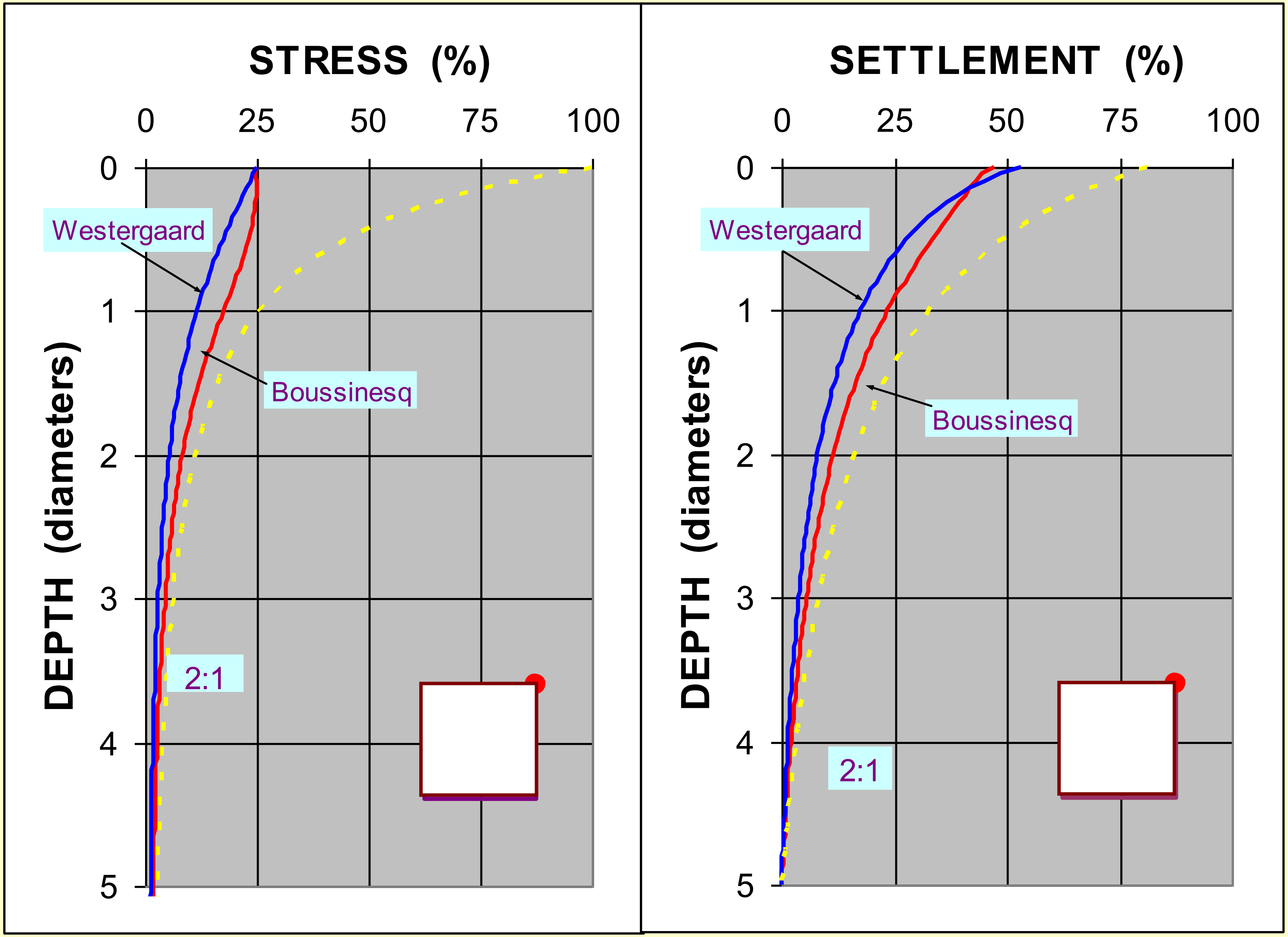


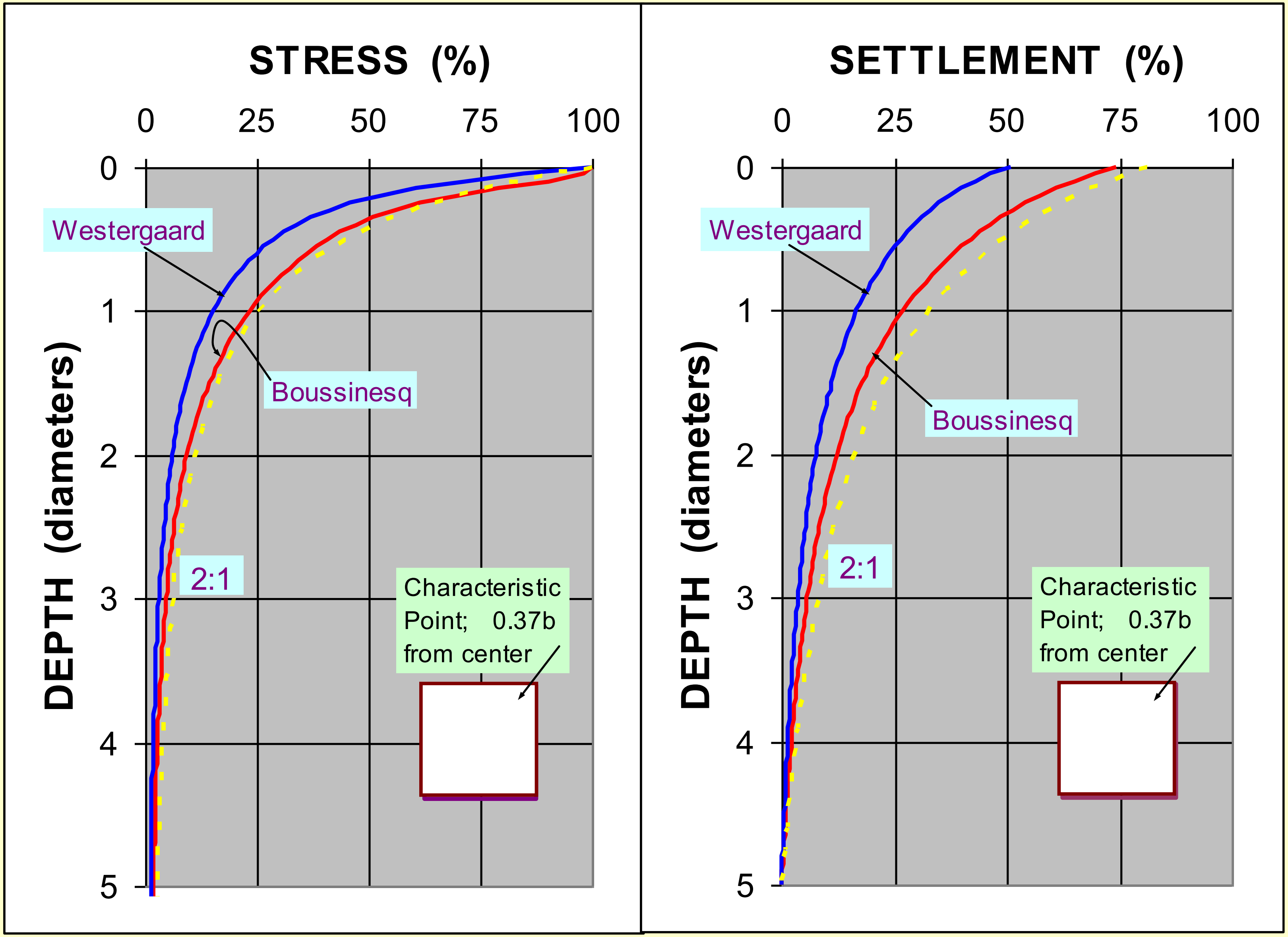






Comparison between Boussinesq, Westergaard, and 2:1 distributions


STRESS (%)
0
25
50
75
100
0
1
2
3
4
5
DEPTH (diameters)
Westergaard
Boussinesq
2:1
SETTLEMENT (%)
0
25
50
75
100
0
1
2
3
4
5
DEPTH (diameters)
Westergaard
Boussinesq
2:1

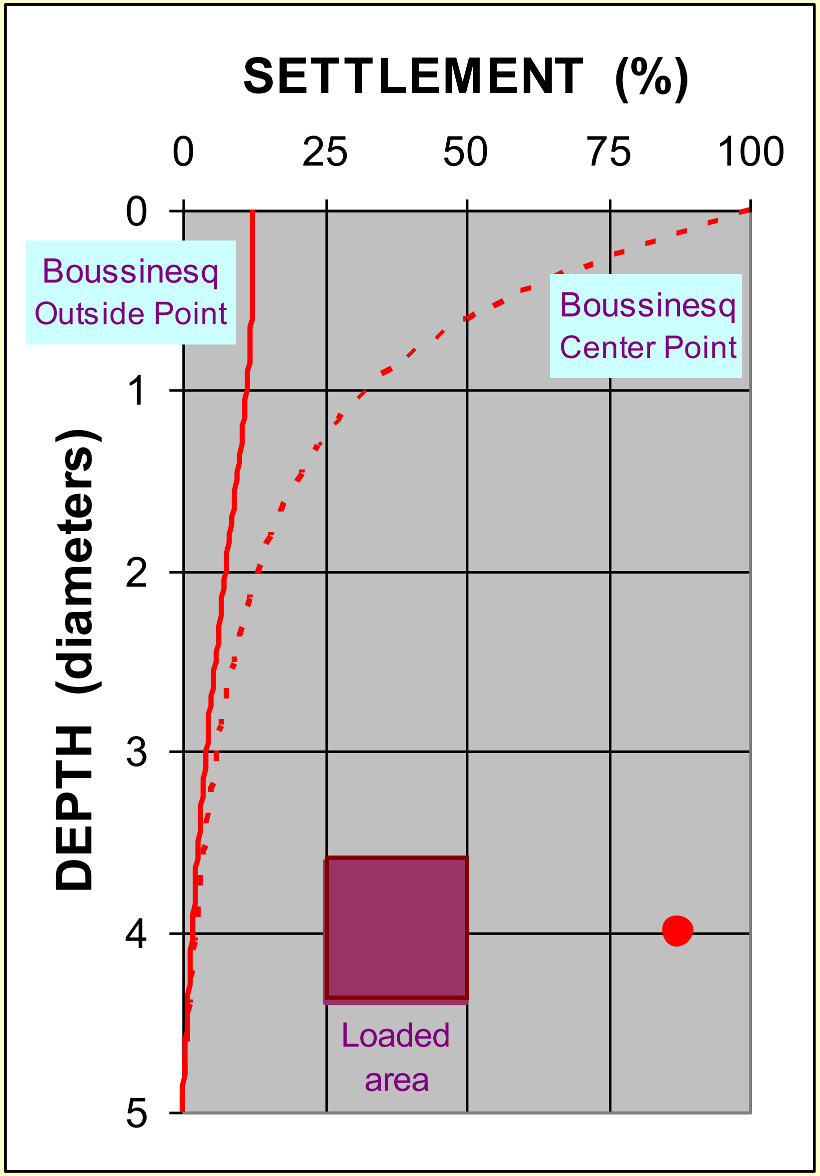


Below the characteristic point, stresses for flexible and stiff footings are equal

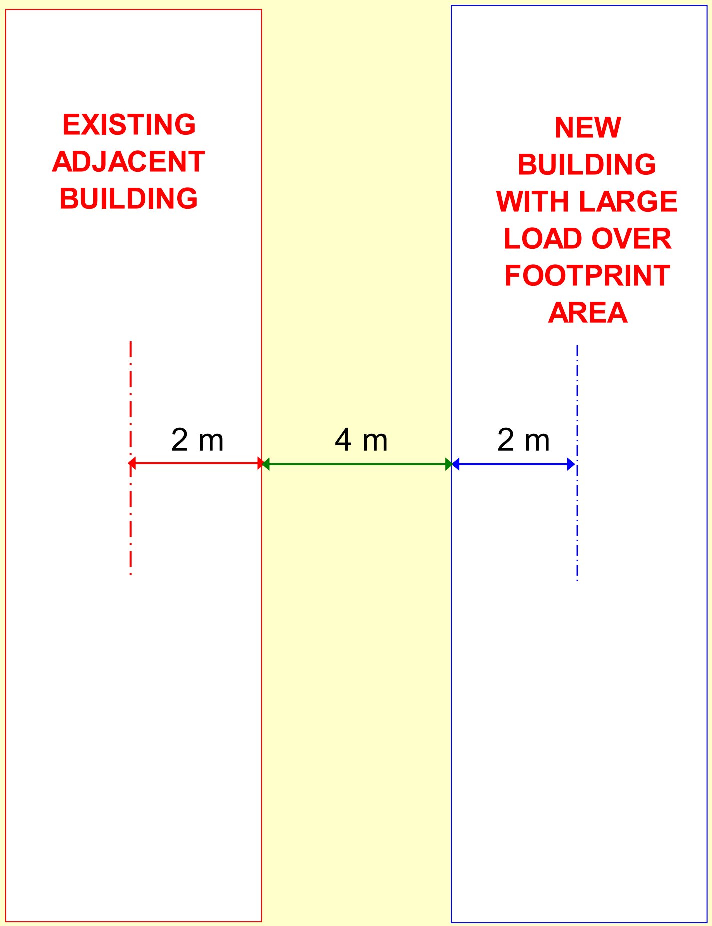
Now, if the settlement distributions are so similar, why do we persist in using Boussinesq stress distribution instead of the much simpler 2:1 distribution?

Because a footing is not alone in this world; near by, there are other footings, and fills, and excavation, etc., for example:

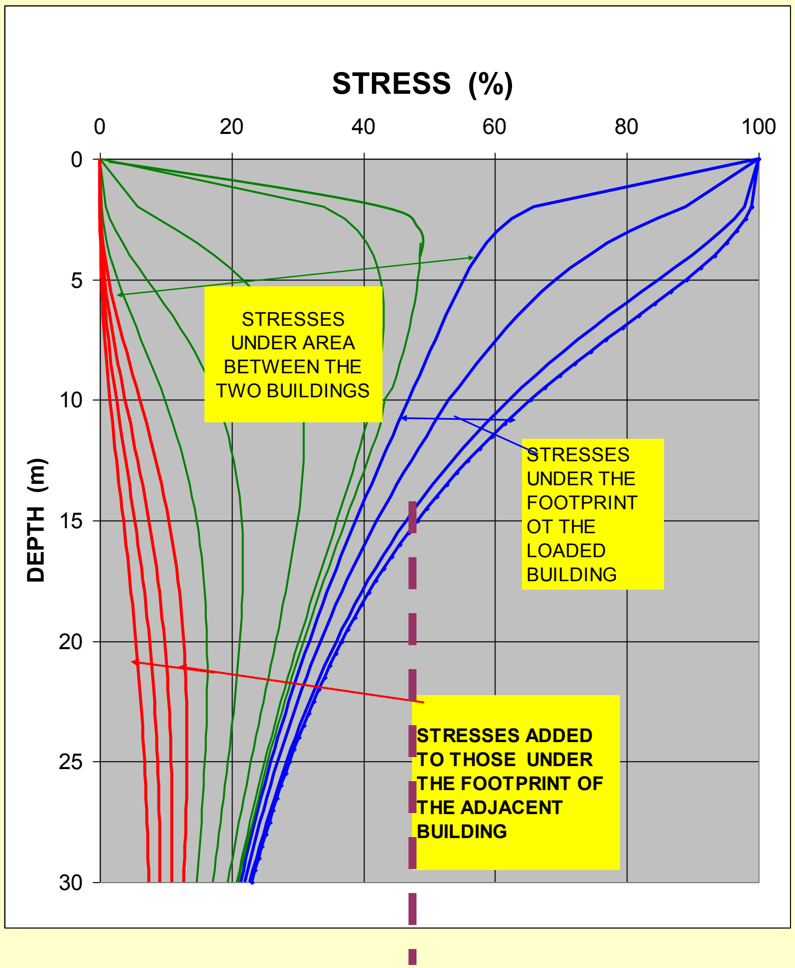
The settlement imposed outside the loaded foundation is often critical



Calculations using Boussinesq distribution can be used to determine how stress applied to the soil from one building may affect an adjacent existing building (having the same loading as the new building).



The soils consist of slightly preconsolidated moderately compressible silt and clay



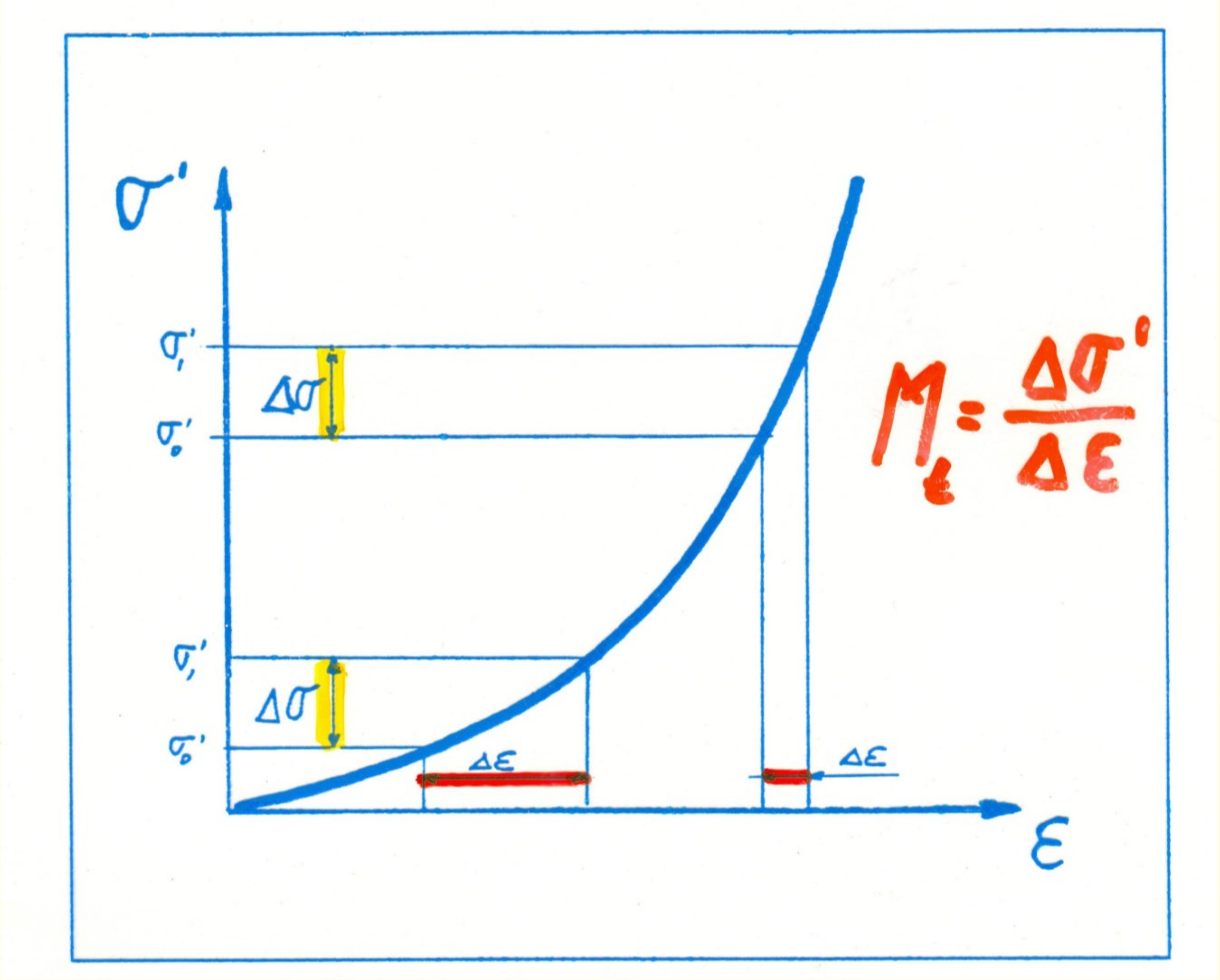
Calculations by means of UniSettle

The end result of a geotechnical design analysis

is

Settlement

Stress-Strain



Stress-strain behavior is non-linear for most soils. The non-linearity cannot be disregarded when analyzing compressible soils, such as silts and clays, that is, the elastic modulus approach is not appropriate for these soils.

Non-linear stress-strain behavior of compressible soils, is conventionally modeled as follows.

$$\varepsilon = \frac{C_c}{1+e_0} \lg \frac{\sigma'_1}{\sigma'_0} \qquad = \quad CR \lg \frac{\sigma'_1}{\sigma'_0}$$

where ε = strain induced by increase of effective stress from σ'_0 to σ'_1
C_c = compression index
e_0 = void ratio
σ'_0 = original (or initial) effective stress
σ'_1 = final effective stress
CR = Compression Ratio = $CR = \frac{C_c}{1+e_0}$ (MIT)

In overconsolidated soils (most soils are)

$$\varepsilon = \frac{1}{1+e_0}\left(C_{cr}\ \lg\frac{\sigma'_p}{\sigma'_0} \quad + \quad C_c\ \lg\frac{\sigma'_1}{\sigma'_p}\right)$$

where σ'_p = preconsolidation stress
C_{cr} = re-compression index

The Janbu Method

The Janbu tangent modulus approach, proposed by Janbu (1963; 1965; 1967; 1998), and referenced by the Canadian Foundation Engineering Manual, CFEM (1985; 1992), applies the same basic principles of linear and non-linear stress-strain behavior. The method applies to all soils, clays as well as sand. By this method, the relation between stress and strain is a function of two non-dimensional parameters which are unique for a soil: a stress exponent, j, and a modulus number, m.

