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 = Standard Penetration Test
TPPT = Texas Penetration Test
VST = Vane Shear Test
PMT = Pressuremeter Test
CPMT = Cone Pressuremeter
DMT = Dilatometer Test
SPLT = Screw Plate Load Test
ISB = Iowa K8 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 = Manistock 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
CPTu = piezodissipation test
DMTu = Dilatometer with A-reading dissipations
SPTT = Standard Penetration Test with Torque
LPT = Large Penetration Test
DEPPT = Dual Element PiezoProbe Test
SCPMTu = Seismic Piezocone Pressuremeter
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.

After CPT Manual for Highway Geotechnical Engineers, MnDOT 2018
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

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 so-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
Why CPT

- You get to the job site
- You push the cone
- You issue a report

All in about 25 minutes
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?
The objective of a soil investigation is to get good information on the soil behavior.
SPT vs. CPT
N vs. tip, sleeve and pore pressure

SPT = Standard Penetration Test
CPT = Cone Penetration Test

\( c_u \) = undrained strength
\( \gamma_T \) = unit weight
\( I_R \) = rigidity index
\( \phi' \) = 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

\( D_R \) = relative density
\( \gamma_T \) = unit weight
\( LI \) = liquefaction index
\( \phi' \) = friction angle
\( c' \) = cohesion intercept
\( e_o \) = void ratio
\( q_a \) = bearing capacity
\( \sigma_p' \) = preconsolidation
\( V_s \) = shear wave
\( E' \) = elastic modulus
\( \Psi \) = dilatancy angle
\( q_b \) = pile end bearing
\( f_s \) = pile skin friction
Reality – Aspect 1

SPT vs. CPT

\( f(N) \) vs. \( f(\text{tip, sleeve and pore pressure}) \)

- \( c_u \) = undrained strength
- \( \gamma_T \) = unit weight
- \( I_r \) = rigidity index
- \( \phi' \) = friction angle
- \( OCR \) = overconsolidation
- \( k_o \) = 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

- \( D_R \) = relative density
- \( \gamma_T \) = unit weight
- \( LI \) = liquefaction index
- \( \phi' \) = friction angle
- \( c' \) = cohesion intercept
- \( e_o \) = void ratio
- \( q_a \) = bearing capacity
- \( \sigma_p' \) = preconsolidation
- \( V_s \) = shear wave
- \( E' \) = elastic modulus
- \( \psi \) = dilatancy angle
- \( q_b \) = pile end bearing
- \( f_s \) = pile skin friction
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
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

“New? It has always been there”

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

Direct design methods
Why not CPT (or so people claim)
Why not CPT (or so people claim)
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
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.
We have known this since the 1930s.

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.
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
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
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
Gerald Verbeek
g.verbeek@verbeekservices.com
(903) 216 5372
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
CPT – Soil profiling

