Penfield Pavilion:
Pile Wave Load Calculation

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Appendix A: CEM Calculation 12" ø Pile

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Appendix C: CEM Calculation 8" ø Pile
I) Purpose:
To determine environmental loads acting on a 12” round pile, a 12” square pile, and an 8” round pile due to waves and other hydrodynamic forces generated from Long Island Sound at the Penfield Pavilion located at 323 Fairfield Beach Road in Fairfield, CT. Loads will be quantified using the method outlined in the Coastal Construction Manual (FEMA P-55) as well as with the Morison Equation per the Coastal Engineering Manual (CEM) and Det Norske Veritas Classification Notes No. 30.5 (DNV).

II) References:


III) **Water Surface Variations:**

![Diagram of water surface variations]

**Note:**

1) Datum is NAVD 88

2) 100-, 50-, 10- & 100-Yr + Setup from FEMA “FIS - Fairfield County, Connecticut” Preliminary 10/2011

3) All other elevations from USACE “Tidal Flood Profiles New England Coastline” 9/1988

IV) **Site Exposure:**

The site has an open exposure to the waters of the Long Island Sound. Wind can generate waves over a fetch greater than 10 miles in several directions, and therefore was not limited in wave height and wave period calculations.

V) **Wind Speed for Design:**

1) 50-yr, 3-second gust at 33’, $U_{50-yr,3-sec} = 110$ MPH (CT Building Code 2009 Amendment – Appendix K)

2) Conversion factor to 100-yr, 3-second gust at 33’, $U_{100-yr,3-sec}$, is 1.07 (ASCE 7-05 – Table C6-7)

\[
U_{100-yr,3-sec} = 110 \text{ mph} \times 1.07 = 117.7 \text{ mph}
\]
3) Convert $U_{100-yr,3-sec}$ to a wind with a 1-hr duration, $U_{100-yr,1-hr}$ using the Wind speed & Wave Growth Application of the Automated Coastal Engineering System (ACES) (see section VI).

$$U_{100-yr,1-hr} = 77.88 \text{ mph}$$

VI) Design Wave:

The 100-yr wave height, $H_{mo}$, and 100-yr wave period, $T_p$, were calculated using the Wind speed & Wave Growth Application of the Automated Coastal Engineering System (ACES). The site exposure and wind conditions discussed in Sections IV & V were input into ACES as seen below:

<table>
<thead>
<tr>
<th>Breaking criteria</th>
<th>Item</th>
<th>Value</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>El of Observed Wind (Zobs)</td>
<td>33.00</td>
<td>feet</td>
</tr>
<tr>
<td></td>
<td>Observed Wind Speed (Uobs)</td>
<td>117.70</td>
<td>mph</td>
</tr>
<tr>
<td></td>
<td>Air Sea Temp. Diff. (dT)</td>
<td>0.00</td>
<td>deg F</td>
</tr>
<tr>
<td></td>
<td>Dur of Observed Wind (DurO)</td>
<td>3.00</td>
<td>sec</td>
</tr>
<tr>
<td></td>
<td>Dur of Final Wind (DurF)</td>
<td>1.00</td>
<td>hours</td>
</tr>
<tr>
<td></td>
<td>Lat. of Observation (LAT)</td>
<td>41.00</td>
<td>deg</td>
</tr>
<tr>
<td>Results</td>
<td>Wind Fetch Length (F)</td>
<td>26.00</td>
<td>MILES</td>
</tr>
<tr>
<td></td>
<td>Eq Neutral Wind Speed (Ue)</td>
<td>77.88</td>
<td>mph</td>
</tr>
<tr>
<td></td>
<td>Adjusted Wind Speed (Us)</td>
<td>136.74</td>
<td>mph</td>
</tr>
<tr>
<td></td>
<td>Wave Height (Hmo)</td>
<td>9.99</td>
<td>feet</td>
</tr>
<tr>
<td></td>
<td>Wave Period (Tp)</td>
<td>5.97</td>
<td>sec</td>
</tr>
</tbody>
</table>

Wave Growth: Deep

100-yr condition wave height & wave period:

$$H_{mo} = 9.99 \text{ ft}$$

$$T_p = 5.97 \text{ sec}$$

VII) Wave Load Analysis:

Wave loads on the piles were determined using two different calculation methods. Analysis per the Coastal Construction Manual (FEMA P-55 2011) is shown in Section VII-1 and analysis utilizing the Morison’s Equation (CEM /DNV) are shown in Section VII-2.
1) FEMA P-55 Analysis:

FEMA-55 Analysis was performed on a 12” round pile (Section VII-1-A), 12” Square pile (Section VII-1-B), and an 8” round pile (Section VII-1-C).

A) 12” ø Pile:

i. Scour Analysis:

\[ S_{max} = 2a \]  \hspace{1cm} \text{(FEMA P-55 2011 – Eq 8.10)}

\( S_{max} \) = Max Localized Scour Depth
\( a \) = the diameter of the longest cross-section dimension or diagonal

\[ S_{max} = 2(1') = 2' \]

Check for pile group scour:

\[ S_{tot} = 6(a) = 6(1') = 6' \]  \hspace{1cm} \text{(FEMA P-55 2011 – Eq 8.11a)}

Pile group scour likely to occur if soil type is predominately silty (FEMA P-55 2011). Assume soil is sandy. Therefore just \( S_{max} \) is used.

Grade at front of piles, \( G = \text{El.} + 7.4' \) NAVD 88. Grade after scour, \( GS = \text{El.} + 5.4' \) NAVD 88.

ii. Design Stillwater & Velocity:

\[ d_s = E_{sw} - GS \]  \hspace{1cm} \text{(FEMA P-55 2011 – Eq 8.1)}

\( d_s \) = Design stillwater flood depth
\( E_{sw} \) = Design stillwater flood elevation
\( GS \) = Ground elevation after scour

To determine if the design stillwater flood elevation should be set to the 100-yr stillwater elevation or the 100-yr stillwater elevation plus setup, check for wave breaking. If \( H_{mo} \) is greater than 0.78h (where h = water depth before scour =
E_{sw(no setup) - G}, wave breaking occurs and 100-yr stillwater flood elevation plus setup will be used for calculations. Otherwise, 100-yr stillwater not including setup should be used.

\[ H_{mo} = 9.99' > 0.78(h) = 0.78(10.1' - 7.4') \]
\[ 9.99' > 2.1' \]
\[ H_{mo} > 0.78(h) \]  

Therefore wave breaking and wave setup occur.

\[ d_s = 11.1' - 5.4' = 5.7' \]

Velocity:

Lower bound velocity:  
\[ V_{lower} = \frac{d_s}{t} \]  
(FEMA P-55 2011 – Eq 8.2a)

Upper bound velocity:  
\[ V_{upper} = \sqrt{gd_s} \]  
(FEMA P-55 2011 – Eq 8.2b)