Janbu's general relation is

$$\varepsilon = \frac{1}{mj}\left[\left(\frac{\sigma'_1}{\sigma'_r}\right)^j - \left(\frac{\sigma'_0}{\sigma'_r}\right)^j\right]$$

where: σ'_r = a "reference stress = 100 KPa

j = a stress exponent

m = the modulus number

The Janbu Method

Dense Coarse-Grained Soil **j = 1**

$$\varepsilon = \frac{1}{m}(\sigma'_1 - \sigma'_0) = \frac{1}{m}\Delta\sigma'$$ σ' in KPa

$$\varepsilon = \frac{1}{2m}(\sigma'_1 - \sigma'_0) = \frac{1}{2m}\Delta\sigma'$$ σ' in ksf

Cohesive Soil **j = 0**

$$\varepsilon = \frac{1}{m}\ln\frac{\sigma'_1}{\sigma'_0}$$

Sandy or Silty Soils **j = 0.5**

$$\varepsilon = \frac{1}{5m}(\sqrt{\sigma'_1} - \sqrt{\sigma'_0})$$ σ' in KPa

$$\varepsilon = \frac{\sqrt{2}}{m}(\sqrt{\sigma'_1} - \sqrt{\sigma'_p}$$ σ' in ksf

There are direct mathematical conversions between m and the E and C_c-e_0

For E given in units of KPa (and ksf), the relation between the modulus number and the E-modulus is

m	=	E/100	(KPa)
m	=	E/2	(ksf)

For C_c-e_0, the relation to the modulus number is

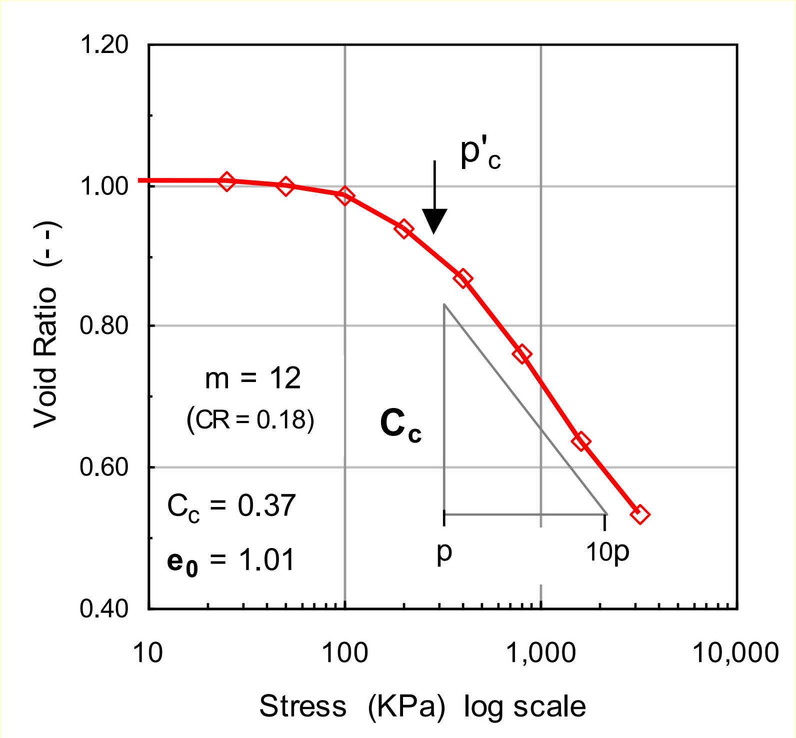
$$m = \ln 10 \frac{1+e_0}{C_c} = 2.3 \frac{1+e_0}{C_c}$$

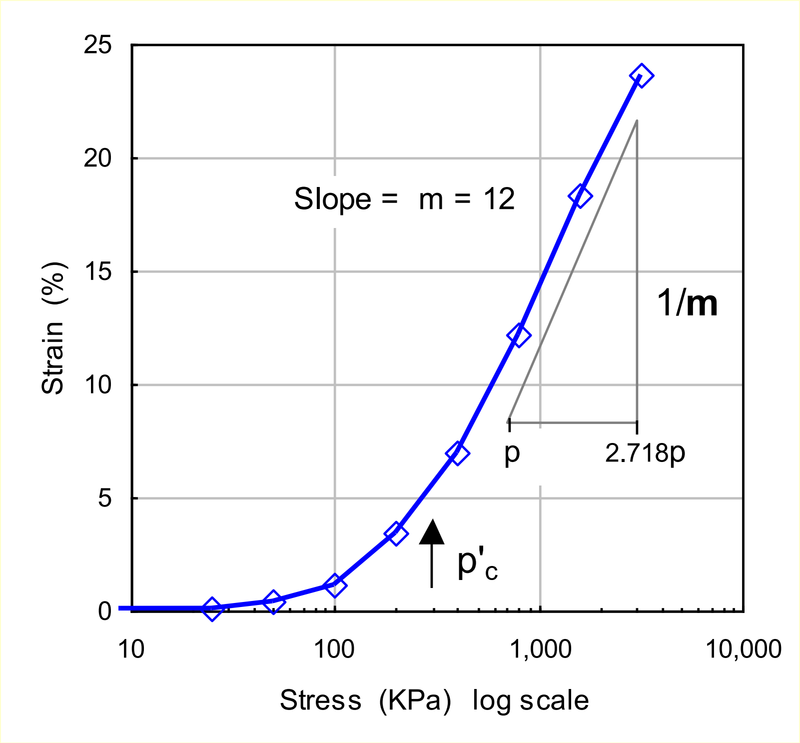
Typical and Normally Conservative Virgin Modulus Numbers

SOIL TYPE		MODULUS NUMBER	STRESS EXP.
Till, very dense to dense		1,000 — 300	(j = 1)
Gravel		400 — 40	(j = 0.5)
Sand	dense	400 — 250	(j = 0.5)
	compact	250 — 150	- " -
	loose	150 — 100	- " -
Silt	dense	200 — 80	(j = 0.5)
	compact	80 — 60	- " -
	loose	60 — 40	- " -
Silty clay	hard, stiff	60 — 20	(j = 0)
and	stiff, firm	20 — 10	- " -
Clayey silt	soft	10 — 5	- " -
Soft marine clays and organic clays		20 — 5	(j = 0)
Peat		5 — 1	(j= 0)

For clays and silts, the recompression modulus, m_r, is often five to ten times greater than the virgin modulus, m, listed in the table

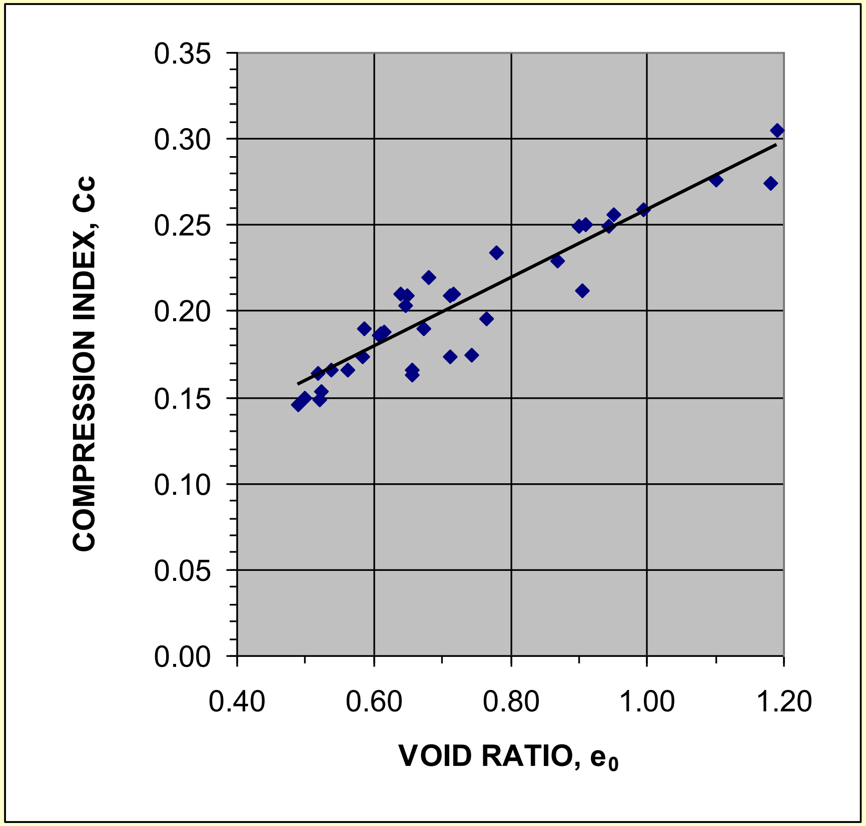
Evaluation of compressibility from oedometer results

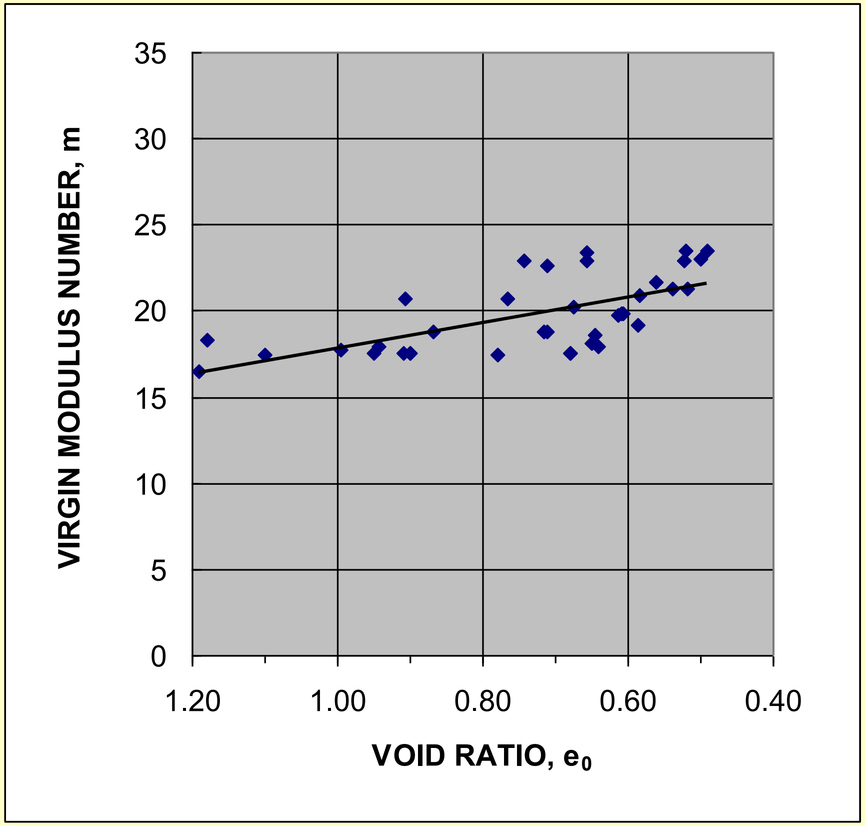




Void-Ratio vs. Stress and Strain vs. Stress — Same test data

Comparison between the C_c/e_0 approach and the Janbu method





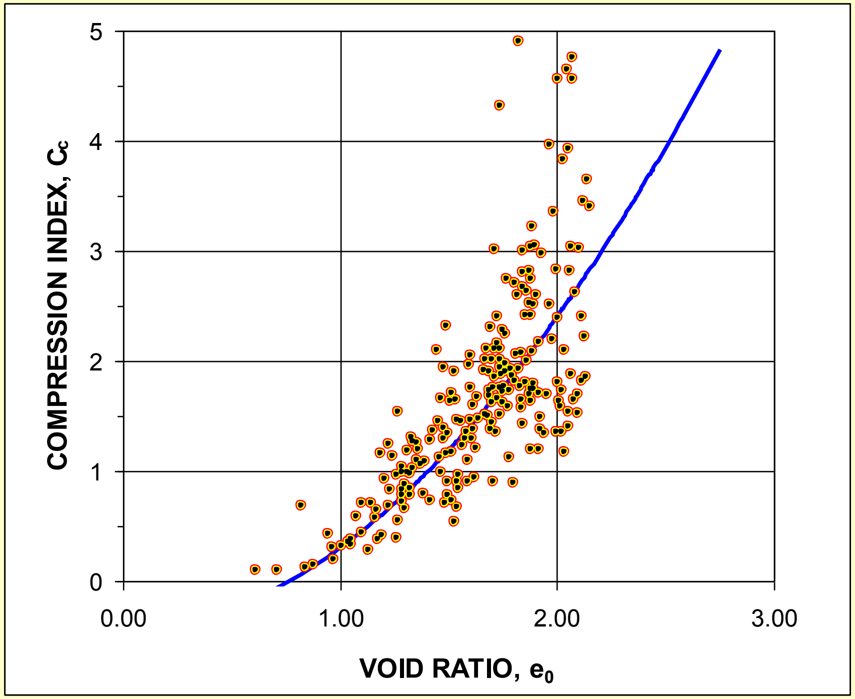
Data from a 20 m thick sedimentary deposit

The C_c-e_0 approach (based on C_c) implies that the compressibility varies by 30± %.

However, the Janbu methods shows it to vary only by 10± %. The modulus number, m, ranges from 18 through 22; It would be unusual to find a clay with less variation.

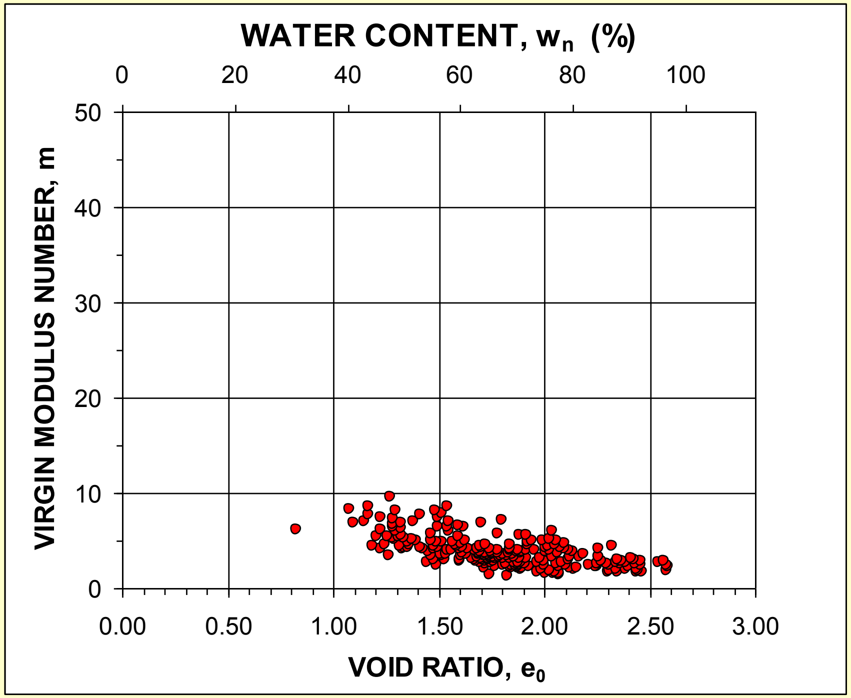
The C_c-values converted via the associated e_0-values to modulus numbers.

Conventional C_c/e_0

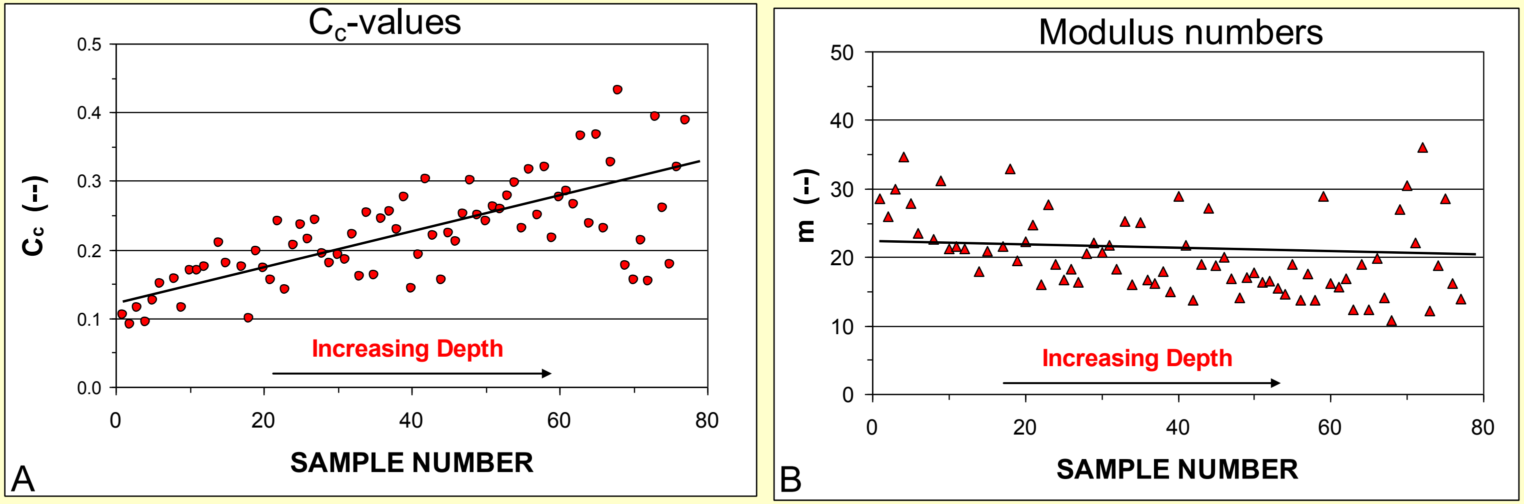


Oedometer test data from Leroueil et al., 1983

Janbu Modulus Number m

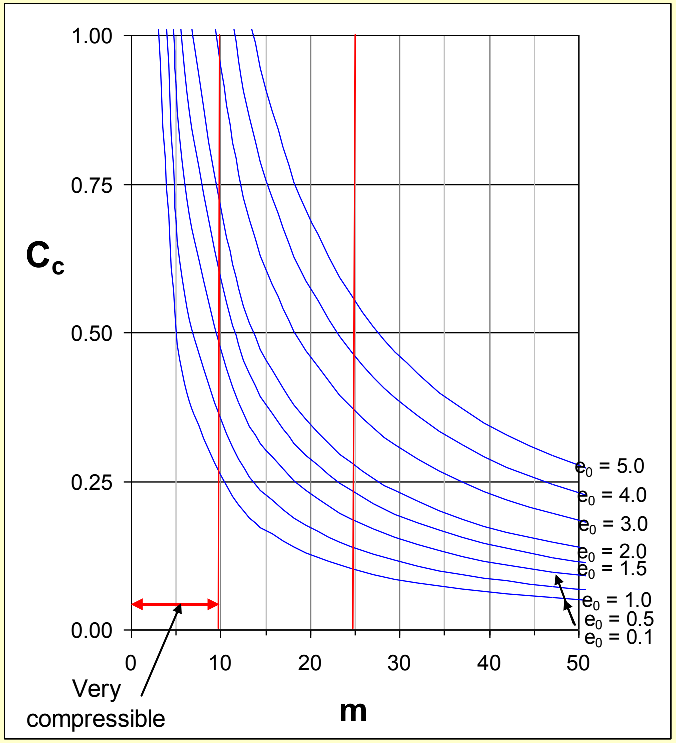


m < 10 ==> Highly compressible

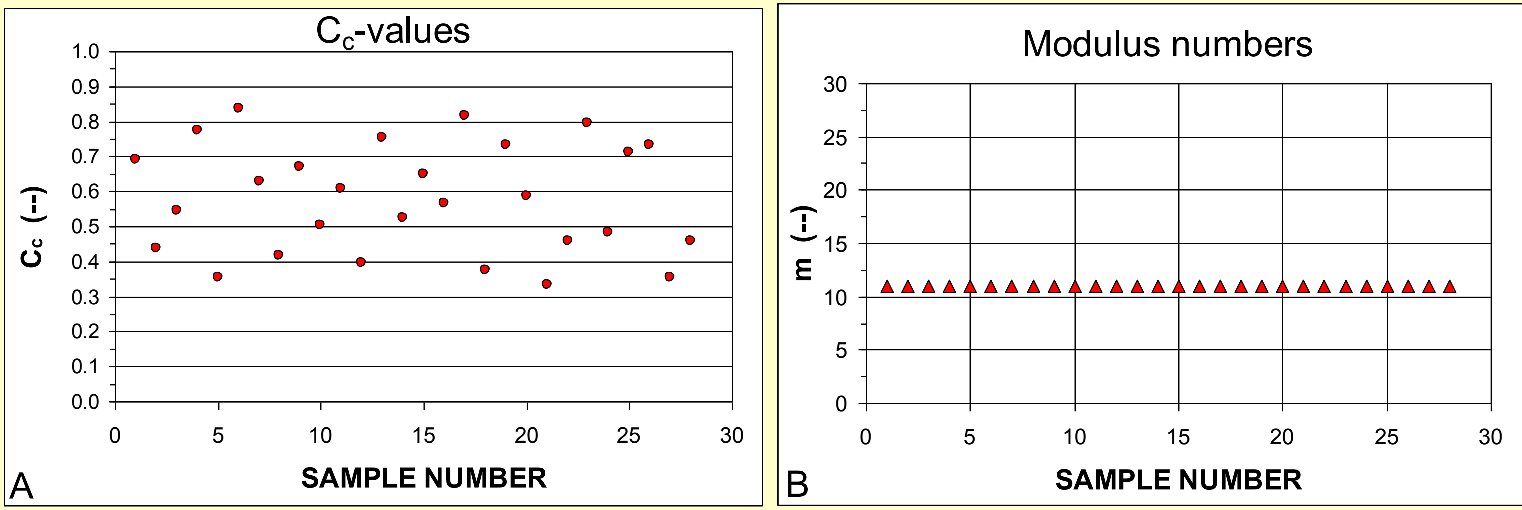


Cc-values and modulus numbers from Beaumont clay. Data from Endley et al. 1996.

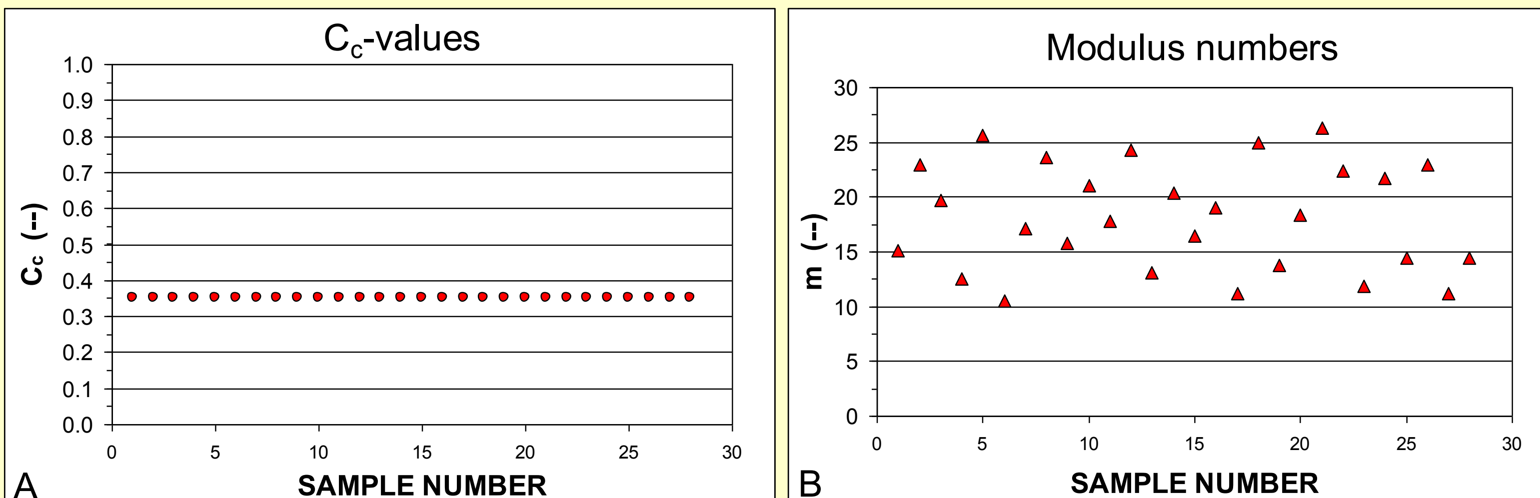
C_c as a function of m and e_0



Variable compressibility?



