Appendix N

Structure Design Calculations
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</tbody>
</table>
SECTION 1: PURPOSE AND SCOPE:

The Milwaukee Dredged Material Management Facility (DMMF) is planned to be constructed on the north side of the existing Jones Island DMDF to provide additional capacity for the disposal of dredged material. Milwaukee Harbor is located on the west shore of Lake Michigan. The new DMMF structure will be comprised of cellular coffer dam wall and a small section of rubble mound dike. The completed and filled DMMF will be used as a berthing facility for Port Milwaukee. Site specific geotechnical borings are reviewed and required geotechnical parameters are extracted to design the structural components of the project.

SECTION 2: ASSUMPTIONS

Assumptions are discussed and justified as deemed necessary throughout this calculation.

SECTION 3: METHODOLOGY AND ACCEPTANCE CRITERIA

The measured N-value is the number of blows required to drive the split spoon sampler a distance of 300 mm (12 in). For routine engineering practice in the United States, correlations for engineering properties are based on SPT N values measured based on a system which is 60 percent efficient (ER=60 percent). These blow counts will be corrected for the energy ratio, borehole diameter, sampling method, and rod length. Standard Penetration Test (SPT) blow counts require correction prior to utilization in soil characterization and determination of properties and behavior.

Since N-values of similar materials increase with increasing effective overburden stress, the corrected blowcount (N 60) is often normalized to 1-atmosphere (or about 100 kPa (14.5 psi)) effective overburden stress using overburden normalization schemes.
SECTION 4: DESIGN INPUTS

Geotechnical log of all the borings is provided in Reference 1a. A profile of uncorrected N-SPT and soil description for all the borings is presented in Attachment 8.1.

SECTION 5: CALCULATIONS

Section 5.1 SPT Correction Factors
Correction factors are calculated following the methodology described earlier. Following table is from Chapter 4 of Reference 2a.

<table>
<thead>
<tr>
<th>Factor</th>
<th>Equipment Variable</th>
<th>Term</th>
<th>Correction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Energy Ratio</td>
<td>Donut Hammer</td>
<td></td>
<td>( C_E = \frac{ER}{60} )</td>
</tr>
<tr>
<td></td>
<td>Safety Hammer</td>
<td></td>
<td>0.5 to 1.0(^{1})</td>
</tr>
<tr>
<td></td>
<td>Automatic Hammer</td>
<td></td>
<td>0.7 to 1.2(^{1})</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>0.8 to 1.5(^{1})</td>
</tr>
<tr>
<td>Borehole Diameter</td>
<td>65 to 115 mm</td>
<td>( C_B )</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>150 mm</td>
<td></td>
<td>1.05</td>
</tr>
<tr>
<td></td>
<td>200 mm</td>
<td></td>
<td>1.15</td>
</tr>
<tr>
<td>Sampling method</td>
<td>Standard sampler</td>
<td>( C_S )</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>Non-standard sampler</td>
<td></td>
<td>1.1 to 1.3</td>
</tr>
<tr>
<td>Rod Length</td>
<td>3 to 4 m</td>
<td>( C_R )</td>
<td>0.75</td>
</tr>
<tr>
<td></td>
<td>4 to 6 m</td>
<td></td>
<td>0.85</td>
</tr>
<tr>
<td></td>
<td>6 to 10 m</td>
<td></td>
<td>0.95</td>
</tr>
<tr>
<td></td>
<td>10 to &gt;30 m</td>
<td></td>
<td>1.0</td>
</tr>
</tbody>
</table>

\(^{1}\) Values presented are for guidance only. Actual ER values should be measured per ASTM D 4633

Among the most important corrections is the energy correction required to adjust the blow counts to 60% energy efficiency. Energy ratio for Automatic Hammer per soil boring log. Additionally, Reference 1c provides the efficiency of the Acker Renegade hammer as 91.8%.

\( C_E := \frac{91.8}{60} \cdot 1 = 1.53 \) Borehole diameter 65 to 115 mm (2 1/2 in to 4 1/2 in) for 4" nominal casing (4.5" OD).

\( C_B := 1.0 \) Sampling method - standard sampler

\( C_S := 1.0 \) Rod Length, 3 to 4 m (9.9 to 13.1 ft) - minimum length of rod is 10 feet (two pieces of 5 feet each). Drilling is offshore with rod length in excess of 30 ft.
\[ C = C_f \cdot C_R \cdot C_S \cdot C_R = 1.454 \]

N 60 = N meas \times C

### 5.1.1 Boring SB-8

<table>
<thead>
<tr>
<th>Elevation of Sample from LWD</th>
<th>N Average</th>
<th>Sample Description</th>
<th>N Corrected</th>
</tr>
</thead>
<tbody>
<tr>
<td>(d_1 = -15.1 \text{ ft})</td>
<td>(N_1 := 0)</td>
<td>CL</td>
<td>(N_{1,60} := N_1 \cdot C)</td>
</tr>
<tr>
<td>(d_2 = -21.8 \text{ ft})</td>
<td>(N_2 := 0)</td>
<td>CL</td>
<td>(N_{2,60} := N_2 \cdot C)</td>
</tr>
<tr>
<td>(d_3 = -26.8 \text{ ft})</td>
<td>(N_3 := 0)</td>
<td>ML / CL, CL</td>
<td>(N_{3,60} := N_3 \cdot C)</td>
</tr>
<tr>
<td>(d_4 = -31.8 \text{ ft})</td>
<td>(N_4 := 9)</td>
<td>CL / ML</td>
<td>(N_{4,60} := N_4 \cdot C)</td>
</tr>
<tr>
<td>(d_5 = -36.8 \text{ ft})</td>
<td>(N_5 := 9)</td>
<td>CL / ML, CL</td>
<td>(N_{5,60} := N_5 \cdot C)</td>
</tr>
<tr>
<td>(d_6 = -41.8 \text{ ft})</td>
<td>(N_6 := 9)</td>
<td>CL</td>
<td>(N_{6,60} := N_6 \cdot C)</td>
</tr>
<tr>
<td>(d_7 = -46.8 \text{ ft})</td>
<td>(N_7 := 9)</td>
<td>SM, CL</td>
<td>(N_{7,60} := N_7 \cdot C)</td>
</tr>
<tr>
<td>(d_8 = -51.3 \text{ ft})</td>
<td>(N_8 := 9)</td>
<td>SW, CL</td>
<td>(N_{8,60} := N_8 \cdot C)</td>
</tr>
<tr>
<td>(d_9 = -56.8 \text{ ft})</td>
<td>(N_9 := 9)</td>
<td>SW / SM, SW / SM, CL</td>
<td>(N_{9,60} := N_9 \cdot C)</td>
</tr>
<tr>
<td>(d_{10} = -61.3 \text{ ft})</td>
<td>(N_{10} := 22)</td>
<td>SL</td>
<td>(N_{10,60} := N_{10} \cdot C)</td>
</tr>
<tr>
<td>(d_{11} = -66.8 \text{ ft})</td>
<td>(N_{11} := 4)</td>
<td>SC/CL</td>
<td>(N_{11,60} := N_{11} \cdot C)</td>
</tr>
</tbody>
</table>
Note:

1. *Middle of sample is considered, see Attachment 8.1.*
2. Corrected SPTs show that using 2 layers are allowed.

### 5.1.2 Boring SB-7

<table>
<thead>
<tr>
<th>Elevation of Sample from LWD</th>
<th>N Average</th>
<th>Sample Description</th>
<th>N Corrected</th>
</tr>
</thead>
<tbody>
<tr>
<td>(d71 = -18.4 \text{ ft})</td>
<td>(N_{71} := 0)</td>
<td>ML, CL</td>
<td>(N_{71,60} := N_{71} \cdot C) (\text{round}\ (N_{71,60}) = 0)</td>
</tr>
<tr>
<td>(d72 = -21.9 \text{ ft})</td>
<td>(N_{72} := 8)</td>
<td>ML / SW</td>
<td>(N_{72,60} := N_{72} \cdot C) (\text{round}\ (N_{72,60}) = 12)</td>
</tr>
<tr>
<td>(d73 = -26.9 \text{ ft})</td>
<td>(N_{73} := 7)</td>
<td>CL</td>
<td>(N_{73,60} := N_{73} \cdot C) (\text{round}\ (N_{73,60}) = 10)</td>
</tr>
<tr>
<td>(d74 = -31.9 \text{ ft})</td>
<td>(N_{74} := 9)</td>
<td>CL</td>
<td>(N_{74,60} := N_{74} \cdot C) (\text{round}\ (N_{74,60}) = 13)</td>
</tr>
<tr>
<td>(d75 = -35.15 \text{ ft})</td>
<td>(N_{75} := 9)</td>
<td>CL</td>
<td>(N_{75,60} := N_{75} \cdot C) (\text{round}\ (N_{75,60}) = 13)</td>
</tr>
<tr>
<td>(d76 = -41.9 \text{ ft})</td>
<td>(N_{76} := 7)</td>
<td>CL</td>
<td>(N_{76,60} := N_{76} \cdot C) (\text{round}\ (N_{76,60}) = 10)</td>
</tr>
<tr>
<td>(d77 = -46.9 \text{ ft})</td>
<td>(N_{77} := 12)</td>
<td>CL</td>
<td>(N_{77,60} := N_{77} \cdot C) (\text{round}\ (N_{77,60}) = 13)</td>
</tr>
<tr>
<td>(d78 = -51.9 \text{ ft})</td>
<td>(N_{78} := 11)</td>
<td>SW, CL</td>
<td>(N_{78,60} := N_{78} \cdot C) (\text{round}\ (N_{78,60}) = 16)</td>
</tr>
<tr>
<td>(d79 = -56.9 \text{ ft})</td>
<td>(N_{79} := 17)</td>
<td>CL</td>
<td>(N_{79,60} := N_{79} \cdot C) (\text{round}\ (N_{79,60}) = 25)</td>
</tr>
<tr>
<td>(d710 := -61.9 \text{ ft})</td>
<td>(N_{710} := 27)</td>
<td>SW, CL</td>
<td>(N_{710,60} := N_{710} \cdot C) (\text{round}\ (N_{710,60}) = 39)</td>
</tr>
<tr>
<td>(d711 := -66.9 \text{ ft})</td>
<td>(N_{711} := 35)</td>
<td>SW, CL</td>
<td>(N_{711,60} := N_{711} \cdot C) (\text{round}\ (N_{711,60}) = 51)</td>
</tr>
</tbody>
</table>
Milwaukee DMMF Project
Structural Soil Properties

Calc. No.: DMMF-SD-01
Project Number: 19W012
Revision: 0A

5.1.2 Boring SB-6

<table>
<thead>
<tr>
<th>Elevation of Sample from LWD</th>
<th>N Average</th>
<th>Sample Description</th>
<th>N Corrected</th>
</tr>
</thead>
<tbody>
<tr>
<td>d712 := −71.9 ft</td>
<td>N_{712} := 27</td>
<td>CL</td>
<td>N_{712,60} := N_{712} \cdot C \quad \text{round}\left(N_{712,60}\right) = 39</td>
</tr>
<tr>
<td>d713 := −76.9 ft</td>
<td>N_{713} := 54</td>
<td>CL</td>
<td>N_{713,60} := N_{713} \cdot C \quad \text{round}\left(N_{713,60}\right) = 78</td>
</tr>
<tr>
<td>d714 := −81.9 ft</td>
<td>N_{714} := 60</td>
<td>CL/ML, CL</td>
<td>N_{714,60} := N_{714} \cdot C \quad \text{round}\left(N_{714,60}\right) = 87</td>
</tr>
</tbody>
</table>

Prepared by: Mark Salehi  Date: 6/28/2020
Reviewed by: Soren Morch  Date: 6/28/2020
### Milwaukee DMMF Project
#### Structural Soil Properties

**Calc. No.:** DMMF-SD-01  
**Project Number:** 19W012  
**Revision:** 0A

\[
\begin{align*}
  d_{611} &= -68.8 \text{ ft} & N_{611} &= 17 \quad \text{SW, CL} & N_{611,60} &= N_{611} \times C \\
  d_{612} &= -73.8 \text{ ft} & N_{612} &= 14 \quad \text{CL} & N_{612,60} &= N_{612} \times C \\
  d_{613} &= -78.8 \text{ ft} & N_{613} &= 23 \quad \text{CL} & N_{613,60} &= N_{613} \times C \\
  d_{614} &= -83.8 \text{ ft} & N_{614} &= 10 \quad \text{CL/ML, CL} & N_{614,60} &= N_{614} \times C \\
  d_{615} &= -88.8 \text{ ft} & N_{615} &= 9 \quad \text{CL} & N_{615,60} &= N_{615} \times C \\
  d_{616} &= -93.8 \text{ ft} & N_{616} &= 17 \quad \text{CL/ML, CL} & N_{616,60} &= N_{614} \times C
\end{align*}
\]

\[
\begin{align*}
  \text{round} \left( N_{611,60} \right) &= 25 \\
  \text{round} \left( N_{612,60} \right) &= 20 \\
  \text{round} \left( N_{613,60} \right) &= 33 \\
  \text{round} \left( N_{614,60} \right) &= 15 \\
  \text{round} \left( N_{615,60} \right) &= 33 \\
  \text{round} \left( N_{616,60} \right) &= 15
\end{align*}
\]

#### 5.1.2 Boring SB-5

<table>
<thead>
<tr>
<th>Elevation of Sample from LWD</th>
<th>N Average</th>
<th>Sample Description</th>
<th>N Corrected</th>
</tr>
</thead>
<tbody>
<tr>
<td>( d_{51} = -23.95 \text{ ft} )</td>
<td>( N_{51} = 9 )</td>
<td>ML, CL</td>
<td>( N_{51,60} = N_{51} \times C ) ( \text{round} \left( N_{51,60} \right) = 13 )</td>
</tr>
<tr>
<td>( d_{52} = -26.45 \text{ ft} )</td>
<td>( N_{52} = 9 )</td>
<td>ML / SW</td>
<td>( N_{52,60} = N_{52} \times C ) ( \text{round} \left( N_{52,60} \right) = 13 )</td>
</tr>
<tr>
<td>( d_{53} = -31.45 \text{ ft} )</td>
<td>( N_{53} = 6 )</td>
<td>CL</td>
<td>( N_{53,60} = N_{53} \times C ) ( \text{round} \left( N_{53,60} \right) = 9 )</td>
</tr>
<tr>
<td>( d_{54} = -36.45 \text{ ft} )</td>
<td>( N_{54} = 7 )</td>
<td>CL</td>
<td>( N_{54,60} = N_{54} \times C ) ( \text{round} \left( N_{54,60} \right) = 10 )</td>
</tr>
<tr>
<td>( d_{55} = -41.45 \text{ ft} )</td>
<td>( N_{55} = 15 )</td>
<td>CL</td>
<td>( N_{55,60} = N_{55} \times C ) ( \text{round} \left( N_{55,60} \right) = 22 )</td>
</tr>
<tr>
<td>( d_{56} = -46.45 \text{ ft} )</td>
<td>( N_{56} = 15 )</td>
<td>CL</td>
<td>( N_{56,60} = N_{56} \times C ) ( \text{round} \left( N_{56,60} \right) = 22 )</td>
</tr>
</tbody>
</table>

Prepared by: Mark Salehi  
Date: 6/28/2020

Reviewed by: Soren Morch  
Date: 6/28/2020
Section 5.2 Preliminary Soil Properties

Table 3-1 of Reference 2b shows approximate relationships between the relative density, standard penetration resistance (SPT), angle of internal friction, and unit weight of granular soils.

<table>
<thead>
<tr>
<th>Granular Soil Properties (after Teng 1962)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compactness</td>
</tr>
<tr>
<td>--------------</td>
</tr>
<tr>
<td>Very Loose</td>
</tr>
<tr>
<td>Loose</td>
</tr>
<tr>
<td>Medium</td>
</tr>
<tr>
<td>Dense</td>
</tr>
<tr>
<td>Very Dense</td>
</tr>
</tbody>
</table>

5.2.1 Boring SB-8

<table>
<thead>
<tr>
<th>Elevation of Sample MLLW</th>
<th>N Corrected</th>
<th>Friction Angle or cohesion</th>
<th>Moist Unit Weight</th>
<th>Sub. Unit Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>φ₁ := 0 deg</td>
<td>γ₁ := 107 pcf</td>
<td>γ₁, sub := 45 pcf</td>
</tr>
<tr>
<td>d₁ = -15.1 ft</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>round (N_{1.60}) = 0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>d₂ = -21.8 ft</td>
<td></td>
<td>φ₂ := 0 deg</td>
<td>γ₂ := 107 pcf</td>
<td>γ₂, sub := 45 pcf</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>round (N_{2.60}) = 0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>d₃ = -26.8 ft</td>
<td></td>
<td>φ₃ := 0 deg</td>
<td>γ₃ := 107 pcf</td>
<td>γ₃, sub := 45 pcf</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>round (N_{3.60}) = 0</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
5.2.2 Boring SB-7

<table>
<thead>
<tr>
<th>Elevation of Sample MLLW</th>
<th>Friction Angle or cohesion</th>
<th>Moist Unit Weight</th>
<th>Sub. Unit Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$d_{71} = -18.4 \text{ ft}$</td>
<td>$\phi_{71} = 0 \text{ deg}$</td>
<td>$\gamma_{71} = 107 \text{ pcf}$</td>
<td>$\gamma_{71,\text{sub}} = 45 \text{ pcf}$</td>
</tr>
<tr>
<td>$d_{72} = -21.9 \text{ ft}$</td>
<td>$\phi_{72} = 31.5 \text{ deg}$</td>
<td>$\gamma_{72} = 111 \text{ pcf}$</td>
<td>$\gamma_{72,\text{sub}} = 63 \text{ pcf}$</td>
</tr>
<tr>
<td>$d_{73} = -26.9 \text{ ft}$</td>
<td>$\phi_{73} = 30 \text{ deg}$</td>
<td>$\gamma_{73} = 125 \text{ pcf}$</td>
<td>$\gamma_{73,\text{sub}} = 65 \text{ pcf}$</td>
</tr>
<tr>
<td>$d_{74} = -31.9 \text{ ft}$</td>
<td>$\phi_{74} = 34 \text{ deg}$</td>
<td>$\gamma_{74} = 128 \text{ pcf}$</td>
<td>$\gamma_{74,\text{sub}} = 67 \text{ pcf}$</td>
</tr>
<tr>
<td>$d_{75} = -35.15 \text{ ft}$</td>
<td>$\phi_{75} = 34 \text{ deg}$</td>
<td>$\gamma_{75} = 128 \text{ pcf}$</td>
<td>$\gamma_{75,\text{sub}} = 67 \text{ pcf}$</td>
</tr>
</tbody>
</table>
\[
d76 = -41.9 \text{ ft} \quad \text{round} \left( N_{76,60} \right) = 10 \quad \phi_{76} := 30 \text{ deg} \quad \gamma_{76} := 125 \text{ pcf} \quad \gamma_{76,\text{sub}} := 65 \text{ pcf}
\]
\[
d77 = -46.9 \text{ ft} \quad \text{round} \left( N_{77,60} \right) = 17 \quad \phi_{77} := 34.5 \text{ deg} \quad \gamma_{77} := 129 \text{ pcf} \quad \gamma_{77,\text{sub}} := 68.5 \text{ pcf}
\]
\[
d78 = -51.9 \text{ ft} \quad \text{round} \left( N_{78,60} \right) = 16 \quad \phi_{78} := 34 \text{ deg} \quad \gamma_{78} := 128 \text{ pcf} \quad \gamma_{78,\text{sub}} := 68 \text{ pcf}
\]
\[
d79 = -56.9 \text{ ft} \quad \text{round} \left( N_{79,60} \right) = 25 \quad \phi_{79} := 36 \text{ deg} \quad \gamma_{79} := 130 \text{ pcf} \quad \gamma_{79,\text{sub}} := 70 \text{ pcf}
\]
\[
d710 = -61.9 \text{ ft} \quad \text{round} \left( N_{710,60} \right) = 39 \quad \phi_{710} := 38 \text{ deg} \quad \gamma_{710} := 135 \text{ pcf} \quad \gamma_{710,\text{sub}} := 75 \text{ pcf}
\]
\[
d711 = -66.9 \text{ ft} \quad \text{round} \left( N_{711,60} \right) = 51 \quad \phi_{711} := 40 \text{ deg} \quad \gamma_{711} := 140 \text{ pcf} \quad \gamma_{711,\text{sub}} := 80 \text{ pcf}
\]
\[
d712 = -71.9 \text{ ft} \quad \text{round} \left( N_{712,60} \right) = 39 \quad \phi_{712} := 38 \text{ deg} \quad \gamma_{712} := 135 \text{ pcf} \quad \gamma_{712,\text{sub}} := 75 \text{ pcf}
\]
\[
d713 = -76.9 \text{ ft} \quad \text{round} \left( N_{713,60} \right) = 78 \quad \phi_{713} := 40 \text{ deg} \quad \gamma_{713} := 140 \text{ pcf} \quad \gamma_{713,\text{sub}} := 80 \text{ pcf}
\]
\[
d714 = -81.9 \text{ ft} \quad \text{round} \left( N_{714,60} \right) = 87 \quad \phi_{714} := 40 \text{ deg} \quad \gamma_{714} := 140 \text{ pcf} \quad \gamma_{714,\text{sub}} := 80 \text{ pcf}
\]

### 5.2.2 Boring SB-6

<table>
<thead>
<tr>
<th>Elevation of Sample MLLW</th>
<th>Friction Angle or cohesion</th>
<th>Moist Unit Weight</th>
<th>Sub. Unit Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>(d61 = -21.3 \text{ ft})</td>
<td>(\phi_{61} := 0 \text{ deg})</td>
<td>(\gamma_{61} := 107 \text{ pcf})</td>
<td>(\gamma_{61,\text{sub}} := 45 \text{ pcf})</td>
</tr>
<tr>
<td>(d62 = -23.8 \text{ ft})</td>
<td>(\phi_{62} := 0 \text{ deg})</td>
<td>(\gamma_{62} := 107 \text{ pcf})</td>
<td>(\gamma_{62,\text{sub}} := 45 \text{ pcf})</td>
</tr>
<tr>
<td>(d63 = -28.8 \text{ ft})</td>
<td>(\phi_{63} := 28.5 \text{ deg})</td>
<td>(\gamma_{63} := 112 \text{ pcf})</td>
<td>(\gamma_{63,\text{sub}} := 62 \text{ pcf})</td>
</tr>
<tr>
<td>(d64 = -33.8 \text{ ft})</td>
<td>(\phi_{64} := 34 \text{ deg})</td>
<td>(\gamma_{64} := 128 \text{ pcf})</td>
<td>(\gamma_{64,\text{sub}} := 67 \text{ pcf})</td>
</tr>
<tr>
<td>Depth (ft)</td>
<td>Soil Properties</td>
<td></td>
<td></td>
</tr>
<tr>
<td>-----------</td>
<td>-----------------</td>
<td></td>
<td></td>
</tr>
<tr>
<td>38.5</td>
<td>$d_{65}$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>43.8</td>
<td>$d_{66}$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>48.8</td>
<td>$d_{67}$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>53.8</td>
<td>$d_{68}$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>58.8</td>
<td>$d_{69}$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>63.8</td>
<td>$d_{70}$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>68.8</td>
<td>$d_{71}$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>73.8</td>
<td>$d_{72}$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>78.8</td>
<td>$d_{73}$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>83.8</td>
<td>$d_{74}$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>88.8</td>
<td>$d_{75}$</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

- $d_{65} = 38.5$ ft
- $d_{66} = 43.8$ ft
- $d_{67} = 48.8$ ft
- $d_{68} = 53.8$ ft
- $d_{69} = 58.8$ ft
- $d_{70} = 63.8$ ft
- $d_{71} = 68.8$ ft
- $d_{72} = 73.8$ ft
- $d_{73} = 78.8$ ft
- $d_{74} = 83.8$ ft
- $d_{75} = 88.8$ ft

$\phi_{65} = 36 \text{ deg}$, $\gamma_{65} = 130 \text{ pcf}$, $\gamma_{65_{\text{sub}}} = 70 \text{ pcf}$

$\phi_{66} = 34 \text{ deg}$, $\gamma_{66} = 126 \text{ pcf}$, $\gamma_{66_{\text{sub}}} = 66 \text{ pcf}$

$\phi_{67} = 34.5 \text{ deg}$, $\gamma_{67} = 128.5 \text{ pcf}$, $\gamma_{67_{\text{sub}}} = 67.5 \text{ pcf}$

$\phi_{68} = 35 \text{ deg}$, $\gamma_{68} = 129 \text{ pcf}$, $\gamma_{68_{\text{sub}}} = 68 \text{ pcf}$

$\phi_{69} = 38 \text{ deg}$, $\gamma_{69} = 135 \text{ pcf}$, $\gamma_{69_{\text{sub}}} = 75 \text{ pcf}$

$\phi_{70} = 38 \text{ deg}$, $\gamma_{70} = 135 \text{ pcf}$, $\gamma_{70_{\text{sub}}} = 75 \text{ pcf}$

$\phi_{71} = 35 \text{ deg}$, $\gamma_{71} = 129 \text{ pcf}$, $\gamma_{71_{\text{sub}}} = 68 \text{ pcf}$

$\phi_{72} = 34 \text{ deg}$, $\gamma_{72} = 126 \text{ pcf}$, $\gamma_{72_{\text{sub}}} = 66 \text{ pcf}$

$\phi_{73} = 38.5 \text{ deg}$, $\gamma_{73} = 135.5 \text{ pcf}$, $\gamma_{73_{\text{sub}}} = 75.5 \text{ pcf}$

$\phi_{74} = 33.5 \text{ deg}$, $\gamma_{74} = 127.5 \text{ pcf}$, $\gamma_{74_{\text{sub}}} = 67.5 \text{ pcf}$

$\phi_{75} = 38 \text{ deg}$, $\gamma_{75} = 135 \text{ pcf}$, $\gamma_{75_{\text{sub}}} = 75 \text{ pcf}$

$\phi_{76} = 33.5 \text{ deg}$, $\gamma_{76} = 127.5 \text{ pcf}$, $\gamma_{76_{\text{sub}}} = 67.5 \text{ pcf}$

Prepared by: Mark Salehi
Date: 6/28/2020

Reviewed by: Soren Morch
Date: 6/28/2020
### 5.2.2 Boring SB-5

<table>
<thead>
<tr>
<th>Elevation of Sample MLLW</th>
<th>N Corrected</th>
<th>Friction Angle or cohesion</th>
<th>Moist Unit Weight</th>
<th>Sub. Unit Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>$d_{51} = -23.95 \text{ ft}$</td>
<td>$\phi_{51} := 34 \text{ deg}$</td>
<td>$\gamma_{51} := 128 \text{pcf}$</td>
<td>$\gamma_{51 _sub} := 67 \text{pcf}$</td>
<td></td>
</tr>
<tr>
<td>$d_{52} = -26.45 \text{ ft}$</td>
<td>$\phi_{52} := 34 \text{ deg}$</td>
<td>$\gamma_{52} := 128 \text{pcf}$</td>
<td>$\gamma_{52 _sub} := 67 \text{pcf}$</td>
<td></td>
</tr>
<tr>
<td>$d_{53} = -31.45 \text{ ft}$</td>
<td>$\phi_{53} := 29.5 \text{ deg}$</td>
<td>$\gamma_{53} := 120 \text{pcf}$</td>
<td>$\gamma_{53 _sub} := 63 \text{pcf}$</td>
<td></td>
</tr>
<tr>
<td>$d_{54} = -36.45 \text{ ft}$</td>
<td>$\phi_{54} := 30 \text{ deg}$</td>
<td>$\gamma_{54} := 125 \text{pcf}$</td>
<td>$\gamma_{54 _sub} := 65 \text{pcf}$</td>
<td></td>
</tr>
<tr>
<td>$d_{55} = -41.45 \text{ ft}$</td>
<td>$\phi_{55} := 36 \text{ deg}$</td>
<td>$\gamma_{55} := 130 \text{pcf}$</td>
<td>$\gamma_{55 _sub} := 70 \text{pcf}$</td>
<td></td>
</tr>
<tr>
<td>$d_{56} = -46.45 \text{ ft}$</td>
<td>$\phi_{56} := 34.5 \text{ deg}$</td>
<td>$\gamma_{56} := 128.5 \text{pcf}$</td>
<td>$\gamma_{56 _sub} := 67.5 \text{pcf}$</td>
<td></td>
</tr>
<tr>
<td>$d_{57} = -51.45 \text{ ft}$</td>
<td>$\phi_{57} := 31.5 \text{ deg}$</td>
<td>$\gamma_{57} := 111 \text{pcf}$</td>
<td>$\gamma_{57 _sub} := 63 \text{pcf}$</td>
<td></td>
</tr>
<tr>
<td>$d_{58} = -56.45 \text{ ft}$</td>
<td>$\phi_{58} := 35 \text{ deg}$</td>
<td>$\gamma_{58} := 129 \text{pcf}$</td>
<td>$\gamma_{58 _sub} := 68 \text{pcf}$</td>
<td></td>
</tr>
<tr>
<td>$d_{59} = -61.45 \text{ ft}$</td>
<td>$\phi_{59} := 38 \text{ deg}$</td>
<td>$\gamma_{59} := 135 \text{pcf}$</td>
<td>$\gamma_{59 _sub} := 75 \text{pcf}$</td>
<td></td>
</tr>
<tr>
<td>$d_{510} = -66.45 \text{ ft}$</td>
<td>$\phi_{510} := 33.5 \text{ deg}$</td>
<td>$\gamma_{610} := 123.5 \text{pcf}$</td>
<td>$\gamma_{510 _sub} := 65.5 \text{pcf}$</td>
<td></td>
</tr>
</tbody>
</table>
### Section 5.3 Normalized Blowcount

Normalization for the overburden pressure is performed as described in the methodology section.

- **Top of grade, and/or, mudline** \( TOG := -12.55 \text{ ft} \)
- **High Water Level** \( HWL := 5 \text{ ft} \) \( \text{Reference 1b} \)
- **Low Water Level** \( LWL := 0 \text{ ft} \)
- **Mean Tide Level** \( MTL := \text{mean}(HWL, LWL) = 2.5 \text{ ft} \)

\( \gamma_{water} := 62 \text{ pcf} \)

#### 5.3.1 Boring SB-8

<table>
<thead>
<tr>
<th>Depth</th>
<th>Effective Overburden Stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>( H_1 := \text{abs}(TOG - d1 - 1 \text{ ft}) = 1.55 \text{ ft} )</td>
<td>( \sigma_1 := H_1 \cdot \gamma_{1, sub} = 0.035 \text{ tonf/ft}^2 )</td>
</tr>
<tr>
<td>( H_2 := \text{abs}(d2 - d1) = 6.7 \text{ ft} )</td>
<td>( \sigma_2 := \sigma_1 + H_2 \cdot \gamma_{2, sub} = 0.186 \text{ tonf/ft}^2 )</td>
</tr>
<tr>
<td>( H_3 := \text{abs}(d3 - d2) = 5 \text{ ft} )</td>
<td>( \sigma_3 := \sigma_2 + H_3 \cdot \gamma_{3, sub} = 0.298 \text{ tonf/ft}^2 )</td>
</tr>
<tr>
<td>( H_4 := \text{abs}(d4 - d3) = 5 \text{ ft} )</td>
<td>( \sigma_4 := \sigma_3 + H_4 \cdot \gamma_{4, sub} = 0.466 \text{ tonf/ft}^2 )</td>
</tr>
<tr>
<td>( H_5 := \text{abs}(d5 - d4) = 5 \text{ ft} )</td>
<td>( \sigma_5 := \sigma_4 + H_5 \cdot \gamma_{5, sub} = 0.633 \text{ tonf/ft}^2 )</td>
</tr>
<tr>
<td>( H_6 := \text{abs}(d6 - d5) = 5 \text{ ft} )</td>
<td>( \sigma_6 := \sigma_5 + H_6 \cdot \gamma_{6, sub} = 0.801 \text{ tonf/ft}^2 )</td>
</tr>
<tr>
<td>( H_7 := \text{abs}(d7 - d6) = 5 \text{ ft} )</td>
<td>( \sigma_7 := \sigma_6 + H_7 \cdot \gamma_{7, sub} = 0.968 \text{ tonf/ft}^2 )</td>
</tr>
<tr>
<td>( H_8 := \text{abs}(d8 - d7) = 4.5 \text{ ft} )</td>
<td>( \sigma_8 := \sigma_7 + H_8 \cdot \gamma_{8, sub} = 1.119 \text{ tonf/ft}^2 )</td>
</tr>
<tr>
<td>( H_9 := \text{abs}(d9 - d8) = 5.5 \text{ ft} )</td>
<td>( \sigma_9 := \sigma_8 + H_9 \cdot \gamma_{9, sub} = 1.303 \text{ tonf/ft}^2 )</td>
</tr>
<tr>
<td>( H_{10} := \text{abs}(d10 - d9) = 4.5 \text{ ft} )</td>
<td>( \sigma_{10} := \sigma_9 + H_{10} \cdot \gamma_{10, sub} = 1.472 \text{ tonf/ft}^2 )</td>
</tr>
<tr>
<td>( H_{11} := \text{abs}(d11 - d10) = 5.5 \text{ ft} )</td>
<td>( \sigma_{11} := \sigma_{10} + H_{11} \cdot \gamma_{11, sub} = 1.642 \text{ tonf/ft}^2 )</td>
</tr>
</tbody>
</table>

Prepared by: Mark Salehi  
Date: 6/28/2020  
Reviewed by: Soren Morch  
Date: 6/28/2020
The exponent $n=0.5$ of the equation below is a stress exponent typically equal to 1 in clays and 0.5 to 0.6 in sands.

