
- AASHTO LRFD Highlights (Bigger Changes)
- AASHTO LRFD Specific Article Changes
- Other Subjects for Discussion
Highlights:

- **Section 5: Concrete Structures**
  - Completely New and Reorganized
    - old 5.8.2.9 - Shear Stress in Concrete, is now 5.7.2.8
    - old 5.13.3 - Footings, is now 5.12.8
    - old 5.13.4 - Concrete Piles, is now 5.12.9
    - old 5.13.4.6 - Seismic Requirements, moved to 5.11.3.2 and 5.11.4.5 under new 5.11 - Seismic Design and Details
Highlights:

- Section 6: Steel Structures
  - Many new changes (94 Articles)

- 6.5.4.2 - Resistance Factors: For axial compression, steel only $\phi_c = 0.95$
  (2014, 7th Ed. change)

- Section 7: Aluminum Structures
  - Also many changes (69 Articles)
Highlights:

- Coulomb Analysis of MSE Walls Ballot Item
  - Formerly, Rankine with $\delta = \beta$ or $B$
  - Now explicitly Coulomb, $\delta \leq \frac{2}{3} \phi$
  - “Horizontal” EH, LS, ES inclined at $\delta$

Affects the following Articles:

- 3.11.5.8.1 - Earth Pressure Distribution for MSE Walls
- 11.10.5.2 - Loading
- 11.10.7.1 - External Stability (Seismic)
AASHTO LRFD 8TH EDITION (2018)

- Highlights:
  - Coulomb Analysis of MSE Walls Ballot Item
  - Article 11.10.7.1 - External Stability
    - Figure 11.10.7.1-1 updated to show $P_{AE}$ inclined at Coulomb $\delta$
o Highlights:
  o Coulomb Analysis of MSE Walls Ballot Item
  o Article 3.11.5.8.1 - Earth Pressure Distribution for MSE Walls
    o For broken back slope use either:
      o Generalized Limit Equilibrium (GLE) Method (see A11.3.3),
      o Coulomb Trial-Wedge Analysis (see A11.3.1, A11.3.2), or
      o “Traditional Method” per Figure C3.11.5.8.1-1 with $\beta = \beta'$
"Traditional Method"

Figure C3.11.5.8.1-1—Earth Pressure Distribution for MSE Wall with Broken Back Earth Surcharge
Method of Calculation | "Horizontal" Earth Pressure Load
--- | ---
Limit Equilibrium Analysis | 45,738 lb/ft
Coulomb Trial-Wedge Analysis | 46,072 lb/ft
Coulomb "Traditional" Method | 52,734 lb/ft
Rankine "Traditional" Method | 53,012 lb/ft

"Traditional Method"

- Coulomb, $\delta = \frac{2}{3} \phi$
- Rankine, $\delta = \beta'$
 Highlights:

 - **Seismic Wall Corners Ballot Item**
  - No longer requires seismic analysis for walls with changes in alignment sharper than 120 degrees, but calls attention to why these changes in alignment are problematic:
Highlights:

- **Seismic Wall Corners Ballot Item**
  - (they “tend to exhibit greater damage than free standing walls with generally straight alignments due to potentially greater stiffness at the corner, or may separate at the corner due to lack of connectivity between wall sections at the corner.”)
Highlights:

- Seismic Wall Corners Ballot Item

Affects the following Articles:

- 11.5.4.2 - Extreme Event I, No Analysis
- 11.6.5.6 - Wall Details for Improved Seismic Performance
- 11.10.7.4 - Wall Details for Improved Seismic Performance
- 11.11.6 - Seismic Design for Prefabricated Modular Walls
Highlights:

- **Seismic Wall Corners Ballot Item**
  - Formerly **no guidance** provided on what specific analyses should be conducted to address the potentially higher loads, nor is any such guidance available. Therefore, these articles have been refocused to provide better guidance on what is expected for **good seismic detailing**.
Highlights:

- **Seismic Wall Corners Ballot Item**
  - Also calls attention to the fact that placement and compaction of fill at corners requires more attention than other sections along the wall due to lack of room there. Inadequate compaction can potentially cause increased deformations at the corners during a seismic event.
Specific Article Changes:

3.8.1.2.1 - Wind Load on Structures: WS

- For Ohio, Design Speed $V = 115$ mph
- Load Factor $\gamma_{WS} = 1.00$
- Generally, design pressure $\approx 0.07$ ksf
- Formerly, $V = 100$ mph, $\gamma_{WS} = 1.40$
- Formerly, design pressure $\approx 0.05$ ksf
- $1.40 \times 0.05 = 0.07$ (no change)
Specific Article Changes:

- 3.11.5.3 - Active Lateral Earth Pressure Coefficient, $k_a$
  - $\delta = 0.67 \phi$ soil-concrete, soil-soil
  - $\delta = 0.33 \phi$ soil-steel
  - $\delta = 0.50 \phi$ * soil-precast concrete
  - $\delta = 0.70 \phi$ * prefabricated modular stepped modules
  * per Table C3.11.5.9-1
Specific Article Changes:

- 3.11.5.6 - Lateral Earth Pressures for Nongravity Cantilevered Walls
  - For Figures 3.11.5.6-3, 3.11.5.6-6, and 3.11.5.6-7 (Teng, 1962), \( D = 1.2 D_0 \)
  - Explicitly states that adding 20% to depth is due to a simplification of the loading model.
Fig. 12-9 Design of cantilever sheetpiling in granular soils: (a) conventional method; (b) simplified method.
Specific Article Changes:

