Session I

Introduction

Topic 1

Opening Remarks & Introductions
Opening Remarks

Welcome!

LRFD for
Highway Bridge Foundations and
Earth Retaining Structures
OBJECTIVES

A. Identify Key Program Personnel
B. Discuss ODOT Training Course Outcomes
C. Concur with Course Ground Rules
D. Navigate Curriculum Material
Key Program Personnel

- Host Agency
- Course Instructor
- Course Participants
- Guests
Host Agency

Ohio Department of Transportation Office of Structures

- Administrator: Tim Keller
- Assistant Administrator: Jawdat Siddiqi
- Foundation Coordinator: Peter Narsavage
Key Program Personnel

Course Instructor

- Robert Liang
  - BSCE, Tamkang University, Taiwan
  - MS, North Carolina State University
  - Ph. D, University of California, Berkeley

- PE / State(s) (Ohio and California)
Key Program Personnel

Participants

- Your Name / Employer?
- Area of Expertise?
- Years of Design Experience?
- LRFD Design Experience?
- Degrees / Licenses / P.E.’s?
Introductions

So Who is Here?

Let's Get Started
ODOT Training Course
Outcomes

Upon Completion of This Course, Participants Should be able to...

1. Explain the fundamentals of LRFD bridge design as it relates to foundations and earth retaining structures.
2. Apply LRFD foundation criteria to shallow and deep foundation designs.

3. Apply LRFD to retaining wall design
Ground Rules

- Attendance taken daily
- Daily schedule times:
  - ✓ Start 8:00 a.m.
  - ✓ Lunch 1-hour (11:30 a.m. - 12:30 p.m.)
  - ✓ Finish 4:30 p.m.
- Ask questions any time
- **Stay focused on the outcomes**
- Restroom locations
- Restaurant locations
Any More

Ground Rules

• Mute Pagers and Cell Phones
• Observe break and lunch time limits
• Only one person talking at a time
• Leave room when necessary
• Smoking in designated areas only
<table>
<thead>
<tr>
<th>Session</th>
<th>Topic</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>Opening Remarks</td>
</tr>
<tr>
<td>II-1</td>
<td>Principle of Limit State Designs</td>
</tr>
<tr>
<td>II-2</td>
<td>Loads and Load Combinations</td>
</tr>
<tr>
<td>II-3</td>
<td>Serviceability Performance Limit</td>
</tr>
<tr>
<td>III</td>
<td>Soil and Rock Materials</td>
</tr>
<tr>
<td>IV</td>
<td>LRFD Design of Shallow Foundations</td>
</tr>
<tr>
<td>V</td>
<td>Guided Design Example of Shallow Foundations. In-Class Example</td>
</tr>
<tr>
<td>Session</td>
<td>Topic</td>
</tr>
<tr>
<td>---------</td>
<td>-------</td>
</tr>
<tr>
<td>VI</td>
<td>LRFD Design of Deep Foundations</td>
</tr>
<tr>
<td>VII</td>
<td>Guided Design of Driven Pile Group In-Class Exercise</td>
</tr>
<tr>
<td>VIII-1</td>
<td>LRFD Design of Cast-in-Place Gravity and Semi-Gravity Walls In-Class Example</td>
</tr>
<tr>
<td>VIII-2</td>
<td>LRFD Design of Mechanically Stabilized Walls</td>
</tr>
<tr>
<td>Session</td>
<td>Topic</td>
</tr>
<tr>
<td>---------</td>
<td>--------------------------------------------</td>
</tr>
<tr>
<td>VIII-3</td>
<td>LRFD Design of Anchored Walls</td>
</tr>
<tr>
<td>IX</td>
<td>Guided Design of MSE Wall and Anchored Wall</td>
</tr>
<tr>
<td>X</td>
<td>Review, Exam, and Course Evaluation</td>
</tr>
</tbody>
</table>
Learning Outcomes

A. Identify Key Program Personnel
B. Discuss ODOT Training Course Outcomes
C. Concur with Course Ground Rules
D. Navigate Curriculum Material
Session II

Fundamentals of LRFD

Topic 1

Principles of Limit State Designs
OBJECTIVES

A. Define the term “Limit State”
B. Define the term “Resistance”
C. Identify the applicability of each of the four primary limit states
OBJECTIVES

D. Identify the three limit states used in substructure design
E. Identify and define the six components of the fundamental LRFD equation
Define the term “Limit State”

A Limit State is a condition beyond which a structural component, ceases to satisfy the provisions for which it is designed.
Define the Term “Resistance”

Resistance is a quantifiable value that defines the point beyond which the particular limit state under investigation for a particular component will be exceeded.
Resistance can be defined in terms of:

- Load/Force
- Stress (normal, shear, torsion)
- Number of cycles
- Temperature
- Strain
- Etc.
Application of the Primary Limit States
Limit States

- Strength Limit State
- Extreme Event Limit State
- Service Limit State
- Fatigue Limit State
Strength Limit State
Extreme Event Limit State
Extreme Event Limit State
Extreme Event Limit State

REPEAT
Service Limit State
Fatigue Limit State
Identify the three limit states used in bridge substructure design
Substructure Design

- Strength Limit State
- Extreme Limit State
- Service Limit State
Identify and define each of the six components of the fundamental equation
\[ \Sigma \eta_i \gamma_i Q_i \leq R_r = \phi R_n \]

- \( \eta_i \) = Load modifier (eta)
- \( \gamma_i \) = Load factor (gamma)
- \( Q_i \) = Force effect
- \( R_r \) = Factored resistance
- \( \phi \) = Resistance factor (phi)
- \( R_n \) = Nominal resistance
Learning Outcomes

A. Define the term “Limit State”
B. Define the term “Resistance”
C. Identify the applicability of each of the four primary limit states
Learning Outcomes

D. Identify the three limit states used in substructure design

E. Identify and define the six components of the fundamental LRFD equation
Session II

Fundamentals of LRFD

Topic 2

Loads and Load Combinations
OBJECTIVES

A. Identify the loads applied to a substructure and present the basic equations for computing load combinations.

B. Identify the loads & load factors that make up the load combinations for each limit state.
OBJECTIVES

C. Identify the applicable LRFD load combination and load factors that will produce maximum and minimum force effects on substructures.
Identify Loads and Basic Equations

Load Designations

- Permanent Loads
- Transient Loads
Loads and Basic Equations

Permanent Loads

- **DC** – Dead Load of Components
- **DW** – Dead Load of Wearing Surface
- **EH** – Earth Load Horizontal
- **EL** – Effect of Locked-In Loads
- **ES** – Earth Load Surcharge
- **EV** – Earth Load Vertical
- **DD** – Downdrag
Lateral earth pressure (p)

- Stiffness of structure
- Characteristics of retained earth

\[ P = k \gamma_s z \]

- \( k = k_o, k_a, \) or \( k_p \)
- \( \gamma_s = \) soil weight

AASHTO 3.5.2, 3.11.5.1
Passive pressure estimate for cohesive soils ($p_p$)

\[ p_p = k_p \gamma_s z + 2c \sqrt{k_p} \]

- Vertical/sloping walls with horizontal backfill
- Vertical walls with sloping backfill

*AASHTO 3.11.5.4*
## Movement Requirements

<table>
<thead>
<tr>
<th>Type of Backfill</th>
<th>$\Delta/H$ (ft/ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Active</td>
</tr>
<tr>
<td>Dense Sand</td>
<td>0.001</td>
</tr>
<tr>
<td>Medium dense sand</td>
<td>0.002</td>
</tr>
<tr>
<td>Loose sand</td>
<td>0.004</td>
</tr>
<tr>
<td>Compacted silt</td>
<td>0.002</td>
</tr>
<tr>
<td>Compacted lean clay</td>
<td>0.010</td>
</tr>
</tbody>
</table>

*Source: AASHTO Table C3.11.1-1*
Equivalent-fluid method pressure estimation (Rankine)

\[ p = \gamma_{eq} z \]

\( \gamma_{eq} = \) Equivalent Fluid Pressure

**AASHTO Table 3.11.5.5-1**

- Wall heights \( \leq 20.0 \) ft
- Sand or gravel (loose, medium or dense)
- Level backfill or \( \beta = 25^\circ \)
## Typical Values for Equivalent Fluid Unit weights of Soils

<table>
<thead>
<tr>
<th>Type of Soil</th>
<th>Level Backfill</th>
<th>Backfill with $\beta = 25^\circ$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>At-Rest $\gamma_{eq}$ (kcf)</td>
<td>Active $\Delta/H = 1/240$ $\gamma_{eq}$ (kcf)</td>
</tr>
<tr>
<td>Loose sand or gravel</td>
<td>0.055</td>
<td>0.040</td>
</tr>
<tr>
<td>Medium dense sand or gravel</td>
<td>0.050</td>
<td>0.035</td>
</tr>
<tr>
<td>Dense sand or gravel</td>
<td>0.045</td>
<td>0.030</td>
</tr>
</tbody>
</table>

*Note: $\gamma_{eq}$ refers to the equivalent fluid unit weight of the soil.*

AASHTO Table 3.11.5.5-1
Equivalent-fluid method with sloping backfill

- **Vertical component**
  - $P_v = P_h \tan \beta$

- **Horizontal component**
  - $P_h = 0.5 \gamma_{eq} H^2$
ES – Surcharge Loads

Uniform surcharge loads

- \( \Delta_p = k_s q_s \)
- \( k_s = \) coefficient of earth pressure due to surcharge (dim)
- \( q_s = \) uniform surcharge applied to upper surface of active earth wedge (ksf)
Uniformly Loaded Strip – Parallel

\[ \Delta_{ph} = \frac{2p}{\pi} \left[ \delta - \sin \delta \cos (\delta + 2\alpha) \right] \]

**AASHTO 3.11.6.2, Figure 3.11.6.2-1**
\[
\Delta_{ph} = \frac{P}{\pi R^2} \left[ \frac{3ZX^2}{R^3} - \frac{R(1-2v)}{R+Z} \right]
\]

AASHTO 3.11.6.2

AASHTO Figure 3.11.6.2-2
\[ \Delta_{ph} = \frac{4Q}{\pi} \frac{X^2Z}{R^4} \]
\[ \Delta_{ph} = \frac{Q}{\pi Z} \left[ \frac{1}{A^3} - \frac{1-2\nu}{A + \frac{Z}{X_2}} - \frac{1}{B^3} + \frac{1-2\nu}{B + \frac{Z}{X_1}} \right] \]

\[ A = \sqrt{1 + \left(\frac{Z}{X_2}\right)^2} \]

\[ B = \sqrt{1 + \left(\frac{Z}{X_1}\right)^2} \]

AASHTO 3.11.6.2

AASHTO Figure 3.11.6.2-4
Flexible wall strip loads

Concentrated vertical loads – use 2V:1H

For strip load:
\[ \Delta \sigma_v = \frac{P_v}{D_1} \]

For isolated footing load:
\[ \Delta \sigma_v = \frac{P'_{f}}{D_{1}(L+Z)} \]

For point load:
\[ \Delta \sigma_v = \frac{P'_{p}}{D_1^2} \text{ with } b_f = 0 \]
Flexible wall strip loads

Concentrated horizontal loads

\[ \Delta \sigma_H \max = 2 \Sigma F/l_1 \]

\[ l_1 = (C_f + b_f - 2e') \tan (45 + \Phi r/2) \]

\[ \Sigma F = P_{H1} + F_1 + F_2 \]

\[ F_1 = \text{lateral force due to earth pressure} \]
\[ F_2 = \text{lateral force due to traffic surcharge} \]
\[ P_{H1} = \text{lateral force due to superstructure or other concentrated lateral loads} \]

\( e' = \text{eccentricity of load on footing (see Figure 11.10.10.1-1 for example of how to calculate this)} \)

a. Distribution of Stress for Internal Stability Calculations.
Flexible wall strip loads

Concentrated horizontal loads

\[ l_2 = (C_f + b_f - 2e')\tan(45 + \phi_f/2) \]

\[ \Delta \sigma_H = 2 \Sigma F/I_2 \]

\[ \Sigma F = P_{H2} + F_1 + F_2 \]

\[ P_{H2} = \text{lateral force due to superstructure or other concentrated lateral loads} \]

If footing is located completely outside active zone behind wall, the footing load does not need to be considered in the external stability calculations.

b. Distribution of Stress for External Stability Calculations.

AASHTO Figure 3.11.6.3-2-b
DD - Downdrag

Ground settlement adjacent to pile/ shaft

- $\alpha$, $\beta$ or $\lambda$ method
  - $DD = \alpha S_u$
  - $DD = \beta \sigma'_v$
  - $DD = \lambda (\sigma'_v + 2S_u)$
Loads and Basic Equations

Transient Loads

- LL
- PL
- IM
- BR
- CE
- CT
- CV
- EQ
- FR
- IC
- LS
- WA
- WS
- CR
- SE
- SH
- TG
- TU
- WL
LL – Vehicular Live Load

- HL-93
- Design truck + lane
- Design tandem + lane
- Multiple presence factors

AASHTO 3.6.1
Live Loads

Design Tandem

- Two 25 kip loads 4' - 0" apart
- 6' - 0" total separation

AASHTO 3.6.1.2.3
Live Loads

AASHTO Standard Spec vs LRFD Spec:

Old Std Spec Loading:
- HS20 Truck, or
- Alternate Military, or
- Lane Load

New LRFD Loading:
- HL-93 Truck and Lane Load, or
- Tandem and Lane Load, or
- 90% of 2 Trucks and Lane Load
LS – Live load surcharge

- Live load within \( \frac{1}{2} \) wall height behind wall

\[
\Delta p = k_s \gamma_s h_{eq} \quad k_s \text{ taken as } k_o \text{ or } k_a
\]

= constant horizontal earth pressure due to live load

- \( h_{eq} \) a function of wall height

Table 3.11.6.4-1 Equivalent Height of Soil for Vehicular Loading on Abutments Perpendicular to Traffic.

<table>
<thead>
<tr>
<th>Abutment Height (ft.)</th>
<th>( h_{eq} ) (ft.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.0</td>
<td>4.0</td>
</tr>
<tr>
<td>10.0</td>
<td>3.0</td>
</tr>
<tr>
<td>≥20.0</td>
<td>2.0</td>
</tr>
</tbody>
</table>

AASHTO 3.11.6.4

- Assume unit weight = 120 pcf (AASHTO Table 3.5.1-1)
Equivalent Height of Soil for Vehicular Loading on Retaining Walls Parallel to Traffic.

<table>
<thead>
<tr>
<th>Retaining Wall Height (ft.)</th>
<th>$h_{eq}$ (ft.) Distance from wall backface to edge of traffic</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.0 ft.</td>
</tr>
<tr>
<td>5.0</td>
<td>5.0</td>
</tr>
<tr>
<td>10.0</td>
<td>3.5</td>
</tr>
<tr>
<td>$\geq 20.0$</td>
<td>2.0</td>
</tr>
</tbody>
</table>

AASHTO Table 3.11.6.4-2
Identify Loads & Load Factors at each Load Combination

- Applicable loads at each limit state
- Load modifiers, $\eta_i$
  - Ductility, redundancy, importance
- Load factors, $\gamma_i$
  - Permanent (max. & min)
  - Transient
Section 1000 – ODOT LRFD Bridge Design Specifications

- $\eta_D$ (Ductility) = 1.0 for all limit states
- $\eta_i$ (operational importance) = 1.0 with exceptions listed in Article 1.3.5
- $\eta_R = 1.05$ for non-redundant elements
- $\eta_R = 1.0$ for redundant elements

- **Non-redundant**
  - Single-column piers
  - Two-column piers
  - T-type piers with a stem height-to-width ratio of 3:1 or greater

- **Redundant**
  - Cap and column piers with three or more columns
Identify Loads & Load Factors at each Load Combination

\[ Q = \sum \eta_i \gamma_i Q_i \]  
(1.3.2.1-1)

\[ \eta \] - Load Modifier

\[ \eta_i = \eta_D \eta_R \eta_I \geq 0.95 \]  
(1.3.2.1-2)

\[ \eta_D \] – Ductility Factor

\[ \eta_R \] – Redundancy Factor

\[ \eta_I \] – Operational Importance

\[ \gamma \] - Load Factor

\[ Q \] - Load Effect
Limit States - Load Modifiers

- **Applicable only to the Strength Limit State**
  - \( \eta_D \) – Ductility Factor:
    - \( \eta_D = 1.05 \) for nonductile members
    - \( \eta_D = 1.00 \) for conventional designs and details complying with specifications
    - \( \eta_D = 0.95 \) for components for which additional ductility measures have been taken
  - \( \eta_R \) – Redundancy Factor:
    - \( \eta_R = 1.05 \) for nonredundant members
    - \( \eta_R = 1.00 \) for conventional levels of redundancy
    - \( \eta_R = 0.95 \) for exceptional levels of redundancy
  - \( \eta_I \) – Operational Importance:
    - \( \eta_I = 1.05 \) for important bridges
    - \( \eta_I = 1.00 \) for typical bridges
    - \( \eta_I = 0.95 \) for relatively less important bridges
Load Factors and Combinations

ODOT Recommended Load Modifiers

- **For the Strength Limit States**
  - $\eta_D$ – Ductility Factor:
    - Use a ductility load modifier of $\eta_D = 1.00$ for all strength limit states
  - $\eta_R$ – Redundancy Factor:
    - Use $\eta_R = 1.05$ for “non-redundant” members
    - Use $\eta_R = 1.00$ for “redundant” members
    - Bridges with 3 or fewer girders should be considered “non-redundant.”
    - Bridges with 4 girders with a spacing of 12’ or more should be considered “non-redundant.”
    - Bridges with 4 girders with a spacing of less than 12’ should be considered “redundant.”
    - Bridge with 5 or more girders should be considered “redundant.”
Load Factors and Combinations

ODOT Recommended Load Modifiers

- For the Strength Limit States
  - Redundancy Factor: \( \eta_R \)
    - Use \( \eta_R = 1.05 \) for “non-redundant” members
    - Use \( \eta_R = 1.00 \) for “redundant” members
    - Single and two column piers should be considered non-redundant.
    - Cap and column piers with three or more columns should be considered redundant.
    - T-type piers with a stem height to width ratio of 3-1 or greater should be considered non-redundant.
    - For information on other substructure types, refer to NCHRP Report 458 *Redundancy in Highway Bridge Substructures*.
    - \( \eta_R \) does NOT apply to foundations. Foundation redundancy is included in the resistance factor.
Non-Redundant Substructures

Single Column Piers
Non-Redundant Substructures

Two Column Piers
Non-Redundant Substructures

T-type Piers
Stem height-to-width = 3-to-1 or greater

Example:
Height = 51.0 ft
Width = 16.0 ft
Ratio = 3.19
NON-REDUNDANT
Redundancy

- For additional information:
- NCHRP Report 458, *Redundancy in Highway Bridge Substructures*
- NCHRP Report 406, *Redundancy in Highway Bridge Superstructures*
Foundation Redundancy

- $\eta_R = 1.0$ for all foundations
- Non-redundant foundations $\rightarrow$ Reduce Resistance Factor ($\varphi$) by 20%
- Non-redundant Pile Foundations:
  - $\leq 4$ piles per substructure
- Non-redundant Drilled Shaft Foundations:
  - $1$ or $2$ shafts per substructure
Load Factors and Combinations

ODOT Recommended Load Modifiers

- **For the Strength Limit States**
  - $\eta_I$ – Operational Importance:
    - In General, use $\eta_I = 1.00$ unless one of the following applies
    - Use $\eta_I = 1.05$ if any of the following apply
      - Design ADT $\geq 60,000$
      - Detour length $\geq 50$ miles
      - Any span length $\geq 500'$
    - Use $\eta_I = 0.95$ if both of the following apply
      - Design ADT $\leq 400$
      - Detour length $\leq 10$ miles

Detour length applies to the shortest, emergency detour route.
## Loads and Load Factors

### Load Factors and Load Combinations

Table 3.4.1-1 Load Combinations and Load Factors

| Load Combination | DC | DD | DW | EH | EV | ES | EL | WA | WS | WL | FR | TU | CR | SH | TG | SE | EQ | IC | CT | CV |
|------------------|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|
| STRENGTH I       | 1.75 | 1.00 | -- | -- | -- | 1.00 | -- | -- | -- | -- | 1.00 | 0.50/1.20 | -- | -- | -- | -- |
| (unless noted)    |     |     |    |    |    |     |    |    |    |    |     |     |     |     |     |    |    |    |
| STRENGTH II      | 1.35 | 1.00 | -- | -- | -- | 1.00 | -- | -- | -- | -- | 1.00 | 0.50/1.20 | -- | -- | -- | -- |
| STRENGTH III     | 1.00 | 1.40 | -- | -- | -- | 1.00 | -- | -- | -- | -- | 1.00 | 0.50/1.20 | -- | -- | -- | -- |
| STRENGTH IV      | 1.00 | 1.00 | -- | -- | -- | 1.00 | -- | -- | -- | -- | 1.00 | 0.50/1.20 | -- | -- | -- | -- |
| STRENGTH V       | 1.35 | 1.00 | 0.40 | 1.0 | 1.00 | 0.50/1.20 | -- | -- | -- | -- | 1.00 | 0.50/1.20 | -- | -- | -- | -- |
# Loads and Load Factors

Factors and Load Combinations

Table 3.4.1-1 Load Combinations and Load Factors (cont.)

| Load Combination          | DC | DD | DW | EH | EV | ES | EL | LL | IM | CE | BR | PL | LS | WA | WS | WL | FR | TU | CR | SH | TG | SE | EQ | IC | CT | CV |
|---------------------------|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|
| EXTREME EVENT I           | γ_p| γ_EQ| 1.00| -- | -- | 1.00| -- | -- | -- | 1.00| -- | -- | -- | 1.00| -- | -- | -- | 1.00| -- | -- | -- | -- | -- | -- |
| EXTREME EVENT II          | γ_p| 0.50| 1.00| -- | -- | 1.00| -- | -- | -- | -- | 1.00| 1.00| 1.00| 1.00| -- | -- | -- | -- | -- | -- | -- | -- | -- | -- |
| FATIGUE – LL, IM, & CE ONLY | -- | 0.75| -- | -- | -- | -- | -- | -- | -- | -- | -- | -- | -- | -- | -- | -- | -- | -- | -- | -- | -- | -- | -- | -- |

Use One of These at a Time
# Loads and Load Factors

## Load Factors and Load Combinations

### Table 3.4.1-1 Load Combinations and Load Factors (cont.)

| Load Combination | DC | DD | DW | EH | EV | ES | EL | WA | WS | WL | FR | TG | SE | TU | CR | SH | Use One of These at a Time |
|------------------|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|-----------------------------|
| SERVICE I        | 1.00 | 1.00 |    |     |    |    |    | 1.00 | 0.30 | 1.0 | 1.00 | 1.00/1.20 | γ<sub>TG</sub> | γ<sub>SE</sub> |    | -- | -- | -- | -- |
| SERVICE II       | 1.00 | 1.30 | 1.00 |    |    |    |    |     | -- | -- | 1.00 | 1.00/1.20 | -- | -- | -- | -- | -- |
| SERVICE III      | 1.00 | 0.80 | 1.00 |    |    |    |    |     | -- | -- | 1.00 | 1.00/1.20 | γ<sub>TG</sub> | γ<sub>SE</sub> |    | -- | -- | -- | -- |
| SERVICE IV       | 1.00 |    |    |    |    |    |    |     | 1.00 | 0.70 | -- | 1.00 | 1.00/1.20 | -- | 1.0 | -- | -- | -- | -- |
### Table 3.4.1-2 Load Factors for Permanent Loads, $\gamma_p$

<table>
<thead>
<tr>
<th>Type of Load, Foundation Type, and Method Used to Calculate Downdrag</th>
<th>Load Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Maximum</td>
</tr>
<tr>
<td>DC: Component and Attachments</td>
<td>1.25</td>
</tr>
<tr>
<td>DC: <strong>Strength IV only</strong></td>
<td>1.50</td>
</tr>
<tr>
<td>DD: Downdrag</td>
<td>1.4</td>
</tr>
<tr>
<td>Piles, $\alpha$ Tomlinson Method</td>
<td>1.05</td>
</tr>
<tr>
<td>Piles, $\lambda$ Method</td>
<td>1.25</td>
</tr>
<tr>
<td>Drilled Shafts, O’Neill and Reese (1999) Method</td>
<td></td>
</tr>
<tr>
<td>DW: Wearing Surfaces and Utilities</td>
<td>1.50</td>
</tr>
<tr>
<td>EH: Horizontal Earth Pressure</td>
<td>1.50</td>
</tr>
<tr>
<td>• Active</td>
<td>1.35</td>
</tr>
</tbody>
</table>
# Loads and Load Factors

## Load Factors and Load Combinations

<table>
<thead>
<tr>
<th>Type of Load, Foundation Type, and Method Used to Calculate Downdrag</th>
<th>Load Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Maximum</td>
</tr>
<tr>
<td>EL: Locked-in Erection Stresses</td>
<td>1.00</td>
</tr>
<tr>
<td>EV: Vertical Earth Pressure</td>
<td>1.00</td>
</tr>
<tr>
<td>• Overall Stability</td>
<td>1.00</td>
</tr>
<tr>
<td>• Retaining Walls and Abutments</td>
<td>1.35</td>
</tr>
<tr>
<td>• Rigid Buried Structure</td>
<td>1.30</td>
</tr>
<tr>
<td>• Rigid Frames</td>
<td>1.35</td>
</tr>
<tr>
<td>• Flexible Buried Structures other than Metal Box Culverts</td>
<td>1.95</td>
</tr>
<tr>
<td>• Flexible Metal Box Culverts</td>
<td>1.50</td>
</tr>
<tr>
<td>ES: Earth Surcharge</td>
<td>1.50</td>
</tr>
</tbody>
</table>
Load Combinations

- **Strength I**
  - Normal vehicular use
  - Basic load combination
  - No wind

- **Strength II**
  - Owner-specified or permit design vehicles
  - No wind
Load Combinations

- **Strength III**
  - Wind load combination (> 55 mph)
  - No live load

- **Strength IV**
  - High dead to live load ratios (> 7.0)
  - Controls for “long span bridges”
  - No live load

*AASHTO 3.4.1*
Load Combinations

- **Strength V**
  - Normal vehicular use
  - Wind velocity of 55 mph

- **Extreme Event I**
  - Earthquake loading combination
Load Combinations

➢ Extreme Event II
  - Ice loading combination
  - Vehicle/vessel collision load combination
  - Certain hydraulic events

➢ Service I
  - Normal operational use of bridge
  - 55 mph wind
Load Combinations

➢ Service II
  ● Control yielding of steel structures and slip in slip-critical connections due to live load

➢ Service III
  ● For tension in prestressed concrete superstructures
  ● For crack control
Load Combinations

- **Service IV**
  - For tension in prestressed concrete columns (substructures)
  - For crack control

- **Fatigue**
  - Repetitive gravitational live load
  - Dynamic load allowance included by a single design truck
Loads Factors for Permanent Loads

- Selected to produce max./min. total extreme force effects
- For maximum force effects, loads that reduce maximum force effects should be factored by minimum load factor

AASHTO 3.4.1
Load Factors for Sliding and Eccentricity

Load Factors for Bearing Resistance

$β$ + $δ$
Typical Application of Live Load Surcharge

CONVENTIONAL STRUCTURE
Learning Outcomes

A. Identify the loads applied to a substructure and present the basic equations for computing load combinations.

B. Identify the loads & load factors that make up the load combinations for each limit state.
Learning Outcomes

C. Identify the applicable LRFD load combination and load factors that will produce maximum and minimum force effects on substructures.
Session II

Fundamentals of LRFD

Topic 3

Serviceability Performance Limits
OBJECTIVES

A. Identify horizontal displacement limits for substructures
B. Identify vertical displacement limits for substructures
C. Identify stability requirements for substructures
Introduction

Service Limit State Considerations

1. Horizontal displacement
2. Vertical displacement
3. Overall stability
Identify Horizontal Displacement Limits

Horizontal Displacements

- Lateral loads (shear)
- Applied moments
Lateral Earth Pressures on Abutments

Estimated displacement depends on:

- Wall height
- Type and compaction of backfill
- Degree wall may be restrained from movement
Rotation Movements

Evaluated at:

- Top of substructure unit (in plan)
- Elevation of deck
Rotation of Abutment

Sand and Gravel

Soft Clay

Dense Gravel

Settlement in Clay
Horizontal Movement

Resulting settlement

- Lateral movement
- Rotation of abutment due to “lateral squeeze”
Lateral “Squeeze” of Soft Subsoil

Abutment Movement

Settlement

Thrust

Abutment Rotates Toward Fill
Identify Vertical Displacement & Limits

Foundation Displacements

- Vertical Displacements
  - Downward – settlement
  - Upward – uplift or pullout

- Caused by:
  - Total settlement
  - Differential settlement
Differential Settlement

- May be more important than total settlement
- Typical values
  - Continuous span – 1” or less
  - Simple span – 1.5” to 2”
"Angular distortion" is defined as the ratio of the differential settlement (δ') between foundation elements to the span length (l), or δ'/l, between elements.
Differential Settlement
Differential Movement

Single Pier or Bent

- Tolerable settlement between adjacent columns within a pier are significantly less than between adjacent piers

- Due to:
  - Rigidity
  - Short length

AASHTO 3.11.1
Differential Settlement of Abutment

- Crack
- Wingwall Rotation
- Sand and Gravel
- Soft Clay
- Dense Gravel
- Settlement in Clay
Maximum Settlements

- Structural and geotechnical engineers should establish:
  - Total permissible settlement
  - Differential settlement
- Basis of Service Limit State Resistances
Identify Overall Stability Performance Limits

- Evaluate at Service I Limit State
- Unfactored loads
- Conventional limiting equilibrium methods
Overall Stability
Performance Limits

- Displacements not computed or estimated
- Displacement criteria not required
- Establish required resistance factors
Learning Outcome

A. Identify horizontal displacement limits for substructures
B. Identify vertical displacement limits for substructures
C. Identify stability requirements for substructures
Session III

Overview of Soil and Rock Materials
OBJECTIVES

A. Use the Mohr-Coulomb equation to determine the shear strength of soils.

B. Explain the difference between drained and undrained strength.

C. Know what field or laboratory test should be performed to obtain the required soil or rock properties.
OBJECTIVES

D. Describe the difference between the intact properties of rock and the rock mass properties.

E. Degradable Shales: additional testing needed

F. Intermediate geomaterials
Soil Characteristics

- Composed of individual grains of rock
- Relatively low strength
- Coarse grained (+ #200)
  - High permeability
- Fine grained (- #200)
  - Low permeability
  - Time dependant effects
Rock Characteristics

- **Strength**
  - Intermediate geomaterials,
    \[ q_u = 50-1500 \text{ psi} \]
  - Rock,
    \[ q_u > 1500 \text{ psi} \]

- **Rock mass properties**
Use the Mohr-Coulomb Equation to Determine Shear Strength
Soil Shear Strength, $\tau'$

$$\tau' = c' + \sigma_n' \tan \phi_f'$$
• Shear Strength = Max. Shear Stress that a Soil Can Withstand

> Shear Strength of soil is controlled by effective stresses, whether failure occurs under drained or undrained conditions
• Drained Shear Strength

- The strength corresponding to failure with no change in effective stress on the failure plane.

