SoCal Regional Meeting, **Irvine March 5th** 2026, 10:00 am -3:00 pm

NorCal Regional Meeting, **Concord March 6th** 2026, 10:00 am -3:00 pm

Application to geotechnical design using (S)DMT results

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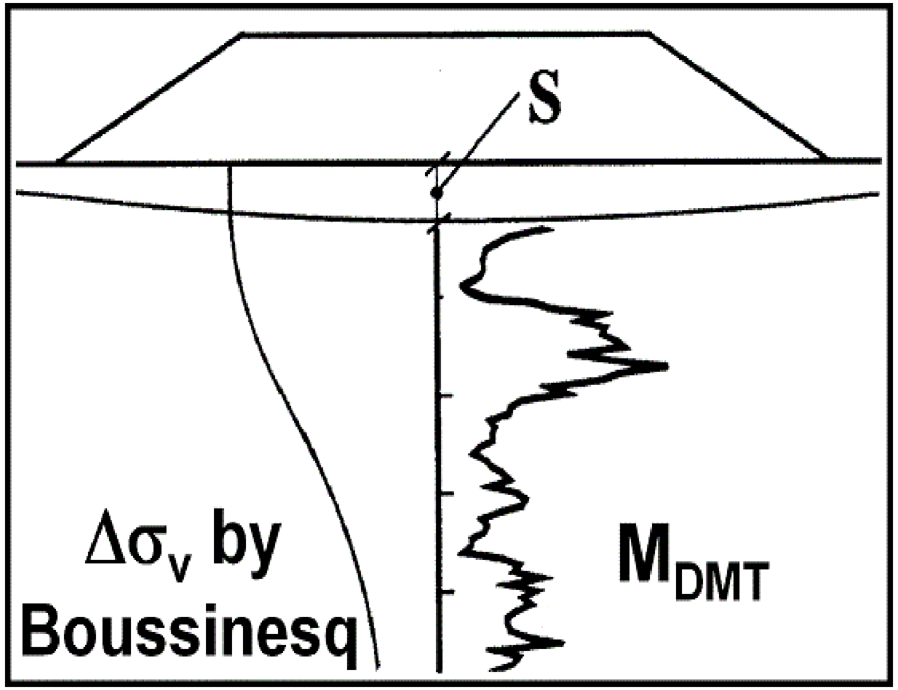
Main SDMT applications

- Settlements prediction
- In situ G- γ decay curves
- FEM input parameters (Plaxis)
- QA of soil improvement
- Slip surface detection in OC clay
- Liquefaction resistance (CRR)
- Laterally loaded piles (P-y curves)
- Diaphragm walls (springs model)
- Vs for quality assessment of soil samples

Settlements Prediction (Modulus)

Settlements Prediction



LOAD | **SOIL**

Boussinesq → $\Delta\sigma_v$

DMT → M

$$S = \sum \frac{\Delta\sigma_v}{M}\,\Delta z$$

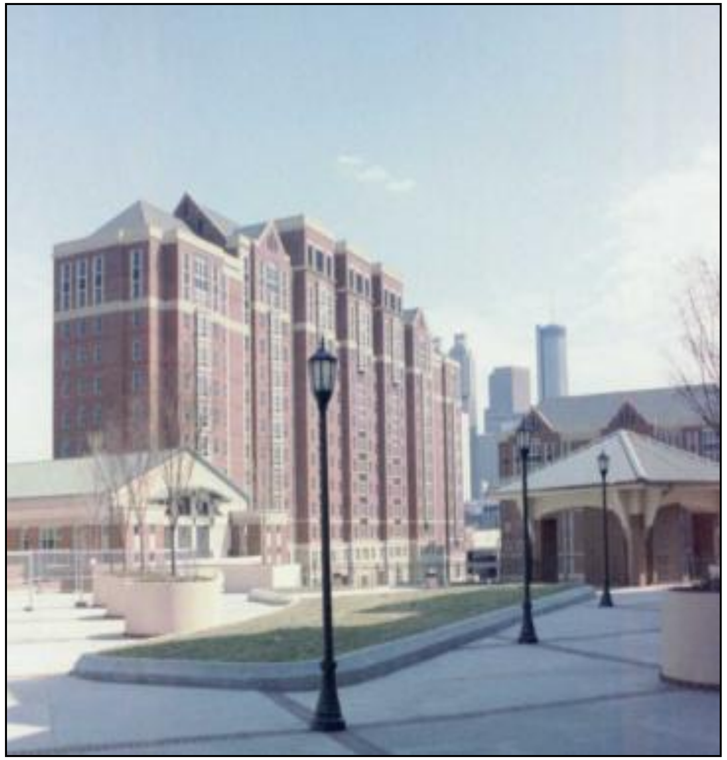
- 1-D approach (classic Terzaghi)
- Primary settlement at working loads

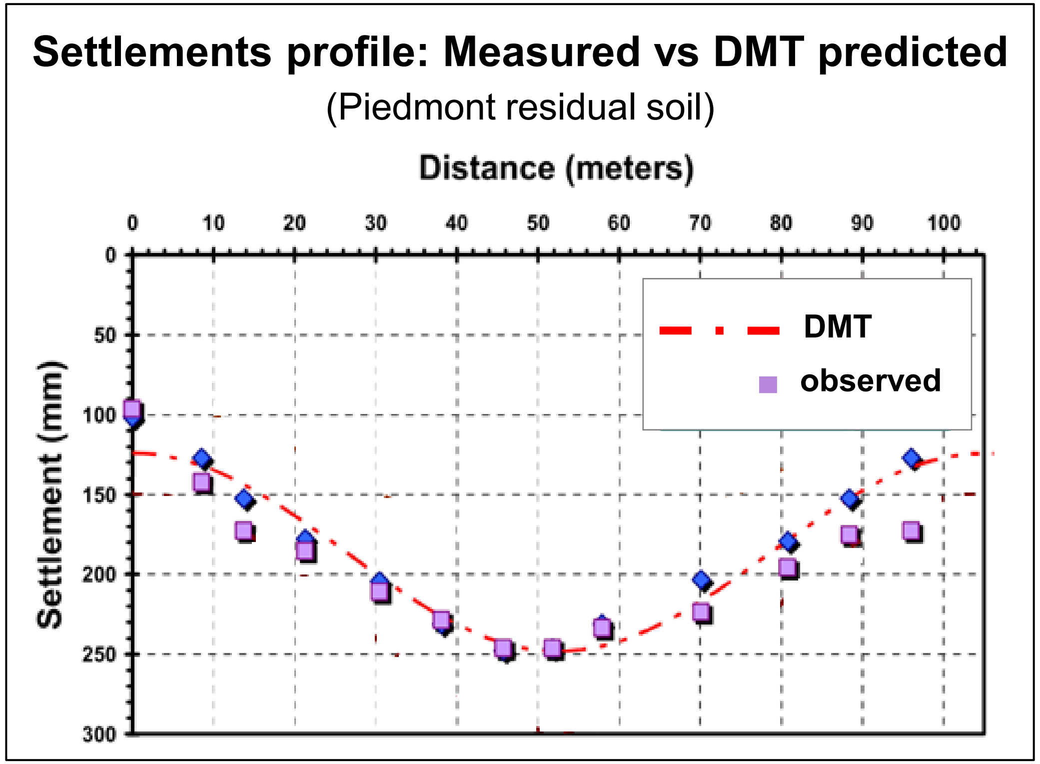
Many publications & case histories of good agreement between measured and DMT-predicted settlements / moduli:

- ***Failmezger** (2020)*
- ***Godlewski** (2018)*
- ***McNulty & Harney** (2014)*
- ***Berisavijevic** (2013)*
- ***Vargas** (2009)*
- ***Bullock** (2008)*
- ***Monaco** (2006)*
- ***Lehane & Fahey** (2004)*
- ***Mayne** (2001, 2004, 2005)*
- ***Failmezger** (1999, 2000, 2001)*
- ***Crapps & Law Engineering** (2001)*
- ***Tice & Knott** (2000)*
- ***Woodward** (1993)*
- ***Iwasaki et al.** (1991)*
- ***Hayes** (1990)*
- ***Mayne & Frost** (1988)*
- ***Schmertmann** 1986,1988)*
- ***Steiner** (1994)*
- ***Leonards** (1988)*
- ***Lacasse** and **Lunne** (1986)*
- ..
- ..

Observed vs. DMT-predicted settlements

Dormitory Building 13 storeys (Atlanta - USA)

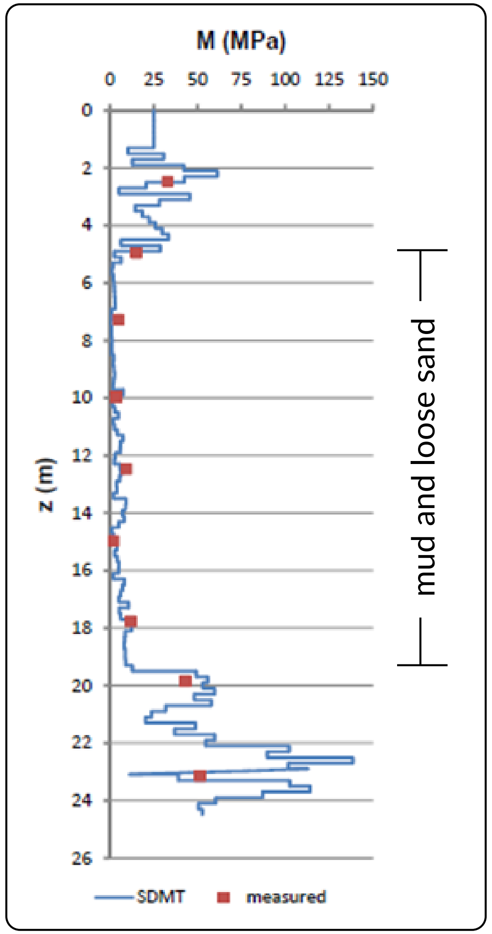




Mayne (2005)

SPT Settlement prediction:	**46 mm**
DMT Settlement prediction:	**250 mm**
Observed Settlement:	**250 mm**
SPT → large error !!!	

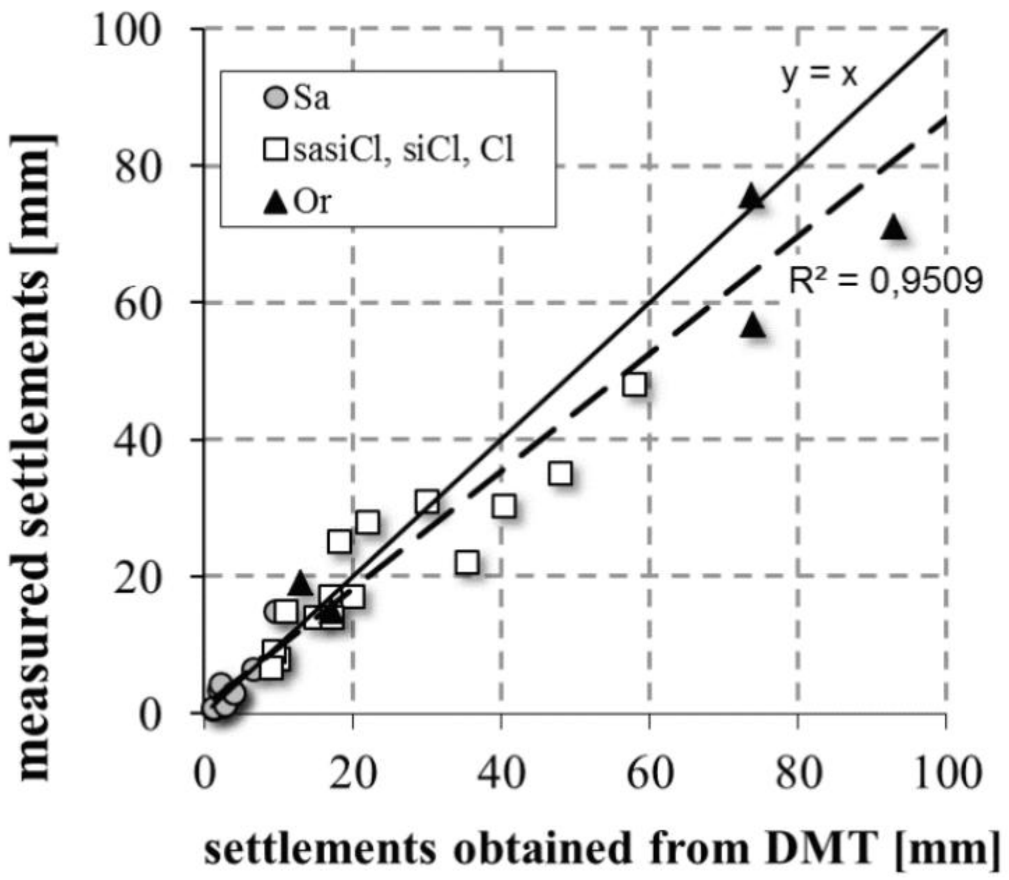
Observed vs. Predicted Settlements by DMT
Silos on Danube Bank (Belgrade)



Silo founded on mat 100 m x 23 m, with qnet = 160 kPa
DMT Settlement prediction: 77 cm
Measured Settlement: 63 cm
DMT +22%

D. Berisavijevic, 2013

28 Projects: observed versus DMT predicted settlements

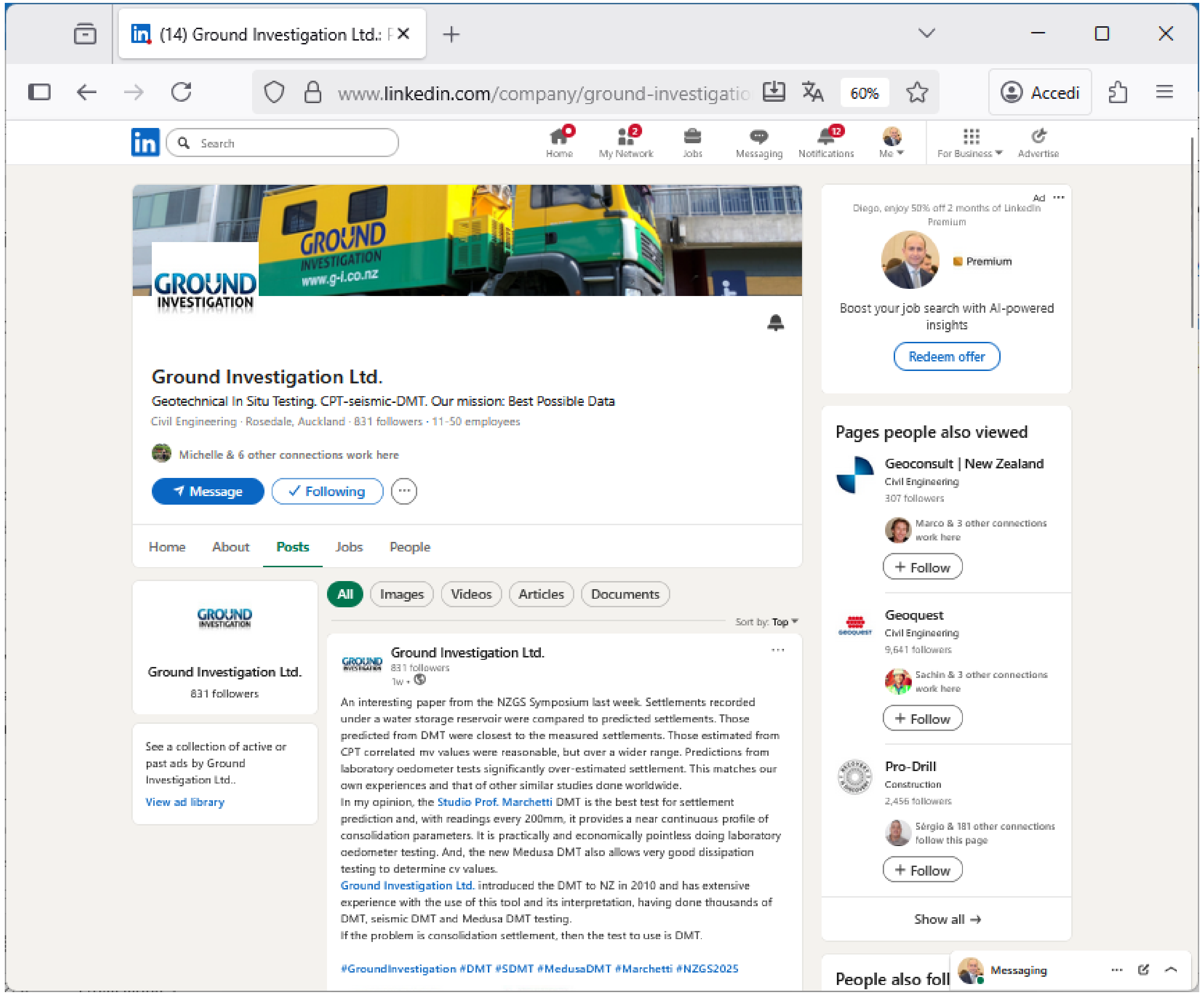


".. comparison of settlement values measured at the structures with respect to those obtained by dilatometer data and observations (28 structures). It should be added that the given set of buildings was limited to structures with shallow foundation.."

different soil types

Godlewski, 2018

Recent Linked In Post from Ground Investigation New Zealand (21/10/2025)



https://www.linkedin.com/posts/ground-investigation-ltd_groundinvestigation-dmt-sdmt-activity-7386242561469931520-MV7_?utm_source=share&utm_medium=member_desktop&rcm=ACoAAAGpd_ABHokCwIcTVXWUQH8_wWHT_qQZG-4

Recent Linked In Post of Ground Investigation (New Zealand 2025)

An interesting paper from the NZGS Symposium last week. Settlements recorded under a water storage reservoir were compared to predicted settlements. Those predicted from DMT were closest to the measured settlements. Those estimated from CPT correlated mv values were reasonable, but over a wider range. Predictions from laboratory oedometer tests significantly over-estimated settlement. This matches our own experiences and that of other similar studies done worldwide.

In my opinion, the Studio Prof. Marchetti DMT is the best test for settlement prediction and, with readings every 200mm, it provides a near continuous profile of consolidation parameters. It is practically and economically pointless doing laboratory oedometer testing. And, the new Medusa DMT also allows very good dissipation testing to determine cv values.

Ground Investigation Ltd. introduced the DMT to NZ in 2010 and has extensive experience with the use of this tool and its interpretation, having done thousands of DMT, seismic DMT and Medusa DMT testing.

If the problem is consolidation settlement, then the test to use is DMT.

hashtag#GroundInvestigation hashtag#DMT hashtag#SDMT hashtag#MedusaDMT hashtag#Marchetti hashtag#NZGS2025

Recent Linked In Post from Ground Investigation New Zealand (21/10/2025)

NZGS Symposium 2025

Geotechnical Horizons: Innovations & Challenges

15 - 18 October 2025 • Auckland, New Zealand

Predicted and Measured Settlement of Peat Loaded by a Reservoir

T.L. Smith, C.D. Gilbert & J.A. Masigan

Riley Consultants Ltd, Christchurch.

ABSTRACT

Te Waihekeora Water Storage Reservoir, completed in 2024, stores 3.2M m^3 of water within a 43 Hectare geomembrane lined basin located 15km south of Dargaville. The site topography is dominated by dune formations of the Kariotahi Group, with recent organic deposits infilling valley sections. The adopted design solution involved installation of a high-density polyethylene (HDPE) liner over natural soils, which included organic deposits up to 12m deep, with only nominal surface preparation. To support the assessment of liner deformations due to reservoir loading, a range of tests were undertaken including in-situ and laboratory techniques. Upon completion of construction, high-resolution bathometric surveys have been undertaken at prescribed reservoir levels. The comparison of available settlement prediction techniques with measured deformations over an extended area provides a range of insights around their applicability and accuracy. Derived settlement coefficients are presented, which may be of interest to those working with similar ground conditions.

1 INTRODUCTION

The Te Waihekeora Reservoir, located 15km south of Dargaville, provides 3.2M m^3 of water storage for the Kaipara Water Scheme. The reservoir is designed to support up to 1100 Ha of new horticultural development on the Poutō Peninsula. One high potential impact classification (PIC) dam and four low PIC dams were built to form the reservoir, with a maximum retained height of 11m.

The work was commissioned by the Te Tai Tokerau Water Trust, which has the objective to develop water schemes to enable the establishment of commercially viable and environmentally sustainable horticulture, providing economic and employment opportunities in the region. The Trust has received funding assistance from Kānoa - Regional Economic Development & Investment Unit - the Northland Regional Council and Far North District Council.

7 CONCLUSIONS

1. A fundamental limitation to the ability to accurately predict primary and secondary settlement across a large and variable site such as the subject site, is the accuracy and resolution of site geological model. As always, the services of an experienced engineering geologist are essential.
2. Settle 3D (RocScience) is able to analyse and present extensive and complex site and loading geometries, though some "workarounds are required".
3. In-situ DMT testing stood out as the most accurate predictor for primary settlement parameters.
4. The large-scale preload trial remains the "gold standard" for such analyses as it considers the full soil profile. However, in this case, and more broadly in the experience of the authors, these site trials require careful supervision and can be undermined by the practicalities of constructing and monitoring them.
5. CPT based settlement correlations provided relatively accurate parameter estimates in this case.
6. Laboratory Oedometer testing significantly overestimated settlement in this case. Sample disturbance was suspected as a leading cause for inaccuracy, and this may have been exacerbated by the high fibrous peat content of the samples.
7. Traditional approaches to Cv estimation relate to permeability and the ability of soil to dissipate porewater pressure. This approach is not applicable to the partially saturated peat being assessed. This was confirmed by the preload trial, which more accurately estimated time related effects.

Example of SDMT measurements and a ‘real time’ Settlements Prediction at a demonstration site for a workshop

Bogotà (Colombia - 2015)

Workshop on DMT and CPT

Bogotà, Colombia May 2015

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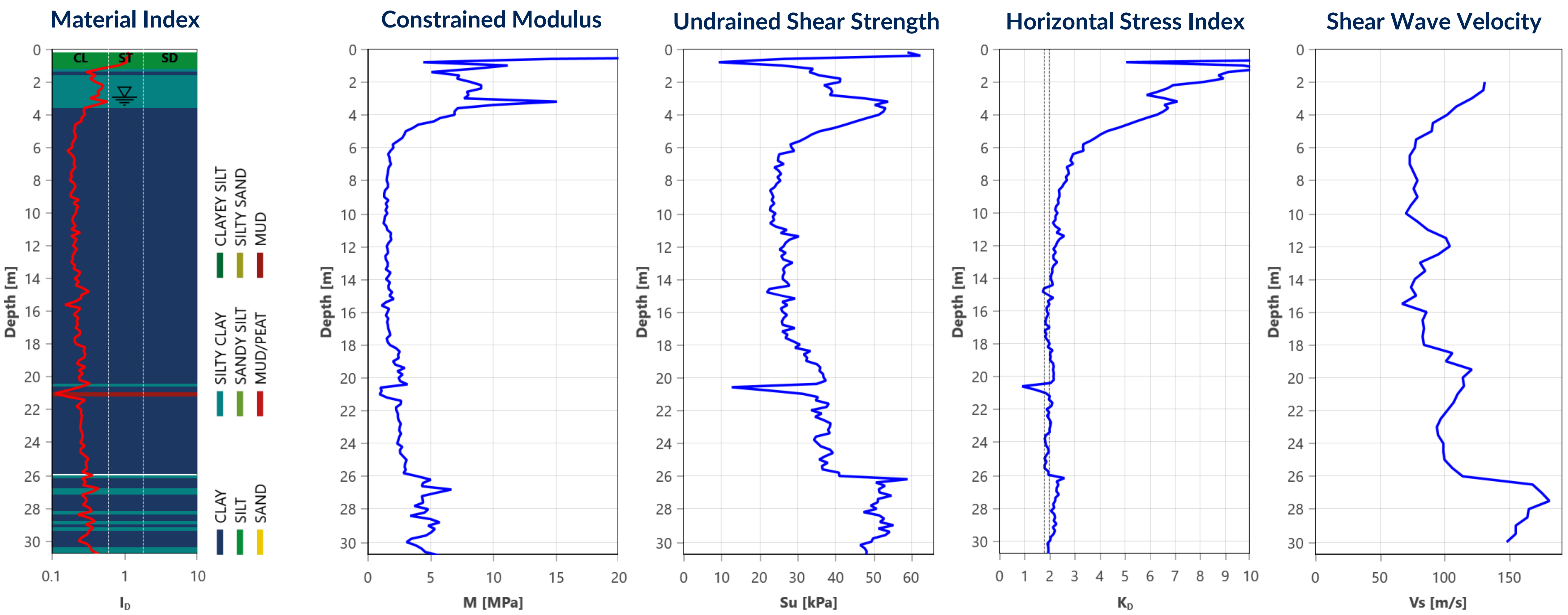
SDMT main soil profiles

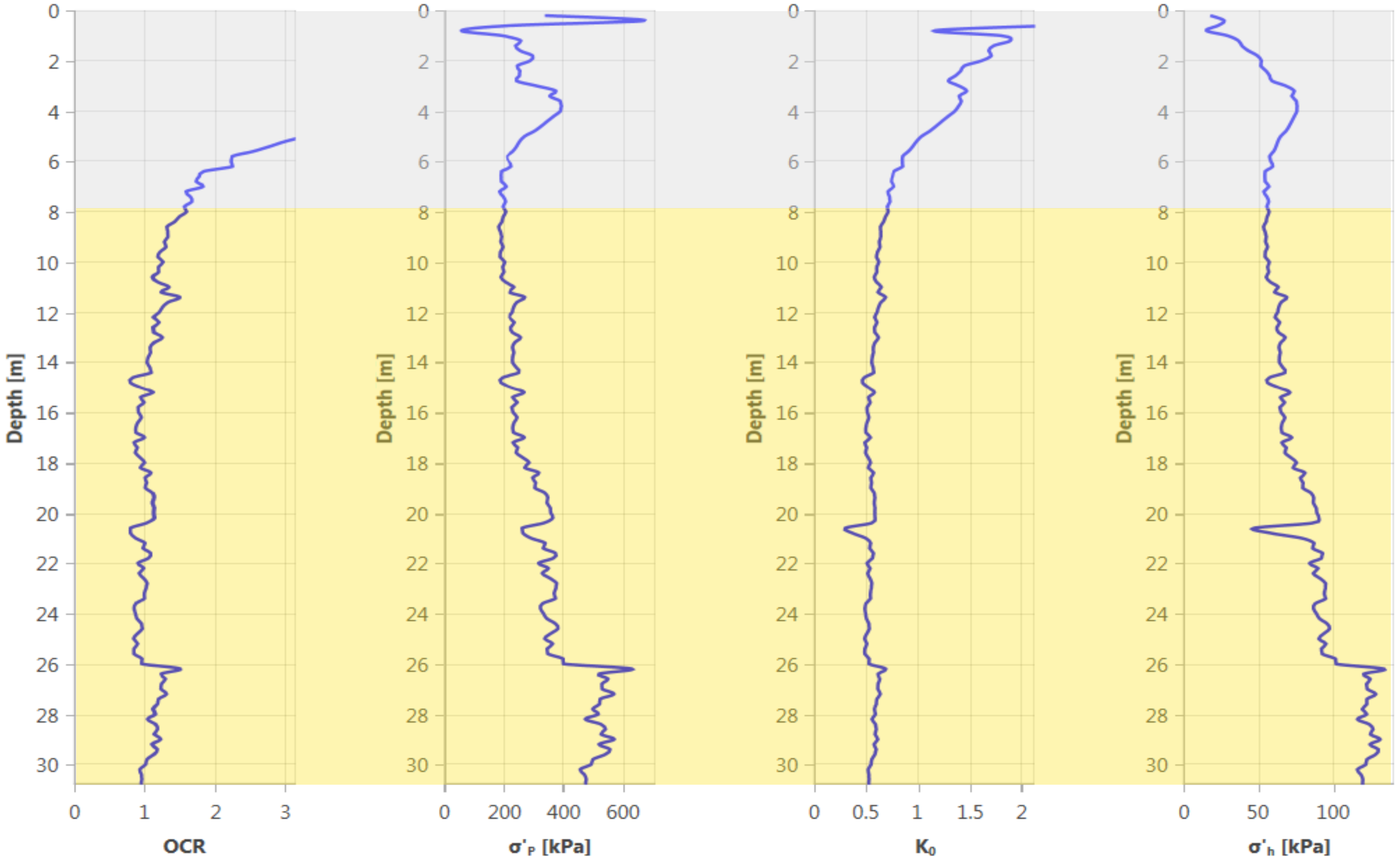


Stress History Parameters

OCR >> 1 → TOP CRUST

OCR ~ 1 → NC Clay

Overconsolidation Ratio | Preconsolidation Pressure | Earth Pressure Coefficient | Horizontal Effective Stress