V = design water velocity  
t = 1 second  
g = gravitational acceleration (32.2ft/s²)

FEMA P-55 recommends that for buildings directly on a beach or in a VE Zone (such as the Penfield Pavilion) \( V_{upper} \) be used for the design water velocity, V.

\[ V = \sqrt{\frac{32.2 \text{ ft}}{s^2} (5.7 \text{ ft})} = 13.5 \frac{\text{ft}}{s} \]

iii. Breaking Wave Load on Seaward Row of Piles:

\[ F_{brkp} = \frac{1}{2} \left( C_{db} \gamma_w D H_b^2 \right) \]  
(FEMA P-55 2011 – Eq 8.5)

\( F_{brkp} \) = Breaking wave force acting at SWL (per pile)  
\( C_{db} = 2.25 \) for square piles & 1.75 for round piles  
\( \gamma_w = \) specific weight of water (64.4 lb/ft³)  
\( D = \) diameter or the longest cross-section dimension or diagonal  
\( H_b = \) wave breaking height = 0.78\( d_s \) = 0.78*5.7 ft = 4.4 ft

\[ F_{brkp} = \frac{1}{2} \left( 1.75 * 64.4 \frac{\text{lb}}{\text{ft}^3} 1\text{ft}(4.4\text{ ft})^2 \right) = 1084 \text{ lbs} \]

iv. Hydrodynamic Load on Landward Rows of Piles

\[ F_{dyn} = \frac{1}{2} (C_d \rho D V^2 A) \]  
(FEMA P-55 2011 – Eq 8.8)
F_{dyn} = \text{Hydrodynamic load}
\nonumber
C_d = 1.2 \text{ for round piles}
\nonumber
A = \text{Surface area perpendicular to flow }= D \cdot d_s
\nonumber
\rho = \text{water density }= 1.99 \text{ slugs/ft}^3
\nonumber
\nonumber
\begin{align*}
F_{dyn} &= \frac{1}{2} \left( 1.2 \cdot 1.99 \frac{\text{slugs}}{\text{ft}^3} \cdot 1 \text{ft}(5.7 \text{ ft})(13.5 \frac{\text{ft}}{s})^2 \right) = 1190 \text{ lbs}
\end{align*}

v. Impact Load from Debris
\nonumber
\nonumber
F_i = W V C_d C_b C_{str} \quad \text{(FEMA P-55 2011 – Eq 8.9)}
\nonumber
\nonumber
F_i = \text{Impact force acting at the stillwater elevation}
\nonumber
W = \text{weight of object (assume 1000lbs)}
\nonumber
V = \text{water velocity }= \frac{1}{2} V_{upper} = 6.75 \text{ ft/s}
\nonumber
C_d = \text{Depth coefficient }= 1 \text{ in VE Zones}
\nonumber
C_b = \text{Blockage coefficient }= 1 \text{ in first row }& 0.6 \text{ in second row of piles}
\nonumber
C_{str} = \text{Building Structure Coefficient }= 0.2
\nonumber
\nonumber
\begin{align*}
\text{First row of piles:} \\
F_i &= 1000 \text{lbs} \cdot 6.75 \frac{\text{ft}}{s} \cdot 1 \cdot 1 \cdot 0.2 = 1350 \text{ lbs}
\end{align*}
\nonumber
\nonumber
\begin{align*}
\text{Second row of piles:} \\
F_i &= 1000 \text{lbs} \cdot 6.75 \frac{\text{ft}}{s} \cdot 1 \cdot 0.6 \cdot 0.2 = 810 \text{ lbs}
\end{align*}

vi. $F_{\text{design}}$
\nonumber
\nonumber
The design load is the greater of the breaking wave load or impact load for the seaward (first) row of piles. $F_{\text{design}}$ acts at El. +11.1’ for the first row of piles. The greater of the hydrodynamic load or impact load for the landward piles was chosen as $F_{\text{design}}$ for the second row of piles. The hydrodynamic load acts at El. +8.3’ and the impact load acts at El. +11.1’. $F_{\text{design}}$ for the second row of piles should be adjusted if the lower load generates a higher moment in the area of concern due to the loads acting at different elevations.
\nonumber
\begin{align*}
F_{\text{design, first row}} &= 1350 \text{ lbf} \\
F_{\text{design, second row}} &= 1190 \text{ lbf}
\end{align*}
B) 12” Square Pile:

i. Scour Analysis:

\[ S_{\text{max}} = 2a \quad \text{(FEMA P-55 2011 – Eq 8.10)} \]

\( S_{\text{max}} \) = Max Localized Scour Depth
\( a \) = the diameter of the longest cross-section dimension or diagonal

\[ S_{\text{max}} = 2 \left( \sqrt{1^2 + 1^2} \right) = 2.8' \]

Check for pile group scour:

\[ S_{\text{tot}} = 6(a) = 6(1.4') = 8.4' \quad \text{(FEMA P-55 2011 – Eq 8.11a)} \]

Pile group scour likely to occur if soil type is predominately silty (FEMA P-55 2011). Assume soil is sandy. Therefore just \( S_{\text{max}} \) is used.

Grade at front of piles, \( G = \text{El.} + 7.4' \text{ NAVD 88.} \) Grade after scour, \( G_S = \text{El.} + 4.6' \text{ NAVD 88.} \)

ii. Design Stillwater & Velocity:

\[ d_s = E_{sw} - G_S \quad \text{(FEMA P-55 2011 – Eq 8.1)} \]

\( d_s \) = Design stillwater flood depth
\( E_{sw} \) = Design stillwater flood elevation
\( G_S \) = Ground elevation after scour

To determine if the design stillwater flood elevation should be set to the 100-yr stillwater elevation or the 100-yr stillwater elevation plus setup, check for wave breaking. If \( H_{\text{mo}} \) is greater than 0.78h (where h = water depth before scour = \( E_{sw(\text{no setup})} - G \)), wave breaking occurs and 100-yr stillwater flood elevation plus setup will be used for calculations. Otherwise, 100-yr stillwater not including setup should be used.
\[ H_{mo} = 9.99' > 0.78(h) = 0.78(10.1' - 7.4') \]
\[ 9.99' > 2.1' \]
\[ H_{mo} > 0.78(h) \quad \checkmark \]

Therefore wave breaking and wave setup occur.

\[ d_s = 11.1' - 4.6' = 6.5' \]

Velocity:

Lower bound velocity: \[ V_{lower} = \frac{d_s}{t} \]
Upper bound velocity: \[ V_{upper} = \sqrt{g d_s} \]

\[ V = \text{design water velocity} \]
\[ t = 1 \text{ second} \]
\[ g = \text{gravitational acceleration (32.2ft/s}^2) \]

