Cone Penetration Testing

a proven, but underutilized site investigation method

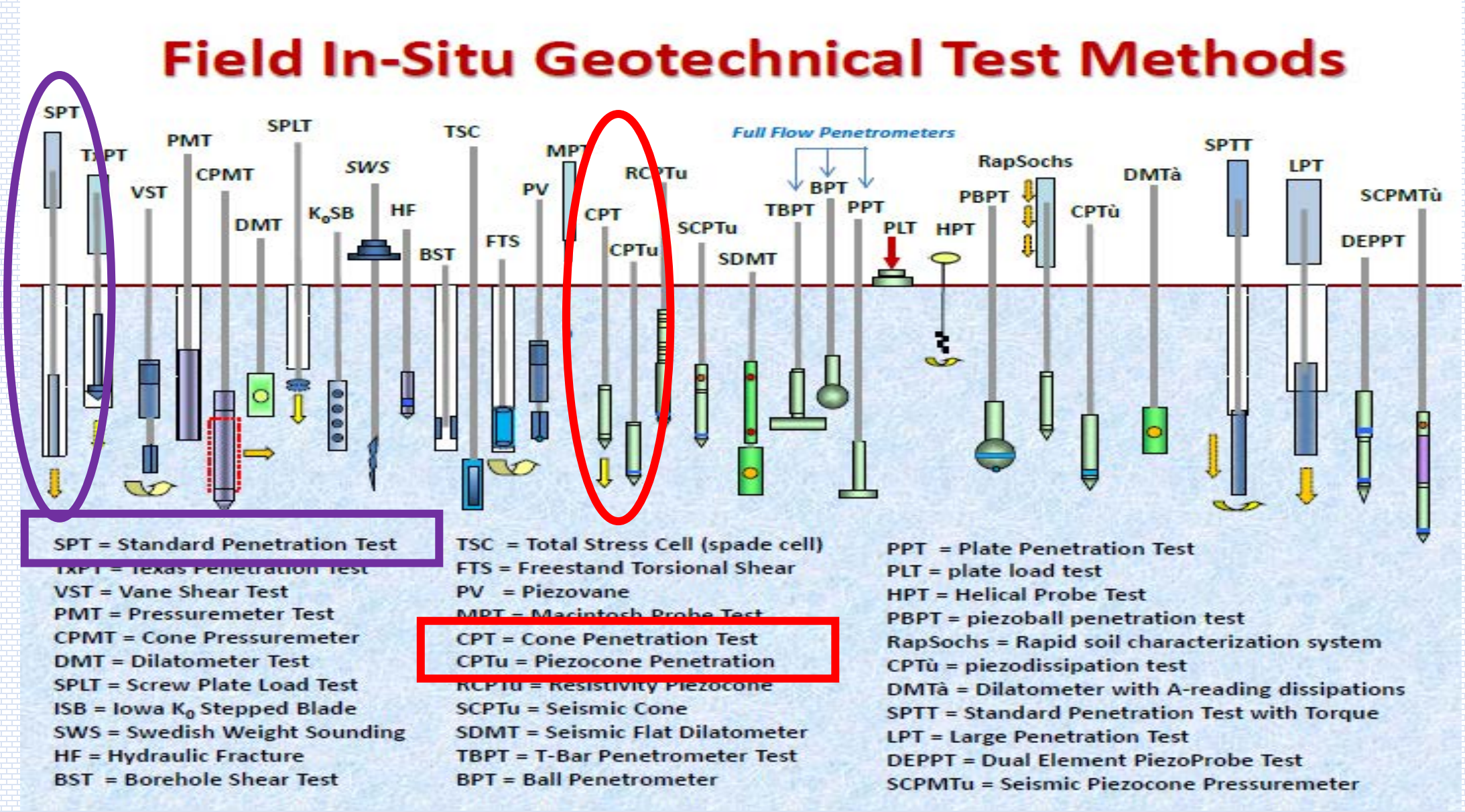
Gerald Verbeek – Verbeek Management Services

The disclaimers

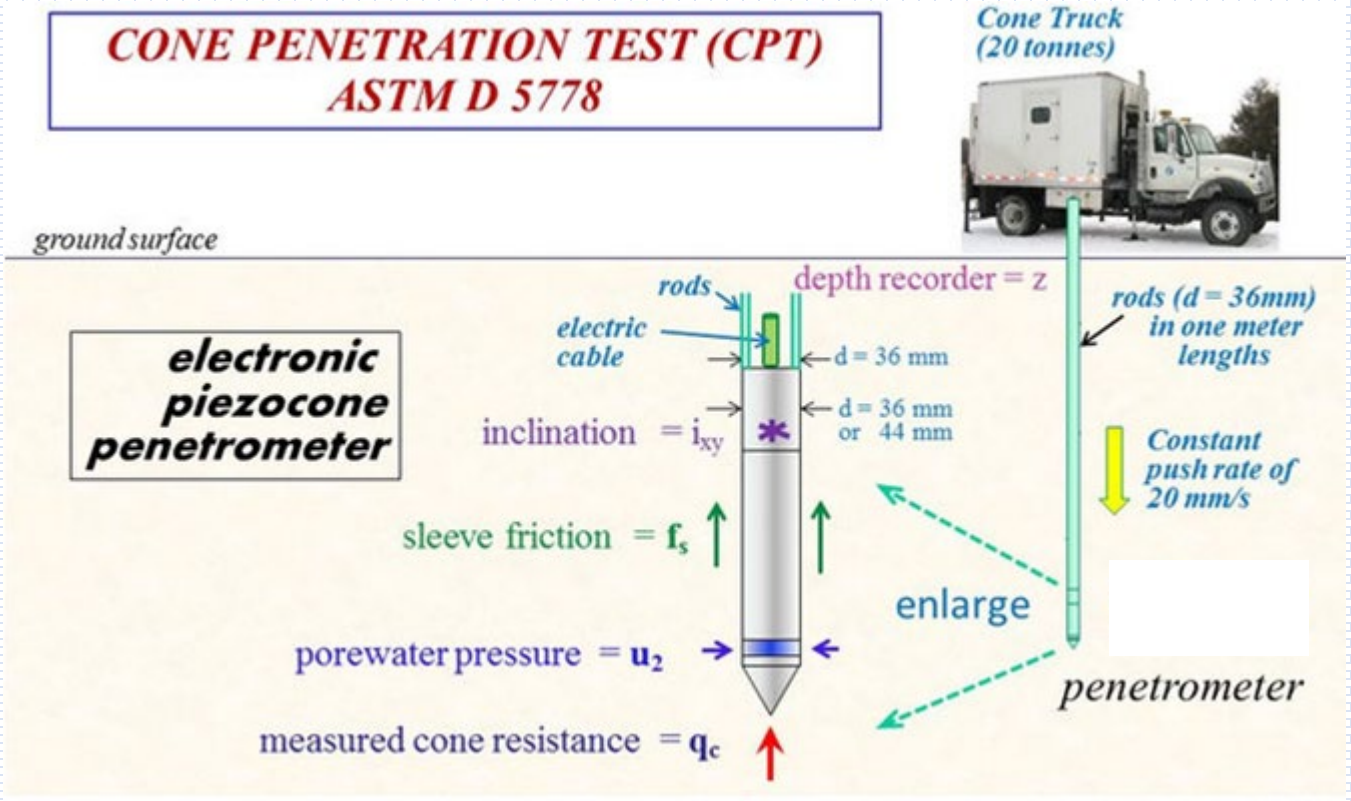
- **I'm Dutch**
- **I'm a Conehead**

But I will try not to be biased

What is CPT


Field In-Situ Geotechnical Test Methods
SPT
TxPT
VST
PMT
CPMT
DMT
SPLT
K_oSB
SWS
HF
BST
TSC
FTS
PV
MPT
CPT
CPTu
RCPTu
SCPTu
SDMT
Full Flow Penetrometers
TBPT
BPT
PPT
PLT
HPT
PBPT
RapSochs
CPTù
DMTà
SPTT
LPT
DEPPT
SCPMTù
SPT = Standard Penetration Test
TxPT = Texas Penetration Test
VST = Vane Shear Test
PMT = Pressuremeter Test
CPMT = Cone Pressuremeter
DMT = Dilatometer Test
SPLT = Screw Plate Load Test
ISB = Iowa K_0 Stepped Blade
SWS = Swedish Weight Sounding
HF = Hydraulic Fracture
BST = Borehole Shear Test
TSC = Total Stress Cell (spade cell)
FTS = Freestand Torsional Shear
PV = Piezovane
MPT = Macintosh Probe Test
CPT = Cone Penetration Test
CPTu = Piezocone Penetration
RCPTu = Resistivity Piezocone
SCPTu = Seismic Cone
SDMT = Seismic Flat Dilatometer
TBPT = T-Bar Penetrometer Test
BPT = Ball Penetrometer
PPT = Plate Penetration Test
PLT = plate load test
HPT = Helical Probe Test
PBPT = piezoball penetration test
RapSochs = Rapid soil characterization system
CPTù = piezodissipation test
DMTà = Dilatometer with A-reading dissipations
SPTT = Standard Penetration Test with Torque
LPT = Large Penetration Test
DEPPT = Dual Element PiezoProbe Test
SCPMTu = Seismic Piezocone Pressuremeter

What is CPT



After CPT Manual for Highway Geotechnical Engineers , MnDOT 2018

The cone penetration test uses a highly standardized and instrumented cone that is pushed into the ground at a continuous rate of 0.8 in/s (20 mm/s) to record the tip resistance, sleeve friction and pore pressure every 0.4 in (10 mm) until the cone reaches the target depth or refusal.

What is CPT

1932 -1934

Pieter Barentsen develops the first internationally recognized cone model for Cone Penetration Testing (CPT). The Dutch Cone was born and also the first patent in CPT history, applied in 1934 and granted in 1938 to Goudsche Machinefabriek and Pieter Barentsen.

1959

GMF Gouda introduces the first hydraulic pushing rigs for 10 ton and later also 20 ton capacity setting a new standard for efficient CPT soundings.

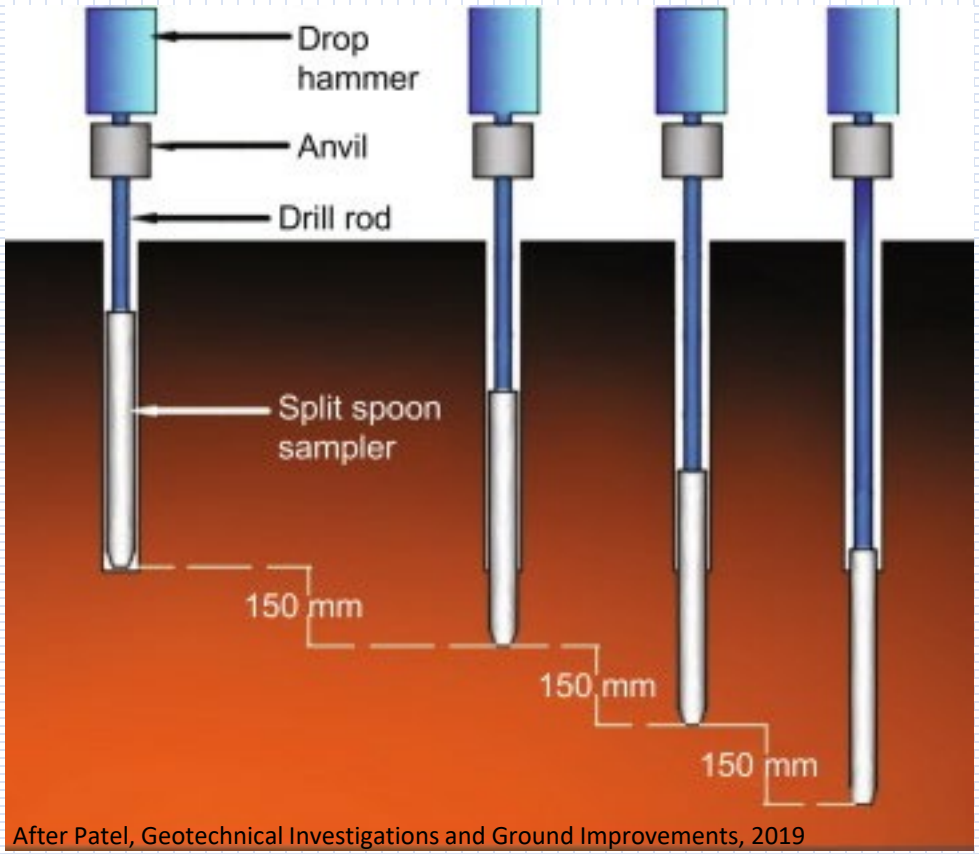
1965

H.K.S. Begemann improved the Dutch cone and added an extra sliding shaft for measuring the sleeve friction, resulting in the Friction Jacket Cone, also known as Begemann Cone.

1971

After a period of testing, failing and improving the electric cone penetrometers with strain gauged measuring bodies become more reliable and popular.

What is SPT



After Patel, Geotechnical Investigations and Ground Improvements, 2019

The standard penetration test uses a split spoon sampler. A hammer of 140 lbs (63.5 kg) is dropped from a height of 30 in (760 mm) to drive the sampler into the ground to a depth of 6 in (150 mm) below the clean-out depth, the so-called "seating drive".

Next the split spoon sampler is driven another 12 in (300 mm) into the ground until it has penetrated a total depth of 18 in (450 mm).

The number of blows required to penetrate these last 12 in (300 mm) is termed the penetration resistance (the s-called N value).

What is SPT

1902

Colonel Charles R. Gow, owner of the Gow Construction Co. in Boston, began making exploratory borings using 1-inch diameter drive samplers

Around 1930

During the late 1920s and early 1930s, SPT was standardized by Harry Mohr, one of Gow's engineers, then with Raymond Concrete Pile Co.

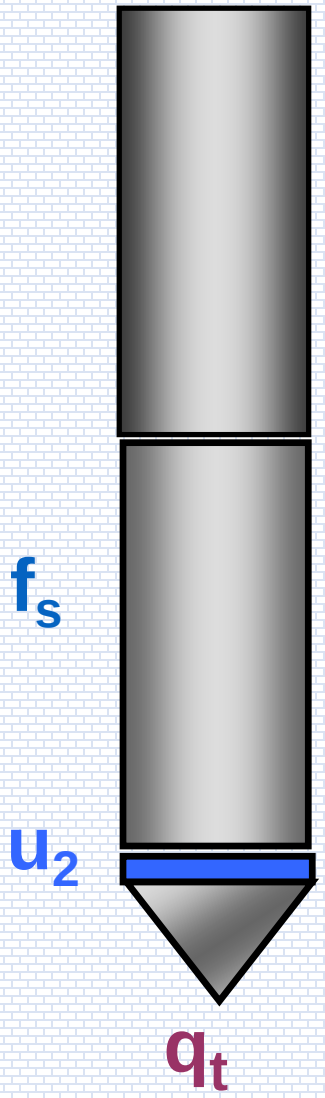
Late 1930s

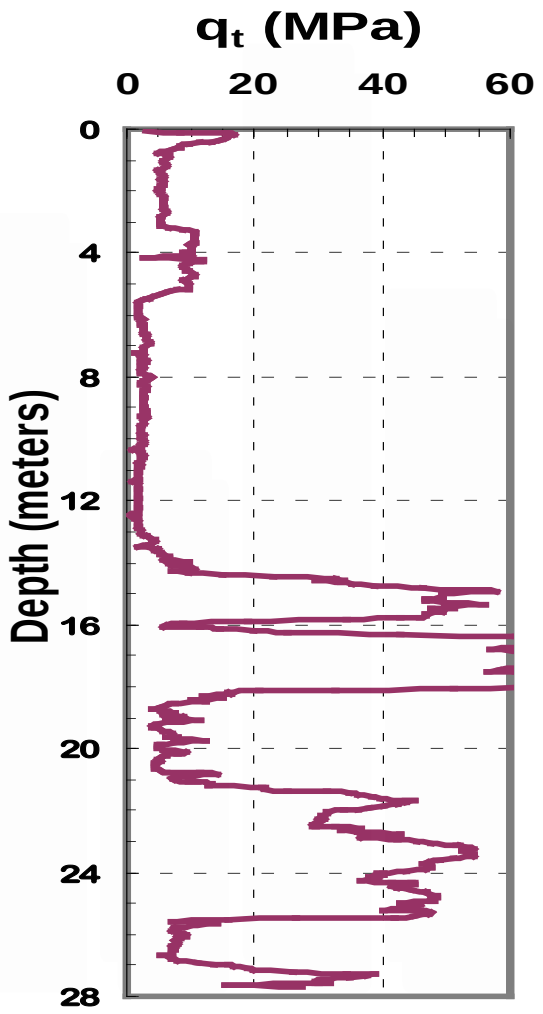
Karl von Terzaghi and Arthur Casagrande promoted adoption of the split spoon sampling procedure

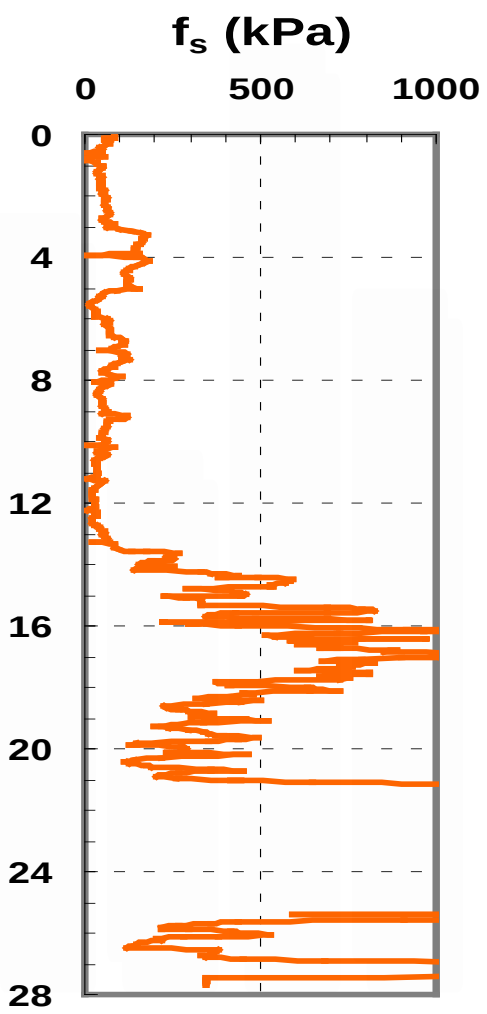
1947

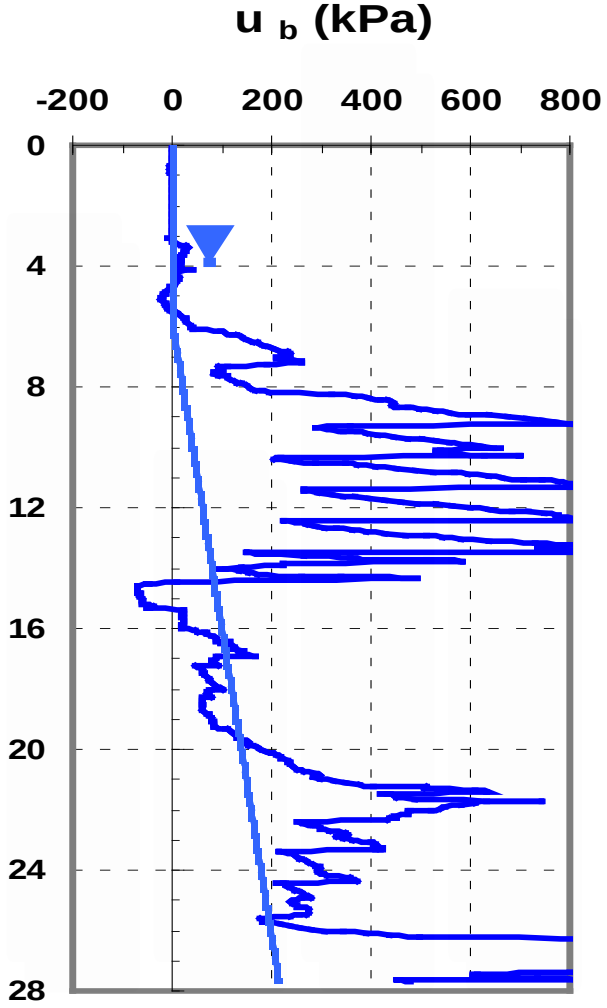
The concept of using "standard" blow counts to estimate soil properties was introduced when Terzaghi and Mohr developed correlations between allowable bearing pressure and blowcounts in sands.