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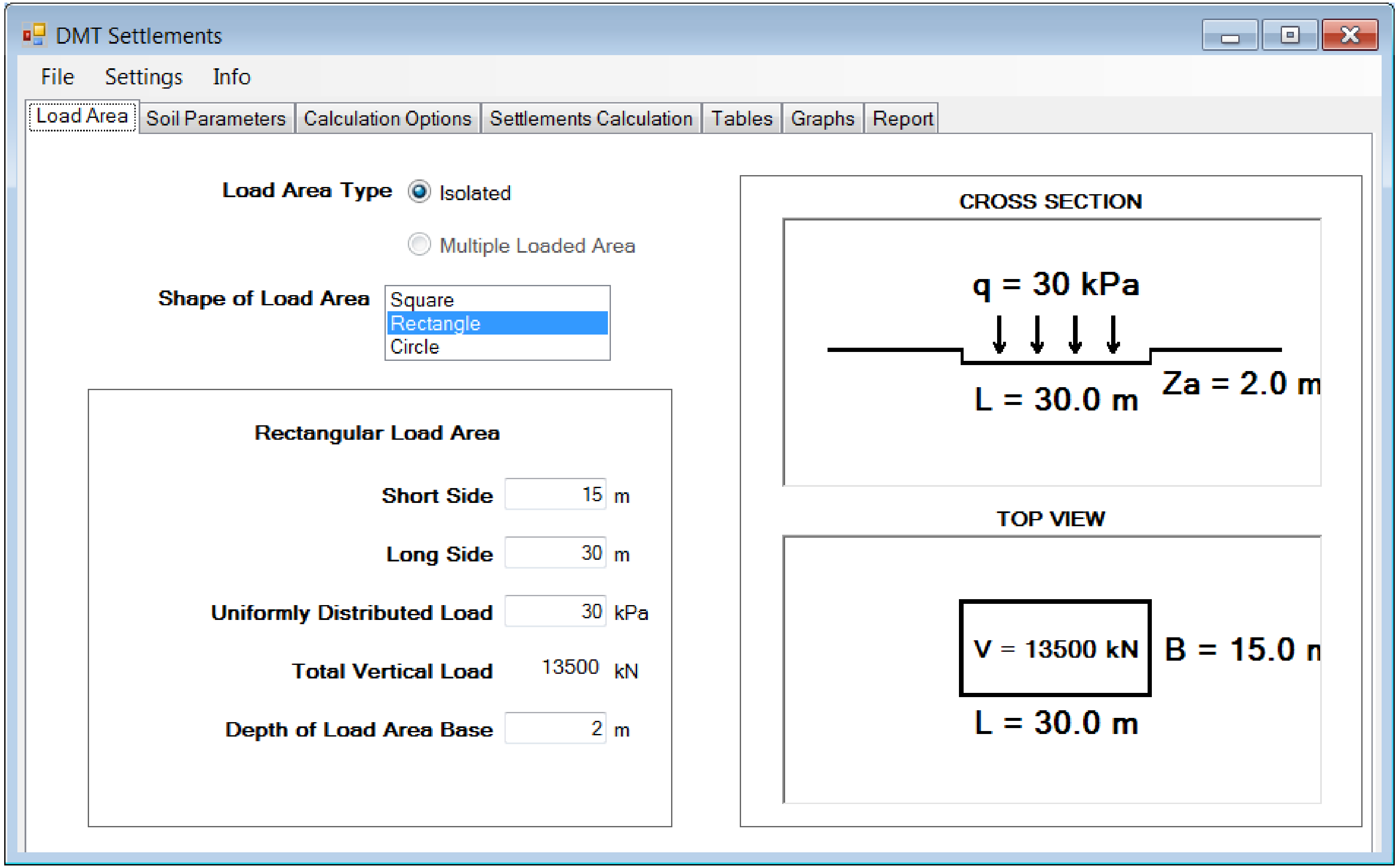
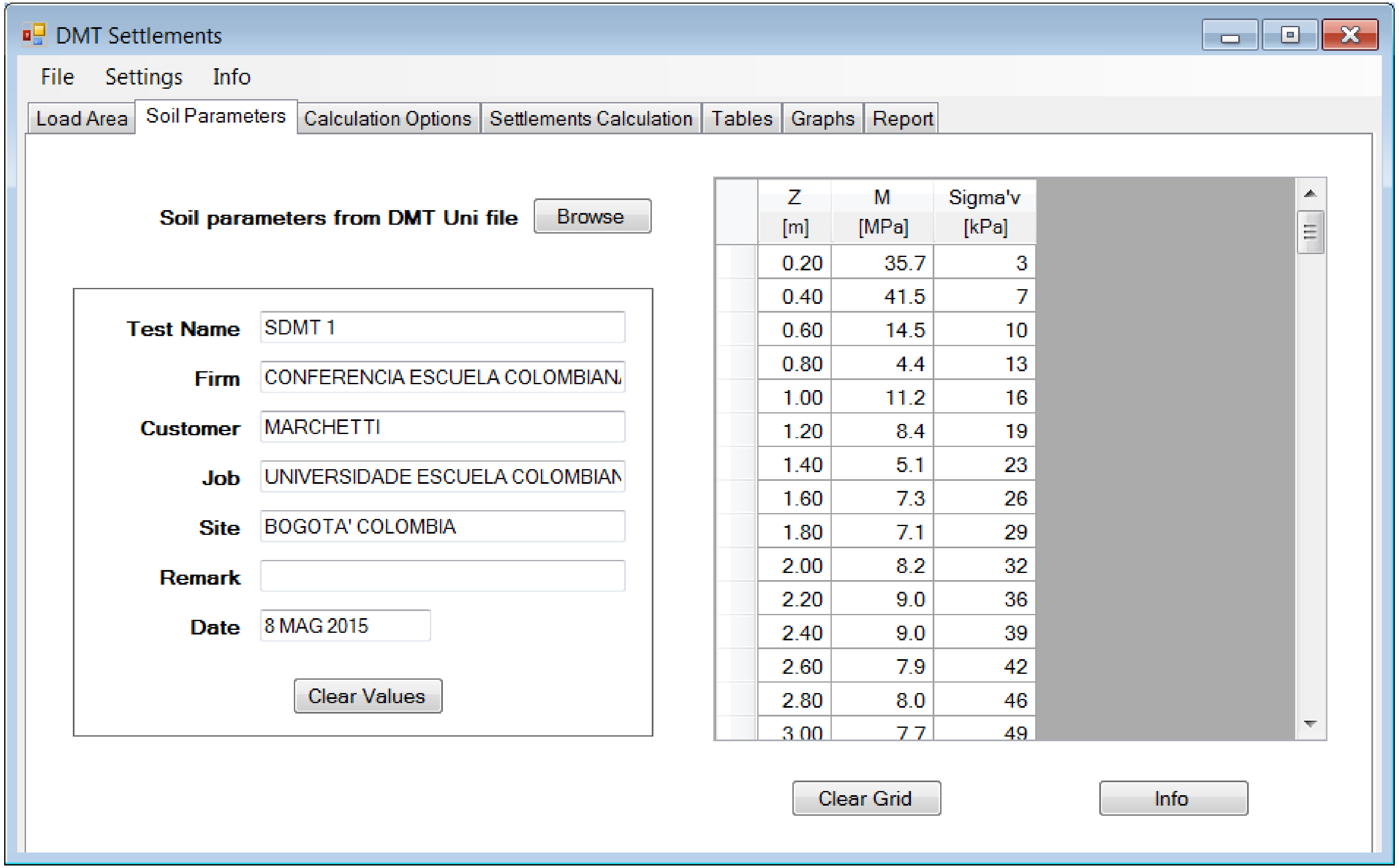
Settlements Prediction: Load information



Settlements Prediction: Soil information (modulus)

Workshop in Colombia (May 2015, Bogotà)



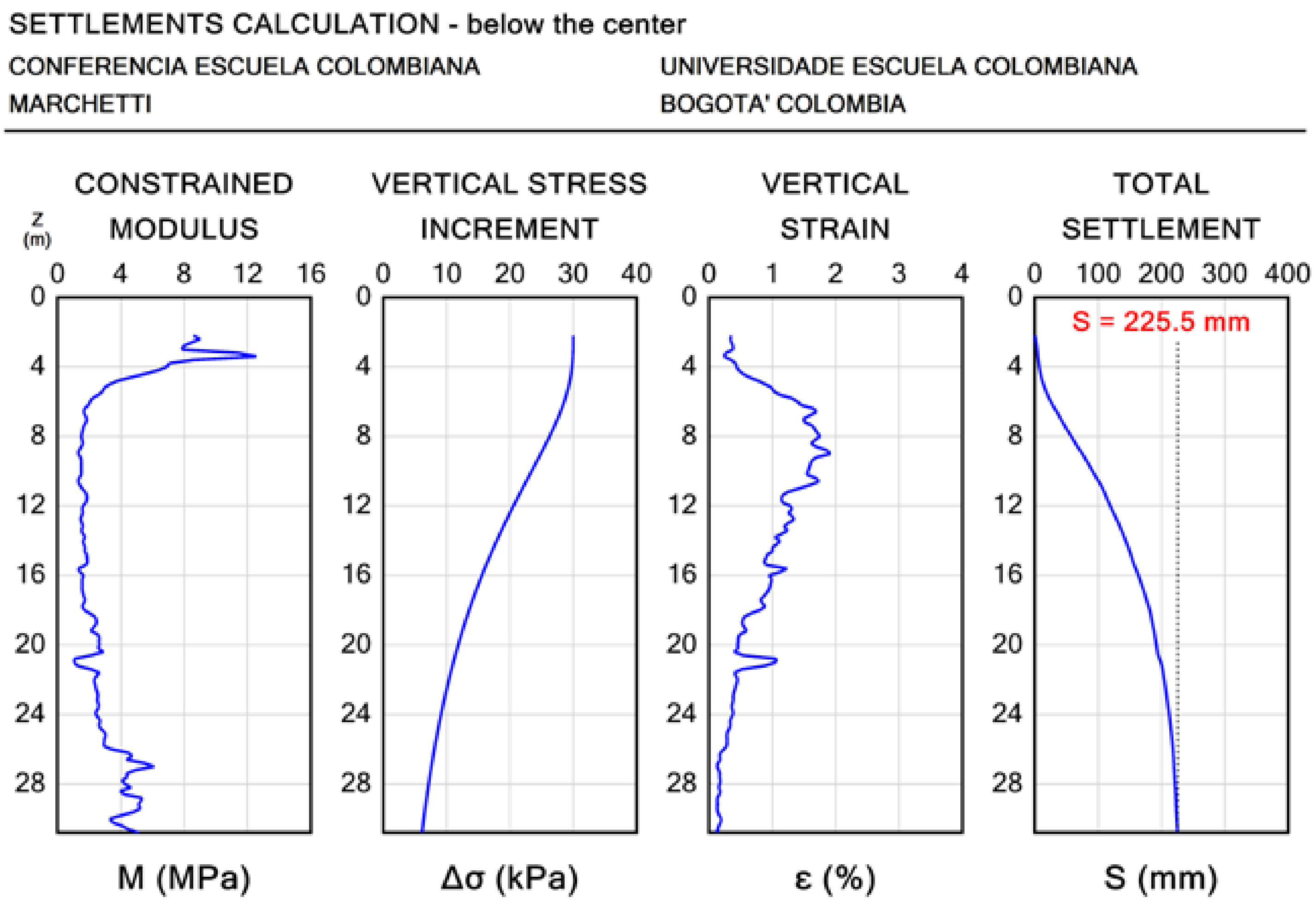
Z [m]	M [MPa]	Sigma'v [kPa]
0.20	35.7	3
0.40	41.5	7
0.60	14.5	10
0.80	4.4	13
1.00	11.2	16
1.20	8.4	19
1.40	5.1	23
1.60	7.3	26
1.80	7.1	29
2.00	8.2	32
2.20	9.0	36
2.40	9.0	39
2.60	7.9	42
2.80	8.0	46
3.00	7.7	49

Settlements Prediction: evaluation of the settlement

	Settlements Calculation Point	Settlements [mm]	Z Stop [m]	Δσ/σ'v
▶	below the center	225.5	30.80	0.027
	below the corner	79.8	30.80	0.017
	below the median point of short side	121.5	30.80	0.019
	below the median point of long side	144.6	30.80	0.024

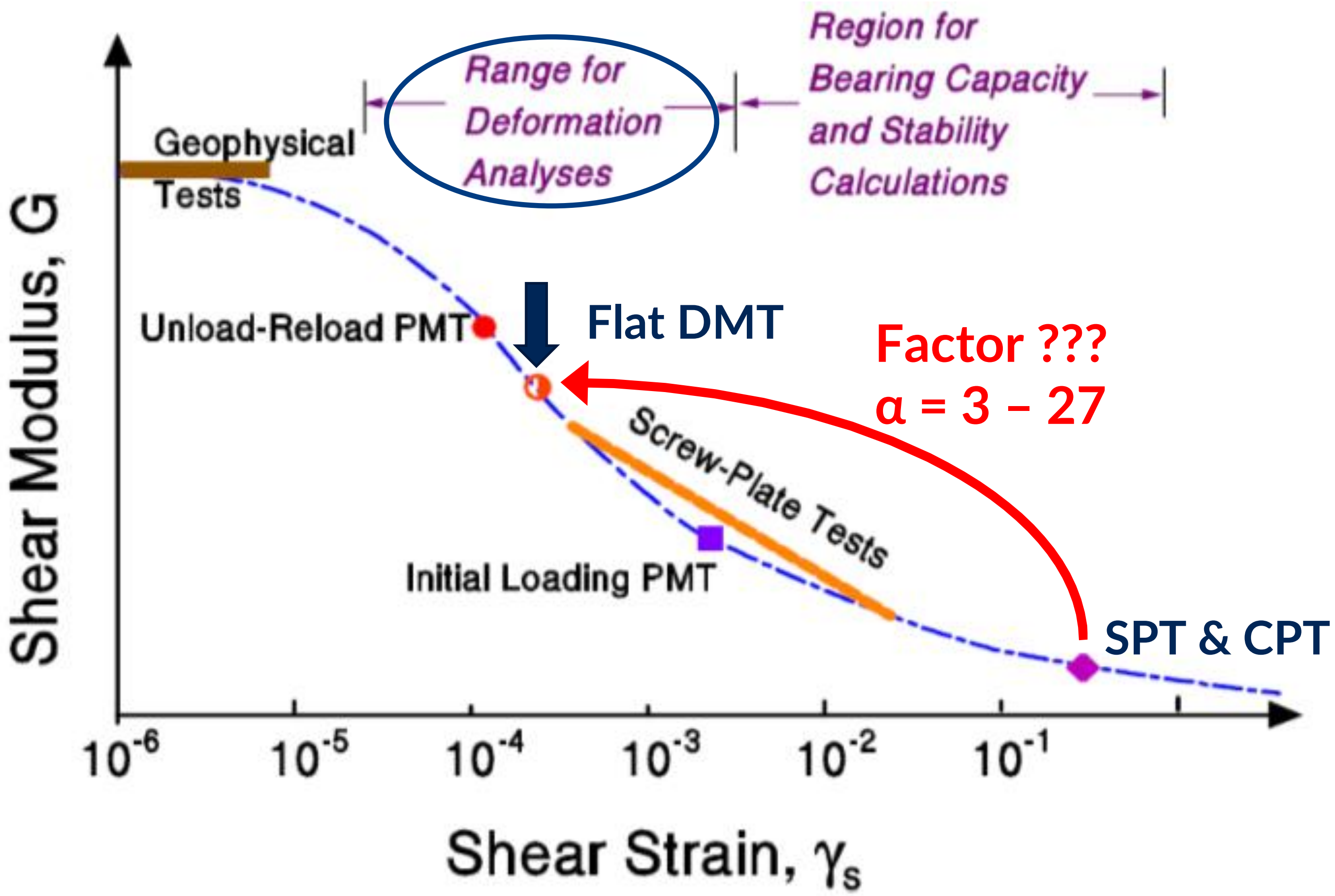
Settlements Prediction: graph below center of load

Workshop in Colombia (May 2015, Bogotà)



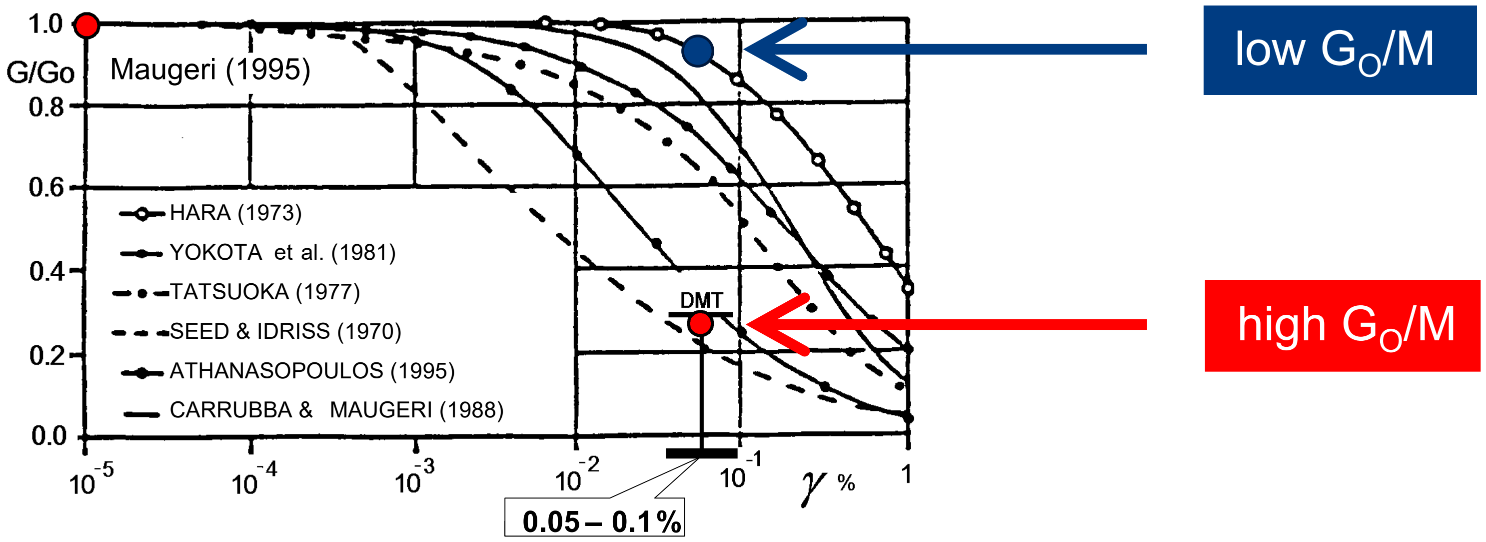
In situ estimation of G-gamma decay curve

DMT: direct measurement of modulus in the soil loaded at the strain level for deformation analysis



Mayne (2001)

G_0 and M_{DMT} on the G - γ decay curve



SDMT ➡ G_0 - *small strain modulus from Vs* ($\gamma \leq 10^{-5}$ %)

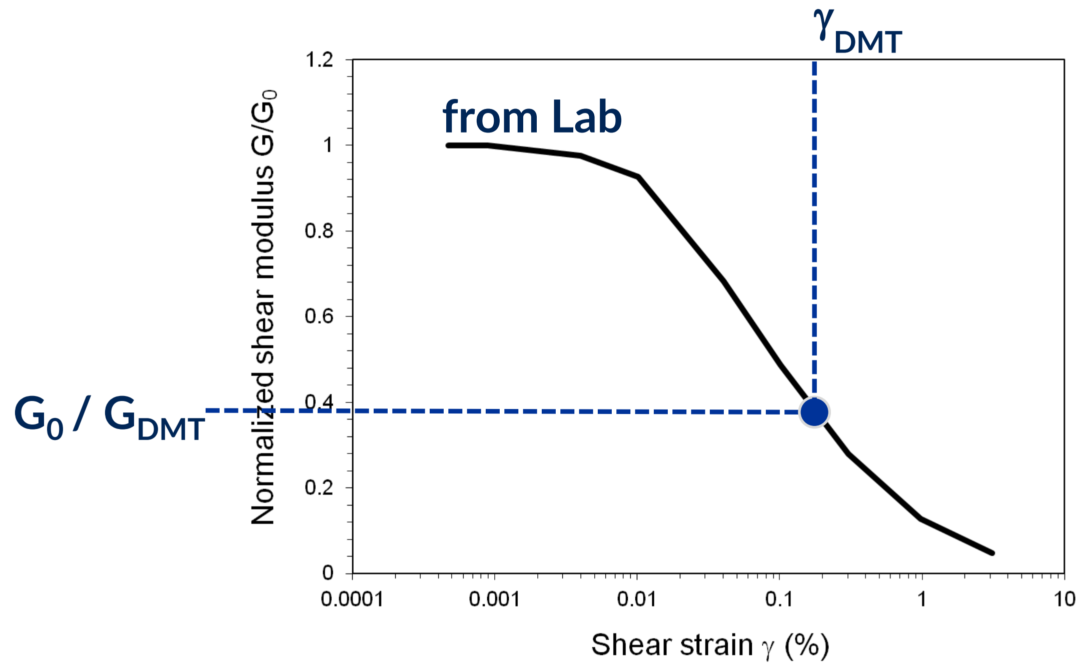
two points M_{DMT} - *working strain modulus* (γ = 0.05 – 0.1 %) *Mayne (2001)* *Ishihara (2001)*

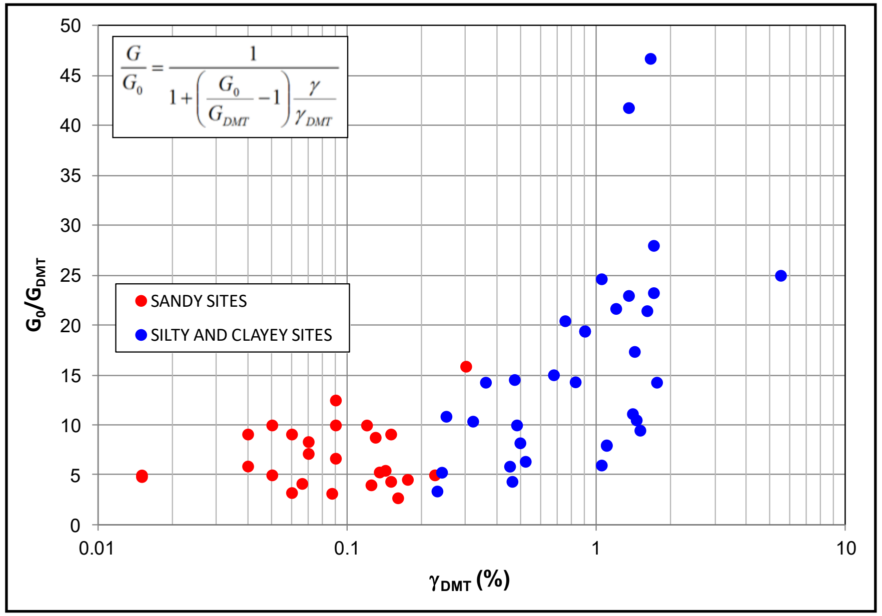
G_0 / M_{DMT} may provide an in situ estimate of the G-γ decay curve

Pubblications: Rodriguez et al (2019), Amoroso et al (2012, 2014), Marchetti et al (2008), Lehane & Fahey (2004)

Tentative estimation of G - γ decay curve (Amoroso et al 2014)

SDMT and Lab data: intersection method for the construction of a **hyperbolic equation to estimate curve**





G_{DMT} from M_{DMT} (elastic theory)

G_0 from Vs by SDMT

G-γ reference curve from laboratory

γ_{DMT} = 0.015-0.30 % (**sand**)

γ_{DMT} = 0.23-1.75 % (**clay**)

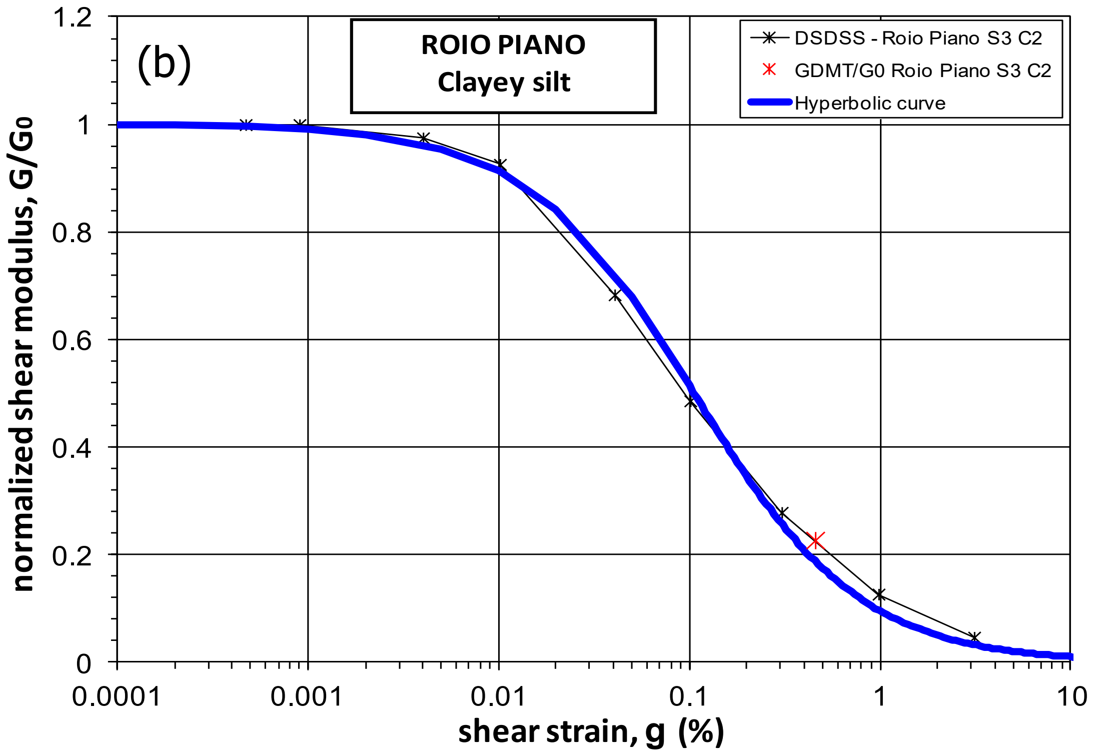
Validation of G - γ decay curve estimation

- First validation in a clayey silt soil
  Roio Piano 2014

- Good agreement between hyperbolic SDMT estimation and laboratory stiffness decay curve

- Potential applications
  - Soil input data for seismic ground response analysis
  - Numerical modelling (e.g. deep excavations Plaxis HSS, Di Mariano et al. 2019)

further validation required



$$\frac{G}{G_0} = \frac{1}{1 + \left(\frac{G_0}{G_{DMT}} - 1\right)\frac{\gamma}{\gamma_{DMT}}}$$

Amoroso et al. 2014

FEM input parameters (Plaxis)

DMT for FEM input parameters
→ PLAXIS Hardening Soil (HS) constitutive model

- **PLAXIS** is a finite element software developed at Delft University of Technology for the analysis of soil and rock behavior under static and dynamic loading conditions

- Simulations of:
  - ground–structure interaction by incorporating advanced constitutive soil models
  - staged construction
  - time-dependent effects such as consolidation and creep

- Evaluates deformation, stability and pore pressure response in:
  - foundations, excavations
  - embankments
  - tunnels
  - slopes
  - offshore structures

DMT for FEM input parameters
→ PLAXIS Hardening Soil (HS) constitutive model

- Constitutive models in PLAXIS describe the stress–strain–time behavior of soils and rocks under various loading and environmental conditions

- These models form the core of finite element analysis by defining how materials respond to changes in stress, pore pressure, strain rate and loading history

- Broadly, the constitutive models in PLAXIS can be classified into
  - **elastic models, (Linear elastic (LE))**
  - **elasto-plastic models, (Mohr–Coulomb, (MC))**
  - **advanced soil models, (Hardening soil (HS), Hardening soil with small strain (HSSmall), Soft soil (SS), Soft Soil creep (SSC))**
  - **rock models**
  - **special-purpose models. (UBC3D PLM, PM4 Sand, PM4 Silt)**

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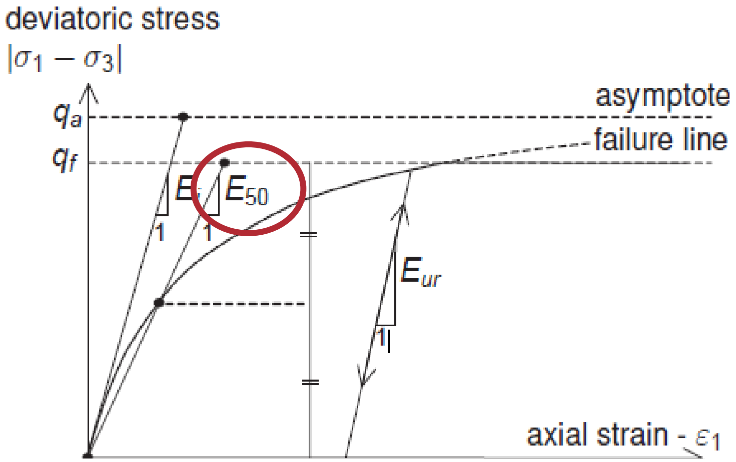
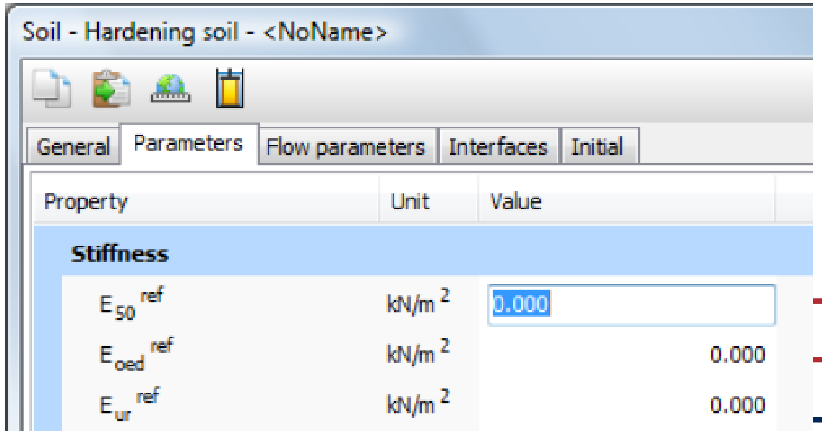




E_{50} is the Modulus for primary loading

$$E_{50} = E_{50}^{ref}\left(\frac{c\cos\varphi - \sigma'_3\sin\varphi}{c\cos\varphi + p^{ref}\sin\varphi}\right)^m$$

***E_{50}^{ref}* is a reference stiffness modulus corresponding to the reference confining pressure *p_{ref}* (*p_{ref}* =1 bar)**