- 3.11.6.2 - Point, Line, and Strip Loads (ES) Walls Restrained from Movement
- For Figure 3.11.6.2-4 - Horizontal Pressure on a Wall Caused by a Finite Line Load Perpendicular to the Wall; redefinition of $X_2$ to be in-line with the original Boussinesq definition.
Figure 3.11.6.2-4—Horizontal Pressure on a Wall Caused by a Finite Line Load Perpendicular to the Wall
Specific Article Changes:

10.6.3.1.2c - Considerations for Footings on Slopes

- Formerly no convergence of $N_c$ (to 5.14 or drained value for flat slope)

- Now use Tables for $RC_{BC}$
  - $q_n$-sloping ground = $RC_{BC} q_n$

- or use Limit Equilibrium
\[ N_s = \frac{\gamma H_s}{c} \]  

(10.6.3.1.2c-2)

where:

\[ N_s = \text{slope stability factor (dim)} \]

\[ H_s = \text{height of sloping ground surface below bottom of footing (ft)} \]

and other variables are as defined in Article 10.6.3.1.2a.

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**Figure 10.6.3.1.2c-1—Definition of Footing and Slope Geometric Parameters for Determination of RC\textsubscript{BC}**
## AASHTO LRFD 8TH EDITION (2018)

### Table 10.6.3.1.2c-1—Reduction Coefficients ($RC_{BC}$) for Footings Placed on Slopes Composed of either Purely Cohesive Soils, ($\phi = 0$); Purely Cohesionless Soils ($c' = 0$); or Soils with both Cohesive and Cohesionless Strength Components

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<th>$\phi$ (°)</th>
<th>$B/H$</th>
<th>$b/B$</th>
<th>$\beta=10^\circ$</th>
<th>$\beta=20^\circ$</th>
<th>$\beta=30^\circ$</th>
<th>$\beta=40^\circ$</th>
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<td></td>
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<td>$N_5$</td>
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Table 10.6.3.1c-2—Reduction Coefficients ($R_{cd}$) for Footings Placed Adjacent to Slopes Composed of either Purely Cohesive Soils, ($\phi = 0$); Purely Cohesionless Soils ($\phi > 0$); or Soils with both Cohesive and Cohesionless Strength Components

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<th>10</th>
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<th>0</th>
<th>0.5</th>
<th>1.25</th>
<th>2.5</th>
<th>5</th>
<th>10</th>
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<td>0.91</td>
<td>0.76</td>
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<td>0.81</td>
<td>0.80</td>
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Table 10.6.3.1c-2 (cont.)
H = 8.0 ft

\( \beta = 20^\circ \)

FOUNDATION SOIL

\( \phi = 22^\circ, c' = 150 \text{ psf}, \gamma = 120 \text{ pcf} \)

B = 8.0 ft
\[ q_n = c_f' N_{cm} + \gamma_q D_f N_{qm} C_{wq} + 1/2 \gamma_f B' N_{\gamma m} C_{w\gamma} \]  
(AASHTO LRFD Eq. 10.6.3.1.2a-1)

\[ B' = B = 8 \text{ ft} \]
\[ \phi_f' = 22^\circ \]
\[ \gamma_f = 120 \text{ pcf} \]
\[ s_c = s_q = s_\gamma = 1.00 \text{ (for infinite strip footing)} \]
\[ i_c = i_q = i_\gamma = 1.00 \text{ (for inclined eccentric loading)} \]
\[ d_q = 1 + 2 \tan(\phi_f)(1-\sin \phi_f)^2 \arctan(D_f/B') \]  
(Hansen, 1970)

\[ N_c = 16.88 \]
\[ N_q = 7.82 \]
\[ N_{\gamma} = 7.13 \]
\[ C_{wq} = 1.00 \text{ (deep water, } D_w > D_f) \]
\[ C_{w\gamma} = 1.00 \text{ (deep water, } D_w > D_f + 1.5B) \]

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<th>(d_q)</th>
<th>(N_{qm})</th>
<th>(q_n)</th>
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<tbody>
<tr>
<td>0 ft</td>
<td>1.00</td>
<td>7.82</td>
<td>5954 psf</td>
</tr>
<tr>
<td>3 ft</td>
<td>1.11</td>
<td>8.71</td>
<td>9090 psf</td>
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Table 10.6.3.1.c-2—Reduction Coefficients (RC_{bc}) for Footings Placed Adjacent to Slopes Composed of either Purely Cohesive Soils, (\phi = 0); Purely Cohesionless Soils (\phi > 0); or Soils with both Cohesive and Cohesionless Strength Components

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<th>7.5</th>
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<td>5.0</td>
<td>7.5</td>
<td>10.0</td>
<td>15.0</td>
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<td>30.0</td>
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<td>b/B = 0.5</td>
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<td>0.5</td>
<td>0.76</td>
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<tr>
<td>1.25</td>
<td>( \beta = 20^\circ )</td>
<td>20^\circ</td>
<td>20^\circ</td>
<td>0.76</td>
<td>0.40</td>
<td>0.40</td>
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<tr>
<td>1.5</td>
<td>( \phi_f = 22^\circ )</td>
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<td>22^\circ</td>
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Note: The table continues on the next page.
\[ q_{n\text{-sloping ground}} = RC_{BC} q_n = 0.78 \times 5954 \text{ psf} = 4644 \text{ psf} \]

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<th>(N_{qm})</th>
<th>(q_n)</th>
<th>(q_{n\text{-sloping ground}})</th>
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<td>3 ft</td>
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H = 4.0 ft
\[ \beta = 20^\circ \]
B = 8.0 ft