Typical, but "selected" C_c-values, and converted via associated "selected" e_0-values to "m"-values.



"Selected" constant C_c-values, and converted via associated "selected" e_0-values to "m"-values.

Strain

Linear Elastic Deformation (Hooke's Law)

$$\varepsilon = \frac{\Delta\sigma'}{E}$$

ε = induced strain in a soil layer

$\Delta\sigma'$ = imposed change of effective stress in the soil layer

E = elastic modulus of the soil layer (Young's Modulus)

Young's modulus is the modulus for when lateral expansion is allowed, which may be the case for soil loaded by a small footing, but not when the load is applied over a large area. In the latter case, the lateral expansion is constrained (or confined). The constrained modulus, D, is larger than the E-modulus. The constrained modulus is also called the "oedometer modulus". For ideally elastic soils, the ratio between D and E is:

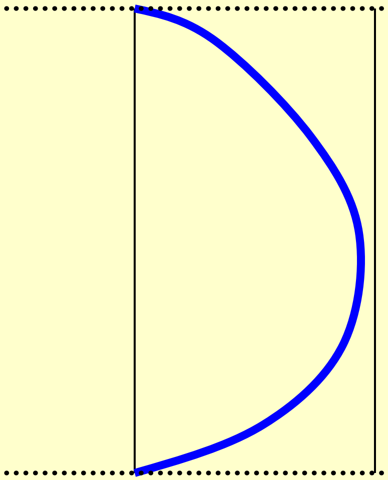
$$\frac{D}{E} = \frac{(1-\nu)}{(1+\nu)\,(1-2\nu)}$$

ν = *Poisson's ratio*

Approximate values of T_v for different average

values of the degree of consolidation, U_{AVG}

U_{AVG} (%)	25	50	70	80	90	"100"
T_v	0.05	0.20	0.40	0.57	0.85	$\approx$1.00

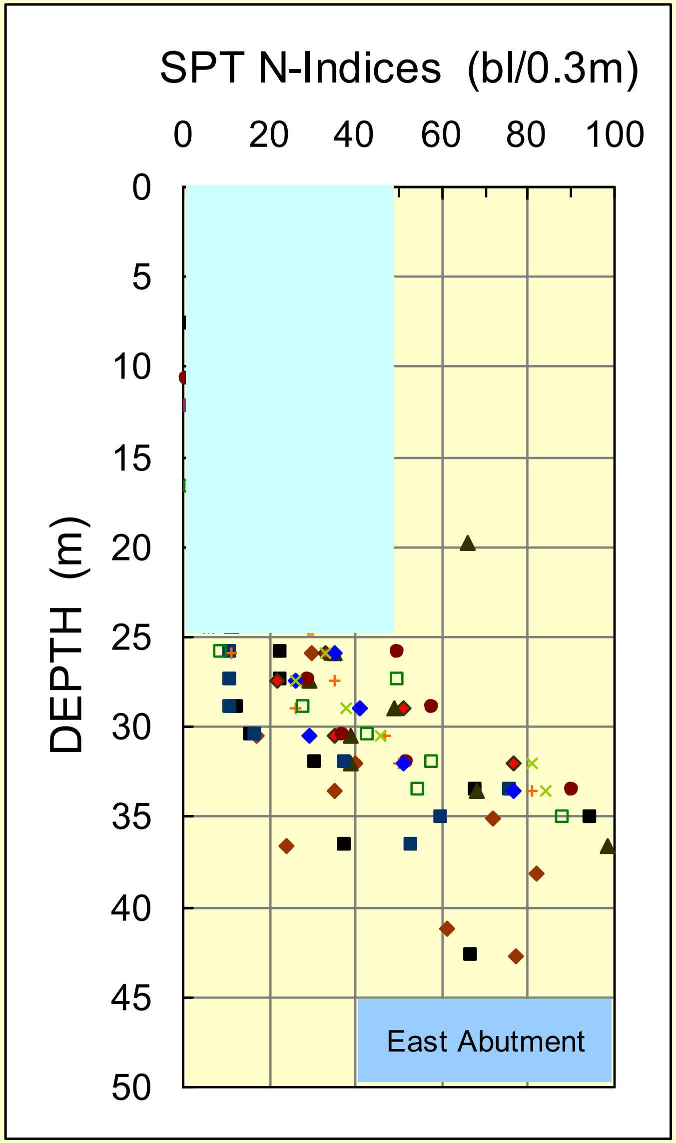


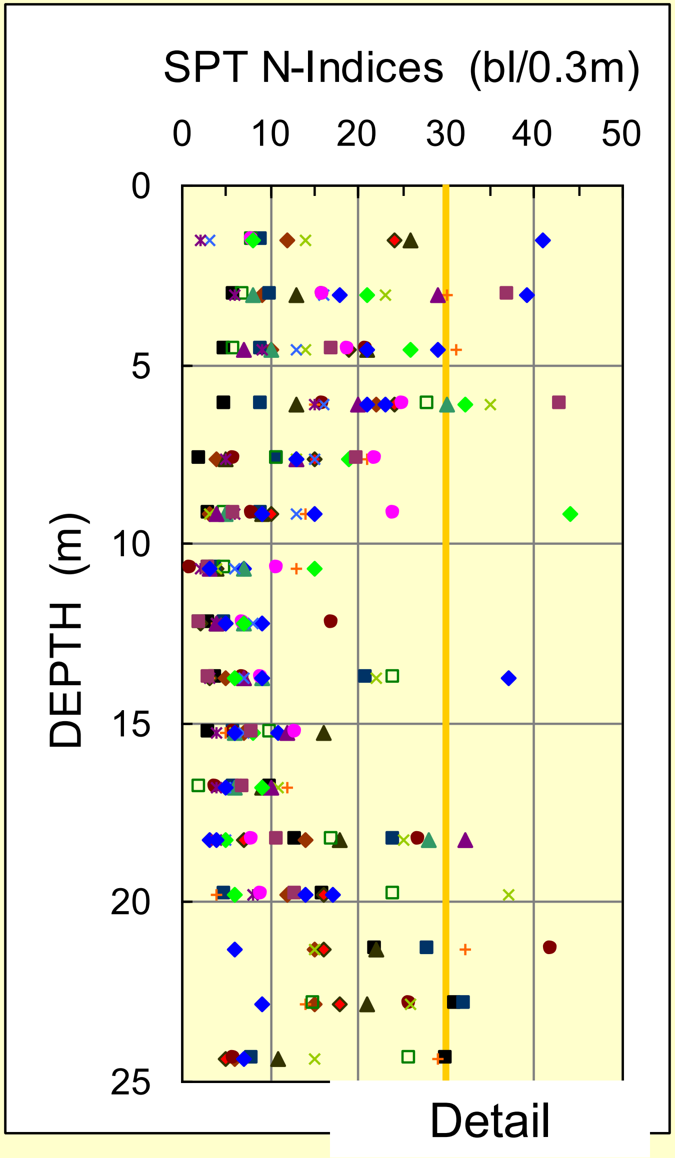
Double-side drainage distribution of excess pore pressure at Time t in a soil profile consisting of a single consolidating soil layer

HOW TO HANDLE A MULTILAYERED PROFILE?

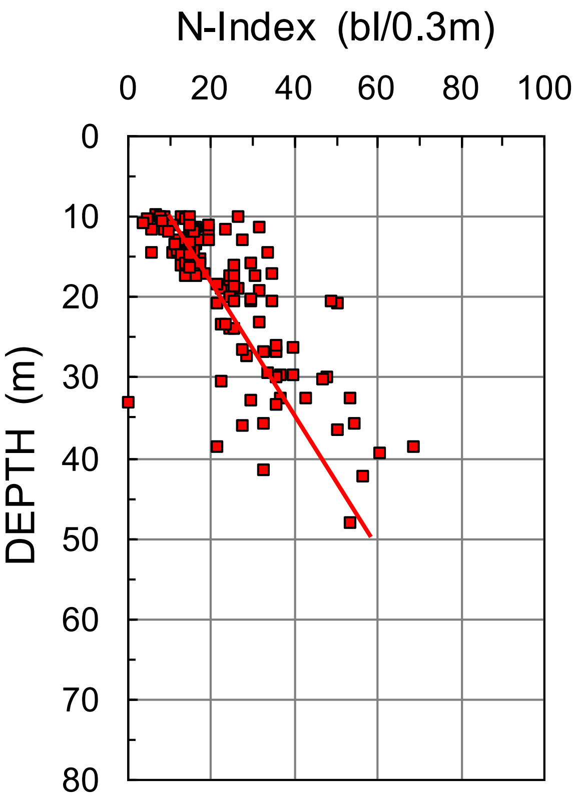
The SPT

Example from Atlantic coast of South USA

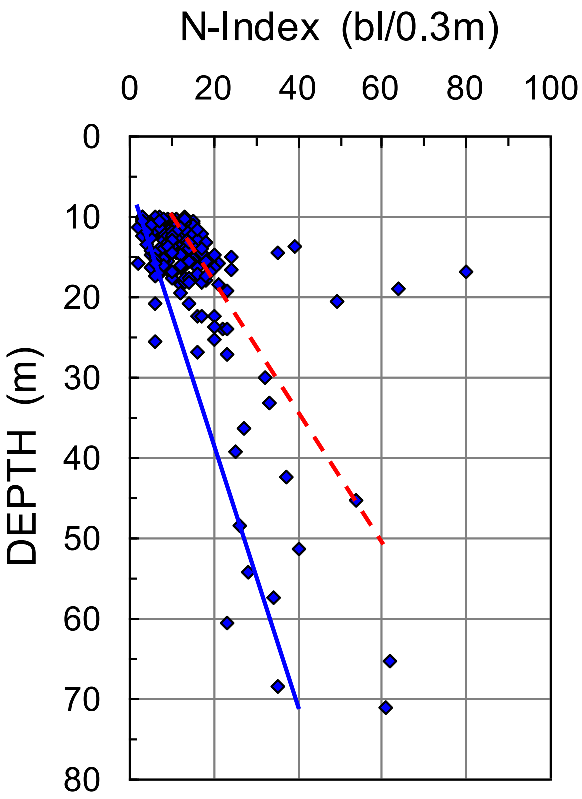




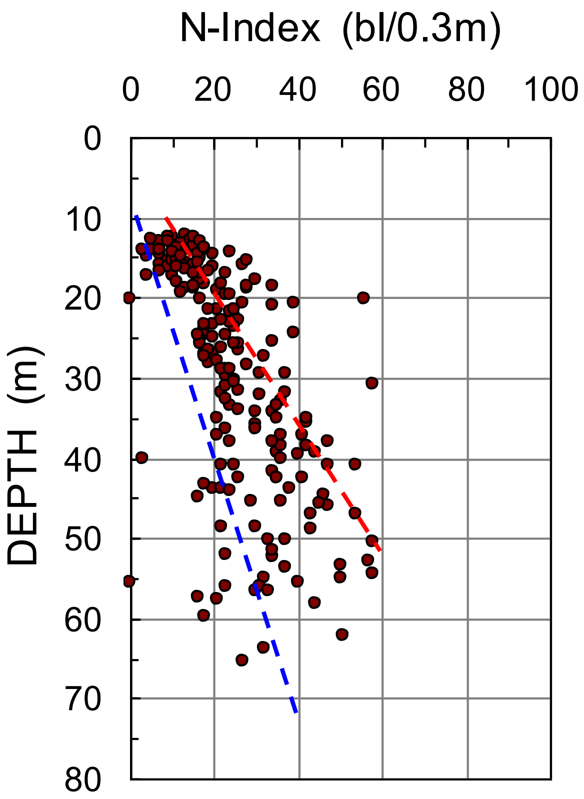
Example from Atlantic coast of Canada



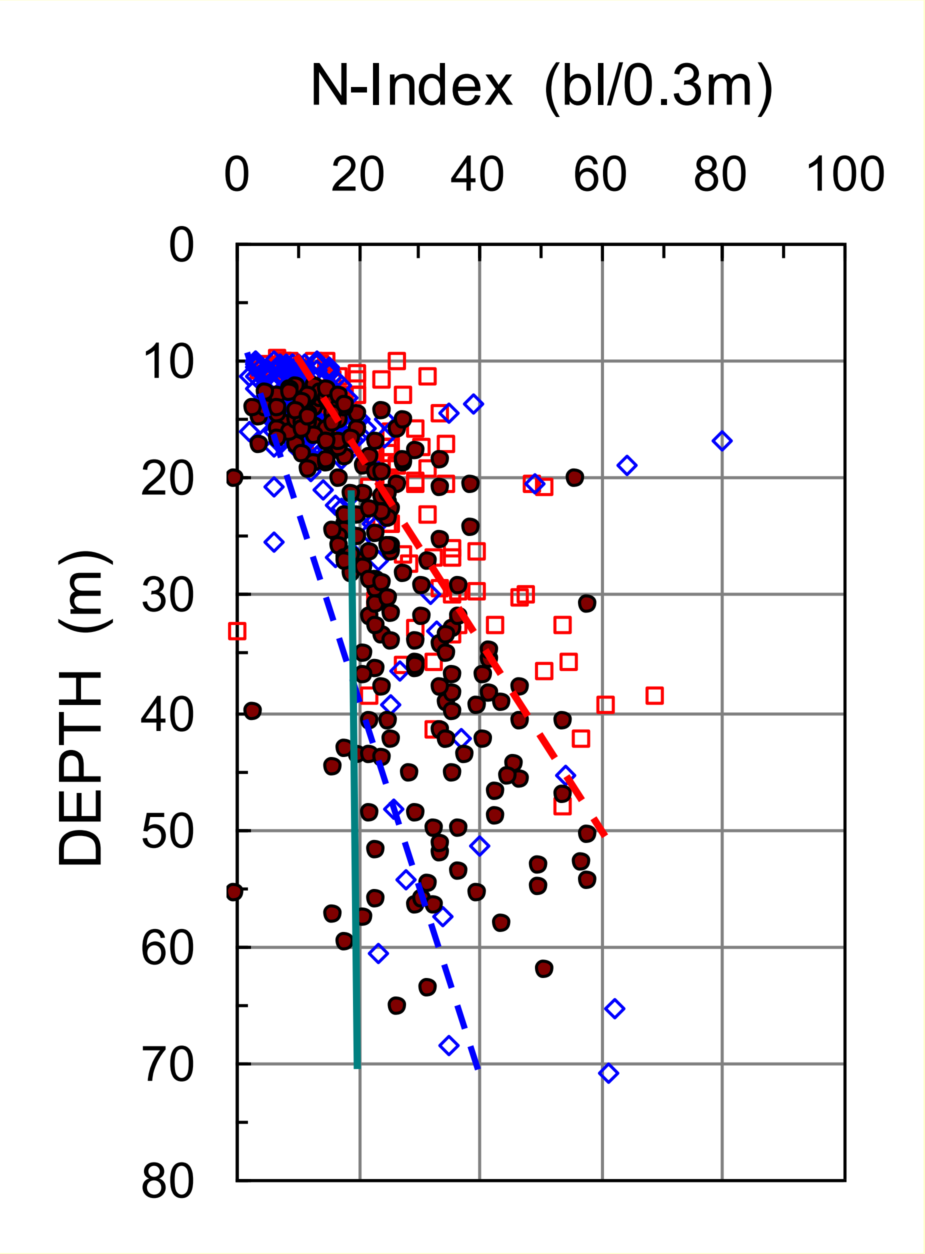
SPT for design



After problems arose

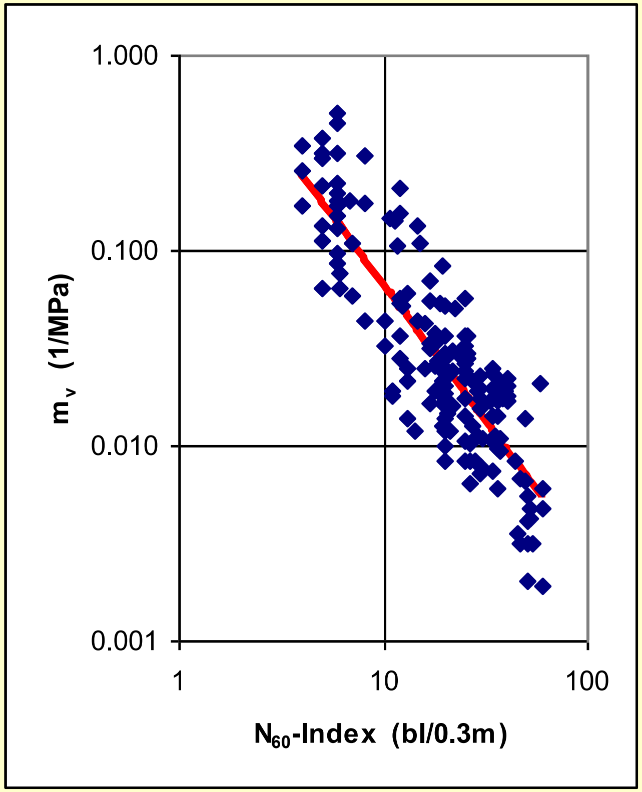


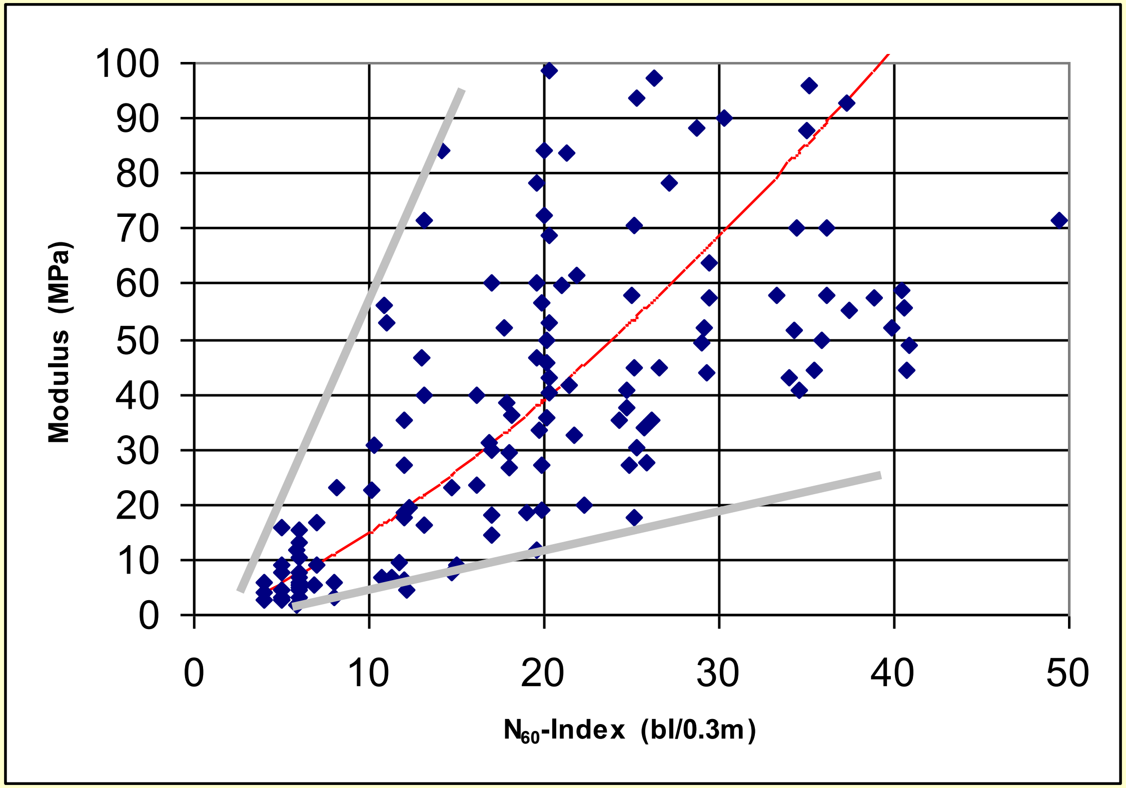
Forensics



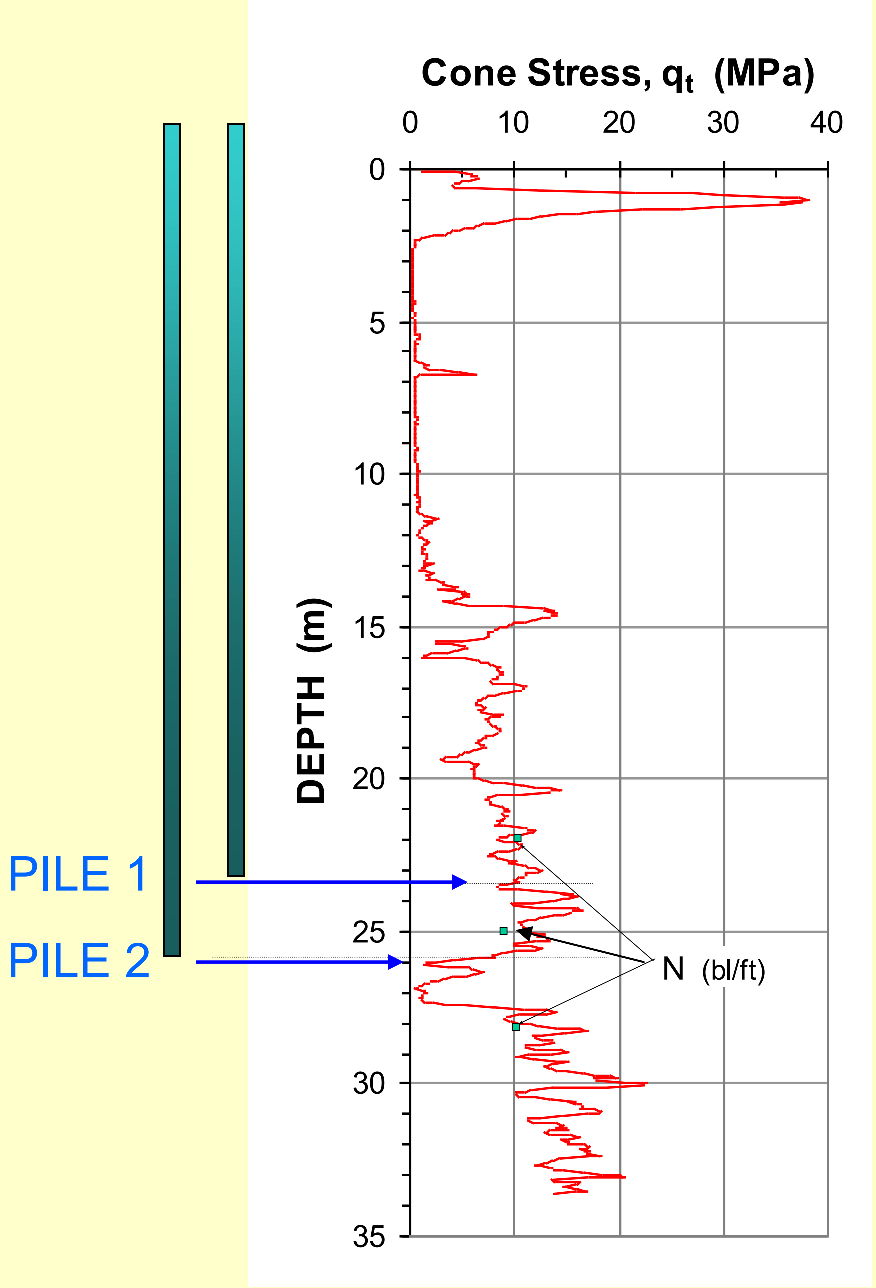
With all data points

Direct numerical use of the SPT N-index





(after Terzaghi, Peck, and Mesri 1996
from Burland and Burbidge 1985)


