

Interpretation in fine grained deposits



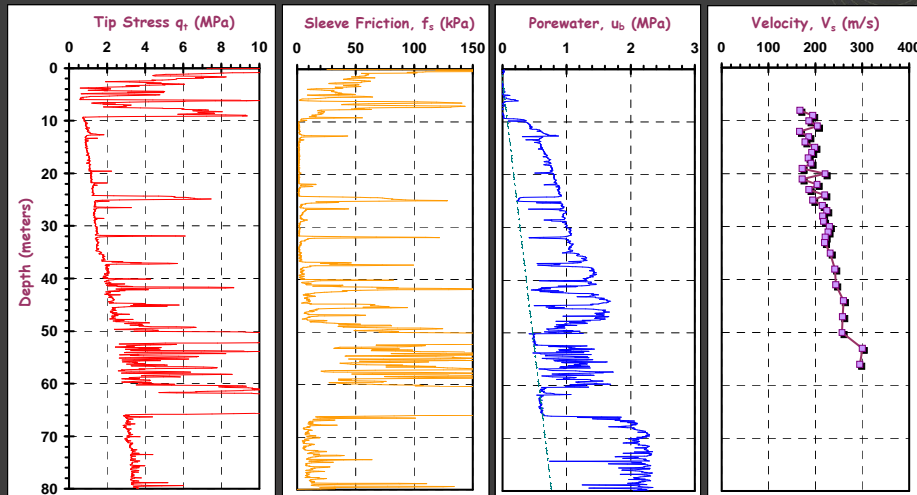
Interpretation in fine grained deposits



- Cone penetration in fine grained soils is generally undrained and so pore pressures will be generated and can be measured.
- The pore pressures generated affect measurements of both cone resistance and sleeve friction.
- So Cone resistance and (if relevant) sleeve friction should be corrected using the measured pore pressures.
- The measured pore pressures can also be used directly for interpretation in terms of soil design parameters.



SCPTu in Soft Clays,



Sandpoint, Idaho



Interpretation of CPTU data in clay

Parameters that we can try to estimate from CPTs in fine grained soils are:

- State characteristics (In situ stresses and stress history)
- Strength characteristics
- Deformation characteristics
- Flow and consolidation characteristics
- In situ pore pressure



Interpretation of CPTU data in clay

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State characteristics

1. Soil Unit weight: γ_w for computation of in situ vertical effective stress (σ'_{v0})
2. Stress history
 σ'_p and OCR = σ'_p / σ'_{v0}
3. In situ horizontal effective stress
 $\sigma'_{h0} = K_0 \sigma'_{v0}$



State characteristics

- *Soil unit weight*
- *Stress History*
- *In situ horizontal stress*



Soil unit weight

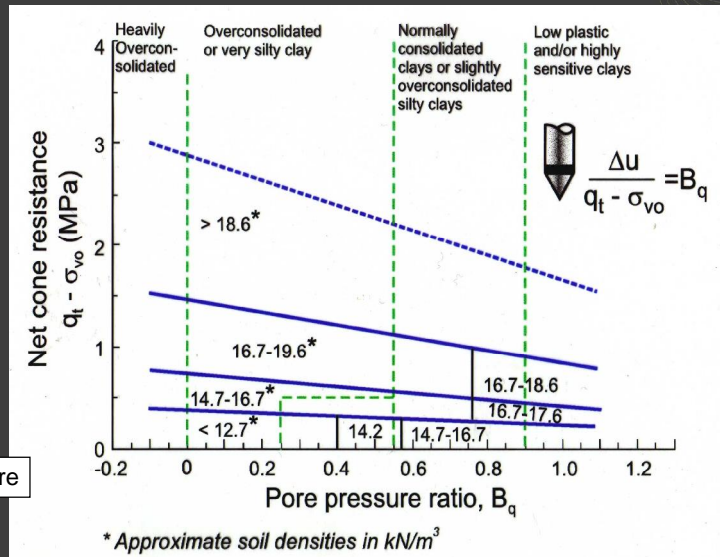
- Larsson and Mulabdic proposed a chart for obtaining a rough estimate of soil density for clays. An iteration is necessary since soil density is also needed for computation of net cone resistance ($q_t - \sigma_{v0}$) and B_q .



Soil unit weight

Larsson and Mulabdic

Iterative procedure



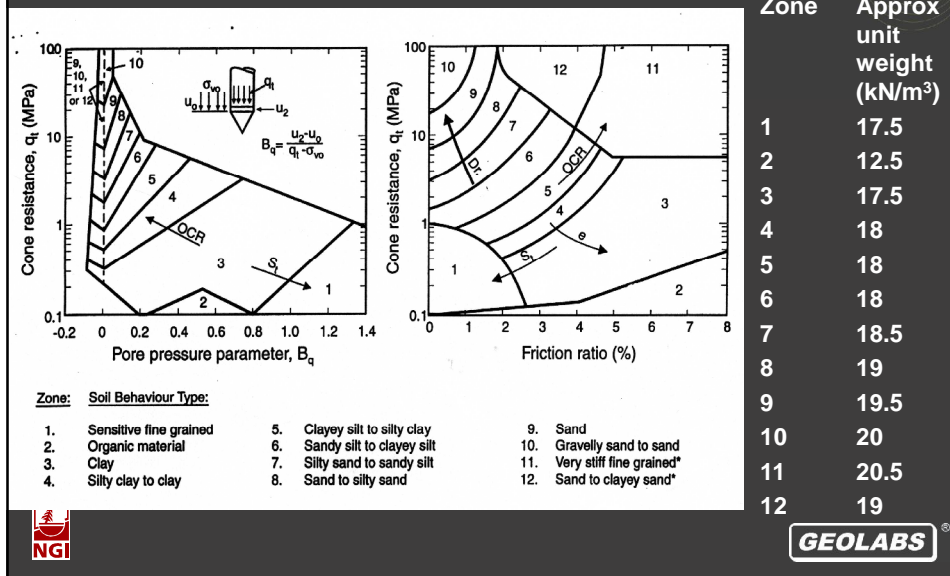
Soil unit weight

- Soil unit weight can also be estimated using the soil classification chart and using the unit weights for each soil zone.



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Soil behaviour type classification chart from Robertson et al. (1986)

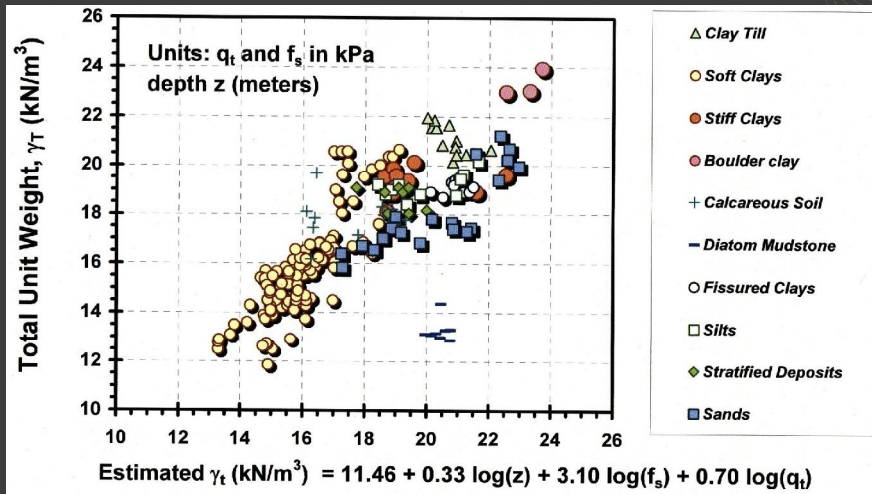


Soil unit weight

- If no other information, such as adjacent boreholes or local experience, is available then these charts can be used as a preliminary 'approximate' estimate of soil unit weight.



Unit weights



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Robertson and Cabal (2010)

- $\gamma / \gamma_w = 0.27 [\log R_f] + 0.36 [\log(q_t/p_a)] + 1.236$



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State characteristics

- *Soil unit weight*
- *Stress History*
- *In situ horizontal stress*



Overconsolidation Ratio

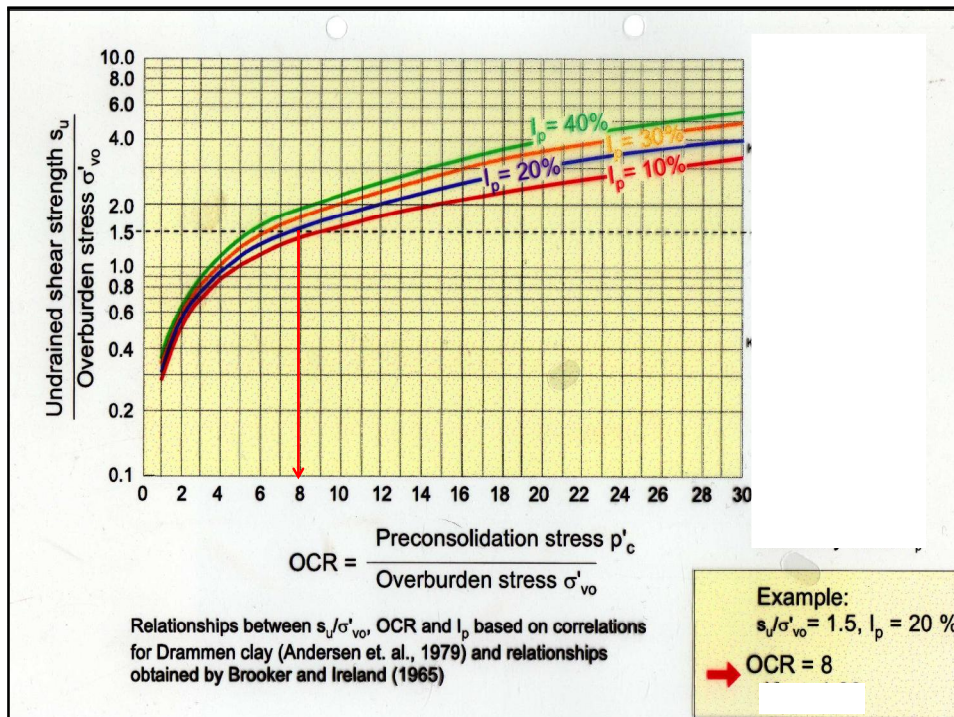
Interpretation of CPTU data for OCR can use methods either:

- *Indirectly from undrained shear strength*
- *directly on CPTU data*



Overconsolidation Ratio

- From undrained shear strength
 - Estimate s_u from CPT/CPTU data
 - Estimate effective vertical stress, σ_{vo}' , from soil profile (using laboratory density data if possible) and compute s_u/σ_{vo}' .
 - Estimate the corresponding normally consolidated (NC) value of s_u/σ_{vo}' using measured or estimated plasticity index (I_p).
 - Estimate OCR from published correlations.



List of piezocone parameters for estimating OCR (modified from Mayne, 1991).