FOUNDATION SOIL
\[ \phi = 22^\circ, c' = 150 \text{ psf}, \gamma = 120 \text{ pcf} \]
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<td>β = 20°</td>
<td>20°</td>
<td>20°</td>
<td>20°</td>
<td>20°</td>
<td>20°</td>
<td>20°</td>
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<td>20°</td>
</tr>
<tr>
<td>Φr = 22°</td>
<td>20°</td>
<td>20°</td>
<td>20°</td>
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<td>30°</td>
<td>30°</td>
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<tr>
<td>RC_{BC}</td>
<td>0.80</td>
<td>0.75</td>
<td>0.77</td>
<td>0.72</td>
<td>0.73</td>
<td>0.69</td>
<td>0.74</td>
<td>0.70</td>
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</table>
$$q_n\text{-sloping ground} = RC_{BC} q_n = 0.719467 \times 5954 \text{ psf} = 4283 \text{ psf}$$

<table>
<thead>
<tr>
<th>$D_f$</th>
<th>$d_q$</th>
<th>$N_{qm}$</th>
<th>$q_n$</th>
<th>$q_n$-sloping ground</th>
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<tbody>
<tr>
<td>0 ft</td>
<td>1.00</td>
<td>7.82</td>
<td>5954 psf</td>
<td>4283 psf</td>
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<tr>
<td>3 ft</td>
<td>1.11</td>
<td>8.71</td>
<td>9090 psf</td>
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### Table

<table>
<thead>
<tr>
<th>Color</th>
<th>Name</th>
<th>Unit Weight (pcf)</th>
<th>Cohesion' (psf)</th>
<th>Phi' (°)</th>
<th>Constant Unit Wt. Above Water Table (pcf)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Bearing Soil</td>
<td>125</td>
<td>150</td>
<td>22</td>
<td>120</td>
</tr>
</tbody>
</table>

![Diagram](image)
### AASHTO LRFD 8th Edition (2018)

#### Table of Soil Properties

<table>
<thead>
<tr>
<th>Color</th>
<th>Name</th>
<th>Unit Weight (pcf)</th>
<th>Cohesion' (psf)</th>
<th>Phi° (°)</th>
<th>Constant Unit Wt. Above Water Table (pcf)</th>
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</thead>
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<tr>
<td>Brown</td>
<td>Bearing Soil</td>
<td>125</td>
<td>150</td>
<td>22</td>
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</tbody>
</table>

#### Diagram

- **1.000**
  - 4,096 pcf
  - Water Table

#### Footnotes

- [AASHTO LRFD 8th Edition](#)
<table>
<thead>
<tr>
<th>Color</th>
<th>Name</th>
<th>Unit Weight (pcf)</th>
<th>Cohesion (psf)</th>
<th>Phi' (°)</th>
<th>Constant Unit Wt. Above Water Table (pcf)</th>
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</thead>
<tbody>
<tr>
<td></td>
<td>Bearing Soil</td>
<td>125</td>
<td>150</td>
<td>22</td>
<td>120</td>
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</tbody>
</table>
### Method

<table>
<thead>
<tr>
<th>Method</th>
<th>Top of Slope</th>
<th>Middle of Slope</th>
<th>Flat Ground</th>
</tr>
</thead>
<tbody>
<tr>
<td>$R_{BC}$ Table</td>
<td>4644</td>
<td>78%</td>
<td>4283</td>
</tr>
<tr>
<td>Limit Equilibrium</td>
<td>4720</td>
<td>65%</td>
<td>4096</td>
</tr>
<tr>
<td>Percent Difference</td>
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<td>2%</td>
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### Table of Soil Properties

<table>
<thead>
<tr>
<th>Color</th>
<th>Name</th>
<th>Unit Weight (pcf)</th>
<th>Cohesion' (psf)</th>
<th>Phi° (°)</th>
<th>Constant Unit Wt. Above Water Table (pcf)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Bearing Soil</td>
<td>125</td>
<td>150</td>
<td>22</td>
<td>120</td>
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</tbody>
</table>

### Diagram:

- **1.000**
- **7,869 pcf**

## Bearing Soil Properties

<table>
<thead>
<tr>
<th>Color</th>
<th>Name</th>
<th>Unit Weight (pcf)</th>
<th>Cohesion' (psf)</th>
<th>Phi° ('')</th>
<th>Constant Unit Wt. Above Water Table (pcf)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Bearing Soil</td>
<td>125</td>
<td>150</td>
<td>22</td>
<td>120</td>
</tr>
</tbody>
</table>

![Diagram showing bearing soil properties](image)

1.000

10,500 pcf
<table>
<thead>
<tr>
<th>Method</th>
<th>Top of Slope</th>
<th>Middle of Slope</th>
<th>Flat Ground</th>
</tr>
</thead>
<tbody>
<tr>
<td>$R_{BC}$ Table</td>
<td>N/A</td>
<td>N/A</td>
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<tr>
<td>Limit Equilibrium</td>
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<td>71%</td>
<td>7869</td>
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<td></td>
<td>75%</td>
<td></td>
<td>10500</td>
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<tr>
<td>Percent Difference</td>
<td>N/A</td>
<td>N/A</td>
<td>16%</td>
</tr>
</tbody>
</table>
Specific Article Changes:

- **10.6.3.4 - Failure by Sliding**
  - No more $\delta$, replaced explicitly with $\phi_f$.
  - Now, $R_\tau = CV \tan \phi_f$.
  - $C = 1.0$ for CIP, $C = 0.8$ for precast.
  - For cohesive, rewrote so that footings on clay are always on 6 in. granular material, and $0.5 \sigma'_v$ is not only a limit when the granular material is present.
For footings that rest on clay, where footings are supported on at least 6.0 in. of compacted granular material, the sliding resistance may be taken as the lesser of:

- the cohesion of the clay, or
- one-half the normal stress on the interface between the footing and soil, as shown in Figure 10.6.3.4-1 for retaining walls.