### Normalized Corrected Blowcount

<table>
<thead>
<tr>
<th>$C_N$</th>
<th>Expression</th>
<th>$N_1$</th>
<th>$N_2$</th>
<th>$N_3$</th>
<th>$N_4$</th>
<th>$N_5$</th>
<th>$N_6$</th>
<th>$N_7$</th>
<th>$N_8$</th>
<th>$N_9$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$C_{N1}$</td>
<td>$\left( \frac{\text{tonf}}{ft^2} \right)^{0.5}$</td>
<td>$\frac{ft^2}{\sigma_1}$</td>
<td>5.355</td>
<td>$N_{1,60,1} := C_{N1} \cdot N_{1,60}$</td>
<td>round $(N_{1,60,1}) = 0$</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$C_{N2}$</td>
<td>$\left( \frac{\text{tonf}}{ft^2} \right)^{0.5}$</td>
<td>$\frac{ft^2}{\sigma_2}$</td>
<td>2.321</td>
<td>$N_{2,60,1} := C_{N2} \cdot N_{2,60}$</td>
<td>round $(N_{2,60,1}) = 0$</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$C_{N3}$</td>
<td>$\left( \frac{\text{tonf}}{ft^2} \right)^{0.5}$</td>
<td>$\frac{ft^2}{\sigma_3}$</td>
<td>1.831</td>
<td>$N_{3,60,1} := C_{N3} \cdot N_{3,60}$</td>
<td>round $(N_{3,60,1}) = 0$</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$C_{N4}$</td>
<td>$\left( \frac{\text{tonf}}{ft^2} \right)^{0.5}$</td>
<td>$\frac{ft^2}{\sigma_4}$</td>
<td>1.465</td>
<td>$N_{4,60,1} := C_{N4} \cdot N_{4,60}$</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$C_{N5}$</td>
<td>$\left( \frac{\text{tonf}}{ft^2} \right)^{0.5}$</td>
<td>$\frac{ft^2}{\sigma_5}$</td>
<td>1.257</td>
<td>$N_{5,60,1} := C_{N5} \cdot N_{5,60}$</td>
<td>round $(N_{5,60,1}) = 19$</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$C_{N6}$</td>
<td>$\left( \frac{\text{tonf}}{ft^2} \right)^{0.5}$</td>
<td>$\frac{ft^2}{\sigma_6}$</td>
<td>1.118</td>
<td>$N_{6,60,1} := C_{N6} \cdot N_{6,60}$</td>
<td>round $(N_{6,60,1}) = 16$</td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$C_{N7}$</td>
<td>$\left( \frac{\text{tonf}}{ft^2} \right)^{0.5}$</td>
<td>$\frac{ft^2}{\sigma_7}$</td>
<td>1.016</td>
<td>$N_{7,60,1} := C_{N7} \cdot N_{7,60}$</td>
<td>round $(N_{7,60,1}) = 15$</td>
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</tr>
<tr>
<td>$C_{N8}$</td>
<td>$\left( \frac{\text{tonf}}{ft^2} \right)^{0.5}$</td>
<td>$\frac{ft^2}{\sigma_8}$</td>
<td>0.945</td>
<td>$N_{8,60,1} := C_{N8} \cdot N_{8,60}$</td>
<td>round $(N_{8,60,1}) = 13$</td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$C_{N9}$</td>
<td>$\left( \frac{\text{tonf}}{ft^2} \right)^{0.5}$</td>
<td>$\frac{ft^2}{\sigma_9}$</td>
<td>0.876</td>
<td>$N_{9,60,1} := C_{N9} \cdot N_{9,60}$</td>
<td>round $(N_{9,60,1}) = 11$</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Prepared by: Mark Salehi  
Date: 6/28/2020

Reviewed by: Soren Morch  
Date: 6/28/2020
5.3.1.1 Cohesion SB-8

The soil boring log describes samples with low plasticity, which indicates existing of clays in the soil sample. The cohesion from this clay content will be estimated assuming a low plasticity index (PI) of 20% for the Undrained Consolidated Triaxial Compression (TC) curve in Figure 3-2, Relationship between the ratio $\frac{S_u}{p}$ and plasticity index for normally consolidated clays, in the Engineering Manual EM 1110-2-2504.

For a PI = 20%, the $\frac{S_u}{p}$ is 0.35.

Where $S_u$ is the undrained shear strength and $p$ is the vertical effective stress.
### Estimated Cohesion for SB-8

\[ Su_1 := 0.35 \cdot \sigma 1 = 24.413 \text{ psf} \]
\[ C_1 := \frac{Su_1}{2} = 12.206 \text{ psf} \]
\[ Su_2 := 0.35 \cdot \sigma 2 = 129.938 \text{ psf} \]
\[ C_2 := \frac{Su_2}{2} = 64.969 \text{ psf} \]
\[ Su_3 := 0.35 \cdot \sigma 3 = 208.688 \text{ psf} \]
\[ C_3 := \frac{Su_3}{2} = 104.344 \text{ psf} \]
\[ Su_4 := 0.35 \cdot \sigma 4 = 325.938 \text{ psf} \]
\[ C_4 := \frac{Su_4}{2} = 162.969 \text{ psf} \]
\[ Su_5 := 0.35 \cdot \sigma 5 = 443.188 \text{ psf} \]
\[ C_5 := \frac{Su_5}{2} = 221.594 \text{ psf} \]
\[ Su_6 := 0.35 \cdot \sigma 6 = 560.438 \text{ psf} \]
\[ C_6 := \frac{Su_6}{2} = 280.219 \text{ psf} \]
\[ Su_7 := 0.35 \cdot \sigma 7 = 677.688 \text{ psf} \]
\[ C_7 := \frac{Su_7}{2} = 338.844 \text{ psf} \]
\[ Su_8 := 0.35 \cdot \sigma 8 = 783.213 \text{ psf} \]
\[ C_8 := \frac{Su_8}{2} = 391.606 \text{ psf} \]
\[ Su_9 := 0.35 \cdot \sigma 9 = 912.188 \text{ psf} \]
\[ C_9 := \frac{Su_9}{2} = 456.094 \text{ psf} \]
\[ Su_{10} := 0.35 \cdot \sigma 10 = (1.03 \cdot 10^3) \text{ psf} \]
\[ C_{10} := \frac{Su_{10}}{2} = 515.156 \text{ psf} \]
\[ Su_{11} := 0.35 \cdot \sigma 11 = (1.15 \cdot 10^3) \text{ psf} \]
\[ C_{10} := \frac{Su_{11}}{2} = 574.831 \text{ psf} \]
5.3.2 Boring SB-7

<table>
<thead>
<tr>
<th>Depth</th>
<th>Effective Overburden Stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>(H_{71} = \text{abs}(TOG - d_{71} - 1\ ft) = 4.85\ ft)</td>
<td>(\sigma_{71} = H_{71} \cdot \gamma_{71_sub} = 0.109\ \text{tonf/ft}^2)</td>
</tr>
<tr>
<td>(H_{72} = \text{abs}(d_{72} - d_{71}) = 3.5\ ft)</td>
<td>(\sigma_{72} = \sigma_{71} + H_{72} \cdot \gamma_{72_sub} = 0.219\ \text{tonf/ft}^2)</td>
</tr>
<tr>
<td>(H_{73} = \text{abs}(d_{73} - d_{72}) = 5\ ft)</td>
<td>(\sigma_{73} = \sigma_{72} + H_{73} \cdot \gamma_{73_sub} = 0.382\ \text{tonf/ft}^2)</td>
</tr>
<tr>
<td>(H_{74} = \text{abs}(d_{74} - d_{73}) = 5\ ft)</td>
<td>(\sigma_{74} = \sigma_{73} + H_{74} \cdot \gamma_{74_sub} = 0.549\ \text{tonf/ft}^2)</td>
</tr>
<tr>
<td>(H_{75} = \text{abs}(d_{75} - d_{74}) = 3.25\ ft)</td>
<td>(\sigma_{75} = \sigma_{74} + H_{75} \cdot \gamma_{75_sub} = 0.658\ \text{tonf/ft}^2)</td>
</tr>
<tr>
<td>(H_{76} = \text{abs}(d_{76} - d_{75}) = 6.75\ ft)</td>
<td>(\sigma_{76} = \sigma_{75} + H_{76} \cdot \gamma_{76_sub} = 0.878\ \text{tonf/ft}^2)</td>
</tr>
<tr>
<td>(H_{77} = \text{abs}(d_{77} - d_{76}) = 5\ ft)</td>
<td>(\sigma_{77} = \sigma_{76} + H_{77} \cdot \gamma_{77_sub} = 1.049\ \text{tonf/ft}^2)</td>
</tr>
<tr>
<td>(H_{78} = \text{abs}(d_{78} - d_{77}) = 5\ ft)</td>
<td>(\sigma_{78} = \sigma_{77} + H_{78} \cdot \gamma_{78_sub} = 1.219\ \text{tonf/ft}^2)</td>
</tr>
<tr>
<td>(H_{79} = \text{abs}(d_{79} - d_{78}) = 5\ ft)</td>
<td>(\sigma_{79} = \sigma_{78} + H_{79} \cdot \gamma_{79_sub} = 1.394\ \text{tonf/ft}^2)</td>
</tr>
<tr>
<td>(H_{710} = \text{abs}(d_{710} - d_{79}) = 5\ ft)</td>
<td>(\sigma_{710} = \sigma_{79} + H_{710} \cdot \gamma_{710_sub} = 1.581\ \text{tonf/ft}^2)</td>
</tr>
<tr>
<td>(H_{711} = \text{abs}(d_{711} - d_{710}) = 5\ ft)</td>
<td>(\sigma_{711} = \sigma_{710} + H_{711} \cdot \gamma_{711_sub} = 1.781\ \text{tonf/ft}^2)</td>
</tr>
<tr>
<td>(H_{712} = \text{abs}(d_{712} - d_{711}) = 5\ ft)</td>
<td>(\sigma_{712} = \sigma_{711} + H_{712} \cdot \gamma_{712_sub} = 1.969\ \text{tonf/ft}^2)</td>
</tr>
<tr>
<td>(H_{713} = \text{abs}(d_{713} - d_{712}) = 5\ ft)</td>
<td>(\sigma_{713} = \sigma_{712} + H_{713} \cdot \gamma_{713_sub} = 2.169\ \text{tonf/ft}^2)</td>
</tr>
<tr>
<td>(H_{714} = \text{abs}(d_{714} - d_{713}) = 5\ ft)</td>
<td>(\sigma_{714} = \sigma_{713} + H_{714} \cdot \gamma_{714_sub} = 2.369\ \text{tonf/ft}^2)</td>
</tr>
</tbody>
</table>
The exponent $n=0.5$ of the equation below is a stress exponent typically equal to 1 in clays and 0.5 to 0.6 in sands.

\[
C_{N71} := \left( \frac{\text{tonf}}{\text{ft}^2} \right)^{0.5} \sigma_{71} = 3.027
\]

\[
N_{71,60,1} := C_{N71} \cdot N_{71,60}
\]

\[
\text{round} \left( N_{71,60,1} \right) = 0
\]

\[
C_{N72} := \left( \frac{\text{tonf}}{\text{ft}^2} \right)^{0.5} \sigma_{72} = 2.135
\]

\[
N_{72,60,1} := C_{N72} \cdot N_{72,60}
\]

\[
\text{round} \left( N_{72,60,1} \right) = 25
\]

\[
C_{N73} := \left( \frac{\text{tonf}}{\text{ft}^2} \right)^{0.5} \sigma_{73} = 1.618
\]

\[
N_{73,60,1} := C_{N73} \cdot N_{73,60}
\]

\[
\text{round} \left( N_{73,60,1} \right) = 16
\]

\[
C_{N74} := \left( \frac{\text{tonf}}{\text{ft}^2} \right)^{0.5} \sigma_{74} = 1.349
\]

\[
N_{74,60,1} := C_{N74} \cdot N_{74,60}
\]

\[
\text{round} \left( N_{74,60,1} \right) = 18
\]

\[
C_{N75} := \left( \frac{\text{tonf}}{\text{ft}^2} \right)^{0.5} \sigma_{75} = 1.233
\]

\[
N_{75,60,1} := C_{N75} \cdot N_{75,60}
\]

\[
\text{round} \left( N_{75,60,1} \right) = 16
\]

\[
C_{N76} := \left( \frac{\text{tonf}}{\text{ft}^2} \right)^{0.5} \sigma_{76} = 1.067
\]

\[
N_{76,60,1} := C_{N76} \cdot N_{76,60}
\]

\[
\text{round} \left( N_{76,60,1} \right) = 11
\]

\[
C_{N77} := \left( \frac{\text{tonf}}{\text{ft}^2} \right)^{0.5} \sigma_{77} = 0.976
\]

\[
N_{77,60,1} := C_{N77} \cdot N_{77,60}
\]

\[
\text{round} \left( N_{77,60,1} \right) = 17
\]

\[
C_{N78} := \left( \frac{\text{tonf}}{\text{ft}^2} \right)^{0.5} \sigma_{78} = 0.906
\]

\[
N_{78,60,1} := C_{N78} \cdot N_{78,60}
\]

\[
\text{round} \left( N_{78,60,1} \right) = 14
\]
5.3.2 Boring SB-6

Depth | Effective Overburden Stress

| $H_{61}$ | $\text{abs}(TOG - d61 - 1\ ft) = 7.75\ ft$ | $\sigma_{61} = \frac{\text{tonf}}{ft^2}$ | $\gamma_{61\_sub} = 0.174$ | $\sigma_{61} = \sigma_{61} + H_{61} \cdot \gamma_{61\_sub} = 0.231\ \frac{\text{tonf}}{ft^2}$ |
| $H_{62}$ | $\text{abs}(d62 - d61) = 2.5\ ft$ | $\sigma_{62} = \sigma_{62} + H_{62} \cdot \gamma_{62\_sub} = 0.386\ \frac{tonf}{ft^2}$ |
| $H_{63}$ | $\text{abs}(d63 - d62) = 5\ ft$ | $\sigma_{63} = \sigma_{63} + H_{63} \cdot \gamma_{63\_sub} = 0.553\ \frac{tonf}{ft^2}$ |
| $H_{64}$ | $\text{abs}(d64 - d63) = 5\ ft$ | $\sigma_{64} = \sigma_{64} + H_{64} \cdot \gamma_{64\_sub} = 0.718\ \frac{tonf}{ft^2}$ |
| $H_{65}$ | $\text{abs}(d65 - d64) = 4.7\ ft$ | $\sigma_{65} = \sigma_{65} + H_{65} \cdot \gamma_{65\_sub} = 0.893\ \frac{tonf}{ft^2}$ |
| $H_{66}$ | $\text{abs}(d66 - d65) = 5.3\ ft$ | $\sigma_{66} = \sigma_{66} + H_{66} \cdot \gamma_{66\_sub} = 0.893\ \frac{tonf}{ft^2}$ |
\[ H_{67} := \text{abs}(d67 - d66) = 5 \text{ ft} \]
\[ \sigma_{67} := \sigma_{66} + H_{67} \cdot \gamma_{67}\text{,sub} = 1.061 \frac{\text{tonf}}{\text{ft}^2} \]
\[ H_{68} := \text{abs}(d68 - d67) = 5 \text{ ft} \]
\[ \sigma_{68} := \sigma_{67} + H_{68} \cdot \gamma_{68}\text{,sub} = 1.231 \frac{\text{tonf}}{\text{ft}^2} \]
\[ H_{69} := \text{abs}(d69 - d68) = 5 \text{ ft} \]
\[ \sigma_{69} := \sigma_{68} + H_{69} \cdot \gamma_{69}\text{,sub} = 1.419 \frac{\text{tonf}}{\text{ft}^2} \]
\[ H_{610} := \text{abs}(d610 - d69) = 5 \text{ ft} \]
\[ \sigma_{610} := \sigma_{69} + H_{610} \cdot \gamma_{610}\text{,sub} = 1.606 \frac{\text{tonf}}{\text{ft}^2} \]
\[ H_{611} := \text{abs}(d611 - d610) = 5 \text{ ft} \]
\[ \sigma_{611} := \sigma_{610} + H_{611} \cdot \gamma_{611}\text{,sub} = 1.776 \frac{\text{tonf}}{\text{ft}^2} \]
\[ H_{612} := \text{abs}(d612 - d611) = 5 \text{ ft} \]
\[ \sigma_{612} := \sigma_{611} + H_{612} \cdot \gamma_{612}\text{,sub} = 1.941 \frac{\text{tonf}}{\text{ft}^2} \]
\[ H_{613} := \text{abs}(d613 - d612) = 5 \text{ ft} \]
\[ \sigma_{613} := \sigma_{612} + H_{613} \cdot \gamma_{613}\text{,sub} = 2.13 \frac{\text{tonf}}{\text{ft}^2} \]
\[ H_{614} := \text{abs}(d614 - d613) = 5 \text{ ft} \]
\[ \sigma_{614} := \sigma_{613} + H_{614} \cdot \gamma_{614}\text{,sub} = 2.299 \frac{\text{tonf}}{\text{ft}^2} \]
\[ H_{615} := \text{abs}(d615 - d614) = 5 \text{ ft} \]
\[ \sigma_{615} := \sigma_{614} + H_{615} \cdot \gamma_{615}\text{,sub} = 2.486 \frac{\text{tonf}}{\text{ft}^2} \]
\[ H_{616} := \text{abs}(d616 - d615) = 5 \text{ ft} \]
\[ \sigma_{616} := \sigma_{615} + H_{616} \cdot \gamma_{616}\text{,sub} = 2.655 \frac{\text{tonf}}{\text{ft}^2} \]

The exponent \( n=0.5 \) of the equation below is a stress exponent typically equal to 1 in clays and 0.5 to 0.6 in sands.

\[
C_{N61} := \left( \frac{\text{tonf}}{\text{ft}^2} \right)^{0.5} \sigma_{61} = 2.395
\]

\[ N_{61\_60\_1} := C_{N61} \cdot N_{61\_60} \]

\[ \text{round} \left( N_{61\_60\_1} \right) = 0 \]

\[
C_{N62} := \left( \frac{\text{tonf}}{\text{ft}^2} \right)^{0.5} \sigma_{62} = 2.082
\]

\[ N_{62\_60\_1} := C_{N62} \cdot N_{62\_60} \]

\[ \text{round} \left( N_{62\_60\_1} \right) = 0 \]

\[
C_{N63} := \left( \frac{\text{tonf}}{\text{ft}^2} \right)^{0.5} \sigma_{63} = 1.61
\]

\[ N_{63\_60\_1} := C_{N63} \cdot N_{63\_60} \]

\[ \text{round} \left( N_{63\_60\_1} \right) = 9 \]

---

**Prepared by:** Mark Salehi  
**Date:** 6/28/2020

**Reviewed by:** Soren Morch  
**Date:** 6/28/2020
Calc. No.: DMMF-SD-01
Project Number: 19W012
Revision: 0A

Milwaukee DMMF Project
Structural Soil Properties

\[ C_{N64} = \left( \frac{\text{tonf}}{ft^2 \sigma_{64}} \right)^{0.5} = 1.345 \]

\[ N_{64,60,1} := C_{N64} \cdot N_{64,60} \]

\[ \text{round} (N_{64,60,1}) = 18 \]

\[ C_{N65} = \left( \frac{\text{tonf}}{ft^2 \sigma_{65}} \right)^{0.5} = 1.18 \]

\[ N_{65,60,1} := C_{N65} \cdot N_{65,60} \]

\[ \text{round} (N_{65,60,1}) = 33 \]

\[ C_{N66} = \left( \frac{\text{tonf}}{ft^2 \sigma_{66}} \right)^{0.5} = 1.058 \]

\[ N_{66,60,1} := C_{N66} \cdot N_{66,60} \]

\[ \text{round} (N_{66,60,1}) = 22 \]

\[ C_{N67} = \left( \frac{\text{tonf}}{ft^2 \sigma_{67}} \right)^{0.5} = 0.971 \]

\[ N_{67,60,1} := C_{N67} \cdot N_{67,60} \]

\[ \text{round} (N_{67,60,1}) = 21 \]

\[ C_{N68} = \left( \frac{\text{tonf}}{ft^2 \sigma_{68}} \right)^{0.5} = 0.901 \]

\[ N_{68,60,1} := C_{N68} \cdot N_{68,60} \]

\[ \text{round} (N_{68,60,1}) = 22 \]

\[ C_{N69} = \left( \frac{\text{tonf}}{ft^2 \sigma_{69}} \right)^{0.5} = 0.84 \]

\[ N_{69,60,1} := C_{N69} \cdot N_{69,60} \]

\[ \text{round} (N_{69,60,1}) = 34 \]

\[ C_{N610} = \left( \frac{\text{tonf}}{ft^2 \sigma_{610}} \right)^{0.5} = 0.789 \]

\[ N_{610,60,1} := C_{N610} \cdot N_{610,60} \]

\[ \text{round} (N_{610,60,1}) = 25 \]

\[ C_{N611} = \left( \frac{\text{tonf}}{ft^2 \sigma_{611}} \right)^{0.5} = 0.75 \]

\[ N_{611,60,1} := C_{N611} \cdot N_{611,60} \]

\[ \text{round} (N_{611,60,1}) = 19 \]
5.3.2 Boring SB-5

<table>
<thead>
<tr>
<th>Depth</th>
<th>Effective Overburden Stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>$H_{51} = \text{abs} \left( TOG - d_{51} - 1 \text{ ft} \right) = 10.4 \text{ ft}$</td>
<td>$\sigma_{51} = H_{51} \cdot \gamma_{51_{\text{sub}}} = 0.348 \frac{\text{tonf}}{\text{ft}^2}$</td>
</tr>
<tr>
<td>$H_{52} = \text{abs} \left( d_{52} - d_{51} \right) = 2.5 \text{ ft}$</td>
<td>$\sigma_{52} = \sigma_{51} + H_{52} \cdot \gamma_{52_{\text{sub}}} = 0.432 \frac{\text{tonf}}{\text{ft}^2}$</td>
</tr>
<tr>
<td>$H_{53} = \text{abs} \left( d_{53} - d_{52} \right) = 5 \text{ ft}$</td>
<td>$\sigma_{53} = \sigma_{52} + H_{53} \cdot \gamma_{53_{\text{sub}}} = 0.59 \frac{\text{tonf}}{\text{ft}^2}$</td>
</tr>
<tr>
<td>$H_{54} = \text{abs} \left( d_{54} - d_{53} \right) = 5 \text{ ft}$</td>
<td>$\sigma_{54} = \sigma_{53} + H_{54} \cdot \gamma_{54_{\text{sub}}} = 0.752 \frac{\text{tonf}}{\text{ft}^2}$</td>
</tr>
<tr>
<td>$H_{55} = \text{abs} \left( d_{55} - d_{54} \right) = 5 \text{ ft}$</td>
<td>$\sigma_{55} = \sigma_{54} + H_{55} \cdot \gamma_{55_{\text{sub}}} = 0.927 \frac{\text{tonf}}{\text{ft}^2}$</td>
</tr>
<tr>
<td>$H_{56} = \text{abs} \left( d_{56} - d_{55} \right) = 5 \text{ ft}$</td>
<td>$\sigma_{56} = \sigma_{55} + H_{56} \cdot \gamma_{56_{\text{sub}}} = 1.096 \frac{\text{tonf}}{\text{ft}^2}$</td>
</tr>
<tr>
<td>$H_{57} = \text{abs} \left( d_{57} - d_{56} \right) = 5 \text{ ft}$</td>
<td>$\sigma_{57} = \sigma_{56} + H_{57} \cdot \gamma_{57_{\text{sub}}} = 1.253 \frac{\text{tonf}}{\text{ft}^2}$</td>
</tr>
</tbody>
</table>
The exponent \(n=0.5\) of the equation below is a stress exponent typically equal to 1 in clays and 0.5 to 0.6 in sands.

\[
H_{58} := \text{abs}(d_{58} - d_{57}) = 5 \text{ ft} \\
\sigma_{58} := \sigma_{57} + H_{58} \cdot \gamma_{58, \text{sub}} = 1.423 \frac{\text{tonf}}{\text{ft}^2} \\
H_{59} := \text{abs}(d_{59} - d_{58}) = 5 \text{ ft} \\
\sigma_{59} := \sigma_{58} + H_{59} \cdot \gamma_{59, \text{sub}} = 1.611 \frac{\text{tonf}}{\text{ft}^2} \\
H_{510} := \text{abs}(d_{510} - d_{59}) = 5 \text{ ft} \\
\sigma_{510} := \sigma_{59} + H_{510} \cdot \gamma_{510, \text{sub}} = 1.775 \frac{\text{tonf}}{\text{ft}^2}
\]

Normalized Corrected Blowcount

\[
C_{N51} := \left( \frac{\text{tonf}}{\text{ft}^2} \right) \left( \frac{\sigma_{51}}{\text{ft}^2} \right)^{0.5} = 1.694 \quad N_{51.60.1} := C_{N51} \cdot N_{51.60} \]

\[
\text{round } (N_{51.60.1}) = 22
\]

\[
C_{N52} := \left( \frac{\text{tonf}}{\text{ft}^2} \right) \left( \frac{\sigma_{52}}{\text{ft}^2} \right)^{0.5} = 1.521 \quad N_{52.60.1} := C_{N52} \cdot N_{52.60} \]

\[
\text{round } (N_{52.60.1}) = 20
\]

\[
C_{N53} := \left( \frac{\text{tonf}}{\text{ft}^2} \right) \left( \frac{\sigma_{53}}{\text{ft}^2} \right)^{0.5} = 1.302 \quad N_{53.60.1} := C_{N53} \cdot N_{53.60} \]

\[
\text{round } (N_{53.60.1}) = 11
\]

\[
C_{N54} := \left( \frac{\text{tonf}}{\text{ft}^2} \right) \left( \frac{\sigma_{54}}{\text{ft}^2} \right)^{0.5} = 1.153 \quad N_{54.60.1} := C_{N54} \cdot N_{54.60} \]

\[
\text{round } (N_{54.60.1}) = 12
\]

\[
C_{N55} := \left( \frac{\text{tonf}}{\text{ft}^2} \right) \left( \frac{\sigma_{55}}{\text{ft}^2} \right)^{0.5} = 1.039 \quad N_{55.60.1} := C_{N55} \cdot N_{55.60} \]

\[
\text{round } (N_{55.60.1}) = 23
\]

\[
C_{N56} := \left( \frac{\text{tonf}}{\text{ft}^2} \right) \left( \frac{\sigma_{56}}{\text{ft}^2} \right)^{0.5} = 0.955 \quad N_{56.60.1} := C_{N56} \cdot N_{56.60} \]

\[
\text{round } (N_{56.60.1}) = 21
\]
\[ C_{N57} \left( \frac{\text{tonf}}{\text{ft}^2} \right)^{0.5} = 0.893 \]
\[ N_{57,60,1} := C_{N57} \cdot N_{57,60} \]
\[ \text{round} (N_{57,60,1}) = 10 \]

\[ C_{N58} \left( \frac{\text{tonf}}{\text{ft}^2} \right)^{0.5} = 0.838 \]
\[ N_{58,60,1} := C_{N58} \cdot N_{58,60} \]
\[ \text{round} (N_{58,60,1}) = 22 \]

\[ C_{N59} \left( \frac{\text{tonf}}{\text{ft}^2} \right)^{0.5} = 0.788 \]
\[ N_{59,60,1} := C_{N59} \cdot N_{59,60} \]
\[ \text{round} (N_{59,60,1}) = 38 \]

\[ C_{N510} \left( \frac{\text{tonf}}{\text{ft}^2} \right)^{0.5} = 0.751 \]
\[ N_{510,60,1} := C_{N510} \cdot N_{510,60} \]
\[ \text{round} (N_{510,60,1}) = 14 \]

Section 6. Results

6.1 Boring SB-8

<table>
<thead>
<tr>
<th>Depth of Layer</th>
<th>Normalized N</th>
<th>Friction Angle</th>
<th>Moist Unit Weight</th>
<th>Sub. Unit Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>( d_1 = -15.1 \text{ ft} )</td>
<td>( \text{round} (N_{1,60,1}) = 0 )</td>
<td>( \phi_1 := 0 \text{ deg} )</td>
<td>( \gamma_1 := 107 \text{ pcf} )</td>
<td>( \gamma_{1,\text{sub}} := 45 \text{ pcf} )</td>
</tr>
<tr>
<td>( d_2 = -21.8 \text{ ft} )</td>
<td>( \text{round} (N_{2,60,1}) = 0 )</td>
<td>( \phi_2 := 0 \text{ deg} )</td>
<td>( \gamma_2 := 107 \text{ pcf} )</td>
<td>( \gamma_{2,\text{sub}} := 45 \text{ pcf} )</td>
</tr>
<tr>
<td>( d_3 = -26.8 \text{ ft} )</td>
<td>( \text{round} (N_{3,60,1}) = 0 )</td>
<td>( \phi_3 := 0 \text{ deg} )</td>
<td>( \gamma_3 := 107 \text{ pcf} )</td>
<td>( \gamma_{3,\text{sub}} := 45 \text{ pcf} )</td>
</tr>
<tr>
<td>( d_4 = -31.8 \text{ ft} )</td>
<td>( \text{round} (N_{4,60,1}) = 19 )</td>
<td>( \phi_4 := 35 \text{ deg} )</td>
<td>( \gamma_4 := 129 \text{ pcf} )</td>
<td>( \gamma_{4,\text{sub}} := 70 \text{ pcf} )</td>
</tr>
<tr>
<td>( d_5 = -36.8 \text{ ft} )</td>
<td>( \text{round} (N_{5,60,1}) = 16 )</td>
<td>( \phi_5 := 34 \text{ deg} )</td>
<td>( \gamma_5 := 127 \text{ pcf} )</td>
<td>( \gamma_{5,\text{sub}} := 68 \text{ pcf} )</td>
</tr>
<tr>
<td>( d_6 = -41.8 \text{ ft} )</td>
<td>( \text{round} (N_{6,60,1}) = 15 )</td>
<td>( \phi_6 := 33 \text{ deg} )</td>
<td>( \gamma_6 := 125 \text{ pcf} )</td>
<td>( \gamma_{6,\text{sub}} := 65 \text{ pcf} )</td>
</tr>
<tr>
<td>( d_7 = -46.8 \text{ ft} )</td>
<td>( \text{round} (N_{7,60,1}) = 13 )</td>
<td>( \phi_7 := 32 \text{ deg} )</td>
<td>( \gamma_7 := 120 \text{ pcf} )</td>
<td>( \gamma_{7,\text{sub}} := 64 \text{ pcf} )</td>
</tr>
</tbody>
</table>
Layers are bundled into 4 layers from mudline to elevation of -60 ft LWD, which is below the estimated embedment. A weighted average SPT-N value is estimated as per IBC guideline.