FEMA P-55 recommends that for buildings directly on a beach or in a VE Zone (such as the Penfield Pavilion) \( V_{upper} \) be used for the design water velocity, \( V \).

\[ V = \sqrt{32.2 \frac{ft}{s^2} (6.5ft)} = 13.5 \frac{ft}{s} \]

iii. Breaking Wave Load on Seaward Row of Piles:

\[ F_{brkp} = \frac{1}{2} (C_{db} \gamma_w D H_b)^2 \]

\( F_{brkp} = \) Breaking wave force acting at SWL (per pile)
\( C_{db} = 2.25 \) for square piles & 1.75 for round piles
\( \gamma_w = \) specific weight of water (64.4 lb/ft\(^3\))
\( D = \) diameter or the longest cross-section dimension or diagonal
\( H_b = \) wave breaking height = 0.78\( d_s = 0.78 \times 6.5 \text{ ft} = 5.1 \text{ ft} \)

\[ F_{brkp} = \frac{1}{2} \left( 2.25 \times 64.4 \frac{lb}{ft^3} 1.4ft(5.1 \text{ ft})^2 \right) = 2640 \text{ lbs} \]

iv. Hydrodynamic Load on Landward Rows of Piles

\[ F_{dyn} = \frac{1}{2} (C_d \rho D V^2 A) \]

\( F_{dyn} = \) Hydrodynamic load
\( C_d = 1.25 \) per FEMA P-55 Table 8-2
\( A = \) Surface area perpendicular to flow =\( D \times d_s \)
\( \rho = \) water density = 1.99 slugs/ft\(^3\)
\[ F_{\text{dyn}} = \frac{1}{2} \left( 1.25 \times 1.99 \frac{\text{slugs}}{\text{ft}^3} \times 1.4 \frac{\text{ft}}{\text{s}} \times (6.5 \text{ ft}) (14.5 \frac{\text{ft}}{\text{s}})^2 \right) = 2280 \text{ lbs} \]

v. Impact Load from Debris

\[ F_i = W V C_d C_b C_{\text{str}} \]  
\[ (\text{FEMA P-55 2011 – Eq 8.9}) \]

\( F_i \) = Impact force acting at the stillwater elevation  
\( W \) = weight of object (assume 1000lbs)  
\( V \) = water velocity = \( \frac{1}{2} V_{\text{upper}} = 7.25 \text{ ft/s} \)  
\( C_d \) = Depth coefficient = 1 in VE Zones  
\( C_b \) = Blockage coefficient = 1 in first row & 0.6 in second row of piles  
\( C_{\text{str}} \) = Building Structure Coefficient = 0.2

First row of piles:

\[ F_i = 1000 \text{lbs} \times 7.25 \frac{\text{ft}}{\text{s}} \times 1 \times 1 \times 0.2 = 1450 \text{ lbs} \]

Second row of piles:

\[ F_i = 1000 \text{lbs} \times 7.25 \frac{\text{ft}}{\text{s}} \times 1 \times 0.6 \times 0.2 = 870 \text{ lbs} \]

vi. \( F_{\text{design}} \)

The design load is the greater of the breaking wave load or impact load for the seaward (first) row of piles. \( F_{\text{design}} \) acts at El. +11.1’ for the first row of piles.  
The greater of the hydrodynamic load or impact load for the landward piles was chosen as \( F_{\text{design}} \) for the second row of piles. The hydrodynamic load acts at El. +7.9’ and the impact load acts at El. +11.1’. \( F_{\text{design}} \) for the second row of piles should be adjusted if the lower load generates a higher moment in the area of concern due to the loads acting at different elevations.

\[ F_{\text{design \ first row}} = 2640 \text{ lbf} \]

\[ F_{\text{design \ second row}} = 2280 \text{ lbf} \]
C) 8” Ø Pile:

i. Scour Analysis:

\[ S_{\text{max}} = 2a \]  
(FEMA P-55 2011 – Eq 8.10)

\[ S_{\text{max}} = \text{Max Localized Scour Depth} \]

\[ a = \text{the diameter of the longest cross-section dimension or diagonal} \]

\[ S_{\text{max}} = 2(0.667\text{′}) = 1.33\text{′} \]

Check for pile group scour:

\[ S_{\text{tot}} = 6(a) = 6(0.667\text{′}) = 4\text{′} \]  
(FEMA P-55 2011 – Eq 8.11a)

Pile group scour likely to occur if soil type is predominately silty (FEMA P-55 2011). Assume soil is sandy. Therefore just \( S_{\text{max}} \) is used.

Grade at front of piles, \( G = \text{El.} + 7.4\text{′} \) NAVD 88. Grade after scour, \( GS = \text{El.} + 6.1\text{′} \) NAVD 88.

ii. Design Stillwater & Velocity:

\[ d_s = E_{\text{sw}} - GS \]  
(FEMA P-55 2011 – Eq 8.1)

\( d_s = \text{Design stillwater flood depth} \)

\( E_{\text{sw}} = \text{Design stillwater flood elevation} \)

\( GS = \text{Ground elevation after scour} \)

To determine if the design stillwater flood elevation should be set to the 100-yr stillwater elevation or the 100-yr stillwater elevation plus setup, check for wave breaking. If \( H_{\text{mo}} \) is greater than 0.78h (where \( h = \text{water depth before scour} = E_{\text{sw(no setup)}} - G \)), wave breaking occurs and 100-yr stillwater flood elevation plus setup will be used for calculations. Otherwise, 100-yr stillwater not including setup should be used.

\[ H_{\text{mo}} = 9.99\text{′} > 0.78(h) = 0.78(10.1\text{′} - 7.4\text{′}) \]
9.99′ > 2.1′

\[ H_{mo} > 0.78(h) \]

Therefore wave breaking and wave setup occur.

\[ d_s = 11.1′ - 6.1′ = 5.0′ \]

Velocity:

Lower bound velocity: \[ V_{lower} = \frac{d_s}{t} \] (FEMA P-55 2011 – Eq 8.2a)

Upper bound velocity: \[ V_{upper} = \sqrt{g d_s} \] (FEMA P-55 2011 – Eq 8.2b)

\[ V = \text{design water velocity} \]
\[ t = 1 \text{ second} \]
\[ g = \text{gravitational acceleration (32.2ft/s}^2\text{)} \]

FEMA P-55 recommends that for buildings directly on a beach or in a VE Zone (such as the Penfield Pavilion) \( V_{upper} \) be used for the design water velocity, \( V \).

\[ V = \sqrt{32.2 \frac{ft}{s^2}(5.0 ft)} = 12.7 \frac{ft}{s} \]

iii. Breaking Wave Load on Seaward Row of Piles:

\[ F_{brkp} = \frac{1}{2} \left( C_{db} \gamma_w D H_b^2 \right) \] (FEMA P-55 2011 – Eq 8.5)