The following notation shall be taken to apply to Figure 10.6.3.4-1:

\[ q_s = \text{unit shear resistance, equal to } S_u \text{ or } 0.5 \sigma'_v, \text{ whichever is less} \]
\[ R_s = \text{nominal sliding resistance between soil and foundation (kips) expressed as the shaded area under the } q_s \text{ diagram} \]
\[ S_u = \text{undrained shear strength (ksf)} \]
\[ \sigma'_v = \text{vertical effective stress (ksf)} \]
Specific Article Changes:

- 11.8.4.1 - Overall Stability
  - Commentary states the design procedure applied to Figure 3.11.5.6-3 (Teng, 1962) is for continuous walls embedded in granular soils.
  - States that similar procedures may also be performed for temporary walls using Figures 3.11.5.6-6 and 3.11.5.6-7 (Teng, 1962).
In designing permanent nongravity cantilevered walls with continuous vertical elements embedded in granular soil, the simplified earth pressure distributions in Figure 3.11.5.6-3 may be used with the following simplified design procedure (Teng, 1962):

- Determine the magnitude of lateral pressure on the wall due to earth pressure, surcharge loads and differential water pressure over the design height of the wall using \( k_{n1} \).
- Determine the magnitude of lateral pressure on the wall due to earth pressure, surcharge loads and differential water pressure over the design height of the wall using \( k_{n2} \).
- Determine in the following equation the value \( x \) as defined in Figure 3.11.5.6-3 to determine the distribution of net passive pressure in front of the wall below the design height:

\[
x = \left[ \frac{\gamma k_{a2} \gamma'_{s1} H}{(\phi k_{p2} - \gamma k_{a2}) \gamma'_{s2}} \right] \tag{C11.8.4.1-1}
\]
Note: For walls embedded in granular soil, refer to Figure 3.11.5.6.3-3 and use Figure 3.11.5.6-7 for retained cohesive soil when appropriate.

Figure 3.11.5.6-6—Unfactored Simplified Earth Pressure Distributions for Temporary Nongravity Cantilevered Walls with Continuous Vertical Wall Elements Embedded in Cohesive Soil and Retaining Granular Soil Modified after Teng (1962)

Figure 3.11.5.6-7—Unfactored Simplified Earth Pressure Distributions for Temporary Nongravity Cantilevered Walls with Continuous Vertical Wall Elements Embedded in Cohesive Soil and Retaining Cohesive Soil Modified after Teng (1962)
OTHER SUBJECTS FOR DISCUSSION

- Precast Wingwall Alternatives
- Prefabricated Modular Walls
- Predrilled Piles in Bedrock
- MSE Walls (various subjects)
- CCTV Tower Foundations
- Downdrag on Piles
OTHER SUBJECTS FOR DISCUSSION

- Settlement Time
- Spread Footing Foundations in BDM versus L&D Manual
- Micropiles
- Monotube Piles
Typically two options:

- **18” (WING “A”)**
  - 12” TYP.

**SLOPE PER PLANS**

**FREE DRAINING GRANULAR BACK**

**12”**

**SECTION A–A**

**WING WALL SECTION**

_TYPE ‘D’ = 6”–13”_

_TYPE ‘D’ = 3”–4”_
Plan Note D118:

Item 511 Wingwalls or Headwalls for 611 Items

“For Items 706.05, 706.051, 706.052 and 706.053 with a cast-in-place wingwall or headwall a precast alternative may be furnished per 602.03. The precast alternative will meet the cast-in-place structural design loadings, design height, and design length dimensions. ... (continued)
Plan Note D118:

- Item 511 Wingwalls or Headwalls for 611 Items

... Full compensation for the precast wingwall or headwall is the number of cubic yards of Item 511 and pounds of Item 509 for the corresponding cast-in-place structure.”
PRECAST WINGWALL ALTERNATIVES

- Plan Note D118 pulled and revised:
  - No permanent tension on epoxy anchors
  - Limit on angle between stream and walls with no stem-footing moment connection
  - Requirement of engineered drawings including: backfill details, sequence of construction, any changes to footing
  - Payment quantity a lump sum
o Precast Semigravity Wall suppliers encouraged to submit to Prefabricated Retaining Wall Systems Approval Process.

o Pre-Approval will be required within the next couple of years.
Prefabricated Modular Wall Supplemental Specification

- Currently SS 840 provides design and construction specification for MSE Walls
- Need new SS ### for Prefabricated Modular Walls, to provide design and construction specifications, so wall systems can be submitted to the Prefabricated Retaining Wall Systems Approval Process on an even basis.
Prefabricated Modular Wall suppliers encouraged to submit to Prefabricated Retaining Wall Systems Approval Process.

Pre-Approval will be required within the next couple of years.
Prefabricated Modular Walls Include

- Segmental (Modular Block) Walls
  - (Dry-cast concrete modular systems will not be allowed for highway construction)

- Large Gravity Block Walls

- Crib Walls

- Bin Walls

- Gabion Walls, per SS 838 (no preapproval)
**ITEM 507 – PREBORED HOLES, AS PER PLAN:**

PREBORED HOLES SHALL EXTEND AT LEAST TEN (10) FEET INTO BEDROCK AT EACH PILE. THE CONTRACTOR IS RESPONSIBLE FOR MAINTAINING AN OPEN HOLE.

BACKFILL THE VOID BETWEEN THE PILE AND THE PREBORED HOLE WITH CLASS QC MISC CONCRETE UP TO THE TOP OF ROCK ELEVATION. ABOVE THE TOP OF ROCK, BACKFILL THE VOID TO THE BOTTOM OF FOOTING ELEVATION WITH GRANULAR MATERIAL CONFORMING TO 703.11, STRUCTURAL BACKFILL TYPE 2, EXCEPT 100 PERCENT OF THE MATERIAL SHALL PASS THROUGH A ¾-INCH (19.0 mm) SIEVE. PAYMENT FOR THE PREBORED HOLES INCLUDES THE BACKFILL MATERIAL.