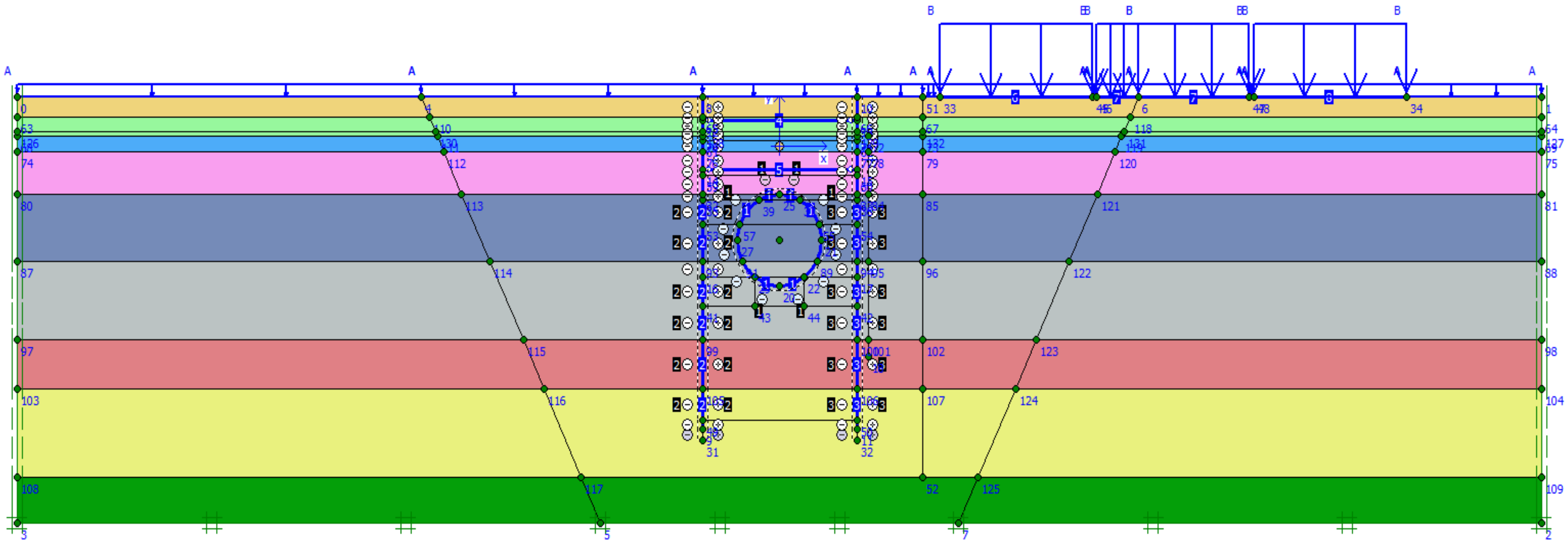
- E_{50}^{ref} correlated to M_{DMT} and E_{oed}^{ref} (Schanz & Vermeer, 1997)
- literature: E_{50}^{ref} = 15 to 75 MPa, like M_{DMT} by DMT (quartz sand, loose to very dense sand)

$$E_{50}^{ref} = E_{oed}^{ref} = M_{DMT}$$

$$E_{ur}^{ref} = 4 \cdot E_{oed}^{ref} = 4 \cdot M_{DM}$$

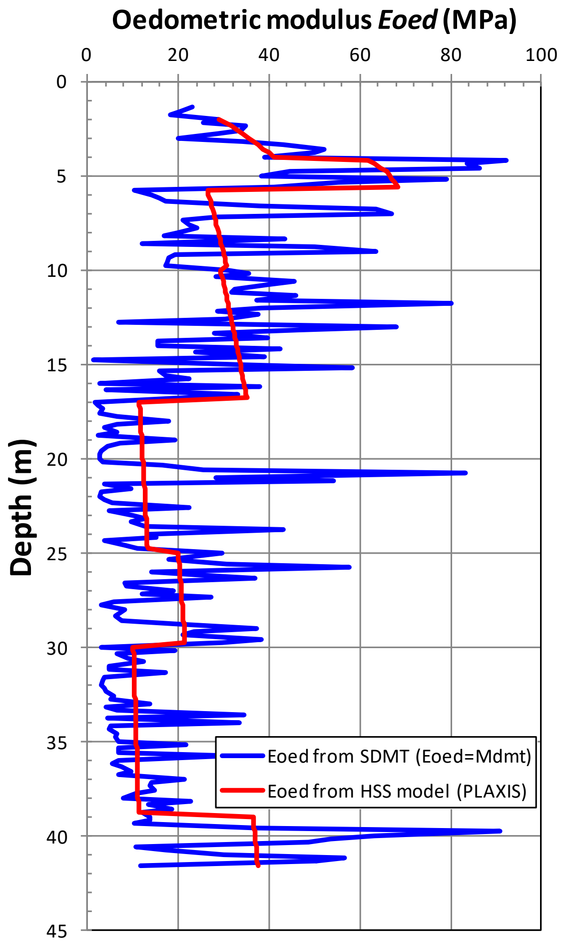
SDMT for FEM input parameters
PLAXIS Hardening Soil Small Strain (HSS) constitutive model

Comparison between HSS model – PLAXIS from SDMT parameters and monitoring activities for the excavation of Verge de Montserrat Station (Barcelona, Spain)

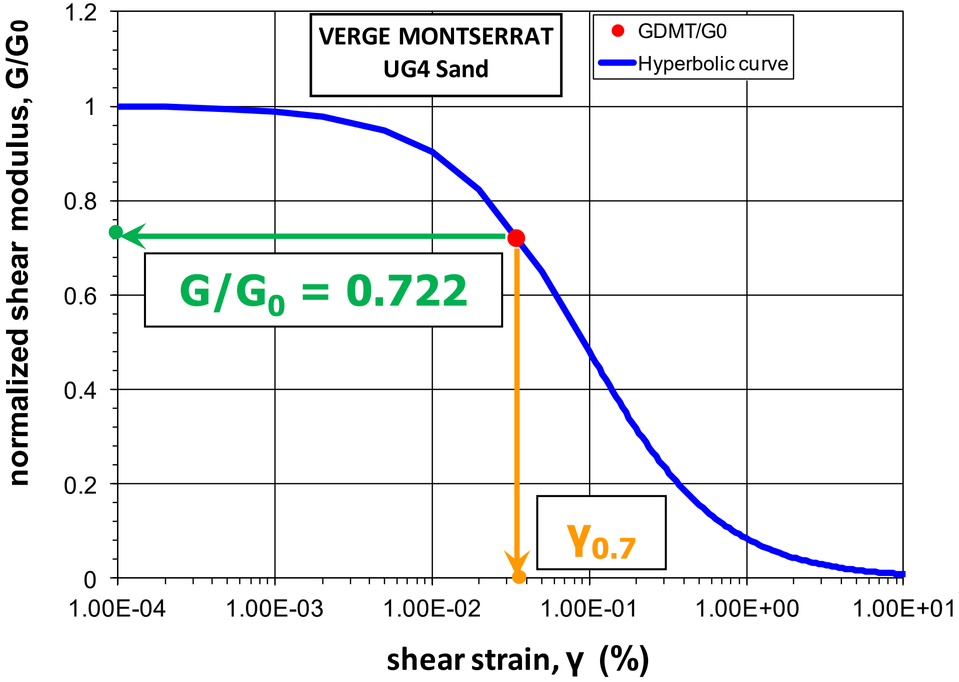


Di Mariano et al. (2019)

SDMT for FEM input parameters
PLAXIS Hardening Soil Small (HSS) constitutive model



Di Mariano et al. (2019)



'.. significant variability due to the fine interlayering of silts, sands, and clays ..'

HSS model – PLAXIS

$$E_{oed} = E_{oed}^{ref}\left(\sigma_1'/p^{ref}\right)^m$$

$$E_{50} = E_{50}^{ref}\left(\sigma_3'/p^{ref}\right)^m$$

$$E_{ur} = E_{ur}^{ref}\left(\sigma_3'/p^{ref}\right)^m$$

$$G_0 = G_0^{ref}\left(\sigma_1'/p^{ref}\right)^m$$

Assumptions:

$$M_{DMT} = E_{oed} = E_{50} = E_{ur}/4$$

Work in progress

Our Geotechnical Team lead by Dr. Kaustav Das is working on a document containing clear specific guidelines for evaluating the input parameters of Plaxis using DMT and SDMT test results, subdivided for each of the different constitutive models (current deadline June 2026)

Draft 1: Constitutive models in Plaxis (LE)

The Linear Elastic (LE) model assumes a linear relationship between stress and strain governed by Young's modulus (E) and Poisson's ratio (ν).

Model name	Applications	Basic Input Parameters	DMT Parameter	Conversion
LE	Structural elements (plates, beams) & Preliminary analyses Very stiff soils or rock masses where deformations are small	Young's Modulus (E)	M_{DMT}	$E = \frac{(1+\nu)(1-2\nu)}{(1-\nu)} M$ (for ν = 0.25 – 0.30, E ≈ 0.8 M_{DMT})
		Poisson's Ratio (ν)	-	Assumed
		Unit weight (γ)	γ	Directly obtained from SDMT Pro

Draft 2: Constitutive models in Plaxis (MC)

The Mohr–Coulomb (MC) model is the most widely used elasto-plastic model in PLAXIS. It is based on a linear elastic response up to failure, followed by perfectly plastic behavior governed by the Mohr–Coulomb failure criterion.

Model name	Applications	Basic Input Parameters	DMT Parameter	Conversion
MC	Preliminary analysis at the beginning of any project, Suitable for stiff clay where shear strength is governed by only S_u Good for sandy soils	Young's Modulus (E)	M_{DMT}	$E = \frac{(1+\nu)(1-2\nu)}{(1-\nu)} M$ (for ν = 0.25 – 0.30, E ≈ 0.8 M_{DMT})
		Poisson's Ratio (ν)	-	Assumed
		Unit weight (γ)	γ	Directly obtained from SDMT Pro
		Cohesion/Undrained shear strength (S_u)	S_u	Directly obtained from SDMT Pro
		Peak Frictional Angle (φ)	φ	Directly obtained from SDMT Pro (only sand/silt)
		Coefficient of permeability (k)	k_h	Directly obtained from SDMT Pro with dissipation tests (only for clay/silt)

Quality Assessment of Soil Improvement

In the last decades the DMT has been increasingly used in compaction jobs to quantify the gain in soil improvement

Ground Reinforcement

- Stone Columns
- Soil Nails
- Micropiles
- Jet Grouting
- Ground Anchors
- Geosynthetics
- Fibers
- Lime Columns
- Vibro-Concrete Columns
- ..

Ground Improvement

- Surface Compaction
- Drainage/Surcharge
- Electro-osmosis
- Compaction grouting
- Blasting
- Dynamic Compaction
- ..

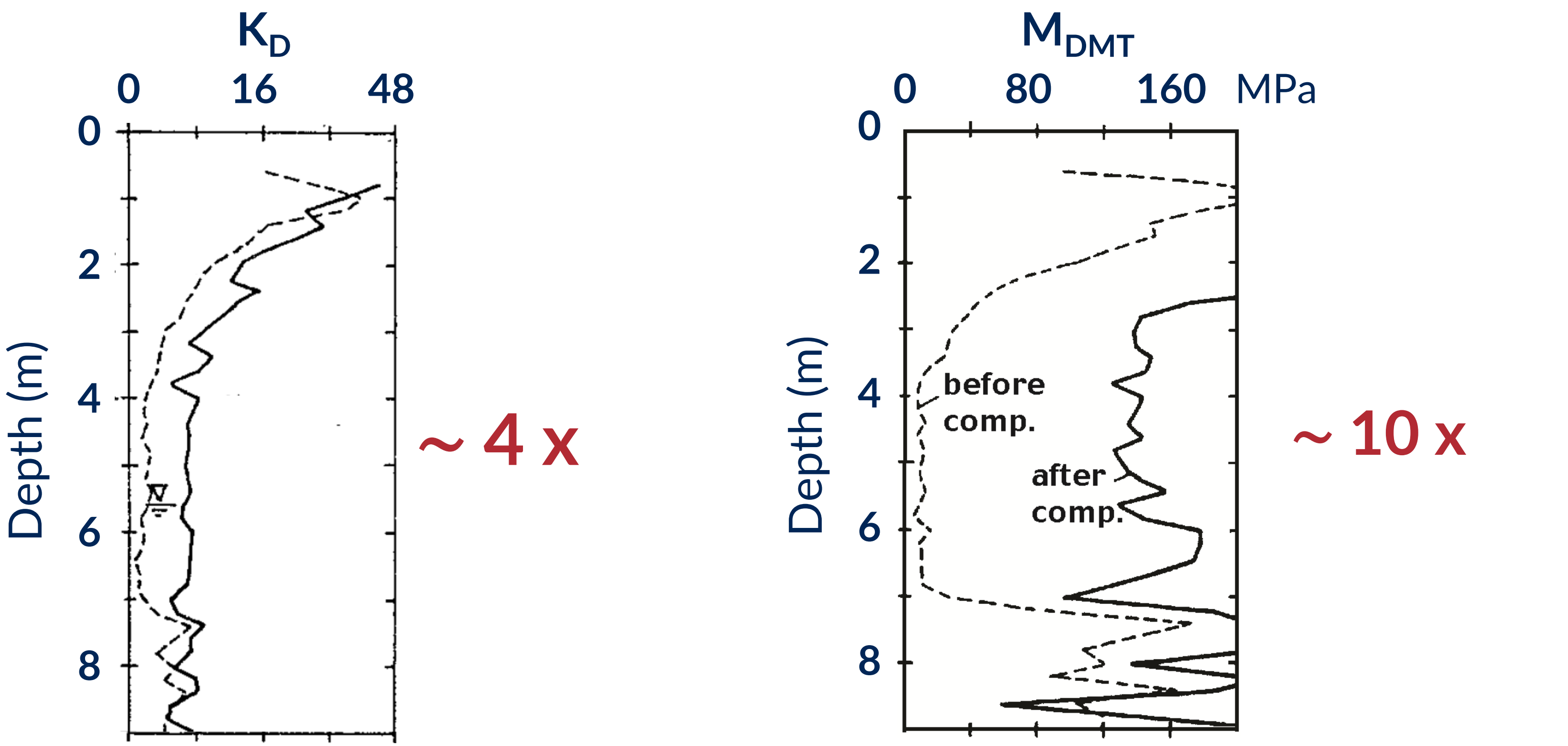
Ground Treatment

- Soil Cement
- Lime Admixtures
- Flyash
- Dewatering
- Heating/Freezing
- Vitrification
- ..

QA of soil improvement with DMT (case history 1)

Belgium – container terminal (loose sandfill)

Resonant vibrocompaction technique



Van Impe, De Cock, Massarsch, Mengé - New Delhi (1994)

QA of soil improvement with DMT (case history 2)

North Carolina (USA)

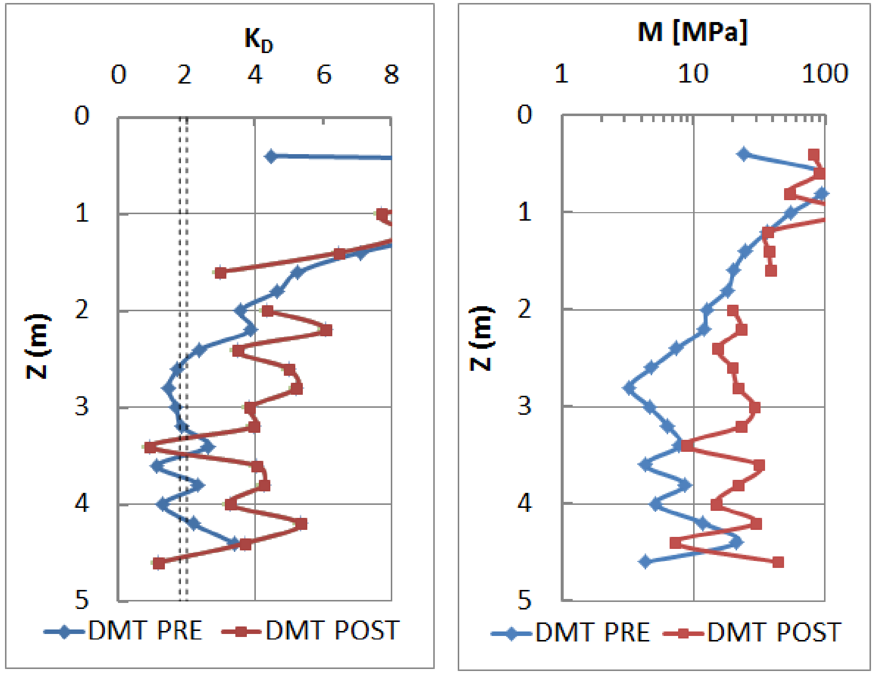


Figure 3

Resin Injection (in sands)

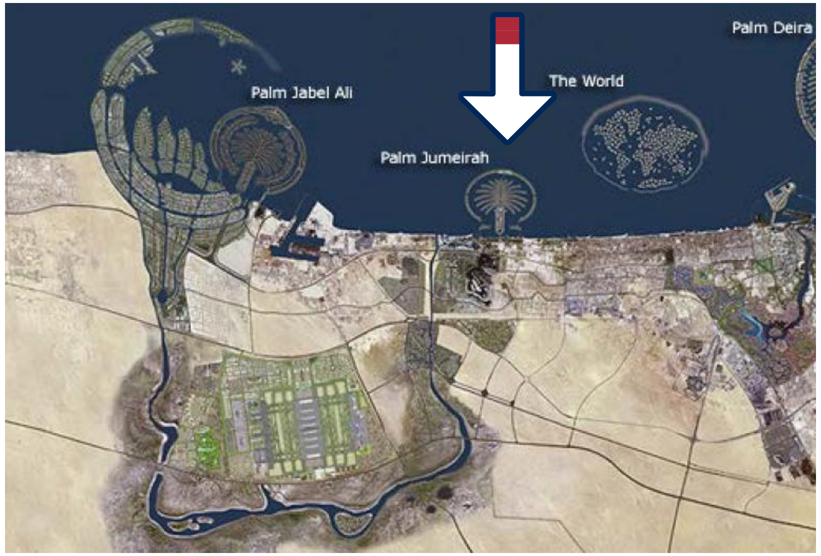
"The DMT tests were performed near the ground improvement and about 15 feet away from the improvement"

"Figure 3 illustrates how well resin injections improved the soil and how well K_D and M detected such improvements" (Failmezger 2017)

Grifton School Project

QA of soil improvement with DMT (case history 3)

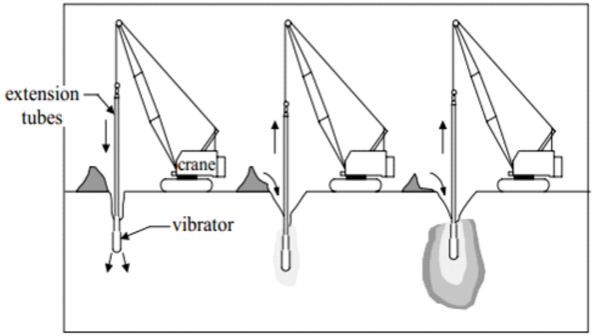
Palma Jumeirah Dubai



"..hydraulically filled silty fine calcareous sand dredged from sea bed, underlain by sedimentary rock of very weak sandstone and siltstone.."

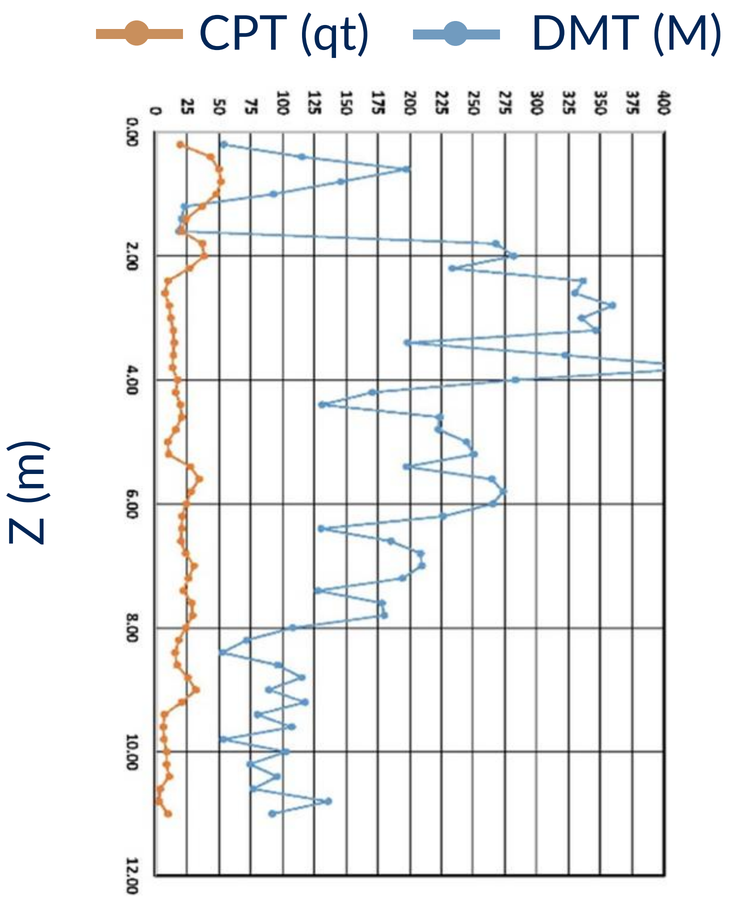
Aim of DMT & CPT tests: to confirm OC of vibrocompaction, detected also by very high Vs (400-500 m/s)

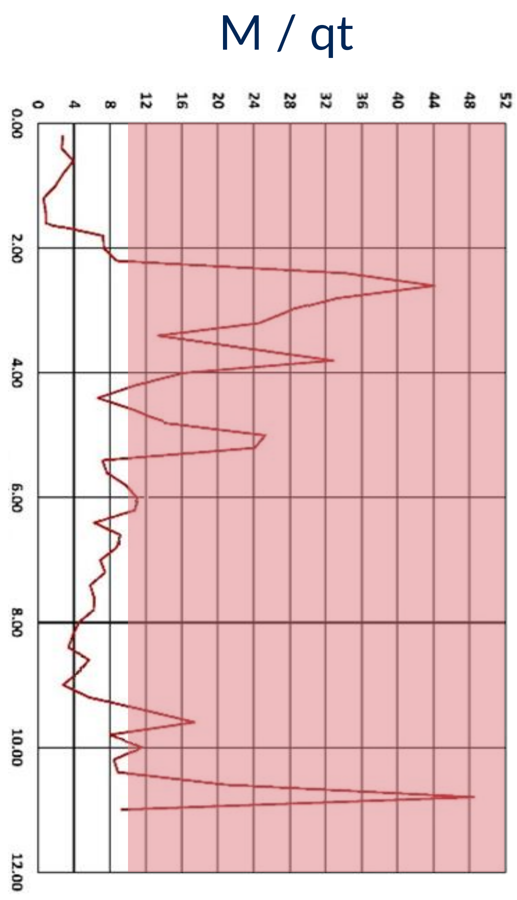
E. Sharif (2015)



QA of soil improvement with DMT (case history 3)

Palma Jumeirah Dubai





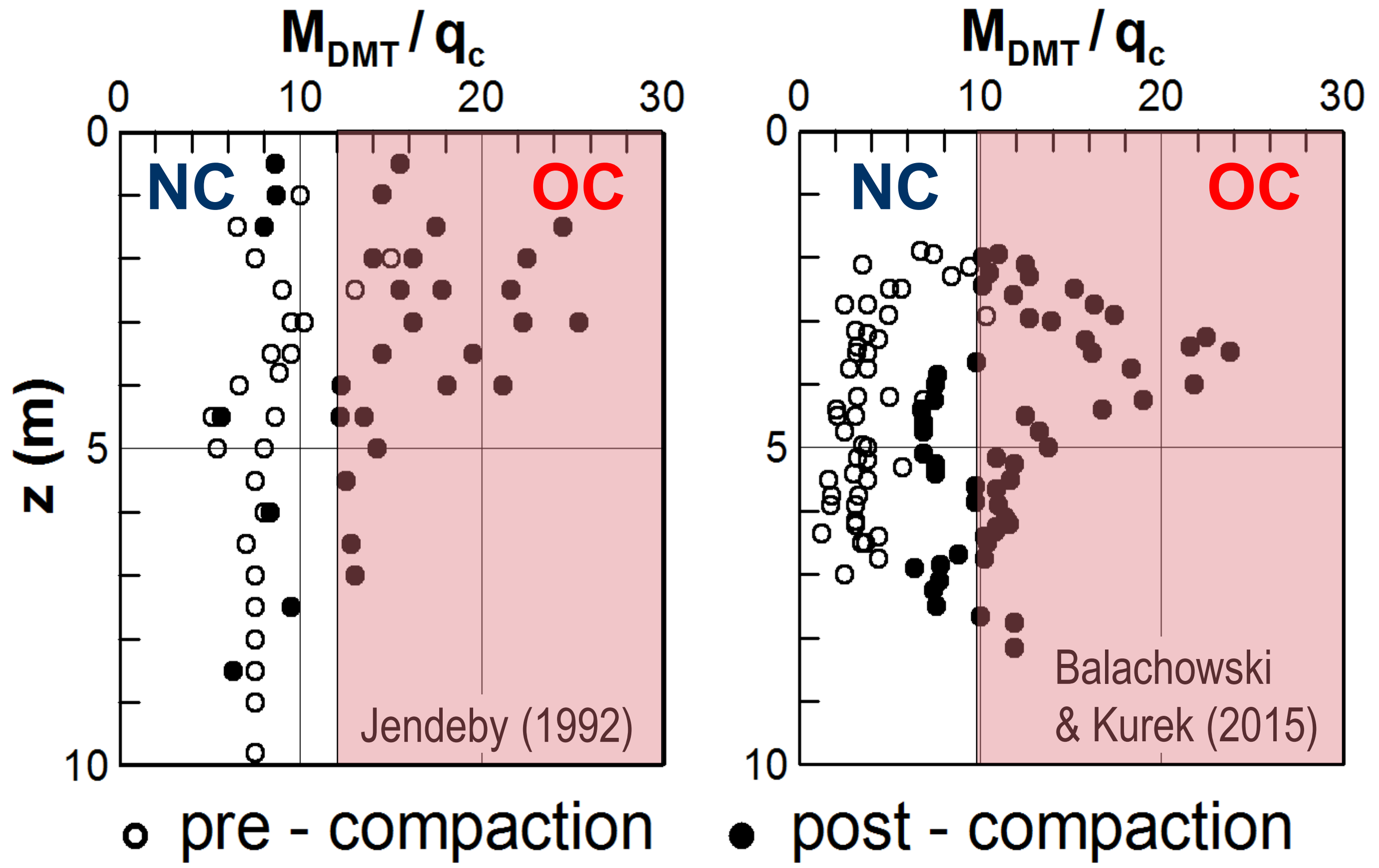
Conclusion:

M / qt ≤ 10 → poor quality compaction

M / qt ≥ 10 → high quality compaction

E. Sharif (2015)

QA of soil improvement with DMT (case history 4 and 5)



Conclusion:

$M_{DMT} / q_c \leq 10$ → poor quality compaction

$M_{DMT} / q_c \geq 10$ → high quality compaction

QA of soil improvement with DMT: indications

Recommended:

- DMT/SDMT before and after the treatment
- compare before/after profiles of K_D, M_{DMT}, Vs

When before treatment tests are not possible:

- both DMT and CPT (after treatment)
- examine ratio M_{DMT}/qt (must be higher than ~ 10)

Slip surface detection in OC clay slopes

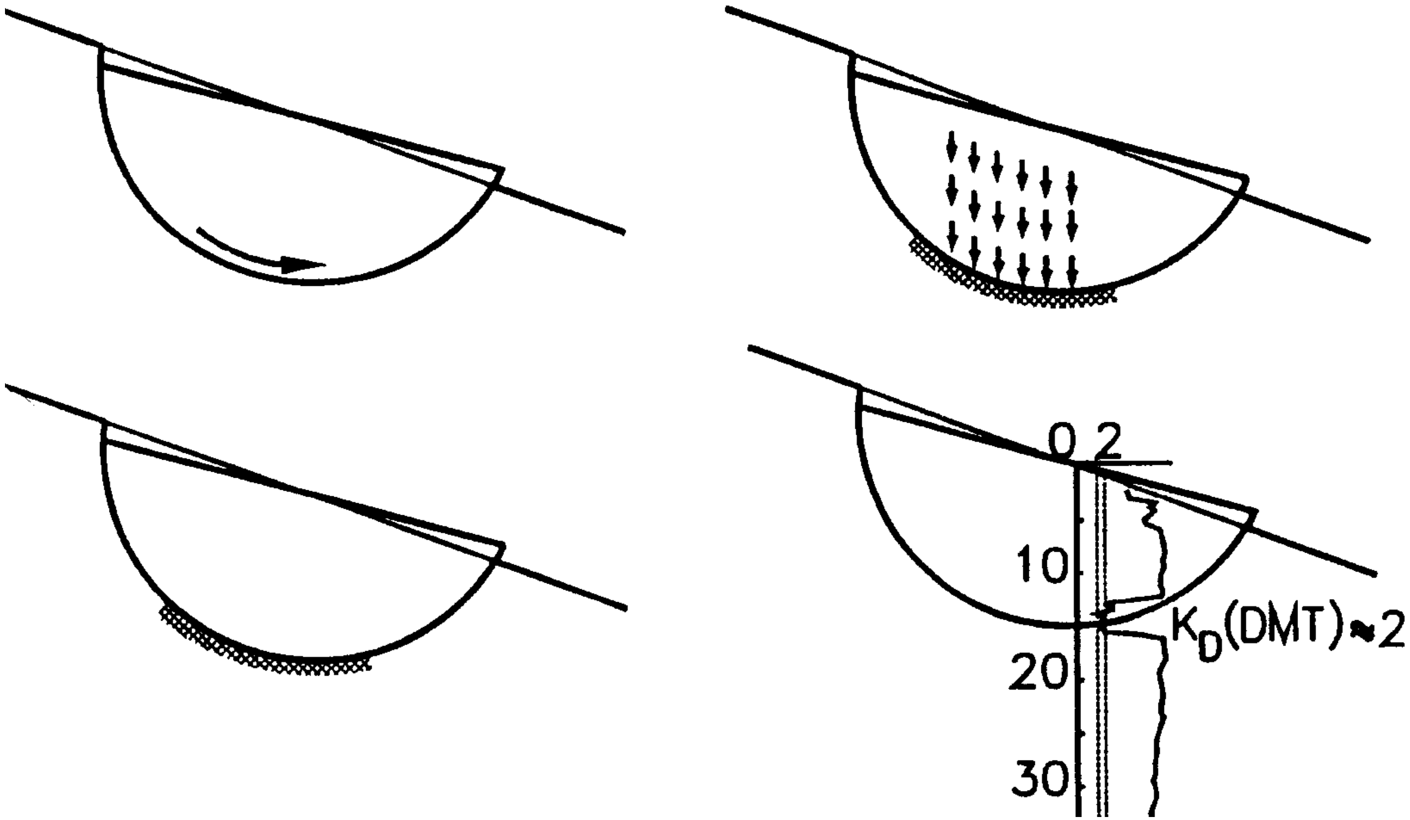
DMT-K_D = 2 method → Verify if an OC clay slope contains active (or old quiescent) slip surface