original plan for CPTs
CPT – Soil profiling

site variability detected by multiple CPTs

< 100 m (330 ft)
CPT – Soil profiling

site variability detected by multiple CPTs
Design Parameters

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

<table>
<thead>
<tr>
<th>Grondsoort</th>
<th>Bijmengsel</th>
<th>Consisten-</th>
<th>$k',\text{MPa}$</th>
<th>$\lambda_\text{ac}$</th>
<th>$q_0,\text{MPa}$</th>
<th>$C_p$</th>
<th>$C_v$</th>
<th>$C_r$</th>
<th>$C_s$</th>
<th>$E_w$</th>
<th>$\varphi'$</th>
<th>$c'$</th>
<th>$c_u$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>tientie a</td>
<td>kN/m$^3$</td>
<td>kN/m$^3$</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>MPa</td>
<td>Graden</td>
<td>kPa</td>
<td>kPa</td>
</tr>
<tr>
<td>Grind</td>
<td>Zwart silig</td>
<td>Los</td>
<td>17</td>
<td>19</td>
<td>15</td>
<td>500</td>
<td>∞</td>
<td>0.004</td>
<td>0.0015</td>
<td>45</td>
<td>32.5</td>
<td>0</td>
<td>N.v.t.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Matig</td>
<td>18</td>
<td>20</td>
<td>25</td>
<td>1000</td>
<td>∞</td>
<td>0.0023</td>
<td>0.0008</td>
<td>75</td>
<td>35.0</td>
<td>0</td>
<td>N.v.t.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Vast</td>
<td>19</td>
<td>20</td>
<td>22</td>
<td>1400</td>
<td>∞</td>
<td>0.0019</td>
<td>0.0005</td>
<td>90</td>
<td>37.5</td>
<td>40.0</td>
<td>0</td>
</tr>
<tr>
<td>Sterk zilt</td>
<td>Zwart silig</td>
<td>Los</td>
<td>18</td>
<td>20</td>
<td>10</td>
<td>400</td>
<td>∞</td>
<td>0.0058</td>
<td>0.0019</td>
<td>30</td>
<td>30.0</td>
<td>0</td>
<td>N.v.t.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Matig</td>
<td>19</td>
<td>21</td>
<td>15</td>
<td>600</td>
<td>∞</td>
<td>0.0038</td>
<td>0.0013</td>
<td>45</td>
<td>32.5</td>
<td>0</td>
<td>N.v.t.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Vast</td>
<td>20</td>
<td>21</td>
<td>22</td>
<td>1500</td>
<td>∞</td>
<td>0.0023</td>
<td>0.0009</td>
<td>75</td>
<td>35.0</td>
<td>40.0</td>
<td>0</td>
</tr>
<tr>
<td>Zand</td>
<td>Schoon</td>
<td>Los</td>
<td>17</td>
<td>19</td>
<td>5</td>
<td>200</td>
<td>∞</td>
<td>0.0115</td>
<td>0.0038</td>
<td>15</td>
<td>30.0</td>
<td>0</td>
<td>N.v.t.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Matig</td>
<td>18</td>
<td>20</td>
<td>15</td>
<td>600</td>
<td>∞</td>
<td>0.0038</td>
<td>0.0013</td>
<td>45</td>
<td>32.5</td>
<td>0</td>
<td>N.v.t.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Vast</td>
<td>19</td>
<td>20</td>
<td>21</td>
<td>1200</td>
<td>∞</td>
<td>0.0005</td>
<td>0.0017</td>
<td>75</td>
<td>35.0</td>
<td>40.0</td>
<td>0</td>
</tr>
<tr>
<td>Zwart zilt, kiezel</td>
<td></td>
<td>Los</td>
<td>18</td>
<td>19</td>
<td>20</td>
<td>450</td>
<td>600</td>
<td>0.0005</td>
<td>0.0017</td>
<td>15</td>
<td>50.0</td>
<td>0</td>
<td>N.v.t.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Matig</td>
<td>19</td>
<td>20</td>
<td>21</td>
<td>400</td>
<td>∞</td>
<td>0.0015</td>
<td>0.0008</td>
<td>45</td>
<td>25.0</td>
<td>30.0</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Vast</td>
<td>20</td>
<td>21</td>
<td>22</td>
<td>200</td>
<td>∞</td>
<td>0.0003</td>
<td>0.0019</td>