Cone Stress, q_t (MPa)
0
10
20
30
40
DEPTH (m)
0
5
10
15
20
25
30
35
PILE 1
PILE 2
N (bl/ft)

100 years of development

(slides pinched from Paul Mayne)

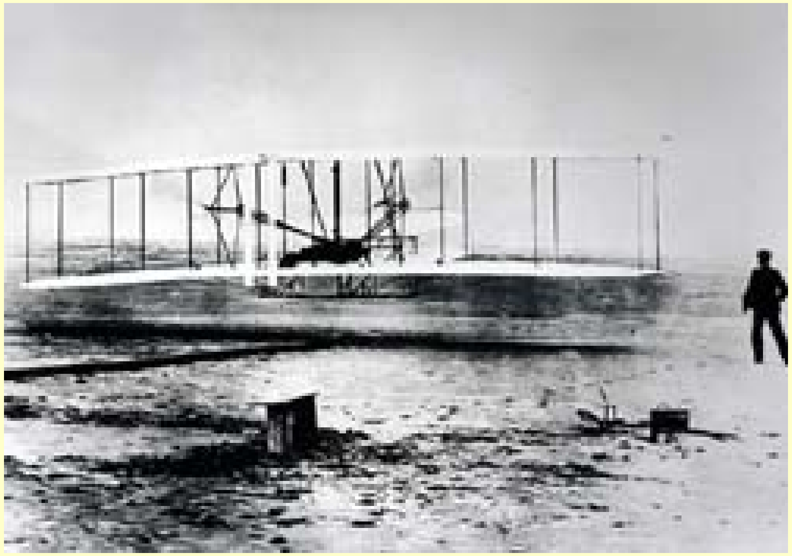
Telephone
1909

Cell phone

2001

Wright Plane

1903



Boeing 717

2001

Oldfield Auto
1903

BMW

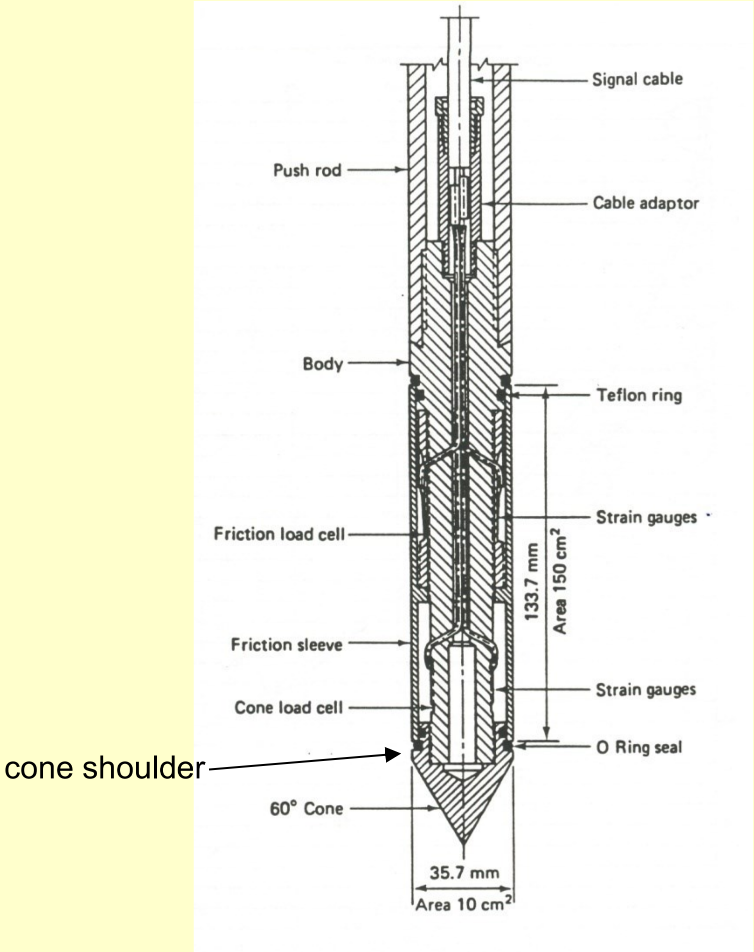
2001

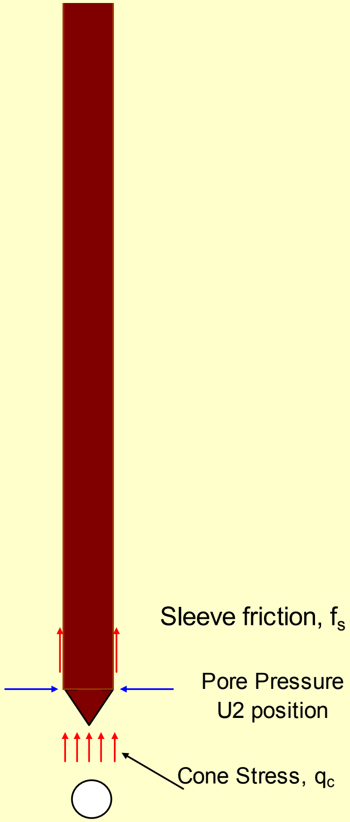
Geotechnical Test 1902

1902 - Colonial Charles Gow of Raymond Pile Company

Geotech Test 2002 and today?

Principles of the CPT and CPTU





The Cone

Penetrometer

"U2 Position" = pore pressure measured on the cone "shoulder"


DAU-CPT

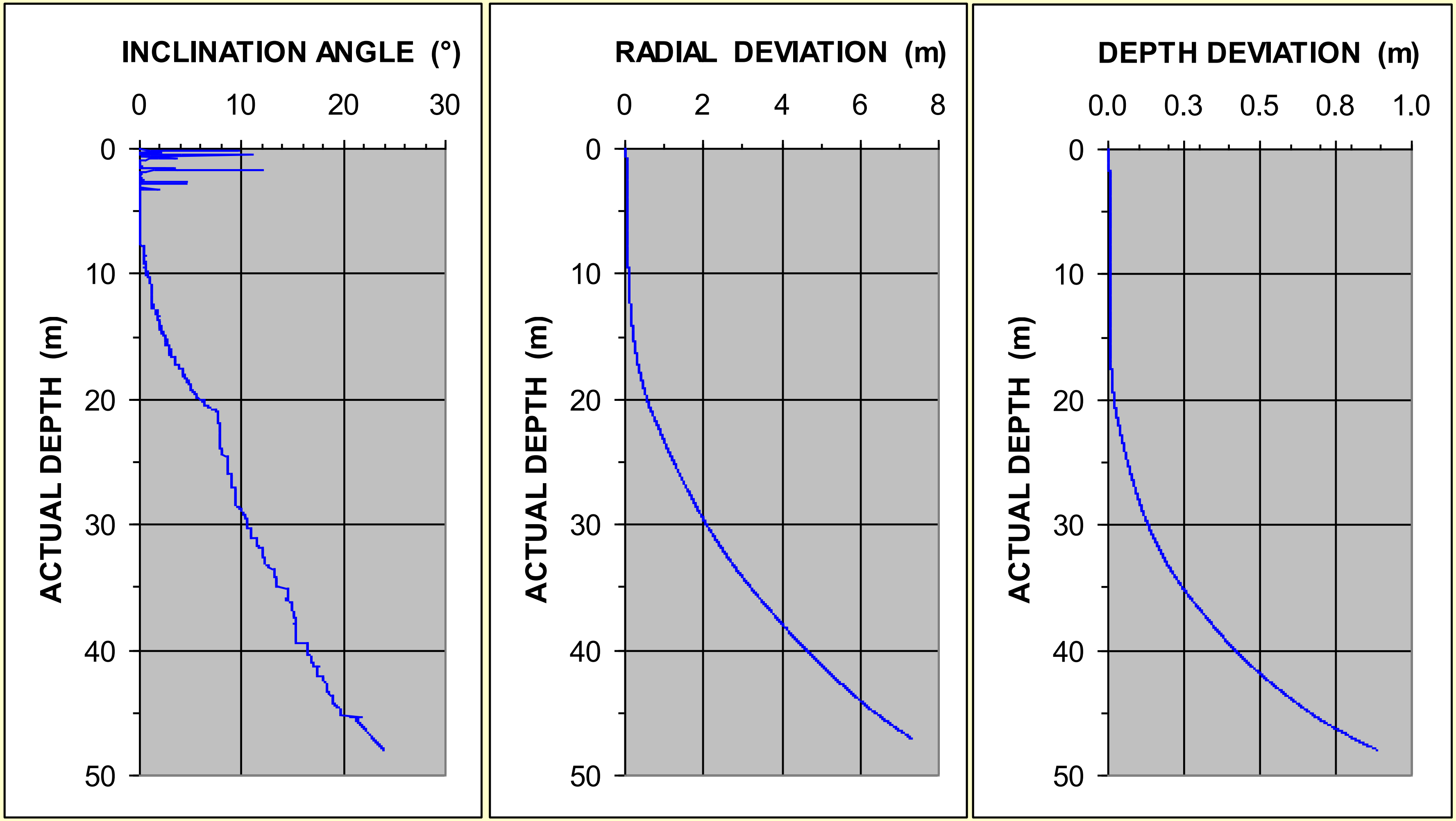

F550
Advantage
Probe & Injection Corp.
SASKATOON, SASK
306-956-3374

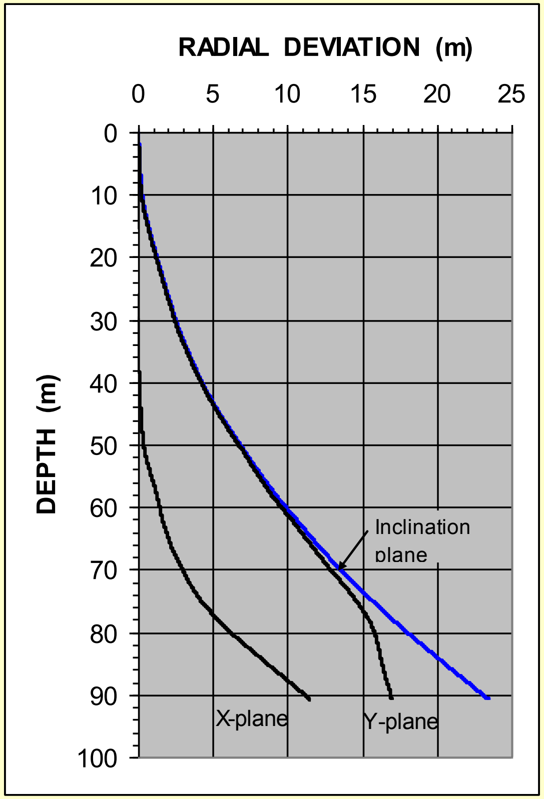

6600
6600
Geoprobe
Geoprobe

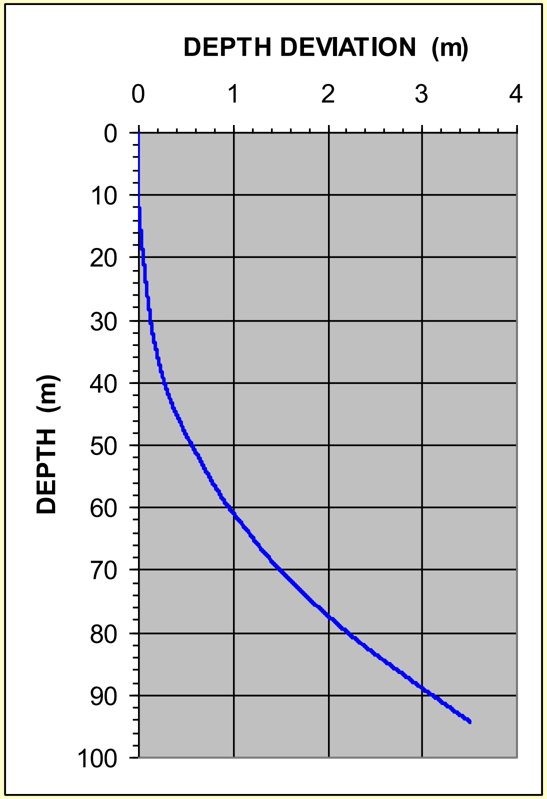

Geoprobe

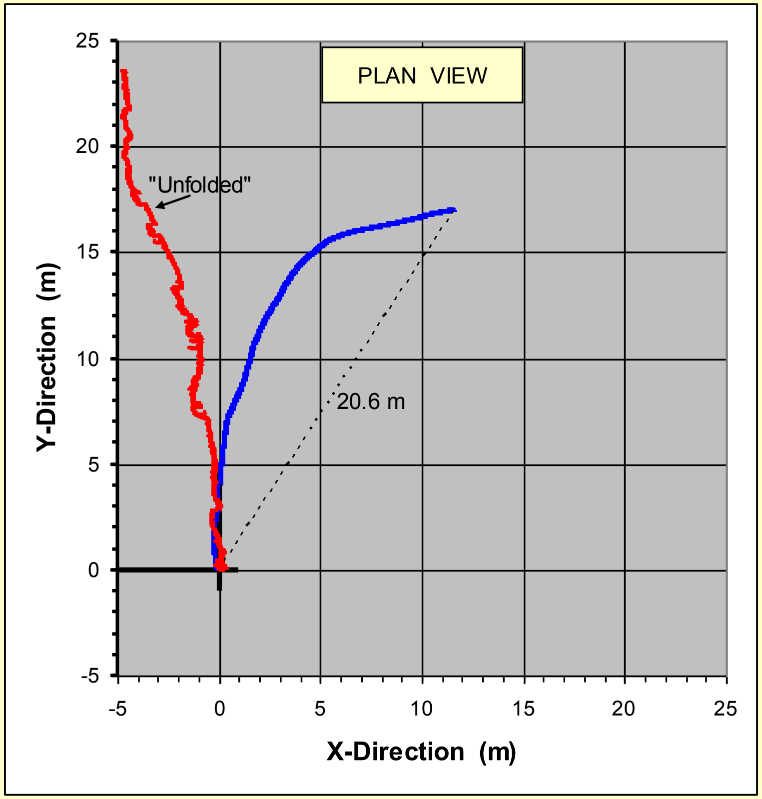
The CPT sounding rod is never truly vertical, of course.

How much can it be off?



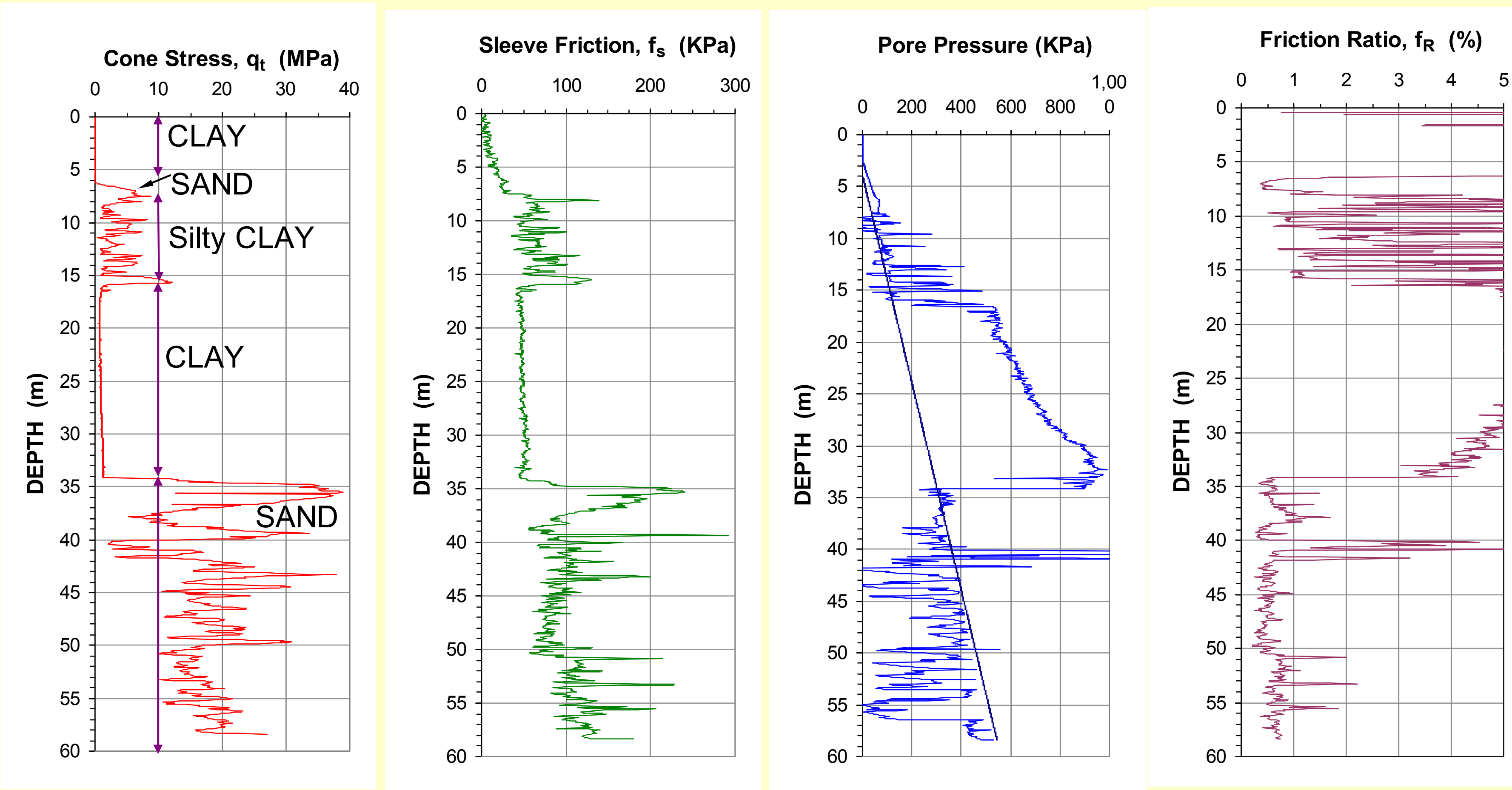

RADIAL DEVIATION (m)
0 5 10 15 20 25
DEPTH (m)
0 10 20 30 40 50 60 70 80 90 100
Inclination plane
X-plane
Y-plane


DEPTH DEVIATION (m)
0 1 2 3 4
DEPTH (m)
0 10 20 30 40 50 60 70 80 90 100


PLAN VIEW
"Unfolded"
20.6 m
Y-Direction (m)
25 20 15 10 5 0 -5
X-Direction (m)
-5 0 5 10 15 20 25

Example 2

Example of a CPTU sounding from a river estuary delta (Nakdong River, Pusan, Korea)

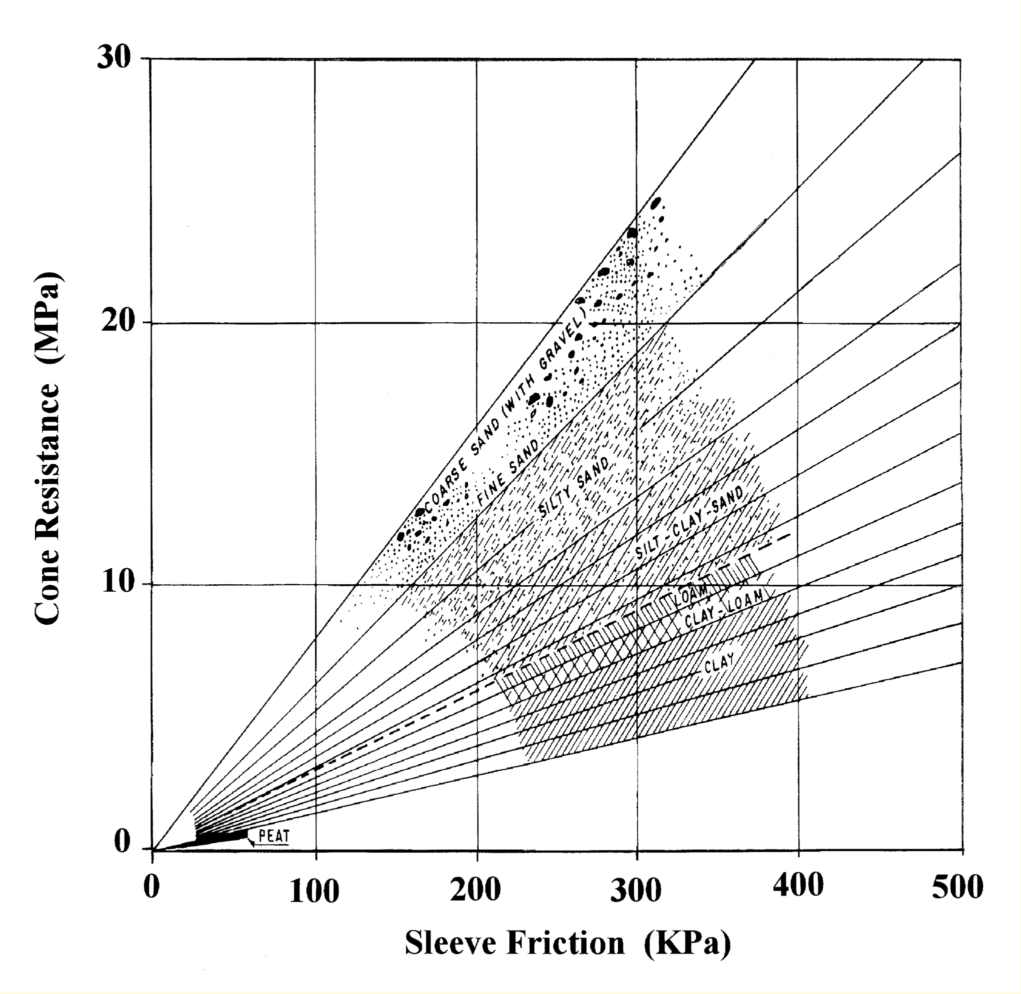


The sand layer between 6 m and 8 m depth is potentially liquefiable.