1 Sliding

2 Remoulding

3 Reconsolidation (NC State)

4 Inspect K_D profile



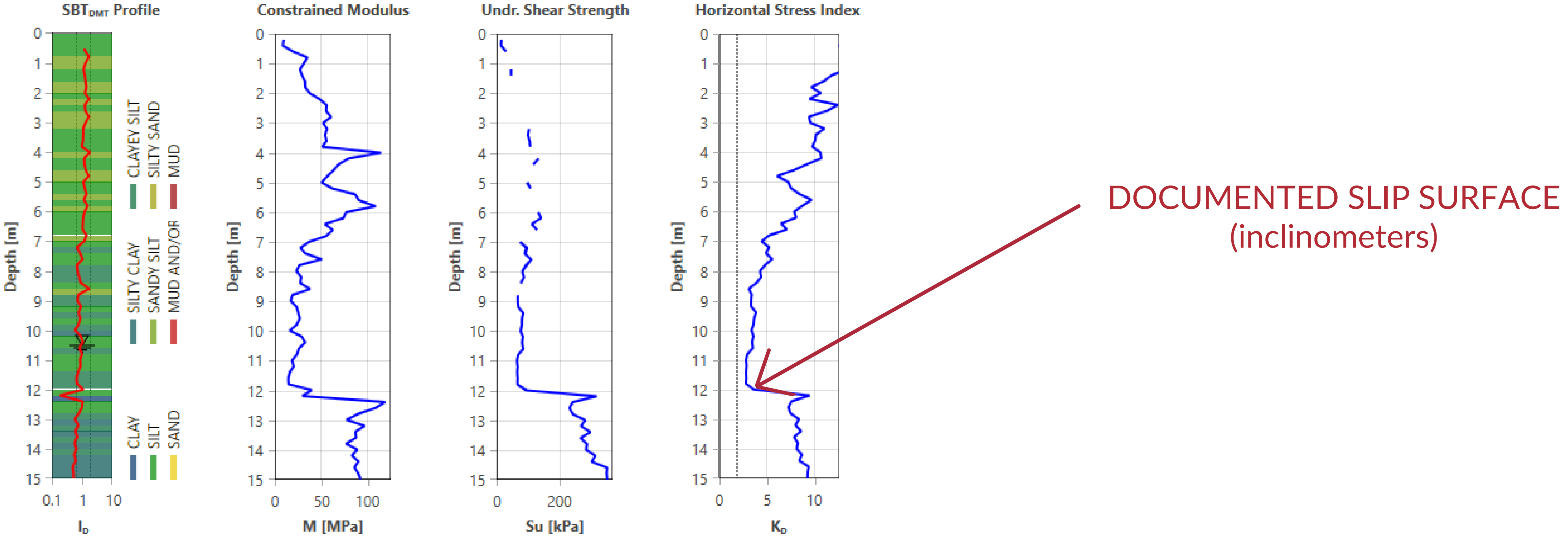
(Totani et al. 1997)

DMT-K_D method for detecting slip surfaces in OC clay slopes

- Use K_D = 2 as identifier of zones of NC clay in a slope which otherwise exhibits an OC profile (K_D >> 2)
- Specific value K_D = 2 (not simply "weak zones " with low K_D)
- Detects also quiescent slip surfaces which could reactivate (inclinometers cannot as movement is required)
- But cannot establish if slope is now moving and how fast (inclinometers can)
- In many cases DMT + inclinometers used in combination (e.g. use K_D profiles to optimize location/depth of inclinometers)

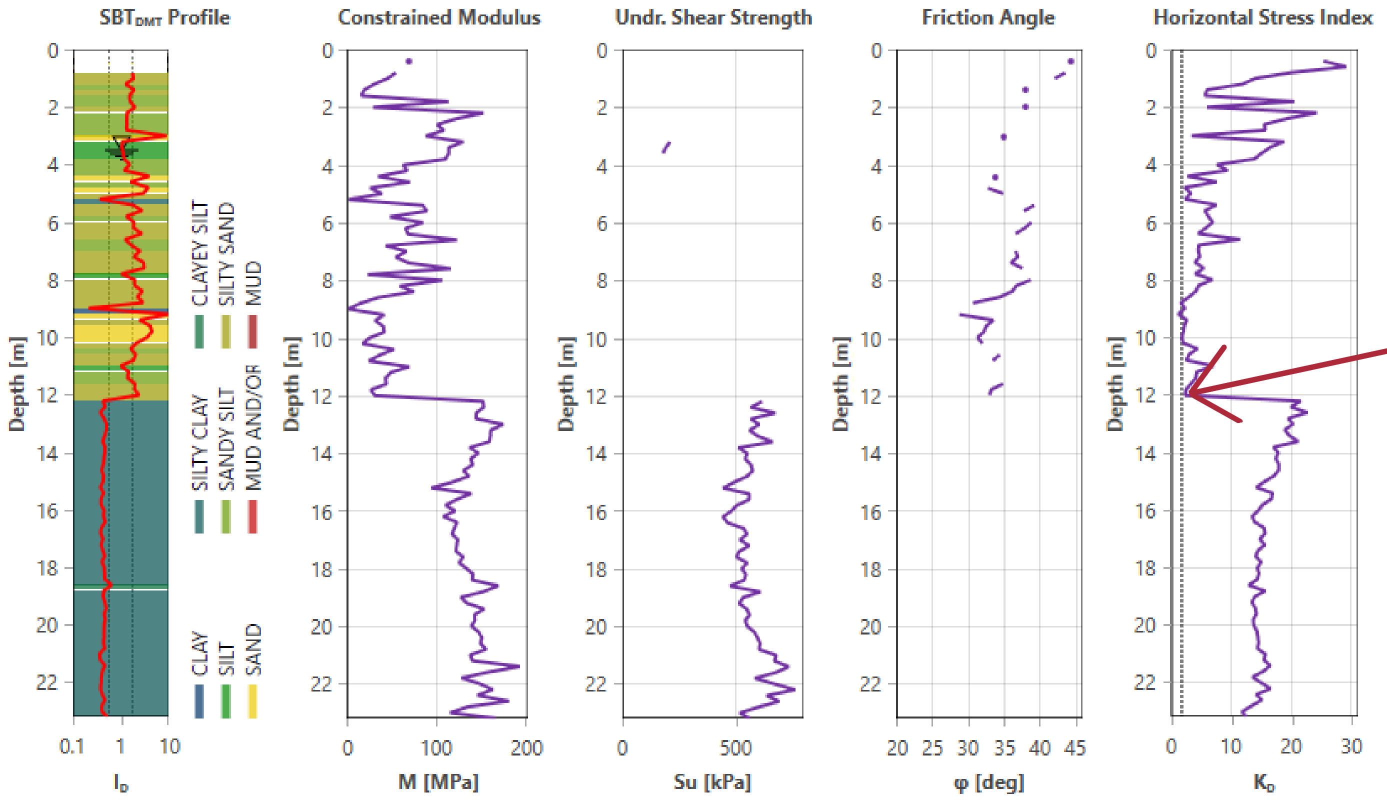
Slip Surface detection (case history 2)

Landslide ‘Filippone’ (Chieti 1997)



Slip Surface detection (case history 2)

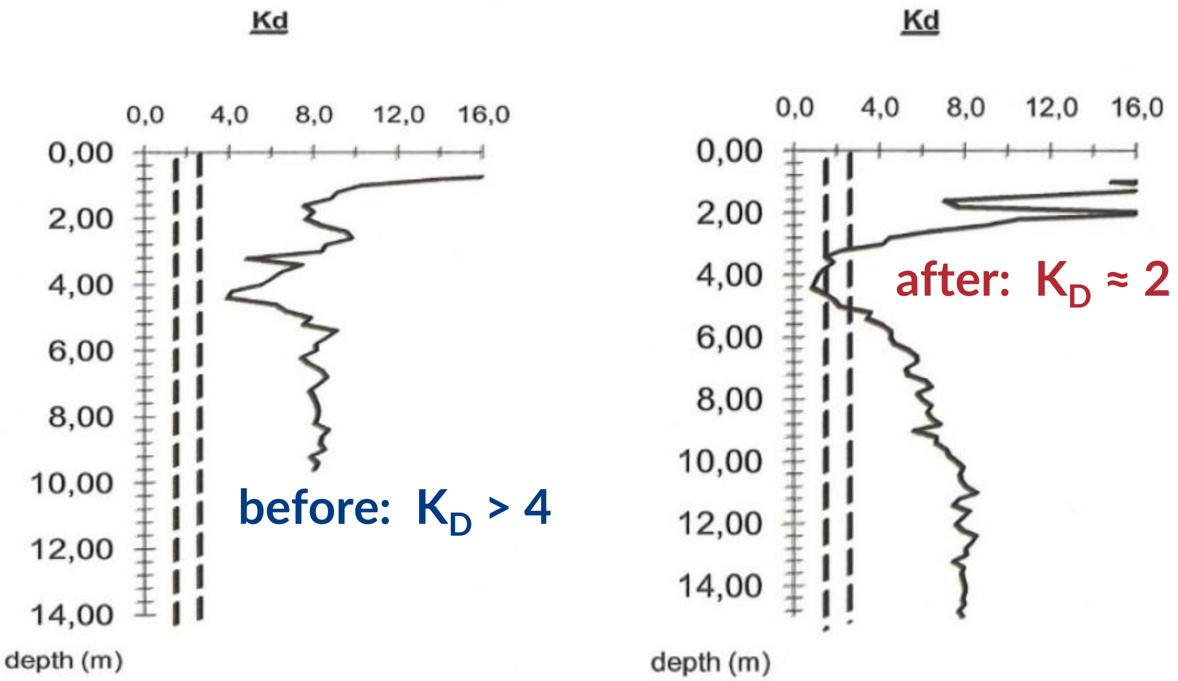
Landslide ‘St. Barbara’ (AR)



DOCUMENTED SLIP SURFACE

K_D to detect slip surface

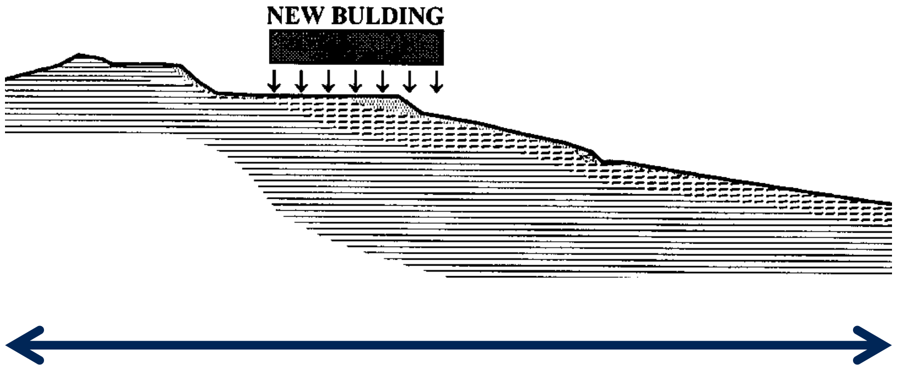
Inspection of K_D profile before and after the landslide



Peiffer, 2016 - ISC'5 Conf

Chieti Hill - Italy

SAFETY PROBLEM: heavy NEW BUILDING to be constructed on OC CLAY SLOPE

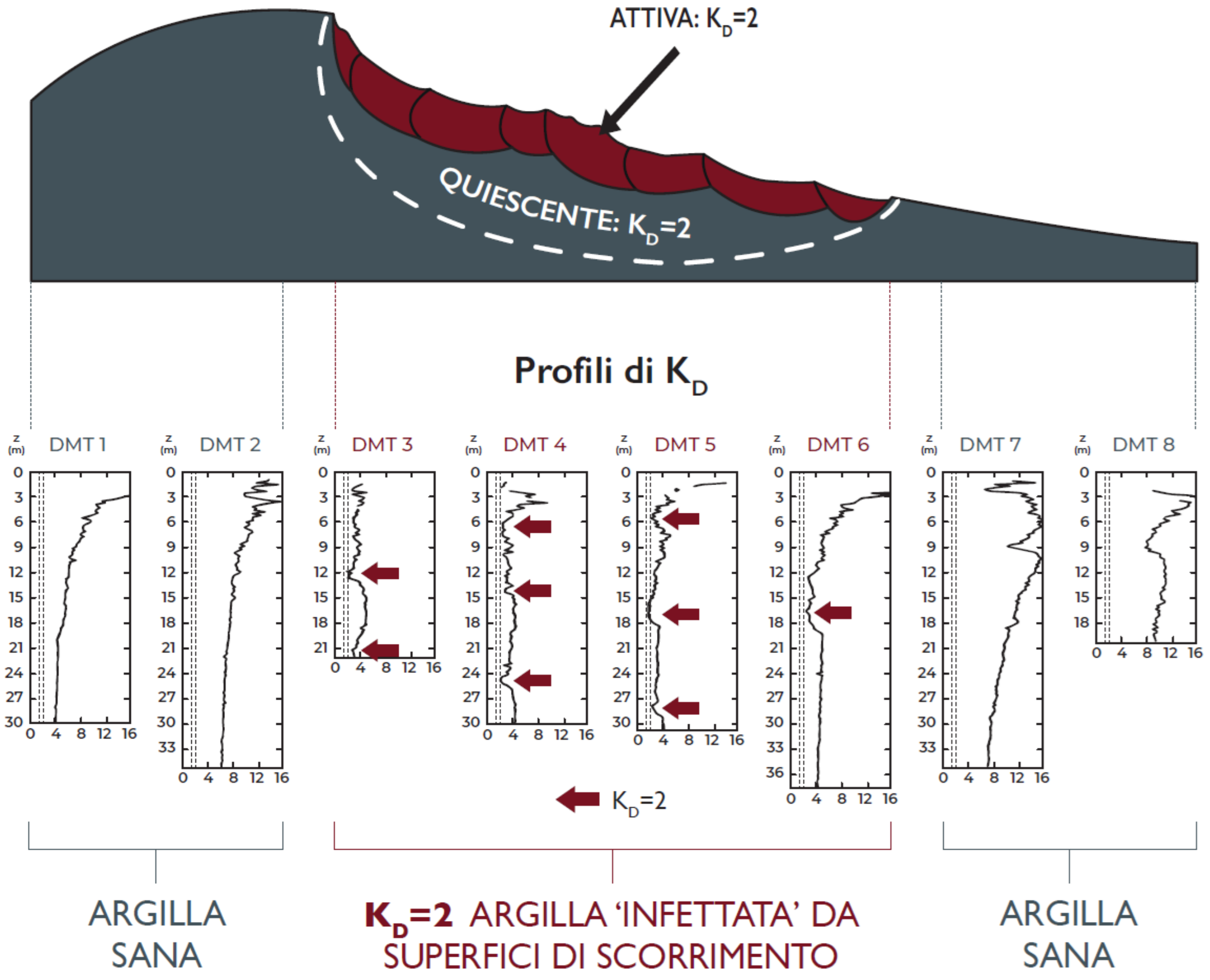


spaced DMT tests

Slope stability assessment:

Detect past and present shear planes inspecting K_D profiles

Reconstruction of multiple slip surfaces in Chieti Hill



Chieti Hill project in Rankine Lecture (Leroueil 2001)

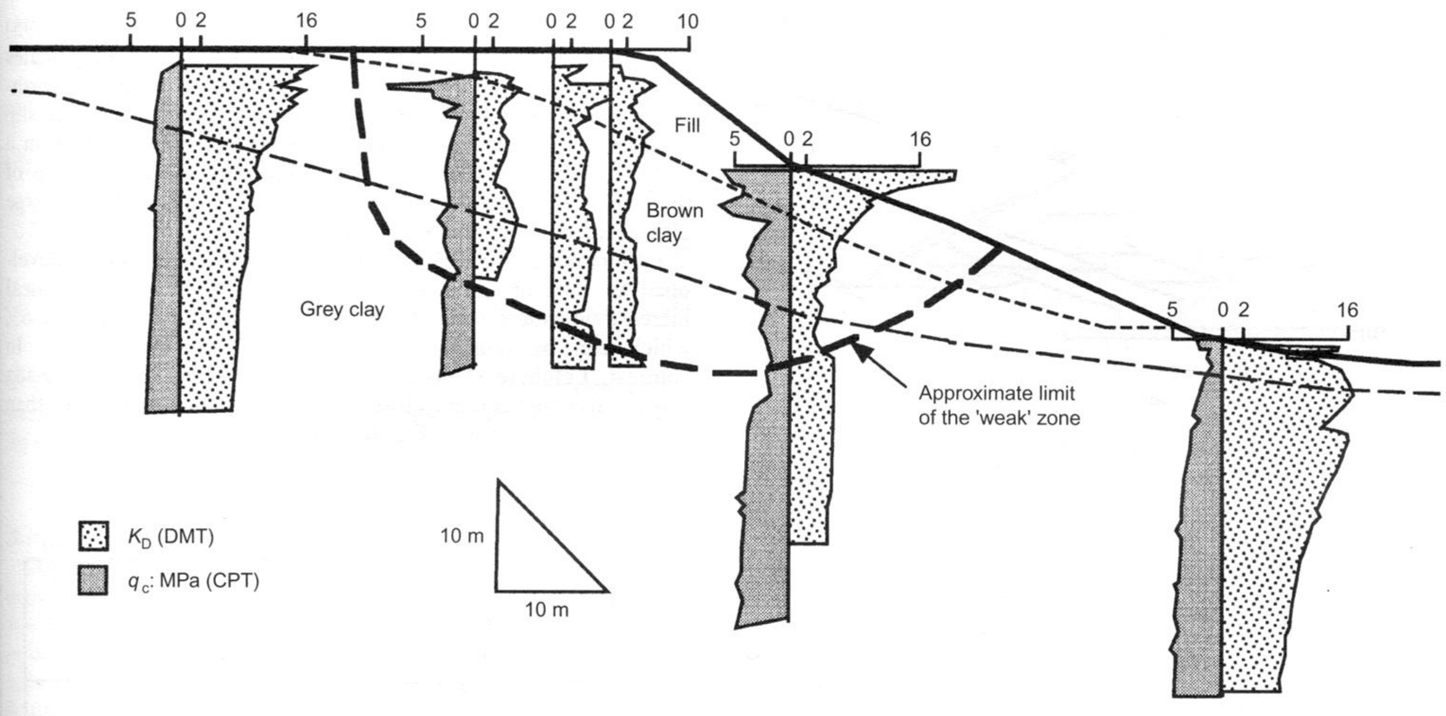


Fig. 58. CPT (q_c) and DMT (K_D) profiles at Chieti, Italy (after Totani *et al.*, 1997)

Leroueil (2001) *"Natural slopes and cuts: movement and failure mechanisms". Rankine Lecture, Géothecnique, 51(3)*

DMT for Liquefaction Resistance

Simplified Procedure (Seed & Idriss 1971)

Liquefaction occurs if: CRR ≤ CSR

CSR (Cyclic Stress Ratio) = τ_{av} / σ'_{v0} = $\mathbf{0.65\,(a_{max} / g)\,(\sigma_{v0} / \sigma'_{v0})\,r_d}$

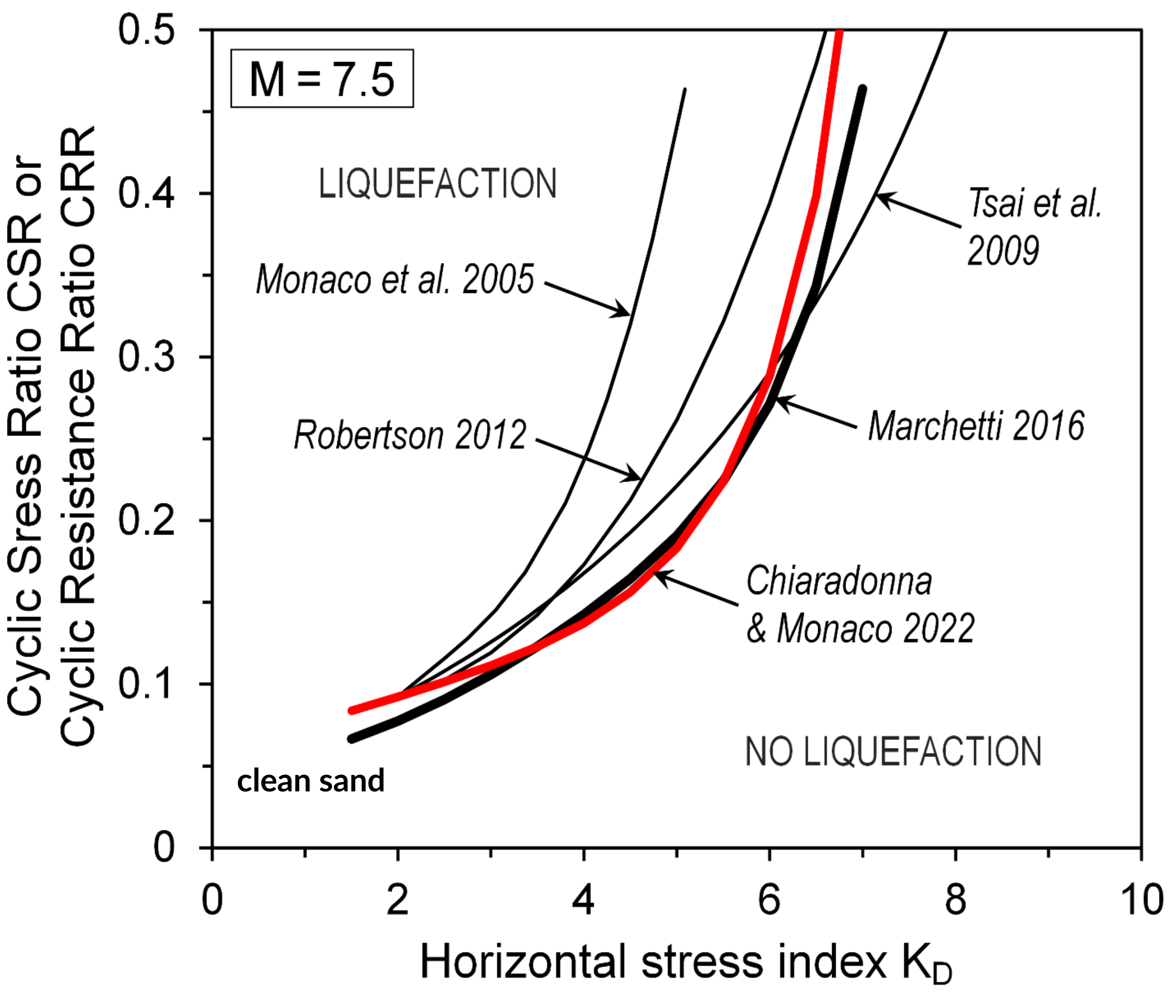
CRR (Cyclic Resistance Ratio) estimated from CPT, SPT, Vs, DMT, Lab

Youd et al. (2001) *"Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils". ASCE J. GGE 127(10)*

Idriss & Boulanger (2008) *"Soil liquefaction during earthquakes". EERI Report No. MNO-12, Earthquake Engineering Research Institute*

...

CRR-K_D most recent curves (2005-2022)



Monaco et al. (2005)

$$CRR = 0.0107\,K_D^{\,3} - 0.0741\,K_D^{\,2} + 0.2169\,K_D - 0.1306$$

Tsai et al. (2009)

$$CRR = exp\left[\left(\frac{K_D}{8.8}\right)^3 - \left(\frac{K_D}{6.5}\right)^2 + \left(\frac{K_D}{2.5}\right) - 3.1\right]$$

Robertson (2012)

$$CRR = 93\,(0.025\,K_D)^3 + 0.08$$

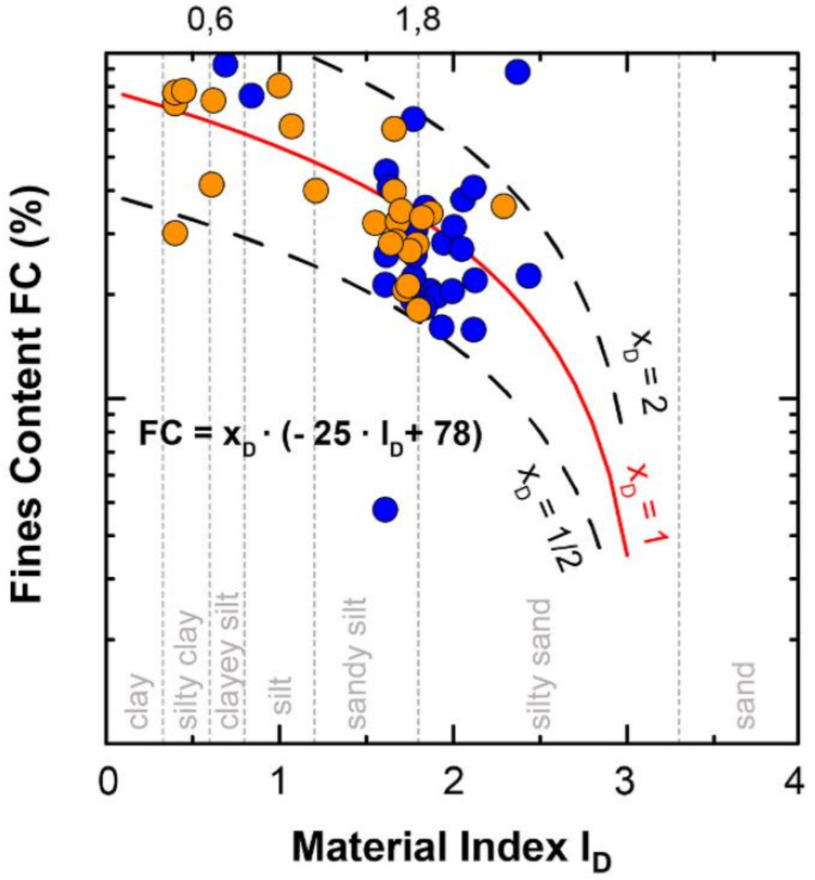
Marchetti (2016)

$$\begin{aligned}\mathrm{CRR} = \exp[&(Q_{cn}/540) + (Q_{cn}/67)^2 - (Q_{cn}/80)^3 \\ &+ (Q_{cn}/114)^4 - 3]\end{aligned} \quad \text{with} \quad Q_{cn} = 25K_D$$

Chiaradonna & Monaco (2022)

$$CRR = \exp(0.001109K_D^4 - 0.00569K_D^3 + 0.000625K_D^2 + 0.221K_D - 2.8)$$

Estimations of FC from DMT results (I_D)

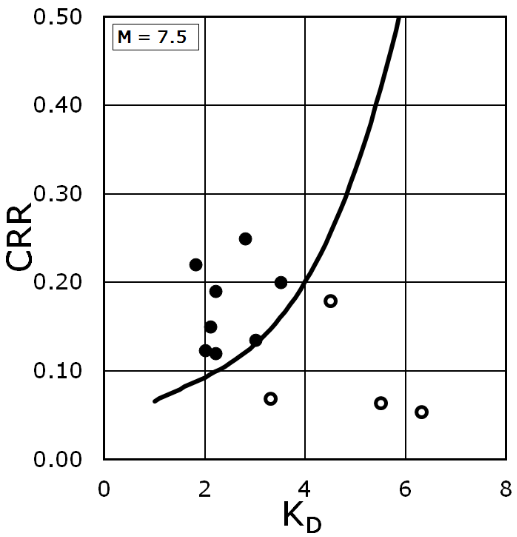


- Di Buccio et al. (2023) – first attempt of estimating FC from I_D
- Research in progress on sensitivity of other parameters (U_D, K_D , E_D) to the FC estimation

SDMT for LIQUEFACTION

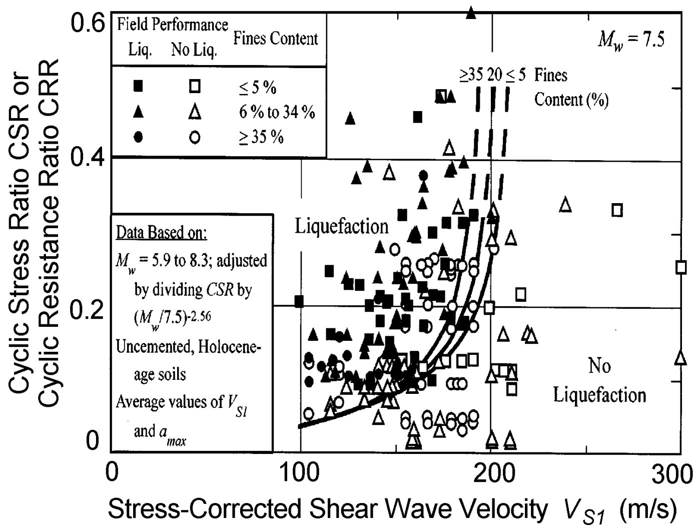
SDMT provides 2 independent evaluations of CRR

CRR from K_D



Marchetti ASCE (2013)
Chiaradonna & Monaco (2022)

CRR from V_s



Andrus & Stokoe (2000)
Andrus et al. (2004)

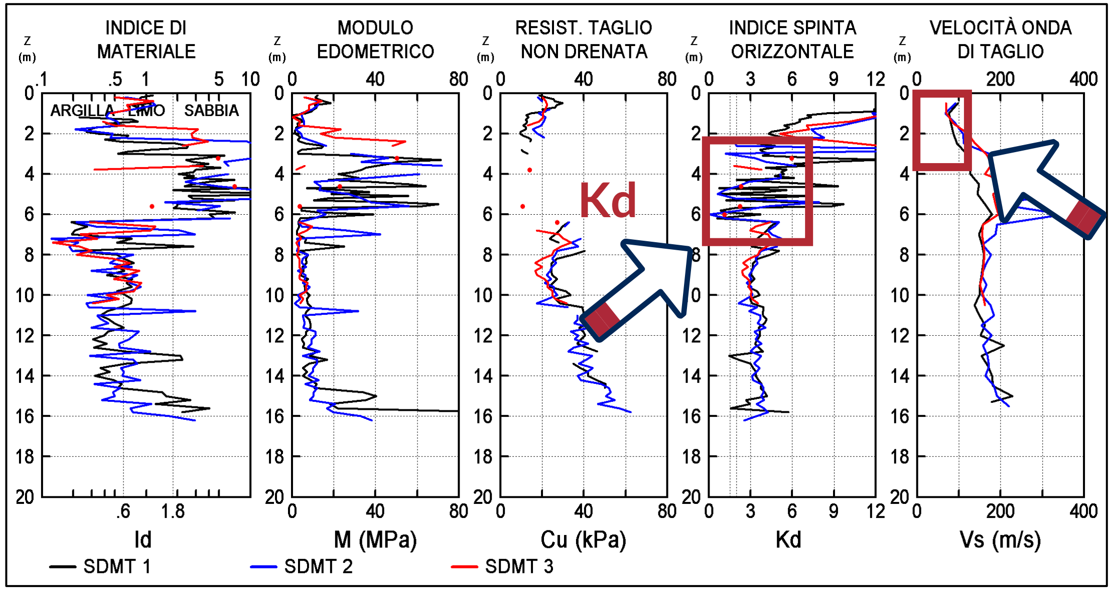
... Two independent methods for evaluating CRR