Parameter	Basis	Reference
1. u_m/q_c	Empirical	Baligh <i>et al.</i> , 1980
2. $\Delta u/q_c$	Empirical	Campanella & Robertson, 1981
3. $B_q = \Delta u/(q_c - \sigma_{v0})$	Empirical	Senne set, Janbu & Svane, 1982
4. $B_q = \Delta u/(q_t - \sigma_{v0})$	Empirical	Wroth, 1984
5. $\Delta u/(q_c - u_0)$	Empirical	Smits, 1982
6. $\Delta u/\sigma'_{v0}$	Empirical Theory	Azzouz <i>et al.</i> , 1983 Mayne & Bachus, 1988
7. $N_u = \Delta u/\sigma'_{v0}$	Empirical	Tavenas & Leroueil, 1988
8. $q_t - \sigma_{v0}$	Empirical	Tavenas & Leroueil, 1988
9. $q_t - u_m$	Theory	Konrad & Law, 1987
10. Δu	Empirical	Mayne & Holtz, 1988
11. $q_t - u_0$	Theory	Sandven, Senne set & Janbu, 1988
12. $(q_t - \sigma_{v0})/\sigma'_{v0}$	Theory	Wroth, 1988
13. $(u_1/u_0) - (u_2/u_0)$	Empirical	Sully, <i>et al.</i> , 1988
14. q_t, u_1, f_s	Empirical	Rad & Lunne, 1988
15. $(q_t - u_2)/\sigma'_{v0}$	Theory	Houlsby, 1988 and Mayne, 1991
16. $f_t/(q_t - \sigma_{v0})$	Empirical	Wroth 1984



CPTU Stress History Correlations

Wroth (1984), Mayne(1991) and others proposed theoretical basis (cavity expansion; critical state soil mechanics) for the following potential correlations between CPTU data and σ'_p or OCR:

$$\sigma'_p = f(\Delta u_1 \text{ or } \Delta u_2)$$

$$\sigma'_p = f(q_t - \sigma_{v0})$$

$$\sigma'_p = f(q_t - u_2)$$

$$\text{OCR} = f(B_q = \Delta u_2/(q_t - \sigma_{v0}))$$

$$\text{OCR} = f(Q_t = (q_t - \sigma_{v0})/\sigma'_{v0})$$

$$\text{OCR} = f((q_t - u_2)/\sigma'_{v0})$$

Most Common:

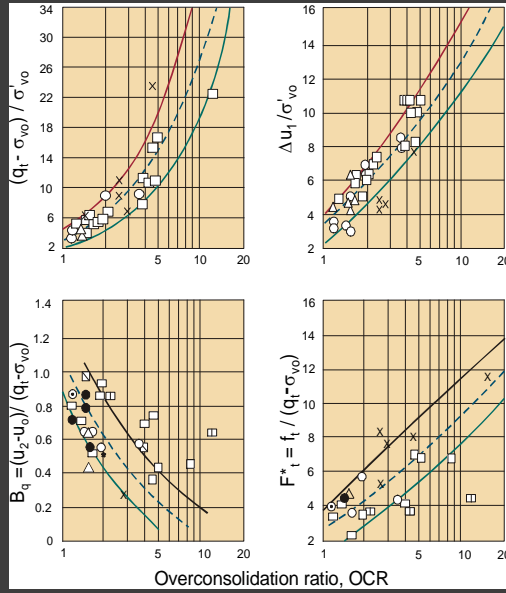
$$\sigma'_p = k (q_t - \sigma_{v0})$$

or

$$\text{OCR} = k [(q_t - \sigma_{v0})/\sigma'_{v0}]$$



Correlations to OCR



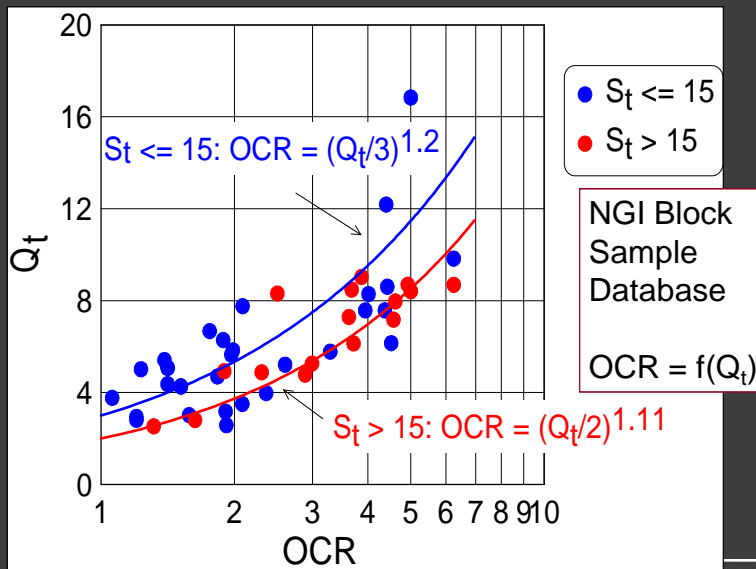
Following recommendations by Prof. P.Wroth 1984 Rankine lecture

Legend:

- △ Troll
- Brage
- Haltenbanken
- Haga
- * Rio
- ⊖ Vancouver
- X Cowden
- ▽ Brent Cross
- Onsøy
- Emmerstad
- ⊙ Drammen lean clay
- Drammen plastic clay



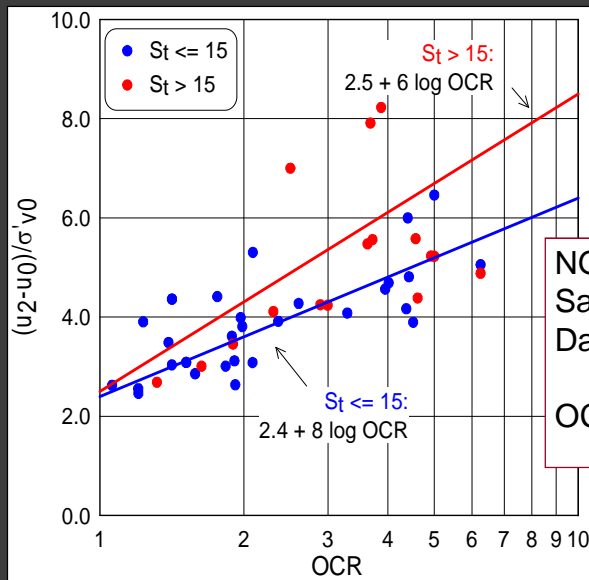
CPTU Stress History Correlations



[Karlsrud et al. 2005]



CPTU Stress History Correlations

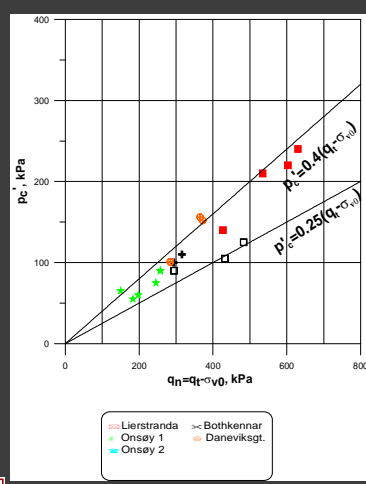


NGI Block Sample Database
 $OCR = f(\Delta u_2 / \sigma'_{v0})$

[Karlsrud et al. 2005]



Preconsolidation stress from CPTU parameters

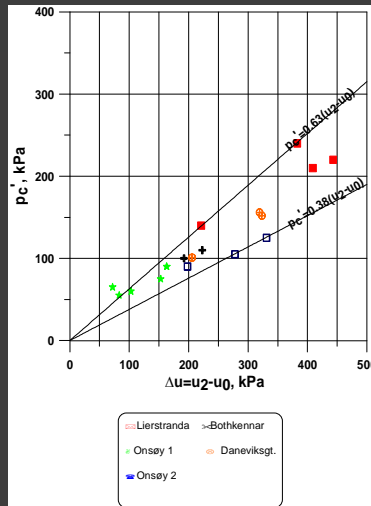


From data base of block samples and CPTU at Norwegian and UK reference test sites

Chen and Mayne(1996)
 $p_c' = 0.305 (q_t - \sigma_{v0})$
 $n = 1256, r^2 = 0.82$



Preconsolidation stress from CPTU parameters



From data base of block samples and CPTU at Norwegian and UK reference test sites

Chen and Mayne(1996)

$$p_c' = 0.53 (u_2 - u_0)$$

$$n = 811, r^2 = 0.722$$



Direct Evaluation of σ_p' in Clays from CPTu

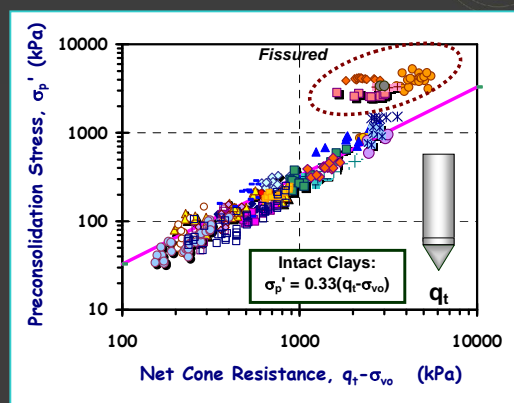
$$\sigma_p' = 0.33 (q_t - \sigma_{vo})$$

$$\sigma_p' = 0.47 (u_1 - u_0)$$

$$\sigma_p' = 0.53 (u_2 - u_0)$$

$$\sigma_p' = 0.60 (q_t - u_2)$$

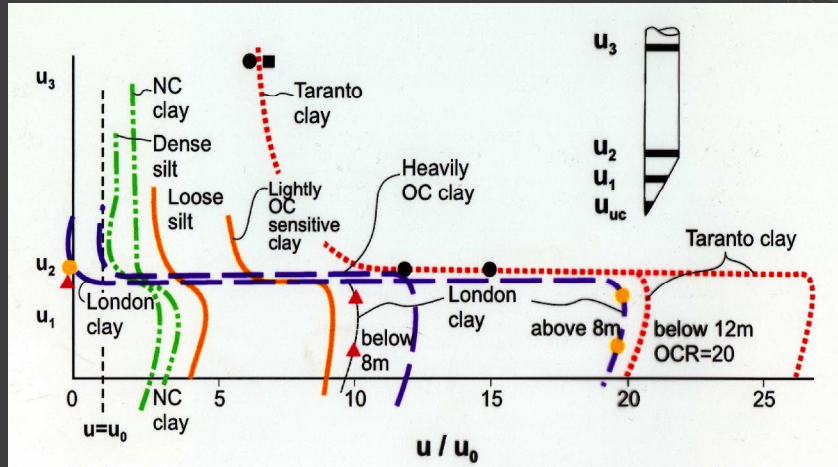
As a start



Redundancy is a good thing in geotechnics



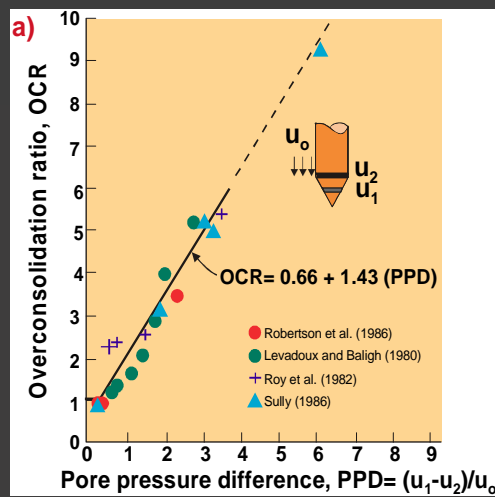
Measured pore pressure distributions in clay



After Robertson et al. (1986)