\( F_{brkp} = \) Breaking wave force acting at SWL (per pile)
\( C_{db} = 2.25 \) for square piles & 1.75 for round piles
\( \gamma_w = \) specific weight of water (64.4 lb/ft\(^3\))
\( D = \) diameter or the longest cross-section dimension or diagonal
\( H_b = \) wave breaking height = 0.78\( d_s \) = 0.78 * 5 ft = 3.9 ft

\[ F_{brkp} = \frac{1}{2} \left( 1.75 * 64.4 \frac{lb}{ft^3} * 0.667 ft (3.9 ft)^2 \right) = 575 \text{ lbs} \]

iv. Hydrodynamic Load on Landward Rows of Piles

\[ F_{dyn} = \frac{1}{2} \left( C_d \rho D V^2 A \right) \] (FEMA P-55 2011 – Eq 8.8)

\( F_{dyn} = \) Hydrodynamic load
\( C_d = 1.2 \) for round piles
\( A = \) Surface area perpendicular to flow =\( D d_s \)
\( \rho = \) water density = 1.99 slugs/ft\(^3\)
\[ F_{dyn} = \frac{1}{2} \left( 1.2 \times 1.99 \frac{\text{slugs}}{\text{ft}^3} \times 0.667 \text{ft/s}(5 \text{ ft})(12.7 \frac{\text{ft}}{\text{s}})^2 \right) = 640 \text{ lbs} \]

v. Impact Load from Debris

\[ F_i = W V C_d C_b C_{str} \quad \text{(FEMA P-55 2011 – Eq 8.9)} \]

- \( F_i \) = Impact force acting at the stillwater elevation
- \( W \) = weight of object (assume 1000 lbs)
- \( V \) = water velocity \( = \frac{1}{2} V_{\text{upper}} = 6.35 \text{ ft/s} \)
- \( C_d \) = Depth coefficient = 1 in VE Zones
- \( C_b \) = Blockage coefficient = 1 in first row & 0.6 in second row of piles
- \( C_{str} \) = Building Structure Coefficient = 0.2

First row of piles:

\[ F_i = 1000 \text{ lbs} \times 6.35 \frac{\text{ft}}{\text{s}} \times 1 \times 1 \times 0.2 = 1270 \text{ lbs} \]

Second row of piles:

\[ F_i = 1000 \text{ lbs} \times 6.35 \frac{\text{ft}}{\text{s}} \times 1 \times 0.6 \times 0.2 = 760 \text{ lbs} \]

vi. \( F_{\text{design}} \)

The design load is the greater of the breaking wave load or impact load for the seaward (first) row of piles. \( F_{\text{design}} \) acts at El. +11.1’ for the first row of piles. The greater of the hydrodynamic load or impact load for the landward piles was chosen as \( F_{\text{design}} \) for the second row of piles. The hydrodynamic load acts at El. +8.6’ and the impact load acts at El. +11.1’. \( F_{\text{design}} \) for the second row of piles should be adjusted if the lower load generates a higher moment in the area of concern due to the loads acting at different elevations.

\[ F_{\text{design \ first \ row}} = 1270 \text{ lbf} \]

\[ F_{\text{design \ second \ row}} = 760 \text{ lbf} \]
2) Morison’s Equation

Morison’s Equation Analysis was performed on a 12” round pile (Section VII-2-A), 12” Square pile (Section VII-2-B), and an 8” round pile (Section VII-2-C).

A) 12” ø Pile:

i. **Check for Slenderness:**

If wavelength, $\lambda > 5D$, member is slender and Morison’s Equation is applicable (DNV 1991).

$$\left(\frac{2\pi}{t}\right)^2 = g \left(\frac{2\pi}{\lambda}\right) \tanh \left(\frac{2\pi}{\lambda} h\right)$$  \hspace{1cm} (Dean & Dalrymple 1991)

$t = \text{Wave period} = 5.97 \text{ sec (see Section IV)}$

$g = \text{acceleration due to gravity}$

$\lambda = \text{wave length}$

$h = \text{water depth (use h prior to setup because it is more conservative) = 2.7'}$ (see Section VII-1-A-ii)

$$\left(\frac{2\pi}{5.97} \right)^2 = 32.2 \frac{ft}{s^2} \left(\frac{2\pi}{\lambda}\right) \tanh \left(\frac{2\pi}{\lambda} \cdot 2.7 ft\right)$$

$\lambda = 78 \text{ ft}$

$\lambda = 78 \text{ ft} > 5(1 \text{ ft}) = 5 \text{ ft}$ \hspace{1cm} \checkmark

Pile is slender & Morison’s Equation is valid.

ii. **Scour Analysis:**

$$S = S_{mc} + S_{mw}$$

$S = \text{Scour depth}$

$S_{mc} = \text{Max scour depth due to current}$

$S_{mw} = \text{Max scour depth due to wave action}$

$$S_{mc} = 2K_1 K_2 \left(\frac{b}{h}\right)^{0.65} \left(F_r\right)^{0.43} h$$  \hspace{1cm} (CEM 2003 VI-5-6-b-3-a)

$h = \text{water depth} = 2.7'$

$b = \text{pile width} = 1'$

$F_r = \text{Flow Froude no.} = \frac{U}{(gh)^{0.5}} = 0.377$

$U = \text{mean current velocity mag} = 3.28 \text{ ft/s (per calculation in CEM interactive equations)}$

$K_1 = \text{pile shape factor} = 1$ for cylindrical piles (per CEM)

$K_2 = \text{pile shape factor} = 1$ for cylindrical piles (per CEM)
\[ S_{mw} = 1.3[1 - e^{-0.03(k_c-6)}]D \]  
(CEM 2003 VI-5-6-b-3-a)

\( K_c = \) Keulegan-Carpenter no. = 1.2 (per CEM Chart)
\( D = \) Pile Diameter

Because \( K_c \) is less than 6, assume \( S_{mw} = 0 \) (\( S_{mw} \) is not negative)

\[ S = S_{mc} = 2(1)(1) \left( \frac{1\text{ft}}{2.7\text{ft}} \right)^{0.65} = 0.377^{0.43}2.7\text{ft} = 1.8\text{ft} \]

iii. Check for Wave breaking:

If \( H_{mo} \) is greater than 0.78h (where \( h = \) water depth before scour = \( E_{sw}(\text{no setup}) - G \)), wave breaking occurs and 100-yr stillwater flood elevation plus setup will be used for calculations. Otherwise, 100-yr stillwater not including setup should be used.

\[ H_{mo} = 9.99' > 0.78(h) = 0.78(10.1' - 7.4') \]
\[ 9.99' > 2.1' \]
\[ H_{mo} > 0.78(h) \checkmark \]

Therefore wave breaking and wave setup occur.

iv. Check Wave Theory:

If \( h/\lambda \geq 0.3 \) linear wave equations can be used (DNV 1991).

