Sheet Piling Walls:
Cantilever Walls
Anchored Walls
Braced Cuts
Overview of Sheet Piling as a Retaining Wall

- Sheet piling is a structural “in-situ” type of retaining wall
  - Does not rely on its mass to retain the soil, as opposed to a gravity wall
  - In-situ walls rely on their flexural strength to retain soil, supported either by their own penetration into the soil or by an anchoring system
- Other types of structural in-situ walls
  - Soldier pile walls – use H-beams to hold timber or concrete lagging to retain soil on a temporary or permanent basis
  - Slurry walls – bentonite slurry is injected into a trench after which reinforcement and concrete are placed into the trench, forming a wall
Materials for Sheet Piling

- Steel
  - Cold formed
  - Hot rolled
- Aluminium
  - Extruded
- Vinyl
  - Extruded
- Fibreglass
  - Pultruded
- Concrete
- Wood
Steel Sheet Piles

- **Hot rolled**
  - Panel and interlocks rolled in one operation
  - “Traditional” form of steel sheet piling

- **Cold formed**
  - Form rolled cold from steel plate
  - Common with lighter sheet pile profiles
  - Interlocks more prone to breakage

a. Hot-rolled Z-section

b. Cold-rolled Z-section
Concrete and Wood Sheeting

- Concrete Sheeting
  - Grouted
  - Tongue and Groove

- Wood Sheeting
  - Butt-ended
  - Tongue and Groove
  - Splint Fastened
Aluminium, Vinyl and Fibreglass Sheeting

- Made for lightweight and light load applications
- Common substitute for wood or concrete walls
- Require special handling in setting and driving
- Vinyl sheets can be obtained in various colours, but is subject to long term creep
Sections of Sheet Piling

- **Z-shaped sheeting**
  - Popular in north America
  - Usually drive two at a time with split clamp
  - Wall stiffness developed with each sheet without assumed assistance from the interlocks

- **U-shaped sheeting** (Larssen, etc.)
  - Very popular in Europe
  - Usually driven one at a time
  - Wall stiffness developed with two sheets and load transferred using the interlocks (European practice; U.S. practice does not assume this load transfer)
Sections of Sheet Piling

- **Arched shaped**
  - Used for shallower wall construction
  - Used in cold formed steel and aluminium sheeting

- **Flat-web sheeting**
  - Almost exclusively used for cellular cofferdams
  - Main stress is tensile through the web and interlocks
  - Can be driven singly or two at a time
Transitional Sections and Interlock Styles

- **Transitional Sections**
  - Fabricated corners
  - Rolled corners

- **Interlock Styles**
  - Hot rolled and extruded sections
    - Ball and socket
    - Single or double jaw
    - Double hook
    - Thumb and finger
      - One point contact
      - Three point contact
  - Cold formed sections
    - Hook and grip

Ball and Socket (BS)
Double Jaw (DJ)
Single Jaw (SJ)
Double Hook (DH)
Thumb and Finger - three point contact (TF)
Thumb and Finger - one point contact (TFX)
Hook and Grip (HG)
# Specifications for Steel Sheet Piling (USS)

<table>
<thead>
<tr>
<th>Steel Grade</th>
<th>Allowable Stress, ksi</th>
<th>Allowable Stress, MPa</th>
</tr>
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<tbody>
<tr>
<td>ASTM A 328</td>
<td>25</td>
<td>172</td>
</tr>
<tr>
<td>ASTM A 572 Gr. 45</td>
<td>29</td>
<td>200</td>
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<tr>
<td>ASTM A 572 Gr. 50</td>
<td>32</td>
<td>220</td>
</tr>
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<td>ASTM A 572 Gr. 55</td>
<td>35</td>
<td>241</td>
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<tr>
<td>ASTM A 690</td>
<td>32</td>
<td>220</td>
</tr>
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### Steel Sheet Piling Sections