Why CPT

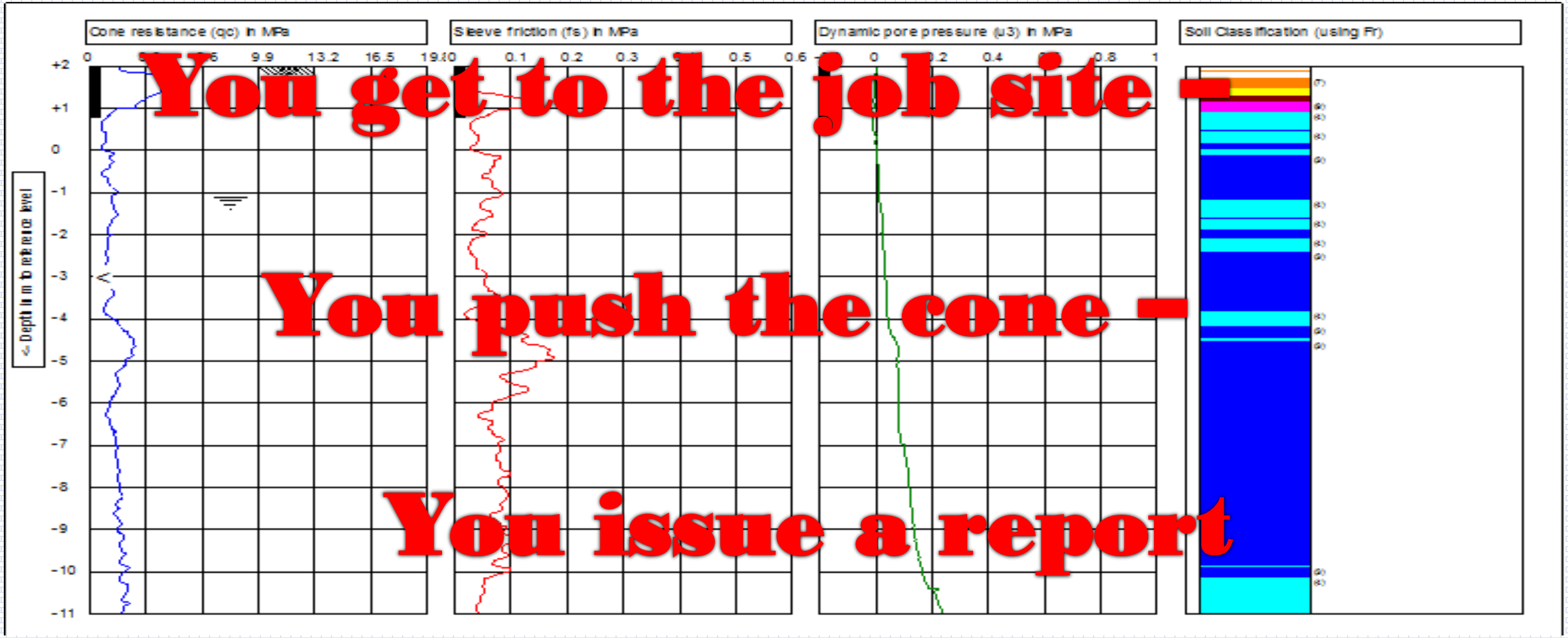

f_s
u_2
q_t


q_t (MPa)
0
20
40
60
Depth (meters)
0
4
8
12
16
20
24
28


f_s (kPa)
0
500
1000
0
4
8
12
16
20
24
28


u_b (kPa)
-200
0
200
400
600
800
0
4
8
12
16
20
24
28

Why CPT



You get to the job site –

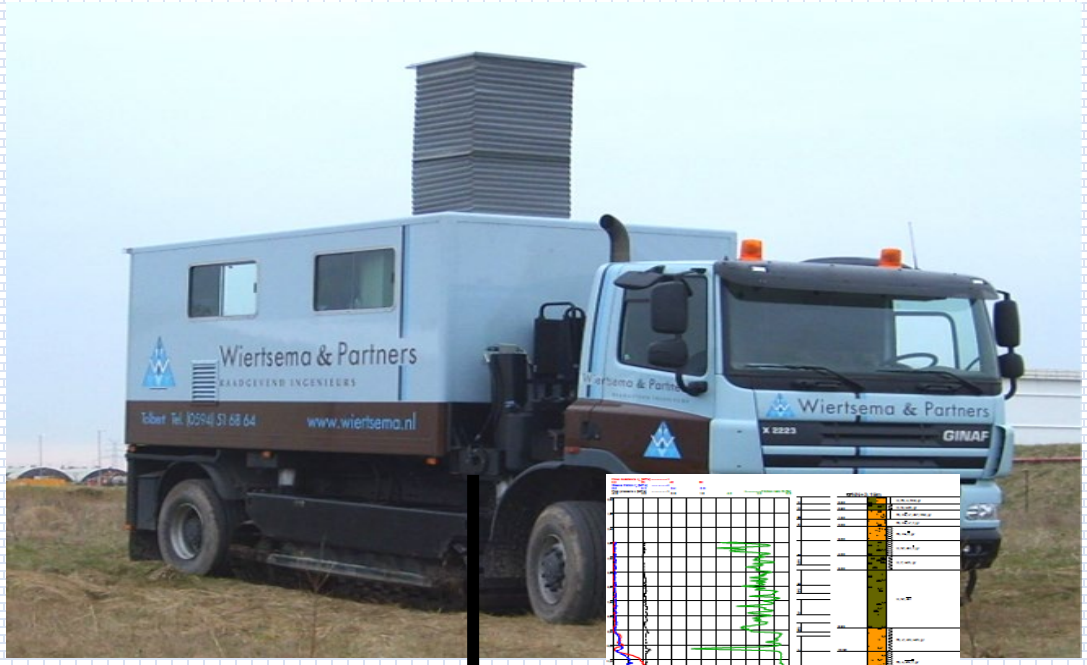
You push the cone –

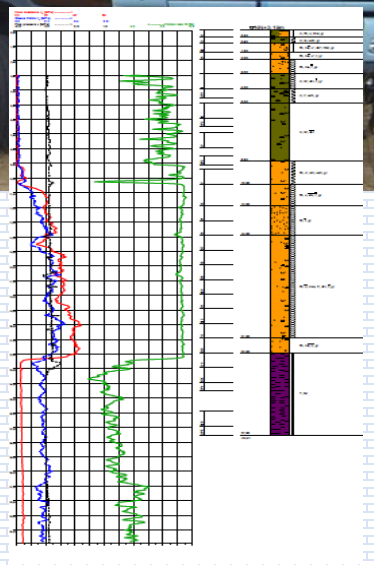
You issue a report

All in about 25 minutes

Why CPT

For site investigations CPT's are usually carried out in combination with a few boreholes for soil sampling. Next to an almost continuous profile is speed another advantage. Per unit 650 to 1000 ft (200 to 300 meter) of soil profile can be collected per day.

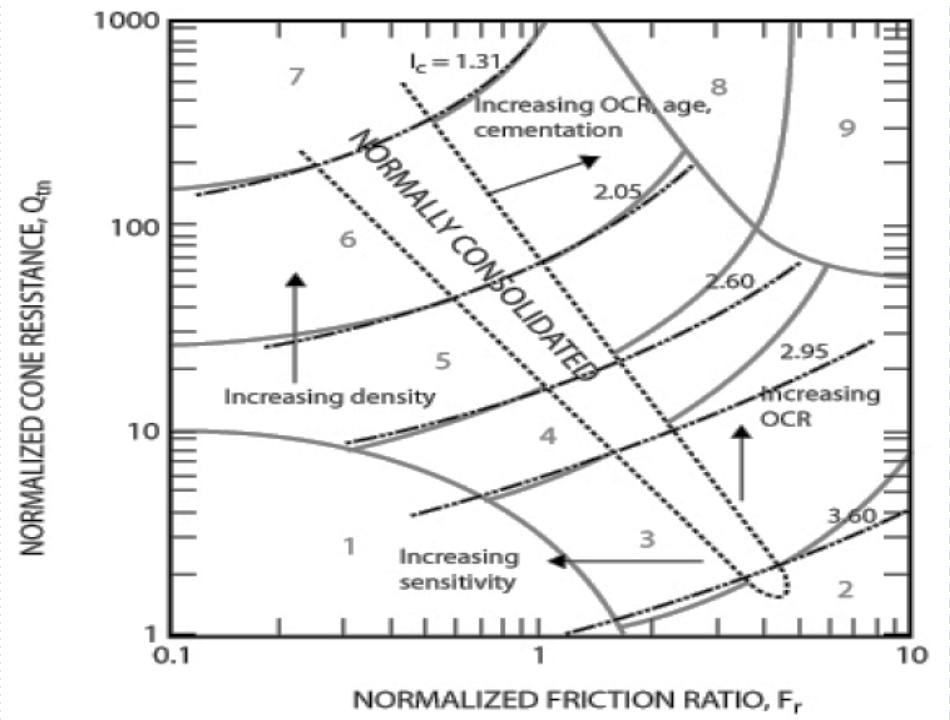




Why not CPT (or so people claim)

- **You need to "feel" the soil … really?**

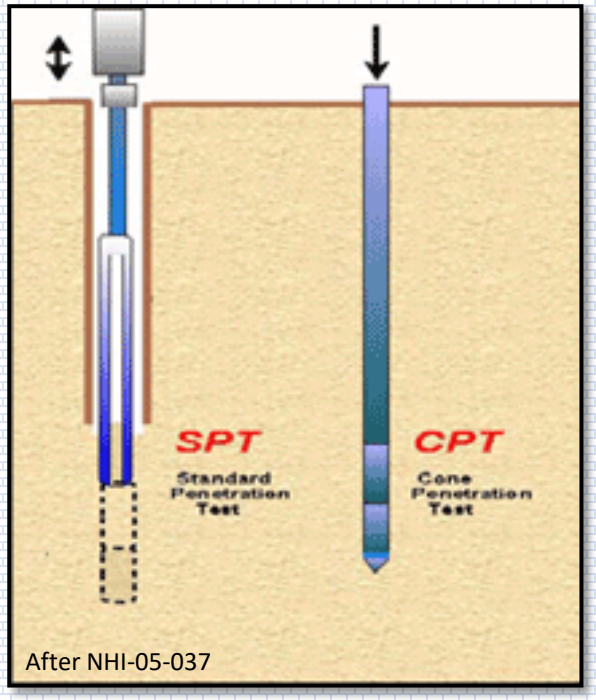
Vs.



Reality – Aspect 1

c_u = undrained strength
γ_T = unit weight
I_R = rigidity index
ϕ' = friction angle
OCR = overconsolidation
K_0 = lateral stress state
e_o = void ratio
V_s = shear wave
E' = elastic modulus
C_c = compression index
q_b = pile end bearing
f_s = pile skin friction
k = permeability
q_a = bearing stress

CLAY



After NHI-05-037

SAND

D_R = relative density
γ_T = unit weight
LI = liquefaction index
ϕ' = friction angle
c' = cohesion intercept
e_o = void ratio
q_a = bearing capacity
σ_p' = preconsolidation
V_s = shear wave
E' = elastic modulus
Ψ = dilatancy angle
q_b = pile end bearing
f_s = pile skin friction

The objective of a soil investigation is to get good information on the soil behavior

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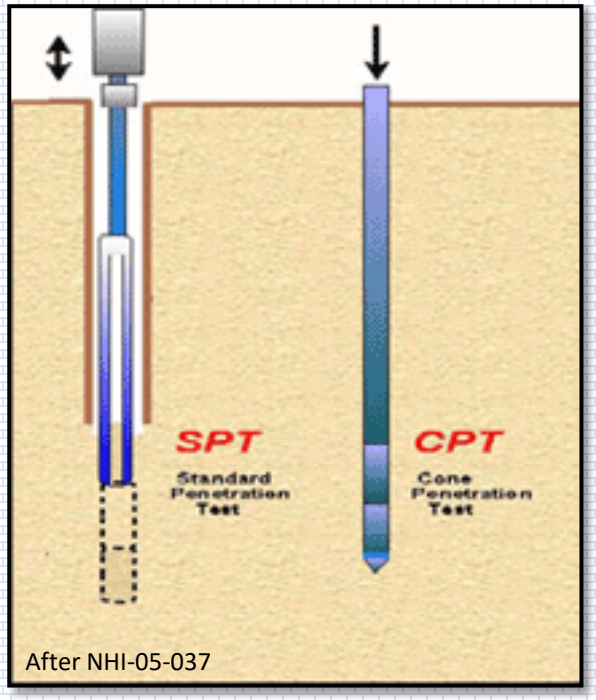

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SPT vs. CPT
N vs. tip, sleeve and pore pressure

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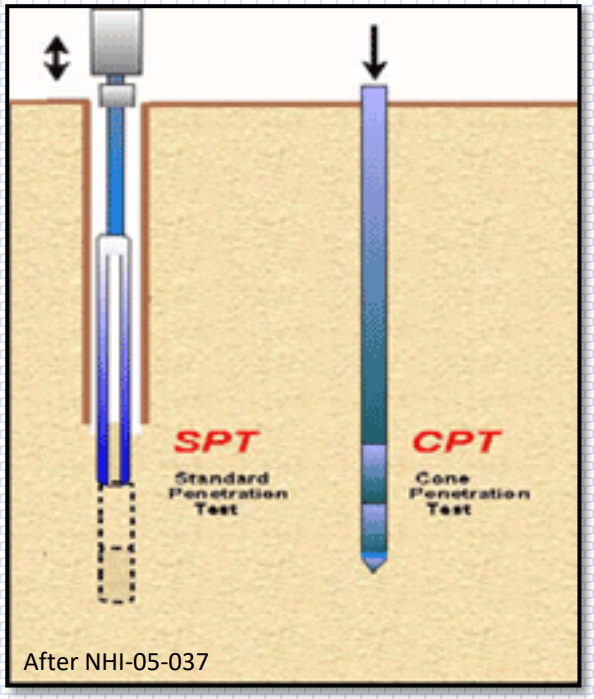

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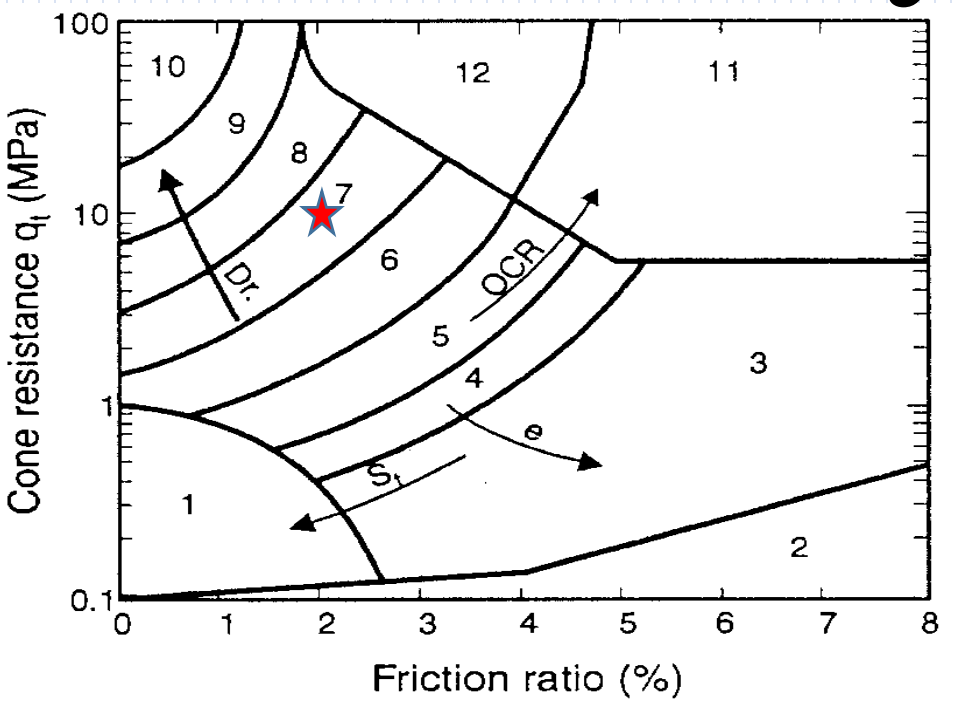
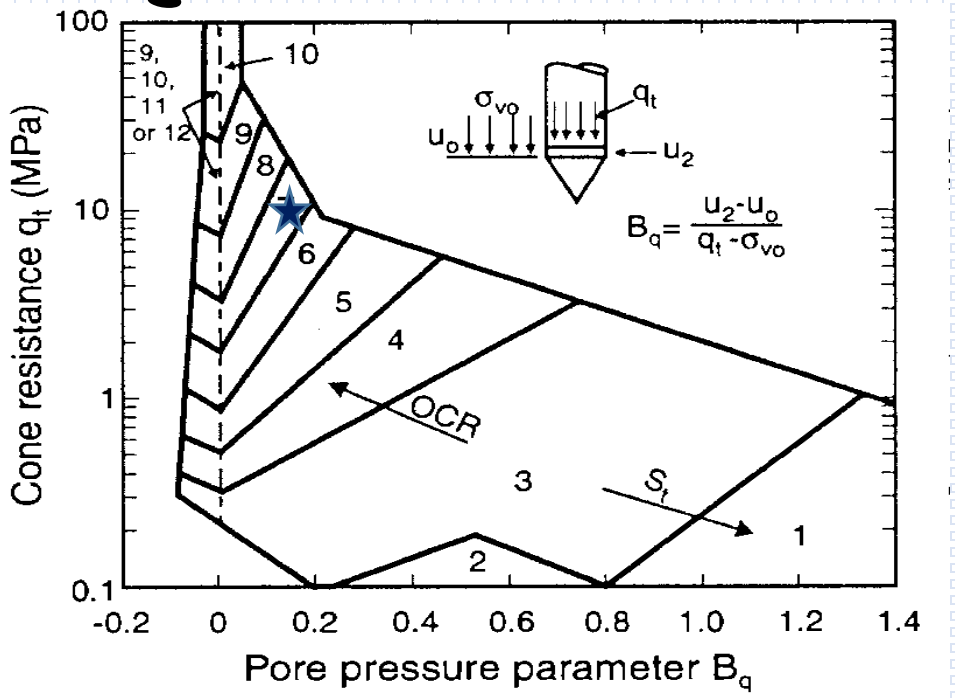
SPT vs. CPT

***f*(N) vs. *f*(tip, sleeve and pore pressure)**

Reality – Aspect 1





Soil Behavior Type (Robertson et al., 1986; Robertson & Campanella, 1988)

1 – Sensitive fine grained	**5 – Clayey silt to silty clay**	**9 – sand**
2 – Organic material	**6 – Sandy silt to silty sand**	**10 – Gravelly sand to sand**
3 – Clay	**7 – Silty sand to sandy silt**	**11 – Very stiff fine grained***
4 – Silty clay to clay	**8 – Sand to silty sand**	**12 – Sand to clayey sand***

Note: Overconsolidated or cemented

Typical Soil Behavior Classification

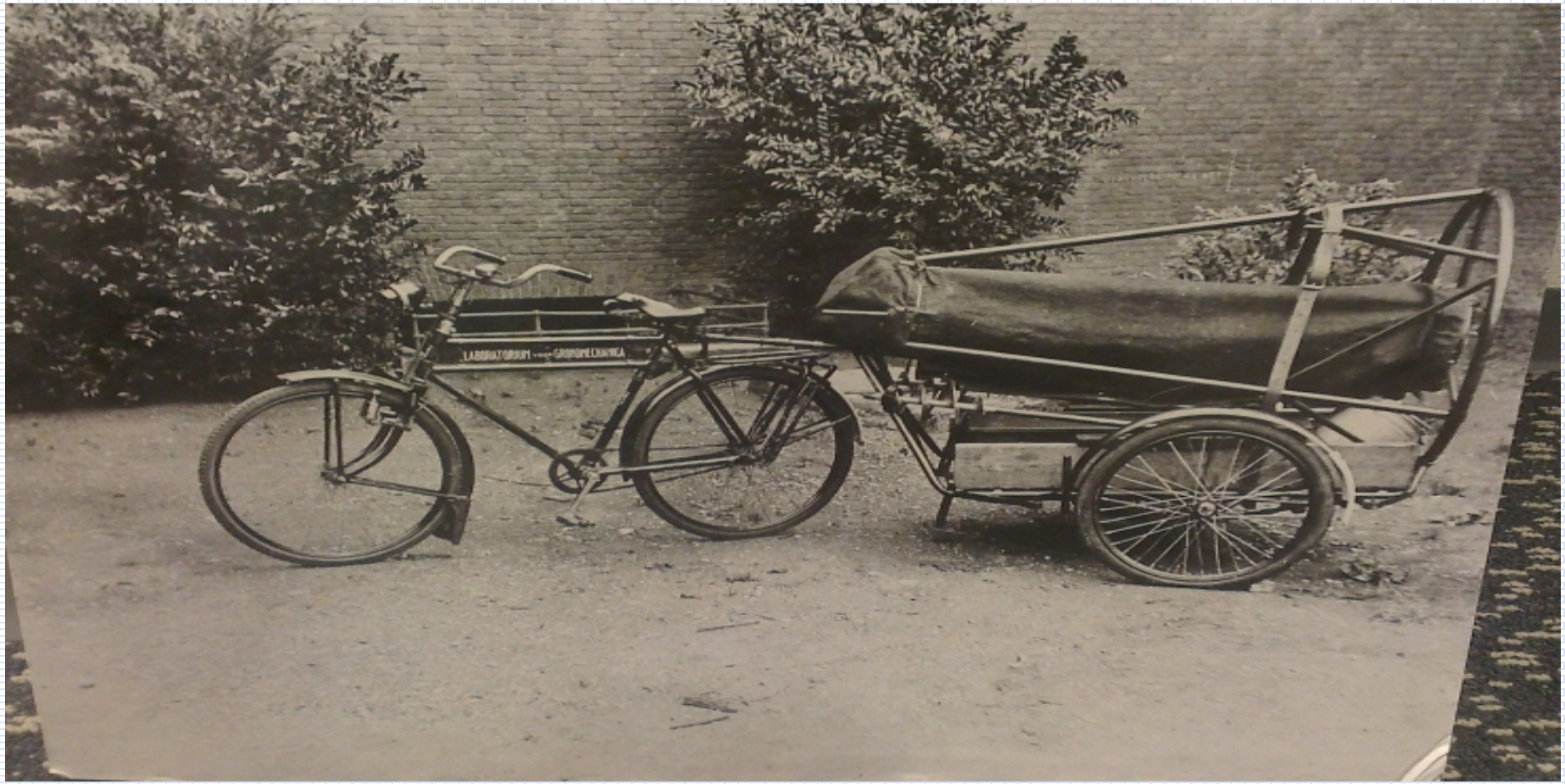
Why not CPT (or so people claim)

- **You need to "feel" the soil … really?**

- **It is supposedly a new method … or is it?**

CPT as a method to assess the bearing capacity of a foundation is older than SPT (by about 15 years)

Reality – Aspect 2
"New? It has always been there"