\[ \frac{h}{\lambda} = \frac{2.7\text{ft}}{78.1\text{ft}} = 0.03 \]

Linear wave theory equations cannot be used – use nonlinear equations.

v. Wave Load on Piles:

Morison’s Equation:

\[ f = f_i + f_d = C_m\rho \frac{2\pi D^2}{4} \frac{dU}{dt} + C_d \frac{1}{2}\rho D U |U| \]

\( f = \) wave force around piles
\( f_i = \) Inertial force per length of pile
\( f_d = \) Drag force per unit length of pile
\( \rho = \) mass density of fluid
D = Pile Diameter
U = horizontal water particle velocity

For linear waves:

\[ U = \frac{HGT}{2\lambda} \frac{\cosh \left( \frac{2\pi}{\lambda} (z + h) \right) \cos \left( \frac{2\pi t}{T} \right)}{\cosh \left( \frac{2\pi}{\lambda} h \right)} \]

\[ \frac{dU}{dt} = \frac{HG\pi}{\lambda} \frac{\cosh \left( \frac{2\pi}{\lambda} (z + h) \right)}{\cosh \left( \frac{2\pi}{\lambda} h \right)} \sin \left( \frac{-2\pi t}{T} \right) \]

For non-linear waves use CEM Tables

\[ C_m = \text{Inertial or mass hydrodynamic force coefficient (1 for round piles (DNV 91))} \]
\[ C_d = \text{Drag hydrodynamic force coefficient (1.1 (DNV 91))} \]
\[ H = \text{depth limited wave height} = 0.78(2.7' + 1' \text{ (wavesetup)} + 1.8' \text{ (scour)}) = 4.3' \]

Waves are non-linear. Interactive tables from CEM Section VI-5-7-b used to solve for maximum force and moment arm (see appendix A: Coastal Engineering Manual calculations – 1’ round pile).

\[ f = 332 \text{ lb} \]

Multiply by 1.5 factor of safety (per CEM guidelines)

\[ f_{\text{design}} = 500 \text{ lb} \]

\( f_{\text{design}} \) acts at 2.54’ above design grade before scour (El. +9.94).
B) 12" Square Pile:

i. Check for Slenderness:

If wavelength, $\lambda > 5D$, member is slender and Morison’s Equation is applicable (DNV 1991).

$$\left(\frac{2\pi}{t}\right)^2 = g \left(\frac{2\pi}{\lambda}\right) \tanh \left(\frac{2\pi}{\lambda} h\right)$$  \hspace{1cm} (Dean & Dalrymple 1991)

$t = \text{Wave period} = 5.97 \text{ sec (see Section IV)}$
$g = \text{acceleration due to gravity}$
$\lambda = \text{wave length}$
$h = \text{water depth (use h prior to setup because it is more conservative)} = 2.7' \text{ (see Section VII-1-A-ii)}$

$$\left(\frac{2\pi}{5.97 s}\right)^2 = 32.2 \frac{ft}{s^2} \left(\frac{2\pi}{\lambda}\right) \tanh \left(\frac{2\pi}{\lambda} 2.7 ft\right)$$

$$\lambda = 78 \text{ ft}$$

$$\lambda = 78 \text{ ft} > 5(1.4 \text{ ft}) = 7 \text{ ft} \quad \checkmark$$

Pile is slender & Morison’s Equation is valid.

ii. Scour Analysis:

$$S = S_{mc} + S_{mw}$$

$S = \text{Scour depth}$
$S_{mc} = \text{Max scour depth due to current}$
$S_{mw} = \text{Max scour depth due to wave action}$

$$S_{mc} = 2K_1K_2 \left(\frac{b}{h}\right)^{0.65} F_r^{0.43} h$$  \hspace{1cm} (CEM 2003 VI-5-6-b-3-a)

$h = \text{water depth} = 2.7'$
$b = \text{pile width} = 1'$
$F_r = \text{Flow Froude no.} = U/(gh)^{0.5} = 0.377$
$U = \text{mean current velocity mag} = 3.28 \text{ ft/s (per calculation in CEM interactive equations)}$
$K_1 = \text{pile shape factor} = 1.1 \text{ (per CEM)}$
$K_2 = \text{Pile shape factor} = 1.24 \text{ (per CEM)}$

$$S_{mw} = 1.3\left[1 - e^{-0.03(K_c-6)}\right]D$$  \hspace{1cm} (CEM 2003 VI-5-6-b-3-a)

$K_c = \text{Keulegan-Carpenter no.} = 1.2 \text{ (per CEM Chart)}$
D = Pile Dimension = 1.4’

Because $K_c$ is less than 6, assume $S_{mw} = 0$ ($S_{mw}$ is not negative)

$$S = S_{mc} = 2(1.1)(1.24)\left(\frac{1\,\text{ft}}{2.7\,\text{ft}}\right)^{0.65}0.377^{0.43}2.7\,\text{ft} = 2.5\,\text{ft}$$

**iii. Check for Wave breaking:**

If $H_{mo}$ is greater than 0.78$h$ (where $h$ = water depth before scour = $E_{sw(no\,setup)} - G$), wave breaking occurs and 100-yr stillwater flood elevation plus setup will be used for calculations. Otherwise, 100-yr stillwater not including setup should be used.

$$H_{mo} = 9.99' > 0.78(h) = 0.78(10.1' - 7.4')$$

$$9.99' > 2.1'$$

$$H_{mo} > 0.78(h) \checkmark$$

Therefore wave breaking and wave setup occur.

**iv. Check Wave Theory:**

If $h/\lambda \geq 0.3$ linear wave equations can be used (DNV 1991).

$$\frac{h}{\lambda} = \frac{2.7\,\text{ft}}{78.1\,\text{ft}} = 0.03$$

Linear wave theory equations cannot be used – use nonlinear equations.