- \((\Delta u = 0, \Delta v \neq 0)\)

- CD Test Or CU Test with Pore Pressure measurement.
• **Undrained Shear Strength**

- The strength corresponding to failure with no change in water content.
- \((\Delta v = 0, \Delta u \neq 0)\)

- UU, CU Tests.
Effective Stress Shear Strength envelope
(b) Clay Mohr-Coulomb Strength Criterion
Effective Stress Shear Strength envelope
(b) Sand, Gravel and rock fill
Saturated Clays

\[ S_u = C, \quad \phi = \phi_u = 0 \]
Field Tests to Obtain the Required Soil or Rock Properties
Undrained Strength of Cohesive Soils, $s_u$

Unconfined Compression

$$s_u = q_u/2$$

Vane Shear Test

Typical Values

$$s_u = 250 - 4000 \text{ psf}$$
Drained Strength of Cohesive Soils, $c'$ and $f'_f$

Triaxial Compression CU Test

Typical Values

$c' = 100 - 500$ psf

$\phi'_f = 20^\circ - 35^\circ$
Drained Strength of Cohesionless Soils, $f'_f$

Friction angle is correlated to SPT results.

Standard Penetration Test (SPT)

Typical Values

$\phi'_f = 25^\circ - 45^\circ$
\[ N_1 = C_N \cdot N \]

\[ N_1 = \text{SPT blow count corrected for overburden Pressure, } \sigma'_v \]

\[ C_N = 0.77 \log_{10} \left( \frac{40}{\sigma'_v} \right) \]

\[ \sigma'_v = \text{Vertical Effective Stress (ksf)} \]

\[ N = \text{Uncorrected SPT blow count (blows/ft)} \]
\[ N_{60} = \left( \frac{ER}{60\%} \right) N \]

\[ N_{60} = \text{SPT blow count corrected for hammer efficiency} \]

\[ ER = \text{Hammer Efficiency} \]

\[ = 60\% \text{ conventional drop hammer using rope and cathead} \]

\[ = 80\% \text{ automatic trip hammer} \]

\[ Nl_{60} = C_N N_{60} \]

- ODOT is Requiring SPT Energy Measurement (Effective January 07)
Correlation of SPT $N1_{60}$ values to drained friction angle of granular soils (modified after Bowles, 1977)

<table>
<thead>
<tr>
<th>$N1_{60}$</th>
<th>$\phi_f$</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt;4</td>
<td>25 - 30</td>
</tr>
<tr>
<td>4</td>
<td>27 - 32</td>
</tr>
<tr>
<td>10</td>
<td>30 - 35</td>
</tr>
<tr>
<td>30</td>
<td>35 - 40</td>
</tr>
<tr>
<td>50</td>
<td>38 - 43</td>
</tr>
</tbody>
</table>

AASHTO Table 10.4.6.2.4-1
Soil Deformation

- Initial elastic settlement (all soils)
- Primary consolidation
- Secondary consolidation
- Fine-grained (cohesive) soils
Consolidation Test

Oedometer
Consolidation Properties

Void Ratio (e)

$\sigma_p' = \text{Preconsolidation Stress}$

$C_r$

$C_s$

$C_c$

Log$_{10} \sigma_v'$
One log cycle

\[ \Delta e = C_\alpha = 0.06 \]

<table>
<thead>
<tr>
<th>Elapsed Time (min)</th>
<th>Void ratio (e)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.1</td>
<td>2.65</td>
</tr>
<tr>
<td>1</td>
<td>2.62</td>
</tr>
<tr>
<td>10</td>
<td>2.55</td>
</tr>
<tr>
<td>100</td>
<td>2.52</td>
</tr>
<tr>
<td>1000</td>
<td>2.45</td>
</tr>
<tr>
<td>10000</td>
<td>2.42</td>
</tr>
</tbody>
</table>

Stress Range, 40 – 80 kPa
### Typical Consolidation Properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Typical Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$C_c$</td>
<td>0.1 to 1.0</td>
</tr>
<tr>
<td>$C_r$</td>
<td>10 % of $C_c$</td>
</tr>
<tr>
<td>$C_s$</td>
<td>Approximately $C_r$</td>
</tr>
<tr>
<td>$C_\alpha$</td>
<td>4% to 6% of $C_c$</td>
</tr>
<tr>
<td>$C_v$</td>
<td>0.01 to 1.0 ft$^2$/day</td>
</tr>
</tbody>
</table>
Elastic Properties of Soil

- **Young’s Modulus,** $E_s$
  - Typical values, 0.278 – 27.78 ksi

- **Poisson’s Ratio,** $\nu$
  - Typical values, 0.2 – 0.5

- **Shear Modulus,** $G$
  - Typical values, $E_s / [2 (1 + \nu)]$

- **Determination by correlation to** $N_{160}$ or $s_u$, or in-situ tests
# Elastic Constants of Various Soils

(Modified after U.S. Department of the Navy, 1982, and Bowles, 1988)

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Typical Range of Young’s Modulus Values, $E_s$ (ksi)</th>
<th>Poisson’s Ratio, $\nu$ (dim.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Soft sensitive</td>
<td>0.347–2.08</td>
<td>0.4–0.5 (undrained)</td>
</tr>
<tr>
<td>Medium stiff</td>
<td>2.08–6.94</td>
<td></td>
</tr>
<tr>
<td>Very stiff</td>
<td>6.94–13.89</td>
<td></td>
</tr>
<tr>
<td>Loess</td>
<td>2.08–8.33</td>
<td>0.1–0.3</td>
</tr>
<tr>
<td>Silt</td>
<td>0.278–2.78</td>
<td>0.3–0.35</td>
</tr>
<tr>
<td>Fine Sand:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Loose</td>
<td>1.11–1.67</td>
<td>0.25</td>
</tr>
<tr>
<td>Medium dense</td>
<td>1.67–2.78</td>
<td></td>
</tr>
<tr>
<td>Dense</td>
<td>2.78–4.17</td>
<td></td>
</tr>
<tr>
<td>Sand:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Loose</td>
<td>1.39–4.17</td>
<td>0.20–0.36</td>
</tr>
<tr>
<td>Medium dense</td>
<td>4.17–6.94</td>
<td></td>
</tr>
<tr>
<td>Dense</td>
<td>6.94–11.11</td>
<td>0.30–0.40</td>
</tr>
<tr>
<td>Gravel:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Loose</td>
<td>4.17–11.11</td>
<td>0.20–0.35</td>
</tr>
<tr>
<td>Medium dense</td>
<td>11.11–13.89</td>
<td></td>
</tr>
<tr>
<td>Dense</td>
<td>13.89–27.78</td>
<td>0.30–0.40</td>
</tr>
</tbody>
</table>
### Elastic Constants of Various Soils

(Modified after U.S. Department of the Navy, 1982, and Bowles, 1988)

#### Estimating $E_s$ from SPT $N$ Value

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>$E_s$ (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Silts, sandy silts, slightly cohesive mixes</td>
<td>$0.056 N_{160}$</td>
</tr>
<tr>
<td>Clean fine to medium sands and slightly silty sands</td>
<td>$0.097 N_{160}$</td>
</tr>
<tr>
<td>Coarse sands and sands with little gravel</td>
<td>$0.139 N_{160}$</td>
</tr>
<tr>
<td>Sandy gravel and gravels</td>
<td>$0.167 N_{160}$</td>
</tr>
</tbody>
</table>

#### Estimating $E_s$ from $q_c$ (static cone resistance)

| Sandy soils | $0.028 q_c$ |

(Continue)

AASHTO Table 10.4.6.3-1
Describe Differences Between Intact Rock Properties and Rock Mass Properties
Rock Properties

- Laboratory testing is for small intact rock specimens
- Rock mass is too large to be tested in lab or field
- Rock mass properties are obtained by correlating intact rock to large-scale rock mass behavior – failures in tunnels and mine slopes
- Requires geologic expertise
Intact Rock Strength

Point Load Test

Unconfined Compression, $q_u$

Typical Values

$q_u = 1500 - 50000$ psi
Rock Quality

Length, L

0.820 ft
- Sound

0.656 ft
- Not sound, highly weathered

0.820 ft
- Not sound, centerline pieces < 100 mm, highly weathered

0.623 ft
- Sound

0.197 ft
- Not sound

0.656 ft
- Sound

Core Run
Total = 4 ft

CR = 96%  RQD = 53%
Recovery and RQD Calculation

Core Recovery, \( CR = \frac{\text{Total length of rock recovered}}{\text{Total core run length}} \)

\[ CR = \frac{(250 + 200 + 250 + 190 + 60 + 80 + 120) \text{ mm}}{1,200 \text{ mm}} \]

\( CR = 96\% \)

\[
RQD = \frac{\sum \text{Length of sound pieces} > 100 \text{ mm}}{\text{Total core run length}} \]

\[ RQD = \frac{(250 + 190 + 200) \text{ mm}}{1,200 \text{ mm}} \times 100\% \]

\( RQD = 53\% \)
RMR Rock Mass Rating System

- Compressive strength of intact rock, $q_u$
- RQD
- Joint spacing
- Joint condition
- Ground water condition
# Geomechanics Classification of Rock Masses

<table>
<thead>
<tr>
<th>PARAMETER</th>
<th>RANGES OF VALUES</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strength of intact rock material</td>
<td></td>
</tr>
<tr>
<td>Point load strength index</td>
<td>&gt;175 ksf</td>
</tr>
<tr>
<td>85–175 ksf</td>
<td>85–175 ksf</td>
</tr>
<tr>
<td>45–85 ksf</td>
<td>45–85 ksf</td>
</tr>
<tr>
<td>20–45 ksf</td>
<td>20–45 ksf</td>
</tr>
<tr>
<td>For this low range, uniaxial compressive test is preferred</td>
<td>For this low range, uniaxial compressive test is preferred</td>
</tr>
<tr>
<td>Uniaxial compressive strength</td>
<td>&gt;4320 ksf</td>
</tr>
<tr>
<td>2160–4320 ksf</td>
<td>2160–4320 ksf</td>
</tr>
<tr>
<td>1080–2160 ksf</td>
<td>1080–2160 ksf</td>
</tr>
<tr>
<td>520–1080 ksf</td>
<td>520–1080 ksf</td>
</tr>
<tr>
<td>215–520 ksf</td>
<td>215–520 ksf</td>
</tr>
<tr>
<td>70–215 ksf</td>
<td>70–215 ksf</td>
</tr>
<tr>
<td>20–70 ksf</td>
<td>20–70 ksf</td>
</tr>
<tr>
<td>Relative Rating</td>
<td>15</td>
</tr>
<tr>
<td></td>
<td>12</td>
</tr>
<tr>
<td></td>
<td>7</td>
</tr>
<tr>
<td></td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>0</td>
</tr>
<tr>
<td>Drill core quality RQD</td>
<td>90% to 100%</td>
</tr>
<tr>
<td>Relative Rating</td>
<td>20</td>
</tr>
<tr>
<td></td>
<td>17</td>
</tr>
<tr>
<td></td>
<td>13</td>
</tr>
<tr>
<td></td>
<td>8</td>
</tr>
<tr>
<td></td>
<td>3</td>
</tr>
<tr>
<td>Spacing of joints</td>
<td>&gt;10 ft.</td>
</tr>
<tr>
<td></td>
<td>3–10 ft.</td>
</tr>
<tr>
<td></td>
<td>1–3 ft.</td>
</tr>
<tr>
<td></td>
<td>2 in.–1 ft.</td>
</tr>
<tr>
<td></td>
<td>&lt;2 in.</td>
</tr>
<tr>
<td>Relative Rating</td>
<td>30</td>
</tr>
<tr>
<td></td>
<td>25</td>
</tr>
<tr>
<td></td>
<td>20</td>
</tr>
<tr>
<td></td>
<td>10</td>
</tr>
<tr>
<td></td>
<td>5</td>
</tr>
<tr>
<td>Condition of joints</td>
<td>• Very rough surfaces</td>
</tr>
<tr>
<td></td>
<td>• Not continuous</td>
</tr>
<tr>
<td></td>
<td>• No separation</td>
</tr>
<tr>
<td></td>
<td>• Hard joint wall rock</td>
</tr>
<tr>
<td></td>
<td>• Slightly rough surfaces</td>
</tr>
<tr>
<td></td>
<td>• Separation &lt;0.05 in.</td>
</tr>
<tr>
<td></td>
<td>• Hard joint wall rock</td>
</tr>
<tr>
<td></td>
<td>• Slightly rough surfaces</td>
</tr>
<tr>
<td></td>
<td>• Separation &lt;0.05 in.</td>
</tr>
<tr>
<td></td>
<td>• Soft joint wall rock</td>
</tr>
<tr>
<td></td>
<td>• Slicken-sided surfaces or</td>
</tr>
<tr>
<td></td>
<td>• Gouge &lt;0.2 in. thick or</td>
</tr>
<tr>
<td></td>
<td>• Joints open &gt;0.2 in.</td>
</tr>
<tr>
<td></td>
<td>• Continuous joints</td>
</tr>
<tr>
<td>Relative Rating</td>
<td>25</td>
</tr>
<tr>
<td></td>
<td>20</td>
</tr>
<tr>
<td></td>
<td>12</td>
</tr>
<tr>
<td></td>
<td>6</td>
</tr>
<tr>
<td></td>
<td>0</td>
</tr>
</tbody>
</table>

AASHTO Table 10.4.6.4-1
### Geomechanics Classification of Rock Masses (Continue)

<table>
<thead>
<tr>
<th></th>
<th>Ground water conditions (use one of the three evaluation criteria as appropriate to the method of exploration)</th>
<th>Inflow per 30 ft. tunnel length</th>
<th>None</th>
<th>&lt;400 gal./hr.</th>
<th>400–2000 gal./hr.</th>
<th>&gt;2000 gal./hr.</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>Ratio = joint water pressure/major principal stress</td>
<td>0</td>
<td>0.0–0.2</td>
<td>0.2–0.5</td>
<td>&gt;0.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>General Conditions</td>
<td>Completely Dry</td>
<td>Moist only (interstitial water)</td>
<td>Water under moderate pressure</td>
<td>Severe water problems</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Relative Rating</td>
<td>10</td>
<td>7</td>
<td>4</td>
<td>0</td>
<td></td>
</tr>
</tbody>
</table>

AASHTO Table 10.4.6.4-1
### Geomechanics Rating Adjustment for Jointed Orientations

AASHTO Table 10.4.6.4-2

<table>
<thead>
<tr>
<th>Strike and Dip Orientations of Joints</th>
<th>Very Favorable</th>
<th>Favorable</th>
<th>Fair</th>
<th>Unfavorable</th>
<th>Very Unfavorable</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tunnels</td>
<td>0</td>
<td>-2</td>
<td>-5</td>
<td>-10</td>
<td>-12</td>
</tr>
<tr>
<td>Foundations</td>
<td>0</td>
<td>-2</td>
<td>-7</td>
<td>-15</td>
<td>-25</td>
</tr>
<tr>
<td>Slopes</td>
<td>0</td>
<td>-5</td>
<td>-25</td>
<td>-50</td>
<td>-60</td>
</tr>
</tbody>
</table>

### Geomechanics Rock Mass Classes Determined From Total Ratings

AASHTO Table 10.4.6.4-3

<table>
<thead>
<tr>
<th>RMR Rating</th>
<th>100–81</th>
<th>80–61</th>
<th>60–41</th>
<th>40–21</th>
<th>&lt;20</th>
</tr>
</thead>
<tbody>
<tr>
<td>Class No.</td>
<td>I</td>
<td>II</td>
<td>III</td>
<td>IV</td>
<td>V</td>
</tr>
<tr>
<td>Description</td>
<td>Very good rock</td>
<td>Good rock</td>
<td>Fair rock</td>
<td>Poor rock</td>
<td>Very poor rock</td>
</tr>
</tbody>
</table>
• Rock Mass Strength

\[
\phi'_{i} = \tan^{-1}(4 h \cos^{2}[30+0.33\sin^{-1}(h^{-3/2})]-1)^{-1/2}
\]
\[
\tau = (\cot \phi'_{i} - \cos \phi'_{i})mq_{u}/8
\]
\[
h = 1 + 16(m\sigma'_{n}+sq_{u})/(3m^{2}q_{u})
\]

Values of the parameters m and s are determined based on empirical correlation to rock type and RMR.
# Approximate Relationship Between Rock-Mass Quality and Material Constants Used in Defining Nonlinear Strength (Hoek and Brown, 1988)

<table>
<thead>
<tr>
<th>Rock Quality</th>
<th>Constants</th>
<th>Rock Type</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A =</td>
<td>Carbonate rocks with well developed crystal cleavage—dolomite, limestone and marble</td>
</tr>
<tr>
<td></td>
<td>B =</td>
<td>Lithified argillaceous rocks—mudstone, siltstone, shale and slate (normal to cleavage)</td>
</tr>
<tr>
<td></td>
<td>C =</td>
<td>Arenaceous rocks with strong crystals and poorly developed crystal cleavage—sandstone and quartzite</td>
</tr>
<tr>
<td></td>
<td>D =</td>
<td>Fine grained polyminerallic igneous crystalline rocks—andesite, dolerite, diabase and rhyolite</td>
</tr>
<tr>
<td></td>
<td>E =</td>
<td>Coarse grained polyminerallic igneous &amp; metamorphic crystalline rocks—amphibolite, gabbro gneiss, granite, norite, quartz-diorite</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th></th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>E</th>
</tr>
</thead>
<tbody>
<tr>
<td>INTACT ROCK SAMPLES</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Laboratory size specimens free from discontinuities</td>
<td>m</td>
<td>7.00</td>
<td>10.00</td>
<td>15.00</td>
<td>17.00</td>
</tr>
<tr>
<td>CSIR rating: $RMR = 100$</td>
<td>s</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th></th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>E</th>
</tr>
</thead>
<tbody>
<tr>
<td>VERY GOOD QUALITY ROCK MASS</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tightly interlocking undisturbed rock with unweathered joints at 3–10 ft.</td>
<td>m</td>
<td>2.40</td>
<td>3.43</td>
<td>5.14</td>
<td>5.82</td>
</tr>
<tr>
<td>CSIR rating: $RMR = 85$</td>
<td>s</td>
<td>0.082</td>
<td>0.082</td>
<td>0.082</td>
<td>0.082</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>GOOD QUALITY ROCK MASS</th>
<th>m</th>
<th>0.575</th>
<th>0.821</th>
<th>1.231</th>
<th>1.395</th>
<th>2.052</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>s</td>
<td>0.00293</td>
<td>0.00293</td>
<td>0.00293</td>
<td>0.00293</td>
<td>0.00293</td>
</tr>
<tr>
<td>FAIR QUALITY ROCK MASS</td>
<td>m</td>
<td>0.128</td>
<td>0.183</td>
<td>0.275</td>
<td>0.311</td>
<td>0.458</td>
</tr>
<tr>
<td></td>
<td>s</td>
<td>0.00009</td>
<td>0.00009</td>
<td>0.00009</td>
<td>0.00009</td>
<td>0.00009</td>
</tr>
<tr>
<td>POOR QUALITY ROCK MASS</td>
<td>m</td>
<td>0.029</td>
<td>0.041</td>
<td>0.061</td>
<td>0.069</td>
<td>0.102</td>
</tr>
<tr>
<td></td>
<td>s</td>
<td>3 x 10^{-6}</td>
<td>3 x 10^{-6}</td>
<td>3 x 10^{-6}</td>
<td>3 x 10^{-6}</td>
<td>3 x 10^{-6}</td>
</tr>
<tr>
<td>VERY POOR QUALITY ROCK MASS</td>
<td>m</td>
<td>0.007</td>
<td>0.010</td>
<td>0.015</td>
<td>0.017</td>
<td>0.025</td>
</tr>
<tr>
<td></td>
<td>s</td>
<td>1 x 10^{-7}</td>
<td>1 x 10^{-7}</td>
<td>1 x 10^{-7}</td>
<td>1 x 10^{-7}</td>
<td>1 x 10^{-7}</td>
</tr>
</tbody>
</table>

CSIR rating: $RMR = 65$

CSIR rating: $RMR = 44$

CSIR rating: $RMR = 23$

CSIR rating: $RMR = 3$
### Typical Elastic Constants for Intact Rock

#### Summary of Elastic Moduli for Intact Rock (Modified after Kulhawy, 1978)

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>No. of Values</th>
<th>No. of Rock Types</th>
<th>Elastic Modulus, $E_i$ (ksi $\times 10^3$)</th>
<th>Standard Deviation (ksi $\times 10^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Maximum</td>
<td>Minimum</td>
</tr>
<tr>
<td>Granite</td>
<td>26</td>
<td>26</td>
<td>14.5</td>
<td>0.93</td>
</tr>
<tr>
<td>Diorite</td>
<td>3</td>
<td>3</td>
<td>16.2</td>
<td>2.48</td>
</tr>
<tr>
<td>Gabbro</td>
<td>3</td>
<td>3</td>
<td>12.2</td>
<td>9.8</td>
</tr>
<tr>
<td>Diabase</td>
<td>7</td>
<td>7</td>
<td>15.1</td>
<td>10.0</td>
</tr>
<tr>
<td>Basalt</td>
<td>12</td>
<td>12</td>
<td>12.2</td>
<td>4.20</td>
</tr>
<tr>
<td>Quartzite</td>
<td>7</td>
<td>7</td>
<td>12.8</td>
<td>5.29</td>
</tr>
<tr>
<td>Marble</td>
<td>14</td>
<td>13</td>
<td>10.7</td>
<td>0.58</td>
</tr>
<tr>
<td>Gneiss</td>
<td>13</td>
<td>13</td>
<td>11.9</td>
<td>4.13</td>
</tr>
<tr>
<td>Slate</td>
<td>11</td>
<td>2</td>
<td>3.79</td>
<td>0.35</td>
</tr>
<tr>
<td>Schist</td>
<td>13</td>
<td>12</td>
<td>10.0</td>
<td>0.86</td>
</tr>
<tr>
<td>Phyllite</td>
<td>3</td>
<td>3</td>
<td>2.51</td>
<td>1.25</td>
</tr>
<tr>
<td>Sandstone</td>
<td>27</td>
<td>19</td>
<td>5.68</td>
<td>0.09</td>
</tr>
<tr>
<td>Siltstone</td>
<td>5</td>
<td>5</td>
<td>4.76</td>
<td>0.38</td>
</tr>
<tr>
<td>Shale</td>
<td>30</td>
<td>14</td>
<td>5.60</td>
<td>0.001</td>
</tr>
<tr>
<td>Limestone</td>
<td>30</td>
<td>30</td>
<td>13.0</td>
<td>0.65</td>
</tr>
<tr>
<td>Dolostone</td>
<td>17</td>
<td>16</td>
<td>11.4</td>
<td>0.83</td>
</tr>
</tbody>
</table>
### Summary of Poisson’s Ratio for Intact Rock (Modified after Kulhawy, 1978)

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>No. of Values</th>
<th>No. of Rock Types</th>
<th>Poisson’s Ratio, $\nu$</th>
<th>Standard Deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Minimum</td>
<td>Maximum</td>
<td>Mean</td>
<td></td>
</tr>
<tr>
<td>Granite</td>
<td>22</td>
<td>22</td>
<td>0.39</td>
<td>0.20</td>
</tr>
<tr>
<td>Gabbro</td>
<td>3</td>
<td>3</td>
<td>0.20</td>
<td>0.18</td>
</tr>
<tr>
<td>Diabase</td>
<td>6</td>
<td>6</td>
<td>0.38</td>
<td>0.29</td>
</tr>
<tr>
<td>Basalt</td>
<td>11</td>
<td>11</td>
<td>0.32</td>
<td>0.23</td>
</tr>
<tr>
<td>Quartzite</td>
<td>6</td>
<td>6</td>
<td>0.22</td>
<td>0.14</td>
</tr>
<tr>
<td>Marble</td>
<td>5</td>
<td>5</td>
<td>0.40</td>
<td>0.28</td>
</tr>
<tr>
<td>Gneiss</td>
<td>11</td>
<td>11</td>
<td>0.40</td>
<td>0.22</td>
</tr>
<tr>
<td>Schist</td>
<td>12</td>
<td>11</td>
<td>0.31</td>
<td>0.12</td>
</tr>
<tr>
<td>Sandstone</td>
<td>12</td>
<td>9</td>
<td>0.46</td>
<td>0.20</td>
</tr>
<tr>
<td>Siltstone</td>
<td>3</td>
<td>3</td>
<td>0.23</td>
<td>0.18</td>
</tr>
<tr>
<td>Shale</td>
<td>3</td>
<td>3</td>
<td>0.18</td>
<td>0.09</td>
</tr>
<tr>
<td>Limestone</td>
<td>19</td>
<td>19</td>
<td>0.33</td>
<td>0.23</td>
</tr>
<tr>
<td>Dolostone</td>
<td>5</td>
<td>5</td>
<td>0.35</td>
<td>0.29</td>
</tr>
</tbody>
</table>

AASHTO Table C10.4.6.5-2
Rock Mass Deformation

- Typical values of $E_M$ range from 1,000 to 10,000 ksi

\[ E_M = 145,000 \times 10^{\left(\frac{RMR-10}{40}\right)} \]

\[ E_\alpha = 2 \times RMR - 100 \]
Rock Mass Deformation

\[ E_m = \left( \frac{E_m}{E_i} \right) E_i \]

Estimation of \( E_m \) based on RQD (after O’Neill and Reese, 1999)

<table>
<thead>
<tr>
<th>RQD (percent)</th>
<th>( E_m/E_i )</th>
<th>Closed Joints</th>
<th>Open Joints</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>1.00</td>
<td>0.60</td>
<td></td>
</tr>
<tr>
<td>70</td>
<td>0.70</td>
<td>0.10</td>
<td></td>
</tr>
<tr>
<td>50</td>
<td>0.15</td>
<td>0.10</td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>0.05</td>
<td>0.05</td>
<td></td>
</tr>
</tbody>
</table>

AASHTO Table 10.4.6.5-1
• Erodibility of Rock

- AASHTO 10.4.6.6
- Rock’s susceptibility to erosion should be considered.
  - Cementing agent.
  - Mineralogy.
  - Jointed spacing.
  - Weathering.
• Identifying Rock using Geological Origin

- Igneous Rock – Granite, Diorite, Basalt.

- Sedimentary Rock – Sandstone, Limestone and Shale.