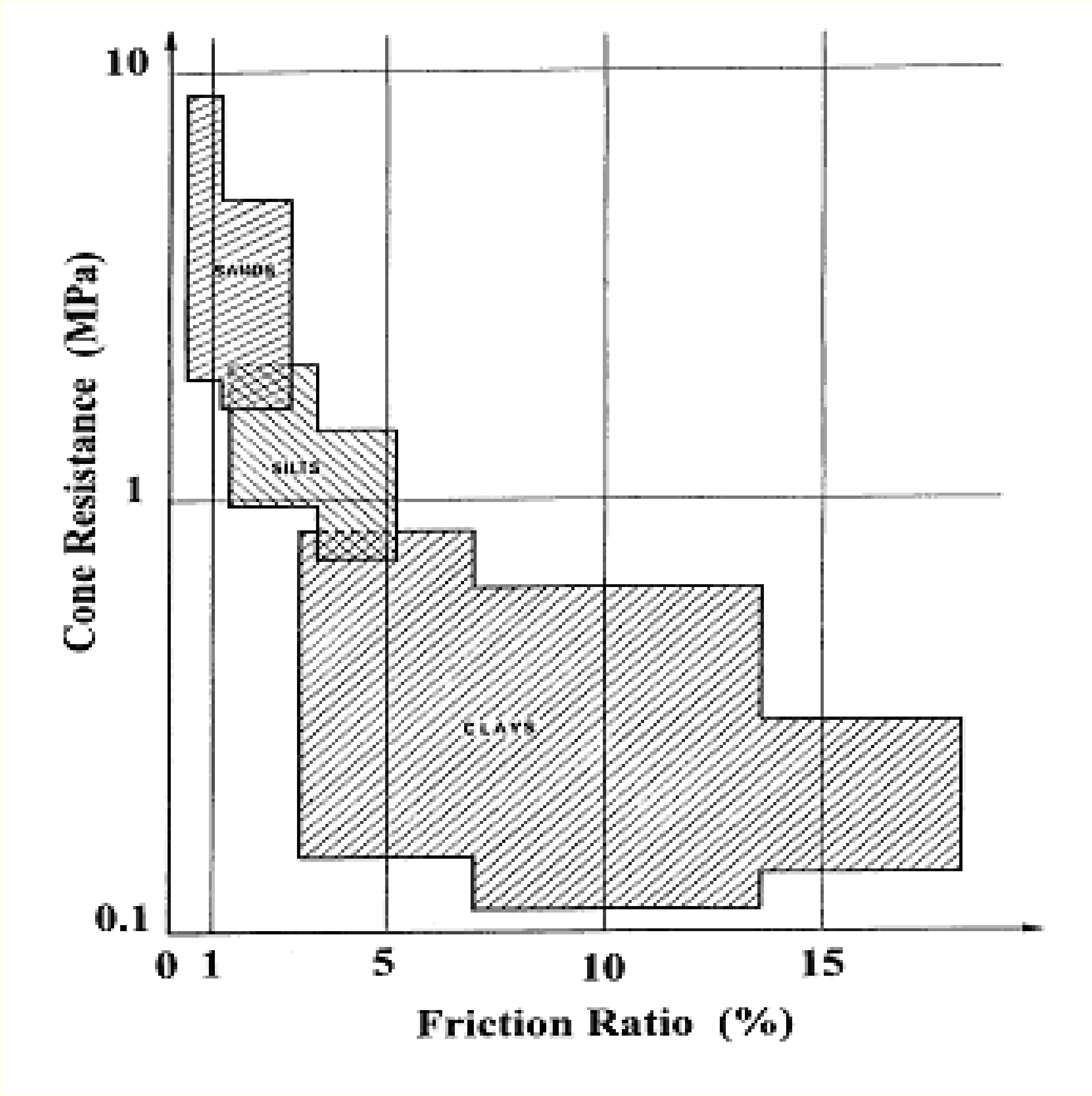
The two clay layers are very soft.

The sand below 34 m depth is very dense and dilative, i.e., overconsolidated and providing sudden large penetration resistance to driven piles and relaxation problems.

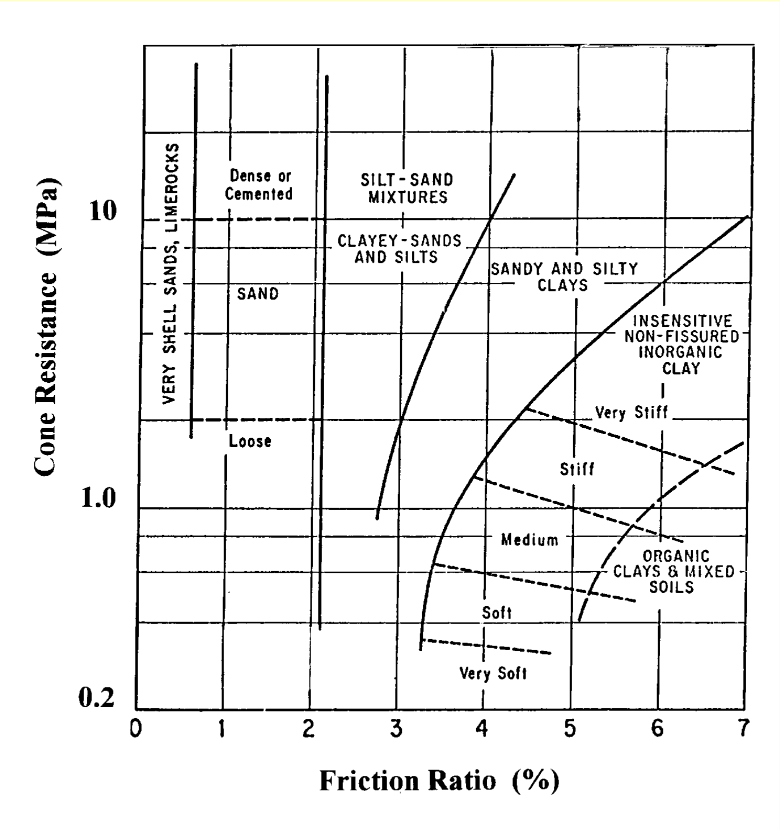
Soil profiling

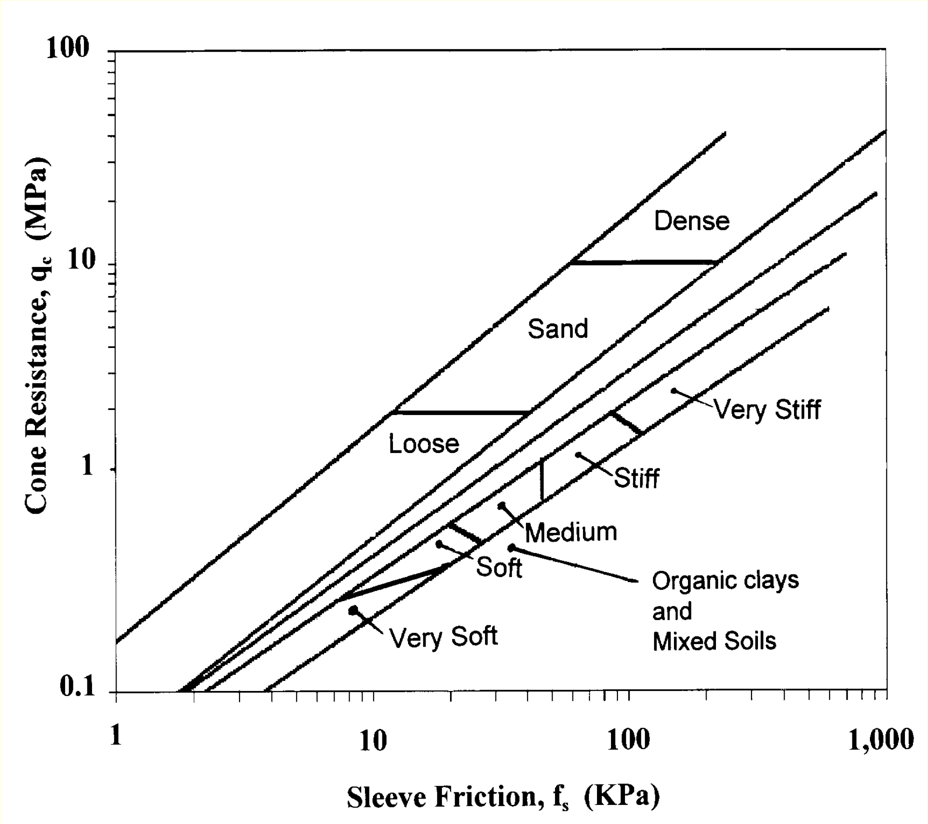


The Begemann original profiling chart (Begemann, 1965)

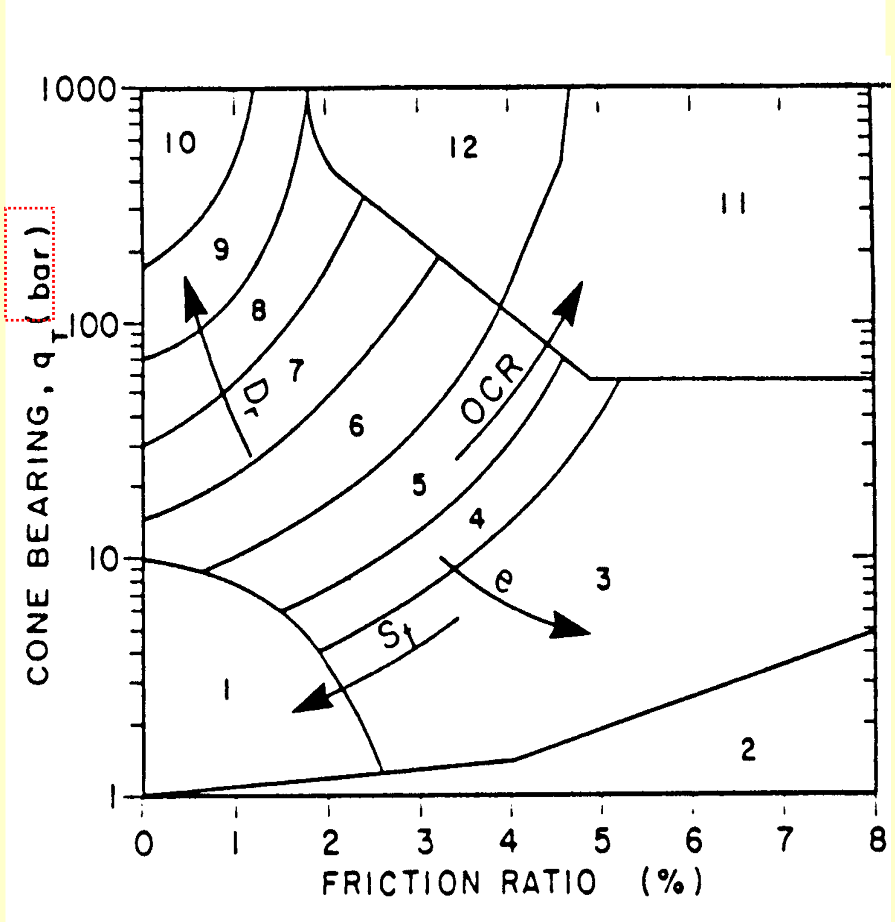


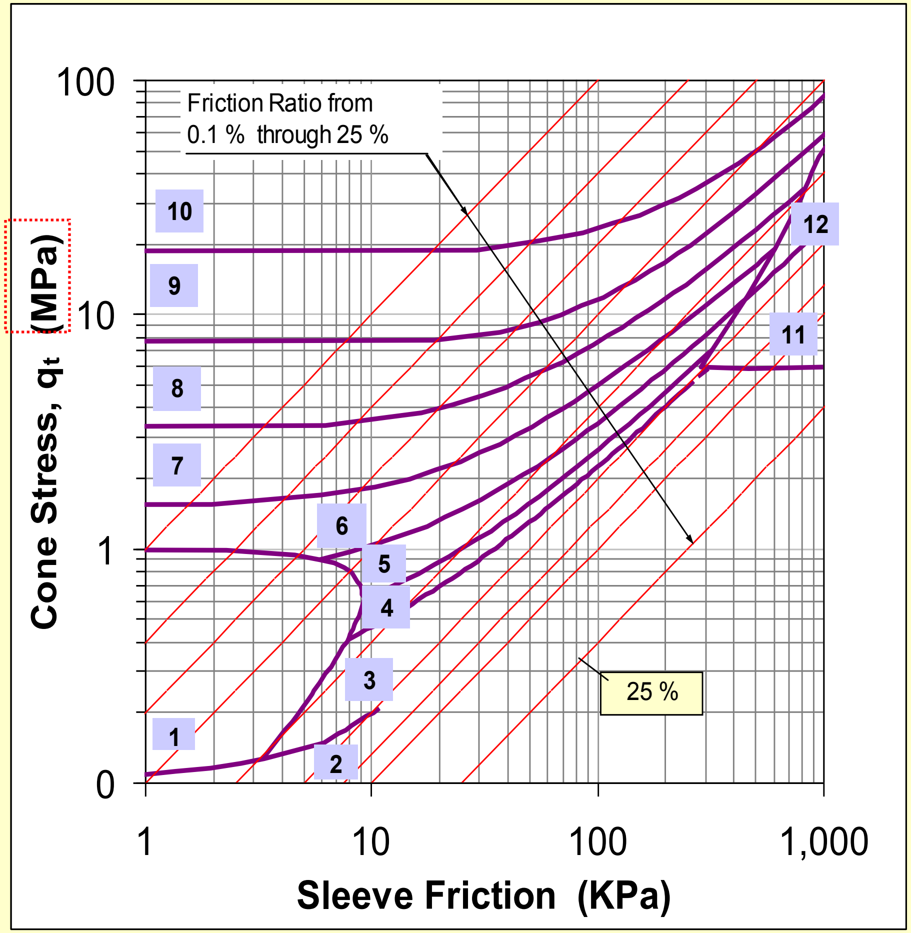
Plot of data from research penetrometer (Sanglerat et al., 1974)



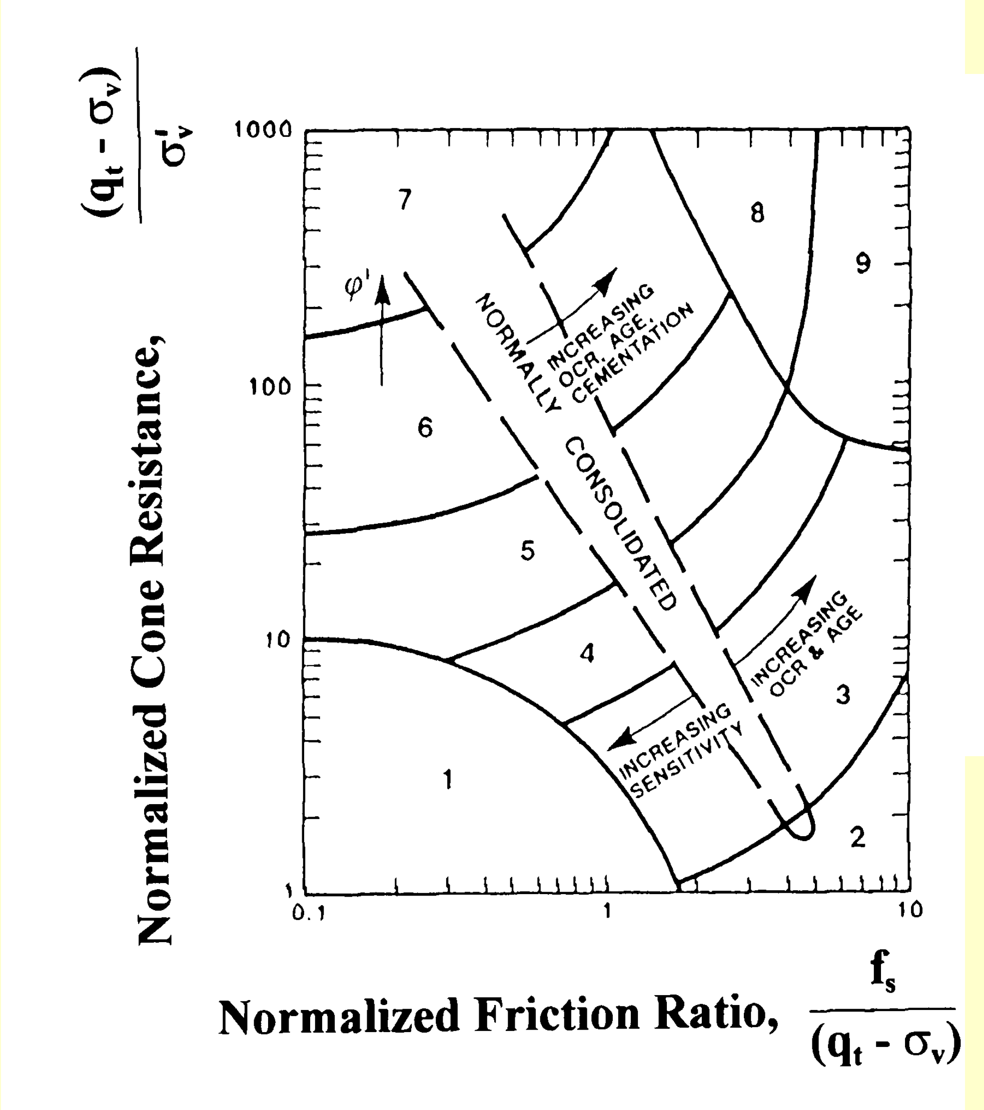


The Schmertmann profiling chart (Schmertmann, 1978)

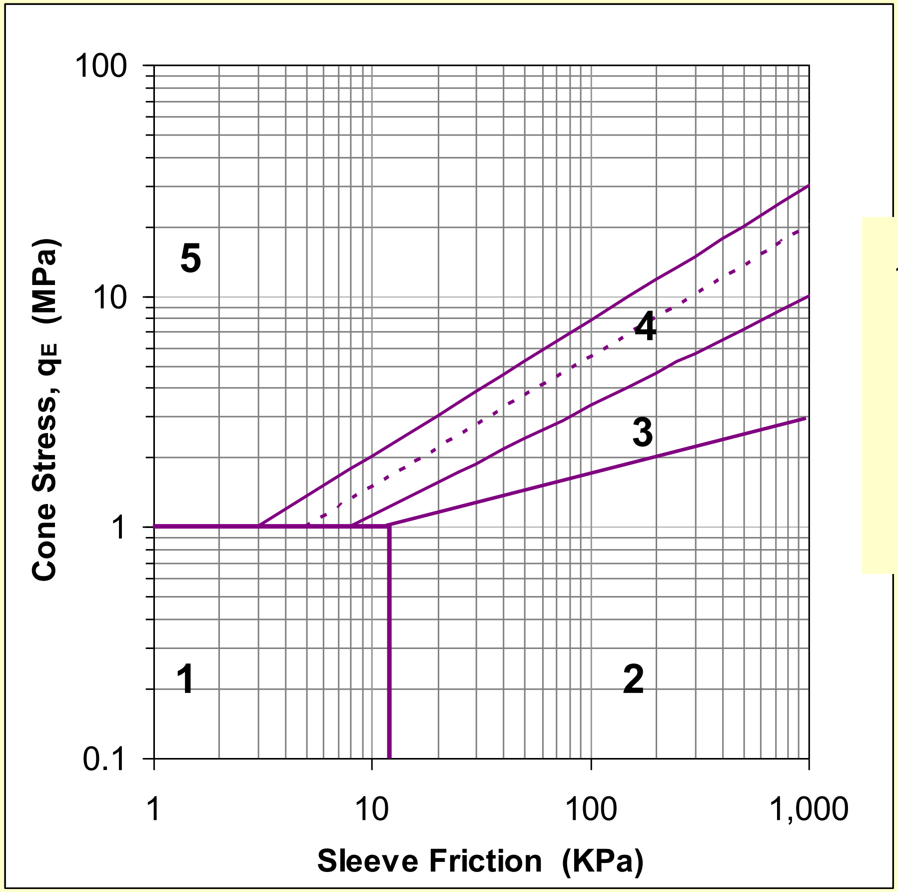




Profiling chart per Robertson et al. (1986)

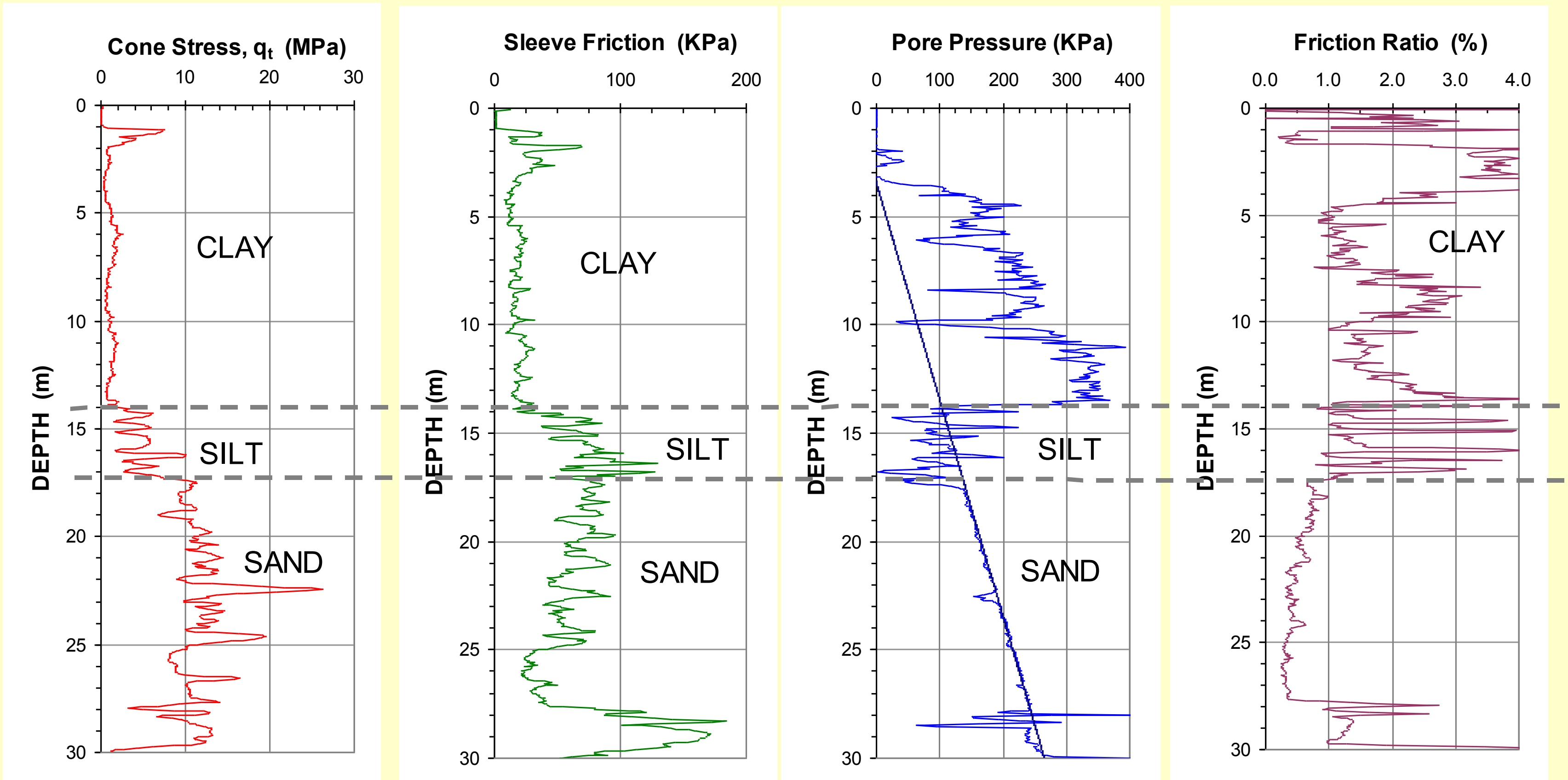


Profiling chart per Robertson (1990)

