JAPANESE GEOTECHNICAL SOCIETY

International Seminar

THE FLAT DILATOMETER (DMT) and its APPLICATIONS to GEOTECHNICAL DESIGN

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Tokyo, 12 Feb 1999

INDEX

Revised version with expanded section on liquefaction (see also key paper by Sladen referenced there) and addition of the "Table of operative moduli by textbooks".

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Many papers mentioned in this report may be downloaded at the bibliographic site : www.marchetti-dmt.it

1980 ASCE paper by Marchetti
Describes **original correlations** (details ➔ paper)
Principle : calibrate DMT results vs high quality parameters from well documented sites

STANDARDS - RECOMMENDATIONS

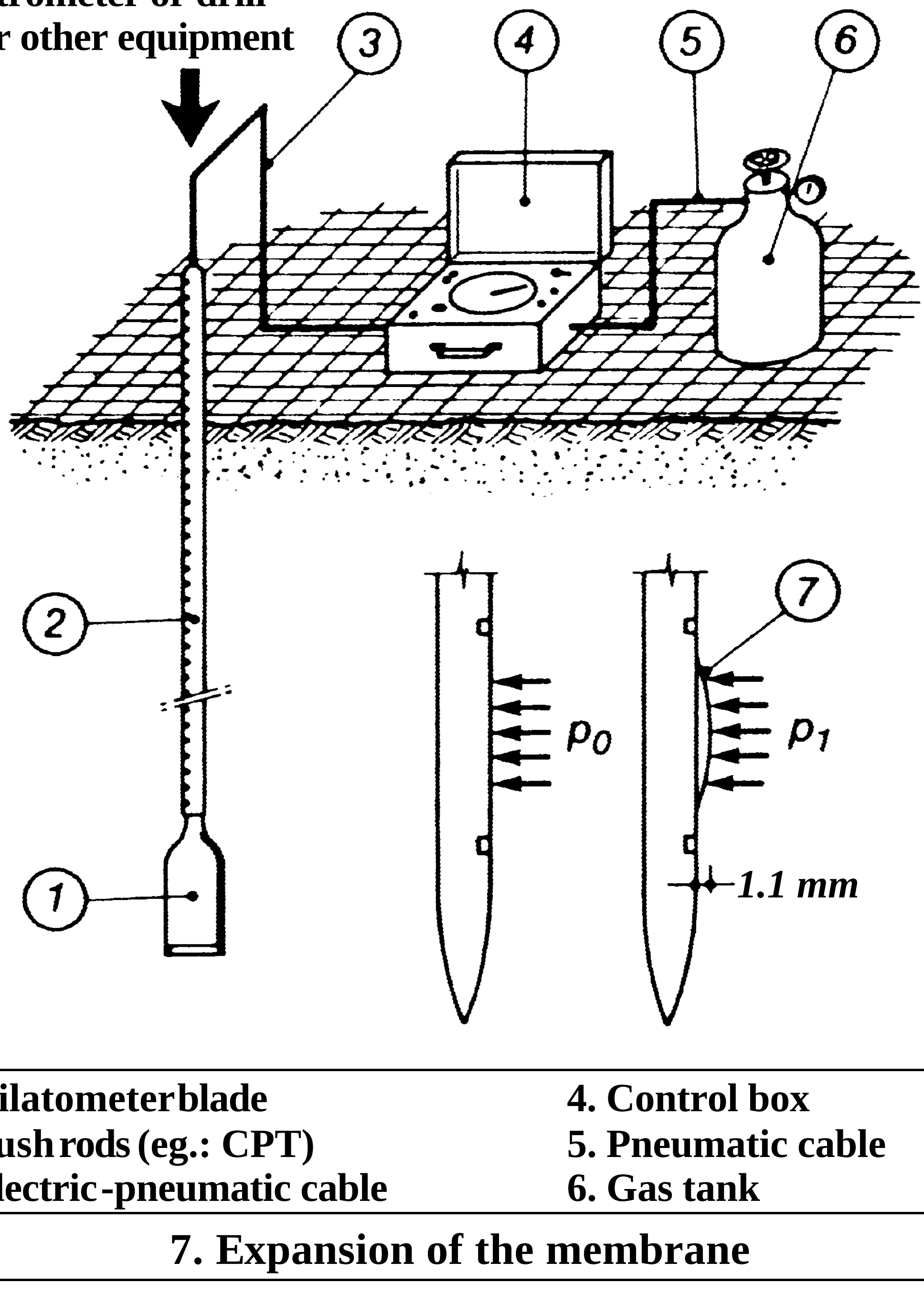
1986 ASTM suggested method
1997 Eurocode 7
1992 US DOT FHA, Briaud & Miran (a manual)

SOA - COMPREHENSIVE REPORTS

1989 Lunne et al, Rio de Janeiro
1988 Lutenegger, Orlando
1997 Marchetti, Cairo
2001 Totani et al. , IN SITU 2001 Bali Indonesia

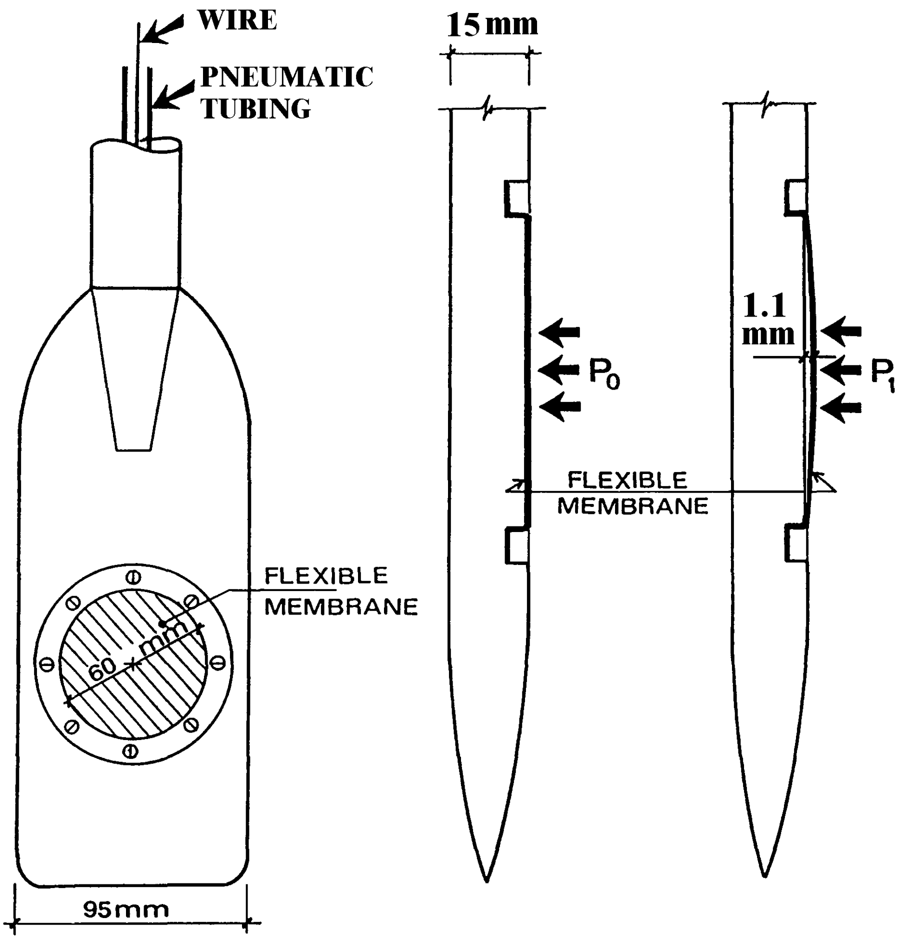
GENERAL LAYOUT OF THE DILATOMETER TEST (DMT)

Push force provided by penetrometer or drill rig or other equipment



1. Dilatometer blade	**4. Control box**
2. Push rods (eg.: CPT)	**5. Pneumatic cable**
3. Electric-pneumatic cable	**6. Gas tank**
7. Expansion of the membrane	

INTERMEDIATE DMT INDEXES



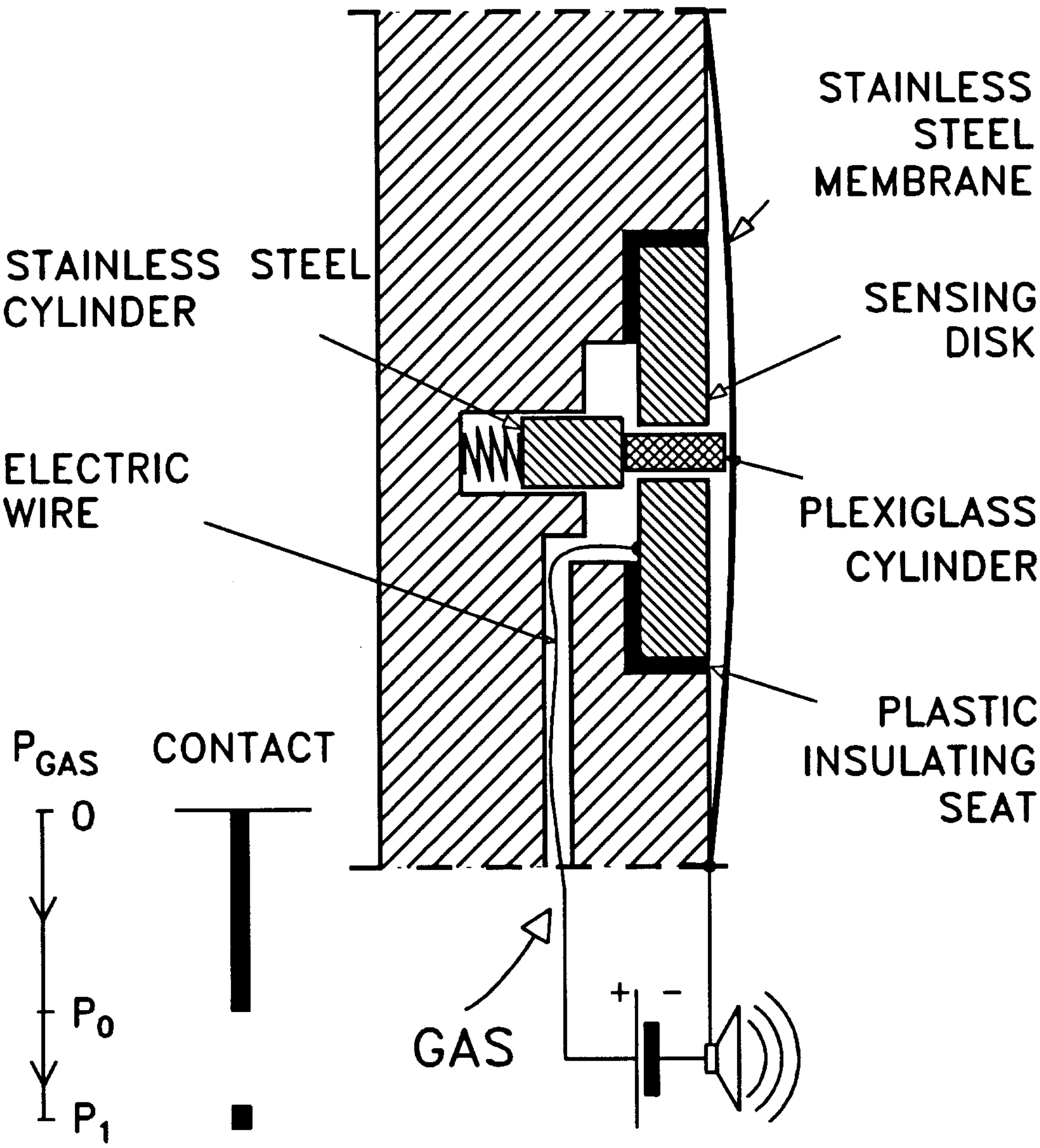
$$I_D = \frac{p_1 - p_0}{p_0 - u_0}$$ **MATERIAL INDEX**

$$K_D = \frac{p_0 - u_0}{\sigma'_{v0}}$$ **HORIZONTAL STRESS INDEX**

$$E_D = 34.7(p_1 - p_0)$$ **DILATOMETER MODULUS**

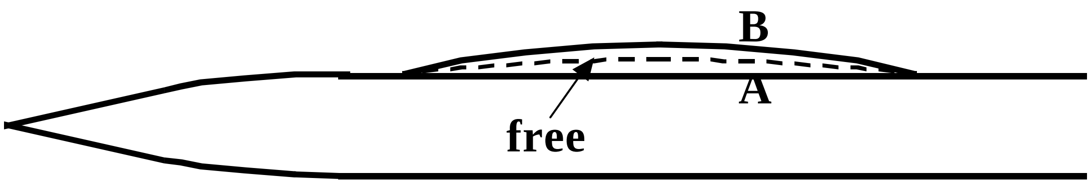
- **BUT DMT IS A <u>TWO PARAMETER</u> TEST !!!.** (ONLY TWO INDEPENDENT, NO CREATION OF INFO)
- **MEMBRANE NOT A MEASURING ORGAN (= GAGE at GROUND SURFACE). ACCURACY IS THAT OF GAGE**
- **MEMBRANE = PASSIVE SEPARATOR SOIL-GAS**
- **BALANCE OF ZERO (NULL METHOD) ACCURATE**
- **BLADE = ELECTRIC SWITCH (ON/OFF) (non electronic)**

WORKING PRINCIPLE

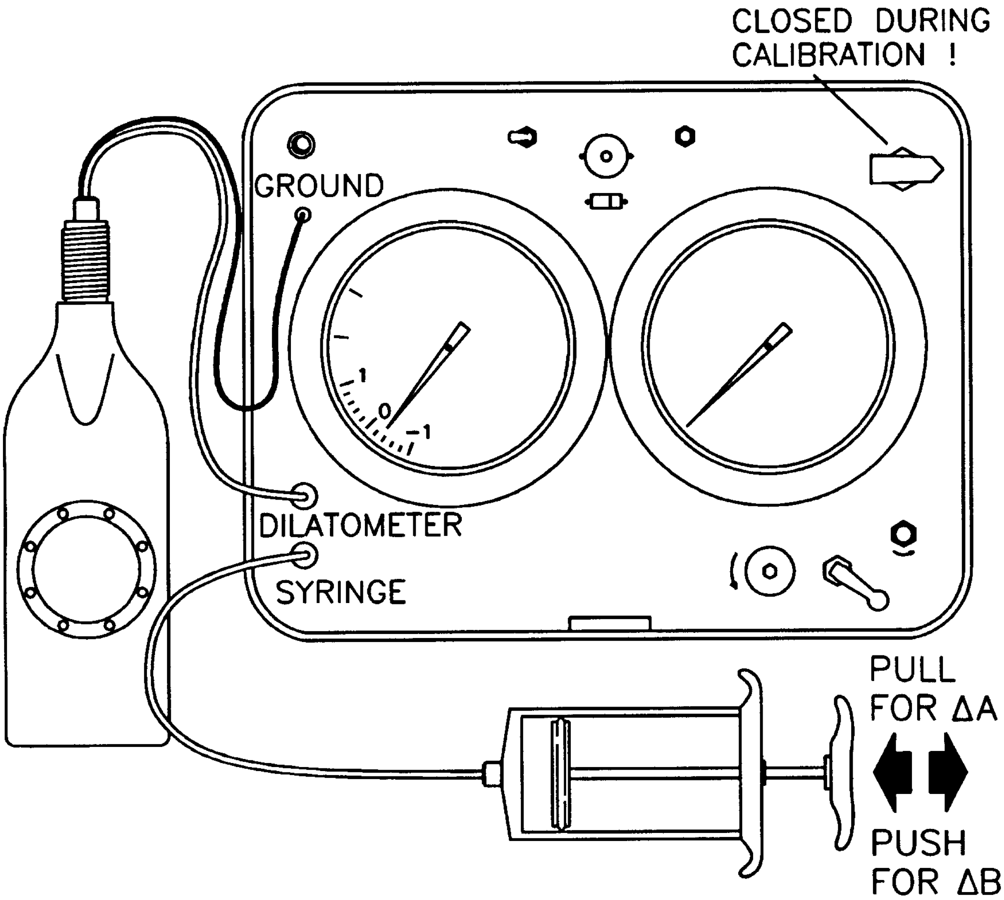


CALIBRATION OF MEMBRANE : ΔA & ΔB » *TARES* to be detracted

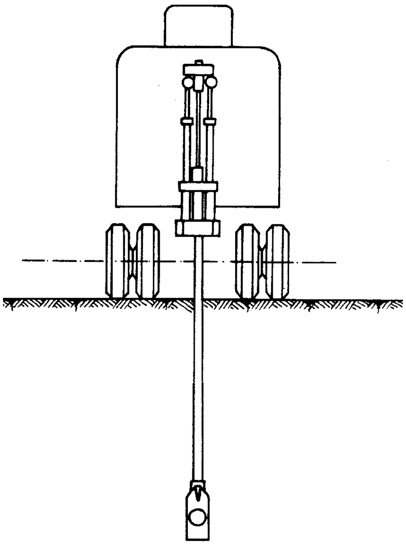
ΔA, ΔB needed to correct A,B into Po and P1



Layout of connections during calibration

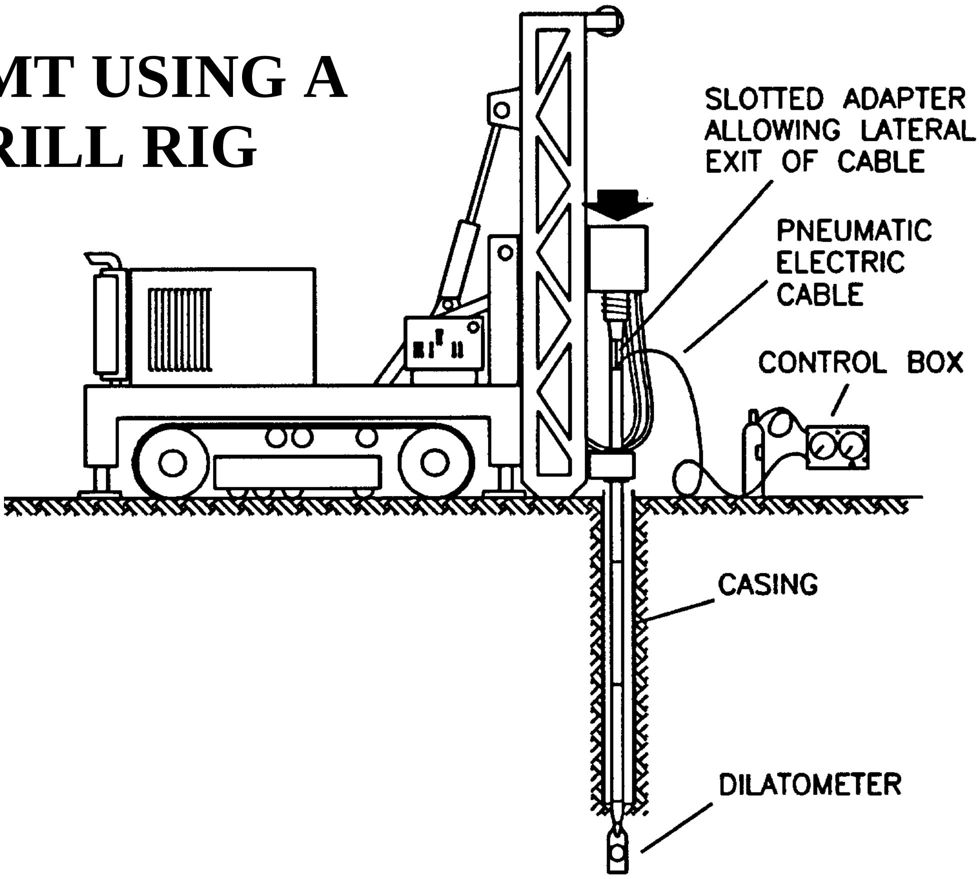


INSERTION OF THE DMT BLADE



DMT USING A PENETROMETER

DMT USING A DRILL RIG



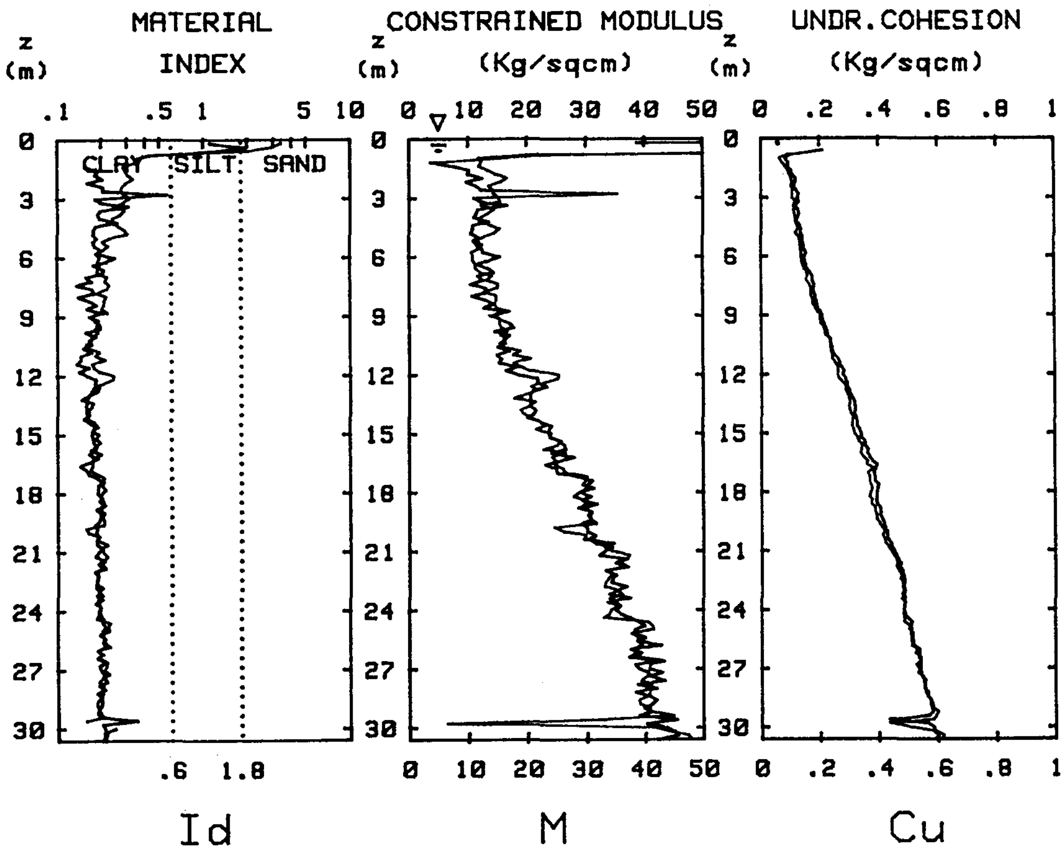
PERCUSSION (e.g. **SPT) : tolerated (except v. loose sands and sensitive clays) but not recommended**

SOILS that CAN BE TESTED by DMT

- **Suitable for SANDS, SILTS, CLAY (when grains small vs membrane D=60 mm). But can cross through GRAVEL layers » 0.5 m**
- **Due to the *balance of zero* (null method) : high resolution even in nearly liquid soils**
- **Very robust, can penetrate soft rocks (safe push on blade 25 ton)**
- **Clays : Cu = 2- 4 KPa   to Cu= 10 bar (marls)**
- **Moduli : 5 to 4000 bar (0.5 to 400 MPa)**
- **Firms with insufficient pushing can do DMT only in soft soils. But LIMIT is push capacity.**
  20 ton trucks do DMT fast and easily in hard soils.

REPRODUCIBILITY of DMT

Performed by 4 alternating operators
Cestari (SGI), Lacasse (NGI), Lunne (NGI), Marchetti (Aq)



Marine NC sensitive clay
(Onsoy, Norway)

WHAT TO DO with P_0 and P_1

			STEP 1			STEP 2				
			CALCULATE INTERMEDIATE (OBJECTIVE)			Convert Id Kd Ed to COMMON PARAMETERS (via CORRELATIONS)				
Z	**Po**	**P1**	**Id**	**Kd**	**Ed**	**Ko**	**OCR**	**M**	**Cu**	**f**
m	Bar	bar			bar			bar	bar	
1.0	1.1	3.3	1.87	6.3	73			151		38.3
1.2	1.3	1.8	.33	6.6	15	1.4	6.5	31	.19	
1.4	1.2	1.7	.37	5.7	15	1.3	5.1	29	.17	
1.6	1.2	1.6	.28	5.3	11	1.2	4.6	21	.16	
1.8	1.1	1.4	.21	4.6	8	1.1	3.6	13	.14	

- **Basic philosophy : evaluate familiar <u>parameters</u> (users can check vs other tests). Design via parameters.**
- **No correlations to *bearing capacity, foundations* etc.**

NOTE in TABLE

-In clay (Id < 1.2) : **Cu** In sand (Id > 1.2) : **f**

-In Sand the **automatic** reduction by PC gives **no Ko** & **OCR**.

Methods exist, but require cautious use!

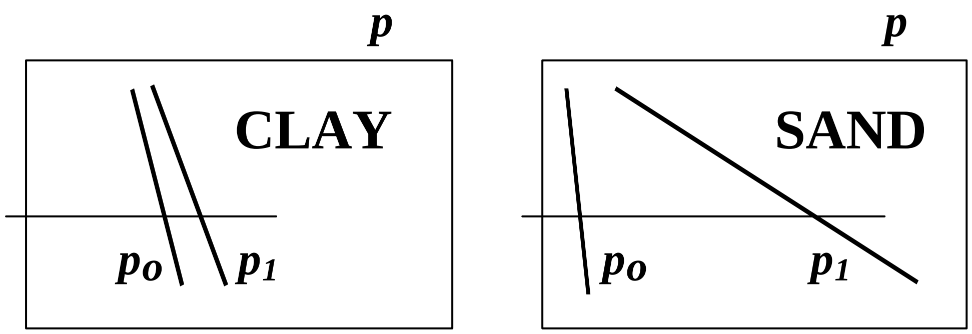
BASIC DMT REDUCTION FORMULAE

p_0 and p_1	p_0	Corrected First Reading	$p_0 = 1.05(A - Z_M + \Delta A) - 0.05(B - Z_M - \Delta B)$
	p_1	Corrected Second Reading	$p_1 = B - Z_M - \Delta B$
Inter-mediate parameters	I_D	Material Index	$I_D = (p_1 - p_0) / (p_0 - u_0)$
	K_D	Horizontal Stress Index	$K_D = (p_0 - u_0) / \sigma'_{vo}$
	E_D	Dilatometer Modulus	$E_D = 34.7 (p_1 - p_0)$
Interpreted parameters	K_0	Coeff. Earth Pressure in Situ	$K_{0,DMT} = (K_D / 1.5)^{0.47} - 0.6$
	OCR	Overconsolidation Ratio	$OCR_{DMT} = (0.5\, K_D)^{1.56}$
	C_u	Undrained Shear Strength	$C_{u,DMT} = 0.22\, \sigma'_{vo} (0.5\, K_D)^{1.25}$
	φ	Friction Angle	$\varphi_{safe,DMT} = 28 + 14.6 \log K_D - 2.1 \log^2 K_D$
	c_h	Coefficient of Consolidation	$C_{h,DMTA} \approx 7cm^2 / T_{flex}$
	k_h	Coefficient of permeability	$k_h = c_h \gamma_w / M_h \quad (M_h \approx K_0 M_{DMT})$
	γ	Unit Weight and Description	(see chart)
	M	Vertical Drained Constrained Modulus	$M_{DMT} = R_M E_D$ if $I_D \leq 0.6$ $R_M = 0.14 + 2.36 \log K_D$ if $I_D \geq 3$ $R_M = 0.5 + 2 \log K_D$ if $0.6 < I_D < 3$ $R_M = R_{M,0} + (2.5 - R_{M,0}) \log K_D$ where $R_{M,0} = 0.14 + 0.15(I_D - 0.6)$ If $K_D > 10$ $R_M = 0.32 + 2.18 \log K_D$ If $R_M < 0.85$ set $R_M = 0.85$
	U_o	Equilibrium pore pressure	$U_o = p_2 \approx C - Z_M + \Delta A$

The outlined parameters (M and Cu) are generally quite accurate and the DMT most widely used.

