Steel Sheet Piling | RETAINING WALL COMPARISON
TECHNICAL REPORT
Prepared by EIC Group, Inc., LLC
NARRATIVE

Comparison Retaining Wall Design and Cost Study
Steel Sheet Piling vs. Various Walls

Prepared By:

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NARRATIVE

We are pleased to present herein the results of our comparison study of steel sheet piling vs. various wall types for the North American Steel Sheet Piling Association (NASSPA). The purpose of this work was to determine the feasibility of utilizing permanent steel sheeting for retaining walls that traditionally used concrete, slurry, or other materials. The following walls were studied:

- Tied Back Steel Sheet Piling
- Reinforced Concrete Cantilever
- Concrete Modular Unit
- Mechanically Stabilized Earth
- Soldier Pile and Concrete Lagging
- Slurry Wall

A hypothetical retaining wall case study was developed based on conditions that often occur in practical situations. The proposed wall has an exposed face of 19 feet and retains dense fine sand with no water table present. Above the wall, the embankment slopes up at an 18-degree angle. The study assumed the walls were to be built in a cut situation with available space for open excavation. The cost of the cut in front of the wall is common to all cases and not included.

The above listed wall options were designed for the case study. Design criteria are based on AASHTO Standard Specifications for Highway Bridges, 17th Edition, 2002, ASD. No temporary retaining system during construction was assumed for any option. Drawings depicting the proposed configurations were developed along with engineering calculations. The excavation required in front of the walls to obtain the desired cut configuration was not included in the comparison since it was necessary for all options and not dependent on the wall type. A complete listing of quantities is given in the following summary tables.

Costs and construction durations were computed for each option. Reference data was taken from the current edition of “RS Means Heavy Construction Cost Data” and then compared to bid prices for recent NJDOT and NJTA projects for reasonableness. Cost estimates and construction durations are given in the following tables.

The results of the study reveal the following:

- The steel sheet pile option provides a minimum 35% cost savings over other wall type options. It provides a 65% savings over a traditional cast-in-place concrete wall.

- The steel sheet pile option has the shortest construction duration of all options.

It should also be noted that although the modular unit wall option was closest in cost to the sheet pile wall, it is often not feasible for situations where groundwater is present in the retained soil. Therefore, it may not be appropriate in many situations. In summary, the results of the study indicate that for the appropriate site conditions, a permanent steel sheet piling retaining wall is the least costly option over the other walls studied and has a significantly shorter construction duration.
North American Steel Sheet Piling Association

Retaining Wall Study
Calculations

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CHAPTER 1 Summary

Notes:

1. All design performed utilizing the Service Load Method.
2. Designs performed in accordance with the AASHTO Standard Specifications for Highway Bridges, 17th Edition 2002, ASD.

The designs presented herein are conceptual in nature to illustrate and compare construction methods and costs for the various walls studied. They should not be used for actual construction.

1.1 Conceptual Model

Wall Properties

\[ H := 19 \text{-ft} \quad \text{Exposed Wall height} \]
\[ L := 100 \text{-ft} \quad \text{Wall Length} \]

1.1.1 Soil and Site Parameters

Retained Soil - Existing above Excavation Level

\[ \gamma := 120 \text{-pcf} \quad \text{Soil Density} \]
\[ \phi_f := 30 \text{-deg} \quad \text{Angle of internal friction} \]
\[ \delta := 0 \quad \text{Angle of friction between soil and wall or per AASHTO table 5.5.2B} \]
\[ \beta := 90 \text{-deg} \quad \text{Batter of Wall, where 90 degrees is vertical except at Concrete Modular Units} \]
\[ \alpha := 18 \text{-deg} \quad \text{Slope of Retained Soil (approx 1:3 slope)} \]
\[ c := 0 \quad \text{Soil Cohesion} \]

Foundation Soil - Below Excavation Level - same as Retained Soil above Excavation Level

Determine Coulomb's Passive Earth Pressure Coefficient, $K_p$

$$K_p := \frac{(\sin(\beta - \phi))}{(\sin(\beta))^2 \cdot \sin(\beta + \delta) \left[ 1 - \left( \sin(\phi f + \delta) \cdot \frac{\sin(\phi f + \alpha)}{\sin(\beta + \delta) \cdot \sin(\beta + \alpha)} \right)^{0.5} \right]^2}$$

$K_p = 5.33$ Coulomb's passive earth pressure coefficient

Determine Coulomb's Active Earth Pressure Coefficient, $K_a$

$$K_a := \frac{\sin(\beta + \phi f)}{(\sin(\beta))^2 \cdot \sin(\beta - \delta) \left[ 1 + \left( \sin(\phi f + \delta) \cdot \frac{\sin(\phi f - \alpha)}{\sin(\beta - \delta) \cdot \sin(\alpha + \beta)} \right) \right]^2}$$

$K_a = 0.424$ Coulomb's active earth pressure coefficient for fill material

Where:

$\beta = 90\,\text{deg}$ $\delta = 0\,\text{deg}$

$\phi f = 30\,\text{deg}$ $\alpha = 18\,\text{deg}$
## Section 1.2 Summary of Costs and Construction Time

### All Walls

<table>
<thead>
<tr>
<th>Retaining Wall Type</th>
<th>Construction Duration (Days)</th>
<th>Total Cost for 100 ft. Wall</th>
<th>Cost per Linear Ft.</th>
<th>Cost per Square Ft.</th>
</tr>
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<tbody>
<tr>
<td>Grouted Anchor Steel Sheet Pile Wall</td>
<td>13</td>
<td>$90,607</td>
<td>$906.07</td>
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<tr>
<td>Cast-In-Place Reinforced Concrete Wall</td>
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<td>$258,572</td>
<td>$2,585.72</td>
<td>$136.09</td>
</tr>
<tr>
<td>Concrete Modular Unit Gravity Wall</td>
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<td>$144,741</td>
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<tr>
<td>Mechanically Stabilized Earth Wall</td>
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<td>$181,593</td>
<td>$1,815.93</td>
<td>$95.58</td>
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<tr>
<td>Soldier Pile and Lagging Wall</td>
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<td>$171,856</td>
<td>$1,718.56</td>
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<td>Slurry Wall*</td>
<td>64</td>
<td>$400,145</td>
<td>$4,001.45</td>
<td>$210.60</td>
</tr>
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</table>

*Concept model - not typical application for slurry wall but included in study to give comprehensive range of options
NORTH AMERICAN STEEL SHEET PILING ASSOCIATION  
RETAINING WALL STUDY  
Section 1.3 Summary of Costs and Construction Time, Each Wall

Grouted Anchor Steel Sheet Pile Wall

<table>
<thead>
<tr>
<th>Pay Item No.</th>
<th>Item</th>
<th>Unit</th>
<th>Quantity</th>
<th>Daily Output (unit/day)</th>
<th>Time (day)</th>
<th>Unit Cost</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>Grouted Anchors - 1&quot; Dia</td>
<td>LF</td>
<td>286.0</td>
<td>120</td>
<td>3</td>
<td>$20.20</td>
<td>$5,777.20</td>
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<td>02</td>
<td>Sheet piling, 19 ft deep excavation</td>
<td>TN</td>
<td>29.0</td>
<td>12.95</td>
<td>3</td>
<td>$1,950.00</td>
<td>$56,550.00</td>
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<tr>
<td>03</td>
<td>Wales, connections &amp; struts</td>
<td>TN</td>
<td>1.5</td>
<td>NA</td>
<td>-</td>
<td>$300.00</td>
<td>$459.00</td>
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<td>04</td>
<td>Anchors</td>
<td>TN</td>
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<td>-</td>
<td>$2,700.00</td>
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<tr>
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<td>TCY</td>
<td>211.0</td>
<td>670</td>
<td>-</td>
<td>$2.02</td>
<td>$426.22</td>
</tr>
<tr>
<td>08</td>
<td>Borrow loading</td>
<td>TCY</td>
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<td>NA</td>
<td>-</td>
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<td>-</td>
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<td>-</td>
<td>$0.23</td>
<td>-</td>
</tr>
<tr>
<td>10</td>
<td>Compaction, walk behind vibrating plate</td>
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<td>$0.78</td>
<td>$164.58</td>
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<tr>
<td>12</td>
<td>Excavation, trench, common earth</td>
<td>BCY</td>
<td>211.0</td>
<td>480</td>
<td>1</td>
<td>$3.86</td>
<td>$814.46</td>
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<tr>
<td>15</td>
<td>Driven piles, complete pile driving setup</td>
<td>EA</td>
<td>1.0</td>
<td>0.27</td>
<td>4</td>
<td>$22,000.00</td>
<td>$22,000.00</td>
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<tr>
<td>16</td>
<td>Geotextile for subsurface drainage</td>
<td>SY</td>
<td>244.4</td>
<td>1600</td>
<td>1</td>
<td>$2.18</td>
<td>$532.79</td>
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**Totals**: 13 $90,607.21  
Cost per LF $906.07  
Cost per SF $47.69
## Cast-In-Place Reinforced Concrete Wall

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<th>Pay Item No.</th>
<th>Item</th>
<th>Unit</th>
<th>Quantity</th>
<th>Daily Output (unit/day)</th>
<th>Time (day)</th>
<th>Unit Cost</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
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<td>07</td>
<td>Backfill structural</td>
<td>LCY</td>
<td>4,100.0</td>
<td>670</td>
<td>6</td>
<td>$ 2.02</td>
<td>$ 8,282.00</td>
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<tr>
<td>08</td>
<td>Borrow loading</td>
<td>BCY</td>
<td>4,100.0</td>
<td>NA</td>
<td>-</td>
<td>$ 13.86</td>
<td>$ 56,826.00</td>
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<tr>
<td>09</td>
<td>Compaction, riding, vibrating roller</td>
<td>ECY</td>
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<td>5200</td>
<td>1</td>
<td>$ 0.23</td>
<td>$ 943.00</td>
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<tr>
<td>10</td>
<td>Compaction, walk behind vibrating plate</td>
<td>ECY</td>
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<td>560</td>
<td>1</td>
<td>$ 0.78</td>
<td>$ 112.32</td>
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<td>12</td>
<td>Excavation, trench, common earth</td>
<td>BCY</td>
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<td>480</td>
<td>9</td>
<td>$ 3.86</td>
<td>$ 15,826.00</td>
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<td>3</td>
<td>$ 2.80</td>
<td>$ 2,660.00</td>
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<td>19</td>
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<td>100.0</td>
<td>400</td>
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<td>$ 555.00</td>
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<td>11</td>
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<td>$ 35,045.00</td>
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<td>21</td>
<td>Reinforcing steel, A615 Gr 60</td>
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<td>10</td>
<td>$ 2,825.00</td>
<td>$ 60,172.50</td>
</tr>
<tr>
<td>23</td>
<td>Concrete, ready mix</td>
<td>CY</td>
<td>540.0</td>
<td>NA</td>
<td>-</td>
<td>$ 114.00</td>
<td>$ 61,560.00</td>
</tr>
<tr>
<td>25</td>
<td>Placing concrete, footings</td>
<td>CY</td>
<td>330.0</td>
<td>150</td>
<td>3</td>
<td>$ 28.00</td>
<td>$ 9,240.00</td>
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<tr>
<td>26</td>
<td>Placing concrete, walls</td>
<td>CY</td>
<td>210.0</td>
<td>120</td>
<td>2</td>
<td>$ 35.00</td>
<td>$ 7,350.00</td>
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<tr>
<td>Totals</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>47</td>
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<td></td>
<td></td>
<td></td>
<td>Cost per LF $ 2,585.72</td>
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</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Cost per SF $ 136.09</td>
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</tr>
</tbody>
</table>
## Section 1.3 Summary of Costs and Construction Time, Each Wall

### Concrete Modular Unit Gravity Wall

<table>
<thead>
<tr>
<th>Pay Item No.</th>
<th>Item</th>
<th>Unit</th>
<th>Quantity</th>
<th>Daily Output (unit/day)</th>
<th>Time (day)</th>
<th>Unit Cost</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>07</td>
<td>Backfill structural</td>
<td>LCY</td>
<td>2,791.0</td>
<td>670</td>
<td>5</td>
<td>$2.02</td>
<td>$5,637.82</td>
</tr>
<tr>
<td>08</td>
<td>Borrow loading</td>
<td>BCY</td>
<td>2,724.0</td>
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<td>-</td>
<td>$13.86</td>
<td>$37,754.64</td>
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<tr>
<td>09</td>
<td>Compaction, riding, vibrating roller</td>
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<td>Compaction, walk behind, vibrating plate</td>
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<td>$0.78</td>
<td>$91.26</td>
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<tr>
<td>12</td>
<td>Excavation, trench, common earth</td>
<td>BCY</td>
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<td>7</td>
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<td>300.0</td>
<td>1600</td>
<td>1</td>
<td>$2.18</td>
<td>$654.00</td>
</tr>
<tr>
<td>18</td>
<td>Forms in place, footing</td>
<td>SFCA</td>
<td>400.0</td>
<td>440</td>
<td>1</td>
<td>$2.80</td>
<td>$1,120.00</td>
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<td>TN</td>
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<td>$2,825.00</td>
<td>$60,737.50</td>
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<td>23</td>
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<td>CY</td>
<td>146.5</td>
<td>NA</td>
<td>-</td>
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<td>24</td>
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<td>120</td>
<td>2</td>
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<tr>
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<td>Placing concrete, footings</td>
<td>CY</td>
<td>18.5</td>
<td>150</td>
<td>1</td>
<td>$28.00</td>
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<tr>
<td>27</td>
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<td>128.0</td>
<td>95</td>
<td>2</td>
<td>$53.50</td>
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</table>

**Totals**: 31 $144,740.58

Cost per LF $1,447.41

Cost per SF $76.18
# NORTH AMERICAN STEEL SHEET PILING ASSOCIATION
## RETAINING WALL STUDY
### Section 1.3 Summary of Costs and Construction Time, Each Wall

#### Mechanically Stabilized Earth Wall

<table>
<thead>
<tr>
<th>Pay Item No.</th>
<th>Item</th>
<th>Unit</th>
<th>Quantity</th>
<th>Daily Output (unit/day)</th>
<th>Time (day)</th>
<th>Unit Cost</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>07</td>
<td>Backfill structural 105 H.P., 150 ft. haul, sand &amp; gravel</td>
<td>LCY</td>
<td>3,593.0</td>
<td>670</td>
<td>6</td>
<td>2.02</td>
<td>7,257.86</td>
</tr>
<tr>
<td>08</td>
<td>Borrow loading Select granular fill</td>
<td>BCY</td>
<td>3,593.0</td>
<td>NA</td>
<td>-</td>
<td>13.86</td>
<td>49,798.98</td>
</tr>
<tr>
<td>09</td>
<td>Compaction, riding, vibrating roller 12 in. lift, 2 passes</td>
<td>ECY</td>
<td>3,593.0</td>
<td>5200</td>
<td>1</td>
<td>0.23</td>
<td>826.39</td>
</tr>
<tr>
<td>10</td>
<td>Compaction, walk behind, vibrating plate 12 in. lift, 2 passes</td>
<td>ECY</td>
<td>117.0</td>
<td>560</td>
<td>1</td>
<td>0.78</td>
<td>91.26</td>
</tr>
<tr>
<td>12</td>
<td>Excavation, trench, common earth 14 ft to 20 ft deep, 1.5 cy hydraulic backhoe</td>
<td>BCY</td>
<td>3,593.0</td>
<td>480</td>
<td>8</td>
<td>3.86</td>
<td>13,868.98</td>
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<tr>
<td>16</td>
<td>Geotextile for subsurface drainage Fabric, laid in trench, adverse conditions</td>
<td>SY</td>
<td>438.9</td>
<td>1600</td>
<td>1</td>
<td>2.18</td>
<td>956.80</td>
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<tr>
<td>21</td>
<td>Reinforcing steel, A615 Gr 60 10 - 50 ton job, # 3 to # 7 bars</td>
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<td>2.1</td>
<td>4</td>
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<td>20,057.50</td>
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<tr>
<td>22</td>
<td>Welded wire fabric 6x6, W4xW4, 58psf/csf</td>
<td>CSF</td>
<td>193.0</td>
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<td>8</td>
<td>94.00</td>
<td>18,142.00</td>
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<tr>
<td>23</td>
<td>Concrete, ready mix Normal weight, 3500 psi</td>
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<td>11.1</td>
<td>NA</td>
<td>-</td>
<td>114.00</td>
<td>1,265.40</td>
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<tr>
<td>25</td>
<td>Placing concrete, footings Continuous, shallow pumped</td>
<td>CY</td>
<td>600.0</td>
<td>150</td>
<td>4</td>
<td>28.00</td>
<td>16,800.00</td>
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<tr>
<td>29</td>
<td>Precast concrete wall panels 10 in. thick</td>
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<td>2</td>
<td>22.68</td>
<td>47,628.00</td>
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<tr>
<td>30</td>
<td>Galvanizing steel in shop 1 ton to 20 tons</td>
<td>TN</td>
<td>5.6</td>
<td>NA</td>
<td>-</td>
<td>875.00</td>
<td>4,900.00</td>
</tr>
</tbody>
</table>