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OCR from dual CPTU data

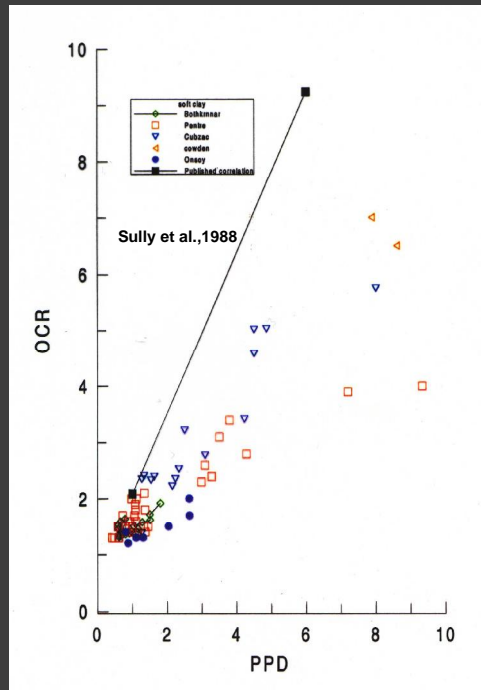


Sully et al., 1988



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PPD



Be careful, may not apply to your site



Overconsolidation Ratio

- Directly from CPTU data

It is recommended to estimate OCR in cohesive soils from CPT/CPTU data as follows:

- For deposits where little experience is available, estimate OCR based on the shape of the normalised cone resistance profile (Q_t). This relationship can be expressed by the following simple formula

$$OCR = k (q_t - \sigma_{vo}) / \sigma_{vo}'$$

- Or via σ_p'

$$\sigma_p' / \sigma_{vo}' = k (q_t - \sigma_{vo}) / \sigma_{vo}'$$

- an average value of $k = 0.3$ with a range of 0.2 - 0.5. Higher values of k are recommended in aged heavily overconsolidated clays



- Check also with correlations based on s_u / σ_{vo}' , B_q and $\Delta u / \sigma_{vo}'$



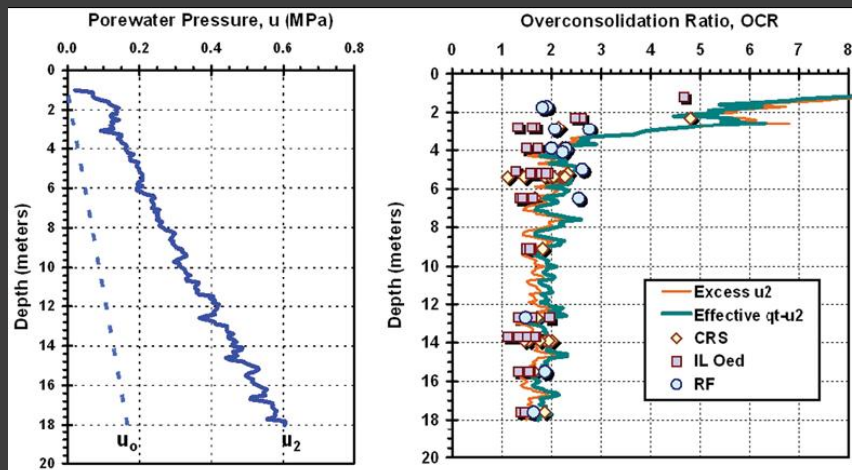
Overconsolidation Ratio and p_c'

- For projects where CPTU data are available it is recommended to also apply any correlations you can. Then estimate OCR based on a conservative average of consistent data.
- If previous experience is available in the same deposit, then the values suggested above should be adjusted to reflect this experience and to provide a more reliable profile of OCR.
- For larger projects, where additional high quality field and laboratory data may be available, site specific correlations should be developed based on consistent and relevant values of OCR.
- Consider the use of the new correlations for p_c' and convert to OCR



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OCR assessment

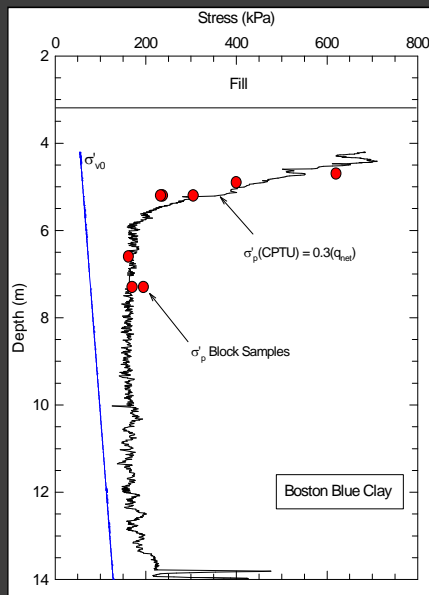


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Example - CPTU Stress History Correlation

Boston Blue Clay Site –
Newbury, MA.

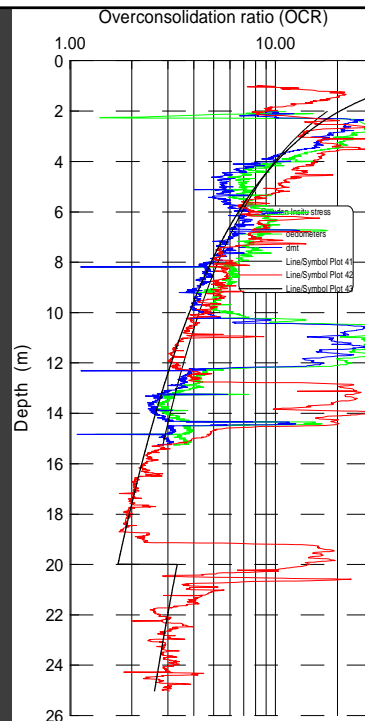
σ'_p values obtained from
Constant Rate of Strain
(CRS) Consolidation
tests conducted on high
quality Sherbrooke Block
samples



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OCR: Cowden

helping to establish history
sand layer vs gravel layer



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State characteristics

- *Soil unit weight*
- *Stress History*
- *In situ horizontal stress*



In situ horizontal stress

- Currently no truly reliable methods of determining in situ horizontal stress or coefficient of lateral earth pressure from CPTU data.

However to arrive at a rough estimate we can use methods based on:

- OCR
- measured pore pressure difference PPSV
- measurement of lateral stress or sleeve friction



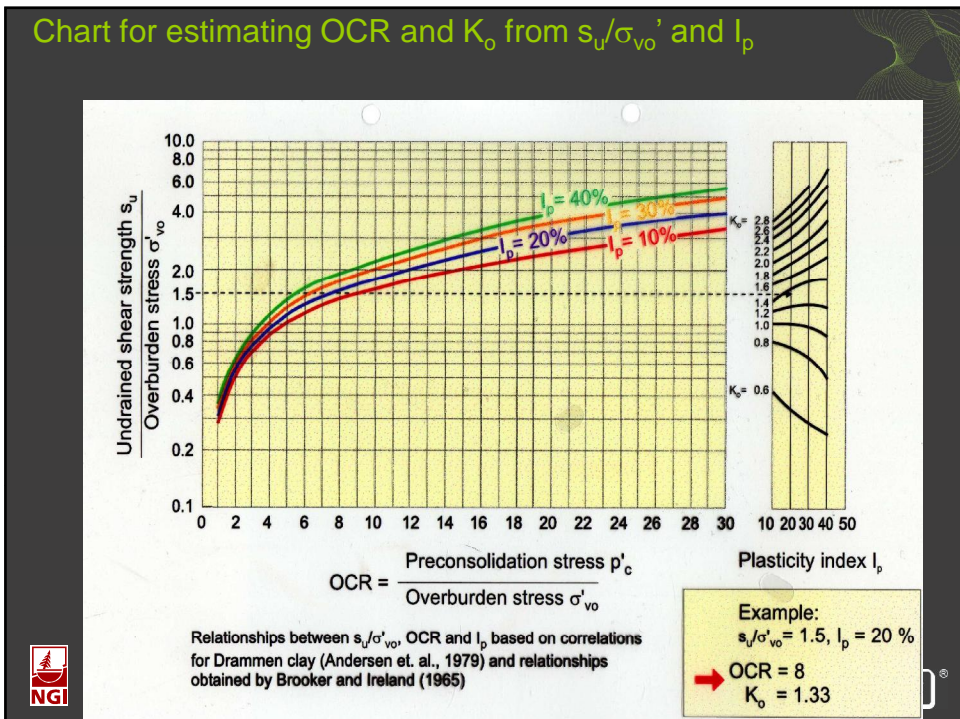
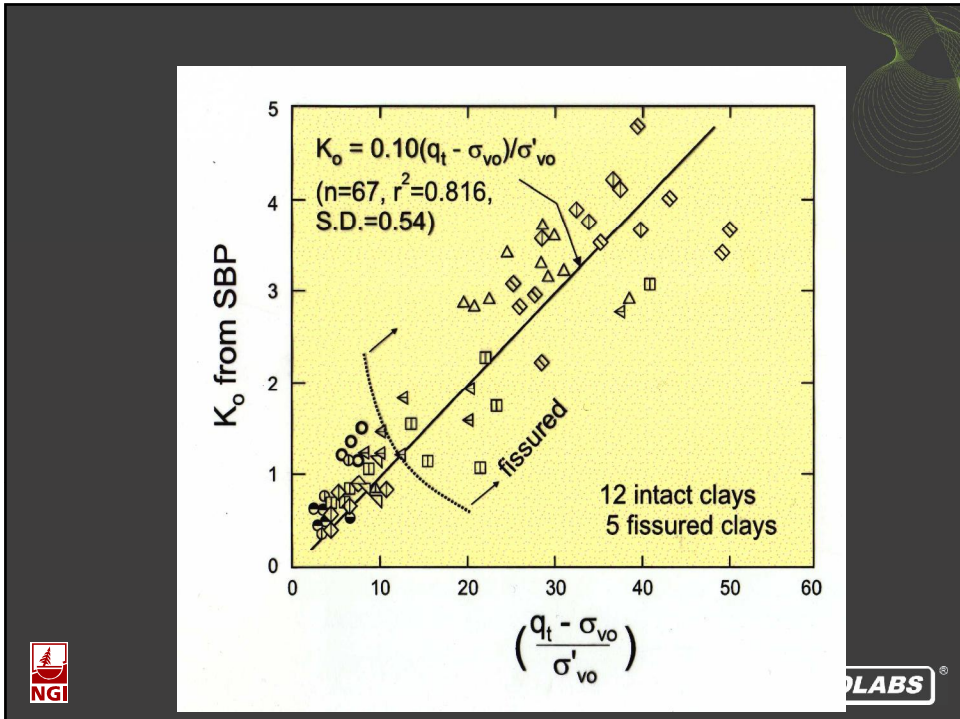
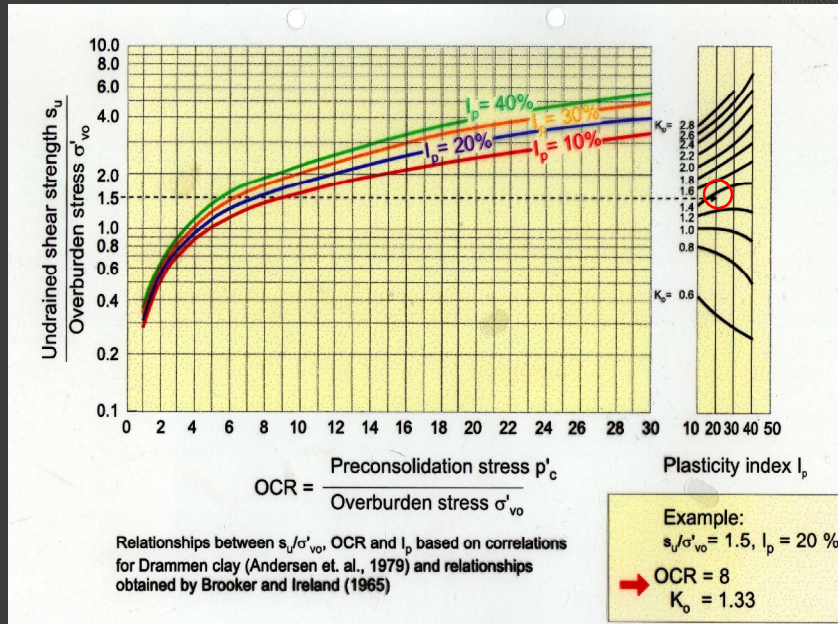
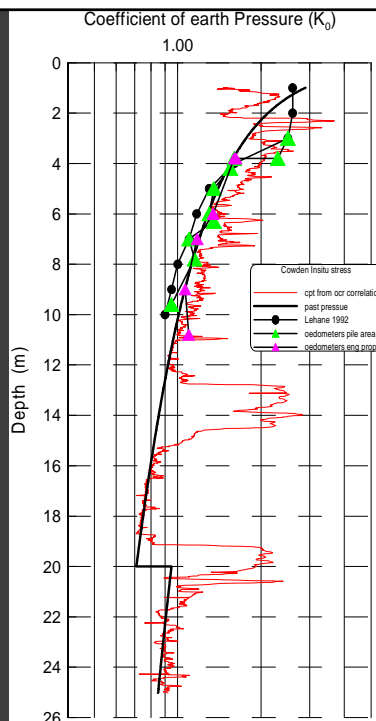