One of the first CPT devices (around 1940)

Reality – Aspect 2
"New? It has always been there"

50 kN hand operated CPT device

Reality – Aspect 2
"New? It has always been there"

50 kN hand operated CPT device

Reality – Aspect 2
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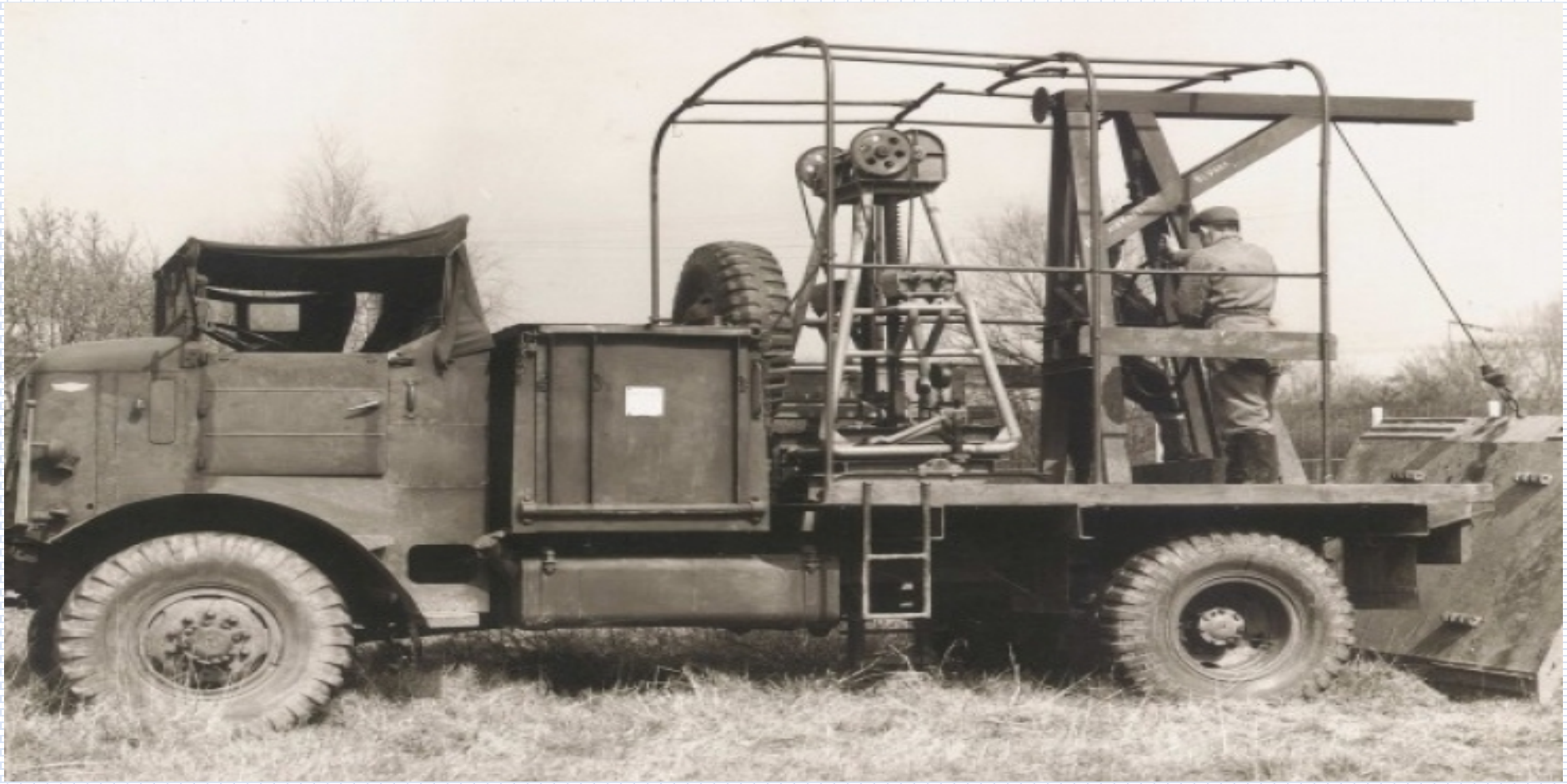
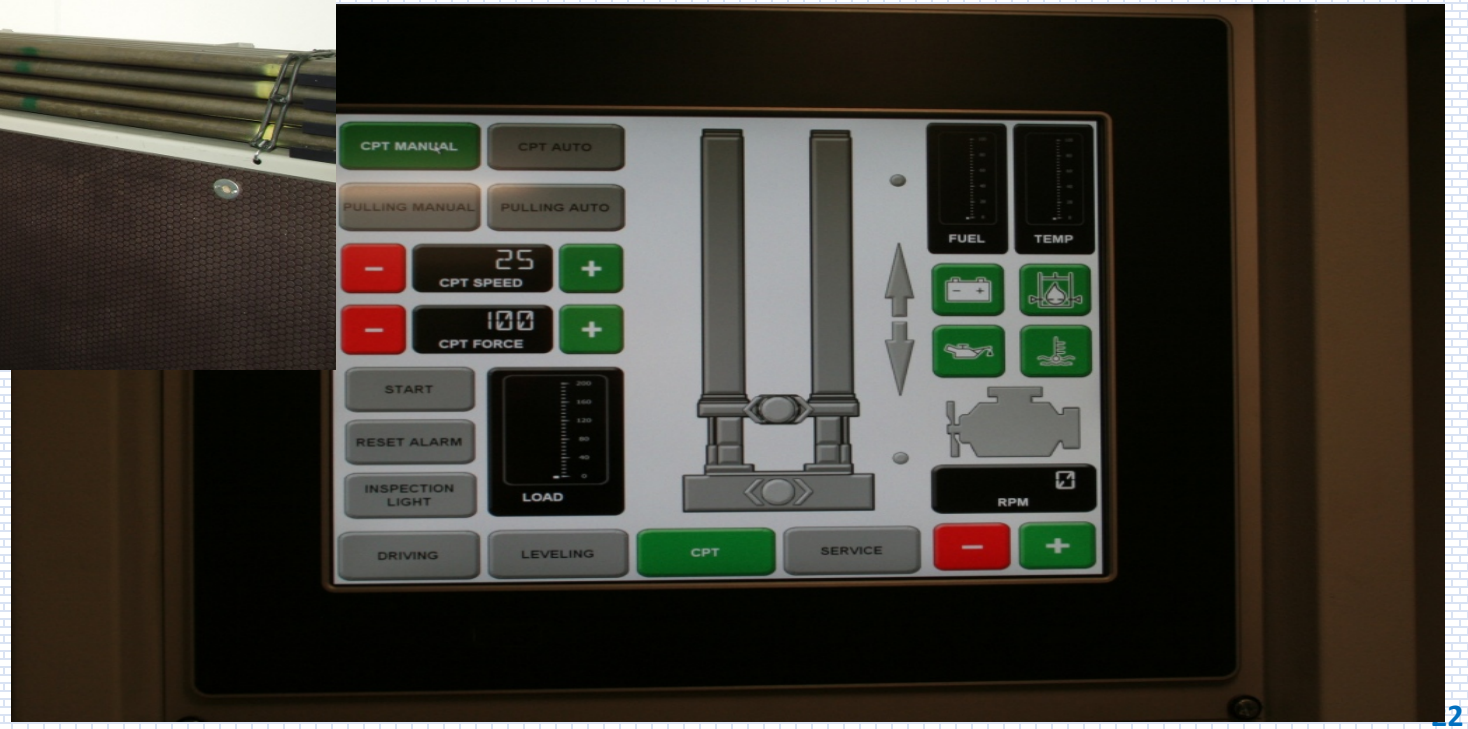


Hand operated 50 kN pusher installed on old army truck

Reality – Aspect 2
"New? It has always been there"

Manual pusher inside a truck

Reality – Aspect 2
"New? It has always been there"



Interior of a "dated" CPT truck (left) and HMI screen (right)

Why not CPT (or so people claim)

- You need to "feel" the soil … really?
- It is a supposedly a new method … or is it?
- Not specified … but things are changing

DEPARTMENT OF TRANSPORTATION

Cone Penetration Test
Design Guide for
State Geotechnical Engineers

Direct design methods

Why not CPT (or so people claim)

DEPARTMENT OF TRANSPORTATION

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U.S. Department of Transportation
Federal Highway Administration

Advanced Geotechnical Methods in Exploration (A-GaME)

DEPARTMENT OF TRANSPORTATION

Cone Penetration Test Design Guide for State Geotechnical Engineers

Reality – Aspect 3
Things are changing – consider the a-game

Up to 50 percent of major infrastructure projects suffer impacts to schedule or cost due to geotechnical issues. Many of these issues relate to risks identified directly or indirectly to the scope and quality of site characterization work. Effective site characterization is critical for recognizing potential problems that may affect design and construction and for ensuring safe, well-performing, and cost-effective projects.

Current practice for characterizing a project site will typically include a minimum number of borings with samples obtained every few feet. Drilling and sampling at discrete locations requires engineers to construct profiles of the subsurface using interpolation, which may result in uncertainty in design and construction.

Reality – Aspect 3
Things are changing – consider the a-game

Several proven, effective, and underutilized technologies are available that, when combined with processes that assess risk and variability, allow optimization of subsurface exploration programs for improved site characterization and maximum return-on-investment. These technologies include cone penetration testing.

U.S. Department of Transportation
Federal Highway Administration

We have known this since the 1930s

Why not CPT (or so people claim)

- **You need to "feel" the soil … really?**
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U.S. Department of Transportation
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Advanced Geotechnical Methods in Exploration (A-GaME)

DEPARTMENT OF TRANSPORTATION

Cone Penetration Test Design Guide for State Geotechnical Engineers

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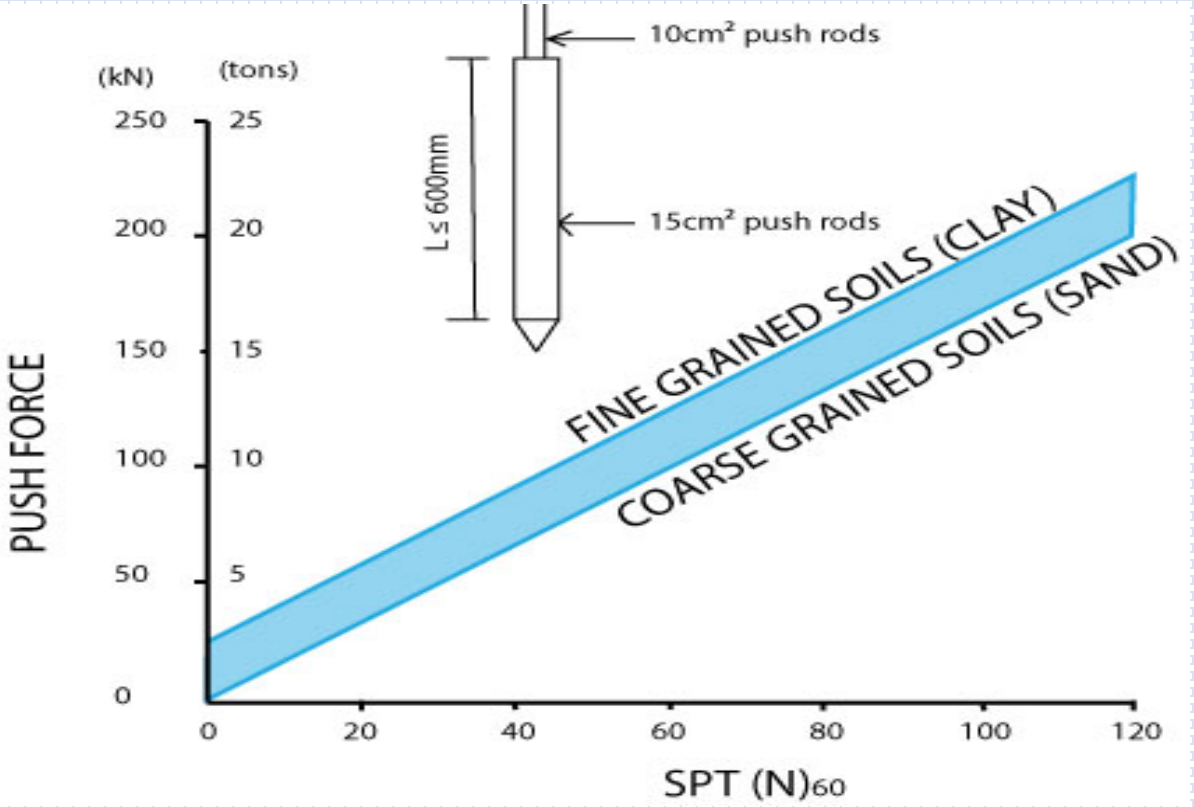
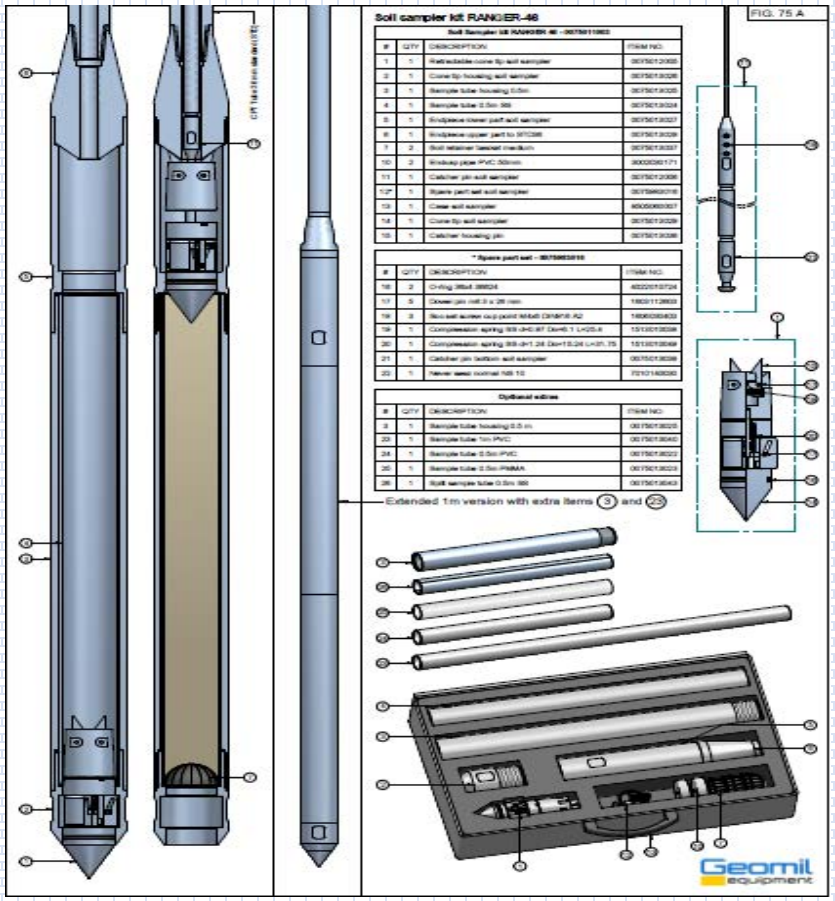
Why not CPT (or so people claim)

- **You need to "feel" the soil … really?**
- **It is a supposedly a new method … or is it?**
- **Not specified … but things are changing**
- **Too hard … or is it?**



Why not CPT (or so people claim)

- You need to “feel” the soil … really?
- It is a supposedly a new method … or is it?
- Not specified … but things are changing
- Too hard … or is it?
- No samples …. why not?



Why not CPT (or so people claim)

- You need to "feel" the soil … really?
- It is a supposedly a new method … or is it?
- Not specified … but things are changing
- Too hard … or is it?
- No samples …. why not?
- Boulders or debris … yes, that's a problem

You can break a cone!

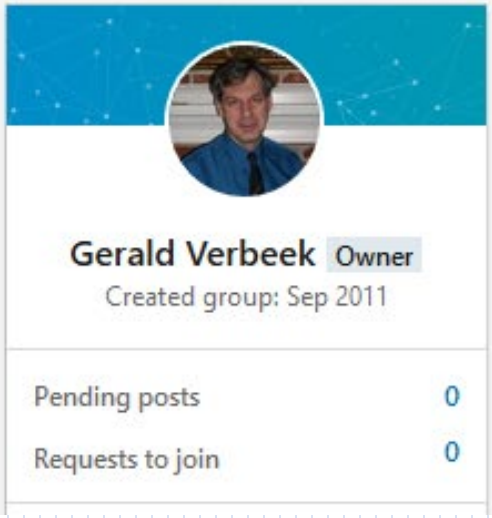
Why not CPT (or so people claim)

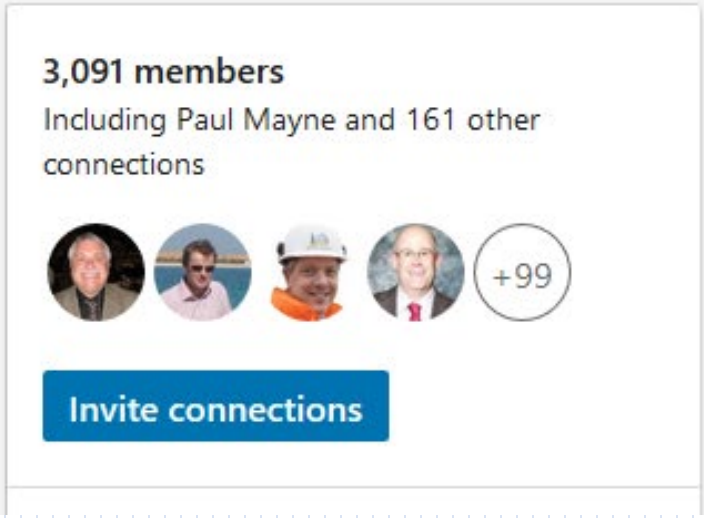
- You need to "feel" the soil … really?
- It is a supposedly a new method … or is it?
- Not specified … but things are changing
- Too hard … or is it?
- No samples …. why not?
- Boulders or debris … yes, that's a problem

No method is perfect …

but give it a try and see what it can do

CPT Discussion Group on LinkedIn





This group is established to exchange knowledge and information on CPT, and the solutions it offers for a wide variety of geotechnical engineering applications. With the support of our partner Eijkelkamp GeoPoint SoilSolutions we intend to make this group not only a discussion forum for CPT issues, but also a tool to promote CPT as the preferred soil investigation method around the world.

Cone Penetration Testing

a proven, but underutilized site investigation method

For more information
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(903) 216 5372

Allnamics

CPT based engineering in The Netherlands

R.F. van Dorp & M.W. Bielefeld

Contents

- Engineering applications for CPT
  - Soil profiling
  - Estimation of soil properties & parameters for (preliminary) geotechnical design
  - Pile design
  - Trouble shooting

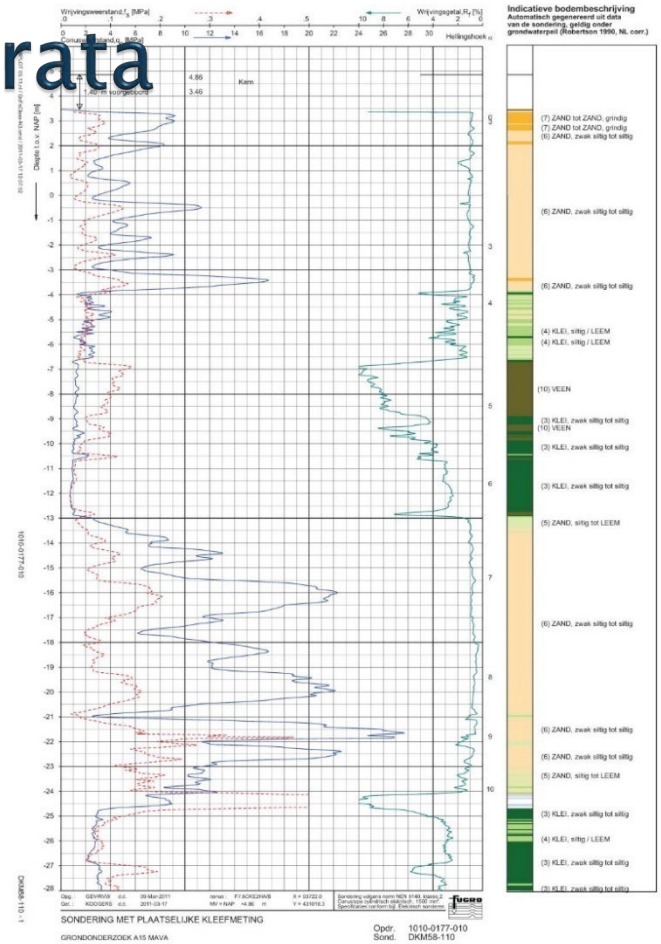
- Case study
  - Highway design based on CPT

Caveat

Since the design and engineering standards in The Netherlands are different from those in the USA this presentation will focus on the approach rather than numerical values

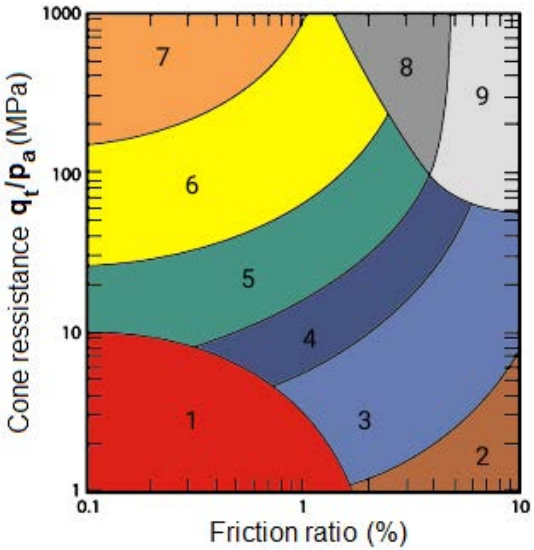
CPT – Detection bearing strata

- Cone resistance is a reliable parameter for:
  - Soil strength
  - Soil density
  - Calculation of pile bearing capacity