Id - Material Index (soil type)

Whoever does DMT 1st time notes :



\ came natural (apart theory) define Id as a "vicinity ratio"

$$I_d = (P_1 - P_o)/(P_o - U_o)$$

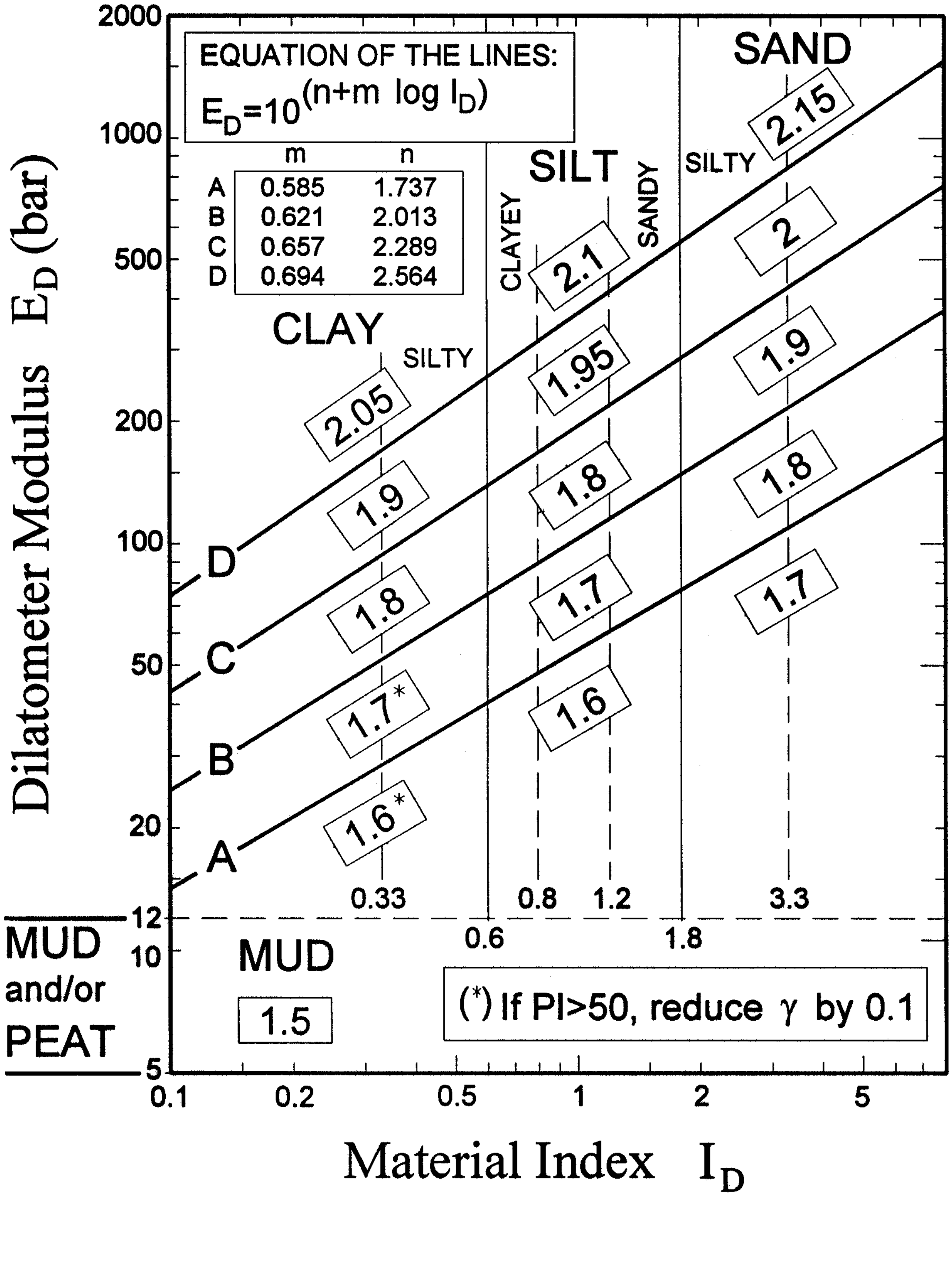
Experience has shown

- **Id v. sensitive, 0.1 to 10 (2 log cycles)**

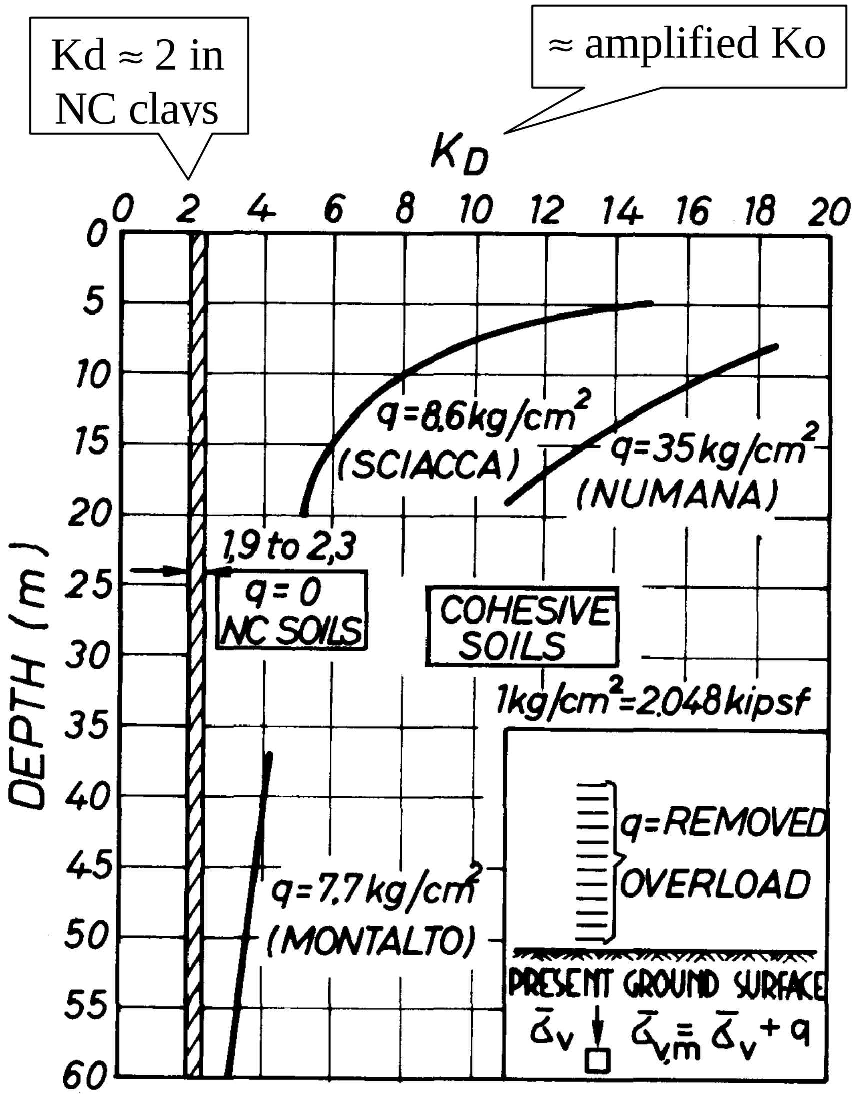
0.1	0.6	1.8	10
CLAY	SILT	SAND	

- **Like FR in CPT <u>but</u> : amplified, highly reproducible**
- **Not primary scope, but a nice extra - generally reliable**
- **I_D not result of sieve analysis, but from mechanical response (» *rigidity index*)**
- **Eg clay + sand described by I_D as *silt* Þ behaves mechanically as... (incorrect for grain size, + relevant mechanical behavior)**
- **If interest in permeability, (besides I_D) other index U_D**

SOIL DESCRIPTION and estimated g / g_w



Kd - Horiz Stress Index (soil type)



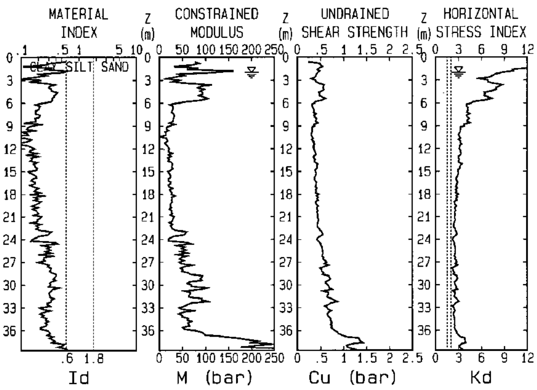
Note similarity Kd - OCR. (This prompted in 1980 the <u>correlation</u> Kd-OCR)

(next step : from OCR Þ Cu via Ladd 1977 - Soa Tokyo V.2 p. 440)

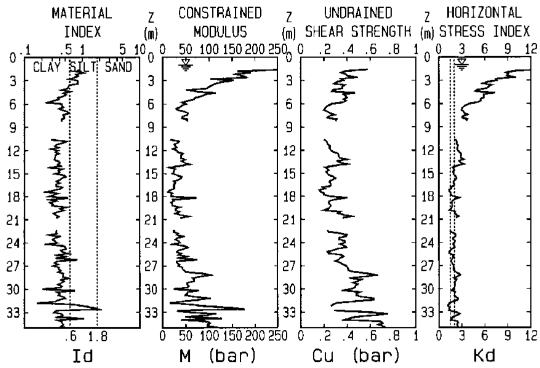
$$Cu/p' \approx (Cu/p')_{NC} \cdot OCR^{\approx 0.8}$$

Kd » 2 in NC clays (1.8-2.3)

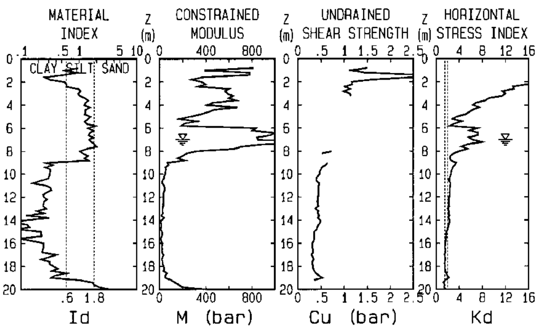
Kd evidences clearly crusts (current/ old)



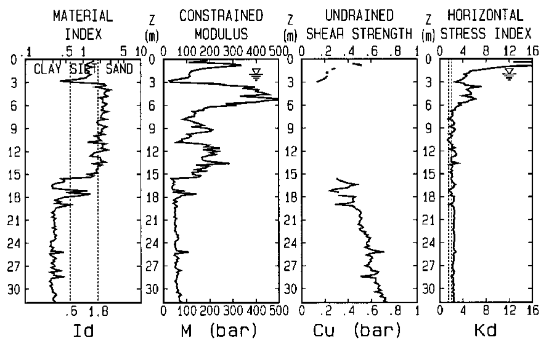
GUASTICCE (Livorno)



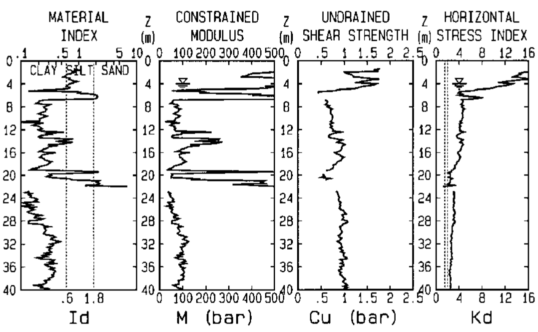
PESCARA



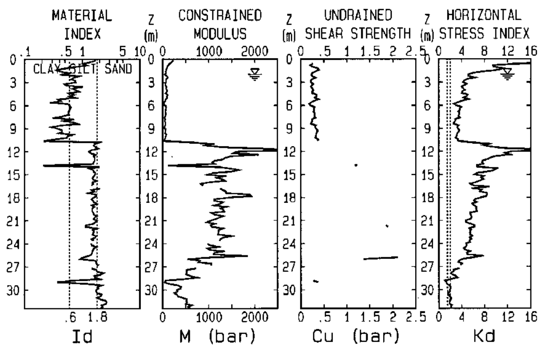
AREZZO



RIETI

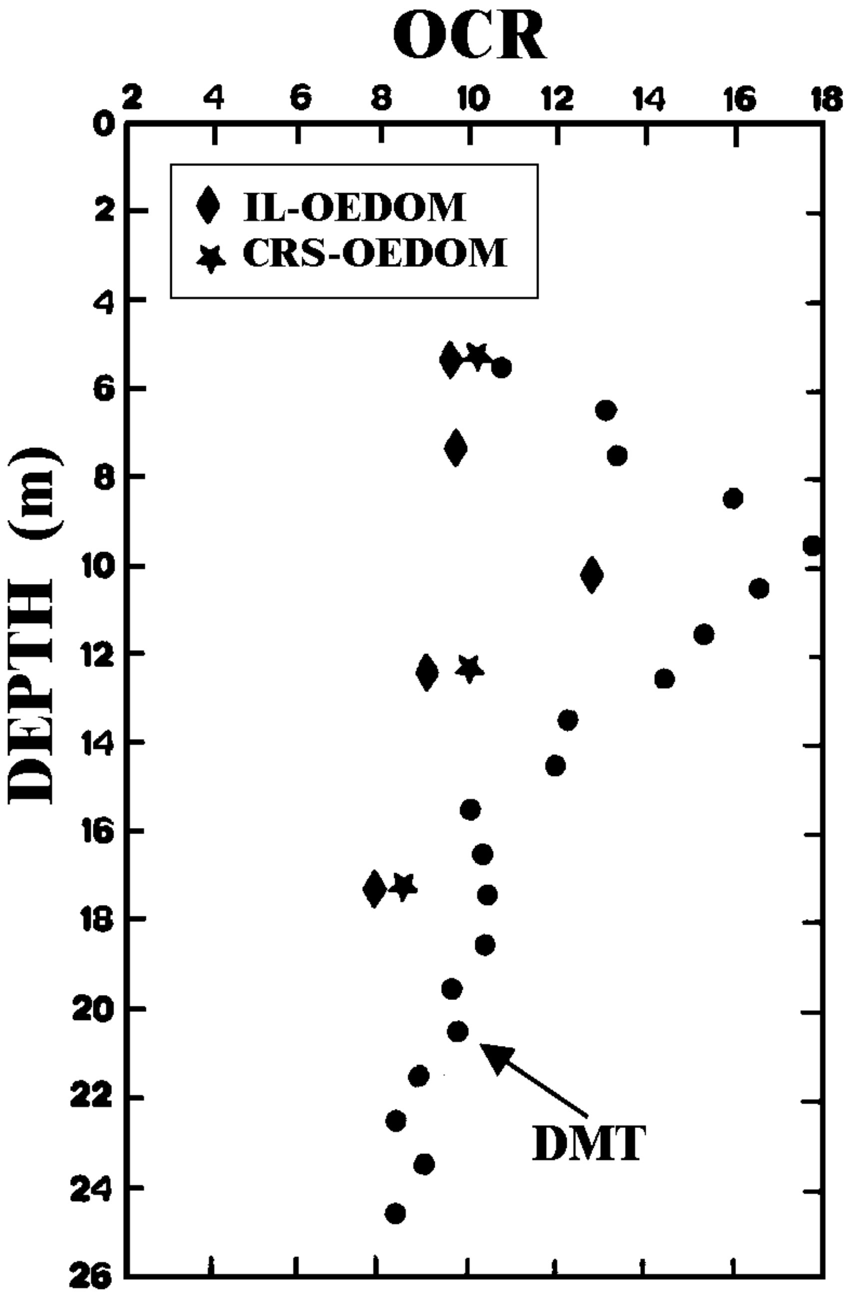


LIVORNO



BOLOGNA- VERONA

OCR from oedometer vs OCR from DMT



Overconsolidated Augusta clay (Sicily)

Jamiolkowski, 1988, ISOPT I,1, p. 271

OCR from DMT at BOTHKENNAR

National Research Site in UK, soft clay

- **Investigated for years : most refined sampling techniques (piston samplers, Laval samplers, Landva's trimming technique..) and also DMT.**
- **Results summarized in 1992 (June) Géotechnique**
- **Based on all other tests, Authors stated (p. 171): "despite scatter, it is apparent OCR » constant" (see in their Fig. 8b the vertical line labeled "suggested profile"). DMT, however (also in Fig. 8b) indicated differently, namely two branches with a break at » 11 m.**

See diagrams next page

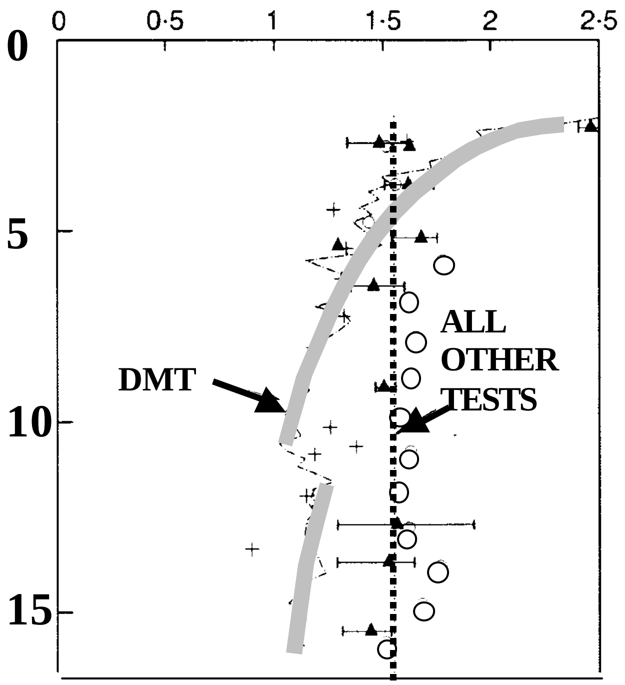
- **One year later Building Res. Establishment + Imperial College reviewed all tests, added new results, sent a questionnaire inviting research proposals worldwide. Documentation included an <u>updated OCR</u> very similar to OCR_{DMT}**

****

- **In this case DMT captured details of <u>OCR-SHAPE</u> undetected by refined sampling/lab work by scrupoulos technicians (non commercial !).**
- **OCR_{DMT} : negligible cost / couple of hours.**

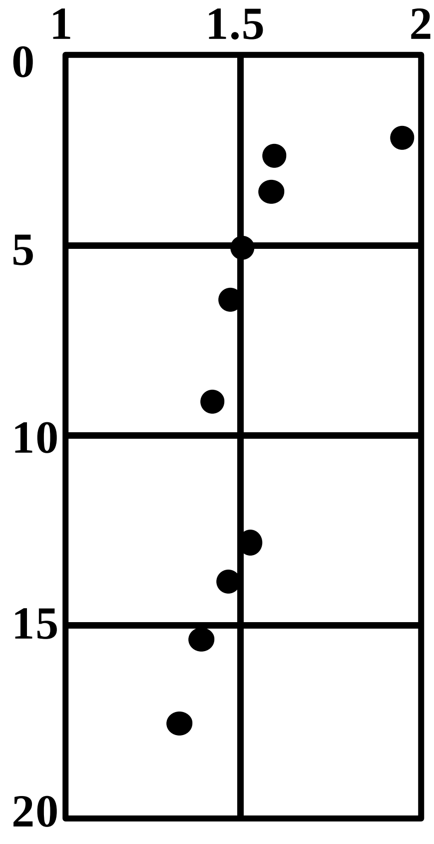
OCR estimates at BOTHKENNAR

OCR from investigations up to 1992



Géotechnique, June 1992, p.171, Fig. 8b

OCR after additional investigations (1993)



CORRELATION OCR vs Kd

Kamei T. and Iwasaki K.(1995). "Evaluation of Undrained shear strenght of cohesive soils using a flat dilatometer"

SOILS AND FOUNDATIONS Vol. 35, No. 2, 1-13, June 1995
Japanese Society of Soil Mechanics and Foundation Engineering

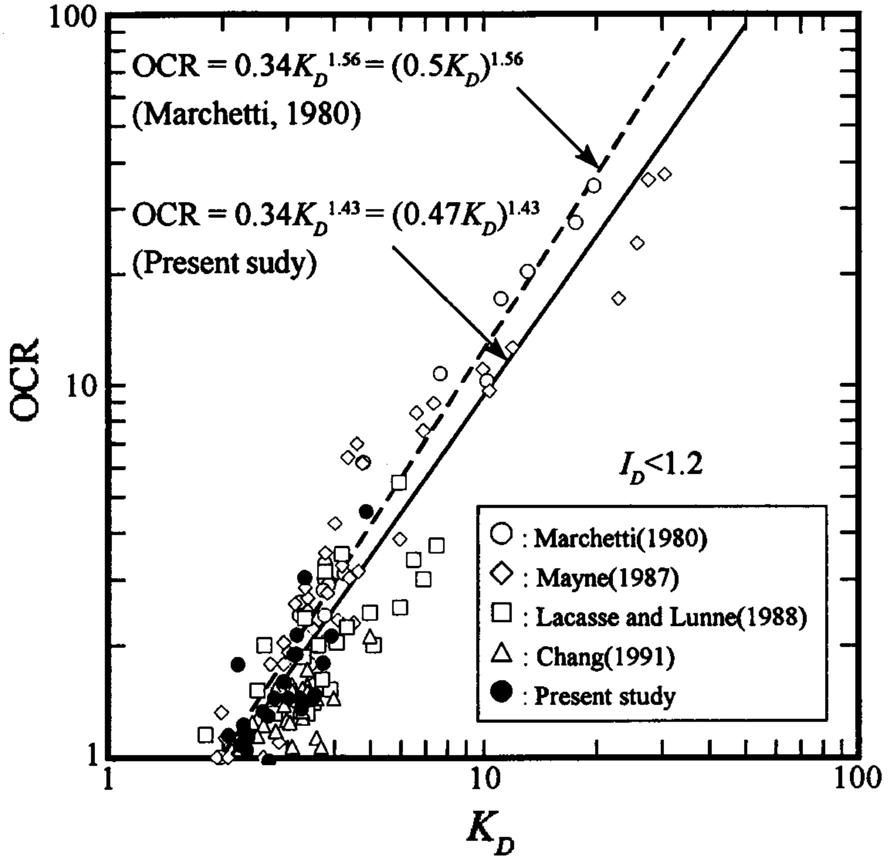


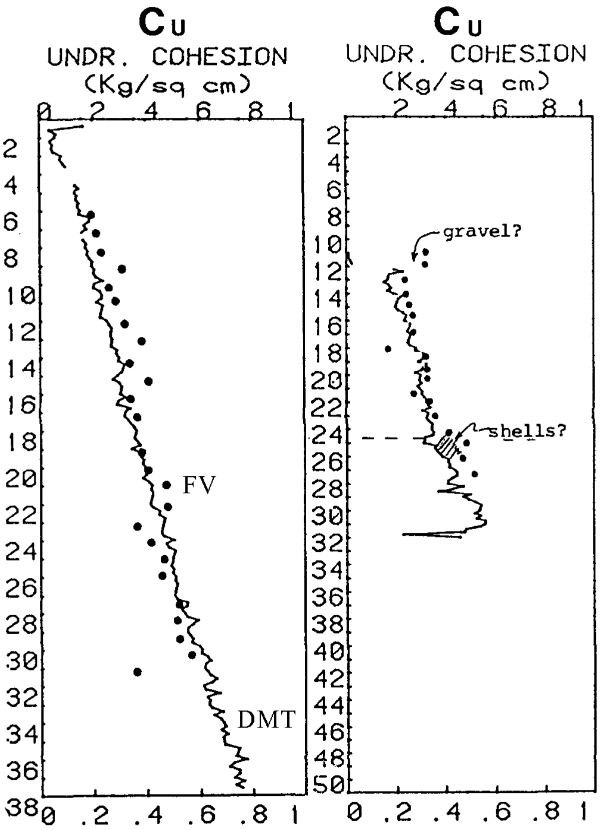
Fig. 3. Correlation between K_D and OCR for cohesive soils all over the world

COMMENT

Practically no appreciable difference vs 1980 correlation

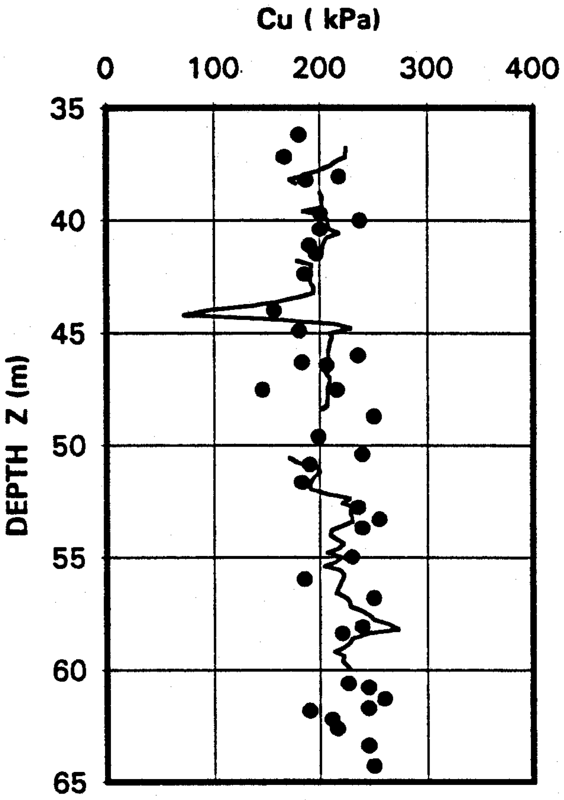
Cu predicted by DMT vs Cu by FV/UU

NC



(Cu)DMT vs (Cu)Field Vane
(Skeena Ontario Canada, Soft clay)

OC



(Cu)DMT vs (Cu)Lab Trx UU
(Montalto di Castro, Italy)

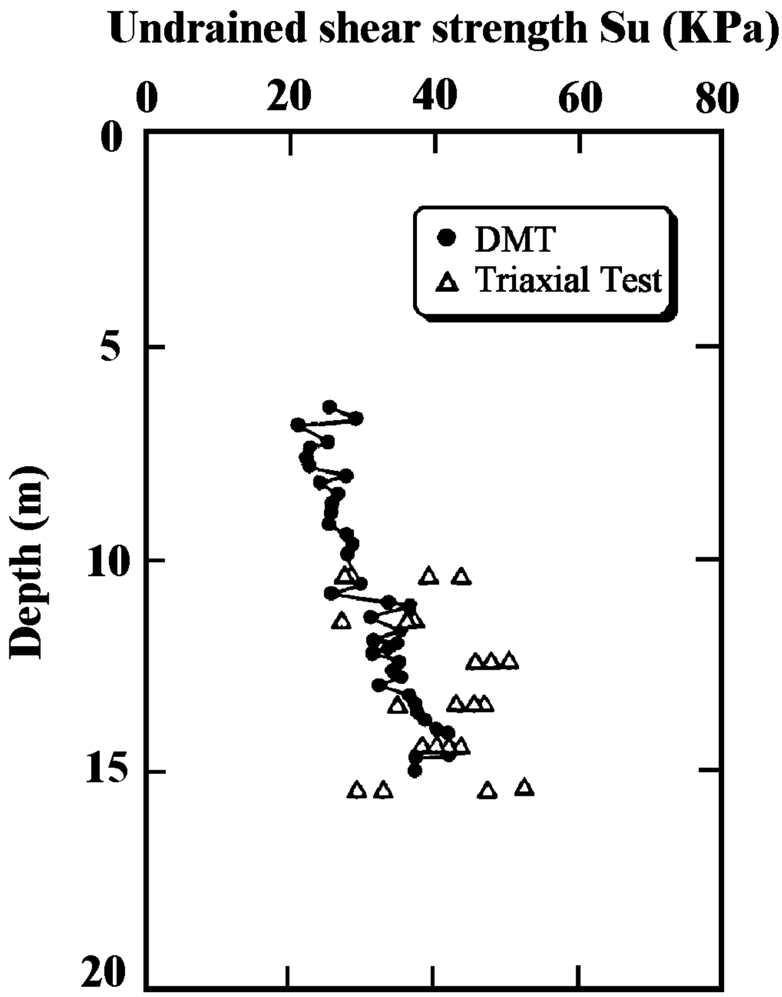
Mekechuk J. "DMT Use on C.N. Rail Line British Columbia", First Int.Conf. on the Flat Dilatometer, Edmonton, Canada, Feb 83, 50

UNDRAINED SHEAR STRENGTH from UU TRIAXIAL and from DMT

Iwasaki, Tsuchiya, Sakai, Yamamoto (1991)

Geotechnical Research Center
Kiso-Jiban Consultants Company, Tokyo

TOKYO BAY COHESIVE ALLUVIAL DEPOSITS



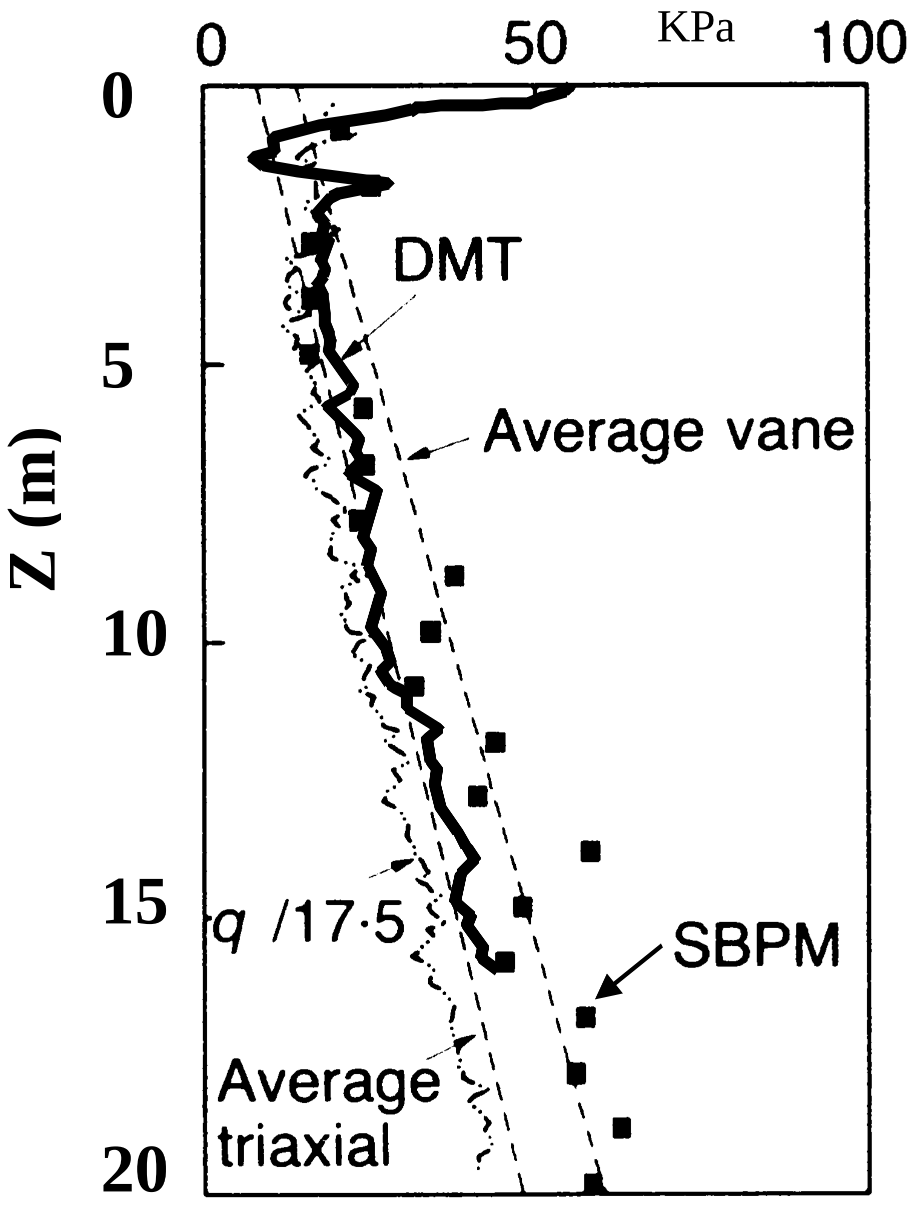
Iwasaki, K Tsuchiya H., Sakai Y., Yamamoto Y. (1991) "Applicability of the Marchetti Dilatometer Test to Soft Ground in Japan", GEOCOAST '91, Sept. 1991, Yokohama 1/6