**Totals** 35 $ 181,593.17

Cost per LF $ 1,815.93

Cost per SF $ 95.58
## Soldier Pile and Lagging Wall

<table>
<thead>
<tr>
<th>Pay Item No.</th>
<th>Item</th>
<th>Unit</th>
<th>Quantity</th>
<th>Daily Output (unit/day)</th>
<th>Time (day)</th>
<th>Unit Cost</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>05</td>
<td>Grouted Anchors 1&quot; dia</td>
<td>LF</td>
<td>350.0</td>
<td>120</td>
<td>3</td>
<td>20.20</td>
<td>7,070.00</td>
</tr>
<tr>
<td>04</td>
<td>Anchors</td>
<td>TN</td>
<td>0.5</td>
<td>NA</td>
<td>-</td>
<td>2,700.00</td>
<td>1,269.00</td>
</tr>
<tr>
<td>07</td>
<td>Backfill structural</td>
<td>LCY</td>
<td>2,266.0</td>
<td>670</td>
<td>4</td>
<td>2.02</td>
<td>4,577.32</td>
</tr>
<tr>
<td>08</td>
<td>Borrow loading</td>
<td>BCY</td>
<td>2,266.0</td>
<td>NA</td>
<td>-</td>
<td>13.86</td>
<td>31,406.76</td>
</tr>
<tr>
<td>09</td>
<td>Compaction, riding, vibrating roller</td>
<td>ELY</td>
<td>2,266.0</td>
<td>5200</td>
<td>1</td>
<td>0.23</td>
<td>521.18</td>
</tr>
<tr>
<td>10</td>
<td>Compaction, walk behind vibrating plate</td>
<td>ELY</td>
<td>106.0</td>
<td>560</td>
<td>1</td>
<td>0.78</td>
<td>82.68</td>
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<tr>
<td>11</td>
<td>Excavation, trench, common earth</td>
<td>BCY</td>
<td>968.0</td>
<td>600</td>
<td>2</td>
<td>3.10</td>
<td>3,000.80</td>
</tr>
<tr>
<td>12</td>
<td>Excavation, trench, common earth</td>
<td>BCY</td>
<td>1,298.0</td>
<td>480</td>
<td>3</td>
<td>3.86</td>
<td>5,010.28</td>
</tr>
<tr>
<td>13</td>
<td>Driven piles, complete pile driving setup</td>
<td>VLF</td>
<td>420.0</td>
<td>510</td>
<td>1</td>
<td>76.50</td>
<td>32,130.00</td>
</tr>
<tr>
<td>15</td>
<td>Driven piles, complete pile driving setup</td>
<td>EA</td>
<td>1.0</td>
<td>0.27</td>
<td>4</td>
<td>22,000.00</td>
<td>22,000.00</td>
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<tr>
<td>16</td>
<td>Geotextile for subsurface drainage</td>
<td>SY</td>
<td>233.3</td>
<td>1600</td>
<td>1</td>
<td>2.18</td>
<td>508.59</td>
</tr>
<tr>
<td>21</td>
<td>Reinforcing steel, A615 Gr 60</td>
<td>TN</td>
<td>7.5</td>
<td>2.1</td>
<td>4</td>
<td>2,825.00</td>
<td>21,187.50</td>
</tr>
<tr>
<td>29</td>
<td>Precast concrete wall panels</td>
<td>SF</td>
<td>1,900.0</td>
<td>1550</td>
<td>2</td>
<td>22.68</td>
<td>43,092.00</td>
</tr>
</tbody>
</table>

**Totals** 26 $171,856.11

Cost Per LF $1,718.56

Cost Per SF $90.45
### Slurry Wall

<table>
<thead>
<tr>
<th>Pay Item No.</th>
<th>Item Description</th>
<th>Unit</th>
<th>Quantity</th>
<th>Daily Output (unit/day)</th>
<th>Time (day)</th>
<th>Unit Cost</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>07</td>
<td>Backfill structural 105 H.P., 150 ft. haul, sand &amp; gravel</td>
<td>LCY</td>
<td>515.9</td>
<td>670</td>
<td>1</td>
<td>$2.02</td>
<td>$1,042.12</td>
</tr>
<tr>
<td>08</td>
<td>Borrow loading Select granular fill</td>
<td>BCY</td>
<td>515.9</td>
<td>NA</td>
<td>-</td>
<td>$13.86</td>
<td>$7,150.37</td>
</tr>
<tr>
<td>10</td>
<td>Compaction, walk behind vibrating plate 12 in. lift, 2 passes</td>
<td>ECY</td>
<td>515.9</td>
<td>560</td>
<td>1</td>
<td>$0.78</td>
<td>$402.40</td>
</tr>
<tr>
<td>12</td>
<td>Evacuation, trench, common earth 14 ft to 20 ft deep, 1.5 cy hydraulic backhoe</td>
<td>BCY</td>
<td>515.9</td>
<td>480</td>
<td>2</td>
<td>$3.86</td>
<td>$1,991.37</td>
</tr>
<tr>
<td>16</td>
<td>Geotextile for subsurface drainage Fabric, laid in trench, adverse conditions</td>
<td>SY</td>
<td>288.9</td>
<td>1600</td>
<td>1</td>
<td>$2.18</td>
<td>$629.80</td>
</tr>
<tr>
<td>17</td>
<td>Slurry Trench, excavated in wet soils Backfilled w/3ksi concrete, no reinforcement</td>
<td>CF</td>
<td>11,691.0</td>
<td>333</td>
<td>36</td>
<td>$23.50</td>
<td>$274,738.50</td>
</tr>
<tr>
<td>20</td>
<td>Steel framed plywood 16ft to 20ft high</td>
<td>SFCA</td>
<td>2,000.0</td>
<td>400</td>
<td>5</td>
<td>$8.15</td>
<td>$16,300.00</td>
</tr>
<tr>
<td>21</td>
<td>Reinforcing steel, A615 Gr 60 10 - 50 ton job, # 3 to # 7 bars</td>
<td>TN</td>
<td>32.7</td>
<td>2.1</td>
<td>17</td>
<td>$2,825.00</td>
<td>$92,377.50</td>
</tr>
<tr>
<td>23</td>
<td>Concrete, ready mix Normal weight, 3500 psi</td>
<td>CY</td>
<td>37.0</td>
<td>NA</td>
<td>-</td>
<td>$114.00</td>
<td>$4,218.00</td>
</tr>
<tr>
<td>26</td>
<td>Placing concrete, walls 15 in thk, pumped</td>
<td>CY</td>
<td>37.0</td>
<td>120</td>
<td>1</td>
<td>$35.00</td>
<td>$1,295.00</td>
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</table>

**Totals**

<table>
<thead>
<tr>
<th>Pay Item</th>
<th>Unit Quantity</th>
<th>Daily Output (unit/day)</th>
<th>Time (day)</th>
<th>Unit Cost</th>
<th>Cost</th>
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</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>64</td>
</tr>
</tbody>
</table>

Cost Per LF $4,001.45

9 of 78
CHAPTER 2 TIED BACK STEEL SHEET PILE WALL

2.1 Design Calculations

Wall Properties

\[ H := 19\text{-ft} \quad \text{Exposed Wall height} \]
\[ L := 100\text{-ft} \quad \text{Wall Length} \]

Soil Properties

Retained Soil

\[ \gamma := 120\text{-pcf} \quad \text{Soil Density} \]
\[ \phi_f := 30\text{-deg} \quad \text{Angle of internal friction} \]
\[ \delta := 0 \quad \text{Angle of friction between soil and wall} \]
\[ \beta := 90\text{-deg} \quad \text{Batter of Wall, where 90 degrees is vertical} \]
\[ \alpha := 18\text{-deg} \quad \text{Slope of Retained Soil} \]
\[ c := 0 \quad \text{Soil Cohesion} \]
Determine Coulomb's Passive Earth Pressure Coefficient, Kp

\[
K_p := \frac{(\sin(\beta - \phi_f))^2}{\sin(\beta)^2 \cdot \sin(\beta + \delta) \cdot \left[ 1 - \left( \frac{\sin(\phi_f + \delta) \cdot \sin(\phi_f + \alpha)}{\sin(\beta + \delta) \cdot \sin(\beta + \alpha)} \right)^{0.5} \right]^2}
\]

Kp = 5.33  Coulomb's passive earth pressure coefficient

Determine Coulomb's Active Earth Pressure Coefficient, K_a

\[
K_a := \frac{\sin(\beta + \phi_f)^2}{\sin(\beta)^2 \cdot \sin(\beta - \delta) \cdot \left[ 1 + \sqrt{\left( \frac{\sin(\phi_f + \delta) \cdot \sin(\phi_f - \alpha)}{\sin(\beta - \delta) \cdot \sin(\alpha + \beta)} \right)^2} \right]^2}
\]

K_a = 0.424  Coulomb's active earth pressure coefficient

Where:

\[
\begin{align*}
\beta &= 90 \cdot \text{deg} \\
\delta &= 0 \cdot \text{deg} \\
\phi_f &= 30 \cdot \text{deg} \\
\alpha &= 18 \cdot \text{deg}
\end{align*}
\]

See program results on pp 18 - 21.
2.1.1 Sheet Pile Design

Determine required Section Modulus

\[ M := 30.91 \text{-kip-ft} \quad \text{Maximum Moment from Computer Output} \]
\[ \text{(See Section 2.1.4)} \]

Use A572 Steel
\[ F_y := 50 \text{-ksi} \]

\[ F_b := 0.55 \cdot F_y \quad F_b = 27 \text{ ksi} \quad \text{Allowable Stress AASHTO table 10.32.1A} \]

\[ S_x := \frac{M}{F_b} \quad S_x = 13.74 \text{in}^3 \quad \text{Required Section Modulus} \]

Use
\[ S_x := 22.3 \text{in}^3 \]

Consider Deflection of Sheet Pile

\[ \Delta_{\text{max}} := \frac{H}{360} \quad \Delta_{\text{max}} = 0.63 \text{-in} \quad \text{Allowable deflection} \]

Where:
\[ H = 19 \text{-ft} \quad \text{Exposed Wall Height} \]

\[ \Delta := 0.70 \text{-in} \quad \text{with} \quad I := 132.8 \text{-in}^4 \quad \text{From Computer Output Say OK} \]
2.1.2 Grouted Soil Anchors

Fa = Force per unit length of wall = 4.09 k/ft @ 8’ spacing = 32.7 k

AASHTO Table 5.7.6.2 A

\[ F_u = \text{Presumptive Ultimate Load Transfer in Soil} - 13 \text{ k/lf in Dense Sand} \]

Min Length of Grouted Anchor = \( l_g = \frac{(Fa \cdot SF)}{F_u} \)

SF = Safety Factor = 2.50

\[ l_g = \frac{(32.7 \cdot 2.50)}{13.0} = 6.3' \text{ use 6.5' min} \]

Anchor Steel - ASTM722 Grade 150 \( \text{fa} = 0.55 \times 150 = 82.5 \text{KSI} \)

\[ \text{Asmin} = \frac{32.7}{82.5} = 0.40 \text{ in}^2 \text{ use 1''Ø Rod min} \]

Assume no corrosion protection required.
2.1.3 Waler Design

\[
\begin{align*}
  w &= 5.0 \text{ kip ft} \quad \text{Uniform Load on waler} \\
  s &= 8\text{ ft} \quad \text{Tie Rod spacing} \\
  \frac{M_w}{8} &= \frac{w \cdot s^2}{8} \quad M_w = 40.0 \text{ kip ft} \quad \text{Maximum moment} \\
  \frac{V_w}{2} &= \frac{1}{2} \cdot w \cdot s \quad V_w = 20.0 \text{ kip} \quad \text{Maximum Shear} \\
  \frac{S_w}{0.55 \cdot 50 \cdot \text{ksi}} &= \frac{M_w}{0.55 \cdot 50 \cdot \text{ksi}} \quad S_w = 17.5 \text{ in}^3 \quad \text{Required section Modulus} \\
  \frac{A_w}{0.4 \cdot 50 \cdot \text{ksi}} &= \frac{V_w}{0.4 \cdot 50 \cdot \text{ksi}} \quad A_w = 1.0 \text{ in}^2 \quad \text{Required Web Area} \\
  \text{Use 2-C10x15.3} \quad S_w &= 2 \cdot 13.5 \cdot \text{in}^3 \quad S_w = 27.0 \cdot \text{in}^3 \\
  &\quad A_w = 2 \cdot 10 \cdot 0.240 \cdot \text{in}^2 \quad A_w = 4.8 \cdot \text{in}^2 \quad \text{OK}
\end{align*}
\]
### Input Data

- Depth Of Excavation = 19.00 ft
- Depth Of Active Water = 100.00 ft
- Water Density = 62.43 pcf
- Surcharge = 0.0 psf
- Depth Of Passive Water = 100.00 ft
- Minimum Fluid Density = 31.82 pcf
- Slope (active) = 18.0 degrees

#### Soil Profile

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Soil Name</th>
<th>γ (pcf)</th>
<th>γ’ (pcf)</th>
<th>C (psf)</th>
<th>C_a (psf)</th>
<th>φ (°)</th>
<th>δ (°)</th>
<th>K_a</th>
<th>K_sc</th>
<th>K_p</th>
<th>K_cc</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00</td>
<td>Loose Coarse Sand</td>
<td>106.28</td>
<td>68.73</td>
<td>0.0</td>
<td>0.0</td>
<td>30.0</td>
<td>0.0</td>
<td>0.42</td>
<td>0.0</td>
<td>5.33</td>
<td>0.0</td>
</tr>
</tbody>
</table>

#### Solution

- Sheet Name
  - l (in/ft) = 156.90
  - E (psi) = 3.04E+07
  - k (in²/ft) = 23.20
  - f (psi) = 27000.0
  - Maximum Bending Moment (ft·lb/ft) = 52137.8
  - Upstand (ft) = 0.00
  - Toe (ft) = 10.67
  - Pile Length (ft) = 29.67

#### Load Model: Area Distribution

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Type</th>
<th>Linear Load (lb/ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.00</td>
<td>Water</td>
<td>5103.4</td>
</tr>
</tbody>
</table>

#### Maxima

- Bending Moment: 30907.5 ft·lb/ft, 15.11 ft
- Deflection: 0.7 in, 16.00 ft
- Pressure: 848.1 psf, 19.00 ft
- Shear Force: 4743.0 lb/ft, 4.00 ft
2.2 Quantity Calculations

2.2.1 Sheet Pile Quantity

**Sheet Piling (Pay Item 02)**

\[ L_p := 30\text{-ft} \quad \text{Length of Sheet Pile} \]

\[ H = 19\text{-ft} \quad \text{Height of Retained Soil} \]

Proposed Sheet Pile Properties

\[ W := 30.3\text{-in} \quad \text{Width} \]

\[ H_t := 13.52\text{-in} \quad \text{Height} \]

\[ w_t = 19.31\text{psf} \]

\[ S_x := 23.2\frac{\text{in}^3}{\text{ft}} \quad \text{Section Modulus} \]

\[ I_x := 156.9\frac{\text{in}^4}{\text{ft}} \quad \text{Moment of Inertia} \]

**Total Weight of Sheet Piling (Pay Item 02)**

\[ W_t := L \cdot L_p \cdot w_t \] \[ W_t := 29.0 \text{ Ton} \]

Where:

\[ L = 100\text{-ft} \quad \text{Wall Length} \]

\[ L_p = 30\text{-ft} \quad \text{Pile Length} \]

\[ w_t = 19.31 \text{ psf} \quad \text{Pile Weight/sf} \]
2.2.2 Grouted Anchor and Tie Rod Quantity

Grouted Anchor (Pay Item 01)

12 Spaces @ 8' Spacing + 2' @ Each end = 100'

Install 13 Units @ (15' + 7') = 286LF

Anchor Rod Quantity (Pay Item 04)

\[ D := 1\text{-in} \quad \text{Rod Diameter} \]
\[ ab = 0.85 \text{ in}^2 \quad \text{Area of Rod} \]
\[ wb := 3.01 \frac{\text{lbf}}{\text{ft}} \quad \text{Weight of Rod} \]
\[ Lt := 22\text{-ft} \quad \text{Length of Rod} \]