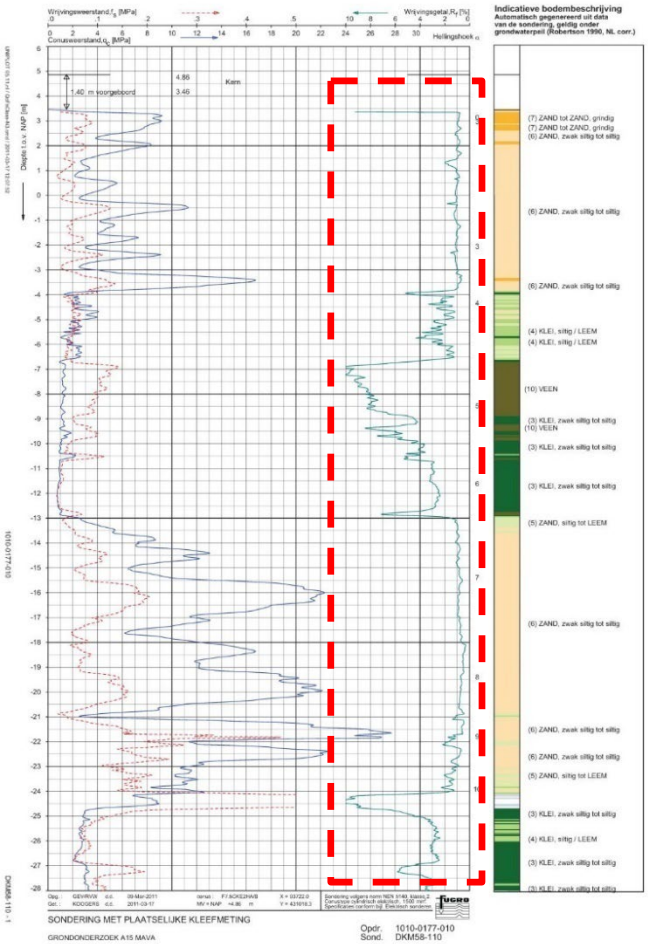


CPT – Soil profiling

- Friction ratio can be used for identification of soil types (Robertson, 1990)
- And thus the soil profile can be established

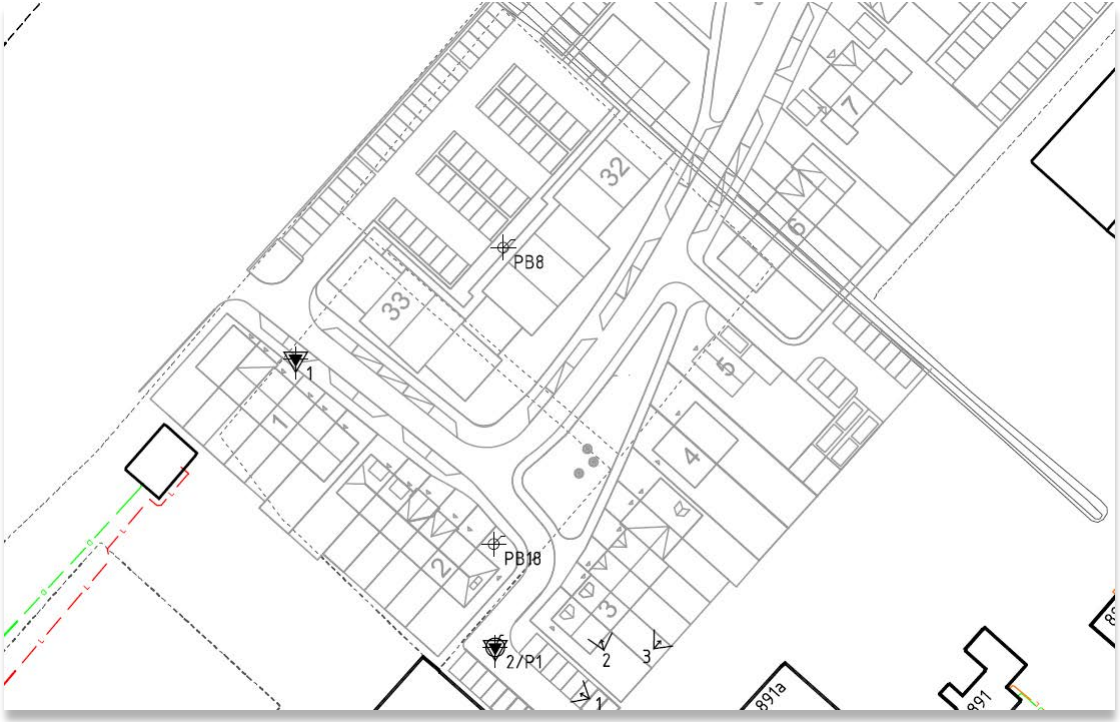


Zone	Soil Behavior Type (SBT)
1	Sensitive fine grained
2	Organic material
3	Clay
4	Silty Clay to clay
5	Clayey silt to silty clay
6	Sandy silt to clayey silt
7	Silty sand to sandy silt
8	Sand to silty sand
9	Sand



CPT – Soil profiling

original plan
for CPTs



CPT – Soil profiling

site variability detected by multiple CPTs

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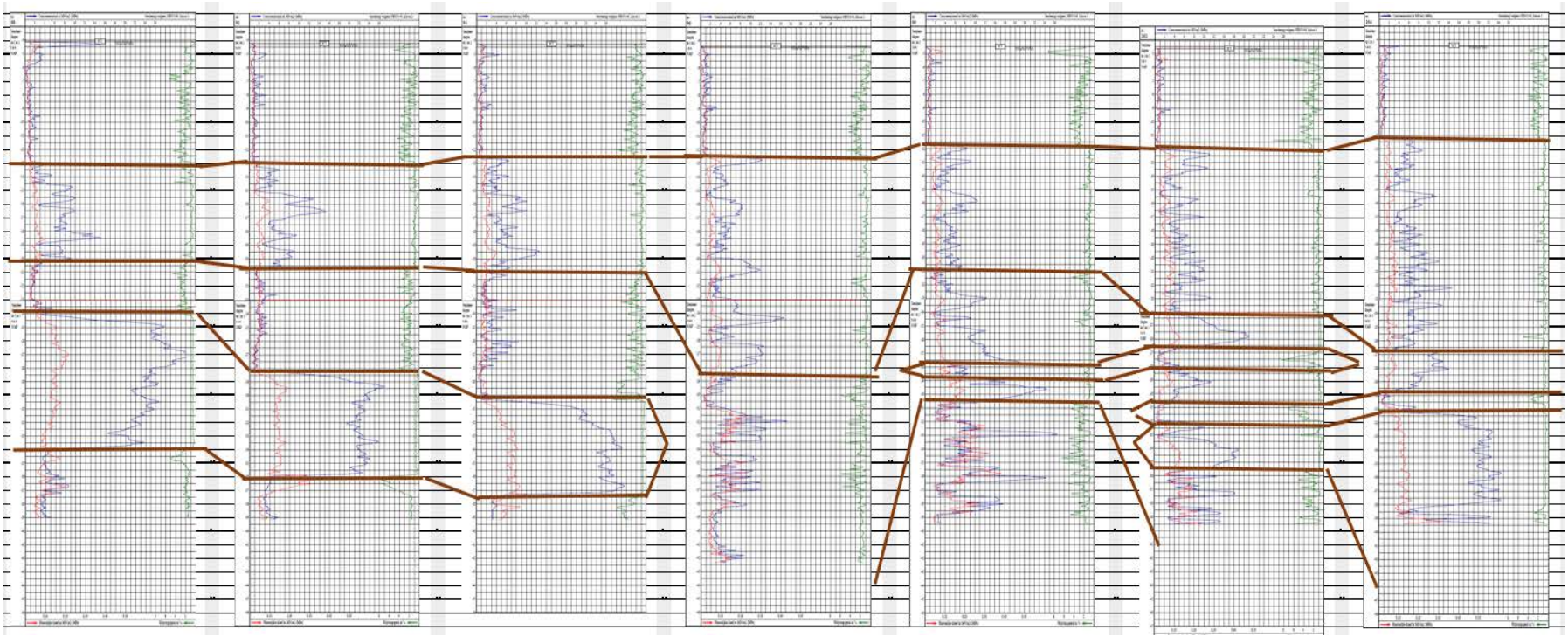
site variability detected by multiple CPTs



Design Parameters

The Dutch code gives indicative design parameters based on identified soil type and Q_c

Grondsoort			Karakteristieke waarde [a] van grondeigenschap																	
Hoofd-naam	Bijmengsel	Consis-tentie [b]	γ [c] kN/m³		γ_{sat} kN/m³		q_c [d g] MPa	C'_p [g]		C'_s		$C_c/(1+e_0)$ [g] [-]		C_α [f] [-]		$C_{sw}/(1+e_0)$ [g] [-]		E_{100} [g h] MPa		φ' [g] Graden		c' kPa		c_u kPa
Grind	Zwak siltig	Los	17		19		15	500		∞		0,004 6		0		0,001 5		45		32,5		0		
		Matig	18		20		25	1 000		∞		0,002 3		0		0,000 8		75		35,0		0		N.v.t.
		Vast	19	20	21	22	30	1 200	1 400	∞		0,001 9	0,001 6	0		0,000 6	0,000 5	90	105	37,5	40,0	0		
	Sterk siltig	Los	18		20		10	400		∞		0,005 8		0		0,001 9		30		30,0		0		
		Matig	19		21		15	600		∞		0,003 8		0		0,001 3		45		32,5		0		N.v.t.
		Vast	20	21	22	22,5	25	1 000	1 500	∞		0,002 3	0,001 5	0		0,000 8	0,000 5	75	110	35,0	40,0	0		
Zand	Schoon	Los	17		19		5	200		∞		0,011 5		0		0,003 8		15		30,0		0		
		Matig	18		20		15	600		∞		0,003 8		0		0,001 3		45		32,5		0		N.v.t.
		Vast	19	20	21	22	25	1 000	1 500	∞		0,002 3	0,001 5	0		0,000 8	0,000 5	75	110	35,0	40,0	0		
	Zwak siltig, kleiig		18	19	20	21	12	450	650	∞		0,005 1	0,003 5	0		0,001 7	0,001 2	35	50	27,0	32,5	0		N.v.t.
	Sterk siltig, kleiig		18	19	20	21	8	200	400	∞		0,011 5	0,005 8	0		0,003 8	0,001 9	15	30	25,0	30,0	0		N.v.t.
Leem [e]	Zwak zandig	Slap	19		19		1	25		650		0,092 0		0,003 7		0,030 7		2		27,5	30,0	0		50	
		Matig	20		20		2	45		1 300		0,051 1		0,002 0		0,017 0		3		27,5	32,5	1		100	
		Vast	21	22	21	22	3	70	100	1 900	2 500	0,032 9	0,023 0	0,001 3	0,000 9	0,011 0	0,007 7	5	7,5	30,0	32,5	3,8		200	300
	Sterk zandig		19	20	19	20	2	45	70	1 300	2 000	0,051 1	0,032 9	0,002 0	0,001 3	0,017 0	0,011 0	3	5	27,5	35,0	0	1	50	100
Klei	Schoon	Slap	14		14		0,5	7		80		0,328 6		0,013 1		0,109 5		1		17,5		0		25	
		Matig	17		17		1,0	15		160		0,153 3		0,006 1		0,051 1		2		17,5		5		50	
		Vast	19	20	19	20	2,0	25	40	320	500	0,092 0	0,076 7	0,003 7	0,003 1	0,030 7	0,025 6	4	10	17,5	25,0	13	15	100	200
	Zwak zandig	Slap	15		15		0,7	10		110		0,230 0		0,009 2		0,076 7		1,5		17,5		5		40	
		Matig	18		18		1,5	20		240		0,115 0		0,004 6		0,038 3		3		22,5		5		80	
		Vast	20	21	20	21	2,5	30	50	400	600	0,076 7	0,046 0	0,003 1	0,001 8	0,025 6	0,015 3	5	10	22,5	27,5	13	15	120	170
	Sterk zandig	-	18	20	18	20	1,0	25	140	320	1 680	0,092 0	0,016 4	0,003 7	0,000 7	0,030 7	0,005 5	2	5	27,5	32,5	0	1	0	10
	Organisch	Slap	13		13		0,2	7,5		20		0,306 7		0,015 3		0,102 2		0,5		15,0		0	1	10	
		Matig	15	16	15	16	0,5	10	15	40	60	0,230 0	0,153 3	0,011 5	0,007 7	0,076 7	0,051 1	1,0	2,0	15,0		0	1	25	30
Veen	Niet voorbelast	Slap	10	12	10	12	0,1	5	7,5	20	30	0,460 0	0,306 7	0,023 0	0,015 3	0,153 3	0,102 2	0,2	0,5	15,0		1	2,5	10	20
	Matig voorbelast	Matig	12	13	12	13	0,2	7,5	10	30	40	0,306 7	0,230 0	0,015 3	0,011 5	0,102 2	0,076 7	0,5	1,0	15,0		2,5	5	20	30
Variatiecoëfficiënt v			0,05				–	0,25										0,10				0,20			

Settlement

Sheet piles

Fills

Footings

Embankments

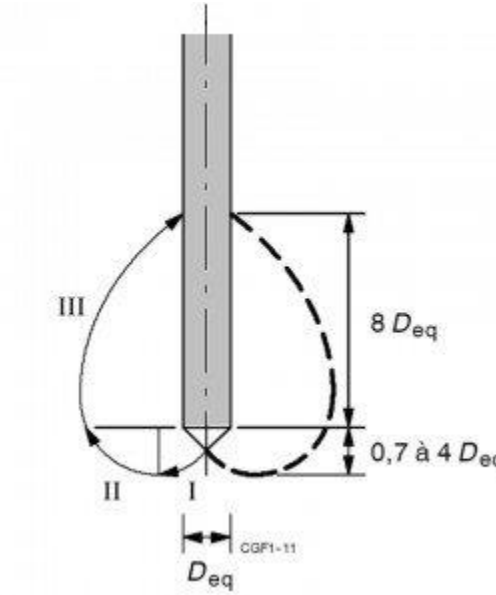
Slope stability

Land reclamations

Toe resistance design method: Koppejan

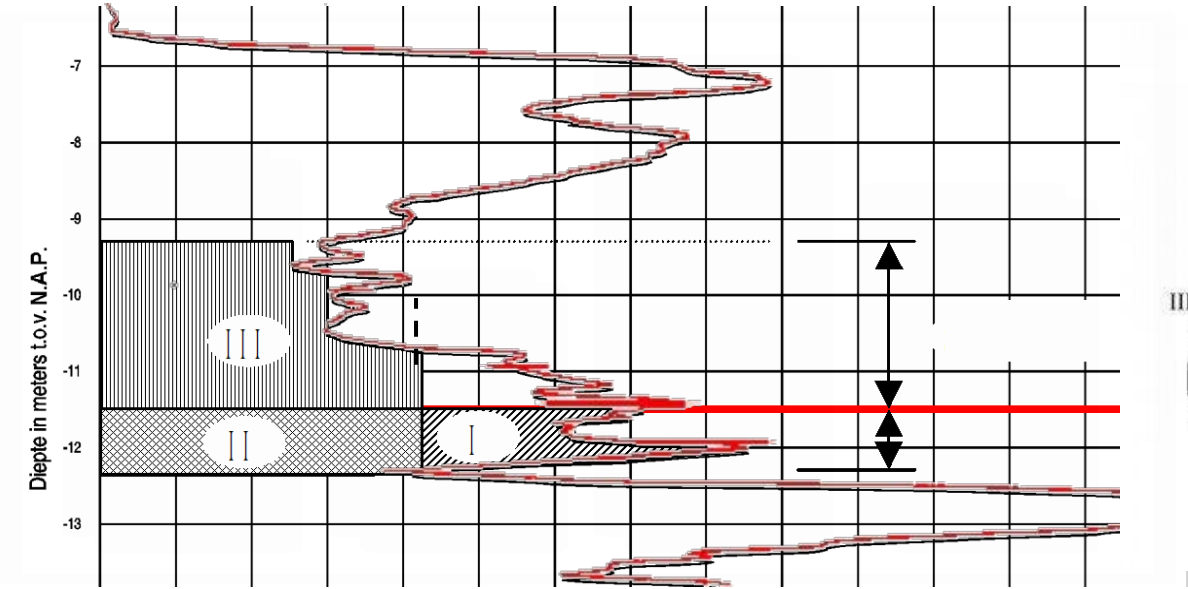
- Named after engineer A.W. Koppejan
  - 1948: “A formula combining the Terzaghi Load-compression relationship and the Buisman time effect
    - Proceeding of the 2nd international conference Soil Mechanics and Foundation Engineering, Rotterdam 1948
  - 1952: “Length and capacity of driven piles”

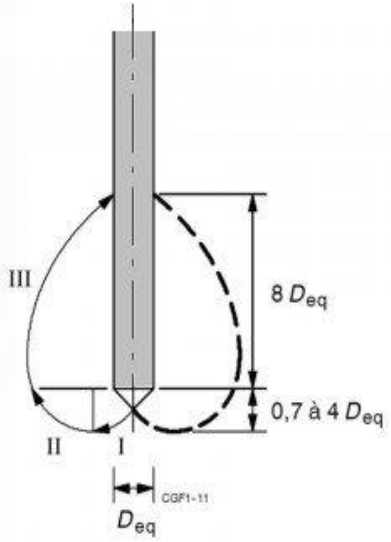
- Conversion of the resistance of a small diameter cone to that of a larger diameter pile based on ratio between penetration depth and diameter



Toe resistance design method: Koppejan

$$p_{r;max} = \frac{1}{4} q_{c;I;gem} + \frac{1}{4} q_{c;II;gem} + \frac{1}{2} q_{c;III;gem}$$





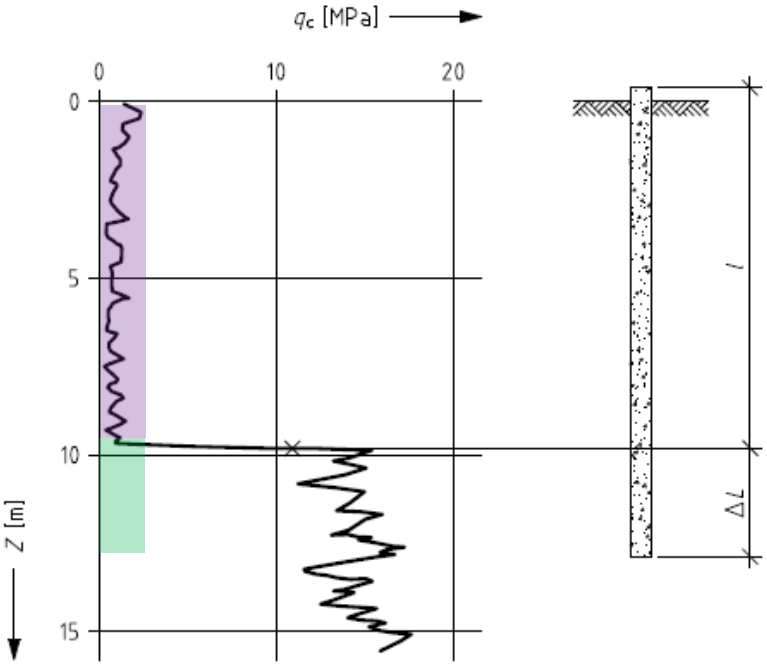
https://www.youtube.com/watch?v=y6Gjl1VhNWw

Dutch pile design code using CPT

- Direct design method

- Total resistance
  - Toe resistance R_b
  - Shaft friction R_s
  - Negative skin friction $F_{s,nk}$

- $R_{pile,nett} = R_b + R_s - F_{s,nk}$



Dutch pile design code using CPT

(basics, unfactored presentation here)

$$R_{pile,nett} = R_b + R_s - F_{s,nk}$$

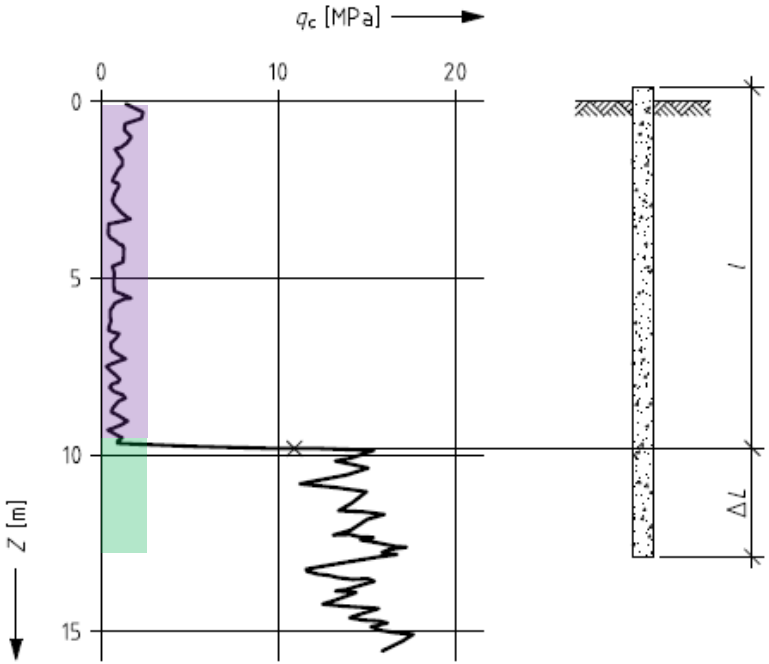
$$R_b = A_{pile} * \beta * s * \alpha_p * p_{r,max}$$

$$p_{r;max} = \frac{1}{4} q_{c;I;gem} + \frac{1}{4} q_{c;II;gem} + \frac{1}{2} q_{c;III;gem}$$

$$R_s = O_{s;\Delta L;gem} \times \int_{\Delta L} \alpha_s \times q_{c;z;a} \times dz$$