QUALITY of Cu,$_{DMT}$

- **Generally v. satisfactory for everyday practice**
- **Of course we know existance of many Cu (TRX compression - ext., simple shear, pl. strain, FV..)**
- **But how often designers use more than one Cu ?**
- **Available results (Fucino, Bothkennar…) indicate Cu_{DMT} fits right in middle of other Cu.**
- **As to $Cu_{(TRX\ UU)}$: Designers (unless samples of exceptional quality) often prefer Cu from insitu.**
- **Cu often from Qc (Cpt), but Nc=10 to 20 (?) relatively wide range**
- **» Accuracy of Cu_{DMT} likely consequence of sensitivity of DMT to s_h , strictly related to Cu (s_h and Cu *both* largely determined by OCR)**

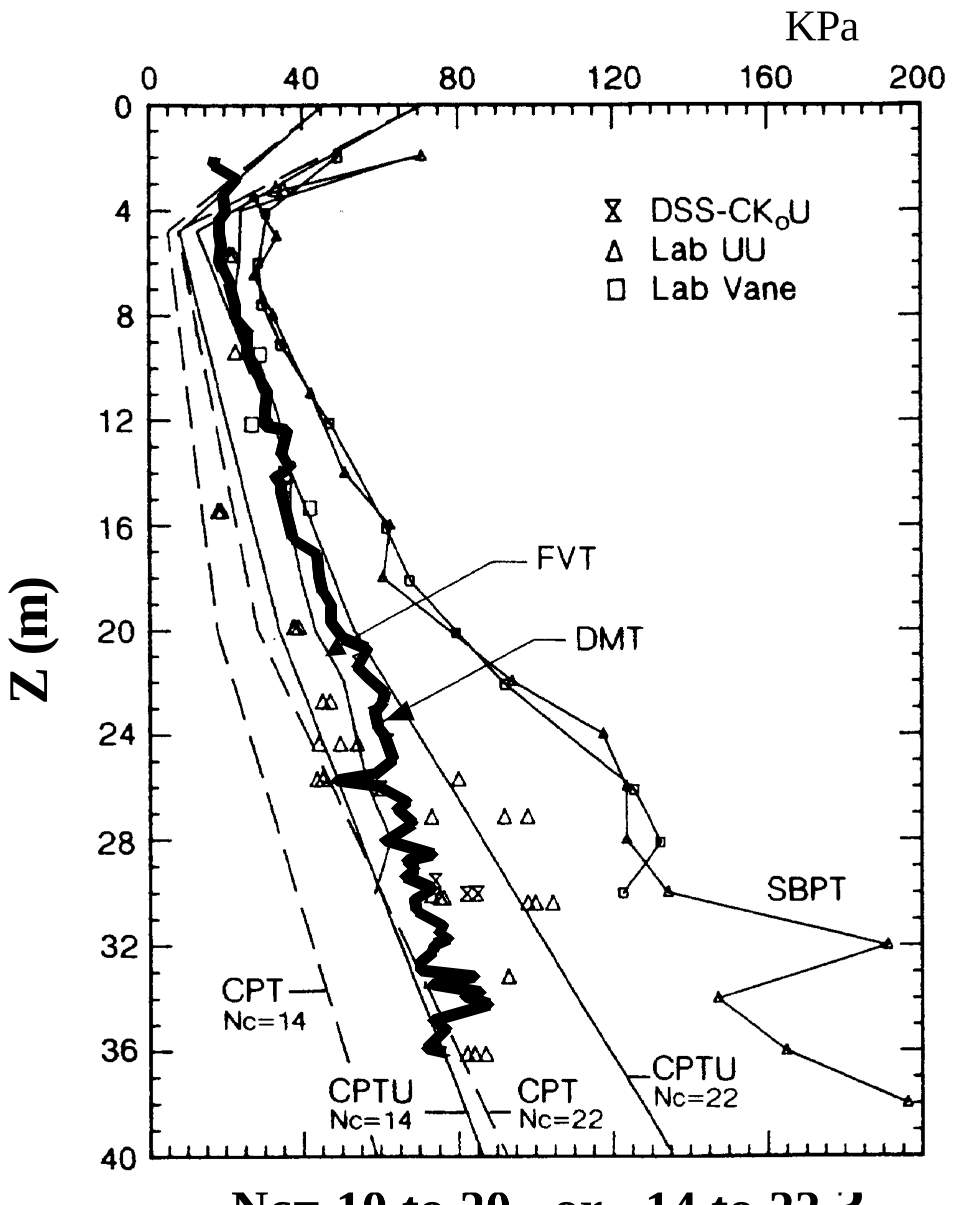
(think of similarities of correlations Cu=f(OCR) and Ko=f(OCR))

Cu at NATIONAL SITE <u>BOTHKENNAR</u> UK



Nash et al., Géotechnique, June 1995, p. 173

Cu at NATIONAL SITE FUCINO



Nc= 10 to 20 or 14 to 22 ?

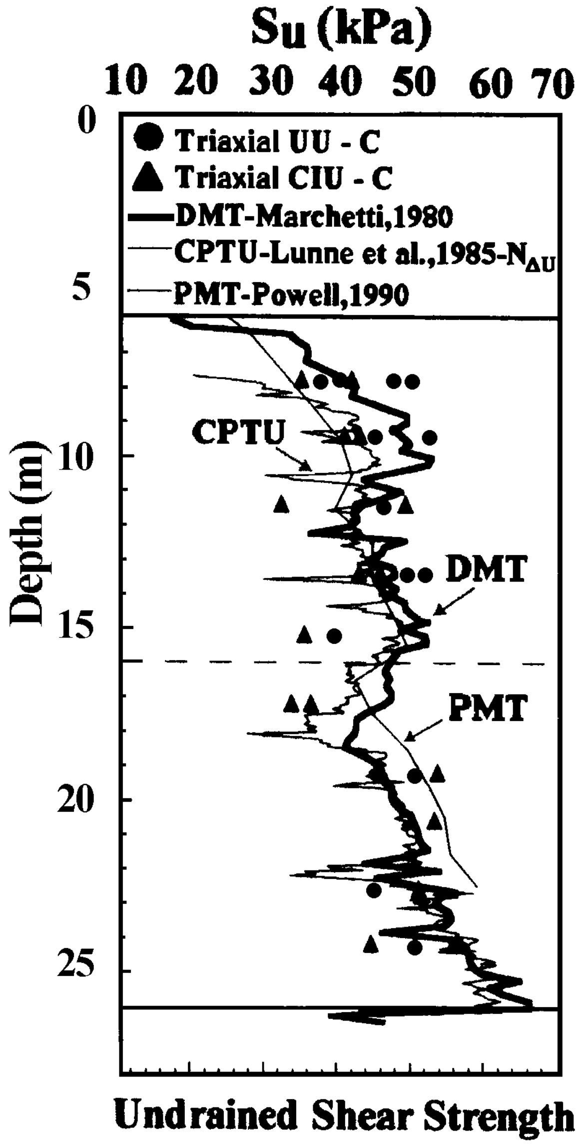
A.G.I., 10th ECSMFE Firenze 1991 Vol. 1, p. 37

Su,dmt vs Su predicted by other tests

University of Pernambuco
BRAZIL

<u>Research site</u> 1
Recife Clay

Coutinho et al., Conf. Atlanta ISC 1999, Vol. 2, 1006

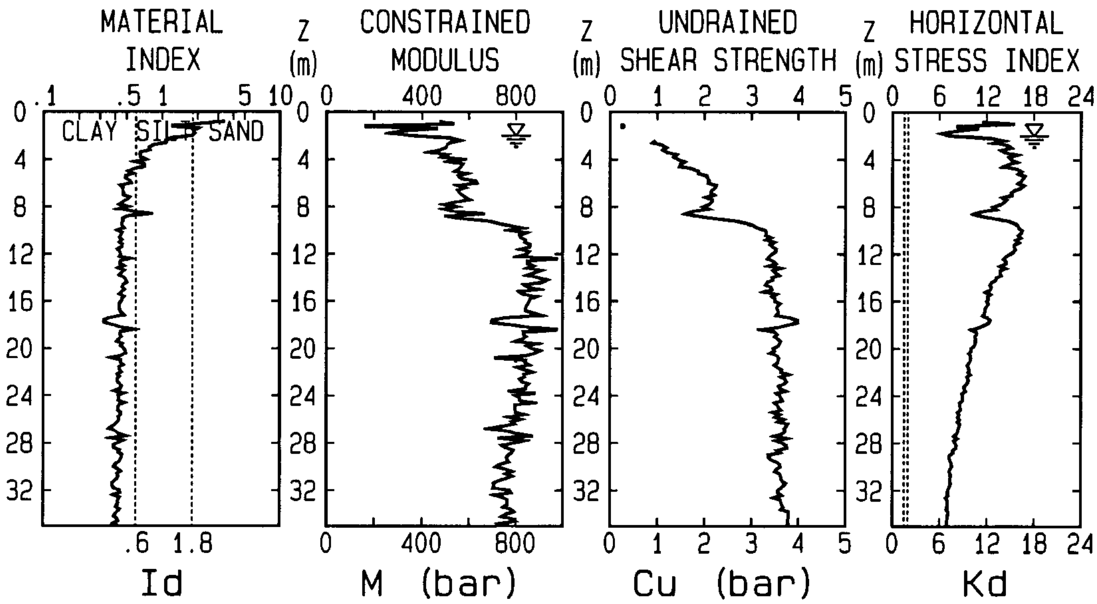


- **<u>COST</u> : 1 or 2 (Trx UU) » as entire DMT profile (+ no worry operator/ sample quality)**
- **<u>TIME</u> : Trx : weeks (months). <u>Insitu</u> : same day**

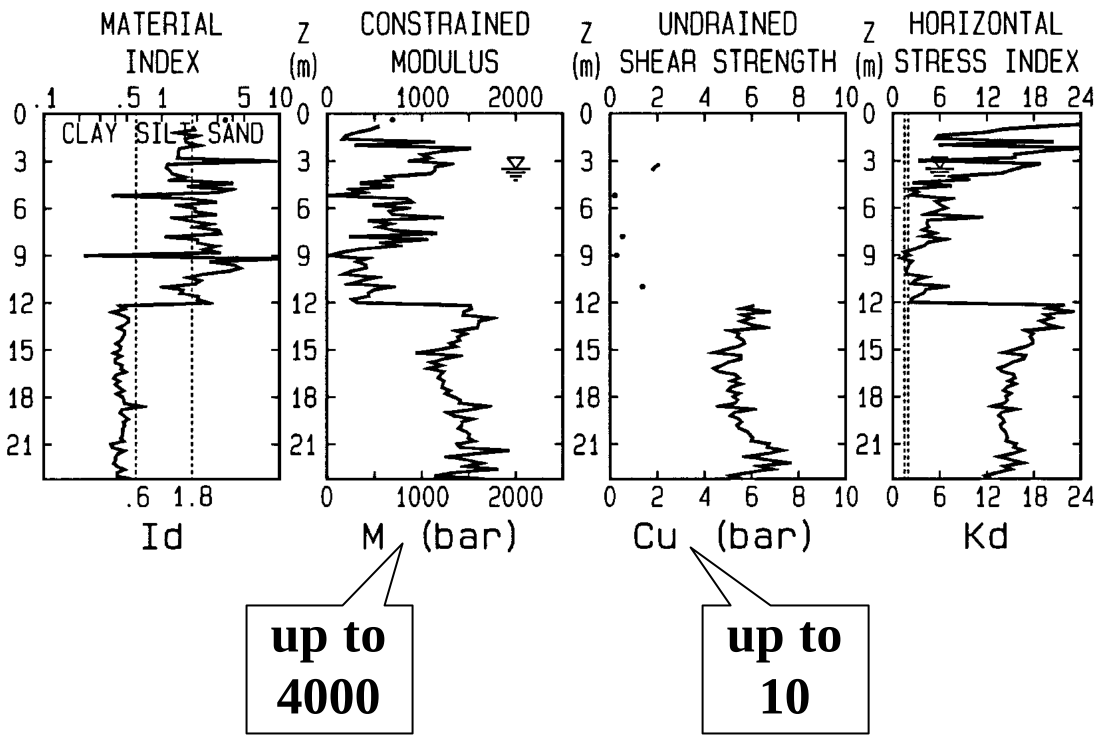
Need compare in <u>many geogr</u>. areas : when in new regions, » no need local correlation

HARDEST CLAYS testable by DMT

CHIETI - D5 (#8995)

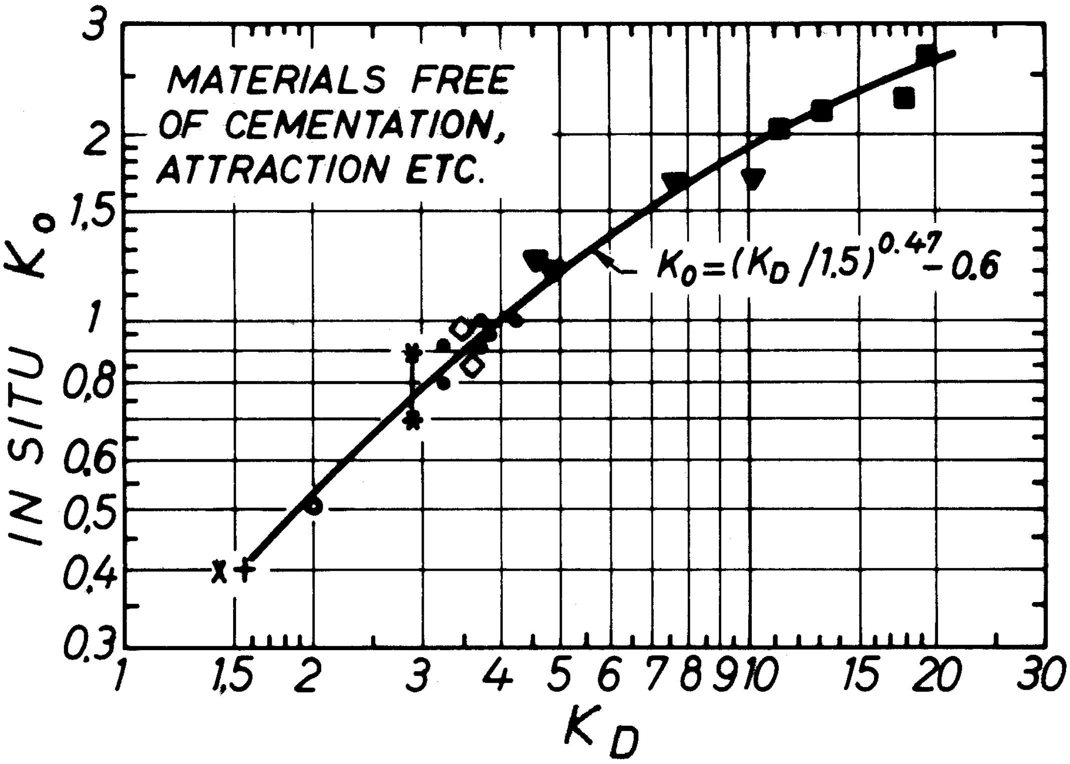


S. BARBARA (AR) - D6 (#8843)



CORRELATION (1980) Ko-Kd generally satisfactory in cohesive soils(Id< 1.2)

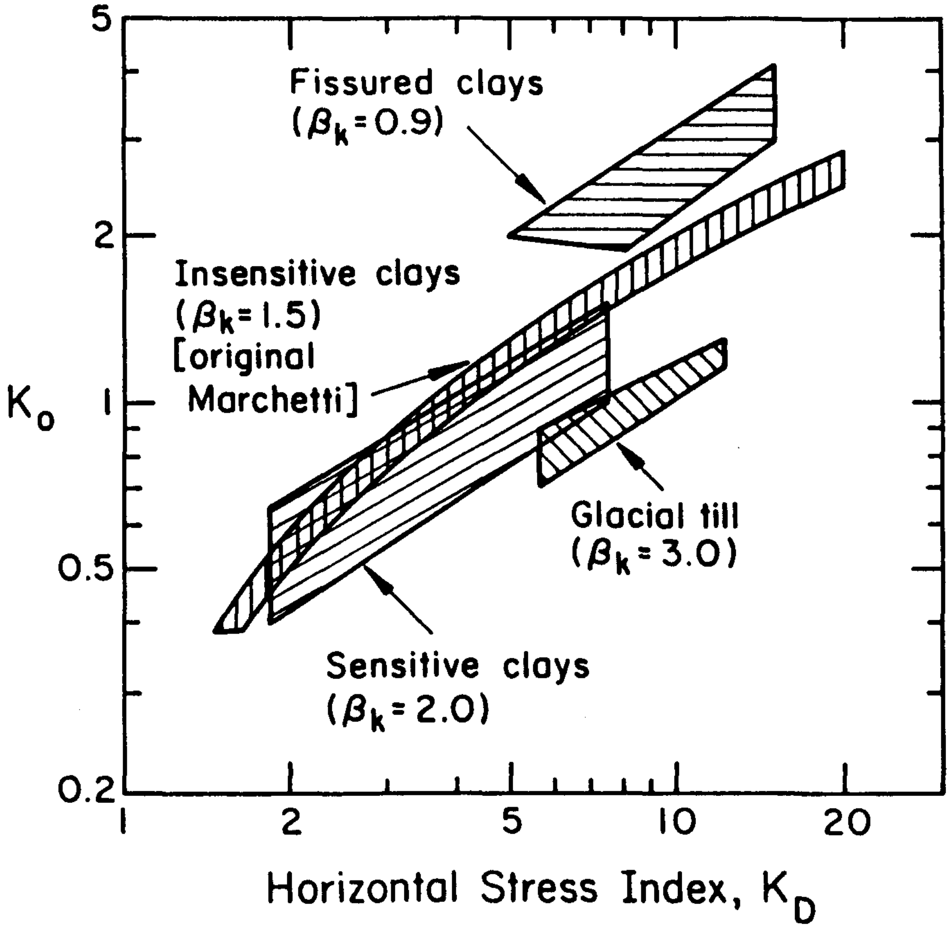
(But expect just a broad Ko evaluation)



Even in "DEVIATING" CLAYS the 1980 Ko-Kd correlation is a REASONABLE AVERAGE

Results on "Special clays" by Kulhawy(1990), Powell & Uglow (1986) and Lacasse & Lunne (1988) indicate:

- **Such clays deviate from original correlation (above or below). Yet similar trend, » parallel.**
- **Original correlation is a good average, fits well in between (hence still the one most used today)**



Ko in Sand: 1980 Correlation unsatisfactory

In sand evaluating Ko is still possible, having available, besides Kd, Qc from CPT:

Ko= C1 + C2 Kd + C3 Qc/Sigmav

with

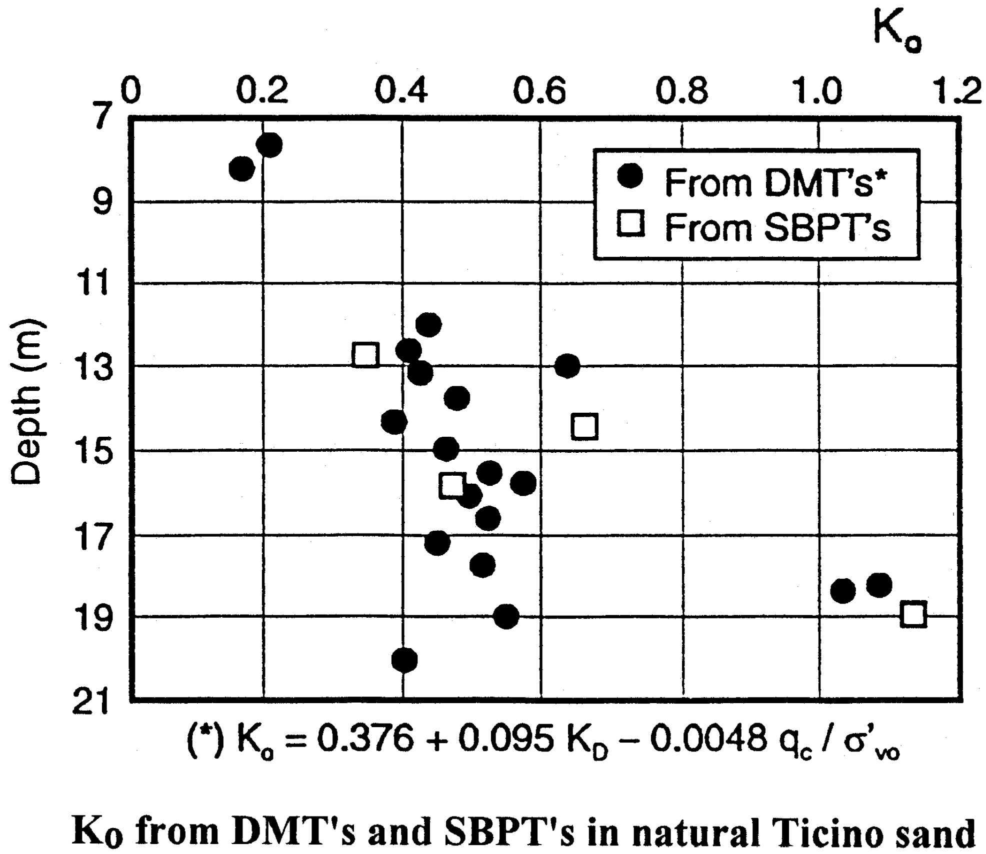
C1= 0.376 C2= 0.095

C3= -0.002 for artificial, freshly placed sand

C3= -0.005 for natural seasoned sand

Disadvantages (a) Subjectivity in the choice of C3 (b) Difficult to match Kd-Qc of same layer (c) Reliability ?

Though unpractical for routine use, some satisfactory Ko predictions have been reported (Jamiolkowski, CPT '95, Linköping - Sweden, Welcome address, Fig. 6):



Ko from DMT's and SBPT's in natural Ticino sand (Jamiolkowski 1995)

f ' from DMT in SAND (OFTEN STUDIED WITH Ko)

1980 ASCE unique Ko-Kd (clay OK), in sand unsatisfactory

1983 SCHMERTMANN combines
(CC) **Ko=f(Kd, f)** + **(**D&M) q_c **=f(Ko, f)** (b.c. with Ko)
By measuring q_c, Kd : 2 eqns, 2 unknowns → Ko, ϕ

1985 MARCHETTI

- Combine the 2 above eqns (exactly as Schmertmann 1983) eliminating ϕ (peak, residual, cv, pl strain, trx), thereby obtaining relation (see chart) Ko, Kd, q_c.
- Translate in graphical form D&M (see chart) q_c, Ko, ϕ

Then (1) Obtain Ko (Kd, q_c) from 1st chart
(2) Obtain ϕ (Ko, q_c) from 2nd chart

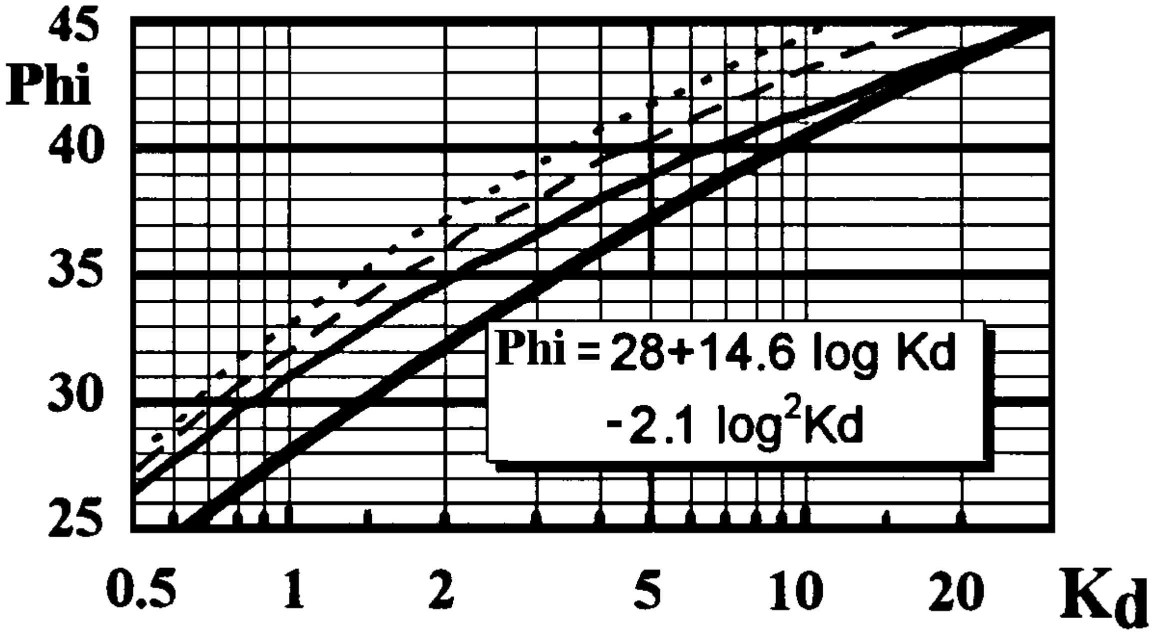
1986 BALDI...MARCHETTI..AL...
Idea still relation Ko, Kd, q_c, but abandoning previous eqns., and fitting all accumulated CC database

$$K_o = 0.376 + 0.095\, K_D - 0.0046\, q_c / \sigma'_{vo}$$

(hence Ko). For ϕ → D&M chart

1991 CAMPANELLA & ROBERTSON
Add scale for Kd in D&M chart, hence can read ϕ (with a rough estimate of Ko)

1996 MARCHETTI
Uses Campanella & R. (1991) to obtain Kd-ϕ curves.(for 3 Ko assumptions, get 3 curves). Since for low ϕ the 3 curves appear to overpredict ϕ (vs CC data) draw a lower line. Tentative recommended line. Not meant to be most likely estimate of ϕ, but *operative low estimate*.(e.g. maybe helpful in case of disturbed samples) (≈ verified for sites qc-Kd)

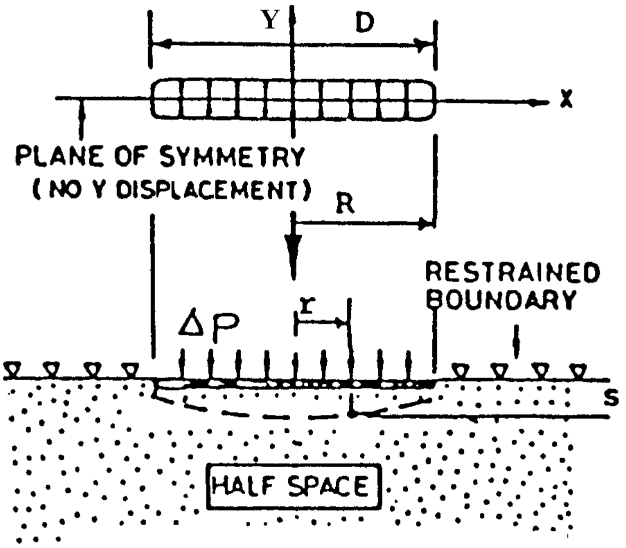


Use lower curve (or equation) for a safe estimate (lower bound) of f (uncemented sands). Marchetti, 1997

Marchetti S. (1997). "The Flat Dilatometer : Design Applications". Third Geotechnical Engineering. Conf. Cairo University, Jan. 1997, Keynote lecture, 26 pp.

Ed	**Dilatometer Modulus** (deduced by theory of elasticity)

CROSS SECTION



$$S = \frac{2\,D\,\Delta p}{\pi}\,\frac{(1-\mu^2)}{E}\sqrt{1-\left(\frac{r}{R}\right)^2}$$

At center, **Dp** has caused **<u>1.1 mm</u>**.