- Metamorphic Rock – Quartzite, Schist, Gneiss.
IGM (Intermediate Geomaterials)

AASHTO 10.8.2.2.3

- For Drilled Shafts in IGMs

- Cohesive IGM – clay shales or mudstones
  with $S_u = 5 \sim 50$ ksf

- Cohesionless IGM – granular tills or granular residual soils
  with $N_{160} > 50$ blow/ft
Learning Outcomes

A. Use the Mohr-Coulomb equation to determine the shear strength of soils.
B. Explain the difference between drained and undrained strength.
C. Know what field or laboratory test should be performed to obtain the required soil or rock properties.
Learning Outcomes

D. Describe the difference between the intact properties of rock and the rock mass properties.
E. degradable Shales: additional testing needed
F. Intermediate geomaterials
Session IV

LRFD Design of Shallow Foundations
Shallow Foundation Design Considerations

1. Overall (global) stability
2. Bearing failure
3. Deformation (settlement)
4. Sliding
5. Overturning
6. Uplift
7. Scour (loss of bearing or stability)
8. Frost
9. External or surcharge loads
10. Effect of ground water
OBJECTIVES

A. Compute nominal geotechnical resistances for spread footings
B. Describe the differences between LRFD and ASD of spread footings
Nominal Geotechnical Resistances

- ASD Failure Modes
- LRFD Limit States
Determine Nominal Geotechnical Bearing Resistance at the Service Limit State

Size Footing at the Service Limit State

Determine Nominal Bearing & Sliding Resistance at the Strength & Extreme Limit States

Check Footing at Strength Limit State for 1. Bearing  2. Sliding  3. Eccentricity


Perform Structural Design Based on Factored Loads and Factored Resistances
Service Limit State

Global Stability

Location A

Stabilize

Location B

Destabilize
Global Stability Factor of Safety – Method of Slices

\[ N \tan \phi \]

\[ W_T \]

\[ N \alpha \]

\[ T \]

\[ \theta \]

\[ c \delta \]

\[ W_T \]

\[ \alpha \]

\[ T \]
Resistance Factors

**ASD Factors of Safety**

<table>
<thead>
<tr>
<th>Soil/Rock Parameters and Ground Water Conditions Based On:</th>
<th>Slope Supports Abutment or Other Structure?</th>
</tr>
</thead>
<tbody>
<tr>
<td>In-situ or Laboratory Tests and Measurements</td>
<td>Yes</td>
</tr>
<tr>
<td></td>
<td>1.5</td>
</tr>
<tr>
<td>No Site-specific Tests</td>
<td>1.8</td>
</tr>
</tbody>
</table>

\[ LRFD \quad \phi = \frac{\gamma}{FS} \]
Stability Wrap-Up

- Unfactored loads
  - Service Limit State

- Applied stress must be limited
  - Footings supported in a slope
  - $\phi \leq 0.65$ (FS $\geq 1.5$)

- Stress criteria for stability can control footing design
Nominal Bearing Resistance at Service I Limit State

\[ \gamma = 1.0 \]
\[ \Phi = 1.0 \]

LRFD = ASD
 Settlement Types

- Elastic ($S_e$)
- Primary ($S_c$)
- Secondary ($S_s$)

$$S_t = S_e + S_c + S_s$$
Elastic Settlement

- Occurs instantaneously at load application
- Distortion of mass
- Bulk modulus (Elastic Modulus + Poison Ratio)
- Generally not elastic
- Typically estimated by elastic theory
- Cohesionless soils and rock
Loads That Contribute to Elastic Settlement

- Permanent
- Transient
Primary Settlement (Consolidation)

- Occurs as water is expelled from the pores
- Volume of voids decreases over time
- Important for fine-grain soils
Primary Settlement

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Volume Solids (%)</th>
<th>Volume Voids (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gravel</td>
<td>100</td>
<td>0</td>
</tr>
<tr>
<td>Sand</td>
<td>75</td>
<td>25</td>
</tr>
<tr>
<td>Silt</td>
<td>50</td>
<td>50</td>
</tr>
<tr>
<td>Clay</td>
<td>25</td>
<td>75</td>
</tr>
<tr>
<td>Muck</td>
<td>0</td>
<td>100</td>
</tr>
<tr>
<td>Peat</td>
<td>0</td>
<td>100</td>
</tr>
</tbody>
</table>

COMPRESSIBILITY

- LOW
- HIGH

Investigate
Consolidation Settlement (Spring Analogy)

Porous Piston

Soil

Piston

Spring

Valve

Closed

Spring compresses

Water pressure decreases

Spring resists

Water pressure relieved

Valve

Water

Pressure

Stress

In Water

In Spring

Time

Piston moves

Escaping

Consolidation Settlement (Spring Analogy)
Secondary Settlement (Creep)

- Continued strain occurs in the soil skeleton
- High moisture, highly plastic, and organic content soil most susceptible
- Negative effects on long term spread footing performance

\[ S_s = \frac{C^\alpha}{1 + e_0} H_c \log \left( \frac{t_2}{t_1} \right) \]

AASHTO Equ. 10.6.2.4.3-9
Settlement of Granular vs. Cohesive Soils

- Relative importance of settlement components for different soil types
- Structural effects of settlement components
## Settlement Considerations

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Elastic Settlement</th>
<th>Primary Settlement</th>
<th>Secondary Settlement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sands/Gravels</td>
<td>Yes</td>
<td>No</td>
<td>No</td>
</tr>
<tr>
<td>Clays</td>
<td>Possible</td>
<td>Yes</td>
<td>Possible</td>
</tr>
<tr>
<td>Organic</td>
<td>Possible</td>
<td>Possible</td>
<td>Yes</td>
</tr>
<tr>
<td>Rock</td>
<td>Yes</td>
<td>No</td>
<td>Possible</td>
</tr>
</tbody>
</table>
Stress Below Footing

Boussinesq Pressure Isobars

AASHTO Fig. 10.6.2.4.1-1
2-to-1 Distribution Method
Strip Footing

\[ \Delta \sigma_v = \frac{B_f}{(B_f + Z)} q \]

Exact Distribution

Approximate Distribution, \( \Delta \sigma_v \)
Rectangular Footing

\[ \Delta \sigma_v = \frac{P}{(B_f + z)(L_f + z)} \]
Layered Profile Settlement Analysis

- Max. Layer thickness ≈ 10 ft (C 10.6.2.4.2)
Calculating Change in Effective Vertical Stress, $\Delta\sigma_v$

$L >> B$

$q = \frac{10 \text{ k/ft}}{5'} = 2 \text{ ksf}$

$Z = 5'$

$\Delta\sigma_v = q \left( \frac{B_f}{B_f + z} \right) = 2 \left( \frac{5}{5+5} \right) = 1 \text{ ksf}$
Elastic Half-Space Method

\[ S_e = \frac{q_0 (1 - \nu^2) \sqrt{A'}}{144 E_s \beta_z} \]

\( A' = \) Effective area (ft\(^2\))
\( \beta_z = \) shape factor
\( E_s = \) Young’s modulus \( \left( \frac{1}{2} - \frac{2}{3} B \right) \)

AASHTO Equ. 10.6.2.4.2-1
# ELASTIC Shapes and Rigidity Factors, EPRI (1983)

<table>
<thead>
<tr>
<th>$L/B$</th>
<th>Flexible, $\beta_z$ (Average)</th>
<th>$\beta_z$ Rigid</th>
</tr>
</thead>
<tbody>
<tr>
<td>Circular</td>
<td>1.04</td>
<td>1.13</td>
</tr>
<tr>
<td>1</td>
<td>1.06</td>
<td>1.08</td>
</tr>
<tr>
<td>2</td>
<td>1.09</td>
<td>1.10</td>
</tr>
<tr>
<td>3</td>
<td>1.13</td>
<td>1.15</td>
</tr>
<tr>
<td>5</td>
<td>1.22</td>
<td>1.24</td>
</tr>
<tr>
<td>10</td>
<td>1.40</td>
<td>1.40</td>
</tr>
</tbody>
</table>

AASHTO Table 10.6.2.4.2-1
\[ \Delta H = H_c \frac{1}{C'} \log \left( \frac{\sigma'_{v_0} + \Delta \sigma_v}{\sigma'_{v_0}} \right) \]

- \( H_c \) = initial height of layer i
- \( C' \) = bearing capacity index

AASHTO Equ. 10.6.2.4.2-2
Hough Chart of Bearing Capacity Index, C’

- Based on soil and structure type, SPT blowcount

AASHTO Fig. 10.6.2.4.2-1
Settlement of Footing on Rock

\[ \rho = q_o \left( 1 - \nu^2 \right) \frac{rI_p}{144 E_m} \]

Circular (Square) footing

\[ I_p = \frac{\sqrt{\pi}}{\beta_z} \]

AASHTO Equ. 10.6.2.4.4-2

Rectangular footing

\[ I_p = \frac{(L/B)^{1/2}}{\beta_z} \]

AASHTO Equ. 10.6.2.4.4-4
Settlement Magnitude
Cohesive (clay) Soils

NC Soil, $\sigma'_p = \sigma'_o$

$$S_c = \left[ \frac{H_c}{1 + e_o} \right] \left[ C_c \log \left( \frac{\sigma'_f}{\sigma'_p} \right) \right]$$

AASHTO Equ. 10.6.2.4.3-2
Settlement Magnitude
Cohesive (clay) Soils

OC Soil, $\sigma'_p > \sigma'_o$

$$S_c = \left[ \frac{H_c}{1 + e_o} \right] \left[ C_r \log \frac{\sigma'_p}{\sigma'_o} + C_c \log \frac{\sigma'_f}{\sigma'_p} \right]$$

AASHTO Equ. 10.6.2.4.3-1
Three Dimensional Consolidation Effect

\[ B \gg H_c \]

\[ S_c(3-D) = \mu_c S_c(1-D) \]

AASHTO Eqe. 10.6.2.4.3-7
Effect of 3-D Consolidation Settlement

\[ S_c(3-D) = \mu_c S_c(1-D) \]

AASHTO Figure 10.6.2.4.3-3
Time for 1-D Consolidation Settlement

\[ t = \frac{TH_d^2}{c} \]  

AASHTO 10.6.2.4.3-8

AASHTO Fig. 10.6.2.4.3-4

Figure 10.6.2.4.3-4  Percentage of Consolidation as a Function of Time Factor, \( T \) (EPRI, 1983).

IV-35
Typical Consolidation Compression Curve for Overconsolidated Soil

AASHTO Figure 10.6.2.4.3-1
Effects of Sample Quality on Consolidation Test Results

AASHTO Figure C10.6.2.4.3-1
Typical Variation of Preconsolidation Stress with Depth

AASHTO Figure C10.6.2.4.3-2

Range of $\sigma'_p$ from laboratory oedometer tests

Soft sensitive gray silty clay; occasional shells and sand seams
PI $\sim$ 15
$w_n \sim$ 50% avg.

Silty sand
Nominal Bearing Resistance at Service Limit State

For a constant value of settlement

$q_n$ vs $B_f$
Eccentricity of Footings on Soil

\[ e_B = \frac{M_B}{P} \]
\[ e_L = \frac{M_L}{P} \]

AASHTO 10.6.1.3
Effective Dimensions for Footings on Soil

- \( B' = B - 2e_B \)
- \( L' = L - 2e_L \)

AASHTO EQU. 10.6.1.3-1
Applied Stress Beneath Effective Footing Area (Soil)

\[ q = \frac{P}{(B')(L')} \]

UNIFORM

AASHTO 10.6.1.4
Trapezoidal Distribution
Footings on Rock

\[ q_{\text{min}} = \sum \frac{P}{B_f} \left( 1 - 6 \frac{e}{B} \right) \]

\[ q_{\text{max}} = \sum \frac{P}{B_f} \left( 1 + 6 \frac{e}{B} \right) \]

AASHTO 10.6.1.4
Triangular Distribution
Footings on Rock

AASHTO 10.6.1.4
Limits on Eccentricity
(Strength Limit States)

\[ e_B < \frac{B}{4} \quad \text{Soil} \]
\[ e_B < \frac{3B}{8} \quad \text{Rock} \]
Use of Eccentricity and Effective Footing Dimensions

- **Service Limit State**
  - Nominal bearing resistance limited by Settlement

- **Strength Limit State**
  - Nominal bearing resistance limited by Bearing Resistance

- **Prevent Overturning (Eccentricity Limit)**
  - Strength Limit States
Bearing Resistance Failure
Bearing Resistance Mechanism

\[ q_n = cN_C + \gamma D_f N_q C_{Wq} + 0.5\gamma B_f N_C W_{Cy} \]

\[ \varepsilon = C + \sigma' \tan \phi \]

Soil Shear Strength

Ground Surface

\[ \sigma_v = \gamma D_f \]
Other Bearing Considerations

- Footing Shape
- Embedment Depth
- Load Inclination
- Location Near a Slope

\[ q_n = cN_c s_{c_i} + \gamma D_f N_{q} s_{q} d_{i} q_{i} C_{wq} + 0.5\gamma B_f N_{\gamma} s_{\gamma} i_{\gamma} C_{w\gamma} \]

AASHTO Equ. 10.6.3.1.2a
### Shape Correction Factor

<table>
<thead>
<tr>
<th>Factor</th>
<th>Friction Angle</th>
<th>Cohesion Term ((s_c))</th>
<th>Unit Weight Term ((s_γ))</th>
<th>Surcharge Term ((s_q))</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shape Factors, (s_c, s_γ, s_q)</td>
<td>(\phi = 0)</td>
<td>(1 + \left(\frac{B_f}{5L_f}\right))</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>(\phi &gt; 0)</td>
<td>(1 + \left(\frac{B_f}{L_f}\right)\left(\frac{N_q}{N_c}\right))</td>
<td>(1 - 0.4\left(\frac{B_f}{L_f}\right))</td>
<td>(1 + \left(\frac{B_f}{L_f}\tan\phi\right))</td>
</tr>
</tbody>
</table>

*Note: Shape (eccentricity) factors, \(s\), should not be applied simultaneously with inclined loading factors, \(i\). Bowles (1982) recommends using the inclination factors, \(i\), with the depth correction factors, \(d\), only.*
**Load Inclination Correction Factor**

\[
\begin{align*}
\text{Friction Angle} & \quad \text{Cohesion Term (} i_c) & \quad \text{Surcharge Term (} i_q) & \quad \text{Unit Weight Term (} i_\gamma) \\
\phi = 0 & \quad i_c = 1 - \left( \frac{nH}{B'_t L'_t c N_c} \right) & \quad 1.0 & \quad 1.0 \\
\phi > 0 & \quad i_c = i_q - \left[ \left( 1 - i_q \right) / N_q - 1 \right] & \quad i_q = \left[ 1 - \frac{H}{(V + B'_t L'_t c \cot \phi)} \right] & \quad i_\gamma = \left[ 1 - \frac{H}{(V + B'_t L'_t c \cot \phi)} \right]^{(z+1)}
\end{align*}
\]

**AASHTO Equ. 10.6.3.1.2a-5 to a-8**
AASHTO C.10.6.3.1.2a

In practice, omit the load inclination factors.
## Correction Factor for Location of Ground Water Table

<table>
<thead>
<tr>
<th>Depth of Ground Water Table, $D_W$</th>
<th>$C_{Wq}$</th>
<th>$C_{W\gamma}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.5</td>
<td>0.5</td>
</tr>
<tr>
<td>$D_f$</td>
<td>1.0</td>
<td>0.5</td>
</tr>
<tr>
<td>$&gt; 1.5B_f + D_f$</td>
<td>1.0</td>
<td>1.0</td>
</tr>
</tbody>
</table>
### Embedment Depth Correction Factor

<table>
<thead>
<tr>
<th>Friction Angle, $\phi$ (degrees)</th>
<th>$D_f/B_f$</th>
<th>$d_q$</th>
</tr>
</thead>
<tbody>
<tr>
<td>32</td>
<td>1</td>
<td>1.20</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>1.30</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>1.35</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>1.40</td>
</tr>
<tr>
<td>37</td>
<td>1</td>
<td>1.20</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>1.25</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>1.30</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>1.35</td>
</tr>
<tr>
<td>42</td>
<td>1</td>
<td>1.15</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>1.20</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>1.25</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>1.30</td>
</tr>
</tbody>
</table>

AASHTO Table 10.6.3.1.2a-4
Modified Bearing Capacity Factors for Footing on Sloping Ground

AASHTO Fig 10.6.3.1.2c-1&c-2

Cohesive soil (Φ=0)
Cohesionless soil (c = 0)
Modified Bearing Capacity Factors for Footing on Sloping Ground

AASHTO Fig 10.6.3.1.2c-1&c-2

Cohesive soil ($\Phi=0$)
Cohesionless soil (c =0)
AASHTO 10.6.3.1.3
Semi-Empirical Procedure for Nominal Bearing Resistance of Soils

**SPT**

\[
q_u = \frac{\overline{N}_{160} B}{5} \left[ C_{wq} \frac{D_f}{B} + C_{w\gamma} \right]
\]

\(\overline{N}_{160}\) (Bottom of footing to 1.5B)

**CPT**

\[
q_n = \frac{\overline{q}_c B}{40} \left( C_{wq} \frac{D_f}{B} + C_{w\gamma} \right)
\]

AASHTO Equ 10.6.3.1.3-1

AASHTO Equ 10.6.3.1.3-2
Nominal Bearing Resistance at Strength Limit State

- Same as calculating ultimate bearing capacity in ASD
- Present equation or graph as a function of effective width, $B'_f$
- Strength Limit State resistance factor is 0.45
- Equation or graph also applicable at Extreme Limit States with $\phi = 1.0$
Nominal Sliding Resistance

\[ R_R = \phi R_n = \phi_T R_T + \phi_{ep} R_{ep} \]

AASHTO Equ 10.6.3.4-1
Passive pressure frequently ignored due to possible excavation in future.

Add shear key to increase nominal sliding resistance.

Passive earth pressure on the shear key depth is used for nominal sliding resistance.
Nominal Sliding Resistance (on Sand)

\[ R_T = V \tan \delta \]

- **Footing cast-in-place:**
  - \( \tan \delta = \tan \phi_f \)
  - \( \phi_T = 0.8 \)

- **Pre-cast footing:**
  - \( \tan \delta = 0.8 \tan \phi_f \)
  - \( \phi_T = 0.9 \)

AASHTO Equ 10.6.3.4-2
Nominal Sliding Resistance (on Clay)

\[ \Phi_T = 0.85 \]

Shaded area \((q_s\) diagram) \(\geq 6.0\) in of compacted granular material

AASHTO Fig 10.6.3.4-1
Footings on Rock

- Service Limit State – use published presumptive bearing values
- Published values are allowable therefore settlement-limited
- Procedures for computing settlement are available (AASHTO 10.6.2.4.4)
# AASHTO Table C10.6.2.6.1-1

<table>
<thead>
<tr>
<th>Type of Bearing Material</th>
<th>Consistency in Place</th>
<th>Bearing Resistance (ksf)</th>
<th>Ordinary Range</th>
<th>Recommended Value of Use</th>
</tr>
</thead>
<tbody>
<tr>
<td>Massive crystalline igneous and metamorphic rock: granite, diorite, basalt, gneiss, thoroughly cemented conglomerate (sound condition allows minor cracks)</td>
<td>Very hard, sound rock</td>
<td>120–200</td>
<td></td>
<td>160</td>
</tr>
<tr>
<td>Foliated metamorphic rock: slate, schist (sound condition allows minor cracks)</td>
<td>Hard sound rock</td>
<td>60–80</td>
<td></td>
<td>70</td>
</tr>
<tr>
<td>Sedimentary rock: hard cemented shales, siltstone, sandstone, limestone without cavities</td>
<td>Hard sound rock</td>
<td>30–50</td>
<td></td>
<td>40</td>
</tr>
<tr>
<td>Weathered or broken bedrock of any kind, except highly argillaceous rock (shale)</td>
<td>Medium hard rock</td>
<td>16–24</td>
<td></td>
<td>20</td>
</tr>
<tr>
<td>Compaction shale or other highly argillaceous rock in sound condition</td>
<td>Medium hard rock</td>
<td>16–24</td>
<td></td>
<td>20</td>
</tr>
<tr>
<td>Well-graded mixture of fine- and coarse-grained soil: glacial till, hardpan, boulder clay (GW-GC, GC, SC)</td>
<td>Very dense</td>
<td>16–24</td>
<td></td>
<td>20</td>
</tr>
<tr>
<td>Gravel, gravel-sand mixture, boulder-gravel mixtures (GW, GP, SW, SP)</td>
<td>Very dense</td>
<td>12–20</td>
<td>14</td>
<td></td>
</tr>
<tr>
<td>-------------------------------------------------------------------</td>
<td>------------</td>
<td>-------</td>
<td>----</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Medium dense to dense</td>
<td>8–14</td>
<td>10</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Loose</td>
<td>4–12</td>
<td>6</td>
<td></td>
</tr>
<tr>
<td>Coarse to medium sand, and with little gravel (SW, SP)</td>
<td>Very dense</td>
<td>8–12</td>
<td>8</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Medium dense to dense</td>
<td>4–8</td>
<td>6</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Loose</td>
<td>2–6</td>
<td>3</td>
<td></td>
</tr>
<tr>
<td>Fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)</td>
<td>Very dense</td>
<td>6–10</td>
<td>6</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Medium dense to dense</td>
<td>4–8</td>
<td>5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Loose</td>
<td>2–4</td>
<td>3</td>
<td></td>
</tr>
<tr>
<td>Fine sand, silty or clayey medium to fine sand (SP, SM, SC)</td>
<td>Very dense</td>
<td>6–10</td>
<td>6</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Medium dense to dense</td>
<td>4–8</td>
<td>5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Loose</td>
<td>2–4</td>
<td>3</td>
<td></td>
</tr>
<tr>
<td>Homogeneous inorganic clay, sandy or silty clay (CL, CH)</td>
<td>Very dense</td>
<td>6–12</td>
<td>8</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Medium dense to dense</td>
<td>2–6</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Loose</td>
<td>1–2</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>Inorganic silt, sandy or clayey silt, varved silt-clay-fine sand (ML, MII)</td>
<td>Very stiff to hard</td>
<td>4–8</td>
<td>6</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Medium stiff to stiff</td>
<td>2–6</td>
<td>3</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Soft</td>
<td>1–2</td>
<td>1</td>
<td></td>
</tr>
</tbody>
</table>
Strength Limit State

- Very little guidance available for bearing resistance of rock
- AASHTO provides for evaluating the cohesion and friction angle of rock using the RMR Rock Mass Rating System
RMR Rock Mass Rating System

- RMR Rock Mass Rating developed for tunnel design
- Includes life safety considerations and therefore, margin of safety
- Use of cohesion and friction angle therefore may be conservative
Comparison of ASD and LRFD
LRFD vs. ASD

- All modes are expressly checked at a limit state in LRFD
- Eccentricity limits replace the overturning Factor of Safety
Geotechnical Design Recommendation

Width vs. Resistance - LRFD

Effective Footing width, $B'$ (m)

Nominal Bearing Resistance (ksf)

Strength Limit State
Service Limit State
Recommended Practice

- For LRFD design of footings on soil and rock;
  - Size footings at the Service Limit State
  - Check footing at all other applicable Limit States
- Settlement typically controls!
1. Global stability – Service Limit State
2. Bearing Resistance – Strength Limit State
3. Settlement – Service Limit State
4. Overturning/eccentricity – Strength States
Learning Outcomes

- Geotechnical theory and methods used in LRFD are the same as used in ASD to determine estimated settlement and nominal bearing resistance (i.e., ultimate bearing capacity).

- Overall stability of the site is checked using traditional methods and unfactored loads at the Service Limit State.

- Overturning is controlled by employing eccentric load limitations.
GUIDED DESIGN OF SHALLOW FOUNDATIONS

INTERIOR BRIDGE PIER ON SPREAD FOOTING
Subsurface profile at the interior bridge pier is shown in Figure 1.
There is no scour potential at this site.
Frost penetration depth is 2 ft.
Tolerable settlement is 1.5 in.
The direction conventions are shown in Figure 2.
Find

- Size the footing based on Load and Resistance Factor Design at the Service I, Strength I and Extreme I limit states

- Assume factor, $\eta = 1.0$
Figure 1: Subsurface Profile

- Column Diameter, $D = 2.0 \text{ ft}$

**Unit 1:** Lean Clay
- $7.7 \text{ ft}$
- $\gamma = 125.0 \text{pcf}$

**Unit 2:** Silty Sand
- $7.0 \text{ ft}$
- $\gamma = 125.0 \text{pcf}$

**Unit 3a:** Well graded Sand
- $15.7 \text{ ft}$
- $\gamma = 125.0 \text{pcf}$

**Unit 3b:** Well graded Sand (saturated)
- $10 \text{ ft}$
- $\gamma = 125.0 \text{pcf}$

**Unit 4:** Clean, uniform Sand
- $10 \text{ ft}$
- $\gamma = 125.0 \text{pcf}$
Figure 2: Sign Conventions and Load Directions
Steps 1 through 4:

Preliminary bridge layout, review of existing geologic and subsurface data, site reconnaissance, scour and frost potential:
The structural, hydraulics and geotechnical engineers completed these steps. The required design information is provided in the problem given statement.
Step 5:
Determine the loads applied to the footing:

The structural engineer has calculated and provided the values shown in Table 1.
### Table 1: Loads at Column Base

<table>
<thead>
<tr>
<th>Load</th>
<th>Axial, P (kip)</th>
<th>Shear, V (kip)</th>
<th>Moment, $M_z$ (kip-ft.)</th>
<th>Moment, $M_y$ (kip-ft.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dead load components (DW)</td>
<td>1439</td>
<td>38</td>
<td>156</td>
<td>552</td>
</tr>
<tr>
<td>Live load (LL)</td>
<td>375</td>
<td>9</td>
<td>302</td>
<td>145</td>
</tr>
<tr>
<td>Vehicular dynamic load allowance (IM)</td>
<td>71</td>
<td>2</td>
<td>57</td>
<td>27</td>
</tr>
<tr>
<td>Wind on structure (WS)</td>
<td>199</td>
<td>11</td>
<td>66</td>
<td>167</td>
</tr>
<tr>
<td>Wind on live load (WL)</td>
<td>4</td>
<td>1</td>
<td>5</td>
<td>19</td>
</tr>
<tr>
<td>Earthquake (EQ)</td>
<td>376</td>
<td>181</td>
<td>1236</td>
<td>4093</td>
</tr>
</tbody>
</table>
The General equation for this set of loads and limit state is

\[ \sum \eta_i \gamma_i Q_i = \eta \left( \gamma_p(DC) + \gamma_{LL}(LL) + \gamma_{IM}(IM) + \gamma_{WS}(WS) + \gamma_{WL}(WL) + \gamma_{EQ}(EQ) \right) \]

After Eqn. 3.4.1-1 AASHTO, 2004

All load factors are 1.0 except:
- Wind on the structure (WS) = 0.3

- Earthquake, is an extreme limit state load and is not included in service limit state checks.

Table 3.4.1-1 AASHTO, 2004
AASHTO provides for taking the vehicular dynamic load allowance (impact) as zero for design of substructures.

η = 1.0

Assume the effect of footing self-weight is negligible
So the Service I limit state loads are as follows:

\[ P = \eta \left[ \gamma_p(DC) + \gamma_{LL}(LL) + \gamma_{IM}(IM) + \gamma_{WS}(WS) + \gamma_{WL}(WL) + \gamma_{EQ}(EQ) \right] \]

Axial load:
\[ P = 1.0(1.0(1439)+1.0(375)+1.0(0)+0.3(199)+1.0(4)+0(376)) \]
\[ P = 1878 \text{ k} \]

Shear load:
\[ V = 1.0(1.0(38)+1.0(9)+1.0(0)+0.3(11)+1.0(1)+0(181)) \]
\[ V = 51 \text{ k} \]
\[ M_Z = 1.0(1.0(156)+1.0(302)+1.0(0)+0.3(66) +1.0(5)+0(1236)) \]

\[ M_Z = 438 \text{ k-ft} \]

\[ M_Y = 1.0(1.0(552)+1.0(145)+1.0(0)+0.3(167)+1.0(19)+0(4093)) \]

\[ M_Y = 766 \text{ k-ft} \]

<table>
<thead>
<tr>
<th>Axial Load $P$ (K)</th>
<th>Shear $V$ (K)</th>
<th>Moment in Z direction $M_Z$ (K-ft)</th>
<th>Moment in Y direction $M_Y$ (K-ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1878</td>
<td>51</td>
<td>483</td>
<td>766</td>
</tr>
</tbody>
</table>
Loads for Strength I Limit State

- All load factors $\gamma_p$ for the dead load components will be taken as minimums or maximums, depending on whether the load acts to stabilize or destabilize the footing.
For this set use the following load factors

<table>
<thead>
<tr>
<th>For checking bearing resistance</th>
<th>For checking sliding</th>
<th>For checking eccentricity</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\gamma_p = \gamma_{\text{max}}$</td>
<td>$\gamma_p = \gamma_{\text{min}}$</td>
<td>$\gamma_p = \gamma_{\text{min}}$</td>
</tr>
<tr>
<td>1.25</td>
<td>0.9</td>
<td>0.9</td>
</tr>
</tbody>
</table>
AASHTO provides for taking the vehicular dynamic load allowance (impact) as zero for design of substructures.

Assume that the effect of footing self-weight is negligible.

The minimum load cases will conservatively ignore live load.