Liquefaction assessment using SDMT (case history 1)

Moment magnitude MW: 6.3
Distance from the epicentre: 45 km
Peak ground acceleration PGA: 0.065 g

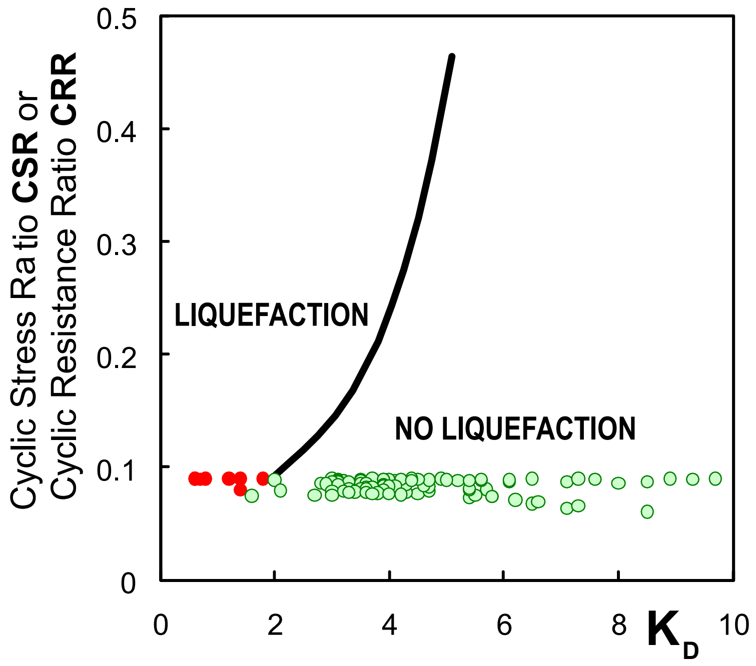
CSR

Vittorito – L'Aquila (April 2009)

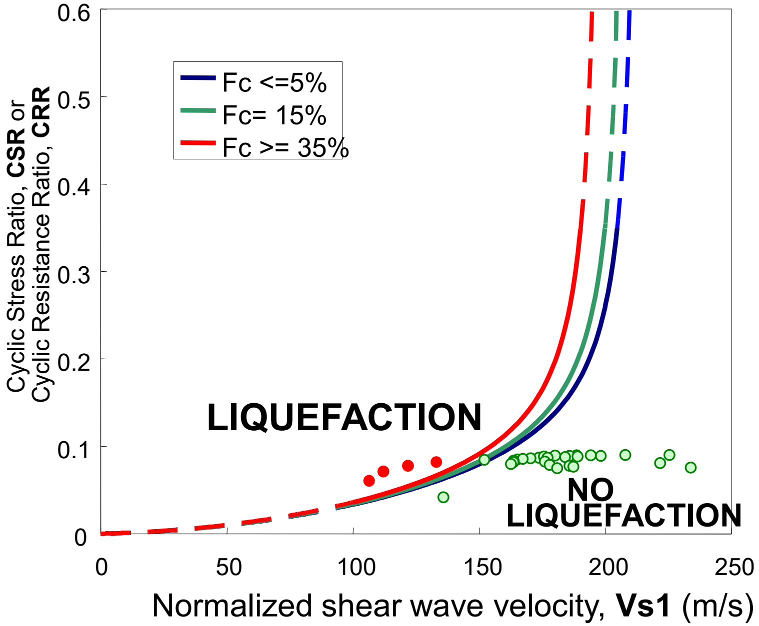


Liquefaction assessment using SDMT (case history 1)

Liquefaction depth from K_D: 2-6 m



Liquefaction depth from V_s: 1-2.5 m

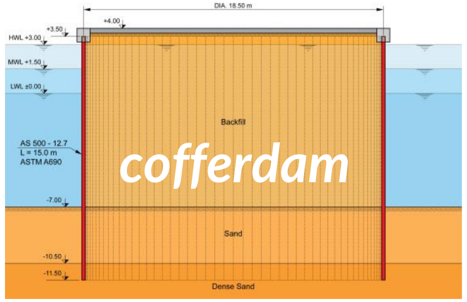


Both K_D and V_s indicate Liquefaction potential between 2.0-2.5 m

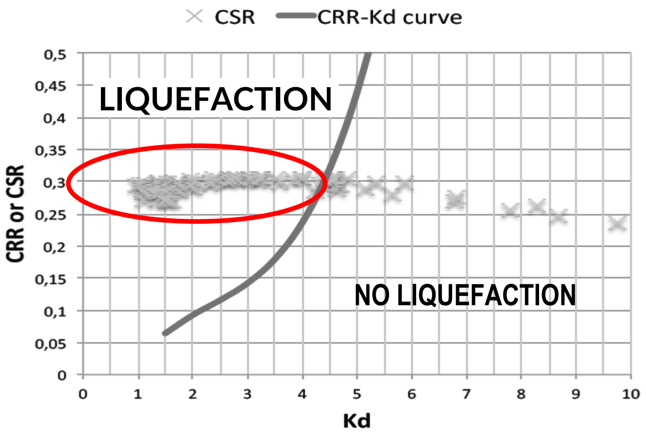
Vittorito – L'Aquila (April 2009)

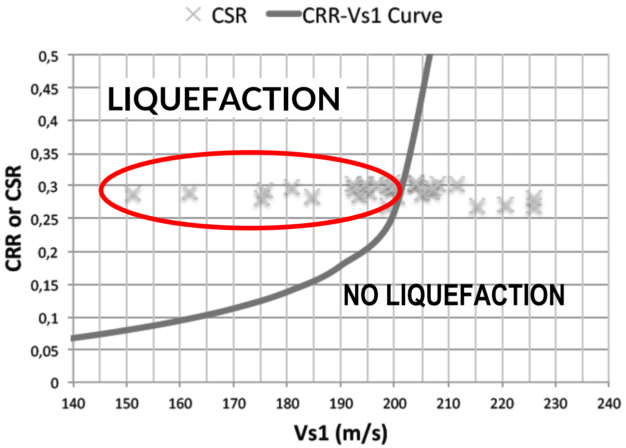
Monaco et al. (2009, 2010)

Liquefaction case history in Costarica



Design Earthquake (M Richter = 7,5 and PGA = 0,25 g)

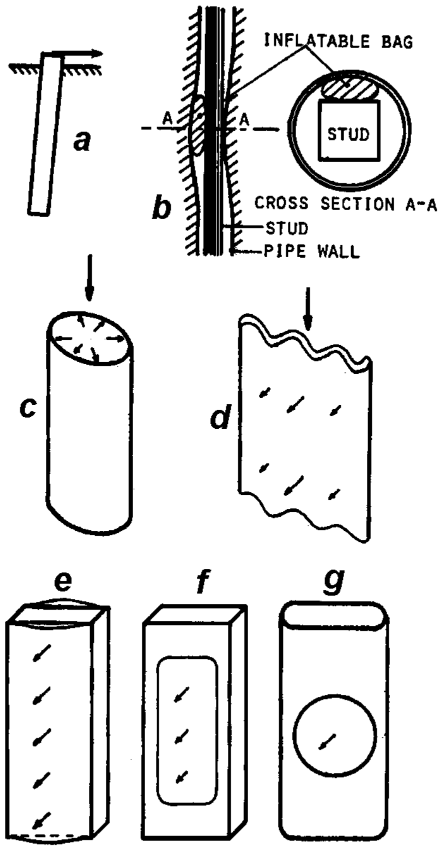




"Just a few weeks after the SDMT execution, the cyclic wave action due to a storm induced liquefaction of the soil deposit.." (Vargas & Coto 2012)

Design of Laterally Loaded Piles (LLP - PY curves)

Background



Original stimulus for development of DMT:

Tools for measuring parameters for the design of **Laterally Loaded Piles**

Marchetti (1977) *"Devices for In Situ Determination of Soil Modulus Es" Proceedings of 9th ICSMFE, Tokyo 1977*

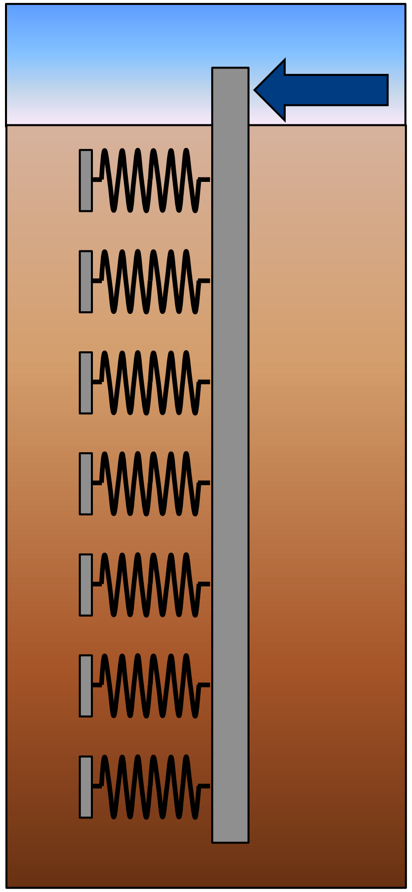
Similarity DMT – LL PILES

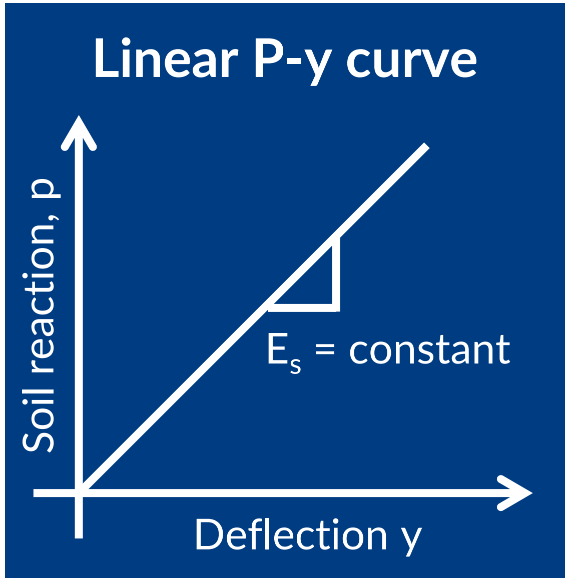
- Structural elements installed in soil
- LATERAL deformation

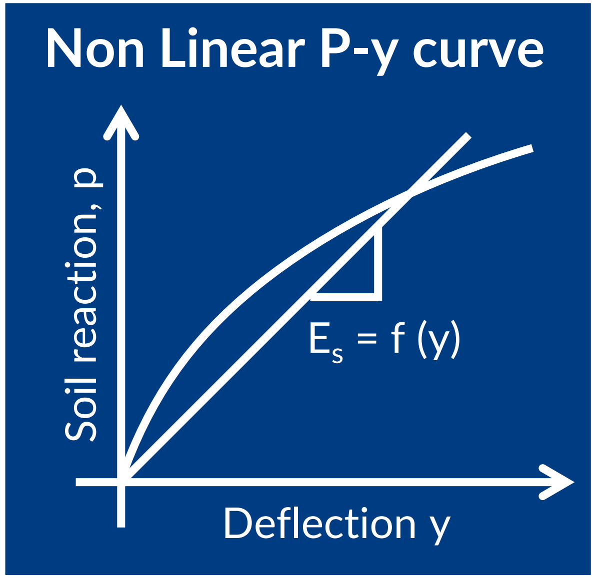
Introduction to LLP and P-Y Curves

- LLP analysis is generally more complex than for axially loaded piles, because of the nonlinear and heterogeneous nature of soil response
- Numerical or semi-empirical methods based on P–Y curve approach are widely adopted
- P-Y curves approach (Hetenyi 1946): soil is modeled with a series of independent and nonlinear springs (Winkler model), where the lateral soil reaction (P) at each depth is expressed as a function of the local lateral deflection (Y)
- Three main methods were developed to estimate P-Y curves from DMT results :
  Gabr and Borden (1988), Robertson (1989), Marchetti (1991)
- Prediction of the two latter methods are compared with measured deflections in the case history LLP test site of Livorno (Italy 1991)

Design of laterally loaded piles (Winkler)







Three different methods using DMT results for evaluating P-y curves for laterally loaded piles

- Gabr & Borden (1988)
- Robertson et al. (1989)
- Marchetti et al. (1991)

→ recommended methods

Gabr and Borden (1988) method

$$\frac{\mathbf{P}}{\mathbf{P_{ult}}} = \mathbf{tanh}\left(\frac{\boldsymbol{E_{s1}\,y}}{\mathbf{P_{ult}}}\right)$$

Ultimate soil resistance P_{ult}

$$P_{ult} = N_p S_u d \qquad N_p = 3 + \frac{\sigma'_{v0}}{S_u} + J\left(\frac{Z}{d}\right) \leq 9$$

N_p = non dimensional ultimate clay resistance coefficient
S_u = undrained shear strength from DMT
σ'_{v0} = effective vertical stress at depth Z
J = empirical coefficient; 0.5 for soft clay, 0.25 for stiff clay
Z = depth
d = pile diameter

Initial Tangent Modulus E_{s1}

$$E_{s1} = 6.5\left(\frac{P_o - \sigma_{h0}}{b}\right) d$$

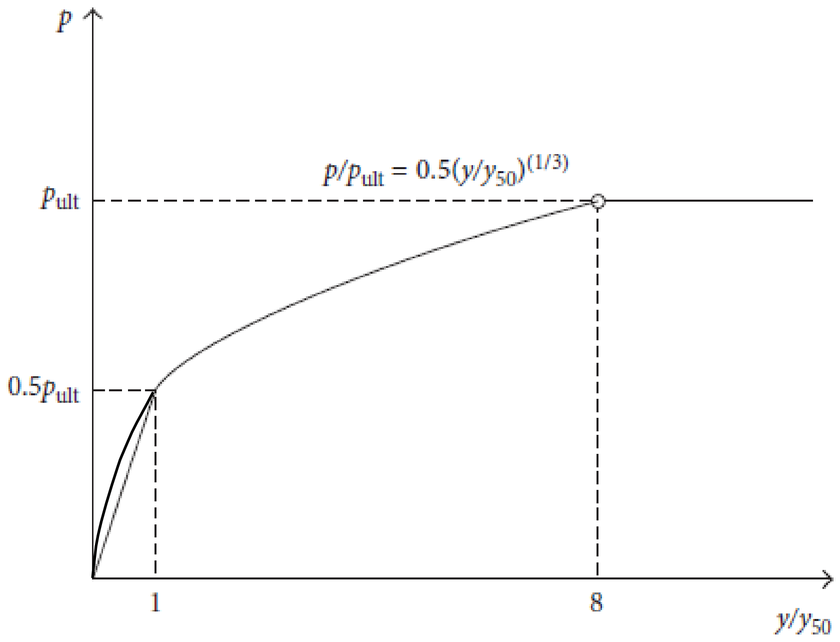
P_o first DMT reading
σ_{h0} in-situ horizontal at-rest pressure
b half blade thickness (7 mm)
d pile diameter

Gabr and Borden (1988) method

Comments:

- First method for P-Y curves from DMT data
- Developed specifically for stiff OC clay and tends to overpredict deflections in non OC clays
- Initial tangent modulus E_{S1} based only on P_0, related to total horizontal stress and not to soil compressibility, which is available from DMT: $E_D = f\,(P_1 - P_0)$

Robertson 1989 method



Parameters:

p : soil reaction

y : pile deflection

p_{ult} : ultimate soil resistance

p/p_{ult} : ratio of soil resistance

y_{50} : pile deflection for $p = 50\%\ p_{ult}$

y/y_{50} : ratio of pile deflection

- Adaptation of early Matlock 1970 method: $y/y_{50} \geq 8 \rightarrow p/p_{ult} = 1$ (asymptot $p = p_{ult}$)
- Applicable in OC and NC clay and based on Su
- Applicable in sand (based on φ)

Robertson 1989 method

$$\frac{\mathbf{P}}{\mathbf{P_{ult}}} = \mathbf{0.5}\left(\frac{\mathbf{y}}{\mathbf{y_{50}}}\right)^{\mathbf{0.33}}$$

Ultimate soil resistance (P_{ult})

$$P_{ult} = N_p S_u d \qquad N_p = 3 + \frac{\sigma'_{v0}}{S_u} + J\left(\frac{Z}{d}\right) \leq 9$$

N_p = non dimensional ultimate clay resistance coefficient

S_u = undrained shear strength from DMT

σ'_{v0} = effective vertical stress at depth Z

J = empirical coefficient; 0.5 for soft clay, 0.25 for stiff clay

Z = depth

d = pile diameter

Reference pile deflection (y_{50})

$$y_{50} = \frac{23.67 k S_u (D^{0.5})}{F_c E_D}$$

F_c = 10 (empirical stiffness factor)

E_D = dilatometer modulus

S_u = undrained shear strength from DMT

k reduction factor to capture the effects of pile installation

$$k = \begin{cases} 1, \ OCR \leq 1 \\ \frac{4}{3} - \frac{OCR}{3}, 1 \leq OCR \leq 2 \\ \frac{2}{3}, OCR > 2 \end{cases}$$

LLP Livorno case history with Robertson 1989 method

- DMT data exported from SDMT Pro (Z, σ'_{v0}, E_D, Su, OCR)
- Evaluate all other parameters (k, Np, Pult, Y50)

Pile Length [m]	18
d [m]	0.5
Fc	10

Z	σ'v0	E_D	Su	OCR	k	Np	P_{ult}	Y50
[m]	[kPa]	[MPa]	[kPa]	▯	▯	▯	[kN/m]	[m]
0.8	14	6.2	38	2.40	0.67	4.17	52.80	0.00684
1	17	5.6	37	2.10	0.67	4.46	55.28	0.00741
1.2	20	5.3	37	1.90	0.70	4.74	61.39	0.00818
1.4	24	5.5	33	1.70	0.77	5.13	64.86	0.00770
1.6	27	4.0	29	1.50	0.83	5.53	66.83	0.01011
1.8	30	2.4	28	1.50	0.83	5.87	68.50	0.01627
2	33	3.4	34	1.50	0.83	5.97	84.58	0.01395
2.2	37	3.7	44	1.60	0.80	6.04	106.32	0.01592
2.4	40	6.0	49	1.60	0.80	6.22	121.84	0.01093
2.6	43	5.1	35	1.30	0.90	6.83	107.55	0.01034
2.8	47	3.1	32	1.20	0.93	7.27	108.55	0.01613
3	50	3.0	29	1.00	1.00	7.72	112.00	0.01618
3.2	53	2.8	25	0.93	1.00	8.32	104.00	0.01494
3.4	56	1.7	22	0.82	1.00	8.95	98.40	0.02166
3.6	59	1.5	21	0.77	1.00	9.00	94.50	0.02343
3.8	62	0.9	22	0.76	1.00	9.00	99.00	0.04091
4	65	0.6	21	0.73	1.00	9.00	94.50	0.05858
4.2	66	1.1	20	0.69	1.00	9.00	90.00	0.03043
4.4	67	0.8	21	0.70	1.00	9.00	94.50	0.04394
4.6	68	0.8	21	0.70	1.00	9.00	94.50	0.04394
4.8	69	1.3	20	0.68	1.00	9.00	90.00	0.02575
5	70	1.1	21	0.68	1.00	9.00	94.50	0.03195

LLP Livorno case history with Robertson 1989 method

Evaluate P-Y values: calculation every 1 m depth and selection of Y/Y50 as in the table below

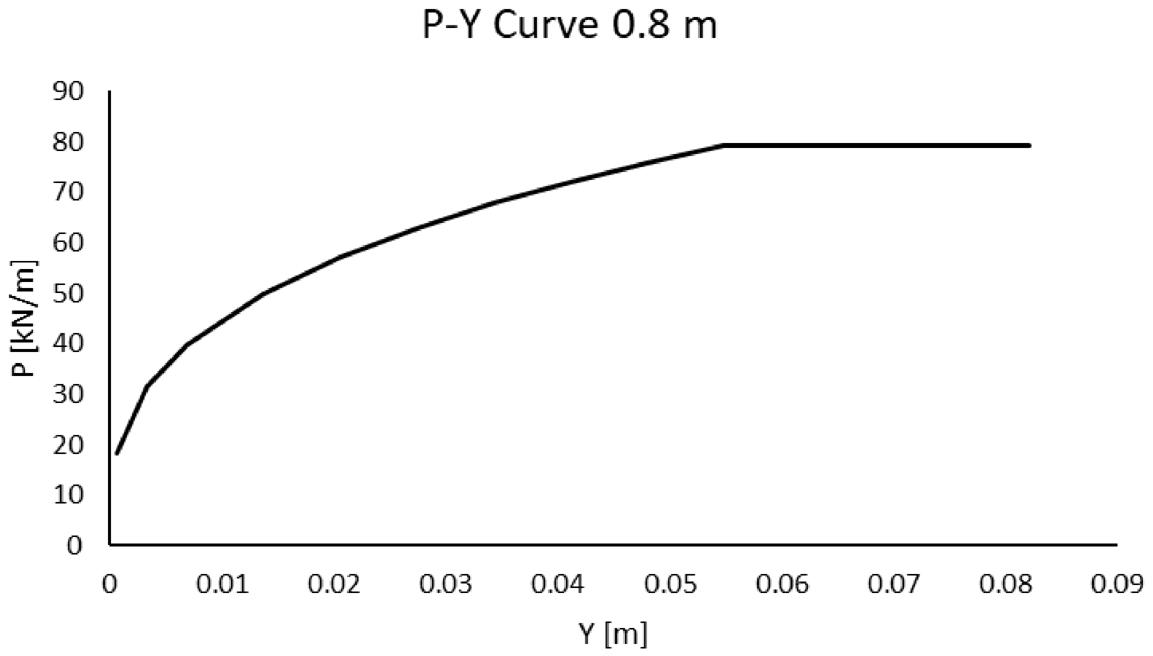
P-Y Graph

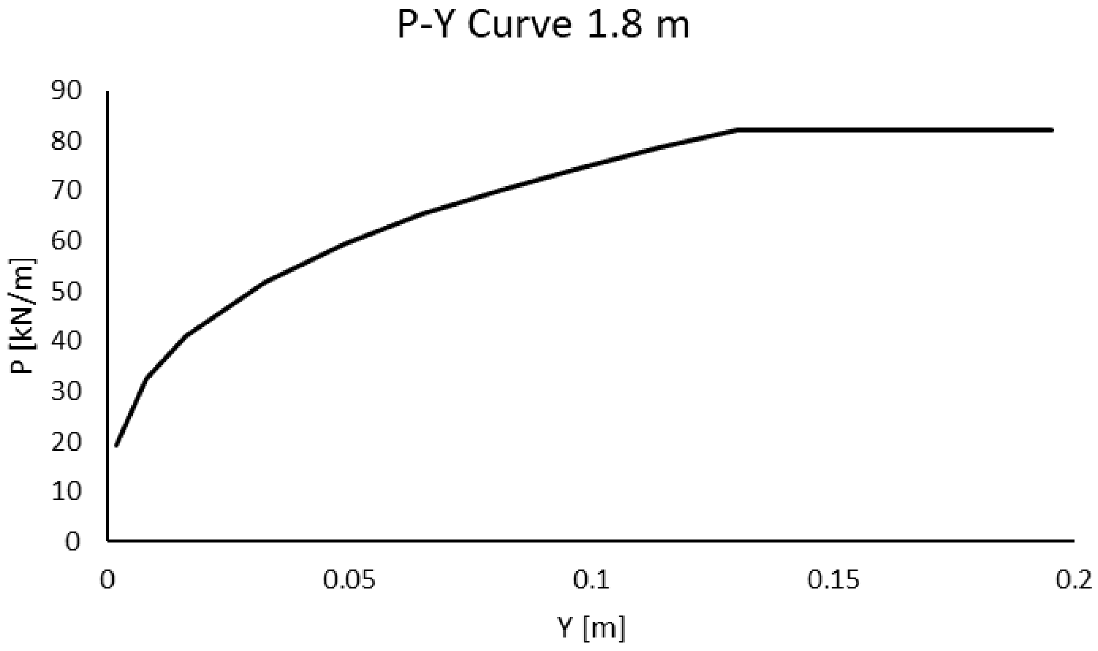
Depth [m]	Y/Y50	y [m]	P/Pu	P [kN/m]
0.8	0.1	0.00068	0.23	18.38
	0.5	0.00342	0.40	31.43
	1	0.00684	0.50	39.60
	2	0.01368	0.63	49.89
	3	0.02052	0.72	57.11
	4	0.02736	0.79	62.86
	5	0.03419	0.85	67.72
	6	0.04103	0.91	71.96
	7	0.04787	0.96	75.75
	8	0.05471	1	79.20
	9	0.06155	1	79.20
	10	0.06839	1	79.20
	11	0.07523	1	79.20
	12	0.08207	1	79.20

Depth [m]	Y/Y50	y [m]	P/Pu	P [kN/m]
1.8	0.1	0.00163	0.23	19.08
	0.5	0.00814	0.40	32.62
	1	0.01627	0.50	41.10
	2	0.03254	0.63	51.78
	3	0.04882	0.72	59.28
	4	0.06509	0.79	65.24
	5	0.08136	0.85	70.28
	6	0.09763	0.91	74.68
	7	0.11391	0.96	78.62
	8	0.13018	1	82.20
	9	0.14645	1	82.20
	10	0.16272	1	82.20
	11	0.179	1	82.20
	12	0.19527	1	82.20

LLP Livorno case history with Robertson 1989 method

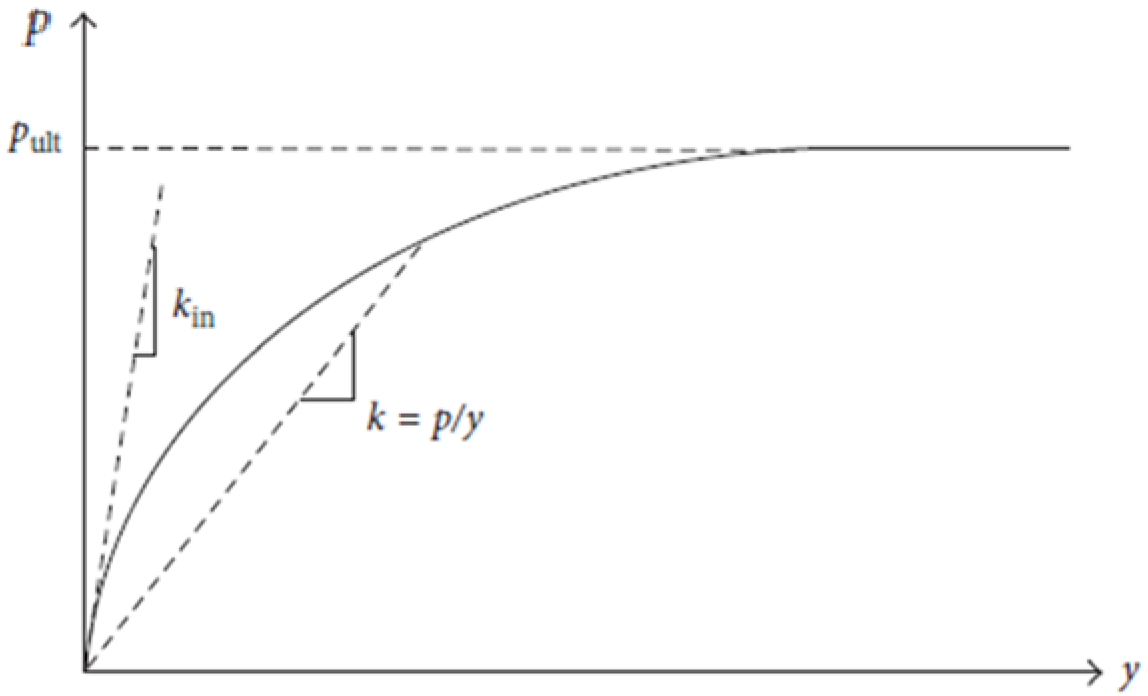
Optional plots of P-Y curves every 1 m





Marchetti 1991 method

This method **simplifies** previous methodologies and **improves** them by adopting tangent hyperbolic curve (Gabr and Borden 1988) and DMT compressibility parameter E_D (Robertson 1989)



Parameters:

- p : soil reaction
- y : pile deflection
- p_{ult} : ultimate soil resistance

Marchetti 1991 method

$$\frac{\mathbf{P}}{\mathbf{P_{ult}}} = \mathbf{tanh}\left(\frac{\boldsymbol{E_{s1}\,y}}{\mathbf{P_{ult}}}\right)$$

Ultimate soil resistance (P_{ult})

$P_{ult} = \alpha\ K1\ (P_0 - U_0)\ d$

$\alpha = \text{reduction factor} = \frac{1}{3} + \frac{2}{3}\left(\frac{Z}{7d}\right) \leq 1$

$K1 = 1.24$

$Z = \text{depth},\ d = \text{pile diameter}$

P_o = Corrected DMT-A reading

U_0=in-situ hydrostatic pressure

Initial Tangent Modulus (E_{s1})

$E_{s1} = \alpha\ K2\ E_D$

E_{s1}= initial tangent soil modulus,

$K2 = 10\left(\frac{d}{0.5}\right)^{0.5}$

E_D= dilatometer modulus

y/d ≤ 4%, for y/d > 4% $P=P_{ult}$

LLP Livorno case history with Marchetti 1991 method

- DMT data exported from SDMT Pro (Z, P_0, U_0, E_D)
- Evaluate or input all other parameters (K_1=1.24, K_2, α, P_{ult}, E_{s1})