**v. Wave Load on Piles:**

Morison’s Equation:

$$f = f_i + f_d = C_m\rho \frac{2\pi D^2}{4} \frac{dU}{dt} + C_d\frac{1}{2}\rho D U |U|$$

$f$ = wave force around piles
$f_i$ = Inertial force per length of pile
$f_d$ = drag force per unit length of pile
$\rho$ = mass density of fluid
$D$ = Pile Dimension = 1.4’
$U$ = horizontal water particle velocity

For linear waves:
\[ U = \frac{HGT \cosh\left(\frac{2\pi}{\lambda}(z + h)\right)}{2\lambda} \cos\left(\frac{2\pi t}{T}\right) \]

\[ \frac{dU}{dt} = \frac{HG\pi \cosh\left(\frac{2\pi}{\lambda}(z + h)\right)}{\lambda} \cosh\left(\frac{2\pi}{\lambda}h\right) \sin\left(\frac{-2\pi t}{T}\right) \]

For non-linear waves use CEM Tables

\[ C_m = \text{Inertial or mass hydrodynamic force coefficient (1.51 (DNV 91))} \]

\[ C_d = \text{Drag hydrodynamic force coefficient (1.71 (DNV 91))} \]

\[ H = \text{depth limited wave height} = 0.78(2.7' + 1' \text{ (wavesetup)} + 2.5\text{ (scour)}) = 4.8' \]

Waves are non-linear. Interactive tables from CEM Section VI-5-7-b used to solve for maximum force and moment arm (see appendix B: Coastal Engineering Manual calculations – 1' square pile).

\[ f = 907 \text{ lbf} \]

Multiply by 1.5 factor of safety (per CEM guidelines)

\[ f_{\text{design}} = 1365 \text{ lbf} \]

\[ f_{\text{design}} \text{ acts at 2.54'} \text{ above design grade before scour (El. +9.94).} \]
C) 8” ø Pile:

i. Check for Slenderness:

If wavelength, $\lambda > 5D$, member is slender and Morison’s Equation is applicable (DNV 1991).

$$\left( \frac{2\pi}{t} \right)^2 = g \left( \frac{2\pi}{\lambda} \right) \tanh \left( \frac{2\pi}{\lambda} h \right)$$  
(Dean & Dalrymple 1991)

$t = \text{Wave period} = 5.97 \text{ sec (see Section IV)}$
$g = \text{acceleration due to gravity}$
$\lambda = \text{wave length}$
$h = \text{water depth (use h prior to setup because it is more conservative)} = 2.7'$ (see Section VII-1-A-ii)

$$\left( \frac{2\pi}{5.97} \right)^2 = 32.2 \frac{ft}{s^2} \left( \frac{2\pi}{\lambda} \right) \tanh \left( \frac{2\pi}{\lambda} 2.7 ft \right)$$

$$\lambda = 78 \text{ ft}$$

$$\lambda = 78 ft > 5 \left( \frac{5}{3} ft \right) = 3.33 \text{ ft} \quad \checkmark$$

Pile is slender & Morison’s Equation is valid.

ii. Scour Analysis:

$$S = S_{mc} + S_{mw}$$

$S =$ Scour depth
$S_{mc} =$ Max scour depth due to current
$S_{mw} =$ Max scour depth due to wave action

$$S_{mc} = 2K_1K_2 \left( \frac{b}{h} \right)^{0.65} Fr^{0.43} h \quad \text{(CEM 2003 VI-5-6-b-3-a)}$$

$h =$ water depth = 2.7'
$b =$ pile width = 0.667'
$Fr =$ Flow Froude no. = $U/(gh)^{0.5} = 0.377$
$U =$ mean current velocity mag = 3.28 ft/s (per calculation in CEM interactive equations)
$K_1 =$ pile shape factor = 1 (per CEM)
$K_2 =$ Pile shape factor = 1 (per CEM)

$$S_{mw} = 1.3 \left[ 1 - e^{-0.03(kc^{-6})} \right] D \quad \text{(CEM 2003 VI-5-6-b-3-a)}$$
Kc = Keulegan-Carpenter no. = 1.2 (per CEM Chart)
D = Pile Dimension = 0.667’

Because Kc is less than 6, assume Smw = 0 (Smw is not negative)

\[ S = S_{mc} = 2(1)(1)
\left( \frac{1\text{ft}}{2.7\text{ft}} \right)^{0.65} 0.377^{0.43} 2.7\text{ft} = 1.4\text{ ft} \]

iii. Check for Wave breaking:

If \( H_{mo} \) is greater than 0.78h (where h = water depth before scour = E_{sw(no setup)} - G), wave breaking occurs and 100-yr stillwater flood elevation plus setup will be used for calculations. Otherwise, 100-yr stillwater not including setup should be used.

\[ H_{mo} = 9.99’ > 0.78(h) = 0.78(10.1’ - 7.4’) \]
\[ 9.99’ > 2.1’ \]
\[ H_{mo} > 0.78(h) \checkmark \]

Therefore wave breaking and wave setup occur.

iv. Check Wave Theory:

If \( h/\lambda \geq 0.3 \) linear wave equations can be used (DNV 1991).

\[ \frac{h}{\lambda} = \frac{2.7\text{ft}}{78.1\text{ft}} = 0.03 \]

Linear wave theory equations cannot be used – use nonlinear equations.

v. Wave Load on Piles:

Morison’s Equation:

\[ f = f_i + f_d = C_m \rho \frac{2\pi D^2}{4} \frac{dU}{dt} + C_d \frac{1}{2} \rho DU|U| \]

\( f = \) wave force around piles
\( f_i = \) inertial force per length of pile
\( f_d = \) drag force per unit length of pile
\( \rho = \) mass density of fluid
\( D = \) Pile Dimension = 0.667’
\( U = \) horizontal water particle velocity

For linear waves:
\[ U = \frac{HGT}{2\lambda} \frac{\cosh \left( \frac{2\pi}{\lambda} (z + h) \right)}{\cosh \left( \frac{2\pi}{\lambda} h \right)} \cos \left( \frac{2\pi t}{T} \right) \]

\[ \frac{dU}{dt} = \frac{HG\pi}{\lambda} \frac{\cosh \left( \frac{2\pi}{\lambda} (z + h) \right)}{\cosh \left( \frac{2\pi}{\lambda} h \right)} \sin \left( \frac{-2\pi t}{T} \right) \]

For non-linear waves use CEM Tables

\( C_m = \) Inertial or mass hydrodynamic force coefficient (1 (DNV 91))
\( C_d = \) Drag hydrodynamic force coefficient (1.1 (DNV 91))
\( H = \) depth limited wave height = 0.78(2.7'+1' (wavesetup) + 1.4' (scour)) = 4'

Waves are non-linear. Interactive tables from CEM Section VI-5-7-b used to solve for maximum force and moment arm (see appendix C: Coastal Engineering Manual calculations – 8” round pile).

\[ f = 191 \text{ lbf} \]

Multiply by 1.5 factor of safety (per CEM guidelines)

\[ f_{design} = 290 \text{ lbf} \]

\( f_{design} \) acts at 2.54’ above design grade before scour (El. +9.94).
Summary:

The design wave loads acting on the first row and landward rows of piles for each case and each pile type are summarized below.

1) 12” ø Piles

The design wave load for the first row of 12” round piles is 1350 lbf per pile acting at El. +11.1’ when FEMA P-55 calculation method is utilized and 500 lbf per pile acting at El. +9.94’ when the Morison’s Equation is utilized to calculate the load. The FEMA P-55 design load was determined to be the debris impact load. The debris impact load was chosen as the design load because it is higher than the wave breaking load (1090 lbf). The wave breaking load acts at El. +11.1’.