Chart for estimating OCR and K_0 from s_u/σ'_{vo} and I_p



K_0 :
Cowden
 helping to establish history
 But be careful



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Interpretation of CPTU data in clay

Parameters that we can try to estimate from CPTs in fine grained soils are:

- State characteristics (In situ stresses and stress history)
- **Strength characteristics**
- Deformation characteristics
- Flow and consolidation characteristics
- In situ pore pressure



Strength characteristics

- undrained shear strength
- sensitivity
- effective stress strength parameters



Strength characteristics

- undrained shear strength
- sensitivity
- effective stress strength parameters



Shear Strength of Clays

For most design problems in clays (especially loading) the critical failure condition is undrained.

1. Undrained Shear strength s_u ($= c_u$)
2. Remoulded undrained shear strength (s_{ur})
or Sensitivity, $S_t = s_u/s_{ur}$



Notes Regarding Undrained Shear Strength

1. The undrained shear strength is not unique.
2. The in situ undrained shear strength depends on many factors with the most important being: mode of shear failure, soil anisotropy, strain rate and stress history.
3. Therefore s_u required for analysis depends on the design problem.
4. Measured CPTU data are also influenced by such factors as anisotropy and rate effects.
5. The CPTU cannot directly measure s_u and therefore CPTU interpretation of s_u relies on a combination of theory and empirical correlations



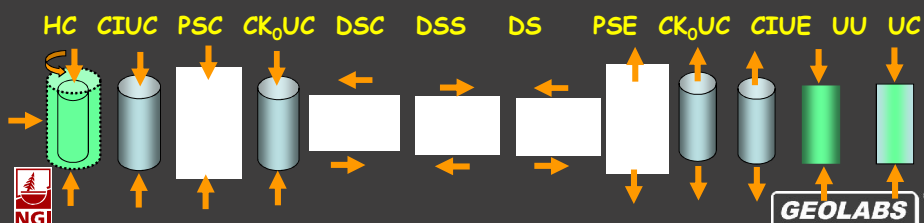
Undrained Shear Strength

- Classical interpretation from CPT in clays: undrained shear strength = $c_u = s_u$

$$s_u = \frac{q_t - \sigma_{vo}}{N_{kt}}$$

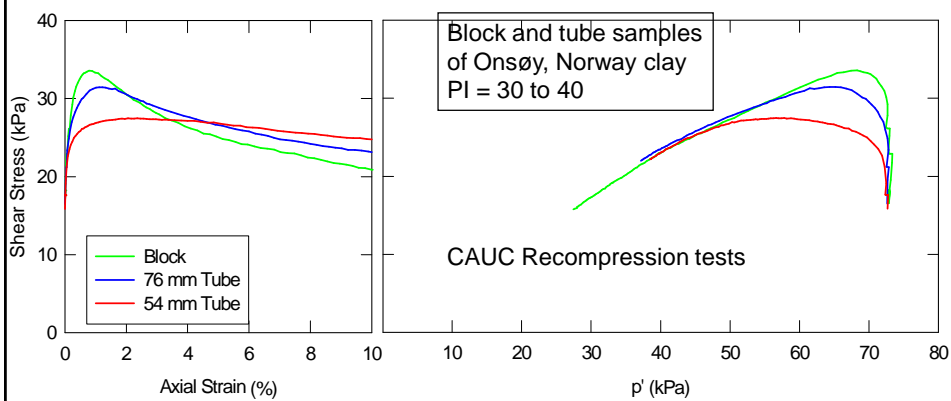
- Which s_u ?

$$N_k = 15$$



CPTU s_u Cone Factors – Karlsrud et al. (2005)

Update of CPTU s_u cone factors using NGI high quality block sample database. Derived cone factors as function: OCR, Sensitivity (S_u) and Plasticity Index (I_p)



Undrained shear strength

- It is very important that any interpretation of CPT/CPTU in terms of undrained shear strength should state which undrained shear strength it refers to; e.g. s_u corresponding to that measured in an anisotropically consolidated undrained triaxial test sheared in compression - CAUC.
- There are two main approaches of interpretation,
 - one based on theoretical solutions
 - one based on empirical correlations.



Undrained shear strength

- The theoretical solutions available can be grouped under the following 5 classes:
- Classical bearing capacity theory.
- Cavity expansion theory.
- Conservation of energy combined with cavity expansion theory
- Analytical and numerical approaches using linear and non-linear stress - strain relationships.
- Strain Path theory

All methods give :

$$q_t = N_c \cdot s_u + \sigma_o$$

($\sigma_o = \sigma_{vo} , \sigma_{ho} , \sigma_{meano}$) In practice use : $q_t = N_c \cdot s_u + \sigma_{vo}$



Undrained shear strength

- The empirical approaches available for interpretation of s_u from CPT/CPTU results can be grouped under 3 main categories as follows:
- s_u estimation using "total" cone resistance
- s_u estimation using "effective" cone resistance
- s_u estimation using excess pore pressure



Undrained Shear Strength from CPTU Data

$$s_u = q_{\text{net}}/N_{\text{kt}} = (q_t - \sigma_{v0})/N_{\text{kt}}$$

Most Common

$$s_u = \Delta u/N_{\Delta u} = (u_2 - u_0)/N_{\Delta u}$$

Often used

$$s_u = q_e/N_{ke} = (q_t - u_2)/N_{ke}$$

Seldom used

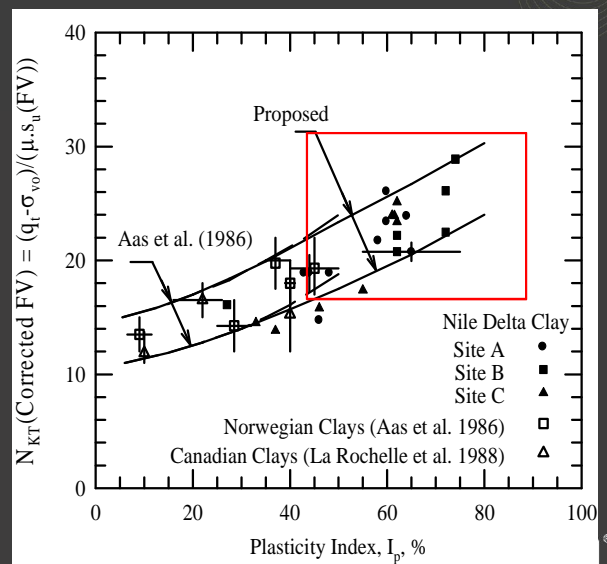
Need empirical correlation factors N_{kt} , $N_{\Delta u}$, or N_{ke} factors to be correlated to a specific measure of undrained shear strength, e.g., $s_u(\text{CAUC})$ or $s_u(\text{ave})$

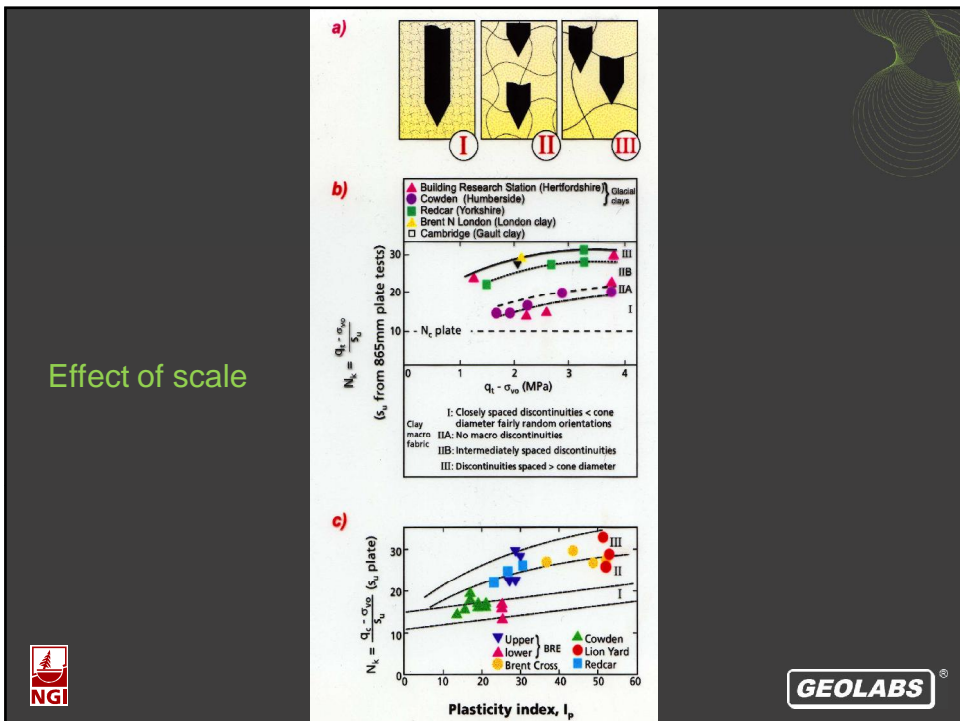
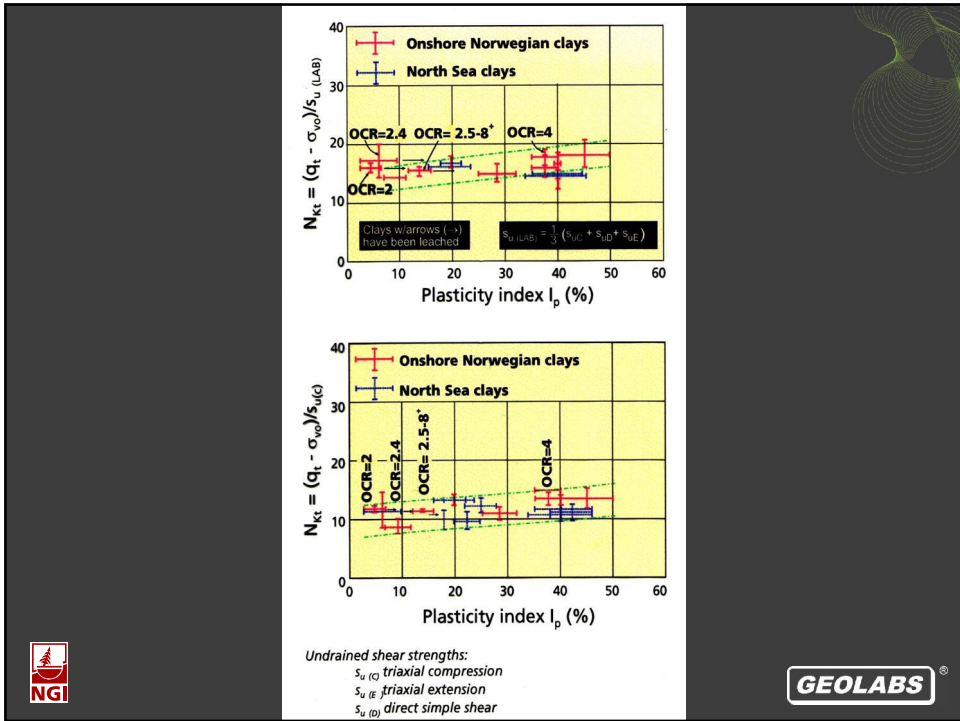


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Hamza et al

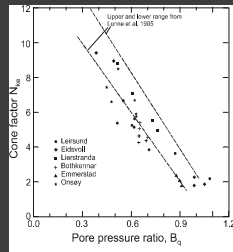
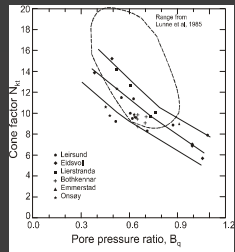
(data quality!!)