Pile Length [m]	18
d [m]	0.5
K1	1.24
K2	10

Z [m]	P_0	U_0	E_D	α	P_{ult}	E_{s1}
[m]	[kPa]	[kPa]	[MPa]	[]	[kN/m]	[kPa]
0.8	206	0	6.2	0.49	62.04	30114
1	211	0	5.6	0.52	68.52	29333
1.2	223	0	5.3	0.56	77.69	29781
1.4	207	0	5.5	0.60	77.00	33000
1.6	194	0	4.0	0.64	76.75	25524
1.8	192	0	2.4	0.68	80.49	16229
2	225	0	3.4	0.71	99.64	24286
2.2	285	0	3.7	0.75	132.95	27838
2.4	317	0	6.0	0.79	155.36	47429
2.6	243	0	5.1	0.83	124.83	42257
2.8	231	0	3.1	0.87	124.12	26867
3	216	0	3.0	0.90	121.17	27143
3.2	196	0	2.8	0.94	114.58	26400
3.4	178	0	1.7	0.98	108.26	16676
3.6	173	0	1.5	1.00	107.26	15000
3.8	179	0	0.9	1.00	110.98	9000
4	179	0	0.6	1.00	110.98	6000
4.2	173	2	1.1	1.00	106.02	11000
4.4	179	4	0.8	1.00	108.50	8000
4.6	184	6	0.8	1.00	110.36	8000
4.8	183	8	1.3	1.00	108.50	13000
5	188	10	1.1	1.00	110.36	11000

LLP Livorno case history with Marchetti 1991 method

Evaluate P-Y values: calculation every 1 m depth and selection of Y/D as in the table below

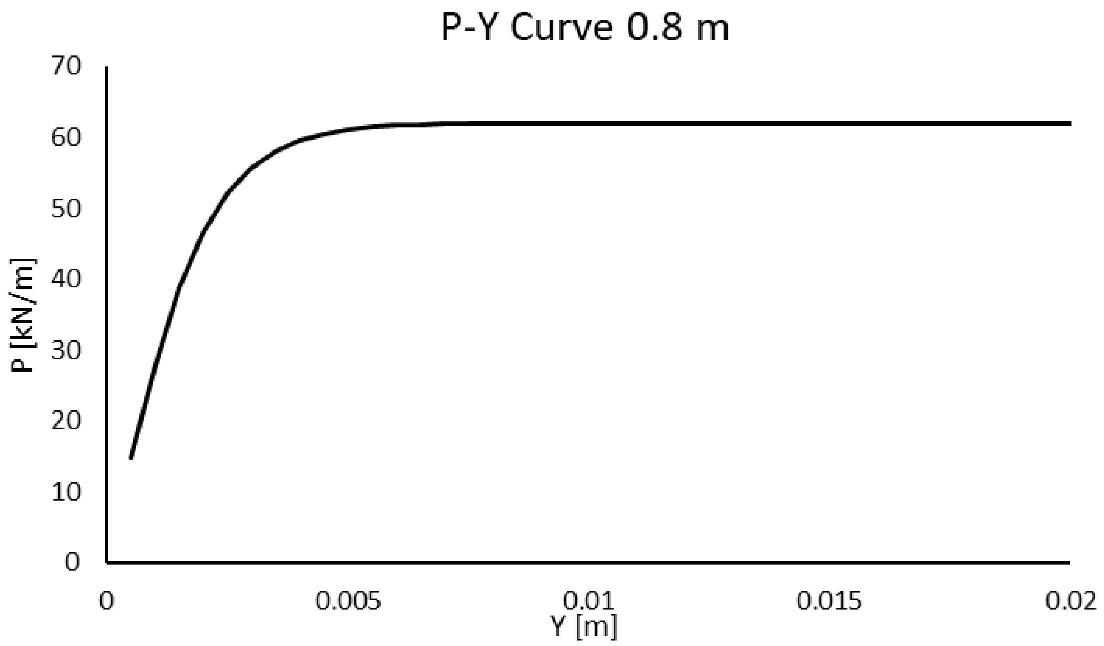
P-Y Graph

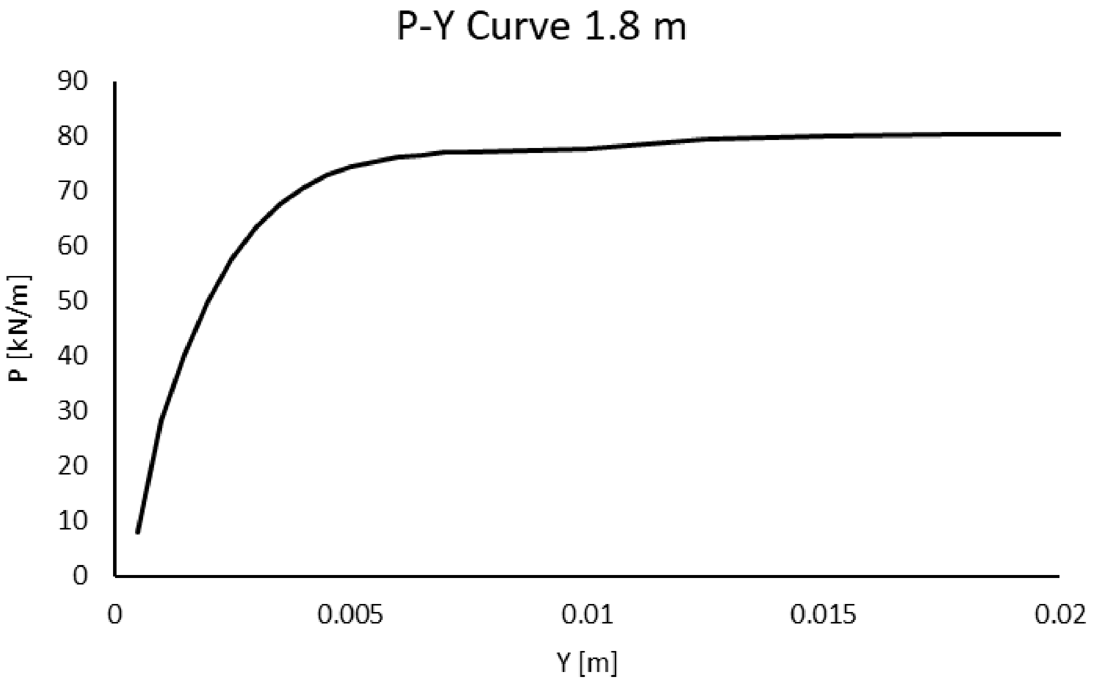
Depth [m]	Y/D [%]	y [m]	P [kN/m]
0.8	0.1	0.0005	14.77
	0.2	0.001	27.95
	0.3	0.0015	38.58
	0.4	0.002	46.47
	0.5	0.0025	51.97
	0.6	0.003	55.64
	0.7	0.0035	58.02
	0.8	0.004	59.53
	0.9	0.0045	60.48
	1	0.005	61.08
	1.1	0.0055	61.44
	1.2	0.006	61.67
	1.3	0.0065	61.81
	1.4	0.007	61.90
	1.5	0.0075	61.95
	2	0.01	62.03
	2.5	0.0125	62.03
	3	0.015	62.04
	3.5	0.0175	62.04
	4	0.02	62.04

Depth [m]	Y/D [%]	y [m]	P [kN/m]
1.8	0.1	0.0005	8.09
	0.2	0.001	16.01
	0.3	0.0015	23.63
	0.4	0.002	30.81
	0.5	0.0025	37.45
	0.6	0.003	43.51
	0.7	0.0035	48.94
	0.8	0.004	53.74
	0.9	0.0045	57.94
	1	0.005	61.57
	1.1	0.0055	64.69
	1.2	0.006	67.34
	1.3	0.0065	69.58
	1.4	0.007	71.46
	1.5	0.0075	73.03
	2	0.01	77.69
	2.5	0.0125	79.46
	3	0.015	80.11
	3.5	0.0175	80.36
	4	0.02	80.4431

LLP Livorno case history with Marchetti 1991 method

Optional plots of P-Y curves every 1 m





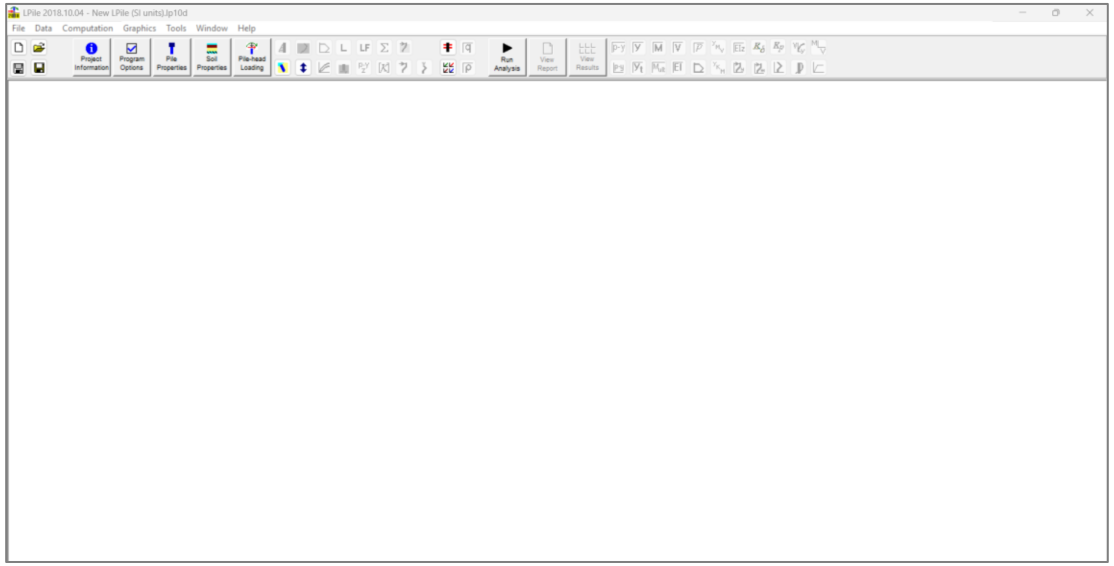
Details of LLP case history in Livorno (CH clay)

- Pile diameter = 0.5 m
- Pile length = 18 m
- Two different pile conditions:
  - Free head loading (kN): 100, 180, 220, 260
  - Fixed head loading (kN): 50, 100, 140, 180, 220, 260, 300
- Pile casing: ID = 495.4 mm, OD = 508 mm
- Compressive strength of concrete = 30 MPa
- Reinforcement steel:
  - 20 bars Φ 24 mm up to depth 5.35 m
  - 8 bars Φ 24 mm from depth 5.35 m to 18 m

Ensoft Lpi 2018 software for LLP - Input and Processing

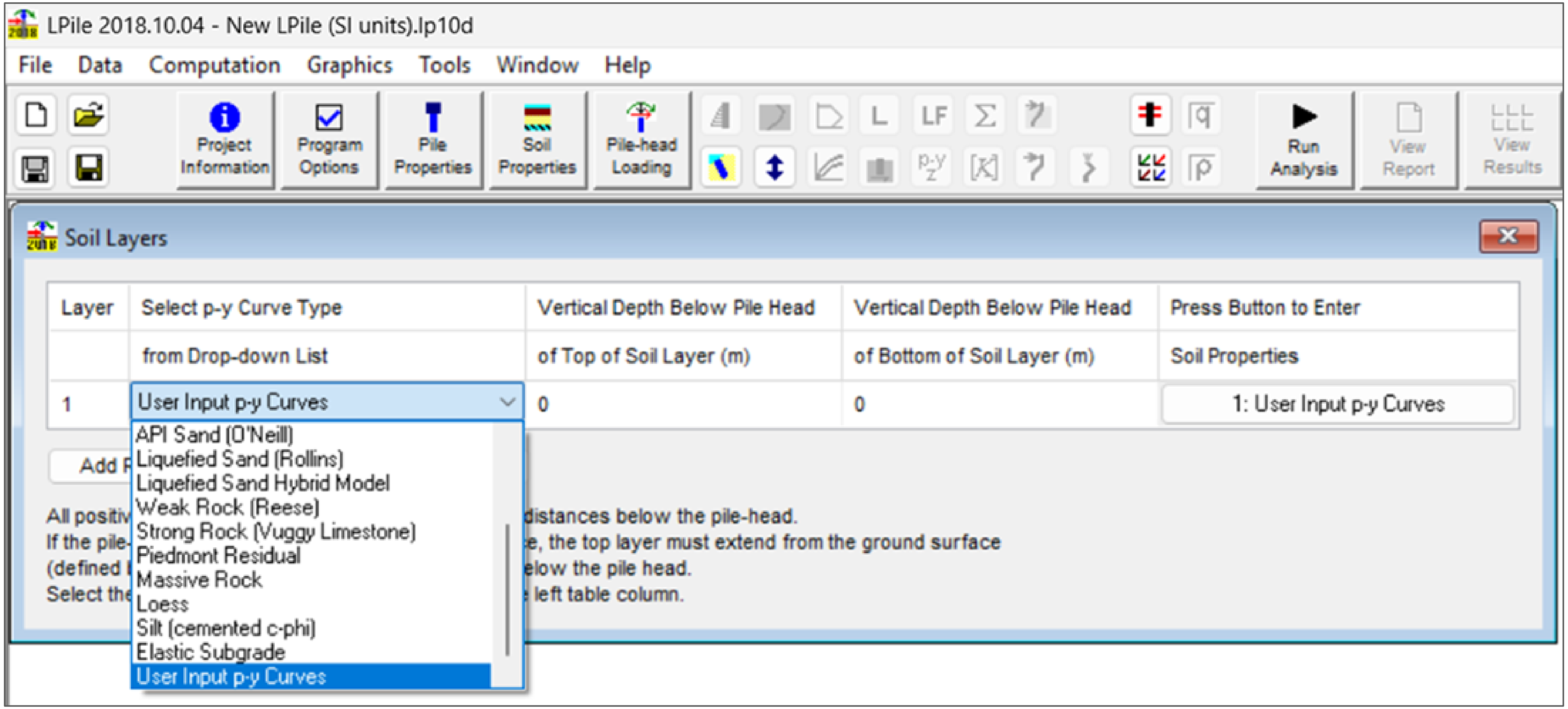
Software for LLP analysis using P-Y curves

- LLP analysis using Ensoft LPi 2018
- The freeware version of the software may be downloaded from https://getintopc.com/softwares/ensoft-lpi-2018-free-download/



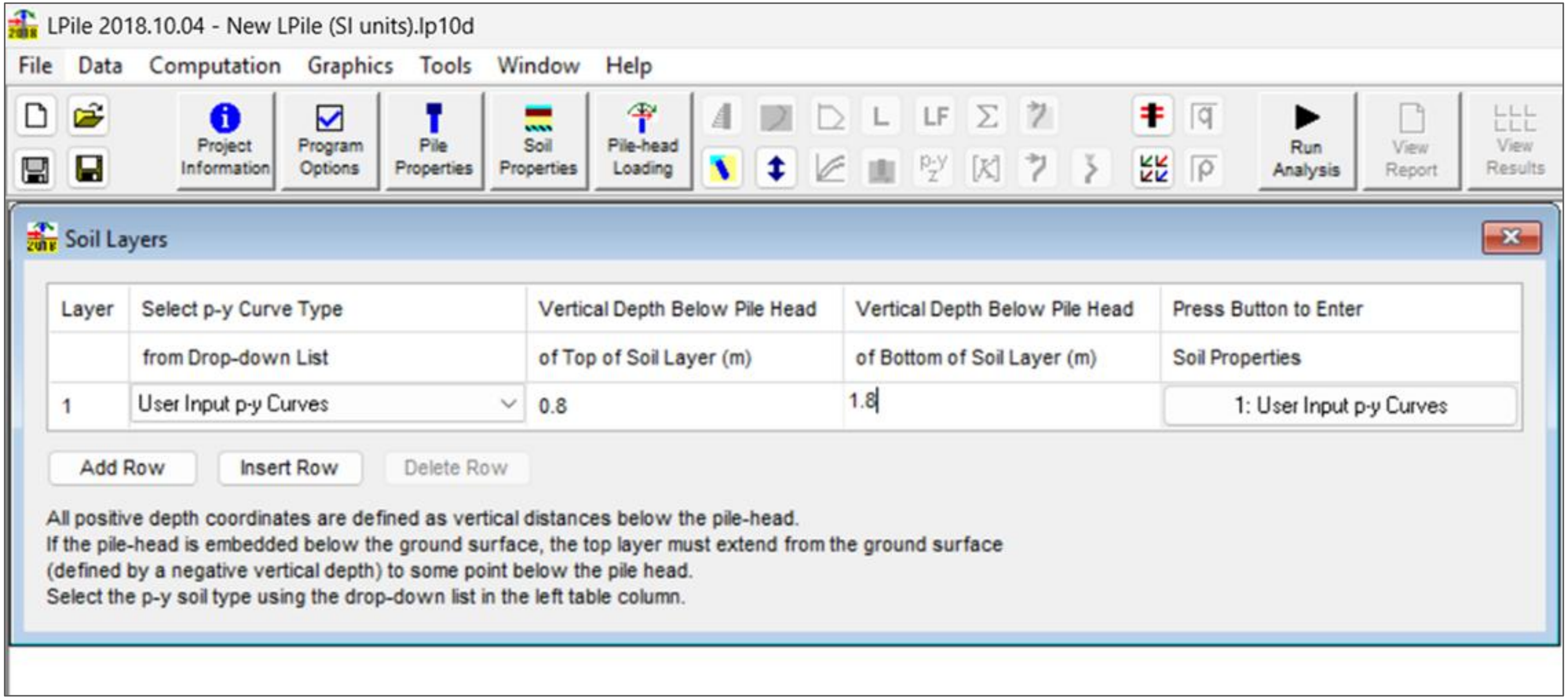
Input P-Y curves in Ensoft Lpi 2018 software

Step 1 Create the subsoil profile by using user input P-Y curves



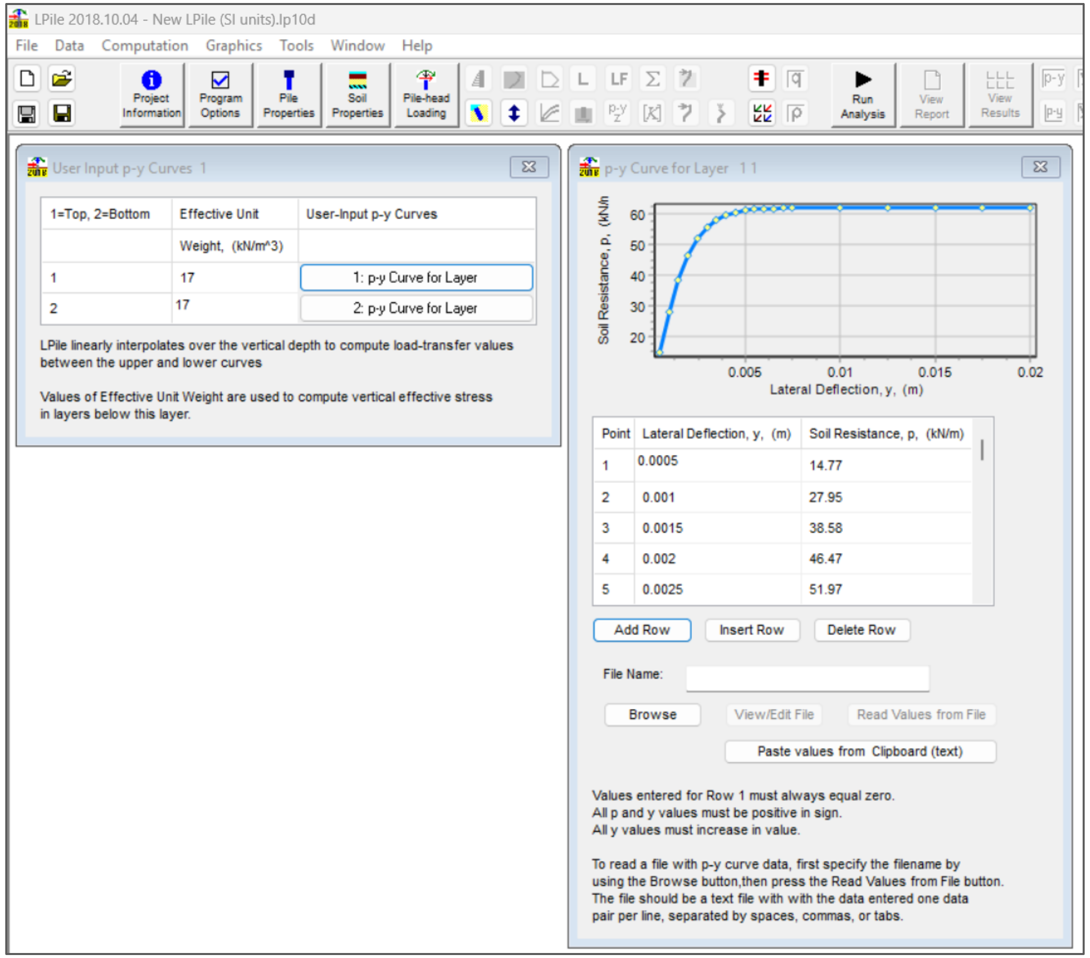
Input P-Y curves in Ensoft Lpi 2018 software

Step 2 Create layers starting from first test depth (generally 1.0 m interval – max 20 layers)



Input P-Y curves in Ensoft Lpi 2018 software

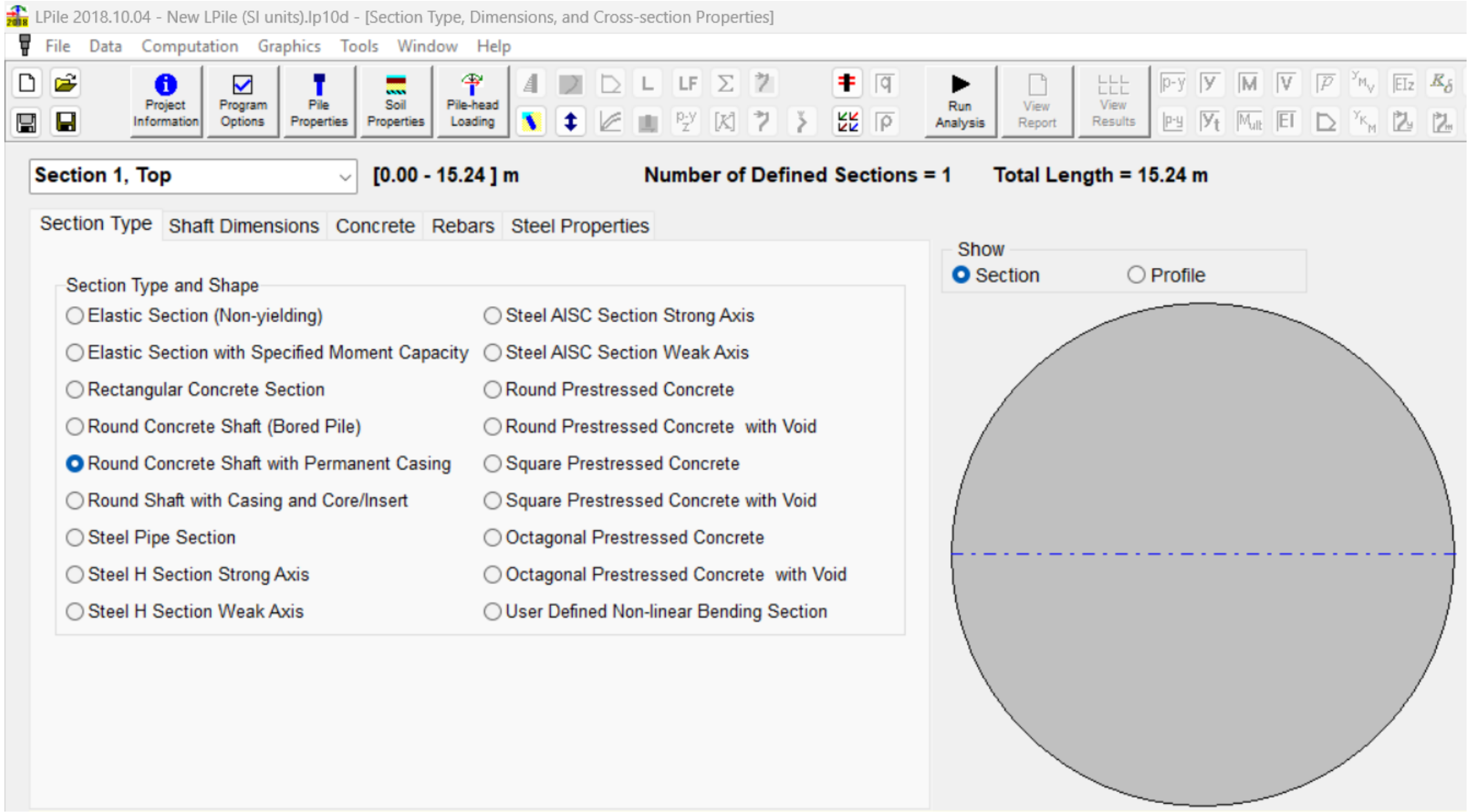
Step 3 For each layer, input unit weight for top and bottom depth of layer and P-Y curves



P-Y Graph			
Depth [m]	**Y/D [%]**	**y [m]**	**P [kN/m]**
0.8	0.1	0.0005	14.77
	0.2	0.001	27.95
	0.3	0.0015	38.58
	0.4	0.002	46.47
	0.5	0.0025	51.97
	0.6	0.003	55.64
	0.7	0.0035	58.02
	0.8	0.004	59.53
	0.9	0.0045	60.48
	1	0.005	61.08
	1.1	0.0055	61.44
	1.2	0.006	61.67
	1.3	0.0065	61.81
	1.4	0.007	61.90
	1.5	0.0075	61.95
	2	0.01	62.03
	2.5	0.0125	62.03
	3	0.015	62.04
	3.5	0.0175	62.04
	4	0.02	62.04

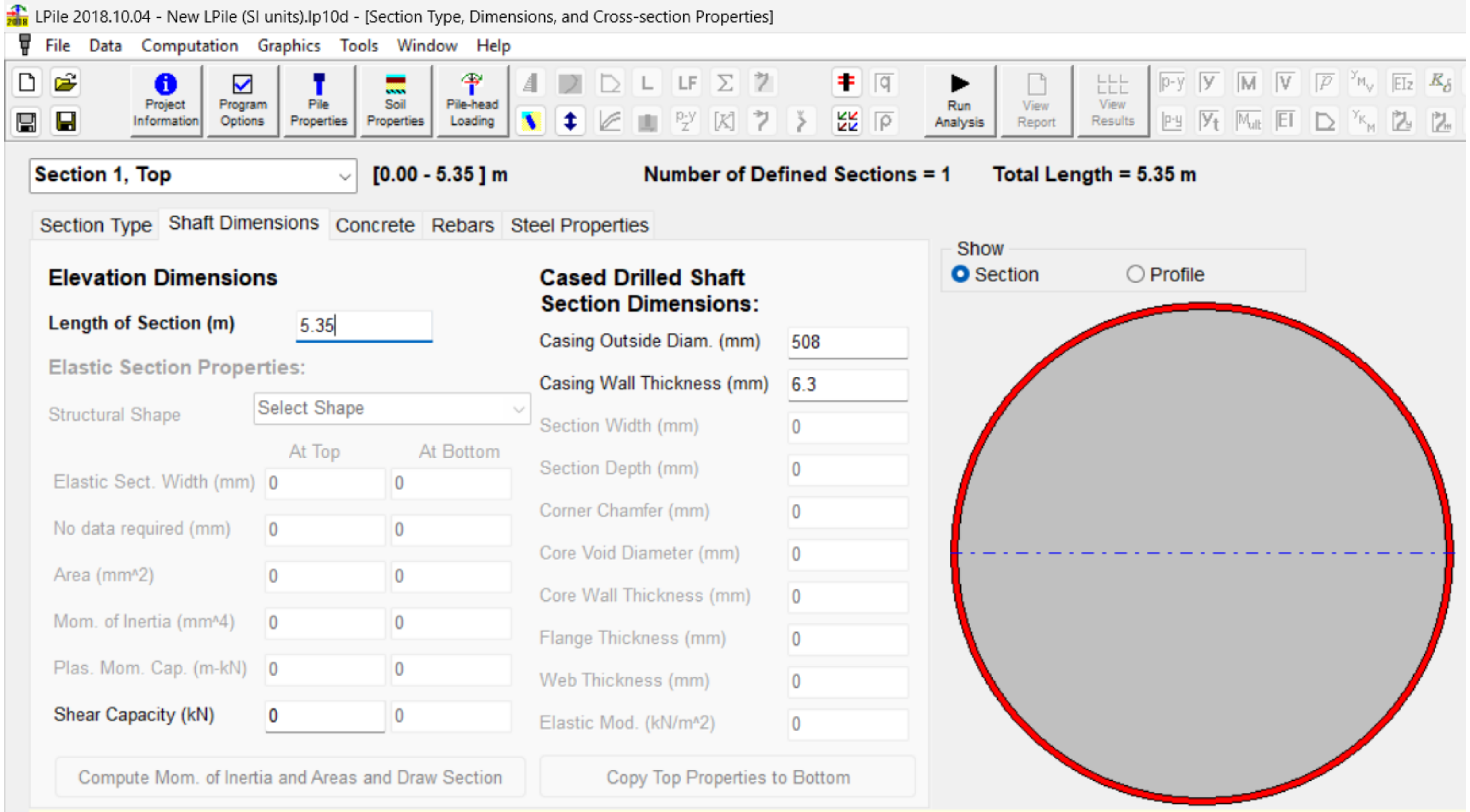
Input Pile properties in Ensoft Lpi 2018 software

