SOILS AND FOUNDATIONS
Lesson 09
Chapter 9 – Deep Foundations
Lesson Plan

- **Topic 1 (Section 9.0 to 9.8)**
  - Driven piles
  - Static capacity

- **Topic 2 (Section 9.9)**
  - Driven Piles - Construction Monitoring and QA

- **Topic 3 (Section 9.9.10)**
  - Driven Piles – Load Tests

- **Topic 4 (Section 9.10)**
  - Drilled shafts
  - Static capacity
  - Construction
Deep Foundations

Lesson 09 - Topic 1
Driven Piles and Static Capacity
Section 9.0 to 9.8
Learning Outcomes

At the end of this session, the participant will be able to:

- Describe types of driven piles and applications for use
- Compute static capacity for driven piles in granular and cohesive soils
- Identify 2 of the design steps in pile groups
- Discuss negative skin friction
Deep foundations may be used at pier and abutment locations.
Establishment of need for deep foundations
Structural Foundation Topics

- Shallow Foundations (Spread Footings)
  - Bearing Capacity
  - Settlement

- Deep Foundations
  - Load Capacity
  - Settlement
  - Negative Skin Friction
The foundation designer must define at what depth suitable soil layers begin in the soil profile.
Situations Where a Deep Foundation is Needed

Figure 9-1
Situations Where a Deep Foundation is Needed

Figure 9-1
Deep Foundation Classification System

- Figure 9-2
Specific terminology for deep foundations

- Static pile capacity
- Ultimate pile capacity
- Driving capacity
- Restrike capacity
- Shaft resistance in piles
- Side resistance in drilled shafts
- Toe resistance for piles
- Tip or base resistance for shafts
- And more……..
Structural Terminology

- **Allowable load**
- **Design load**
  - Equal to or less than allowable load
- **Ultimate (Nominal) load**
- **Table 9-10 for maximum structural design stress and maximum structural driving stress**
Types of Piling

- Concrete
- Steel Pipe
- Timber
- Steel H
- Pre-cast Concrete
- Composite
<table>
<thead>
<tr>
<th>Type of Pile</th>
<th>Typical Axial Design Loads</th>
<th>Typical Lengths</th>
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<tbody>
<tr>
<td>Timber</td>
<td>20-110 kips (100 – 500 kN)</td>
<td>15-120 ft (5-37 m)*</td>
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<tr>
<td>Precast / Prestressed</td>
<td>90-225 kips (400-1000 kN) for reinforced</td>
<td>30-50 ft (10-15m) for reinforced</td>
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<tr>
<td>Reinforced Concrete</td>
<td>90-1000 kips (400-4500 kN) for prestressed</td>
<td>50-130 ft (15-40m) for prestressed</td>
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<tr>
<td>Steel H</td>
<td>130-560 kips (600-2500 kN)</td>
<td>15-130 ft (5-40 m)</td>
</tr>
<tr>
<td>Steel Pipe (without concrete core)</td>
<td>180-560 kips (800-2500 kN)</td>
<td>15-130 ft (5-40 m)</td>
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<tr>
<td>Steel Pipe (with concrete core)</td>
<td>560-3400 kips (2500-15000 kN)</td>
<td>15-130 ft (5-40 m)</td>
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</tbody>
</table>

* 15-75 ft (5-23 m) for Southern Pine; 15-120 ft (5-37 m) for Douglas Fir
**Effect of Subsurface and Hydraulic Conditions on Piles**

### Table 9-2

<table>
<thead>
<tr>
<th>Typical Problem</th>
<th>Recommendations</th>
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<tr>
<td>Boulders overlying bearing stratum</td>
<td>Use heavy nondisplacement pile with a reinforced tip or manufactured point and include con-tingent predrilling item in contract.</td>
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<tr>
<td>Loose cohesionless soil</td>
<td>Use tapered pile to develop maximum skin friction.</td>
</tr>
<tr>
<td>Negative skin friction</td>
<td>Use smooth steel pile to minimize drag adhesion, and avoid battered piles. Minimize the magnitude of drag force when possible.</td>
</tr>
<tr>
<td>Deep soft clay</td>
<td>Use rough concrete pile to increase adhesion and rate of pore water dissipation.</td>
</tr>
<tr>
<td>Artesian Pressure</td>
<td>Do not use mandrel driven thin-wall shells as generated hydrostatic pressure may cause shell collapse; pile heave common to closed-end pipe.</td>
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<tr>
<td>Scour</td>
<td>Do not use tapered piles unless large part of taper extends well below scour depth. Design permanent pile capacity to mobilize soil resistance below scour depth.</td>
</tr>
<tr>
<td>Coarse Gravel Deposits</td>
<td>Use precast concrete piles where hard driving expected in coarse soils. DO NOT use H-piles or open end pipes as nondisplacement piles will penetrate at low blow count and cause unnecessary overruns.</td>
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</table>
## Pile Shape Effects