Determine Number of Rods

\[ N := \frac{L}{s} + 1 \quad N = 13.5 \quad \text{Use} \quad N := 13 \quad \text{Rods} \]

Where: \( L = 100\text{-ft} \quad \text{Wall Length} \)

Determine Total Rod Weight (Pay Item 04)

\[ Wt := N\cdot Lt\cdot wb \quad Wt = 0.43 \text{ Ton} \]

2.2.3 Waler Quantity (Pay Item 03)

Use 2-C10x15.3 Walers

\[ w := 15.3 \frac{\text{lbf}}{\text{ft}} \quad \text{Weight of each waler} \]

Determine Total Weight of Walers (Pay Item 03)

\[ Wt := 2\cdot w\cdot L \quad Wt = 1.53 \text{ Ton} \]

Where: \( L = 100\text{-ft} \quad \text{Wall Length} \)
2.2.4 Excavation and Backfill

Refer to Figure 2-2

Additional Excavation for drainage stone
\[ A_3 = 3 \times 19 = 57 \text{ft}^2 \] Cross-sectional area of drainage stone
\[ V_d := A_3 \cdot L \quad V_d = 211 \text{yd}^3 \] Volume of stone for drainage behind wall

**Total Excavation**
\[ V := V + V_d \quad V = 500 \text{yd}^3 \]

**Excavation Related Pay Items**

- Item 07 - Backfill Structural \[ V = 0 \text{yd}^3 \]
- Item 08 - Select Granular Fill \[ V = 211 \text{yd}^3 \]
- Item 09 - Compaction, Roller \[ V = 0 \text{yd}^3 \]
- Item 10 - Compaction, Plate \[ V_{10} := 3 \cdot \text{ft} \cdot 19 \cdot \text{ft} \cdot L \quad V_{10} = 211 \text{yd}^3 \]
- Item 12 - Excavation \[ V = 211 \text{yd}^3 \]
- Item 16 - Geotextile \[ A_{16} := (3 \cdot \text{ft} + 19 \cdot \text{ft}) \cdot L \quad A_{16} = 244.4 \text{yd}^2 \]
Figure 2-1
Grouted Anchor Configuration
Figure 2-2
Excavation and Backfill Diagram
CHAPTER 3 CAST-IN-PLACE REINFORCED CONCRETE WALL

3.1 Design Calculations, Refer to Figures 3-1 & 3-2

WALL HEIGHT (TOP OF WALL TO BOTTOM OF FOOTING)= 26.00 FT
ANGLE OF INCLINED BACKFILL= 18 DEGREES
ANGLE OF INTERNAL FRICTION= 30 DEGREES
UNIT WEIGHT OF SOIL= 120 PCF

3.1.1 Stability

VERTICAL LOADS

<table>
<thead>
<tr>
<th>PART</th>
<th>WIDTH (FT)</th>
<th>HEIGHT (FT)</th>
<th>AREA (FT^2)</th>
<th>UNIT WT. (KCF)</th>
<th>FORCE (K/FT)</th>
<th>ARM (FT)</th>
<th>MOMENT (K FT/FT)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.75</td>
<td>21.00</td>
<td>36.75</td>
<td>0.150</td>
<td>5.51</td>
<td>5.63</td>
<td>31.01</td>
</tr>
<tr>
<td>2</td>
<td>2.25</td>
<td>18.00</td>
<td>20.25</td>
<td>0.150</td>
<td>3.04</td>
<td>7.25</td>
<td>22.02</td>
</tr>
<tr>
<td>3</td>
<td>6.50</td>
<td>5.00</td>
<td>32.50</td>
<td>0.150</td>
<td>4.88</td>
<td>3.25</td>
<td>15.84</td>
</tr>
<tr>
<td>4</td>
<td>12.50</td>
<td>4.50</td>
<td>56.25</td>
<td>0.150</td>
<td>8.44</td>
<td>12.75</td>
<td>107.58</td>
</tr>
<tr>
<td>5</td>
<td>2.25</td>
<td>18.00</td>
<td>20.25</td>
<td>0.120</td>
<td>2.43</td>
<td>8.00</td>
<td>19.44</td>
</tr>
<tr>
<td>6</td>
<td>2.25</td>
<td>3.50</td>
<td>7.88</td>
<td>0.120</td>
<td>0.95</td>
<td>7.63</td>
<td>7.21</td>
</tr>
<tr>
<td>7</td>
<td>10.25</td>
<td>21.50</td>
<td>220.38</td>
<td>0.120</td>
<td>26.45</td>
<td>13.88</td>
<td>366.92</td>
</tr>
<tr>
<td>8</td>
<td>12.50</td>
<td>4.06</td>
<td>25.38</td>
<td>0.120</td>
<td>3.05</td>
<td>14.83</td>
<td>45.18</td>
</tr>
<tr>
<td>Pv</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>6.53</td>
<td>19.00</td>
<td>124.16</td>
</tr>
</tbody>
</table>

\[
\begin{align*}
F_v &= 61.26 \\
M_r &= 739.36
\end{align*}
\]

\[K_a = 0.39 \text{ (RANKINE COEFFICIENT WITH INCLINED BACKFILL)}\]

\[P_a = 0.5 \times \text{SOIL WT} \times H^2 \times K_a\]
\[P_v = P_a \times \sin(\text{BACKFILL ANGLE})\]

\[P_h = P_a \times \cos(\text{BACKFILL ANGLE})\]

HORIZONTAL LOADS

\[P_h = P_a \times \cos(\text{BACKFILL ANGLE})\]
\[M_o = P_h \times (H/3)\]
STABILITY CHECK

RESULTANT = \( \frac{\text{Mr} - \text{Mo}}{\text{Fv}} \) 8.78 FT YES In middle third

OVERTURNING FS: Mr/Mo 3.67 YES >2.0

SLIDING FS = \( \frac{\text{Fv}*\text{(Coefficient)}}{\text{Ph}} \) 1.52 YES >1.5
Coeff. = 0.50 (AASHTO Table 5.5.2B) - Dense M-F Sand

BRG. PRESSURE

\( B = 19.00 \) FT \( e = 0.72 \) FT

\( q(\text{max}) = \frac{\text{Fv}}{B^*}(1 + 6e/B) \) 3.96 KSF (T)

\( q(\text{min}) = \frac{\text{Fv}}{B^*}(1 - 6e/B) \) 2.49 KSF (H)

1.98 TSF (T) YES
1.25 TSF (H) YES

Assumed allowable bearing capacity for Dense Med-Fine Sand = 2TSF
3.1.2 Heel Design

MATERIAL PROPERTIES

\[
\begin{align*}
fc' &= 3,000 \text{ PSI} \\
fv &= 0.95 * fc'^{1/2} \\
fv &= 0.4 * fc' \\
fy &= 60,000 \text{ PSI} \\
fs &= 24,000 \text{ PSI} \\
n &= Es / Ec \\
k &= fc / (fc + fs / n) \\
j &= 1 - k / 3 \\
K &= 0.169 \text{ (ACI TABLE)}
\end{align*}
\]

VERTICAL LOADS (NEGLECT VERTICAL COMPONENT OF ACTIVE EARTH FORCE)

\[
\begin{array}{ccccccccc}
\text{PART} & \text{WIDTH} & \text{HEIGHT} & \text{AREA} & \text{UNIT WT.} & \text{FORCE} & \text{ARM} & \text{MOMENT} \\
(\text{FT}) & (\text{FT}) & (\text{FT}^2) & (\text{KCF}) & (\text{K/FT}) & (\text{FT}) & (\text{K FT/FT}) \\
4 & 10.25 & 4.50 & 46.13 & 0.150 & 6.92 & 5.13 & 35.46 \\
7 & 10.25 & 21.50 & 220.38 & 0.120 & 26.45 & 5.13 & 135.53 \\
8 & 10.50 & 2.03 & 20.81 & 0.120 & 2.50 & 6.83 & 17.06 \\
\end{array}
\]

Fv = \frac{35.86}{M} = \frac{188.05}{\text{OK > dmin}}

ASSUME BAR SIZE \boxed{10} \text{ DIAMETER = 1.270 IN}

\[
\begin{align*}
d &= \text{HEEL DEPTH} - 2^\prime - (1/2^*\text{BAR DIAMETER}) \\
dmin &= \frac{M(K)^{1/2}}{d}
\end{align*}
\]

STEEL AREA
\[
As = \frac{M}{(fs*j*d)} = 2.05 \text{ IN}^2
\]

NOMINAL MOMENT CAPACITY (AASHTO 8.17.1.1)
\[
0.9*Mn = 0.9*(As*fy*d*(1-0.59*(As*fy/fe^b*d))) = 326.84 \text{ K FT}
\]

CRACKING MOMENT (AASHTO 8.17.1.1)
\[
1.2*Mcr = 1.2*(7.5*fc'^{1/2})*S = 239.57 \text{ K FT}
\]

IS 0.9*Mn >= 1.2*Mcr ? \boxed{YES}

As (required) = \boxed{2.05} \text{ IN}^2

PROVIDE BAR SIZE \boxed{10} \text{ AT \boxed{6} IN SPACING}

BAR AREA = 1.27 \text{ IN}^2 \text{ DIAMETER = 1.270 IN}

As (provided) = 2.54 \text{ IN}^2
HEEL DESIGN (CONTINUED)

CHECK SHEAR IN HEEL AT BACK FACE OF STEM (CRITICAL SECTION)

\[ v = \frac{V}{b \cdot d} \quad 58.18 \text{ PSI} \quad \text{NG} > v_c \]

CHECK DEVELOPMENT LENGTH OF BAR SIZE \(10\)

\[ L_{db} = \text{MAX.} \ 0.04 \cdot A_b \cdot f_y / (f'c)^{1/2} \ OR \ 0.0004 \cdot d \cdot b^4 \ (\text{AASHTO 8.25.1}) \]
\[ L_{db} = 55.6 \ \text{IN} \quad \text{OR} \quad 30.5 \ \text{IN} \]
\[ L_{db} = 55.6 \ \text{IN} \]

\[ L_d = L_{db} \cdot \left( \frac{A_s \text{ (required)}}{A_s \text{ (provided)}} \right) \quad 44.9 \ \text{IN} \quad \text{(12 IN MIN.)} \]  
\[ \text{(AASHTO 8.25.3.2)} \]  
\[ \text{(AASHTO 8.25.3.3)} \]
3.1.3 Toe Design

ASSUME CRITICAL SECTION AT FACE OF STEM FOR M & V

q(max)= 3.96 KSF (T)
q(min)= 2.49 KSF (H)

BEARING PRESSURE AT FRONT FACE OF STEM

\[ y = \text{DISTANCE FROM PRESSURE DIAGRAM AT STEM TO MINIMUM PRESSURE} \]
\[ y = (q(\text{max})-q(\text{min}))*((\text{FOOTING WIDTH-TOE WIDTH})/\text{FOOTING WIDTH}) \]
\[ y = 1.10 \]

PRESSURE AT FACE OF STEM:\[ y+q(\text{min}) = 3.59 \text{ KSF} \]

<table>
<thead>
<tr>
<th>PART</th>
<th>WIDTH (FT)</th>
<th>PRESS. (KSF)</th>
<th>FORCE (K/FT)</th>
<th>ARM (FT)</th>
<th>MOMENT (K FT/FT)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>4.75</td>
<td>0.37</td>
<td>1.76</td>
<td>3.17</td>
<td>5.57</td>
</tr>
<tr>
<td>2</td>
<td>4.75</td>
<td>3.59</td>
<td>17.06</td>
<td>2.38</td>
<td>40.52</td>
</tr>
</tbody>
</table>

\[ Fv = 18.82 \]
\[ M = 46.09 \]

ASSUME BAR SIZE \[ 9 \]
DIAMETER= 1.128 IN

\[ d = \text{TOE DEPTH} - 3" -(1/2*BAR DIAMETER) \]
\[ d = 56.44 \text{ IN} \]

\[ d_{\text{min}} = \frac{(M/K)^{1/2}}{\text{E}} \]
\[ d_{\text{min}} = 19.91 \text{ IN} \]

STEEL AREA
\[ A_s = \frac{M}{(f_s*j*d)} \]
\[ A_s = 0.46 \text{ IN}^2 \]

NOMINAL MOMENT CAPACITY (AASHTO 8.17.1.1)
\[ 0.9*\text{Mn} = 0.9*(A_s*fy*d^2*(1-0.59*(A_s*fy/fc^2*b*d))) = 166.51 \text{ K FT} \]

CRACKING MOMENT (AASHTO 8.17.1.1)
\[ 1.2*\text{McR} = 1.2*(7.5*fc^2/b^2)*S \]
\[ 1.2*\text{McR} = 295.77 \text{ K FT} \]

IS \[ 0.9*\text{Mn} >= 1.2*\text{McR} ? \]
NO, INCREASE AREA BY 33%

As (required)= 0.61 \text{ IN}^2

PROVIDE BAR SIZE \[ 8.00 \]
AT \[ 12 \] IN SPACING
BAR AREA= 0.79 \text{ IN}^2
DIAMETER= 1.000 IN

As (provided)= 0.79 \text{ IN}^2

CHECK SHEAR IN TOE AT FRONT FACE OF STEM CRITICAL SECTION

\[ v = V/(b*d) = 33.33 \text{ PSI} \]
OK < vc
**TOE DESIGN (CONTINUED)**

**CHECK DEVELOPMENT LENGTH OF BAR SIZE**

\[
L_{db} = \text{MAX. } 0.04 \times A_b \times f_y / (f_c')^{1/2} \text{ OR } 0.0004 \times d_b^3 \text{ (AASHTO 8.25.1)}
\]

<table>
<thead>
<tr>
<th>L_{db}</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>34.6</td>
<td>IN</td>
<td></td>
</tr>
<tr>
<td>34.6</td>
<td>IN</td>
<td></td>
</tr>
</tbody>
</table>

\[
L_d = L_{db} \times (A_s \text{ (required)}/A_s \text{ (provided)}) \text{ (AASHTO 8.25.3.2)}
\]

<table>
<thead>
<tr>
<th>L_d</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>26.7</td>
<td>IN</td>
<td></td>
</tr>
</tbody>
</table>

(12 IN MIN.)