No wind load (WS & WL) and EQ.
So the Strength I limit state loads are as follows:

\[ P = \eta [\gamma_p (DC) + \gamma_{LL} (LL) + \gamma_{IM} (IM) + \gamma_{WS} (WS) + \gamma_{WL} (WL) + \gamma_{EQ} (EQ)] \]

Axial:

\[ P_{\text{max}} = 1.0 [1.25(1439) + 1.75(375) + 1.0(0) + 0(199) + 0(4) + 0(376)] \]

\[ P_{\text{max}} = 2455 \text{ k} \]

\[ P_{\text{min}} = 1.0 [0.9(1439) + 1.75(0) + 1.0(0) + 0(199) + 0(4) + 0(376)] \]

\[ P_{\text{min}} = 1295 \text{ k} \]
Shear:

\[
P = \eta \left[ \gamma_p(\text{DC}) + \gamma_{LL}(\text{LL}) + \gamma_{IM}(\text{IM}) + \gamma_{WS}(\text{WS}) + \gamma_{WL}(\text{WL}) + \gamma_{EQ}(\text{EQ}) \right]
\]

\[
V_{\text{max}} = 1.0[1.25(38)+1.75(9)+1.0(0)+0(11)+0(1)+0(181)]
\]

\[
V_{\text{max}} = 63k
\]

\[
V_{\text{min}} = 1.0[0.9(38)+1.75(0)+1.0(0)+0(11)+0(1)+0(181)]
\]

\[
V_{\text{min}} = 34k
\]
Moment about \( Z \):

\[
P = \eta \left[ \gamma_p(DC) + \gamma_{LL}(LL) + \gamma_{IM}(IM) + \gamma_{WS}(WS) + \gamma_{WL}(WL) + \gamma_{EQ}(EQ) \right]
\]

\[M_{Z\text{max}} = 1.25[1.0(156)+1.75(302)+1.0(0)+0(66)+0(5)+0(1236)]\]

\[M_{Z\text{max}} = 724 \text{ k-ft}\]

\[M_{Z\text{min}} = 1.0[0.9(156)+1.75(0)+1.0(0)+0(66)+0(5)+0(1236)]\]

\[M_{Z\text{min}} = 140 \text{ k-ft}\]
Moment about $Y$:

$$P = \eta \left[ \gamma_p(\text{DC}) + \gamma_{LL}(\text{LL}) + \gamma_{IM}(\text{IM}) + \gamma_{WS}(\text{WS}) + \gamma_{WL}(\text{WL}) + \gamma_{EQ}(\text{EQ}) \right]$$

$$M_{Y_{\text{max}}} = 1.0[1.25(522)+1.75(145)+1.0(0)+0(167)+0(19)+0(4093)]$$

$$M_{Y_{\text{max}}} = 944 \text{ k-ft}$$

$$M_{Y_{\text{min}}} = 1.0[0.9(552)+1.75(0)+1.0(0)+0(167)+0(19)+0(4093)]$$

$$M_{Y_{\text{min}}} = 497 \text{ k-ft}$$
<table>
<thead>
<tr>
<th></th>
<th>Axial Load $P$ (K)</th>
<th>Shear $V$ (K)</th>
<th>Moment in Z direction $M_Z$ (K-ft)</th>
<th>Moment in Y direction $M_Y$ (K-ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max</td>
<td>2455</td>
<td>63</td>
<td>724</td>
<td>944</td>
</tr>
<tr>
<td>Min</td>
<td>1295</td>
<td>34</td>
<td>140</td>
<td>497</td>
</tr>
</tbody>
</table>
Loads for Extreme I Limit State

- All load factors for the dead load components will be taken as maximums. (Dynamic model used maximum factored loads to compute the earthquake forces)
- A separate check using earthquake loads computed using minimum load factors should also be performed.
For this set, use the following load factors

<table>
<thead>
<tr>
<th>For checking bearing resistance</th>
<th>For checking sliding</th>
<th>For checking eccentricity</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\gamma_p = \gamma_{\text{max}}$</td>
<td>$\gamma_p = \gamma_{\text{max}}$</td>
<td>$\gamma_p = \gamma_{\text{max}}$</td>
</tr>
<tr>
<td>1.25</td>
<td>1.25</td>
<td>1.25</td>
</tr>
</tbody>
</table>
AASHTO provides for taking the vehicular dynamic load allowance (impact) as zero for design of substructures (IM = 0.0)

Assume that the effect of footing self-weight is negligible

The minimum load cases will conservatively ignore live load (\(\gamma_{LL} = 0.0\))

No wind load (WS & WL)
So the Extreme I limit state loads are as follows:

Axial load for evaluating bearing stress (Seismic force acting downward)

\[ P_{\text{E}_{\text{max}}} = 1.0[1.25(1439)+0(375)+1.0(0)+0(199)+0(4)+1.0(376)] \]

\[ P_{\text{E}_{\text{max}}} = 2174 \text{ k} \]

\[ P_{\text{E}_{\text{min}}} = 1.0[1.25(1439)+0(375)+1.0(0)+0(199)+0(4)-1.0(376)] \]

\[ P_{\text{E}_{\text{min}}} = 1423 \text{ k} \]
So the Extreme I limit state loads are as follows:

\[
V_{Emax} = 1.0[1.25(38) + 0(9) + 1.0(0) + 0(11) + 0(1) + 1.0(181)]
\]

\[
V_{Emax} = 228k
\]

\[
M_{ZE} = 1.0[1.25(156) + 0(302) + 1.0(0) + 0(66) + 0(5) + 1.0(1236)]
\]

\[
M_{ZE} = 1431 \text{ k-ft}
\]

\[
M_{YE} = 1.0[1.25(552) + 0(145) + 1.0(0) + 0(167) + 0(19) + 1.0(4093)]
\]

\[
M_{YE} = 4783 \text{ k-ft}
\]
<table>
<thead>
<tr>
<th>Axial Load $P_{max}$ (K) (Bearing Stress)</th>
<th>Axial Load $P_{min}$ (K) (Sliding eccentricity)</th>
<th>Shear V (K)</th>
<th>Moment in Z direction $M_z$ (K-ft)</th>
<th>Moment in Y direction $M_y$ (K-ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2174</td>
<td>1423</td>
<td>228</td>
<td>1431</td>
<td>4783</td>
</tr>
</tbody>
</table>
Step 6:

Conduct Field Exploration and Laboratory Testing:

This step is completed, and the subsurface data are shown in Figure 1.
Due to the granular nature of the soils encountered in the boring, laboratory testing was limited to soil classifications, and moisture content, and SPT.

The foundation design will be based largely on design parameters correlated to the soil classifications and the SPT N-values.
Compute the initial vertical effective stresses at the midpoint of each layer:

<table>
<thead>
<tr>
<th>Layer 2</th>
<th>Layer 3a</th>
<th>Layer 3b</th>
<th>Layer 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \sigma'_{vo} ) (ksf)</td>
<td>1.375</td>
<td>2.7625</td>
<td>4.038</td>
</tr>
</tbody>
</table>

The effective stress diagram for the profile at the pier location is plotted in Figure 3.
Figure 3: The effective stress diagram for the profile at the pier location.
Step 7:

Calculate Nominal Bearing Resistance at the Strength and Extreme Limit State
The General Bearing Capacity Equation is

\[ q_n = cN_c s c i_c + \gamma D_f N_q s_q d_q i_q C_{W_q} + 0.5\gamma B N_{\gamma} s_{\gamma} i_{\gamma} C_{W_{\gamma}} \]

Equ. 10.6.3.1.2-1, AASHTO

- The first term goes to zero since the foundation materials are granular & noncohesive.

- The load inclination factors will also be taken as 1.0, as this is consistent with common practice.
For the interior pier, we will assume that the footing is essentially rectangular (L/B < 5)

**the shape and inclination factors should not be applied together.** Since the inclination are taken as 1.0, we will only apply the shape factors as follows:

\[
S_\gamma = 1 - 0.4 \frac{B_f}{L_f}
\]

\[
S_\gamma = 0.6 \text{ Assume the effect of eccentricity is small for now, so } B_f \approx L_f
\]

Table 10.6.3.1.2-3, AASHTO
The footing would be cast against the firm Layer 2 as Geotech. Engineer has recommended.

- From Fig. 1, SPT N-value for this layer is 20.
- From Fig. 3, $\sigma'_v$ in Layer 2 is about 1.4 ksf.

$\varphi = 35^\circ$ is selected for computing bearing Capacity

$$S_q = 1 + \frac{B_f}{L_f} \tan \varphi$$  
Table 10.6.3.1.2-3, AASHTO
Since the overburden materials above the footing are cohesive soils, then

\[ d_q = 1.0 \ldots \text{Table 10.6.3.1.2-4 AASHTO} \]

\[ q = \gamma' D_f = 0.9375 \text{ ksf} \]

Check ground water depth
$D_f = 7.5 \text{ ft}$ ; Assumes a 3-ft-thick footing and 4.5-ft cover

$D_w = \text{Depth of ground water below the surface} = 29.9 \text{ ft}$

Conservatively estimate the footing width with an upper bound value of $B_f = 20 \text{ ft}$

$C_w = 0.9$ ; $C_w = 1.0$  

$N = 48.0$ ; $N = 33.3$  

(Table 10.6.3.1.2-2, AASHTO)
Then

\[ q_n = \gamma' D_f \nu N_{q_s} s_{q_d} i_q C_{W_q} + 0.5 \gamma B \nu N_{s_i} C_{W_s} \]

\[ q_n = 937.5(33.3)(1.0)(1.7)(1.0)(1.0) + 0.5(125)(B)(48)(0.9)(0.6)(1.0) \]

\[ q_n = 53072 + 1620B \text{ (lb/ft}^2\text{)} \]
Since the nominal bearing resistance was computed using a soil friction angle correlated to SPT N-values, a resistance factor $\Phi = 0.45$ should be applied to resistances computed using this equation when checking strength limit states

\[(AASHTO \text{ Table } 10.5.5.2.1-I)\]
Step 8:

Calculate Nominal Bearing Resistance at the Service Limit State

Calculate stress increase at the midpoint of each soil layer using the 2-on-1 stress distribution method for various footing widths; say $B_f = 10$ ft, 15 ft and 20 ft as a function of the stress applied by the footing, $q$: 
\[ \Delta \sigma_v / q = B_f \cdot L_f / (B_f + Z)(L_f + Z) \]

But \( B_f \approx L_f \)

\[ \Delta \sigma_v / q = B_f^2 / (B_f + Z)^2 \]
Table 2: Stress Increase with Depth as a Function of Footing Width and Stress Applied by the Footing (q)

<table>
<thead>
<tr>
<th>Soil Layer</th>
<th>Z (ft) (Below footing base)</th>
<th>Stress Increase, $\Delta \sigma_v$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$B_f = 10\text{ft}$</td>
</tr>
<tr>
<td>2</td>
<td>3.4</td>
<td>0.55q</td>
</tr>
<tr>
<td>3a</td>
<td>14.6</td>
<td>0.16q</td>
</tr>
<tr>
<td>3b</td>
<td>27.2</td>
<td>0.07q</td>
</tr>
<tr>
<td>4</td>
<td>37.0</td>
<td>0.04q</td>
</tr>
</tbody>
</table>
For these widths, determine the stress that would need to be applied to generate 1.5 in settlement using the Hough method.

\[
\Delta H = H \left( \frac{1}{C'} \right) \log \left[ \left( \sigma'_{V_o} + \Delta \sigma'_V \right) / \sigma'_{V_o} \right]
\]

From AASHTO Equ. 10.6.2.4.1-3

<table>
<thead>
<tr>
<th>Layer</th>
<th>2</th>
<th>3a</th>
<th>3b</th>
<th>4</th>
</tr>
</thead>
<tbody>
<tr>
<td>C'</td>
<td>65</td>
<td>120</td>
<td>102</td>
<td>110</td>
</tr>
</tbody>
</table>

From AASHTO Fig. 10.6.2.4.1-1
Calculate settlement in each layer and sum the settlements to find the applied stress, $q$, required to yield a total settlement 1.5 inches at a footing width. This is an iterative process.
B = 10 ft  find q = 12.5 ksf to produce $\Sigma \Delta H = 1.5$ in

<table>
<thead>
<tr>
<th>$\Delta H_2$</th>
<th>$\Delta H_3a$</th>
<th>$\Delta H_3b$</th>
<th>$\Delta H_4$</th>
<th>$\Sigma \Delta H$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0 in</td>
<td>0.35 in</td>
<td>0.12 in</td>
<td>0.04 in</td>
<td>1.5 in</td>
</tr>
</tbody>
</table>

$$\Delta H_2 = H_2 \left( \frac{1}{C'_2} \right) \log \left[ \left( \sigma'_{vo} + \Delta \sigma_v \right) / \sigma'_v \right]_2$$

$$\Delta H_2 = 7 \text{ ft} \left( \frac{1}{65} \right) \log \left[ \left( 1.375 \text{ ksf} + 0.55(12.5 \text{ ksf}) \right) / 1.375 \text{ ksf} \right]$$

= 0.08 ft
= 1.0 inch
Repeat this process for several footing widths over the expected range of footing dimensions, say 5 to 23 feet. Plot the results as a function of effective footing width as shown in Fig. 4.
Figure 4: Nominal Bearing Resistance vs. Effective Footing Width at Service I Limit State for 1.5 in settlement
Step 9:

Calculate Nominal Sliding and Passive Soil Resistance at the Strength and Extreme Limit State
Sliding resistance of concrete footings is given by:

\[ R_R = \phi_\tau R_\tau + \phi_{ep} R_{ep} \]

\[ R_\tau = V \tan \delta \]  \hspace{1cm} \text{(Eqn. 10.3.3.4-2, AASHTO)}

\( V = \text{Axial load} \)

\( \delta = \phi = 35^\circ \)

\[ R_\tau = 0.7(V) \]
The resistance factor associated with strength limit state checks is $\Phi = 0.8$ (Table 10.5.5.2.1-1, AASHTO).

Passive pressure will be ignored

$$R_R = (0.8)(0.7)(1295^k) = 725 \text{ kips}$$
Step 10:

Check Global Stability of the Footing

The geotechnical engineer has determined that since this is an interior pier with level ground conditions, global stability is not a concern.
Step 11:

Size the Footing at the Service Limit State

First determine the eccentricity of the loads applied to the footing in both directions. Then find $B'_f$ and $L'_f$. 
Estimate trial footing dimensions. Try 16.4 ft by 16.4 ft footing and calculate $B'_f$ and $L'_f$. The smaller will be used to enter Fig 4 to determine the nominal bearing resistance at the service limit state.

\[ e_y = \frac{M_z}{P} \]
\[ e_y = \frac{483}{1878} \]
\[ e_y = 0.26 \text{ ft} \]

\[ e_z = \frac{M_y}{P} \]
\[ e_z = \frac{766}{1878} \]
\[ e_z = 0.41 \text{ ft} \]
\[ B'_f = B_f - 2e_y \]  
(Eqn. 10.6.1.3-1, AASHTO)

\[ B'_f = 15.88 \text{ft} \]

\[ L'_f = L_f - 2e_z \]  
(Eqn. 10.6.1.3-2, AASHTO)

\[ L'_f = 15.58 \text{ ft} \]
Since $L'_f$ is less than $B'_f$, use $L'_f = 15.58$ ft to check nominal bearing resistance at the service limit state.

From Fig. 4, the nominal bearing resistance is about 8.145 ksf.
Compute applied bearing stress over effective footing area, $A'$

$$A' = B'_f L'_f = 2475 \text{ft}^2$$

$$q_{\text{applied}} = \frac{P}{A'} = \frac{1878k}{247.5 \text{ ft}^2} = 7.6 \text{ ksf}$$

This is less than 8.145 ksf from Fig 4 for settlement less than 1.5 in. Use 16.4 ft by 16.4 ft footing to perform the rest of the design checks.
Check assumption to neglect self-weight of footing assuming a footing thickness of 3.3 ft:

\[ W_{ftg} = V_{ftg}(\gamma_{\text{concrete}} - \gamma_{\text{soil}}) = 22k \]

This should be less than 1 to 2% of the axial load applied to the footing:

\[ 2\%P = 0.02(1878k) = 37.6k \quad (\text{OK}) \]
Step 12:

Check the bearing pressure, maximum eccentricity and sliding at the Strength limit state.
Bearing Pressure

Compute effective footing dimensions and bearing area using maximum load factors applied to the dead load components:

\[ e_y = \frac{M_{Z_{\text{max}}}}{P_{\text{max}}} \]
\[ e_y = \frac{724}{2455} \]
\[ e_y = 0.29 \text{ ft} \]

\[ e_z = \frac{M_{Y_{\text{max}}}}{P_{\text{max}}} \]
\[ e_z = \frac{944}{2455} \]
\[ e_z = 0.38 \text{ ft} \]
\[ B'_f = B_f - 2e_y \]
\[ B'_f = 16.4 - 2(0.29) = 15.82\text{ft} \]

\[ L'_f = L_f - 2e_y \]
\[ L'_f = 16.4 - 2(0.38) = 15.64\text{ft} \]

\[ A' = L'_f \cdot B'_f = (15.82)(15.64) = 247.5\text{ ft}^2 \]
\[ q_{\text{applied}} = \frac{P}{A'} = \frac{2455}{247.5} = 9.5\text{ ksf} \]
Nominal bearing resistance at this effective footing width, $B'_f$

\[ q_n = 53072 + 1620 B'_f \quad \text{lb/ft}^2 \]

$B_f = 15.64\text{ft}$

$q_n = 78.4 \text{ ksf}$
The factored resistance at the extreme limit state is then:

\[ R_n = \phi(q_n) \quad , \quad \phi = 0.45 \]

\[ R_n = 35.2 \text{ ksf} \]

This is greater than \( q_{\text{applied}} = 9.9 \text{ ksf} \)
**Maximum eccentricity**

The maximum eccentricity in any direction should be less than one-fourth of the actual footing dimension (AASHTO). The maximum eccentricity is evaluated using the minimum load factor combination:

\[ e_y = \frac{M_{Z_{min}}}{P_{min}} \]
\[ e_y = \frac{140}{1295} \]
\[ e_y = 0.11 \text{ ft} \]

\[ e_z = \frac{M_{Y_{min}}}{P_{min}} \]
\[ e_z = \frac{497}{1295} \]
\[ e_z = 0.38 \text{ ft} \]
\[ \frac{1}{4} B_f = \frac{1}{4} L_f \]

\[ = \frac{1}{4}(16.4\text{ft}) = 4.1\text{ft} \]

\[ e_y < e_z < 4.1\text{ft} \]
The factored horizontal shear force must be less than the factored nominal sliding resistance. This is evaluated using minimum load factors. Equation for $R_\tau$ is provided in Step 9. Passive resistance is ignored, so $R_{ep}$ is zero.

$$R_R = \phi_\tau R_\tau + \phi_{ep} R_{ep}$$

$$R_R = 0.8(0.7(1295)) = 725 \text{ (k)}$$

$$V_{min} = 34 \text{ (k)} < 725 \text{ (k)} \quad \text{OK}$$
Step 13:

Check the bearing pressure, maximum eccentricity and sliding at the extreme limit state
Bearing Pressure

Compute effective footing dimensions and bearing area using maximum load factors applied to dead load components

\[ e_y = \frac{M_{ZE}}{P_{E_{max}}} \]
\[ e_y = \frac{1431}{2174} \]
\[ e_y = 0.66 \text{ ft} \]

\[ e_z = \frac{M_{YE}}{P_{E_{max}}} \]
\[ e_z = \frac{4783}{2174} \]
\[ e_z = 2.2 \text{ ft} \]
B'\(_f\) = B\(_f\) - 2e\(_y\)

B'\(_f\) = 16.4 - 2(0.66) = 15.08 ft

L'\(_f\) = L\(_f\) - 2e\(_y\)

L'\(_f\) = 16.4 - 2(2.2) = 12.0 ft

A' = L'\(_f\) \times B'\(_f\) = (15.08)(12.0) = 181 ft\(^2\)

---

<table>
<thead>
<tr>
<th>(e_y)</th>
<th>(e_z)</th>
<th>(B'(_f))</th>
<th>(L'(_f))</th>
<th>(A')</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.66 ft</td>
<td>2.2 ft</td>
<td>15.08 ft</td>
<td>12.0 ft</td>
<td>181 ft(^2)</td>
</tr>
</tbody>
</table>

\(q_{applied} = \frac{P}{A'} = \frac{2174}{181} = 12\text{ ksf}\)
Nominal bearing resistance at $B'_f = 12.0$ ft

$$q_n = 53072 + 1620B'_f$$

$q_n = 72.5$ ksf

$$R_R = \varphi q_n = 1.0(72.5)$$

$R_R = 72.5$ ksf $> q_{\text{applied}}$  

OK
Maximum eccentricity

The maximum eccentricity in any direction should be less than one-fourth of the actual footing dimension (AASHTO). The maximum eccentricity is evaluated using the minimum load factor combination:

\[
e_y = \frac{M_{ZE}}{P_{Emin}}
\]

\[
e_y = \frac{1431}{1423} = 1.0 \text{ ft}
\]

\[
e_z = \frac{M_{YE}}{P_{Emin}}
\]

\[
e_z = \frac{4783}{1423} = 3.4 \text{ ft}
\]
\[
\frac{1}{4} B_f = \frac{1}{4} L_f = \frac{1}{4}(16.4\text{ft}) = 4.1\text{ft}
\]

\[e_y < e_Z < 4.1\text{ft}\]
The factored horizontal shear force must be less than the factored nominal sliding resistance. This is evaluated using minimum load factors.

\[
R_R = \phi_\tau R_\tau + \phi_{ep} R_{ep}
\]

\[
R_R = 1.0(0.7)(1423) = 996 \text{ (k)}
\]

\[
V_E = 228 \text{ k} < 996 \text{ k} \quad \text{OK}
\]
Step 14:

Complete Structural Design of Footing

The structural engineer completes this step
Session VI

LRFD Design of Deep Foundations
OBJECTIVES

A. State the performance limits that should be evaluated when designing a deep foundation
B. Be able to select a deep foundation type
C. Be able to select the appropriate resistance factor for each performance limit evaluated
Start

Define subsurface conditions
geometric constraints

Define loads
load combinations

Factor loads for each combination

Verify need for deep foundation

Select suitable pile type
Select suitable pile type

Determine nominal axial structural resistance

Determine Factored axial structural. resistance for single pile

Determine Factored axial geot. resistance for single pile

Check drivability

Driven pile found.

Pile Group Design
Deep Foundation Design Process

1. Decide deep foundation type
2. Select resistance factor
3. Compute resistances
4. Layout foundation group and analyze at the strength limit state
5. Check the service limit state
Deep Foundation Performance Limits
Strength Limit State Checks

**Driven Piles**
- Structural resistance
- Axial geotechnical resistance
- Driven resistance

**Drilled Shafts**
- Structural resistance
- Axial geotechnical resistance
Driven Resistance

Pile damage
Driven Performance Limit
Driven Performance Limit
Service Limit State Checks

**Driven Piles**
- Global Stability
- Vertical Displacement
- Horizontal Displacement

**Drilled Shafts**
- Global Stability
- Vertical Displacement
- Horizontal Displacement
Displacement

\[ \Delta z \]

\[ \Delta x \]
LRFD Differences from ASD

- **Same**
  - Determining Resistance
  - Determining Deflection

- **Different**
  - Comparison of load and resistance
  - Specific separation of resistance and deflection
Deep foundation type selection

- Method of support
- Bearing material depth
- Load type, direction and magnitude
- Constructability
- Cost
### Deep Foundation Types

<table>
<thead>
<tr>
<th>Driven Piles</th>
<th>Deep Foundation Material</th>
</tr>
</thead>
<tbody>
<tr>
<td>Driven</td>
<td>Premised Concrete</td>
</tr>
<tr>
<td></td>
<td>Post-tension Concrete</td>
</tr>
<tr>
<td></td>
<td>Pre-cast Concrete</td>
</tr>
<tr>
<td></td>
<td>Cast-in-place Concrete</td>
</tr>
<tr>
<td></td>
<td>Steel</td>
</tr>
<tr>
<td></td>
<td>Wood</td>
</tr>
<tr>
<td></td>
<td>Specialty / Composites</td>
</tr>
</tbody>
</table>

- **Drilled Shafts**
- **Driven**
  - X
  - X
  - X
  - X
  - X
  - X
  - X
  - X
  - X
- **Drilled or Bored**
  - --
  - X
  - --
  - X
  - X
  - X
  - --
  - X
- **Jacked / Special**
  - X
  - --
  - --
  - X
  - X
  - X
  - --
  - X
Method of Support

End Bearing  Side Friction  Combined
Driven Low Displacement Piles
Driven High Displacement Piles
Drilled Shafts
Depth to Bearing/ Scour
Load Type and Direction

- Permanent/Transient/Cyclic
- Horizontal or Vertical
Load Type and Direction

- Wood is better for transient resistance than permanent
- Steel pile better cyclic resistance
- High horizontal loads better resisted by stiffer piles or shafts
## Load Magnitude

<table>
<thead>
<tr>
<th>Deep foundation type</th>
<th>Typical range of nominal (ultimate) resistance (kips)</th>
<th>Typical length (feet)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Timber pile</td>
<td>75 – 200</td>
<td>20 – 40</td>
</tr>
<tr>
<td>Concrete pile</td>
<td>200 – 2,000</td>
<td>20 – 150</td>
</tr>
<tr>
<td>Steel H-pile</td>
<td>200 – 1,000</td>
<td>20 – 160</td>
</tr>
<tr>
<td>Pipe pile</td>
<td>175 – 2,500</td>
<td>20 – 100</td>
</tr>
<tr>
<td>Drilled shaft</td>
<td>750 – 10,000</td>
<td>20 – 160</td>
</tr>
</tbody>
</table>
Obstructions/ Rock

Use low displacement steel piles
-or-
Drilled shafts
**Equipment access**

- Low headroom requires pile splicing
- Equipment size a function of pile/shaft size
Selection of Resistance factors

- **Strength limit state**
  - Structural Resistance
  - Geotechnical Resistance
  - Driven Resistance (piles only)

- **Service limit state**
  - Resistance factor = 1.0
Methods for determining structural resistance

- Axial compression
- Combined axial and flexure
- Shear

Concrete - Section 5
Steel - Section 6
Wood - Section 8
Structural resistance factors

Concrete (5.5.4.2.1)
Axial Comp. = 0.75
Flexure = 0.9
Shear = 0.9
Pile Driving = 1.0
Structural resistance factors

Steel (6.5.4.2)

Axial = 0.5 (H-pile)
  0.6 (pipe pile)

Combined

  Axial = 0.7 (H-pile)
  0.8 (pipe pile)

  Flexure = 1.0

Shear = 1.0

Resistance during pile driving = 1.0
Structural resistance factors

**Timber (8.5.2.2)**
- Compression = 0.9
- Tension = 0.8
- Flexure = 0.85
- Shear = 0.75
- Driving = 1.15

LRFD Specifications
H-Pile Driven to Refusal

For any single pile the total factored load shall not exceed the maximum factored structural resistance.

<table>
<thead>
<tr>
<th>Pile Size</th>
<th>$R_{R_{\text{max}}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>HP10x42</td>
<td>310 kips</td>
</tr>
<tr>
<td>HP12x53</td>
<td>380 kips</td>
</tr>
<tr>
<td>HP14x73</td>
<td>530 kips</td>
</tr>
</tbody>
</table>

Assume $F_y = 50$ ksi

- No unbraced length
- Axial load with no moment / bending
- Not suitable for capped pile piers or pile in soils subjected to scour
When estimating pile length, the depth to refusal shall be assumed as the elevation on the nearest soil boring where the rock core begins.
ODOT BDM 202.2.3.2.b
Piles Not Driven to Refusal on Bedrock
(Friction Piles)

Prefer cast-in-place reinforced concrete piles

\[ R_{ndr} = \frac{Q}{\phi_{dyn}} \]

- \( R_{ndr} \) = ultimate bearing value
- \( Q \) = total factored load per the highest loaded pile at each substructure unit (kips)
- \( \phi_{dyn} \) = 0.70 for piles installed according to CMS507 and CMS523
## ODOT BDM 202.2.3.2.b
### Maximum Ultimate Bearing Value

<table>
<thead>
<tr>
<th>Pipe Pile Diameter (in.)</th>
<th>Maximum $R_{ndr}$ (kips)</th>
<th>H-Pile Size</th>
<th>Maximum $R_{ndr}$ (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>12</td>
<td>330</td>
<td>HP 10x42</td>
<td>350</td>
</tr>
<tr>
<td>14</td>
<td>390</td>
<td>HP 12x53</td>
<td>380</td>
</tr>
<tr>
<td>16</td>
<td>450</td>
<td>HP 14x73</td>
<td>440</td>
</tr>
</tbody>
</table>

- **CMS Wall thickness**
- **Drivability Basis**
**ODOT BDM 202.2.3.2.b**  
**Maximum Ultimate Bearing Value**

<table>
<thead>
<tr>
<th>Pipe Pile Diameter (in.)</th>
<th>Maximum $R_{ndr}$ (kips)</th>
<th>H-Pile Size</th>
<th>Maximum $R_{ndr}$ (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>12</td>
<td>250</td>
<td>HP 10x42</td>
<td>475</td>
</tr>
<tr>
<td>14</td>
<td>300</td>
<td>HP 12x53</td>
<td>595</td>
</tr>
<tr>
<td>16</td>
<td>390</td>
<td>HP 14x73</td>
<td>820</td>
</tr>
</tbody>
</table>
Determining Geotechnical Resistance of Piles

- Field methods
  - Static load test
  - Dynamic load test (PDA)
  - Driving Formulae

- Static analysis methods
Determining Geotechnical Resistance of Piles
Static Load Test

Load

Settlement

Elastic pile compression

Pile top settlement

AASHTO 10.7.3.8.2

VI-39
Dynamic Load Test (PDA)
# Geotechnical Resistance Factors for Piles

<table>
<thead>
<tr>
<th>Nominal Resistance of Single Pile in Axial Compression—Dynamic Analysis and Static Load Test Methods, $\phi_{dyn}$</th>
<th>Condition/Resistance Determination Method</th>
<th>Resistance Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Driving criteria established by static load test(s); quality control by dynamic testing and/or calibrated wave equation, or minimum driving resistance combined with minimum delivered hammer energy from the load test(s). For the last case, the hammer used for the test pile(s) shall be used for the production piles.</td>
<td>Values in Table 2</td>
</tr>
<tr>
<td></td>
<td>Driving criteria established by dynamic test with signal matching at beginning of redrive conditions only of at least one production pile per pier, but no less than the number of tests per site provided in Table 3. Quality control of remaining piles by calibrated wave equation and/or dynamic testing.</td>
<td>0.65</td>
</tr>
<tr>
<td></td>
<td>Wave equation analysis, without pile dynamic measurements or load test, at end of drive conditions only</td>
<td>0.40</td>
</tr>
<tr>
<td></td>
<td>FHWA-modified Gates dynamic pile formula (End of Drive condition only)</td>
<td>0.40</td>
</tr>
<tr>
<td></td>
<td>Engineering News Record (as defined in Article 10.7.3.8.5) dynamic pile formula (End of Drive condition only)</td>
<td>0.10</td>
</tr>
</tbody>
</table>
### Geotechnical Resistance Factors for Piles