For **D=60 mm**, **S** = 1.1 mm formula ⇒

<u>Dilatometer Modulus</u> = E_d = $E/(1-n^2)$ = 37.4 Dp

E_d=34.7 Dp

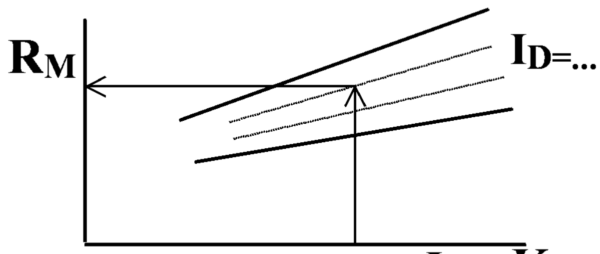
Elastic formula by **Gravesen** S. "Elastic Semi-Infinite Medium bounded by a Rigid Wall with a Circular Hole", Danmarks Tekniske Højskole, No. 11, Copenhagen, 1960, p. 110.

Before use, E_D must be converted to M

Correction factor $\underline{R_M} = f(K_D, I_D)$ Soil type
Stress history

$$M = R_M E_D$$

(M=1-D modulus)



ASCE '80 Eq. 9/ Fig. 13a

- Rm similar to K in M=K q_C (CPT) with the difference
  - -K= 2-20 = f(OCR ??, Dr)
  - -Rm = 1-3 = f(Id, Kd)

- Ed should not evoke Young's E'
  - -Ed lacks Stress Hist. (not to be used in deform analysis)
  (same situation as trying to derive E from Qc without Stress History)
  - -Use M. If E' required, infer it from M (E' » 0.8 M)

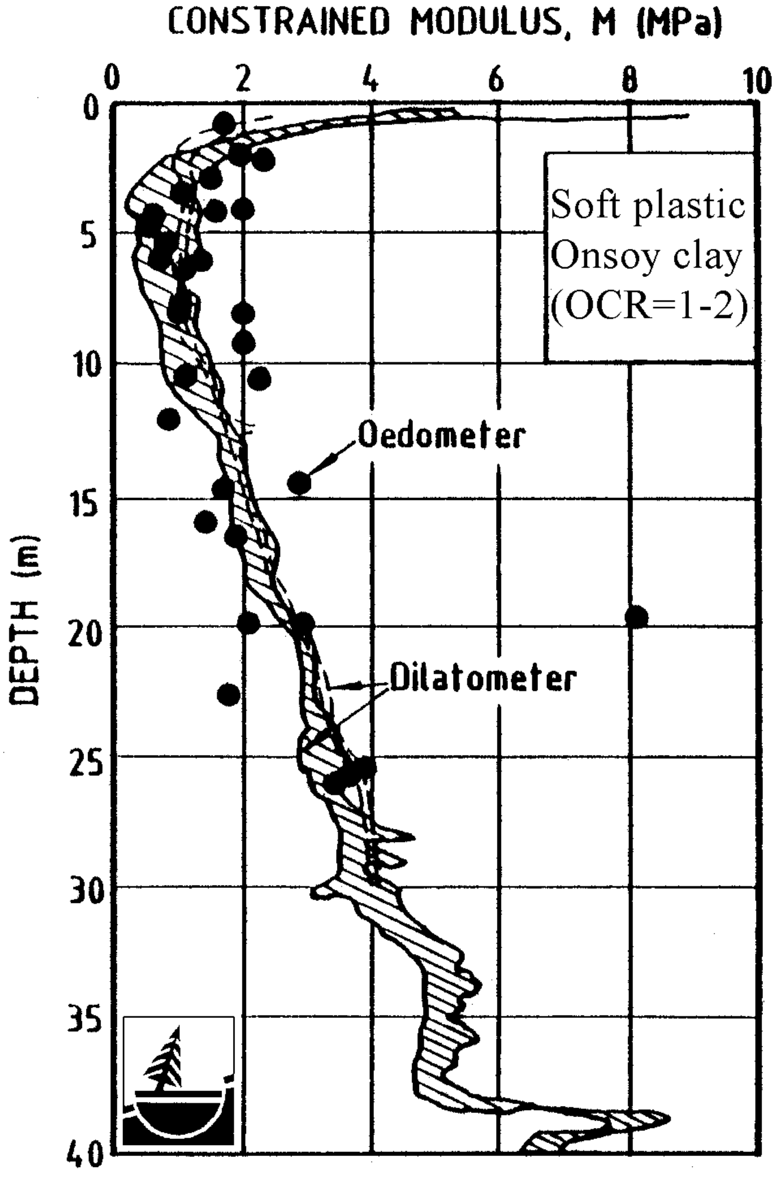
REASONS OF $\underline{R_M}$: MANY TASKS (CONVERSIONS)

• Ed on distorted soil • Horizontal-vertical • Drained-Undrained **(Skempton param.)**	function of ➔	• Soil type (**ID**) • Stress Hist. (**KD**)

Above reasons support correl. M-Ed (**ID, KD pa**rameters), but last word to real life e.g. comparing highly qualified:

(1) M/ Mref (2) Settlement observed/ predicted **» OK**

CONSTRAINED 1-D MODULI FROM OEDOMETER AND FROM DMT



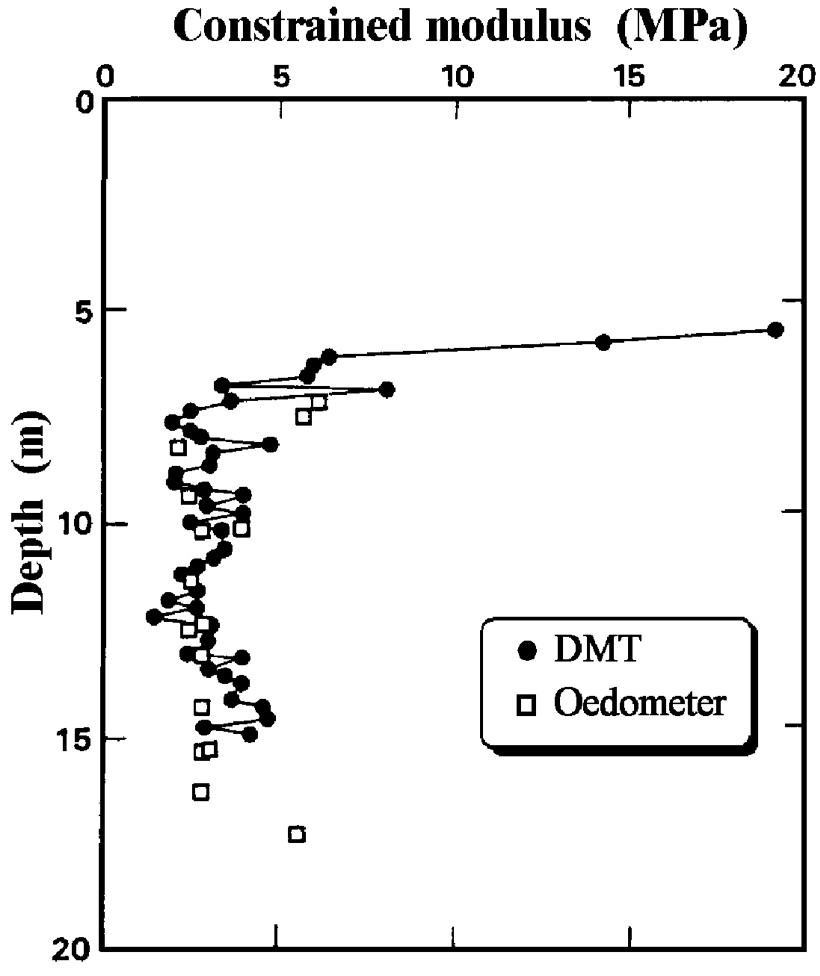
Norwegian Geotechnical Institute (1986). "In Situ Site Investigation Techniques and interpretation for offshore practice" Report 40019-28 by S. Lacasse, Fig. 16a, 8 Sept 86

CONSTRAINED 1-D MODULI FROM OEDOMETER AND FROM DMT

Iwasaki, Tsuchiya, Sakai, Yamamoto (1991)

Geotechnical Research Center
Kiso-Jiban Consultants Company, Tokyo

TOKYO BAY COHESIVE ALLUVIAL DEPOSITS



Iwasaki, K Tsuchiya H., Sakai Y., Yamamoto Y. (1991) "Applicability of the Marchetti Dilatometer Test to Soft Ground in Japan", GEOCOAST '91, Sept. 1991, Yokohama 1/6

SOIL **PROPERTIES** FROM IN SITU TESTS by "***TRIANGULATION***"

-**UNABLE** MEASURE IN SITU **PURE** SOIL PROPERTIES

-**RESPONSES** IN SITU: **MIXED FUNCTION** *PURE* S.P.

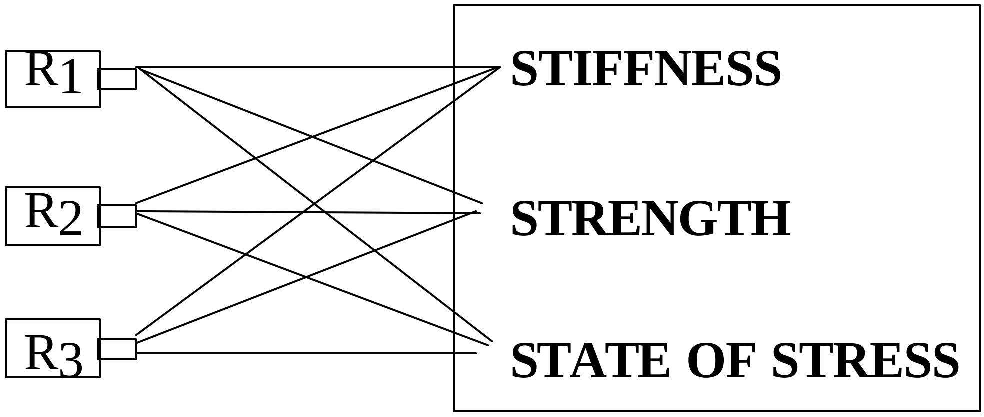
-TO **ISOLATE** *PURE* S.P. : "***TRIANGULATION***"

-say DOMINANT **STIFFNESS**, **STRENGTH**, **STATE STRESS**

-NEED 3 INDEP. IN SITU RESPONSES R1, R2, R3

$R_1 = f_1$ (M, Strength, σ_h)	**INVERT**	M $= g_1 (R_1,R_2,R_3)$
$R_2 = f_2$ (M, Strength, σ_h)	**MATRIX**	Strength $= g_2 (R_1,R_2,R_3)$
$R_3 = f_3$ (M, Strength, σ_h)	**GET**	$\sigma_h = g_3 (R_1,R_2,R_3)$

TRIANGULATION ≈ 3-Chromia to re-create colors

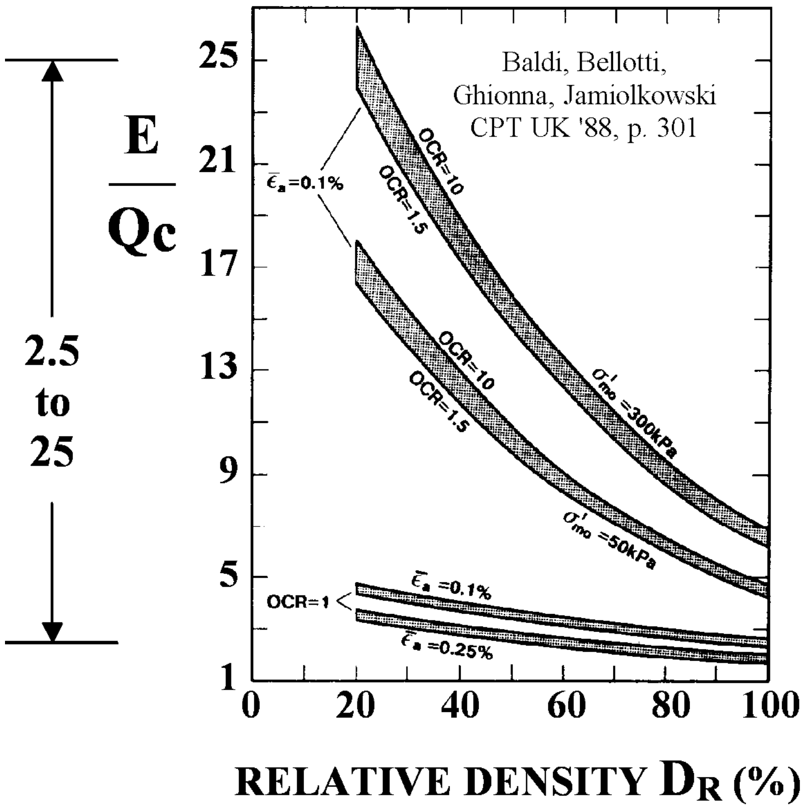


DMT is a TWO-PARAMETER TEST

Makes a lot of difference compared with 1-parameter tests as CPT or SPT (3 would be better, but diminishing returns… Moreover 3rd parameter would be valuable if truly independent - DMT tip q_D not sure…)

1-parameter test Þ wide UNCERTAINTY

- **1 information : NO MATRIX to INVERT**
- **E=2.5 to 25 Q_C (factor 10) = f(Dr, OCR-s_h)**
- **Impossible get 3 *unknowns* (E, Dr, OCR) from just 1 information (Q_C)**

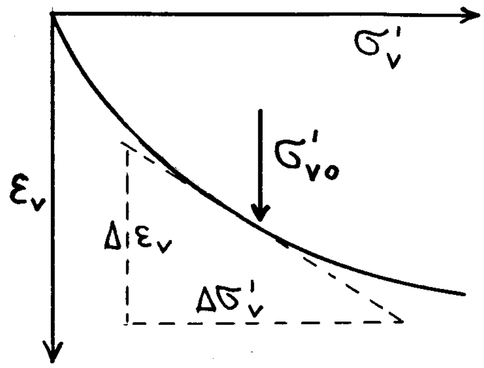


- **If we adopt ave. E= 7 Q_C, E could be off (settlements) by factor 3 (wide).**
- **Jamiolkowski** et al. **conclude (Isopt-1, '88, Vol. 1, p.263) :**

"w/o **Stress History, impossible to select reliable E (or M) from Q_C"**

1. Definition of M (no ambiguity)

M = Eoed=1/m_V= $\Delta\sigma'_v/\Delta\varepsilon_v$ (at σ'_{vo}).



Vert. drained confined tangent (at σ'_{vo}) modulus

Name : Eoed OK, traditionally measured by oedometer. *Improper* if not by oed (correlations). Hence "M", but same.

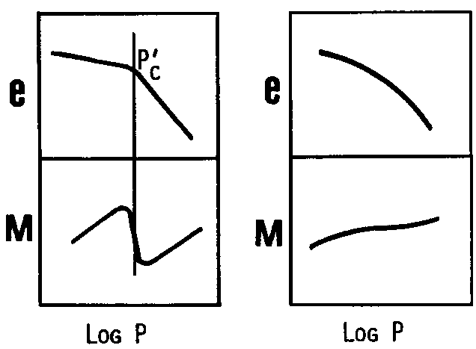
Usual range M_{DMT} **<u>4 - 4000 bar</u>**

2. M for what settlement (initial, 1^{ry}, 2^{ry}) ?

M is just for **<u>primary</u>**. Correlations were established by calibrating vs Eoed (1-D). **M_{DMT}** **<u>must be treated as if by oedometer</u>**.

Use same methods as with oedometers, including, if applicable, usual corrections (depth, shape, rigidity, possibly Skempton-Bjerrum).

3. May use M = constant if D s '$_v$ large ?



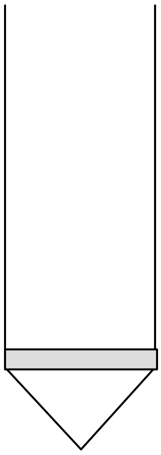
If $\Delta\sigma'_v$ large : σ'_v may exceed p_c. **(?)**

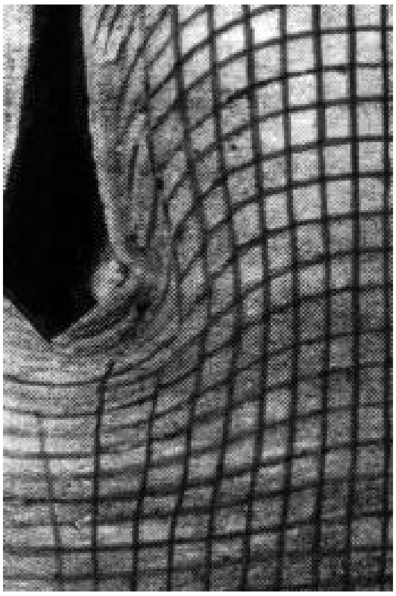
Many structured NC clays (eg some Canadian) : sharp break in e-log p curve ⇒ marked drop in M at p_c. There M_{DMT} may be too high.

<u>But in many common clays</u>, (in most sand?) M accross p_c mild fluctuation, hence M=const. ≈ OK

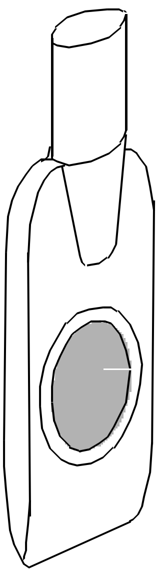
DISTORTIONS due to INSERTION

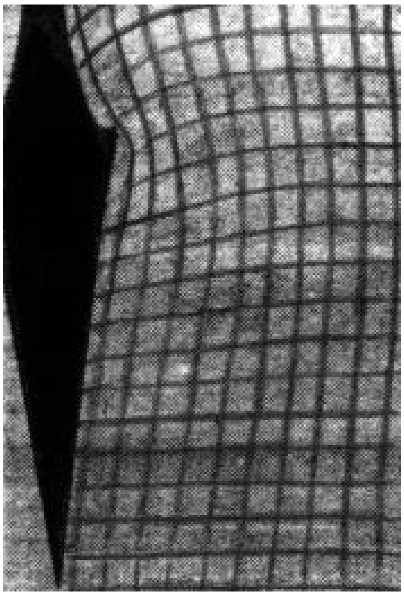
CONE





WEDGE





Photographs of distortions in clay from : Baligh & Scott (Nov. 1975) "Quasi-Static Deep Penetration in Clay", Jnl. ASCE Geot. Eng. Div.

USE OF IN SITU / LAB

SCHMERTMANN **(**BANQUET talk **CPT '95** Sweden)

...while in the PAST **LAB** had **PRIMARY** ROLE in a SITE INVESTIGATION, and **IN SITU** TESTING **COMPLEMENTARY** today often **IN SITU PRIMARY**, **LAB COMPLEMENTARY**

EXAMPLE: SUNSHINE SKYWAY SUSPENSION BRIDGE IN TAMPA (Schmertmann & Crapps RESPONSIBLE FOR FOUNDATIONS) **:**

99% IN SITU, 1% LAB

(Reason of LAB **:** AVOID COLLEAGUE CRITICISM !)

(Provocative : We all respect lab = Source of our understanding, many behavioural details can be studied only in lab...)

TESTING IN THE LABORATORY

- **TO STUDY PATTERN OF BEHAVIOUR / FORMAT OF THE MODELS**
- **TO MEASURE, IN CONTROLLED CONDITIONS, MANY PARAMETERS**

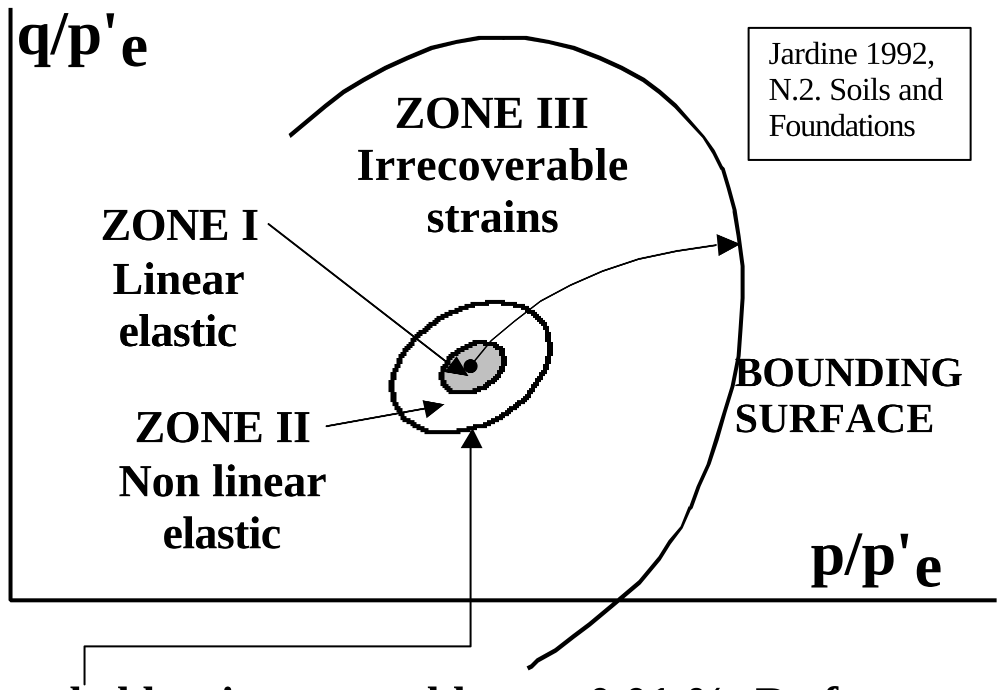
BUT

- **LAB Element 1 INSITU Element (missing LINK)**
- **HIATUS : no matter how careful the sampling**
- **DIFFERENCE : Unknown, Variable, Non Reproducible**

ELASTIC REGION very SMALL
THRESHOLD to irricoverable » 0.01%

Exceeded when element INSITU ➔ LAB

Recent low-strain research, starting from a lab TRX specimen in given stress state, identified 3 zones



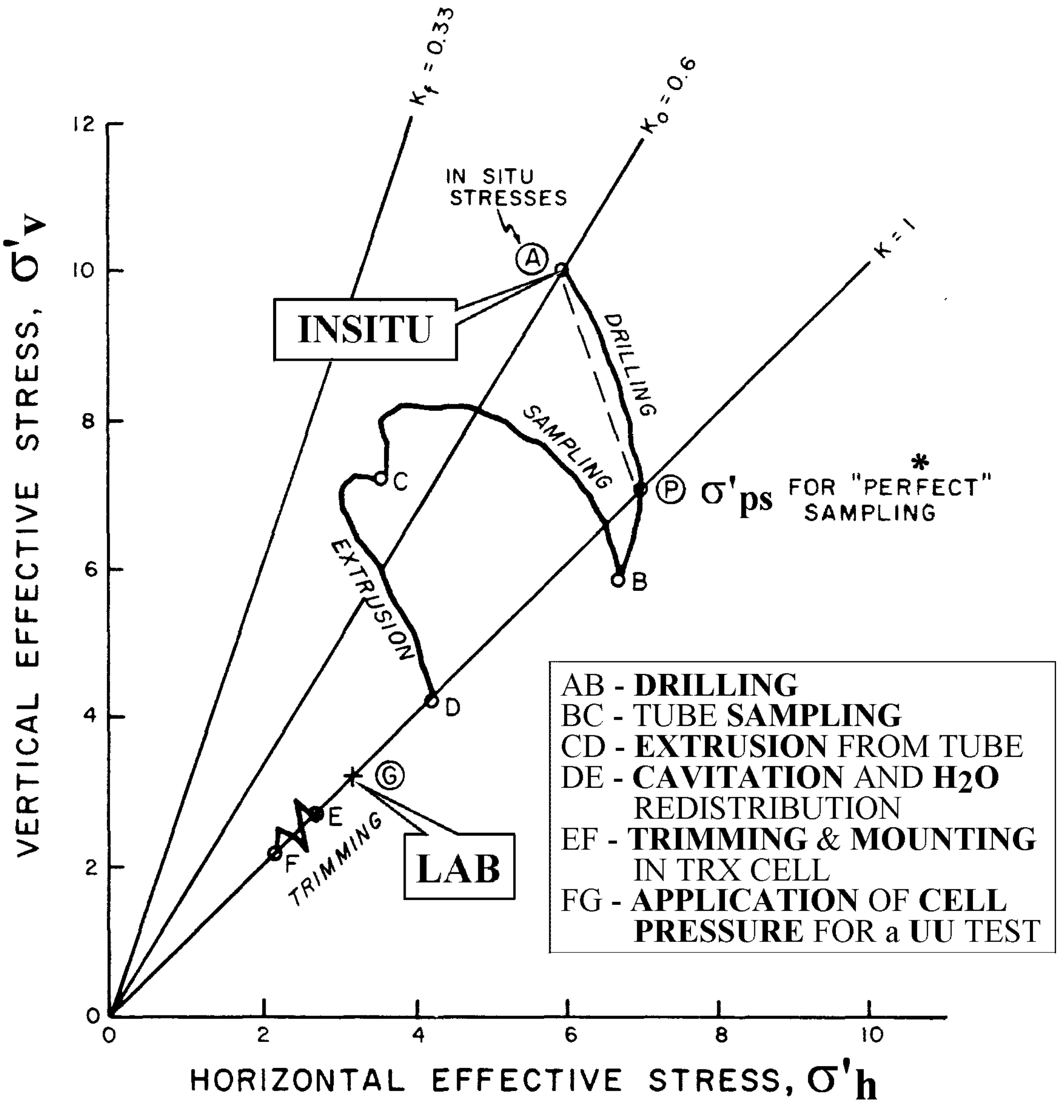
<u>Threshold</u> to irrecoverable e » 0.01 %. By far exceeded by sampling

---

Not completely new ! Terzaghi-Peck (1967):
"Compressibility of <u>even good</u> OC samples may be 2 to 5 times larger than the in situ compressibility".