(AASHTO 8.25.3.3)
3.1.4 Stem Design

Flexure

1) Horizontal Loads (At Tenth Points From Top Of Wall To Top Of Heel)
2) Assume A No. 8 Bar Size For Initial Calc. Of D And Revise Based On Bar Size Chosen For As (p)

<table>
<thead>
<tr>
<th>HT (FT)</th>
<th>Ph (K/FT)</th>
<th>ARM (FT)</th>
<th>M (K FT/FT)</th>
<th>d (IN)</th>
<th>dmin (IN)</th>
<th>As (IN^2)</th>
<th>0.9*Mn (K FT/FT)</th>
<th>1.2*Mct (K FT/FT)</th>
<th>As(r) (IN^2)</th>
<th>bar (p)</th>
<th>spc (p)</th>
<th>As (p) (IN^2)</th>
</tr>
</thead>
<tbody>
<tr>
<td>TOP</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.00</td>
<td>0.27</td>
<td>1.16</td>
<td>0.32</td>
<td>18.63</td>
<td>1.37</td>
<td>0.01</td>
<td>0.79</td>
<td>36.23</td>
<td>0.01</td>
<td>6</td>
<td>12</td>
<td>0.44</td>
</tr>
<tr>
<td>2.15</td>
<td>0.71</td>
<td>1.88</td>
<td>1.33</td>
<td>18.63</td>
<td>2.81</td>
<td>0.04</td>
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<td>6</td>
<td>12</td>
<td>0.44</td>
</tr>
<tr>
<td>4.30</td>
<td>1.35</td>
<td>2.60</td>
<td>3.51</td>
<td>19.83</td>
<td>4.56</td>
<td>0.10</td>
<td>8.78</td>
<td>40.49</td>
<td>0.13</td>
<td>6</td>
<td>12</td>
<td>0.44</td>
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<tr>
<td>6.45</td>
<td>2.20</td>
<td>3.31</td>
<td>7.29</td>
<td>23.05</td>
<td>6.57</td>
<td>0.18</td>
<td>18.19</td>
<td>53.11</td>
<td>0.24</td>
<td>6</td>
<td>12</td>
<td>0.44</td>
</tr>
<tr>
<td>8.60</td>
<td>3.25</td>
<td>4.03</td>
<td>13.10</td>
<td>26.28</td>
<td>8.81</td>
<td>0.28</td>
<td>32.64</td>
<td>67.44</td>
<td>0.37</td>
<td>6</td>
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<td>0.44</td>
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<tr>
<td>10.75</td>
<td>4.51</td>
<td>4.75</td>
<td>21.43</td>
<td>29.44</td>
<td>11.26</td>
<td>0.41</td>
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<td>83.47</td>
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<td>6</td>
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<td>5.46</td>
<td>32.68</td>
<td>32.66</td>
<td>13.91</td>
<td>0.56</td>
<td>80.77</td>
<td>101.22</td>
<td>0.74</td>
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<td>6.18</td>
<td>47.30</td>
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<td>0.74</td>
<td>116.50</td>
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<td>1.20</td>
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<td>17.20</td>
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<td>65.73</td>
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<td>7.61</td>
<td>88.42</td>
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<td>22.87</td>
<td>1.17</td>
<td>216.23</td>
<td>164.71</td>
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<tr>
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<td>13.90</td>
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<td>115.81</td>
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<td>26.18</td>
<td>1.42</td>
<td>282.17</td>
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<tr>
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<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td>3.50</td>
<td>1.09</td>
<td>2.33</td>
<td>2.54</td>
<td>18.63</td>
<td>3.87</td>
<td>0.08</td>
<td>6.35</td>
<td>36.23</td>
<td>0.10</td>
<td>6</td>
<td>12</td>
<td>0.44</td>
</tr>
</tbody>
</table>

BATTER IS LOCATED AT 3.50 FT FROM THE TOP OF THE WALL
BATTER WIDTH AT BOTTOM OF STEM AT HEEL IS 2.25 FT THICK
BATTER HEIGHT = 18.00 FT
STEM DESIGN (CONTINUED)

SHEAR
HORIZONTAL LOADS (AT TENTH POINTS FROM TOP OF WALL TO TOP OF HEEL)

<table>
<thead>
<tr>
<th>HT. (FT)</th>
<th>Ph (K/FT)</th>
<th>v (PSI)</th>
<th>OK &lt; vc</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00</td>
<td>0.27</td>
<td>1.21</td>
<td>OK &lt; vc</td>
</tr>
<tr>
<td>2.15</td>
<td>0.71</td>
<td>3.17</td>
<td>OK &lt; vc</td>
</tr>
<tr>
<td>4.30</td>
<td>1.35</td>
<td>5.68</td>
<td>OK &lt; vc</td>
</tr>
<tr>
<td>6.45</td>
<td>2.20</td>
<td>7.95</td>
<td>OK &lt; vc</td>
</tr>
<tr>
<td>8.60</td>
<td>3.25</td>
<td>10.32</td>
<td>OK &lt; vc</td>
</tr>
<tr>
<td>10.75</td>
<td>4.51</td>
<td>12.77</td>
<td>OK &lt; vc</td>
</tr>
<tr>
<td>12.90</td>
<td>5.98</td>
<td>15.26</td>
<td>OK &lt; vc</td>
</tr>
<tr>
<td>15.05</td>
<td>7.65</td>
<td>17.76</td>
<td>OK &lt; vc</td>
</tr>
<tr>
<td>17.20</td>
<td>9.53</td>
<td>20.31</td>
<td>OK &lt; vc</td>
</tr>
<tr>
<td>19.35</td>
<td>11.61</td>
<td>22.92</td>
<td>OK &lt; vc</td>
</tr>
<tr>
<td>21.50</td>
<td>13.90</td>
<td>25.49</td>
<td>OK &lt; vc</td>
</tr>
</tbody>
</table>

CHECK DEVELOPMENT LENGTH OF BAR SIZE

9 BAR AREA= 1.00 IN^2
BAR DIA.= 1.128 IN

Ldb= MAX. 0.04*Ab*fy/(fc')^1/2 OR 0.0004*db*fy (AASHTO 8.25.1)
Ldb= 43.8 IN OR 27.1 IN
Ldb= 43.8 IN

Ld= Ldb*(As (required)/As (provided)) (AASHTO 8.25.3.2)
Ld= 31.2 IN (12 IN MIN.) (AASHTO 8.25.3.3)

CHECK LAP SPLICE LENGTH FOR DOWEL BAR

CLASS C SPLICE= 1.7*Ld 53.0 IN (12 IN MIN.) (AASHTO 8.32.31)
**SUMMARY**

**SECTION GEOMETRY**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>TOTAL WALL HEIGHT</td>
<td>26.00 ft</td>
</tr>
<tr>
<td>FOOTING WIDTH</td>
<td>19.00 ft</td>
</tr>
<tr>
<td>TOE WIDTH</td>
<td>4.75 ft</td>
</tr>
<tr>
<td>TOE THICKNESS</td>
<td>5.00 ft</td>
</tr>
<tr>
<td>HEEL WIDTH</td>
<td>10.25 ft</td>
</tr>
<tr>
<td>HEEL THICKNESS</td>
<td>4.50 ft</td>
</tr>
<tr>
<td>STEM THICKNESS</td>
<td>1.75 ft</td>
</tr>
<tr>
<td>BATTER THICKNESS</td>
<td>2.25 ft</td>
</tr>
<tr>
<td>BATTER HEIGHT</td>
<td>18.00 ft</td>
</tr>
</tbody>
</table>

**REINFORCING**

<table>
<thead>
<tr>
<th>Section</th>
<th>Provide Number</th>
<th>Bars At</th>
<th>Spacing</th>
</tr>
</thead>
<tbody>
<tr>
<td>HEEL</td>
<td>10</td>
<td>6</td>
<td>IN SPACES</td>
</tr>
<tr>
<td>TOE</td>
<td>8</td>
<td>12</td>
<td>IN SPACES</td>
</tr>
<tr>
<td>STEM</td>
<td>9</td>
<td>DOWELS</td>
<td>6</td>
</tr>
</tbody>
</table>

Development Length:

- **HEEL**: 44.9 in
- **TOE**: 26.7 in
- **STEM**: 53.0 in
3.2 Quantity Calculations
3.2.1 Concrete Quantity:

Footing: 4.5ft× (19ft- 4.75ft- 1.75ft) = 5ft × (4.75ft + 1.75ft) = 88.8ft³ = 3.3yd³
  Pay Item 25 = 3.3 × 100 = 330 yd³

Stem: 1.75ft × (26ft - 5ft) + ½ × 18ft × 2.25ft = 57ft³ = 2.1yd³
  Pay Item 26 = 2.1 × 100 = 210 yd³

Total Concrete Quantity (Pay Item 23):
  (89.6ft³ + 57ft³) = 146.6 ft³ = 5.4yd³/ft × 100 = 540 yd³

Form in place, footing (Pay Item 18):
  Contact area = (5ft + 4.5ft) × 100ft = 950ft²

Forms in place, footing, Integral starter wall (Pay Item 19):
  Length = 100ft

Forms in place, Steel Framed Plywood (Pay Item 20):
  Contact area = 2 × 21.5ft × 100ft = 4300ft²

3.2.2 Reinforcement Quantity:
(Pay Item 21)

Stem Portion
EL. 105 to EL. 126

Front Bars: #4 @ 18”
  L = 26ft - 5ft - 0.5ft = 20.5ft
  ↑(3” CLR. E. end)

WT = 0.668 lb/ft × 20.5ft × 12”/18” = 9.13lb/ft width

Bars on Back Face:
  #7 @ 6”
  L = 14.5ft - 0.25ft = 14.25ft

WT = 2 × 2.044lb/ft × 14.25ft = 58.3 lb/ft
  ↑(wt. #7 bar)
  ↑(per 12”width)

-#6 @ 12”
  L = 22.5ft + 1ft - 14.5ft - 4.5ft + 3ft
  ↑↑ E1.122.5 Above ↑↑↑ #7 fng Lap
  L = 7.5’

WT= 1.502#/1 x 7.5’ = 11.3lb/ft

-Longitudinal Bars: #4 @ 12” E Face
  L=1 ft

  for front face: 26ft - 5ft = 21ft ie 21 spaces, use 22 bars
  for back face: 23 bars

  WT: (22 + 23) × 0.668lb/ft × 1 ft = 30lbs
Top Portion of Stem: #6@12" L = 26ft - 22.5ft + 1ft-0.25ft = 4.25ft

WT = 1.502lb/ft x 4.25ft = 6.4lb/ft

Total Reinforcement in Stem:

WT = 9.13lb + 58.3lb + 11.3lb + 30lb + 6.4lb = 115lb per 1 ft width

Footing: Transverse Bars  #8@12" Bott  
#10@6" Top

(8.6lb/ft + 2.7lb/ft)(19 - 0.5') = 209#
\[\text{Ea. side}\]

Longitudinal Bars:  #5 @18" T & B  
2 x 13 x 1.043lb/ft x 1ft = 27lb
\[\text{T of B bars/mat wt #5 width}\]

Dowels: #9@6" 6.5ft + 4.5ft = 11ft

2 x 3.4lb/ft x 11ft = 75lb
\[\text{WT. #9}\]

Total Reinforcement in Footing:

W = 209lb + 27lb + 75lb = 311lb per 1ft width

Total Reinforcement in Retaining Wall (Pay Item 21):

WT = 115lb + 311lb = 426lb per ft width x 100ft = 21.3 tons
3.2.3 Excavation and Backfill (See Figure 3-3)

Excavation:
Limits: From front face of wall to 2' beyond heel of footing
Then on 1 to 1 slope

Area 1: (19ft − 4.75ft + 2ft) x 26ft + ½ x 26ft x 26ft = 761 ft³ per ft

Area 2: (4.75ft + 2ft) x (5ft + 2ft) + ½ x (7)²ft² = 72 ft³ per ft

Area 3: L = 19ft − 6.5ft + 2ft + 26ft = 40.5ft
H = 40.5ft Tan 18° = 13.2ft
½ x 40.5ft x 13.2ft = 267 ft³ per ft

Total Excavation: (761 ft³ + 72 ft³ + 267 ft³) 1/27 = 41 yd³ per ft x 100ft = 4100 yd³

Excavation Related Pay Items

Item 07 – Backfill Structural  V = 4100 yd³
Item 08 – Select Granular Fill  V = 4100 yd³
Item 09 – Compaction Roller  V = 4100 yd³
Item 10 – Compaction Plate  V = 1.5ft x 26ft x 100ft = 144 yd³
Item 12 – Excavation  V = 4100 yd³
Figure 3-1
Proposed Reinforced Concrete Retaining Wall Configuration
Figure 3-2
Proposed Reinforced Concrete Retaining Wall Section
Figure 3-3
Retaining Wall Quantities
CHAPTER 4 CONCRETE MODULAR UNIT GRAVITY WALL

4.1 Design Calculations

Wall Properties - Stepped Modules

\[ H := 19\cdot \text{ft} + 2\cdot \text{ft} \quad \text{Wall height + Distance below grade} \]
\[ L := 100\cdot \text{ft} \quad \text{Wall Length} \]

Soil Properties

Infill Soil Granular Backfill to fill voids of each unit

\[ \gamma_i := 105\cdot \text{pcf} \quad \text{Soil Density} \quad \text{Coarse Sand & Gravel} \]
\[ \phi_i := 36\cdot \text{deg} \quad \text{Angle of Internal Friction} \]

Foundation Soil & Retained Soil

\[ \gamma_f := 120\cdot \text{pcf} \quad \text{Soil Density} \]
\[ \phi_f := 30\cdot \text{deg} \quad \text{Angle of internal friction} \]
\[ \delta := 22.5\ \text{deg} \quad \text{Angle of friction between soil and wall} \]
\[ \text{AASHTO 5.9.2} = 3/4 \phi_f \]
\[ \beta := 64\cdot \text{deg} \quad \text{Batter of Wall, where 90 degrees is vertical} \]
\[ \text{Match Modules} \]
\[ \alpha := 18\cdot \text{deg} \quad \text{Slope of Retained Soil} \]
\[ c := 0 \quad \text{Soil Cohesion} \]
Determine Coulomb's Active Earth Pressure Coefficient, $K_a$
for retained granular fill

$$K_a := \frac{\sin(\beta + \phi_i)^2}{\sin(\beta)^2 \cdot \sin(\beta - \delta) \cdot \left[ 1 + \sqrt{\frac{\sin(\phi_i - \alpha) \sin(\beta - \delta) \cdot \sin(\alpha + \beta)}{\sin(\phi_i + \delta) \cdot \sin(\beta + \delta)}} \right]^2}$$

$K_a = 0.825$ Coulomb's active earth pressure coefficient for fill material

Where:

$\beta = 64\degree$ deg  $\delta := 22.5$ deg
$\phi_f := 30$ deg  $\alpha = 18\degree$ deg

4.1.1 Stability

Consider Overturning

$M_r := 264.6 \ \frac{\text{kip} \cdot \text{ft}}{\text{ft}}$ Resisting Moment, See Table 4.1

$P_h := \frac{1}{2} \gamma_f \cdot K_a \cdot H^2 \cdot \cos(90 - \beta + \delta)$ Horizontal Load due to Active Soil Pressure

$P_h = 14.5 \ \frac{\text{kip}}{\text{ft}}$ Table 4.1

$z := \frac{H}{3}$  $z = 7.0\footnotesize{\text{ft}}$ Overturning moment arm

$M_o := P \cdot z$ Overturning Moment due to active soil pressure

$M_o := 101.3 \ \frac{\text{kip} \cdot \text{ft}}{\text{ft}}$

Where:

$\gamma_f := 120 \ \text{pcf}$  $H = 21\footnotesize{\text{ft}}$

$K_a = 0.825$  $\beta = 64\degree$ deg
Determine Factor of Safety against Overturning, AASHTO 5.5.5

\[ F_{So} = \frac{Mr}{Mo} \]

\[ F_{So} = 2.6 \quad > \quad 2.0, \text{ O.K.} \]

Where:

\[ Mr := 264.6 \, \frac{\text{kip} \cdot \text{ft}}{\text{ft}} \]

\[ Mo := 101.3 \, \frac{\text{kip} \cdot \text{ft}}{\text{ft}} \]

Consider Sliding

\[ k := \frac{2}{3} \quad \text{See Appendix, Ref. 2, Pp. 434} \]

\[ w := 45.7 \, \frac{\text{kip}}{\text{ft}} \quad \text{Total Weight per ft of Wall, See Table 4.1 -100\% Fill Weight} \]

Determine Factor of Safety against Sliding, AASHTO 5.5.5

\[ F_{Ss} := \frac{w \cdot \tan(k \cdot \phi_f) + P_v}{P_h} \]

\[ F_{Ss} = 2.11 \quad > \quad 1.5, \text{ OK} \]

Where \( P_v = \frac{1}{2} \gamma_f K_a H^2 \sin(90 - \beta + \delta) \)
<table>
<thead>
<tr>
<th>Parameters</th>
<th>See Fig. 4-1 for Wall Configuration</th>
</tr>
</thead>
<tbody>
<tr>
<td>Retained Soil, Gravel &amp; Sand</td>
<td></td>
</tr>
<tr>
<td>Unit Weight</td>
<td>105 pcf</td>
</tr>
<tr>
<td>Ka</td>
<td>0.825</td>
</tr>
<tr>
<td>Soil In Concrete Voids, Gravel &amp; Sand</td>
<td></td>
</tr>
<tr>
<td>Unit Weight</td>
<td>105 pcf</td>
</tr>
<tr>
<td>Batter of Wall</td>
<td>29 degrees from 90 - equivalent</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Concrete Modular Units</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit</td>
<td>Unit Height</td>
</tr>
<tr>
<td>1</td>
<td>4</td>
</tr>
<tr>
<td>2</td>
<td>6</td>
</tr>
<tr>
<td>3</td>
<td>8</td>
</tr>
<tr>
<td>4</td>
<td>10</td>
</tr>
<tr>
<td>5</td>
<td>14</td>
</tr>
<tr>
<td>Totals:</td>
<td></td>
</tr>
</tbody>
</table>

* Weights given for Doublewall II Units for this example

<table>
<thead>
<tr>
<th>Resisting Moment, Mr</th>
<th>Overturning Moment, Mo</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit</td>
<td>Wt</td>
</tr>
<tr>
<td>1</td>
<td>1,609</td>
</tr>
<tr>
<td>Soil above 24</td>
<td>2.8</td>
</tr>
<tr>
<td>2</td>
<td>3,250</td>
</tr>
<tr>
<td>Soil above 1,110</td>
<td>5.5</td>
</tr>
<tr>
<td>3</td>
<td>3,086</td>
</tr>
<tr>
<td>Soil above 2,382</td>
<td>6.0</td>
</tr>
<tr>
<td>4</td>
<td>3,827</td>
</tr>
<tr>
<td>Soil above 3,498</td>
<td>9.0</td>
</tr>
<tr>
<td>5</td>
<td>7,879</td>
</tr>
<tr>
<td>Soil above 9,685</td>
<td>12.0</td>
</tr>
<tr>
<td>Totals:</td>
<td>35,459</td>
</tr>
</tbody>
</table>