#### AASHTO Table 10.5.5.2.3-1

<table>
<thead>
<tr>
<th>Condition/Resistance Determination Method</th>
<th>Resistance Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Skin Friction and End Bearing: Clay and Mixed Soils</td>
<td>0.35</td>
</tr>
<tr>
<td>α-method (Tomlinson, 1987; Skempton, 1951)</td>
<td>0.25</td>
</tr>
<tr>
<td>β-method (Esrig &amp; Kirby, 1979; Skempton, 1951)</td>
<td>0.30</td>
</tr>
<tr>
<td>λ-method (Vijayvergiya &amp; Focht, 1972; Skempton, 1951)</td>
<td>0.40</td>
</tr>
<tr>
<td>Skin Friction and End Bearing: Sand</td>
<td>0.45</td>
</tr>
<tr>
<td>Nordlund/Thurman Method (Hannigan et al., 2005)</td>
<td>0.30</td>
</tr>
<tr>
<td>SPT-method (Meyerhof)</td>
<td>0.25</td>
</tr>
<tr>
<td>CPT-method (Schmertmann)</td>
<td>0.40</td>
</tr>
<tr>
<td>End bearing in rock (Canadian Geotech. Society, 1985)</td>
<td>0.50</td>
</tr>
<tr>
<td>Block Failure, $\varphi_{bl}$</td>
<td>0.60</td>
</tr>
<tr>
<td>Uplift Resistance of Single Piles, $\varphi_{up}$</td>
<td>0.35</td>
</tr>
<tr>
<td>Nordlund Method</td>
<td>0.25</td>
</tr>
<tr>
<td>α-method</td>
<td>0.20</td>
</tr>
<tr>
<td>β-method</td>
<td>0.30</td>
</tr>
<tr>
<td>λ-method</td>
<td>0.25</td>
</tr>
<tr>
<td>SPT-method</td>
<td>0.40</td>
</tr>
<tr>
<td>CPT-method</td>
<td>0.60</td>
</tr>
<tr>
<td>Load test</td>
<td>0.50</td>
</tr>
<tr>
<td>Group Uplift Resistance, $\varphi_{ug}$</td>
<td>0.35</td>
</tr>
<tr>
<td>Sand and clay</td>
<td>0.50</td>
</tr>
<tr>
<td>Horizontal Geotechnical Resistance of Single Pile or Pile Group</td>
<td>0.35</td>
</tr>
<tr>
<td>All soils and rock</td>
<td>1.0</td>
</tr>
<tr>
<td>Structural Limit State</td>
<td>See the provisions of Article 6.5.4.2</td>
</tr>
<tr>
<td>Concrete piles</td>
<td>See the provisions of Article 5.5.4.2.1</td>
</tr>
<tr>
<td>Timber piles</td>
<td>See the provisions of Article 8.5.2.2 and 8.5.2.3</td>
</tr>
<tr>
<td>Pile Drivability Analysis, $\varphi_{ad}$</td>
<td>See the provisions of Article 6.5.4.2</td>
</tr>
<tr>
<td>Steel piles</td>
<td>See the provisions of Article 5.5.4.2.1</td>
</tr>
<tr>
<td>Concrete piles</td>
<td>See the provisions of Article 8.5.2.2</td>
</tr>
<tr>
<td>Timber piles</td>
<td>In all three Articles identified above, use $\varphi$ identified as “resistance during pile driving”</td>
</tr>
</tbody>
</table>

- Site Variability Defined in NCHRP Report 507
- Range of Values of Resistance Factors Depends on Number of Static Load Tests
### Relationship between Number of Static Load Tests Conducted per Site and $\phi$

<table>
<thead>
<tr>
<th>Number of Static Load Tests per Site</th>
<th>Resistance Factor, $\phi$</th>
<th>Site Variability$^a$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Low$^a$</td>
<td>Medium$^a$</td>
</tr>
<tr>
<td>1</td>
<td>0.80</td>
<td>0.70</td>
</tr>
<tr>
<td>2</td>
<td>0.90</td>
<td>0.75</td>
</tr>
<tr>
<td>3</td>
<td>0.90</td>
<td>0.85</td>
</tr>
<tr>
<td>&gt;4</td>
<td>0.90</td>
<td>0.90</td>
</tr>
</tbody>
</table>

$^a$ See commentary.
Number of Dynamic Tests with Signal Matching Analysis per Site to Be Conducted During Production Pile Driving

<table>
<thead>
<tr>
<th>Site Variability&lt;sup&gt;a&lt;/sup&gt;</th>
<th>Low&lt;sup&gt;a&lt;/sup&gt;</th>
<th>Medium&lt;sup&gt;a&lt;/sup&gt;</th>
<th>High&lt;sup&gt;a&lt;/sup&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of Piles Located Within Site</td>
<td>Number of Piles with Dynamic Tests and Signal Matching Analysis Required (BOR)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>≤15</td>
<td>3</td>
<td>4</td>
<td>6</td>
</tr>
<tr>
<td>16–25</td>
<td>3</td>
<td>5</td>
<td>8</td>
</tr>
<tr>
<td>26–50</td>
<td>4</td>
<td>6</td>
<td>9</td>
</tr>
<tr>
<td>51–100</td>
<td>4</td>
<td>7</td>
<td>10</td>
</tr>
<tr>
<td>101–500</td>
<td>4</td>
<td>7</td>
<td>12</td>
</tr>
<tr>
<td>&gt;500</td>
<td>4</td>
<td>7</td>
<td>12</td>
</tr>
</tbody>
</table>

<sup>a</sup> See commentary.
ODOT BDM 303.4.2.6 Dynamic Load Test

- one dynamic load testing item for each pile size
- If multiple pile capacities are required for a given pile size, specify one testing item for each ultimate bearing value ($R_{ndr}$)
- When static load tests are required, provide one dynamic load testing item and three restrike items for each load test item
One dynamic load testing item = a minimum of 2 piles and performing a CAPWAP analysis on one of the two piles.

One restrike item = performing a dynamic test on one pre-driven pile.
Computation of Static Geotechnical Resistance

\[ R_R = \phi R_n \]

\[ \phi R_n = \phi_{qp} R_p + \phi_{qs} R_s \]

\[ R_p = A_p q_p \]

\[ R_s = A_s q_s \]

AASHTO 10.7.3.7.5-2
Static Analysis Methods

Driven Piles
- $\alpha$ method
- $\beta$ method
- $\lambda$ method
- Nordlund - Thurman method
- SPT-method
- CPT-method

Drilled Shafts
- $\alpha$ method
- $\beta$ method
- Side friction in Rock
- Tip Resistance in Rock
Driven Piles - $\alpha$ Method

- Cohesive soil
- Total stress method
- $q_s = \alpha S_u$
- $q_p = 9S_u$
- AASHTO 10.7.3.8.6b
Driven Piles - $\beta$ Method

- Effective stress method

- $q_s = \beta \sigma_v'$

- AASHTO 10.7.3.8.6c

Figure 10.7.3.8.6c-1 $\beta$ Versus OCR for Displacement Piles after Esrig and Kirby (1979)
Driven Piles - $\lambda$ Method

- Effective stress method
- Containing a total stress parameter
- $q_s = \lambda (\sigma_v' + 2S_u)$
- AASHTO 10.7.3.8.6d
Driven Piles - Nordlund/Thurman Method (Sand)

\[ q_s = K_\delta C_F \sigma'_v \frac{\sin(\delta + \omega)}{\cos \omega} \]

- \( K_\delta \) = Coefficient of lateral earth pressure at midpoint of soil layer (dim.)
- \( C_F \) = Correction factor for \( K_\delta \) (dim.)
- \( \sigma'_v \) = Effective overburden stress at midpoint of soil layer (ksf)
- \( \delta \) = Friction angle between pile and soil (deg.)
- \( \omega \) = Angle of pile taper from vertical (deg.)

\[ q_p = \alpha_t N_q' \sigma'_v \leq q_L \]

- \( \alpha_t \) = Coefficient as a function of soil friction angle and the ratio of embedded pile length to pile diameter (dim.)
- \( N_q' \) = Bearing capacity factor
- \( \sigma'_v \) = Effective overburden stress at pile tip (ksf) \( \leq 3.2 \) ksf
- \( q_L \) = Limiting unit tip resistance (ksf)

AASHTO 10.7.3.8.6e
Design Curve for Evaluating $K_\delta$ for Piles where $\phi_f = 25$

Figure 10.7.3.8.6f-1
Design Curve for Evaluating $K_\delta$ for Piles where $\phi_f = 30$

Figure 10.7.3.8.6f-2
Design Curve for Evaluating $K_\delta$ for Piles where $\phi_f = 35$

Figure 10.7.3.8.6f-3
Design Curve for Evaluating $K_\delta$ for Piles where $\phi_f = 40$

Figure 10.7.3.8.6f-4
Correction Factor for $K_\delta$ where $\delta = \phi_f$

Figure 10.7.3.8.6f-5
Relation of $\delta/\phi_f$ and Pile Displacement, $V$, for Various Types of Piles

Figure 10.7.3.8.6f-6
Bearing Capacity Factor, $N'_q$

Figure 10.7.3.8.6f-8
Figure 10.7.3.8.6f-9  Limiting Unit Pile Tip Resistance
Driven Piles - SPT Method (Meyerhof 1976) (Sand)

For displacement piles

\[ q_s = \overline{N_{160}} / 25 \]

For nondisplacement piles

\[ q_s = \overline{N_{160}} / 50 \]

\( \overline{N_{160}} = \) Average corrected SPT-blow count along the pile side (blows/ft)

\[ q_p = \frac{0.8(N_{160})D_b}{D} \leq q_l \]

\( N_{160} = \) Representative SPT blow count near the pile tip corrected for overburden pressure (blows/ft)

\( D = \) Pile width or diameter (ft)

\( D_b = \) Depth of penetration in bearing strata (ft)

\( q_l = \) limiting tip resistance taken as 8\( N_{160} \) for sands and 6\( N_{160} \) for nonplastic silt (ksf)
Driven Piles - CPT Method (Nottingham and Schmertmann 1975) (Sand)

Nominal skin friction resistance

\[
R_s = K_{s,c} \sum_{i=1}^{N_1} \left( \frac{L_i}{8D_i} \right) f_{s_i} a_{s_i} h_i + \sum_{i=1}^{N_2} f_{s_i} a_{s_i} h_i
\]

\(K_{s,c}\) = Correction factors: \(K_c\) for clays and \(K_s\) for sands (dim.)
\(L_i\) = Depth to middle of length interval at the point considered (ft)
\(D_i\) = Pile width or diameter at the point considered (ft)
\(f_{s_i}\) = Unit local sleeve friction resistance from CPT (ksf)
\(a_{s_i}\) = Pile perimeter at the point considered (ft)
\(h_i\) = Length interval at the point considered (ft)
\(N_1\) = Number of intervals between the ground surface and a point 8D below the ground surface
\(N_2\) = Number of intervals between 8D below the ground surface and the tip of the pile
Driven Piles - CPT Method (Nottingham and Schmertmann 1975)

Tip resistance

\[ q_p = \frac{q_{c1} + q_{c2}}{2} \]

- \( q_{c1} \) = average \( q_c \) over a distance of \( yD \) below the pile tip (path a-b-c); sum \( q_c \) values in both the downward (path a-b) and upward (path b-c) direction; use actual \( q_c \) values along path a-b and the minimum path rule along path b-c; compute \( q_{c1} \) for \( y \)-values from 0.7 to 4.0 and use the minimum \( q_{c1} \) value obtained (ksf)

- \( q_{c2} \) = average \( q_c \) over a distance of \( 8D \) above the pile tip (path c-e); use the minimum path rule as for path b-c in the \( q_{c1} \) computations; ignore any minor “x” peak depressions if in sand but include in minimum path if in clay (ksf)
Drilled Shafts in Clay – \( \alpha \) Method

\[ q_s = \alpha S_u \]

\( \alpha = 0.55 \) for \( S_u/p_a \leq 1.5 \)

\( \alpha = 0.55 - 0.1(S_u/p_a - 1.5) \)

for \( 1.5 \leq S_u/p_a \leq 2.5 \)

\( S_u = \) Undrained shear strength \( (\text{ksf}) \)

\( \alpha = \) Adhesion factor \( (\text{dim.}) \)

\( p_a = \) Atmosphere pressure \( (= 2.12 \text{ ksf}) \)

\[ q_p = N_c S_u \leq 80.0 \]

\[ N_c = 6 \left[ 1 + 0.2 \left( \frac{Z}{D} \right) \right] \leq 9 \]

\( D = \) Diameter of shaft \( (\text{ft}) \)

\( Z = \) Penetration of shaft \( (\text{ft}) \)

\( S_u = \) Undrained shear strength \( (\text{ksf}) \)

If \( S_u < 0.5 \text{ ksf} \), use \( 0.67 N_c \)

AASHTO 10.8.3.5.1b

AASHTO 10.8.3.5.1c
Ignore Skin Friction

Explanation of Portions of Drilled Shafts Not Considered in Computing Side Resistance

Figure 10.8.3.5.1b-1

- Per ODOT BDM, do not use Belled Shaft unless consulted with ODOT structures office
Drilled Shafts in Sand – $\beta$ Method

$q_s = \beta \sigma'_v \leq 4.0$

for $0.25 \leq \beta \leq 1.2$

for sandy soils:
for $N_{60} \geq 15$: $\beta = 1.5 - 0.135\sqrt{z}$
for $N_{60} < 15$: $\beta = \frac{N_{60}}{15} (1.5 - 0.135\sqrt{z})$

$\sigma'_v = $ Vertical effective stress at soil layer mid-depth (ksf)
$\beta = $ Load transfer coefficient (dim.)
$z = $ Depth below ground, at soil layer mid-depth (ft)
$N_{60} = $ Average SPT blow count in the design zone under consideration (blows/ft)
$\beta = 2.0-0.06(z)^{0.75} $ for gravel

$q_p = 1.2N_{60}$

for $N_{60} \leq 50$

$q_p = 0.59 \left[ N_{60} \left( \frac{p_a}{\sigma'_v} \right) \right]^{0.8} \sigma'_v$

for $N_{60} > 50$

$p_a = $ Atmosphere pressure (= 2.12 ksf)

AASHTO 10.8.3.5.2b

AASHTO 10.8.3.5.2c
Drilled Shafts in Rock
- Shaft Resistance

\[ q_s = 0.65\alpha_E p_a \left(\frac{q_u}{p_a}\right)^{0.5} < 7.8p_a \left(\frac{f_c'}{p_a}\right)^{0.5} \]

- \( q_u \): Uniaxial compressive strength of rock (ksf)
- \( p_a \): Atmosphere pressure (=2.12 ksf)
- \( \alpha_E \): Reduction factor to account for jointing in rock
- \( f_c' \): Concrete compressive strength (ksi)

<table>
<thead>
<tr>
<th>( E_m/E_i )</th>
<th>( \alpha_E )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>0.5</td>
<td>0.8</td>
</tr>
<tr>
<td>0.3</td>
<td>0.7</td>
</tr>
<tr>
<td>0.1</td>
<td>0.55</td>
</tr>
<tr>
<td>0.05</td>
<td>0.45</td>
</tr>
</tbody>
</table>

AASHTO 10.8.3.5.4

VI-69
Drilled Shafts in Rock – Tip Resistance

\[ q_p = 2.5q_u \]

For the condition that the rock below the base of the drilled shaft to a depth of 2.0B is either intact or tightly jointed, i.e., noncompressive material or gouge-filled seams, and the depth of the socket is greater than 1.5B.

\[ q_p = \left[ \sqrt{s} + \sqrt{(m\sqrt{s} + s)} \right] q_u \]

For the condition that the rock below the base of the drilled shaft to a depth of 2.0B is jointed, and the joints have random orientation.

\( s, m = \text{Fractured rock mass parameters} \)

\( q_u = \text{Unconfined compressive strength of rock (ksf)} \)

AASHTO 10.8.3.5.4c
**Combined Side Friction and End Bearing**

**Figure c10.8.3.5.4d-1**

Use O’Neill and Reese (1999) procedure

\[ R_R = \phi R_n = \phi_{qp} R_p + \phi_{qs} R_s \]

**Figure 10.7.3.8.6f-1**
Consult O’Neill and Reese (1999) for Drilled Shaft Resistance in Intermediate Geo Materials (IGMs)

- Cohesive IGM - clay, shales or mudstone with
  \[ S_u = 5-50 \text{ ksf} \]
- Cohesionless IGM - granular tills or granular residual soils with \( N_{160} > 50 \) blow/ft
# Resistance Factors for Geotechnical Resistance of Drilled Shafts

<table>
<thead>
<tr>
<th>Method/Soil/Condition</th>
<th>Resistance Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Side resistance in clay $\alpha$-method ($O'\text{Neill and Reese, 1999}$)</td>
<td>0.45</td>
</tr>
<tr>
<td>Tip resistance in clay Total Stress ($O'\text{Neill and Reese, 1999}$)</td>
<td>0.40</td>
</tr>
<tr>
<td>Side resistance in sand $\beta$-method ($O'\text{Neill and Reese, 1999}$)</td>
<td>0.55</td>
</tr>
<tr>
<td>Tip resistance in sand $O'\text{Neill and Reese (1999)}$</td>
<td>0.50</td>
</tr>
<tr>
<td>Side resistance in IGMs $O'\text{Neill and Reese (1999)}$</td>
<td>0.60</td>
</tr>
<tr>
<td>Tip resistance in IGMs $O'\text{Neill and Reese (1999)}$</td>
<td>0.55</td>
</tr>
<tr>
<td>Side resistance in rock Horvath and Kenney (1979) $O'\text{Neill and Reese (1999)}$</td>
<td>0.55</td>
</tr>
<tr>
<td>Side resistance in rock Carter and Kulhawy (1988)</td>
<td>0.50</td>
</tr>
<tr>
<td>Block Failure, $\varphi_{bl}$ Clay $O'\text{Neill and Reese (1999)}$</td>
<td>0.55</td>
</tr>
</tbody>
</table>
| Uplift Resistance of Single-Drilled Shafts, $\phi_{up}$ | Clay | $\alpha$-method  
(O'Neill and Reese, 1999) | 0.35 |
|---|---|---|---|
| Sand | $\beta$-method  
(O'Neill and Reese, 1999) | 0.45 |
| Rock | Horvath and Kenney (1979)  
Carter and Kulhawy (1988) | 0.40 |
| Group Uplift Resistance, $\phi_{ug}$ | Sand and clay | 0.45 |
| Horizontal Geotechnical Resistance of Single Shaft or Shaft Group | All materials | 1.0 |
| Static Load Test (compression), $\phi_{load}$ | All Materials | Values in Table 10.5.5.2.3-2, but no greater than 0.70 |
| Static Load Test (uplift), $\phi_{upload}$ | All Materials | 0.60 |
Driven Resistance

Driven Resistance
Wave Equation Results
Unique for assumed

- Hammer Model Selected
- Soil resistance distribution.
- Penetration length
- Smith model parameters
Driven Resistance Factors

- **Concrete piles**, $\phi_{da} = 1.00$
  - *AASHTO* Article 5.5.4.2.1
- **Steel piles**, $\phi_{da} = 1.00$
  - *AASHTO* Article 6.5.4.2
- **Timber piles**, $\phi_{da} = 1.15$
  - *AASHTO* Article 8.5.2.2
Steps to perform drivability analysis:

- Estimate total soil resistance and distribution
- Select hammer
- Model driving system and soil resistance
- Run wave equation analysis
<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>Skin quake</td>
<td>0.1</td>
<td>default per WEAP manual</td>
</tr>
<tr>
<td>Skin damping</td>
<td>0.2</td>
<td>From WEAP manual</td>
</tr>
<tr>
<td>Toe quake</td>
<td>0.1</td>
<td>1/120 of pile width</td>
</tr>
<tr>
<td>Toe damping</td>
<td>0.15</td>
<td>per FHWA NHI-05-042 page 17-68</td>
</tr>
</tbody>
</table>
Identify pile properties (HP12x53)

\[ A_s = 15.5 \text{ in}^2 \]
\[ E_s = 300000 \text{ ksi} \]
\[ \gamma_s = 490 \text{ pcf} \]
Identify Hammer Properties
Use Single Acting Diesel Hammer

Energy = 40,000 ft-lb
Example = ICE 42-S
    Delmag D 19-32
    Pileco D 19-42
Drivability Analysis
(AASHTO 10.7.8)

Driving stress $\sigma_{dr}$ less than the following limits:

- **Steel Piles (Compression & Tension)**
  - $\sigma_{dr} = 0.9 \phi_{da} f_y$
  - $\Phi_{da} = 1.0$
Drivability Analysis (AASHTO 10.7.8)

- Concrete Piles (Compression)
  - \( \sigma_{dr} = \phi_{da} (0.85) f'_c \)

- Concrete Piles (tension)
  - \( \sigma_{dr} = 0.7 \phi_{da} f_y \)
  - \( \phi_{da} = 1.0 \)
Drivability Analysis (AASHTO 10.7.8)

- Timber Piles (compression and tension)
  - $\sigma_{dr} = \phi_{da} (3F_{co})$
  - $\phi_{da} = 1.15$
Other Considerations

- Accounting for Scour
- Accounting for Downdrag
- Accounting for Setup
Scour Considerations
(ODOT BDM 202.2.3.2.h)

- Friction resistance in scour zone shall be neglected.
- Drive piles to a larger ultimate bearing value

\[ R_{ndr} = \frac{Q}{\phi_{dyn}} + R_{nsc} \]

\( Q = \) total factored load (highest loaded pile at each substructure unit)

\( \phi_{dyn} = 0.70 \) (ODOT BDM 202.2.3.2.b)

\( R_{nsc} = \) unfactored skin resistance for the scour zone, estimated by static analysis method.
Downdrag Forces on Piles
(AASHTO 3.11.8)

Figure C3.11.8-1 Common Downdrag Situation Due to Fill Weight (Hannigan, et al. 2005).

Figure C3.11.8-2 Common Downdrag Situation Due to Causes Other than Recent Fill Placement.
Static Methods to estimate downdrag forces

Piles
- Cohesive soils:
  - $\alpha$ or $\lambda$ methods
  - $\beta$ method for long-term conditions
- Sand:
  - Effective stress method

Drilled Shafts
- Cohesive soils:
  - $\alpha$ method
- Granular Soils:
  - $\beta$ method
Pile driven to refusal – add the downdrag forces in the total factored load
Friction piles - the resistance to the extra downdrag force (load) is achieved by driving the piles to a larger ultimate Bearing Value ($R_{ndr}$).
Nominal Pile Driving Resistance Required, $R_{ndr}$

$$\frac{(\Sigma \gamma_i Q_i)}{\varphi_{dyn}} + \gamma_p DD/\varphi_{dyn}$$

Static skin friction component of driving resistance

Total pile resistance during driving

Downdrag Zone

Bearing Zone

Figure C10.7.3.7-1 Design of Pile Foundations for Downdrag.
\[ R_n = \sum \frac{\gamma_i Q_i}{\phi_{dy}} + \frac{\gamma_p DD}{\phi_{dy}} \]

\[ R_{ndr} = R_{sdd} + R_n \]

\( R_{sdd} = \text{skin friction which must be overcome during driving through downdrag zone (kips)} \)
Set-up is not typically accounted for by ODOT.

Contact ODOT Office of Structural Engineering, if consultants or contractors want to consider set up.
Pile Group Resistance
Static Geotechnical Resistance in Clay

Take lesser of
Pile Group in Compression

Efficiency Factor

\[ \eta = \begin{cases} 0.65 & \text{c.c. Spacing} = 2.5 \text{ B} \\ 1.0 & \text{c.c. Spacing} = 6.0 \text{ B} \end{cases} \]

Only when

a) cap is not in firm contact with ground
b) soil at the surface is soft
For clay:
Same as in pile groups

Drilled Shaft Group Resistance

\[ R_{n\text{ group}} = \eta \times R_{n\text{ single}} \]

where:
- \( \eta = 0.65 \) at c-c spacing of 2.5 diameters
- \( \eta = 1.0 \) at c-c spacing of 6 diameters

For cohesionless soils:
use group efficiency factor approach, regardless cap contact
Foundation Redundancy

- $\eta_R = 1.0$ for all foundations
- Non-redundant foundations $\rightarrow$ Reduce Resistance Factor ($\varphi$) by 20%
- Non-redundant Pile Foundations:
  - $\leq 4$ piles per substructure
- Non-redundant Drilled Shaft Foundations:
  - 1 or 2 shafts per substructure
SERVICE LIMIT CHECK
SETTLEMENT COMPUTATION
Axial Response of a Single Element (Approximate method)

\[ \Delta z_{\text{top}} = \Delta z_p + Q_{\text{top}} \frac{L}{(A \cdot E)} \]

Point bearing only

\[ \Delta z_{\text{top}} = \Delta z_p + Q_{\text{top}} \frac{L}{(2 \cdot A \cdot E)} \]

Constant side friction only

\[ \Delta z_{\text{top}} = \Delta z_p + Q_{\text{top}} \frac{L}{(3 \cdot A \cdot E)} \]

Linear increasing friction only

Pile Properties A, E
Settlement of Single Drilled Shaft (AASHTO 10.8.2.2)

Short-Term Settlement

Figure 10.8.2.2.2-1 Normalized Load Transfer in Side Resistance Versus Settlement in Cohesive Soils (from O’Neill and Reese, 1999).
Settlement of Single Drilled Shaft
(AASHTO 10.8.2.2)
Short-Term Settlement

Figure 10.8.2.2.2-2 Normalized Load Transfer in End Bearing Versus Settlement in Cohesive Soils (from O’Neill and Reese, 1999).
Settlement of Single Drilled Shaft
(AASHTO 10.8.2.2)
Short-Term Settlement

Figure 10.8.2.2.2-3 Normalized Load Transfer in Side Resistance Versus Settlement in Cohesionless Soils (from O’Neill and Reese, 1999).
Settlement of Single Drilled Shaft
(AASHTO 10.8.2.2)
Short-Term Settlement

Figure 10.8.2.2.2-4 Normalized Load Transfer in End Bearing Versus Settlement in Cohesionless Soils (from O’Neill and Reese, 1999).
Settlement of pile group
(AASHTO 10.7.2.3)
Equivalent Footing Analogy in cohesive soils

Equivalent Footing at Depth D
Settlement of Pile Group = Compression of Layers $H_1$ and $H_2$ Under Pressure Distribution Shown.

a) Toe Bearing Piles in Hard Clay or in Sand Underlain by Soft Clay
Settlement of pile group
(AASHTO 10.7.2.3)
Equivalent Footing Analogy in cohesive soils

 Equivalent Footing Analogy

Equivalence Footing at Depth 2/3D

Settlement of Pile Group = Compression of Layer H Under Pressure Distribution Shown.

b) Piles Supported by Shaft Resistance in Clay
Settlement of pile group (AASHTO 10.7.2.3)
Equivalent Footing Analogy in cohesive soils

Equivalent Footing at Depth 8/9D
Settlement of Pile Group = Compression of Layers H1, H2, and H3 Under Pressure Distribution Shown.
nQa is Limited by Bearing Capacity of Clay Layers

o) Piles Supported by Shaft Resistance in Sand Underlain by Clay
Settlement of pile group
(AASHTO 10.7.2.3)
Equivalent Footing Analogy in cohesive soils

Equivalent Footing at Depth 2/3D

Settlement of Pile Group = Compression of Layers $H_1$, $H_2$, and $H_3$ Under Pressure Distribution Shown.

d) Piles Supported by Shaft and Toe Resistance in Layered Soil Profile
Settlement of Pile Group in Sand

\[ \rho = \frac{qI \sqrt{B}}{N1_{60}} \quad \text{OR} \quad \rho = \frac{qBI}{2q_c} \]

\[ I = 1 - 0.125 \frac{D'}{B} \geq 0.5 \]

Figure 10.7.2.3.1-2 Location of Equivalent Footing (after Duncan and Buchignani, 1976).
Rigid Cap Model for Pile Group

Computation of Load Effects in Each Pile
Rigid Cap Model

Assumptions:

a. The cap is free to translate and rotate about all axes

b. The cap will not bend

May not apply to:

a. Large pile group

b. Thin cap

c. Widely spaced foundation elements
Distribution of Axial Loads (Simplified)

\[
P_i = \frac{F_z}{n} + \frac{M_x y_i}{\sum_{i=1}^{n} y_i^2} + \frac{M_y x_i}{\sum_{i=1}^{n} x_i^2}
\]
Limitations

a) The axial stiffness of all foundation elements are equal

b) The pile group is symmetrical about at least one of the horizontal axes
Lateral Loads (Equal Displacement)

(AASHTO Article 10.7.3.11)

In a group containing only vertical deep foundation elements, horizontal loads are resisted by bending of the foundation elements. The bending of deep foundation elements is analyzed using the P-Y method.
Due to assumption of a rigid cap, the horizontal deflection at the top of each foundation element in the group will be equal
A deflection is assumed, the horizontal load at the top of each pile can be computed from the results of the individual P-Y analyses.

The loads are totaled and compared to the applied horizontal load.