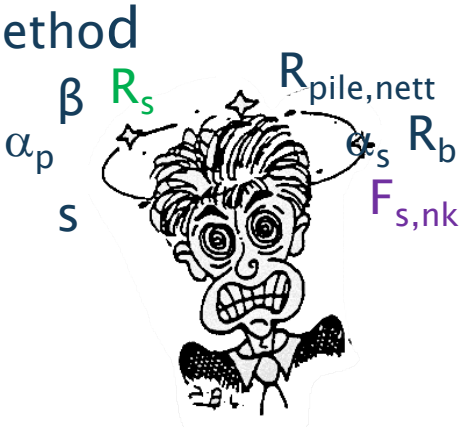
$$F_{s,nk} = O_{s;gem} \times \sum_{j=1}^{j=n} d_j \times K_{0;j;k} \times \tan(\delta_{j;k}) \times \frac{\sigma'_{v;j-1;rep} + \sigma'_{v;j;rep}}{2}$$

$$\approx C_{pile} * L_{nk} * 0{,}25 * \sigma_{k,avg}$$



Dutch pile design code using CPT

- Relationship between resistance and CPT
- Except diameter, cross sectional area, penetration depth L and ΔL
- Factors:
  - β = factor depending on ratio toe diameter / shaft diameter
  - α_p = factor depending on pile type & installation method
  - s = factor depending cross section shape
  - α_s = factor depending on pile type & installation method
- Factors are defined in table 7c of the code



Dutch pile design code using CPT

(basics, unfactored presentation here)

Tabel 7.c — Waarden voor de paalklassefactor α_p, α_s en voor α_t
(in het geval van zand en zand-grindhoudende grond)

Paaltype			Paalklassefactor [a]			Last-zakkingslijn figuren 7.n en 7.o
Type	Nadere specificatie	Wijze van installeren	α_p	α_s	α_t	
Beton-paal	Geprefabriceerd; met constante dwarsafmeting	Geheid	0,7	0,010	0,007	1
	beton direct tegen de grond dr	heid; de mantelbuis dt terugheiend in nbinatie met statisch trekken uit de	0,7	0,014	0,012[c]	1
	In de grond gevormd met een gladde mantelbuis op een voetplaat, waarbij het beton direct tegen de grond drukt	Gehe wordt comb statisch trekken uit de grond verwijderd; de voetplaat blijft in de grond achter				
	In de grond gevormd met een gladde mantelbuis op een schroefpunt, waarbij het beton direct tegen de grond drukt	Geschroefd; bij het trekken van de mantelbuis blijft de schroefpunt in de grond achter	0,63	0,009	0,009	1
	In de grond gevormd met behulp van een avegaar	Geschroefd	0,56	0,006	0,0045	

Pile type

Pile subtype

Pile installation method

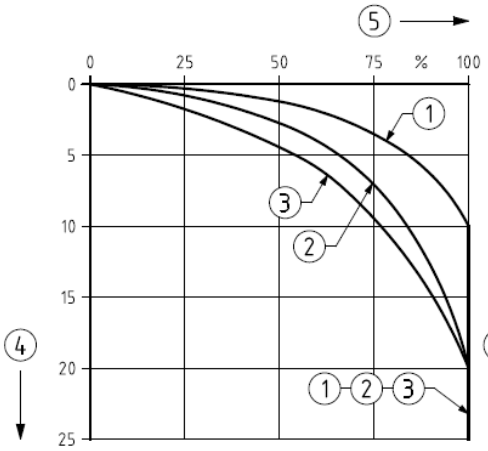
Applicable factors

Applicable load-displacement curve

Dutch pile design code using CPT

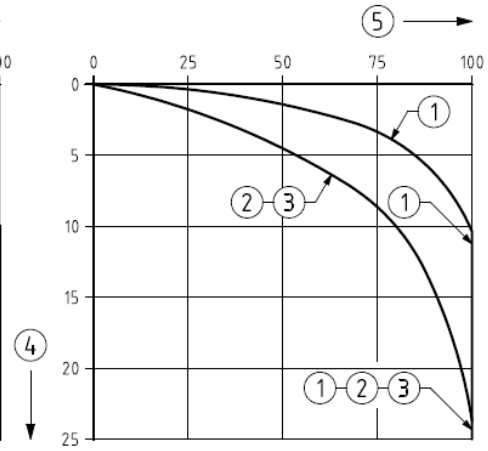
Calculation method includes estimation of displacement and mobilized resistance

- Curves based on post analysis of (decades of) static load tests
- Separate curves for shaft friction and toe resistance (to be combined)
- Different curves depending on characteristic of pile & installation method
- Curves contain relative values, except displacement for shaft friction



Legenda

1 last-zakkingslijn 1 volgens tabel 7.c
2 last-zakkingslijn 2 volgens tabel 7.c
3 last-zakkingslijn 3 volgens tabel 7.c
4 $\frac{s_b}{D_{eq}}$, in %
5 $\frac{R_{b;k}}{R_{b;k;max}}$, in %



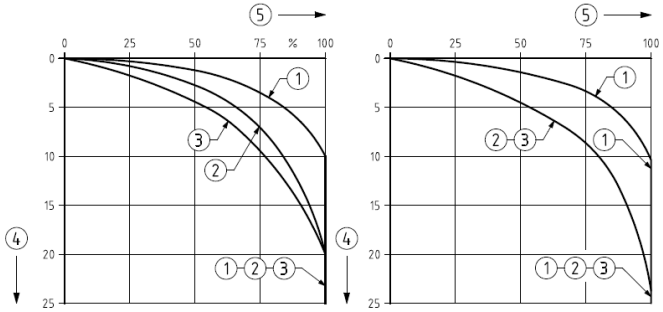
Legenda

1 last-zakkingslijn 1 volgens tabel 7.c
2 last-zakkingslijn 2 volgens tabel 7.c
3 last-zakkingslijn 3 volgens tabel 7.c
4 s_b, in mm
5 $\frac{R_{s;k}}{R_{s;k;max}}$, in %

Dutch pile design code using CPT

Calculation method includes estimation of displacement and mobilized resistance

- Curves for shaft friction and toe resistance combined to total pile head displacement
- Elastic deformation included



Legenda

1 last-zakkingslijn 1 volgens tabel 7.c
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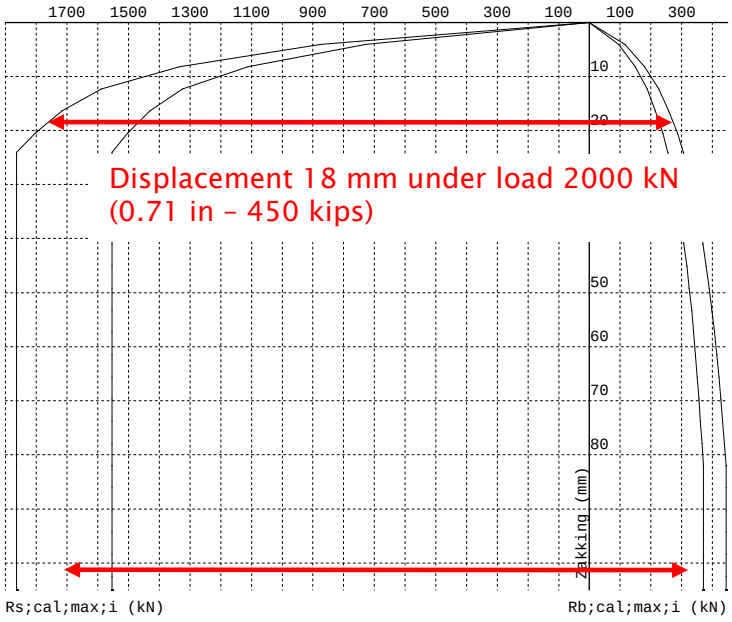
3 last-zakkingslijn 3 volgens tabel 7.c
4 $\frac{s_b}{D_{eq}}$, in %
5 $\frac{R_{b;k}}{R_{b;k;max}}$, in %

Legenda

1 last-zakkingslijn 1 volgens tabel 7.c
2 last-zakkingslijn 2 volgens tabel 7.c
3 last-zakkingslijn 3 volgens tabel 7.c
4 s_b, in mm
5 $\frac{R_{s;k}}{R_{s;k;max}}$, in %



Dutch pile design code using CPT

Engineers can rely confidently on this direct design method without the formal need for load testing

- the same method has been used for more than 70 years
- the method is used for every pile foundation in The Netherlands, for public and private projects
- the method has been validated in the past by load testing

Dutch pile design code using CPT

- Design based on CPT
- Control of pile installation by
  driving records and
  low strain dynamic integrity testing
- No load tests to confirm pile capacity

CPT used for trouble shooting

Check next to or between piles

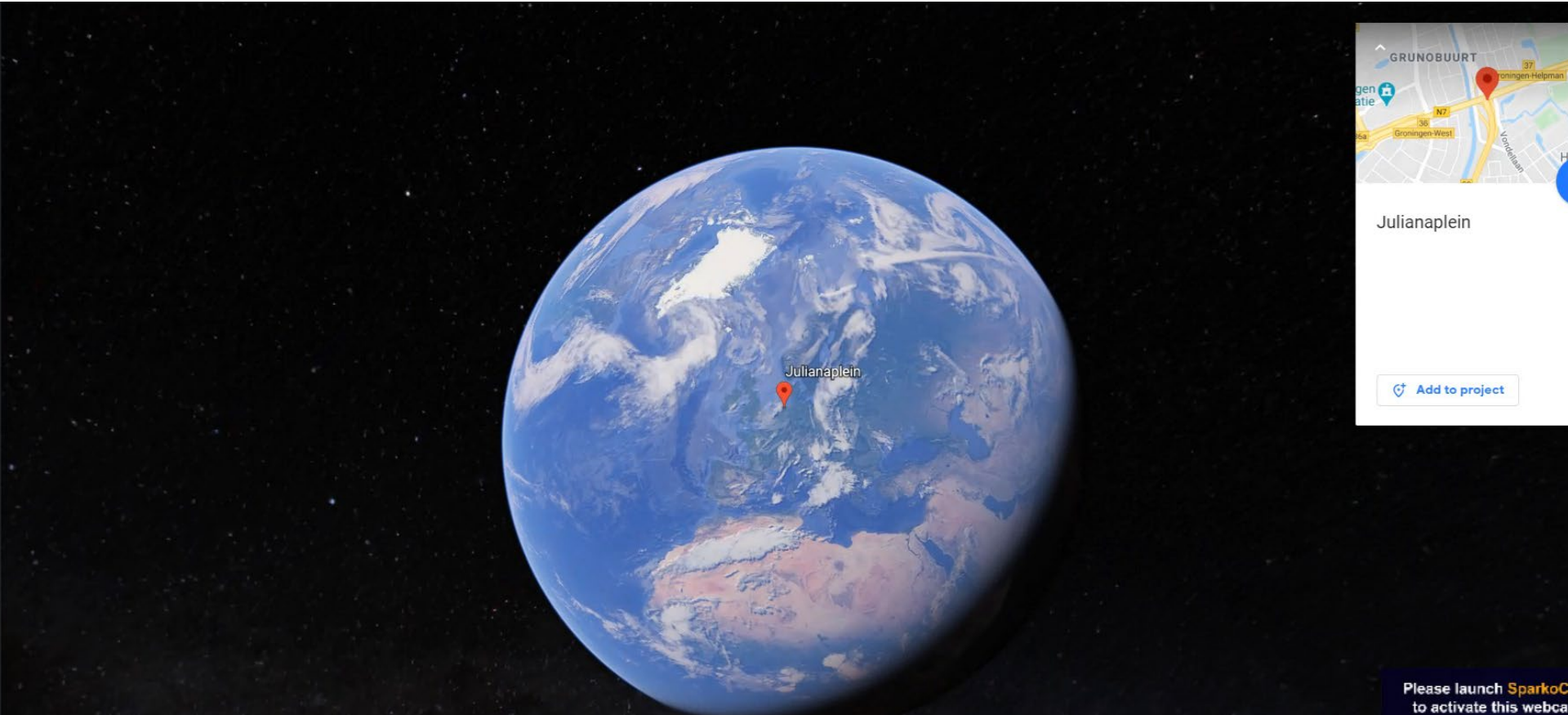
- Disturbances due to pile installation
- Suspicious low strain dynamic integrity testing results
- Suspicious driving records
- Recalculation of the capacity of critical piles

CPT used for trouble shooting

Check directly adjacent to a pile

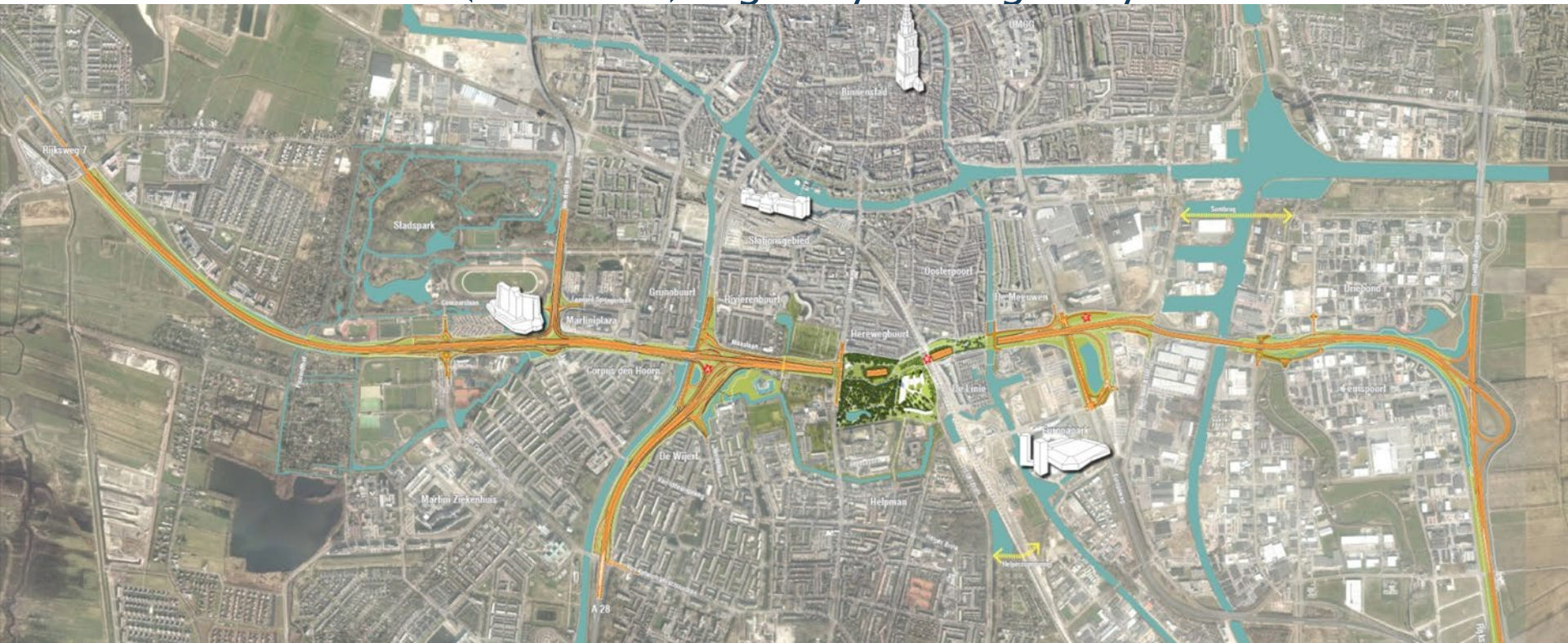
- for obstacles after refusal
- soil disturbance due to foundation installation