**ITEM 507 – STEEL PILES HP14X73, FURNISHED, AS PER PLAN:**

THIS WORK CONSISTS OF FURNISHING AND PLACING STEEL PILES INTO PREBORED HOLES. PLACE EACH PILE VERTICALLY WITHIN THE HOLE SO IT IS NOT INCLINED MORE THAN ONE INCH BETWEEN THE TOP AND BOTTOM. SUPPORT THE PILE SO THAT IT DOES NOT MOVE DURING PLACEMENT OF BACKFILL MATERIAL.

THE TOTAL FACTORED LOAD IS 454 KIPS PER PILE FOR THE ABUTMENT PILES, INCLUDING DOWNDRAG EFFECTS. THE ABUTMENT PILES INCLUDE AN ADDITIONAL 25 KIPS OF FACTORED LOAD PER PILE TO ACCOUNT FOR POSSIBLE DOWNDRAG LOADING.

REAR ABUTMENT PILES:
HP14x73 PILES 35 FEET LONG, ORDER LENGTH

FORWARD ABUTMENT PILES:
HP14x73 PILES 35 FEET LONG, ORDER LENGTH
PREDRILLED PILES IN BEDROCK

- Predrill (pre-bore) hole to larger diameter than pile
  - Item 507 Prebored Holes, As Per Plan
- Place H-pile in hole
  - Item 507 Steel Piles HP__x__, Furnished
- Concrete Pile to Top of Rock
  - Use Class CQ Misc. Concrete
  - Concrete included in Prebored Holes
PREDRILLED PILES IN BEDROCK

- Backfill to bottom of pile cap
  - If MSE Wall, sleeve pile and use granular fill per SS 840.03.K (include in MSE Wall)
  - Otherwise:
    - If conventional abutment, use LSM
    - If integral abutment, use granular
    - Backfill included in Prebored Holes Item
PREDRILLED PILES IN BEDROCK

- **Design Points:**
  - **Order Length:** Do not add an additional 5’ to Estimated Length
  - **Piles Furnished, As Per Plan**
    - Includes placement in prebored hole
  - **No Item 505 Pile Equipment Mobilization**
  - **No Item 507 Piles Driven (no driving)**
  - **Can use \( \varphi_c = 0.95 \) for steel only column in axial compression**
MSE WALL DESIGN CONSIDERATIONS

- MSE Walls
  - Acute angle limits for temporary construction
  - Bearing elevation, sliding considerations with granular undercut
  - Two-stage construction
  - Staged demolition: slope w/ short wire-faced wall rather than tall wall
MSE WALL DESIGN CONSIDERATIONS

- Acute angle limits:
  - For permanent construction, no acute corner angles (under 90°) allowed
  - For temporary construction (wire-faced MSE walls) acute angles to 45° allowed
  - Acute angles must be buried or demolished at end of construction
  - If sharper acute angles are required for MOT, then MSE walls cannot be utilized
MSE WALL DESIGN CONSIDERATIONS

- MSE granular undercut
  - Deeper granular undercut than standard SS 840.06.D foundation preparation is sometimes utilized
  - *This does not* change the foundation material to Granular Material Type C
  - Bearing is calculated at bottom of undercut with deeper embedment
  - Sliding is also calculated at bottom of undercut with passive resistance
MSE WALL DESIGN CONSIDERATIONS

- MSE granular undercut
MSE WALL DESIGN CONSIDERATIONS

- Two-stage MSE Walls

---

OGE Foundations and Retaining Walls Update • June 05, 2018
MSE WALL DESIGN CONSIDERATIONS

- Two-stage MSE Walls

**Figure 2 – Typical anchor assemblage (panel side coil loop to the right)**
MSE WALL DESIGN CONSIDERATIONS

- Two-stage MSE Walls
  - Avoid galvanic corrosion (bimetallic corrosion) in the connectors
    - All metallic components hot-dip galvanized or epoxy coated
  - Avoid macrocell corrosion between backfill and wall face
    - Assure low chloride and sulfate content of all wall backfill
  - Provide drainage protection for connector zone
MSE WALL DESIGN CONSIDERATIONS

- Staged bridge demolition
MSE WALL DESIGN CONSIDERATIONS

- Staged bridge demolition
CCTV TOWER FOUNDATIONS

- Office of Traffic Operations
  - Have installed numerous poles (several hundred?) around the state over the last several years
  - CCTV Camera on top of 70-foot tall spun precast concrete pole
  - For camera stability, Serviceability requirement of less than one inch deflection at top of pole in 30 mph wind
Service Limit State Controls

Still, check Strength Limit State at 100 mph wind

Strength Limit State Design by Broms Method, per “Design of Laterally Loaded Piles” (Broms, 1965)
CCTV TOWER FOUNDATIONS

Broms Method
CCTV TOWER FOUNDATIONS

Broms Method

\[ CDR = \frac{\varphi_{cp} M_R}{\gamma_{WS} M_{Pa}} \]

\[ R = \frac{1}{2} \gamma' k_p L^2 3D \]
Load and Resistance Factor Design (LRFD) Capacity-Demand Ratio (CDR)

Wind Load Factor $\gamma_p = \gamma_{WS} = 1.15$ (for 100 mph wind)
Passive Earth Pressure Resistance Factor $\varphi_{ep} = 0.75$

$$\frac{\gamma_{WS}}{\varphi_{ep}} \approx 1.5 \approx FS_{min}$$

$$CDR = \frac{\varphi_{ep} M_R}{\gamma_{WS} M_{Pa}}$$

$$CDR_{min} = 1.00$$
Strength Limit State Design by Broms Method

Define: Moment Arm (force F) = \( MA_F \)