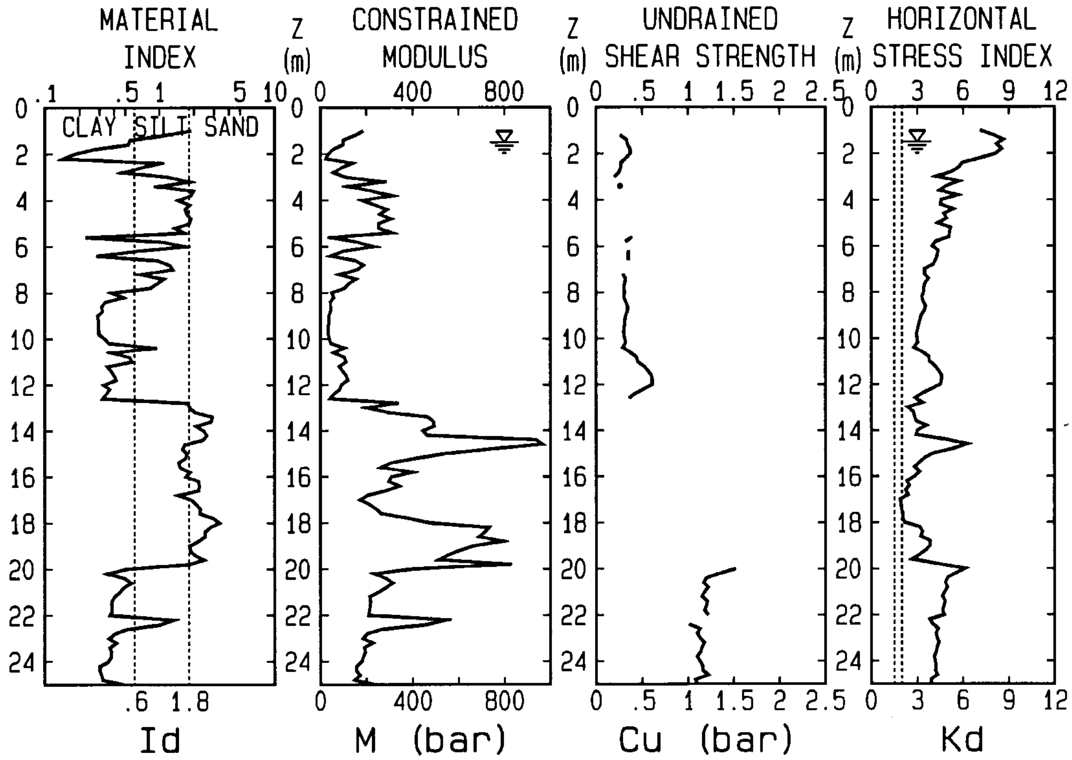
(to be sure, oedometer samples - especially OCR - resent, besides disturbance, also **deficient** σ_h ! Oed starts with σ_h =0. Even at σ'_{vo} : $\sigma_{h,oed} << \sigma_{h,insitu}$)

Stress-Strain TRAUMA from INSITU to LAB
"IDEAL SAMPLE" (Ladd and Lambe 1963)



Ladd and Lambe (1963) "The Strenght of 'Undisturbed' Clay" from Undrained tests". ASTM-NRC Symposium, Ottawa

EXAMPLE OF DMT RESULTS



HOW TO USE DMT RESULTS

- **M and Cu : common, usual way**
- **Id : soil type (sand, silt, clay)**
- **Kd same shape OCR (useful to *understand* history of deposit). NOTE : Kd=2 ® OCR » 1**

EXAMPLE OF TABULAR OUTPUT

SOIL TEST   D M T : D3 - 4 OCT 1989   Reg 1003
LIVORNO HARBOUR   NEW QUAY   WATERTABLE m 1.5

Reduction formulae according to ASCE Geot.Jnl.,Mar. 1980, Vol.109, 299-321
NOTE : OCR = ''relative OCR''. OCR below often reasonable. Accuracy can be improved if precise OCR values are available. Then factorize all OCR below by the ratio OCRreference/OCR

Symbol	Definition	Unit
Po	= Corrected A reading	bar
P1	= Corrected B reading	bar
Gamma	= Bulk unit weight/GammaH2O	(-)
Sigma'	= Effective overb. stress	bar
U	= Pore pressure	bar
Id	= Material Index	(-)
Kd	= Horizontal stress index	(-)
Ed	= Dilatometer modulus	bar

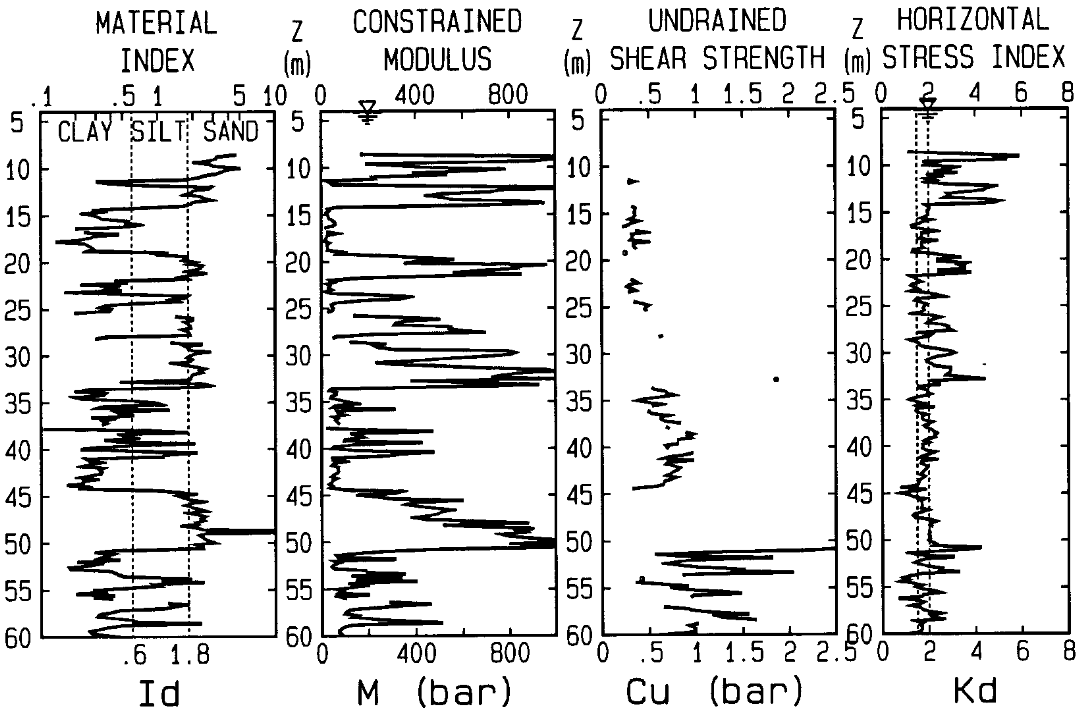
INTERPRETED GEOTECHNICAL PARAMETERS

Symbol	Definition	Unit
Ko	= In situ earth press. coeff.	(-)
Ocr	= Overconsolidation ratio	(-)
Phi	= Safe floor value of friction angle	(-)
M	= Constrained modulus (at Sigma')	bar
Cu	= Undrained shear strength	bar

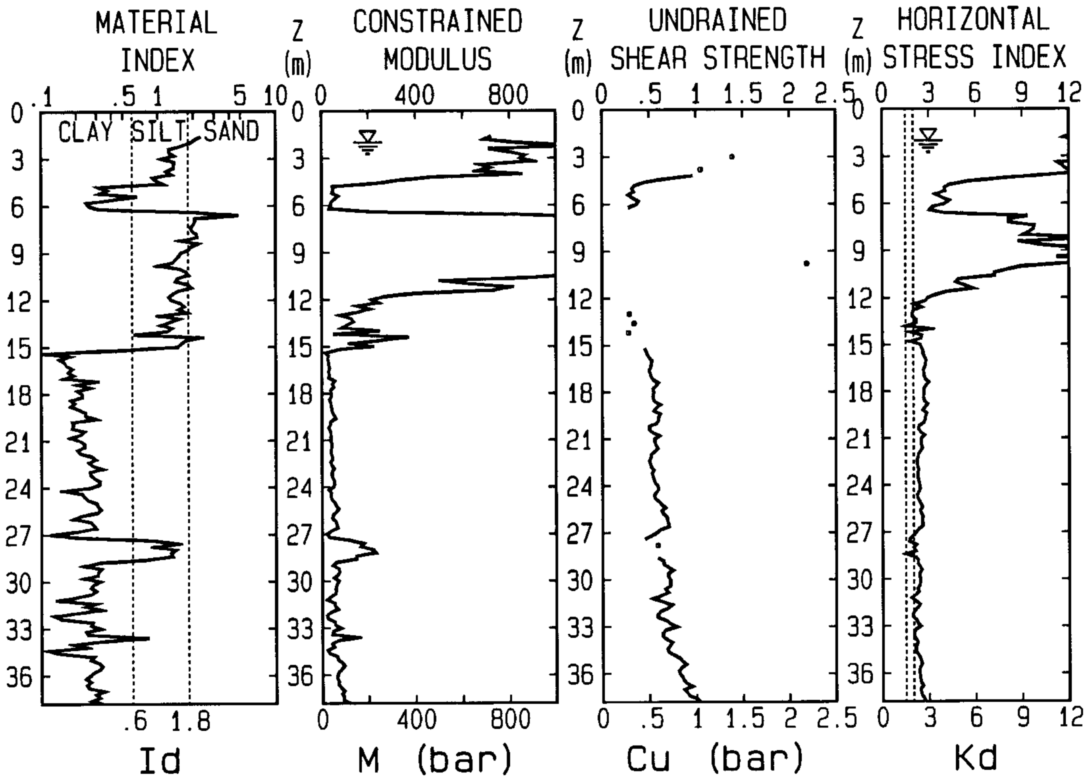
Z (m)	Po	P1	Gamma	Sigma'	U	Id	Kd	Ed	Ko	Ocr	Phi	M	Cu	DESCRIPTION
1.00	1.2	3.5	1.80	0.17	0.00	1.89	7.1	85			39	186		SILTY SAND
1.20	1.6	3.4	1.70	0.21	0.00	1.09	7.9	66	1.6	8.6		150	0.26	SILT
1.40	2.1	3.2	1.70	0.24	0.00	0.56	8.6	43	1.7	9.8		100	0.33	SILTY CLAY
1.60	2.2	3.3	1.70	0.26	0.01	0.53	8.2	43	1.6	9.0		98	0.34	SILTY CLAY
1.80	2.4	3.0	1.60	0.28	0.03	0.27	8.5	23	1.7	9.6		55	0.37	CLAY
2.00	2.4	2.8	1.60	0.29	0.05	0.18	8.2	16	1.6	9.0		36	0.37	CLAY
2.20	2.2	2.5	1.50	0.30	0.07	0.15	7.1	12	1.5	7.3		25	0.32	MUD
2.40	1.9	3.8	1.70	0.31	0.09	1.02	6.0	70	1.3	5.5		139	0.27	SILT
2.60	2.0	3.2	1.70	0.32	0.11	0.68	5.7	47	1.3	5.2		90	0.27	CLAYEY SILT
2.80	1.9	2.7	1.60	0.34	0.13	0.48	5.2	31	1.2	4.4		57	0.25	SILTY CLAY
3.00	1.6	3.4	1.70	0.35	0.15	1.19	4.3	66	1.0	3.3		110	0.20	SILT
3.20	2.2	6.0	1.80	0.36	0.17	1.81	5.7	140			38	276		SILTY SAND
3.40	2.0	3.8	1.70	0.38	0.19	0.96	4.9	66	1.1	4.0		118	0.26	SILT
3.60	2.0	5.4	1.80	0.39	0.21	1.97	4.5	128			37	224		SILTY SAND
3.80	2.5	6.8	1.90	0.41	0.23	1.87	5.6	159			38	312		SILTY SAND
4.00	2.2	5.0	1.70	0.43	0.25	1.45	4.6	105			37	183		SANDY SILT
4.20	2.3	5.9	1.80	0.44	0.26	1.85	4.5	136			37	238		SILTY SAND
4.40	2.7	6.8	1.80	0.46	0.28	1.67	5.4	152			38	289		SANDY SILT
4.60	2.5	6.4	1.70	0.47	0.30	1.73	4.7	144			37	258		SANDY SILT
4.80	2.7	7.2	1.90	0.49	0.32	1.89	4.9	167			37	306		SILTY SAND
5.00	2.5	6.5	1.80	0.50	0.34	1.82	4.4	148			36	253		SILTY SAND
5.20	3.1	6.7	1.80	0.52	0.36	1.37	5.2	136			37	253		SANDY SILT
5.40	3.1	7.6	1.80	0.53	0.38	1.65	5.1	167			37	310		SANDY SILT
5.60	3.2	3.8	1.70	0.55	0.40	0.23	5.1	23	1.2	4.3		42	0.39	CLAY
5.80	2.8	5.4	1.70	0.56	0.42	1.10	4.2	97	1.0	3.2		160	0.32	SILT
6.00	2.7	6.6	1.80	0.58	0.44	1.69	4.0	144			36	233		SANDY SILT
6.20	3.1	4.6	1.70	0.59	0.46	0.61	4.4	58	1.1	3.4		96	0.35	CLAYEY SILT
6.40	3.1	3.8	1.70	0.61	0.48	0.28	4.3	27	1.0	3.3		44	0.35	CLAY
6.60	3.1	5.6	1.70	0.62	0.50	0.97	4.2	93	1.0	3.2		152	0.35	SILT
6.80	3.1	6.2	1.70	0.63	0.52	1.23	4.0	117			36	187		SANDY SILT
7.00	2.8	5.7	1.70	0.65	0.54	1.31	3.5	109			35	159		SANDY SILT
7.20	2.9	4.4	1.70	0.66	0.56	0.69	3.5	58	0.88	2.4		83	0.29	CLAYEY SILT
7.40	3.1	5.8	1.70	0.68	0.58	1.08	3.7	101	0.93	2.6		154	0.32	SILT
7.60	3.0	5.3	1.70	0.69	0.60	0.95	3.5	85	0.89	2.4		124	0.31	SILT
7.80	3.0	5.0	1.70	0.70	0.62	0.83	3.4	74	0.88	2.3		105	0.30	SILT
8.00	3.1	4.0	1.70	0.72	0.64	0.39	3.4	35	0.87	2.3		49	0.31	SILTY CLAY
8.20	3.1	4.2	1.70	0.73	0.66	0.48	3.3	43	0.85	2.2		58	0.30	SILTY CLAY
8.40	3.2	4.0	1.70	0.74	0.68	0.33	3.4	31	0.86	2.3		43	0.32	SILTY CLAY
8.60	3.4	4.2	1.70	0.76	0.70	0.31	3.6	31	0.90	2.4		45	0.34	CLAY
8.80	3.4	4.2	1.70	0.77	0.72	0.31	3.5	31	0.88	2.4		44	0.34	CLAY
9.00	3.3	4.0	1.70	0.79	0.74	0.29	3.3	27	0.84	2.1		37	0.32	CLAY
9.20	3.3	4.0	1.70	0.80	0.76	0.29	3.2	27	0.82	2.1		36	0.31	CLAY
9.40	3.3	4.0	1.70	0.81	0.77	0.29	3.1	27	0.81	2.0		35	0.31	CLAY
9.60	3.3	4.0	1.70	0.83	0.79	0.29	3.0	27	0.79	1.9		35	0.30	CLAY
9.80	3.3	4.0	1.70	0.84	0.81	0.30	3.0	27	0.77	1.8		34	0.30	CLAY
10.00	3.4	4.2	1.70	0.85	0.83	0.33	3.0	31	0.78	1.9		39	0.31	CLAY
10.20	3.5	4.4	1.70	0.87	0.85	0.36	3.0	35	0.79	1.9		45	0.32	SILTY CLAY
10.40	3.3	5.6	1.70	0.88	0.87	0.94	2.8	85	0.74	1.7		104	0.29	SILT
10.60	3.9	5.0	1.70	0.90	0.89	0.39	3.3	43	0.85	2.2		59	0.37	SILTY CLAY

DMT in NC sites

VENEZIA LIDO

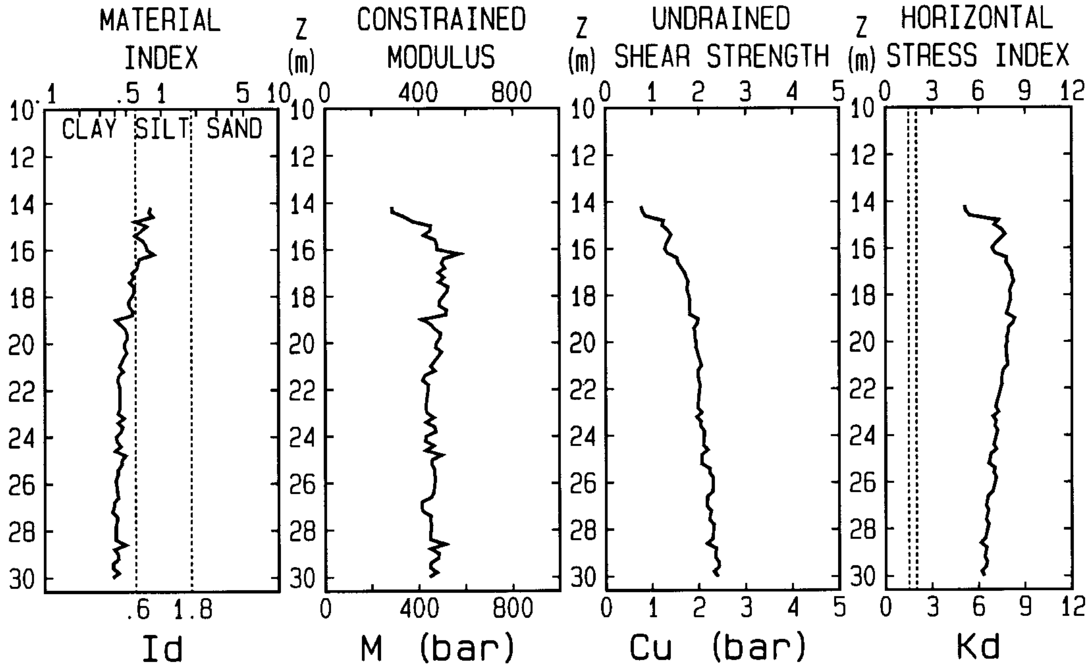


STAGNO LIVORNO

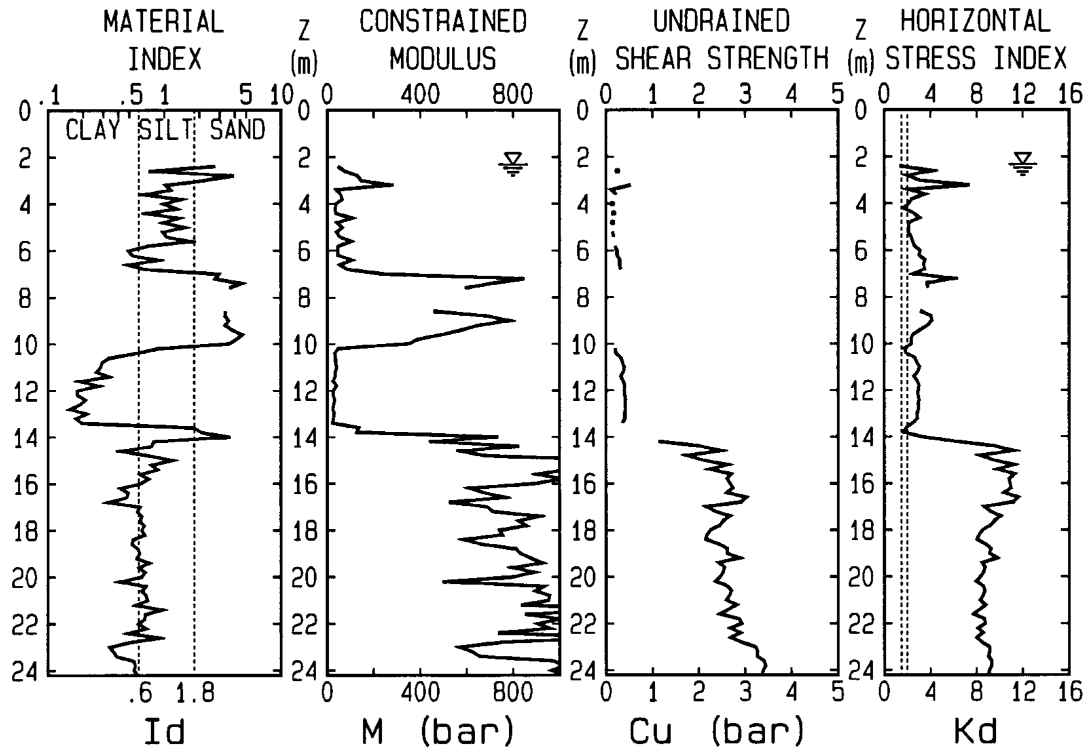


DMT in OC sites

AUGUSTA



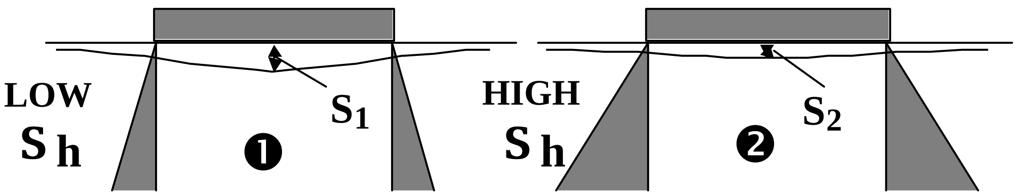
TARANTO

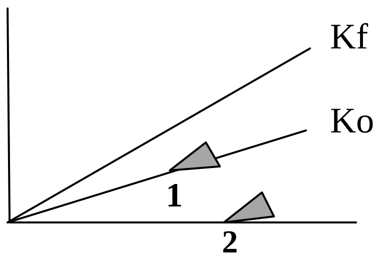


S_h	High S_h Reduce SETTLEMENTS

- **Well documented by Massarsch (1994) in a LOOSE SANDFILL Compaction case** (see also Leonards & Frost 1988...)
- **Compaction /densification causes OC, increasing s_h, which reduces <u>substantially</u> settlements.**

i.e. same sand, same q : $S_2 << S_1$





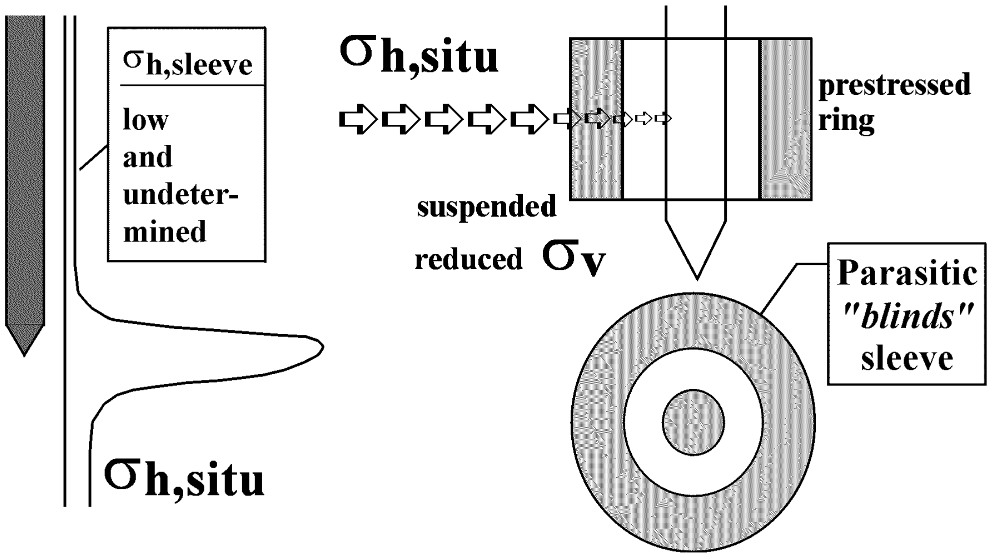
Reasons why Sand #2 is stiffer:
(a) More distant from K_f (b) Higher s_{ave} (> M)
For a given sand Modulus = f (s_o, Ds_h, Path)

Since s_h important for settlements, better be aware !

Cylindrical Probes Scarcely Sensitive to S_h due to ARCHING

Hughes & Robertson (Can. G. Jn. Aug. 1985) **analysed stresses around circular probes in sand :**

- **Behind tip, enormous stress reduction**
- **Ring of high residual stresses (at some distance)**
- **"Parasitic" RING obstacle to s_h (blinding effect) limits and makes undetermined $s_{h,sleeve}$**
- **Moreover $s_{h,sleeve}$ itself is NOT measured horizontally, but from fs (vert. force), via d (?)**



Laterally suspended **floating** "**ring**" also reduces s_v at base
⇒ could be reason why Qc (z) < linear

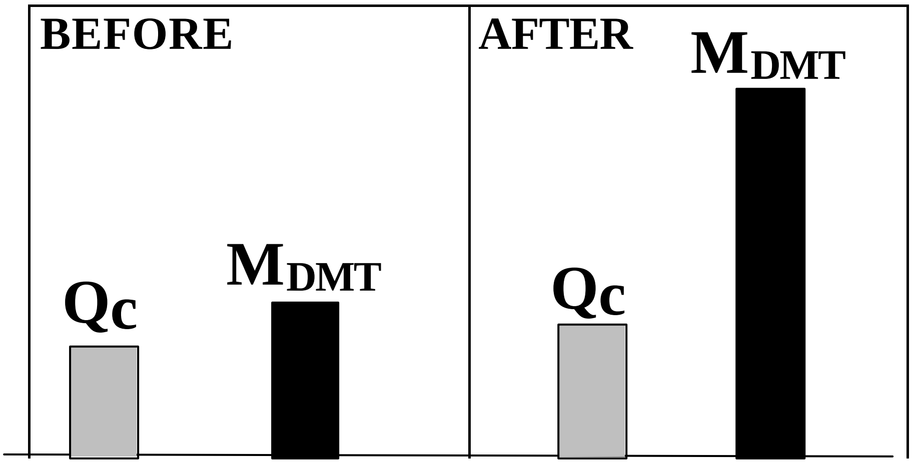
Similar conclusions (sand) Huang, Can. G. Jn. Feb 1994
...$s_{h,sleeve}$ <u>INSENSITIVE</u> to $s_{h,in\ situ}$ + <u>UNSTABLE</u>
(reason why f_S » unused)

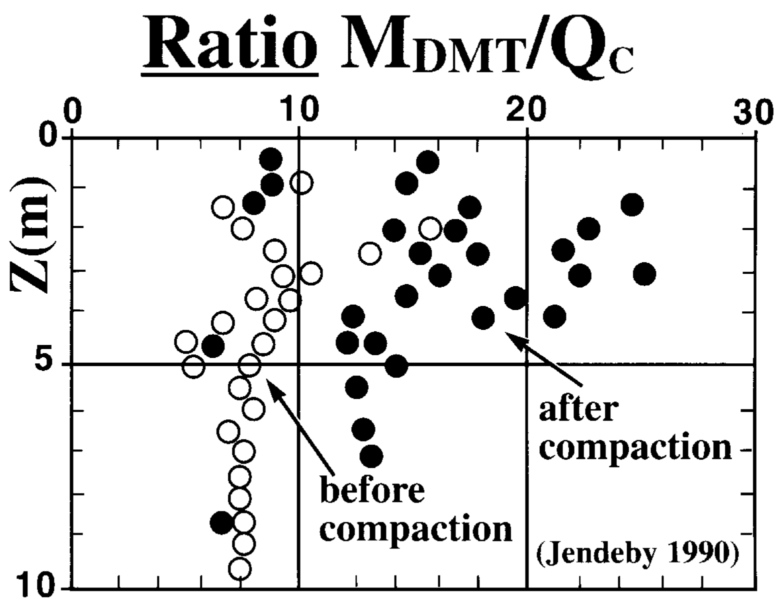
\ <u>PROBES circular + angular transition point</u> (LSC, PIP) difficult to feel s_h Þ problem, s_h important for settlements, liquefaction etc.

FLAT SHAPE MORE REACTIVE to σ_h

1. **Eliminates (reduces) arching (**L/B=6)
2. **No angular transition point**
3. **Stress <u>measured</u> are <u>horizontal</u> stresses**

Higher reactivity of DMT to **$\Delta\sigma_h$-COMPACTION** demonstrated by **Jendeby 92**. He measured both **Qc** and **M_{DMT}** before and after compaction of a loose sandfill





DMT MORE REACTIVE to s_h

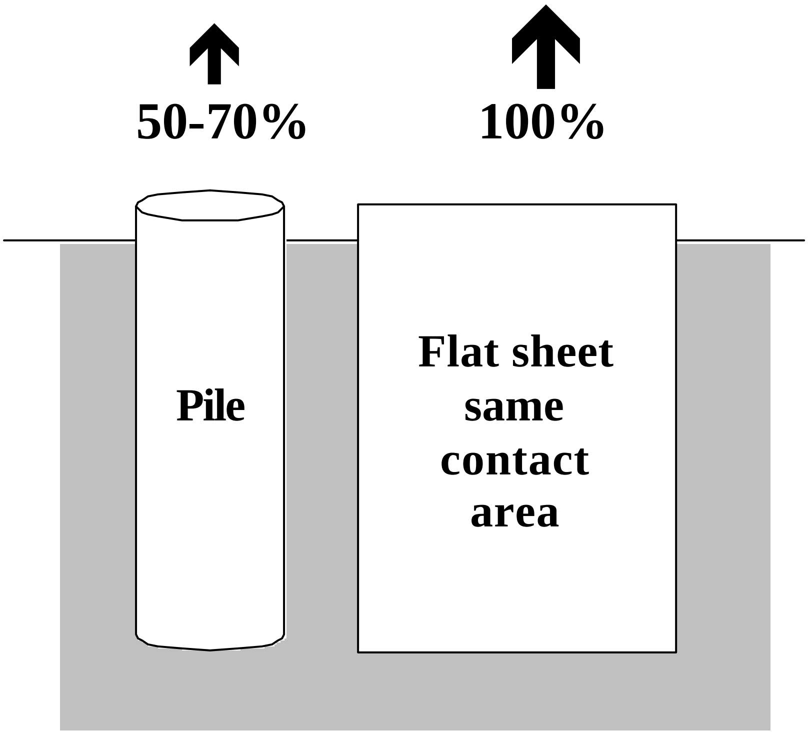
- **Compaction increases s_h, OCR, Dr.**
- **In principle, higher reactivity of DMT to compaction could be due <u>not only</u> to s_h.**
- **However we know Qc is quite reactive to Dr, DMT moderately reactive to OCR, hence <u>higher reactivity of DMT primarily due to s_h</u>.**
- **Higher reactivity of DMT to s_h also demonstrated by CC results by Jamiolkowski's group.**

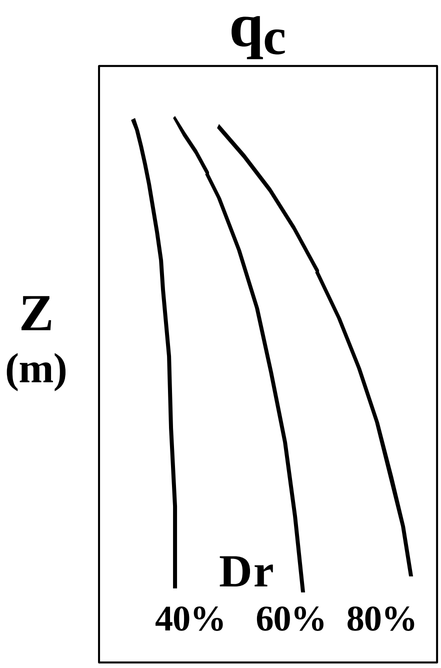
<u>NOTE</u>: higher reactivity of DMT to s_h is NOT as saying we are able to derive quantitavely s_h from DMT (the various effects must be separated).

On the other hand since many behaviours (settlements, liquefaction…) depend on s_h, better use tools <u>sensitive to s_h</u> than insensitive to s_h.

ARCHING is HIGH for CIRCULAR SHAPE, but » 0 for FLAT SHAPE

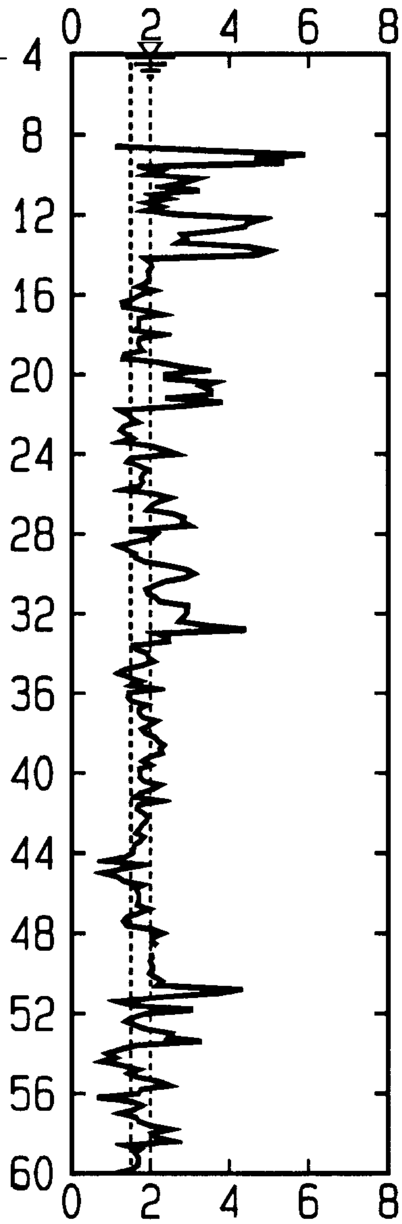
Pile : extraction force substantially less (arching)





for a given Dr
p_o less than linear with Z

VENICE : » NC SANDS & SILTS



$$K_D = \frac{p_o - u_o}{\sigma'_{vo}}$$

DMT sensitive to COMPACTION and $\Delta\sigma_h$

Before-after CPT/DMTs used to monitor compaction

often $$\frac{\Delta Mdmt / Mdmt}{\Delta Qc / Qc} \approx 2$$

<u>Schmertmann</u> (1986) DYNAMIC COMPACTION of sand site. M_{DMT} % increase ≈ *<u>twice</u>* % increase in q_c.