** Ph = Ka*gamma*z^2/2*cos(90-beta+delta)**

Total Weight = 38,847 (i.e. no reduction of fill weight) Check Sliding = 2.11 including vertical component of Pactive

Minimum Factor of Safety for Overturning = 2.0 (AASHTO 16th - 5.5.5)
4.1.2 Bearing Capacity of Substrate Soil

\[ \phi_f = 30\text{-deg} \quad \text{Angle of internal friction} \]

\[ N_e := 30.14 \quad \text{From AASHTO Table 4.4.7.1A} \]

\[ N_g := 22.4 \]

\[ B := 14\text{-ft} \quad \text{Width of bottom unit of wall, See Figure 4-1} \]

\[ e := \frac{B}{2} - \frac{M_r - M_o}{w} \quad e = 1.27\text{ft} \quad \text{Eccentricity of resultant load from the midpoint of the bottom unit} \]

\[ B = 2.33\text{ft} \quad \text{Kern distance} \quad \text{OK, e is within Kern} \]

\[ L_p := B - e \cdot 2 \quad L_p = 11.46\text{ft} \quad \text{Effective Bearing Length} \]

\[ \sigma := \frac{w}{B} \quad \sigma := .326 \text{ ksf} \quad \text{Bearing Stress on Foundation Soil} \]

Where:

\[ w = 45.7 \text{ kip} \quad \text{Weight of the Wall} \]

\[ q := \frac{1}{2} \gamma_f \cdot L_p \cdot N_g \quad q = 15.4 \text{ksf} \quad \text{Allowable Bearing Stress on Soil} \]

Where:

\[ \gamma_f := 120 \text{pcf} \quad \text{Unit weight of soil} \]

Determine Factor of Safety for Bearing, AASHTO 4.4.7.1.2

\[ F_{SB} := \frac{q}{\sigma} \quad F_{SB} = 4.7 \quad > \quad 3 \quad \text{OK} \]
4.2 Quantity Calculations

4.2.1 Modular Units

Concrete Quantity (Pay Item 23, 24 & 27)

\[ A := 277 \text{ft}^2 \quad \text{Volume per 8' Section} \]

\[ L = 100 \text{ft} \quad \text{Length of Wall} \]

\[ \text{Volume} = V = \frac{L}{8} \quad V = 128 \text{yd}^3 \]

Reinforcement Quantity (Pay Item 21)

\[ \text{Ratio} := \frac{293 \text{lbf}}{\text{yd}^3} \quad \text{Assumed ratio of reinforcement to concrete} \]

\[ Wt := V \cdot \text{Ratio} \quad Wt = 18.8 \text{ ton} \]

4.2.2 Footing

Concrete in Leveling Pad (Pay Item 23 & 25)

\[ A := 31 \cdot \text{ft}^2 + 21 \cdot \text{ft}^2 \quad A = 5 \text{ft}^2 \quad \text{Cross-Section Area} \]

\[ L = 100 \text{ft} \quad \text{Length of Wall} \]

\[ \text{Volume} = V := A \cdot L \quad V = 18.5 \text{yd}^3 \]

Reinforcement Quantity (Pay Item 21)

\[ 128 \text{yd}^3 \]

\[ \text{Ratio} := \frac{293 \text{lbf}}{\text{yd}^3} \]

\[ Wt := V \cdot \text{Ratio} \quad Wt = 2.7 \text{ Ton} \]

Concrete related Pay Items

Item 18 - Forms in place, footing \[ 41 \cdot \text{ft} \cdot L = 400 \text{ft}^2 \]

Item 21 - Reinforcing Steel \[ 18.8 \text{ Ton} + 2.7 \text{ Ton} = 21.5 \text{ Ton} \]

Item 23 - Concrete, Ready Mix \[ 128 \text{yd}^3 + 18.5 \text{yd}^3 = 146.5 \text{yd}^3 \]

Item 24 - Placing concrete, chute \[ 128 \text{yd}^3 \quad (\text{Assume for plant operations}) \]

Item 25 - Placing concrete, pumped \[ 18.5 \text{yd}^3 \]

Item 27 - Placing concrete, with crane \[ 128 \text{yd}^3 \quad (\text{Assume for field ops}) \]
4.2.3 Excavation and Backfill

Excavation (Pay Item 12)

\[ A := 795 \text{-ft}^2 \quad \text{Cross-Section Area measured in CAD Sketch} \]
\[ L = 100 \text{-ft} \quad \text{Length of Wall} \]
\[ \text{Volume} = A \cdot L = 2944 \text{-yd}^3 \]

Backfill: Structural Fill (Pay Items 07 & 08)

\[ A := 500 \text{-ft}^2 \quad \text{Cross-Section Area measured in CAD Sketch} \]
\[ L = 100 \text{-ft} \quad \text{Length of Wall} \]
\[ \text{Volume} = A \cdot L = 1852 \text{-yd}^3 \]

Volume of Granular Backfill behind wall units (Pay Items 07 & 08)

\[ A := 92 \text{-ft}^2 \quad \text{Cross-Section Area measured in CAD Sketch} \]
\[ L = 100 \text{-ft} \quad \text{Length of Wall} \]
\[ \text{Volume} = A \cdot L = 341 \text{-yd}^3 \]

Volume of Granular Backfill inside wall units (Pay Item 07 & 08)

\[ A := 1291 \text{-ft}^2 \quad \text{CF per 8' Section} \]
\[ L = 100 \text{-ft} \quad \text{Length of Wall} \]
\[ \text{Volume} = \frac{1291 \cdot 100}{8 \cdot 27} = 598 \text{ CY} \]

Total Volume of Granular Backfill (Pay Items 07 & 08)

\[ \text{Volume} = 341 \text{ yd}^3 + 598 \text{yd}^3 = 939 \text{yd}^3 \]

Geotextile around granular fill pocket (Pay Item 16)

\[ W := 19 \cdot \text{ft} + 6 \cdot \text{ft} + 2 \cdot \text{ft} \quad \text{Length of Geotextile along Wall} \]
\[ \text{Cross-Section Area} \]
\[ L = 100 \text{-ft} \quad \text{Length of Wall} \]
\[ \text{Area} = W \cdot L = 300 \text{-yd}^2 \]
Excavation related Pay Items

Item 07 - Backfill Structural \[1852 \text{ yd}^3 + 939 \text{ yd}^3 = 2791 \text{ yd}^3\]

Item 08 - Select Granular Fill \[2724 \text{ yd}^3\]

Item 09 - Compaction, Roller \[2724 \text{ yd}^3\]

Item 10 - Compaction, Plate \[1.5 \cdot \text{ft} \cdot 21 \cdot \text{ft} \cdot L = 117 \text{ yd}^3\]

Item 12 - Excavation \[2944 \text{ yd}^3\]
CONCRETE MODULAR UNIT GRAVITY WALL

FIGURE 4-1

PARTIAL PLAN – CONCRETE MODULAR UNIT GRAVITY WALL

* DIMENSIONS VARY AS PER MANUFACTURER
CHAPTER 5 MECHANICALLY STABILIZED EARTH WALL

5.1 Design Calculations

Wall Properties

\[ H := 19 \text{-ft} + 2 \text{-ft} \quad \text{Wall height + Embedment below grade} \]
\[ L := 100 \text{-ft} \quad \text{Wall Length} \]

Soil Properties

Infill Soil: Granular Backfill used as fill around reinforcement

\[ \gamma_i := 105 \text{-pcf} \quad \text{Soil Density} \]
\[ \phi_i := 36 \text{-deg} \quad \text{Angle of Internal Friction} \]

Foundation & Retained Soil

\[ \gamma_f := 120 \text{-pcf} \quad \text{Soil Density} \]
\[ \phi_f := 30 \text{-deg} \quad \text{Angle of internal friction} \]
\[ \delta := 0 \quad \text{Angle of friction between soil and wall} \]
\[ \beta := 90 \text{-deg} \quad \text{Batter of Wall, where 90 degrees is vertical} \]
\[ \alpha := 18 \text{-deg} \quad \text{Slope of Retained Soil} \]
\[ c := 0 \quad \text{Soil Cohesion} \]

Galvanized Steel Reinforcement Properties

\[ w := 2 \text{-in} \quad \text{Width of Reinforcement Strip} \]
\[ s_v := 2.5 \text{-ft} \quad \text{Vertical Spacing, Center to Center} \]
\[ s_h := 2 \text{-ft} \quad \text{Horizontal Spacing, Center to Center} \]
\[ f_y := 60 \text{-ksi} \quad \text{Yield Stress} \]
\[ \phi_u := 20 \text{-deg} \quad \text{Soil tie friction angle} \]
Determine Coulomb's Active Earth Pressure Coefficient, Ka

\[ Ka := \frac{\sin(\beta + \phi_i)^2}{\sin(\beta)^2 \cdot \sin(\beta - \delta) \left( 1 + \sqrt{\sin(\phi_i + \delta) - \frac{\sin(\phi_i - \alpha)}{\sin(\beta - \delta) \cdot \sin(\alpha + \beta)}} \right)^2} \]

Ka = 0.32  Coulomb's active earth pressure coefficient for fill material

Where:
\[ \beta = 90 \cdot \text{deg} \quad \delta = 0 \cdot \text{deg} \]
\[ \phi_i = 36 \cdot \text{deg} \quad \alpha = 18 \cdot \text{deg} \]

5.1.1 Stability

Determine Required Tie Requirements

\[ FS := 1.5 \quad \text{Factor of Safety for bearing} \]

\[ t := \frac{\gamma_i \cdot H \cdot Ka \cdot sv \cdot sh \cdot FS}{w \cdot fy} \quad t = 0.44 \text{ in} \quad \text{Required Tie Thickness} \]

Where:
\[ \gamma_i = 105 \text{ pcf} \quad sv = 3 \text{ ft} \quad fy = 35 \text{ ksi} \]
\[ H = 21 \text{ ft} \quad sh = 2 \text{ ft} \]
\[ Ka = 0.32 \quad w = 2 \text{ in} \]

Consider Corrosion of Reinforcement: Assume the rate of corrosion is 0.001 in. per year and that there is a 100 year life span or per AASHTO

\[ t = 0.16 \text{ in min} \]

Use 1/4 in. thick ties \[ t := 0.25 \cdot \text{in} \]

Analysis follows per FHWA software MSEW

Per output, use a Tie Length \[ Lt = 15 \text{ ft} \]
SOIL DATA

REINFORCED SOIL
Unit weight, $\gamma = 105.0$ lb/ft$^2$
Net design value of internal angle of friction, $\phi = 36.0$°

RETAINED SOIL
Unit weight, $\gamma = 120.0$ lb/ft$^2$
Net design value of internal angle of friction, $\phi = 30.0$°

FOUNDATION SOIL (Considered as an equivalent uniform soil)
Equivalent unit weight, $\gamma_{equiv.} = 120.0$ lb/ft$^2$
Equivalent internal angle of friction, $\phi_{equiv.} = 30.0$°
Equivalent cohesion, $c_{equiv.} = 0.0$ lb/ft$^2$

Water table does not affect bearing capacity

LATERAL EARTH PRESSURE COEFFICIENTS
$K_a$ (internal stability) = 0.2596 (if batter is less than 10°, $K_a$ is calculated from eq. 15. Otherwise, eq. 38 is utilized)
$K_a$ (external stability) = 0.3948 (if batter is less than 10°, $K_a$ is calculated from eq. 16. Otherwise, eq. 17 is utilized)

BEARING CAPACITY
Bearing capacity coefficients (calculated by MSEW): $N_c = 30.14$
$N_{gamma} = 22.40$

SEISMICITY
----- Not Applicable -----

INPUT DATA: Metal strips (Analysis)

<table>
<thead>
<tr>
<th>Metal strip type #1</th>
<th>Metal strip type #2</th>
<th>Metal strip type #3</th>
<th>Metal strip type #4</th>
<th>Metal strip type #5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yield strength of steel, $F_y$ [ksi]</td>
<td>65.3</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Gross width of strip, $b$ [in]</td>
<td>2.0</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Vertical spacing, $s_v$ [ft]</td>
<td>varies</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Design cross section area, $A_c$ [in$^2$]</td>
<td>0.16</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Ribbed steel strips. Uniformity Coefficient of reinforced soil, $C_u = D_{60}/D_{10} = 4.0$</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Friction angle along reinforcement-soil interface, $\delta$</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>@ the top</td>
<td>60.97</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>@ 19.7 ft or below</td>
<td>36.00</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Pullout resistance factor, $F^*$</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>@ the top</td>
<td>1.80</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>@ 19.7 ft or below</td>
<td>0.73</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Scale-effect correct. factor, $\alpha$</td>
<td>1.00</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
</tbody>
</table>

Variation of Lateral Earth Pressure Coefficient With Depth

<table>
<thead>
<tr>
<th>$z$</th>
<th>$K / K_a$</th>
</tr>
</thead>
</table>

Page 1
### INPUT DATA: Facia and Connection (Analysis)

**FACIA type:** Segmented precast concrete panels.
- Depth of panel is 0.50 ft. Horizontal distance to Center of Gravity of panel is 0.25 ft.
- Average unit weight of panel is \( \gamma_f = 152.78 \text{ lb/ft}^3 \)

<table>
<thead>
<tr>
<th>Z / H</th>
<th>To-static / Tmax</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00</td>
<td>1.00</td>
</tr>
<tr>
<td>0.25</td>
<td>1.00</td>
</tr>
<tr>
<td>0.50</td>
<td>1.00</td>
</tr>
<tr>
<td>0.75</td>
<td>1.00</td>
</tr>
<tr>
<td>1.00</td>
<td>1.00</td>
</tr>
</tbody>
</table>

### DAT A (for connection only)

<table>
<thead>
<tr>
<th>Type #1</th>
<th>Type #2</th>
<th>Type #3</th>
<th>Type #4</th>
<th>Type #5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strength reduction at the connection, ( \tau_{Cru} = F_{yc} / F_y )</td>
<td>0.90</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
</tbody>
</table>

### INPUT DATA: Geometry and Surcharge loads (of a SIMPLE STRUCTURE)

- **Design height, \( H_d \):** 21.00 [ft] (Embedded depth is \( E = 2.00 \) ft, and height above top of finished bottom grade is \( H = 19.00 \) ft.)
- **Batter, \( \omega \):** 0.0 [deg]
- **Backslope, \( \beta \):** 18.0 [deg]
- **Backslope rise:** 100.0 [ft] Broken back equiv. angle, \( I = 18.00^\circ \) (see Fig. 25 in DEMO 82)

**Uniformly distributed dead load is 0.0 [lb/ft²]**

### ANALYSIS: CALCULATED FACTORS (static conditions)

- **Bearing capacity, \( F_s = 3.69 \)**, Foundation Interface: Direct sliding, \( F_s = 1.580 \), Eccentricity, \( e/L = 0.1392 \).