Therefore, summing moments about the base of the Pole Foundation,

\[ MA_R = \frac{1}{3} L \quad \text{and} \quad MA_{Pa} = e + L \]

\[ M_R = MA_R R = (\frac{1}{3} L) (\frac{1}{2} \gamma' k_p L^2 3D) = \frac{1}{2} \gamma' k_p L^3 D \]

\[ M_{Pa} = MA_{Pa} P_a = (e + L) P_a \]

Therefore, for LRFD,

\[ CDR = \frac{\varphi_{ep} \frac{1}{2} \gamma' k_p L^3 D}{\gamma_{WS} (e + L) P_a} \]
CCTV TOWER FOUNDATIONS

Run LPILE analyses on Pole Foundation for Service Limit State
a. Pile Head Loading: $V = 244$ lb; $M = 95,727$ in-lb; $W = 11,061$ lb
b. Vary $L$ typically from 20 feet to point of failure in one-foot increments
c. Recalculate LPILE Pile Properties for each increment of $L$
d. Determine foundation length $L$ to achieve Fixed-Tip Condition, $L_{\text{Fixed}}$
e. If $L_{\text{Fixed}} > 25$ feet, then consider Non-Fixed Tip
f. Limit Deflection Ratio to $\text{Ratio}_{\Delta y} \leq 1.05$

Deflection Ratio

Ratio of deflection between $\Delta y_{\text{diff}}$ and $\Delta y_{\text{head}}$

$$\text{Ratio}_{\Delta y} = \frac{\Delta y_{\text{diff}}}{\Delta y_{\text{head}}}$$

<table>
<thead>
<tr>
<th></th>
<th>$\text{Ratio}_{\Delta y}$</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Fixed-Tip</strong></td>
<td>$&lt; 1.005$</td>
</tr>
<tr>
<td><strong>Non-Fixed Tip</strong></td>
<td>$1.005 \leq \text{Ratio}_{\Delta y} &lt; 1.150$</td>
</tr>
<tr>
<td><strong>Rigid Pole</strong></td>
<td>$\text{Ratio}_{\Delta y} \geq 1.150$</td>
</tr>
<tr>
<td><strong>Failure</strong></td>
<td>$\Delta y_{\text{head}}$ indefinite</td>
</tr>
</tbody>
</table>
DOWNDRAG ON PILES

- Downdrag on Piles (Service versus Strength)
  - Friction Piles are driven into the ground, continuously deforming the soil at the base, and sliding along the side soil.
  - Full geotechnical capacity (minus any setup) is mobilized at driving.
  - If the axial load exceeds this value, the pile will move again, remobilizing the full geotechnical capacity.
DOWNDRAG ON PILES

- **Typical 12-inch CIP Pipe Pile, 0.25” shell, 60 feet long, 35 ksi Steel**
  - 676.1 kips Nominal Axial Structural Resistance
  - Say, ultimate resistance is tip = 20 kips + side = 260 kips = 280 kips total (geotech.)
  - Say, settlement occurs in top 30 feet, neutral plane at 20 feet.
  - Friction in upper 20 feet = 80 kips
  - Resistance below 20 feet = 200 kips
Bridge Load
= 220 kips
Downdrag
= 80 kips
Total Load
= 300 kips
> Total Resistance
= 200 kips
DOWNDRAG ON PILES

- **Structural Resistance?**
  - Total Load = 300 kips < 676.1 kips **OK**

- **Geotechnical Resistance?**
  - Total Load = 300 kips > 200 kips **No Good**

- **But will the Pile Fail?**
  - Depends on definition of “failure”
  - Structurally, it is okay
  - Geotechnically, it will move
    (Service Limit State)
Bridge Load = 220 kips
Downdrag = 80 kips
Total Load = 300 kips
> Total Resistance = 200 kips
Bridge Load = 220 kips
Downdrag = 30 kips
Total Load = 250 kips
Total Resistance = 250 kips
DOWNDRAG ON PILES

- Pile will stop moving when neutral plane climbs to point where:
  bridge load + downdrag load = total resistance

- Pile only needs to move a small distance for this to happen (around 1 cm = 0.4 inch)
Total Pile settlement will be settlement of soil below neutral plane + ≈0.4 inch

This is a Service Limit issue

See Amal Goza at OTEC 2018

See GEC 12 FHWA-NHI-16-009

See MnDOT Geotechnical Manual
A profile of settling soil is typically divided into multiple layers with differing consolidation characteristics:
- layer thickness
- overburden pressure
- maximum past pressure (OCR, virgin)
- drainage paths
- $c_c$, $c_r$, $C'$, $c_v$ (magnitude and rate)
SETTLEMENT TIME

- Some layers will take a long time to consolidate, and some will do so nearly immediately.
- The slowest layer does not control the time to achieve consolidation (waiting period), i.e. 90% or 0.4” remaining.
- Consider all layers individually, and then build a composite of behavior.
- Consider the following 50-foot profile:
## Settlement Time