<u>Jendeby</u> (1992) monitored DEEP COMPACTION in a sand fill by *VIBROWING*. M_{DMT} increase ≈ *<u>twice</u>* increase in q_c.

<u>Pasqualini & Rosi</u> (1993) VIBROFLOTATION job : "*<u>DMT clearly</u> detected improvement <u>even in layers</u> where <u>benefits</u> were <u>undetected</u> by CPT*".

<u>Ghent group</u> (1993) before-after CPTs DMTs to evaluate effects (±$\Delta\sigma_h$, Dr) by <u>PILE</u> (Atlas) <u>INSTALLATION</u> "*<u>DMTs before-after</u> installation demonstrate <u>more clearly</u> [than CPT] <u>beneficial effects</u> of Atlas installation*".

Above concurrently suggest DMT:

- **uniquely sensitive to slight ±$\Delta\sigma_h$, Dr.**
- **can feel changes undetected by other methods**
  e.g. relaxation behind **diaphragm walls.**

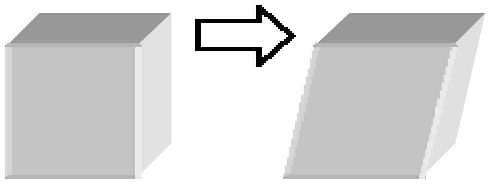
EVALUATING <u>OCR in SAND</u>

Since Mdmt increases faster than Qc when compacting (=imparting OCR to a loose ≈ NC fill), the ratio Mdmt/Qc increases with OCR. Such ratio may in fact be used as a rough indicator of OCR : Mdmt/Qc = 5-10 means NC sand, 12-24 means OC sand

A SCRUPOLOUS INVESTIGATOR SHOULD MEASURE 36 PARAMETERS

1. **To predict deformations, convert**

   s_{ij} (6) **e_{ij} (6)**

   $$\{e_{ij}\} = \begin{bmatrix} 6x6 \\ = \\ 36 \end{bmatrix} \{s_{ij}\}$$

   Matrix 36 non constant (!) parameters (constitutive laws)

2. **Investigation : <u>36</u> parameters!!! at each Z !**
3. **Should remember when trying predict behaviour from 1 parameter such as penetration resistance !**
4. **With DMT 2 parameters, still << 36, but + 100%**
5. **Note : first parameters most important.**
   The 36th → negligible gain.

Difference DMT vs PMT (pressuremeter)

DMT is *not* a PMT. PMT in a predrilled hole, DMT pushed in fresh soil.

- **Act 1 PMT : borehole = destroy S_h *drastic !***
  (DMT amplifies s_h , hence at least Þ correlations)
- **Borehole ⇒ Disturbance ⇒ Dependent on operator & soil ⇒ repeatibility : *strong limitation!***
- **Slower, + costly, less soil layers tested**
- **Multiplicity of methods for correcting for disturbance (subjectivity)**

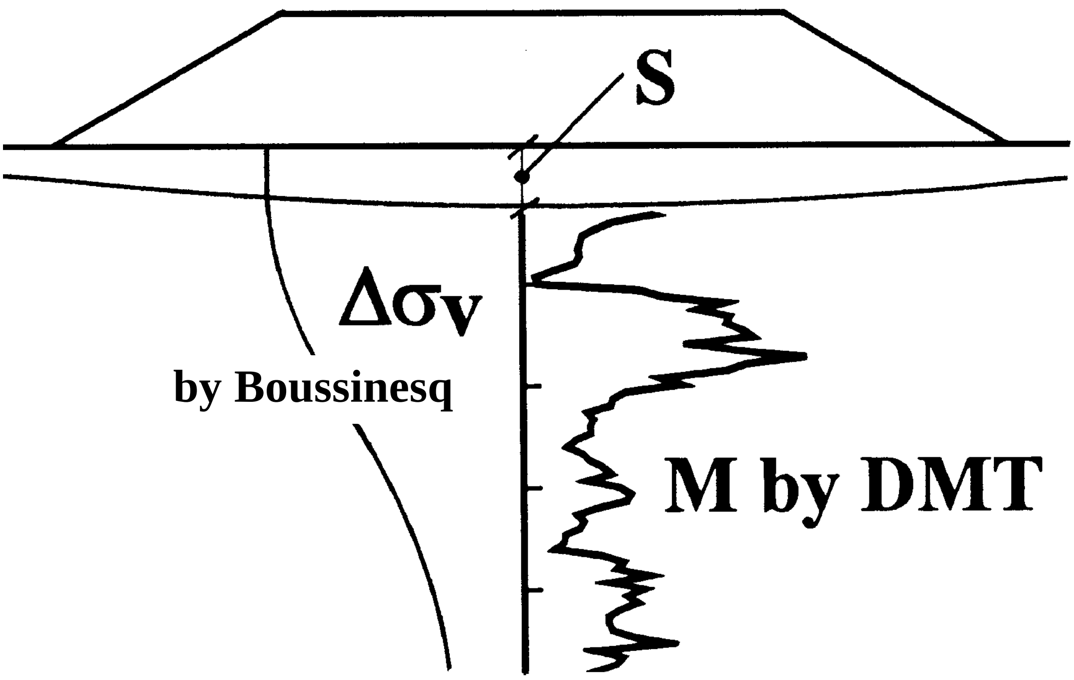
Dumas and Ortigao report very similar settlement predictions by DMT-PMT *provided "special" correction-curve-fitting applied to PMT...*

As to SBPM

- **» Prohibitive (PhD required?)**
- **After 2 decades research, Jamiolkowski concludes (CPT'95 Linkoping and 1998) : SBPM cannot be used - as previously hoped - as a standard for s_h i.e. $s_{h,SBPM}$ dubious reliability.**

Some colleagues not sure it is advantageous to run PMT in place of DMT (except rocks).

DMT APPLICATION N° 1
SETTLEMENT CALCULATION



$$S = \sum \frac{\Delta s_v}{M} \cdot \Delta Z$$

1-D vs 3-D SETTLEMENT FORMULAE

In general, in conventional elasticity :

- 1-D formula in 1-D problems (*large* rafts)
- 3-D formula in 3-D problems (*small* isolated footings)

$$S_{1-D} = \Sigma \frac{\Delta s_v}{M} \cdot \Delta Z$$

$$S_{3-D} = \Sigma \frac{1}{E} \cdot \left[\Delta s_v - n \cdot \left(\Delta s_x + \Delta s_y \right) \right]$$

However **our recommendation : use 1-D in *all* cases:**

- $S_{3\text{-}D}$ involves ν and σ_h (uncertainties)
- $S_{1\text{-}D}$ v. similar $S_{3\text{-}D}$ (10%) (≈ compensation)

- Since results similar, preferable 1-D: simpler, conventional, engineer independent.
- If engineer wants to use 3-D, E' ≈ 0.8 M (elasticity).

$S_{1\text{-}D}$ above = end product of a seamless **<u>non subjective</u>** chain, from in situ to the office :

- Field raw data are independent from **operator**
- R_M for converting E_D to M not chosen by **person making interpretation**, but R_M =M / E_D =f (I_D, K_D)
- $S_{1\text{-}D}$ independent from **person calculating settlements** (once M_{DMT} in computer, push button and by Boussinesq get S)

Such settlement cannot be manipulated !

RECOMMENDED METHOD OF SETTLEMENT CALCULATION : "$\mathbf{M_{DMT}}$ + **1-D** method"

$$S = \Sigma \frac{\Delta s_V}{M} \cdot \Delta Z$$

Why 1-D preferred to 3-D : see ASCE Jnl GE Jan 1991, Marchetti discussion to Leonards. A side benefit of the 1-D method : **CONTINUOUS NON-SUBJECTIVE CHAIN FROM IN SITU TESTING TO $S_{1\text{-}D}$:**

- Ed independent from operator
- M independent from person making interpretation, being Rm=M/Ed=f(Kd)
- $S_{1\text{-}D}$ independent from person calculating settlements

SETTLEMENT CALCULATION (ONE DIMENSIONAL METHOD)
UNDER THE CENTER OF THE LOADED AREA

PESCARA PLANT
WATER TREATMENT TANK

DATA

SHAPE OF LOADED AREA	CIRCULAR
DIAMETER D	D = 25.3 m
UNIFORM APPLIED LOADING	Q = .5 Kg/sqcm
Z INITIAL SETTLEM. CALCULATION	Zfoundat = 1.4 m
PROFILE SOIL MODULUS M	DILATOMETER TEST D3

LEGEND

Z	= DEPTH	(m)
M	= 1-DIMENS. MODULUS	(Kg/sqcm)
Sigma	= EFFECTIVE OVERBURDEN PRESSURE	(Kg/sqcm)
Dsigma	= VERTIC.STRESS INCREM.(Boussinesq)	(Kg/sqcm)
Epsilon	= STRAIN OF INDIVID. LAYER	(%)
Deltas	= SETTLEM. OF INDIVID. LAYER	(cm)
S	= ACCUMULATED SETTLEMENT (SUM)	(cm)

Z	M	Sigma	Dsigma	Epsilon	Deltas	S
1.40	172	.24	.500	.290	.058	0.0
1.80	157	.30	.500	.318	.064	.1
2.20	250	.37	.500	.200	.040	.2
2.60	204	.45	.499	.244	.049	.3
3.00	248	.52	.499	.201	.040	.4
3.40	227	.55	.498	.219	.044	.5
3.80	161	.58	.496	.308	.062	.6
4.20	174	.61	.494	.284	.057	.7
4.60	95	.64	.492	.518	.104	.9
5.00	188	.67	.489	.261	.052	1.0
5.40	58	.70	.485	.844	.169	1.1
5.80	151	.73	.481	.319	.064	1.4
6.20	191	.76	.476	.250	.050	1.5
6.60	153	.79	.471	.309	.062	1.6
7.00	201	.83	.465	.232	.046	1.7
7.40	272	.86	.459	.169	.034	1.8
7.80	170	.89	.452	.266	.053	1.9
8.20	98	.92	.445	.452	.090	2.1
8.60	160	.95	.438	.274	.055	2.3
9.00	76	.98	.430	.568	.114	2.4
9.40	87	1.01	.422	.482	.096	2.6
9.80	46	1.04	.413	.907	.181	2.8
10.20	35	1.07	.405	1.167	.233	3.2
10.60	37	1.09	.396	1.079	.216	3.7
11.00	30	1.12	.387	1.303	.261	4.2
11.40	40	1.15	.379	.958	.192	4.7
11.80	63	1.18	.370	.587	.117	5.1
12.20	69	1.20	.361	.520	.104	5.4
12.60	67	1.23	.352	.528	.106	5.6
13.00	22	1.26	.343	1.587	.317	5.9
13.40	66	1.29	.335	.506	.101	6.3
13.80	72	1.32	.326	.455	.091	6.5
14.20	67	1.34	.318	.478	.096	6.7
14.60	101	1.37	.310	.307	.061	6.9
15.00	181	1.41	.302	.167	.033	7.0
15.40	122	1.44	.294	.241	.048	7.1
15.80	88	1.47	.286	.327	.065	7.2
16.20	29	1.50	.278	.974	.195	7.3
16.60	55	1.53	.271	.490	.098	7.6
17.00	49	1.56	.264	.540	.108	7.8 ← SETTLEMENT

Z	M	Sigma	Dsigma	Epsilon	Deltas	S
1.60	166	.27	.500	.301	.060	.1
2.00	218	.34	.500	.229	.046	.2
2.40	241	.41	.500	.207	.041	.3
2.80	287	.48	.499	.174	.035	.4
3.20	239	.53	.498	.209	.042	.4
3.60	151	.57	.497	.328	.066	.5
4.00	175	.60	.495	.283	.057	.6
4.40	89	.63	.493	.557	.111	.8
4.80	150	.66	.491	.326	.065	1.0
5.20	190	.69	.487	.256	.051	1.1
5.60	102	.72	.483	.472	.094	1.3
6.00	186	.75	.479	.257	.051	1.5
6.40	146	.78	.474	.325	.065	1.6
6.80	203	.81	.468	.231	.046	1.7
7.20	217	.84	.462	.213	.043	1.8
7.60	160	.87	.456	.284	.057	1.9
8.00	74	.91	.449	.604	.121	2.0
8.40	134	.93	.441	.330	.066	2.2
8.80	87	.97	.434	.501	.100	2.3
9.20	105	.99	.426	.407	.081	2.5
9.60	72	1.02	.417	.581	.116	2.7
10.00	38	1.05	.409	1.075	.215	3.0
10.40	29	1.08	.400	1.376	.275	3.4
10.80	26	1.11	.392	1.529	.306	3.9
11.20	31	1.13	.383	1.218	.244	4.5
11.60	48	1.16	.374	.777	.155	4.9
12.00	47	1.19	.365	.784	.157	5.2
12.40	77	1.22	.357	.463	.093	5.5
12.80	37	1.25	.348	.937	.187	5.7
13.20	43	1.27	.339	.784	.157	6.2
13.60	82	1.30	.331	.403	.081	6.4
14.00	60	1.33	.322	.537	.107	6.6
14.40	80	1.36	.314	.394	.079	6.8
14.80	95	1.39	.306	.322	.064	6.9
15.20	116	1.42	.298	.258	.052	7.0
15.60	58	1.45	.290	.500	.100	7.1
16.00	153	1.48	.282	.185	.037	7.3
16.40	57	1.51	.275	.480	.096	7.5
16.80	61	1.54	.267	.436	.087	7.7

DMT-calculated vs observed SETTLEMENTS

SCHMERTMANN, 1986 - 16 CASE-HISTORY

Proc. In Situ '86 ASCE Spec. Conf. VIP, Blacksburg, p.303.

No	Location	Structure	Compressible soil	Settlement (mm)			Ratio DMT/ meas.
				DMT	**	meas	
1	Tampa	Bridge pier	HOC Clay	*25	b,d	15	1.67
2	Jacksonville	Power Plant	Compacted sand	*15	b,o	14	1.07 (ave.3)
3	Lynn Haven	Factory	Peaty sd.	188	a	185	1.02
4	British Columbia	Test embankment	Peat org. sd.	2030	a	2850	0.71
5a	Fredricton	Surcharge	Sand	*11	a	15	0.73
b	"	3' plate	Sand	*22	a	28	0.79
c	"	building	Quick cl. Silt	*78	a	35	2.23
6a	Ontario	Road embankment	Peat	*300	a,o	275	1.09
b	"	building	Peat	*262	a,o	270	0.97
7	Miami	4' plate	Peat	93	b	71	1.31
8a	Peterborough	Apt. bldg	Sd. & si.	*58	a, o	48	1.21
b	"	Factory		*20	a, o	17	1.18
9	"	Water tank	Si. clay	*30	b,o	31	0.97
10a	Linkoping	2x3 m plate	Si. sand	*9	a,o	6.7	1.34
b	"	1.1x1.3m plate	Si. sand	*4	a,o	3	1.33
11	Sunne	House	Silt & sand	*10	b,o	8	1.25

-30%
+50%

CALCULATED/OBSERVED AVE : 1.18

Similar agreement by many others, e.g.:

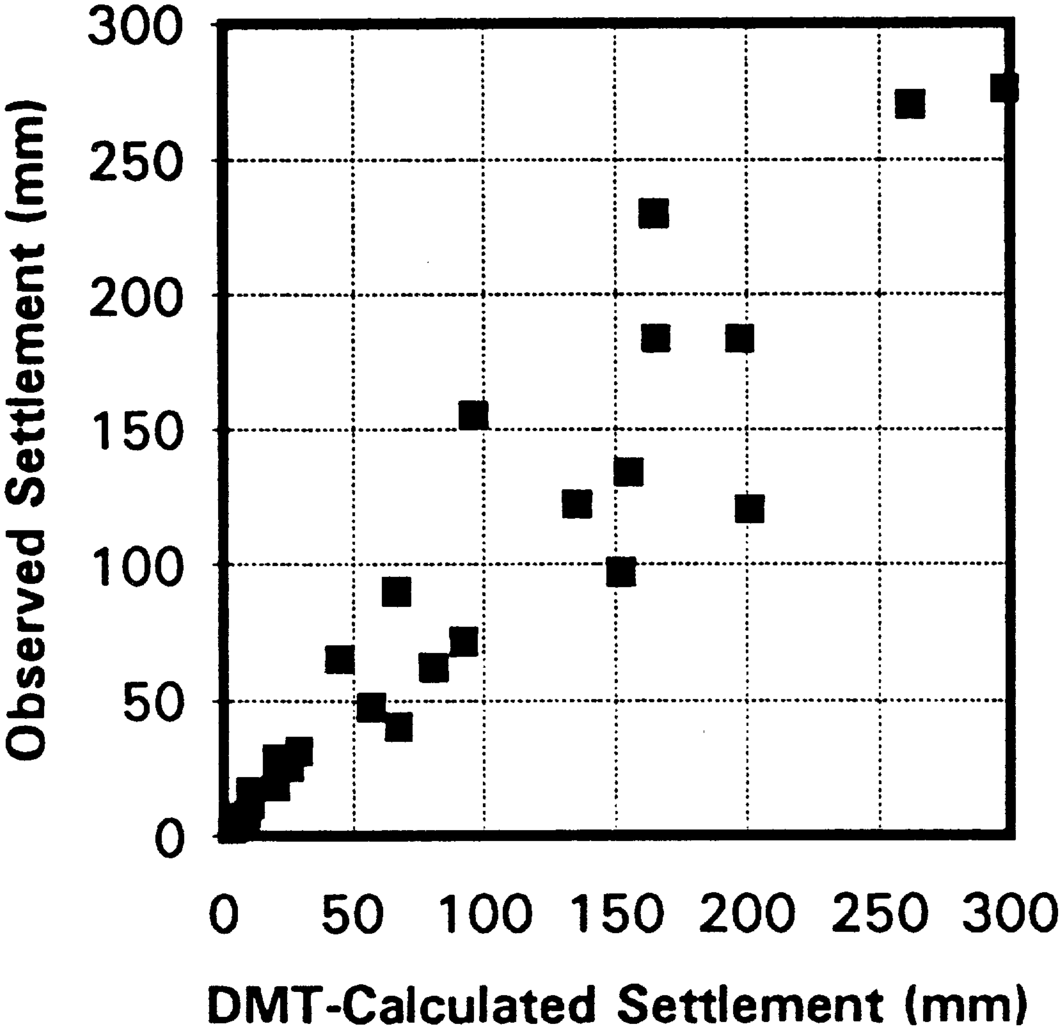
Lacasse, S. & **Lunne**, T. 1986. Dilatometer Tests in Sand. Proc. In Situ '86 ASCE Spec. Conf. Virginia Tech, Blacksburg.

Sallfors G. (1988) "Validity of compression modulus determined by dilatometer tests", Proc. of two-day seminar at NGI on calibration of in situ tests.

Steiner ('94), Woodward ('96), Mayne, Failmezger ('98), Didaskalou ('99), Tice (2000) ...

OBSERVED vs DMT CALCULATED SETTLEMENT

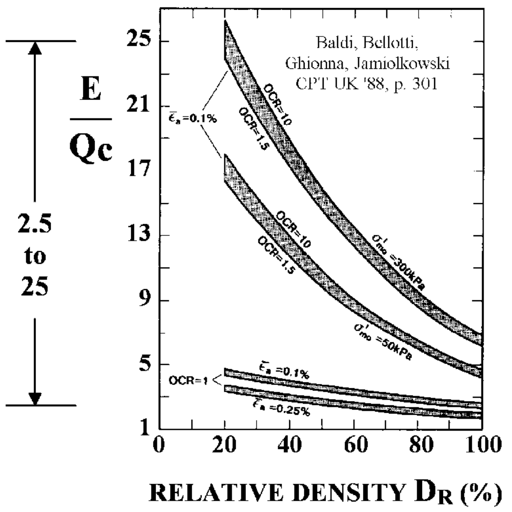
HAYES, 1990 (°)

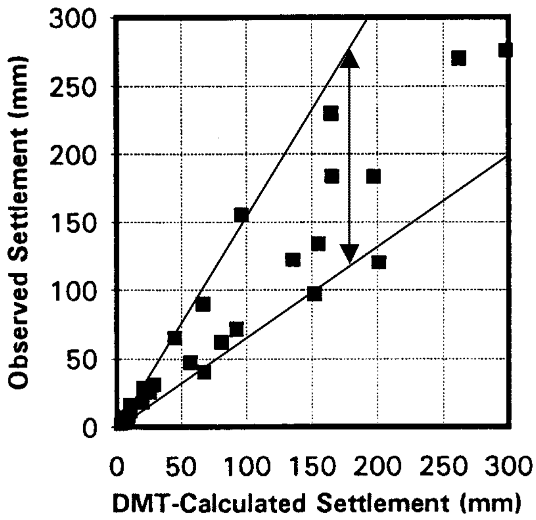


HAYES J.A. (1990) "The Marchetti Dilatometer and Compressibility" Paper to Southern Ontario Section of Canad. Geot. Soc. Seminar on "In Situ Testing and Monitoring". Sept.

ACCURACY OF SETTLEMENT PREDICTIONS

CPT **DMT**



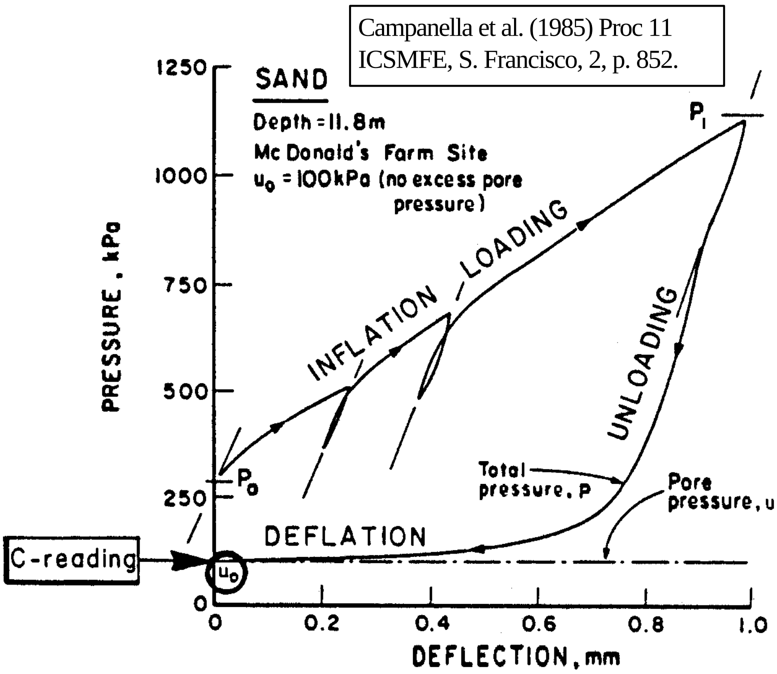


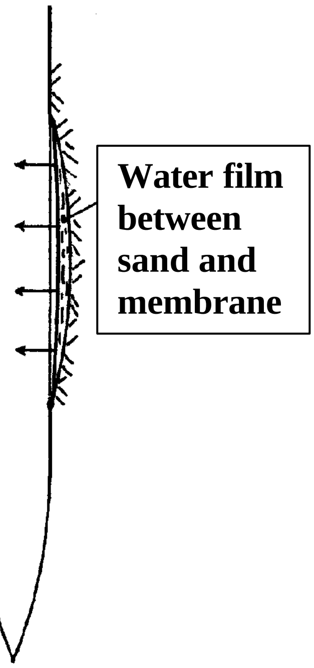
A comparison of band amplitude in Jamiolkowski and Hayes data indicates a settlement prediction bandwidth (max/min) for DMT 3 times narrower than CPT

Higher accuracy of DMT believed due to :

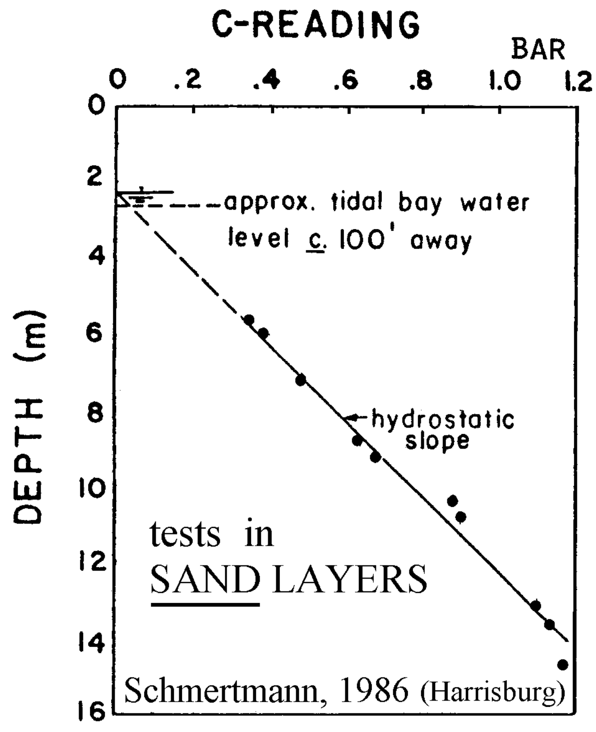
1. Sensitivity to s_h, Stress History
2. Wedge shaped tips deform soil << than cones
3. The modulus obtained by expanding a membrane (a mini load test) is more closely correlated to insitu soil modulus than is a penetration resistance

C-READING (pressure on membrane at "membrane closure") in <u>SAND</u> = Uo





» no contact Sand/Membrane $\Rightarrow \sigma = \sigma' + U_o$



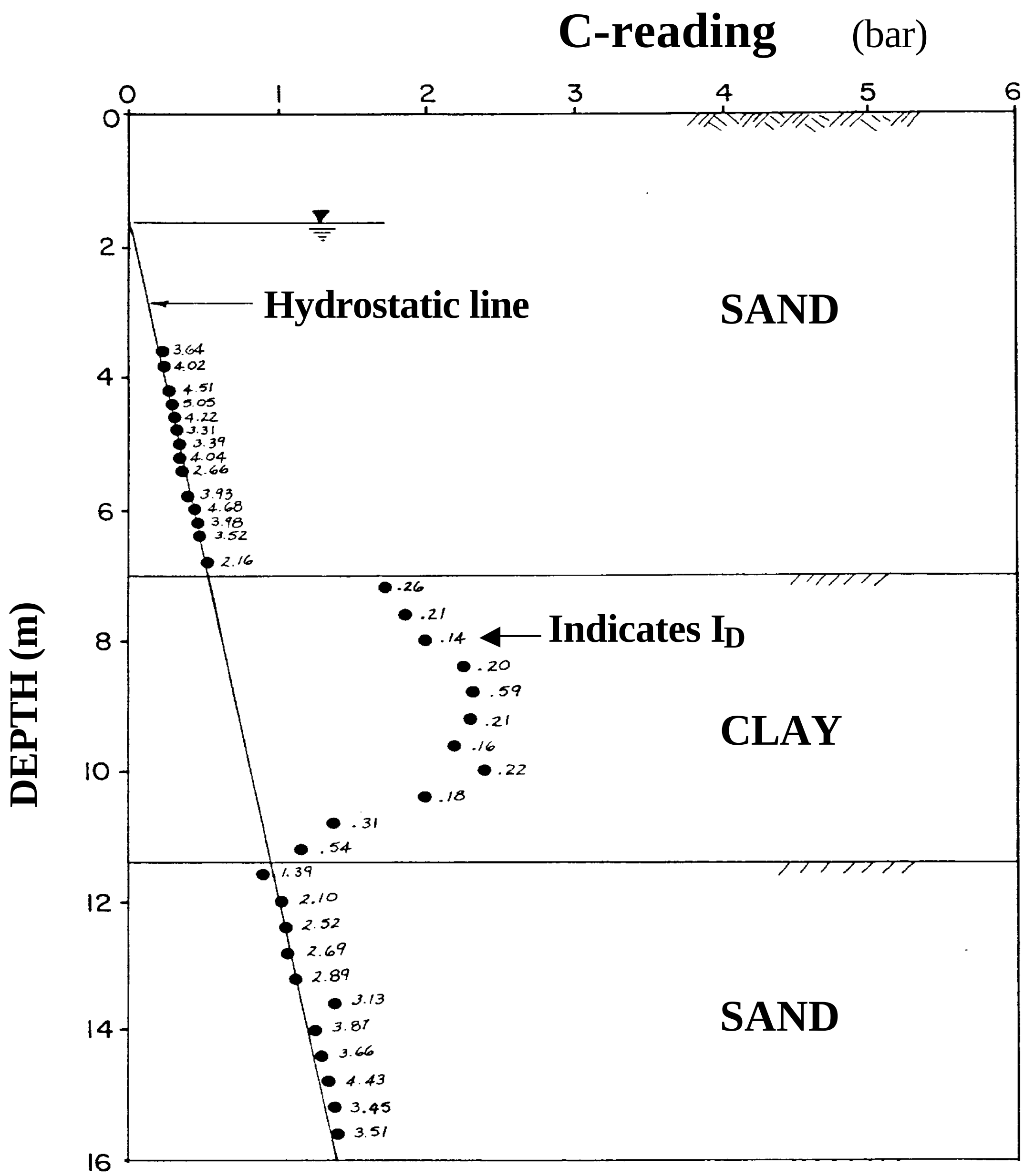
Same Uo as from a piezometer, without problems of :

- Filter clog
- Smearing
- Saturation

Schmertmann, J.H.S.(1986). Some 1985-86 Development in Dilatometer Testing and Analysis. Proc. PennDOT and ASCE Conf. on Geotechnical Engineering Practice, Harrisburg, PA.