Step 4 Choose pile type from the list



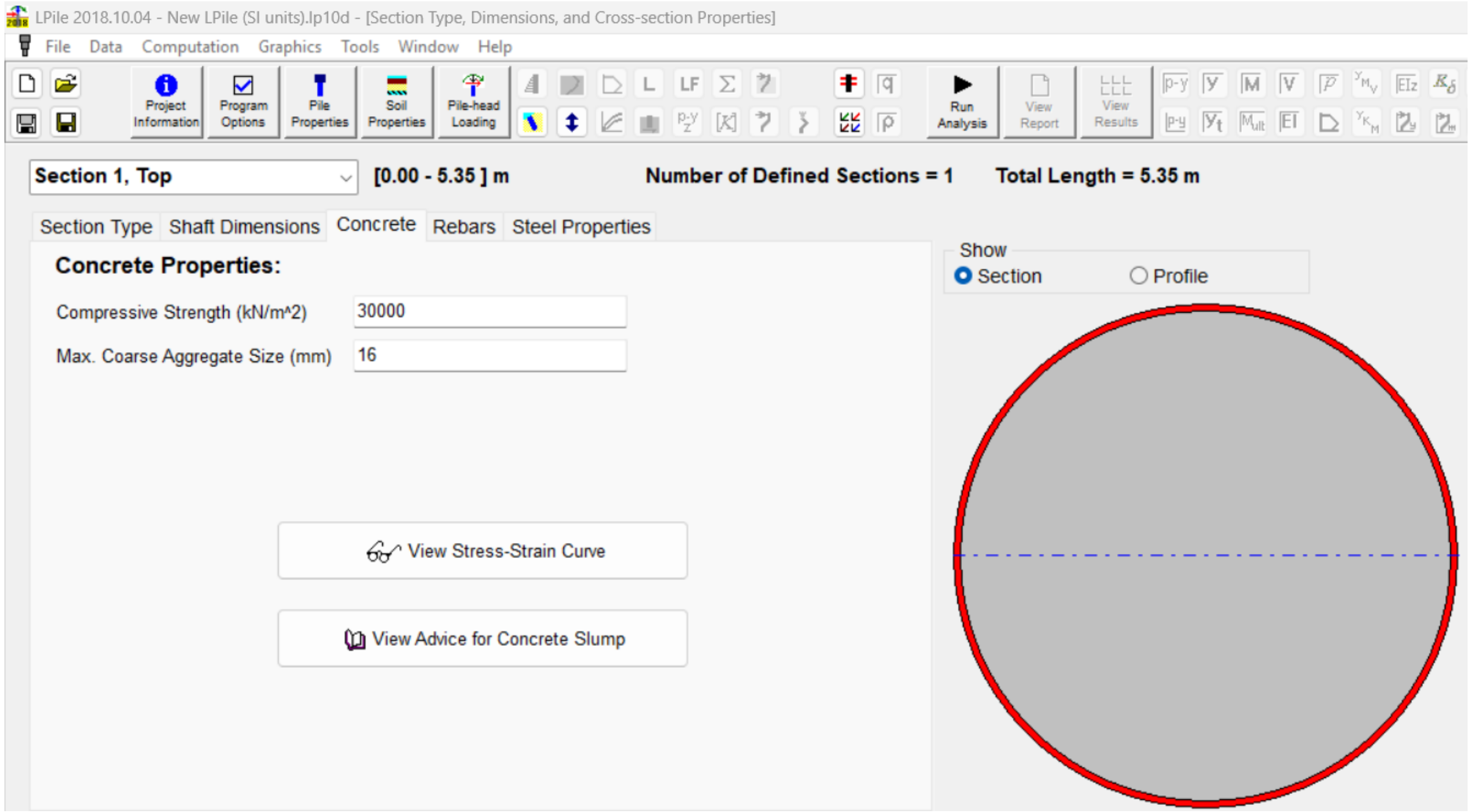
Input Pile properties in Ensoft Lpi 2018 software

Step 5 Input shaft dimensions



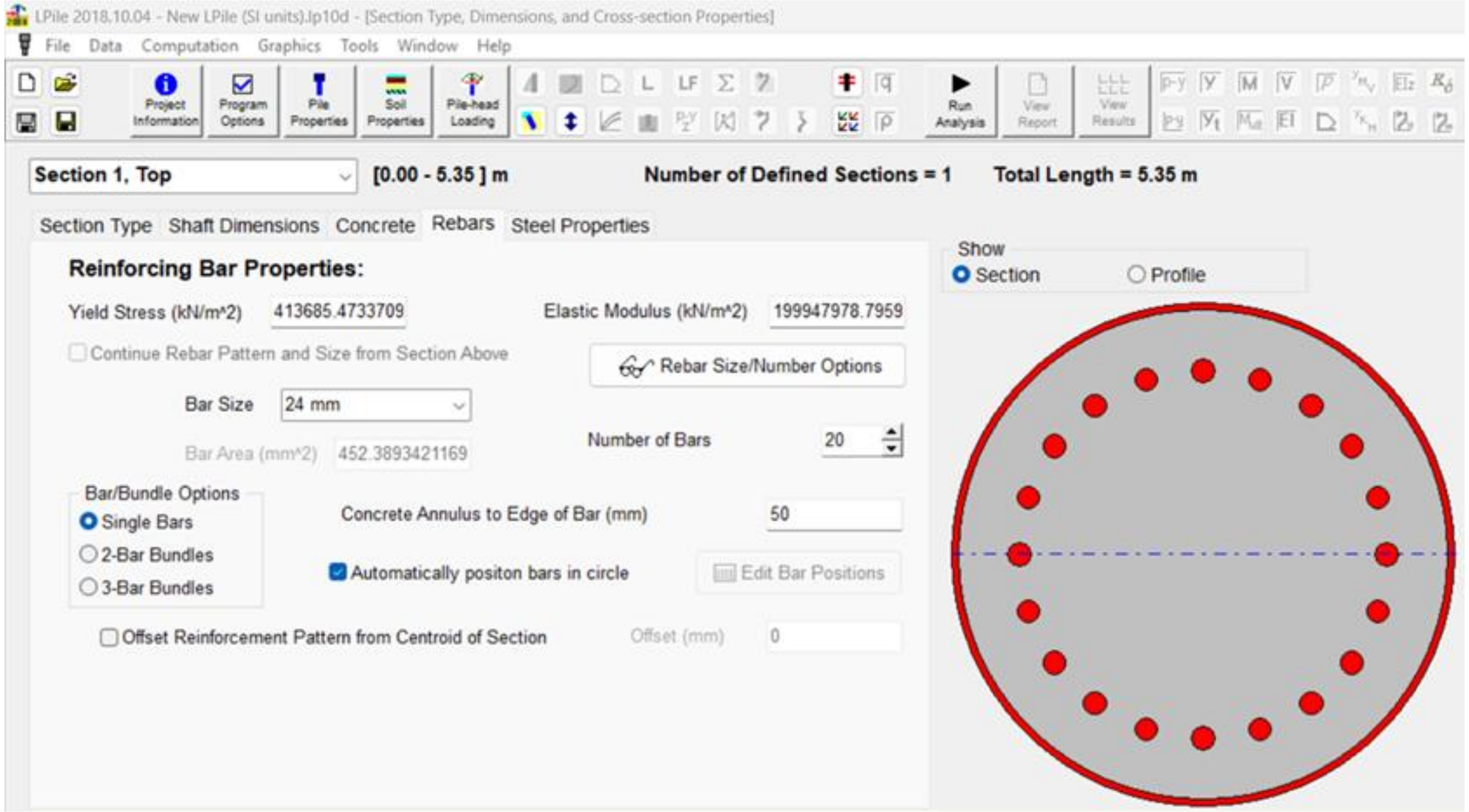
Input Pile properties in Ensoft Lpi 2018 software

Step 6 Input concrete properties, Max. coarse aggregate size of the employed concrete



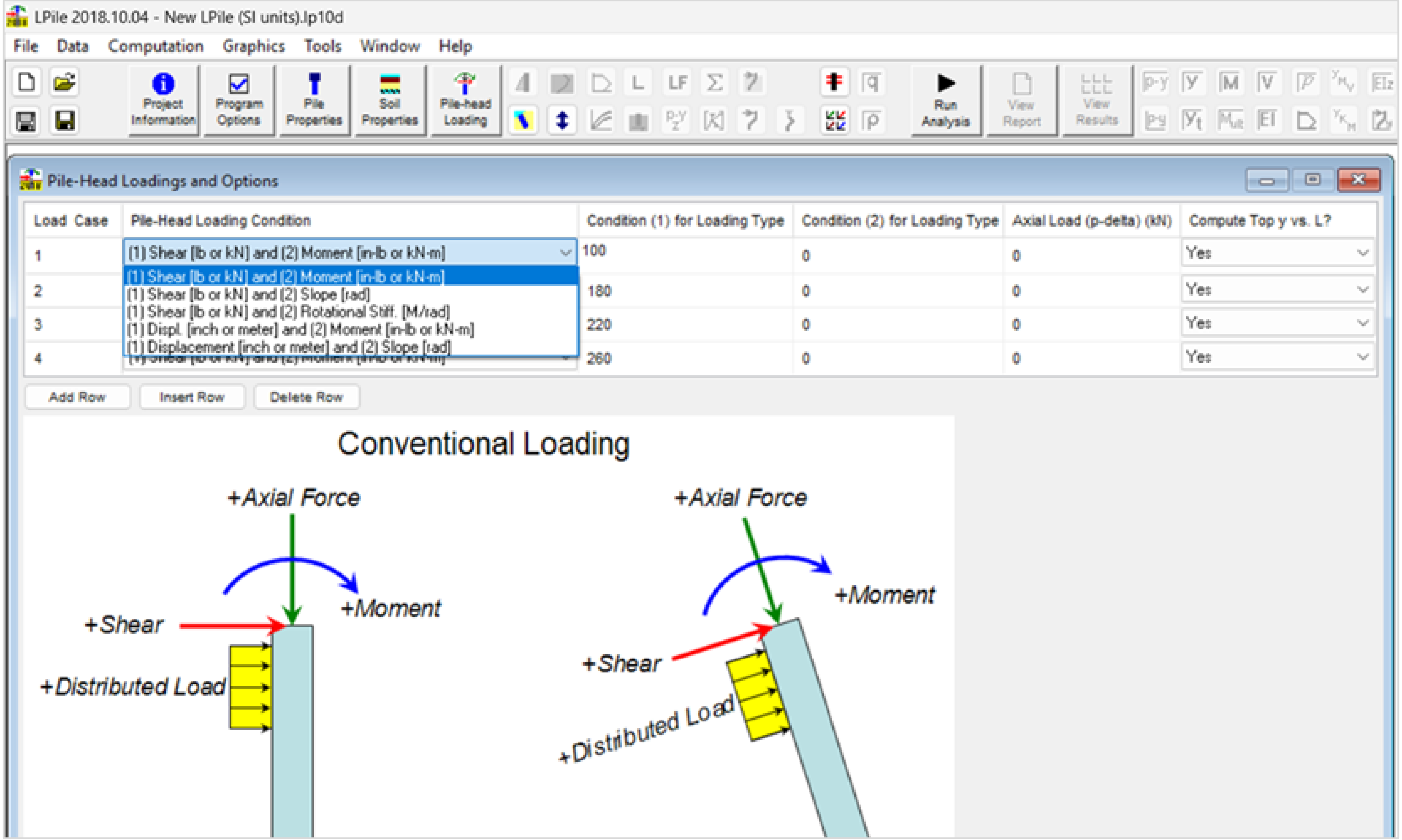
Input Pile properties in Ensoft Lpi 2018 software

Step 7 Input reinforcing bar properties



Input Pile properties in Ensoft Lpi 2018 software

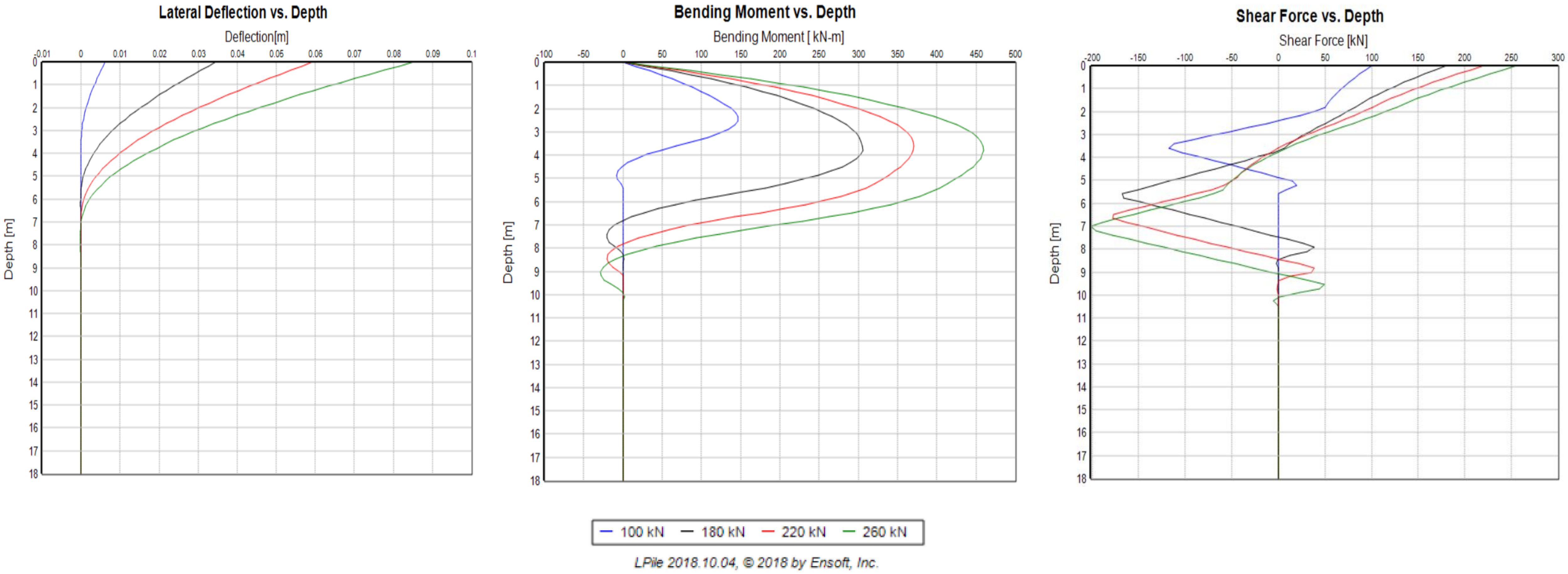
Step 8a In section 'Pile Head Loading', select condition 1 for free head and input loads



LLP Livorno case history: validation of Robertson 1989, Marchetti 1991 compared with field test measurements

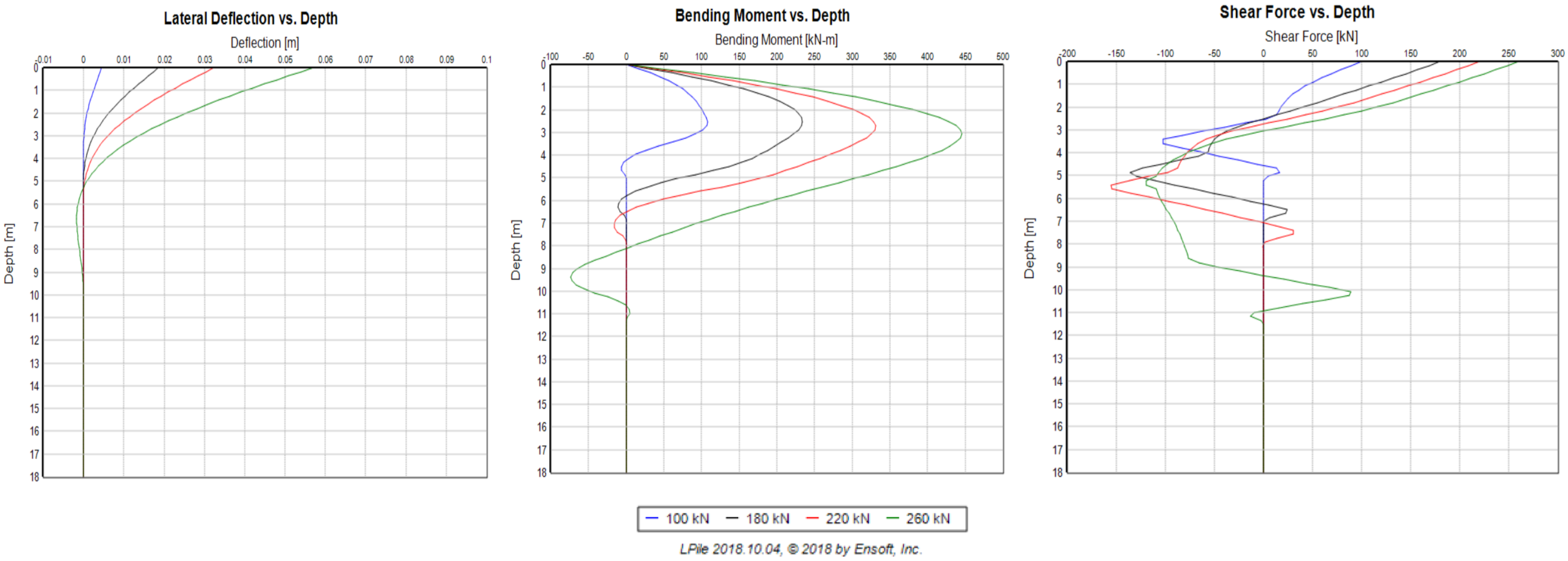
LLP analysis output P-Y curves Robertson 1989

LLP Livorno case history free head condition



LLP analysis output P-Y curves Marchetti 1991

LLP Livorno case history free head condition

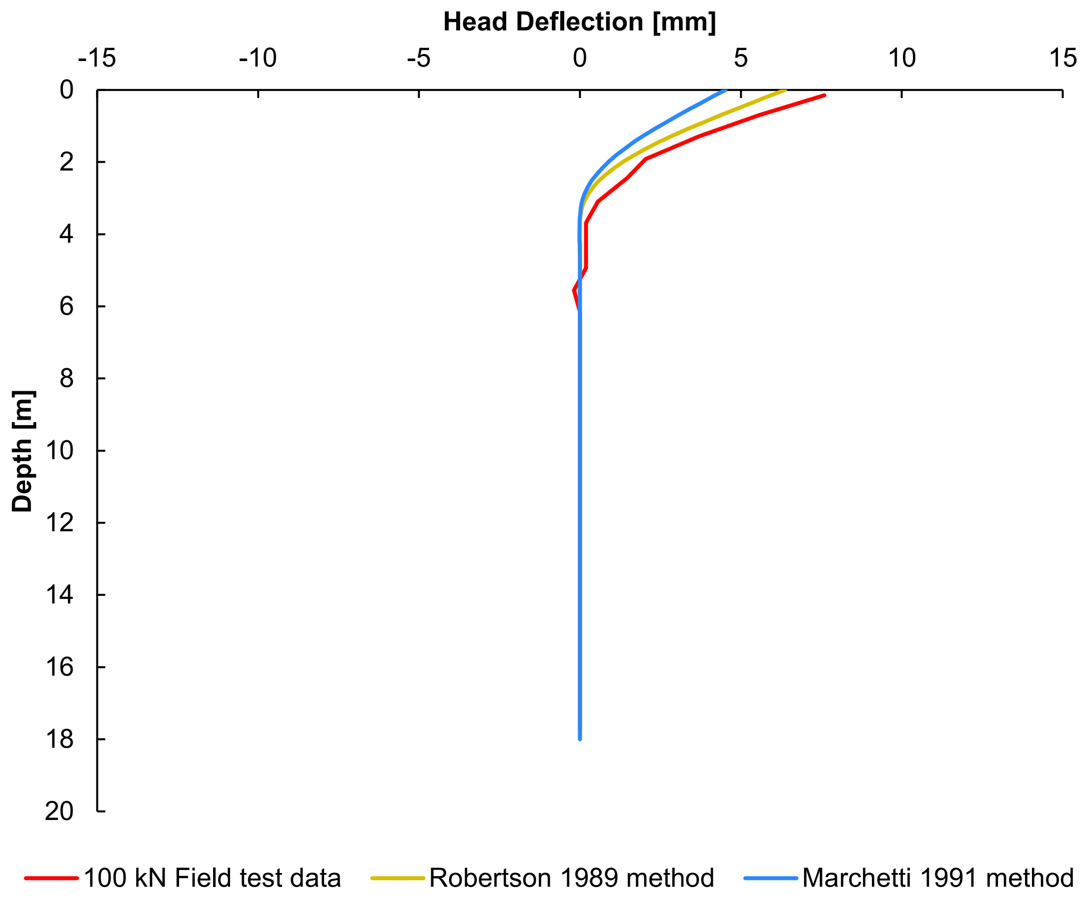


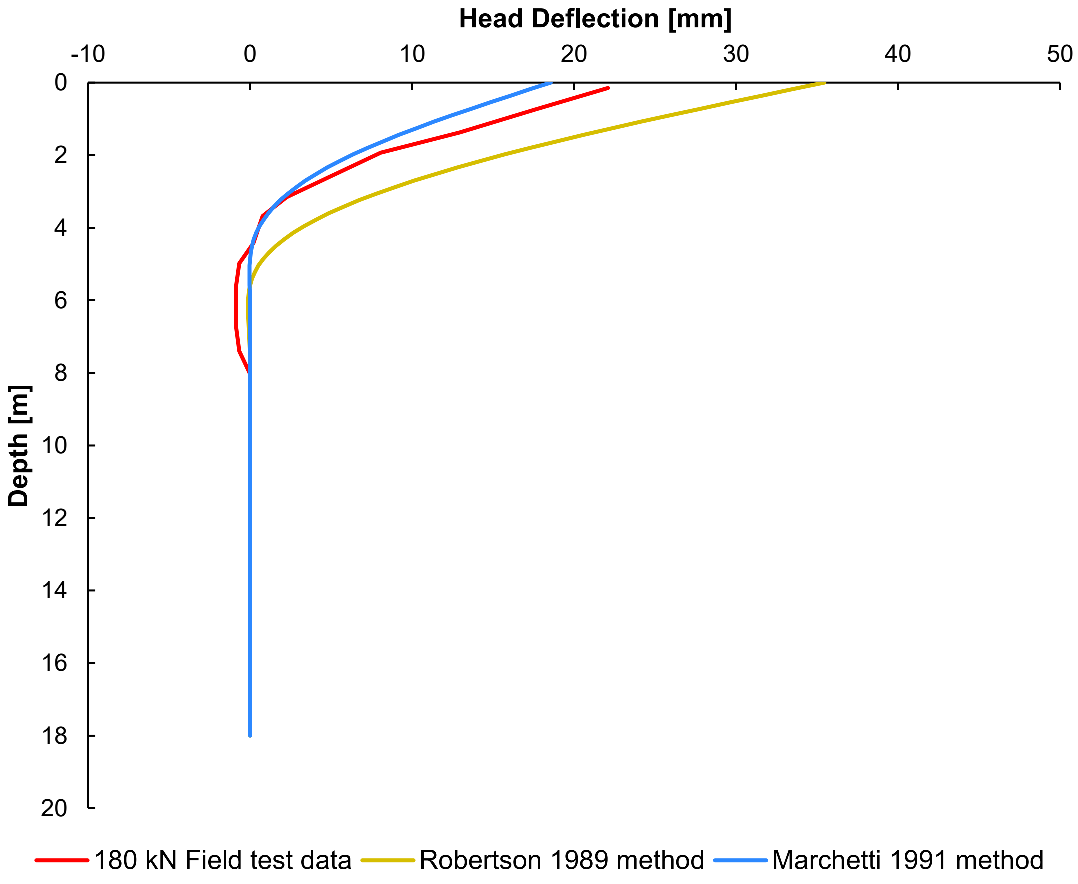
Free Head Pile Robertson 1989 - Marchetti 1991 - measured

100 kN horizontal load

LLP tests Livorno

180 kN horizontal load



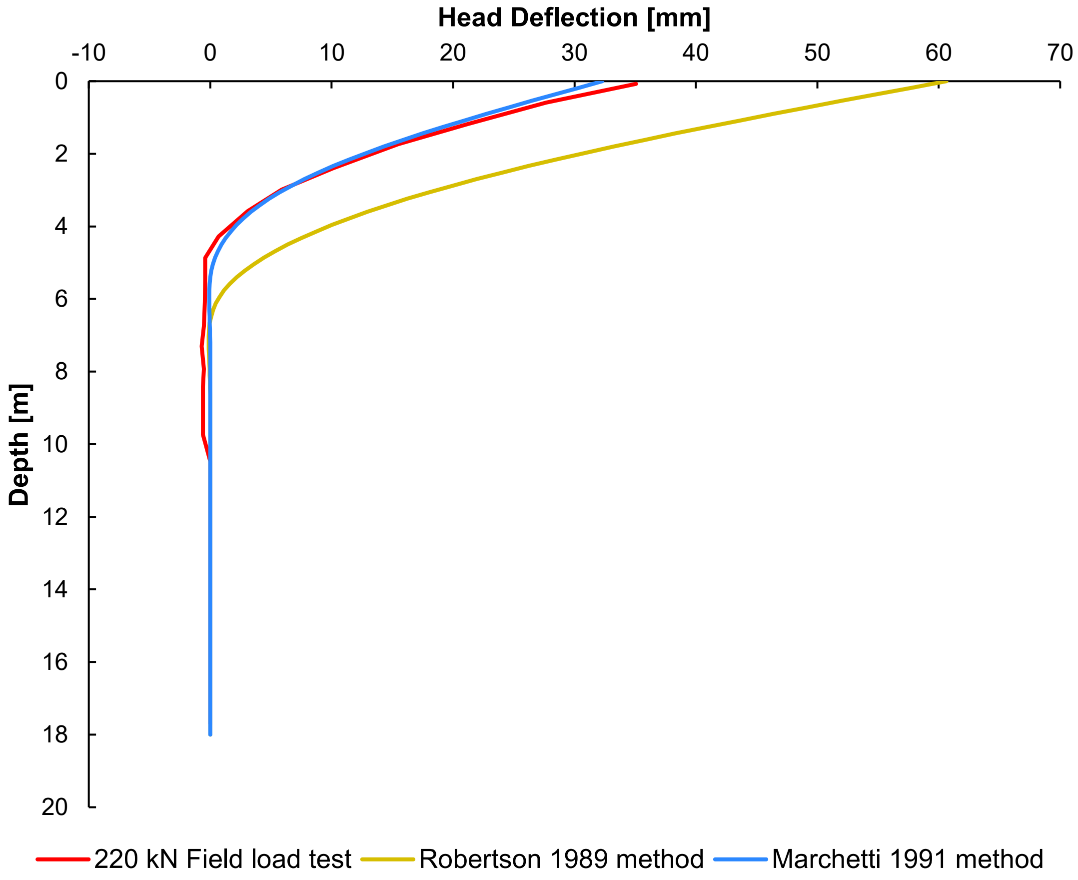


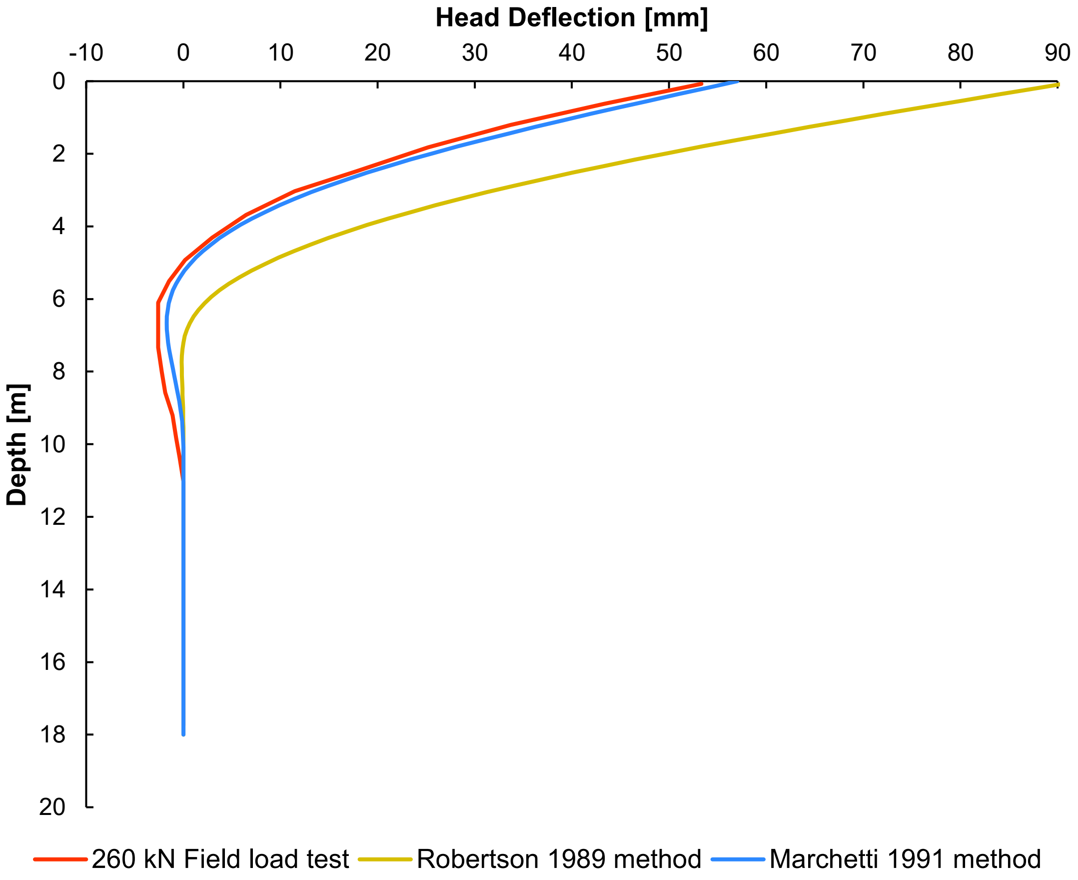
Free Head Pile Robertson 1989 - Marchetti 1991 - measured

