What Are Additional Pile Design Considerations?

- Time Dependent Soil Strength Changes
- Scour
- Plugging
- Densification
- Predrilling / Jetting
- Drivability
Time Effects on Soil Resistance

Time effects are changes in soil resistance that occur with time.

Soil Setup

Relaxation
Soil Setup

**Soil setup** is a time dependent increase in the soil resistance or capacity.

In clay soils, setup is attributed to increases in effective stress as large excess positive pore pressures generated during driving dissipate as well as due to thixotropic effects.

In sands, setup is attributed primarily to aging effects and / or release of arching effects with time.
Soil setup frequently occurs for piles driven in **saturated clays** as well as **loose to medium dense silts and fine sands** as the excess pore pressures dissipate.

The magnitude of soil setup depends on soil characteristics as well as the pile material and type.
Relaxation

Relaxation is a time dependent decrease in the soil resistance or capacity.

During pile driving, dense soils may dilate thereby generating negative pore pressures and temporarily higher soil resistance.

Relaxation has been observed for piles driven in dense, saturated non-cohesive silts, fine sands, and some shales.
How to Quantify Time Effects

- Non-instrumented Restrike Tests
  - uncertain if blow count change due to change in hammer performance or soil strength
How to Quantify Time Effects

- Multiple Static Load Tests (time, $$)
How to Quantify Time Effects

- Dynamic Tests
  - inexpensive, cost-effective
  - check for setup or relaxation
  - short-term restrikes establish setup trend
  - long-term restrikes provide confidence in capacity estimates

\[ L = \text{Pile Length (ft)} \]
\[ c = \text{wave speed (ft/s)} \]
Soil Setup

End of Initial Driving

Restrike 8 Days Later

Shaft = 3 x EOID
Relaxation

End of Initial Driving

Restrike 24 Hours Later

Toe = \frac{1}{2} \text{ EOID}
<table>
<thead>
<tr>
<th>Static Analysis Methods</th>
<th>Setup - Yes</th>
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</thead>
<tbody>
<tr>
<td>Dynamic Formulas</td>
<td>Setup - Yes</td>
</tr>
<tr>
<td>Wave Equation Analysis</td>
<td>Depends</td>
</tr>
<tr>
<td>Dynamic Testing</td>
<td>Time of Test</td>
</tr>
<tr>
<td>Static Load Testing</td>
<td>Time of Test</td>
</tr>
</tbody>
</table>
Quantification of Time Effects

Restrike testing generally performed 1 to 10 days after installation
Efforts to Predict Setup

Skov and Denver (1988) – *Time-Dependence of Bearing Capacity of Piles*

\[ Q = Q_o (1 + A \log \left[ \frac{t}{t_o} \right]) \]

Sand: \( t_o = 0.5, A=0.2 \)
Clay: \( t_o = 1.0, A=0.6 \)

Bullock et al (2005) – *Side Shear Setup: Results from Florida Test Piles*

\[ \frac{Q_s}{Q_{s0}} = \frac{f_s}{f_{s0}} = A \log \left( \frac{t}{t_o} \right) + 1 = \left( \frac{m_s}{Q_{s0}} \right) \log \left( \frac{t}{t_o} \right) + 1 \]
where:  
\[ \frac{Q_s}{Q_{s0}} = \frac{f_s}{f_{s0}} = A \log \left( \frac{t}{t_0} \right) + 1 = \left( \frac{m_s}{Q_{s0}} \right) \log \left( \frac{t}{t_0} \right) + 1 \]

- \( Q_s \) = Side shear capacity at time \( t \)
- \( Q_{s0} \) = Side shear capacity at reference time \( t_0 \)
- \( f_s \) = Unit side shear capacity at time \( t \)
- \( f_{s0} \) = Unit side shear capacity at reference time \( t_0 \)
- \( t \) = Time elapsed since EOD, days
- \( t_0 \) = Reference time, recommended to use 1 day
- \( m_s \) = Semilog-linear slope of \( Q_s \) vs. \( \log t \)
18” PSC with O-Cell at bottom

- Aucilla, Dynamic Test
- Aucilla, Static Test

Bullock et al (2005) – Side Shear Setup: Results from Florida Test Piles

\[ Q_{S0} = 1021 \text{ kN (at } t_0 = 1\text{ day) } \]

\[ m_S = 293.4 \text{ kN} \]
For Closed-end Pipe Piles in Ohio Soils

\[ Q = 0.9957 \, Q_o \, t^{0.087} \]

\( Q \) = pile capacity (kips) at time \( t \) (hours) since EOID
  
  ( \( Q \) includes shaft + toe)