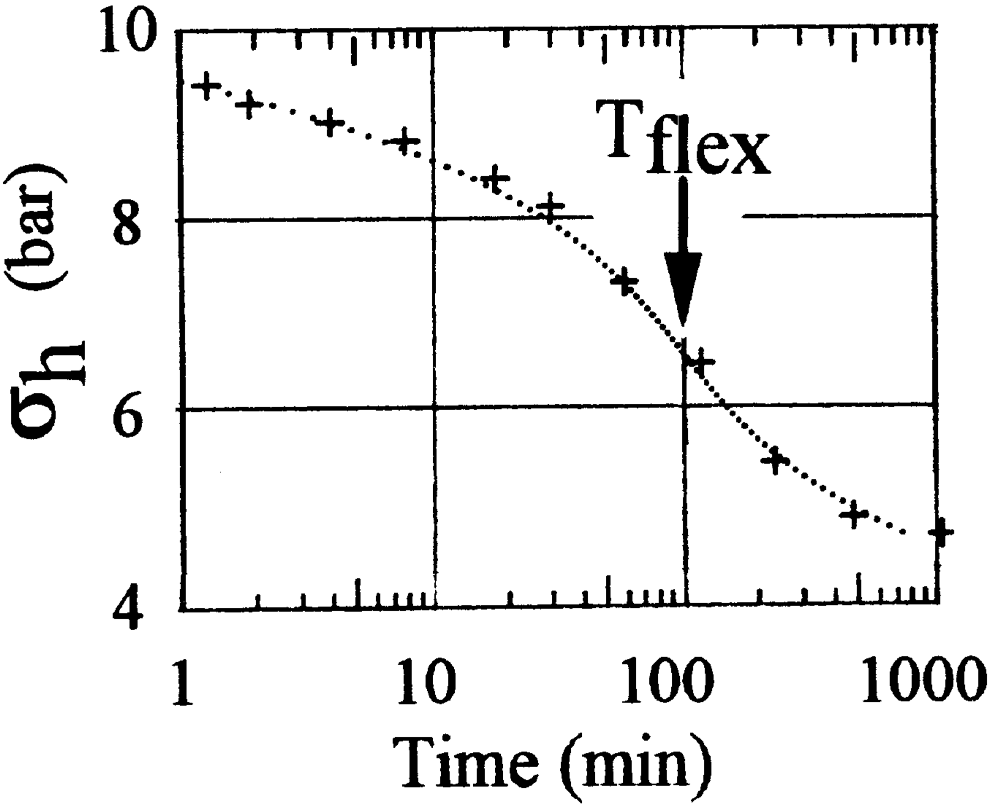
C-reading (P2)

- **in SANDS measures Uo (» piezometer)**
- **in CLAYS evidences Du (i.e. non freely draining)**



Schmertmann, 1988 **(DMT Digest No. 10, May 1988, Fig.3)**

Coefficient of CONSOLIDATION / PERMEABILITY from Tflex



$$C_h \cong \frac{7cm^2}{T_{flex}} \quad k = \frac{C \cdot g_w}{M}$$

Marchetti, S. & Totani, G. 1989.
C_h Evaluations from DMTA Dissipation Curves.
Proc. XII ICSMFE, Rio de Janeiro, Vol. 1: 281-286.

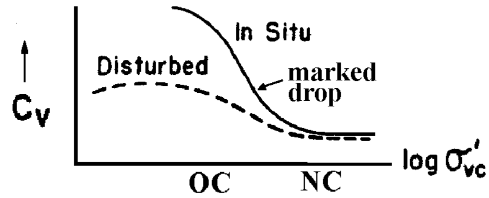
(Interestingly Mesri et al (ASCE Jnl GGE, Aug **1999**, 716) **advocate use of "inflection point method" even for Cv from oedometer, in place of the usual Casagrande or Taylor methods).**

RELIABILITY OF Ch BY DMT

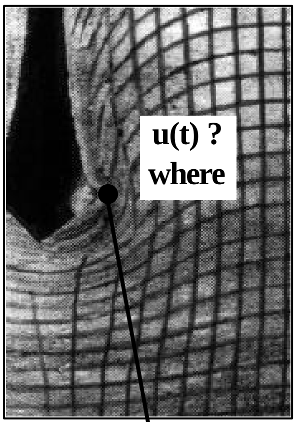
Difficult to evaluate : scarcity reliable reference Ch

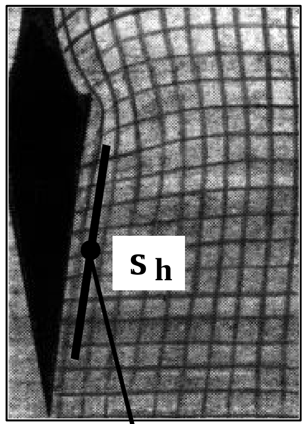
Comparisons complicated by:

- **$C_{horizontal} \neq C_{vertical}$**
  (oedometer gives C_{V}!)
- **C_{OC} ¹ C_{NC}** (Load Range)

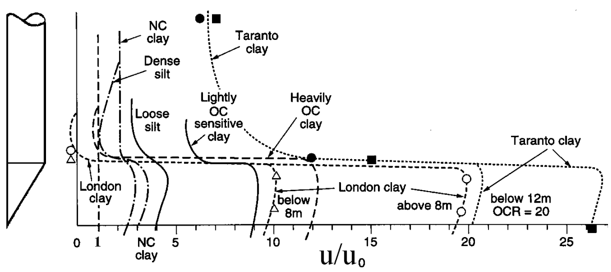


But *Common sense* : **If we had to backcalculate Ch under an embankment, would we prefer to infer Ch…**





…from **u**(t) in a singular ***POINT*** in highly distorted clay...?



PORE PRESSURE AT INTERFACE

Robertson et al. (1986). "Use of Piezometer Cone Data", Proc. "In Situ '86" ASCE Spec. Conf., Virginia Tech, Blacksburg, p. 1265

...or from **s h** - decay under this ***MINIEMBANKMENT*** ?

- Integral (more stable)
- Larger volume

Instrumentally, the *bloodless* (dry) DMT dissipations **easier**:

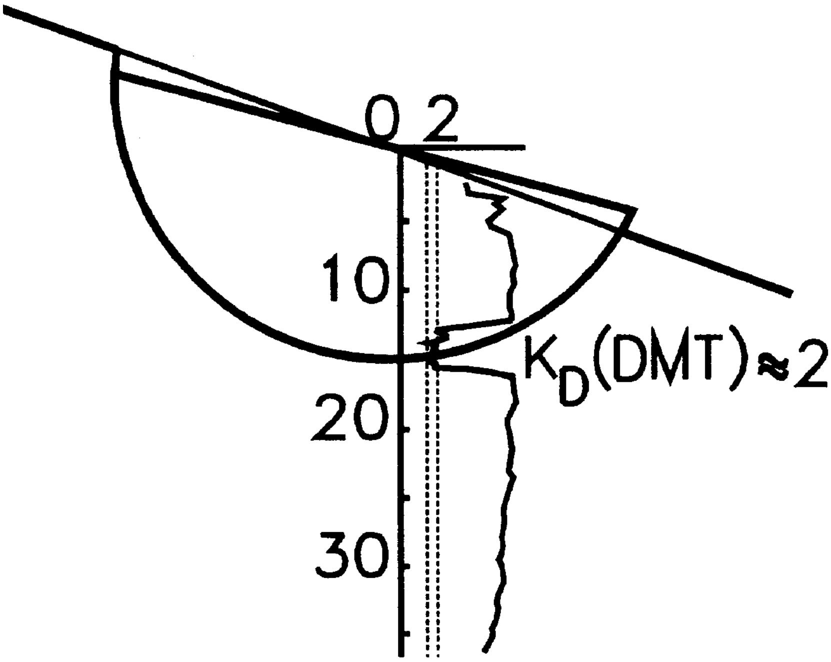
-No Filter clogging/ smearing
-No Loss of saturation
» No Dependent on push force

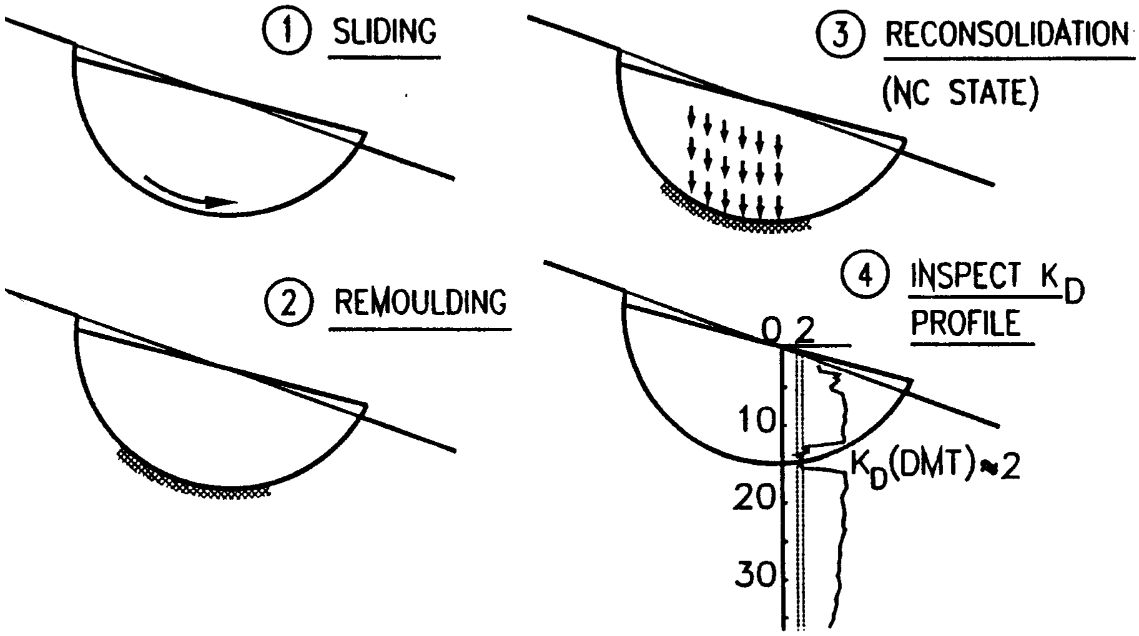
CASE HISTORIES : $C_{h,dmt}$ underpredicts Ch from real scale works by a factor 1 to 3. (Still on the "slow" side, but much faster than the usually too slow oedometer predictions)

Totani et al. 1998, In situ determination of Ch by DMT", Proc. ISC '98, Atlanta, 2, 607.

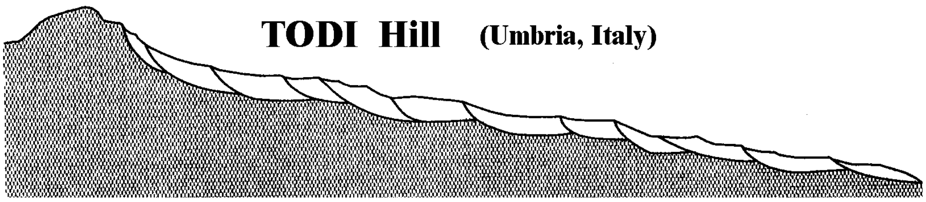
Evaluating STABILITY of a SLOPE

Verifyies if an OC Clay SLOPE contains ACTIVE (or old quiescent) SLIP surfaces

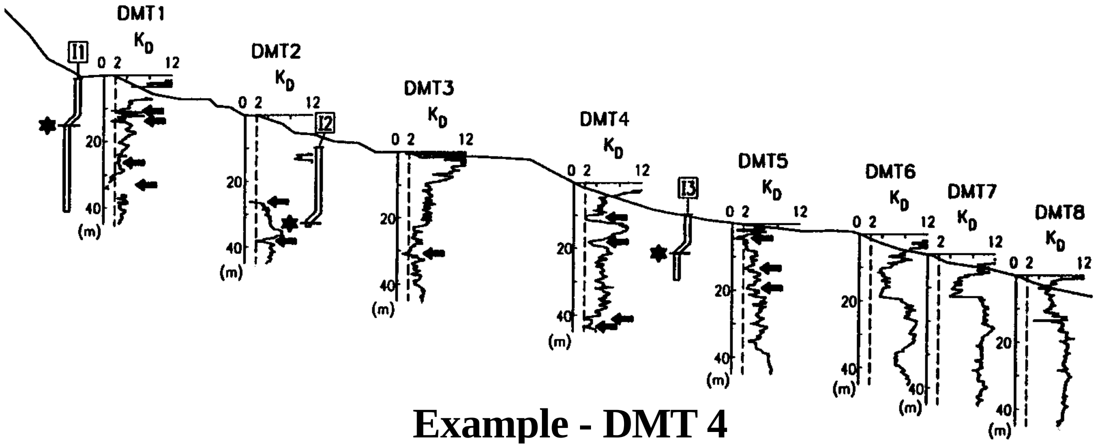




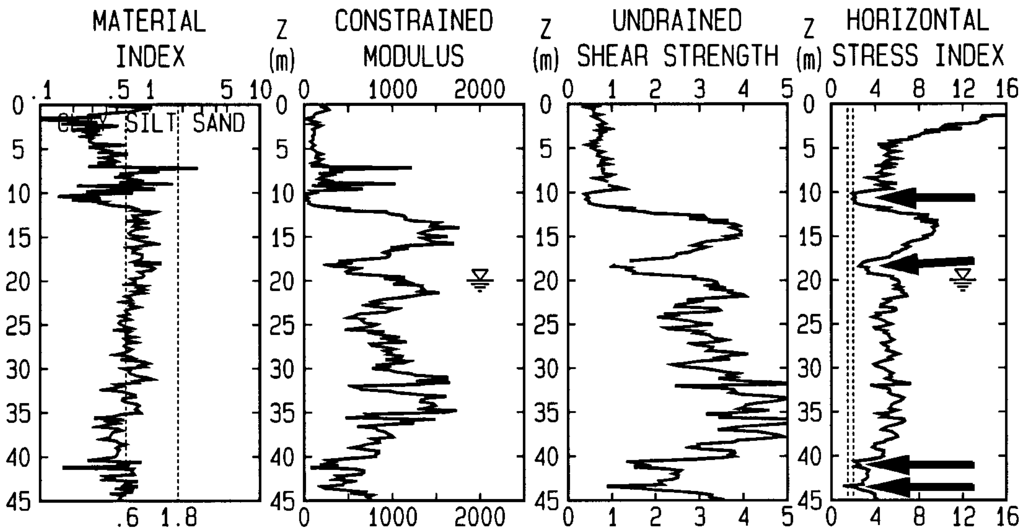
KD=2 LAYERS CORRESPOND TO SLIP SURFACES BY INCLINOMETERS



Qualitative reconstruction (Tonnetti, 1978)



Example - DMT 4

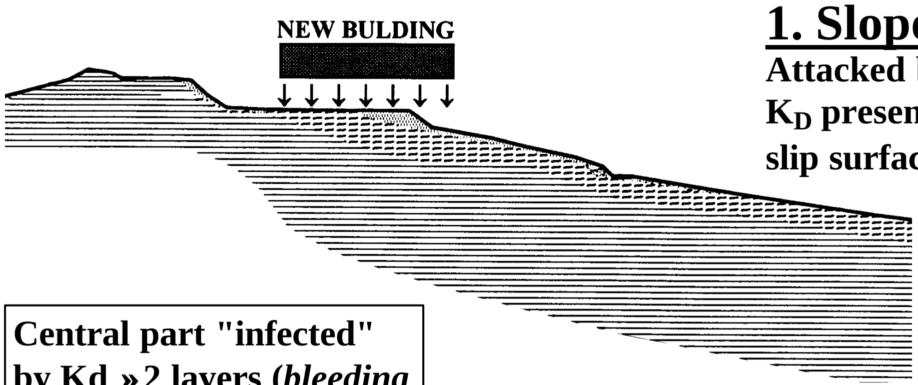


- **Specific value Kd » 2**
- **Detects quiescents planes (inclinometers can't - must move)**

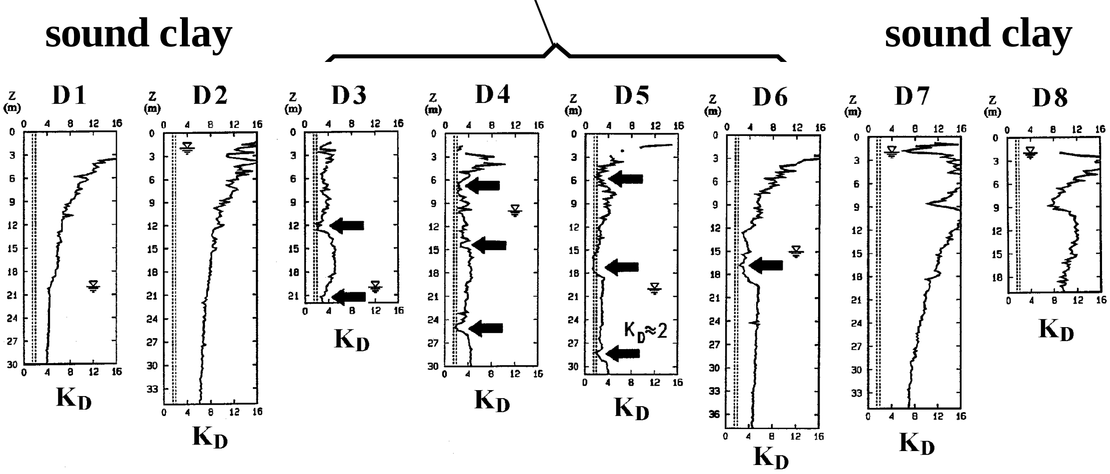
Case history, Chieti Hills, 1996. SAFETY of proposed VERY <u>HEAVY</u> BUILDING on an OC Clay SLOPE

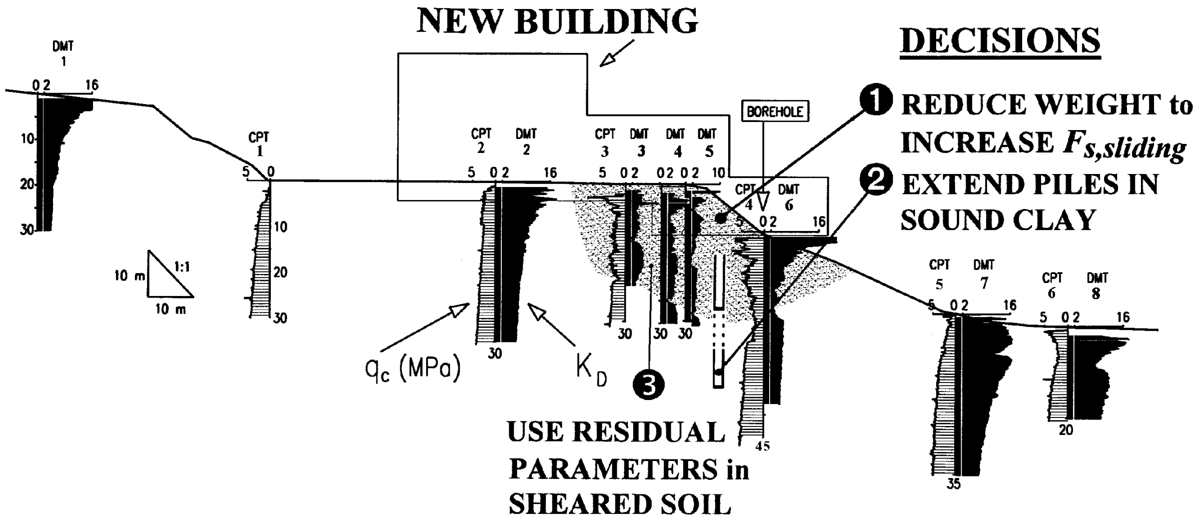
<u>1. Slope stable ???</u>

Attacked by verifying via K_D presence of previous slip surfaces



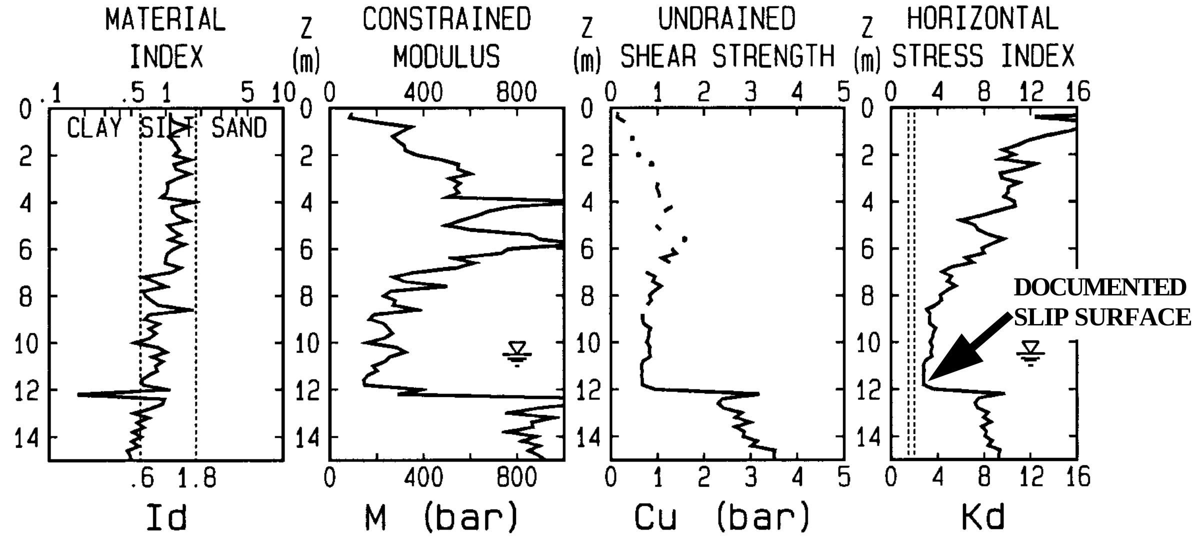
Central part "infected" by Kd » 2 layers (*bleeding wounds*) = past/ present shear planes



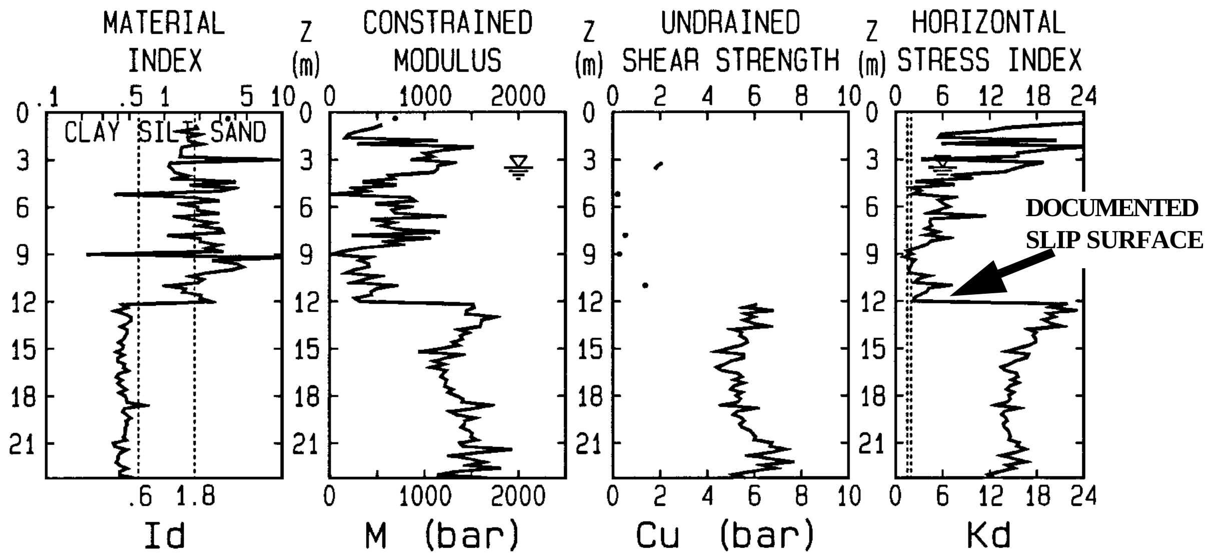


Examples of Kd » 2 in SLIP SURFACES

LANDSLIDE "FILIPPONE" (CHIETI)



LANDSLIDE "CAVE VECCHIE" (S.Barbara -AR)



DMT for LATERALLY LOADED PILES

Recommended methods p-y curves from DMT:
Robertson et al. (1987)
Marchetti et al. (1991)

VALIDATIONS on FULL SCALE PILES by
Marchetti et al. (1991)
NGI (1998)
Georgia Tech (1998)

- **The two methods similar results**
- **V. good agreement predicted-observed behavior**

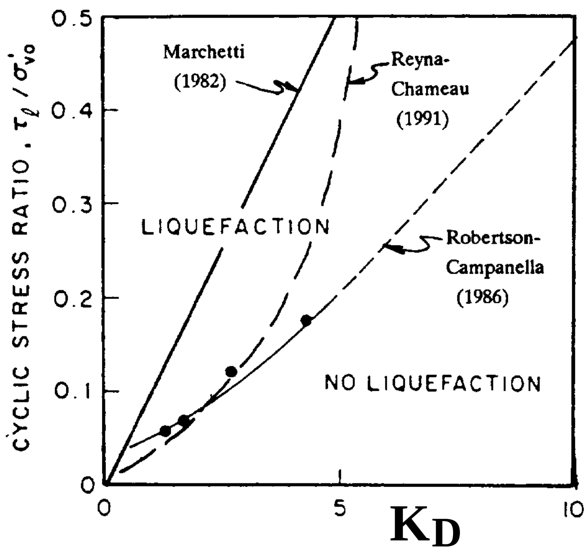
Note : DMT initially conceived for LL PILES
Similarity DMT - LL piles :
a. Structural element installed in soil
b. Lateral deformation

LL PILE GROUPS :
Ruesta & Townsend Jnl ASCE GGE, Dec. 1997

DMT FOR LIQUEFACTION

Summarizes available knowledge →

1. **Enter K_D in Reyna & Chameau curve (°) to estimate cyclic stress ratio to cause liquefaction**
2. **Use Seed-like method.**



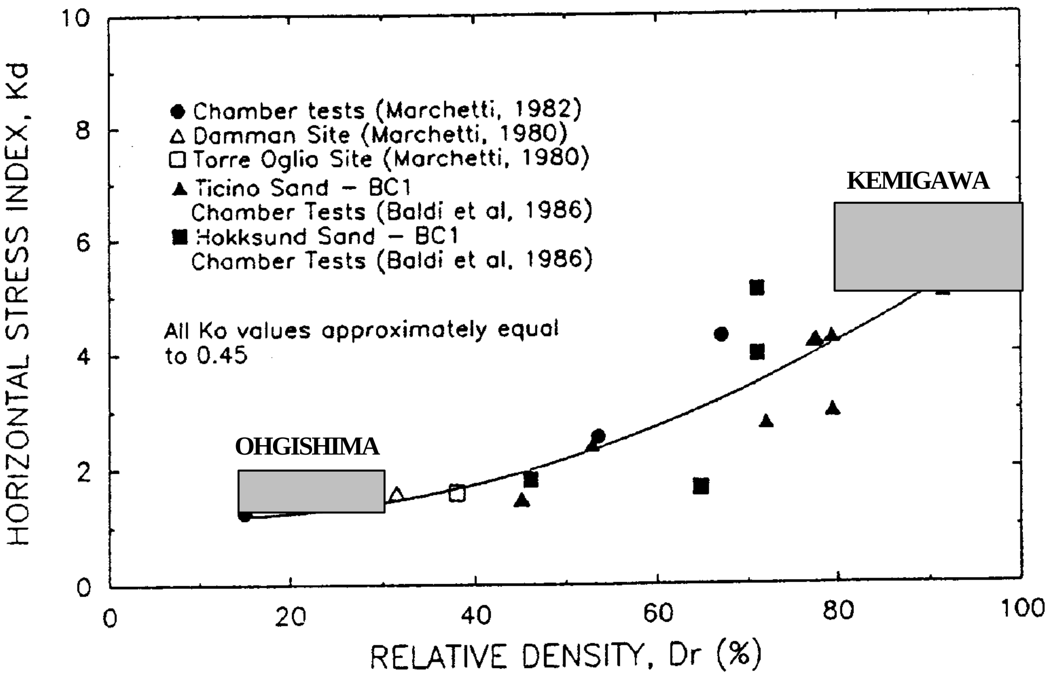
- **Above diagram + available information suggests that a clean sand (natural or sandfill) is adequately safe against liquefaction (M=7.5 earthquakes) if Kd :**

Seismicity of the area	a_{max}/g	*Kd,min*
Nonseismic	/	***1.7***
Low seismicity	***0.15***	***4.2***
Average seismicity	***0.25***	***5.0***
High seismicity	***0.35***	***5.5***

(Compacted sandfills normally easily exceed these Kd).