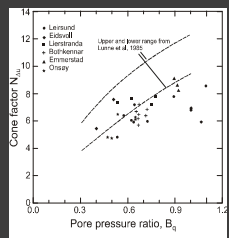


Cone factors from Norwegian and UK soft clay test bed sites

$$N_{kt} = (q_t - \sigma_{vo}) / s_{uCAUC}$$



$$N_{ke} = (q_t - u_2) / s_{uCAUC}$$

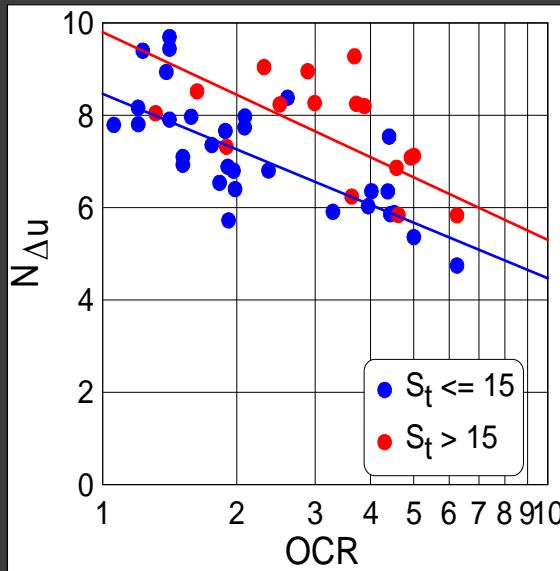


$$N_{\Delta u} = (u_2 - u_0) / s_{uCAUC}$$

Undrained shear strength from CAUC triaxial tests on Sherbrooke block samples



CPTU s_u Cone Factors – Karlsrud et al. (2005)

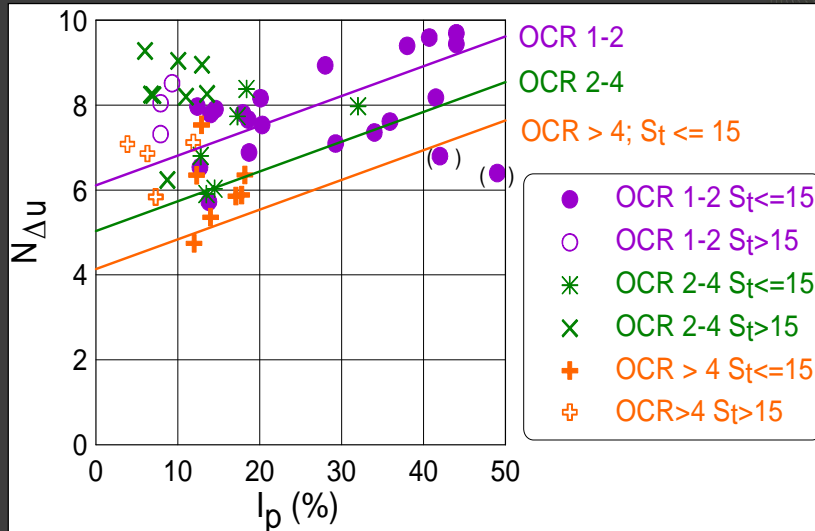


$$s_u = (u_2 - u_0) / N_{\Delta u}$$

[Karlsrud et al. 2005]



CPTU s_u Cone Factors – Karlsrud et al. (2005)



[Karlsrud et al. 2005]



CPTU s_u Cone Factors – Karlsrud et al. (2005)

Best fit regression lines to plotted data for s_u (CAUC)

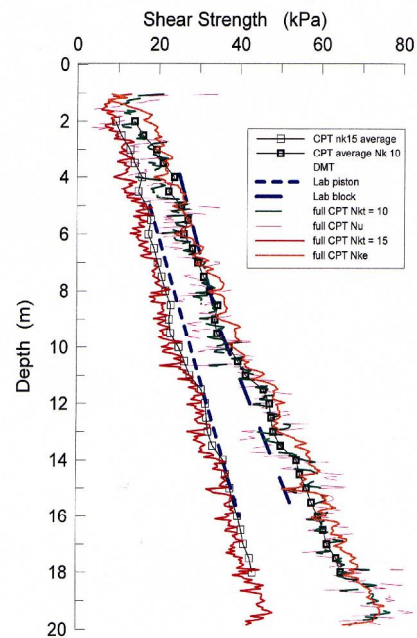
Cone Factor	Sensitivity S_t	Regression Equation	Standard Deviation
N_{kt}	≤ 15	$7.8 + 2.5\log\text{OCR} + 0.082I_p$	0.197
	> 15	$8.5 + 2.5\log\text{OCR}$	
$N_{\Delta u}$	≤ 15	$6.9 - 4.0\log\text{OCR} + 0.07I_p$	0.128
	> 15	$9.8 - 4.5\log\text{OCR}$	
N_{ke}	≤ 15	$11.5 - 9.05B_q$	0.172
	> 15	$12.5 - 11.0B_q$	



Best relationship (statistically) = $N_{\Delta u}$. Note: $N_{\Delta u}$ correlation uses direct measurement (u_2) and does not require use of q_t which must be corrected for overburden stress in other correlations.



Shear strength - Bothkennar correlation to match source



Undrained shear strength from CPTU data in clays

1. In deposits where little experience is available: use N_{kt} equal to 15-20. Use upper limit for conservative value. In soft clays N_{kt} can be as low as 10. In stiff fissured clay can be as high as 30. In soft clays s_u can also be estimated from excess pore pressure (u_2) with $N_{\Delta v}$ from 7-10.
2. If previous experience is available in same deposit, the values suggested above should be adjusted to reflect this experience.
3. For larger projects, where high quality field and laboratory data may be available, site specific correlations should be developed based on appropriate and reliable values of s_u .



Shear strength from CPTU in clays

- Historical scatter in correlations for clays may have been due to mixing of q_c and q_t ???
- Also quality of base data, source of c_u
- Correlate to what you have confidence in
 - Block samples
 - Piston samples
 - Type of test
 - Do you have confidence to use peak strength
 - What are your design procedures based on e.g. use αc_u what was the previous alpha derived from



Strength characteristics

- undrained shear strength
- Sensitivity (not today)
- effective stress strength parameters



Remoulded shear strength or Sensitivity

It has been suggested that it may be possible to estimate remoulded shear strength from measured CPT sleeve friction, be very careful!! Friction measurements are least reliable



Strength characteristics

- undrained shear strength
- sensitivity
- effective stress strength parameters

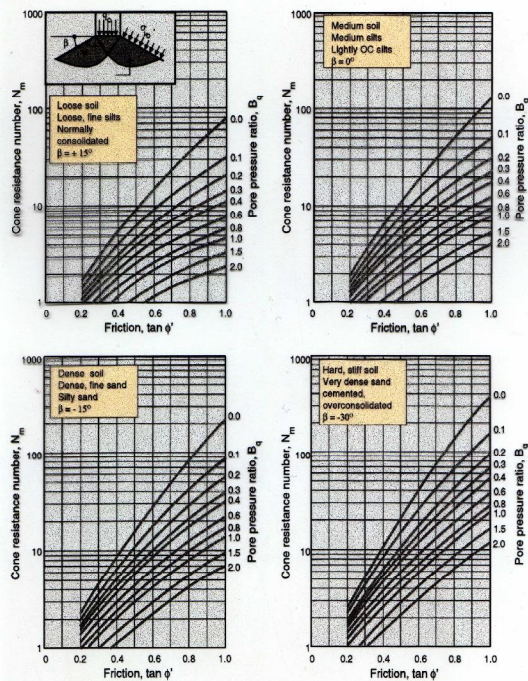


Effective stress strength parameters

- An effective stress interpretation method has been developed by Senneset et al. (1982, 1988), Senneset and Janbu (1985), and Sandven et al. (1988)



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NTH Effective Penetration Theory

Ref: Senneset and Janbu (1982, ESOPT)

- Cone Resistance Number (N_m)

$$N_m = \frac{N_q - 1}{1 + N_u \cdot B_q} = \frac{q_t - \sigma_{vo}}{\sigma_{vo}' + a'}$$

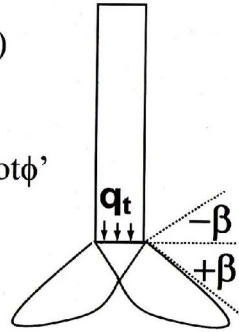
- Intercept = attraction $a' = c' \cot \phi'$

- Porewater Bearing Factor, N_u

$$N_u = 6 \cdot \tan \phi' \cdot (1 + \tan \phi')$$

- End Bearing Factor, N_q

$$N_q = \tan^2(45^\circ + \frac{1}{2} \phi') \exp[(\pi - 2\beta) \tan \phi']$$



Drained Penetration: $q_t + a' = N_q (\sigma_{vo}' + a')$



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NTH Effective Penetration Theory

- Cone Resistance Number (N_m) = measured slope of $(q_t - \sigma_{vo})$ vs. σ_{vo}'
- Intercept = attraction $a' = c' \cot \phi'$
- If $a' = 0$ assumed: $N_m = Q = (q_t - \sigma_{vo}) / \sigma_{vo}'$
- Porewater Parameter: $B_q = \Delta u_2 / (q_t - \sigma_{vo})$
- Use solution for $\beta = 0$ (angle of plastification)
- Q and B_q same as in Robertson (1990) soil behavioral classification charts.