\[ N_{8\_avg\_L1} := \frac{H_1 \cdot N_{1\_60\_1} + H_2 \cdot N_{2\_60\_1} + H_3 \cdot N_{3\_60\_1}}{H_1 + H_2 + H_3} = 0 \]

\[ \phi_{8\_avg\_L1} := 0 \ \text{deg} \quad \gamma_{8\_avg\_L1} := 107 \ \text{pcf} \quad \gamma_{8\_sub\_avg\_L1} := 45 \ \text{pcf} \]

\[ N_{8\_avg\_L2} := \frac{H_4 \cdot N_{4\_60\_1}}{H_4} = 19.171 \]

\[ \phi_{8\_avg\_L2} := 35 \ \text{deg} \quad \gamma_{8\_avg\_L2} := 129 \ \text{pcf} \quad \gamma_{8\_sub\_avg\_L2} := 70 \ \text{pcf} \]

\[ N_{8\_avg\_L3} := \frac{H_5 \cdot N_{5\_60\_1} + H_6 \cdot N_{6\_60\_1}}{H_5 + H_6} = 15.53 \]

\[ \phi_{8\_avg\_L3} := 33 \ \text{deg} \quad \gamma_{8\_avg\_L3} := 125 \ \text{pcf} \quad \gamma_{8\_sub\_avg\_L3} := 65 \ \text{pcf} \]

\[ N_{8\_avg\_L4} := \frac{H_7 \cdot N_{7\_60\_1} + H_8 \cdot N_{8\_60\_1} + H_9 \cdot N_{9\_60\_1}}{H_7 + H_8 + H_9} = 12.344 \]

\[ \phi_{8\_avg\_L4} := 30 \ \text{deg} \quad \gamma_{8\_avg\_L4} := 115 \ \text{pcf} \quad \gamma_{8\_sub\_avg\_L4} := 63 \ \text{pcf} \]
### 6.2 Boring SB-7

<table>
<thead>
<tr>
<th>Depth of Layer</th>
<th>Normalized N</th>
<th>Friction Angle</th>
<th>Moist Unit Weight</th>
<th>Sub. Unit Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>$d71 = -18.4 \text{ ft}$</td>
<td>$\text{round} \left( N_{71, 60.1} \right) = 0$</td>
<td>$\phi_{71} := 0 \text{ deg}$</td>
<td>$\gamma_{71} := 107 \text{pcf}$</td>
<td>$\gamma_{71_{\text{sub}}} := 45 \text{pcf}$</td>
</tr>
<tr>
<td>$d72 = -21.9 \text{ ft}$</td>
<td>$\text{round} \left( N_{72, 60.1} \right) = 25$</td>
<td>$\phi_{72} := 33 \text{ deg}$</td>
<td>$\gamma_{72} := 125 \text{pcf}$</td>
<td>$\gamma_{72_{\text{sub}}} := 65 \text{pcf}$</td>
</tr>
<tr>
<td>$d73 = -26.9 \text{ ft}$</td>
<td>$\text{round} \left( N_{73, 60.1} \right) = 16$</td>
<td>$\phi_{73} := 30 \text{ deg}$</td>
<td>$\gamma_{73} := 115 \text{pcf}$</td>
<td>$\gamma_{73_{\text{sub}}} := 63 \text{pcf}$</td>
</tr>
<tr>
<td>$d74 = -31.9 \text{ ft}$</td>
<td>$\text{round} \left( N_{74, 60.1} \right) = 18$</td>
<td>$\phi_{74} := 32 \text{ deg}$</td>
<td>$\gamma_{74} := 120 \text{pcf}$</td>
<td>$\gamma_{74_{\text{sub}}} := 64 \text{pcf}$</td>
</tr>
<tr>
<td>$d75 = -35.15 \text{ ft}$</td>
<td>$\text{round} \left( N_{75, 60.1} \right) = 16$</td>
<td>$\phi_{75} := 30 \text{ deg}$</td>
<td>$\gamma_{75} := 115 \text{pcf}$</td>
<td>$\gamma_{75_{\text{sub}}} := 63 \text{pcf}$</td>
</tr>
<tr>
<td>$d76 = -41.9 \text{ ft}$</td>
<td>$\text{round} \left( N_{76, 60.1} \right) = 11$</td>
<td>$\phi_{76} := 29.5 \text{ deg}$</td>
<td>$\gamma_{76} := 120 \text{pcf}$</td>
<td>$\gamma_{76_{\text{sub}}} := 62.5 \text{pcf}$</td>
</tr>
<tr>
<td>$d77 = -46.9 \text{ ft}$</td>
<td>$\text{round} \left( N_{77, 60.1} \right) = 17$</td>
<td>$\phi_{77} := 32.5 \text{ deg}$</td>
<td>$\gamma_{77} := 124 \text{pcf}$</td>
<td>$\gamma_{77_{\text{sub}}} := 64.5 \text{pcf}$</td>
</tr>
<tr>
<td>$d78 = -51.9 \text{ ft}$</td>
<td>$\text{round} \left( N_{78, 60.1} \right) = 14$</td>
<td>$\phi_{78} := 36 \text{ deg}$</td>
<td>$\gamma_{78} := 130 \text{pcf}$</td>
<td>$\gamma_{78_{\text{sub}}} := 70 \text{pcf}$</td>
</tr>
<tr>
<td>$d79 = -56.9 \text{ ft}$</td>
<td>$\text{round} \left( N_{79, 60.1} \right) = 21$</td>
<td>$\phi_{79} := 38 \text{ deg}$</td>
<td>$\gamma_{79} := 135 \text{pcf}$</td>
<td>$\gamma_{79_{\text{sub}}} := 75 \text{pcf}$</td>
</tr>
<tr>
<td>$d710 = -61.9 \text{ ft}$</td>
<td>$\text{round} \left( N_{710, 60.1} \right) = 31$</td>
<td>$\phi_{710} := 40 \text{ deg}$</td>
<td>$\gamma_{710} := 140 \text{pcf}$</td>
<td>$\gamma_{710_{\text{sub}}} := 80 \text{pcf}$</td>
</tr>
<tr>
<td>$d711 = -66.9 \text{ ft}$</td>
<td>$\text{round} \left( N_{711, 60.1} \right) = 38$</td>
<td>$\phi_{711} := 40 \text{ deg}$</td>
<td>$\gamma_{711} := 140 \text{pcf}$</td>
<td>$\gamma_{711_{\text{sub}}} := 80 \text{pcf}$</td>
</tr>
<tr>
<td>$d712 = -71.9 \text{ ft}$</td>
<td>$\text{round} \left( N_{712, 60.1} \right) = 28$</td>
<td>$\phi_{712} := 39 \text{ deg}$</td>
<td>$\gamma_{712} := 137 \text{pcf}$</td>
<td>$\gamma_{712_{\text{sub}}} := 77 \text{pcf}$</td>
</tr>
<tr>
<td>$d713 = -76.9 \text{ ft}$</td>
<td>$\text{round} \left( N_{713, 60.1} \right) = 53$</td>
<td>$\phi_{713} := 40 \text{ deg}$</td>
<td>$\gamma_{713} := 140 \text{pcf}$</td>
<td>$\gamma_{713_{\text{sub}}} := 80 \text{pcf}$</td>
</tr>
<tr>
<td>$d714 = -81.9 \text{ ft}$</td>
<td>$\text{round} \left( N_{714, 60.1} \right) = 57$</td>
<td>$\phi_{714} := 40 \text{ deg}$</td>
<td>$\gamma_{714} := 140 \text{pcf}$</td>
<td>$\gamma_{714_{\text{sub}}} := 80 \text{pcf}$</td>
</tr>
</tbody>
</table>

Layers are bundled into 4 layers from mudline to elevation of -60 ft LWD, which is below the estimated embedment. A weighted average SPT-N value is estimated as per IBC guideline.

\[
N_{7\_avg\_L1} := \frac{H_{71} \cdot N_{71, 60.1}}{H_{71}} = 0
\]

\[
\phi_{7\_avg\_L1} := 0 \text{ deg} \quad \gamma_{7\_avg\_L1} := 107 \text{pcf} \quad \gamma_{7\_sub\_avg\_L1} := 45 \text{pcf}
\]
\[ N_{\text{avg}, L2} := \frac{H_{72} \cdot N_{72, 60, 1}}{H_{72}} = 24.826 \]

\[ \phi_{\text{avg}, L2} := 33 \text{ deg} \quad \gamma_{\text{avg}, L2} := 125 \text{ pcf} \quad \gamma_{\text{sub, avg}, L2} := 65 \text{ pcf} \]

\[ \frac{H_{73} \cdot N_{73, 60, 1} + H_{74} \cdot N_{74, 60, 1} + H_{75} \cdot N_{75, 60, 1} + H_{76} \cdot N_{76, 60, 1}}{H_{73} + H_{74} + H_{75} + H_{76} + H_{77} + H_{78} + H_{79}} = 15.957 \]

\[ N_{\text{avg}, L3} := \frac{H_{710} \cdot N_{710, 60, 1} + H_{711} \cdot N_{711, 60, 1}}{H_{78} + H_{79} + H_{710} + H_{711} + H_{712} + H_{713} + H_{714}} = 29.607 \]

\[ \phi_{\text{avg}, L3} := 33 \text{ deg} \quad \gamma_{\text{avg}, L3} := 125 \text{ pcf} \quad \gamma_{\text{sub, avg}, L3} := 65 \text{ pcf} \]

\[ \phi_{\text{avg}, L4} := 40 \text{ deg} \quad \gamma_{\text{avg}, L4} := 140 \text{ pcf} \quad \gamma_{\text{sub, avg}, L4} := 80 \text{ pcf} \]

### 6.2 Boring SB-6

<table>
<thead>
<tr>
<th>Depth of Layer</th>
<th>Normalized N</th>
<th>Friction Angle</th>
<th>Moist Unit Weight</th>
<th>Sub. Unit Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>(d61) = -21.3 ft</td>
<td>(N_{61, 60, 1}) = 0</td>
<td>(\phi_{61} = 0) deg</td>
<td>(\gamma_{61} = 107) pcf</td>
<td>(\gamma_{61, sub} = 45) pcf</td>
</tr>
<tr>
<td>(d62) = -23.8 ft</td>
<td>(N_{62, 60, 1}) = 0</td>
<td>(\phi_{62} = 0) deg</td>
<td>(\gamma_{62} = 107) pcf</td>
<td>(\gamma_{62, sub} = 45) pcf</td>
</tr>
<tr>
<td>(d63) = -28.8 ft</td>
<td>(N_{63, 60, 1}) = 9</td>
<td>(\phi_{63} = 29) deg</td>
<td>(\gamma_{63} = 113) pcf</td>
<td>(\gamma_{63, sub} = 62) pcf</td>
</tr>
<tr>
<td>(d64) = -33.8 ft</td>
<td>(N_{64, 60, 1}) = 18</td>
<td>(\phi_{64} = 32) deg</td>
<td>(\gamma_{64} = 120) pcf</td>
<td>(\gamma_{64, sub} = 64) pcf</td>
</tr>
<tr>
<td>(d65) = -38.5 ft</td>
<td>(N_{65, 60, 1}) = 33</td>
<td>(\phi_{65} = 34.5) deg</td>
<td>(\gamma_{65} = 128.5) pcf</td>
<td>(\gamma_{65, sub} = 67.5) pcf</td>
</tr>
<tr>
<td>(d66) = -43.8 ft</td>
<td>(N_{66, 60, 1}) = 22</td>
<td>(\phi_{66} = 33.5) deg</td>
<td>(\gamma_{66} = 123.5) pcf</td>
<td>(\gamma_{66, sub} = 65.5) pcf</td>
</tr>
<tr>
<td>(d67) = -48.8 ft</td>
<td>(N_{67, 60, 1}) = 21</td>
<td>(\phi_{67} = 33.5) deg</td>
<td>(\gamma_{67} = 123.5) pcf</td>
<td>(\gamma_{67, sub} = 65.5) pcf</td>
</tr>
<tr>
<td>(d68) = -53.8 ft</td>
<td>(N_{68, 60, 1}) = 22</td>
<td>(\phi_{68} = 39) deg</td>
<td>(\gamma_{68} = 137) pcf</td>
<td>(\gamma_{68, sub} = 77) pcf</td>
</tr>
<tr>
<td>(d69) = -58.8 ft</td>
<td>(N_{69, 60, 1}) = 34</td>
<td>(\phi_{69} = 40) deg</td>
<td>(\gamma_{69} = 140) pcf</td>
<td>(\gamma_{69, sub} = 80) pcf</td>
</tr>
</tbody>
</table>

Prepared by: Mark Salehi \hspace{1cm} Date: 6/28/2020

Reviewed by: Soren Morch \hspace{1cm} Date: 6/28/2020
Layers are bundled into 4 layers from mudline to elevation of -60 ft LWD, which is below the estimated embedment. A weighted average SPT-N value is estimated as per IBC guideline. Layer 5 and 9 are ignored as it might be a bolder in the middle of a medium layer.

\[ N_{6_{\text{avg},L1}} = \frac{H_6 \cdot N_{61_{60,1}} + H_6 \cdot N_{62_{60,1}}}{H_6 + H_6 + H_6} = 0 \]

\[ \phi_{7_{\text{avg},L1}} = 0 \text{ deg} \quad \gamma_{7_{\text{avg},L1}} = 107 \text{ pcf} \quad \gamma_{7_{\text{sub},avg,L1}} = 45 \text{ pcf} \]

\[ N_{6_{\text{avg},L2}} = \frac{H_6 \cdot N_{63_{60,1}}}{H_6} = 9.363 \]

\[ \phi_{6_{\text{avg},L2}} = 29 \text{ deg} \quad \gamma_{6_{\text{avg},L2}} = 113 \text{ pcf} \quad \gamma_{6_{\text{sub},avg,L2}} = 62 \text{ pcf} \]

\[ N_{6_{\text{avg},L3}} = \frac{H_6 \cdot N_{64_{60,1}}}{H_6} = 17.589 \]

\[ \phi_{6_{\text{avg},L3}} = 32 \text{ deg} \quad \gamma_{6_{\text{avg},L3}} = 120 \text{ pcf} \quad \gamma_{6_{\text{sub},avg,L3}} = 64 \text{ pcf} \]

\[ N_{6_{\text{avg},L4}} = \frac{H_6 \cdot N_{66_{60,1}} + H_6 \cdot N_{67_{60,1}} + H_6 \cdot N_{68_{60,1}}}{H_6 + H_6 + H_6} = 21.655 \]

\[ \phi_{6_{\text{avg},L4}} = 36 \text{ deg} \quad \gamma_{6_{\text{avg},L4}} = 130 \text{ pcf} \quad \gamma_{6_{\text{sub},avg,L4}} = 69 \text{ pcf} \]
6.2 Boring SB-5

<table>
<thead>
<tr>
<th>Depth of Layer</th>
<th>Normalized N</th>
<th>Friction Angle</th>
<th>Moist Unit Weight</th>
<th>Sub. Unit Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>$d_{51}$ = -23.95 ft</td>
<td>round ($N_{51,60,1}$) = 22</td>
<td>$\phi_{51}$ := 33.5 deg</td>
<td>$\gamma_{51}$ := 123.5 pcf</td>
<td>$\gamma_{51,sub}$ := 65.5 pcf</td>
</tr>
<tr>
<td>$d_{52}$ = -26.45 ft</td>
<td>round ($N_{52,60,1}$) = 20</td>
<td>$\phi_{52}$ := 33 deg</td>
<td>$\gamma_{52}$ := 125 pcf</td>
<td>$\gamma_{52,sub}$ := 65 pcf</td>
</tr>
<tr>
<td>$d_{53}$ = -31.45 ft</td>
<td>round ($N_{53,60,1}$) = 11</td>
<td>$\phi_{53}$ := 29 deg</td>
<td>$\gamma_{53}$ := 113 pcf</td>
<td>$\gamma_{53,sub}$ := 62 pcf</td>
</tr>
<tr>
<td>$d_{54}$ = -36.45 ft</td>
<td>round ($N_{54,60,1}$) = 12</td>
<td>$\phi_{54}$ := 29.5 deg</td>
<td>$\gamma_{54}$ := 120 pcf</td>
<td>$\gamma_{54,sub}$ := 62.5 pcf</td>
</tr>
<tr>
<td>$d_{55}$ = -41.45 ft</td>
<td>round ($N_{55,60,1}$) = 23</td>
<td>$\phi_{55}$ := 34 deg</td>
<td>$\gamma_{55}$ := 124 pcf</td>
<td>$\gamma_{55,sub}$ := 66.5 pcf</td>
</tr>
<tr>
<td>$d_{56}$ = -46.45 ft</td>
<td>round ($N_{56,60,1}$) = 21</td>
<td>$\phi_{56}$ := 34.5 deg</td>
<td>$\gamma_{56}$ := 123.5 pcf</td>
<td>$\gamma_{56,sub}$ := 66 pcf</td>
</tr>
<tr>
<td>$d_{57}$ = -51.45 ft</td>
<td>round ($N_{57,60,1}$) = 10</td>
<td>$\phi_{57}$ := 29 deg</td>
<td>$\gamma_{57}$ := 113 pcf</td>
<td>$\gamma_{57,sub}$ := 62 pcf</td>
</tr>
<tr>
<td>$d_{58}$ = -56.45 ft</td>
<td>round ($N_{58,60,1}$) = 22</td>
<td>$\phi_{58}$ := 39 deg</td>
<td>$\gamma_{58}$ := 137 pcf</td>
<td>$\gamma_{58,sub}$ := 77 pcf</td>
</tr>
<tr>
<td>$d_{59}$ = -61.45 ft</td>
<td>round ($N_{59,60,1}$) = 38</td>
<td>$\phi_{59}$ := 40 deg</td>
<td>$\gamma_{59}$ := 140 pcf</td>
<td>$\gamma_{59,sub}$ := 80 pcf</td>
</tr>
<tr>
<td>$d_{510}$ = -66.45 ft</td>
<td>round ($N_{510,60,1}$) = 14</td>
<td>$\phi_{510}$ := 35.5 deg</td>
<td>$\gamma_{510}$ := 126 pcf</td>
<td>$\gamma_{510,sub}$ := 64 pcf</td>
</tr>
</tbody>
</table>

Layers are bundled into 4 layers from mudline to elevation of -60 ft LWD, which is below the estimated embedment. A weighted average SPT-N value is estimated as per IBC guideline. Layer 9 is ignored as it might be a bolder in the middle of a medium layer.

$$N_{5,avg,L1} := \frac{H_{51} \cdot N_{51,60,1} + H_{52} \cdot N_{52,60,1}}{H_{51} + H_{52}} = 21.724$$

$$\phi_{5,avg,L1} := 34.5 \text{ deg} \quad \gamma_{5,avg,L1} := 123.5 \text{ pcf} \quad \gamma_{5,sub,avg,L1} := 66 \text{ pcf}$$

$$N_{5,avg,L2} := \frac{H_{53} \cdot N_{53,60,1} + H_{54} \cdot N_{54,60,1}}{H_{53} + H_{54}} = 11.544$$

$$\phi_{5,avg,L2} := 30 \text{ deg} \quad \gamma_{5,avg,L2} := 115 \text{ pcf} \quad \gamma_{5,sub,avg,L2} := 63 \text{ pcf}$$

Prepared by: Mark Salehi

Reviewed by: Soren Morch
\[
N_{5\text{, avg}_L3} := \frac{H_{55} \cdot N_{55\text{, }60\_1} + H_{56} \cdot N_{56\text{, }60\_1}}{H_{55} + H_{56}} = 21.735
\]

\[
\phi_{5\text{, avg}_L3} := 36 \text{ deg} \quad \gamma_{5\text{, avg}_L3} := 130 \text{ pcf} \quad \gamma_{5\text{, sub\_avg}_L3} := 70 \text{ pcf}
\]

\[
N_{5\text{, avg}_L4} := \frac{H_{57} \cdot N_{57\text{, }60\_1} + H_{58} \cdot N_{58\text{, }60\_1} + H_{510} \cdot N_{510\text{, }60\_1}}{H_{57} + H_{58} + H_{510}} = 15.5
\]

\[
\phi_{5\text{, avg}_L4} := 33 \text{ deg} \quad \gamma_{5\text{, avg}_L4} := 125 \text{ pcf} \quad \gamma_{5\text{, sub\_avg}_L4} := 65 \text{ pcf}
\]

Section 7. References

PROJECT DOCUMENTS
1a Geotechnical Borings Prepared by Foth (2019).
1b DMMF Basis of Design.

GUIDELINES
2a Geotechnical Engineering Circular No. 5, Evaluation of Soil and Rock Properties, FHWA-IF-02-034, April 2002
2b Design of Sheet Pile Walls, EM 1110-2-2504.
Section 8. Attachments

Attachment 8.1: Soil Boring Profiles
TITLE: Structural Design of Cellular Retaining Structure

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SECTION 3 METHODOLOGY AND ACCEPTANCE CRITERIA 2
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Prepared by: Mark Salehi Date: 10/19/2020
Reviewed by: Soren Morch / Alex Mora Date: 10/19/2020
SECTION 1: PURPOSE AND SCOPE:
The Milwaukee Dredged Material Management Facility (DMMF) is planned to be constructed on the north side of the existing Jones Island DMMF to provide additional capacity for the disposal of dredged material. Milwaukee Harbor is located on the west shore of Lake Michigan. The new DMMF structure will be comprised of cellular coffer dam wall and a small section of rubble mound dike. The completed and filled DMMF will be used as a berthing facility for Port Milwaukee. The scope of this calculation is to present the structural design of the cells.

SECTION 2: ASSUMPTIONS
Foth performed a site-specific geotechnical investigation. Parameters extracted from those borings combined with the geotechnical investigation used for the Jones Island CDF, designed by USACE will be the basis of soil parameters.

SECTION 3: METHODOLOGY AND ACCEPTANCE CRITERIA
Cellular retaining structures are self-supporting gravity structures constructed using straight web sheet piles to form various shapes. The piles are interlocked and driven to form closed cells which are then filled with clean granular material (References 3a and 3b). To achieve continuity of the wall, the circular cells are connected together using fabricated junction piles and short arcs. The straight web pile section and particularly the interlocks have been designed to resist the circumferential tension which is developed in the cells due to the radial pressure of the contained fill. Wailings and tie rods are not required in these structures.

Two main types of cellular structure are circular and diaphragm cells with the former one having following advantages:

- Each cell is self-supporting and independent of the next
- Require fewer piles per linear foot of the structure

For this project, the design and construction of circular cells are considered to develop a cost-effective design. Cellular structures consist of two very different materials, steel and soil, resulting in a complex interaction that requires close attention to detail in the design process. The design of such cellular structures is, to a great extent, affected by the site's geotechnical conditions. The type of foundation material affects the cell interaction with both the foundation and the backfill soil and is greatly influenced by the vertical and horizontal displacements of the cell. Based on available geotechnical data (see Section 4), cell sheets will be extended into the lakebed to competent soil instead of rock. For such cells that are sitting on soil the following general steps, based on References 2b and 2c, are used for the design:
Milwaukee DMMF Project

1. Performing geotechnical evaluation to establish design inputs,
2. Setting height of structure based on environmental factors, berthing, and mooring purposes,
3. Establish saturation line,
4. Calculation of external forces and stability against,
   4.1 Overturning
   4.2 Bursting and hoop tension (interlock)
   4.3 Internal stability at cell centerline
   4.4 Slipping between fill and cell (tilting)
   4.5 Cell fill
   4.6 Horizontal shear
   4.7 Bearing capacity control
   4.8 Pull out control
   4.9 Settlement

Acceptance criteria and safety factor of each stability check are provided in Section 5.

SECTION 4: DESIGN INPUTS

4.1 Constant Parameters

Freshwater Unit Weight \( \gamma_{\text{water}} = 62.4 \frac{lbf}{ft^3} \)

High Water Level (HWL) \( HWL = 4 \cdot ft \) Reference 1b

Low Water Level (LLW) \( LWL = 0 \cdot ft \) Reference 1b

Mean water level (MLW) \( MWL = 1.4 \cdot ft \) Reference 1b

100-year Wave Height \( WavH = 3 \cdot ft \) Reference 1b

100-year Wave Period \( WavP = 12 \cdot sec \) Reference 1b

100 years is used for thickness reduction of steel for the design. Values are extracted from Reference 3c (see following tables). Applying 100 years thickness reduction is highly conservative as

- preventive measure such as cathodic protection is considered in the design.
Thickness reduction water contact (first row of Table 6.2)

\[ \text{THKR}_w := \frac{1.4 \text{ mm}}{25.4 \text{ mm} / \text{in}} = 0.03 \text{ in} \]

Thickness reduction cell fill contact (first row of Table 6.1 divided by two for compaction)

\[ \text{THKR}_{cf} := \frac{1.2 \text{ mm}}{25.4 \text{ mm} / \text{in}} = 0.02 \text{ in} \]

Thickness reduction backfill contact (second row of Table 6.1 divided by two for compaction)

\[ \text{THKR}_{bfc} := \frac{3 \text{ mm}}{25.4 \text{ mm} / \text{in}} = 0.06 \text{ in} \]

Probable reduction in flat sheet thickness:

\[ \text{THKR}_1 := \text{THKR}_w + \text{THKR}_{cf} = 0.05 \text{ in} \]
\[ \text{THKR}_2 := \text{THKR}_{cf} + \text{THKR}_{bfc} = 0.08 \text{ in} \]

Design thickness reduction:

\[ \text{THKR}_D := \max (\text{THKR}_1, \text{THKR}_2) = 0.08 \text{ in} \]

Yield strength of sheet pile

\[ f_y := 50 \cdot \text{ksi} \]
4.2 Geotechnical Data

A site-specific geotechnical investigation was performed by Foth and design parameters are established as per Reference 1e. In addition, the associated geotechnical borings were reviewed to make sure that the most reasonable parameters are assigned to soil layers. Additionally, USACE borings (Reference 1c and 1d) performed for the design of the existing Jones Island DMDF were reviewed, and data from boring SB-8 of Reference 1a is also included. Active and passive earth pressures are calculated using the Rankine method. It should be noted that the Rankine method tends to overestimate the active pressure and underestimate the passive pressure, which results in conservative design forces. This is an added factor of safety included in the design.

Soil Layers:

**Layer 1:** -12.55 to -26.8 ft LWD

- Angle of internal friction: \( \phi_{L1} = 0 \cdot \text{deg} \)
- Shear strength: \( c_{L1} = 240 \cdot \text{psf} \)
- Friction coefficient: \( \delta_{L1} = 0.54 \cdot \phi_{L1} = 0.00 \text{ deg} \)
- Top elev. of layer: \( TL_{1,EL} = -12.55 \cdot \text{ft} \)
- Bottom elev. of layer: \( BL_{1,EL} = -26.8 \cdot \text{ft} \)
- Moist unit weight: \( \gamma_{L1,wet} = 120 \cdot \text{pcf} \)
- Saturated unit weight: \( \gamma_{L1,sat} = 140 \cdot \text{pcf} \)
- Submerged unit weight: \( \gamma_{L1,sub} = \gamma_{L1,sat} - \gamma_{water} = 77.60 \text{ pcf} \)
Active pressure coeff.: \[ KaL1 := \frac{1 - \sin (\phi L1)}{1 + \sin (\phi L1)} = 1.00 \]

At rest pressure coeff.: \[ Kol1 := \frac{1 - \sin (\phi L1)}{1 + \sin (\phi L1)} \cdot \left(1 + \frac{2}{3} \cdot \sin (\phi L1)\right) = 1.00 \]

Passive pressure coeff.: \[ KpL1 := \frac{1 + \sin (\phi L1)}{1 - \sin (\phi L1)} = 1.00 \]

Layer 2: -26.8 to -31.8 ft LWD

Angle of internal friction: \[ \phi L2 := 30 \cdot \text{deg} \]

Shear strength: \[ cL2 := 240 \cdot \text{psf} \]

Friction coefficient: \[ \delta L2 := 0.54 \cdot \phi L2 = 16.20 \text{deg} \]

Top elev. of layer \[ TL2_{EL} := -26.8 \cdot \text{ft} \]

Bottom elev. of layer \[ BL2_{EL} := -31.8 \cdot \text{ft} \]

Moist unit weight: \[ \gamma L2_{wet} := 120 \cdot \frac{\text{lbf}}{\text{ft}^3} \]

Saturated unit weight: \[ \gamma L2_{sat} := 140 \cdot \text{pcf} \]

Submerged unit weight: \[ \gamma L2_{sub} := \gamma L2_{sat} - \gamma_{water} = 77.60 \text{pcf} \]
Active pressure coeff.: \[ KaL2 := \frac{1 - \sin(\phi L2)}{1 + \sin(\phi L2)} = 0.33 \]

At rest pressure coeff.: \[ KOL2 := \frac{1 - \sin(\phi L2)}{1 + \sin(\phi L2)} \cdot \left(1 + \left(\frac{2}{3}\right) \cdot \sin(\phi L2)\right) = 0.44 \]

Passive pressure coeff.: \[ KpL2 := \frac{1 + \sin(\phi L2)}{1 - \sin(\phi L2)} = 3.00 \]

**Layer 3: -31.8 to -41.8 ft LWD**

Angle of internal friction: \[ \phi L3 := 30 \cdot \text{deg} \]

Shear strength: \[ cL3 := 240 \cdot \frac{\text{lbf}}{\text{ft}^2} \]

Friction coefficient: \[ \delta L3 := 0.54 \cdot \phi L3 = 16.20 \text{ deg} \]

Top elev. of layer \[ TL3_{EL} := -31.8 \cdot \text{ft} \]

Bottom elev. of layer \[ BL3_{EL} := -41.8 \cdot \text{ft} \]

Moist unit weight: \[ \gamma L3_{wet} := 125 \cdot \frac{\text{lbf}}{\text{ft}^3} \]

Saturated unit weight: \[ \gamma L3_{sat} := 140 \cdot \text{pcf} \]

Submerged unit weight: \[ \gamma L3_{sub} := \gamma L3_{sat} - \gamma_{\text{water}} = 77.60 \text{ pcf} \]

Active pressure coeff.: \[ KaL3 := \frac{1 - \sin(\phi L3)}{1 + \sin(\phi L3)} = 0.33 \]
Milwaukee DMMF Project

At rest pressure coeff.: 
\[ K_{OL3} := \left( \frac{1 - \sin (\phi_{L3})}{1 + \sin (\phi_{L3})} \right) \cdot \left( 1 + \left( \frac{2}{3} \right) \cdot \sin (\phi_{L3}) \right) = 0.44 \]

Passive pressure coeff.: 
\[ K_{pL3} := \left( \frac{1 + \sin (\phi_{L3})}{1 - \sin (\phi_{L3})} \right) = 3.00 \]

Layer 4: -41.8 to -56.8 ft LWD

Angle of internal friction: 
\[ \phi_{L4} := 30 \cdot \text{deg} \]

Shear strength: 
\[ c_{L4} := 240 \cdot \frac{\text{lbf}}{\text{ft}^2} \]

Friction coefficient: 
\[ \delta_{L4} := 0.54 \cdot \phi_{L4} = 16.20 \text{ deg} \]

Top elev. of layer: 
\[ T_{L4_{EL}} := -41.8 \cdot \text{ft} \]

Bottom elev. of layer: 
\[ B_{L4_{EL}} := -56.8 \cdot \text{ft} \]

Moist unit weight: 
\[ \gamma_{L4_{wet}} := 115 \cdot \frac{\text{lbf}}{\text{ft}^2} \]

Saturated unit weight: 
\[ \gamma_{L4_{sat}} := 140 \cdot \text{pcf} \]

Submerged unit weight: 
\[ \gamma_{L4_{sub}} := \gamma_{L4_{sat}} - \gamma_{water} = 77.60 \text{ pcf} \]

Active pressure coeff.: 
\[ K_{aL4} := \left( \frac{1 - \sin (\phi_{L4})}{1 + \sin (\phi_{L4})} \right) = 0.33 \]

At rest pressure coeff.: 
\[ K_{OL4} := \left( \frac{1 - \sin (\phi_{L4})}{1 + \sin (\phi_{L4})} \right) \cdot \left( 1 + \left( \frac{2}{3} \right) \cdot \sin (\phi_{L4}) \right) = 0.44 \]
Passive pressure coeff.:  \[ K_{pL4} := \frac{1 + \sin(\phi_{L4})}{1 - \sin(\phi_{L4})} = 3.00 \]

4.3 Fill Material Properties
Cells should be filled with free draining granular material. Soils with less than 5 percent of the particles by weight passing the No. 200 sieve and 15 percent passing the No. 100 sieve are considered (Reference 2b).