<td>15</td>
<td>50.0</td>
<td>0</td>
<td>N.v.t.</td>
</tr>
<tr>
<td>Leem e</td>
<td>Zwart zandig</td>
<td>Slap</td>
<td>19</td>
<td>19</td>
<td>1</td>
<td>25</td>
<td>650</td>
<td>0.0020</td>
<td>0.0037</td>
<td>2</td>
<td>27.5</td>
<td>30.0</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Matig</td>
<td>20</td>
<td>20</td>
<td>2</td>
<td>1000</td>
<td>600</td>
<td>0.0011</td>
<td>0.0037</td>
<td>3</td>
<td>27.5</td>
<td>32.5</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Vast</td>
<td>21</td>
<td>21</td>
<td>2</td>
<td>2500</td>
<td>1500</td>
<td>0.0029</td>
<td>0.0013</td>
<td>5</td>
<td>27.5</td>
<td>37.5</td>
<td>1</td>
</tr>
<tr>
<td>Sterk zandig</td>
<td>Zwart zandig</td>
<td>Slap</td>
<td>19</td>
<td>20</td>
<td>2</td>
<td>25</td>
<td>70</td>
<td>0.0012</td>
<td>0.0170</td>
<td>2</td>
<td>27.5</td>
<td>37.5</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Matig</td>
<td>20</td>
<td>20</td>
<td>2</td>
<td>100</td>
<td>200</td>
<td>0.0005</td>
<td>0.0011</td>
<td>5</td>
<td>27.5</td>
<td>37.5</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Vast</td>
<td>21</td>
<td>21</td>
<td>2</td>
<td>1000</td>
<td>2500</td>
<td>0.0031</td>
<td>0.0011</td>
<td>5</td>
<td>27.5</td>
<td>37.5</td>
<td>1</td>
</tr>
<tr>
<td>Klei</td>
<td>Schoon</td>
<td>Slap</td>
<td>14</td>
<td>14</td>
<td>0.5</td>
<td>7</td>
<td>80</td>
<td>0.3286</td>
<td>0.0131</td>
<td>1</td>
<td>17.5</td>
<td>0</td>
<td>25</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Matig</td>
<td>15</td>
<td>17</td>
<td>1.0</td>
<td>150</td>
<td>180</td>
<td>0.0061</td>
<td>0.0055</td>
<td>2</td>
<td>17.5</td>
<td>0</td>
<td>50</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Vast</td>
<td>16</td>
<td>20</td>
<td>20</td>
<td>50</td>
<td>800</td>
<td>0.0076</td>
<td>0.0003</td>
<td>3</td>
<td>22.5</td>
<td>0</td>
<td>80</td>
</tr>
<tr>
<td>Sterk zandig</td>
<td>Zwart zandig</td>
<td>Slap</td>
<td>15</td>
<td>15</td>
<td>0.5</td>
<td>25</td>
<td>140</td>
<td>0.0052</td>
<td>0.0077</td>
<td>1.5</td>
<td>22.5</td>
<td>0</td>
<td>120</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Matig</td>
<td>16</td>
<td>18</td>
<td>1.9</td>
<td>70</td>
<td>200</td>
<td>0.0012</td>
<td>0.0007</td>
<td>2</td>
<td>22.5</td>
<td>0</td>
<td>120</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Vast</td>
<td>17</td>
<td>20</td>
<td>20</td>
<td>30</td>
<td>500</td>
<td>0.0020</td>
<td>0.0019</td>
<td>2</td>
<td>22.5</td>
<td>0</td>
<td>120</td>
</tr>
<tr>
<td>Organisch</td>
<td>Zwart zandig</td>
<td>Slap</td>
<td>13</td>
<td>13</td>
<td>0.2</td>
<td>7.5</td>
<td>10</td>
<td>0.150</td>
<td>0.0064</td>
<td>1.5</td>
<td>22.5</td>
<td>0</td>
<td>120</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Matig</td>
<td>15</td>
<td>18</td>
<td>15</td>
<td>50</td>
<td>200</td>
<td>0.0053</td>
<td>0.0007</td>
<td>2</td>
<td>15.0</td>
<td>0</td>
<td>120</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Vast</td>
<td>17</td>
<td>21</td>
<td>21</td>
<td>20</td>
<td>500</td>
<td>0.0026</td>
<td>0.0015</td>
<td>2</td>
<td>5.0</td>
<td>0</td>
<td>120</td>
</tr>
</tbody>
</table>