### Table 9-3

<table>
<thead>
<tr>
<th>Shape Characteristics</th>
<th>Pile Types</th>
<th>Placement Effects</th>
</tr>
</thead>
</table>
| **Displacement**      | Steel Pipe (Closed end), Precast Concrete | • Increase lateral ground stress  
  • Densify cohesionless soils, remolds and weakens cohesive soils temporarily  
  • Set-up time may be 6 months in clays for pile groups |
| **Nondisplacement**   | Steel H, Steel Pipe (Open end) | • Minimal disturbance to soil  
  • Not suited for friction piles in coarse granular soils. Piles often have low driving resistances in these deposits making field capacity verification difficult thereby often resulting in excessive pile lengths. |
| **Tapered**           | Timber, Monotube, Tapertube, Thin-wall shell | • Increased densification of soils with less disturbance, high capacity for short length in granular soils |
Other issues

- Noise and vibrations during installation
- Remote areas may restrict driving equipment size
- Local availability of certain materials
- Waterborne operations may dictate some handling limitations (e.g., shorter pile sections)
- Steep terrain may make use of certain pile equipment costly or impossible
Cost Evaluation of Alternate Deep Foundation Types

- Often several deep foundation types meet project requirements
- Final choice must be made on cost analysis
- In cost analysis include ALL costs related to a given pile type
  - Uncertainties in execution, time delays, cost of load testing, cost of pile caps, noise and vibrations, etc
Cost Evaluation of Alternate Pile Types

- Three major categories of cost for driven piles
  - Pile support cost
  - Pile cap support cost
  - Construction control method support cost

- For most piles, the pile cost is usually linear with depth based on unit price
  - May not be true for very long concrete or long, large section steel piles
    • Special handling, splicing, etc.
Cost Evaluation of Alternate Pile Types

- Express costs in terms of $/ton capacity for each alternative as discussed in Chapter 8

- For cost savings recommendations, see Table 9-4
Computation of Pile Capacity

- **Ultimate pile capacity**, $Q_u$
- **Shaft resistance**, $R_s$
- **Toe resistance**, $R_t$

$$Q_u = R_s + R_t$$
Computation of Pile Capacity

- **Shaft resistance**, \( R_s = f_s A_s \)
  
  - \( f_s \) is unit shaft resistance
  - \( A_s \) is shaft surface area

- **Toe resistance**, \( R_t = q_t A_t \)
  
  - \( q_t \) is unit toe resistance
  - \( A_t \) is pile toe area

- \( Q_u = R_s + R_t = f_s A_s + q_t A_t \)
Allowable Geotechnical Pile Load

The allowable geotechnical pile load, $Q_a$, is defined as follows in terms of $Q_u$ and factor of safety, $FS$

$$Q_a = \frac{Q_u}{\text{Factor of Safety}}$$
**Factor of Safety**

- **FS depends on the following:**
  - Level of confidence in input parameters
  - Variability of soil and rock
  - Method of static analysis
  - Proposed pile installation method
  - Level of construction monitoring

- **The FS used in static analysis should be based upon the construction control method specified**
## Factor of Safety (Table 9-5)

<table>
<thead>
<tr>
<th>Construction Control Method</th>
<th>Factor of Safety</th>
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</thead>
<tbody>
<tr>
<td>Static load test (ASTM D-1143) with wave equation analysis</td>
<td>2.00</td>
</tr>
<tr>
<td>Dynamic testing (ASTM D-4945) with wave equation analysis</td>
<td>2.25</td>
</tr>
<tr>
<td>Indicator piles with wave equation analysis</td>
<td>2.50</td>
</tr>
<tr>
<td>Wave equation analysis</td>
<td>2.75</td>
</tr>
<tr>
<td>Gates dynamic formula</td>
<td>3.50</td>
</tr>
</tbody>
</table>
**FS as a function of Soil Resistance**

\[ Q_u = R_{s1} + R_{s2} + R_{s3} + R_t \]

*Assume static load test*

**For design**

\[ Q_a = \frac{(R_{s3} + R_t)}{(FS=2)} \]

\[ Q_a = \frac{(Q_u - R_{s1} - R_{s2})}{(FS=2)} \]