<table>
<thead>
<tr>
<th>Layer</th>
<th>Description</th>
<th>Drainage</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>A-7-6</td>
<td>Non-Consolidating</td>
</tr>
<tr>
<td>2</td>
<td>A-1-b</td>
<td>Non-Consolidating</td>
</tr>
<tr>
<td>3</td>
<td>A-6b</td>
<td>Cohesive (Double Drainage)</td>
</tr>
<tr>
<td>4</td>
<td>A-3a</td>
<td>Granular</td>
</tr>
<tr>
<td>5</td>
<td>A-1-b</td>
<td>Granular</td>
</tr>
<tr>
<td>6</td>
<td>A-4b</td>
<td>Cohesive (Double Drainage)</td>
</tr>
<tr>
<td>7</td>
<td>A-6b</td>
<td>Cohesive (Single Drainage)</td>
</tr>
<tr>
<td>8</td>
<td>A-6a</td>
<td>Cohesive (Single Drainage)</td>
</tr>
<tr>
<td>9</td>
<td>A-4a</td>
<td>Cohesive (Single Drainage)</td>
</tr>
</tbody>
</table>
SETTLEMENT TIME

- Determine the amount of settlement completed and remaining for each layer at specific time intervals.
- At each time interval, sum the settlement completed and remaining for all layers, and plot this result versus time.
- Determine the $T_{90}$ time for the entire soil profile.
SETTLEMENT TIME

- If the total settlement remaining at the $T_{90}$ time is greater than 0.4 inches, then use the $T_{90}$ time for the waiting period, rounding up to an even interval of days/weeks/months.

- If the total settlement remaining at the $T_{90}$ time is less than 0.4 inches, then find the time at which 0.4 inches remains, and use this time for the waiting period, once again rounding up to an even interval of days/weeks/months.
**BDM VS L&D MANUAL: SPREAD FOOTINGS**

- **BDM vs L&D Manual regarding Spread Footing Foundations**

<table>
<thead>
<tr>
<th></th>
<th>Spread Footings on Soil</th>
<th>Spread Footings on Erodible Rock</th>
<th>Spread Footings on Non-Erodible Rock</th>
<th>Driven Pile Foundations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bridges (BDM)</td>
<td>X</td>
<td>7’ below Thalweg Elevation</td>
<td>On Rock, Dowels for Lateral</td>
<td>202.2.3.2.h</td>
</tr>
<tr>
<td>Three-Sided Culverts (L&amp;D Vol.2)</td>
<td>See OGE HEC-18, HEC-23</td>
<td>HEC-18, HEC-23</td>
<td>HEC-18</td>
<td>BDM, HEC-23</td>
</tr>
</tbody>
</table>

- **Multiple-span three-sided culverts are considered as bridges**
BDM VS L&D MANUAL: SPREAD FOOTINGS

- **BDM Spread Footings on Rock**
  - **BDM Section 202.2.3.1.b** is strict on scour resistant (non-erodible) rock:
    - \( Q_u \geq 2,500 \text{ psi} \) (“slightly strong” rock)
    - Remain intact when immersed in water (e.g. insoluble) **no bucket slaking**
    - Unit weight \( \geq 150 \text{ pcf} \)
    - Joint or bedding plane spacing that define large blocks (> 4-ft)
BDM VS L&D MANUAL: SPREAD FOOTINGS

○ BDM Spread Footings on Rock

○ BDM Section 202.2.3.1.b eliminates nearly all rock in Ohio as non-erodible

○ We propose to change the erodibility requirements to HEC-18, FHWA-HIF-12-003
  ○ Erodibility Index for strong and durable rocks (Quarrying and Plucking)
  ○ Abrasion Scour of rock due to bedload
  ○ We still need to define the use of these for Ohio
HEC-18 Scour of Soils

- Granular Soils: \( \tau_c \) (Pa) = \( D_{50} \) (mm)
- Holds down to \( D_{50} \approx 0.1 \) mm
- HEC-18 Scour of Soils
  - Cohesive Soils not so straightforward

Figure 4.6. Critical shear stress vs. particle grain size (Briaud et al. 2011).
FHWA-HRT-15-033 “Scour in Cohesive Soils”

\[ \tau_c = \alpha \left( \frac{w}{F} \right)^{-2.0} PI^{1.3} q_u^{0.4} \]

**Figure 54. Equation. Predictive relation for critical shear stress.**

Where:
- \( \tau_c \) = Critical shear stress, lbf/ft\(^2\) (Pa).
- \( w \) = Water content, dimensionless ratio.
- \( F \) = Fraction of fines by mass (<75μm), dimensionless ratio.
- \( PI \) = Plasticity index, dimensionless ratio.
- \( q_u \) = Unconfined compressive strength, lbf/ft\(^2\) (Pa).
- \( \alpha \) = Unit conversion constant, 0.01 in U.S. customary units and 0.1 in S.I.
Erosion Rate (Briaud, 2011)

Figure 6.11. Generalized relationships for scour in cohesive materials (Briaud et al. 2011).
Erosion Rate (Briaud, 2011)

Figure 6.11. Generalized relationships for scour in cohesive materials (Briaud et al. 2011).
Erosion Rate (Briaud, 2011)

Figure 6.11. Generalized relationships for scour in cohesive materials (Briaud et al. 2011).
BDM VS L&D MANUAL: SPREAD FOOTINGS

Flood Event Hydrograph

\[ Q_{\text{max}} \]

\[ Q \]

\[ q_o \]

\[ t_o \]

\[ t \]

\[ t_{\text{equiv}} \]

\[ t_r \]
BDM VS L&D MANUAL: SPREAD FOOTINGS

- HEC-23, FHWA-NHI-09-112, Bridge Scour and Stream Instability Countermeasures
Micropiles

- Typically used for:
  - Underpinning Inadequate Foundations
  - Supplementing Existing Foundations
  - Adding Lateral Stability
  - Tight Overhead Working Conditions
- Generally not a cost-effective new-build foundation option
MICROPILES

- Design Micropiles per:
  - AASHTO LRFD Bridge Design Specifications Article 10.9 (LRFD)
  - and AASHTO LRFD Bridge Construction Specifications Section 33 (LRFD)
  - or FHWA-NHI-05-039 “Micropile Design and Construction Reference Manual” (ASD)
Mono tube Piles

- It consists of tapered fluted steel shell without mandrel. The pile shells are driven to the required depth and then the interior of the shell is inspected.
- The shell is then filled with concrete and the excess shell if any is cut off. The extension of shell up to the required length is carried out by the welding.
- Shell are rigid and water tight.
- Suitable for a wide variety of soil conditions ranging from end bearing to friction load carrying piles.