Angle of internal friction:  \( \phi_{F} := 35 \cdot \text{deg} \)

Friction coefficient:  \( \delta_{F} := 0.54 \cdot \phi_{F} = 18.90 \cdot \text{deg} \)

Moist unit weight:  \( \gamma_{F\text{, wet}} := 125 \cdot \text{pcf} \)

Saturated unit weight:  \( \gamma_{F\text{, sat}} := 130 \cdot \text{pcf} \)

Submerged unit weight:  \( \gamma_{F\text{, sub}} := \gamma_{F\text{, sat}} - \gamma_{\text{water}} = 67.60 \cdot \text{pcf} \)

Active pressure coeff.:  \[ K_{aF} := \frac{1 - \sin(\phi_{F})}{1 + \sin(\phi_{F})} = 0.27 \]

At rest pressure coeff.:  \[ K_{0F} := \left( \frac{1 - \sin(\phi_{F})}{1 + \sin(\phi_{F})} \right) \cdot \left( 1 + \frac{2}{3} \cdot \sin(\phi_{F}) \right) = 0.37 \]

Passive pressure coeff.:  \[ K_{pF} := \frac{1 + \sin(\phi_{F})}{1 - \sin(\phi_{F})} = 3.69 \]
4.4 Soil Behind the Wall
Soil properties behind the wall from water level to the mudline are extracted from the characteristics of potential dredged material that will be disposed behind the cofferdam and inside the DMMF.

Table 2: Soil Condition Behind the Wall

<table>
<thead>
<tr>
<th>Description</th>
<th>$\phi$</th>
<th>$\delta$</th>
<th>$C$</th>
<th>$\gamma_{\text{wet}}$</th>
<th>$\gamma_{\text{sat}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Value</td>
<td>30</td>
<td>16.2</td>
<td>375</td>
<td>125</td>
<td>135</td>
</tr>
</tbody>
</table>

Angle of internal friction: \( \phi_{BW} := 30 \cdot \text{deg} \)

Shear strength: \( c_{BW} := 375 \cdot \text{psf} \)

Wet unit weight: \( \gamma_{BW_{\text{wet}}} := 125 \cdot \text{pcf} \)

Saturated unit weight: \( \gamma_{BW_{\text{sat}}} := 135 \cdot \text{pcf} \)

Submerged unit weight: \( \gamma_{BW_{\text{sub}}} := \gamma_{BW_{\text{sat}}} - \gamma_{\text{water}} = 72.60 \cdot \text{pcf} \)

Active pressure coeff.: \( K_{aBW} := \left( \frac{1 - \sin \left( \phi_{BW} \right)}{1 + \sin \left( \phi_{BW} \right)} \right) = 0.33 \)

At rest pressure coeff.: \( K_{0BW} := \left( \frac{1 - \sin \left( \phi_{BW} \right)}{1 + \sin \left( \phi_{BW} \right)} \right) \cdot \left( 1 + \left( \frac{2}{3} \right) \cdot \sin \left( \phi_{BW} \right) \right) = 0.44 \)

Passive pressure coeff.: \( K_{pBW} := \left( \frac{1 + \sin \left( \phi_{BW} \right)}{1 - \sin \left( \phi_{BW} \right)} \right) = 3.00 \)
4.5 Loads

- Surcharge starting 30 feet behind bulkhead:  \( q_{sur} = \frac{1000 \text{ lbf}}{\text{ft}^2} \) Reference 1b

- Surcharge on the bulkhead and immediately behind the bulkhead up to 30 feet:
  \( q_{sur_{cell}} = \frac{500 \text{ lbf}}{\text{ft}^2} \) Reference 1b

- Berthing and mooring loads are transferred to a separate dolphin and piled relieving platform system that supports both bollards and fenders. Therefore, they are not considered in the design of cellular retaining wall.

- For crane load refer to Attachment 8a. It should be noted that the crane load is transferred to the relieving platform and only shown in the design to confirm that the cells are not designed to withstand the project specified crane. Approaching other type of crane to the structure, should be evaluated case by case.

- Wave load:  \( q_{wave} = 12.4 \text{ kip/ft} \) Reference 1b

The probable scenario for the waves to be considered is that the cells are constructed and filled compacted, but the backfilling is not yet performed. Therefore, a case (called Case II) is considered with the waves applying at the high water level to mainly check the interlock strength. Applying the waves, when the cells are backfilled compacted will reduce the overturning moment and is not controlling.

4.6 Other Elevations

Following cell elevations are established in an iterative fashion to address all the mooring and stability criteria.

- Top of cell sheets  \( TCS_{EL} = 12 \cdot \text{ft} \)

- Top of fill  \( TF_{EL} = 12 \cdot \text{ft} \)

- Top of fill with relieving platform  \( TF_{EL2} = 6.4 \cdot \text{ft} \)

Bottom of cell sheets will be adjusted as required. This is an iterative process.
- Bottom of cell sheets (Inboard) \( BCS_{EL} := -52 \cdot ft \)

- Bottom of cell sheets (Outboard) \( BOCS_{EL} := -52 \cdot ft \)

- Dredged mudline \( DM_{EL} := -27 \cdot ft \)

Conservatively, saturation line is set to the high water level and self draining material will drain any excess rainfall or water to the lake through interlocks.

- Water level inside the cell \( WL_{cell} := HWL = 4.00 \ ft \)

Saturation slope is considered based on Reference 2b considering Silty Coarse Grained Fill.

- Elevation of saturated line \( Sat_{EL} := HWL = 4.00 \ ft \)
SECTION 5: CALCULATIONS

5.1 Physical Dimensions

- Height of selected lake water level above the dredged mudline
  \[ H_{\text{case1}} := \text{HWL} - \text{DM}_{\text{EL}} = 31.00 \text{ ft} \]

- Height of the top of cell from mudline
  \[ H_{\text{cell}} := \text{TCS}_{\text{EL}} - \text{DM}_{\text{EL}} = 39.00 \text{ ft} \]

- Height of the fill in the cell
  \[ H_{\text{fill}} := \text{TF}_{\text{EL}} - \text{DM}_{\text{EL}} = 39.00 \text{ ft} \]

- Height of the fill in the cell for cells with relieving platform that results in lesser of fill in the cell. Only used for controlling of overturning.
  \[ H_{\text{fill2}} := \text{TF}_{\text{EL2}} - \text{DM}_{\text{EL}} = 33.40 \text{ ft} \]

- Embedment depth
  \[ D := \text{DM}_{\text{EL}} - \text{BCS}_{\text{EL}} = 25.00 \text{ ft} \]

Since the internal face of the cells are planned to be sealed, conservatively, the internal face is considered saturated with the excess hydrostatic pressure applied to the wall.

- Excess hydrostatic pressure from backfilling
  \[ H_{\text{exc}} := \text{TCS}_{\text{EL}} - \text{Sat}_{\text{EL}} = 8.00 \text{ ft} \]

- Cell equivalent width:

The equivalent width, "We", of the structure is defined as the width of an equivalent rectangular section having a section modulus equal to that of the actual structure. Here the equivalent width is extracted from Section 4.3.1 of Reference 3c by specifying the radius of the cell and using \( \theta \) as 35 degrees.

Table 4.1.: Circular cells with \( \theta = 35^\circ \) standard junction piles

<table>
<thead>
<tr>
<th>No. of piles per cell</th>
<th>Geometrical values</th>
<th>Interlock deviation</th>
<th>Design values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cell</td>
<td>Arc</td>
<td>System</td>
<td>( d = 2 - r_n )</td>
</tr>
<tr>
<td>88</td>
<td>27</td>
<td>15</td>
<td>1</td>
</tr>
<tr>
<td>88</td>
<td>29</td>
<td>13</td>
<td>1</td>
</tr>
<tr>
<td>88</td>
<td>31</td>
<td>11</td>
<td>1</td>
</tr>
<tr>
<td>88</td>
<td>31</td>
<td>11</td>
<td>1</td>
</tr>
</tbody>
</table>

Should be read from Table 4.1 of Reference 3c (see above)
• Cell radius: $R_m := \frac{14.09}{2 \cdot 0.3048} \cdot \text{ft} = 23.11 \text{ ft}$

• Cell equivalent width $W_e := \frac{12.3}{0.3048} \cdot \text{ft} = 40.35 \text{ ft}$

• Arc radius: $R_a := \frac{3.28}{0.3048} \cdot \text{ft} = 10.76 \text{ ft}$

• Center-to-center distance of cells $CL := \frac{18.42}{0.3048} \cdot \text{ft} = 60.43 \text{ ft}$

• Cell connection angle $\alpha_{cell} := 32.73 \text{ deg}$

• Height of the saturation line above mudline $H_{sat} := \text{Sat}_{EL} - \text{DM}_{EL} = 31.00 \text{ ft}$

• Thickness of layer 1 $Thk_{L1} := \text{TL1}_{EL} - \text{TL2}_{EL} = 14.25 \text{ ft}$

• Thickness of layer 2 $Thk_{L2} := \text{TL2}_{EL} - \text{TL3}_{EL} = 5.00 \text{ ft}$

• Thickness of layer 3 $Thk_{L3} := \text{TL3}_{EL} - \text{TL4}_{EL} = 10.00 \text{ ft}$

• Thickness of layer 4 $Thk_{L4} := 50 \text{ ft}$
5.2 Overturning Stability

It is assumed that the cell is entirely filled with engineered fill material. The deriving force is the result of surcharge load on the wharf yard plus the pressure generated by the retained material. Sliding is generally not an issue for cells on soil, but evaluated for documentation purposes.

- Cell weight:

  Cell weight is calculated suing the combination of equivalent cell width and cell fill density above the mudline.

  For a case with cell filled to the top

  \[
  Cell_{weight} := W_e \cdot (H_{F_{cell}} - H_{sat}) \cdot \gamma F_{wet} + W_e \cdot (H_{sat}) \cdot \gamma F_{sub} = 124.92 \text{ kip/ft}
  \]

  For a case with cell filled to the bottom of relieving platform

  \[
  Cell_{weight2} := W_e \cdot (H_{F_{cell}} - H_{sat}) \cdot \gamma F_{wet} + W_e \cdot (H_{sat}) \cdot \gamma F_{sub} = 96.67 \text{ kip/ft}
  \]

- Stabilizing moment

  \[
  M_R := Cell_{weight} \cdot W_e \cdot 0.5 = 2520548.97 \text{ lbf-ft}
  \]

  \[
  M_{R2} := Cell_{weight2} \cdot W_e \cdot 0.5 = 1950583.77 \text{ lbf-ft}
  \]

- Overturning moment

  - Soil pressure at the top of the disposed material or top of cell. For simplicity, the surcharge is only multiplied by the active coefficient of the top layer

    \[
    SP_{tdm} := K_e B_w \cdot q_{sur} = 333.33 \text{ lbf/ft}^2
    \]

  - Soil pressure at the saturation level

    \[
    SP_{sat} := (TCS_{EL} - Sat_{EL}) \cdot K_e B_w \cdot \gamma B_{wet} + SP_{tdm} - 2 \cdot c_{BW} \cdot \sqrt{K_e B_w} = 233.65 \text{ lbf/ft}^2
    \]
• Soil pressure at the mudline

\[ SP_{mdln} := (Sat_{EL} - DM_{EL}) \cdot KaBW \cdot \gamma BW_{sub} + SP_{sat} - 2 \cdot cBW \cdot \sqrt{KaBW} = 550.84 \text{ lbf/ft}^2 \]

• Excess hydrostatic pressure at the saturation line

\[ SP_{exc} := H_{exc} \cdot \gamma_{water} = 499.20 \text{ lbf/ft}^2 \]

\[ M_O := SP_{tdm} \cdot H_{cell}^2 \cdot 0.5 + (SP_{sat} - SP_{tdm}) \cdot (H_{cell} - H_{sat}) \cdot 0.5 \cdot \left( H_{sat} + \frac{(H_{cell} - H_{sat})}{3} \right) \]

\[ + (SP_{sat} - SP_{tdm}) \cdot H_{sat}^2 \cdot 0.5 + (SP_{mdln} - SP_{sat}) \cdot \frac{H_{sat}^2}{6} \]

\[ + \left( H_{sat}^3 - H_{case1}^3 \right) \cdot \frac{\gamma_{water}}{6} + \left( SP_{exc} \cdot H_{exc} \right) \cdot \left( H_{cell} - \frac{2}{3} H_{exc} \right) \]

\[ M_O = 310.21 \frac{kip}{ft} \cdot \frac{ft}{ft} \]

\[ M_R = 2520.55 \frac{kip}{ft} \cdot \frac{ft}{ft} \]

\[ M_{R2} = 1950.58 \frac{kip}{ft} \cdot \frac{ft}{ft} \]

\[ FS_{overturning} := \left( \frac{M_R}{M_O} \right) = 8.13 \]

if \( FS_{overturning} > 2 \), “O.K.”, “N.G.” = ”O.K.”

\[ FS_{overturning2} := \left( \frac{M_{R2}}{M_O} \right) = 6.29 \]

if \( FS_{overturning} > 2 \), “O.K.”, “N.G.” = ”O.K.”

Another required control is when the cells are constructed and filled, but not backfilled, the wave condition is applicable and overturning moment is calculated below to check the stability. Also the resisting moment with partial cell fill is considered.

\[ M_{OW} := q\text{wave} \cdot H_{case1} = 384.40 \frac{kip}{ft} \cdot \frac{ft}{ft} \]
$FS_{\text{overt turning}} = \left( \frac{M_{R2}}{M_{OW}} \right) = 5.07$

\[ \text{if} \left( FS_{\text{overt turning}} > 2 \right), \text{"O.K."}, \text{"N.G."} = \text{"O.K."} \]

5.3 Bursting and Interlock Tension

The minimum required size of sheet piling is selected to satisfy a factor of safety of 1.5. Pressure calculated for the internal stability (i.e. $E_i$) applies to this control as well. However, the maximum pressure is going to be used instead of the resultant force. Also, the fill is considered wet rather than submerged. The lateral pressure coefficient of 0.5 is used based on Reference 3b. For sheets driving into soil, the hoop tension reaches zero around 0.1 times the exposed height of the cell below mudline. Also, the hoop tension is maximum around 1/3rd of the height of the cell from the fixity point. Therefore, first the fixity point is calculated and then the pressures calculated earlier are adjusted. Allowable interlock strength is calculated using a reduction factor of 0.8 based on Reference 3c.

\[ d_{\text{fixity}} := 0.1 \cdot (TCS_{EL} - D_{MEL}) = 3.90 \text{ ft} \]

Distance of fixity below the mudline

\[ y_{\text{fixity}} := D_{MEL} - d_{\text{fixity}} = -30.90 \text{ ft} \]

Elevation of fixity point
Elevation of maximum pressure

\[ y_{\text{max}} := y_{\text{fixity}} + \frac{(TCS_{EL} - y_{\text{fixity}})}{3} = -16.60 \text{ ft} \]

Maximum earth pressure at the interlocks

\[ a_{\text{max}} := (TCS_{EL} - y_{\text{max}}) \cdot \gamma_{\text{Fwet}} \cdot 0.5 = 1.79 \frac{\text{kip}}{\text{ft}^2} \]

The Tennessee Valley Authority (TVA) suggested that the maximum interlock tension can be computed from the free body diagram of the cell as follow:

\[ T_{\text{max}1} := a_{\text{max}} \cdot R_m = 3.44 \frac{\text{kip}}{\text{in}} \]

Another interlock tension control is through combined active soil pressure, surcharge, and differential water pressure.

\[ P_b := K_a B W \cdot \left( \gamma_{BW_{wet}} \cdot (H_{\text{cell}} - H_{\text{sat}}) + \gamma_{BW_{sub}} \cdot (H_{\text{sat}} - 0.25 \cdot H_{\text{cell}}) + q_{\text{sur}} \right) - \gamma_{\text{water}} \cdot (Sat_{EL} - LWL) \]

\[ P_b = 1.43 \frac{\text{kip}}{\text{ft}^2} \]

\[ T_{\text{max}3} := P_b \cdot R_m = 2.76 \frac{\text{kip}}{\text{in}} \]

\[ T_{\text{max}} := \text{max}(T_{\text{max}1}, T_{\text{max}2}, T_{\text{max}3}) = 5.35 \frac{\text{kip}}{\text{in}} \]

Per vertical inch
Based on above Table from Reference 3b, PS 31 will have the following allowable interlock strength and safety factor:

\[ \text{Intrlc}_{\text{Strength}} := 24 \cdot \frac{\text{kip}}{\text{in}} \]  
Max. interlock strength from Table above

\[ t_w := 0.5 \cdot \text{in} \]  
Web thickness from Table above

### 5.3.1 Reduction Factor for Internal Resistance
Reduction factor for internal resistance is provided by Reference 3c.

\[ \beta_R := 0.8 \]

### 5.3.2 Junction Reduction Factor
This reduction factor takes into account the behavior of the welded junction pile at ultimate limit stress.

\[ \beta_T := 0.9 \cdot \left( 1.3 - 0.8 \cdot \frac{R_a}{R_m} \right) \cdot \left( 1 - 0.3 \cdot \tan(\phi F) \right) = 0.66 \]
5.3.3 Calculating the Durability Factor

Durability factor is calculated based on the project design life and corrosion rates established in Reference 1b.

\[ \Delta t := THKR_D = 0.08 \text{ in} \]

Total web thickness reduction front and back

\[ \beta_{cor} := \frac{t_w - \Delta t}{t_w} = 0.83 \]

Correction factor for durability

5.3.4 Controlling the Safety Factor

Safety factors are controlled for both interlocks at the main cell and junction piles.

\[ T_{allow, intrinsic} := \beta_R \cdot IntrlC_{strength} = 19.20 \text{ kip in} \]

Interlock allowable resistance from table above

\[ T_{allow, web} := f_y \cdot (t_w - \Delta t) = 20.87 \text{ kip in} \]

Web allowable resistance

\[ T_{allow} := \min (T_{allow, intrinsic}, T_{allow, web}) = 19.20 \text{ kip in} \]

\[ T_{allow, cor} := T_{allow} \cdot \beta_{cor} = 16.03 \text{ kip in} \]

\[ FS_{interlock} := \left( \frac{T_{allow, cor}}{T_{max}} \right) = 3.00 \]

\[ \text{if } (FS_{interlock} > 1.5, "O.K.", "N.G." ) = "O.K." \]

\[ T_{allow, junp} := T_{allow, cor} \cdot \beta_T = 10.57 \text{ kip in} \]

Junction pile allowable resistance

\[ FS_{interlock, juncp} := \left( \frac{T_{allow, junp}}{T_{max}} \right) = 1.98 \]
5.4 Internal Stability at Cell Centerline
Soil shear resistance along the plane through cell centerline (internal stability of the cell) is another possible mode of failure. The shearing resistance along cell centerline plane, that is, the sum of the fill shear resistance and resistance in the interlocks, must be equal to or greater than the shear due to the overturning effects. A safety factor of 1.25 is required in this case as the extreme condition of high water level and total submergence of the fill is considered.

\[
Q := \frac{1.5 \cdot (MO)}{W_e} = 11.53 \text{ kip/ft} \quad \text{Shearing force on the centerline}
\]

To calculate the lateral pressure at the cell centerline a higher value of lateral pressure coefficient should be used. In this analysis, the percent increase in the lateral pressure coefficient is used to increase the lateral pressure that was calculated earlier. In order to calculate the friction resistance in the sheet-pile interlocks a friction coefficient of 0.3 is used.

\[
K_{aFadj} := \frac{\cos(\phi F)^2}{2 - \cos(\phi F)} = 0.50
\]

\[
f_i := 0.3 \quad \text{Reference 2c}
\]

\[
P_{sat} := K_{aFadj} \cdot (\gamma_{F\text{wet}} \cdot (H_{cell} - H_{sat})) = 0.50 \text{ ksf} \quad \text{Pressure inside cell at the water level}
\]

\[
P_c := K_{aFadj} \cdot (\gamma_{F\text{wet}} \cdot (H_{cell} - H_{sat}) + H_{sat} \cdot \gamma_{F_{\text{sub}}}) = 1.56 \text{ ksf} \quad \text{Pressure inside cell at the mudline}
\]

\[
P_{\text{diff.water}} := \begin{cases} 
0 \text{ lbf/ft}^2 & \text{if } H_{\text{case1}} \leq H_{sat} \\
(\gamma_{\text{water}} \cdot (H_{sat} - H_{case1})) \cdot \gamma_{\text{water}} & \text{if } H_{\text{case1}} < H_{sat}
\end{cases} = 0.00 \text{ lbf/ft}^2
\]

Prepared by: Mark Salehi Date: 10/19/2020
Reviewed by: Soren Morch / Alex Mora Date: 10/19/2020
5.5 Slipping between Fill and Cell (Tilting)

The structure must also be stable against tilting. The friction force created between sheeting and cell fill acts downward to combat this lifting action.

\[
Y_E := \left( \gamma F_{\text{wet}} \cdot (H_{\text{cell}} - H_{\text{sat}}) + \gamma F_{\text{sub}} \cdot H_{\text{sat}} \right) / H_{\text{cell}}
\]

Weighted average of fill cell densities

\[
K_{a_{\text{eff}}} := \tan \left( \frac{45 - \phi_F}{2} \right)^2
\]

\[
FS_{\text{tilting}} := \frac{1}{M_O} \cdot \frac{1}{6} \cdot Y_E \cdot W_e \cdot H_{\text{cell}} \cdot \left( 3 \cdot \tan(\phi_F)^2 - \frac{W_e}{H_{\text{cell}}} \cdot \tan(\phi_F)^3 \right) + 3 \cdot K_{a_{\text{eff}}} \cdot f_i \cdot \frac{H_{\text{cell}}}{W_e}
\]

\[
FS_{\text{tilting}} = 4.78
\]

\[
\text{if} \ (FS_{\text{tilting}} \geq 1.5, \text{"O.K."}, \text{"N.G."}) = \text{"O.K."}
\]
**5.6 Shear at Cell Fill**

Factor of safety against shear at cell fill, sheet pile interface.

\[ P_{pzero} = \gamma F_{sub} \cdot H_{case 1}^2 \cdot 0.5 \]

k=1 is used based on Reference 2c

\[ FS_{cellfill} = \frac{W_e}{M_o} \cdot \left( \left( P_{pzero} + \frac{P_{pcl} \cdot W_e}{CL} \right) \cdot \tan(\delta F) + P_{pcl} \cdot f_i \cdot \frac{W_e}{CL} \right) = 3.35 \]

if \( FS_{cellfill} > 1.5 \), "O.K.", "N.G." = "O.K."

**5.7 Horizontal Shear**

Horizontal shear force acting on a sheet pile structure can cause the cell to tilt. The check for horizontal shear is completed following the Cummings' method described in Reference 2b (Page C-16).

\[ \phi = 30 \cdot \text{deg} \]

\[ c_c = W_e \cdot \tan(\phi) = 23.30 \text{ ft} \]

\[ a = H_{cell} - c_c = 15.70 \text{ ft} \]

\[ R1 = \gamma F_{sub} \cdot a \cdot c_c = 24.73 \text{ kip/ft} \]

\[ R2 = \gamma F_{sub} \cdot c_c^2 = 36.69 \text{ kip/ft} \]

\[ h_r = \frac{\left( R2 \cdot \frac{c_c}{3} + R1 \cdot \frac{c_c}{2} \right)}{R1 + R2} = 9.33 \text{ ft} \]

\[ M_r = (R1 + R2) \cdot h_r = 573.06 \text{ kip-ft} \]
\[ P_{\text{max}} := P_{\text{sat}} + \left( H_{\text{sat}} - y_{\text{max}} \right) \cdot y_{\text{sub}} \cdot K_{\text{adj}} \]

\[ E_i := P_{\text{sat}} \cdot \left( H_{\text{cell}} - H_{\text{sat}} \right) \cdot 0.5 + \left( P_{\text{max}} + P_{\text{sat}} \right) \cdot \left( H_{\text{sat}} - y_{\text{max}} \right) \cdot 0.5 \quad j = 79.95 \text{ kip/ft} \]

\[ M_i := E_i \cdot f_i \cdot W_e = 967.85 \text{ kip-ft} \]

\[ FS_{\text{horshr}} := \frac{(M_r + M_i)}{M_O} = 4.97 \]

\[ \text{if } (FS_{\text{horshr}} > 1.5, \text{“O.K.”, “N.G.”}) = \text{“O.K.”} \]

### 5.8 Bearing Capacity

Bearing capacity for the combination impact of weight and overturning for the sandy clay layer are evaluated here. Also, from earlier, the bottom of the cell is extending into **Layer 4** and therefore its characteristics are used. Conservatively, a shearing resistance angle of 20 degrees is used. The commonly acceptable factor of safety is 3 based on Reference 2b.

\[ \phi' := 25 \text{ deg} \]

\[ a := e^{\left\{0.75 \cdot n - \frac{\phi'}{2}\right\} \cdot \tan (\phi')} = 2.71 \]

\[ N_q := \frac{a^2}{2 \cdot \left( \cos \left(45 \text{ deg} + \frac{\phi'}{2}\right) \right)^2} = 12.72 \]

\[ N_c := (N_q - 1) \cdot \cot (\phi') = 25.13 \]

\[ k := 3 \cdot \left( \tan \left(45 \text{ deg} + \frac{\phi' + 33 \text{ deg}}{2}\right) \right)^2 = 36.49 \]

\[ N_y := \frac{\tan (\phi')}{2} \cdot \left( \frac{k}{\left( \cos (\phi') \right)^2 - 1} \right) = 10.12 \]
$\frac{\text{Cell weight}}{W_e} = 3.10 \text{ ksf}$

$6 \cdot \frac{(M_o)}{W_e^2} = 1.14 \text{ ksf}$

$q_f := cL2 \cdot N_c + 0.5 \cdot \gamma L2_{sub} \cdot W_e \cdot N_q + (DM_{EL} - BCS_{EL}) \cdot \gamma L2_{sub} \cdot N_q = 46.56 \text{ ksf}$

$FS_{bearing} := \frac{q_f}{\frac{Cell weight}{W_e} + 6 \cdot \frac{(M_o)}{W_e^2} + q_{sur\_cell}} = 9.83$

\[ \text{if } (FS_{bearing} > 3, "O.K.", "N.G." ) = "O.K." \]

### 5.9 Pullout of Inboard (Land Side) Sheets

The penetration must be adequate to insure stability with respect to pull-out of the outboard sheeting due to tilting.

$D_{emb} := (DM_{EL} - BCS_{EL}) = 25.00 \text{ ft}$

$D_{embL1} := Thk_{L1} = 14.25 \text{ ft}$

$D_{embL2} := D_{emb} - Thk_{L1} = 10.75 \text{ ft}$

\[ \text{if } (D_{embL2} < Thk_{L2}, "O.K.", "Cells extend to Layer 3") = "Cells extend to Layer 3" \]

$D_{embL2} := Thk_{L2} = 5.00 \text{ ft}$

$D_{embL3} := D_{emb} - Thk_{L1} - Thk_{L2} = 5.75 \text{ ft}$

\[ \text{if } (D_{embL3} < Thk_{L3}, "O.K.", "Cells extend to Layer 4") = "O.K." \]
Calculations:

\[ Q_u := \left[ 0.5 \cdot K_{aL1} \cdot \gamma_{L1_{sub}} \cdot D_{embrL1}^2 \cdot \tan(\delta L1) \right] + 0.5 \cdot K_{aL2} \cdot \gamma_{L2_{sub}} \cdot D_{embrL2}^2 \cdot \tan(\delta L2) + 0.5 \cdot K_{aL3} \cdot \gamma_{L3_{sub}} \cdot D_{embrL3}^2 \cdot \tan(\delta L3) + cL2 \cdot D_{embrL2} + cL3 \cdot D_{embrL3} \right] \cdot 2 \cdot 1 \cdot \text{ft} = 12.44 \text{ kip} \\
\]

\[ Q_p := \frac{(M_0 \cdot 1 \cdot \text{ft})}{3 \cdot W_e \cdot \left( 1 + \frac{W_e}{4 \cdot CL} \right)} = 2.20 \text{ kip} \]

Passive pressure is not included

\[ FS_{pullout} := \frac{Q_u}{Q_p} = 5.66 \]

If \( FS_{pullout} > 1.5 \), “O.K.”, “N.G.” = “O.K.”

5.10 Pullout of Outboard (Lake side) Sheets

USACE methodology described on Page C-18 of Reference 2b is used.

\[ DM_{in} := DM_{EL} - BOCS_{EL} = 25.00 \text{ ft} \]

\[ P_1 := (HWL - DM_{EL}) \cdot \gamma_{water} = 1.93 \text{ kip ft}^{-2} \]

\[ P_2 := P_1 + DM_{in} \cdot (\gamma_{water} + \gamma L2_{sub} \cdot K_{aL2}) = 4.14 \text{ kip ft}^{-2} \]

\[ P_3 := (Sat_{EL} - DM_{EL}) \cdot (\gamma_{water} + \gamma F_{sub} \cdot K_{aF}) + (TCS_{EL} - Sat_{EL}) \cdot \gamma F_{wet} \cdot K_{aF} = 2.77 \text{ kip ft}^{-2} \]
Recalculating the outboard sheeting forces based on USACE methodology described on Page C-15 through C-18 of Reference 2b.

\[
P'_p := Y_{L3_{sub}} \cdot K_{pL3} = 232.80 \text{ psf/ft}
\]

\[
K_{outboard} := KaL3 \cdot 1.4 = 0.47
\]

\[
P := Y_{L3_{sub}} \cdot K_{outboard} + Y_{water} = 98.61 \text{ psf/ft}
\]

\[
P' := Y_{L3_{sub}} \cdot K_{outboard} = 36.21 \text{ psf/ft}
\]

\[
P_x := P'_p - P' = 196.59 \text{ psf/ft}
\]

\[
h_f := \frac{SP_{mdlin}}{P_x} = 2.80 \text{ ft} \quad \text{Point of fixity}
\]

\[
H' := TF_{EL} - DM_{EL} + h_f = 41.80 \text{ ft}
\]
5.11 Foundation Load Eccentricity

Eccentricity of the load applied to the foundation, i.e. bottom of the embedded flat sheets, is calculated here. Total soil pressure is calculated in ProSheet using the input provided earlier. First the moments acting on the
First the moments acting on the bottom of the cell is calculated using active and passive pressures calculated above and then the eccentricity is estimated by dividing the sum of them by the total net vertical pressure due to weight of the material inside of the cell. Surcharge is considered on the back as per BOD (i.e. 500 psf up to 30 feet and then 1000 psf). Total cell weight is considered instead of equivalent cell weight. Since full surcharge is considered in the back, realistically, one-quarter of the surcharge is also applied on the cell.

\[
M_1 := 0.144 \text{ ksf} \cdot 1 \text{ ft} \cdot 39 \text{ ft} \left(25 \text{ ft} + 39 \text{ ft} \cdot \frac{ft}{2}\right) = 249.91 \text{ kip} \cdot \text{ft}
\]
\[
M_2 := \left(0.378 \text{ ksf} - 0.144 \text{ ksf}\right) \cdot 1 \text{ ft} \cdot 6 \text{ ft} \cdot \left(64 \text{ ft} - \frac{2}{3} \cdot 6 \text{ ft}\right) = 42.12 \text{ kip} \cdot \text{ft}
\]
\[
M_3 := (0.378 \text{ ksf} - 0.144 \text{ ksf}) \cdot (39 \text{ ft} - 6 \text{ ft}) \cdot 1 \text{ ft} \cdot \left(25 \text{ ft} + \frac{31 \text{ ft}}{2}\right) = 312.74 \text{ kip} \cdot \text{ft}
\]
\[
M_4 := (0.546 \text{ ksf} - 0.378 \text{ ksf}) \cdot 1 \text{ ft} \cdot 2 \text{ ft} \cdot \left(64 \text{ ft} - 6 \text{ ft} - \frac{2}{3} \cdot 2 \text{ ft}\right) = 9.52 \text{ kip} \cdot \text{ft}
\]
\[
M_5 := (0.546 \text{ ksf} - 0.378 \text{ ksf}) \cdot 1 \text{ ft} \cdot (39 \text{ ft} - 8 \text{ ft}) \cdot \left(25 \text{ ft} + 31 \text{ ft} \cdot \frac{ft}{2}\right) = 210.92 \text{ kip} \cdot \text{ft}
\]
\[
M_6 := \left(1.238 \text{ ksf} - 0.546 \text{ ksf}\right) \cdot 31 \text{ ft} \cdot 1 \text{ ft} \cdot \left(25 \text{ ft} + 31 \text{ ft} \cdot \frac{ft}{3}\right) = 378.99 \text{ kip} \cdot \text{ft}
\]
\[
M_7 := \left(1.022 \text{ ksf} - 1 \text{ ft} \cdot 31 \text{ ft}\right) \cdot \left(25 \text{ ft} - 4 \text{ ft} - \frac{2}{3} \cdot 4 \text{ ft}\right) = 37.47 \text{ kip} \cdot \text{ft}
\]
\[
M_8 := -6.72 \text{ ksf} \cdot 21 \text{ ft} \cdot 1 \text{ ft} \cdot 21 \text{ ft} \cdot \frac{ft}{3} = -493.92 \text{ kip} \cdot \text{ft}
\]
\[
M_{\text{total}} := M_1 + M_2 + M_3 + M_4 + M_5 + M_6 + M_7 + M_8 = 747.76 \text{ kip} \cdot \text{ft}
\]
\[
P := R_m \cdot 2 \cdot 1 \text{ ft} \cdot \left(\frac{H_{\text{cell}} - H_{\text{sat}}}{\gamma_{\text{wet}}} + \frac{H_{\text{sat}}}{\gamma_{\text{sub}}}\right) + \frac{(D_{\text{EL}} - B_{\text{L3EL}}) \cdot \gamma_{L2_{\text{sub}}} + (B_{\text{L3EL}} - B_{\text{L4EL}}) \cdot \gamma_{L3_{\text{sub}}} + (B_{\text{L4EL}} - B_{\text{CSel}}) \cdot \gamma_{L4_{\text{sub}}}}{4} = 190.53 \text{ kip}
\]
\[
e := \frac{M_{\text{total}}}{P} = 3.92 \text{ ft}
\]
\[
e_{\text{all}} := \frac{R_m \cdot 2}{6} = 7.70 \text{ ft}
\]
\[
\text{if } (e_{\text{all}} > e, \text{"O.K."}, \text{"N.G."}) = \text{"O.K."}
\]
Overturning is also calculated using this method to check the acceptability.

\[ MO := M_1 + M_2 + M_3 + M_4 + M_5 + M_6 + M_7 = 1241.68 \text{ kip} \cdot \text{ft} \]

\[ MR := -M_8 + P \cdot R_m = 4897.72 \text{ kip} \cdot \text{ft} \]

\[ FS_{\text{overturning}} := \frac{MR}{MO} = 3.94 \]

\[
\begin{align*}
\text{if } (FS_{\text{overturning}} \geq 2, "O.K."; "N.G." ) = "O.K." \end{align*}
\]

Maximum and minimum soil pressures at the bottom of the cell are.

\[
\begin{align*}
\Sigma_{\text{max}} &= \frac{M_{\text{total}} \cdot R_m}{1 \cdot 1 \text{ ft} \cdot (R_m \cdot 2)^3} + \frac{P}{R_m \cdot 2 \cdot 1 \text{ ft}} = 6.22 \text{ ksf} \\
\Sigma_{\text{min}} &= \frac{-M_{\text{total}} \cdot R_m}{1 \cdot 1 \text{ ft} \cdot (R_m \cdot 2)^3} + \frac{P}{R_m \cdot 2 \cdot 1 \text{ ft}} = 2.02 \text{ ksf}
\end{align*}
\]

**5.12 Settlement**

Immediate as well as primary and secondary consolidation settlements are performed using site-specific consolidation tests and guidelines provided by References 2b and 3d. It is provided as a stand alone analysis.
SECTION 6: RESULTS

Analysis results show that the interlock safety factor is less than required standard values when the crane is on the wall. Therefore, a relieving platform is going to be designed to carry part of the crane loads.