The design wave load for the landward rows of 12” round piles is 1190 lbf per pile acting at El. +8.3’ when FEMA P-55 calculation method is utilized and 500 lbf per pile acting at El. +9.94’ when the Morison’s Equation is utilized to calculate the load. The FEMA P-55 design load was determined to be the hydrodynamic load. The hydrodynamic load was chosen as the design load because it is higher than the debris impact load (810 lbf). The debris impact load acts at El. +11.1’. If the debris impact load causes a greater critical moment due to the loads acting at different elevations it should be used for design.

2) 12” Square Piles

The design wave load for the first row of 12” square piles is 2640 lbf per pile acting at El. +11.1’ when FEMA P-55 calculation method is utilized and 1365 lbf per pile acting at El. +9.94’ when the Morison’s Equation is utilized to calculate the load. The FEMA P-55 design load was determined to be the wave breaking load. The wave breaking load was chosen as the design load because it is higher than the debris impact load (1450 lbf). The debris impact load acts at El. +11.1’.

The design wave load for the landward rows of 12” square piles is 2280 lbf per pile acting at El. +7.9’ when FEMA P-55 calculation method is utilized and 1365 lbf per pile acting at El. +9.94’ when the Morison’s Equation is utilized to calculate the load. The FEMA P-55 design load was determined to be the hydrodynamic load because it is higher than the debris impact load (870 lbf). The debris impact load acts at El. +11.1’. If the debris impact load causes a greater critical moment due to the loads acting at different elevations it should be used for design.

3) 8” ø Piles

The design wave load for the first row of 8” round piles is 1270 lbf per pile acting at El. +11.1’ when FEMA P-55 calculation method is utilized and 290 lbf per pile acting at El. +9.94’ when the Morison’s Equation is utilized to calculate the load. The FEMA P-55 design load was determined to be the debris impact load. The debris impact load was chosen as the design load because it is higher than the wave breaking load (575 lbf). The wave breaking load acts at El. +11.1’.
The design wave load for the first row of 8” round piles is 760 lbf per pile acting at El. +11.1’ when FEMA P-55 calculation method is utilized and 290 lbf per pile acting at El. +9.94’ when the Morison’s Equation is utilized to calculate the load. The FEMA P-55 design load was determined to be the debris impact load. The debris impact load was chosen as the design load because it is higher than the hydrodynamic load (640 lbf). The hydrodynamic load acts at El. +8.6’. If the hydrodynamic load causes a greater critical moment due to the loads acting at different elevations it should be used for design.

A 1.5 factor of safety was used when utilizing the Morison’s Equation as per guidelines outlined in the Coastal Engineering Manual. No factor of safety was used when utilizing the FEMA P-55 calculation method because per the Coastal Construction Manual (FEMA P-55), these loads are already conservative. On several cases listed above, the hydrodynamic load was greater than the wave breaking load. The hydrodynamic load is dependent on the horizontal water velocity. The upper limit of the velocity was used because of the vicinity of the building to the shoreline and because the building is located in a VE-Zone. This may have resulted in a very high water velocity and a conservative hydrodynamic load calculation. All loads are summarized on the attached definition sketch, Wave Load & Scour Diagram.
**WAVE LOAD & SCOUR DIAGRAM**

**WAVE LOAD SUMMARY**

<table>
<thead>
<tr>
<th>CASE</th>
<th>PILE TYPE</th>
<th>F_{\text{Design}} FIRST ROW</th>
<th>F_{\text{Design}} LANDWARD PILES</th>
<th>S</th>
<th>X_{\text{FIRSTROW}}</th>
<th>X_{\text{LANDWARD}}</th>
</tr>
</thead>
<tbody>
<tr>
<td>FEMA 55</td>
<td>ROUND 12&quot;</td>
<td>1350* lbf</td>
<td>1140 lbf</td>
<td>2.0'</td>
<td>3.7'</td>
<td>0.9'</td>
</tr>
<tr>
<td>FEMA 55</td>
<td>SQUARE 12&quot;</td>
<td>2640 lbf</td>
<td>2280 lbf</td>
<td>2.3'</td>
<td>3.7'</td>
<td>0.5'</td>
</tr>
<tr>
<td>FEMA 55</td>
<td>ROUND 8&quot;</td>
<td>1270* lbf</td>
<td>160* lbf</td>
<td>1.8'</td>
<td>3.7'</td>
<td>3.7'</td>
</tr>
<tr>
<td>MORISON'S EQUATION</td>
<td>ROUND 12&quot;</td>
<td>500 lbf</td>
<td>500 lbf</td>
<td>1.8'</td>
<td>2.5'</td>
<td>2.5'</td>
</tr>
<tr>
<td>MORISON'S EQUATION</td>
<td>SQUARE 12&quot;</td>
<td>1365 lbf</td>
<td>1365 lbf</td>
<td>2.5'</td>
<td>2.5'</td>
<td>2.5'</td>
</tr>
<tr>
<td>MORISON'S EQUATION</td>
<td>ROUND 8&quot;</td>
<td>290 lbf</td>
<td>240 lbf</td>
<td>1.4'</td>
<td>2.5'</td>
<td>2.5'</td>
</tr>
</tbody>
</table>

* Load due to debris impact see calculations for loads generated by wave breaking (1st row) or hydrodynamic load (landward piles).

**NOTE:**

F_{\text{design}} FOR THE LANDWARD PILES IS LISTED AS THE HIGHER OF THE HYDRODYNAMIC LOAD AND DEBRIS IMPACT LOAD. THESE LOADS ACT AT DIFFERENT ELEVATIONS. SHOULD THE LOWER LOAD CREATE A HIGHER CRITICAL MOMENT IT SHOULD BE USED FOR DESIGN. ALL THE LOADS AND THE ELEVATIONS THEY ACT AT ARE LISTED IN THE SUMMARY (P. 22-23).
APPENDIX A

CEM Calculation – 12” Round Pile
Equations VI-5-281 to 5-304
Forces on piles using linear wave theory

<table>
<thead>
<tr>
<th>Input Parameter</th>
<th>Value</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pile diameter (D)</td>
<td>1</td>
<td>ft</td>
</tr>
<tr>
<td>Drag coefficient (CD)</td>
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</tr>
<tr>
<td>Inertia coefficient (CM)</td>
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<td></td>
</tr>
<tr>
<td>Wave period (T)</td>
<td>5.97 sec</td>
<td></td>
</tr>
<tr>
<td>Wave height (H)</td>
<td>4.3</td>
<td>ft</td>
</tr>
<tr>
<td>Water depth (d)</td>
<td>3.7</td>
<td>ft</td>
</tr>
<tr>
<td>Scour depth (SM)</td>
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<td>ft</td>
</tr>
<tr>
<td>Deepwater wavelength (Lo)</td>
<td>182.493</td>
<td>ft</td>
</tr>
<tr>
<td>Wavelength (L)</td>
<td>63.7493</td>
<td>ft</td>
</tr>
<tr>
<td>Si</td>
<td>0.505468</td>
<td></td>
</tr>
<tr>
<td>Sd</td>
<td>0.510981</td>
<td></td>
</tr>
<tr>
<td>n</td>
<td>0.958276</td>
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</tr>
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</table>