**For plans and specs**

\[ Q_u = R_{s1} + R_{s2} + (Q_a)(FS=2) \]
Soil Driving Resistance (SRD)

In SRD, FS is not used

\[ \text{SRD} = R_{s1} + R_{s2} + R_{s3} + R_t \]

Soil Setup and Relaxation are considered in SRD

Assume soft clay layer has sensitivity of 2

\[ \text{SRD} = R_{s1} + \frac{R_{s2}}{2} + R_{s3} + R_t \]

**SRD should also include resistance to penetrate hard or dense layers**
**Example 9-1**

**Compute ultimate capacity and driving capacity**

<table>
<thead>
<tr>
<th>Layer</th>
<th>R&lt;sub&gt;s1&lt;/sub&gt;</th>
<th>R&lt;sub&gt;s2&lt;/sub&gt;</th>
<th>R&lt;sub&gt;s3&lt;/sub&gt;</th>
<th>Sensitivity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand</td>
<td>20 tons</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Soft Clay</td>
<td>R&lt;sub&gt;s2&lt;/sub&gt; = 20 tons</td>
<td>Sensitivity = 4</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Gravel</td>
<td>R&lt;sub&gt;s3&lt;/sub&gt; = 60 tons</td>
<td>R&lt;sub&gt;t&lt;/sub&gt; = 40 tons</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Design of Single Piles

- Cohesionless Soils
- Cohesive Soils
- Rocks
Single Piles in Cohesionless Soils
Nordlund’s Method

\[ Q_u = \sum_{d=0}^{d=D} K_\delta C_F p_d \frac{\sin(\delta + \omega)}{\cos \omega} C_d \Delta d + \alpha_t N'q A_t p_t \]

Figure 9-5
For a pile of uniform cross section ($\omega=0$) and embedded length $D$, driven in soil layers of the same effective unit weight and friction angle, the Nordlund equation becomes:

\[ Q_u = (K_\delta C_F p_d \sin \delta C_d D) + (\alpha_t N'_q A_t \rho_t) \]

\[ R_S \]

\[ R_T \]
Nordlund Shaft Resistance

\[ R_s = K_\delta C_F \rho_d \sin \delta C_d D \]

- \( K_\delta \) = coefficient of lateral earth pressure
- \( C_F \) = correction factor for \( K_\delta \) when \( \delta \neq \phi \)
- \( \rho_d \) = effective overburden pressure at center of layer
- \( \delta \) = friction angle between pile and soil
- \( C_d \) = pile perimeter
- \( D \) = embedded pile length

Figures 9.7 - 9.10, Figure 9.11, Figure 9.9
Nordlund Toe Resistance

\[ R_t = \alpha_t \ N'_q \ p_t \ A_t \]

Lesser of

\[ R_t = q_L \ A_t \]

\( \alpha_t \) = dimensionless factor  \hspace{1cm} \text{Figure 9.12a}

\( N'_q \) = bearing capacity factor \hspace{1cm} \text{Figure 9.12b}

\( A_t \) = pile toe area

\( p_t \) = effective overburden pressure at pile toe \( \leq 3 \) ksf

\( q_L \) = limiting unit toe resistance \hspace{1cm} \text{Figure 9.17}
Arching at Pile Tip

Ground Surface

Arching Action

Zone of Shear & Volume Decrease

$p_0 = \alpha \gamma D_f$

$\gamma D_f$

$D_f$

$B$

Arching at Pile Tip
Nordlund Method

\[ Q_u = R_S + R_T \]

and

\[ Q_a = \frac{Q_U}{FS} \]

FS based on construction control method as in Table 9-5
Steps 1 through 6 are for computing shaft resistance and steps 7 through 9 are for computing the pile toe resistance.

**STEP 1** Delineate the soil profile into layers and determine the $\phi$ angle for each layer.

1. Construct $p_o$ diagram using procedure described in Chapter 2.
2. Correct SPT field N values for overburden pressure using Figure 3-23 from Chapter 3 and obtain corrected $N_{160}$ values. Delineate soil profile into layers based on corrected $N_{160}$ values.
3. Determine $\phi$ angle for each layer from laboratory tests or in-situ data.
4. In the absence of laboratory or in-situ test data, determine the average corrected $N_{160}$ value, $N'$, for each soil layer and estimate $\phi$ angle from Table 8-3 in Chapter 8.
STEP 2  Determine \( \delta \), the friction angle between the pile and soil based on the displaced soil volume, \( V \), and the soil friction angle, \( \phi \).