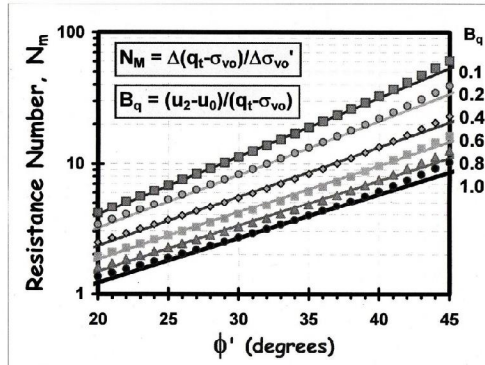


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Approximate NTH CPTU - ϕ' Method ($0.1 < B_q < 1.0$)

Senneset, Sandven, & Janbu (1989, Transportation Research Record 1235)

For $a' = 0$:
 Then: $N_m = Q$
 $= (q_t - \sigma_{vo}) / \sigma_{vo}'$
 For all soil types



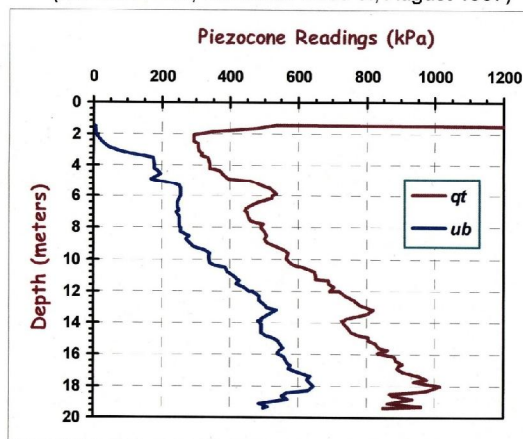
$$\phi' \approx 29.5 \cdot B_q^{0.121} \cdot [0.256 + 0.336 B_q + \log(N_m)]$$

where $20^\circ < \phi' < 45^\circ$ and $0.1 < B_q < 1.0$



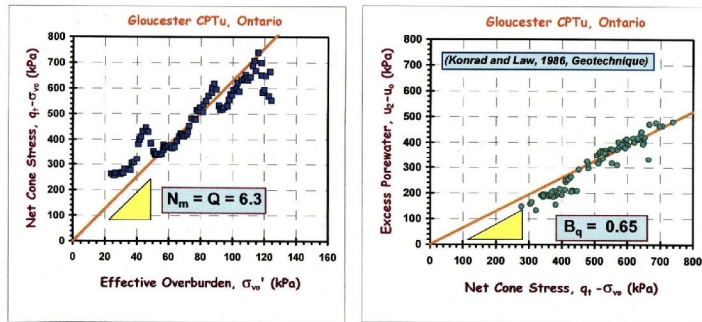
Gloucester Test Site, Ontario

(Konrad & Law, Canadian Geot. J., August 1987)



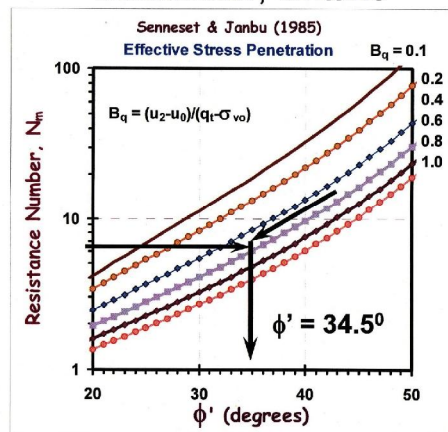
Gloucester Ontario

Canadian National Test Site, Gloucester, Ontario



Gloucester Ontario

Gloucester, Ontario

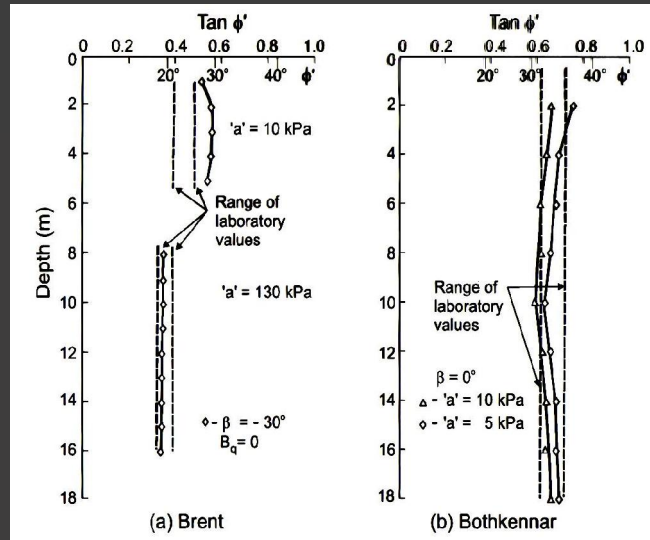


Konrad & Law (Canadian Geot. J. 1987):

$$\phi' = 35^\circ$$



Comparison of effective stress strength parameters from CPTU and laboratory tests



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Interpretation of CPTU data in clay

Parameters that we can try to estimate from CPTs in fine grained soils are:

- State characteristics (In situ stresses and stress history)
- Strength characteristics
- **Deformation characteristics**
- Flow and consolidation characteristics
- In situ pore pressure



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Deformation characteristics

- *Constrained modulus*
 - *undrained Young's modulus*
 - *small strain shear modulus*
- There are basically two approaches for estimating moduli from CPT/CPTU data:
 - Indirect methods that require an estimate of another parameter such as undrained shear strength s_u .
 - Direct methods that relate cone resistance directly to modulus.



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Constrained modulus

$$M = 1/m_v$$

many examples exist of trying to link M to q_c

$$M = 1/m_v = \alpha_m \cdot q_c \quad q_t???$$

values of α_m from table



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Estimation of Constrained Modulus, M, for clays (Adapted from Sanglerat, 1972) (after Mitchell and Gardner, 1975)

$M = 1/m_v = \alpha_m \cdot q_c$		
$q_c < 0.7 \text{ MPa}$	$3 < \alpha_m < 8$	Clay of low plasticity (CL)
$0.7 < q_c < 2.0 \text{ MPa}$	$2 < \alpha_m < 5$	
$q_c > 2.0 \text{ MPa}$	$1 < \alpha_m < 2.5$	
$q_c > 2 \text{ MPa}$	$3 < \alpha_m < 6$	Silts of low plasticity (ML)
$q_c < 2 \text{ MPa}$	$1 < \alpha_m < 3$	
$q_c < 2 \text{ MPa}$	$2 < \alpha_m < 6$	Highly plastic silts and clays (MH, CH)
$q_c < 1.2 \text{ MPa}$	$2 < \alpha_m < 8$	Organic silts (OL)
$q_c < 0.7 \text{ MPa}$		Peat and organic clay (P, OH)
$50 < w < 100$	$1.5 < \alpha_m < 4$	
$100 < w < 200$	$1 < \alpha_m < 1.5$	
$w > 200$	$0.4 < \alpha_m < 1$	

Jones and Rust for South African Alluvial clays suggest 2.75 ± 0.55 for α_m



Do these historical ranges relate to the use of q_c and not q_u

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Estimation of Constrained Modulus, M , for clays (Adapted from Sanglerat, 1972) (after Mitchell and Gardner, 1975)

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$q_c > 2.0 \text{ MPa}$	$1 < \alpha_m < 2.5$	
$q_c \geq 2 \text{ MPa}$	$3 < \alpha_m < 6$	Silts of low plasticity (ML)
$q_c < 2 \text{ MPa}$	$1 < \alpha_m < 3$	
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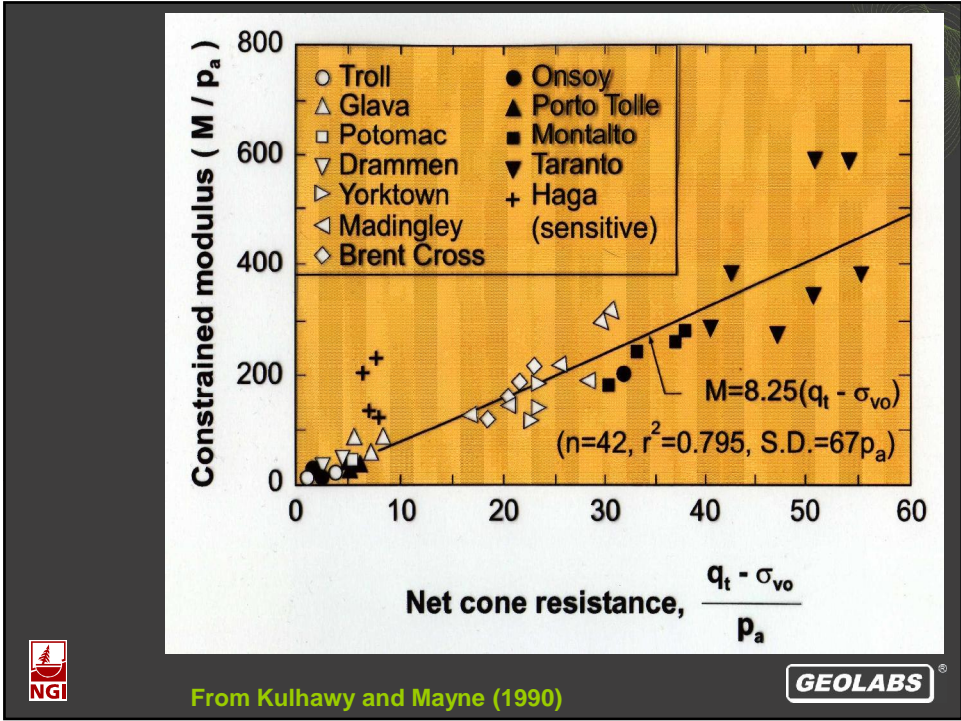


Constrained (oedometer) modulus

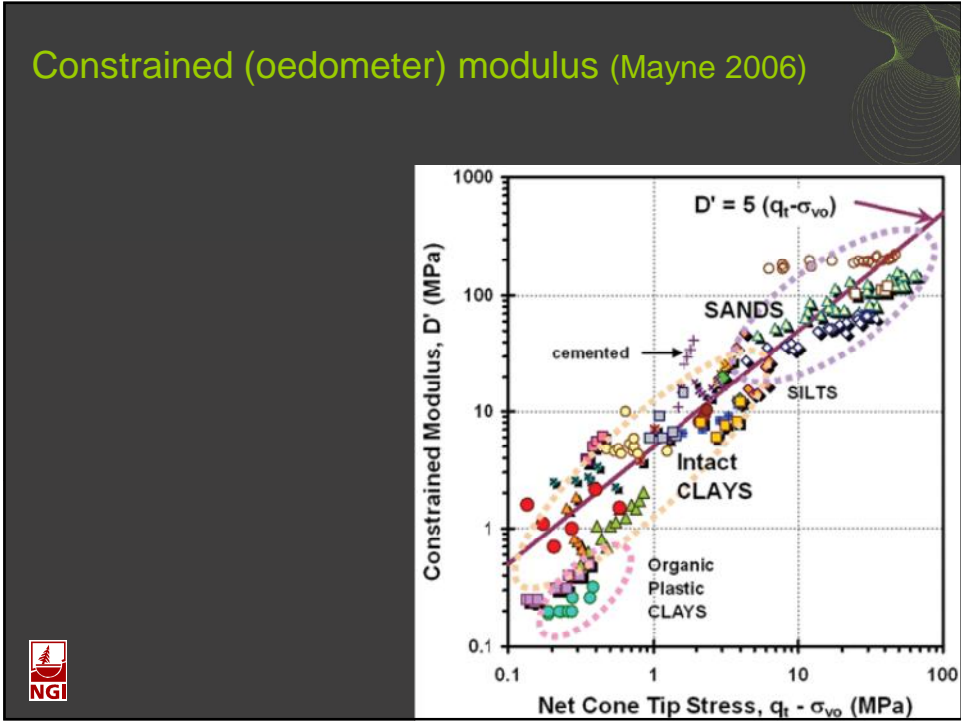
Kulhawy and Mayne (1990) suggested a more general relationship of:

$$M = 8.25 (q_t - \sigma_{vo})$$





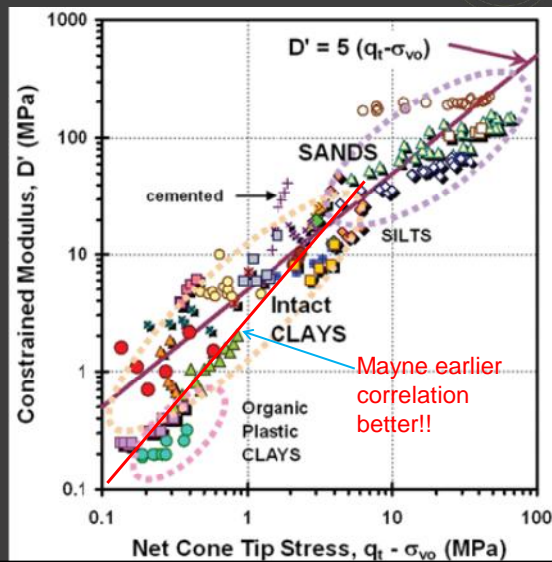
Constrained (oedometer) modulus (Mayne 2006)



Constrained (oedometer) modulus (Mayne 2006)

Take care!