Settlement | Fills | Sheet piles | 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 2\textsuperscript{nd} 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

\[ p_{r, \text{max}} = \frac{1}{4} q_{c;I;\text{gem}} + \frac{1}{4} q_{c;II;\text{gem}} + \frac{1}{2} q_{c;III;\text{gem}} \]

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 $F_{s,nk}$

- $R_{\text{pile,nett}} = R_b + R_s - F_{s,nk}$
Dutch pile design code using CPT
(basics, unfactored presentation here)

\[ R_{\text{pile,nett}} = R_b + R_s - F_{s,nk} \]

\[ R_b = A_{\text{pile}} \times \beta \times s \times \alpha_p \times p_{r,\text{max}} \]

\[ p_{r,\text{max}} = \frac{1}{4} q_{\text{c,II,geom}} + \frac{1}{4} q_{\text{c,III,geom}} + \frac{1}{2} q_{\text{c,III,geom}} \]

\[ R_s = O_{s,\Delta L,\text{geom}} \times \int_{\Delta L} \alpha_s \times q_{c,z,a} \times d z \]

\[ F_{s,nk} = O_{s,\text{geom}} \times \sum_{j=1}^{j=n} d_j \times K_{0,j,k} \times \tan(\delta_{ijk}) \times \frac{\sigma'_{v,j,\text{rep}} + \sigma'_{v,j,\text{rep}}}{2} \]

\[ \approx C_{\text{pile}} \times L_{nk} \times 0.25 \times \sigma_{k,\text{avg}} \]
Dutch pile design code using CPT

- Relationship between resistance and CPT
- Except diameter, cross sectional area, penetration depth L and ΔL
- Factors:
  - $\beta = \text{factor depending on ratio toe diameter / shaft diameter}$
  - $\alpha_p = \text{factor depending on pile type & installation method}$
  - $s = \text{factor depending cross section shape}$
  - $\alpha_s = \text{factor depending on pile type & installation method}$
- Factors are defined in table 7c of the code
# Dutch pile design code using CPT

(*basics, unfactored presentation here*)

<table>
<thead>
<tr>
<th>Pile type</th>
<th>Pile subtype</th>
<th>Pile installation method</th>
<th>Applicable factors</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beton-paal</td>
<td>Geprefabriceerd; met constante dwarsafmeting</td>
<td>Geheid; de mantelbuis wordt teruggevonden in combinatie met beton direct tegen de grond drijft.</td>
<td>Applicable load-displacement curve</td>
</tr>
</tbody>
</table>
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
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
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

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

CPT Correlation

(Robertson, 1990)

<table>
<thead>
<tr>
<th>Zone</th>
<th>Soil Behavior Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Sensitive Fine-grained</td>
</tr>
<tr>
<td>2</td>
<td>Peats</td>
</tr>
<tr>
<td>3</td>
<td>Silty Clay to Clay</td>
</tr>
<tr>
<td>4</td>
<td>Clayey Silt to Silty Clay</td>
</tr>
<tr>
<td>5</td>
<td>Silty Sand to Sandy Silt</td>
</tr>
<tr>
<td>6</td>
<td>Clean Sand to Silty Sand</td>
</tr>
<tr>
<td>7</td>
<td>Gravelly Sand to Dense Sand</td>
</tr>
<tr>
<td>8</td>
<td>Very Stiff Sand to Clayey Sand*</td>
</tr>
<tr>
<td>9</td>
<td>Very Stiff Fine-Grained*</td>
</tr>
</tbody>
</table>

*heavily overconsolidated or cemented
CPT
Tip Resistance

Cone Tip Resistance (tsf) vs Depth (ft)
CPT Friction Ratio
Depth (ft) vs Pore Pressure (tsf)
CPTGRAPHIC
PROFILE

CPT at a Highway Bridge Abutment
Effective Friction Angle (Kulhawy and Mayne, 1990)

\[ \phi = 17.6 + 11 \log (Q_{tn}) \]

\( Q_{tn} = \) Normalized Cone Tip Resistance, where

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

\( q_t \): Cone tip resistance
\( \sigma_{atm} : 1 \text{ atm} = 100 \text{kPa} \)
\( \sigma'_{v0} : \) Vertical effective stress
CPT Based undrained Shear Strength

Undrained Shear Strength (Robertson and Cabal, 2015)

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

- $q_t =$ Cone tip resistance
- $\sigma_{v0} =$ Vertical total stress
- $N_{kt} =$ Empirical cone factor
CPT Based Soil Behavior Type

Direct CPT Based Methods

Bustamante & Gianeselli or LCPC Method

Pile Data Analysis (PDA)