Case Study: Road Project Groningen



Case Study: Road Project Groningen

Renewal 12 km (7.5 miles) highway through city center



Case Study: Road Project Groningen

Old and New

Case Study: Road Project Groningen

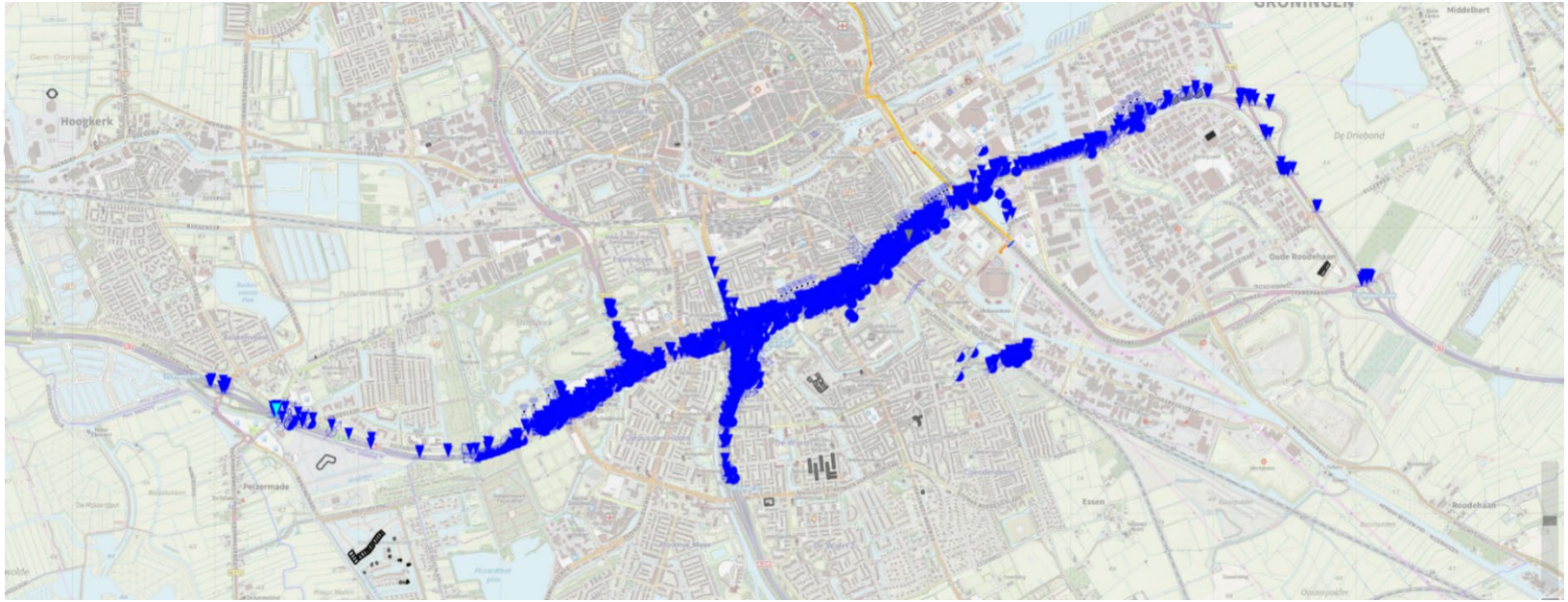
Old and New

Case Study: Road Project Groningen

12 km (7.5 miles) tunnels and viaducts

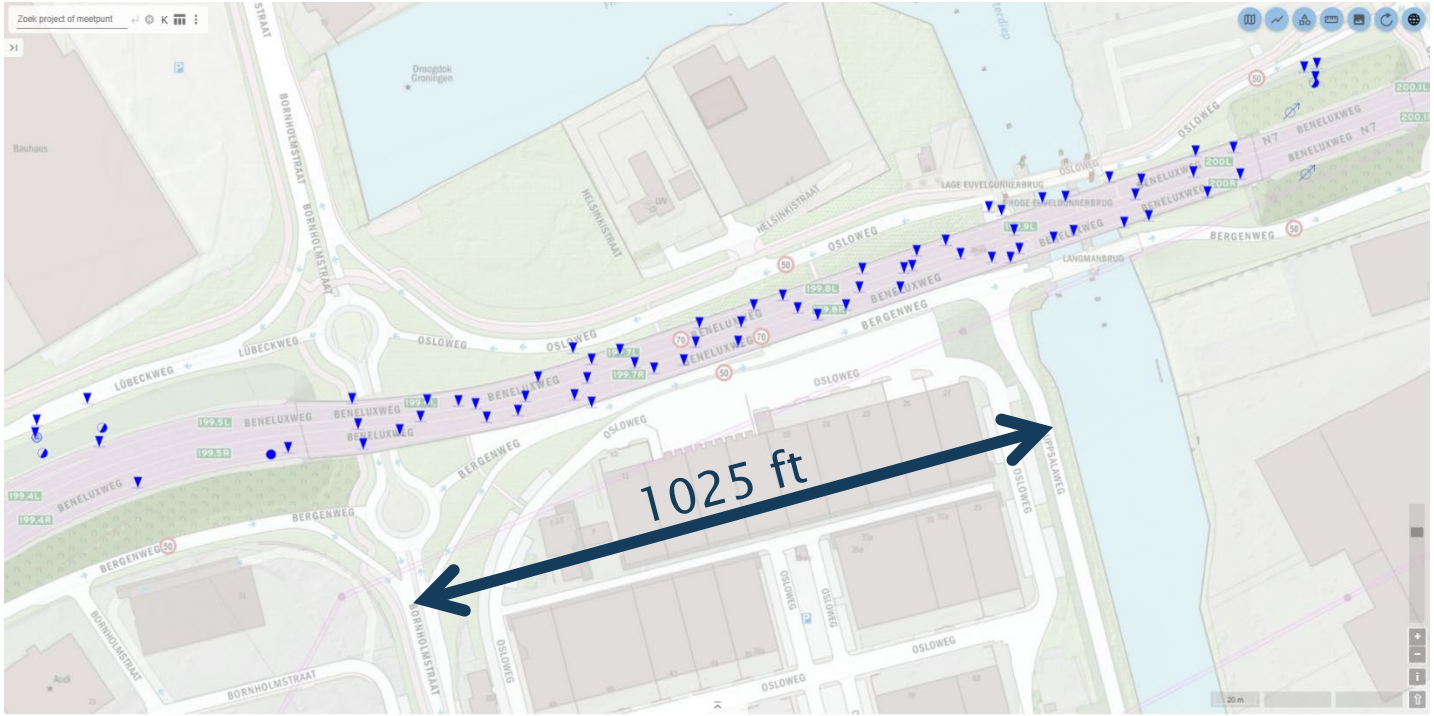
Case Study: Road Project Groningen

Soil investigation: CPTs



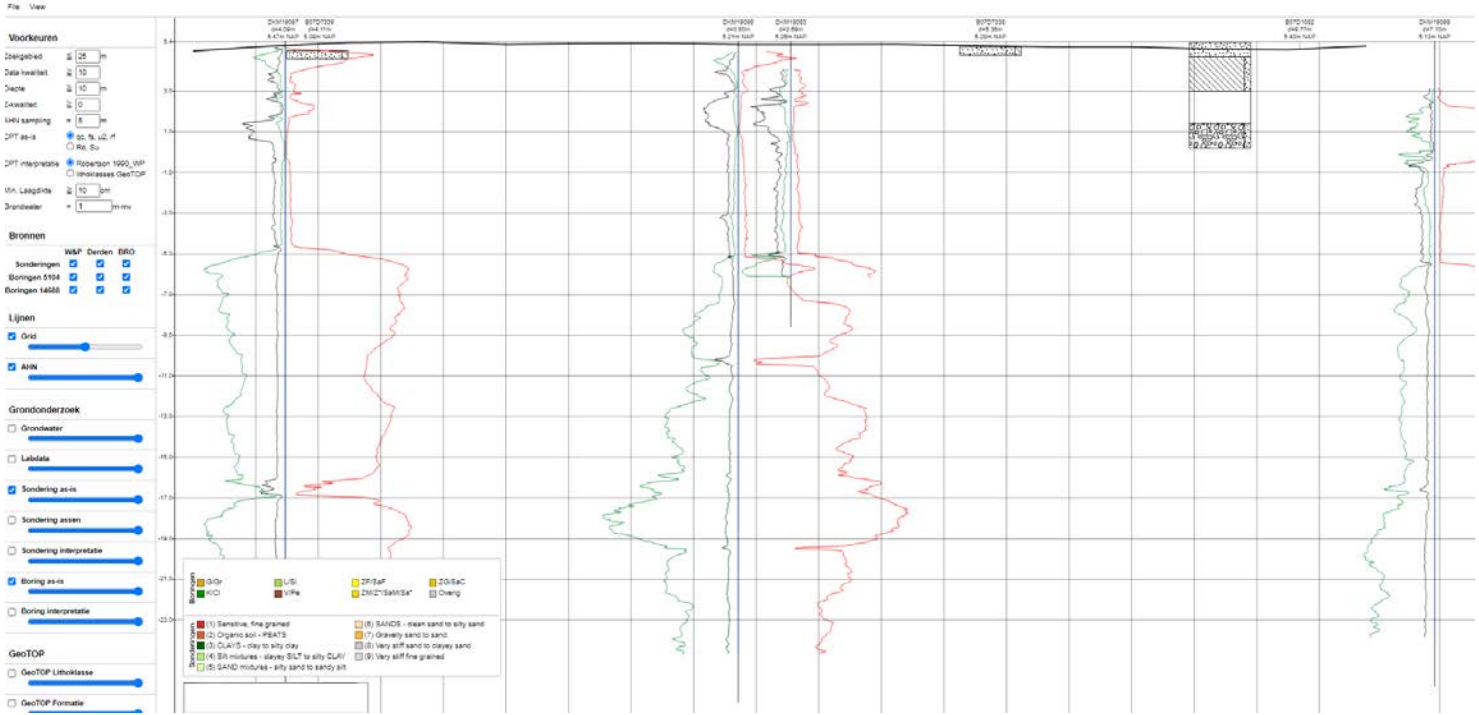
Case Study: Road Project Groningen

Soil investigation: CPTs



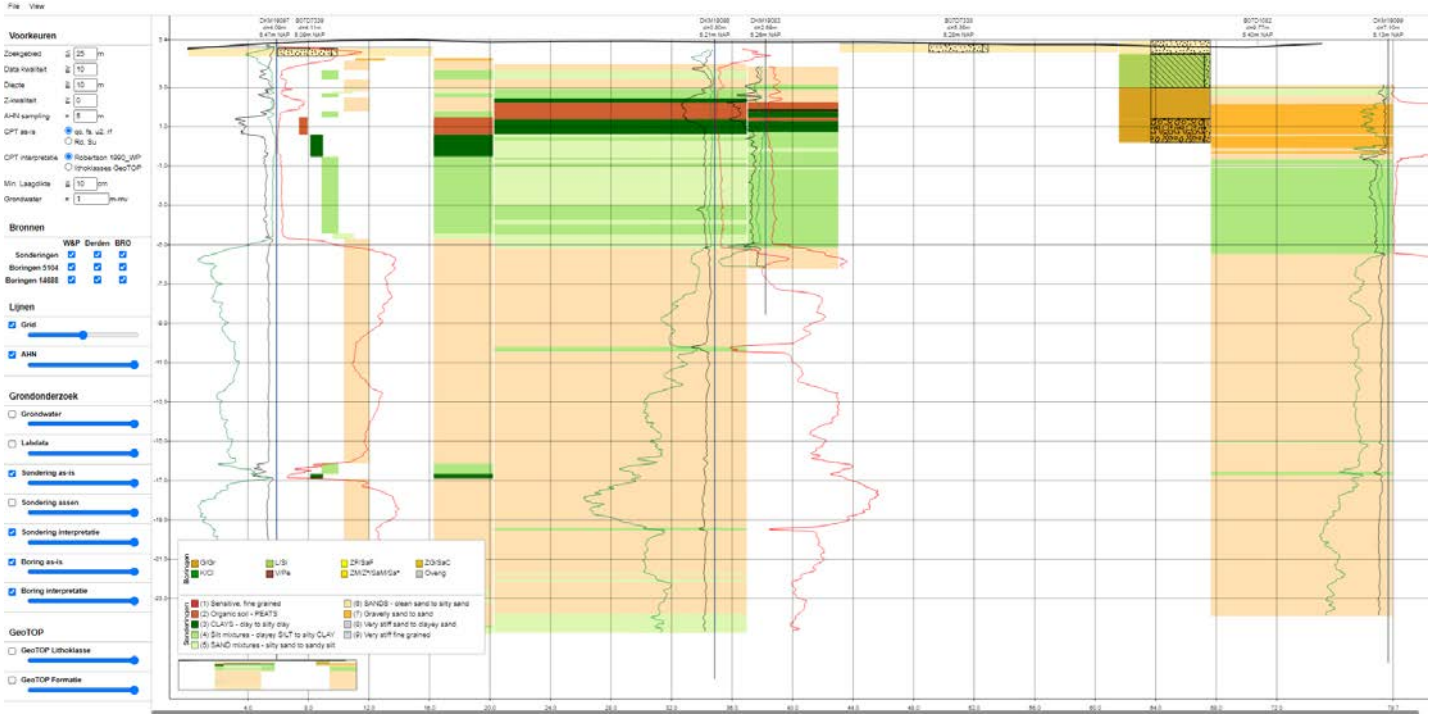
Case Study: Road Project Groningen

Soil investigation: CPTs



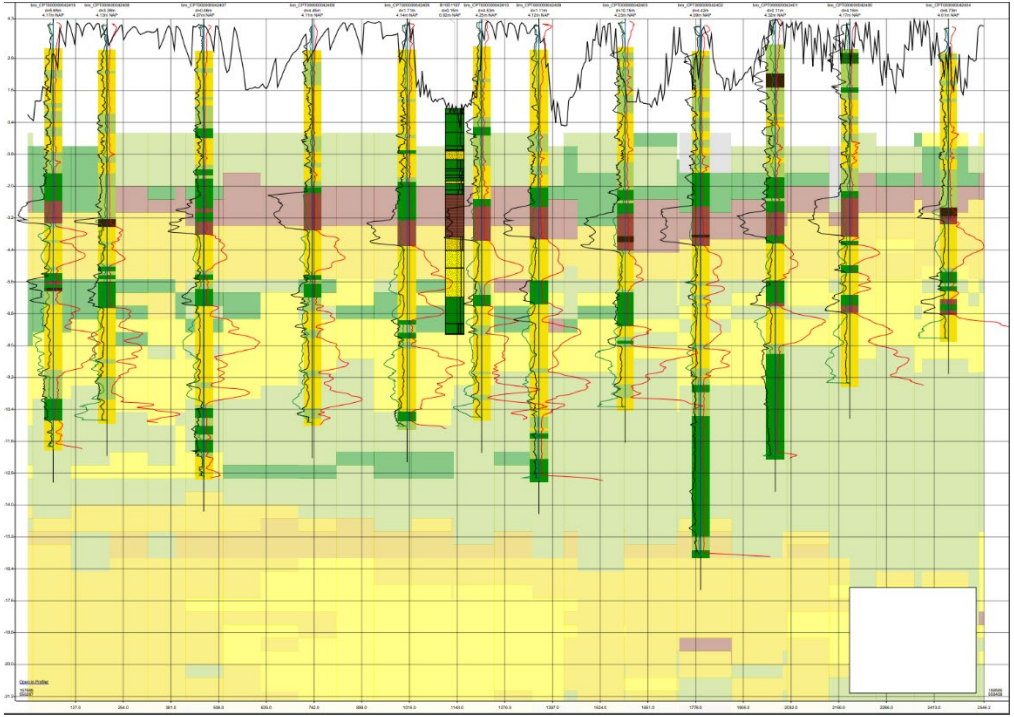
Case Study: Road Project Groningen

Soil profiling based on CPTs



Case Study: Road Project Groningen

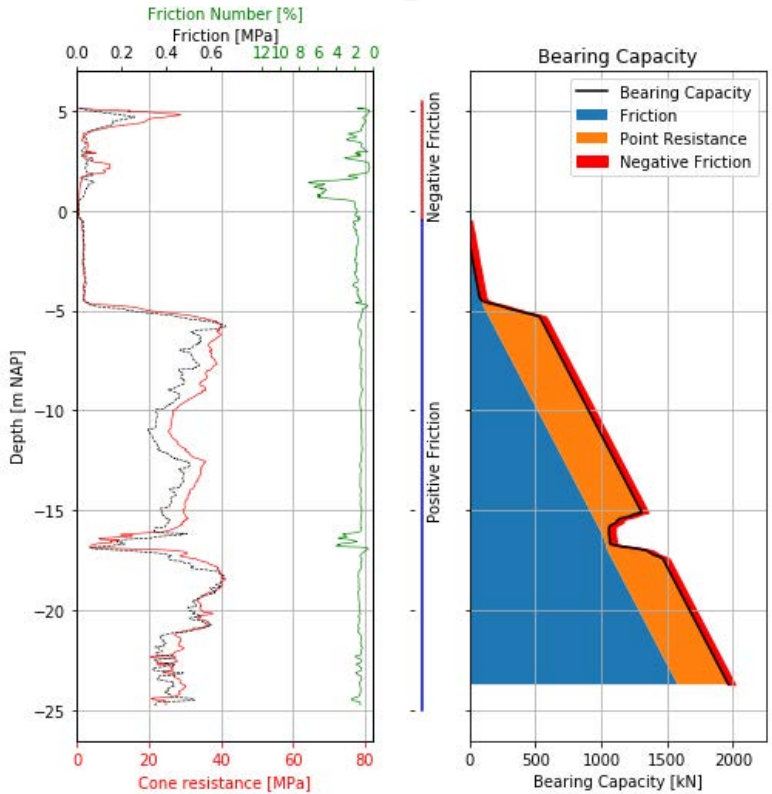
Soil profiling based on CPTs



Case Study: Road Project Groningen

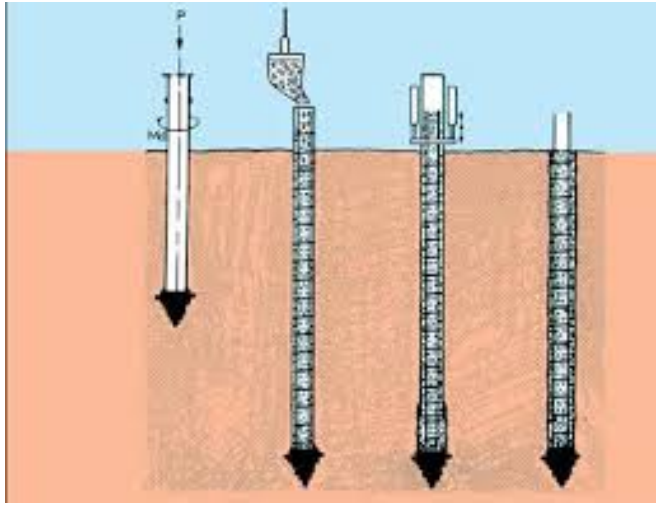
Pile design:

- based on the method of Koppejan
- Pile capacity directly derived from the CPT tip resistance (red line) and sleeve friction (blue line)
- Engineering judgement (as always) still required
  - Site variability
  - Serviceability (it's not just about bearing capacity, displacement must be considered as well)



Case Study: Road Project Groningen

- Foundation design based on CPTs only
- Bearing capacity 1500-2500 kN (340 – 560 kips) per pile
- Fundex and Tubex piles



Allnamics

Thank you for your attention!

Rob van Dorp & Marcel Bielefeld

vandorp@allnamics.com

bielefeld@allnamics.com

Thanks to:

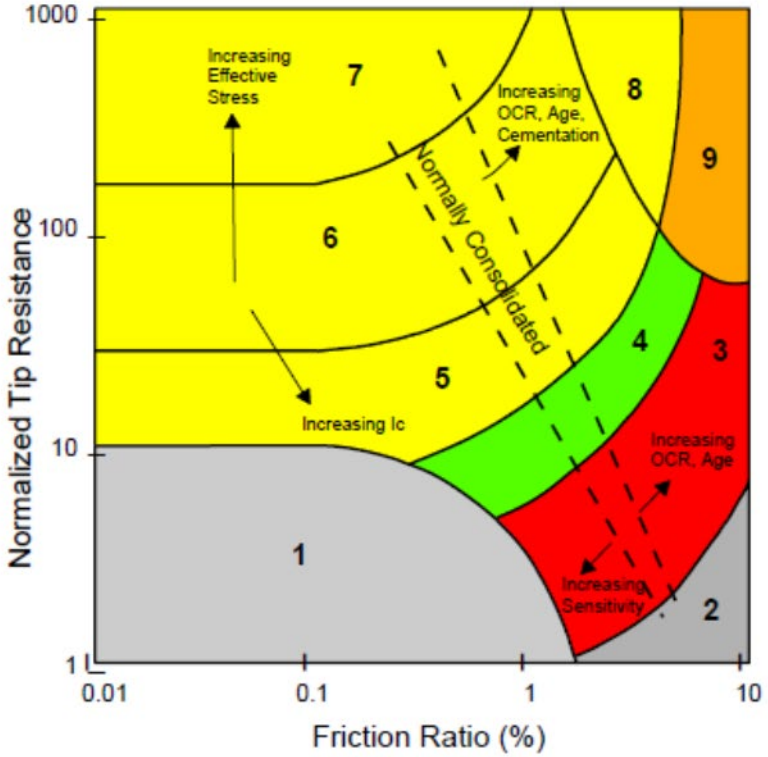
Wiertsema & Partners

Direct CPT Based Design
Bridge Pile Foundation
California Case Study

Sharid K. Amiri

Foundation

CPT Correlation



(Robertson, 1990)

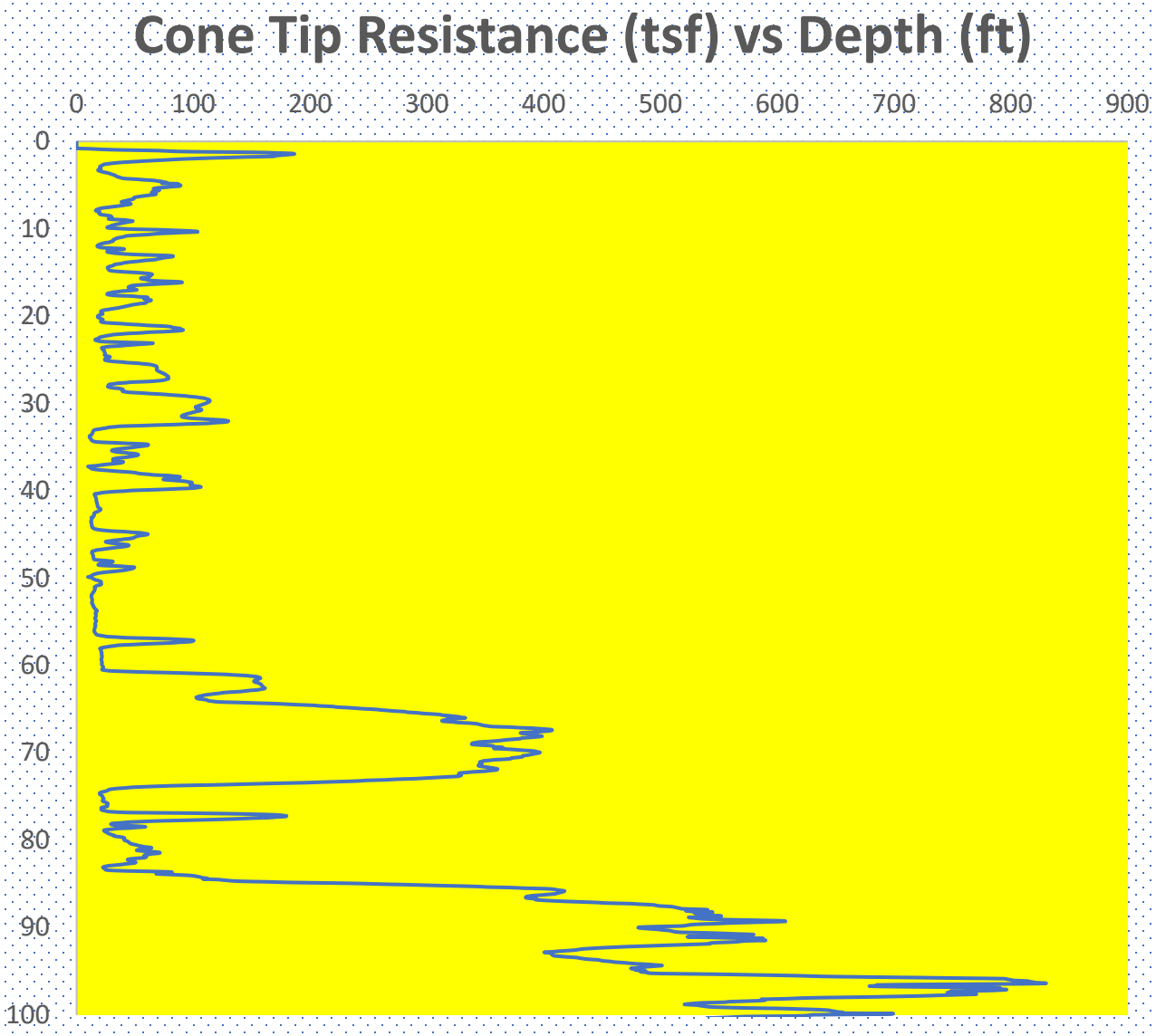
Zone	Soil Behavior Type
1	Sensitive Fine-grained
2	Peats
3	Silty Clay to Clay
4	Clayey Silt to Silty Clay
5	Silty Sand to Sandy Silt
6	Clean Sand to Silty Sand
7	Gravelly Sand to Dense Sand
8	Very Stiff Sand to Clayey Sand*
9	Very Stiff Fine-Grained*