Maximum Component Forces

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
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<tbody>
<tr>
<td>FIMax</td>
<td>37.7463 lbf</td>
</tr>
<tr>
<td>FDMax</td>
<td>162.688 lbf</td>
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Maximum Component Moments

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<tr>
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</thead>
<tbody>
<tr>
<td>MiMax</td>
<td>70.5944 ft-lbf</td>
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<tr>
<td>MDMax</td>
<td>307.583 ft-lbf</td>
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Maximum Force

<p>| | |</p>
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<tbody>
<tr>
<td>Maximum Force</td>
<td>164.863 lbf</td>
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<tr>
<td>Maximum Moment</td>
<td>311.599 ft-lbf</td>
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</table>

Nonlinear Wave Theory

<p>| | |</p>
<table>
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<th></th>
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<tbody>
<tr>
<td>W parameter</td>
<td>0.211416</td>
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<tr>
<td>D/(gT^2)</td>
<td>0.00322683</td>
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<tr>
<td>H/(gT^2)</td>
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</tr>
<tr>
<td>(\phi_m)</td>
<td>0.255</td>
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<tr>
<td>(\alpha_m)</td>
<td>0.175</td>
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<tr>
<td>Max. force</td>
<td>331.883 lbf</td>
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<tr>
<td>Max. moment</td>
<td>842.724 ft-lbf</td>
</tr>
<tr>
<td>Description</td>
<td>Value</td>
</tr>
<tr>
<td>----------------------------------</td>
<td>-------------</td>
</tr>
<tr>
<td>Moment arm</td>
<td>2.53922</td>
</tr>
<tr>
<td>Modified max. total moment</td>
<td>1440.11 ft-lbf</td>
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</table>
APPENDIX B

CEM Calculation – 12” Square Pile
Equations VI-5-281 to 5-304
Forces on piles using linear wave theory

<table>
<thead>
<tr>
<th>Input Parameter</th>
<th>Value</th>
<th>Units</th>
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<td>Drag coefficient (CD)</td>
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<tr>
<td>Inertia coefficient (CM)</td>
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</tr>
<tr>
<td>Wave period (T)</td>
<td>5.97 sec</td>
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</tr>
<tr>
<td>Wave height (H)</td>
<td>4.8 ft</td>
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<tr>
<td>Water depth (d)</td>
<td>3.7 ft</td>
<td></td>
</tr>
<tr>
<td>Scour depth (SM)</td>
<td>2.5 ft</td>
<td></td>
</tr>
<tr>
<td>Deepwater wavelength (Lo)</td>
<td>182.493</td>
<td>ft</td>
</tr>
<tr>
<td>Wavelength (L)</td>
<td>63.7493</td>
<td>ft</td>
</tr>
<tr>
<td>Si</td>
<td>0.505468</td>
<td></td>
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<tr>
<td>Sd</td>
<td>0.510981</td>
<td></td>
</tr>
<tr>
<td>n</td>
<td>0.958276</td>
<td></td>
</tr>
</tbody>
</table>

Maximum Component Forces

| FiMax            | 126.492 lbf|
| FDMax            | 444.348 lbf|

Maximum Component Moments

| MiMax            | 236.569 ft-lbf|
| MDMax            | 840.098 ft-lbf|

Maximum Force

| Max. force       | 453.222 lbf   |
| Max. moment      | 856.552 ft-lbf|

Nonlinear Wave Theory

<p>| W parameter      | 0.259393   |
| D/(gT²)          | 0.00322683 |
| H/(gT²)          | 0.00418616 |
| φ_m              | 0.255      |
| α_m              | 0.175      |
| Max. force       | 906.471 lbf|
| Max. moment      | 2301.72 ft-lbf|</p>
<table>
<thead>
<tr>
<th>Moment arm</th>
<th>2.53922</th>
</tr>
</thead>
<tbody>
<tr>
<td>Modified max. total moment</td>
<td>4567.9 ft-lbf</td>
</tr>
</tbody>
</table>
APPENDIX C

CEM Calculation – 8” Round Pile
**Equations VI-5-281 to 5-304**

Forces on piles using linear wave theory

<table>
<thead>
<tr>
<th>Input Parameter</th>
<th>Value</th>
<th>Units</th>
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</thead>
<tbody>
<tr>
<td>Pile diameter (D)</td>
<td>0.6667 ft</td>
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</tr>
<tr>
<td>Drag coefficient (CD)</td>
<td>1.1</td>
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<tr>
<td>Inertia coefficient (CM)</td>
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<td></td>
</tr>
<tr>
<td>Wave period (T)</td>
<td>5.97 sec</td>
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</tr>
<tr>
<td>Wave height (H)</td>
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</tr>
<tr>
<td>Water depth (d)</td>
<td>3.7 ft</td>
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</tr>
<tr>
<td>Scour depth (SM)</td>
<td>1.4 ft</td>
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</tr>
<tr>
<td>Deepwater wavelength (Lo)</td>
<td>182.493 ft</td>
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</tr>
<tr>
<td>Wavelength (L)</td>
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</tr>
<tr>
<td>Si</td>
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</tr>
<tr>
<td>Sd</td>
<td>0.510981</td>
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<tr>
<td>n</td>
<td>0.958276</td>
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**Maximum Component Forces**

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<th>Units</th>
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<tr>
<td>Fimax</td>
<td>15.6073 lbf</td>
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<tr>
<td>FDMax</td>
<td>93.8575 lbf</td>
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**Maximum Component Moments**

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<tr>
<td>MiMax</td>
<td>29.1892 ft-lbf</td>
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<tr>
<td>MDMMax</td>
<td>177.45 ft-lbf</td>
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<table>
<thead>
<tr>
<th></th>
<th>Value</th>
<th>Units</th>
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<tbody>
<tr>
<td>Maximum Force</td>
<td>94.4674 lbf</td>
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</tr>
<tr>
<td>Maximum Moment</td>
<td>178.583 ft-lbf</td>
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**Nonlinear Wave Theory**

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<tr>
<td>D/(gT²)</td>
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<tr>
<td>H/(gT²)</td>
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<tr>
<td>φₘ</td>
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</tr>
<tr>
<td>αₘ</td>
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<tr>
<td>Max. force</td>
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<tr>
<td>Max. moment</td>
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</tr>
<tr>
<td>Description</td>
<td>Value</td>
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</tr>
<tr>
<td>---------------------------</td>
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<tr>
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</tr>
<tr>
<td>Modified max. total moment</td>
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