\( Q_o \) = pile capacity (kips) at EOID

Khan (2011) – *Prediction of Pile Set-up for Ohio Soils*
For H-piles in Cohesive Iowa Soils

\[
R_t = R_{EOD} \left[ \left( \frac{f_c C_{ha}}{N_a r_p^2} + f_r \right) \log_{10} \left( \frac{t}{t_{EOD}} \right) + 1 \right] \left( \frac{L_t}{L_{EOD}} \right)
\]

- \( R_t \) = Resistance at time \( t \) (kN)
- \( R_{EOD} \) = Resistance at EOD from WEAP-SA or CAPWAP (kN)
- \( C_{ha} \) = Weighted average horizontal coefficient of consolidation (cm\(^2\)/min)
- \( N_a \) = Weighted average uncorrected SPT N value
- \( r_p^2 \) = Equivalent pile radius (cm)
- \( f_c \) = Consolidation factor
- \( f_r \) = Remolding recovery factor
- \( t \) = Elapsed time since EOD, min
- \( t_{EOD} \) = Reference time of 1 min
- \( L_t / L_{EOD} \) = Normalized pile embedded length
For Puget Sound Lowlands

\[
R_s = \frac{R_{so} A \log\left(\frac{t}{t_0}\right)}{k_1 + k_2 R_{so} A \log\left(\frac{t}{t_0}\right)} + R_{so}
\]

- \( R_s \): shaft resistance at time “t” of driving (kips)
- \( R_{so} \): initial shaft resistance at “t_o” of driving (kips)
- \( A \): constant based on pile type (1.72 for PSC, 0.70 for CEP, 0.77 for OEP)
- \( t \): time after driving (hours)
- \( t_o \): time of driving (hours) (\( t_o = 1 \) hour for all tests)
- \( k_1 \): regression factor 1 (0.17 for PSC, 0.12 for CEP, 0.15 for OEP)
- \( k_2 \): regression factor 2 (0.00044 for PSC, 0.00078 for CEP, 0.00060 for OEP)

Reddy and Stueda (2014) – Time Dependent Capacity Increases of Piles Driven in Puget Sound Lowlands
Soil Setup Factor

The soil setup factor is defined as failure load determined from a static load test divided by the capacity at the end of driving.
<table>
<thead>
<tr>
<th>Predominant Soil Type Along Pile Shaft</th>
<th>Range in Soil Set-up Factor</th>
<th>Recommended Soil Set-up Factors*</th>
<th>Number of Sites and (Percentage of Data Base)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay</td>
<td>1.2 - 5.5</td>
<td>2.0</td>
<td>7 (15%)</td>
</tr>
<tr>
<td>Silt - Clay</td>
<td>1.0 - 2.0</td>
<td>1.0</td>
<td>10 (22%)</td>
</tr>
<tr>
<td>Silt</td>
<td>1.5 - 5.0</td>
<td>1.5</td>
<td>2 (4%)</td>
</tr>
<tr>
<td>Sand - Clay</td>
<td>1.0 - 6.0</td>
<td>1.5</td>
<td>13 (28%)</td>
</tr>
<tr>
<td>Sand - Silt</td>
<td>1.2 - 2.0</td>
<td>1.2</td>
<td>8 (18%)</td>
</tr>
<tr>
<td>Fine Sand</td>
<td>1.2 - 2.0</td>
<td>1.2</td>
<td>2 (4%)</td>
</tr>
<tr>
<td>Sand</td>
<td>0.8 - 2.0</td>
<td>1.0</td>
<td>3 (7%)</td>
</tr>
<tr>
<td>Sand - Gravel</td>
<td>1.2 - 2.0</td>
<td>1.0</td>
<td>1 (2%)</td>
</tr>
</tbody>
</table>

* - Confirmation with Local Experience Recommended
EOID, \( t = 0.0007 \text{ Days} \)
Rs = 453, Rt = 1846, Ru = 2299 kN

BOR #1, \( t = 0.06 \text{ Days} \)
Rs = 818, Rt = 2047, Ru = 2865 kN

BOR #2, \( t = 0.69 \text{ Days} \)
Rs = 1046, Rt = 2077, Ru = 3123 kN

BOR #3, \( t = 14.65 \text{ Days} \)
Rs = 1530, Rt = 2113, Ru = 3643 kN

BOR #4, \( t = 48.81 \text{ Days} \)
Rs = 2121, Rt = 2092, Ru = 4213 kN
For Closed-end Pipe Piles in Wisconsin

Capacity (kN)

Total
Shaft

Time (log)

0.0001 0.001 0.01 0.1 1 10 100

DLT
SLT
Relaxation is a time dependent decrease in the soil resistance or capacity.