Open Ended Driven Steel Pipe Piles: Plugged & Unplugged
<table>
<thead>
<tr>
<th>Parameter</th>
</tr>
</thead>
<tbody>
<tr>
<td>Measured Cone Resistance, $q_c$</td>
</tr>
<tr>
<td>Friction Coefficient, $\alpha$</td>
</tr>
<tr>
<td>Pile Unit Side Friction, $f_p$</td>
</tr>
<tr>
<td>End Bearing Capacity Factor, $K_c$</td>
</tr>
<tr>
<td>Equivalent Average Cone Resistance, $q_{ca}$</td>
</tr>
<tr>
<td>Pile Unit End Bearing, $q_p$</td>
</tr>
</tbody>
</table>
$f_p = \frac{q_c}{\alpha}$

$q_p = k_c \ q_{ca}$
# Pile Categories

<table>
<thead>
<tr>
<th>Pile Category</th>
<th>Pile Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>IA</td>
<td>plain bored piles; mud bored piles, hollow auger bored piles, micropiles (grouted under low pressure), cast screwed piles, piers, barrettes</td>
</tr>
<tr>
<td>IB</td>
<td>cased bored piles, driven cast piles</td>
</tr>
<tr>
<td>IIA</td>
<td>driven pre-cast piles, prestressed tubular piles, jacket concrete piles</td>
</tr>
<tr>
<td>IIB</td>
<td>driven metal piles, jacked metal piles</td>
</tr>
<tr>
<td>IIIA</td>
<td>driven grouted piles, driven rammed piles</td>
</tr>
<tr>
<td>IIIB</td>
<td>high pressure grouted piles of large diameter &gt; 250 mm; micropiles grouted under high pressure</td>
</tr>
</tbody>
</table>
## Bearing Capacity Factors, $k_c$

<table>
<thead>
<tr>
<th>Nature of Soil</th>
<th>$q_c$ (MPa)</th>
<th>$q_c$ (tsf)</th>
<th>Group I</th>
<th>Group II</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soft clay and mud</td>
<td>&lt; 1</td>
<td>&lt;10</td>
<td>0.4</td>
<td>0.5</td>
</tr>
<tr>
<td>Moderately compact clay</td>
<td>1 to 5</td>
<td>10 to 52</td>
<td>0.35</td>
<td>0.45</td>
</tr>
<tr>
<td>Silty and Loose Sand</td>
<td>≤ 5</td>
<td>≤ 52</td>
<td>0.4</td>
<td>0.5</td>
</tr>
<tr>
<td>Compact to stiff clay and compact silt</td>
<td>&gt; 5</td>
<td>&gt; 52</td>
<td>0.45</td>
<td>0.55</td>
</tr>
<tr>
<td>Soft chalk</td>
<td>≤ 5</td>
<td>≤ 52</td>
<td>0.2</td>
<td>0.3</td>
</tr>
<tr>
<td>Moderately compact sand and gravel</td>
<td>5 to 12</td>
<td>52 to 125</td>
<td>0.4</td>
<td>0.5</td>
</tr>
<tr>
<td>Weathered to fragmented chalk</td>
<td>&gt; 5</td>
<td>&gt; 52</td>
<td>0.2</td>
<td>0.4</td>
</tr>
<tr>
<td>Compact to very compact sand and gravel</td>
<td>&gt; 12</td>
<td>&gt; 125</td>
<td>0.3</td>
<td>0.4</td>
</tr>
</tbody>
</table>
Friction Coefficient, $\alpha$

<table>
<thead>
<tr>
<th>Nature of Soil</th>
<th>qc (MPa)</th>
<th>qc (tsf)</th>
<th>A</th>
<th>B</th>
<th>A</th>
<th>B</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soft clay and mud</td>
<td>&lt; 1</td>
<td>&lt;10</td>
<td>30</td>
<td>90</td>
<td>90</td>
<td>30</td>
</tr>
<tr>
<td>Moderately compact clay</td>
<td>1 to 5</td>
<td>10 to 52</td>
<td>40</td>
<td>80</td>
<td>40</td>
<td>80</td>
</tr>
<tr>
<td>Silty and Loose Sand</td>
<td>≤ 5</td>
<td>≤ 52</td>
<td>60</td>
<td>150</td>
<td>60</td>
<td>120</td>
</tr>
<tr>
<td>Compact to stiff clay and compact silt</td>
<td>&gt; 5</td>
<td>&gt; 52</td>
<td>60</td>
<td>120</td>
<td>60</td>
<td>120</td>
</tr>
<tr>
<td>Soft chalk</td>
<td>≤ 5</td>
<td>≤ 52</td>
<td>100</td>
<td>120</td>
<td>100</td>
<td>120</td>
</tr>
<tr>
<td>Moderately compact sand and gravel</td>
<td>5 to 12</td>
<td>52 to 125</td>
<td>100</td>
<td>200</td>
<td>100</td>
<td>200</td>
</tr>
<tr>
<td>Weathered to fragmented chalk</td>
<td>&gt; 5</td>
<td>&gt; 52</td>
<td>60</td>
<td>80</td>
<td>60</td>
<td>80</td>
</tr>
<tr>
<td>Compact to very compact sand and gravel</td>
<td>&gt; 12</td>
<td>&gt; 125</td>
<td>150</td>
<td>300</td>
<td>150</td>
<td>200</td>
</tr>
</tbody>
</table>
### Maximum Limit of $f_p$ (tsf)