- Maximum embedment below the mudline (Landside) \( D_{\text{emb}} = 25.00 \text{ ft} \)
- Maximum embedment below the mudline (Lakeside) \( D_{\text{m}} = 25.00 \text{ ft} \)
- Cell diameter \( R_m \cdot 2 = 46.23 \text{ ft} \)

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<th>ID</th>
<th>Failure Mechanism</th>
<th>Req. SF</th>
<th>Prov. SF</th>
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<tr>
<td>1</td>
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<td>5.07</td>
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<tr>
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<tr>
<td>4</td>
<td>Tilting</td>
<td>2</td>
<td>2.99</td>
</tr>
<tr>
<td>5</td>
<td>Cell Fill</td>
<td>2</td>
<td>2.09</td>
</tr>
<tr>
<td>6</td>
<td>Horizontal Shear</td>
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<td>7</td>
<td>Soil Bearing</td>
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<td>4.46</td>
</tr>
<tr>
<td>8</td>
<td>Outboard Pull-out</td>
<td>1.5</td>
<td>3.54</td>
</tr>
<tr>
<td>9</td>
<td>Inboard Pull-out</td>
<td>1.5</td>
<td>6.04</td>
</tr>
</tbody>
</table>

\[
\text{FailMech1} \equiv \begin{bmatrix} \text{FS}_{\text{overturing}} & \text{FS}_{\text{interlock}} & \text{FS}_{\text{InterFric}} & \text{FS}_{\text{tilting}} \end{bmatrix}
\]

\[
\text{FailMech2} \equiv \begin{bmatrix} \text{FS}_{\text{cellfill}} & \text{FS}_{\text{horshr}} & \text{FS}_{\text{bearing}} & \text{FS}_{\text{pullout}} & \text{FS}_{\text{ipullout}} \end{bmatrix}
\]

\[
\text{FailMech1} = \begin{bmatrix} 5.07 & 1.98 & 2.75 & 4.78 \end{bmatrix}
\]

\[
\text{FailMech2} = \begin{bmatrix} 3.35 & 4.97 & 9.83 & 5.66 & 9.85 \end{bmatrix}
\]

\[
\text{ReqSF} = \begin{bmatrix} 2 & 1.5 & 1.5 & 2 & 2 & 3 & 1.5 & 1.5 \end{bmatrix}
\]
SECTION 7: REFERENCES

PROJECT DOCUMENTS
1b Milwaukee Estuary DMMF Basis of Design
1c Milwaukee CDF, Geotechnical Data, USACE, July 2007
1d Milwaukee CDF Drilling Logs, August 2006.
1e Foth Calculation, Structural Soil Characterization and Properties Determination.

CODES AND STANDARDS
2a Design of Sheet Pile Walls, USACE, EM 1110-2-2504
2b Design of Sheet Pile Cellular Structures, USACE, EM-1110-2-2503
2c Foundations & Earth Structures, Design Manual 7.02, NAVFAC

GUIDELINES
3a Pile Buck Steel Sheet Piling Design Manual
3b Handbook of Port and Harbor Engineering, Geotechnical and Structural Aspects

Prepared by: Mark Salehi Date: 10/19/2020
Reviewed by: Soren Morch / Alex Mora Date: 10/19/2020
# Structural Design and Analysis of Concrete Deck

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SECTION 1: PURPOSE AND SCOPE:

The Milwaukee Dredged Material Management Facility (DMMF) is planned to be constructed on the north side of the existing Jones Island DMDF to provide additional capacity for the disposal of dredged material. Milwaukee Harbor is located on the west shore of Lake Michigan. The new DMMF structure will be comprised of cellular coffer dam wall on all two (2) sides with a docking platform on a portion of the north side. The scope of this calculation is to present the structural analysis of the concrete deck of the relieving platform.

SECTION 2: ASSUMPTIONS

Assumptions are discussed and justified as deemed necessary throughout this calculation.

SECTION 3: METHODOLOGY AND ACCEPTANCE CRITERIA

American Concrete Institute (ACI) guideline from Reference 2a along with applicable loads and load combinations defined by the UFC guideline (Reference 2b) are used to design the platform.

The concrete deck is analyzed using the output of STAAD.Pro platform model and this MathCAD document. Forces will be extracted from the model to check the ACI code requirement and compute the required reinforcement. In the STAAD mode, plate elements are used to simulate the concrete deck and beam elements are used for modeling beam and pipes. Following items are specifically considered in the analysis:

1. Since the vertical loads are being transferred to the ground/seabed through pipe piles, a fixity point of 8D, where D is the diameter of the pipe, is estimated for the fixity point. This will indirectly model the soil-structure interaction. Piles that fall outside of the cells are designed as a length of 8D +27 ft + (distance from LWD to the bottom of portal).

2. For modeling of the platform a two-way slab with peripheral beam is used. During lateral loads, it acts as a diaphragm and transfer the load to the piles.

3. Clear cover to reinforcement shall be 3 inches, minimum (Reference 2c).
SECTION 4: DESIGN INPUTS

4.1 Applicable Loads

Only applicable loads are presented. Any other loads shown on the above table that are not discussed are not applicable to this project.

1. Dead Load (D) - Self weight of construction materials and other structural components.
2. Live Load (Lu) - Uniform distributed live load.
3. Live Load (Lc) - Concentrated live loads.
4. Buoyancy Load (B) - Buoyancy load will be considered for the piles from fixity point to mean low water level.
5. Berthing (Be) - Berthing loads are equivalent to the rubber fender reaction times 1.1 tolerance.
6. Mooring/Breasting Load (M) - bollard loads are considered.
7. Earthquake Load (EQ) is calculated.
8. Earth Load (H) from back of the platform is considered.
9. Wave Load (W) considered on the structure.
10. Ice (Ice) load is applied to the structure.
4.2 Material

1. Concrete shall have compressive strength at 28 days of 5000 psi minimum.

2. Steel pipe pile shall be fabricated in compliance with ASTM A252, with a minimum yield strength of 50 ksi.

4.3 Loading

4.3.1 Dead Load

Dead load will be applied based on the weight of the structure in STAAD.Pro Software.

\[ Y_{\text{concrete}} := 150 \text{ pcf} \]  
Density of Concrete

\[ Y_{\text{fill}} := 125 \text{ pcf} \]  
Density of saturated fill over the deck

4.3.2 Live Load

A uniform live load is considered on the platform. Additionally, the load from a Manitowoc 2250 series 3 crawler crane is applicable to the deck.

\[ LL := 500 \text{ psf} \]

\[ T_{cw} := 47.3 \text{ in} \]

\[ L_{\text{tread}} := 328.9 \text{ in} \]
\[ P_{23} := 15.9 \text{ psi} \cdot T_{cw} \cdot L_{tread} + 0.5 \cdot (36.9 - 15.9) \text{ psi} \cdot T_{cw} \cdot L_{tread} = 410.70 \text{ kip} \]

\[ P_{\text{side}} := 28.6 \text{ psi} \cdot T_{cw} \cdot L_{tread} + 0.5 \cdot (30.6 - 28.6) \text{ psi} \cdot T_{cw} \cdot L_{tread} = 460.49 \text{ kip} \]

\[ P_{\text{front}} := 12.9 \text{ psi} \cdot T_{cw} \cdot L_{tread} + 0.5 \cdot (35.9 - 12.9) \text{ psi} \cdot T_{cw} \cdot L_{tread} = 379.59 \text{ kip} \]

Maximum pressure is for the boom over side position. This pressure will distributed over an area equivalent of a 30 degree spread through structural fill.

\[ p_{\text{min}} := 28.6 \text{ psi} \]

\[ p_{\text{max}} := 30.6 \text{ psi} \]

\[ h_{\text{fill}} := 2 \text{ ft} \]

\[ A_{\text{spread}} := 30 \text{ deg} \]

\[ T'_{cw} := T_{cw} + 2 \cdot \tan (A_{\text{spread}}) \cdot h_{\text{fill}} = 6.25 \text{ ft} \]

\[ L'_{\text{tread}} := L_{tread} + 2 \cdot \tan (A_{\text{spread}}) \cdot h_{\text{fill}} = 29.72 \text{ ft} \]

\[ p'_{\text{min}} := \frac{p_{\text{min}} \cdot T_{cw}}{T'_{cw}} = \frac{83552.59}{\text{lb}} \cdot \frac{1}{\text{ft} \cdot \text{s}^2} \]

\[ p'_{\text{max}} := \frac{p_{\text{max}} \cdot T_{cw}}{T'_{cw}} = \frac{89395.43}{\text{lb}} \cdot \frac{1}{\text{ft} \cdot \text{s}^2} \]

The crane tread pressure will be applied on the concrete deck to develop the worst effect. Spacing of treads is 22 ft c/c (See Manitowoc 2250 Product Guide).

\[ p_{\text{ave}} := 0.5 \cdot (p'_{\text{min}} + p'_{\text{max}}) = 2.69 \text{ ksf} \]

Apply pressure on an area 6' wide x 30' long.
4.3.3 Buoyancy

$\gamma_{\text{seawater}} = 64 \text{pcf}$

For 20" diameter pipe pile.

$D_{\text{pile}} = 20 \text{ in}$

$V_{\text{pile}} = \frac{n}{4} \cdot D_{\text{pile}}^2$

$V_{\text{pile}} = 2.18 \text{ ft}^2$  
Volume of pile per foot basis.

$B_{20} = V_{\text{pile}} \cdot \gamma_{\text{seawater}} = 0.14 \frac{\text{kip}}{\text{ft}}$  
Buoyancy on concrete filled piles.

For 24" diameter pipe pile.

$D_{\text{pile}} = 24 \text{ in}$

$V_{\text{pile}} = \frac{n}{4} \cdot D_{\text{pile}}^2$

$V_{\text{pile}} = 3.14 \text{ ft}^2$  
Volume of pile per foot basis.

$B_{24} = V_{\text{pile}} \cdot \gamma_{\text{seawater}} = 0.20 \frac{\text{kip}}{\text{ft}}$  
Buoyancy on concrete filled piles.

For 32" diameter pipe pile.

$D_{\text{pile}} = 32 \text{ in}$

$V_{\text{pile}} = \frac{n}{4} \cdot D_{\text{pile}}^2$

$V_{\text{pile}} = 5.59 \text{ ft}^2$  
Volume of pile per foot basis.

$B_{32} = V_{\text{pile}} \cdot \gamma_{\text{seawater}} = 0.36 \frac{\text{kip}}{\text{ft}}$  
Buoyancy on concrete filled piles.
4.3.4 Berthing

\[ Be := 1.1 \times 1196.41 \text{ kN} \]

A tolerance factor of 1.1 is applied to the fender reaction for a SC 1450 H1.

\[ Be = 295.86 \text{ kip} \]

\[ \mu = 0.2 \]

Coefficient of static friction between steel and UHMWPE.

\[ Be_f = Be \times \mu = 59.17 \text{ kip} \]

Note: Berthing load will consider the following loads.

- The total fender reaction load applied perpendicular to the face of the fender panel.
- The lateral load from the friction of the total fender reaction applied parallel to the face of the fender panel.

Berthing loads will be applied at El. +4.0'.

Additional berthing load requested by Port are:

- 16 kip/ft over 40 feet impact length for Ore carrier
- 22 kip/ft over 30 feet impact length for other carrier - Control

These cases will be applied as independent load combinations.

4.3.5 Earthquake

\[ I := 1 \]

Importance factor for Risk Category II, Table 1.5-2 of ASCE 7

\[ R := 2 \]

Response modification coefficient

\[ C_d := 2 \]

Deflection amplification coefficient

\[ S_s := 0.076 \]

Ground motion (period = 0.2s)
\[ S_i := 0.048 \quad \text{Ground motion (period = 0.1s)} \]
\[ S_{MS} := 0.121 \quad \text{Site-modified spectral acceleration value} \]
\[ S_{M1} := 0.116 \quad \text{Site-modified spectral acceleration value} \]
\[ S_{DS} := 0.081 \quad \text{Numeric seismic design value at 0.2s SA} \]
\[ S_{D1} := 0.077 \quad \text{Numeric seismic design value at 1.0s SA} \]
\[ PGA := 0.037 \quad \text{Peak ground acceleration} \]

\[ C_s := \frac{S_{DS}}{\left( \frac{R}{I} \right)} = 0.04 \quad \text{Seismic Response Coefficient} \]

\[ T_L := 12 \, s \quad \text{Long period transition period TL} \]

\[ C_t := 0.016 \quad \text{Concrete moment resisting frame, Table 12.8-2 of ASCE 7} \]
\[ x := 0.9 \]
\[ h_n := 45 \quad \text{Distance to fixity point of piles} \]

\[ T_a := C_t \cdot h_n^x = 0.49 \quad \text{Approximate fundamental period} \]

\[ C_{s_{\max}} := \frac{S_{D1}}{\left( T_a \cdot \frac{R}{I} \right)} = 0.08 \]

\[ C_{s_{\min}} := 0.44 \cdot S_{DS} \cdot I = 0.04 \]

\[ W_{20\_pipe} := 104.23 \, \text{lbf/ft} \quad W_{24\_pipe} := 171.45 \, \text{lbf/ft} \]
\[ W_{\text{tran}} := (2 \, \text{ft} \cdot Y_{\text{fill}} + 3 \, \text{ft} \cdot Y_{\text{concrete}}) \cdot 15 \, \text{ft} \cdot 30 \, \text{ft} = 325.06 \, \text{kip} \]
\[ + (2 \cdot W_{24\_pipe} + W_{20\_pipe}) \cdot 0.5 \cdot h \cdot \text{ft} \]

\[ W_{\text{long}} := (2 \, \text{ft} \cdot Y_{\text{fill}} + 3 \, \text{ft} \cdot Y_{\text{concrete}}) \cdot 30 \, \text{ft} \cdot 120.85 \, \text{ft} = 2628.39 \, \text{kip} \]
\[ + (9 \cdot W_{20\_pipe} + 18 \cdot W_{24\_pipe}) \cdot 0.5 \cdot h \cdot \text{ft} \]

\[ V_{\text{tran}} := \frac{C_s \cdot W_{\text{tran}}}{15 \, \text{ft}} = 0.88 \, \frac{\text{kip}}{\text{ft}} \]

\[ V_{\text{long}} := \frac{C_s \cdot W_{\text{long}}}{30 \, \text{ft}} = 3.55 \, \frac{\text{kip}}{\text{ft}} \]

\[ k := 0.5 \cdot PGA = 0.02 \]

\[ k' := 0.7 \cdot k = 0.01 \]

### 4.3.6 Mooring

\[ M := 120 \, \text{kip} \]

Mooring load will be applied to generate the worst effect on the structure. 60 Ton bollards.

It will be applied at 60 degrees respect berth/fender line and at an angle of 60 degrees up or 30 degrees down.

### 4.3.7 Notional Loads

Notional loads shall be applied as lateral loads at the top level for this platform. The notional loads shall be additive to other lateral loads and shall be applied in all load combinations.

\[ a := 1.0 \]

For LRFD equations (1.6 for ASD equations).

\[ Y_{l_{\text{tran}}} := W_{\text{tran}} + LL \cdot 15 \, \text{ft} \cdot 30 \, \text{ft} \]

Gravity load applied at level i from the LRFD load combination.
4.3.8 Earth Loads

Consider earth loads from 5 feet deep backfill subject to 500 psf UDL.

\[ K_a = 0.333 \text{ (Rankine)} \]
\[ \text{Depth of Fill} = 5\text{ft} \]

\[ H = 500 \text{ psf} \cdot 5 \text{ ft} + 0.5 \cdot Y_{\text{fill}} \cdot (5 \text{ ft})^2 \cdot 0.333 = 3.02 \frac{\text{kip}}{\text{ft}} \]

4.3.9 Wave

A horizontal wave load of 6.2 kip/ft will be applied at El. +2.0'.

\[ W = 6.2 \frac{\text{kip}}{\text{ft}} \]

4.3.10 Ice

A horizontal ice load of 38.0 kip/ft will be applied at El. -2.0' on piles.

A vertical ice load of 160 kip upward and 135 kip downward will be applied to piles.
**SECTION 5: CALCULATION**

Following figure shows the geometry of the structural model as simulated in the STAAD.Pro software.

\[ Ice_h := 38 \, \text{kip/ft} \]

Horizontal ice load - general.

\[ Ice_{v,up} := 160 \, \text{kip} \]

Upward ice load on piles.

\[ Ice_{v,down} := 135 \, \text{kip} \]

Downward ice load on piles.
5.1 Plate Stresses and Reactions

Plate Centre Stress Summary

<table>
<thead>
<tr>
<th>Plate</th>
<th>L/C</th>
<th>Qx (psi)</th>
<th>Qy (psi)</th>
<th>Sx (psi)</th>
<th>Sy (psi)</th>
<th>Mx (kip ft)</th>
<th>My (kip ft)</th>
<th>Mxy (kip ft)</th>
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<tr>
<td>Max Ox</td>
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<td>38 USG 1.2D+</td>
<td>-169.9</td>
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<tr>
<td>Max Qy</td>
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<td>53 USG 1.2D+</td>
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<td>114.2</td>
<td>-2.7</td>
<td>-9.6</td>
<td>1.4</td>
<td>47.4</td>
</tr>
<tr>
<td>Min Qy</td>
<td>449</td>
<td>40 USI 1.2D+</td>
<td>-149.3</td>
<td>-131.3</td>
<td>-9.2</td>
<td>4.3</td>
<td>4.7</td>
<td>46.4</td>
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<tr>
<td>Max Sx</td>
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<td>41 USJ 1.2D+</td>
<td>0.2</td>
<td>0.6</td>
<td>167.6</td>
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<td>-0.5</td>
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<tr>
<td>Min Sx</td>
<td>1265</td>
<td>40 USI 1.2D+</td>
<td>24.3</td>
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<td>-17.3</td>
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<tr>
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<tr>
<td>Min Sy</td>
<td>308</td>
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<td>-10.8</td>
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<tr>
<td>Max Sxy</td>
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<td>21.2</td>
<td>60.7</td>
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<td>40 USI 1.2D+</td>
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<tr>
<td>Max My</td>
<td>154</td>
<td>7.8RTHING</td>
<td>47.1</td>
<td>56.8</td>
<td>4.1</td>
<td>12.4</td>
<td>8.4</td>
<td>17.0</td>
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<tr>
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<td>47.4</td>
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<tr>
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<td>154</td>
<td>25 USJ 1.2D+</td>
<td>30.1</td>
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<td>4.7</td>
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<tr>
<td>Min Mxy</td>
<td>47</td>
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Reaction Summary

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<tr>
<th>Node</th>
<th>L/C</th>
<th>FX (kip)</th>
<th>FY (kip)</th>
<th>FZ (kip)</th>
<th>MX (kip ft)</th>
<th>MY (kip ft)</th>
<th>MZ (kip ft)</th>
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<tbody>
<tr>
<td>Max FX</td>
<td>4</td>
<td>74 USG D+LCF</td>
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<td>-250.9</td>
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<td>Min FX</td>
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<td>76 USI D+LCR</td>
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<td>-1.9</td>
</tr>
<tr>
<td>Max FY</td>
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<td>89 USG D+LCF</td>
<td>0.1</td>
<td>385.1</td>
<td>8.9</td>
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<tr>
<td>Min FY</td>
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<tr>
<td>Max FZ</td>
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<td>62 USG D+LCF</td>
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<td>85.7</td>
<td>66.3</td>
<td>675.1</td>
<td>17.8</td>
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<tr>
<td>Min FZ</td>
<td>2</td>
<td>75 USH D+LCF</td>
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<td>-35.0</td>
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</tr>
<tr>
<td>Max MX</td>
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<td>62 USH D+LCF</td>
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<td>85.7</td>
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<td>675.1</td>
<td>17.8</td>
</tr>
<tr>
<td>Min MX</td>
<td>2</td>
<td>75 USH D+LCF</td>
<td>2.2</td>
<td>-5.4</td>
<td>-35.0</td>
<td>-371.9</td>
<td>-3.4</td>
</tr>
<tr>
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<td>62 USH D+LCF</td>
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<td>63.5</td>
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<td>-31.9</td>
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<td>Max MZ</td>
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<td>-1.9</td>
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<td>Min MZ</td>
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<td>74 USG D+LCF</td>
<td>7.7</td>
<td>122.9</td>
<td>-24.5</td>
<td>-263.2</td>
<td>-0.1</td>
</tr>
</tbody>
</table>

Description of output:

1. Qx and Qy are shear stresses (Force/unit length/thickness). These stresses may be compared to allowable shear stress. These shear stresses are applied to the plate side perpendicular to the respective axis.
2. Sx and Sy are membrane stresses (Force/unit length/thickness). These stresses may be compared to allowable compressive or tensile stress.

3. Sxy is in-plane shear stress (Force/unit length/thickness).

4. Mx, My, Mxy are moments per unit width (Force x Length/length). These moments act on the plate side perpendicular to the respective axis.

For purposes of this analysis/design, the main direction is the longitudinal direction. The secondary direction is the transverse direction.

### 5.2 Section Properties

- $f_c := 5000 \text{ psi}$  \hspace{1cm} Concrete compression at 28 days
- $f_y := 60000 \text{ psi}$ \hspace{1cm} Yield of reinforcing steel
- $b_w := 1 \text{ ft}$
- $CL := 3 \text{ in}$ \hspace{1cm} Rebar cover
- $h := 36 \text{ in}$ \hspace{1cm} Thickness of the slab
- $d_{#4\_bar} := 0.75 \text{ in}$
- $d_{#8\_bar} := 1 \text{ in}$
- \begin{align*}
    d_{\text{tran}} &= h - CL - 0.5 \cdot d_{#8\_bar} = 32.50 \text{ in} \\
    d_{\text{long}} &= h - CL - 1.5 \cdot d_{#8\_bar} = 31.50 \text{ in}
\end{align*}
5.3 Concrete Deck Design

5.3.1 Design Parameters

\[ \lambda := 1.0 \] 
Modification factor, ACI Table 19.2.4.2

\[ M_{u,x} := 66 \text{ kip} \cdot \text{ft} \] 
Ultimate moment per foot of length, longitudinal direction (local X)

\[ M_{u,y} := 158 \text{ kip} \cdot \text{ft} \] 
Ultimate moment per foot of length, transverse direction (local Y)

\[ V_{u,x} := 0.17 \text{ ksi} = 24.48 \text{ ksf} \] 
Ultimate shear per square foot, longitudinal direction

\[ V_{u,y} := 0.131 \text{ ksi} = 18.86 \text{ ksf} \] 
Ultimate shear per square foot, transverse direction

\[ T_u := 54 \text{ kip} \cdot \text{ft} \] 
Ultimate torsion per foot of length

\[ E := 57000 \cdot \sqrt{f'c \cdot \text{psi}} = 4030.51 \text{ ksi} \] 
ACI 19.2.2.1.b

5.3.2 ACI Beam Requirements - ACI Chapter 9

Minimum Flexural Reinforcement - Transverse Direction

\[ b_w = 1.00 \text{ ft} \] 
Assumes one layer of #8 flexure bars.

\[ d_{\text{tran}} = 32.50 \text{ in} \]

\[ A_{s,\text{min,tran}} := \max \left( \frac{3 \cdot \sqrt{f'c \cdot \text{psi}}}{f_y} \cdot b_w \cdot d_{\text{tran}}, \frac{200 \text{ psi} \cdot b_w \cdot d_{\text{tran}}}{f_y} \right) \]

\[ A_{s,\text{min,tran}} = 1.38 \text{ in}^2 \] 
ACI 9.6.1.2
Minimum Flexural Reinforcement - Longitudinal Direction

\[ b_w = 1.00 \text{ ft} \]

\[ d_{long} = 31.50 \text{ in} \quad \text{Assumes one layer of #8 flexure bars.} \]

\[
A_{s\_min\_long} \geq \max \left( \frac{3 \cdot \sqrt{f'c \cdot \text{psi}}}{f_y \cdot b_w \cdot d_{long}}, \frac{200 \text{ psi}}{f_y \cdot b_w \cdot d_{long}} \right) \quad \text{ACI 9.6.1.2}
\]

\[ A_{s\_min\_long} = 1.34 \text{ in}^2 \]

\[ A_{s\_min} = \max \left( A_{s\_min\_tran}, A_{s\_min\_long} \right) \]

\[ A_{s\_min} = 1.38 \text{ in}^2 \]

Minimum Shear Reinforcement

ACI Table 9.6.3.3

\[
A_{v\_min} \geq \max \left( 0.75 \cdot \sqrt{f'c \cdot \text{psi}} \cdot \frac{b_w}{f_y} \text{ in}, 50 \cdot \frac{b_w}{f_y} \text{ in} \cdot \text{psi} \right) = 0.01 \text{ in}^2
\]
### 5.3.3 ACI Strength Reduction Factors - ACI Chapter 21

#### Table 21.2.1—Strength reduction factors $\phi$

<table>
<thead>
<tr>
<th>Action or structural element</th>
<th>$\phi$</th>
<th>Exceptions</th>
</tr>
</thead>
<tbody>
<tr>
<td>(a) Moment, axial force, or combined moment and axial force</td>
<td>0.65 to 0.90 in accordance with 21.2.2</td>
<td>Near ends of pretensioned members where strands are not fully developed, $\phi$ shall be in accordance with 21.2.3.</td>
</tr>
<tr>
<td>(b) Shear</td>
<td>0.75</td>
<td>Additional requirements are given in 21.2.4 for structures designed to resist earthquake effects.</td>
</tr>
<tr>
<td>(c) Torsion</td>
<td>0.75</td>
<td>—</td>
</tr>
<tr>
<td>(d) Bearing</td>
<td>0.65</td>
<td>—</td>
</tr>
<tr>
<td>(e) Post-tensioned anchorage zones</td>
<td>0.85</td>
<td>—</td>
</tr>
<tr>
<td>(f) Brackets and corbels</td>
<td>0.75</td>
<td>—</td>
</tr>
<tr>
<td>(g) Struts, ties, nodal zones, and bearing areas designed in accordance with strut-and-tie method in Chapter 23</td>
<td>0.75</td>
<td>—</td>
</tr>
<tr>
<td>(h) Components of connections of precast members controlled by yielding of steel elements in tension</td>
<td>0.90</td>
<td>—</td>
</tr>
<tr>
<td>(i) Plain concrete elements</td>
<td>0.60</td>
<td>—</td>
</tr>
<tr>
<td>(j) Anchors in concrete elements</td>
<td>0.45 to 0.75 in accordance with Chapter 17</td>
<td>—</td>
</tr>
</tbody>
</table>

*Fig. R21.2.2a—Strain distribution and net tensile strain in a nonprestressed member.*

*Fig. R21.2.2b—Variation of $\phi$ with net tensile strain in extreme tension reinforcement, $\varepsilon_t$."

$\phi_b := 0.9$

For tension-controlled sections.

$\phi_v := 0.75$

$\phi_t := 0.75$
5.3.4 ACI Sectional Strength - ACI Chapter 22 - Flexure

\[
\begin{array}{|c|c|c|c|c|c|c|c|}
\hline
\text{Imperial Bar Size} & \text{"Soft" Metric Size} & \text{Weight per Unit Length (lb/ft)} & \text{Mass per Unit Length (kg/m)} & \text{Nominal Diameter (in)} & \text{Nominal Diameter (mm)} & \text{Nominal Area (in}^2\text{)} & \text{Nominal Area (cm}^2\text{)} \\
\hline
\#10 & \#8 & 0.376 & 0.791 & 0.375 & 9.523 & 0.11 & 7.1 \\
\#12 & \#10 & 0.668 & 0.999 & 0.506 & 12.7 & 0.2 & 129 \\
\#16 & \#12 & 1.043 & 1.556 & 0.626 & 15.875 & 0.31 & 206 \\
\#18 & \#14 & 1.502 & 2.241 & 0.756 & 19.05 & 0.44 & 284 \\
\#20 & \#16 & 2.041 & 3.049 & 0.979 & 22.226 & 0.6 & 367 \\
\#25 & \#20 & 2.67 & 3.982 & 1.066 & 25.4 & 0.79 & 509 \\
\#29 & \#24 & 3.4 & 5.071 & 1.128 & 28.59 & 1 & 643 \\
\#32 & \#28 & 4.203 & 6.418 & 1.27 & 32.26 & 1.27 & 819 \\
\#36 & \#32 & 5.313 & 7.924 & 1.41 & 35.61 & 1.56 & 1005 \\
\#40 & \#36 & 7.60 & 11.47 & 1.693 & 4.5 & 2.25 & 1432 \\
\#50 & \#50 & 13.6 & 26.29 & 2.257 & 57.34 & 4 & 2581 \\
\hline
\end{array}
\]

\[d_{\#8\text{-}bar} = 1\text{ in}\] Diameter of reinforcing bar is 0.75 in or #6 bar.

\[A_{\text{bar}} = 0.79\text{ in}^2\] Area of reinforcing steel bar.

\[n = 2\] Number of reinforcing bars per foot of width.

\[A_{s,\text{tran}} = \begin{cases} 
& n \cdot A_{\text{bar}} < A_{s,\text{min}} \\
& \frac{n \cdot A_{\text{bar}}}{A_{s,\text{min}}} \\
& \text{else}
\end{cases} = 1.58\text{ in}^2\]

\[n_{\text{min}} = \text{round}\left(\frac{A_{s,\text{tran}}}{A_{\text{bar}}}\right) = 2.00\]

\[A_s = n_{\text{min}} \cdot A_{\text{bar}} = 1.58\text{ in}^2\] Use 2-#6 bars per foot of width.

\[\beta_1 = \begin{cases} 
& \text{if } 2500\text{ psi} < f'c < 4000\text{ psi} \\
& 0.85 \\
& \text{else if } 4000\text{ psi} < f'c < 8000\text{ psi} \\
& 0.85 - 0.05 \cdot \frac{f'c - 4000\text{ psi}}{1000\text{ psi}} \\
& 0.65
\end{cases}\] ACI Table 22.2.2.4.3

Prepared by: Mark Salehi (Rev. 1), Alex Mora (Rev. 0) Date: 7/14/2020

Reviewed by: Soren Morch (Rev. 1), Mark Salehi (Rev. 0) Date: 7/14/2020
\( a := \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b_w} = 1.86 \text{ in} \)

\[
\phi b Mn_y := \phi b \cdot A_s \cdot f_y \cdot \left( d_{\text{tr}} - \frac{a}{2} \right) = 224.47 \text{ kip \cdot ft}
\]

**Ratio** := abs \( \frac{M_{u,y}}{\phi b Mn_y} \) = 0.70

Consider acceptable. Use same reinforcement for positive steel in the longitudinal direction.

\[
\phi b Mn_x := \phi b \cdot A_s \cdot f_y \cdot \left( d_{\text{long}} - \frac{a}{2} \right) = 217.36 \text{ kip \cdot ft}
\]

**Ratio** := abs \( \frac{M_{u,x}}{\phi b Mn_x} \) = 0.30

---

**5.3.5 ACI Sectional Strength - ACI Chapter 22 - Two Way Shear**

\( d := \text{mean}(d_{\text{tr}}, d_{\text{long}}) = 32.00 \text{ in} \)  

ACI 22.6.2.1

Critical Section

\( D_{\text{col}} := 24 \text{ in} \)

\( A_{\text{col}} := \frac{1}{4} \cdot n \cdot D_{\text{col}}^2 = 3.14 \text{ ft}^2 \)

\( l_{\text{col}} := \sqrt{A_{\text{col}}} = 1.77 \text{ ft} \)

Equivalent side of square column.