220 kN horizontal load

LLP tests Livorno

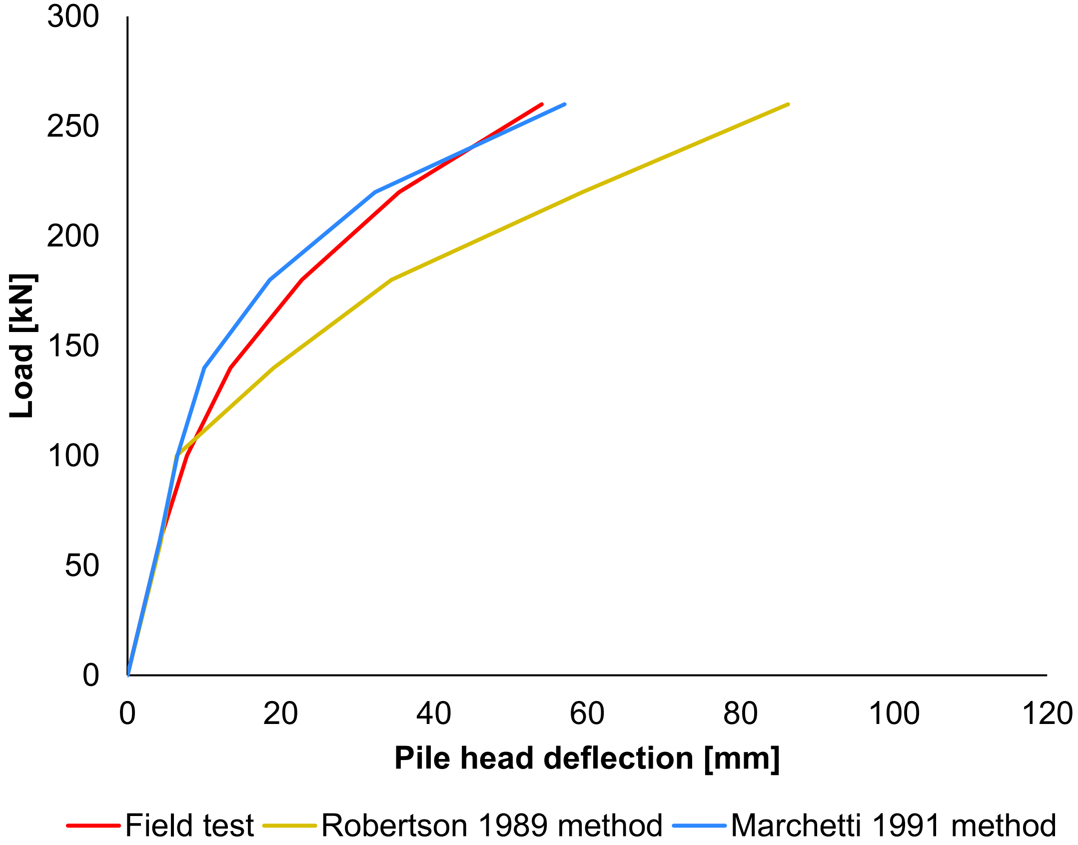
260 kN horizontal load





Free Head Pile Robertson 1989 - Marchetti 1991 - measured

LLP tests Livorno



Applied Load	Field measure	Robertson (1989)	Error Robertson	Marchetti (1991)	Error Marchetti
[kN]	[mm]	[mm]	[%]	[mm]	[%]
0	0	0	**0**	0	**0**
60	4	4	**0**	4	**0**
100	8	6	**-25**	7	**-13**
140	13	19	**46**	10	**-23**
180	23	34	**48**	19	**-17**
220	35	59	**41**	32	**-9**
260	54	86	**59**	57	**5**

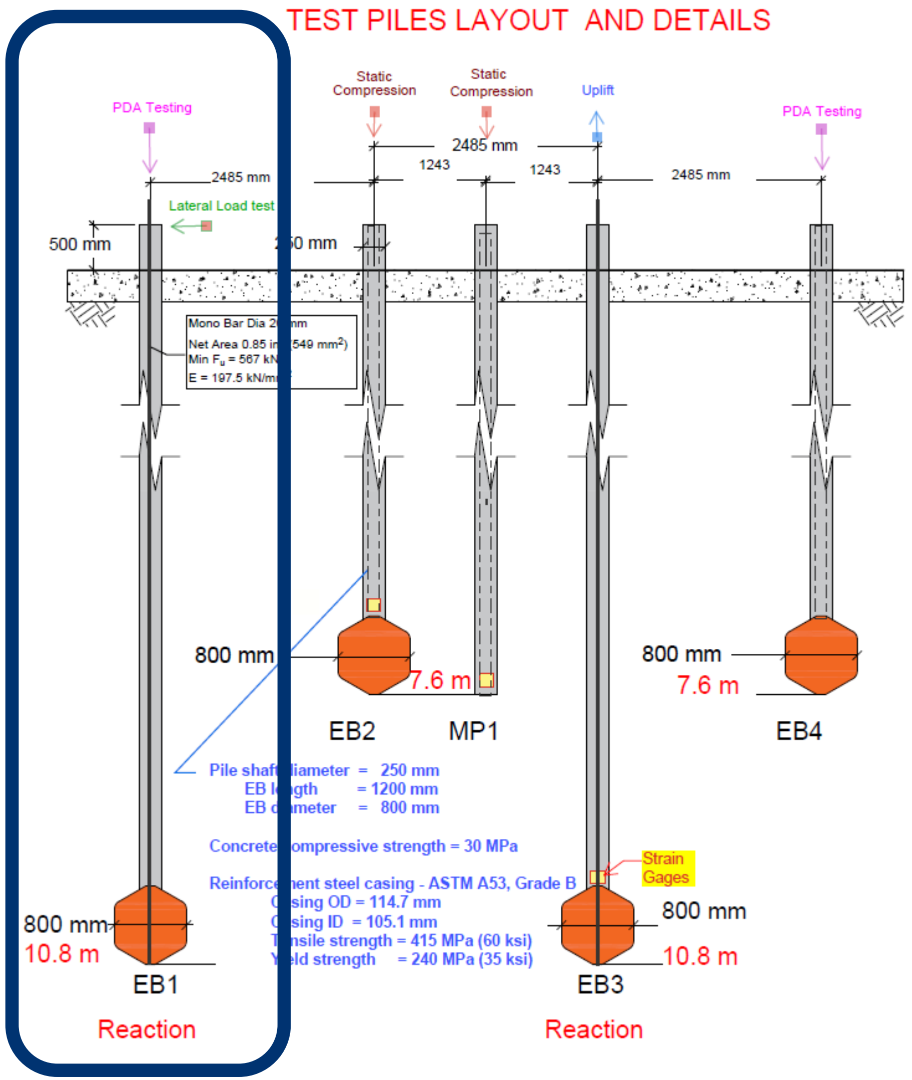
LLP test in Livorno (1991)

Comments on Robertson (1989) and Marchetti (1991) LLP performance

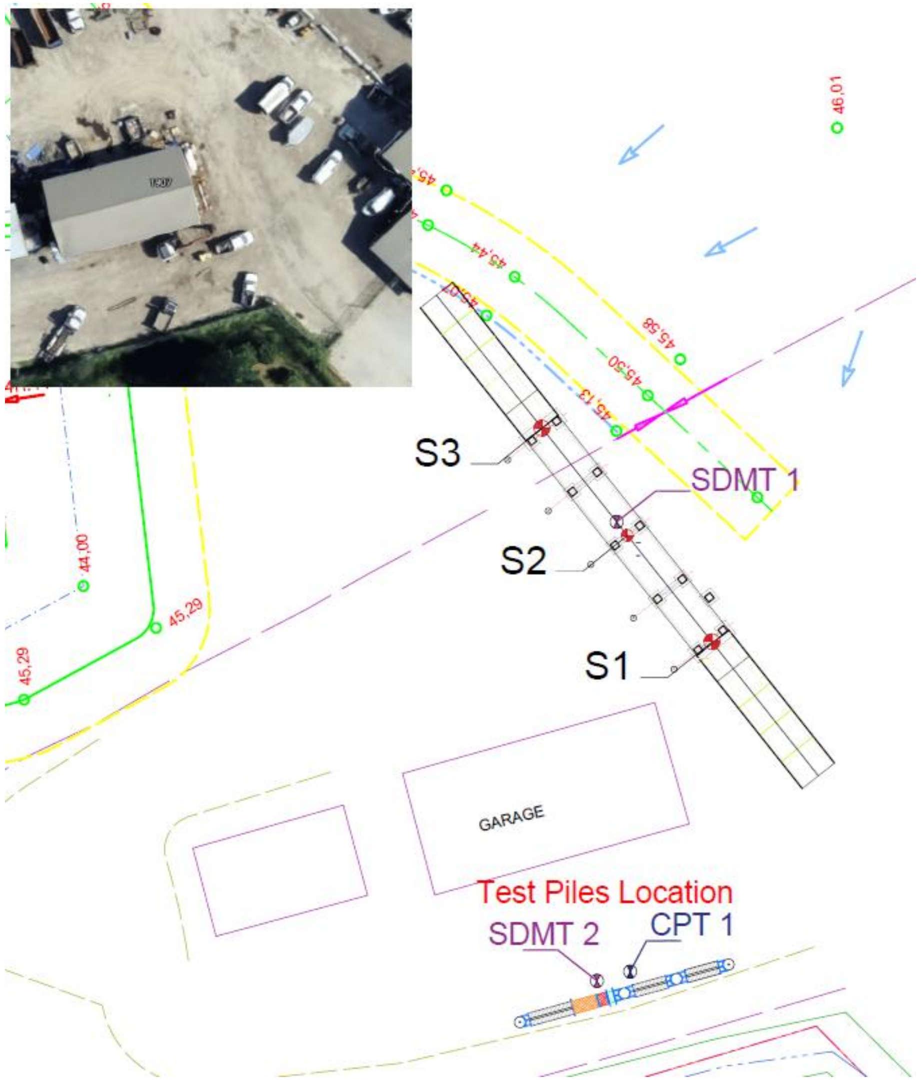
- At the lowest load, where initial tangent modulus is estimated by E_D, both Roberston and Marchetti method provide good prediction
- At medium to high loads, Robertson method overpredicts deflections. Possible reason: $p = p_{ult}$ only for $y/y_{50} \geq 8$, too conservative !!
- Marchetti method (tanh function) provides very good predictions in all loads in the LLP test of Livorno

LLP tests in Gatineau (2025)

Pile testing Specifications for the Gatineau Pile Testing Event (2025)

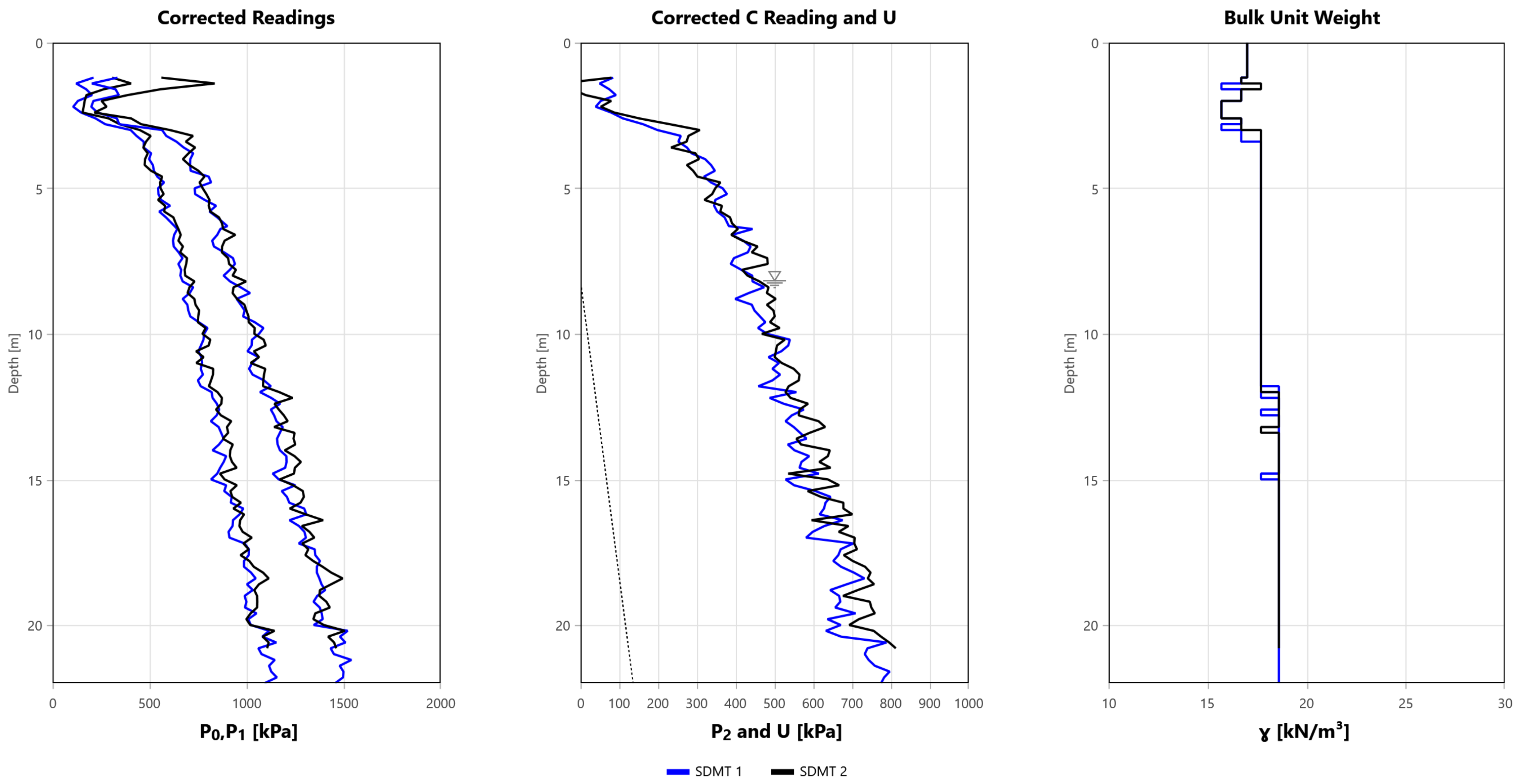


Site characterization tests for the Gatineau Pile Testing Event (2025)

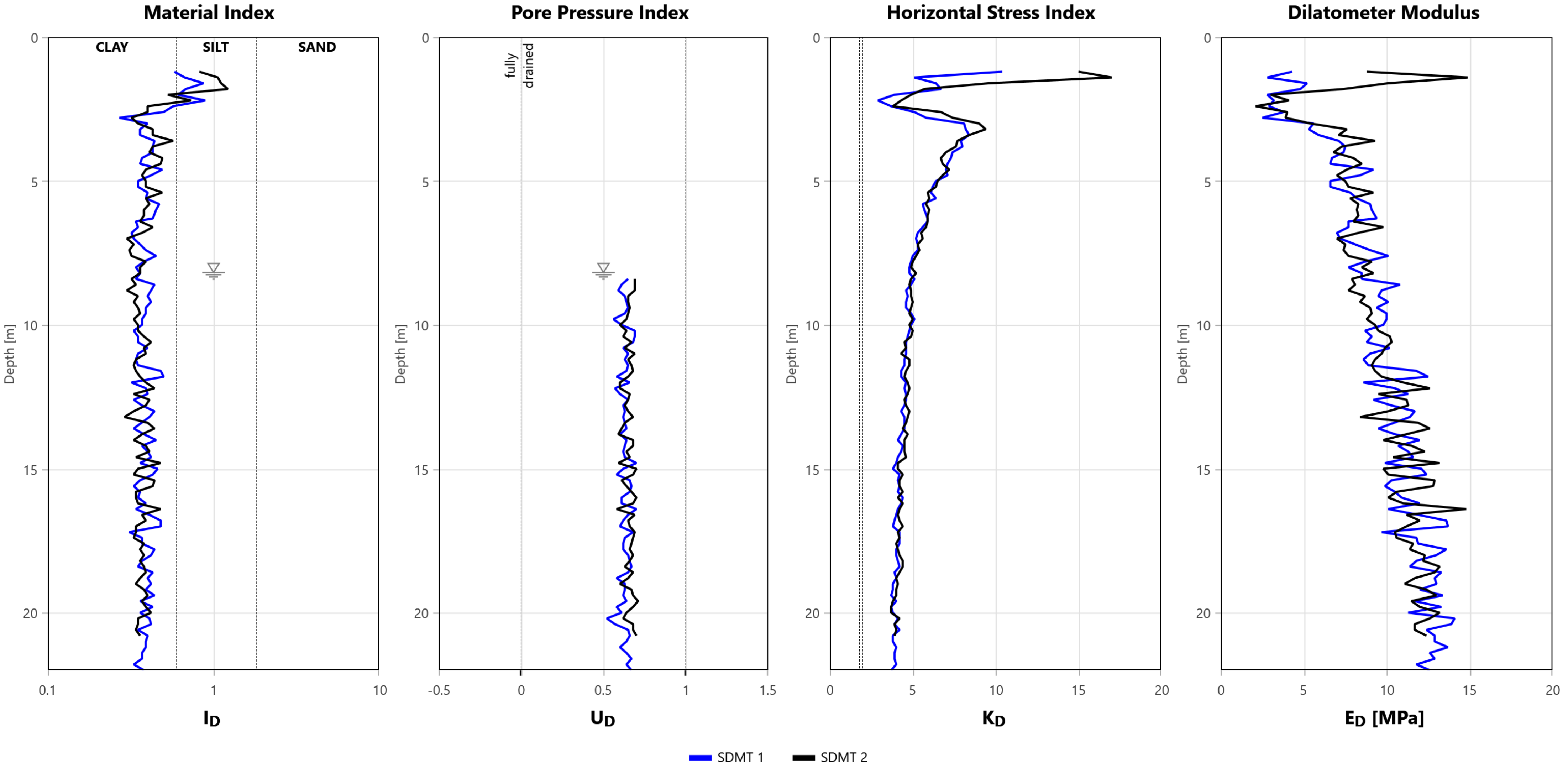


Gatineau LLP test site (September 2025): DMT test results

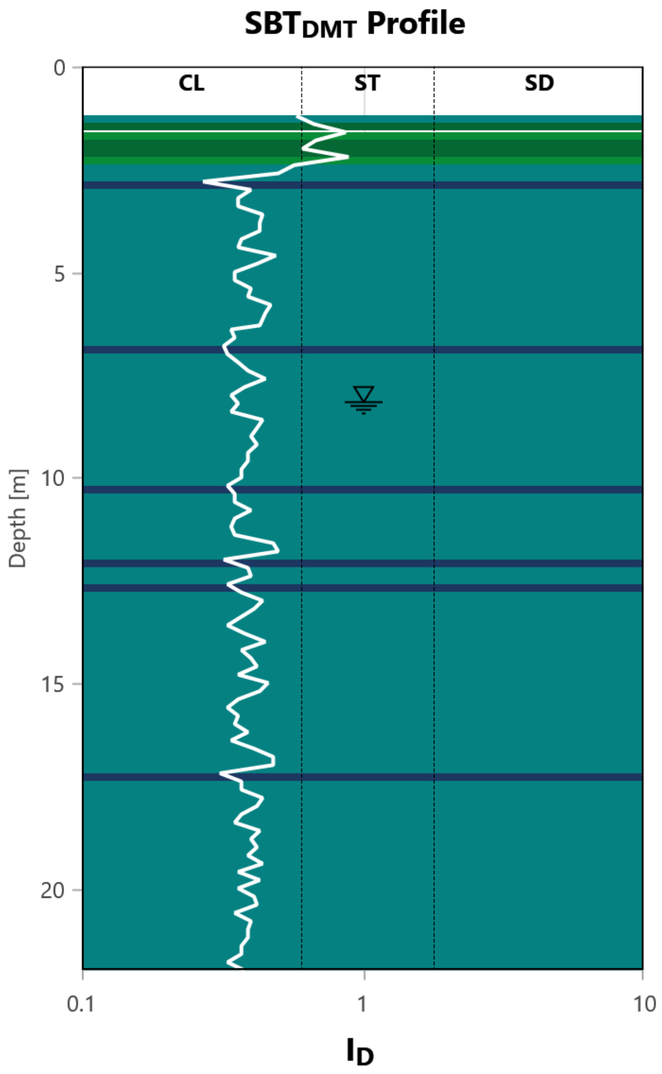
Medusa DMT tests results in Gatineau (2025/9)

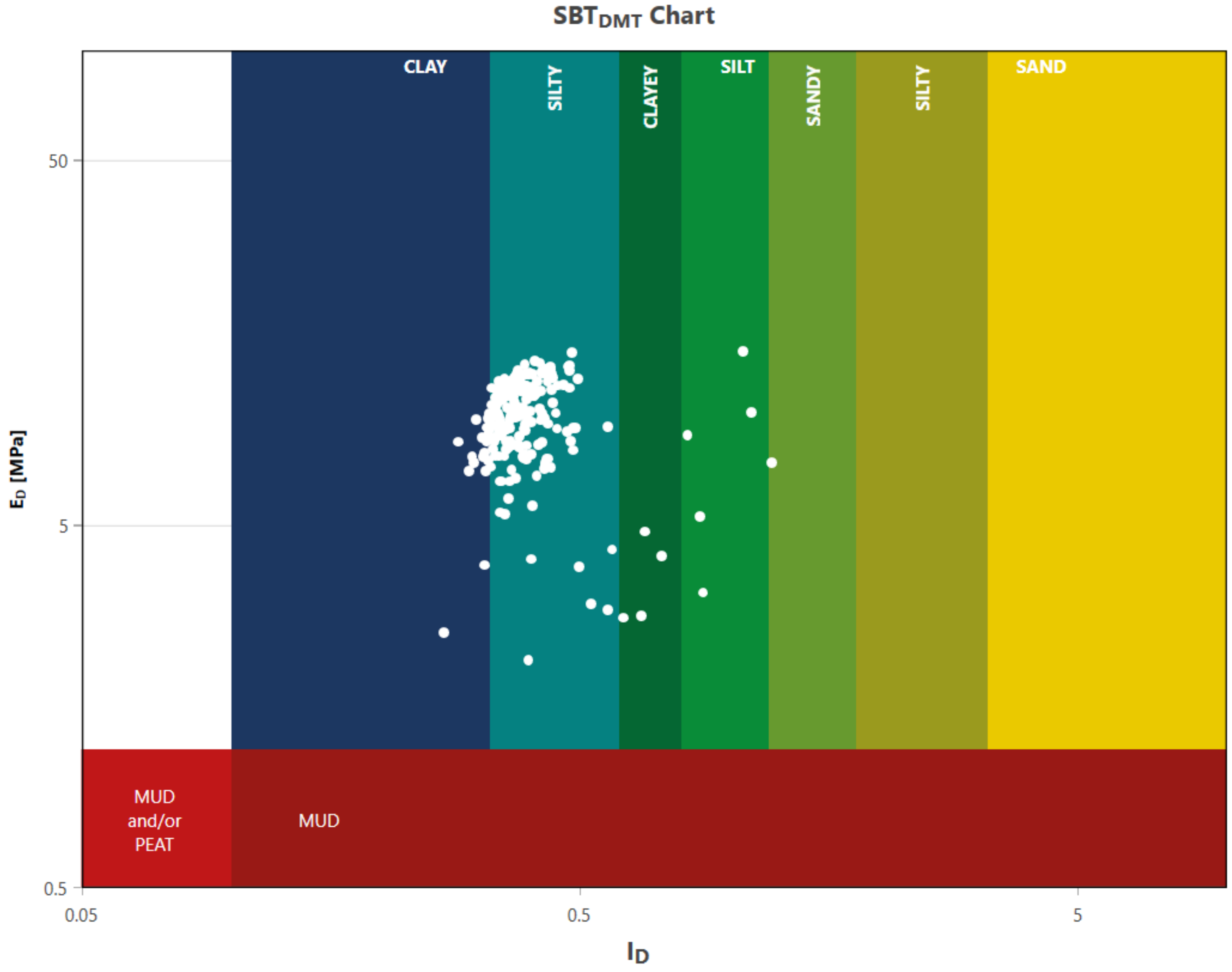


Medusa DMT tests results in Gatineau (2025/9)

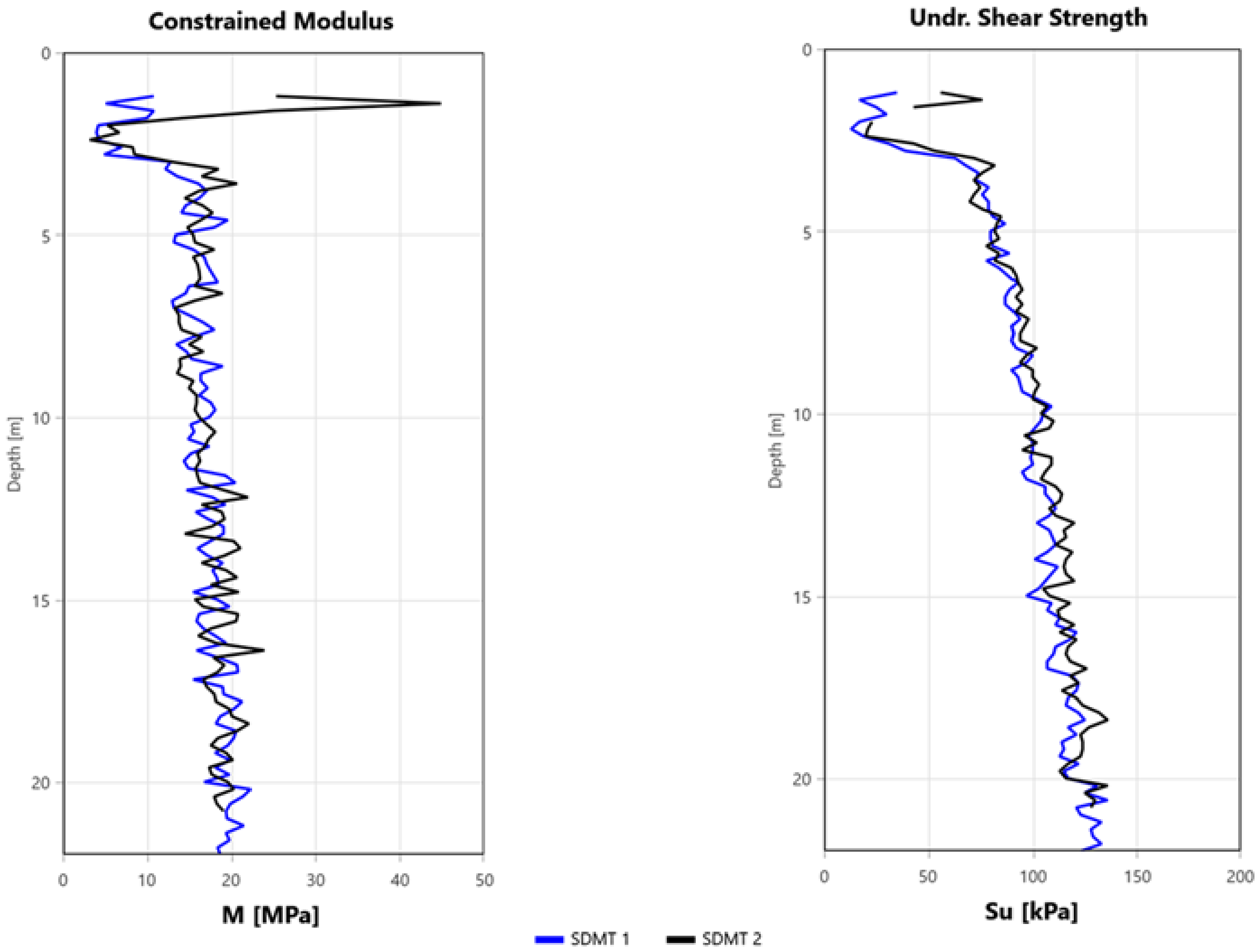


Medusa DMT tests results in Gatineau (2025/9)

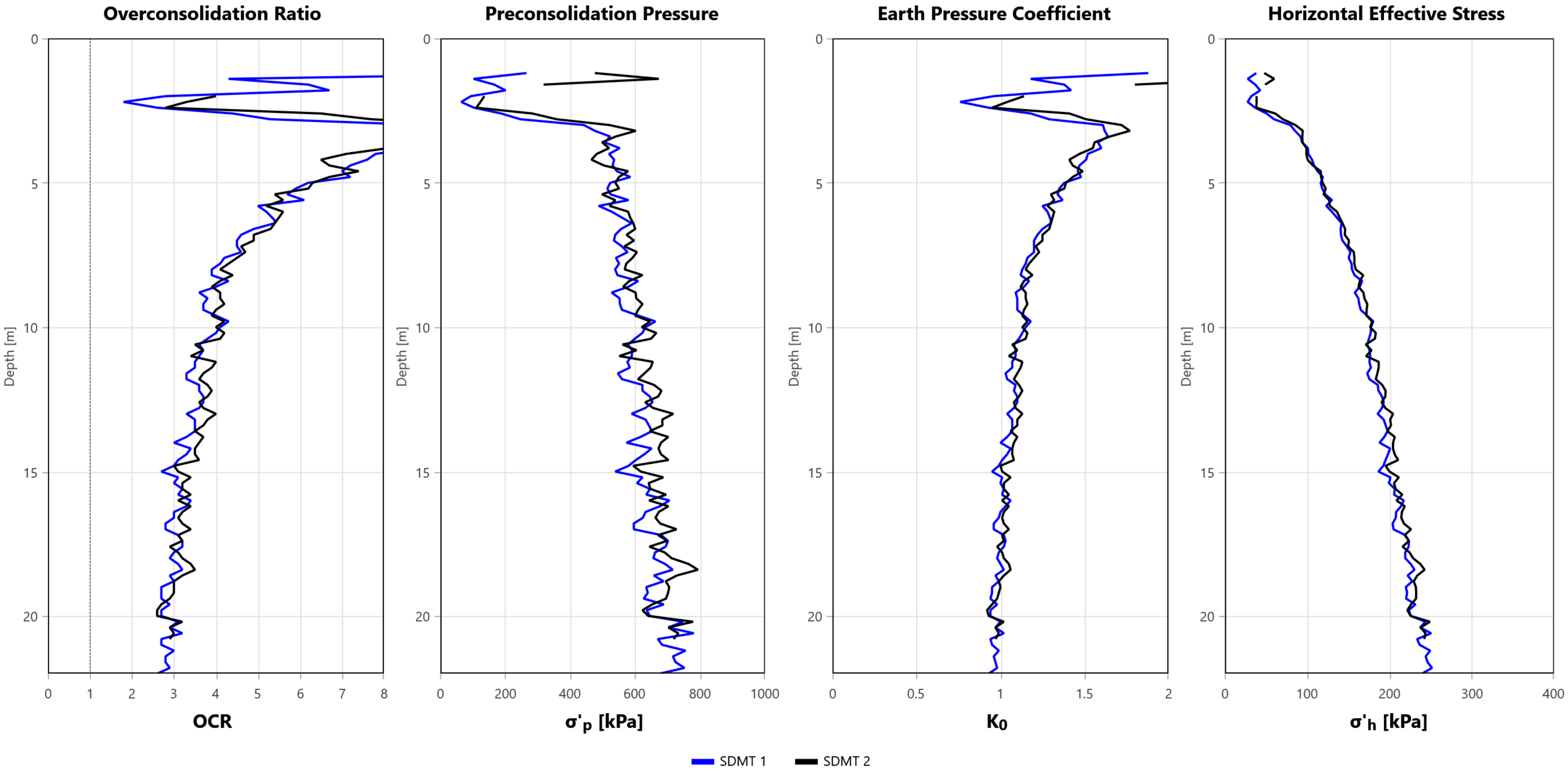




Medusa DMT tests results in Gatineau (2025/9)



Medusa DMT tests results in Gatineau (2025/9)



Summary and Conclusions

- DMT, SDMT, Medusa (S)DMT equipment and test procedures
- Field machines for penetration
- Data reduction and interpretation of geotechnical parameters, with comparisons in well documented research test sites
- DMT & CPT: similarities, differences and integration for soil characterization
- Main applications for geotechnical design

THANK YOU FOR YOUR ATTENTION

Questions

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