*heavily overconsolidated or cemented

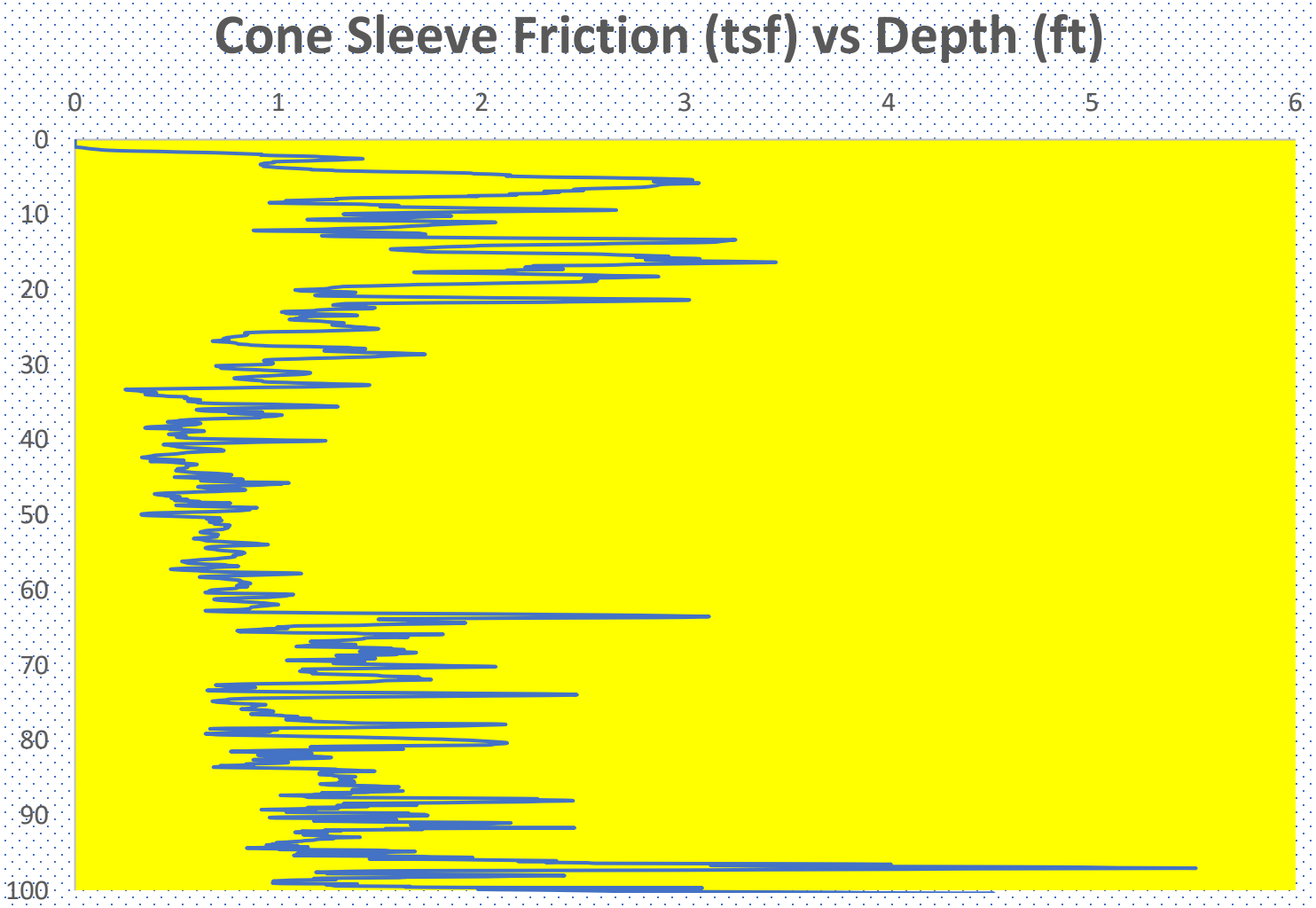
Soil Classification Using The Cone Penetration Test
P.K. Robertson
Canadian Geotechnical Journal

CPT

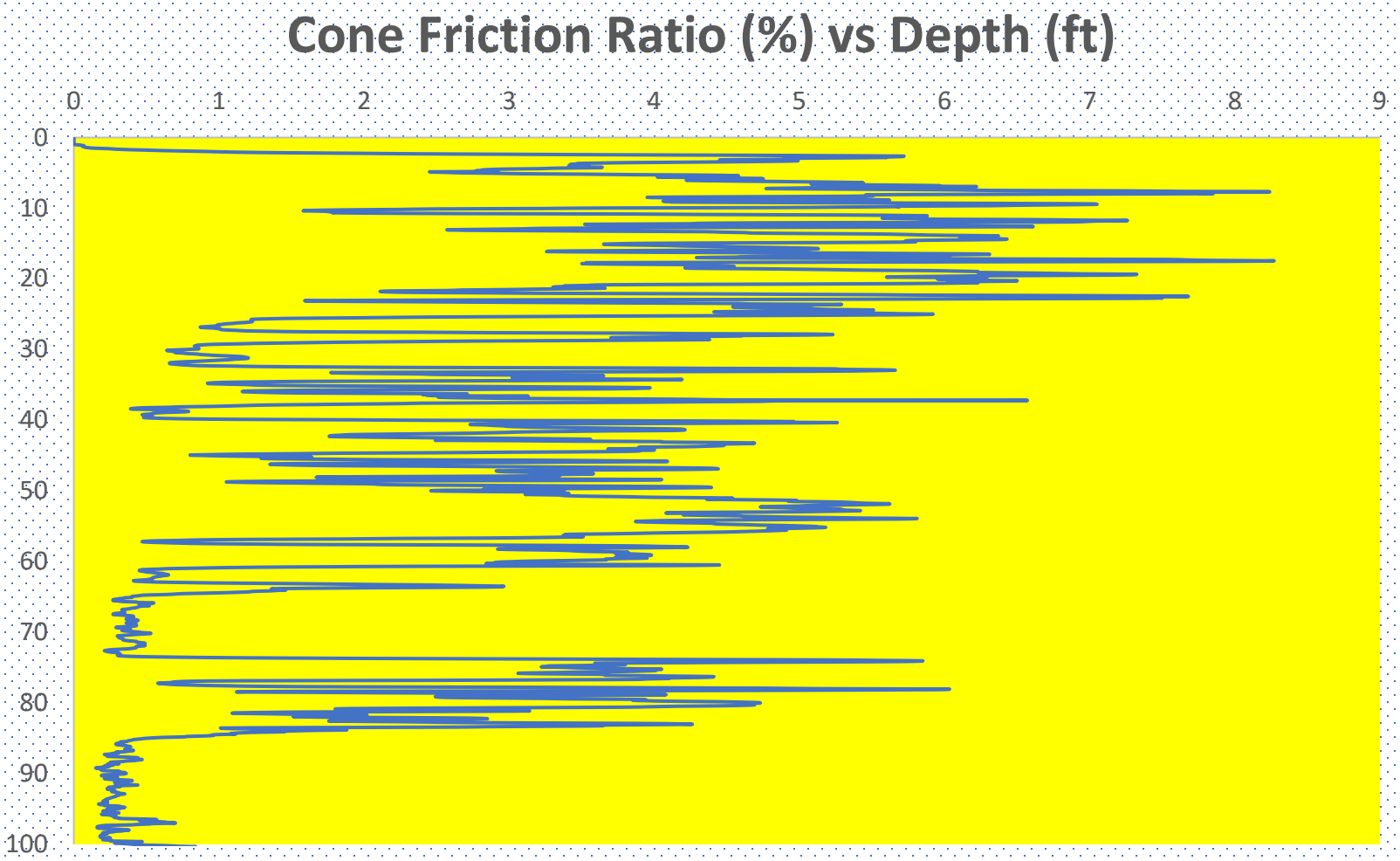
Tip Resistance



CPT
Sleeve Friction

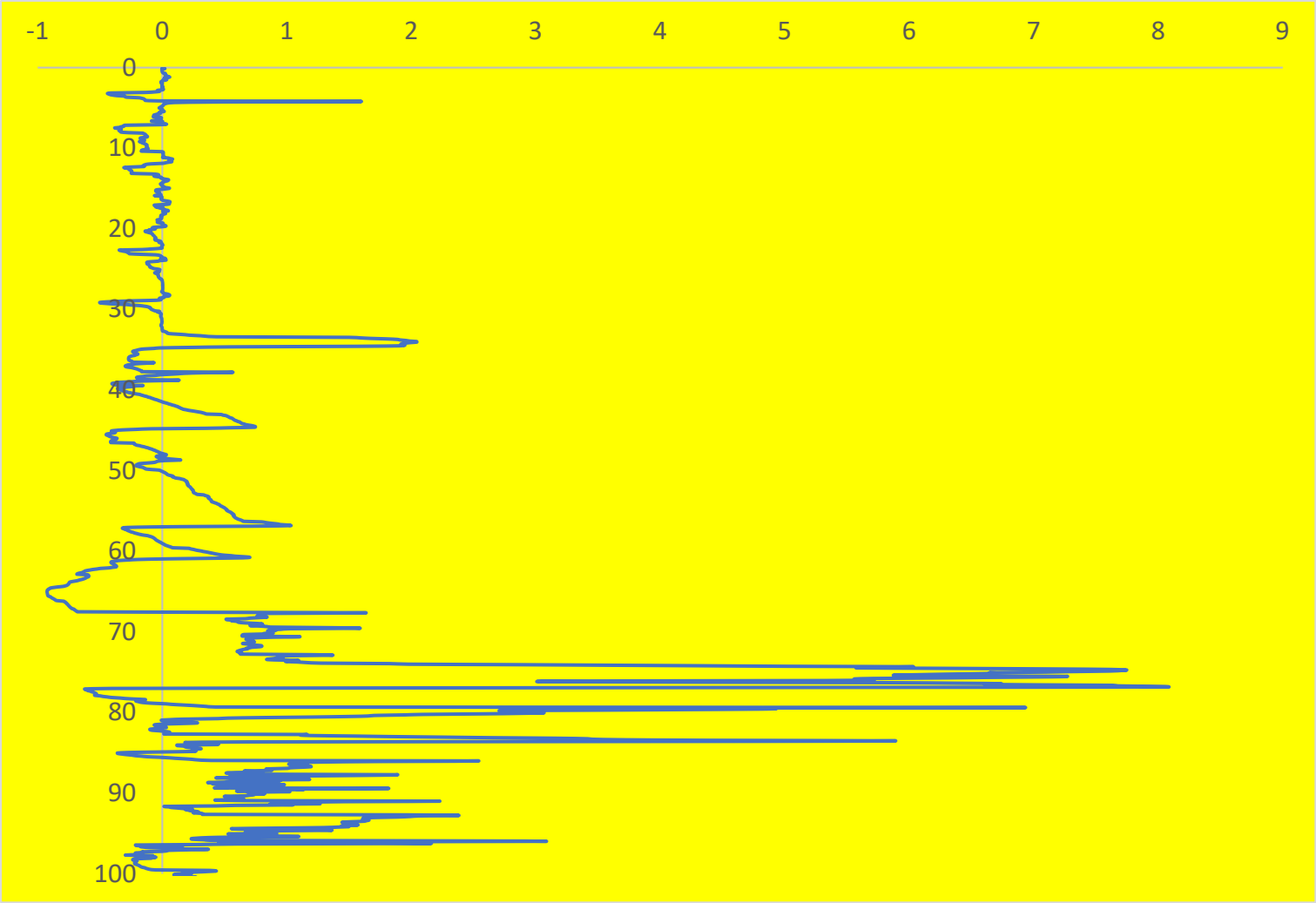


CPT
Friction Ratio

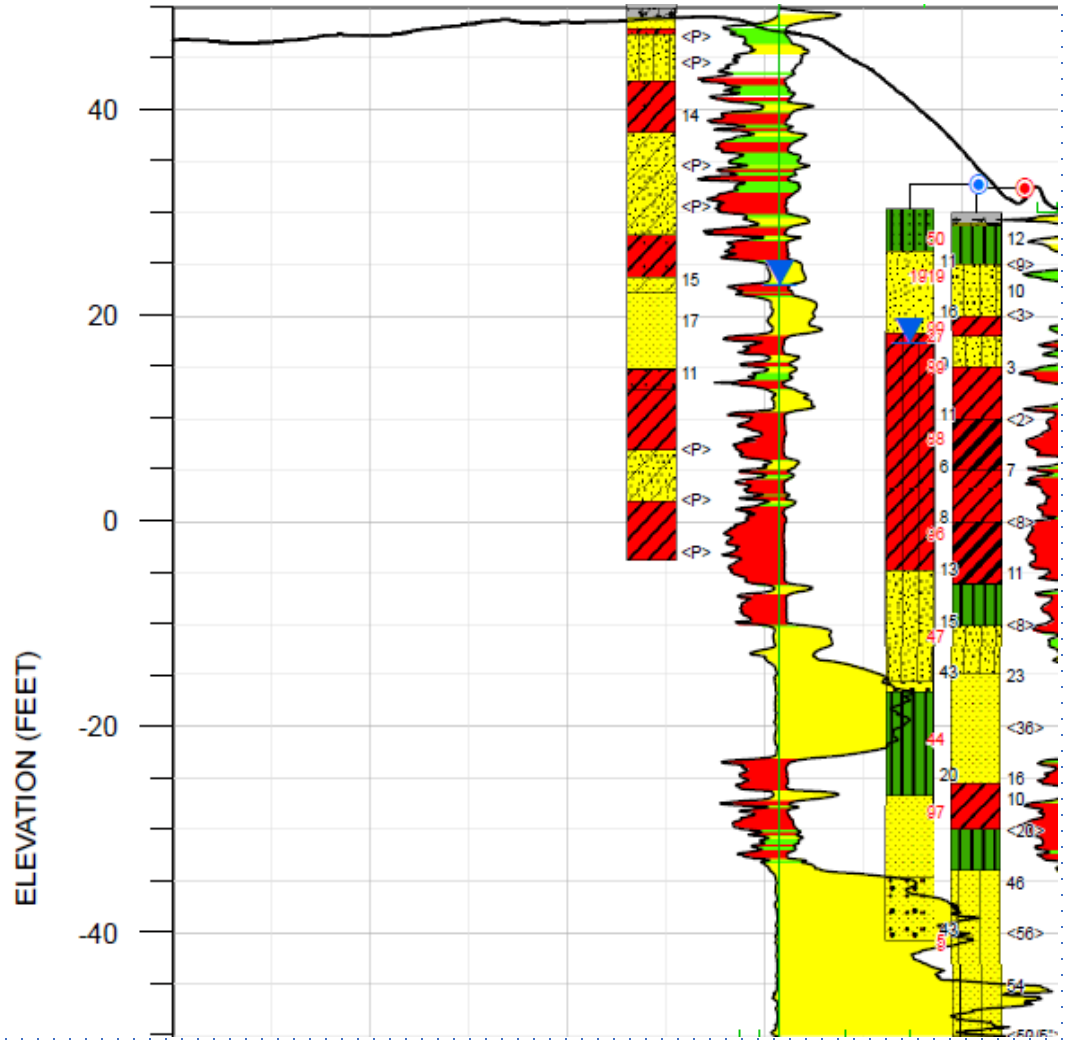


CPT
Pore Pressure

Depth (ft) vs Pore Pressure (tsf)



CPT GRAPHIC PROFILE



CPT at a Highway Bridge Abutment

CPT Based Friction Angle

Effective Friction Angle (Kulhawy and Mayne, 1990)

$\acute{\emptyset} = 17.6 + 11 \log (Q_{tn})$

Q_{tn} = Normalized Cone Tip Resistance, where

$Q_{tn} = [(q_t / \sigma_{atm})/(\sigma'_{v0} / \sigma_{atm})]^{0.5}$

q_t: Cone tip resistance

σ_{atm} : 1 atm = 100kPa

σ'_{v0} : Vertical effective stress

Manual on Estimating Soil Properties for Foundation Design

CPT Based undrained Shear Strength

Undrained Shear Strength (Robertson and Cabal, 2015)

$S_u = (q_t - \sigma_{v0})/N_{kt}$

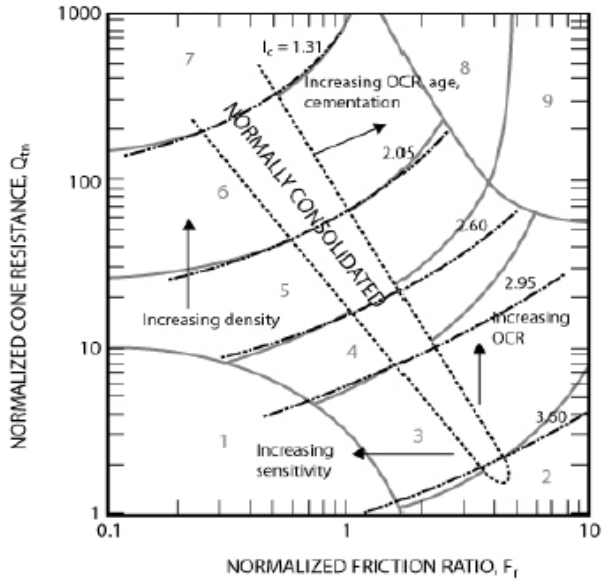
q_t = Cone tip resistance

σ_{v0}= Vertical total stress

N_{kt} = Empirical cone factor

Guide to Cone Penetration Testing for Geotechnical Engineering

CPT Based Soil Behavior Type



Zone	Soil Behavior Type	I_c
1	*Sensitive, fine grained*	N/A
2	*Organic soils – clay*	> 3.6
3	*Clays – silty clay to clay*	2.95 – 3.6
4	*Silt mixtures – clayey silt to silty clay*	2.60 – 2.95
5	*Sand mixtures – silty sand to sandy silt*	2.05 – 2.6
6	*Sands – clean sand to silty sand*	1.31 – 2.05
7	*Gravelly sand to dense sand*	< 1.31
8	*Very stiff sand to clayey sand**	N/A
9	*Very stiff, fine grained**	N/A

** Heavily overconsolidated or cemented*

Guide to Cone Penetration Testing For Geotechnical Engineering, Robertson, P.K. & Cabal, K.L., 2015

Direct CPT Based Methods

Bustamante & Gianeselli or LCPC Method

Pile Data Analysis (PDA)

Open Ended Driven Steel Pipe Piles : Plugged & Unplugged

Measured Cone Resistance, q_c

Friction Coefficient, α

Pile Unit Side Friction , f_p

End Bearing Capacity Factor, K_c

Equivalent Average Cone Resistance, q_{ca}

Pile Unit End Bearing, q_p

$$f_p = q_c/\alpha$$

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

Pile Categories

Pile Category	Pile Type
IA	plain bored piles; mud bored piles, hollow auger bored piles, micropiles (grouted under low pressure), cast screwed piles, piers, barrettes
IB	cased bored piles, driven cast piles
IIA	driven pre-cast piles, prestressed tubular piles, jacket concrete piles
IIB	driven metal piles, jacked metal piles
IIIA	driven grouted piles, driven rammed piles
IIIB	high pressure grouted piles of large diameter > 250 mm ; micropiles grouted under high pressure

Bearing Capacity Factors, k_c

Nature of Soil	q_c (MPa)	q_c (tsf)	Factors k_c Group I	Factors k_c Group II
Soft clay and mud	< 1	<10	0.4	0.5
Moderately compact clay	1 to 5	10 to 52	0.35	0.45
Silty and Loose Sand	≤ 5	≤ 52	0.4	0.5
Compact to stiff clay and compact silt	> 5	> 52	0.45	0.55
Soft chalk	≤ 5	≤ 52	0.2	0.3
Moderately compact sand and gravel	5 to 12	52 to 125	0.4	0.5
Weathered to fragmented chalk	> 5	> 52	0.2	0.4
Compact to very compact sand and gravel	> 12	> 125	0.3	0.4

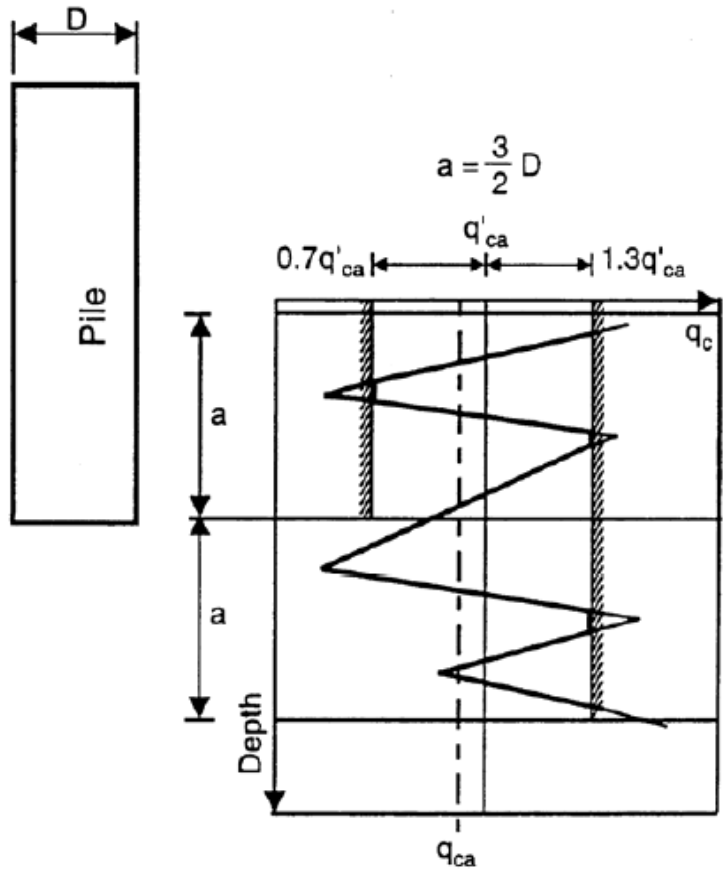
Friction Coefficient, α

			Category Coefficients, α			
			I		II	
Nature of Soil	qc (MPa)	qc (tsf)	A	B	A	B
Soft clay and mud	< 1	<10	30	90	90	30
Moderately compact clay	1 to 5	10 to 52	40	80	40	80
Silty and Loose Sand	≤ 5	≤ 52	60	150	60	120
Compact to stiff clay and compact silt	> 5	> 52	60	120	60	120
Soft chalk	≤ 5	≤ 52	100	120	100	120
Moderately compact sand and gravel	5 to 12	52 to 125	100	200	100	200
Weathered to fragmented chalk	> 5	> 52	60	80	60	80
Compact to very compact sand and gravel	> 12	> 125	150	300	150	200