Do not rely on correlations at face value



Constrained (oedometer) modulus

The estimation of drained consolidation or deformation parameters, such as M , from an undrained penetration test may be liable for serious error, especially when based on general empirical correlations.

Conceptually, total stress undrained measurements from a CPT are difficult to correlate to drained parameters without the addition of pore pressure measurements.



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Deformation characteristics

- *Constrained modulus*
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- There are basically two approaches for estimating moduli from CPT/CPTU data:
 - Indirect methods that require an estimate of another parameter such as undrained shear strength s_u .
 - Direct methods that relate cone resistance directly to modulus.



Undrained Young's modulus

- The estimation of undrained Young's modulus, E_u , is usually made using empirical correlations with the undrained shear strength, s_u , in the form:

$$E_u = n \cdot s_u$$

where n is a constant that depends on shear stress level, overconsolidation ratio, clay sensitivity and other factors

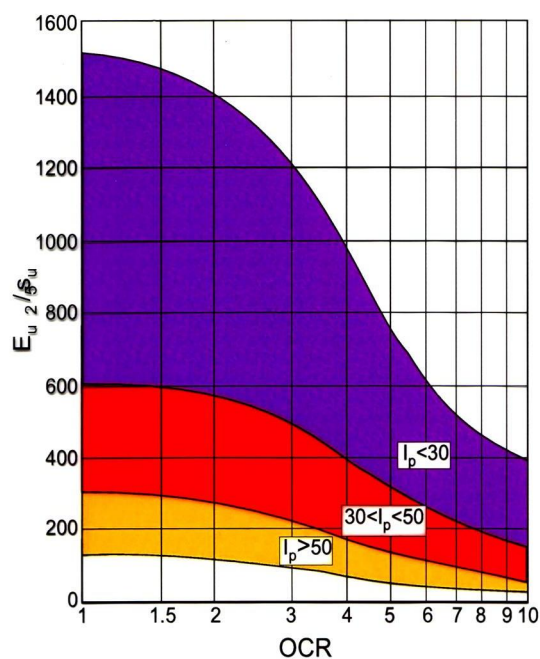


Undrained Young's modulus

- The recommended procedure for estimation of the undrained Young's modulus, E_u , is to
 - first estimate the undrained shear strength, s_u , from CPT/CPTU profiles
 - then estimate the stress history (OCR).
 - Then estimate E_u for the relevant shear stress level appropriate to the particular problem.
 - A knowledge of the plasticity index (I_p) would significantly improve the estimate.



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Small strain shear modulus

- The shear modulus is largest at very low strains. This initial, small strain modulus is often denoted G_o .



Small strain shear modulus

Mayne and Rix (1993) showed that the small strain shear modulus varied with void ratio (e) and cone penetration resistance (q_t) for a wide range of clays and can be expressed as:

$$G_o = 99.5 (p_a)^{0.305} \frac{(q_t)^{0.695}}{(e)^{1.130}}$$

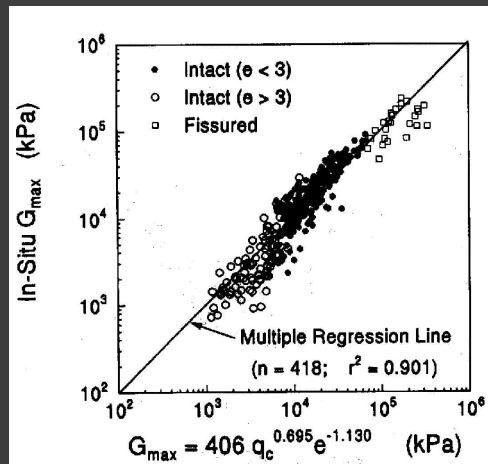
Be careful with this correlation, see seismic cone later

where:

p_a = atmospheric reference stress in the same units as G_o and q_t .



Small strain shear modulus from CPTU data



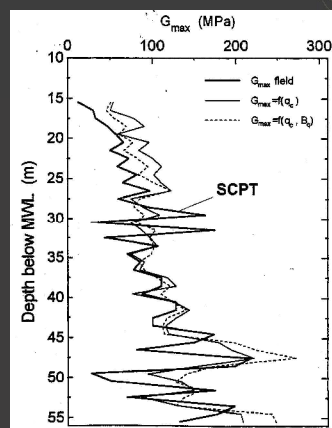
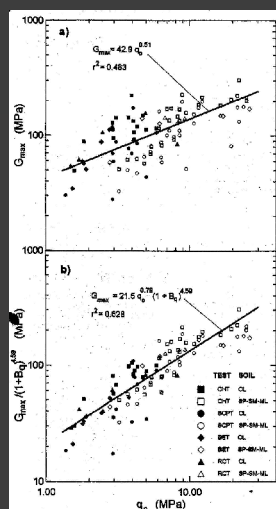
From Mayne and Rix (1993)
What $G_{??}$



($r^2 = 0.901$) $G_{max} = 406(q_c)^{0.695}/e^{1.130}$



CPTU - G_{max} correlations for Venetian soils -



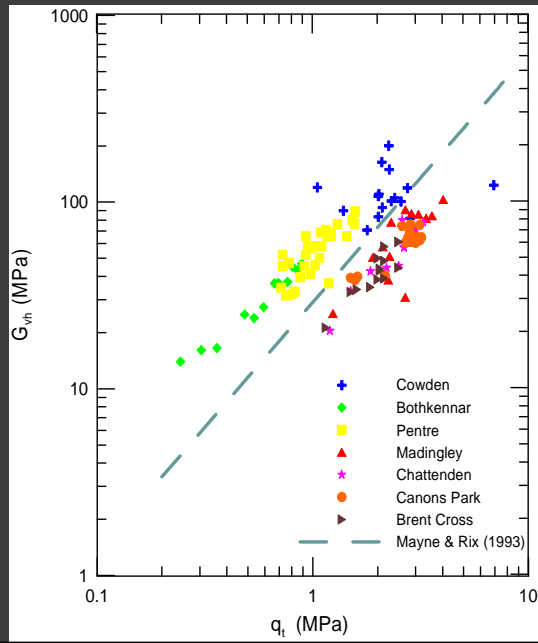
Comparison with G_{max} measured with seismic cone



Correlations from Simonini and Cola, 2000

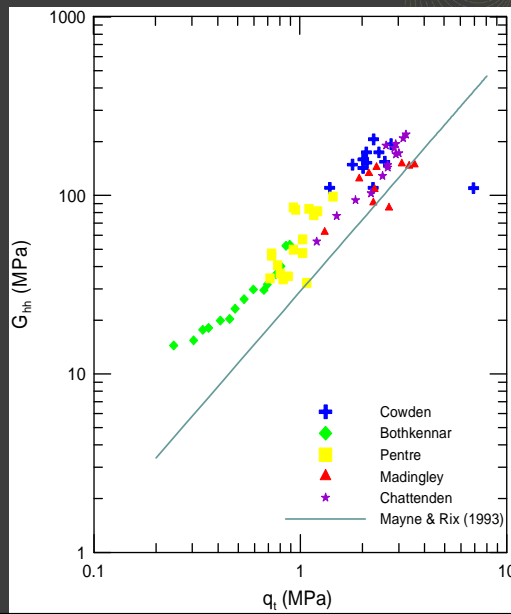


G_{vh} in UK soils



G_{hh} in UK soils

See seismic cone later



Small strain shear modulus

- Is there in fact a strong dependence of G_0 upon void ratio
- This will be discussed further later



Interpretation of CPTU data in clay

Parameters that we can try to estimate from CPTs in fine grained soils are:

- State characteristics (In situ stresses and stress history)
- Strength characteristics
- Deformation characteristics
- Flow and consolidation characteristics
- In situ pore pressure



Flow and consolidation characteristics

- *coefficient of consolidation*
- *coefficient of permeability*



Piezocene Dissipation test

A test when the decay of the pore pressure is monitored during a pause in penetration (ideally the other cone parameters should also be recorded)

- The measured pore pressure comprises two parts, an element due to the penetration process and the equilibrium pore pressure
- With time the measured pore pressure would decay to the equilibrium value.

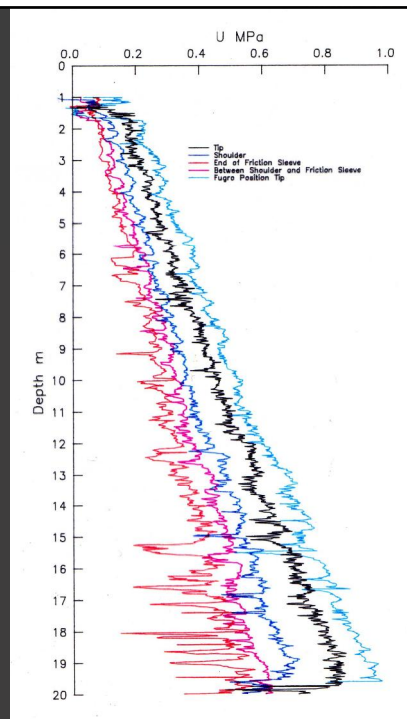


The results will vary with filter position.