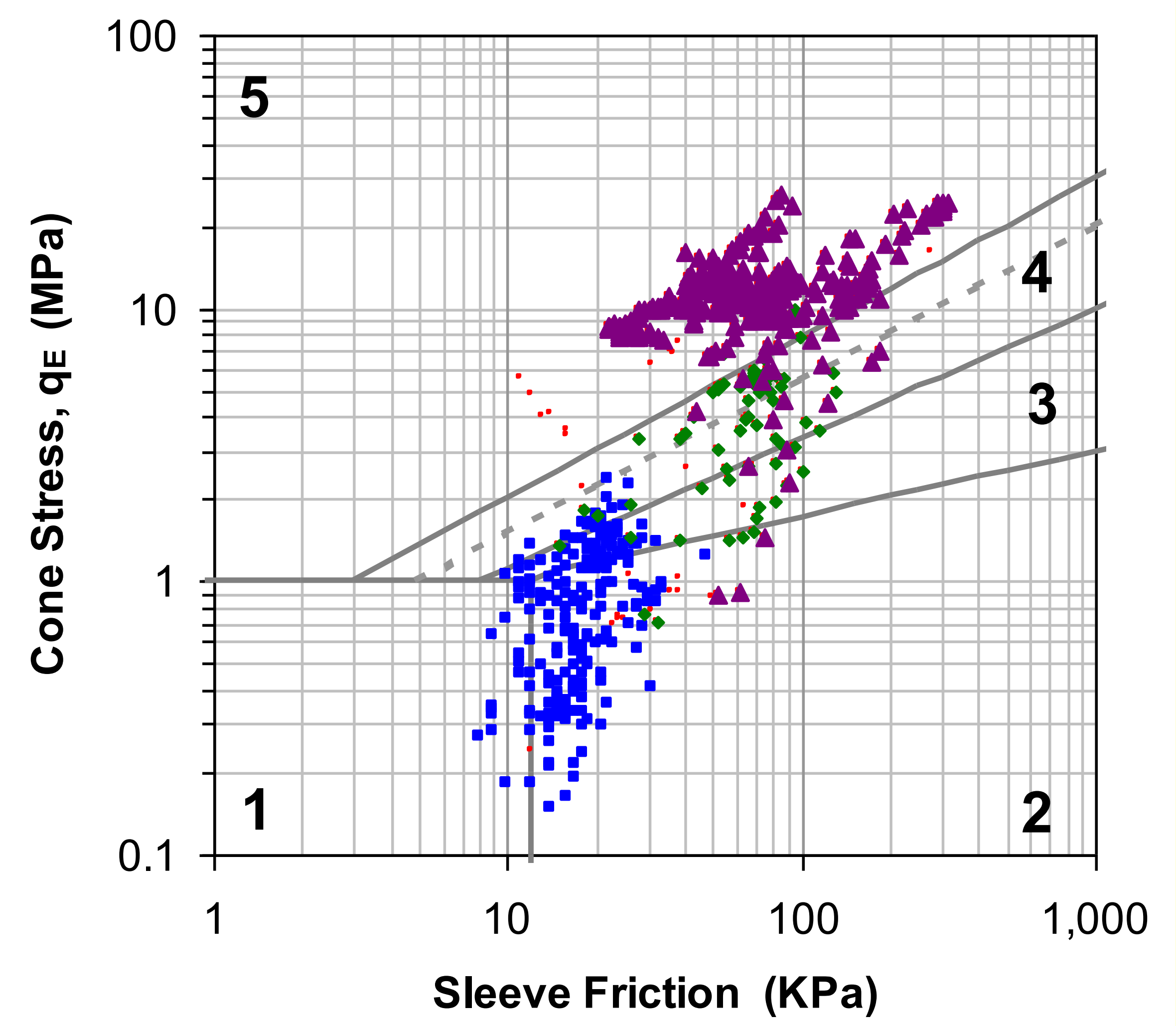


1 = Very Soft Clays, or Sensitive or Collapsible Soils
2 = Clay and/or Silt
3 = Clayey Silt and/or Silty Clay
4a = Sandy Silt
4b = Silty Sand
5 = Sand to Sandy Gravel

The Eslami-Fellenius profiling chart (Eslami 1996; Eslami and Fellenius, 1997)



Results of a CPTU sounding



The CPTU data of the Preceding Slide plotted in an Eslami-Fellenius Chart

The CPTU is an excellent and reliable tool for soil identification, but there is more to geotechnical site investigation than just establishing the soil type.

And, the CPTU can deliver much more than soil profiling

Determining the E-Modulus

$$E_{25} = \alpha\, q_t$$

Where E_{25} = secant modulus for a stress equal to about 25 % of "ultimate stress"

α = **an empirical coefficient**

q_t = cone stress

Soil Type	α
Silt and sand	1.5
Compact sand	2.0
Dense sand	3.0
Sand and gravel	4.0

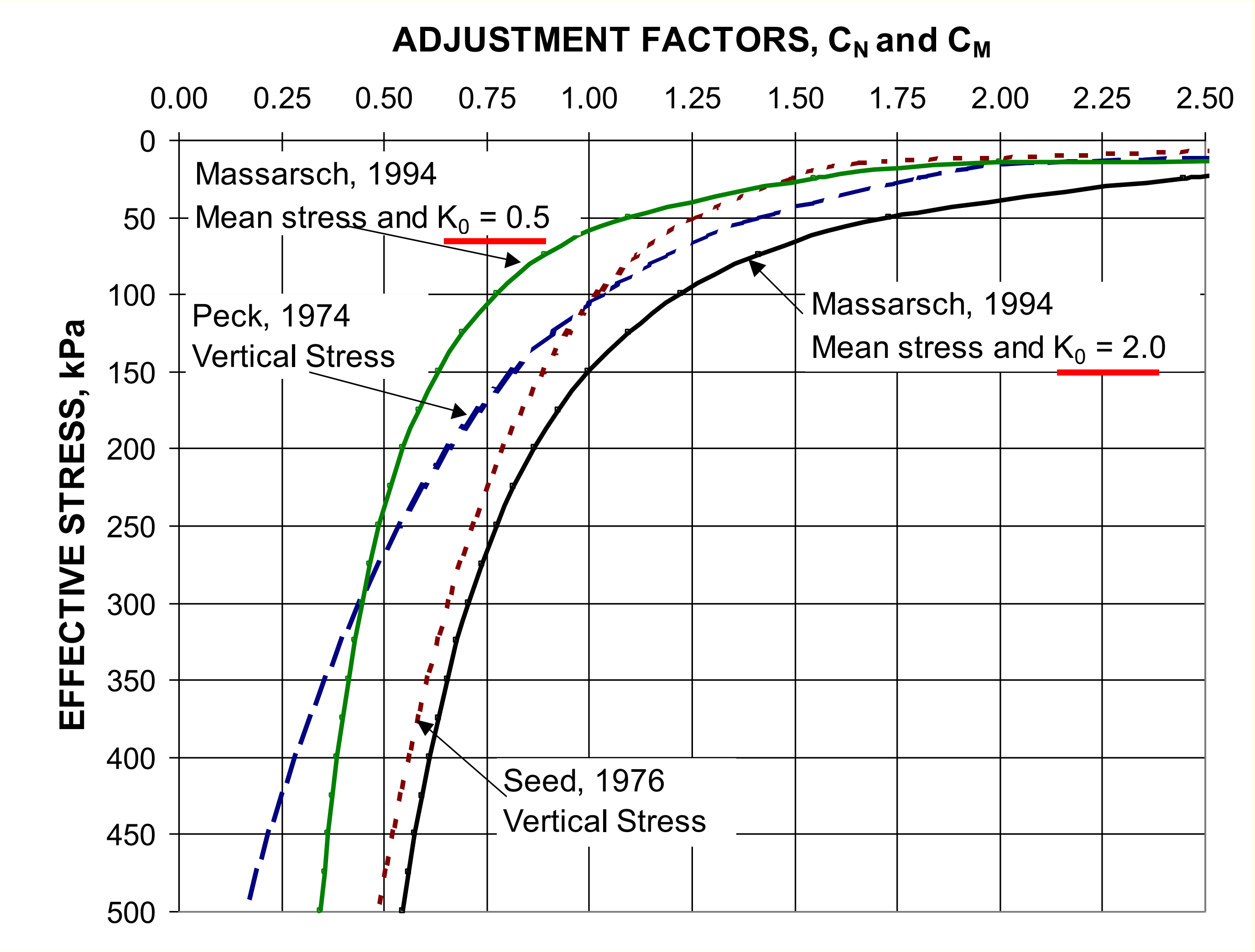
Settlement Analysis and Adjustment to Overburden Stress

$$q_{tM} = q_t \; C_M \qquad C_M = \left(\frac{\sigma_r}{\sigma'_m}\right)^{0.5} \qquad \sigma'_m = \frac{\sigma'_v \left(1 + 2\, K_0\right)}{3}$$

Where
- q_{tM} = adjusted cone stress
- C_M = stress adjustment factor ≤ 2.5
- σ_r = a reference stress = 100 KPa
- σ'_m = mean effective stress

$$q_{tM} = q_t \left(\frac{\sigma_r}{\sigma'_m}\right)^{0.5}$$

Massarsch 1994



Adjustment Factor C_M for CPT with K_0 = 0.5 and 2.0 and
Adjustment Factors C_N for SPT index according to Eqs. 4 and 5

Modulus Number, m, from CPT

$$m = a \left(\frac{q_{tM}}{\sigma_r}\right)^{0.5}$$

where

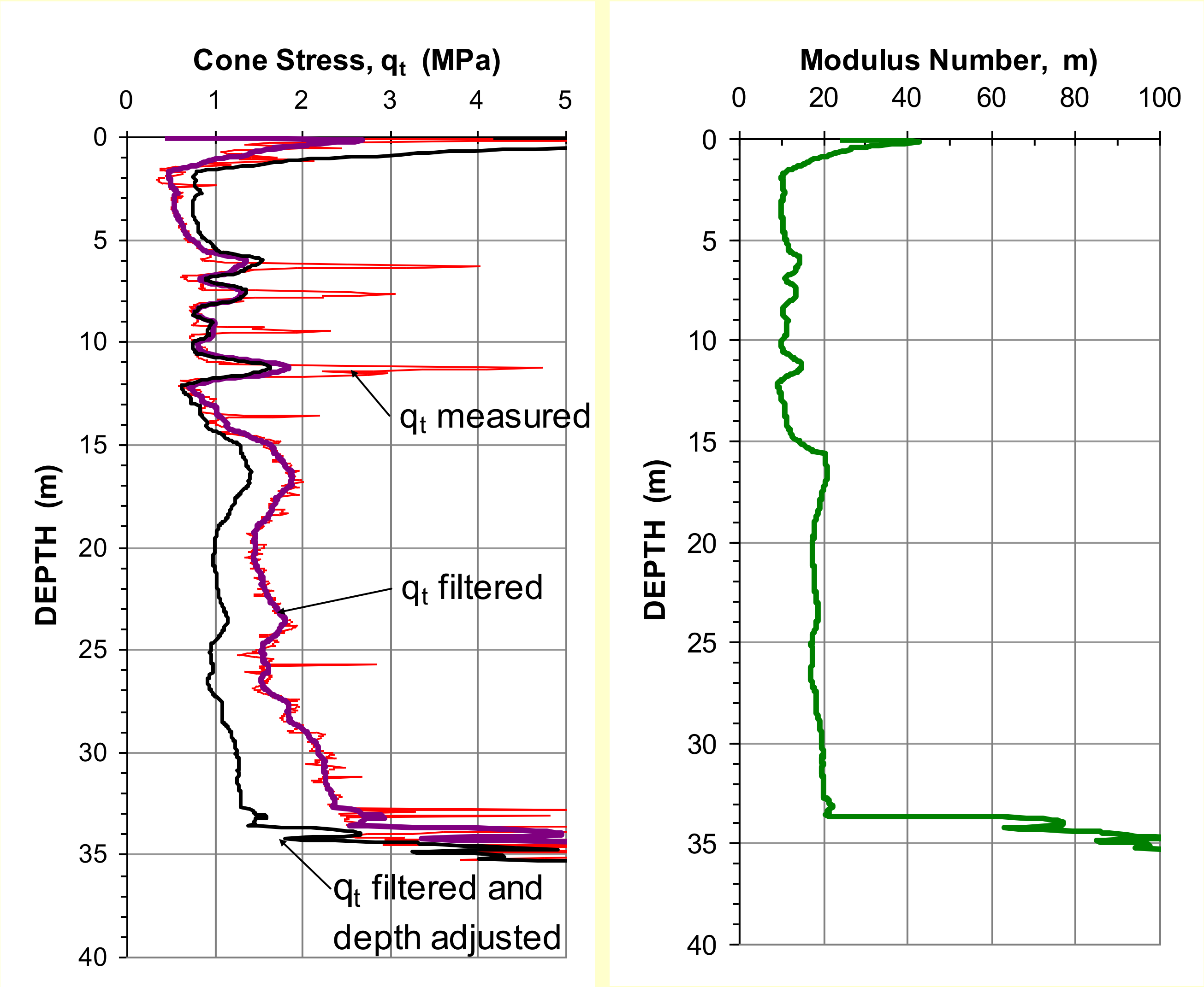
	m	=	modulus number
	a	=	an empirical modulus modifier, which depends on soil type
	q_{tM}	=	stress-adjusted cone stress *)
	σ_r	=	reference stress = 100 KPa

*) Note, the adjustment requires an estimate of the overconsolidation ratio, OCR, and K_0

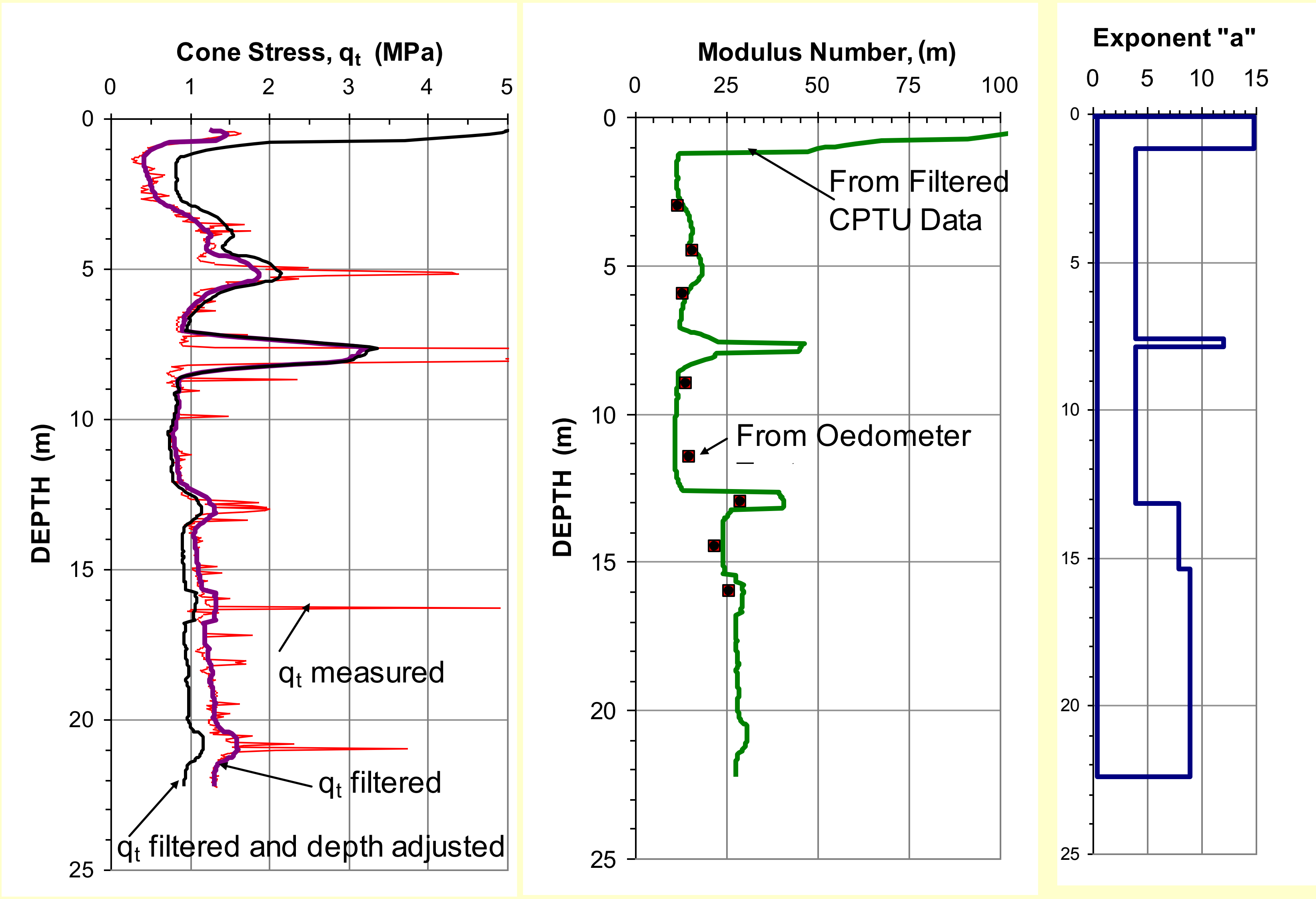
Soil Type	Modulus Modifier, a
Clay, soft	3
Clay, firm	5
Silt, organic soft	7
Silt, loose	12
Silt, compact	15
Silt, dense	20
Sand, silty loose	20
Sand, loose	22
Sand, compact	28
Sand, dense	35
Gravel, loose	35
Gravel, dense	45

Massarsch and Fellenius, 2001

Example of Modulus Number, m, determined from a CPTU Sounding



Massarsch and Fellenius, 2001

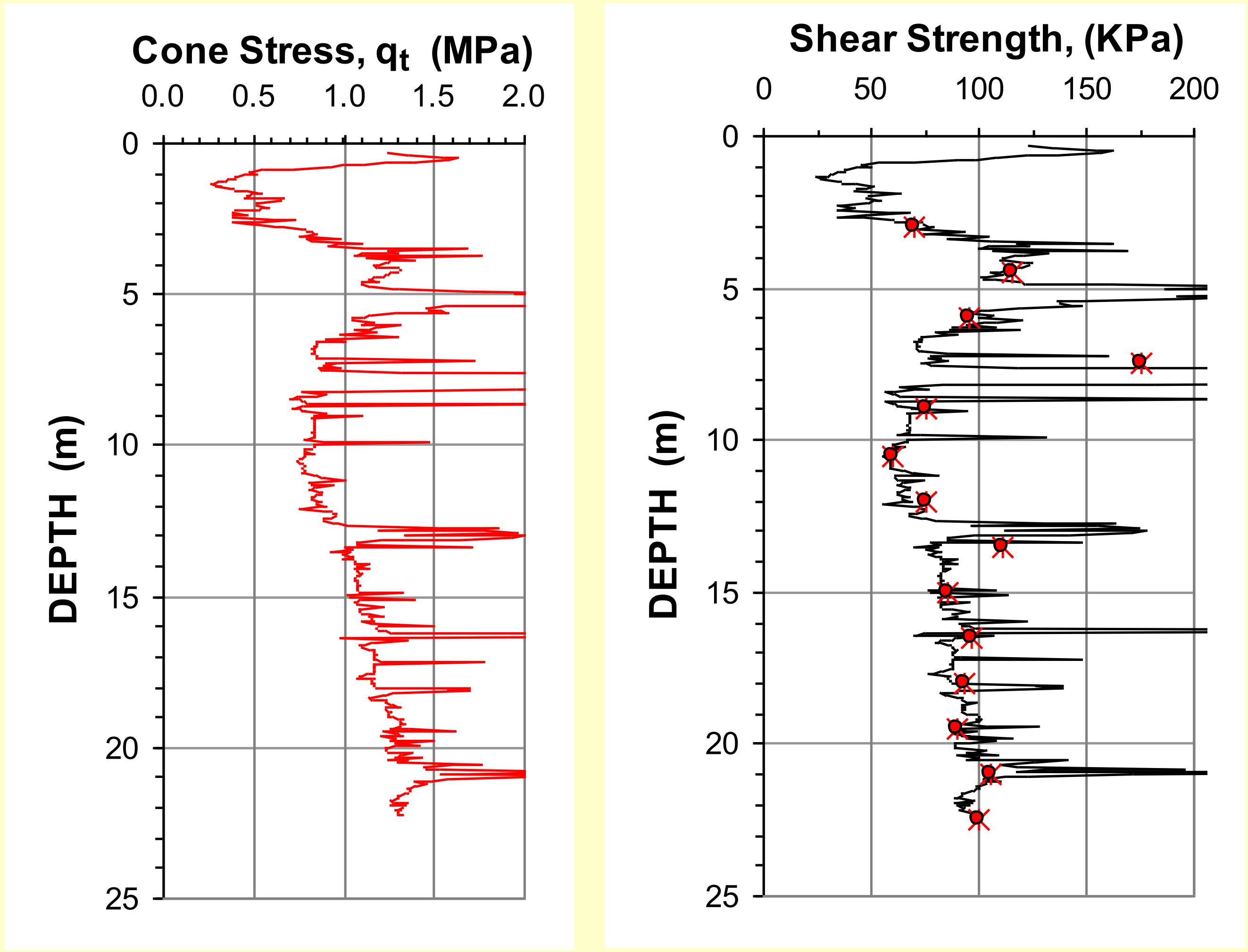


Example 3. Paddle River, Alberta

Determining Undrained Shear Strength

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

τ_u	=	undrained shear strength
q_t	=	cone resistance corrected for pore water pressure on shoulder
σ_v	=	total overburden stress
N_{kt}	=	a coefficient; $10 < N_{kt} < 20$



Cone stress (q_t) and undrained shear strength profiles. The latter is **fitted** to a vane shear profile from a test next to the CPTU sounding using $N_{kt} = 10$.