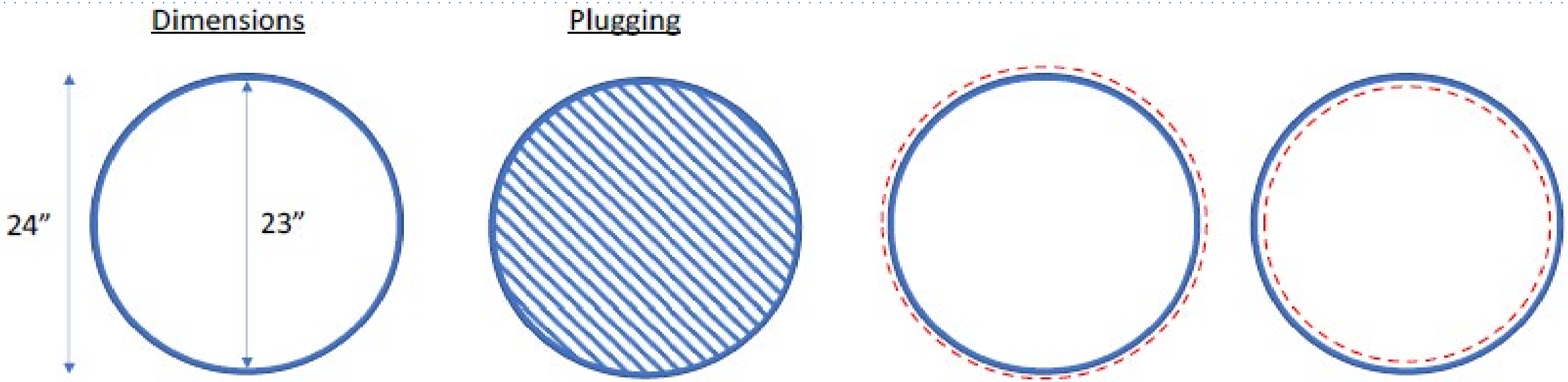
Maximum Limit of f_p (tsf)

			Category					
			Maximum Limit of f_p, tsf					
			I		II		III	
Nature of Soil	qc (MPa)	qc (tsf)	A	B	A	B	A	B
Soft clay and mud	< 1	<10	0.157	0.157	0.157	0.157	0.365	
Moderately compact clay	1 to 5	10 to 52	0.365	0.365	0.365	0.365	0.835	≥ 1.253
			(0.835)	(0.835)	(0.835)			
Silty and Loose Sand	≤ 5	≤ 52	0.365	0.365	0.365	0.365	0.835	-
Compact to stiff clay and compact silt	> 5	> 52	0.365	0.365	0.365	0.365	0.835	≥ 2.088
			(0.835)	(0.835)	(0.835)			
Soft chalk	≤ 5	≤ 52	0.365	0.365	0.365	0.365	0.835	-
Moderately compact sand and gravel	5 to 12	52 to 125	0.835	0.365	0.835	0.835	1.253	≥ 2.088
			(1.253)	(0.835)	(1.253)			
Weathered to fragmented chalk	> 5	> 52	1.253	0.835	1.253	1.253	1.566	≥ 2.088
			(1.566)	(1.253)	(1.566)			
Compact to very compact sand and gravel	> 12	> 125	1.253	0.835	1.253	1.253	1.566	≥ 2.088
			(1.566)	(1.253)	(1.566)			

Equivalent Average Cone Resistance



Pile Plugging



Unplugged Tip Area (steel only)

Plugged Tip Area

Outer Perimeter

Inner Perimeter

LCPC Method (Step by Step)

Tip Resistance from Raw CPT file @Pile Depth corresponding to the design tip elevation

→ Average Tip Resistance over 3D (1.5 D above and below the design tip elevation)

→ 70% and 130% of the average tip resistance value

→ Equivalent Average Cone Resistance, q_{ca}

→ Nature of Soil , based on the I_c Value

→ K_c value based on the soil type and tip resistance

LCPC Method

Coefficient α, based on Soil Type and tip resistance

Unit Side Resistance = Tip Resistance/α

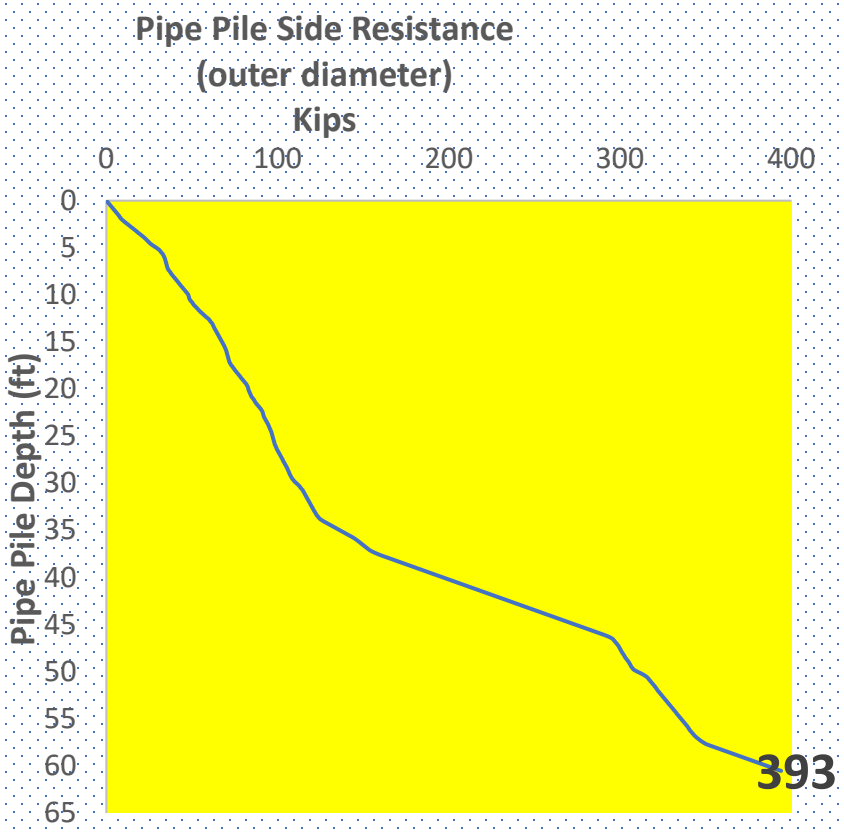
Maximum Limit of Side Resistance (f_p)

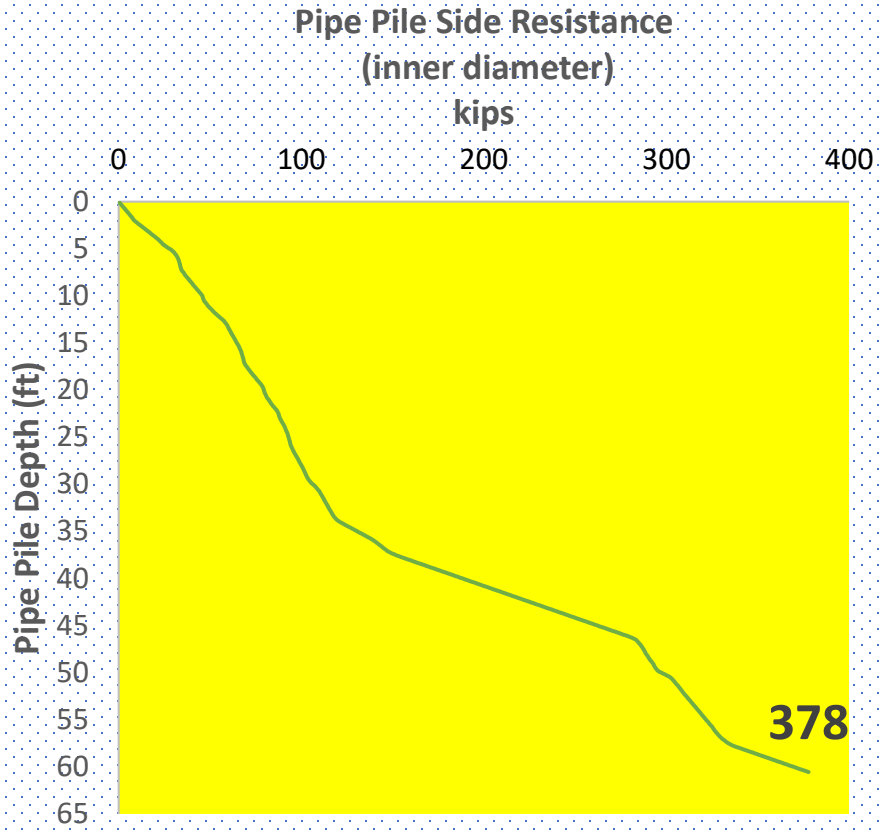
Actual Unit Side Pile Resistance: (Lower of Unit Side Resistance and Maximum Limit of Side Resistance)

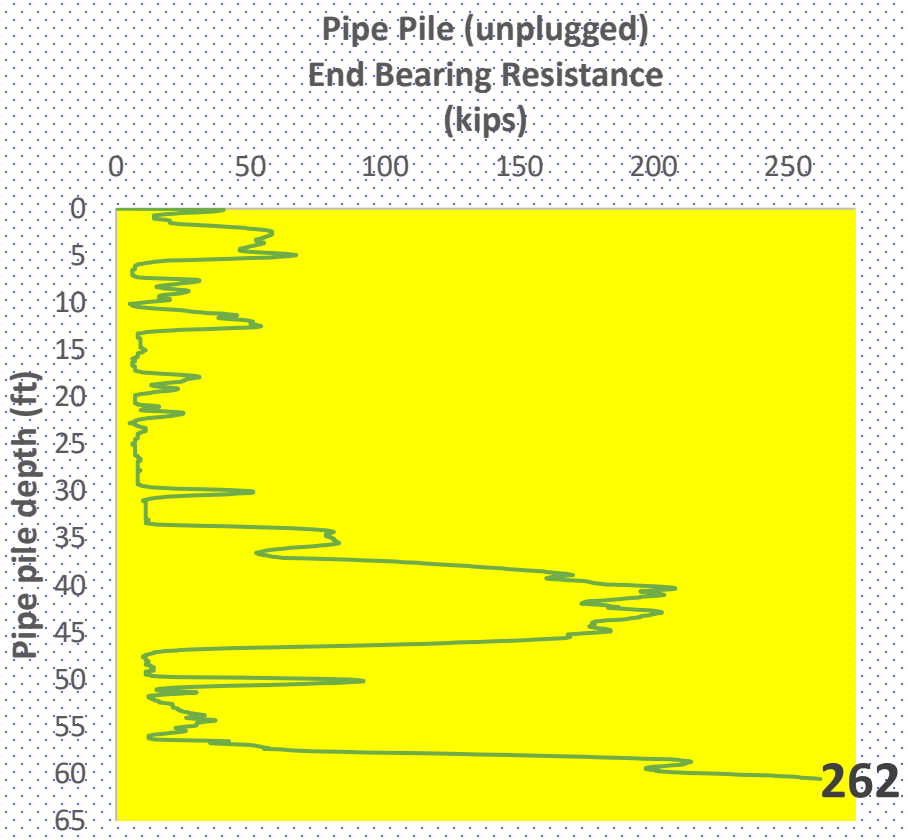
Pile Unit End Bearing : Equivalent Tip Resistance * K_c

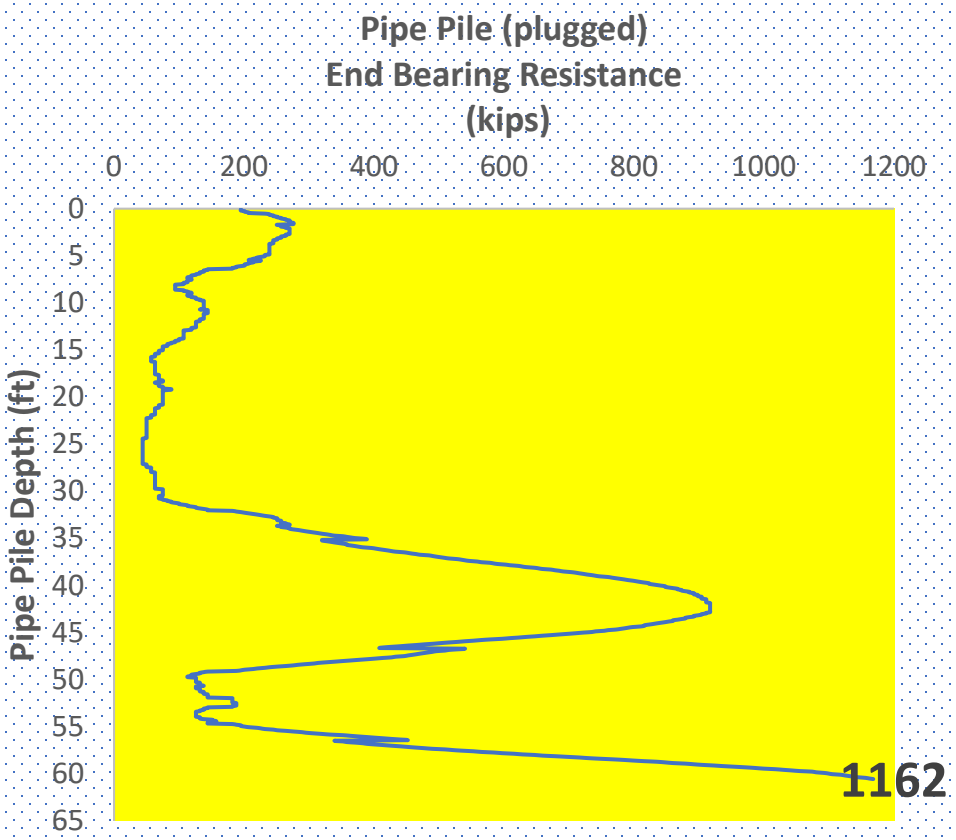
Unplugged Pile End Bearing: Tip Resistance* unplugged end bearing

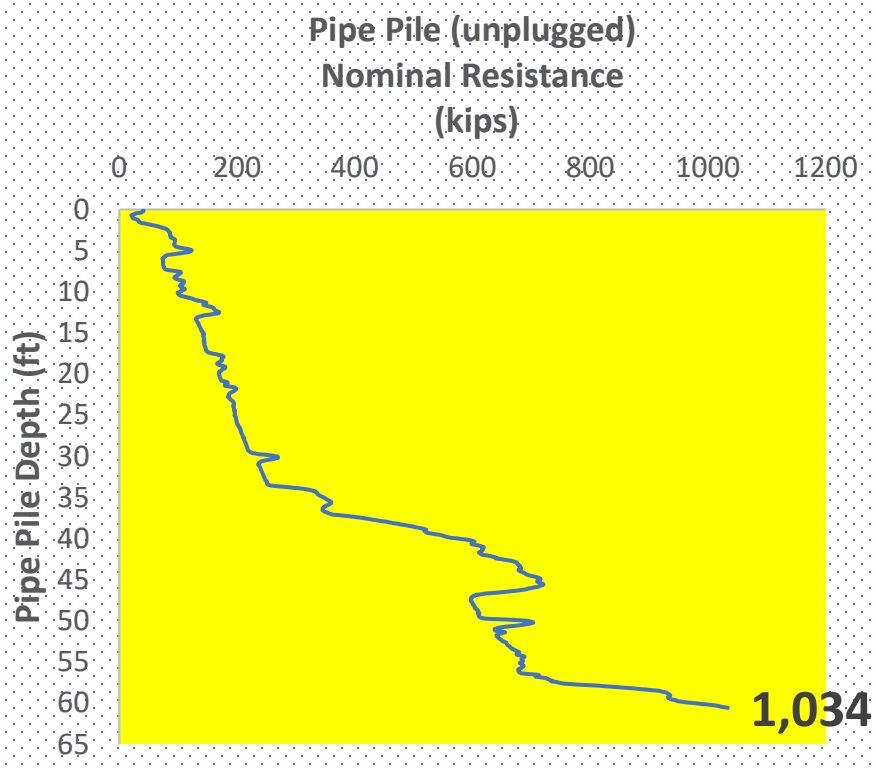
LCPC Method

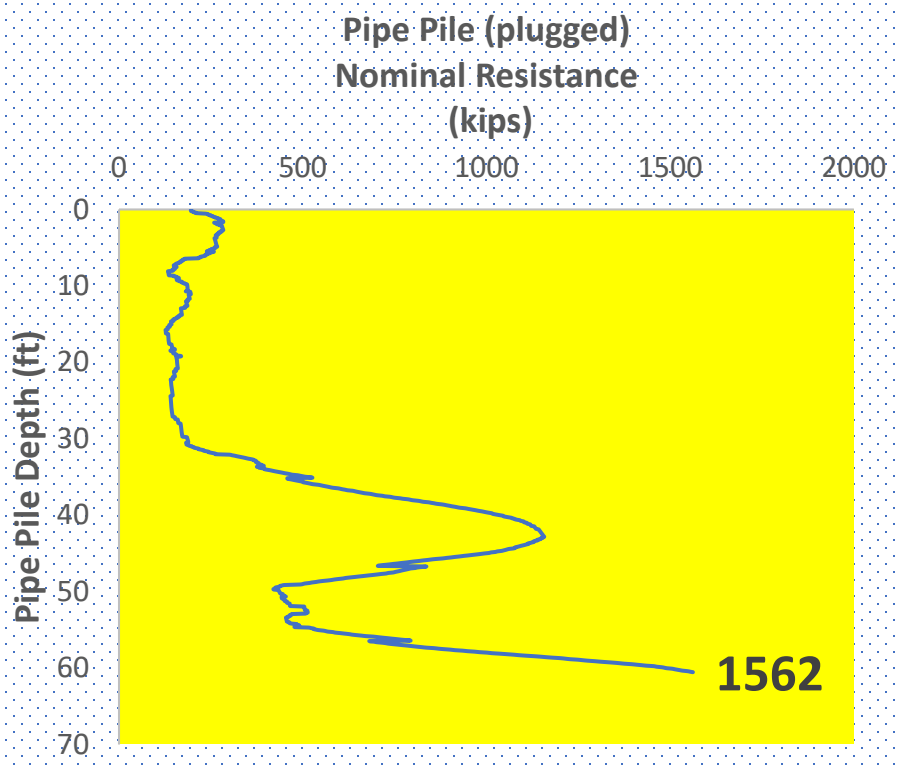

Pipe Pile Side Resistance
(outer diameter)
Kips
0
100
200
300
400
Pipe Pile Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
65
393


Pipe Pile Side Resistance
(inner diameter)
kips
0
100
200
300
400
Pipe Pile Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
65
378


Pipe Pile (unplugged)
End Bearing Resistance
(kips)
0
50
100
150
200
250
Pipe pile depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
65
262

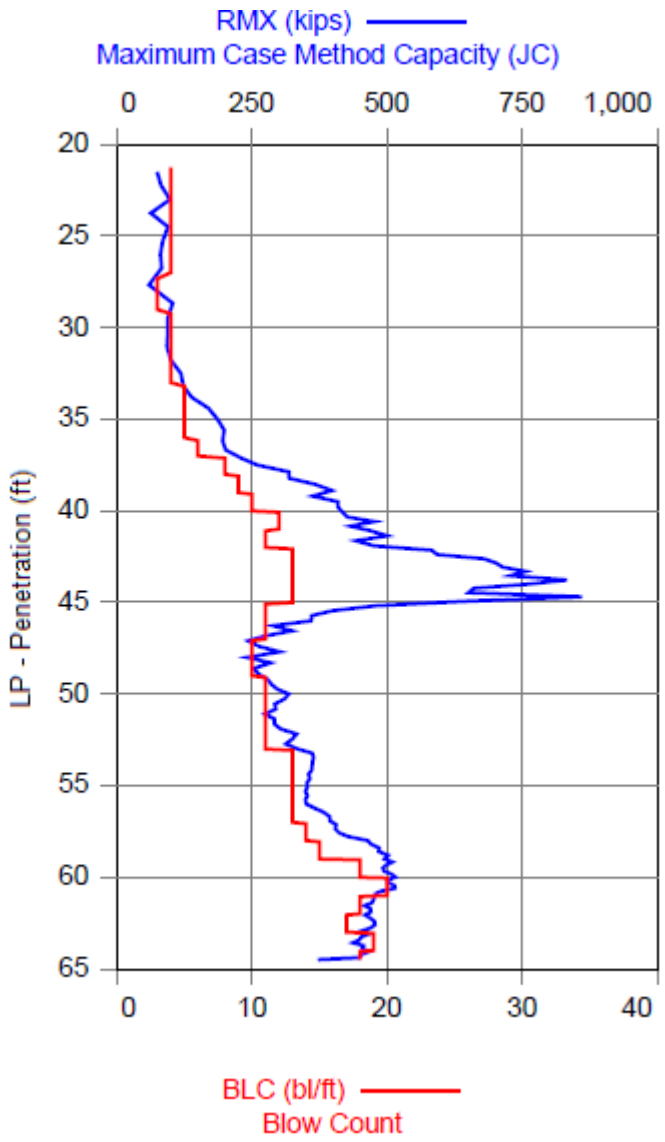

Pipe Pile (plugged)
End Bearing Resistance
(kips)
0
200
400
600
800
1000
1200
Pipe Pile Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
65
1162


Pipe Pile (unplugged)
Nominal Resistance
(kips)
0
200
400
600
800
1000
1200
Pipe Pile Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
65
1,034


Pipe Pile (plugged)
Nominal Resistance
(kips)
0
500
1000
1500
2000
Pipe Pile Depth (ft)
0
10
20
30
40
50
60
70
1562

Pile Data Analysis (PDA)

- 24 inch nominal diameter pipe pile with a wall thickness of 0.5 inch and a total length of approximately 70 feet was driven with an APE D-46-32 open ended single acting diesel hammer with a rated energy of 114 kp-ft. The PDA plot shows the pile capacity for the initial drive.



SIGNAL MATCHING ANALYSIS

(Initial Drive)

- estimate of ultimate toe resistance : 214 kips
- estimate of ultimate shaft resistance: 275 kips
- estimate of ultimate total resistance: 489 kips

SIGNAL MATCHING ANALYSIS (Beginning of Restrike)

- estimate of ultimate toe resistance : 29 kips
- estimate of ultimate shaft resistance: 992 kips
- estimate of ultimate total resistance: 1021 kips

Soil Setup

- Case study on the use of direct CPT based design of driven steel pipe pile
- LCPC method was used in design to evaluate the pile axial nominal resistance
- PDA testing was performed to verify the pile nominal resistance
- SIGNAL MATCHING ANALYSIS evaluation of side shaft resistance and pile toe resistance
- Setup played a significant role for pipe pile to reach its required design capacity

Acknowledgment

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