The deflection is adjusted and the process repeated until the total horizontal resistance supplied by the piles is equal to the applied horizontal load.
Distribution of Horizontal Loads
Horizontal Response

Properties
A, E, I

$H_t$, $Q_t$, $M_t$, $y$

Graph with axes $P$ and $y$
P-y Curve development

Typical required soil parameters:

- $S_u$
- $\phi_f$
- $\gamma$
- $k$
- $\varepsilon_{50}$

$k$ – coefficient of subgrade reaction

$\varepsilon_{50}$ - strain at 50% of ultimate strength
P-y Results for Single Element

1740 k
8000 in-k
10.1 k

Deflection, in.
Moment, in. -kx10^2
Shear, k

0.84
8640
65.5

Depth, ft
Variation of Stiffness (EI)

Stiffness EI (kip-in^2)

Reinforced Concrete Shaft

Moment (in-kip)
Pile Head Fixity

Service Limit State

Moment

Strength Limit State

Moment

VI-121
Group Effects
P-y Interaction Effects

Pile Load Modifiers, $P_m$

<table>
<thead>
<tr>
<th>Pile CTC spacing (in the direction of loading)</th>
<th>Pile Load Modifiers, $P_m$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Row 1</td>
</tr>
<tr>
<td>3B</td>
<td>0.7</td>
</tr>
<tr>
<td>5B</td>
<td>1.0</td>
</tr>
</tbody>
</table>
Explanation of Row definition
Output for multiple loads

<table>
<thead>
<tr>
<th>Load Type</th>
<th>Condition</th>
<th>Boundary Axial Load</th>
<th>Pile-Head Deflection</th>
<th>Maximum Moment</th>
<th>Maximum Shear</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 V= 10000, M= 0.000</td>
<td>36000.0000</td>
<td>.0783522</td>
<td>338542.0000</td>
<td>10000.0000</td>
<td></td>
</tr>
<tr>
<td>1 V= 15000, M= 0.000</td>
<td>36000.0000</td>
<td>.1216378</td>
<td>514228.0000</td>
<td>15000.0000</td>
<td></td>
</tr>
<tr>
<td>1 V= 20000, M= 0.000</td>
<td>36000.0000</td>
<td>.1785663</td>
<td>713956.0000</td>
<td>20000.0000</td>
<td></td>
</tr>
<tr>
<td>1 V= 25000, M= 0.000</td>
<td>36000.0000</td>
<td>.2600870</td>
<td>933068.0000</td>
<td>25000.0000</td>
<td></td>
</tr>
<tr>
<td>1 V= 30000, M= 0.000</td>
<td>36000.0000</td>
<td>.3724842</td>
<td>1149125.0000</td>
<td>30000.0000</td>
<td></td>
</tr>
</tbody>
</table>
Computer P-y Modeling Programs

( SINGLE PILE OR DRILLED SHAFT )

- COM624P
- LPILE
Pile Group Modeling Programs

( Soil Structure Interaction Method )

- GROUP
- FB PIER
General Module Used by Soil Structure Interaction Software

Non-linear Column and Cap Beam (may not be included)
Flexible Membrane or Rigid Pile Cap
Non-linear Pile Material

Non-linear Soil Response Springs
T-z
τ-θ
P-y (& P-x)
Q-z
Nominal Structural Resistance

- Axial Compression
- Flexure Resistance
- Shear Resistance
HP 12 X 53 Driven Pile

Structural Resistance - Axial compression

\[ A_s = 15.5 \text{ in}^2 \text{ (after corrosion loss)} \]

\[ F_y = 50 \text{ ksi} \]

\[ \lambda = 0 \text{ (For fully embedded piles)} \]
\[ P_n = \text{Nominal axial structural resistance} \]

\[ P_n = 0.66 \lambda \sigma_y A_s = 0.66^0(50)(15.5) \]

\[ P_n = 775 \text{ kips} \]

\[ P_r = \text{Factored structural resistance} \]

\[ = \Phi_c P_n = (0.6)(775) = 465 \text{ kips} \]
Nominal Structural Resistance
- Flexure Resistance

\[ z_x = 74 \text{ in}^3 \]
\[ z_y = 32.2 \text{ in}^3 \]
\[ F_y = 50 \text{ ksi} \]

Nominal moment capacity
\[ M_{nx} = F_y Z_x \]
\[ M_{ny} = F_y Z_x \]

\[ M_{nx} = (50 \text{ ksi})(74 \text{ in}^3) = 3700 \text{ k-in} \]
\[ M_{ny} = (50 \text{ ksi})(32.2 \text{ in}^3) = 1610 \text{ k-in} \]
Structural Resistance – Shear Resistance

D = 11.78 in
\(t_w = 0.435\) in
\(F_y = 50\) ksi
C = 1.0

\[V_n = CV_p\]
\[V_p = 0.58F_yw D t_w\]

\[V_p = (0.58)(50\) ksi\)(11.78 in\)(0.435 in)\]

\[V_n = V_p C = 149(1.0) = 149\) kips\]

AASHTO Articles 6.10.7.2-1, 6.10.7.2-2, 6.10.7.3.3a  VI-134
Combined Compression and Flexure

If \( \frac{P_u}{2P_r} \leq 0.2 \), then

\[
\frac{P_u}{2P_r} + \left( \frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}} \right) \leq 1.0
\]

If \( \frac{P_u}{2P_r} \geq 0.2 \), then

\[
\frac{P_u}{P_r} + \frac{8}{9} \left( \frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}} \right) \leq 1.0
\]
\[ P_r = \text{Factored compressive resistance} \]

\[ P_u = \text{Axial compressive load from factored loading} \]

\[ M_{rx}, M_{ry} = \text{The factored flexural resistance about the } x\text{-axis and the } y\text{-axis respectively taken equal to } \Phi_f \text{ times the nominal flexural resistance about the } x\text{-axis and the } y\text{-axis} \]

\[ M_{ux}, M_{uy} = \text{Concurrent moments calculated from factored loading} \]
$P_r = \Phi_c P_n = (0.7)(775) = 543 \text{ kips}$

$M_{rx} = \Phi_f M_{nx} = (1.0)(3700) = 3700\text{k-in}$

$M_{ry} = \Phi_f M_{ny} = (1.0)(1610) = 1610\text{k-in}$

$$\frac{P_u}{P_r} + \frac{8}{9} \left( \frac{M_{ux}}{M_{rx}} + \frac{M_{wy}}{M_{ry}} \right) \leq 1.0$$
Session VII

Guided Design of Driven Pile Group
Step 1: Define Subsurface Condition and Geometric Constrains
### Step 1:

#### Parameters For Design

<table>
<thead>
<tr>
<th>Layer 1</th>
<th>Layer 2-Rock (at Ele.70)</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \gamma_{\text{dry}} ) = (90 pcf)</td>
<td>( \gamma_{\text{avg}} ) = (150.4 pcf)</td>
</tr>
<tr>
<td>( \gamma_{\text{wet}} ) = (110 pcf)</td>
<td>(( q_{u} ))\text{avg} = (11.485 ksi)</td>
</tr>
<tr>
<td>( \gamma' ) = (47.6 pcf)</td>
<td>( \nu ) = 0.2</td>
</tr>
<tr>
<td>( \phi' ) = 31</td>
<td>( E ) = (2125 ksi)</td>
</tr>
<tr>
<td>( \nu ) = 0.25</td>
<td>( G_{0} ) = (885.417 ksi)</td>
</tr>
<tr>
<td>( E ) = (0.833 ksi)</td>
<td>RQD (%) = 90.4</td>
</tr>
<tr>
<td>( G_{0} ) = (0.33 ksi)</td>
<td>Recovery (%) = 98.4</td>
</tr>
<tr>
<td>( K ) = (20 psi)</td>
<td></td>
</tr>
</tbody>
</table>

(Coefficient of variation of subgrade reaction)
Step 2: Determine Applicable Load, and Load Combinations

- Load combination that produces the maximum vertical load on the foundation system-strength I (Max load factor) and Service I load case

- Load combination that produces the maximum overturning on the foundation

- Load combination that produces the maximum lateral load.
Step 3: Factor Loads for Each Combination

Application of Loads
## Step 3:

### Summary of Factored Loads

<table>
<thead>
<tr>
<th>Limit State</th>
<th>Str-I Max/Fin</th>
<th>Str-I Min/Fin</th>
<th>Ser-I Max/Fin</th>
<th>Ser-I Min/Fin</th>
<th>Str-Ill Max/Fin</th>
<th>Ser-I Max/Fin</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum Vertical Load</td>
<td>2253</td>
<td>1791</td>
<td>1791</td>
<td>1860</td>
<td>1815</td>
<td>1791</td>
</tr>
<tr>
<td>Lateral Load (in Trans. Dir.)</td>
<td>7693</td>
<td>4774</td>
<td>4709</td>
<td>7291</td>
<td>6374</td>
<td>4774</td>
</tr>
<tr>
<td>Lateral Load (in Long. Dir.)</td>
<td>0</td>
<td>162</td>
<td>162</td>
<td>0</td>
<td>508</td>
<td>162</td>
</tr>
<tr>
<td>Lateral Load (in Long. Dir.)</td>
<td>855</td>
<td>571</td>
<td>568</td>
<td>855</td>
<td>787</td>
<td>571</td>
</tr>
<tr>
<td>Lateral Load (in Trans. Dir.)</td>
<td>0</td>
<td>10</td>
<td>10</td>
<td>0</td>
<td>37</td>
<td>10</td>
</tr>
</tbody>
</table>
Step 4: Verify Need for a Pile Foundation

Evaluate a spread footing design based on shallow foundation design principles
Step 5: Select Suitable Pile Type and Size

Assumes that drilled shaft foundations have been shown to be more costly than driven pile foundation (granular soil, water table, bearing strata, etc.)
Step 5:

H Pile (12 inch) end bearing on rock is selected

- A low displacement pile to minimize friction in the overlying soils
- Can be driven to high capacity on rock
- Relatively stiff in bending, thus less lateral deflection for comparably sized concrete or timber piles
- Soils are not corrosive
Step 5:

Determine Optimum Pile Size

- **H-Pile range**
  Pile Spacing is controlled by the greater of 30 inches or 2.5 times the pile diameter.
  The minimum spacing for pile sizes 12 inches and under = 30 inches

- **Absolute Minimum Spacing**
  Spacing should be no less than 2.5 D (D=12 inch)
  2.5 D = 30 inch
Step 5:

Determine Optimum Pile Size

- Minimum pile spacing to reduce group effects
  - Min. c-c spacing of 3D or 3 feet in granular soils to optimize group capacity and minimize installation problem

Lateral group effects are controlled by pile spacing in the direction of loading and perpendicular to the direction of loading

  - Perpendicular to the direction of loading = 3D
  - In the direction of loading = 5D or 8D
Determine Optimum Pile Size

- Maximum pile spacing
  Spacing the piles more than 10 feet c-c results in higher bending moments in the pile cap between each pile and negative bending moments over the top of each pile that may result in additional steel reinforcing or thicker pile caps.
  Keep the pile spacing less than 10 feet c-c.
Step 5:

Determine Optimum Pile Size

- Edge Clearance
  Minimum cover: \( \text{Cover}_{\text{min}} = 9 \text{ inch} \)

For a 12 inch pile, minimum distance from edge of footing to center of pile:

\[ \text{Dist}_{\text{min}} = \text{Cover}_{\text{min}} + \frac{D}{2} = 15 \text{ inch} \]
Step 5:

Determine Optimum Pile Size

- Maximum pile cap dimensions
  The length of the pile cap in the direction perpendicular to the centerline (L) is limited to the width of the abutment.

  The width of the pile cap in the direction parallel to the centerline of the bridge (B) can be made wider as required.
Step 5:
Step 5:

\[ L = L_{\text{max}} = 46.875 \text{ ft} \]

\[ B (\text{initial starting point}) = 10.25 \text{ ft} \]

\[ S_B = B - (2)(\text{Edge Distance}) = 7.75 \text{ ft} \]

\[ N_B = 1 \ (\text{Two rows of Piles}) \text{ for 5 diameter spacing} \]

\[ N_L = 5 \text{ to } 14 \ (6 \text{ to } 15 \text{ rows of pile}) \]
Step 5:

Determine maximum axial load acting on piles

Summing the forces in the z-direction and the moments about point B

\[ \Sigma F_z = 0 \]
\[ \Sigma F_z = P_{\text{vert}} - R_{\text{BACK}} - R_{\text{FRONT}} \]
\[ \Sigma M_B = 0 \]
\[ \Sigma M_B = -M_{\text{long}} - \frac{S_B}{2} \cdot P_{\text{vert}} + S_B \cdot R_{\text{FRONT}} \]
Factored Pile Load (Q)

Step 5:

STR I max

\( P_{\text{vert}} = 2253 \text{ k} \)  \( M_{\text{long}} = 7963 \text{k-ft} \)

\( R_{\text{FRONT}} = 2119 \text{ k} \) \( R_{\text{back}} = 134 \text{ k} \)

\( R_{\text{FRONT}} \) (6 piles in front row)  \( = \frac{R_{\text{FRONT}}}{6} = 353.167 \text{ k} \)

\( R_{\text{FRONT}} \) (7 piles in front row)  \( = \frac{R_{\text{FRONT}}}{7} = 302.7 \text{ k} \)

\( R_{\text{FRONT}} \) (15 piles in front row)  \( = \frac{R_{\text{FRONT}}}{15} = 141.267 \text{ k} \)
Step 6: Determine Maximum Factored Structural Resistance ($R_{R_{\text{max}}}$)

Use 7 piles in front row $Q=302.7$ k
Based on structural resistance,
Select $\text{HP 12} \times 53$ ($F_y = 50$ ksi)

$R_{R_{\text{max}}} = 380$ kips
Per ODOT BDM Section 202.2.3.2.a
Step 7: Check Drivability of Pile

Pile drivability is checked using the computer program WEAP (GRLWEAP)

Developing input parameters for WEAP

• Driving lengths of piles
• Distribution and magnitude of side friction
  Skin Friction can be computed using DRIVEN program
• Smith model parameters
• Hammer Model
Step 7:

Wave Analysis Results (For Example)

Refusal pile ultimate capacity

\[ Q_{\text{ult}} = 660 \text{ kips} > 302.7 \text{ kips} \]

O.K.

Refusal driving stress

\[ S_{\text{driving}} = 36 \text{ ksi} < 0.9 \Phi_{\text{da}} \cdot F_y(=45 \text{ ksi}) \]

O.K.
Step 8: Preliminary Pile Layout Based on Factored Loads and Drivability Analysis Results

HP $12 \times 53$

$F_y = 50$ ksi
Step 9: Calculate individual pile loads

\[ P_i = \frac{F_z}{n} + \frac{M_x y_i}{\sum_{i=1}^{n} y_i^2} + \frac{M_y x_i}{\sum_{i=1}^{n} x_i^2} \]
Step 9:

- The maximum compressive load is reasonably close to the factored resistance for the selected pile.
- The tension load is minimized.
- A reasonable layout with respect to axial load.
Step 10: Evaluate lateral loads (Preliminary)

• Looking at the maximum horizontal loads, transverse loads are relatively small and can be ignored for the purposes of this step.

• The maximum longitudinal service load is:
  \[ P_{\text{long}} = 571 \text{ k} \]
  Number of piles, \( N_{\text{pile}} = 14 \)
  Thus, load per pile = 40.8 k

• To handle the anticipated horizontal loads, battering the piles makes more sense. Front row at 3V: 1H battering
Step 11: Evaluate Pile Head Fixity

• The performance of the pile group and the resulting pile stresses are greatly influenced by the degree to which piles are fixed against rotation at the pile head.

• Fixity is provided by the pile cap and depends on:
  - Embedment of the pile into the cap
  - Geometry of the pile group
  - The stiffness of the pile cap
  - Deflection of the pile.

• Typical 25 to 75% fixity
  - Analyzed with 0 and 100% fixity to determine the critical conditions for pile stress (usually 100% fixity) and deflection (0% fixity)
Step 12: Perform Pile Soil Interaction Analysis

Use
Computer Program FB-Pier
or
Computer Program GROUP
## Loads for Each Limit State

<table>
<thead>
<tr>
<th>LIMIT STATE</th>
<th>FB-Pier Load Case</th>
<th>Fz (K)</th>
<th>My (K-FT)</th>
<th>Mx (K-FT)</th>
<th>Fx (K)</th>
<th>Fy (K)</th>
</tr>
</thead>
<tbody>
<tr>
<td>STR-I MAX/FIN</td>
<td>1</td>
<td>751.0</td>
<td>-2564.3</td>
<td>0.0</td>
<td>285.0</td>
<td>0.0</td>
</tr>
<tr>
<td>SER-I MAX/FIN</td>
<td>2</td>
<td>597.0</td>
<td>-1591.3</td>
<td>54.0</td>
<td>190.3</td>
<td>3.3</td>
</tr>
<tr>
<td>STR-I MIN/FIN</td>
<td>3</td>
<td>620.0</td>
<td>-2430.3</td>
<td>0.0</td>
<td>285.0</td>
<td>0.0</td>
</tr>
<tr>
<td>SER-I MIN/FIN</td>
<td>4</td>
<td>597.0</td>
<td>-1569.7</td>
<td>54.0</td>
<td>189.3</td>
<td>3.3</td>
</tr>
<tr>
<td>STR-III MAX/FIN</td>
<td>5</td>
<td>605.0</td>
<td>-2124.7</td>
<td>169.3</td>
<td>262.3</td>
<td>12.3</td>
</tr>
<tr>
<td>SER-I MAX/FIN</td>
<td>6</td>
<td>597.0</td>
<td>-1591.3</td>
<td>54.0</td>
<td>190.3</td>
<td>3.3</td>
</tr>
</tbody>
</table>
### FB-Pier Run Results (Strength Limit State)

<table>
<thead>
<tr>
<th>Run #</th>
<th>Units</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pile head condition</td>
<td>Fixed</td>
<td>Pinned</td>
<td>Fixed</td>
<td>Pinned</td>
<td></td>
</tr>
<tr>
<td>Soil Friction</td>
<td>No</td>
<td>No</td>
<td>Yes</td>
<td>Yes</td>
<td></td>
</tr>
<tr>
<td><strong>Strength Limit State</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Maximum Axial load</td>
<td>Kip</td>
<td>340</td>
<td>332</td>
<td>340</td>
<td>332</td>
</tr>
<tr>
<td>Pile number and LC</td>
<td>Pile 8 LC1</td>
<td>Pile 8 LC1</td>
<td>Pile 8 LC1</td>
<td>Pile 8 LC1</td>
<td></td>
</tr>
<tr>
<td>Maximum Tension</td>
<td>Kip</td>
<td>0.06</td>
<td>1.45</td>
<td>15.3</td>
<td>2.25</td>
</tr>
<tr>
<td>Pile number and LC</td>
<td>Pile 7 LC3</td>
<td>Pile 7 LC3</td>
<td>Pile 1 LC3</td>
<td>Pile 13 LC3</td>
<td></td>
</tr>
<tr>
<td>Max combined load</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Axial</td>
<td>kip</td>
<td>288</td>
<td>289</td>
<td>336</td>
<td>290</td>
</tr>
<tr>
<td>M2</td>
<td>kip-ft</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>M3</td>
<td>kip-ft</td>
<td>107</td>
<td>100</td>
<td>26</td>
<td>97</td>
</tr>
<tr>
<td>Pile number and LC</td>
<td>Pile 8 LC3</td>
<td>Pile 8 LC3</td>
<td>Pile 6 LC1</td>
<td>Pile 8 LC3</td>
<td></td>
</tr>
<tr>
<td>Depth</td>
<td>FT</td>
<td>8</td>
<td>8</td>
<td>0</td>
<td>8</td>
</tr>
<tr>
<td>Max V2</td>
<td>Kips</td>
<td>18.1</td>
<td>18.2</td>
<td>15.9</td>
<td>18.1</td>
</tr>
<tr>
<td>Pile number and LC</td>
<td>Pile 7 LC3</td>
<td>Pile 7 LC3</td>
<td>Pile 7 LC3</td>
<td>Pile 7 LC3</td>
<td></td>
</tr>
<tr>
<td>Max V3</td>
<td>Kips</td>
<td>3.4</td>
<td>3</td>
<td>3.3</td>
<td>3</td>
</tr>
<tr>
<td>Pile number and LC</td>
<td>Pile 2 LC5</td>
<td>Pile 13 LC5</td>
<td>Pile 2 LC5</td>
<td>Pile 13 LC5</td>
<td></td>
</tr>
</tbody>
</table>
### FB-Pier Run Results (Service Limit State)

<table>
<thead>
<tr>
<th>Run #</th>
<th>Units</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pile head condition</td>
<td>Fixed</td>
<td>Pinned</td>
<td>Fixed</td>
<td>Pinned</td>
<td></td>
</tr>
<tr>
<td>Soil Friction</td>
<td>No</td>
<td>No</td>
<td>Yes</td>
<td>Yes</td>
<td></td>
</tr>
</tbody>
</table>

#### Service Limit State

<table>
<thead>
<tr>
<th></th>
<th>Units</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max X Displacement</td>
<td>IN</td>
<td>0.481</td>
<td>0.489</td>
<td>0.46</td>
<td>0.474</td>
</tr>
<tr>
<td>Max Vertical Displacement</td>
<td>IN</td>
<td>0.133</td>
<td>0.122</td>
<td>0.123</td>
<td>0.108</td>
</tr>
<tr>
<td>Load Case</td>
<td>LC6</td>
<td>LC6</td>
<td>LC6</td>
<td>LC6</td>
<td>LC6</td>
</tr>
<tr>
<td>Max Y displacement</td>
<td>IN</td>
<td>0.02</td>
<td>0.053</td>
<td>0.02</td>
<td>0.053</td>
</tr>
<tr>
<td>Load Case</td>
<td>LC6</td>
<td>LC6</td>
<td>LC6</td>
<td>LC6</td>
<td>LC6</td>
</tr>
</tbody>
</table>
Step 12:

View of Model

Beam Seat Elevation
(Displacement Measurement Location)
Step 13: Check Geotechnical Axial Capacity

Max factored axial pile load = 340 k
Max factored tension pile load = 15.3 k
Max factored Shear = 18.2 k

These occurred when the pile was assumed to be fully fixed in the pile cap and when soil friction was considered.
Step 14: Check Structural Axial Capacity
(in lower portion of pile)

Step 15: Check Structural Axial Capacity in
Combined Bending and Axial Load
(upper portion of pile)

Step 16: Check Structural Shear Capacity

Step 17: Check Maximum Horizontal and Vertical
Deflection of Pile Group at Beam Seats
Using Service Load Case

Step 18: Pile Cap Design
Session VIII

LRFD Design of Earth Retaining Walls

Topic 1

Cast-In-Place Gravity and Semi-Gravity Walls

(AASHTO 11.6)
OBJECTIVE

A. Explain basic components and recall typical dimensions for Cast-In-Place Gravity and Semi-Gravity Walls

B. Identify primary specification checks for strength and service limit state
Cast-In-Place Gravity and Semi-Gravity Walls

- Basic Components
- Typical Dimensions
Cast-In-Place Gravity Wall

- Cast in Place (CIP) concrete retaining walls are classified as rigid gravity or semi-gravity walls.
- CIP gravity walls rely on self-weight to resist overturning and sliding.
- CIP gravity walls are generally trapezoidal in shape.
CIP Gravity Wall

Min. Batter 1H:48V

Granular Soil Backfill

Mass Concrete

0.5H to 0.7H
CIP Semi-Gravity Walls

- Cast in Place (CIP) semi-gravity walls rely on self-weight and bending action of the wall stem to resist lateral earth pressures
- CIP semi-gravity walls include cantilever, counterfort, and buttress walls
- Used for earth retaining structures and bridge abutments in fill situations
CIP Cantilever Wall

- Backwall
- Front face
- Back face
- Batter
- Key between successive concrete pours for high walls
- Stem
- Toe
- Base slab, footing or pile cap
- Heel

Key between successive concrete pours for high walls
CIP Counterfort Wall

Front face

Counterfort

Base slab, footing or pile cap
Typical Dimensions
Cantilever Wall

Min. Batter
(1H:48V)

8" min
(12" preferable)

$H/_{10}$ to $H/_{8}$

$H/_{12}$ to $H/_{10}$

$2/_{5}H$ to $3/_{5}H$
Typical Dimensions
Counterfort Wall

$\frac{1}{3}H$ to $\frac{2}{3}H$

8" min
(12" preferable)

1H:48V
Min. Batter

varies

$\frac{H}{12}$

$\frac{2}{5}H$ to $\frac{3}{5}H$
Earth Pressures

- Gravity and Semi-gravity walls designed to support lateral earth pressures and water pressures
- Earth pressures result from:
  - Surcharge loading
  - Backfill and retained soil
  - Compaction (ODOT ignores this)
  - Any others??
- Use theoretical Rankine or Coulomb methods
Lateral Earth Pressure

- At rest earth pressure, $K_o$
- Active earth pressure, $K_a$
- Passive earth pressure, $K_p$

$$\sigma_H = K \sigma_V$$
Earth Pressure

- $K_p > K_o > K_a$
- Movement required to generate passive pressures much greater than for active pressure
Surcharge Loads

- Earth surcharge AASHTO Section 3.11.6.1 and 3.11.6.2
- Live load surcharge AASHTO 3.11.6.4
Water Pressure

- Common to provide positive drainage measures
- If no drainage provided, design for lateral earth pressure and water pressure
Strength Limit State Checks

- **External Stability**
  1. Sliding
  2. Limiting Eccentricity
  3. Bearing Resistance
Service Limit State Checks

- Overall Stability
- Wall settlement
- Lateral displacement
External Failure Mechanisms

- Sliding
- Limiting Eccentricity (Overturning)
- Bearing
Step 1: Unfactored Loads

- **Permanent Loads**
  - Horizontal Earth Pressure (EH)
  - Vertical Earth Pressure (EV)
  - Earth Surcharge (ES)
  - Dead Load of Structures

- **Transient Loads**
  - Live Load Surcharge (LS)
  - Water Load, Stream Pressure (WA)
### Step 2: Load Factors for Factored Loads and Moments

<table>
<thead>
<tr>
<th>Group</th>
<th>$\gamma_{DC}$</th>
<th>$\gamma_{EV}$</th>
<th>$\gamma_{EH}$ (Active)</th>
<th>$\gamma_{ES}$</th>
<th>$\gamma_{LS}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strength Ia</td>
<td>0.90</td>
<td>1.00</td>
<td>1.50</td>
<td>1.50</td>
<td>1.75</td>
</tr>
<tr>
<td>Strength Ib</td>
<td>1.25</td>
<td>1.35</td>
<td>1.50</td>
<td>1.50</td>
<td>1.75</td>
</tr>
<tr>
<td>Service I</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
</tbody>
</table>
Typical Application of Live Load Surcharge

CONVENTIONAL STRUCTURE
Step 3: Check Bearing Resistance

- Calculate nominal bearing resistance, $q_{n}$, based on methods for spread footings and use resistance factor from Table 10.5.5.2.2-1
- Live load surcharge included in sum of vertical forces

$$q_r = \phi q_n \geq \frac{\sum V}{B - 2e}$$
\[
e = \frac{(F_T \cos \beta) h/3 - (F_T \sin \beta) B/2}{V_1 X_{V_1} - V_2 X_{V_2} + W_1 X_{W_1}} \frac{V_1 + V_2 + W_1 + W_2 + F_T \sin \beta}{V_1 + V_2 + W_1 + W_2 + F_T \sin \beta}
\]

• Bearing Stress for walls on Soils
\[ e = \frac{(F_T \cos \beta)h/3 - (F_T \sin \beta)B/2 - V_1 X_{v1} - V_2 X_{v2} + W_1 X_{w1}}{V_1 + V_2 + W_1 + W_2 + F_T \sin \beta} \]

- Bearing Stress for walls on Rocks
## Resistance Factor for Geotechnical Resistance of Shallow Foundations at the Strength Limit State

<table>
<thead>
<tr>
<th>Method/Soil/Condition</th>
<th>Resistance Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Theoretical method <em>(Munfakh et al., 2001)</em>, in clay</td>
<td>0.50</td>
</tr>
<tr>
<td>Theoretical method <em>(Munfakh et al., 2001)</em>, in sand, using CPT</td>
<td>0.50</td>
</tr>
<tr>
<td>Theoretical method <em>(Munfakh et al., 2001)</em>, in sand, using SPT</td>
<td>0.45</td>
</tr>
<tr>
<td>Semi-empirical methods <em>(Meyerhof, 1957)</em>, all soils</td>
<td>0.45</td>
</tr>
<tr>
<td>Footings on rock</td>
<td>0.45</td>
</tr>
<tr>
<td>Plate Load Test</td>
<td>0.55</td>
</tr>
<tr>
<td>Precast concrete placed on sand</td>
<td>0.90</td>
</tr>
<tr>
<td>Cast-in-Place Concrete on sand</td>
<td>0.80</td>
</tr>
<tr>
<td>Cast-in-Place or precast Concrete on Clay</td>
<td>0.85</td>
</tr>
<tr>
<td>Soil on soil</td>
<td>0.90</td>
</tr>
<tr>
<td>Passive earth pressure component of sliding resistance</td>
<td>0.50</td>
</tr>
</tbody>
</table>

*AASHTO 10.5.5.2.2-1*
Step 4: Check Eccentricity

- Calculate for each Load Group
- \( e < B/4 \) is acceptable for foundations on soil
- \( e < 3B/8 \) is acceptable for foundations on rock
- This check replaces overturning calculation used in ASD
Step 5: Check Sliding

- \( R_R = \phi_T R_T \) where \( R_R \) is the factored resistance against sliding
- Neglect passive resistance in front of wall
- Neglect live load surcharge
- Same as spread footing
Step 6: Check Overall Stability

- Overall stability always checked at Service I
Service Limit State
Wall Settlement

- Total settlement affects serviceability
- Settlement may be critical at wall interaction with other structures (approach to a pile-supported abutment)
- $\frac{\Delta Y}{\Delta X}$ (distortion) $\leq 1/500$
Service Limit State
Lateral Wall Displacement

- Lateral deformation usually occurs during construction
- Lateral deformation affected by
  - Wall batter
  - Compaction effort
Session VIII

LRFD Design of Earth Retaining Walls

Topic 2

MSE (Mechanically Stabilized Embankment) Walls

AASHTO (11.10)
OBJECTIVES

A. Explain the basic components and design principles for MSE walls

B. Identify the primary specification checks for strength and service limit states

C. Demonstrate the ability to check an MSE wall design
Principal Components of an MSE Wall
Applications of MSE Walls

- Retaining Wall
- Access Ramp
- Waterfront Structure
- Bridge Abutment
Wall Types
Connections
Primary Specification Checks for Strength and Service Limit States
Strength Limit States for MSE Walls

- External Stability
  1. Sliding
  2. Limiting Eccentricity
  3. Bearing Resistance
External Failure Mechanisms

- Sliding
- Limiting Eccentricity (Overturning)
- Bearing
Strength Limit States for MSE Walls

- Internal Stability
  1. Tensile Resistance of Reinforcement
  2. Pullout Resistance of Reinforcement
  3. Structural Resistance of Face Elements
  4. Structural Resistance of Face Element Connection

- All Internal Stability Checks are Done by Wall Suppliers
Service Limit States for MSE Walls

1. Overall Stability
2. Wall Settlement
3. Lateral Displacement
# Resistance Factor for Geotechnical Resistance of Shallow Foundations at the Strength Limit State

<table>
<thead>
<tr>
<th>Method/Soil/Condition</th>
<th>Resistance Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Theoretical method (<em>Munfakh et al.</em>, 2001), in clay</td>
<td>0.50</td>
</tr>
<tr>
<td>Theoretical method (<em>Munfakh et al.</em>, 2001), in sand, using CPT</td>
<td>0.50</td>
</tr>
<tr>
<td>Theoretical method (<em>Munfakh et al.</em>, 2001), in sand, using SPT</td>
<td>0.45</td>
</tr>
<tr>
<td>Semi-empirical methods (<em>Meyerhof</em>, 1957), all soils</td>
<td>0.45</td>
</tr>
<tr>
<td>Footings on rock</td>
<td>0.45</td>
</tr>
<tr>
<td>Plate Load Test</td>
<td>0.55</td>
</tr>
<tr>
<td>Precast concrete placed on sand</td>
<td>0.90</td>
</tr>
<tr>
<td>Cast-in-Place Concrete on sand</td>
<td>0.80</td>
</tr>
<tr>
<td>Cast-in-Place or precast Concrete on Clay</td>
<td>0.85</td>
</tr>
<tr>
<td>Soil on soil</td>
<td>0.90</td>
</tr>
<tr>
<td>Passive earth pressure component of sliding resistance</td>
<td>0.50</td>
</tr>
</tbody>
</table>

*AASHTO 10.5.5.2.2-1*
Step 1: Unfactored Loads

- Estimate reinforcement length
- Determine earth pressures and surcharges
- Determine unfactored loads and moments
MSE Wall Element Dimensions Needed for Design
Preliminary Sizing of Reinforcement

- Minimum reinforcement length
  - $> 0.7H$ or 8 ft.