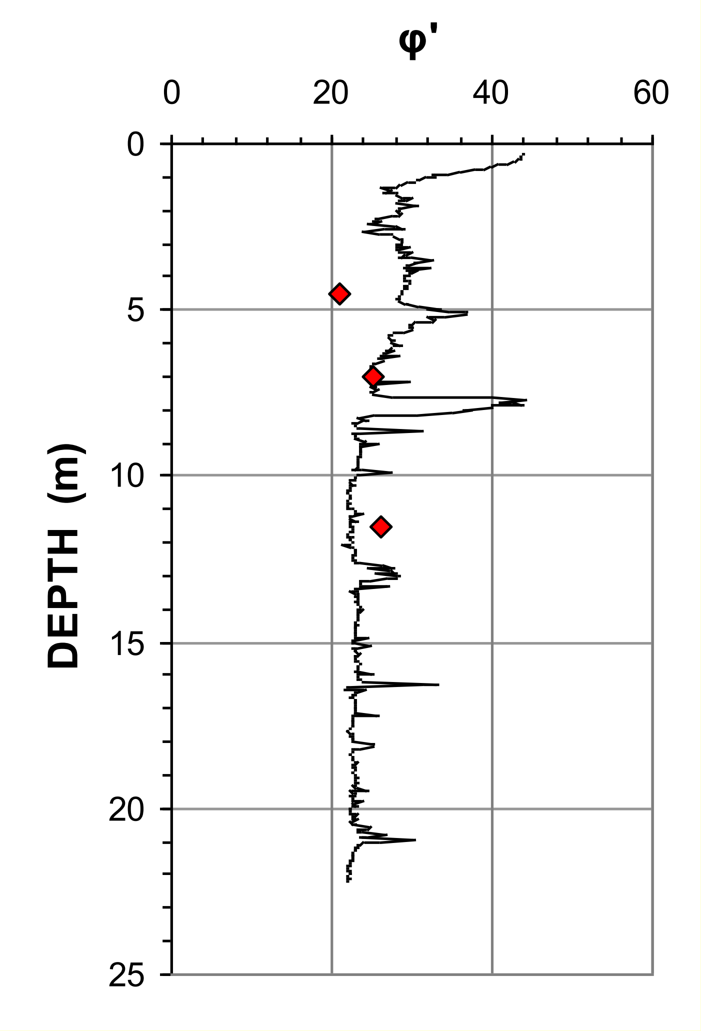
Paddle River, Alberta

Determining Friction Angle

$$tg\phi' = C_\phi \lg \frac{q_t}{\sigma'_v} + K_\phi$$

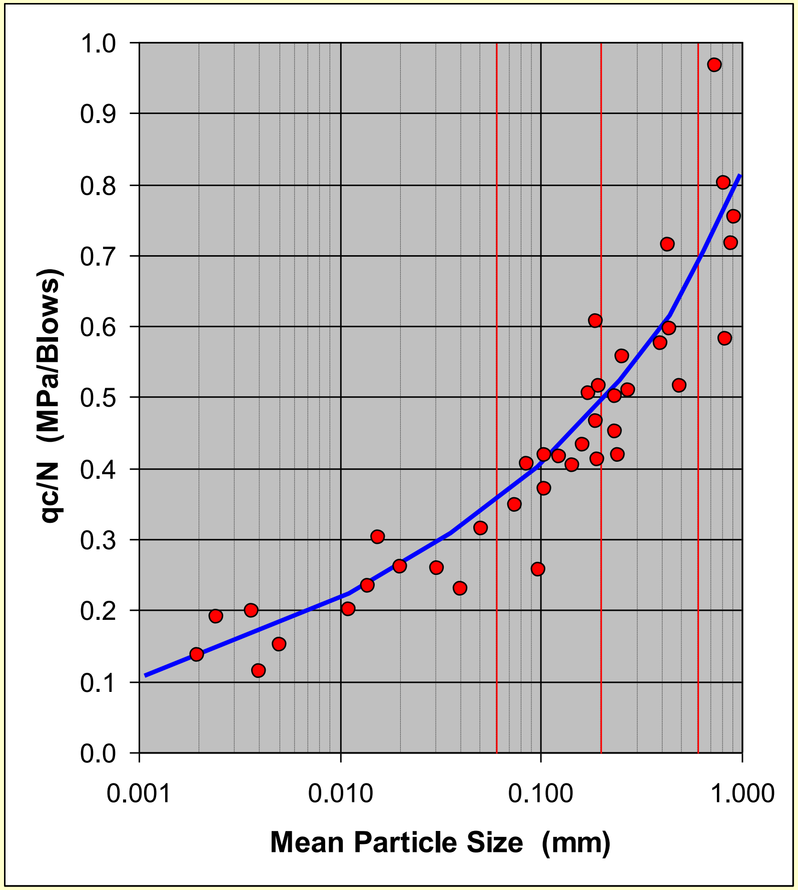
ϕ'	=	effective friction angle
C_φ	=	*a coefficient; C_φ = 0.37 (= 1/2.68)*
K_φ	=	*a coefficient; K_φ = 0.1*
q_t	=	cone resistance corrected for pore water pressure on shoulder
σ'_v	=	effective overburden stress

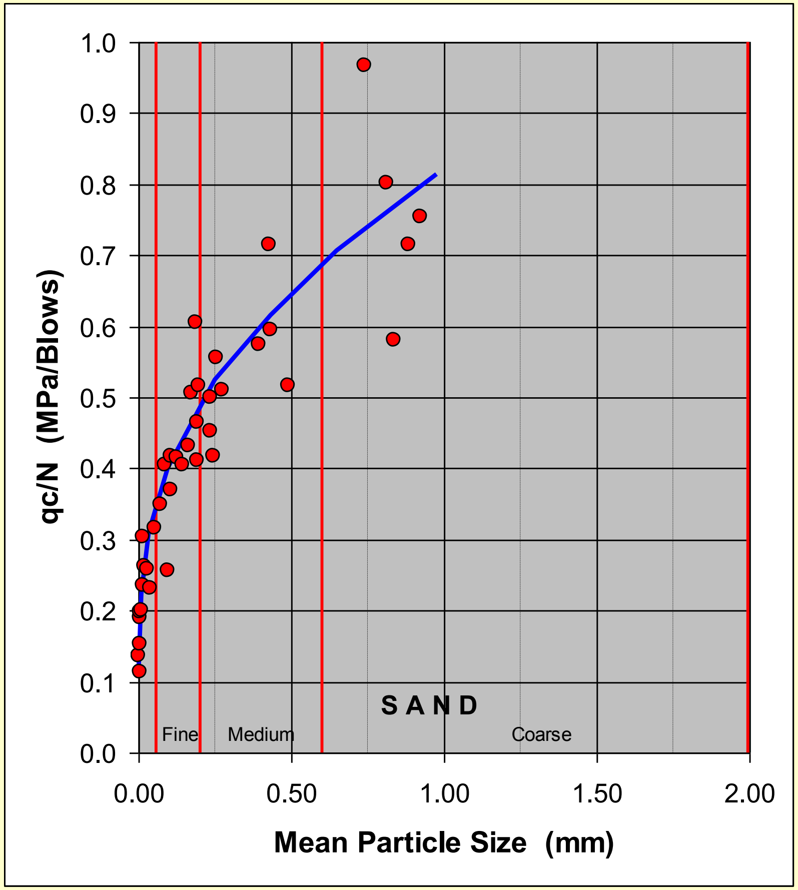
Applications



Friction angle, φ', profile determined from the CPTU sounding with three values from triaxial tests. The basic 0.37 C_φ and K_φ coefficients are used.

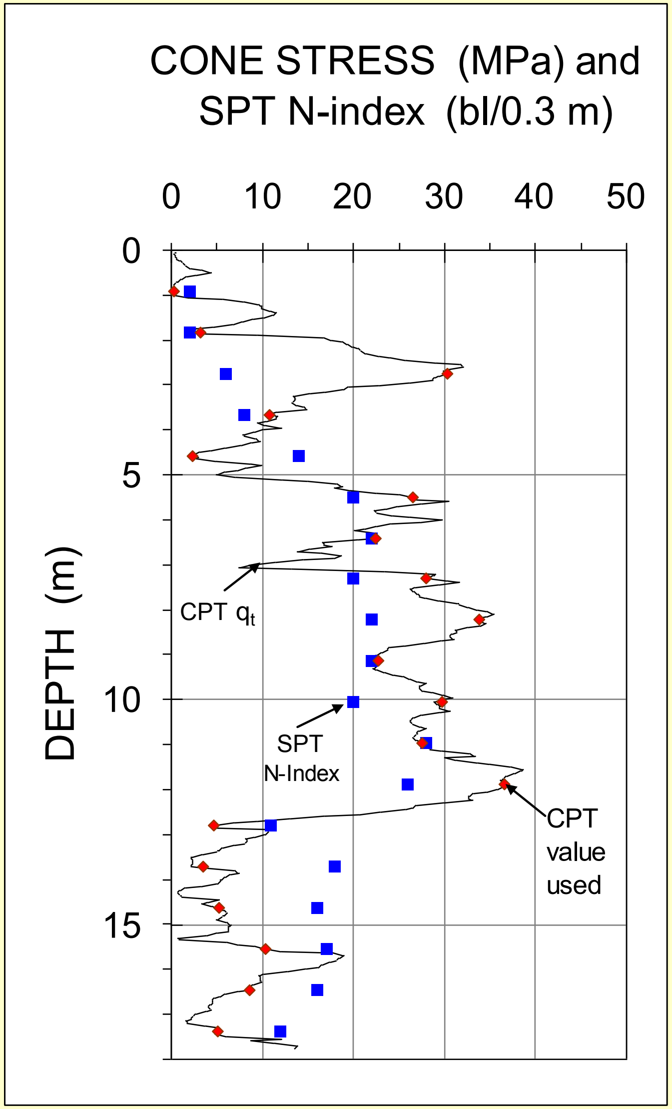
Determining SPT Index

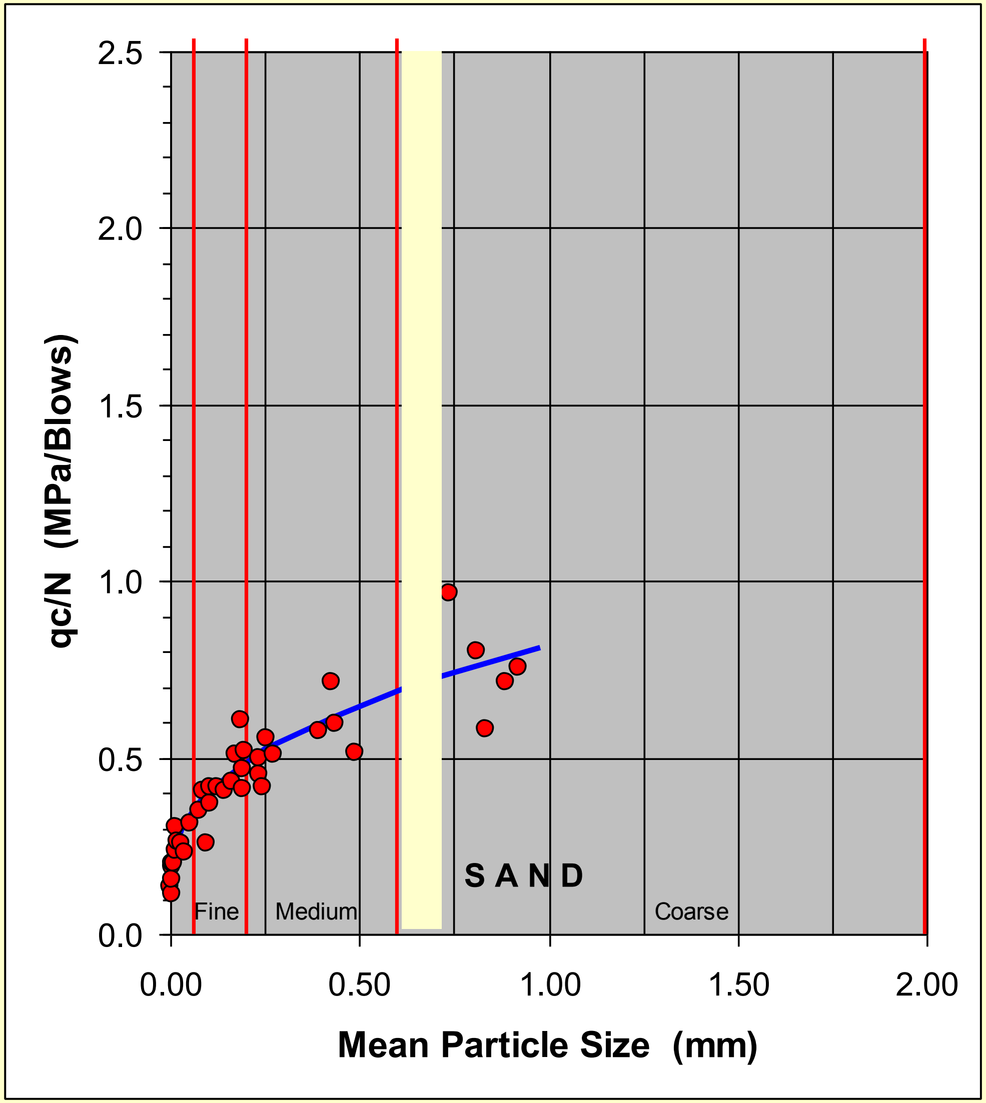




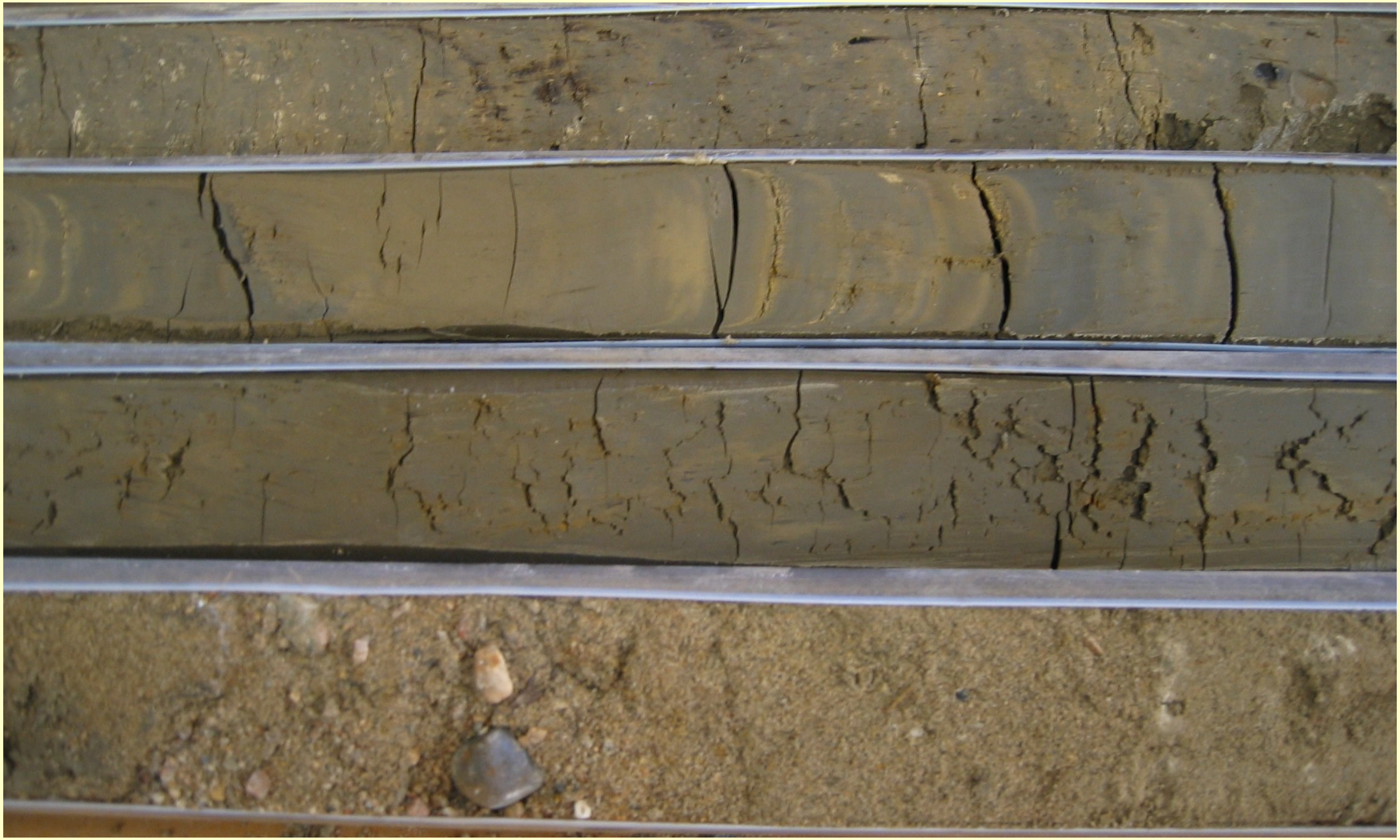
Robertson et al. (1983)

Example from North East Florida





Data from McVay et al. (1999)



Liquefaction

7.4 Magnitude Earthquake of August 17, 1999

Kocaeli, Adapazari, Turkey

Photo courtesy of Noel J. Gardner, Ottawa

Photo courtesy of Noel J. Gardner, Ottawa

Assessment of liquefaction risk from results of a CPTU sounding

An earthquake generates a Cyclic Stress Ratio, CRR

$$CSR = 0.65 \frac{a_{\max}}{g} \frac{\sigma_v}{\sigma_v'} r_d$$

For earthquake magnitude of 7.5

CSR	=	Cyclic Stress Ratio
a_{max}	=	maximum horizontal acceleration at ground surface (m/s^2)
g	=	gravity constant (m/s^2)
r_d	=	stress reduction coefficient for depth, dimensionless
z	=	depth below ground surface (m)

The safety against liquefaction depends on the Cyclic Resistance Ratio, CRR, determined from the CPTU data

$$F_s = \frac{CRR}{CSR}$$

For earthquake magnitude of 7.5

The Cyclic Resistance Ratio, CRR, is expressed in two equations

$$CRR = 0.833\left(\frac{q_{c1}}{100}\right) + 0.05 \quad for \quad q_{c1} < 50 \qquad CRR = 93\left(\frac{q_{c1}}{100}\right)^3 + 0.08 \quad for \quad 50 < q_{c1} < 160$$

The following fitted equation represents both above

$$CRR = 0.045(e^{0.14 q_{c1}})$$

where

$$q_{c1} = q_c\, C_{N\,c1} = q_c \sqrt{\frac{\sigma_r}{\sigma_v^{'}}}$$

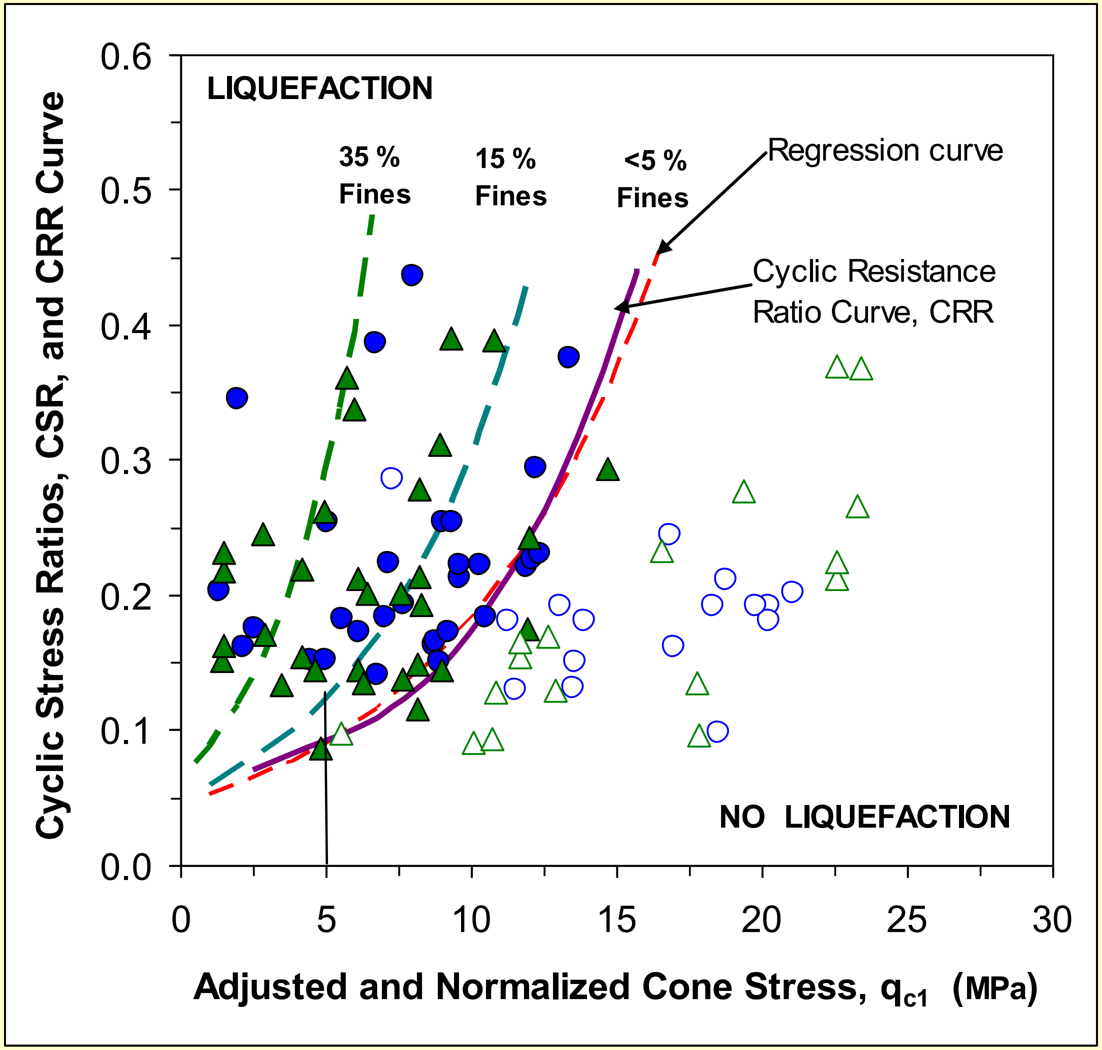
C_{Nc1} = normalization factor

σ_r = reference stress = 100 KPa (= atmospheric pressure)

σ'_v = effective overburden stress at the depth of the cone stress considered (KPa)

Determining seismic risk from CPTU sounding

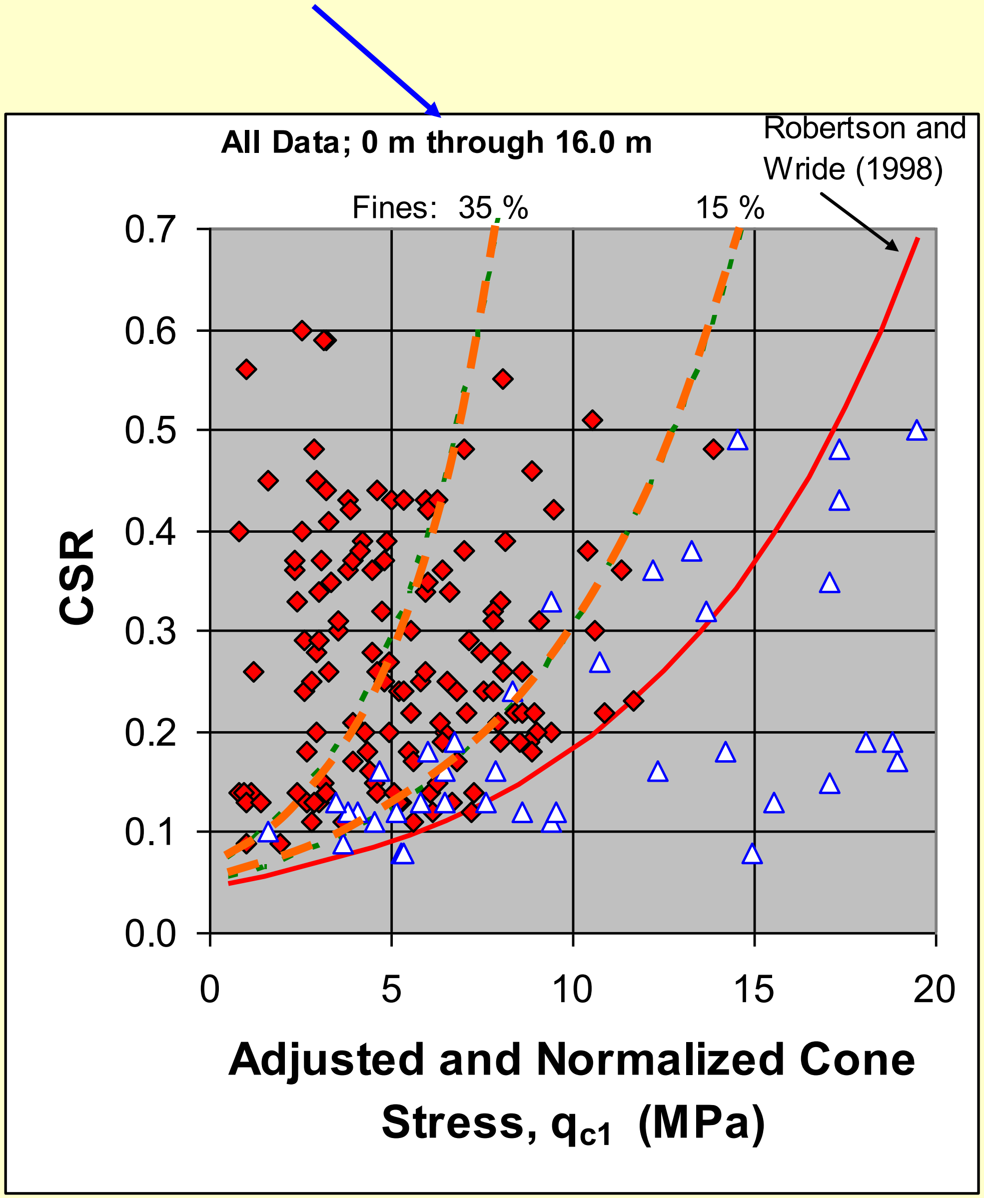
Every plotted point represents an earthquake observation with either no liquefaction of with liquefaction observed



Correlations between CRR-values calculated from actual earthquakes versus q_{c1} values for cases of liquefaction (solid symbols) and no liquefaction (open symbols), and boundary curve (solid line) according to **Robertson and Wride (1998) and Youd et al. (2001)**.