During pile driving, dense soils may dilate thereby generating negative pore pressures and temporarily higher soil resistance.

Relaxation has been observed for piles driven in dense, saturated non-cohesive silts, fine sands, and some shales.
Relaxation Factor

The relaxation factor is defined as failure load determined from a static load test divided by the capacity at the end of driving.

Relaxation factors of 0.5 to 0.9 have been reported in case histories of piles in shales.

Relaxation factors of 0.5 and 0.8 have been observed in dense sands and extremely dense silts, respectively.
Time Effects on Pile Drivability and Pile Capacity

Time dependent soil strength changes should be considered during the design stage.

Tools that have been used include:

- SPT – torque and vane shear tests
- CPTu with dissipation tests
- Model piles
- Soil setup / relaxation factors
Piles Subject to Scour

Types of Scour

Aggradation / Degradation Scour
- Long-term stream bed elevation changes

Local Scour
- Removal of material from immediate vicinity of foundation

Contraction and General Scour
- Erosion across all or most of channel width
Piles Subject to Scour

- Bridge Deck
- Original Bed
- Channel Degradation Scour
- Local Scour
- Piles
Pile Design Recommendations in Soils Subject to Scour

1. Reevaluate foundation design relative to pile length, number, size and type

2. Design piles for additional lateral restraint and column action due to increase in unsupported length

3. Local scour holes may overlap, in which case scour depth is indeterminate and may be deeper.
Pile Design Recommendations in Soils Subject to Scour

4. Perform design assuming all material above scour line has been removed.

5. Place top of footing or cap below long-term scour depth to minimize flood flow obstruction.

6. Piles supporting stub abutments in embankments should be driven below the thalweg elevation.
Open end sections include open end pipe piles and H-piles.

It is generally desired that the open sections remain unplugged during driving and behave plugged under static loading conditions.

Why?
Factors influencing plug development include hammer size and penetration to pile diameter ratio (D/b).

During driving:

- Large hammer $\Rightarrow$ plug slippage
- Small hammer $\Rightarrow$ plug formation

Therefore, size pile for larger hammer.
Plugging of Open Pile Sections

Under **static** conditions, plugging likely in:
- Dense sand & clay $\Rightarrow D/b \geq 20$
- Medium dense sand $\Rightarrow D/b \geq 20$ to 30

Under **dynamic** conditions, plugging may occur (b $\leq$ 30 inch diameter) in very dense sands unless the penetration into the dense layer is very shallow (<3b). For large diameter piles (b $\geq$ 60 inch diameter), plugging will rarely occur during driving.
Plugging of Open Pile Sections

H-piles are usually assumed to be plugged under static loading conditions due to the smaller section size.
Plugging of Open Pile Sections
Use of Constrictor Plates in Open End Pipe Piles to Force Plugging

Bottom View

Top View
Densification Effects on Soil Resistance
Densification Effects on Soil Resistance

Densification can result in the soil resistance as well as the pile penetration resistance (blow count) being significantly greater than that calculated for a single pile.

Added confinement from cofferdams or the sequence of pile installation can further aggravate a densification problem.
Potential densification effects should be considered in the design stage. Studies indicate an increase in soil friction angle of up to $4^\circ$ would not be uncommon for piles in loose to medium dense sands.

A lesser increase in friction angle would be expected in dense sands or cohesionless soils with a significant fine content.
Effects of Predrilling and Jetting on Soil Resistance
Effects of Predrilling and Jetting on Soil Resistance

Predrilling in cohesive soils and jetting in cohesionless soils are sometimes used to satisfy minimum penetration requirements.

Both predrilling and jetting will affect the axial, lateral, and uplift capacity that can be attained.
Effects of Predrilling and Jetting on Pile Capacity

Depending upon the predrilled hole diameter, the shaft resistance in the predrilled zone may be reduced to between 50 and 85% of the shaft resistance without predrilling.

In jetted zones, shaft resistance reductions of up to 50% have been reported.
Effects of Predrilling and Jetting on Pile Capacity

It is important that the effect of predrilling or jetting be evaluated by design personnel whenever it is proposed.
Pile Drivability

The ability of a pile to be driven to the desired depth and / or capacity at a reasonable blow count without exceeding the material driving stress limits.
Factors Affecting Pile Drivability

- Driving system characteristics
- Pile material strength
- Pile impedance, EA/C
- Dynamic soil response

Primary factor controlling drivability
Pile Drivability

Pile drivability should be checked by wave equation analysis during the design stage for all driven piles.

Pile drivability is particularly critical for closed end pipe piles.
Questions