\( b_o := 4 \cdot \left( l_{\text{col}} + 2 \cdot \frac{d}{2} \right) = 17.76 \text{ ft} \)

Critical section for two-way members.

Two-way shear strength provided by concrete (ACI Table 22.6.5.2)

\( V_{c,1} := 4 \cdot \lambda \cdot \sqrt{f'_c \cdot \text{psi}} = 0.28 \text{ ksi} \)
\[ \beta := 1.0 \quad \text{Ratio of equivalent column sides.} \]

\[ V_{c,2} := \left( 2 + \frac{4}{\beta} \right) \cdot \lambda \cdot \sqrt{f'c} \cdot \text{psi} = 0.42 \text{ ksi} \]

\[ a_s := 30 \quad \text{Coefficient for edge columns} \]

\[ V_{c,3} := \left( 2 + \frac{a_s \cdot d}{b_o} \right) \cdot \lambda \cdot \sqrt{f'c} \cdot \text{psi} = 0.46 \text{ ksi} \]

\[ V_c := \min (V_{c,1}, V_{c,2}, V_{c,3}) = 0.28 \text{ ksi} \]

\[ \phi V \cdot V_c = 0.21 \text{ ksi} \]

\[ \phi V P_n := \phi V \cdot V_c \cdot b_o \cdot d = 1446.42 \text{ kip} \]

\[ P_u := 385 \text{ kip} \quad \text{Max. reaction force from STAAD.Pro} \]

\[ \text{Ratio} := \frac{P_u}{\phi V P_n} = 0.27 \]

**SECTION 6: RESULTS**

The platform modeled in STAAD.Pro requires the front piles to be 20" dia. x 0.5" w.t and piles in the cellular cofferdam are 24" dia. x 0.688" w.t.

The pipe pile wall thickness does not consider corrosion allowance.
SECTION 7: REFERENCES

PROJECT DOCUMENTS
1a  DMMF Basis of Design [Appendix A of Design Report].
1b  Foth Structural Design of Cellular Cofferdam, DMMF-SD-02, October 2020 [Appendix N of Design Report].

CODES AND STANDARDS
2a  Building Code Requirements for Structural Concrete, ACI 318-14.
2b  United Facilities Criteria (UFC), Design: Piers and Wharves, UFC 4-152-01, 24 January 2017.
2d  Specifications for Structural Steel Buildings, ANSI/AISC 360-16.
2e  Seismic Design of Piers and Wharves, ASCE 61-14.
# Structural Design and Analysis of Dolphin

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SECTION 1: PURPOSE AND SCOPE:

The Milwaukee Dredged Material Management Facility (DMMF) is planned to be constructed on the north side of the existing Jones Island DMDF to provide additional capacity for the disposal of dredged material. Milwaukee Harbor is located on the west shore of Lake Michigan. The new DMMF structure will be comprised of cellular coffer dam wall on two sides with a docking platform on a portion of north side. Additionally, there are dolphin structures to support the fenders and bollards and aid berthing as well as mooring of the design vessel. The scope of this calculation is to present the structural analysis of the dolphin.

SECTION 2: ASSUMPTIONS
Assumptions are discussed and justified as deemed necessary throughout this calculation.

SECTION 3: METHODOLOGY AND ACCEPTANCE CRITERIA

American Concrete Institute (ACI) guideline from Reference 2a along with applicable loads and load combinations defined by the UFC guideline (Reference 2b) are used to design the platform.

The dolphin is analyzed using the output of STAAD.Pro platform model and this MathCAD document. Forces will be extracted from the model to check the ACI code requirement and compute the required reinforcement. In the STAAD mode, plate elements are used to simulate the concrete deck and beam elements are used for modeling beam and pipes. Following items are specifically considered in the analysis:

1. Since the vertical loads are being transferred to the ground/seabed through pipe piles, a fixity point of 8D, where D is the diameter of the pipe, is estimated for the fixity point. This will indirectly model the soil-structure interaction. Piles that fall outside of the cells are designed as a length of 8D +27 ft + (distance from LWD to the bottom of portal).

2. For modeling of the dolphin a two-way slab with peripheral beam is used. During lateral loads, it acts as a diaphragm and transfer the load to the piles.

3. Clear cover to reinforcement shall be 3 inches, minimum (Reference 2c).
Milwaukee DMMF Project

SECTION 4: DESIGN INPUTS

4.1 Applicable Loads

Only applicable loads are presented. Any other loads shown on the above table that are not discussed are not applicable to this project.

1. Dead Load (D) - Self weight of construction materials and other structural components.
2. Live Load (Lu) - Uniform distributed live load.
3. Live Load (Lc) - Concentrated live loads.
4. Buoyancy Load (B) - Buoyancy load will be considered for the piles from fixity point to mean low water level.
5. Berthing (Be) - Berthing loads are equivalent to the rubber fender reaction times 1.1 tolerance.
6. Mooring/Breasting Load (M) - bollard loads are considered.
7. Earthquake Load (EQ) is calculated.
8. Earth Load (H) from back of the platform is considered.
9. Wave Load (W) considered on the structure.
10. Ice (Ice) load is applied to the structure.
4.2 Material

1. Concrete shall have compressive strength at 28 days of 5000 psi minimum.

2. Steel pipe pile shall be fabricated in compliance with ASTM A252, with a minimum yield strength of 50 ksi.

4.3 Loading

4.3.1 Dead Load

Dead load will be applied based on the weight of the structure in STAAD.Pro Software.

\[ \gamma_{\text{concrete}} = 150 \text{ pcf} \quad \text{Density of Concrete} \]

\[ \gamma_{\text{fill}} = 125 \text{ pcf} \quad \text{Density of saturated fill over the deck} \]

4.3.2 Live Load

A uniform live load is considered on the platform. Additionally, the load from a Manitowoc 2250 series 3 crawler crane is applicable to the deck.

\[ LL = 500 \text{ psf} \]

\[ T_{cw} = 47.3 \text{ in} \]

\[ L_{\text{tread}} = 328.9 \text{ in} \]
\[ P_{23} := 15.9 \text{ psi} \cdot T_{cw} \cdot L_{tread} + 0.5 \cdot (36.9 - 15.9) \text{ psi} \cdot T_{cw} \cdot L_{tread} = 410.70 \text{ kip} \]

\[ P_{side} := 28.6 \text{ psi} \cdot T_{cw} \cdot L_{tread} + 0.5 \cdot (30.6 - 28.6) \text{ psi} \cdot T_{cw} \cdot L_{tread} = 460.49 \text{ kip} \]

\[ P_{front} := 12.9 \text{ psi} \cdot T_{cw} \cdot L_{tread} + 0.5 \cdot (35.9 - 12.9) \text{ psi} \cdot T_{cw} \cdot L_{tread} = 379.59 \text{ kip} \]

Maximum pressure is for the boom over side position. This pressure will distributed over an area equivalent of a 30 degree spread through structural fill.

\[ p_{min} := 28.6 \text{ psi} \]

\[ p_{max} := 30.6 \text{ psi} \]

\[ h_{fill} := 2 \text{ ft} \]

\[ A_{spread} := 30 \text{ deg} \]

\[ T'_{cw} := T_{cw} + 2 \cdot \tan (A_{spread}) \cdot h_{fill} = 6.25 \text{ ft} \]

\[ L'_{tread} := L_{tread} + 2 \cdot \tan (A_{spread}) \cdot h_{fill} = 29.72 \text{ ft} \]

\[ p'_{min} := \frac{p_{min} \cdot T_{cw}}{T'_{cw}} = 18.03 \text{ psi} \]

\[ p'_{max} := \frac{p_{max} \cdot T_{cw}}{T'_{cw}} = 19.30 \text{ psi} \]

The crane tread pressure will be applied on the concrete deck to develop the worst effect. Spacing of treads is 22 ft c/c (See Manitowoc 2250 Product Guide).

\[ p_{ave} := 0.5 \cdot (p'_{min} + p'_{max}) = 2.69 \text{ ksf} \]

Apply pressure on an area 8' wide x 2' long.
### 4.3.3 Buoyancy

\[ Y_{\text{seawater}} = 64 \text{ pcf} \]

For 24" diameter pipe pile.

\[ D_{\text{pile}} = 24 \text{ in} \]

\[ V_{\text{pile}} = \frac{n}{4} \cdot D_{\text{pile}}^2 \]

\[ V_{\text{pile}} = 3.14 \text{ ft}^2 \]  
Volume of pile per foot basis.

\[ B_{24} = V_{\text{pile}} \cdot Y_{\text{seawater}} = 0.20 \frac{\text{kip}}{\text{ft}} \]  
Buoyancy on concrete filled piles.

For 32" diameter pipe pile.

\[ D_{\text{pile}} = 32 \text{ in} \]

\[ V_{\text{pile}} = \frac{n}{4} \cdot D_{\text{pile}}^2 \]

\[ V_{\text{pile}} = 5.59 \text{ ft}^2 \]  
Volume of pile per foot basis.

\[ B_{32} = V_{\text{pile}} \cdot Y_{\text{seawater}} = 0.36 \frac{\text{kip}}{\text{ft}} \]  
Buoyancy on concrete filled piles.

### 4.3.4 Berthing

\[ Be = 1.1 \cdot 1196.41 \text{ kN} \]  
A tolerance factor of 1.1 is applied to the fender reaction for a SC 1450 H1.

\[ Be = 295.86 \text{ kip} \]

\[ \mu = 0.2 \]  
Coefficient of static friction between steel and UHMWPE.

\[ Be_f = Be \cdot \mu = 59.17 \text{ kip} \]
Note: Berthing load will consider the following loads.

- The total fender reaction load applied perpendicular to the face of the fender panel.
- The lateral load from the friction of the total fender reaction applied parallel to the face of the fender panel.

Berthing loads will be applied at El. +4.0’.

Additional berthing load requested by Port are:

- 16 kip/ft over 40 feet impact length for Ore carrier
- 22 kip/ft over 30 feet impact length for other carrier - Control

These cases will be applied as independent load combinations.

### 4.3.5 Earthquake

\[ I := 1 \]

Importance factor for Risk Category II, Table 1.5-2 of ASCE 7

\[ R := 2 \]

Response modification coefficient

\[ C_d := 2 \]

Deflection amplification coefficient

\[ S_S := 0.076 \]

Ground motion (period = 0.2s)

\[ S_I := 0.048 \]

Ground motion (period = 0.1s)

\[ S_{MS} := 0.121 \]

Site-modified spectral acceleration value

\[ S_{M1} := 0.116 \]

Site-modified spectral acceleration value

\[ S_{DS} := 0.081 \]

Numeric seismic design value at 0.2s SA

\[ S_{DI} := 0.077 \]

Numeric seismic design value at 1.0s SA

\[ PGA := 0.037 \]

Peak ground acceleration
Calc. No.: DMMF-SD-04  
Project Number: 19W012  
Revision: 1

Milwaukee DMMF Project

\[
C_s := \frac{S_{DS}}{R/I} = 0.04 \quad \text{Seismic Response Coefficient}
\]

\[
T_L := 12 \, s \quad \text{Long period transition period TL}
\]

\[
C_t := 0.016 \quad \text{Concrete moment resisting frame, Table 12.8-2 of ASCE 7}
\]

\[
x := 0.9
\]

\[
h_{n24} := 45 \quad \text{Distance to fixity point of piles}
\]

\[
h_{n32} := 20 \quad \text{Distance to fixity point of piles}
\]

\[
h_n := \frac{(h_{n24} + h_{n32})}{2} = 32.50
\]

\[
T_a := C_t \cdot h_n^x = 0.37 \quad \text{Approximate fundamental period}
\]

\[
C_{s_{\text{max}}} := \frac{S_{DL}}{T_a \cdot \left(\frac{R}{I}\right)} = 0.10
\]

\[
C_{s_{\text{min}}} := 0.44 \cdot S_{DS} \cdot I = 0.04
\]

\[
W_{24_{\text{pipe}}} := 104.23 \frac{lbf}{ft}
\]

\[
W_{32_{\text{pipe}}} := 171.45 \frac{lbf}{ft}
\]

\[
W_{dolphin} := \left(2 \, ft \cdot \gamma_{\text{fill}} + 3 \, ft \cdot \gamma_{\text{concrete}}\right) \cdot 16 \, ft \cdot 13.5 \, ft \cdot 16 + \left(3 \cdot W_{24_{\text{pipe}}} \cdot h_{n24} + 3 \cdot W_{32_{\text{pipe}}} \cdot h_{n32}\right) \cdot 0.5 \cdot ft + 3 \, ft \cdot \gamma_{\text{concrete}} \cdot 5 \, ft \cdot 5 \, ft
\]

\[
W_{dolphin} = 174.63 \, kip
\]

\[
V_{\text{tran}} := \frac{C_s \cdot W_{dolphin}}{16 \, ft} = 0.44 \frac{kip}{ft}
\]

\[
V_{\text{long}} := \frac{C_s \cdot W_{dolphin}}{13.5 \, ft} = 0.52 \frac{kip}{ft}
\]
4.3.6 Mooring

Mooring load will be applied to generate the worst effect on the structure. 60 Ton bollards.

It will be applied at 60 degrees respect berth/fender line and at an angle of 60 degrees up or 30 degrees down.

4.3.7 Notional Loads

Notional loads shall be applied as lateral loads at the top level for this platform. The notional loads shall be additive to other lateral loads and shall be applied in all load combinations.

For LRFD equations (1.6 for ASD equations).

\[ Y_i = W_{dolphin} + LL \cdot 16 \text{ ft} \cdot 13.5 \text{ ft} \]

\[ Y_i = 282.63 \text{ kip} \]

\[ N_{i_{\text{tran}}} = \frac{0.003 \cdot Y_i \cdot \alpha}{14 \text{ ft}} = 0.06 \text{ kip/ft} \]

\[ N_{i_{\text{long}}} = \frac{0.003 \cdot Y_i \cdot \alpha}{10 \text{ ft}} = 0.08 \text{ kip/ft} \]

4.3.8 Earth Loads

Consider earth loads from 5 feet deep backfill subject to 500 psf UDL.

\[ K_a = 0.333 \text{ (Rankine)} \]

Depth of Fill = 5ft

Concrete thickness in the back: 2 ft
4.3.9 Wave

A horizontal wave load of 6.2 kip/ft will be applied at El. +2.0'.

\[ W = 6.2 \, \text{kip/ft} \]

4.3.10 Ice

A horizontal ice load of 38.0 kip/ft will be applied at El. -2.0' on piles.

A vertical ice load of 160 kip upward and 135 kip downward will be applied to piles.

\[ \text{Ice}_{h} = 38 \, \text{kip/ft} \quad \text{Horizontal ice load - general.} \]

\[ \text{Ice}_{v_{\text{up}}} = 160 \, \text{kip} \quad \text{Upward ice load on piles.} \]

\[ \text{Ice}_{v_{\text{dn}}} = 135 \, \text{kip} \quad \text{Downward ice load on piles.} \]
SECTION 5: CALCULATION

Following figure shows the geometry of the structural model as simulated in the STAAD.Pro software.

### 5.1 Plate Stresses and Reactions

<table>
<thead>
<tr>
<th>Plate L/C</th>
<th>Qx (psi)</th>
<th>Qy (psi)</th>
<th>Sx (psi)</th>
<th>Sy (psi)</th>
<th>Sxy (psi)</th>
<th>Mx (Kip ft)</th>
<th>My (Kip ft)</th>
<th>Mxy (Kip ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max Qx</td>
<td>26 U2B 1.2D+1</td>
<td>427.6</td>
<td>438.3</td>
<td>-27.5</td>
<td>-126.2</td>
<td>-108.7</td>
<td>-71.3</td>
<td>-401.0</td>
</tr>
<tr>
<td>Min Qx</td>
<td>27 U2A 1.2D+1</td>
<td>-524.8</td>
<td>535.5</td>
<td>-59.5</td>
<td>-139.0</td>
<td>146.9</td>
<td>-149.1</td>
<td>-433.4</td>
</tr>
<tr>
<td>Max Qy</td>
<td>27 U2A 1.2D+1</td>
<td>-524.8</td>
<td>535.5</td>
<td>-59.5</td>
<td>-139.0</td>
<td>146.9</td>
<td>-149.1</td>
<td>-433.4</td>
</tr>
<tr>
<td>Min Qy</td>
<td>22 U5A 1.2D+1</td>
<td>35.9</td>
<td>-479.4</td>
<td>17.6</td>
<td>-30.4</td>
<td>48.6</td>
<td>158.2</td>
<td>178.2</td>
</tr>
<tr>
<td>Max Sx</td>
<td>23 U2A 1.2D+1</td>
<td>303.8</td>
<td>300.2</td>
<td>235.1</td>
<td>-295.1</td>
<td>-50.3</td>
<td>5.8</td>
<td>-307.8</td>
</tr>
<tr>
<td>Min Sx</td>
<td>22 U2A 1.2D+1</td>
<td>-416.5</td>
<td>431.0</td>
<td>-181.3</td>
<td>70.2</td>
<td>129.9</td>
<td>-129.3</td>
<td>-362.4</td>
</tr>
<tr>
<td>Max Sy</td>
<td>21 U2A 1.2D+1</td>
<td>-164.6</td>
<td>153.2</td>
<td>-174.5</td>
<td>216.1</td>
<td>-71.7</td>
<td>36.5</td>
<td>169.6</td>
</tr>
<tr>
<td>Min Sy</td>
<td>56 U2A 1.2D+1</td>
<td>-6.9</td>
<td>-4.7</td>
<td>-24.2</td>
<td>-318.2</td>
<td>25.8</td>
<td>16.4</td>
<td>1.0</td>
</tr>
<tr>
<td>Max Sxy</td>
<td>27 U2A 1.2D+1</td>
<td>-524.8</td>
<td>535.5</td>
<td>-59.5</td>
<td>-139.0</td>
<td>146.9</td>
<td>-149.1</td>
<td>-433.4</td>
</tr>
<tr>
<td>Min Sxy</td>
<td>82 U5D 1.2D+1</td>
<td>-33.2</td>
<td>33.2</td>
<td>143.7</td>
<td>19.2</td>
<td>-143.7</td>
<td>11.9</td>
<td>0.2</td>
</tr>
<tr>
<td>Max Mx</td>
<td>79 U5B 1.2D+1</td>
<td>-133.3</td>
<td>0.2</td>
<td>-3.1</td>
<td>-5.7</td>
<td>0.0</td>
<td>240.0</td>
<td>-23.4</td>
</tr>
<tr>
<td>Min Mx</td>
<td>27 U2A 1.2D+1</td>
<td>-254.8</td>
<td>535.5</td>
<td>-59.5</td>
<td>-139.0</td>
<td>146.9</td>
<td>-149.1</td>
<td>-433.4</td>
</tr>
<tr>
<td>Max My</td>
<td>22 U5B 1.2D+1</td>
<td>173.2</td>
<td>-284.4</td>
<td>0.8</td>
<td>-78.9</td>
<td>-36.0</td>
<td>55.1</td>
<td>284.9</td>
</tr>
<tr>
<td>Min My</td>
<td>27 U2A 1.2D+1</td>
<td>-524.8</td>
<td>535.5</td>
<td>-59.5</td>
<td>-139.0</td>
<td>146.9</td>
<td>-149.1</td>
<td>-433.4</td>
</tr>
<tr>
<td>Max Mxy</td>
<td>27 U2A 1.2D+1</td>
<td>-524.8</td>
<td>535.5</td>
<td>-59.5</td>
<td>-139.0</td>
<td>146.9</td>
<td>-149.1</td>
<td>-433.4</td>
</tr>
<tr>
<td>Min Mxy</td>
<td>23 U2B 1.2D+1</td>
<td>427.1</td>
<td>433.8</td>
<td>-0.7</td>
<td>-132.0</td>
<td>-111.5</td>
<td>-77.9</td>
<td>-398.7</td>
</tr>
</tbody>
</table>
Description of output:

1. Qx and Qy are shear stresses (Force/unit length/thickness). These stresses may be compared to allowable shear stress. These shear stresses are applied to the plate side perpendicular to the respective axis.

2. Sx and Sy are membrane stresses (Force/unit length/thickness). These stresses may be compared to allowable compressive or tensile stress.

3. Sxy is in-plane shear stress (Force/unit length/thickness).

4. Mx, My, Mxy are moments per unit width (Force x Length/length). These moments act on the plate side perpendicular to the respective axis.

For purposes of this analysis/design, the main direction is the longitudinal direction. The secondary direction is the transverse direction.

5.2 Section Properties

\[ f'c := 5000 \text{ psi} \]

Concrete compression at 28 days

\[ fy := 60000 \text{ psi} \]

Yield of reinforcing steel

\[ b_w := 1 \text{ ft} \]
\[ CL := 3 \text{ in} \quad \text{Rebar cover} \]
\[ h := 60 \text{ in} \quad \text{Thickness of the slab} \]
\[ d_{#4\text{_bar}} := 0.75 \text{ in} \]
\[ d_{#8\text{_bar}} := 1 \text{ in} \]
\[ d_{\text{trans}} := h - CL - 0.5 \cdot d_{#8\text{_bar}} = 56.50 \text{ in} \]
\[ d_{\text{long}} := h - CL - 1.5 \cdot d_{#8\text{_bar}} = 55.50 \text{ in} \]

### 5.3 Concrete Deck Design

#### 5.3.1 Design Parameters

\[ \lambda := 1.0 \quad \text{Modification factor, ACI Table 19.2.4.2} \]
\[ M_{u\_x} := 149 \text{ kip} \cdot \text{ft} \quad \text{Ultimate moment per foot of length, longitudinal direction (local X)} \]
\[ M_{u\_y} := 433 \text{ kip} \cdot \text{ft} \quad \text{Ultimate moment per foot of length, transverse direction (local Y)} \]
\[ V_{u\_x} := 0.524 \text{ ksi} = 75.46 \text{ ksf} \quad \text{Ultimate shear per square foot, longitudinal direction} \]
\[ V_{u\_y} := 0.535 \text{ ksi} = 77.04 \text{ ksf} \quad \text{Ultimate shear per square foot, transverse direction} \]
\[ T_u := 413 \text{ kip} \cdot \text{ft} \quad \text{Ultimate torsion per foot of length} \]
\[ E := 57000 \cdot \sqrt{f'c \cdot \text{psi}} = 4030.51 \text{ ksi} \quad \text{ACI 19.2.2.1.b} \]
5.3.2 ACI Beam Requirements - ACI Chapter 9

Minimum Flexural Reinforcement - Transverse Direction

\[ b_w = 1.00 \text{ ft} \quad \text{Assumes one layer of \#8 flexure bars.} \]
\[ d_{\text{tran}} = 56.50 \text{ in} \]
\[ A_{s_{\text{min, tran}}} := \max \left( 3 \cdot \sqrt{\frac{f'c \cdot psi}{fy}} \cdot b_w \cdot d_{\text{tran}}, \frac{200 \ psi}{fy} \cdot b_w \cdot d_{\text{tran}} \right) \]
\[ A_{s_{\text{min, tran}}} = 2.40 \text{ in}^2 \quad \text{ACI 9.6.1.2} \]

Minimum Flexural Reinforcement - Longitudinal Direction

\[ b_w = 1.00 \text{ ft} \quad \text{Assumes one layer of \#8 flexure bars.} \]
\[ d_{\text{long}} = 55.50 \text{ in} \]
\[ A_{s_{\text{min, long}}} := \max \left( 3 \cdot \sqrt{\frac{f'c \cdot psi}{fy}} \cdot b_w \cdot d_{\text{long}}, \frac{200 \ psi}{fy} \cdot b_w \cdot d_{\text{long}} \right) \quad \text{ACI 9.6.1.2} \]
\[ A_{s_{\text{min, long}}} = 2.35 \text{ in}^2 \]
\[ A_{s_{\text{min}}} := \max \left( A_{s_{\text{min, tran}}}, A_{s_{\text{min, long}}} \right) \]
\[ A_{s_{\text{min}}} = 2.40 \text{ in}^2 \]

Minimum Shear Reinforcement \quad \text{ACI Table 9.6.3.3}

\[ A_{v_{\text{min}}} := \max \left( 0.75 \cdot \sqrt{\frac{f'c \cdot psi}{fy}} \cdot b_w \frac{\text{in}}{\text{in}}, 50 \cdot \frac{b_w}{fy} \cdot \text{in} \cdot \text{psi} \right) = 0.01 \text{ in}^2 \]
5.3.3 ACI Strength Reduction Factors - ACI Chapter 21

Table 21.2.1—Strength reduction factors $\phi$

<table>
<thead>
<tr>
<th>Action or structural element</th>
<th>$\phi$</th>
<th>Exceptions</th>
</tr>
</thead>
<tbody>
<tr>
<td>(a) Moment, axial force, or combined moment and axial force</td>
<td>$0.65$ to $0.90$</td>
<td>Near ends of pretensioned members where strands are not fully developed, $\phi$ shall be in accordance with 21.2.2.</td>
</tr>
<tr>
<td>(b) Shear</td>
<td>$0.75$</td>
<td>Additional requirements are given in 21.2.4 for structures designed to resist earthquake effects.</td>
</tr>
<tr>
<td>(c) Torsion</td>
<td>$0.75$</td>
<td>—</td>
</tr>
<tr>
<td>(d) Bearing</td>
<td>$0.65$</td>
<td>—</td>
</tr>
<tr>
<td>(e) Post-tensioned anchorage zones</td>
<td>$0.85$</td>
<td>—</td>
</tr>
<tr>
<td>(f) Brackets and corbels</td>
<td>$0.75$</td>
<td>—</td>
</tr>
<tr>
<td>(g) Struts, ties, nodal zones, and bearing areas designed in accordance with strut-and-tie method in Chapter 23</td>
<td>$0.75$</td>
<td>—</td>
</tr>
<tr>
<td>(h) Components of connections of precast members controlled by yielding of steel elements in tension</td>
<td>$0.90$</td>
<td>—</td>
</tr>
<tr>
<td>(i) Plain concrete elements</td>
<td>$0.60$</td>
<td>—</td>
</tr>
<tr>
<td>(j) Anchors in concrete elements</td>
<td>$0.45$ to $0.75$</td>
<td>In accordance with Chapter 17</td>
</tr>
</tbody>
</table>

$\phi_b := 0.9$

For tension-controlled sections.

$\phi_v := 0.75$

$\phi_t := 0.75$

Fig. R21.2.2a—Strain distribution and net tensile strain in a nonprestressed member.

Fig. R21.2.2b—Variation of $\phi$ with net tensile strain in extreme tension reinforcement, $\varepsilon_t$. 

Prepared by: Mark Salehi (Rev. 1), Alex Mora (Rev. 0)                  Date: 11/10/2020
Reviewed by: Soren Morch (Rev. 1), Mark Salehi (Rev. 0)                 Date: 11/10/2020
5.3.4 ACI Sectional Strength - ACI Chapter 22 - Flexure

<table>
<thead>
<tr>
<th>Imperial Bar Diameter</th>
<th>&quot;Soft&quot; Metric Bar Diameter</th>
<th>Weight per unit length (lb/ft)</th>
<th>Mass per unit length (lb/ft²)</th>
<th>Nominal Diameter (in)</th>
<th>Nominal Diameter (mm)</th>
<th>Nominal Area (in²)</th>
<th>Nominal Area (cm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>#2</td>
<td>#16</td>
<td>0.376</td>
<td>0.900</td>
<td>9.752</td>
<td>9.752</td>
<td>0.11</td>
<td>0.11</td>
</tr>
<tr>
<td>#4</td>
<td>#13</td>
<td>0.868</td>
<td>0.999</td>
<td>12.7</td>
<td>12.7</td>
<td>0.2</td>
<td>0.2</td>
</tr>
<tr>
<td>#5</td>
<td>#16</td>
<td>1.043</td>
<td>1.556</td>
<td>15.475</td>
<td>15.475</td>
<td>0.31</td>
<td>0.31</td>
</tr>
<tr>
<td>#6</td>
<td>#19</td>
<td>1.502</td>
<td>1.24</td>
<td>12.7</td>
<td>12.7</td>
<td>0.44</td>
<td>0.44</td>
</tr>
<tr>
<td>#7</td>
<td>#22</td>
<td>2.044</td>
<td>3.149</td>
<td>18.03</td>
<td>18.03</td>
<td>0.6</td>
<td>0.6</td>
</tr>
<tr>
<td>#8</td>
<td>#25</td>
<td>2.67</td>
<td>3.982</td>
<td>22.226</td>
<td>22.226</td>
<td>0.79</td>
<td>0.79</td>
</tr>
<tr>
<td>#9</td>
<td>#29</td>
<td>3.4</td>
<td>5.071</td>
<td>28.03</td>
<td>28.03</td>
<td>1.1</td>
<td>1.1</td>
</tr>
<tr>
<td>#10</td>
<td>#32</td>
<td>4.303</td>
<td>6.418</td>
<td>32.26</td>
<td>32.26</td>
<td>1.27</td>
<td>1.27</td>
</tr>
<tr>
<td>#11</td>
<td>#36</td>
<td>5.313</td>
<td>7.924</td>
<td>35.81</td>
<td>35.81</td>
<td>1.6</td>
<td>1.6</td>
</tr>
<tr>
<td>#16</td>
<td>#43</td>
<td>7.65</td>
<td>11.41</td>
<td>43.05</td>
<td>43.05</td>
<td>2.25</td>
<td>2.25</td>
</tr>
<tr>
<td>#19</td>
<td>#57</td>
<td>13.6</td>
<td>26.284</td>
<td>57.33</td>
<td>57.33</td>
<td>4.0</td>
<td>4.0</td>
</tr>
</tbody>
</table>

$\text{d_{#8\text{-}bar}} := 1 \text{ in}$ Diameter of reinforcing bar is 0.75 in or #6 bar.

$A_{\text{bar}} := 0.79 \text{ in}^2$ Area of reinforcing steel bar.

$n := 2$ Number of reinforcing bars per foot of width.

$A_{s_{\text{tran}}} := \begin{cases} 
\frac{n \cdot A_{\text{bar}}}{A_{s_{\text{min}}}} & \text{if } n \cdot A_{\text{bar}} > A_{s_{\text{min}}} \\
A_{s_{\text{min}}} & \text{else} 
\end{cases} = 2.40 \text{ in}^2$

$n_{\text{min}} := \text{round} \left( \frac{A_{s_{\text{tran}}}}{A_{\text{bar}}} \right) = 3.00$

$A_{s} := n_{\text{min}} \cdot A_{\text{bar}} = 2.37 \text{ in}^2$ Use 2-#6 bars per foot of width.