The boundary line is the Cyclic Resistance Ratio Curve, CRR, which is also shown as a linear regression curve for the boundary values. The two dashed curves show the boundary curves for sand with fines contents of 15% and 35%, respectively (Stark and Olsen 1995). The original diagram has the cone stress, q_c, divided by atmospheric pressure to make the number non-dimensional.

Note, the effect of fines contents has lately become challenged.

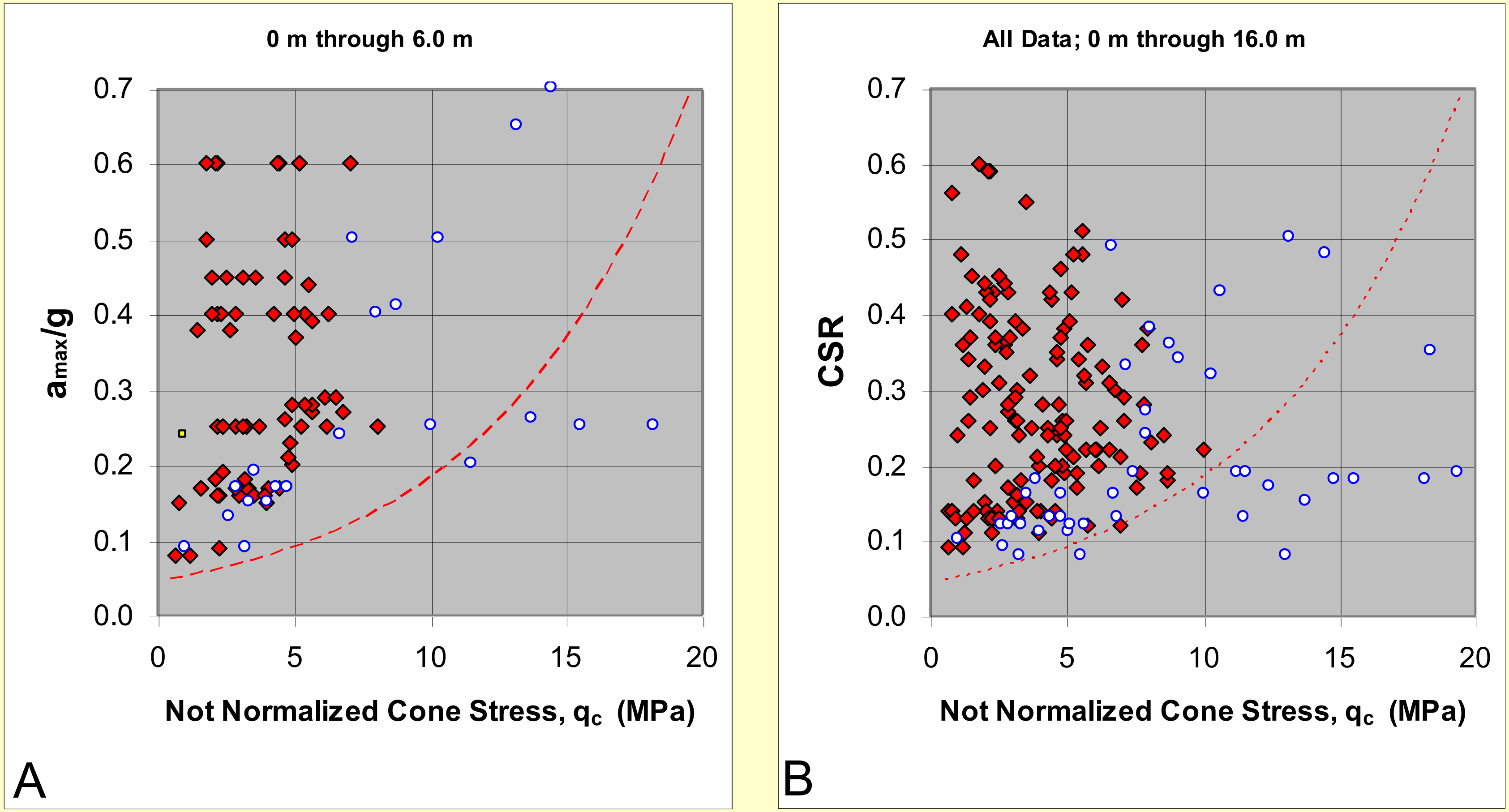


Correlations between CRR-values calculated from actual earthquakes versus q_{c1} values for cases of liquefaction (solid symbols) and no liquefaction (open symbols), according to Moss et al. (2006) and boundary curve (solid line) according to Robertson and Wride (1998) and Youd et al. (2001).

Again, note that the effect of fines contents has lately become challenged.

Separating on two depths and looking at relative seismic force versus not-normalized cone stress.

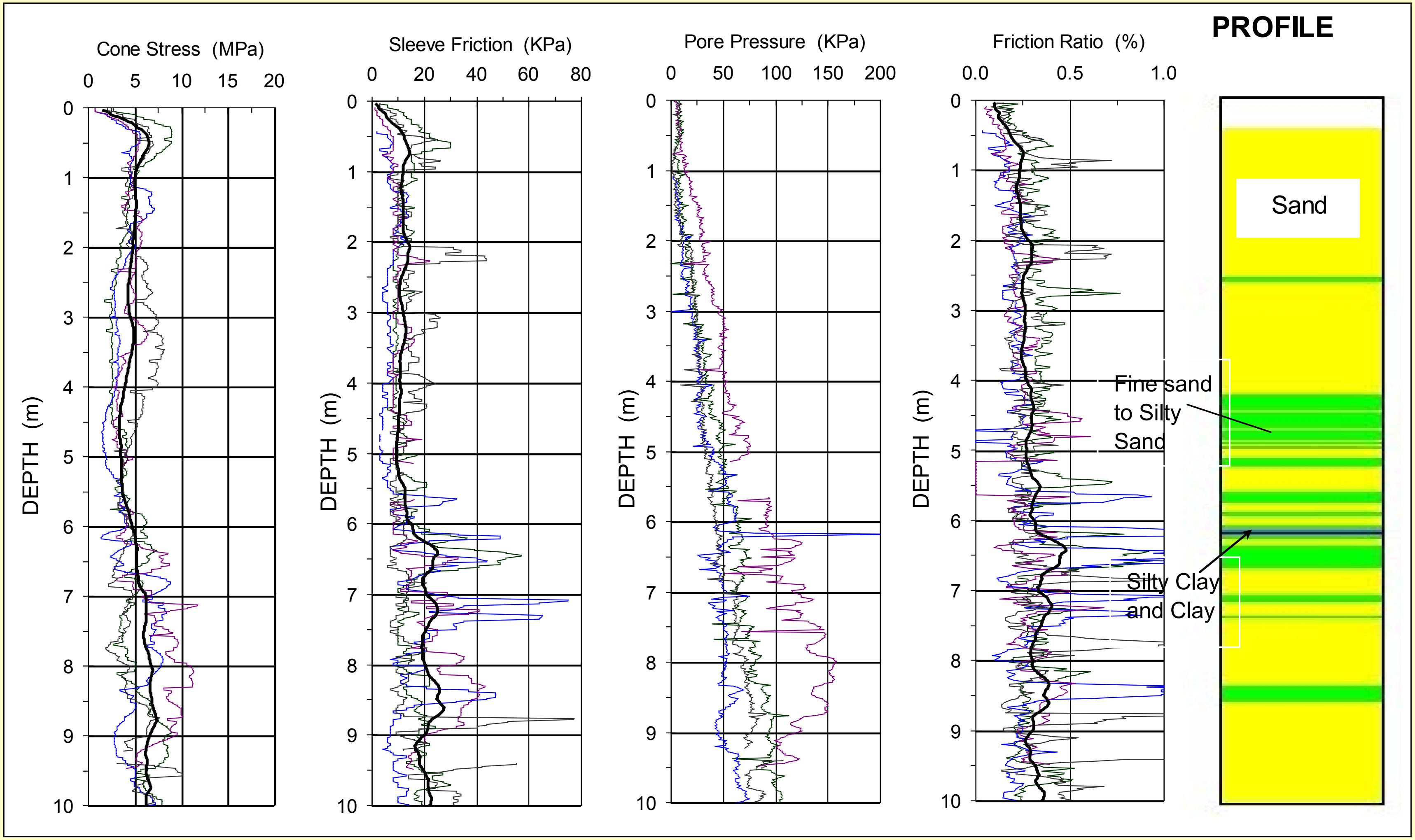
Re-analysis of data from Moss et al. (2006)



The 'old' rule that liquefaction risk is small for shallow depth where the cone stress is ≥5 MPa appears to hold for quake ratio < 0.25.

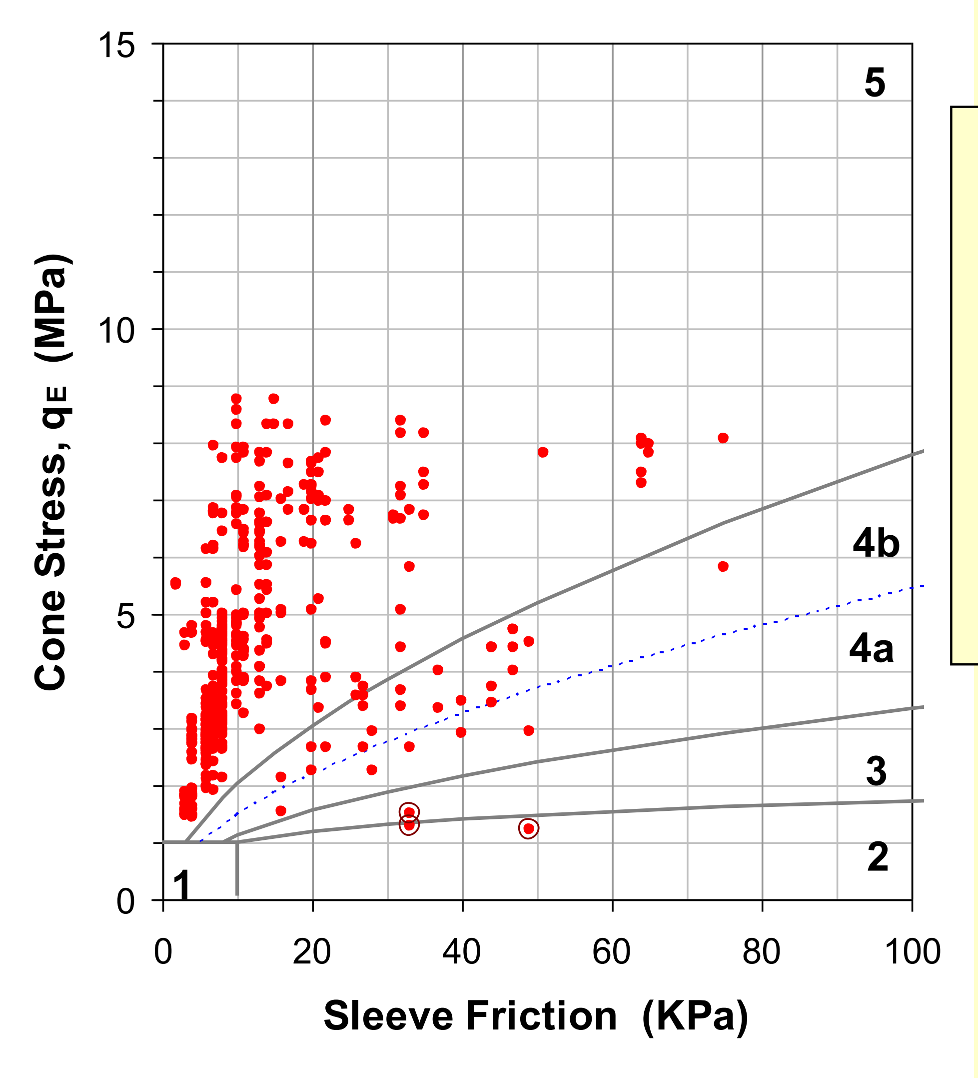
In the past, liquefaction risk was based on values of the SPT N-index. Correlations between the CPTU, q_c, and the N-index indicate a ratio between q_c and N of about 5. However, that ratio has a very large range between low and high. It is questionable how relevant and useful a conversion from an SPT Index value to a cone stress would be for an actual site. One would be better served pushing a cone in the first place.

Example of determining liquefaction susceptibility



Four CPTU initial (before compaction) soundings at Chek Lap Kok Airport. The heavy lines in the cone stress, sleeve friction, and friction ratio diagrams are the geometric averages for each depth of the four soundings.

Soil chart



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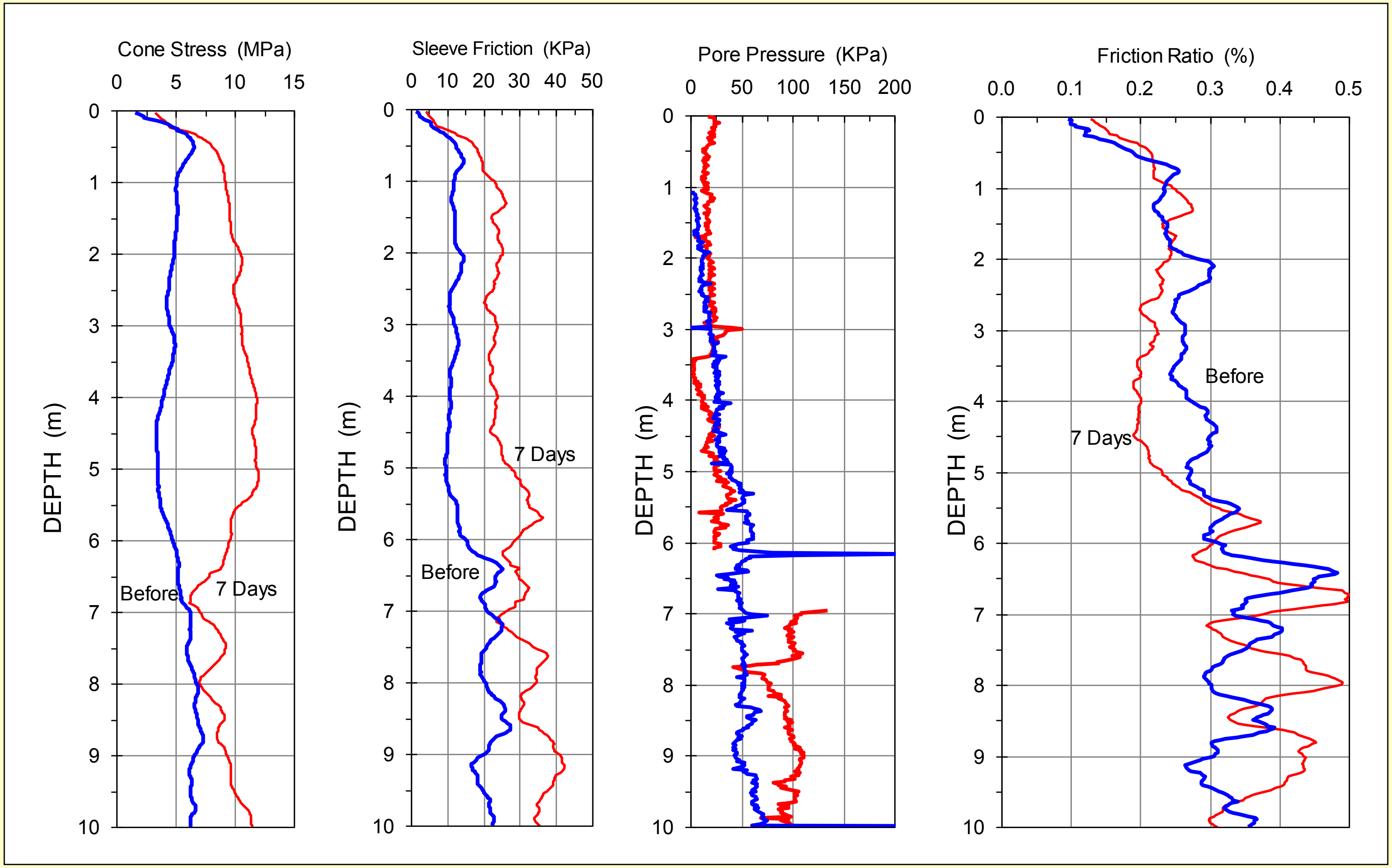
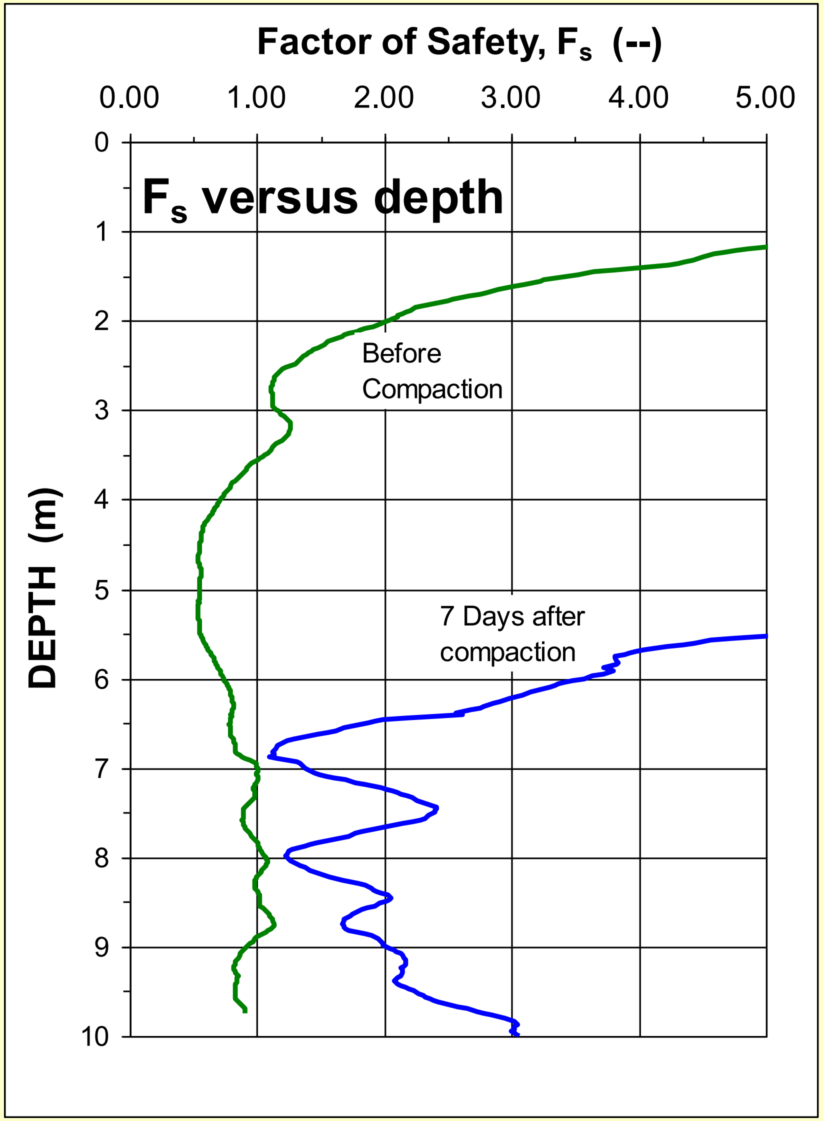


Geometric average values of cone stress, sleeve friction, and friction ratios and measured pore pressures from CPTU soundings at Chek Lap Kok Airport before and seven days after the vibratory compaction.

$$F_s = \frac{CRR}{CSR}$$



Factor of safety against liquefaction **before** and **after** vibratory compaction

CPT and CPTU Methods for Calculating the Ultimate Resistance (Capacity) of a Pile

Schmertmann and Nottingham (1975 and 1978)

Meyerhof (1976)

deRuiter and Beringen (1979)

LCPC, Bustamante and Gianeselli (1982)

Eslami and Fellenius (1997)

ICP, Jardine, Chow, Overy, and Standing (2005)

But we will save those methods for later

Vaughani Shores, Vanuatu *[www.DiveVanuatu.org]*