- Sloping fill or surcharges
  - use $0.8H$ to $1.1H$
• ODOT MSE Wall Detail

• Consult ODOT BDM for specific design parameters
Active Earth Pressure

- Vertical Walls & Horizontal Backslope
  \[ k_a = \tan^2 \left[ 45 - \frac{\phi'_f}{2} \right] \]

- Vertical Walls with Sloping Backfill Surface
  \[ K_a = \frac{\sin^2 (\theta + \phi')}{\Gamma \sin^2 \theta \sin (\theta - \delta)} \]
  \[ \Gamma = 1 + \sqrt{\frac{\sin (\phi' + \delta) \sin (\phi' - \beta)}{\sin (\theta - \delta) \sin (\theta + \beta)}}^2 \]
Earth Pressure Distribution for MSE Wall with Level Backfill Surface

AASHTO Fig. 3.11.5.8.1-1
Earth Pressure for MSE Wall with Sloping Backfill Surface
Earth Pressure Distribution for MSE Wall with Broken Back
Backfill Surface

AASHTO Fig. 3.11.5.8.1-3
## Step 2: Load Factors for Factored Loads and Moments

<table>
<thead>
<tr>
<th>GROUP</th>
<th>$\gamma_{EV}$</th>
<th>$\gamma_{EH}$ (Active)</th>
<th>$\gamma_{ES}$</th>
<th>$\gamma_{LS}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strength – Ia</td>
<td>1.00</td>
<td>1.5</td>
<td>1.5</td>
<td>1.75</td>
</tr>
<tr>
<td>Strength – Ib</td>
<td>1.35</td>
<td>1.5</td>
<td>1.5</td>
<td>1.75</td>
</tr>
<tr>
<td>Service – I</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
</tbody>
</table>
Pressure Diagram for External Stability Analysis

Horizontal Backslope with Traffic Surcharge

Assumed for bearing resistance

Assumed for sliding and eccentricity

\[ F_1 = \frac{1}{2} \gamma f H^2 K_{af} \]

\[ F_2 = qHK_{af} \]

where:
- \( e \): Eccentricity
- \( R \): Resultant of vertical forces \((V_i + qL)\)
- \( K_{af} \): Factor of safety

\[ K_{af} = \frac{1 - \sin \phi_f}{1 + \sin \phi_f} \]
Pressure Diagram for External Stability Analysis

Sloping Backslope

\[ V_2 = \gamma_t L (h-H) / 2 \]

\[ F_V = L \gamma_t h^2 K_{st} / 2 \]

\[ \delta = \beta \]

\[ F_{H} \]

\[ \phi, \gamma_t, K_{st} \]

\[ V_1 = \gamma_s H L \]

\[ h, H, L, B \]
Step 3: Check Eccentricity

- Bearing Resistance
  - Equivalent footing; width = length of reinforcing elements at foundation level
- Triangular pressure distribution cannot develop at base of wall due to flexibility of MSE walls
Step 3: Check Eccentricity

- Location of resultant from toe of wall
  \[ X_o = \frac{(M_{EV} - M_{HTOT})}{P_{EV}} \]

- Eccentricity can be calculated for each load group
  \[ e = \frac{B}{2} - X_o \]
Step 4: Check Sliding

- Factored Resistance against sliding
  \[ R_R = \phi_T R_T \quad \phi_T = 0.9 \]

- Neglect potential passive resistance in front of wall
Step 4: Check Sliding

- Discontinuous reinforcement: minimum $\phi'$ from reinforced backfill and foundation soils
- Continuous reinforcement: same as above and minimum interface friction between reinforcement and soil
Step 5: Check Bearing Resistance

- Nominal bearing resistance

\[ q_R = \phi q_n \geq \frac{P_{EV} + P_{LSV}}{B - 2e} \]

- Include effects of live load surcharges
Step 6: Check Overall Stability

- Limit equilibrium procedures
- Reinforced soil mass – rigid body
- Failure surfaces
- Check at Service I
- Check not included in designs performed by MSE wall suppliers
- Long-term water pressures
Overall and Compound Stability of Complex MES Wall Systems
Service Limit States – Wall Settlement

- Total settlement affects serviceability
- Differential settlement at face generates stress and bending in connection
- Settlement may be critical at wall interaction with other structures
- Use conventional settlement analysis
Limiting Differential Settlements

MSE walls with Panel Size Less than 30 ft$^2$

<table>
<thead>
<tr>
<th>Joint Width (in.)</th>
<th>Limiting Differential Settlement</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.75</td>
<td>1/100</td>
</tr>
</tbody>
</table>
Lateral Wall Displacement

- Usually occurs during construction
- Differential movements along base and lateral wall movements
- Greater displacement with extensible reinforcements
- Cantilever type movements because walls are built from bottom up
Lateral Wall Displacement

Lateral Deformations Affected by:

- Compaction effort
- Reinforcement extensibility
- Length of reinforcement
- Connection details
- Details of wall facing
Session VIII

LRFD Design of Earth Retaining Walls

Topic 3

Anchored Walls

AASHTO (11.9)
OBJECTIVE

A. Explain the basic components of an anchored wall

B. Identify specification checks that must be satisfied for strength and service limit states

C. Demonstrate the ability to evaluate anchor bond length and check for tensile breakage of anchor tendon
Anchored Wall Nomenclature and Anchor Embedment Guidelines
Strength Limit States for Anchored Walls

a. Tensile resistance of tendon steel
b. Ground anchor pullout
c. Flexural resistance of vertical wall element, lagging and permanent facing
d. Passive resistance of vertical wall element
e. Bearing resistance of vertical wall element
Failure Mechanisms for Anchored Walls

a.) Tensile failure of tendon

b.) Pullout failure of grout/grout bond
Failure Mechanisms for Anchored Walls

c.) Failure of wall in bending
d.) Failure of wall due to insufficient passive resistance
Failure Mechanisms for Anchored Walls

e.) Failure due to insufficient axial resistance

f.) Rotational failure of ground mass (Service Limit)
Service Limit States for Anchored Walls

1. Ground surface settlement
2. Lateral wall movement
3. Overall stability
Anchored Wall Design Steps

1. Project requirements and feasibility
2. Subsurface profile and soil/rock parameters
3. Corrosion protection requirements
4. Unfactored total lateral pressure envelope (soil, water, surcharge)
5. Unfactored loads and subgrade reaction
Anchored Wall Design Steps

6. Factored loads for all applicable limit states
7. Anchor inclination and spacing
8. Select tendon type and check tensile resistance
9. Evaluate bond length
10. Flexural resistance of wall
Anchored Wall Design Steps

11. Bearing resistance of wall
12. Passive resistance of wall
13. Overall stability
14. Design timber lagging and permanent facing (if appropriate)
15. Check wall movements at Service Limit State I
Step 1: Project Requirements and Feasibility

Step 2: Subsurface Profile and Soil/Rock Parameters
Step 3: Corrosion Protection

In accordance with AASHTO LRFD Bridge Construction Specifications, Section 6, “Ground Anchors”.

- **Permanent Anchors**
  - Protect free length
  - Protect anchor head

- **Routine Applications**
  - Grease and sheath in free length
  - Only grout in bond length
Step 4: Earth Pressure Diagrams for Anchored Walls

- For “flexible” walls, use apparent earth pressure (AEP) diagrams
- For “stiff” walls, use active or at-rest triangular Rankine
- Many anchored highway walls are flexible walls
Step 5: Calculation of Unfactored Horizontal Loads - One Level

Tributary area method
\[ T_1 = \text{Load over length } H_1 + \frac{H_2}{2} \]
\[ R = \text{Load over length } \frac{H_2}{2} \]

Hinge method
\[ T_1 \text{ Calculated from } \Sigma M_C = 0 \]
\[ R = \text{Total earth pressure} - T_1 \]
Calculation of Unfactored Horizontal Loads – Multi-Level

Tributary Area Method
- $T_1$ from $\Sigma M_c = 0$
- $T_{2u} = ABCGF - T_1$
- $T_{2L} = \Sigma M_D$
- $T_2 = T_{2u} = T_{2L}$
Apparent Earth Pressure Diagram cohesionless soils

\[ P_a = k_a \gamma' s H \]

(a) Wall with one level of ground anchors

\[ P_a = \frac{k_a \gamma' s H^2}{1.5H - 0.5H_1 - 0.5H_{n+1}} \]

(b) Walls with multiple levels of ground anchors
Apparent Earth Pressure (cohesionless soils)

• For Walls with one anchor level:

\[ P_a = k_a \gamma'_s H \]

\[ k_a = \tan^2(45 - \phi_f / 2) \]

• For Walls with Multiple anchor levels:

\[ P_a = \frac{k_a \gamma'_s H^2}{1.5H - 0.5H_1 - 0.5H_{n+1}} \]
The apparent earth Pressure distribution for cohesive soils is related to the stability number, $N_s$:

$$N_s = \frac{\gamma_s H}{S_u}$$

; H is the total excavation depth
Stiff To Hard:

For temporarily anchored walls in stiff to hard cohesive soils \((N_s \leq 4)\):

\[
P_a = 0.2 \gamma_s H \text{ to } 0.4 \gamma_s H
\]

- \(k_a\) may be used based on drained friction angle of cohesive soils
(a) Wall with one level of ground anchors

(b) Walls with multiple levels of ground anchors
Soft to Medium Stiff:

The earth pressure on temporarily or permanent walls is soft to medium stiff cohesive soils \( (N_s \geq 6) \):

\[
P_a = k_a \gamma_s H
\]

\[
k_a = 1 - \frac{4S_u}{\gamma_s H} + 2\sqrt{2} \frac{d}{H} \left( 1 - \frac{5.14S_{ub}}{\gamma_s H} \right) \geq 0.22
\]
Apparent Earth Pressure Diagram cohesive soils

AASHTO Figure
3.11.5.7.2b-1

Soft to Medium Stiff Cohesive Soils
Step 6: Load Combinations and Load Factors

- Strength-I and Service-I
- Extreme Event-I (for seismic)
- Typical loads include horizontal earth pressure (EH), earth pressure surcharge loads (ES), and live load surcharge (LS)
Calculation of Anchor Load

- Calculate additional lateral force due to surcharges
- Calculate factored horizontal anchor forces and factored subgrade reaction force
- Resolve factored horizontal anchor forces to factored anchor force, $T_n$, (for tendon selection) and factored vertical anchor force (for axial resistance evaluation)
Step 7: Anchor Length Criteria

- Bond length: 0.2 H (min.)
- Minimum free length: 15 ft
- Or 5 ft
- $S_v = 8$ to 12 ft (commonly used)
- $45^\circ - \phi/2$
Step 8. Select Tendon/Check Tensile Breakage

- Guaranteed Ultimate Tensile Strength (GUTS)

- Select tendon with:

\[
\text{GUTS} \geq \frac{T_n}{\phi} \\
\text{GUTS} > \frac{\Sigma \gamma_i Q_i}{\phi}
\]
Resistance Factors for Ground Anchors – Tensile Rupture

<table>
<thead>
<tr>
<th>Material</th>
<th>Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mild Steel</td>
<td>0.90</td>
</tr>
<tr>
<td>High Strength Steel</td>
<td>0.80</td>
</tr>
</tbody>
</table>

- For high strength steel, apply resistance factor to Guaranteed Ultimate Tensile Strength (GUTS).
- For mild steel, apply $\phi$ to $F_y$.

AASHTO 11.5.6-1
Step 9: Anchor Bond Length

\[ L_{b\text{(min)}} = \frac{T_n}{\phi Q_a} \]

- \( L_b \) = anchor bond length
- \( T_n \) = factored anchor load
- \( Q_a \) = nominal anchor pullout resistance per unit bond length
Nominal Anchor Pullout Resistance Per Unit Bond Length

\[ Q_a = \pi \times d \times \tau_a \]

- \( Q_a \) = nominal anchor pullout capacity per unit bond length
- \( d \) = anchor hole diameter
- \( \tau_a \) = nominal anchor bond stress
Preliminary Evaluation Only

- Bond stress values in AASHTO should be used for FEASIBILITY evaluation
- AASHTO values for cohesionless and cohesive soil and rock
- Actual bond stress verified in the field through load testing every anchor to at least one times the factored anchor load
Contract documents shall require that verification tests or pullout tests on sacrificial anchors in each soil unit be conducted to establish anchor length and capacity consistent with contractor’s method of anchor installation.

Conduct proof tests on every production anchor to the one times the factored load.
Anchor Load Test

(1) Proof Load Tests
(2) Performance Load Tests
(3) Creep Tests
Presumptive Ultimate Unit Bond Stress for Anchors in Cohesive Soils

AASHTO Table C11.9.4.2-1

<table>
<thead>
<tr>
<th>Anchor/Soil Type (Grout Pressure)</th>
<th>Soil Stiffness or Unconfined Compressive Strength (tsf)</th>
<th>Presumptive Ultimate Unit Bond Stress, $\tau_u$ (ksf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gravity Grouted Anchors (&lt;50 psi)</td>
<td>Stiff to Very Stiff 1.0–4.0</td>
<td>0.6 to 1.5</td>
</tr>
<tr>
<td>Silt-Clay Mixtures</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pressure Grouted Anchors (50 psi–400 psi)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>High Plasticity Clay</td>
<td>Stiff 1.0–2.5 V. Stiff 2.5–4.0</td>
<td>0.6 to 2 V. 1.5 to 3.6</td>
</tr>
<tr>
<td>Medium Plasticity Clay</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Medium Plasticity Sandy Silt</td>
<td>V. Stiff 2.5–4.0</td>
<td>2.0 to 5.2 2.9 to 7.3</td>
</tr>
<tr>
<td>Medium Plasticity Sandy Silt</td>
<td></td>
<td>5.8 to 7.9</td>
</tr>
</tbody>
</table>
Presumptive Ultimate Unit Bond Stress for Anchors in Cohesionless Soils

<table>
<thead>
<tr>
<th>Anchor/Soil Type (Grout Pressure)</th>
<th>Soil Compactness or SPT Resistance$^{(1)}$</th>
<th>Presumptive Ultimate Unit Bond Stress, $\tau_u$ (ksf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gravity Grouted Anchors (&lt;50 psi)</td>
<td>Medium Dense to Dense 11–50</td>
<td>1.5 to 2.9</td>
</tr>
<tr>
<td>Sand or Sand-Gravel Mixtures</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pressure Grouted Anchors (50 psi–400 psi)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fine to Medium Sand</td>
<td>Medium Dense to Dense 11–50</td>
<td>1.7 to 7.9</td>
</tr>
<tr>
<td>Medium to Coarse Sand w/ Gravel</td>
<td>Medium Dense 11–30</td>
<td>2.3 to 14</td>
</tr>
<tr>
<td></td>
<td>Dense to Very Dense 30–50</td>
<td>5.2 to 20</td>
</tr>
<tr>
<td>Silty Sands</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sandy Gravel</td>
<td>Medium Dense to Dense 11–40</td>
<td>4.4 to 29</td>
</tr>
<tr>
<td></td>
<td>Dense to Very Dense 40–50+</td>
<td>5.8 to 29</td>
</tr>
<tr>
<td>Glacial Till</td>
<td>Dense 31–50</td>
<td>6.3 to 11</td>
</tr>
</tbody>
</table>

AASHTO Table C11.9.4.2-2
### Presumptive Ultimate Unit Bond Stress for Anchors in Rocks

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>Presumptive Ultimate Unit Bond Stress, $\tau_n$ (ksf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Granite or Basalt</td>
<td>36 to 65</td>
</tr>
<tr>
<td>Dolomitic Limestone</td>
<td>29 to 44</td>
</tr>
<tr>
<td>Soft Limestone</td>
<td>21 to 29</td>
</tr>
<tr>
<td>Slates &amp; Hard Shales</td>
<td>17 to 29</td>
</tr>
<tr>
<td>Sandstones</td>
<td>17 to 36</td>
</tr>
<tr>
<td>Weathered Sandstones</td>
<td>15 to 17</td>
</tr>
<tr>
<td>Soft Shales</td>
<td>4.2 to 17</td>
</tr>
</tbody>
</table>
## Resistance Factors – Anchor Pullout

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cohesionless (Granular) Soils</td>
<td>0.65</td>
</tr>
<tr>
<td>Cohesive Soils</td>
<td>0.70</td>
</tr>
<tr>
<td>Rock</td>
<td>0.50</td>
</tr>
<tr>
<td>Where Proof Tests Preformed</td>
<td>1.00</td>
</tr>
</tbody>
</table>

1) Using presumptive values for preliminary design only

2) Where proof tests conducted to at least 1.0 times the factored anchor load

AASHTO Table 11.5.6-1
Plans need to show minimum unbonded length
Strength Limit States for Vertical Wall Element

- Flexural Resistance of Wall
- Bearing Resistance of Wall
- Passive Resistance of Wall
Step 10: Flexural Structural Resistance of Wall

- Calculate bending moments
- Use factored vertical loads and maximum factored bending moment
- AASHTO 6.9.2.2 Equations 6.9.2.2-1 and -2 (Combined Axial Compression and Flexure)
- Resistance factor for axial compression

(see AASHTO 6.5.4.2)
For combined axial and flexural resistance of undamaged pile

- Axial resistance for H-piles \( \varphi_c = 0.70 \)
- Axial resistance for pipe piles \( \varphi_c = 0.80 \)
- Flexural resistance \( \varphi_f = 1.00 \)

*AASHTO 6.5.4.2.*

- Indicated values of \( \varphi_c \) and \( \varphi_f \) for combined axial and flexural resistance are for use in interaction equations in Article 6.9.2.2.
Wall Bending Moments

Walls with single level of ground anchors

Walls with multiple levels of ground anchors
Step 11: Bearing Resistance of Vertical Wall Element

- Factored axial loads are carried by portion of wall below excavation grade
- Use $\gamma_p = 1.25$ for dead loads
- Use LRFD-based design analyses for static capacity of driven piles or drilled shafts
- Resistance factors specific to walls need to refer to AASHTO Article 10.5
Step 12: Passive Resistance of Vertical Wall Element

- Required embedment based on LRFD > ASD since load factor for surcharge is 1.5
- $\phi = 1.0$ for passive resistance of vertical elements
Broms Method

**Sand**

Pile width = b

1.5 b

3K_p \sigma_v b

9S_u b

**Clay**

Pile width = b

D

D

D
# Resistance Factors for Permanent Retaining Walls

<table>
<thead>
<tr>
<th>WALL-TYPE AND CONDITION</th>
<th>RESISTANCE FACTOR</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nongravity Cantilevered and Anchored Walls</td>
<td>Article 10.5 applies</td>
</tr>
<tr>
<td>Bearing resistance of vertical elements</td>
<td>0.75</td>
</tr>
<tr>
<td>Passive resistance of vertical elements</td>
<td>0.65 (1)</td>
</tr>
<tr>
<td>Pullout resistance of anchors (1)</td>
<td>0.70 (1)</td>
</tr>
<tr>
<td>Pullout resistance of anchors (2)</td>
<td>0.50 (1)</td>
</tr>
<tr>
<td>Pullout resistance of anchors (2)</td>
<td>1.0 (2)</td>
</tr>
<tr>
<td>Tensile resistance of anchor tendon</td>
<td>0.90 (3)</td>
</tr>
<tr>
<td>Flexural capacity of vertical elements</td>
<td>0.80 (3)</td>
</tr>
<tr>
<td>Tensile resistance of anchor tendon</td>
<td>0.90</td>
</tr>
</tbody>
</table>

*Note: AASHTO Table 11.5.6-1*
Step 13: Overall Stability

Dotted line denotes potential failure surface for external stability analysis.
Step 14: Timber Lagging

- Do not “design” temporary timber lagging, select from experience or use FHWA-RD-75-130 Table 6.3.3b
<table>
<thead>
<tr>
<th>Soil Description</th>
<th>Unified Soil Classification</th>
<th>Depth (ft)</th>
<th>5 ft</th>
<th>6 ft</th>
<th>7 ft</th>
<th>8 ft</th>
<th>9 ft</th>
<th>10 ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>Silt or fine sand and silt above water table</td>
<td>ML, SM-ML</td>
<td>0 - 25</td>
<td>2 in</td>
<td>3 in</td>
<td>3 in</td>
<td>3 in</td>
<td>4 in</td>
<td>4 in</td>
</tr>
<tr>
<td>Sands and gravels (medium dense to dense)</td>
<td>GW, GP, GM, GS, SW, SP, SM</td>
<td>0 - 25</td>
<td>2 in</td>
<td>3 in</td>
<td>3 in</td>
<td>3 in</td>
<td>4 in</td>
<td>4 in</td>
</tr>
<tr>
<td>Clays (stiff to very stiff); non-fissured</td>
<td>CL, CH</td>
<td>25 - 60</td>
<td>3 in</td>
<td>3 in</td>
<td>3 in</td>
<td>4 in</td>
<td>4 in</td>
<td>5 in</td>
</tr>
<tr>
<td>Clays, medium consistency and ( \frac{y_H}{S_u} &lt; 5 )</td>
<td>CL, CH</td>
<td>25 - 60</td>
<td>3 in</td>
<td>3 in</td>
<td>3 in</td>
<td>4 in</td>
<td>4 in</td>
<td>5 in</td>
</tr>
<tr>
<td>Sand and silty sand (loose)</td>
<td>SW, SP, SM</td>
<td>0 - 25</td>
<td>3 in</td>
<td>3 in</td>
<td>3 in</td>
<td>4 in</td>
<td>4 in</td>
<td>5 in</td>
</tr>
<tr>
<td>Clayey sands (medium dense to dense) below water table</td>
<td>SC</td>
<td>0 - 25</td>
<td>3 in</td>
<td>3 in</td>
<td>3 in</td>
<td>4 in</td>
<td>4 in</td>
<td>5 in</td>
</tr>
<tr>
<td>Clay, heavily overconsolidated, fissured</td>
<td>CL, CH</td>
<td>25 - 60</td>
<td>3 in</td>
<td>3 in</td>
<td>4 in</td>
<td>4 in</td>
<td>5 in</td>
<td>5 in</td>
</tr>
<tr>
<td>Cohesionless silt or fine sand and silt below water table</td>
<td>ML, SM-SL</td>
<td>25 - 60</td>
<td>3 in</td>
<td>3 in</td>
<td>4 in</td>
<td>4 in</td>
<td>5 in</td>
<td>5 in</td>
</tr>
</tbody>
</table>
Recommended Thickness of Temporary Timber Lagging (After FHWA-RD-75-130-1976). Continue

<table>
<thead>
<tr>
<th>POTENTIALLY DANGEROUS SOILS</th>
<th>CL, CH 0 – 15</th>
<th>3 in</th>
<th>3 in</th>
<th>4 in</th>
<th>5 in</th>
<th>6 in</th>
<th>8 in</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soft clays $\gamma H &gt; 5 \frac{S_i}{S_a}$</td>
<td>ML 15 – 25</td>
<td>3 in</td>
<td>4 in</td>
<td>5 in</td>
<td>6 in</td>
<td>7 in</td>
<td>8 in</td>
</tr>
<tr>
<td>Slightly plastic silts below water table</td>
<td>SC 25 – 35</td>
<td>4 in</td>
<td>5 in</td>
<td>6 in</td>
<td>7 in</td>
<td>8 in</td>
<td>8 in</td>
</tr>
<tr>
<td>Clayey Sands (loose), below water table</td>
<td>SC 25 – 35</td>
<td>4 in</td>
<td>5 in</td>
<td>6 in</td>
<td>7 in</td>
<td>8 in</td>
<td>8 in</td>
</tr>
</tbody>
</table>

Notes:
1) In the category of "potentially dangerous soils", use of soldier beam and lagging wall systems is questionable.
2) The values shown are based on construction grade lumber.

Local experience may take precedence over recommended values in this table.
Step 15: Service Limit States for Anchored Walls

- Overall stability (already checked)
- Ground surface settlement
- Lateral wall movement
Settlement Profile

AASHTO Fig. C11.9.3.1

Curve I – Sand
Curve II – Stiff Clay
Curves III and IV – Soft to Medium Clay
Maximum Lateral Wall Movement

Graph showing the relationship between maximum lateral wall movement and depth of excavation for different types of walls:

- Soldier Pile & Log or Sheetpiles
- Diaphragm Walls
- Soil Nail Walls
- Drilled Pier Walls
- Soil Cement Walls

The graph plots the maximum lateral wall movement ($s_{H_{m}}$ in mm) against the depth of excavation ($H$, in m), with trend lines indicating the percentage of movement at $0.2\%$ and $0.5\%$.
Appendix
Internal Stability

At each level of reinforcement:

- Calculate maximum load, $T_{\text{max}}$
- Maximum load is compared to available pullout resistance and to the tensile resistance of the reinforcement
Step 1: Critical Failure Surface
Extensible Reinforcement

Zone of maximum stress or potential failure surface

For vertical face

$$\psi = 45 + \frac{\phi}{2}$$
Step 1: Critical Failure Surface

Inextensible Reinforcement

Zone of maximum stress or potential failure surface

\[ H_1 = H + \frac{\tan \beta \times 0.3H}{1 - 0.3\tan \beta} \]

* If wall face is battered, an offset of 0.3\(H_1\) is still required, and the upper portion of the zone of maximum stress should be parallel to the wall face.
Step 2: Factored Horizontal Stresses

- Factored Horizontal Stress

\[ \sigma_H = \gamma_P (\sigma_V k_r + \Delta\sigma_H) \]

- \( \gamma_P \) = load factor
- \( k_r \) = pressure coefficient
- \( \sigma_V \) = pressure due to resultant of gravity forces from soil self weight
- \( \Delta\sigma_H \) = horizontal stress
Step 2: Factored Horizontal Stresses

- Determine $K_a = \tan^2(45 - \phi/2)$
- Evaluate $K_r/K_a$ at each level
- Evaluate $K_r$ at each level
- Evaluate $\sigma_H$ at each level
- Use Figure 11.10.6.2.1-3 AASHTO
  (Coming slide)
Variation of the coefficient of lateral stress ratio $K_r/K_a$ with depth in a MSE

*Does not apply to polymer strip reinforcement*
Step 3: Maximum Tensile Force in Reinforcement

\[ T_{\text{max}} = \sigma_H S_v \]

- Per unit width of wall
- \( \sigma_H = \) factored horizontal soil stress at reinforcement (ksf)
- \( S_v = \) vertical spacing of reinforcement
- Assume factored tensile load at reinforcement to wall connection

AASHTO 11.10.6.2.1-2
Step 4: Reinforcement Pullout Resistance

- Length of reinforcement in resisting zone, $L_e$

$$L_e \geq \frac{T_{\text{max}}}{\phi F^* \alpha \sigma_v C R_c}$$

- $\phi$ = resistance factor
- $F^*$ = pullout friction factor
- $\alpha$ = scale correction factor
- $C = 2$
- $R_c$ = reinforcement coverage ratio
Step 5: Reinforcement Tensile Resistance

\[ T_{\text{max}} \leq \phi \ T_{al} \ R_c \]

- \( T_{al} = \text{Nominal long-term reinforcement design strength} \)
- \( \phi = \text{Resistance factor for tensile resistance} \)
## Resistance Factors for Tensile Resistance

<table>
<thead>
<tr>
<th>Reinforcement Type</th>
<th>Static Loading</th>
<th>Combined Static/Earthquake Loading</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Metallic Reinforcement</strong></td>
<td>0.75</td>
<td>1.00</td>
</tr>
<tr>
<td><strong>Strip Reinforcement</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Static loading</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Combined static/earthquake</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Grid Reinforcement</strong></td>
<td>0.65</td>
<td>0.85</td>
</tr>
<tr>
<td><strong>Static loading</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Combined static/earthquake</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Geosynthetic Reinforcement</strong></td>
<td>0.90</td>
<td>1.20</td>
</tr>
<tr>
<td>• Static loading</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Combined static/earthquake</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
T_{al} for Inextensible Reinforcement

\[ T_{al} = \frac{A_c F_y}{b} \]

- Metallic reinforcement
- \( F_y \) = Minimum yield strength of steel
- \( A_c \) = Design cross section area corrected for corrosion loss
- \( b \) = unit width of sheet/grid/bar mat

AASHTO 11.10.6.4.3a-1
Design Cross Section Area - Strips

\[ A_c = bt_c = b(t_n - t_s) \]

- \( t_c \) = Thickness at end of design life
- \( t_n \) = Thickness at end of construction
- \( t_s \) = Sacrificial thickness due to corrosion

AASHTO 11.10.6.4.1a
Design Cross Section Area - Bars

\[ A_c = N_b \left( \frac{\pi D^*^2}{4} \right) \]

- \( A_c \) = Cross section of reinforcement
- \( N_b \) = No. of bars per unit width \( b \)
- \( D^* \) = Bar diameter after corrosion loss
- \( b \) = Unit width of reinforcement
$T_{al}$ for Extensible Reinforcement

$$T_{al} = \frac{T_{ult}}{RF}$$

$$RF = RF_{ID} \times RF_{CR} \times RF_D$$

- $T_{ult}$ = minimum average roll value
  ultimate tensile strength
- RF = combined strength reduction factor

*Reference:* AASHTO 11.10.6.4.3b-1
### ASD/LRFD Tensile Breakage

#### Example for Steel Strip Reinforcement

<table>
<thead>
<tr>
<th>ASD</th>
<th>LRFD</th>
</tr>
</thead>
<tbody>
<tr>
<td>( T_{\text{max}} = \sigma_h S_v )</td>
<td>( T_{\text{max}} = \gamma_p (\sigma_h S_v) )</td>
</tr>
<tr>
<td>( T_{\text{max}} = (\sigma_v k_r + \Delta\sigma_h) S_v )</td>
<td>( T_{\text{max}} = 1.35 (\sigma_v k_r + \Delta\sigma_h) S_v )</td>
</tr>
<tr>
<td>( T_{\text{al}} = (0.55 F_y A_c) / b )</td>
<td>( \phi T_{\text{al}} = (\phi F_y A_c) / b ) (with ( \phi = 0.75 ))</td>
</tr>
<tr>
<td>( T_{\text{al}} / T_{\text{max}} = 0.55 / 1 = 0.55 )</td>
<td>( T_{\text{al}} / T_{\text{max}} = 0.75 / 1.35 = 0.55 )</td>
</tr>
</tbody>
</table>
Session IX

GUIDED DESIGN
OF RETAINING WALLS

TOPIC 1: MSE WALL DESIGN

• For Illustration Purpose:

Consult ODOT BDM for design parameters specific to ODOT project.
Problem Geometry

Assumed for bearing capacity, overall (global) stability and tensile resistance comps.