a. Compute volume of soil displaced per unit length of pile, \( V \).

b. Enter Figure 9-6 with \( V \) and determine \( \delta/\phi \) ratio for pile type.

c. Calculate \( \delta \) from \( \delta/\phi \) ratio.
STEP 3  Determine the coefficient of lateral earth pressure $K_\delta$ for each soil friction angle, $\phi$.

a. Determine $K_\delta$ for each $\phi$ angle based on displaced volume $V$, and pile taper angle, $\omega$, using appropriate procedure in steps 3b, 3c, 3d, or 3e.

b. If displaced volume is 0.1, 1.0, 10 ft$^3$/ft and the friction angle is 25, 30, 35, or 40, use Figures 9-7 to 9-10.

c. If displaced volume is given but $\phi$ angle is not. Linear interpolation is required to determine $K_\delta$ for $\phi$ angle.
$K_\delta$ versus $\omega$

$\phi = 25^\circ$

Figure 9-7
Nordlund Method Procedure

STEP 3 Determine the coefficient of lateral earth pressure $K_\delta$ for each soil friction angle, $\phi$.

d. If displaced volume is not given but $\phi$ angle is given, log linear interpolation is required to determine $K_\delta$ for displaced volume $V$.

e. If neither the displaced volume or $\phi$ angle are given, first use linear interpolation to determine $K_\delta$ for $\phi$ angle and then use log linear interpolation to determine $K_\delta$ for the displaced volume, $V$.

See Table 9-6 for $K_\delta$ as function of $\phi$ angle and displaced volume $V$. 
### Table 9-6(a) Design Table for Evaluating $K_\delta$ for Piles when $\omega = 0^\circ$ and $V = 0.10$ to 1.00 ft$^3$/ft

<table>
<thead>
<tr>
<th>$\phi$</th>
<th>0.10</th>
<th>0.20</th>
<th>0.30</th>
<th>0.40</th>
<th>0.50</th>
<th>0.60</th>
<th>0.70</th>
<th>0.80</th>
<th>0.90</th>
<th>1.00</th>
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<tr>
<td>25</td>
<td>0.70</td>
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<td>0.79</td>
<td>0.80</td>
<td>0.82</td>
<td>0.83</td>
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<td>26</td>
<td>0.73</td>
<td>0.78</td>
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<td>0.84</td>
<td>0.86</td>
<td>0.87</td>
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<td>2.87</td>
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<td>3.0</td>
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</table>
Nordlund Method Procedure

STEP 4  Determine the correction factor $C_F$ to be applied to $K_\delta$ if $\delta \neq \phi$.

Use Figure 9-11 to determine the correction factor for each $K_\delta$. Enter figure with $\phi$ angle and $\delta/\phi$ ratio to determine $C_F$. 
Correction Factor for $K_\delta$ when $\delta \neq \phi$

![Figure 9-11](image)
Nordlund Method Procedure

STEP 5  Compute the average effective overburden pressure at the midpoint of each soil layer.

STEP 6  Compute the shaft resistance in each soil layer. Sum the shaft from each layer to obtain the ultimate shaft resistance, \( R_s \).

\[ R_s = K \delta C_F p_d \sin \delta C_d D \]
STEP 7  Determine the $\alpha_t$ coefficient and the bearing capacity factor, $N'_q$, from the $\phi$ angle near the pile toe.

a. Enter Figure 9-12(a) with $\phi$ angle near pile toe to determine $\alpha_t$ coefficient based on pile length to diameter ratio.

b. Enter Figure 9-12(b) with $\phi$ angle near pile toe to determine, $N'_q$.

c. If $\phi$ angle is estimated from SPT data, compute the average corrected SPT $N_{160}$ value over the zone from the pile toe to 3 diameters below the pile toe. Use this average corrected SPT $N_{160}$ value to estimate $\phi$ angle near pile toe from Table 8-3.
STEP 8  Compute the effective overburden pressure at the pile toe.

NOTE: The limiting value of $p_t$ is 3 ksf (150 kPa)

STEP 9  Compute the ultimate toe resistance, $R_t$.

Use lesser of:

\[
R_t = \alpha_t N'_q p_t A_t
\]

\[
R_t = q_L A_t
\]

Figure 9-12a and 9-12b

Figure 9-13
\( \alpha_t \) Coefficient versus \( \phi \)

Figure 9-12a
Limiting Unit Toe Resistance

Figure 9-13

Limiting Unit Toe Resistance, $q_L$ (ksf)

Angle of Internal Friction, $\phi$ (degrees)