### METAL STRIP

<table>
<thead>
<tr>
<th># Elevation</th>
<th>Length</th>
<th>Type</th>
<th>Connection</th>
</tr>
</thead>
<tbody>
<tr>
<td>[ft]</td>
<td>[ft]</td>
<td></td>
<td>[pullout] [connect</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>resistance] [break]</td>
</tr>
<tr>
<td>1</td>
<td>1.00</td>
<td>1</td>
<td>N/A</td>
</tr>
<tr>
<td>2</td>
<td>3.46</td>
<td>1</td>
<td>N/A</td>
</tr>
<tr>
<td>3</td>
<td>5.92</td>
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<td>N/A</td>
</tr>
<tr>
<td>4</td>
<td>8.38</td>
<td>1</td>
<td>N/A</td>
</tr>
<tr>
<td>5</td>
<td>10.84</td>
<td>1</td>
<td>N/A</td>
</tr>
<tr>
<td>6</td>
<td>13.30</td>
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<td>N/A</td>
</tr>
<tr>
<td>7</td>
<td>15.76</td>
<td>1</td>
<td>N/A</td>
</tr>
<tr>
<td>8</td>
<td>18.22</td>
<td>1</td>
<td>N/A</td>
</tr>
<tr>
<td>9</td>
<td>20.69</td>
<td>1</td>
<td>N/A</td>
</tr>
</tbody>
</table>

**Length of reinforce = 14.7 ft**

### BEARING CAPACITY for GIVEN LAYOUT

<table>
<thead>
<tr>
<th>STATIC</th>
<th>SEISMIC</th>
<th>UNITS</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Ultimate bearing capacity, ( q_{ult} ):</strong></td>
<td>14257</td>
<td>N/A</td>
</tr>
<tr>
<td><strong>Meyerhof stress, ( \sigma_m ):</strong></td>
<td>3863.9</td>
<td>N/A</td>
</tr>
<tr>
<td><strong>Eccentricity, ( e ):</strong></td>
<td>2.05</td>
<td>N/A</td>
</tr>
<tr>
<td><strong>Eccentricity, ( e/L ):</strong></td>
<td>0.139</td>
<td>N/A</td>
</tr>
</tbody>
</table>

Page 2
DIRECT SLIDING for GIVEN LAYOUT

Along reinforced and foundation soils interface: \( Fs_{\text{static}} = 1.580 \)

<table>
<thead>
<tr>
<th>#</th>
<th>Metal strip Elevation [ft]</th>
<th>Metal strip Length [ft]</th>
<th>Static Force [kips]</th>
<th>Seismic</th>
<th>Metal strip type #</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.00</td>
<td>14.70</td>
<td>2.051</td>
<td>N/A</td>
<td>1</td>
</tr>
<tr>
<td>2</td>
<td>3.46</td>
<td>14.70</td>
<td>2.228</td>
<td>N/A</td>
<td>1</td>
</tr>
<tr>
<td>3</td>
<td>5.92</td>
<td>14.70</td>
<td>2.442</td>
<td>N/A</td>
<td>1</td>
</tr>
<tr>
<td>4</td>
<td>8.38</td>
<td>14.70</td>
<td>2.705</td>
<td>N/A</td>
<td>1</td>
</tr>
<tr>
<td>5</td>
<td>10.84</td>
<td>14.70</td>
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ECCENTRICITY for GIVEN LAYOUT

Along reinforced and foundation soils interface: \( e/L_{\text{static}} = 0.1392 \)

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RESULTS for STRENGTH

[Note: Actual Fs-overall = (yield stress) / (Actual stress)]

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RESULTS for CONNECTION (static conditions)

| # M. Strips Coverage Ratio Rec=b/Sh Horizontal spacing, Sh Horizontal force, To Reduce Factor L for connect. Long-term M. Strips connection strength strength break | Long-term M. Strips connection strength strength break | Fs-overall M. strips connection strength strength break | Fs-overall M. strips strength |
|---|---------------------------------------------------------------------------------------------------------------------------------|------------------------------------------------------|------------------------------------------------------|-----------------------------------|
| 1 | 1.00                                                                                                                          | 0.067                                                | 2.461                                                | 1643                              | 0.90                              | 3700                                | 4111 N/A                           | 2.25 N/A                          |
| 2 | 3.46                                                                                                                          | 0.067                                                | 2.461                                                | 1687                              | 0.90                              | 3700                                | 4111 N/A                           | 2.19 N/A                          |
| 3 | 5.92                                                                                                                          | 0.067                                                | 2.461                                                | 1545                              | 0.90                              | 3700                                | 4111 N/A                           | 2.29 N/A                          |
| 4 | 8.38                                                                                                                          | 0.067                                                | 2.461                                                | 1377                              | 0.90                              | 3700                                | 4111 N/A                           | 2.69 N/A                          |
| 5 | 10.84                                                                                                                         | 0.067                                                | 2.461                                                | 1209                              | 0.90                              | 3700                                | 4111 N/A                           | 3.06 N/A                          |

Page 3
### RESULTS for PULLOUT

| # | Elevation [ft] | Metal strip Coverage Ratio [lb/ft] | Le [ft] | La [ft] | Tmax [lb/ft] | Tmd [lb/ft] | NASSPA1 | 0.90 | 3700 | 4111 | N/A | 3.62 | N/A | 4.02 | 0.90 | 3700 | 4111 | N/A | 4.62 | N/A | 5.13 | 0.90 | 3700 | 4111 | N/A | 6.62 | N/A | 7.36 | 0.90 | 3700 | 4111 | N/A | 16.63 | N/A | 18.48 |

### Table

| # | Elevation [ft] | Metal strip Coverage Ratio [lb/ft] | Le [ft] | La [ft] | Tmax [lb/ft] | Tmd [lb/ft] | NASSPA1 | 0.90 | 3700 | 4111 | N/A | 3.62 | N/A | 4.02 | 0.90 | 3700 | 4111 | N/A | 4.62 | N/A | 5.13 | 0.90 | 3700 | 4111 | N/A | 6.62 | N/A | 7.36 | 0.90 | 3700 | 4111 | N/A | 16.63 | N/A | 18.48 |

| # | Elevation [ft] | Metal strip Coverage Ratio [lb/ft] | Le [ft] | La [ft] | Tmax [lb/ft] | Tmd [lb/ft] | NASSPA1 | 0.90 | 3700 | 4111 | N/A | 3.62 | N/A | 4.02 | 0.90 | 3700 | 4111 | N/A | 4.62 | N/A | 5.13 | 0.90 | 3700 | 4111 | N/A | 6.62 | N/A | 7.36 | 0.90 | 3700 | 4111 | N/A | 16.63 | N/A | 18.48 |
Consider Overturning

FS := 2.0  \text{ Factor of Safety against overturning}

Determine Resisting Moment

\[ W := \gamma_i \cdot L \cdot t \cdot H \quad W = \frac{33 \text{ kip}}{\text{ft}} \quad \text{Weight of soil resisting overturning} \]

\[ x := \frac{L_t}{2} \quad x = 7.5\text{ft} \quad \text{Moment arm of resisting soil from face of wall} \]

\[ Mr := W \cdot x \quad Mr = \frac{248 \text{ kip ft}}{\text{ft}} \quad \text{Resisting Moment} \]

Determine Overturning Moment

\[ P := \frac{1}{2} \gamma_i \cdot K_a \cdot H^2 \cdot \sin(\beta) \quad P = \frac{7.4 \text{ kip}}{\text{ft}} \quad \text{Horizontal Load due to Soil} \]

\[ z := \frac{H}{3} \quad z = 7\text{ft} \quad \text{Overturning moment arm} \]

\[ Mo := P \cdot z \quad Mo = \frac{51.9 \text{ kip ft}}{\text{ft}} \quad \text{Overturning Moment} \]

Determine Factor of Safety

\[ \frac{Mr}{Mo} = 4.80 > 2.0, \text{ OK} \]
Check Sliding

\[ k := \frac{2}{3} \quad \text{See Appendix, Ref. 2, Pp. 434} \]

Determine Factor of Safety against Sliding, AASHTO 5.5.5

\[ FS_s := \frac{W \cdot \tan(k \cdot \phi_i)}{p} \quad FS_s = 2.0 > 1.5 \quad \text{OK} \]

Where:

\[ P = 7.4 \, \text{kip} \]
\[ ft \]
\[ W = 33 \, \text{kip} \]
\[ ft \]

5.1.2 Bearing Capacity of Substrate Soil

\[ \phi_f = 30^\circ \text{deg} \quad \text{Angle of internal friction} \]

\[ N_c := 30.14 \quad \text{From AASHTO Table 4.4.7.1A} \]

\[ N_g := 22.4 \]

\[ e := \frac{L_t}{2} - \frac{M_r - M_o}{W} \quad e = 1.56 \, \text{ft} \quad \text{Eccentricity of resultant load from the midpoint of the retained soil} \]

\[ L_p := L_t - e \cdot 2 \quad L_p = 11.9 \quad \text{Effective Length} \]

\[ q := \frac{1}{2} \gamma_f \cdot L_p \cdot N_g \quad q = 16.0 \, \text{ksf} \quad \text{Allowable Bearing Stress on Soil} \]

\[ \sigma := \gamma_i \cdot H \quad \sigma := 2.21 \, \text{ksf} \quad \text{Bearing Stress on Foundation Soil} \]

Determine Factor of Safety for Bearing, AASHTO 4.4.7.1.2

\[ FS_b := \frac{q}{\sigma} \quad FS_b = 7.3 > 3 \quad \text{OK} \]
5.2 Quantity Calculations

5.2.1 Concrete

Wall Panels (Pay Item 29)

\[ A := H \cdot L \quad A = 2100 \text{ ft}^2 \quad \text{Exposed wall area} \]
\[ L = 100 \text{ ft} \quad \text{Length of Wall} \]
\[ H = 21 \text{ ft} \quad \text{Height of Wall} \]
\[ T := 7.5 \text{ in} \quad \text{Assumed Wall Panel Thickness} \]

\[ V := A \cdot T \quad V = 48.6 \text{ yd}^3 \quad \text{Volume of Concrete in Wall Panels} \]

Concrete in Leveling Pad (Pay Item 23)

\[ v := 3 \text{ ft} \cdot 1 \text{ ft} \cdot L \quad v = 11.1 \text{ yd}^3 \quad \text{Volume in Concrete Leveling Pad} \]

Formwork for Leveling Pad (Pay Item 25)

\[ A_f := 2 \cdot 3 \text{ ft} \cdot 100 \text{ ft} \quad A_f = 600 \text{ ft}^2 \]

5.2.2 Reinforcement Quantity in Wall Panels

Galvanized Ties (Pay Item 22 & 30)

\[ w = 2 \text{ in} \quad \text{Tie Width} \]
\[ t = 0.25 \text{ in} \quad \text{Tie Thickness} \]

Determine Tie Grid Matrix

\[ sv := 2.5 \text{ ft} \quad \text{Vertical Spacing of Ties} \]
\[ sh := 2 \text{ ft} \quad \text{Horizontal Spacing of Ties} \]

\[ S_v := \frac{H}{sv} \quad S_v := 9 \quad \text{Number of Vertical Ties} \]
\[ S_h := \frac{L}{sh} \quad S_h = 50 \quad \text{Number of Horizontal Ties} \]

Determine Total Weight of Tie Steel (Pay Item 30)

\[ W_t := S_v \cdot S_h \cdot w \cdot t \cdot L \cdot 490 \frac{\text{lb ft}}{\text{ft}^3} \quad W_t = 5.6 \text{ Ton} \]

Determine equivalent area of welded wire fabric for estimating (Pay Item 22)

\[ A_w := \frac{W_t}{58 \text{ lb ft}} \quad A_w = 193 \quad \text{CSF} \]
Reinforcement Steel in Wall Panels and Leveling Pad (Pay Item 21)

\[\text{Ratio} := 293 \frac{\text{lbf}}{\text{yd}^3}\]
Assumed Ratio of Reinforcement Steel to Concrete

\[\text{Wt} := V \cdot \text{Ratio}\]
Wt = 71. Ton

Where \( V = 48.6 \text{yd}^3 \)
Volume of Concrete in Wall Panels

5.2.3 Excavation and Backfill

Volume of Granular Fill (Pay Item 07 & 08)

\[V := H \cdot (15 \cdot \text{ft} + 3.5 \cdot \text{ft}) \cdot L\]
\[V = 1439 \text{yd}^3\]

Where:

\( H = 21 \text{ ft} \)
Wall Height

\( 15 \cdot \text{ft} + 3.5 \cdot \text{ft} \)
Length Granular Fill

\( L = 100 \text{ ft} \)
Wall Length

Volume of Structural Fill (Pay Item 07 & 08)

\[A := 970 \cdot \text{ft}^2 - H \cdot (15 \cdot \text{ft} + 3.5 \cdot \text{ft})\]
\[A = 582 \text{ft}^2\]
Area of Fill

\[V := A \cdot L\]
\[V = 2154 \text{yd}^3\]
Total Volume of Structural Fill

Where:

\( H = 21 \text{ ft} \)
Wall Height

\( 15 \cdot \text{ft} + 3.5 \cdot \text{ft} \)
Length Granular Fill

\( L = 100 \text{ ft} \)
Wall Length

Geotextile (Pay Item 16)

\[A := (H + 18.5 \cdot \text{ft}) \cdot L\]
\[A = 438.9 \text{yd}^2\]
Excavation related Pay Items

Item 07 - Backfill Structural  
1439·yd$^3$ + 2154·yd$^3$ = 3593·yd$^3$

Item 08 - Select Granular Fill  
3593·yd$^3$

Item 09 - Compaction, Roller  
3593·yd$^3$

Item 10 - Compaction, Plate  
1.5·ft·21·ft·L = 117·yd$^3$

Item 12 - Excavation  
3593·yd$^3$
FIGURE 5-1
MECHANICALLY STABILIZED EARTH WALL
CHAPTER 6 SOLDIER PILE AND LAGGING WALL

6.1 Design Calculations

Wall Properties

- $H := 19\cdot$ft Wall height
- $L := 100\cdot$ft Wall Length

Retained Soil Properties

- $\gamma_f := 120\cdot$pcf Soil Density
- $\phi_f := 30\cdot$deg Angle of internal friction
- $\delta := 0$ Angle of friction between soil and wall
- $\beta := 90\cdot$deg Batter of Wall, where 90 degrees is vertical
- $\alpha := 18\cdot$deg Slope of Retained Soil
- $c := 0$ Soil Cohesion

Pile Properties

- $F_y := 50\cdot$ksi Yield Strength
Determine Coulomb's Earth Pressure Coefficients

For passive pressure

\[
K_p := \frac{(\sin(\beta - \phi))^2}{\sin(\beta)^2 \cdot \sin(\beta + \delta) \cdot \left[ 1 - \left( \frac{\sin(\phi + \delta) \cdot \sin(\phi + \alpha)}{\sin(\beta + \delta) \cdot \sin(\beta + \alpha)} \right) \right]^{0.5}^2}
\]

\(K_p = 5.33\)  Coulomb's passive earth pressure coefficient

FS := 1.5  Factor of Safety used for \(K_p\)
(Use in lieu of lengthening the pile later)

\(K_p := \frac{K_p}{FS}\)  \(K_p = 3.56\)  Value used for design

For active pressure

\[
K_a := \frac{\sin(\beta + \phi)^2}{\sin(\beta)^2 \cdot \sin(\beta - \delta) \cdot \left[ 1 + \left( \frac{\sin(\phi + \delta) \cdot \sin(\phi - \alpha)}{\sin(\beta - \delta) \cdot \sin(\alpha + \beta)} \right) \right]^{2}}
\]

\(K_a = 0.424\)  Coulomb's active earth pressure coefficient
6.1.1 Pile Design

Determine Pressure Diagram

\[ S := 8 \text{ ft} \quad \text{Spacing of soldier pile} \]
\[ B = 14 \text{ in} \quad \text{Flange width of soldier pile} \]

\[ \text{bf} = \text{effective width} = 3.0B \text{bf} = 3.5 \text{ ft} \quad \text{Effective width of pile including adjustment factor, Value must be less than soldier pile spacing} \]

Consider Area 1, See Figure 6-2

Determine the pressure and force

\[ p_1 := \text{H} \gamma f \cdot K_a \quad \text{pl} = 0.966 \text{ ksf} \quad \text{Active pressure at exc. line} \]

\[ f_1 := \frac{1}{2} p_1 \cdot H \quad fl = 9.18 \text{ kip ft} \quad \text{Total Force per ft. of width due to p1} \]

\[ w_1 := p_1 \cdot S \quad \text{wl} = 7.73 \text{ kip ft} \quad \text{Maximum value of uniform Load (Used for lagging design)} \]

\[ F_1 := f_1 \cdot S \quad \text{Fl} = 73.4 \text{ kip} \quad \text{Force of retained earth due to pressure p1} \]

Determine associated moment arm

\[ T := 4 \text{ ft} \quad \text{Distance of Tie Back from top of pile} \]

\[ A_1 := \text{H} - T - \frac{H}{3} \quad A_1 = 8.67 \text{ ft} \quad \text{Moment Arm for F1 from tie back} \]

\[ M_1 := -F_1 \cdot A_1 \quad M_1 = -636.4 \text{ kip ft} \quad \text{Moment about Tie Back due to P1} \]
Consider Area 2, See Sketch of Pressure Diagram, Fig. 6-2