$\beta_{1} := \begin{cases} 
0.80 & \text{if } 2500 \text{ psi} \leq f'c \leq 4000 \text{ psi} \\
0.85 & \text{else if } 4000 \text{ psi} < f'c < 8000 \text{ psi} \\
0.85 - 0.05 \cdot \frac{(f'c - 4000 \text{ psi})}{1000 \text{ psi}} & \text{ACI Table 22.2.2.4.3} \\
0.65 & \text{else if } f'c \geq 8000 \text{ psi} 
\end{cases}$
\[ a := \frac{A_s \cdot f_y}{0.85 \cdot f'c \cdot b_w} = 2.79 \text{ in} \]

\[ \phi bMn_y := \phi b \cdot A_s \cdot f_y \cdot \left( d_{\text{tran}} - \frac{a}{2} \right) = 587.70 \text{ kip \cdot ft} \]

\[ \text{Ratio} := \text{abs} \left( \frac{M_{u,y}}{\phi bMn_y} \right) = 0.74 \]

Consider acceptable. Use same reinforcement for positive steel in the longitudinal direction.

\[ \phi bMn_x := \phi b \cdot A_s \cdot f_y \cdot \left( d_{\text{long}} - \frac{a}{2} \right) = 577.04 \text{ kip \cdot ft} \]

\[ \text{Ratio} := \text{abs} \left( \frac{M_{u,x}}{\phi bMn_x} \right) = 0.26 \]

5.3.5 ACI Sectional Strength - ACI Chapter 22 - Two Way Shear

\[ d := \text{mean} \left( d_{\text{tran}}, d_{\text{long}} \right) = 56.00 \text{ in} \]

ACI 22.6.2.1

Critical Section

\[ D_{\text{col}} := 24 \text{ in} \]

\[ A_{\text{col}} := \frac{1}{4} \cdot n \cdot D_{\text{col}}^2 = 3.14 \text{ ft}^2 \]

\[ l_{\text{col}} := \sqrt{A_{\text{col}}} = 1.77 \text{ ft} \]

Equivalent side of square column.

\[ b_o := 4 \cdot \left( l_{\text{col}} + 2 \cdot \frac{d}{2} \right) = 25.76 \text{ ft} \]

Critical section for two-way members.

Two-way shear strength provided by concrete (ACI Table 22.6.5.2)

\[ V_{c,1} := 4 \cdot \lambda \cdot \sqrt{f'c} \cdot \text{psi} = 0.28 \text{ ksi} \]
\[ \beta := 1.0 \quad \text{Ratio of equivalent column sides.} \]

\[ V_{c,2} := \left( 2 + \frac{4}{\beta} \right) \cdot \lambda \cdot \sqrt{f'c} \cdot \text{psi} = 0.42 \text{ ksi} \]

\[ a_s := 30 \quad \text{Coefficient for edge columns} \]

\[ V_{c,3} := \left( 2 + \frac{a_s \cdot d}{b_o} \right) \cdot \lambda \cdot \sqrt{f'c} \cdot \text{psi} = 0.53 \text{ ksi} \]

\[ V_c := \min (V_{c,1}, V_{c,2}, V_{c,3}) = 0.28 \text{ ksi} \]

\[ \phi \cdot V_c = 0.21 \text{ ksi} \]

\[ \phi \cdot V_{nP} := \phi \cdot V_c \cdot b_o \cdot d = 3671.66 \text{ kip} \]

\[ Pu := 296.6 \text{ kip} \quad \text{Max. reaction force from STAAD.Pro} \]

\[ \text{Ratio} := \frac{Pu}{\phi \cdot V_{nP}} = 0.08 \]

**SECTION 6: RESULTS**

The platform modeled in STAAD.Pro requires the front piles to be 24" dia. x 0.688" w.t and piles in the cellular cofferdam are 32" dia. x 0.688" w.t.

The pipe pile wall thickness does not consider corrosion allowance.
SECTION 7: REFERENCES

PROJECT DOCUMENTS
1a DMMF Basis of Design [Appendix A of Design Report].
1b Foth Structural Design of Cellular Cofferdam, DMMF-SD-02, October 2020 [Appendix N of Design Report].

CODES AND STANDARDS
2a Building Code Requirements for Structural Concrete, ACI 318-14.
2b United Facilities Criteria (UFC), Design: Piers and Wharves, UFC 4-152-01, 24 January 2017.
2d Specifications for Structural Steel Buildings, ANSI/AISC 360-16.
2e Seismic Design of Piers and Wharves, ASCE 61-14.
Structural Design of Pipe Piles

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<td>10</td>
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<td>SECTION 7</td>
<td>REFERENCES</td>
<td>10</td>
</tr>
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</table>
SECTION 1: PURPOSE AND SCOPE:

The Milwaukee Dredged Material Management Facility (DMMF) is planned to be constructed on the north side of the existing Jones Island DMDF to provide additional capacity for the disposal of dredged material. Milwaukee Harbor is located on the west shore of Lake Michigan. The new DMMF structure will be comprised of cellular coffer dam wall on two sides with a docking platform on a portion of north side. Additionally, there are dolphin structures to support the fenders and bollards and aid berthing as well as mooring of the design vessel. The scope of this calculation is to present the connection between the pipe piles and the deck as well as checking the structural capacity of the piles. The purpose of this analysis is also to determine the reinforced concrete connection for steel pipe piles and concrete deck of the relieving platform. Additionally, the adequacy of the diameter and thickness of the steel pipe piles will be checked.

SECTION 2: ASSUMPTIONS

Assumptions are discussed and justified as deemed necessary throughout this calculation.

SECTION 3: METHODOLOGY AND ACCEPTANCE CRITERIA

American Concrete Institute (ACI) guideline from Reference 2a along with applicable loads and load combinations defined by the UFC guideline (Reference 2b) are used to design the platform.

SECTION 4: DESIGN INPUTS

4.1 Applicable Loads

Only applicable loads are presented. Any other loads shown on the above table that is not discussed is not applicable to this project.

1. Dead Load (D) - Self weight of construction materials and other structural components.
2. Live Load (Lu) - Uniform distributed live load.
3. Live Load (Lc) - Concentrated live loads.
4. Buoyancy Load (B) - Buoyancy load will be considered for the piles from fixity point to mean low water level.
5. Berthing (Be) - Berthing loads are equivalent to the rubber fender reaction times 1.1 tolerance.
6. Mooring/Breasting Load (M) - bollard loads are considered.
7. Earthquake Load (EQ) is calculated.
8. Earth Load (H) from back of the platform is considered.
9. Wave Load (W) considered on the structure.
10. Ice (Ice) load is applied to the structure.
4.2 Material

1. Concrete shall have compressive strength at 28 days of 5000 psi minimum.

2. Steel pipe pile shall be fabricated in compliance with ASTM A252, with a minimum yield strength of 60ksi (modified).

4.3 Loading

See References 1a and 1b for loads applicable to this project.
SECTION 5: CALCULATION

Following figure shows the geometry of the structural model as simulated in the STAAD.Pro software for both platform and dolphin.
5.1 Platform

5.1.1 20" Diameter Pipe Pile Demand

Following table presents results for front piles from the STAAD model for LRFD analysis using the loads documented in Reference 1a. The selection of the reinforcement for the concrete connection based on CRSI Design Handbook 2008 tables (Reference 2d).

The column diameter for this particular pipe pile is 20" - 2*0.5" = 19". The nearest column size has a diameter of 18 inches.

The preferred reinforcement is #8 headed bars, 8 bars minimum.

Beam End Force Summary

The signs of the forces at end B of each beam have been reversed. For example: this means that the Min Fx entry gives the largest tension value for an beam.

<table>
<thead>
<tr>
<th>Beam</th>
<th>Node</th>
<th>L/C</th>
<th>Axial</th>
<th>Shear</th>
<th>Torsion</th>
<th>Bending</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Fx</td>
<td>Fy</td>
<td>Fz</td>
<td>Mx</td>
</tr>
<tr>
<td>Max Fx</td>
<td>1634</td>
<td>1588</td>
<td>396.5</td>
<td>0.1</td>
<td>-1.6</td>
<td>-0.1</td>
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<tr>
<td>Min Fx</td>
<td>1634</td>
<td>1588</td>
<td>57.6</td>
<td>0.2</td>
<td>-0.9</td>
<td>-0.0</td>
</tr>
<tr>
<td>Max Fy</td>
<td>1636</td>
<td>1706</td>
<td>211.6</td>
<td>0.7</td>
<td>0.4</td>
<td>-1.3</td>
</tr>
<tr>
<td>Min Fy</td>
<td>1636</td>
<td>1706</td>
<td>96.1</td>
<td>-0.7</td>
<td>-4.8</td>
<td>6.1</td>
</tr>
<tr>
<td>Max Fy</td>
<td>1628</td>
<td>1239</td>
<td>224.3</td>
<td>-0.1</td>
<td>1.7</td>
<td>-1.1</td>
</tr>
<tr>
<td>Min Fy</td>
<td>1628</td>
<td>1239</td>
<td>96.1</td>
<td>-0.7</td>
<td>-4.8</td>
<td>6.1</td>
</tr>
<tr>
<td>Max Mx</td>
<td>1631</td>
<td>1408</td>
<td>161.8</td>
<td>-0.6</td>
<td>-2.8</td>
<td>6.5</td>
</tr>
<tr>
<td>Min Mx</td>
<td>1631</td>
<td>1408</td>
<td>290.8</td>
<td>0.4</td>
<td>1.1</td>
<td>-1.3</td>
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<td>Max Mx</td>
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<td>1239</td>
<td>96.1</td>
<td>-0.7</td>
<td>-4.8</td>
<td>6.1</td>
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<td>Min Mx</td>
<td>1628</td>
<td>1239</td>
<td>94.1</td>
<td>-0.7</td>
<td>-4.8</td>
<td>6.1</td>
</tr>
<tr>
<td>Min Mx</td>
<td>1628</td>
<td>1239</td>
<td>209.6</td>
<td>0.7</td>
<td>0.4</td>
<td>-1.3</td>
</tr>
</tbody>
</table>

Demand Capacity

Pu = 396.5 kip  Phi Pn = 431 kip
Mu = 530 kip-in Phi Mu = 1949 kip-in
Pu = 96 kip  Phi Pn = 273 kip
Mu = 1308 kip-in Phi Mu = 2077 kip-in

Prepared by: Mark Salehi (Rev. 1), Alex Mora, Connection (Rev. 0) Date: 11/10/2020
Reviewed by: Soren Morch (Rev. 1), Mark Salehi (Rev. 0) Date: 11/10/2020
## Milwaukee DMMF Project

### ROUNDED TIED COLUMNS 18" DIA.

Short columns – no sideway

\[ f'_t = 5,000 \text{ psi} \quad f'_s = 60,000 \text{ psi} \]

<table>
<thead>
<tr>
<th>BARS</th>
<th>RHO</th>
<th>Max Cap</th>
<th>0% ( f'_t )</th>
<th>25% ( f'_t )</th>
<th>50% ( f'_t )</th>
<th>100% ( f'_t )</th>
<th>( f'_s )</th>
<th>( f'_s )</th>
<th>( f'_s )</th>
<th>( f'_s )</th>
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<tbody>
<tr>
<td>4-4</td>
<td>1.24</td>
<td>954.0</td>
<td>654.0 1236.0</td>
<td>583.0 1515.0</td>
<td>479.0 1632.0</td>
<td>395.0 1672.0</td>
<td>272.0 1783.0</td>
<td>126.0 1193.0</td>
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<td></td>
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<tr>
<td>4-5</td>
<td>1.57</td>
<td>959.0</td>
<td>678.0 1296.0</td>
<td>597.0 1580.0</td>
<td>489.0 1707.0</td>
<td>401.0 1771.0</td>
<td>271.0 1839.0</td>
<td>103.0 1467.0</td>
<td></td>
<td></td>
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<tr>
<td>4-10</td>
<td>2.45</td>
<td>1033.0</td>
<td>743.0 1447.0</td>
<td>629.0 1743.0</td>
<td>510.0 1890.0</td>
<td>411.0 2005.0</td>
<td>261.0 2133.0</td>
<td>31.0  2062.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4-14</td>
<td>2.54</td>
<td>1167.0</td>
<td>883.0 1730.0</td>
<td>660.0 1818.0</td>
<td>542.0 2114.0</td>
<td>429.0 2250.0</td>
<td>253.0 2435.0</td>
<td>36.0  2724.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4-16</td>
<td>2.69</td>
<td>1196.0</td>
<td>916.0 1766.0</td>
<td>674.0 1830.0</td>
<td>554.0 2142.0</td>
<td>423.0 2270.0</td>
<td>254.0 2435.0</td>
<td>36.0  2724.0</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Zero Axial Load

\[ \phi_M \]

<table>
<thead>
<tr>
<th>BARS</th>
<th>3%</th>
<th>5%</th>
<th>7%</th>
<th>9%</th>
<th>11%</th>
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</thead>
<tbody>
<tr>
<td>4-4</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>4-5</td>
<td></td>
<td></td>
<td></td>
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</tr>
<tr>
<td>4-10</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4-14</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4-16</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

---

(1) See "Control Points for Interaction Curves – Round Columns", Fig. 4-1.

(2) "0% \( f'_t \)" indicates zero tension in bars on the tension side, "50% \( f'_s \)" indicates 50% \( f'_s \) stress in bars on the tension side, and "100% \( f'_t \)" indicates 100% \( f'_s \) stress (i.e., balance point) in bars on the tension side.

Prepared by: Mark Salehi (Rev. 1), Alex Mora, Connection (Rev. 0)  Date: 11/10/2020

Reviewed by: Soren Morch (Rev. 1), Mark Salehi (Rev. 0)  Date: 11/10/2020
The reinforcement is sufficient for the connection therefore no other element is required.

5.1.2 20" Diameter Pipe Pile Structural Capacity

Following table presents results for front piles from the STAAD model for ASD analysis using the loads documented in Reference 1a. Structural capacity of the pipe and stress ratio is calculated using AISC-360 (Reference 2e).

Beam End Force Summary

The signs of the forces at end B of each beam have been reversed. For example: this means that the Min Fx entry gives the largest tension value for an beam.

<table>
<thead>
<tr>
<th>Beam</th>
<th>Node</th>
<th>L/C</th>
<th>Axial</th>
<th>Shear</th>
<th>Torsion</th>
<th>Bending</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Fx (kip)</td>
<td>Fy (kip)</td>
<td>Fz (kip)</td>
<td>Mx (kip-ft)</td>
</tr>
<tr>
<td>Max Fx</td>
<td>1633</td>
<td>1528</td>
<td>89:S8D D+LCF</td>
<td>295.2</td>
<td>-0.0</td>
<td>-1.2</td>
</tr>
<tr>
<td>Min Fx</td>
<td>1636</td>
<td>1717</td>
<td>90:S9A D+0.7E</td>
<td>60.1</td>
<td>0.2</td>
<td>-0.7</td>
</tr>
<tr>
<td>Max Fy</td>
<td>1636</td>
<td>1708</td>
<td>76:S5I D+LCR</td>
<td>182.5</td>
<td>0.5</td>
<td>0.2</td>
</tr>
<tr>
<td>Min Fy</td>
<td>1628</td>
<td>1239</td>
<td>62:S2B D+B+E</td>
<td>90.2</td>
<td>-0.4</td>
<td>-3.0</td>
</tr>
<tr>
<td>Max Fz</td>
<td>1628</td>
<td>1239</td>
<td>69:S5B D+LU+</td>
<td>170.8</td>
<td>-0.1</td>
<td>-1.0</td>
</tr>
<tr>
<td>Min Fz</td>
<td>1628</td>
<td>1239</td>
<td>62:S2B D+B+E</td>
<td>90.2</td>
<td>-0.4</td>
<td>-3.0</td>
</tr>
<tr>
<td>Max Mx</td>
<td>1631</td>
<td>1408</td>
<td>62:S2B D+B+E</td>
<td>140.9</td>
<td>-0.4</td>
<td>-1.9</td>
</tr>
<tr>
<td>Min Mx</td>
<td>1631</td>
<td>1408</td>
<td>76:S5I D+LCR</td>
<td>221.9</td>
<td>0.3</td>
<td>0.6</td>
</tr>
<tr>
<td>Max My</td>
<td>1628</td>
<td>1239</td>
<td>62:S2B D+B+E</td>
<td>90.2</td>
<td>-0.4</td>
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<td>Min My</td>
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<td>62:S2B D+B+E</td>
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<td>-0.4</td>
<td>-3.0</td>
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<tr>
<td>Max Mz</td>
<td>1636</td>
<td>1708</td>
<td>76:S5I D+LCR</td>
<td>162.5</td>
<td>0.5</td>
<td>0.2</td>
</tr>
<tr>
<td>Min Mz</td>
<td>1636</td>
<td>1717</td>
<td>76:S5I D+LCR</td>
<td>160.9</td>
<td>0.5</td>
<td>0.2</td>
</tr>
</tbody>
</table>

\[ P_{uc} := 295 \cdot \text{kip} \quad \text{compression} \]

\[ P_{ut} := 0 \cdot \text{kip} \quad \text{tension} \]

\[ M_{u} := 70.6 \cdot \text{ft} \cdot \text{kip} \quad \text{flexure} \]

Note that above values are conservative as they do not occur at the same time
1. Pipe dimensional and structural properties.

\[ OD := 20 \text{ in} \]

\[ D := OD = 20.00 \text{ in} \]

\[ t_{\text{reduction}} := 0 \text{ in} \]

\[ t := 0.5 \text{ in} - t_{\text{reduction}} = 0.50 \text{ in} \]

\[ ID := OD - 2 \cdot t = 19.00 \text{ in} \]

\[ A := 0.25 \cdot (OD^2 - ID^2) \cdot n = 30.63 \text{ in}^2 \]

\[ I := \frac{n}{64} \cdot (OD^4 - ID^4) = 1456.86 \text{ in}^4 \]

\[ S := \frac{n}{32 \cdot OD} \cdot (OD^4 - ID^4) = 145.69 \text{ in}^3 \]

\[ r := \sqrt{\frac{I}{A}} = 6.90 \text{ in} \]

\[ Z := \frac{OD^3}{6} - \frac{ID^3}{6} = 190.17 \text{ in}^3 \]

\[ E := 29000 \cdot \text{ksi} \]

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

1. Pipe flexural and compaction capacity is checked using AISC Steel Construction Manual 13th Edition

Assume fixed at base, rotation fixed and translation free at top (Table C-C2.2)

\[ \phi := 0.9 \quad L := 45 \cdot \text{ft} \quad K := 1.2 \]
Check Compactness: Table B4.1 Case 15

In uniform compression: \( \text{Compression} := \text{if} \left( \frac{D}{t} \leq 0.11 \cdot \frac{E}{F_y} \right) \), “Compact”, “Slender”

\[ \text{Compression} = \text{“Compact”} \]

In flexure:
\( \text{Flexure} := \text{if} \left( \frac{D}{t} \leq 0.07 \cdot \frac{E}{F_y} \right) \), “Compact”, \( \text{if} \left( \frac{D}{t} \leq 0.31 \cdot \frac{E}{F_y} \right) \), “Noncompact”, “Slender”

\[ \text{Flexure} = \text{“Compact”} \]

Since Compact in Compression, See Section E3 - Members without slender elements:

\[ F_e := \frac{n^2 \cdot E}{(K \cdot L)^2 r^2} \]
\[ F_e = 32.42 \text{ ksi} \]

\[ F_{cr} := \text{if} \left( \frac{K \cdot L}{r} \leq 4.71 \cdot \left( \frac{E}{F_y} \right) \cdot \left( \frac{F_y}{F_e} \cdot 0.658 \right) \cdot F_y \cdot 0.877 \cdot F_e \right) \]
\[ F_{cr} = 26.22 \text{ ksi} \]

\[ A_g := A \] Gross area of member

\[ P_n := F_{cr} \cdot A_g \]
\[ P_n = 803.12 \text{ kip} \] eqn. E3-1

Since Compact in Flexure, Fails by Local Buckling:

Since section Compact:
\[ M_n := F_y \cdot Z \]
\[ M_n = 792.36 \text{ ft} \cdot \text{kip} \] eqn. F8-1

per AISC code

However to avoid taking member into plastic range, use S as a conservative design:
\[ M_n := F_y \cdot S \]
\[ M_n = 607.03 \text{ ft} \cdot \text{kip} \]
Check Member for Combined Flexure and Compression:

\[
\text{Combined} \iff \left( \frac{P_{uc}}{\phi \cdot P_n} \geq 0.2, \frac{P_{uc}}{\phi \cdot P_n} + \frac{8}{9} \cdot \frac{M_u}{\phi \cdot M_n}, 0.5 \cdot \frac{P_{uc}}{\phi \cdot P_n} + \frac{M_u}{\phi \cdot M_n} \right)
\]

\[\text{Combined} = 0.52\]

Check \iff (Combined \leq 1.0, "O.K. Flex. and Comp.", "NOT O.K. Flex. and Comp.")

\[\text{Check} = \text{"O.K. Flex. and Comp."}\]

Check Member for Combined Flexure and Tension:

\[
P_{nt} := F_y \cdot A_g \quad P_{nt} = 1531.53 \text{ kip}
\]

\[
\text{Combined} \iff \left( \frac{P_{ut}}{\phi \cdot P_{nt}} \geq 0.2, \frac{P_{ut}}{\phi \cdot P_{nt}} + \frac{8}{9} \cdot \frac{M_u}{\phi \cdot M_{nt}}, 0.5 \cdot \frac{P_{ut}}{\phi \cdot P_{nt}} + \frac{M_u}{\phi \cdot M_{nt}} \right)
\]

\[\text{Combined} = 0.13\]

Check \iff (Combined \leq 1.0, "O.K. Flex. and Tension", "NOT O.K. Flex. and Tension")

\[\text{Check} = \text{"O.K. Flex. and Tension"}\]
5.1.3 24" Diameter Pipe Pile Demand

Following table presents results for middle and back row piles from the STAAD model for LRFD analysis using the loads documented in Reference 1a. The selection of the reinforcement for the concrete connection based on CRSI Design Handbook 2008 tables (Reference 2d).

The column diameter for this particular pipe pile is 24" - 2*0.688" = 22.6".

The nearest column size has a diameter of 22 inches.

The preferred reinforcement is #11 headed bars, 8 bars minimum.

<table>
<thead>
<tr>
<th>Beam</th>
<th>Node</th>
<th>L/C</th>
<th>Axial</th>
<th>Shear</th>
<th>Torsion</th>
<th>Bending</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Fx (kip)</td>
<td>Fy (kip)</td>
<td>Fz (kip)</td>
<td>Mx (kip-ft)</td>
</tr>
<tr>
<td>Max Fx</td>
<td>8</td>
<td>15</td>
<td>531.2</td>
<td>0.0</td>
<td>-12.0</td>
<td>-0.0</td>
</tr>
<tr>
<td>Min Fx</td>
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<td>50</td>
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<td>-42.3</td>
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<td>38.5</td>
</tr>
<tr>
<td>Max Fy</td>
<td>1869</td>
<td>1723</td>
<td>40</td>
<td>206.1</td>
<td>13.2</td>
<td>24.0</td>
</tr>
<tr>
<td>Min Fy</td>
<td>16</td>
<td>31</td>
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<td>178.8</td>
<td>-12.5</td>
<td>39.1</td>
</tr>
<tr>
<td>Max Fz</td>
<td>1864</td>
<td>1718</td>
<td>39</td>
<td>-13.4</td>
<td>-3.0</td>
<td>55.5</td>
</tr>
<tr>
<td>Min Fz</td>
<td>1865</td>
<td>1719</td>
<td>26</td>
<td>87.0</td>
<td>-7.3</td>
<td>-107.5</td>
</tr>
<tr>
<td>Max Mx</td>
<td>1864</td>
<td>1718</td>
<td>40</td>
<td>151.8</td>
<td>6.8</td>
<td>-103.0</td>
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<tr>
<td>Min Mx</td>
<td>1864</td>
<td>1718</td>
<td>26</td>
<td>-13.8</td>
<td>5.8</td>
<td>50.0</td>
</tr>
<tr>
<td>Max My</td>
<td>5</td>
<td>9</td>
<td>26</td>
<td>158.1</td>
<td>-6.9</td>
<td>-94.1</td>
</tr>
<tr>
<td>Min My</td>
<td>1865</td>
<td>4</td>
<td>26</td>
<td>84.2</td>
<td>-7.3</td>
<td>-107.5</td>
</tr>
<tr>
<td>Max Mz</td>
<td>23</td>
<td>45</td>
<td>40</td>
<td>378.0</td>
<td>13.1</td>
<td>26.8</td>
</tr>
<tr>
<td>Min Mz</td>
<td>1869</td>
<td>38</td>
<td>53</td>
<td>178.8</td>
<td>-12.5</td>
<td>39.1</td>
</tr>
</tbody>
</table>

Demand: 
Pu = 531.2 kip 
Mu = 1680 kip-in 
Pu = 84.2 kip 
Mu = 13020 kip-in

Capacity: 
Phi Pn = 771 kip 
Phi Mu = 5135 kip-in 
Phi Pn = 404 kip 
Phi Mu = 5865 kip-in
Milwaukee DMMF Project

The reinforcement is not sufficient for the connection therefore will use a steel wide flange beam.

\[ M_{u, wf} := 13020 \text{ kip} \cdot \text{in} = 5865 \text{ kip} \cdot \text{in} = 7155.00 \text{ kip} \cdot \text{in} \]

\[ \phi_b := 0.9 \]

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

\[ Z_{req} := \frac{M_{u, wf}}{\phi_b \cdot F_y} = 159.00 \text{ in}^3 \]
Selection of W section

\[ OD_{col} := 24 \text{ in} - 2 \cdot 0.688 \text{ in} - 2 \cdot 1.5 \text{ in} - 2 \cdot \frac{1}{2} \text{ in} - 2 \cdot 1.41 \text{ in} = 15.80 \text{ in} \]

Check W10x112

\[ d := 11.4 \text{ in} \]

\[ bf := 10.4 \text{ in} \]

\[ Diagonal := \left( d^2 + bf^2 \right)^{0.5} = 15.43 \text{ in} \]

Check := \[
\begin{align*}
& \text{if } \text{Diagonal} \geq OD_{col} \text{ then } \text{“OK”} \\
& \quad \text{“NG”} \\
& \text{else } \text{“OK”}
\end{align*}
\]

\[ Z_{W10} := 147 \text{ in}^3 \]

Check := \[
\begin{align*}
& \text{if } Z_{W10} \leq Z_{req} \text{ then } \text{“add cover plates”} \\
& \quad \text{“add cover plates”} \\
& \text{else } \text{“OK”}
\end{align*}
\]

Note:

Column section was drawn in CAD to determine the cover plate size.

\[ Z_{add} := Z_{req} - Z_{W10} = 12.00 \text{ in}^3 \]

\[ b_{cp} := 4 \text{ in} \]

\[ d_1 := d = 11.40 \text{ in} \]

\[ d_{cp} := \left( \frac{4 \cdot Z_{add}}{b_{cp}} + d_1^2 \right)^{0.5} = 11.91 \text{ in} \]

\[ t_{cp} := 0.5 \cdot (d_{cp} - d_1) = 0.26 \text{ in} \]

This is the minimum cover plate thickness. To reduce the D/C ratio, use 1" cover plate.
$$d_{cp} = d_1 + 2 \cdot t_{cp\_final} = 13.40 \text{ in}$$

$$Z_{cp} = \frac{b_{cp}}{4} \cdot (d_{cp}^2 - d_1^2) = 49.60 \text{ in}^3$$

$$Z_{total} = Z_{W10} + Z_{cp} = 196.60 \text{ in}^3$$

$$Check: \begin{cases} 
\text{if } Z_{total} \geq Z_{req} & \Rightarrow \text{"OK"} \\
\text{else} & \Rightarrow \text{"increase cover plate thickness"}
\end{cases}$$

### 5.1.4 24" Diameter Pipe Pile Structural Capacity

Following table presents results for middle and back row piles from the STAAD model for ASD analysis using the loads documented in Reference 1a. Structural capacity of the pipe and stress ratio is calculated using AISC-360 (Reference 2e).

#### Beam End Force Summary

The signs of the forces at end B of each beam have been reversed. For example: this means that the Min Fx entry gives the largest tension value for an beam.