Example of effect of pore pressure location on measured pore pressure

Bothkennar, UK clay



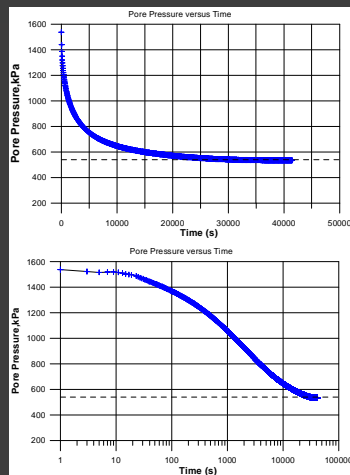
PIEZOCONE DISSIPATION TEST

Stop penetration and measure pore pressure decay with time
Interpret results to get:

- In situ pore pressure - check if different from hydrostatic
- Derive coefficient of consolidation
- guide to permeability



Example CPTU dissipation test nearshore Indonesia



Filter at 53.9 m

Hydrostatic pressure (539 kPa) is relative to mudline



Theory/Interpretation

- For interpretation it is best to normalise the pore pressure relative to the initial pore pressure at the beginning of dissipation, u_i , and the equilibrium in situ pore pressure u_o . The normalised excess pore pressure, U_t , at time t , is thus expressed as:

$$U_t = \frac{u_t - u_o}{u_i - u_o}$$

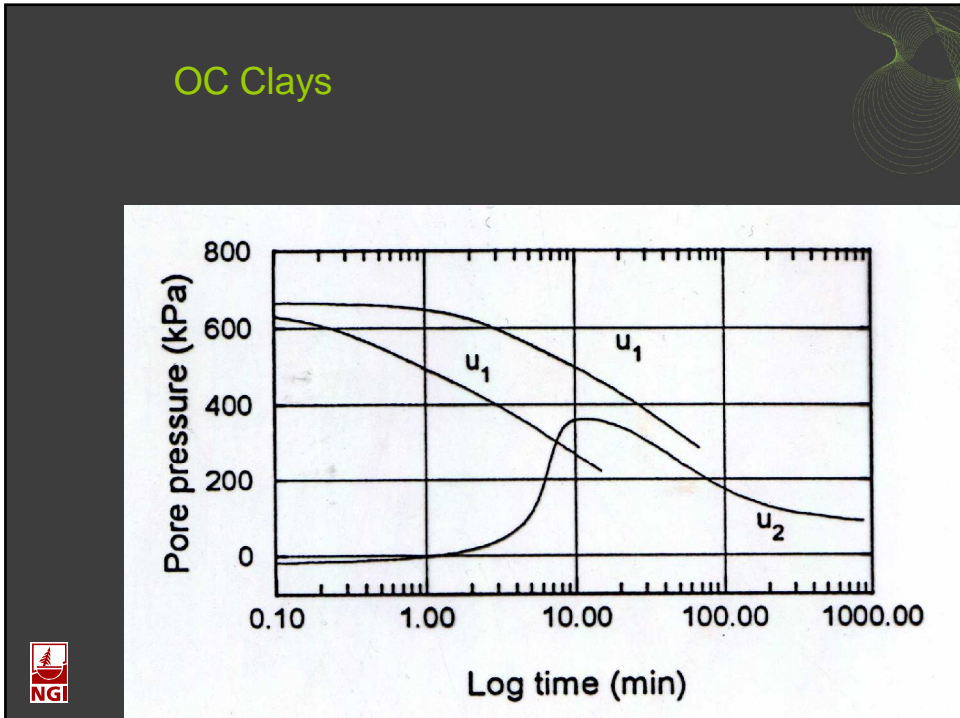
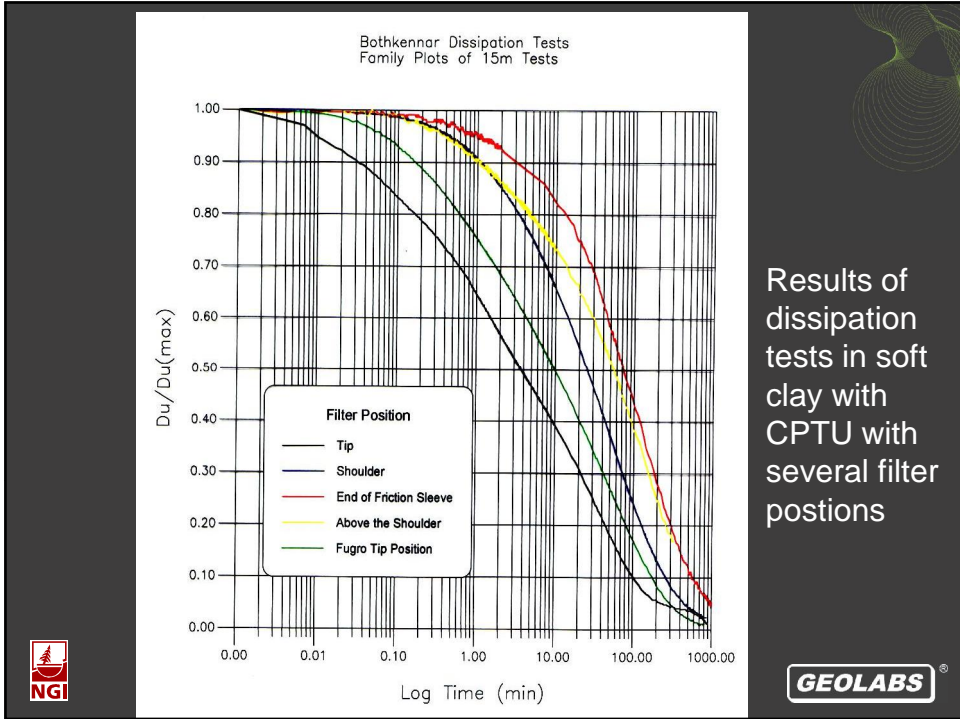
where

- u_t = the pore pressure at time t .
- u_i = initial pore pressure at $t = 0$
- u_o = in-situ pore pressure before penetration



Give curves





Coefficient of consolidation, c_v , from CPTU dissipation tests

Several methods, theoretical and semi-empirical, have been published to find c_h from dissipation test. Several factors influence the relevance of the various methods:

- (i) Estimation of pore pressure at start of dissipation
- (ii) Remoulding of clay around cone during penetration and smearing
- (iii) Importance of both vertical and horizontal dissipation of pore pressure
- (iv) Anisotropy of soil



Coefficient of consolidation, c_v , from CPTU dissipation tests

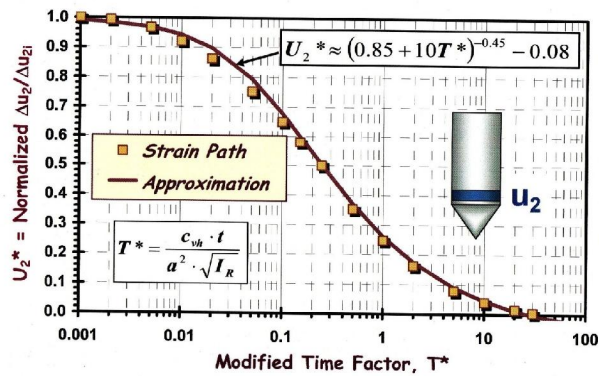
Based on a study by Robertson et al. (1992)

- (i) The method published by Houlsby and Teh (1988) gives reasonable values of c
- (ii) It is mainly the coefficient of consolidation in horizontal direction, c_h , that can be found from a dissipation test
- (iii) A practical and reasonable parameter for the interpretation is the time for 50 % consolidation, t_{50}



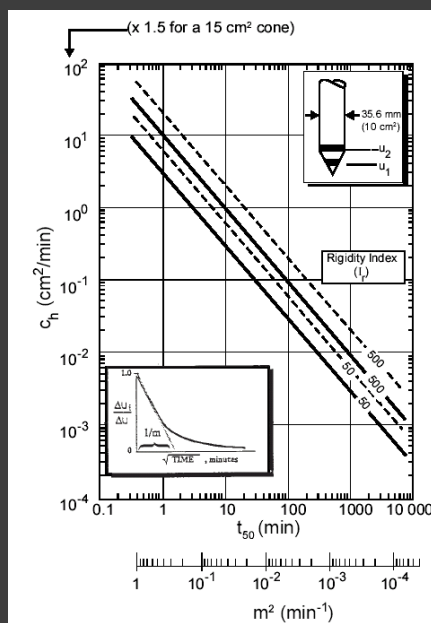
Theory and Practice, modelling

Calibration of Piezo-Dissipations Strain Path Solution (Houlsby & Teh, 1991)



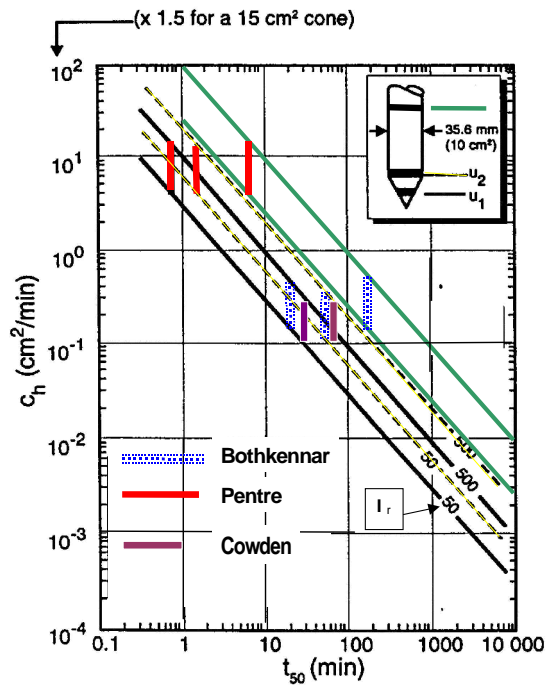
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Chart for finding c_h from t_{50} using Teh's method



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C_h interpreted from CPTU results



Interpretation

The recommended procedure to estimate the coefficient of consolidation is to use dissipation data from the filter location behind the cone (u_2); however, other filter locations may be used even though the data can be somewhat less consistent.

The recommended procedure is as follows:

- a) Plot the early part of the dissipation (less than 10% dissipation) at an enlarged scale, either log or square root time, and evaluate the initial pore pressure, u_i .
- b) Define u_o from available data on ground water level, piezometric readings or data from piezocone tests in adjacent sand layers.



Interpretation

c) plot normalised excess pore pressure

$$U_t = \frac{u_t - u_o}{u_i - u_o}$$

vs time (t) on log and/or \sqrt{t} scale.

d) Define time for 50% dissipation (t_{50}).

e) Use t_{50} and curves in earlier figure to predict c_h . If no other data are available use an average I_r between the ranges

f) If dissipation has not proceeded sufficiently long to define t_{50} the slope of the straight line from the first part of u vs \sqrt{t} plot (m) may be used to predict c_v using values suggested by Teh. But this may be a problem when data are poor.



Interpretation

- Based on available experience, this recommended procedure should provide estimates of c_h to within \pm half an order of magnitude. However, the technique is repeatable and provides an accurate measure of the changes in the consolidation characteristics within a given soil profile.



Conclusions dissipation testing

- The piezocone enhances the ability of the CPT to profile - detection of variability, potential variation in drainage characteristic etc.,
- Good repeatability in dissipation tests can be obtained with good testing practice
- Different filter positions give different decay curves in a similar way to theoretical models
- Square root time plots can be used to better define u_i
- Values of c_h and c_v can be assessed to within an order of magnitude
- Relative changes in c_h can be clearly detected within a profile
- Size effects seem to be accounted for

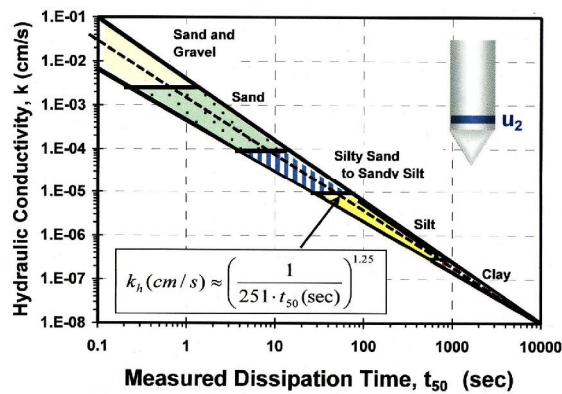


Permeability



Permeability

Direct Permeability Evaluation from CPT_u



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Conclusions on interpretation of CPT_u in fine grained soils

- Check that all results are reliable and that undrained conditions prevail
- Good interpretation methods exist for :
 - Soil Unit weight
 - Overconsolidation ratio
 - Undrained shear strength
 - Small strain shear modulus
 - Coefficient of consolidation
- Rough estimates can be made of :
 - In situ horizontal stress
 - Remoulded shear strength
 - Permeability
- Update correlations locally whenever possible
- Do not eliminate sampling and laboratory testing



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Thank you for your attention

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