---

**MONOTUBE PILES**

The Original Tapered Fluted Steel Pile

---

**MONOTUBE PILES**

Standard Monotube Weights and Volumes

<table>
<thead>
<tr>
<th>TYPE</th>
<th>SIZE POINT DIAMETER x BUTT DIAMETER x LENGTH</th>
<th>WEIGHT (N) per m</th>
<th>EST. CONC. VOL. m³</th>
</tr>
</thead>
<tbody>
<tr>
<td>F Taper</td>
<td>216 mm x 305 mm x 7.62 m</td>
<td>249</td>
<td>0.329</td>
</tr>
<tr>
<td></td>
<td>203 mm x 305 mm x 9.14 m</td>
<td>233</td>
<td>0.420</td>
</tr>
<tr>
<td></td>
<td>216 mm x 350 mm x 12.19 m</td>
<td>277</td>
<td>0.726</td>
</tr>
<tr>
<td></td>
<td>203 mm x 406 mm x 18.29 m</td>
<td>292</td>
<td>1.284</td>
</tr>
<tr>
<td></td>
<td>203 mm x 457 mm x 22.86 m</td>
<td>-</td>
<td>1.979</td>
</tr>
<tr>
<td>J Taper</td>
<td>203 mm x 305 mm x 5.16 m</td>
<td>240</td>
<td>0.244</td>
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<tr>
<td></td>
<td>203 mm x 356 mm x 7.62 m</td>
<td>263</td>
<td>0.443</td>
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<tr>
<td></td>
<td>203 mm x 406 mm x 10.65 m</td>
<td>292</td>
<td>0.726</td>
</tr>
<tr>
<td></td>
<td>203 mm x 457 mm x 12.19 m</td>
<td>-</td>
<td>1.047</td>
</tr>
<tr>
<td>Y Taper</td>
<td>203 mm x 305 mm x 3.05 m</td>
<td>246</td>
<td>0.138</td>
</tr>
<tr>
<td></td>
<td>203 mm x 356 mm x 4.57 m</td>
<td>277</td>
<td>0.260</td>
</tr>
<tr>
<td></td>
<td>203 mm x 406 mm x 6.10 m</td>
<td>292</td>
<td>0.428</td>
</tr>
<tr>
<td></td>
<td>203 mm x 457 mm x 7.62 m</td>
<td>-</td>
<td>0.857</td>
</tr>
</tbody>
</table>

**Extensions (Overall Length 0.305 m Greater than indicated)**

<table>
<thead>
<tr>
<th>TYPE</th>
<th>DIAMETER + LENGTH</th>
<th>9 GA</th>
<th>7 GA</th>
<th>5 GA</th>
<th>3 GA</th>
<th>m³</th>
</tr>
</thead>
<tbody>
<tr>
<td>N 12</td>
<td>305 mm x 305 mm x 6.10 / 12.19 m</td>
<td>290</td>
<td>350</td>
<td>409</td>
<td>492</td>
<td>0.065</td>
</tr>
<tr>
<td>N 14</td>
<td>356 mm x 356 mm x 6.10 / 12.19 m</td>
<td>390</td>
<td>423</td>
<td>466</td>
<td>598</td>
<td>0.088</td>
</tr>
<tr>
<td>N 16</td>
<td>406 mm x 406 mm x 6.10 / 12.19 m</td>
<td>409</td>
<td>482</td>
<td>569</td>
<td>671</td>
<td>0.113</td>
</tr>
<tr>
<td>N 18</td>
<td>457 mm x 457 mm x 6.10 / 12.19 m</td>
<td>-</td>
<td>555</td>
<td>642</td>
<td>750</td>
<td>0.145</td>
</tr>
</tbody>
</table>
MONOTUBE PILES

- Monotube Piles
  - Monotube Pile Corporation out of business
  - Removing vertical fluting, circumferential corrugations, and tapered piles (B,C,D) from 507.06
  - No more replacement of CIP Pipe piles with different types in construction
  - No way to track these replacements, and evidence shows they do not necessarily perform the same (open to claims)
MONOTUBE PILES

- **Miscellaneous Piles**
  - We rarely (if ever) use the piles being removed from 507.06
  - However, future plan is to add a section to C&MS 507 for Miscellaneous Piles, including: Timber, Tapered, Fluted, Corrugated, and Precast Concrete
  - For now, these pile types can still be specified under 507E98000 PILING, MISC.: with appropriate plan notes
MONOTUBE PILES

- **DRIVEN Monotube Pile model**
  - Allows negative cumulative side friction (physically not possible)
  - Slight Taper does not approach behavior of straight pile
  - Magnifies granular side frictional effects in the taper length approximately 2.25x
  - Often underpredicts length substantially
  - **ODOT will disallow use of this analysis model for design**
MONOTUBE PILES

- **Monotube Pile analysis**
  - Analyze as standard CIP Pipe Pile at diameter of straight (non-tapered) part
  - Add an additional 5 feet to Estimated Length (beyond CIP Pipe Estimated)
  - Ensure tapered length starts a minimum of 5 feet below ground surface or pile cap
  - Monotubes are fluted tapered piles, but this also applies to smooth tapered piles
  - Does not apply to Raymond tapered piles