Determine pressure and force

\[ p_2 := p_1 \quad p_2 = 0.966 \text{ ksf} \quad \text{Active pressure at exc line, same as } p_1 \]

\[ L_2 := \frac{H \cdot Ka}{Kp - Ka} \quad L_2 = 2.57 \text{ ft} \quad \text{Depth of A2 pressure} \]

\[ f_2 := \frac{1}{2} p_2 \cdot L_2 \quad f_2 = 1.241 \frac{\text{kip}}{\text{ft}} \quad \text{Total Force per ft. of width for Area 2} \]

\[ w_2 := p_2 \cdot bf \quad w_2 = 3.38 \frac{\text{kip}}{\text{ft}} \quad \text{Uniform Load} \]

\[ F_2 := f_2 \cdot bf \quad F_2 = 4.3 \text{ kip} \quad \text{Force due to pressure } p_2 \]

Determine associated moment arm

\[ A_2 := A_1 + \frac{H}{3} + \frac{L_2}{3} \quad A_2 = 15.86 \text{ ft} \quad \text{Moment arm for } F_2 \text{ from tieback} \]

\[ M_2 := -F_2 \cdot A_2 \quad M_2 = -68.8 \text{ kip ft} \quad \text{Moment due to pressure in area 2} \]

Consider Area 3, See sketch

\[ L_3 := 7.0 \text{ ft} \quad \text{Depth of pressure for area 3, note: this value is determined} \]

\[ p_3 = \gamma f (Kp - Ka) l \quad p_3 = 2.64 \text{ ksf} \quad \text{Pressure of area 3 due to difference of} \]

\[ f_3 := \frac{1}{2} \gamma f (Kp - Ka) L_3^2 \quad f_3 = 9.2 \frac{\text{kip}}{\text{ft}} \quad \text{Total Force per ft. of width} \]

\[ w_3 := p_3 \cdot bf \quad w_3 = 9.24 \frac{\text{kip}}{\text{ft}} \quad \text{Uniform load at bottom} \]

\[ F_3 := f_3 \cdot bf \quad F_3 = 32.2 \text{ kip} \quad \text{Force due to pressure } p_3 \]
Determine associated moment arm

\[ A_3 := H - T + L_2 + \frac{2\cdot L_3}{3} \quad A_3 = 22.26 \text{ ft} \quad \text{Moment arm for force about tieback} \]

\[ M_3 := F_3 \cdot A_3 \quad M_3 = 716.8 \text{ kip ft} \]

Sum moments about tie back - equal close to 0

\[ M := M_1 + M_2 + M_3 \]

\[ M = +11.6 \text{ kip ft} > 0 \quad \text{Therefore value of } L_3 \text{ above is OK} \]

\[ F := F_1 + F_2 - F_3 \quad F_3 = 45.5 \text{ kip} \quad \text{Tie Back Force} \]

Determine Pile length and depth

\[ \text{Depth} := L_2 + L_3 \quad \text{Depth} = 9.6\text{ ft} \]

\[ \text{Length} := L_2 + L_3 + H \quad \text{Length} = 28.6\text{ ft} \]

Use a 30 ft. pile \( L_p := 30\text{ ft} \)

Determine Pile Section Required

Determine the point of zero shear by summing the forces (Note: Assume zero shear is above the mud line)

\[ x := 15.0\text{ ft} \quad \text{Distance from top of grade determined by trial & error} \]

\[ P_x := \frac{1}{2} \cdot w_1 \cdot \frac{x^2}{H} \quad P_x = 45.5 \text{ kip} \quad \text{Horizontal force due to soil at depth } x \]

\[ V := F - P_x \quad V = 0.0 \text{ kip} \quad \text{Net shear at depth } x, \text{ OK} \]

Determine Required Section Modulus

\[ M := F \cdot (x - T) - P_x \cdot \frac{x}{3} \quad M = 273 \text{ kip ft} \]

\[ S_x := \frac{M}{0.55 \cdot F_y} \quad S_x = 121 \text{ in}^3 \quad \text{Use HP14 x 89} \quad S_x := 131 \cdot \text{in}^3 \]

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6.1.2 Lagging Design

Design Lagging based on maximum load applied

Properties
\[ b := 10\text{-in} \quad \text{Height of Lagging Panel} \quad s := 12\text{-in} \quad \text{Space of Shear Reinf.} \]
\[ h := 10\text{-in} \quad \text{Depth of Lagging Panel} \]
\[ f_y := 60\text{-ksi} \quad \text{Use 60 ksi Reinforcing} \]
\[ f_c := 3.5\text{-ksi} \quad \text{28 Day compressive strength of concrete} \]
\[ f_s := 24\text{-ksi} \quad \text{Allowable tension in reinforcement} \]
\[ w := p \cdot b \quad w = 0.805\text{ kip} \quad \text{Uniform Load at exc line on lagging panel, ft} \quad \text{Equal to pressure at excline x panel width} \]
\[ S = 8\text{-ft} \quad \text{Span of Lagging, Conservative} \]

Determine maximum moment, conservative to design as simply supported beam
Include a load factor of 1.3 for Earth Loads as per AASHTO Table 3.22.1A

\[ M := \frac{w \cdot S^2}{8} \cdot 1.3 \quad M = 8.37\text{ kip ft} \quad \text{Maximum Moment} \]
\[ V := 1.15 \cdot \frac{w \cdot S}{2} \cdot 1.3 \quad V = \text{kip} \quad \text{Maximum Shear} \]

Determine the required reinforcement by service load design

\[ A_s := 3 \cdot 0.31\text{-in}^2 \quad A_s = 0.93\text{-in}^2 \quad \text{Try 3 # 5 bars} \]
\[ d := h - 3\text{-in} \quad d = 7\text{-in} \quad \text{Use 3 in. clearance for Reinforcement} \]
\[ T_s := A_s \cdot f_s \quad T_s = 22.3\text{ kip} \quad \text{Tension & Compression Force} \]
\[ k_d := \frac{2 \cdot T_s}{f_c \cdot b} \quad k_d = 1.28\text{ in} \quad \text{Depth of compression block} \]
\[ M_a := T_s \left( d - \frac{k_d}{3} \right) \quad M_a = 12.2\text{ kip ft} \quad > \quad M = 8.37\text{ kip ft} \]

For Shear \[ v_c := 0.95 \cdot \frac{f_c}{\text{psi}} \quad v_c = 56.2\text{ psi} \quad \text{Allow. Concrete Shear} \]
\[ v := \frac{V}{b \cdot d} \quad v = 68.8\text{ psi} \quad \text{Total Shear} \]
\[ A_v := \frac{(v - v_c) \cdot b \cdot s}{f_s} \quad A_v = 0.06\text{ in}^2 \quad < \quad Av := 0.62\text{-in}^2 \quad \text{OK} \]

Use 10"x10" Panels w/ 3 #5 Bars on E. Face
6.1.3 Grouted Anchor Design
   Use 1" dia. Threadbar \( Fa := 76.5 \text{kip} \) See Chapter 2
   Bonded Length = FS \times F/Fu = 44.7 \times 2.5/13.0 = 8.6' Use 10'

6.2 Quantity Calculations

6.2.1 Pile Quantities

   Soldier Piles, HP 14\times89 \text{(Pay Item 14)}
   \[ S = 8.0 \text{ft} \quad \text{Pile Spacing} \]
   \[ L = 100 \text{ft} \quad \text{Wall Length} \]
   \[ Np := \frac{L}{S} + 1 \quad Np = 13.5 \quad \text{Use} \quad Np := 14 \quad \text{Piles} \]
   \[ Lp = 30 \text{ft} \quad \text{Pile Length} \]
   \[ Qp := Lp \cdot Np \quad Qp = 420.0 \text{ft} \quad \text{Total Length of Soldier Pile} \]

   Grouted Anchor Quantity, one per pile \text{(Pay Item 04)}
   14 Units \times (15' + 10') = 350LF

6.2.2 Lagging Panel Quantities

   Concrete Quantity in Lagging Panels
   \[ H = 19 \text{ft} \quad \text{Height of Wall} \]
   \[ h = 10.0 \text{in} \quad \text{Depth of Lagging Panel} \]
   \[ Lp := 7 \text{ft} \quad \text{Length of Lagging Panel} \]
   \[ Nb := 13 \quad \text{Number of Bays of Lagging Panel along wall} \]
   \[ Qc := Nb \cdot H \cdot h \cdot Lp \quad Qc = 53.4 \text{yd}^3 \quad \text{Concrete Quantity} \]
   \[ A := H \cdot L \quad A = 1900 \text{ft}^2 \quad \text{Exposed Wall Area (Pay Item 29)} \]
6.2.3 Reinforcement Quantity in Lagging Panels (Pay Item 21)

\[ L_r := 6.65 \cdot \text{ft} + 14.8 \cdot \text{in} \quad L_r = 48.3 \cdot \text{ft} \quad \text{Total Bar Length Per Panel} \]

\[ N_{pa} := \frac{H}{b} \quad N_{pa} = 22.8 \quad N_{pa} := 23 \quad \text{Panels per bay} \]

\[ w_t := 1.043 \cdot \frac{\text{lbf}}{\text{ft}} \quad \text{Weight of No. 5 bar} \]

\[ N_b = 13 \quad \text{Number of bays} \]

\[ W_r := L_r \cdot N_b \cdot N_{pa} \cdot w_t \]

\[ W_r = 7.5 \text{ Ton} \quad \text{Total Weight of Reinforcement} \]

6.2.4 Excavation and Backfill

**Excavation (Pay Item 12)**

\[ A := 350.4 \cdot \text{ft}^2 \quad \text{Measured in Cad File} \]

\[ V := A \cdot L \quad V = 1298 \cdot \text{yd}^3 \quad \text{Volume of Excavation} \]

**Excavation for Tie Rods (Pay Item 11)**

\[ T := 5 \cdot \text{ft} \quad \text{Assumed Trench width} \]

\[ A_t := 402 \cdot \text{ft}^2 \quad \text{Area of trench beyond Excavation, measured in CAD} \]

\[ N_p = 14 \quad \text{Number of Tie Rod locations} \]

\[ V_t := A_t \cdot T \cdot N_b \quad V_t = 968 \cdot \text{yd}^3 \quad \text{Volume of Tie Rod Trench excavation} \]

**Granular Backfill Behind Wall, for drainage**

\[ Q_g := 2 \cdot \text{ft} \cdot H \cdot L \quad Q_g = 141 \cdot \text{yd}^3 \]

**Structural Fill**

\[ Q_s := V + V_t - Q_g \quad Q_s = 2125 \cdot \text{yd}^3 \]

**Geotextile**

\[ (19 \cdot \text{ft} + 2 \cdot \text{ft}) \cdot L = 233.3 \cdot \text{yd}^2 \quad \text{Pay Item 16} \]
Excavation related Pay Items

<table>
<thead>
<tr>
<th>Item</th>
<th>Calculation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Item 07 - Backfill Structural</td>
<td>$141 \cdot \text{yd}^3 + 2125 \cdot \text{yd}^3 = 2266 \cdot \text{yd}^3$</td>
</tr>
<tr>
<td>Item 08 - Select Granular Fill</td>
<td>$2266 \cdot \text{yd}^3$</td>
</tr>
<tr>
<td>Item 09 - Compaction, Roller</td>
<td>$2266 \cdot \text{yd}^3$</td>
</tr>
<tr>
<td>Item 10 - Compaction, Plate</td>
<td>$1.5 \cdot \text{ft} \cdot 19 \cdot \text{ft} \cdot \text{L} = 106 \cdot \text{yd}^3$</td>
</tr>
<tr>
<td>Item 11 - Excavation for Tie Rods</td>
<td>$968 \cdot \text{yd}^3$</td>
</tr>
<tr>
<td>Item 12 - Excavation</td>
<td>$1298 \cdot \text{yd}^3$</td>
</tr>
</tbody>
</table>
FIGURE 6-1
SOLDIER PILE AND LAGGING WALL
FIGURE 6-2
SOLDIER PILE AND LAGGING WALL
FORCE & PRESSURE DIAGRAM

F3 = 32.2K
W3 = 9.24 K/FT

F1 = 73.4K
W1 = 7.73 K/FT
F2 = 4.3K

L2 = 2.57'
L3 = 7.0'

F = 45.5K
CHAPTER 7 SLURRY WALL

7.1 Design Calculations

Wall Properties

\[ H := 19\cdot\text{ft} \quad \text{Exposed Wall height} \]
\[ L := 100\cdot\text{ft} \quad \text{Wall Length} \]

Retained Soil Properties

\[ \gamma_f := 120\cdot\text{pcf} \quad \text{Soil Density} \]
\[ \phi_f := 30\cdot\text{deg} \quad \text{Angle of internal friction} \]
\[ \delta := 0 \quad \text{Angle of friction between soil and wall} \]
\[ \beta := 90\cdot\text{deg} \quad \text{Batter of Wall, where 90 degrees is vertical} \]
\[ \alpha := 18\cdot\text{deg} \quad \text{Slope of Retained Soil} \]
\[ c := 0 \quad \text{Soil Cohesion} \]

Concrete and Reinforcement Properties

\[ f_c := 4\cdot\text{ksi} \quad \text{28 Day compressive strength} \]
\[ f_y := 60\cdot\text{ksi} \quad \text{Yield strength of reinforcement} \]
Determine Coulomb's Earth Pressure Coefficients

For passive pressure

\[
K_p := \frac{(\sin(\beta - \phi f))^2}{\sin(\beta)^2 \cdot \sin(\beta + \delta) \cdot \left[1 - \left(\frac{\sin(\phi f + \delta) \cdot \sin(\phi f + \alpha)}{\sin(\beta + \delta) \cdot \sin(\beta + \alpha)}\right)^{0.5}\right]^2}
\]

\(K_p = 5.33\)  Coulomb's passive earth pressure coefficient

FS := 1.5 Factor of Safety used for \(K_p\)
(Use in lieu of lengthening wall embedment depth later)

\[
K_p := \frac{K_p}{FS}
\]

\(K_p = 3.56\)  Value used for design

For active pressure

\[
K_a := \frac{\sin(\beta + \phi f)^2}{\sin(\beta)^2 \cdot \sin(\beta - \delta) \cdot \left[1 + \left(\frac{\sin(\phi f + \delta) \cdot \sin(\phi f - \alpha)}{\sin(\beta - \delta) \cdot \sin(\alpha + \beta)}\right)^2\right]}
\]

\(K_a = 0.424\)  Coulomb's active earth pressure coefficient
7.1.1 Cantilever Wall Design

Determine Pressure Diagram, Ref. 2, Pp. 458-462

Consider Area 1

Determine the pressure and force

\[ p_1 := H \cdot \gamma_f \cdot K_a \quad p_1 = 0.966 \text{ ksf} \quad \text{Active pressure at exc line} \]

\[ f_1 := \frac{1}{2} \cdot p_1 \cdot H \quad f_1 = 9.18 \frac{\text{kip}}{\text{ft}} \quad \text{Total Force per ft. of width due to } p_1 \]

Consider Area 2

Determine pressure and force

\[ p_2 := p_1 \quad p_2 = 0.966 \text{ ksf} \quad \text{Active pressure at exc line, same as } p_1 \]

\[ L_2 := \frac{H \cdot K_a}{K_p - K_a} \quad L_2 = 2.57 \text{ ft} \quad \text{Depth of A2 pressure} \]

\[ f_2 := \frac{1}{2} \cdot p_2 \cdot L_2 \quad f_2 = 1.241 \frac{\text{kip}}{\text{ft}} \quad \text{Total Force per ft. of width for Area 2} \]

Determine moment arms from point E for \( f_1 \) & \( f_2 \)

\[ z_1 := L_2 + \frac{H}{3} \quad z_1 = 8.9 \text{ ft} \quad \text{Moment arm for } f_1 \]

\[ z_2 := \frac{2}{3} \cdot L_2 \quad z_2 = 1.7 \text{ ft} \quad \text{Moment arm for } f_2 \]

Determine \( z \) & \( P \)

\[ P := f_1 + f_2 \quad P = 10.4 \frac{\text{kip}}{\text{ft}} \quad \text{Horizontal force due to Passive Pressure} \]

\[ z := \frac{f_1 \cdot z_1 + f_2 \cdot z_2}{f_1 + f_2} \quad z = 8.0 \text{ ft} \quad \text{Distance from Point E} \]
Determine $L4$ by trial and error

$$p5 := \gamma_f \cdot H \cdot Kp + \gamma_f \cdot L2 \cdot (Kp - Ka)$$  \hspace{1cm} p5 = 9.07 \text{ ksf}

$$A1 := \frac{p5}{\gamma_f \cdot (Kp - Ka)} \hspace{1cm} A1 = 24.1 \text{ ft}$$

$$A2 := \frac{8 \cdot (f1 + f2)}{\gamma_f \cdot (Kp - Ka)} \hspace{1cm} A2 = 221.6 \text{ ft}^2$$

$$A3 := \frac{6 \cdot P \cdot (2 \cdot z \cdot \gamma_f \cdot (Kp - Ka) + p5)}{\gamma_f^2 \cdot (Kp - Ka)^2} \hspace{1cm} A3 = 6687.2 \text{ ft}^3$$

$$A4 := \frac{P \cdot (6 \cdot z \cdot p5 + 4 \cdot P)}{\gamma_f^2 \cdot (Kp - Ka)^2} \hspace{1cm} A4 = 35353.1 \text{ ft}^4$$