Assumed for overturning, sliding and pullout resistance comps.

Granular Fill
\( \phi_b = 38^\circ \)
\( c_b = 0 \)
\( \gamma_b = 110 \text{ pcf} \)

Hard Clay
\( \phi_b = 28^\circ \)
\( c_b = 0 \)
\( S_o = 4804 \text{ psf} \)
\( \gamma_f = 130 \text{ pcf} \)

Where:
\( E = \text{ Eccentricity} \)
\( Q = \text{ Traffic Surcharge} \)

L = 16.4 ft

32.8 ft
20 ft

IX-2
Step 1: Unfactored Loads

- Length of Soil Reinforcement
  Minimum soil reinforcement length (L):

  \[ L \geq 0.7 \, H \text{ or } 8 \, \text{ft} \]

For this example use \( L = 16.4 \, \text{ft} \).
Step 1: Unfactored Loads (cont)

- Vertical Earth Pressure (EV)

The weight of the reinforced soil backfill ($P_{EV}$)

$$P_{EV} = \gamma_r H L$$

$$P_{EV} = (110 \text{ pcf}) (23 \text{ ft}) (16.4 \text{ ft})$$

$$= 41.49 \text{ kips/ft length of wall}$$
Step 1: Unfactored Loads (cont)

- Live Load Surcharge (LS)

Using the equivalent height of soil based on AASHTO 3.11.6.4-2

For the wall height of 23 ft, $h_{eq} = 2$ ft.

The vertical force ($P_{LSV}$) due to $h_{eq}$ (effect of vehicular load)

$$P_{LSV} = \gamma_r h_{eq} L$$

$$= (110 \text{ pcf)} \times (2 \text{ ft}) \times (16.4 \text{ ft})$$

$$= 3.61 \text{ kips/ft length of wall}$$
Step 1: Unfactored Loads (cont)

- Horizontal Earth Pressure (EH)
  - Active earth pressure coefficient, $k_a$:

$$
k_a = \frac{\sin^2(\theta + \phi')}{\Gamma \sin^2 \theta \sin(\theta - \delta)}
$$

$$
\Gamma = \left[ 1 + \frac{\sin(\phi' + \delta) \sin(\phi' - \beta)}{\sin(\theta - \delta) \sin(\theta + \beta)} \right]^2
$$

where:
- $\beta = \text{Nominal slope of soil backfill behind wall (deg)} = 0^\circ$
- $\delta = \text{Angle of wall friction (deg)} = 0^\circ$
Step 1: Unfactored Loads (cont)

- Horizontal Earth Pressure (EH) (cont)
  \[ \phi' = \phi'_b = 38^\circ \]
  \[ \theta = 90^\circ \text{ for vertical wall} \]

\[ K_a = \tan^2\left(45 - \frac{\phi}{2}\right) \]

\[ K_a = 0.238 \]

The uniform increase in horizontal earth pressure due to a live load surcharge:

\[ \Delta p = k_a \gamma_b h_{eq} \]

\[ = (0.238)(110 \text{ pcf})(2 \text{ ft}) = 0.0524 \text{ ksf} = 52.4 \text{ psf} \]
Step 1: Unfactored Loads (cont)

- Horizontal Earth Pressure (EH) (cont)

Resultant of the live load surcharge horizontal earth pressure \( P_{LSH} \) acting on the reinforced soil mass is:

\[
P_{LSH} = \Delta p H
\]

\[
= (52.4 \text{ psf})(23 \text{ ft})
\]

\[
= 1.205 \text{ kips/ft length of wall}
\]

Earth pressure \( P_a \) is resultant of Triangular distribution.

\[
P_{EH} = P_{aH} = 0.5 \gamma_b H^2 k_a
\]

\[
= (0.5)(110 \text{ pcf})(23 \text{ ft})^2(0.238)
\]

\[
= 6.924 \text{ kips/ft length of wall}
\]
### Step 1: Unfactored Loads (cont)

#### Summary of Unfactored Vertical Loads and Moments

<table>
<thead>
<tr>
<th>Item</th>
<th>V (kips/ft)</th>
<th>Moment Arm About Toe (ft)</th>
<th>Moment about Toe (kips-ft/ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$P_{EV}$</td>
<td>41.49</td>
<td>8.2</td>
<td>340.2</td>
</tr>
<tr>
<td>$P_{LSV}$</td>
<td>3.61</td>
<td>8.2</td>
<td>29.6</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>45.1</strong></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Step 1: Unfactored Loads (cont)

Unfactored Horizontal Loads/Moments

<table>
<thead>
<tr>
<th>Item</th>
<th>H (kip/ft)</th>
<th>Moment Arm about Toe (ft)</th>
<th>Moment about Toe (kips-ft/ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$P_{EH} = P_{aH}$</td>
<td>6.924</td>
<td>23 ft /3 =7.64</td>
<td>52.9</td>
</tr>
<tr>
<td>$P_{LSH}$</td>
<td>1.205</td>
<td>11.5</td>
<td>113.86</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>8.13</strong></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
## Step 2: Load Factors

The applicable load combinations and load factors

<table>
<thead>
<tr>
<th>Group</th>
<th>$\gamma_{EV}$</th>
<th>$\gamma_{EH}$</th>
<th>$\gamma_{LS}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strength I-a</td>
<td>1.00</td>
<td>1.50</td>
<td>1.75</td>
</tr>
<tr>
<td>(Sliding &amp; Eccentricity)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Strength I-b</td>
<td>1.35</td>
<td>1.50</td>
<td>1.75</td>
</tr>
<tr>
<td>(Bearing resistance)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Service I</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
</tbody>
</table>
### Step 3: Factored Loads and Moments

#### Factored vertical load

<table>
<thead>
<tr>
<th>Group</th>
<th>$P_{EV}$ (kips/ft)</th>
<th>$P_{LSV}$ (kips/ft)</th>
<th>$V_{TOT} = P_{EV1} + P_{LSV}$ (kips/ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unfactored</td>
<td>41.49</td>
<td>3.61</td>
<td>45.1</td>
</tr>
<tr>
<td>Strength I-a</td>
<td>41.49</td>
<td>--</td>
<td>47.81</td>
</tr>
<tr>
<td>(Sliding &amp; Eccentricity)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Strength I-b</td>
<td>56.0</td>
<td>6.32</td>
<td>62.32</td>
</tr>
<tr>
<td>(Bearing resistance)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Service I</td>
<td>41.49</td>
<td>3.61</td>
<td>45.1</td>
</tr>
</tbody>
</table>
### Step 3: Factored Loads and Moments

**Factored horizontal loads**

<table>
<thead>
<tr>
<th>Group</th>
<th>$P_{EH}$ (kips/ft)</th>
<th>$P_{LSH}$ (kips/ft)</th>
<th>$H_{TOT} = P_{EH} + P_{LSH}$ (kips/ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unfactored</td>
<td>6.92</td>
<td>1.21</td>
<td>8.13</td>
</tr>
<tr>
<td>Strength I-a (Sliding &amp; Eccentricity)</td>
<td>10.39</td>
<td>2.11</td>
<td>12.5</td>
</tr>
<tr>
<td>Strength I-b (Bearing resistance)</td>
<td>10.39</td>
<td>2.11</td>
<td>12.5</td>
</tr>
<tr>
<td>Service I</td>
<td>6.92</td>
<td>1.21</td>
<td>8.13</td>
</tr>
</tbody>
</table>
Step 4: MSE Wall settlement/
Lateral Displacement

It is assumed that the MSE wall will not experience unacceptable settlements or lateral displacement.
## Step 5: Eccentricity

### Summary for Eccentric Check

<table>
<thead>
<tr>
<th>Group/Item Units</th>
<th>$P_{EV}$ (kips/ft)</th>
<th>$M_{EV}$ (kips-ft/ft)</th>
<th>$M_{HTOT}$ (kips-ft/ft)</th>
<th>$X_0$ (ft)</th>
<th>$e_B$ (ft)</th>
<th>$q_{uniform}$ ksf</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Strength I-a</strong></td>
<td>41.49</td>
<td>340.2</td>
<td>103.6</td>
<td>5.71</td>
<td>2.49</td>
<td>3.63</td>
</tr>
<tr>
<td>(Sliding &amp; Eccentricity)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Strength I-b</strong></td>
<td>56.0</td>
<td>459.2</td>
<td>103.6</td>
<td>6.35</td>
<td>1.85</td>
<td>4.41</td>
</tr>
<tr>
<td>(Bearing resistance)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Service I</strong></td>
<td>41.49</td>
<td>340.2</td>
<td>66.76</td>
<td>6.59</td>
<td>1.61</td>
<td>3.15</td>
</tr>
</tbody>
</table>
Step 5: Eccentricity (cont)

\[ X_0 = \text{Location of the resultant from toe of wall} \]
\[ = \frac{(M_{EV} - M_{hTOT})}{P_{EV}} \]

\[ B = \text{Base width} = \text{Length of reinforcement strips} \]

\[ e_B = \text{eccentricity} = \frac{B}{2} - X_0 = 8.2 \text{ ft} - X_0 \]

\[ q_{\text{uniform}} = \frac{P_{EV}}{(B - 2e_B)} = \frac{P_{EV}}{2X_0} \]

The location of the resultant must be in the middle half of the base.

\[ e_{\text{max}} = \frac{B}{4} = 16.4 \text{ ft} / 4 = 4.1 \text{ ft} \]

\[ e_B < e_{\text{max}} \text{ for all cases, therefore the design is adequate with regard to eccentricity.} \]
Step 6: Sliding

The factored resistance against failure by sliding \( (R_R) \):

\[
R_R = \phi_r R + \phi_{ep} R_{ep}
\]

Neglect passive resistance: \( \phi_{ep} R_{ep} = 0 \)

\( \phi_r = 0.9 \) for sliding of soil against soil (AASHTO 10.5.5.2.2-1)

Select the lesser friction angle between the reinforced soil and the foundation soil.

Nominal shear resistance is:

\[
R = V \tan \delta
\]

Where:

\[
tan \delta = tan \phi_f
\]

\[
V = P_{EV}
\]
Step 6: Sliding (cont)

\[ R_\tau = P_{EV} \tan 28^\circ = 0.53 P_{EV} \]
\[ P_{EV} = 41.49 \text{ kips/ft (Strength I-a)} \]
\[ R_\tau = (0.53)(41.49 \text{ kips/ft}) = 22 \text{ kips/ft} \]

Applying the resistance factor \( \phi_\tau \) to \( R_\tau \)
\[ R_R = (0.9)(22.0 \text{ kips/ft}) = 19.8 \text{ kips/ft} \]

Sliding resistance is adequate.
\[ R_R > H_{TOT} \ (19.8 \text{ kips/ft} >> 12.5 \text{ kips/ft}) \]
Step 7: Bearing Capacity

The factor unit bearing resistance \( (q_R) \):

\[
q_R = \phi q_u
\]

\[
q_n = c N_{cm} + \gamma_s D_f N_{qm}
\]

Where: \( c = S_u = 4808 \) psf, \( \gamma_s = \gamma_f = 130.5 \) pcf, \( N_{cm} = 5.14 \), and \( D_f = 0 \) ft

Therefore:

\[
q_n = c N_{cm}
\]

\[
q_n = (5.14)(4.8 \text{ ksf}) = 24.7 \text{ ksf}
\]

Use resistance factor \( (\phi) = 0.45 \) (AASHTO 10.5.5.2.2-1)

\[
q_R = \phi q_n = (0.45)(24.7 \text{ ksf}) = 11.1 \text{ ksf}
\]
Step 7: Bearing Capacity (cont)

<table>
<thead>
<tr>
<th>Group/Item Units</th>
<th>$V_{TOT}$ (kips/ft)</th>
<th>$M_{VTOT}$ (kips-ft/ft)</th>
<th>$M_{hTOT}$ (kips-ft/ft)</th>
<th>$X_o$ (ft)</th>
<th>$e_B$ (ft)</th>
<th>$q_{uniform}$ ksf</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strength I-a</td>
<td>47.8</td>
<td>392.0</td>
<td>103.6</td>
<td>6.03</td>
<td>2.17</td>
<td>3.96</td>
</tr>
<tr>
<td></td>
<td>(Sliding &amp; Eccentricity)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Strength I-b</td>
<td>62.3</td>
<td>511.1</td>
<td>103.6</td>
<td>6.54</td>
<td>1.66</td>
<td>4.76</td>
</tr>
<tr>
<td></td>
<td>(Bearing resistance)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Service I</td>
<td>45.1</td>
<td>369.8</td>
<td>66.76</td>
<td>6.72</td>
<td>1.48</td>
<td>3.36</td>
</tr>
</tbody>
</table>
Step 7: Bearing Capacity (cont)

\[ q_{\text{uniform}} = \frac{V_{\text{TOT}}}{(B-2e_B)} = \frac{V_{\text{TOT}}}{2X_0} \]

- The maximum \( q_{\text{uniform}} = 4.76 \text{ ksf} \)
- The bearing capacity is adequate because \( q_R > q_{\text{uniform}} \)
Step 8: Overall Stability

- Check for the over stability against a deep-seat soil failure using a limit equilibrium method of analyses.

- For this example, it is assumed that the overall stability is adequate.
Session IX

GUIDED DESIGN
OF RETAINING WALLS

Topic 2: Single Tier Anchored Wall Example
Problem Geometry

\[ q = 2 \text{ ft} \times 115 \text{pcf} = 230 \text{ psf} \]

\[ \text{H} = 25 \text{ ft} \]

\[ H_1 = 8.4 \text{ ft} \]

\[ \text{H} - H_1 \]

\[ d \]

Cohesion less Soil

\[ \varphi = 33^\circ \]

\[ \gamma_m = 115 \text{ pcf} \]

Pile spacing = 8 ft
Step 1 to Step 3

- Steps 1 through 3 of the design process have been completed ahead of time and are not shown here.
- The design process begins with Step 4, select earth pressure envelope.
Step 4: Select Earth Pressure Envelope

Earth pressure distributions for anchored walls in cohesionless soils (AASHTO 3.11.5.7)
Step 4: Select Earth Pressure Envelope (Cont.)

\[ P = K_A \gamma H \]

Where: \( \phi = 33^\circ \)

\[ K_A = \tan^2 \left( 45 - \frac{\phi}{2} \right) \]

\( K_A = 0.29 \)

\( p = (0.29)(115)(25) \)

\( p = 833.25 \text{ lb/ft} \)
Step 4: Select Earth Pressure Envelope
(Including surcharge load)

\[ p_s = K_A q = (0.29)(115 \text{pcf})(2 \text{ft}) = 66.71 \text{b/ft/ft spacing} \]
Step 5: Evaluate Reaction Force

The lateral forces are calculated using the Tributary Area Method (per ft spacing)

\[
R = \frac{1}{2} \left( \frac{1}{2} (H - H_1) \right) \frac{3}{4} p \quad (1)
\]

\[
R_s = \frac{1}{2} (H - H_1) p_s \quad (2)
\]

\[
T = \frac{1}{2} \left( \frac{2}{3} H_1 p + \frac{1}{3} H p + \frac{1}{6} (H - H_1) \right) \frac{7}{8} p \quad (3)
\]

\[
T_s = p_s (H_1 - \frac{1}{2} (H - H_1)) \quad (4)
\]

Where: \( H_1 = 8.4 \text{ ft}, \quad H = 25 \text{ ft}, \quad p = 833.75 \text{ lb/ft/ft}, \quad p_s = 66.7 \text{ lb/ft/ft} \)
Step 5: Evaluate Reaction Force (Cont.)

Based on the above Eq. 1,2,3,4 (multiply by 8 ft spacing)

<table>
<thead>
<tr>
<th>R (kips)</th>
<th>R_s (kips)</th>
<th>R_T=R+R_s (kips)</th>
<th>T (kips)</th>
<th>T_s (kips)</th>
<th>T_T=T+T_s (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>20.76</td>
<td>4.428</td>
<td>25.2</td>
<td>90.4</td>
<td>8.9</td>
<td>99.3</td>
</tr>
</tbody>
</table>

(Note: Spacing = 8 ft)
Step 6: Load factors

- For active horizontal earth pressure (EH)
  \[ \gamma_p = 1.50 \quad (AASHTO \text{ Table 3.4.1-2}) \]

- For live load surcharge (LS)
  \[ \gamma = 1.75 \quad (AASHTO \text{ Table 3.4.1-1}) \]
### Step 6: Resistance Factors

<table>
<thead>
<tr>
<th>Resistance Factor</th>
<th>$\phi$</th>
<th>Refer to</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pullout resistance of anchors (presumptive unit bond stress)</td>
<td>0.65</td>
<td>AASHTO Table 11.5.6-1</td>
</tr>
<tr>
<td>Pullout resistance of anchors (where proof tests are conducted)</td>
<td>1.0</td>
<td>AASHTO Table 11.5.6-1</td>
</tr>
<tr>
<td>Tensile resistance of anchor tendons (high strength steel)</td>
<td>0.80</td>
<td>AASHTO Table 11.5.6-1</td>
</tr>
<tr>
<td>Passive resistance of vertical elements</td>
<td>1.00</td>
<td>AASHTO Table 11.5.6-1</td>
</tr>
<tr>
<td>Flexural resistance of vertical element</td>
<td>1.00</td>
<td>AASHTO Table 11.5.6-1</td>
</tr>
<tr>
<td>Axial capacity of vertical element (compression)</td>
<td>0.90</td>
<td>AASHTO Article 6.5.4.2</td>
</tr>
<tr>
<td>Bearing resistance of vertical elements for skin friction and end bearing: sand</td>
<td>0.30</td>
<td>AASHTO Table 10.5.5-2</td>
</tr>
<tr>
<td>(SPT Method in sand)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Step 7: Evaluate Anchor Inclination

For this design:

The Anchor Inclination is 30 degree.
Step 8: Select Tendon Type and Check Tensile Resistance

Factored Horizontal Support Load \( \leq \phi T_n \geq \sum \eta \gamma_i Q_i \)

\[ \sum \eta \gamma_i Q_i = 1.5(90.4) + 1.75(8.9) \]

= 151.2 kips/pile
Factored Anchor Force \( \frac{\phi T_n}{\cos 30^\circ} = 174.6 \text{kips/pile} \)

174.6kips \( \leq \Phi F_n = 0.8F_n \) where \( \Phi = 0.8 \)

\[ F_n = 218 \text{kips} \]

Use 4 Standard ASTM A-416 tendon with \( f_{pu} A_{ps} = 234.4 \text{kips} \)

234.4 > 218 kips \( \text{OK} \)

\[ F_n = \text{GUTS} \]
Step 9: Evaluate Anchor Bond Length

Nominal pullout resistance:

\[ Q_a = \pi D \tau_a L_b \]

The range of presumptive ultimate bond stress is 1.7~7.9 ksf.  
(AASHTO Table C11.9.4.2-2)

\[ \tau_a = 5.2 \text{ ksf} \]  
(corresponds to approximately 8.2 kips/ft of bond length for a 0.5 ft diameter, D, drillhole)

\[ Q_a = (8.2) L_b \text{ kips} \]

Factored Anchor Force = 174.6 kips \( \leq \Phi Q_a = 0.65(8.2)L_b \)

\[ L_{bmin} = 174.6/(0.65)(8.2) = 32.8 \text{ ft} \]
If contractor conduct proof tests on each ground anchor to 1.0 times the factored anchor load on the anchor, then higher bond strength from load test may be used. The resistance factor $\Phi = 1.0$
Step 9: Anchor Bond Length (Cont.)

Anchor Geometry
Step 9: Anchor Bond Length (Cont.)

\[ L_1 = \frac{(H - H_1) \sin(45 - \frac{\phi}{2})}{\sin(\theta + 45 + \frac{\phi}{2})} = 7.9 \text{ ft} \]

\[ L_2 = \text{Greater of 5 ft or } H/5 (=5 \text{ ft}) \] (AASHTO Figure 11.9.1-1)

\[ L_u = 15 \text{ ft minimum} \] (AASHTO Figure 11.9.1-1)
Step 10: Flexural Resistance of soldier Pile

Calculation bending moment using Tributary Area Method

\[
M_B = \frac{13}{54} H_1^2 p
\]

\[
T_1 = \frac{(23H^2 - 10HH_1)}{54(H-H_1)} p
\]

\[
R = \frac{2}{3} Hp - T_1
\]

Solve for point of zero shear

\[
x = \frac{1}{9} \sqrt{(26H^2 - 52HH_1)}
\]

\[
M_{BC} = Rx - \frac{px^3}{4(H-H_1)}
\]

\[M_{BC} = \text{Moment at point of zero shear due to earth pressure}\]
Step 10: Flexural Resistance of soldier Pile

For moment about the level of anchor

\[ M_{BT} = M_B + M_{BS} \]

\[ M_B = \frac{13}{54} \cdot pH_1^2 \]

Moment due to earth pressure

\[ M_{BS} = p_s \frac{H_1^2}{2} \]

Moment due to surcharge \( p_s \)

\[ M_{BT} = 132.1 \text{ kip-ft} \]
Step 10: Flexural Resistance of Soldier Pile (Cont.)

For maximum moment at the point of zero shear

\[ M_{BC} = R_x \left( \frac{px^3}{4(H - H_1)} \right) \]
\[ M_{BCS} = R_s x - \frac{ps x^2}{2} \] (surcharge)

Where: \( x \), the point of zero shear, \[ x = \frac{-b \pm \sqrt{b^2 - 4ac}}{2a} \]

\[ a = \frac{3p}{4(H - H_1)} \]
\[ b = p_s \]
\[ c = -\left[H_p \left( \frac{2}{3} - \frac{(23H - 10H_1)}{54(H - H_1)} \right) + p_s \left( \frac{H^2}{2(H - H_1)} \right) \right] \]

\( x = 7.92 \text{ ft} \)
Step 10: Flexural Resistance of Soldier Pile (Cont.)

\[ M_{BC} = 114.5 \text{ kips} \quad M_{BCT} = M_{BC} + M_{BCS} = 114.5 \text{ kips/ft} + 18.33 \text{ kips/ft} = 132.8 \text{ kip-ft} \]

Maximum unfactored bending moment \( M_{BCT} = 132.8 \text{ kip-ft} \)

Maximum factored moment = \( 1.5M_B + 1.75M_{BS} = 201.1 \text{ kip-ft} \)

Maximum factored axial component = \( \left( \frac{T}{\cos(30)} \right)(1.5) + \frac{T_s}{\cos(30)}(1.75) \sin(30) \)

\[ = 174.6 \sin 30 \quad = 87.3 \text{ kips} \]

Where: \( T = 90.4 \text{ kips}, \quad T_s = 8.9 \text{ kips} \)
Step 10: Flexural Resistance of Soldier Pile (Cont.)

Design of Steel Section for Soldier Beam
(combined axial and bending loading)

Use HP 12 x 63

\[ \frac{P_u}{P_r} < 0.2 \quad \text{then} \quad \frac{P_u}{2P_r} + \frac{M_{ux}}{M_{rx}} \leq 1.0 \]

AASHTO Eq.6.9.2.2-1

If \( P_u = 87.3 \text{kips} \)

If \( \lambda \leq 2.25 \), then

\[ P_n = 0.66\lambda F_y A_s \]

AASHTO Eq.6.9.4.1-1
For which

\[ \lambda = \left( \frac{Kl}{r_s \pi} \right)^2 \frac{F_y}{E} \]

AASHTO Eq.6.9.4.1-3

\[ \lambda = \left( \frac{(1.0)(25 \text{ ft} - 8.4 \text{ ft})}{0.423 \pi} \right)^2 \frac{36000 \text{ psi}}{29 \times 10^6 \text{ psi}} = 0.2 \]

\[ P_n = 0.66^{0.2} \times 36 \text{ksi} \times 17.815 \text{in}^2 = 590.2 \text{kips} \]
Step 10: Flexural Resistance of soldier Pile (Cont.)

Design of Steel Section for Soldier Beam

\[ P_r = \phi P_n = 0.9(590.2 \text{ kip}) = 531.2 \text{ kips} \]

\[ \frac{P_u}{P_r} = \frac{87.3 \text{ kip}}{531.2 \text{ kips}} = 0.16 < 0.2 \]

\[ M_{tx} = \phi M_n = 1.0 F_y Z = 1.0(88.3 \text{ in}^3)(36 \text{ ksi}) = 264.9 \text{ kips-ft} \]
\[
\frac{P_u}{2P_r} + \frac{M_{ux}}{M_{rx}} = \frac{87.3\text{kips}}{2*531.2\text{kips}} + \frac{201.1\text{kip-ft}}{264.9\text{kip-ft}} = 0.84 \leq 1.0
\]
Step 11: Evaluate Bearing Resistance of Vertical Wall Element

The nominal bearing resistance

\[ Q_n = f_s A_s + q_t A_t \]  

(AASHTO Eq. 10.7.3.2-2)

Where: \[ f_s = \beta p_0 \], \[ q_t = N_t p_t \]

\[ \beta \approx 0.35, \quad N_t \approx 40 \] from the figure below, for a medium to dense sand, use average values for sand.

\[ P_t = \text{vertical stress at the pile tip} \]
Step 11: Bearing Resistance of Vertical Wall Element (Cont.)

Step 11: Bearing Resistance of Vertical Wall Element (Cont.)

Step 11: Bearing Resistance of Vertical Wall Element (Cont.)

\[ Q_n = (0.35 \gamma (H + d) \times 0.5) \times (4bd) + (40 \gamma d) \times b^2 \]

Where: 
\[ f_s = 0.35 \gamma (H + d) \times 0.5 \quad , \quad q_t = 40 \gamma d \]

\[ A_s = 4bd, \quad A_t = b^2 \quad \text{(Width and depth of HP12x36 are the same)} \]

For a \( b = 0.984 \text{ ft.} \) square pile section,
\[ Q_n = 0.079212d^2 + 6.5803d, \quad \text{Factored axial load} = 87.3 \text{ kips} \]

\[ Q_r = \phi Q_n = (0.3) Q_n = 0.023764d^2 + 1.974d = 87.3 \text{ kips} \]

\[ d_{\text{min}} = 32 \text{ ft} \]

The toe embedment required is 32 ft.
Step 12: Evaluate Passive Resistance of Vertical Wall Element

(Broms Method)
Step 12: Bearing Resistance of Vertical Wall Element (Cont.)

Passive resistance: \( \phi = 1.0 \implies F_n = F_r \)

\[
F_p = 0.5(3K_p \gamma bd)d = 3/2 K_p \gamma bd^2
\]

\[
d = \sqrt{\frac{2R}{3K_p \gamma b}}
\]

where:
- For Rankine passive pressure coefficient: \( K_p = 3.39 \)
- Factored \( R = 1.5(20.76 \text{ kips}) + 1.75(4.428 \text{ kips}) = 38.89 \text{ kips} \)

\( d = 8.15 \text{ ft} \)

The toe embedment required to resist the factored \( R \) is 8.2 ft