(°) Reyna, F. & Chameau, J.L. (1991) "Dilatometer Based Liquefaction Potential of Sites in the Imperial Valley" 2nd Int. Conf. on Recent Advances in Geot. Earthquake Engrg. and Soil Dyn.. St. Louis. May.

Correlation K_d-D_r for NC sands (Reyna-Chameau 1991)



- **Note new datapoints (rectangular areas) by Tanaka & Tanaka (1998) found on <u>top quality freezed samples</u>** **(Sept. 98, Jap. Soils & Found., p.61 Fig. 13)**
- **Since the Reyna-Chameau liquefaction curve largely rests on their K_d-D_r curve above, the new Japanese data reinforce indirectly the Reyna-Chameau liquefaction curve.**

COMMENTS on using DMT for liquefaction

- **DMT demonstrated to be » twice more sensitive than other tools to densification. Arguably also more sensitive to "liquefiability" (a state of "negative compaction").**
- **s_h, *OCR*, aging and cementation (all factors increasing liquefaction resistance) are felt by K_D, but scarcely felt by other tests.**
- **In any case a less disruptive insertion in a liquefiable collapsible sand should increase meaningfulness of what is measured.**

PROBLEMS IN EVALUATING LIQUEFACTION by CPT

- **CPT Research (Sladen 1989, Yu, Schnaid, Collins 1997…) has shown, contrary to attempts by Been et al. (1987), that correlation Qc-SP** (state parameter) **is no unique.**
- **Use in design CRR (cyclic resist. ratio) from CPT ignoring non unicity of the SP-Qc correlation can lead/ has lead (Sladen), to catastrophic consequences (e.g. large scale subsea liquef. flowslides at Nerlerk, Canadian Beaufort sea).**
- **The strong dependence of the Qc-SP correlation on stress level (=non unicity), given the strong link SP-CRR, cast doubts on unicity of Qc-CRR correlation - unicity at the base of routine method of evaluating liquefiability by CPT.**
- **Sladen's conclusion : "while CPT is ideal for providing a qualitative profile, future research for assessing liquefiability should probably be directed towards other tools".**

Sladen Geotechnique 1989, No.2
Yu et al. 1997, ASCE J. GGE, No. 9 Sept.

- **Similar remark by Jamiolkowski's res. group** (1985, 11 ICSMFE, S.F. V.4, p.1896)**, while questioning the use of Nspt and/or Qc for assessing liquefiability of sand deposits with complex stress-strain history. They state : "*reliable predictions of CRR in such deposits require the development of some new in situ device [other than CPT or SPT] much more sensitive to the effects of past stress and strain histories*".**
- **Also Robertson (1998** - Can. G. Jnl. June**) warns that CRR evaluated by CPT (preferred to SPT, due to the poor repeatability) may be adequate for low risk, small scale projects. However for medium to high risk projects he recommends evaluating CRR by more than one method.**

Dependency of Qc-SP correlation on stress level (Sladen), affects reliability of liquef. estimates by CPT.

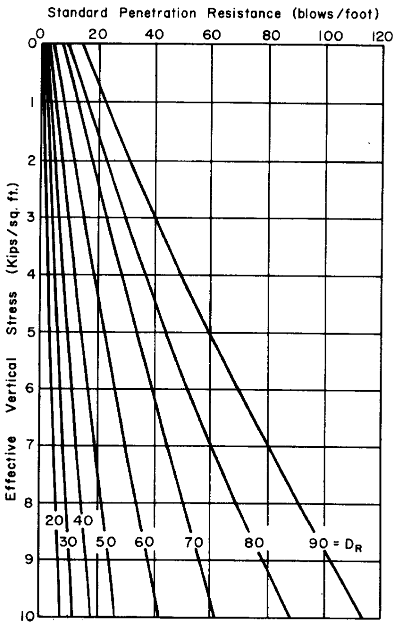
- **Dependency of Qc-SP correlation on stress level could be a problem specific of CPT, where arching, Qc(z) curvature… more pronounced.**
- **SPT & DMT » unaffected by arching because :**
  SPT: blows/vibrations & shallow penetration
  DMT : flat shape of probe.
- **For linearity of SPT lines, see: Gibbs & Holtz, Christian & Swiger (ASCE GE 1975 No. 11), Skempton (Geotechnique Sept 1986).**
- **For linearity of $p_{o,DMT}$ (Kd » constant) see e.g the Venice Kd profile.**

As to CRR from lab "undisturbed" samples:

- **Reconstituted samples may bear little relation to the degree of compactness of the in situ sand (Christian & Swiger ASCE GE 1975 No. 11).**
- **Even good samples do not reflect in situ structure, fabric, aging - affecting significantly CRR**
- **Even high-quality fixed piston samples produce CRR values different from those on top quality freezed samples (Robertson, Can. G. Jnl. June '98).**

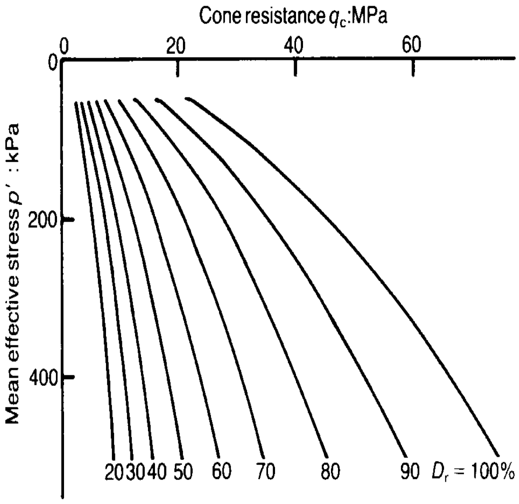
CURVATURE - ARCHING AFFECTS MAINLY CPT ?

SPT



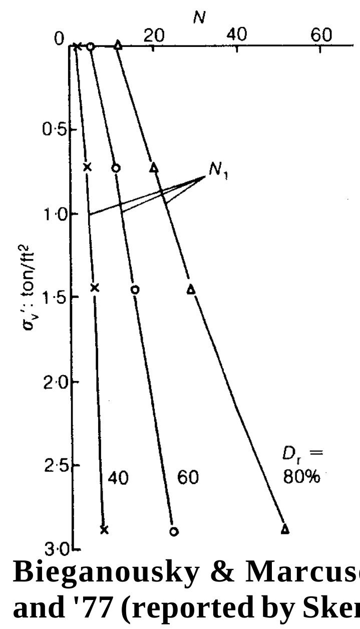
Gibbs & Holtz (reported by Christian & Swiger, Asce GED, Nov '75)

CPT



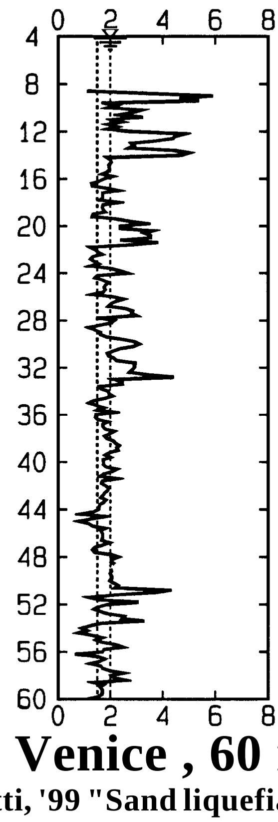
Jamiolkowski's res. group - CC Ticino sand (reported by Sladen, G éotechnique, June '89)

SPT



Bieganousky & Marcuson '76 and '77 (reported by Skempton Géotechnique, Sept '86)

KD-DMT



K_d constant

ß

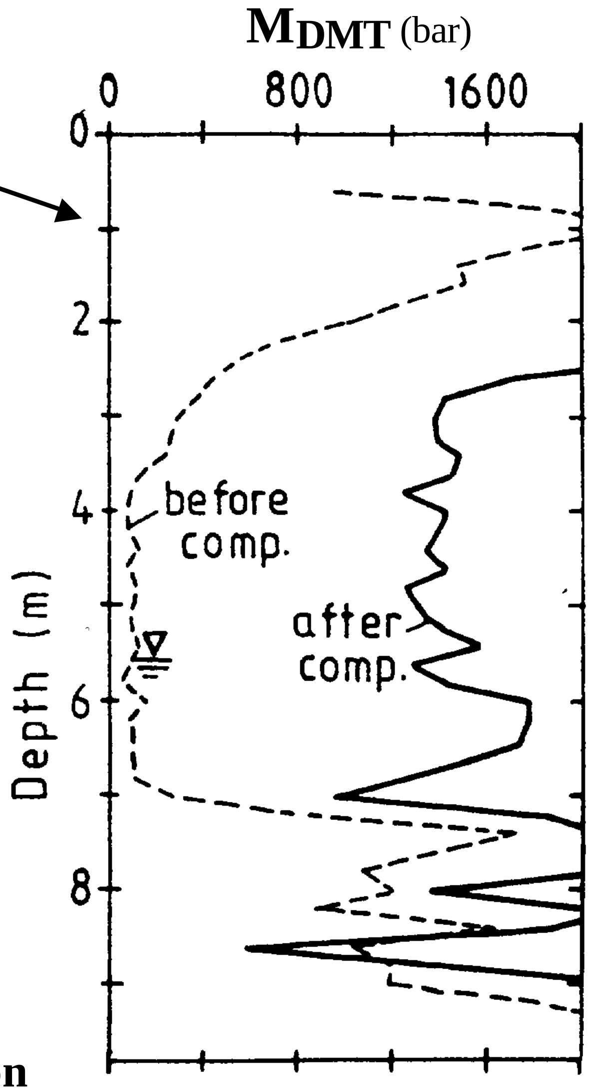
p_o linear

Venice , 60 m

(Marchetti, '99 "Sand liquefiability assessment by DMT", unpublished Techn. Note)

DMT$_{before-after}$ for COMPACTION CONTROL

Reasonant vibrocompaction technique



Since compaction increases M_{DMT} at rate » twice Qc (Schmertmann 1986, Jendeby 1992, Pasqualini & Rosi 1993, De Cock et al. 1993..)...

...DMT suitable when changes of s h, Dr difficult to detect by other methods

DMT usable to detect small changes s h, Dr e.g.:

- **Relaxation behind <u>diaphragm walls</u>** (Hamza '96)
- **Compression/Decompression due to <u>installation</u> of <u>piles</u>** (De Cock, Van Impe, Peiffer 1993.Bap 2 Ghent p.359)

Van Impe, De Cock, Massarsch, Mengé - New Delhi (1994) Vol.3 , p. 1151

COMPACTION CONTROL

Here, sensitivity of DMT to σ_h esp. advantageous.

In fact, <u>if Qc is used to control</u> compaction:

- **Since Qc scarcely sensitive to σ_h**
- **Qc reflects *<u>PART</u>* of the benefit (benefit due to $\Delta\sigma_h$ mostly missed !)**
- **Settlement predicted from Qc after compaction too big : +200-300%** (Massarsch'94). **Consequence : compact more than necessary $\Rightarrow$ waste $**

<u>If Mdmt is used to control</u> compaction:

- **Since Mdmt incorporates $\sigma_{h,compaction}$**
- **Settlement predictions incorporate benefit of increased σ_h + realistic (lower). Waste avoided.**

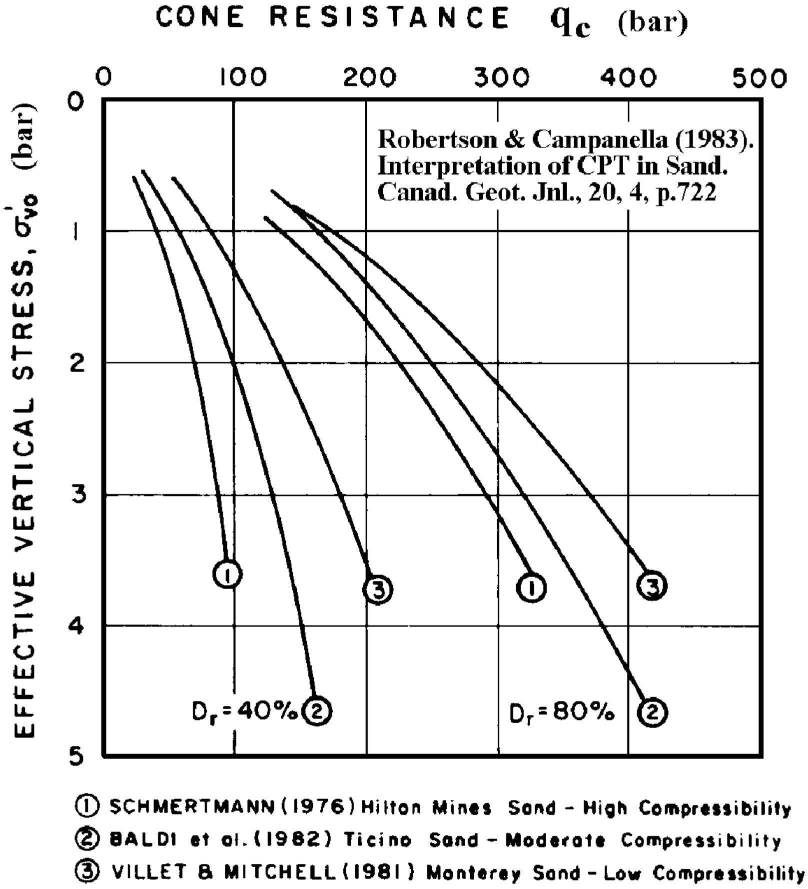
In general, since scope of compaction is to limit settlements, is <u>more rational</u> to base controls on modulus than Dr or Qc (Schmertmann, 1986)

Schmertmann, J.H., Baker, W., Gupta, R. & Kessler, K. (1986). "CPT/DMT Qc of Ground Modification at a Power Plant", Proc. "In Situ '86" ASCE Spec. Conf., Virginia Tech, Blacksburg, p. 985.

Correlations Qc-Dr for one sand may differ 20% for another sand. Specify "Dr = 80%" vague.
<u>Dr</u> elusive and NOT ultimate scope.

Dr In Situ » UNMEASURABLE

Jefferies & Been (CPT '95) : <u>No unique mapping</u> Qc–Dr applicable to all sand. *Hilton mines sand* at Dr=60% has same Qc as *Monterey sand* at Dr=40%.



Is a problem: in most jobs Dr = 40% definitely inacceptable, while 60% could still be acceptable.
(A way: Calibr. Chamber for each job ! non realistic).

***Caveat* for <u>lab</u> research (Ladd <u>1977</u> SOA Tokyo V.2, 461): "The adequacy of using <u>reconstituted</u> lab sand specimens , even <u>exactly</u> (!) at the same in situ density (?), is highly questionable" (fabric/ particle orientation affect results)**
(this even for CC + loss of nat. cementation, aging…)

DMT to control SUBGRADE Compaction

Subgrade = top 30 cm soil supporting superstructure (compaction generally checked by CBR, Proctor, E_{plate}).

Two methods developed for DMT.

METHOD 1

Relate $\mathbf{E_d}$ to **CBR** (Borden 1986), or $\mathbf{E_d}$ to $\mathbf{E_{plate}}$ then use **CBR,** $\mathbf{E_{plate}}$ as usual....**BUT** physical mis-matching :

CBR, E_{plate} apply load on surface, E_d is a profile f(z).

METHOD 2 **(a calibration)**

Is based on the $\boldsymbol{M_{DMT}}$ ***acceptance profile*** (see case history Marchetti 1994, to check subgrade of a 90 km road in Bangladesh).

- Perform a few **preliminary** DMTs in ***areas of accepted*** (vs conventional specs) subgrade, and define a $\boldsymbol{M_{DMT}}$ ***acceptance profile***
- Use such $\boldsymbol{M_{DMT}}$ ***acceptance profile*** as an **economical production** method for quality control of compaction, with only **occasional verifications**

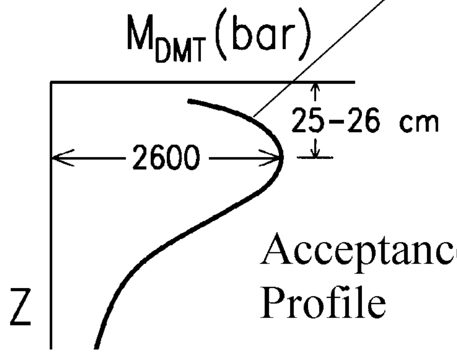
Bangladesh Subgrade Compaction Case History
Specs in terms of (Proctor/ CBR / Eplate) ⇒ in terms of Modulus

Specs were

Modif. **Proctor**=95%
CBR = 10
E$_{plate}$ =300 bar (0.5-1.5 bar)

Typical shape
$(M_{DMT})_{max}$
@ 25-26 cm

DMT
in **accepted**
areas ➔

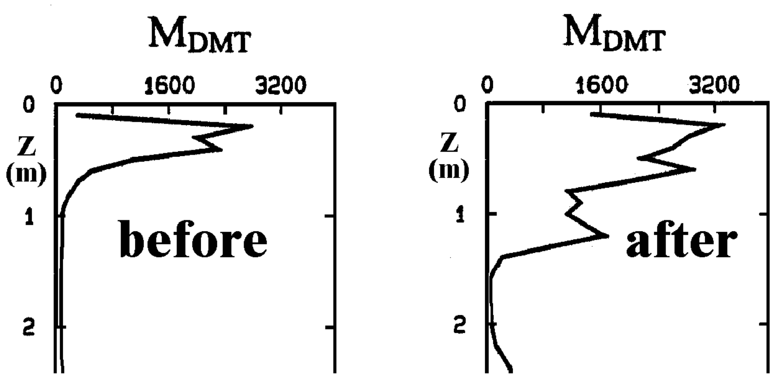


Acceptance
Profile

- V. **fast** (60 DMT profiles to 2-3 m / **10 cm**, in 4 days). **Avoided** large n. **time cons. CBR, Plate** load tests.
- It would be helpful to define **acceptance profiles** similar to Fig. but pertaining to **various classes of roads**.

REASON OF STUDY : **CLAIM**

- **Contractor** unable to compact to specs top 30 cm without **<u>also</u>** compacting underlying **<u>low Dr</u>** sand.
  (Δenergy + Δtime + Δsoil + Δ$ = **CLAIM**)
- $\mathbf{M_{DMT}}$ clearly indicated $\mathbf{DD_r}$ to ≈ 1.2 m >> 30 cm payed



SUBGRADE MODULUS FOR <u>MULTI</u> ANCHORED DIAPHRAGM WALLS

<u>Classic</u> limit pressure methods (Terzaghi, Blum…) are <u>inapplicable</u>, because anchors limit deflections, hence Ka, Kp are not reached. $K_{h,winkler}$ required. True interaction problem. (Pressure ?? not active/passive).

Deriving K_h (springs) from E or M (continuous) not univocal : *<u>models</u>* different. Yet engineers familiar with M, <u>not</u> K_h . Useful even crude relations M to K_h.

Many studies on M to K_h (see next page)**. Found : depends on inter-anchor distance, cantilever height, anchor stiffness… But in essence » all arrive to similar conversion formulae :**

- **Balay '84 K_h » 2M/B (for diaphragm walls)**
- **Viggiani '99 K_h » M/B (for shallow foundations)** (°)

M (1-D mod) can be taken as M_{dmt} (see p. 85).

B is a dimension representative of soil area loaded by the wall (often B taken as : free height for cantilever walls, distance of props for propped walls). Frequently: B » 4m. (Fortunately even a factor 2 on B changes little the results).

Example Kh values : Using the above formula(Balay), a medium clay (M=100 bar) would have Kh= 5000 KN/m^3. A stiff sand (M=1000 bar) would have Kh= 50000 KN/m^3.

(°) The above approximate formula by Viggiani uses Boussinesq formula w=f(q,B,E,m) to calculate settlement of a shallow foundation of width B loaded by q, then Kh=q/w

For less primitive approximations on converting E to Kh for diaphragm walls see next page.

BIBLIOGRAPHY ON COEFFICIENT OF SUBGRADE REACTION (WINKLER) FOR DIAPHRAGM WALLS

Balay Jean (1984). "Recommandations pour la choix des paramètres de calcul des écrans de soutènement par la méthode aux modules de réaction - in French ". Note d'information technique, Ministère Urbanisme, Laborat. Central des Ponts et Chaussées, 18-22.

Potts and Fourie (1986). "A numerical study of the effects of wall deformation on earth pressures", Int. J. for Numerical and Analytical Methods in Geomech. 10, 383-405

Fourie and Potts (1989). "Comparison of finite element and limiting equilibrium analyses for an embedded cantilever retaining wall", Géotechnique, 39, No. 2, 175-188.

Vezole P. (1995). "Retaining walls. Ground - Structure interaction about the reaction coefficients method", in French. Revue Française de Géotechnique, N. 71, 2° trimestre 1995, 31-37

Schmitt. P.(1995). "Estimating the coefficient of subgrade reaction for diaphragm wall and sheet pile wall design", in French. Revue Française de Géotechnique, N. 71, 2° trimestre 1995, 3-10

Simon B (1995). "Comments relating to the choice of the coefficient of subgrade reaction for the design of earth retaining structures", in French. Revue Française de Géotechnique, N. 71, 2° trimestre 1995, 11-19.

Becci e Nova (1987). "Un metodo di calcolo automatico per il progetto di paratie", in Italian. Riv. Ital. Geot. 21, N. 1, 33-46

Viggiani C. (1999). Fondazioni, in Italian. Editor Cuen, p. 225.

DMT for FEM INPUT PARAMETERS

a. Simplest use : lin. elastic, 1 param. E (ν). Hamza reports using E from DMT in Cairo Metro design (with E » 0.8 M_{DMT})

b. "Brute force" approach : model by FEM the complete DMT cycle. Vary input parameters until matching DMT responses. Drawback : need many additional (unknown) parameters

c. A less ambitious - viable - approach :

- **Assume tentative set FEM input parameters, (available soil info + experience of programmer)**
- **Simulate by FEM simple lab tests (e.g. oedometer) . Verify the matching M_{FEM} vs M_{DMT}.**
- **Adjust tentative set of FE parameters to improve matching**

i.e. use M_{DMT} as BENCHMARK. Could avoid gross mistakes in selecting deformation parameters. (Similar "squaring" possible on Cu - clays).

d. Other approaches : identify an "equivalent average" DMT strain. Aim : generating a point in the G-g degradation curve

CHECK FEM PARAMETERS by VERIFYING THEY PREDICT RIGHT SETTLEMENT UNDER SIMPLE FOUNDATIONS

Often FEM are used in complicated cases, when linear elasticity would give inadequate answer (e.g. settements outside diaphragm walls).

Yet, in simple geometries and simple loading , FEM model should yield prediction similar to conventional elastic with tried-and-trusted moduli.

E.g. conventional methods, with conventional moduli , should predict well settlement under a circular tank 5 m diameter with 100 Kpa load.

Hence a way to check FEM parameters.

M_{DMT} values very close to "operative moduli" recommended by textbooks

M_{DMT} are strikingly similar to Moduli recommended by classic textbooks e.g.

Type of soil	E (Kg/cm²)
Very soft clay	3.5-30
Soft clay	20-50
Medium clay	40-80
Hard clay	70-180
Sandy clay	300-400
Silty sand	70-200
Loose sand	100-250
Dense sand	500-800
Dense sand & gravel	1000-2000

- **Table 19.5 on p. 567 of: "Foundation Engineering Handbook". Winterkorn and Fang, Van Nostrand Reinhold Co.**
- **1 Kg/cm^2 » 1 bar = 100 KPa**
- **E » 0.8 M**

- **Agreement important because Moduli by Manuals carry heavy experim. weight. Tables are *distilled normalized* case histories**.
- **E from manuals have predicted *satisfactorily for decades* <u>real</u> settlements in <u>real</u> cases.**

Alternatively, FEM elastic, with judicious choice of moduli (e.g. **E » 0.8 M_{DMT} + reduction** of **E** in **unloading zones - e.g. by 4) may give (at least in simple loading cases) approx right deformation.**

A disadvantage : Modeling DMT more difficult than axisymmetric for 2 reasons

- **Problem 2-D Þ 3-D**
- **DMT involves 2 STAGES : <u>INSERTION</u> + <u>MEMBRANE EXPANSION</u> (not continuation)**

SOLUTIONS AVAIL. SOFAR <u>ONLY for 1st STAGE</u>

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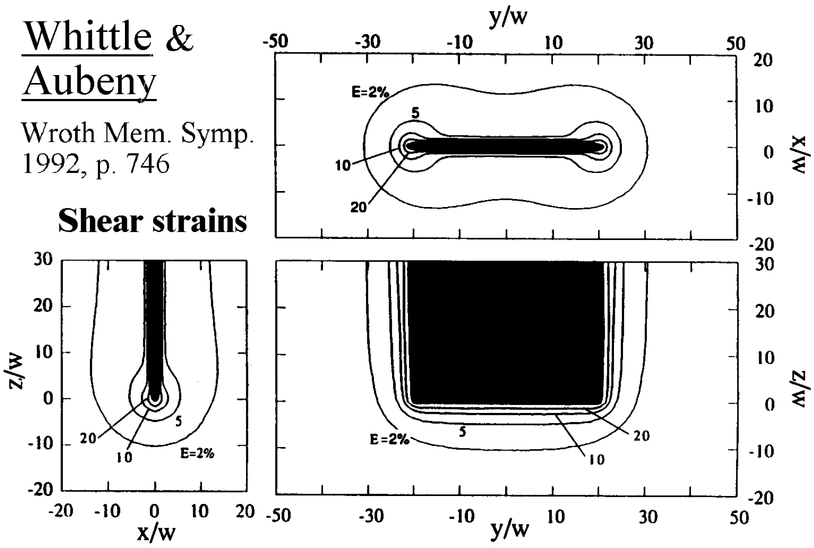
❶



- STRAINS MAX NEAR EDGES
- MEMBRANE IN ZONE OF MIN STRAINS

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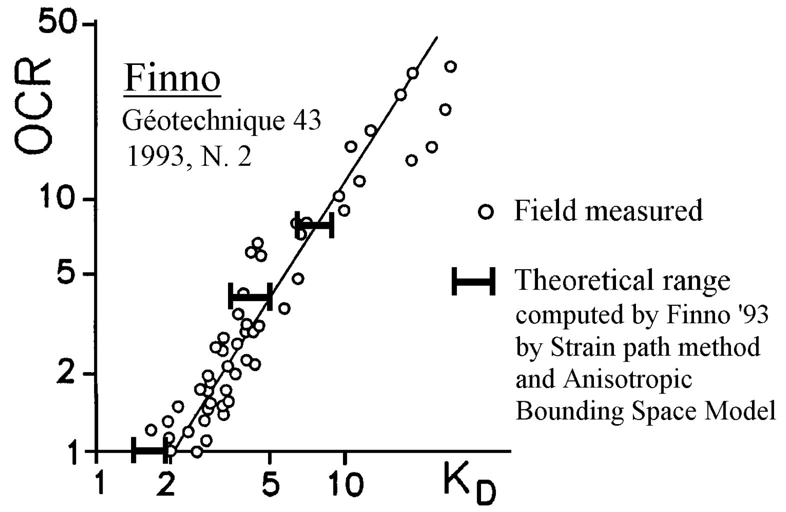
❷



- THEORY SEEMS TO CONFIRM EXPERIM. CALIBRATION
- CALIBR. MOST EXPEDITIOUS WHEN MATH NOT READY, PROVIDED <u>PHYSIC LINK IS CLEAR</u>

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❸ **<u>Yu</u>** <u>et al</u>. Another solution (Wroth Mem. Symp. 1992 p. 783)

DMT BEST APPLICATIONS

- **M and Cu profiles**
- **Estimating settlements, deformation analysis**
- **Monitoring soil improvement**
- **Recognizing soil type**
- **Distinguish freely-draining layers from non f.d.**
- **Verify if a clay slope contains active/old slip surfaces**

Useful information also on:

- **OCR (uniquely sensitive to OCR details) and K_0 in clay**
- **Coefficient of consolidation / permeability**
- **P-y curves for laterally loaded piles**
- **Sand liquefiability**
- **Friction angle in sand**
- **(K_0 in sand + OCR)**

PERCEIVED ADVANTAGES

EXECUTION

Quick, simple, economical, reproducible.
Variety of insertion equipment.
Very short training time
"Not many things can go wrong"

INTERPRETATION

- **DMT a <u>two-parameter</u> test, one related to stress history (dominating soil behavior).**
- **Flat blade » avoids <u>arching</u> typical of cylindrical probes, hence <u>sensitivity to</u> s_h**
- **DMT uniquely sensitive to <u>slight variations</u> of s_h and Dr**
- **Compaction : evidences benefits missed by other tests**
- **Settlement Calculations : Mdmt incorporates s_h / OCR effects. Available documentation indicate:**

  DMT reduces the settlement <u>prediction bandwidth by a factor » 3</u> compared with conical tips

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ACKNOWLEDGMENTS

The writer expresses his appreciation to his longtime coworkers Totani G., Monaco P. and Calabresi M. for the enthusiastic cooperation and for the many original contributions and innovations in the development of the Flat Dilatometer and its Applications to Geotechnical Design.