$L4 := 17.4 \text{ ft} \hspace{1cm}$ Solved by Trial and Error

$$L4^4 + A1 \cdot L4^3 - A2 \cdot L4^2 - A3 \cdot L4 - A4 = 0.8 \text{ ft}^4 \hspace{1cm} \text{Close Enough to 0.0}$$

Total distance below Exc Line Required

$D := L2 + L4 \hspace{1cm} D = 20.0 \text{ ft}$

Determine pressures and distances

$$p4 := p5 + \gamma_f \cdot L4 \cdot (Kp - Ka) \hspace{1cm} p4 = 15.6 \text{ ksf}$$

$$p3 := \gamma_f \cdot L4 \cdot (Kp - Ka) \hspace{1cm} p3 = 6.5 \text{ ksf}$$

$$L5 := \frac{p3 \cdot L4 - 2 \cdot P}{p3 + p4} \hspace{1cm} L5 = 4.2 \text{ ft}$$
7.1.2 Wall Reinforcement Design

Determine Maximum Bending Moment

\[ zp := \frac{2 \cdot P}{(Kp - Ka) \cdot \gamma_f} \quad zp = 7.4 \text{ ft} \quad \text{Point of zero shear} \]

\[ M_{\text{max}} := P \cdot (z + zp) - \left[ \frac{1}{2} \cdot \gamma_f \cdot zp^2 \cdot (Kp - Ka) \right] \cdot \frac{1}{3} \cdot zp \]

\[ M_{\text{max}} = 135.5 \text{ kip ft} \quad \text{Maximum service load moment ft} \]

Design Reinforcement for slurry wall

\[ \rho_{\text{min}} := \frac{200}{f_y} \quad \rho_{\text{min}} = 0.0033 \quad \text{Grade 60} \]

\[ b := 12\text{-in} \quad \text{Consider 12 in. wide wall segment} \]

\[ h := 3\text{-ft} \quad \text{Wall thickness} \]

\[ d := h - 4.5\text{-in} \quad d = 31.5\text{-in} \quad \text{Concrete depth to cl of reinforcement} \]

\[ A_{\text{min}} := b \cdot d \cdot \rho_{\text{min}} \quad A_{\text{min}} = 1.26 \text{ in}^2 \quad \text{per 1 ft. width} \]

Use No. 11 bars at 9 in. spacing

\[ A_s := 1.56 \cdot \frac{\text{in}^2 \cdot 12}{9} \quad A_s = 2.1 \text{ in}^2 \quad \text{per 1 ft. width} \]

\[ a = \frac{A_s \cdot f_y}{0.85 \cdot f_c \cdot b} \quad a = 3.1 \text{ in} \quad \text{Depth of compression block} \]

\[ \phi \cdot M_n = 0.9 \cdot A_s \cdot \left( d - \frac{a}{2} \right) \quad \phi \cdot M_n = 281 \text{ kip ft} \quad \text{OK} \]

\[ \phi \cdot V_c = 0.85 \cdot 2 \cdot \sqrt{\frac{f_c}{\gamma \text{psi}}} \cdot b \cdot d \cdot \rho \quad \phi \cdot V_c = 40.6 \text{ kip} \]

\[ \phi \cdot V_c + \phi \cdot V_s > V \quad \text{OK} \]
7.2 Quantity Calculations

7.2.1 Concrete Quantity (Pay Item 23 & 26)
\[ Q_f := (H + 1 \cdot \text{ft}) \cdot 6 \cdot \text{in} \cdot L \quad Q_f = 37.0 \cdot \text{yd}^3 \quad \text{CIP Finish Wall Portion Only} \]

Where:
- \( H = 19 \cdot \text{ft} \) Exposed Wall Height
- \( 6'' = \text{Finish Wall Thickness} \)
- \( L = 100 \cdot \text{ft} \) Wall Length

Formwork for finish wall (Pay Item 20)
\[ A := (H + 1 \cdot \text{ft}) \cdot L \quad A = 2000 \cdot \text{ft}^2 \]

7.2.2 Reinforcement Quantity (Pay Item 21)

Vertical Bars, No. 11 @ 9'' Each Face
\[ w_{11} := 5.313 \frac{\text{lbf}}{\text{ft}} \quad \text{Bar weight} \]

\[ L_{11} := H + D - 0.5 \cdot \text{ft} \quad L_{11} = 38.5 \cdot \text{ft} \quad \text{Bar Length} \]

\[ N_{11} := \frac{L}{9 \cdot \text{in}} \quad N_{11} = 133.3 \quad N_{11} := 134 \quad \text{No of bars along length Each Face} \]

\[ W_{11} := w_{11} \cdot L_{11} \cdot N_{11} \cdot 2 \quad W_{11} = 54800 \text{ lbf} \quad \text{Weight of vert. bars} \]

Horizontal Bars, No. 5 @ 12'' Each Face
\[ w_5 := 1.043 \frac{\text{lbf}}{\text{ft}} \quad \text{Bar Weight} \]

\[ L_5 := L - 0.5 \cdot \text{ft} \quad L_5 = 99.5 \cdot \text{ft} \quad \text{Bar Length} \]

\[ N_5 := \frac{H + D - 0.5 \cdot \text{ft}}{12 \cdot \text{in}} \quad N_5 = 38.5 \quad N_5 := 39 \quad \text{No of bars along height Each Face} \]

\[ W_5 := w_5 \cdot L_5 \cdot N_5 \cdot 2 \quad W_5 = 8095 \text{ lbf} \quad \text{Weight of horiz bars} \]

Finish Face Bars, #4 @ 12'' Each Face
\[ w_4 := 0.668 \frac{\text{lbf}}{\text{ft}} \quad N_{4h} := \frac{H + 1 \cdot \text{ft} - 0.5 \cdot \text{ft}}{12 \cdot \text{in}} \quad N_{4h} = 19.5 \quad N_{4h} := 20 \]

\[ N_{4v} := \frac{L}{12 \cdot \text{in}} \quad N_{4v} = 100 \quad L_4 := H + 1 \cdot \text{ft} - 0.5 \cdot \text{ft} \quad L_4 = 19.5 \cdot \text{ft} \]

\[ W_4 := w_4 \cdot (L_5 \cdot N_{4h} + L_4 \cdot N_{4v}) \quad W_4 = 2631.9 \text{ lbf} \quad \text{Wt. of #4 bars} \]

Total Weight of Reinforcement (Pay Item 21)
\[ W := W_{11} + W_5 + W_4 \quad W = 32.7 \text{ Ton} \]
FIGURE 7-1
SLURRY WALL
7.2.3 Excavation and Backfill

Determine Quantity for drainage at front face of wall

\[ Ae := 7.3 \cdot \text{ft}^2 \quad \text{Cross-Section Area Measured in CAD File} \]
\[ Qe := Ae \cdot L \quad Qe = 27.0 \cdot \text{yd}^3 \quad \text{Excavation and Backfill Quantity} \]

Determine Quantity for drainage at back face of wall

\[ Agr := 132 \cdot \text{ft}^2 \quad \text{Cross-Section Area Measured in CAD File} \]
\[ Qgr := Agr \cdot L \quad Qgr = 488.9 \cdot \text{yd}^3 \quad \text{Excavation and Backfill Quantity} \]

Total Excavation and Backfill Quantity

\[ Qt := Qe + Qgr \quad Qt = 515.9 \cdot \text{yd}^3 \]

Slurry Trench Excavation and Backfill with concrete (Pay Item 17)

\[ Qt := (H + D) \cdot h \cdot L \quad Qt = 11690.6 \cdot \text{ft}^3 \]

Where:
- \( H = 19 \cdot \text{ft} \) Exposed Wall Height
- \( D = 20 \cdot \text{ft} \) Depth of wall below grade
- \( h = 3 \cdot \text{ft} \) Wall Thickness
- \( L = 100 \cdot \text{ft} \) Wall Length

Geotextile around drainage pocket (Pay Item 16)

\[ Agt := 26 \cdot \text{ft} \cdot L \quad Agt = 288.9 \cdot \text{yd}^2 \]

Excavation related Pay Items

Item 07 - Backfill Structural \quad 515.9 \cdot \text{yd}^3
Item 08 - Select Granular Fill \quad 515.9 \cdot \text{yd}^3
Item 10 - Compaction, Plate \quad 515.9 \cdot \text{yd}^3
Item 12 - Excavation \quad 515.9 \cdot \text{yd}^3
<table>
<thead>
<tr>
<th>Item No.</th>
<th>Pay Item</th>
<th>RS Means Section Referenced</th>
<th>Unit</th>
<th>Daily Output</th>
<th>Unit Cost</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>01</td>
<td>Sheet piling, 15 ft deep excavation</td>
<td>31 41</td>
<td>0020</td>
<td>TN</td>
<td>10.81</td>
<td>$ 2,050.00 Used for anchor wall</td>
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<tr>
<td>02</td>
<td>Sheet piling, 20 ft deep excavation</td>
<td>31 41</td>
<td>0300</td>
<td>TN</td>
<td>12.95</td>
<td>$ 1,950.00</td>
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<tr>
<td>03</td>
<td>Wales, connections &amp; struts</td>
<td></td>
<td>2500</td>
<td>TN</td>
<td>NA</td>
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<tr>
<td>04</td>
<td>Tie rod, upset, 1.75 in. to 4 in. dia with turnbuckle</td>
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<td>31 41</td>
<td>3300</td>
<td>TN NA</td>
<td>$ 2,700.00</td>
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<td>05</td>
<td>Grouted Anchors difficult 30'</td>
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<td>31 32</td>
<td>1420</td>
<td>LF 360.00</td>
<td>$ 20.20</td>
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<td>06</td>
<td>No Item</td>
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<tr>
<td>07</td>
<td>Backfill structural 105 H.P., 150 ft. haul, sand &amp; gravel</td>
<td>31 23</td>
<td>3200</td>
<td>LCY</td>
<td>670.00</td>
<td>$ 2.02 Does Not include materials</td>
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<td>08</td>
<td>Borrow loading Select granular fill</td>
<td>31 23</td>
<td>5000</td>
<td>BCY</td>
<td>NA</td>
<td>$ 13.86 Used to determine material price only (+ 10% profit)</td>
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<td>09</td>
<td>Compaction, riding, vibrating roller 12 in. lift, 2 passes</td>
<td>31 23</td>
<td>5060</td>
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<td>10</td>
<td>Compaction, walk behind vibrating plate 12 in. lift, 2 passes</td>
<td>31 23</td>
<td>7200</td>
<td>ECY</td>
<td>560.00</td>
<td>$ 0.78 Compaction method at wall edges, 18 in. width</td>
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<td>11</td>
<td>Excavation, trench, common earth 6 ft to 10 ft deep, 1.5 cy hydraulic backhoe</td>
<td>31 23</td>
<td>0610</td>
<td>BCY</td>
<td>600.00</td>
<td>$ 3.10 Trench for Tie Backs</td>
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<td>12</td>
<td>Excavation, trench, common earth 14 ft to 20 ft deep, 1.5 cy hydraulic backhoe</td>
<td>31 23</td>
<td>1310</td>
<td>BCY</td>
<td>480.00</td>
<td>$ 3.86 Main Excavating</td>
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<td>13</td>
<td>Driven piles, H sections HP10x42, to 50 ft. length</td>
<td>31 62</td>
<td>0400</td>
<td>VLF</td>
<td>610.00</td>
<td>$ 32.00</td>
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<td>14</td>
<td>Driven piles, H sections HP14x117 to 50 ft. length</td>
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<td>1400</td>
<td>VLF</td>
<td>510.00</td>
<td>$ 76.50</td>
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<td>15</td>
<td>Driven piles, complete pile driving setup Mobilization, large</td>
<td>31 06</td>
<td>1200</td>
<td>EA</td>
<td>0.27</td>
<td>$ 22,000.00</td>
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<td>16</td>
<td>Geotextile for subsurface drainage Fabric, laid in trench, adverse conditions</td>
<td>33 46</td>
<td>0110</td>
<td>SY</td>
<td>1600.00</td>
<td>$ 2.18</td>
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</tbody>
</table>
## NORTH AMERICAN STEEL SHEET PILING ASSOCIATION
### RETAINING WALL STUDY
#### Appendix A: RS Means Pay Items, Heavy Construction 2009

<table>
<thead>
<tr>
<th>Item No.</th>
<th>Pay Item</th>
<th>RS Means Section Referenced</th>
<th>Unit</th>
<th>Daily Output</th>
<th>Unit Cost</th>
<th>Comment</th>
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<tbody>
<tr>
<td>17</td>
<td>Slurry Trench, excavated in wet soils</td>
<td>Backfilled w/ 3ksi concrete, no reinforcement</td>
<td>31 56</td>
<td>23.20</td>
<td>0050 CF</td>
<td>333.00 $ 23.50</td>
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<tr>
<td>18</td>
<td>Forms in place, footing</td>
<td>Continuous wall, plywood, 2x</td>
<td>03 11</td>
<td>13.45</td>
<td>0050 SFCA</td>
<td>440.00 $ 2.80</td>
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<td>19</td>
<td>Forms in place, footing</td>
<td>Integral starter wall, to 4 in</td>
<td>03 11</td>
<td>13.45</td>
<td>1000 LF</td>
<td>400.00 $ 5.55</td>
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<td>20</td>
<td>Steel framed plywood</td>
<td>16 ft to 20 ft high</td>
<td>03 11</td>
<td>13.85</td>
<td>9460 SFCA</td>
<td>400.00 $ 8.15 Used for forming walls</td>
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<tr>
<td>21</td>
<td>Reinforcing steel, A615 Gr 60</td>
<td>10 - 50 ton job, #3 to #7 bars</td>
<td>03 21</td>
<td>10.60</td>
<td>1050 TN</td>
<td>2.10 $ 2,825.00 Reasonable fit for applicable walls</td>
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<tr>
<td>22</td>
<td>Welded wire fabric</td>
<td>6x6, W4xW4, 58psf/csf</td>
<td>03 22</td>
<td>05.50</td>
<td>0400 CSF</td>
<td>27.00 $ 94.00 Best match for strip steel reinforcing in CMU gravity wall</td>
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<td>23</td>
<td>Concrete, ready mix</td>
<td>Normal weight, 3500 psi</td>
<td>03 30</td>
<td>05.35</td>
<td>0200 CY</td>
<td>114.00 $ 114.00 Average strength for various components</td>
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<td>24</td>
<td>Placing concrete, footings</td>
<td>Continuous, shallow, direct chute</td>
<td>03 31</td>
<td>05.70</td>
<td>1900 CY</td>
<td>120.00 $ 21.00 Assume for precasting operations of CMU gravity wall</td>
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<td>25</td>
<td>Placing concrete, footings</td>
<td>Continuous, shallow pumped</td>
<td>03 31</td>
<td>05.70</td>
<td>1950 CY</td>
<td>150.00 $ 28.00</td>
</tr>
<tr>
<td>26</td>
<td>Placing concrete, walls</td>
<td>15 in thk, pumped</td>
<td>03 31</td>
<td>05.70</td>
<td>5350 CY</td>
<td>120.00 $ 35.00</td>
</tr>
<tr>
<td>27</td>
<td>Placing concrete with crane</td>
<td></td>
<td>03 31</td>
<td>05.70</td>
<td>5400 CY</td>
<td>95.00 $ 53.50 Assume for placing precast segments of CMU gravity wall</td>
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<tr>
<td>28</td>
<td>No Item</td>
<td></td>
<td></td>
<td></td>
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<td>29</td>
<td>Precast concrete wall panels</td>
<td>10 in. thick</td>
<td>03 47</td>
<td>13.50</td>
<td>0100 SF</td>
<td>1550.00 $ 22.68 With 33.3% increase in material price for thickness</td>
</tr>
<tr>
<td>30</td>
<td>Galvanizing steel in shop</td>
<td>1 ton to 20 tons</td>
<td>05 05</td>
<td>13.50</td>
<td>5950 TN</td>
<td>875.00 $ 875.00 For strip steel reinforcing in CMU gravity wall</td>
</tr>
</tbody>
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Appendix B: References

   http://www.dot.ca.gov/hq/construction/construc.htm