<table>
<thead>
<tr>
<th>Beam</th>
<th>Node</th>
<th>L/C</th>
<th>Axial Fx (kip)</th>
<th>Shear Fy (kip)</th>
<th>Torsion Fz (kip ft)</th>
<th>Bending Mx (kip ft)</th>
<th>Bending My (kip ft)</th>
<th>Bending Mz (kip ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max Fx</td>
<td>8</td>
<td>15</td>
<td>89.38D + LCF</td>
<td>388.4</td>
<td>0.6</td>
<td>-9.7</td>
<td>-0.1</td>
<td>112.4</td>
</tr>
<tr>
<td>Max Fy</td>
<td>1869</td>
<td>1233</td>
<td>78.55D + LCF</td>
<td>181.9</td>
<td>9.1</td>
<td>15.5</td>
<td>-1.9</td>
<td>-48.0</td>
</tr>
<tr>
<td>Min Fy</td>
<td>1886</td>
<td>1719</td>
<td>74.36D + LCF</td>
<td>213.8</td>
<td>-0.5</td>
<td>23.8</td>
<td>0.2</td>
<td>-79.1</td>
</tr>
<tr>
<td>Max Fx</td>
<td>1864</td>
<td>1718</td>
<td>75.35H + LCF</td>
<td>-3.0</td>
<td>-1.4</td>
<td>35.8</td>
<td>-3.4</td>
<td>-128.9</td>
</tr>
<tr>
<td>Min Fz</td>
<td>1865</td>
<td>1719</td>
<td>52.52B + B+E</td>
<td>88.1</td>
<td>-4.1</td>
<td>-67.1</td>
<td>17.6</td>
<td>264.2</td>
</tr>
<tr>
<td>Max Mz</td>
<td>1864</td>
<td>1718</td>
<td>52.52B + B+E</td>
<td>101.0</td>
<td>4.8</td>
<td>-64.3</td>
<td>18.0</td>
<td>243.2</td>
</tr>
<tr>
<td>Min My</td>
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<td>1718</td>
<td>70.55I + LCF</td>
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<td>4.1</td>
<td>32.7</td>
<td>-3.5</td>
<td>-117.4</td>
</tr>
<tr>
<td>Max My</td>
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<td>9</td>
<td>62.52B + B+E</td>
<td>145.4</td>
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<td>-58.7</td>
<td>14.1</td>
<td>585.3</td>
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<tr>
<td>Min My</td>
<td>1865</td>
<td>4</td>
<td>62.52B + B+E</td>
<td>85.7</td>
<td>-4.1</td>
<td>-67.1</td>
<td>17.6</td>
<td>-675.1</td>
</tr>
<tr>
<td>Max Mz</td>
<td>23</td>
<td>45</td>
<td>76.55I + LCF</td>
<td>282.9</td>
<td>8.9</td>
<td>17.3</td>
<td>-1.8</td>
<td>-158.7</td>
</tr>
<tr>
<td>Min Mz</td>
<td>16</td>
<td>31</td>
<td>74.55D + LCF</td>
<td>125.3</td>
<td>-8.5</td>
<td>25.3</td>
<td>0.1</td>
<td>-242.7</td>
</tr>
</tbody>
</table>

$$P_{uc} = 388.4 \cdot \text{kip}$$ compression

$$P_{ut} = 22.5 \cdot \text{kip}$$ tension

$$M_u = 675.1 \cdot \text{ft} \cdot \text{kip}$$ flexure

Note that above values are conservative as they do not occur at the same time.
1. Pipe dimensional and structural properties.

\[ OD := 24 \text{ in} \]
\[ D := OD = 24.00 \text{ in} \]
\[ t_{\text{reduction}} := 0 \text{ in} \]

\[ t := 0.688 \text{ in} - t_{\text{reduction}} = 0.69 \text{ in} \]

\[ ID := OD - 2 \cdot t = 22.62 \text{ in} \]

\[ A := 0.25 \cdot (OD^2 - ID^2) \cdot n = 50.39 \text{ in}^2 \]

\[ I := \frac{n}{64} \cdot (OD^4 - ID^4) = 3425.82 \text{ in}^4 \]

\[ S := \frac{n}{32 \cdot OD} \cdot (OD^4 - ID^4) = 285.49 \text{ in}^3 \]

\[ r := \sqrt{\frac{I}{A}} = 8.25 \text{ in} \]

\[ Z := \frac{OD^3}{6} - \frac{ID^3}{6} = 374.00 \text{ in}^3 \]

\[ E := 29000 \cdot \text{ksi} \]

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

1. Pipe flexural and compaction capacity is checked using AISC Steel Construction Manual 13th Edition

Assume fixed at base, rotation fixed and translation free at top (Table C-C2.2)

\[ \phi := 0.9 \quad L := 25 \cdot \text{ft} \quad K := 1.2 \]
**Check Compactness:**  
Table B4.1 Case 15

In uniform compression:  
\[ \text{Compression} := \text{if} \left( \frac{D}{t} \leq 0.11 \cdot \frac{E}{F_y} \right) \text{"Compact", "Slender"} \]

**Compression** = "Compact"

In flexure:  
\[ \text{Flexure} := \text{if} \left( \frac{D}{t} \leq 0.07 \cdot \frac{E}{F_y} \right) \text{"Compact"}, \text{if} \left( \frac{D}{t} \leq 0.31 \cdot \frac{E}{F_y} \right) \text{"Noncompact", "Slender"} \]

**Flexure** = "Compact"

**Since Compact in Compression, See Section E3 - Members without slender elements:**

\[ F_e := \frac{n^2 \cdot E}{(K \cdot L)^2 \cdot r} \quad F_e = 150.16 \text{ ksi} \]

\[ F_{cr} := \text{if} \left( \frac{K \cdot L}{r} \leq 4.71 \cdot \frac{E}{F_y} \left( \frac{F_y}{0.658} \cdot F_e \cdot 0.877 \cdot F_e \right) \right) \]

\[ F_{cr} = 43.50 \text{ ksi} \]

\[ A_g := A \quad \text{Gross area of member} \]

\[ P_n := F_{cr} \cdot A_g \quad P_n = 2191.59 \text{ kip} \quad \text{eqn. E3-1} \]

**Since Compact in Flexure, Fails by Local Buckling:**

Since section Compact:  
\[ M_n := F_y \cdot Z \quad M_n = 1558.34 \text{ ft} \cdot \text{kip} \quad \text{eqn. F8-1} \]

per AISC code

However to avoid taking member into plastic range, use S as a conservative design:

\[ M_n := F_y \cdot S \quad M_n = 1189.52 \text{ ft} \cdot \text{kip} \]
Check Member for Combined Flexure and Compression:

\[
\text{Combined} := \text{if } \left( \frac{P_{uc}}{\phi \cdot P_n} \geq 0.2, \frac{P_{uc}}{\phi \cdot P_n} + \frac{8}{9} \cdot \frac{M_u}{\phi \cdot M_n}, 0.5 \cdot \frac{P_{uc}}{\phi \cdot P_n} + \frac{M_u}{\phi \cdot M_n} \right)
\]

\[
\text{Combined} = 0.73
\]

Check := if (Combined \leq 1.0, “O.K. Flex. and Comp.”, “NOT O.K. Flex. and Comp.”)

\[
\text{Check} = “O.K. Flex. and Comp.”
\]

Check Member for Combined Flexure and Tension:

\[
P_{nt} := F_y \cdot A_g \quad P_{nt} = 2519.35 \text{ kip}
\]

\[
\text{Combined} := \text{if } \left( \frac{P_{ut}}{\phi \cdot P_{nt}} \geq 0.2, \frac{P_{ut}}{\phi \cdot P_{nt}} + \frac{8}{9} \cdot \frac{M_u}{\phi \cdot M_n}, 0.5 \cdot \frac{P_{ut}}{\phi \cdot P_{nt}} + \frac{M_u}{\phi \cdot M_n} \right)
\]

\[
\text{Combined} = 0.64
\]

Check := if (Combined \leq 1.0, “O.K. Flex. and Tension”, “NOT O.K. Flex. and Tension”)

\[
\text{Check} = “O.K. Flex. and Tension”
\]
5.2 Dolphin

5.2.1 24" Diameter Pipe Pile Demand

Following table presents results for front piles from the STAAD model for LRFD analysis using the loads documented in Reference 1b. The selection of the reinforcement for the concrete connection based on CRSI Design Handbook 2008 tables (Reference 2d).

The column diameter for this particular pipe pile is 24" - 2*0.688" = 22.6". The nearest column size has a diameter of 22 inches.

The preferred reinforcement is #8 headed bars, 8 bars minimum.

### Beam End Force Summary

The signs of the forces at end B of each beam have been reversed. For example: this means that the Min Fx entry gives the largest tension value for an beam.

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<tr>
<th>Beam</th>
<th>Node</th>
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<th>Shear</th>
<th>Torsion</th>
<th>Bending</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Fx (kip)</td>
<td>Fy (kip)</td>
<td>Fz (kip)</td>
<td>Mx (kip ft)</td>
</tr>
<tr>
<td>Max Fx</td>
<td>4</td>
<td>7</td>
<td>33:USB 1.2D+1</td>
<td>144.0</td>
<td>0.1</td>
<td>2.9</td>
</tr>
<tr>
<td>Min Fx</td>
<td>6</td>
<td>12</td>
<td>26:UZB 1.2D+1</td>
<td>-216.7</td>
<td>0.1</td>
<td>-7.2</td>
</tr>
<tr>
<td>Max Fy</td>
<td>5</td>
<td>9</td>
<td>38:USG 1.2D+1</td>
<td>-150.0</td>
<td>3.7</td>
<td>-7.8</td>
</tr>
<tr>
<td>Min Fy</td>
<td>6</td>
<td>11</td>
<td>25:U2A 1.2D+1</td>
<td>83.7</td>
<td>-2.1</td>
<td>1.7</td>
</tr>
<tr>
<td>Max Fz</td>
<td>4</td>
<td>7</td>
<td>52:UBC 1.2D+1</td>
<td>-138.9</td>
<td>0.0</td>
<td>32.2</td>
</tr>
<tr>
<td>Min Fz</td>
<td>6</td>
<td>11</td>
<td>25:U2A 1.2D+1</td>
<td>-150.0</td>
<td>3.7</td>
<td>-7.8</td>
</tr>
<tr>
<td>Max Mx</td>
<td>4</td>
<td>7</td>
<td>34:USC 1.2D+1</td>
<td>42.6</td>
<td>2.1</td>
<td>1.5</td>
</tr>
<tr>
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<td>6</td>
<td>11</td>
<td>25:U2A 1.2D+1</td>
<td>-191.6</td>
<td>3.6</td>
<td>-4.3</td>
</tr>
<tr>
<td>Max My</td>
<td>6</td>
<td>11</td>
<td>25:U2A 1.2D+1</td>
<td>-150.0</td>
<td>3.7</td>
<td>-7.8</td>
</tr>
<tr>
<td>Min My</td>
<td>4</td>
<td>7</td>
<td>52:UBC 1.2D+1</td>
<td>-138.9</td>
<td>0.0</td>
<td>32.2</td>
</tr>
<tr>
<td>Max Mz</td>
<td>6</td>
<td>11</td>
<td>25:U2A 1.2D+1</td>
<td>-150.0</td>
<td>3.7</td>
<td>-7.8</td>
</tr>
<tr>
<td>Min Mz</td>
<td>6</td>
<td>12</td>
<td>25:U2A 1.2D+1</td>
<td>-160.6</td>
<td>3.7</td>
<td>-7.8</td>
</tr>
</tbody>
</table>

### Demand

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Pu = 216.7 kip</td>
<td>Phi Pn = 198 kip</td>
</tr>
<tr>
<td>Mu = 2179 kip-in</td>
<td>Phi Mu = 3795 kip-in</td>
</tr>
</tbody>
</table>

### Capacity

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Pu = 138.9 kip</td>
<td>Phi Pn = 198 kip</td>
</tr>
<tr>
<td>Mu = 2556 kip-in</td>
<td>Phi Mu = 3795 kip-in</td>
</tr>
</tbody>
</table>
The reinforcement is sufficient for the connection therefore no other element is required.

5.2.2 24" Diameter Pipe Pile Structural Capacity

Following table presents results for front piles from the STAAD model for ASD analysis using the loads documented in Reference 1b. Structural capacity of the pipe and stress ratio is calculated using AISC-360 (Reference 2).
### Beam End Force Summary

The signs of the forces at end B of each beam have been reversed. For example, this means that the Min Fx entry gives the largest tension value for an beam.

<table>
<thead>
<tr>
<th>Beam</th>
<th>Node</th>
<th>L/C</th>
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<th>Shear</th>
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<tr>
<td></td>
<td></td>
<td></td>
<td>Fx (kip)</td>
<td>Fy (kip)</td>
<td>Fz (kip)</td>
<td>Mx (kip-ft)</td>
</tr>
<tr>
<td>Max Fx</td>
<td>4</td>
<td>7</td>
<td>33:U5B 1.2D+</td>
<td>144.0</td>
<td>0.1</td>
<td>2.9</td>
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<tr>
<td>Min Fx</td>
<td>6</td>
<td>12</td>
<td>26:U2B 1.2D+</td>
<td>-216.7</td>
<td>0.1</td>
<td>-7.2</td>
</tr>
<tr>
<td>Max Fy</td>
<td>6</td>
<td>11</td>
<td>25:U2A 1.2D+</td>
<td>-159.0</td>
<td>3.7</td>
<td>-7.8</td>
</tr>
<tr>
<td>Min Fy</td>
<td>5</td>
<td>9</td>
<td>38:USG 1.2D+</td>
<td>83.7</td>
<td>-2.1</td>
<td>1.7</td>
</tr>
<tr>
<td>Max Fz</td>
<td>4</td>
<td>7</td>
<td>52:U8C 1.2D+</td>
<td>-158.9</td>
<td>0.0</td>
<td>32.2</td>
</tr>
<tr>
<td>Min Fz</td>
<td>6</td>
<td>11</td>
<td>25:U2A 1.2D+</td>
<td>-159.0</td>
<td>3.7</td>
<td>-7.8</td>
</tr>
<tr>
<td>Max My</td>
<td>4</td>
<td>7</td>
<td>34:USG 1.2D+</td>
<td>-191.6</td>
<td>3.6</td>
<td>-4.3</td>
</tr>
<tr>
<td>Min My</td>
<td>6</td>
<td>11</td>
<td>25:U2A 1.2D+</td>
<td>-159.0</td>
<td>3.7</td>
<td>-7.8</td>
</tr>
<tr>
<td>Max Mz</td>
<td>4</td>
<td>7</td>
<td>52:U8C 1.2D+</td>
<td>-158.9</td>
<td>0.0</td>
<td>32.2</td>
</tr>
<tr>
<td>Min Mz</td>
<td>6</td>
<td>11</td>
<td>25:U2A 1.2D+</td>
<td>-160.6</td>
<td>3.7</td>
<td>-7.8</td>
</tr>
</tbody>
</table>

\[
P_{uc} := 144 \cdot \text{kip} \quad \text{compression}
\]

\[
P_{ut} := 216 \cdot \text{kip} \quad \text{tension}
\]

\[
M_{u} := 213 \cdot \text{ft} \cdot \text{kip} \quad \text{flexure}
\]

Note that above values are conservative as they do not occur at the same time.

1. Pipe dimensional and structural properties.

\[
OD := 24 \text{ in}
\]

\[
D := OD = 24.00 \text{ in}
\]

\[
t_{\text{reduction}} := 0 \text{ in}
\]

\[
t := 0.688 \text{ in} - t_{\text{reduction}} = 0.69 \text{ in}
\]

\[
ID := OD - 2 \cdot t = 22.62 \text{ in}
\]

\[
A := 0.25 \cdot (OD^2 - ID^2) \cdot n = 50.39 \text{ in}^2
\]
\[ I := \frac{n}{64} \cdot (OD^4 - ID^4) = 3425.82 \text{ in}^4 \]

\[ S := \frac{n}{32 \cdot OD} \cdot (OD^4 - ID^4) = 285.49 \text{ in}^3 \]

\[ r := \sqrt{\frac{I}{A}} = 8.25 \text{ in} \]

\[ Z := \frac{OD^3}{6} - \frac{ID^3}{6} = 374.00 \text{ in}^3 \]

\[ E := 29000 \cdot \text{ksi} \]

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

1. Pipe flexural and compaction capacity is checked using AISC Steel Construction Manual 13th Edition

Assume fixed at base, rotation fixed and translation free at top (Table C-C2.2)

\[ \phi := 0.9 \quad L := 45 \cdot \text{ft} \quad K := 1.2 \]

Check Compactness: Table B4.1 Case 15

In uniform compression:

\[ Compression := \text{if} \left( \frac{D}{t} \leq 0.11 \cdot \frac{E}{F_y} \right), \text{"Compact"}, \text{"Slender"} \]

\[ Compression = \text{"Compact"} \]

In flexure:

\[ Flexure := \text{if} \left( \frac{D}{t} \leq 0.07 \cdot \frac{E}{F_y} \right), \text{"Compact"}, \text{if} \left( \frac{D}{t} \leq 0.31 \cdot \frac{E}{F_y} \right), \text{"Noncompact"}, \text{"Slender"} \]

\[ Flexure = \text{"Compact"} \]
Since Compact in Compression, See Section E3 - Members without slender elements:

\[ F_e := \frac{n^2 \cdot E}{(K \cdot L)^2} \]

\[ F_e = 46.34 \text{ ksi} \]

\[ F_{cr} := \begin{cases} 4.71 \sqrt[4]{\frac{E}{F_y}} \left( \frac{F_y}{F_e} \right) & \text{if } \frac{K \cdot L}{r} \leq 4.71 \sqrt[4]{\frac{E}{F_y}} \left( \frac{F_y}{F_e} \right) \end{cases} \]

\[ F_{cr} = 31.83 \text{ ksi} \]

\[ A_g := A \quad \text{Gross area of member} \]

\[ P_n := F_{cr} \cdot A_g \]

\[ P_n = 1603.89 \text{ kip} \quad \text{eqn. E3-1} \]

Since Compact in Flexure, Fails by Local Buckling:

Since section Compact:

\[ M_n := F_y \cdot Z \]

\[ M_n = 1558.34 \text{ ft} \cdot \text{kip} \quad \text{eqn. F8-1} \]

per AISC code

However to avoid taking member into plastic range, use S as a conservative design:

\[ M_n := F_y \cdot S \]

\[ M_n = 1189.52 \text{ ft} \cdot \text{kip} \]

Check Member for Combined Flexure and Compression:

\[ \text{Combined} := \begin{cases} P_{uc} \geq 0.2, & P_{uc} + \frac{8}{9} \cdot \frac{M_u}{\phi \cdot P_n} + 0.5 \cdot \frac{M_u}{\phi \cdot M_n} \end{cases} \]

\[ \text{Combined} = 0.25 \]
Check := if (Combined ≤ 1.0, "O.K. Flex. and Comp.", "NOT O.K. Flex. and Comp.")

\[ Check = "O.K. Flex. and Comp." \]

**Check Member for Combined Flexure and Tension:**

\[ P_{nt} := F_y \cdot A_g \quad P_{nt} = 2519.35 \text{ kip} \]

\[ Combined := if \left( \frac{P_{ut}}{\phi \cdot P_{nt}} \geq 0.2, \frac{P_{ut}}{\phi \cdot P_{nt}} + \frac{8}{9} \frac{M_u}{\phi \cdot M_n}, 0.5 \cdot \frac{P_{ut}}{\phi \cdot P_{nt}} + \frac{M_u}{\phi \cdot M_n} \right) \]

\[ Combined = 0.25 \]

Check := if (Combined ≤ 1.0, "O.K. Flex. and Tension", "NOT O.K. Flex. and Tension")

\[ Check = "O.K. Flex. and Tension" \]
5.2.3 32" Diameter Pipe Pile Demand

Following table presents results for back piles from the STAAD model for LRFD analysis using the loads documented in Reference 1b. The selection of the reinforcement for the concrete connection based on CRSI Design Handbook 2008 tables (Reference 2d).

The column diameter for this particular pipe pile is 32'' - 2*0.688'' = 30.6''. The nearest column size has a diameter of 30 inches.

The preferred reinforcement is #8 headed bars, 8 bars minimum.

### Beam End Force Summary

The signs of the forces at end B of each beam have been reversed. For example: this means that the Min Fx entry gives the largest tension value for an beam.

<table>
<thead>
<tr>
<th>Beam</th>
<th>Node</th>
<th>L/C</th>
<th>Axial</th>
<th>Shear</th>
<th>Torsion</th>
<th>Bending</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Fx (kip)</td>
<td>Fy (kip)</td>
<td>Fz (kip)</td>
<td>Mx (kip*ft)</td>
</tr>
<tr>
<td>Max Fx</td>
<td>3</td>
<td>5</td>
<td>25:U2A 1,2D+1</td>
<td>304.5</td>
<td>28.4</td>
<td>-182.6</td>
</tr>
<tr>
<td>Min Fx</td>
<td>3</td>
<td>6</td>
<td>32:U5A 1,2D+1</td>
<td>-184.8</td>
<td>-55.0</td>
<td>40.5</td>
</tr>
<tr>
<td>Max Fy</td>
<td>1</td>
<td>1</td>
<td>34:U5C 1,2D+*</td>
<td>-169.7</td>
<td>55.0</td>
<td>40.5</td>
</tr>
<tr>
<td>Min Fy</td>
<td>3</td>
<td>5</td>
<td>32:U5A 1,2D+1</td>
<td>-176.9</td>
<td>-55.0</td>
<td>40.5</td>
</tr>
<tr>
<td>Max Fz</td>
<td>1</td>
<td>1</td>
<td>33:U5B 1,2D+1</td>
<td>-66.6</td>
<td>1.0</td>
<td>73.9</td>
</tr>
<tr>
<td>Min Fz</td>
<td>3</td>
<td>5</td>
<td>25:U2A 1,2D+1</td>
<td>304.5</td>
<td>28.4</td>
<td>-182.6</td>
</tr>
<tr>
<td>Max Mx</td>
<td>2</td>
<td>3</td>
<td>34:U5C 1,2D+*</td>
<td>-23.1</td>
<td>52.2</td>
<td>41.4</td>
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<tr>
<td>Min Mx</td>
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<td>3</td>
<td>25:U2A 1,2D+1</td>
<td>229.7</td>
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<td>25:U2A 1,2D+1</td>
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<td>-182.6</td>
</tr>
<tr>
<td>Min My</td>
<td>3</td>
<td>6</td>
<td>25:U2A 1,2D+1</td>
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<tr>
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<td>32:U5A 1,2D+1</td>
<td>-184.8</td>
<td>-55.0</td>
<td>40.5</td>
</tr>
<tr>
<td>Min Mz</td>
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<td>2</td>
<td>34:U5C 1,2D+*</td>
<td>-177.8</td>
<td>55.0</td>
<td>40.5</td>
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</table>

### Demand

<table>
<thead>
<tr>
<th></th>
<th>Capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pu = 304.5 kip</td>
<td>Phi Pn = 861 kip</td>
</tr>
<tr>
<td>Mu = 19218 kip-in</td>
<td>Phi Mu = 13586 kip-in</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th></th>
<th>Capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pu = 296.6 kip</td>
<td>Phi Pn = 861 kip</td>
</tr>
<tr>
<td>Mu = 24969 kip-in</td>
<td>Phi Mu = 13586 kip-in</td>
</tr>
</tbody>
</table>
## Milwaukee DMMF Project

### ROUND TIED COLUMNS 30° DIA.

**Short columns – no sideways**

- $f'_c = 5,000$ psi
- $f_c = 60,000$ psi

<table>
<thead>
<tr>
<th>BARS</th>
<th>RHO</th>
<th>Max Cap</th>
<th>0% $f_c$</th>
<th>25% $f_c$</th>
<th>50% $f_c$</th>
<th>100% $f_c$</th>
<th>$e_1 = 0.005$</th>
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</thead>
<tbody>
<tr>
<td>8â10</td>
<td>0.67</td>
<td>7115</td>
<td>3686</td>
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<td>1476</td>
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<td>7115</td>
<td>3686</td>
<td>1968</td>
<td>1476</td>
<td>1145</td>
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<td>379</td>
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<td>20.00</td>
<td>7115</td>
<td>3686</td>
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<td>20.00</td>
<td>7115</td>
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<td>1968</td>
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</tr>
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<td>7115</td>
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<td>1968</td>
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<td>1145</td>
<td>379</td>
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<tr>
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<td>20.00</td>
<td>7115</td>
<td>3686</td>
<td>1968</td>
<td>1476</td>
<td>1145</td>
<td>379</td>
</tr>
</tbody>
</table>

(1) See “Control Points for Interaction Curves – Round Columns”; Fig. 4-1.

(2) 0% $f_c$ indicates zero tension in bars on the tension side, 50% $f_c$ indicates 50% $f_c$ stress in bars on the tension side, and 100% $f_c$ indicates 100% $f_c$ stress (i.e., balance point) in bars on the tension side.
The reinforcement is not sufficient for the connection therefore will use a steel wide flange beam.

\[ Mu_{wf} := 24306.66 \text{kip} \cdot \text{in} - 13586 \text{kip} \cdot \text{in} = 10720.66 \text{kip} \cdot \text{in} \]

\[ \phi_b := 0.9 \]

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

\[ Z_{req} := \frac{Mu_{wf}}{\phi_b \cdot F_y} = 238.24 \text{in}^3 \]

Selection of W section

\[ OD_{col} := 32 \text{in} - 2 \cdot 0.688 \text{in} - 2 \cdot 1.5 \text{in} - 2 \cdot \frac{1}{2} \text{in} - 2 \cdot 1.41 \text{in} = 23.80 \text{in} \]

Check W10x112

\[ d' := 11.4 \text{in} \]

\[ bf := 10.4 \text{in} \]

\[ \text{Diagonal} := \left( d'^2 + bf^2 \right)^{0.5} = 15.43 \text{in} \]

Check :=

\[ \begin{array}{c}
\text{if Diagonal} \geq OD_{col} \text{ then } "OK" \\
\text{else } "NG" \\
\text{else } "OK"
\end{array} \]

\[ Z_{W10} := 147 \text{in}^3 \]

Check :=

\[ \begin{array}{c}
\text{if } Z_{W10} \leq Z_{req} \text{ then } "add cover plates" \\
\text{else } "add cover plates" \\
\text{else } "OK"
\end{array} \]
Note:

Column section was drawn in CAD to determine the cover plate size.

\[ Z_{add} := Z_{req} - Z_{W10} = 91.24 \text{ in}^3 \]

\[ b_{cp} := 8 \text{ in} \]

\[ d_1 := d = 11.40 \text{ in} \]

\[ d_{cp} := \left( \frac{4 \cdot Z_{add}}{b_{cp}} + d_1^2 \right)^{0.5} = 13.25 \text{ in} \]

\[ t_{cp} := 0.5 \cdot (d_{cp} - d_1) = 0.93 \text{ in} \]

This is the minimum cover plate thickness. To reduce the D/C ratio, use 1.5" cover plate.

\[ t_{cp\_final} := 1.5 \text{ in} \]

\[ d_{cp} := d_1 + 2 \cdot t_{cp\_final} = 14.40 \text{ in} \]

\[ Z_{cp} := \frac{b_{cp}}{4} \cdot \left( d_{cp}^2 - d_1^2 \right) = 154.80 \text{ in}^3 \]

\[ Z_{total} := Z_{W10} + Z_{cp} = 301.80 \text{ in}^3 \]

\[ \text{Check} := \begin{cases} \text{"OK"} & \text{if } Z_{total} \geq Z_{req} \\ \text{"OK"} & \text{else} \\ \text{"increase cover plate thickness"} & \text{else} \end{cases} \]
5.2.4 32" Diameter Pipe Pile Structural Capacity

Following table presents results for front piles from the STAAD model for ASD analysis using the loads documented in Reference 1b. Structural capacity of the pipe and stress ratio is calculated using AISC-360 (Reference 2e).

Beam End Force Summary

The signs of the forces at end A and B of each beam have been reversed. For example: this means that the Min Fx entry gives the largest tension value for an beam.

<table>
<thead>
<tr>
<th>Beam</th>
<th>Node</th>
<th>L/C</th>
<th>Axial</th>
<th>Shear</th>
<th>Torsion</th>
<th>Bending</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Fx (kip)</td>
<td>Fy (kip)</td>
<td>Fz (kip)</td>
<td>Mx (kip·ft)</td>
</tr>
<tr>
<td>Max Fx</td>
<td>3</td>
<td>5</td>
<td>61.52A D+B+B</td>
<td>197.3</td>
<td>18.1</td>
<td>-114.3</td>
</tr>
<tr>
<td>Min Fx</td>
<td>3</td>
<td>6</td>
<td>66.55A D+L+U+</td>
<td>-112.3</td>
<td>-34.7</td>
<td>25.8</td>
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<tr>
<td>Max Fy</td>
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<td>1</td>
<td>70.55C D+L+U+</td>
<td>-99.7</td>
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<td>25.8</td>
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<td>5</td>
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<td>1</td>
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<td>1.0</td>
<td>46.7</td>
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<tr>
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<td>5</td>
<td>61.55A D+B+B</td>
<td>167.3</td>
<td>18.1</td>
<td>-114.3</td>
</tr>
<tr>
<td>Max Mx</td>
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<td>70.55C D+L+U+</td>
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<td>61.55A D+B+B</td>
<td>107.3</td>
<td>18.1</td>
<td>-114.3</td>
</tr>
<tr>
<td>Min My</td>
<td>3</td>
<td>6</td>
<td>61.55A D+B+B</td>
<td>190.7</td>
<td>18.1</td>
<td>-114.3</td>
</tr>
<tr>
<td>Max Mz</td>
<td>3</td>
<td>6</td>
<td>66.55A D+L+U+</td>
<td>-112.3</td>
<td>-34.7</td>
<td>25.8</td>
</tr>
<tr>
<td>Min Mz</td>
<td>1</td>
<td>2</td>
<td>70.55C D+L+U+</td>
<td>-106.3</td>
<td>34.8</td>
<td>25.8</td>
</tr>
</tbody>
</table>

\[ P_{uc} := 197 \cdot kip \quad \text{compression} \]

\[ P_{ut} := 112 \cdot kip \quad \text{tension} \]

\[ M_u := 1287.7 \cdot \text{ft} \cdot \text{kip} \quad \text{flexure} \]

Note that above values are conservative as they do not occur at the same time

1. Pipe dimensional and structural properties.

\[ OD := 32 \text{ in} \]

\[ D := OD = 32.00 \text{ in} \]

\[ t_{\text{reduction}} := 0 \text{ in} \]
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Revision: 1

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\[ t := 0.688 \text{ in} - t_{\text{reduction}} = 0.69 \text{ in} \]

\[ ID := OD - 2 \cdot t = 30.62 \text{ in} \]

\[ A := 0.25 \cdot (OD^2 - ID^2) \cdot \pi = 67.68 \text{ in}^2 \]

\[ I := \frac{n}{64} \cdot (OD^4 - ID^4) = 8298.32 \text{ in}^4 \]

\[ S := \frac{n}{32 \cdot OD} \cdot (OD^4 - ID^4) = 518.65 \text{ in}^3 \]

\[ r := \sqrt{\frac{I}{A}} = 11.07 \text{ in} \]

\[ Z := \frac{OD^3}{6} - \frac{ID^3}{6} = 674.65 \text{ in}^3 \]

\[ E := 29000 \cdot \text{ksi} \]

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

1. Pipe flexural and compaction capacity is checked using AISC Steel Construction Manual 13th Edition

Assume fixed at base, rotation fixed and translation free at top (Table C-C2.2)

\[ \phi := 0.9 \quad L := 20 \cdot \text{ft} \quad K := 1.2 \]

Check Compactness: Table B4.1 Case 15

In uniform compression: \( Compression := \text{if} \left( \frac{D}{t} \leq 0.11 \cdot \frac{E}{F_y} \right) \), “Compact”, “Slender”

\[ Compression = \text{“Compact”} \]
In flexure:

\[
Flexure := \begin{cases} 
    \text{if} \left( \frac{D}{t} \leq 0.07 \cdot \frac{E}{F_y} \right), & \text{"Compact"}, \\
    \text{if} \left( \frac{D}{t} \leq 0.31 \cdot \frac{E}{F_y} \right), & \text{"Noncompact", "Slender"}
\end{cases}
\]

\[
Flexure = \text{"Noncompact"}
\]

Since Compact in Compression, See Section E3 - Members without slender elements:

\[
F_e := \frac{n^2 \cdot E}{K \cdot L/r} \quad F_e = 423.11 \text{ ksi}
\]

\[
F_{cr} := \begin{cases} 
    \text{if} \left( \frac{K \cdot L}{r} \leq 4.71 \cdot \sqrt{\frac{E}{F_y}}, \left( \frac{F_e}{F_y} \right)^{0.658}, F_y, 0.877 \cdot F_e \right), \\
    \quad \quad F_{cr} = 47.59 \text{ ksi}
\end{cases}
\]

\[
A_g := A \quad \text{Gross area of member}
\]

\[
P_n := F_{cr} \cdot A_g \quad P_n = 3220.61 \text{ kip} \quad \text{eqn. E3-1}
\]

Since Compact in Flexure, Fails by Local Buckling:

Since section Compact:

\[
M_n := F_y \cdot Z \quad M_n = 2811.05 \text{ ft} \cdot \text{kip} \quad \text{eqn. F8-1}
\]

per AISC code

However to avoid taking member into plastic range, use S as a conservative design:

\[
M_n := F_y \cdot S \quad M_n = 2161.02 \text{ ft} \cdot \text{kip}
\]
Check Member for Combined Flexure and Compression:

\[
\text{Combined} = \begin{cases} 
\text{if} & \left( \frac{P_{uc}}{\phi \cdot P_n} \geq 0.2, \frac{P_{uc}}{\phi \cdot P_n} + \frac{8}{9} \cdot \frac{M_u}{\phi \cdot M_n}, 0.5 \cdot \frac{P_{uc}}{\phi \cdot P_n} + \frac{M_u}{\phi \cdot M_n} \right) \\
\text{Combined} = 0.70
\end{cases}
\]

\[
\text{Check} = \text{if} \left( \text{Combined} \leq 1.0, \text{"O.K. Flex. and Comp."}, \text{"NOT O.K. Flex. and Comp."} \right)
\]

Check = "O.K. Flex. and Comp."

Check Member for Combined Flexure and Tension:

\[
P_{nt} = F_y \cdot A_g \\
P_{nt} = 3383.91 \text{ kip}
\]

\[
\text{Combined} = \begin{cases} 
\text{if} & \left( \frac{P_{ut}}{\phi \cdot P_{nt}} \geq 0.2, \frac{P_{ut}}{\phi \cdot P_{nt}} + \frac{8}{9} \cdot \frac{M_u}{\phi \cdot M_n}, 0.5 \cdot \frac{P_{ut}}{\phi \cdot P_{nt}} + \frac{M_u}{\phi \cdot M_n} \right) \\
\text{Combined} = 0.68
\end{cases}
\]

\[
\text{Check} = \text{if} \left( \text{Combined} \leq 1.0, \text{"O.K. Flex. and Tension"}, \text{"NOT O.K. Flex. and Tension"} \right)
\]

Check = "O.K. Flex. and Tension"
SECTION 6: RESULTS

Platform Reinforcement:

20" Dia. Piles - 6 - #8 headed bars with #4 spirals at 3 inch spacing.

24" Dia. Piles - 14 - #11 headed bars with #4 spirals at 3 inch spacing plus W10x112 with 4"x1" cover plates, welded to flanges, full length of member.

Dolphin Reinforcement:

24" Dia. Piles - 8 - #8 headed bars with #3 spirals at 3 inch spacing.

32" Dia. Piles - 20 - #11 headed bars with #4 spirals at 3 inch spacing plus W10x112 with 8"x1.5" cover plates, welded to flanges, full length of member.

All piles meet the required structural capacity based on AISC guideline.

SECTION 7: REFERENCES

PROJECT DOCUMENTS

1a Foth Structural Design of Concrete Platform, NBPA-NT-SD-03, Nov. 2020 [Appendix N of Design Report].

CODES AND STANDARDS

2a Building Code Requirements for Structural Concrete, ACI 318-14.
2b United Facilities Criteria (UFC), Design: Piers and Wharves, UFC 4-152-01, 24 January 2017.
2d Concrete Reinforcing Steel Institute, Design Handbook, 2008.