<table>
<thead>
<tr>
<th>Nature of Soil</th>
<th>qc (MPa)</th>
<th>qc (tsf)</th>
<th>I</th>
<th></th>
<th>II</th>
<th></th>
<th>III</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Soft clay and mud</td>
<td>&lt; 1</td>
<td>&lt;10</td>
<td>0.157</td>
<td>0.157</td>
<td>0.157</td>
<td>0.157</td>
<td>0.365</td>
<td></td>
</tr>
<tr>
<td>Moderately compact clay</td>
<td>1 to 5</td>
<td>10 to 52</td>
<td>0.365</td>
<td>0.365</td>
<td>0.365</td>
<td>0.365</td>
<td>0.835</td>
<td>≥ 1.253</td>
</tr>
<tr>
<td>Silty and Loose Sand</td>
<td>≤ 5</td>
<td>≤ 52</td>
<td>0.365</td>
<td>0.365</td>
<td>0.365</td>
<td>0.365</td>
<td>0.835</td>
<td></td>
</tr>
<tr>
<td>Compact to stiff clay and compact silt</td>
<td>&gt; 5</td>
<td>&gt; 52</td>
<td>0.365</td>
<td>0.365</td>
<td>0.365</td>
<td>0.365</td>
<td>0.835</td>
<td>≥ 2.088</td>
</tr>
<tr>
<td>Soft chalk</td>
<td>≤ 5</td>
<td>≤ 52</td>
<td>0.365</td>
<td>0.365</td>
<td>0.365</td>
<td>0.365</td>
<td>0.835</td>
<td></td>
</tr>
<tr>
<td>Moderately compact sand and gravel</td>
<td>5 to 12</td>
<td>52 to 125</td>
<td>0.835</td>
<td>0.365</td>
<td>0.835</td>
<td>0.835</td>
<td>1.253</td>
<td>≥ 2.088</td>
</tr>
<tr>
<td>Weathered to fragmented chalk</td>
<td>&gt; 5</td>
<td>&gt; 52</td>
<td>1.253</td>
<td>0.835</td>
<td>1.253</td>
<td>1.253</td>
<td>1.566</td>
<td>≥ 2.088</td>
</tr>
<tr>
<td>Compact to very compact sand and gravel</td>
<td>&gt; 12</td>
<td>&gt; 125</td>
<td>1.253</td>
<td>0.835</td>
<td>1.253</td>
<td>1.253</td>
<td>1.566</td>
<td>≥ 2.088</td>
</tr>
</tbody>
</table>
Equivalent Average Cone Resistance
Pile Plugging

Dimensions

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 $\alpha$, based on Soil Type and tip resistance

Unit Side Resistance = Tip Resistance/$\alpha$

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 $\times K_c$

Unplugged Pile End Bearing: Tip Resistance* unplugged end bearing
LCPC Method
• 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.
• estimate of ultimate toe resistance : 214 kips
• estimate of ultimate shaft resistance: 275 kips
• estimate of ultimate total resistance: 489 kips
• 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

• California Department of Transportation (Caltrans)

• Orange County Transportation Authority (OCTA)

• Fugro Consultants, Los Angeles

• Earthspecs, Inc, Orange County, California