<table>
<thead>
<tr>
<th>Profile</th>
<th>Section Index</th>
<th>District Rolled</th>
<th>Driving Distance per Pile</th>
<th>Weight</th>
<th>Web Thickness</th>
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<tbody>
<tr>
<td>PSX32</td>
<td>H.</td>
<td>16½</td>
<td>44.0</td>
<td>32.0</td>
<td>³/₄</td>
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<tr>
<td>PS32*</td>
<td>H.S.</td>
<td>15</td>
<td>40.0</td>
<td>32.0</td>
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<td>H.S.</td>
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<td>35.0</td>
<td>28.0</td>
<td>³/₄</td>
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<td>PSA28*</td>
<td>H.</td>
<td>16</td>
<td>37.3</td>
<td>28.0</td>
<td>½</td>
</tr>
<tr>
<td>PSA23</td>
<td>H.S.</td>
<td>16</td>
<td>30.7</td>
<td>23.0</td>
<td>³/₄</td>
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<tr>
<td>PDA27</td>
<td>H.</td>
<td>16</td>
<td>36.0</td>
<td>27.0</td>
<td>³/₄</td>
</tr>
<tr>
<td>PMA22</td>
<td>H.S.</td>
<td>19½</td>
<td>36.0</td>
<td>22.0</td>
<td>³/₄</td>
</tr>
<tr>
<td>PZ38</td>
<td>H.</td>
<td>18</td>
<td>57.0</td>
<td>38.0</td>
<td>³/₄</td>
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<tr>
<td>PZ32</td>
<td>H.</td>
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<td>PZZ7</td>
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<td>27.0</td>
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</tr>
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</table>

### Suggested Allowable Design Stresses—Sheet Piling

<table>
<thead>
<tr>
<th>Steel Brand or Grade</th>
<th>Minimum Yield Point, psi</th>
<th>Allowable Design Stress, psi*</th>
</tr>
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<tbody>
<tr>
<td>USS-Ext-TEN 55 (ASTM A572 Gr 55)</td>
<td>55,000</td>
<td>35,000</td>
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<tr>
<td>USS-Ext-TEN 50 (ASTM A572 Gr 50)</td>
<td>50,000</td>
<td>32,000</td>
</tr>
<tr>
<td>USS MARINER STEEL</td>
<td>50,000</td>
<td>32,000</td>
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<tr>
<td>USS Ext-TEN 45 (ASTM A572 Gr 45)</td>
<td>45,000</td>
<td>28,000</td>
</tr>
<tr>
<td>Regular Carbon Grade (ASTM A 690)</td>
<td>38,500</td>
<td>25,000</td>
</tr>
</tbody>
</table>

*Based on 65% of minimum yield point. Some increase for temporary overstresses generally permissible.

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*Sections PS32 and PSA28 are infrequently rolled and we do not advise their use in a design unless an adequate tonnage can be ordered at one time to assure a minimum rolling.

Complete data regarding these sections will be found in a separate publication entitled "USS Steel Sheet Piling."*

H—Homestead, Pa. (Pittsburgh District)
S—South Chicago (Chicago District)
# PZ/PS Sections (Skyline)

## PZ/PS

### PZ/PS Hot Rolled Steel Sheet Piling

## TYPICAL PZ/PS SECTION

<table>
<thead>
<tr>
<th></th>
<th></th>
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<tr>
<td><strong>PZ 22</strong></td>
<td>22.0</td>
<td>9.0</td>
<td>0.375</td>
<td>0.375</td>
<td>6.47</td>
<td>0.77</td>
<td>19.1</td>
<td>14.4</td>
<td>64.38</td>
<td>0.28</td>
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<td>0.375</td>
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<td>1.09</td>
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<td>0.35</td>
<td>0.55</td>
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<td><strong>PZ 32</strong></td>
<td>20.0</td>
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<td>0.500</td>
<td>0.500</td>
<td>10.29</td>
<td>1.54</td>
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<td>0.500</td>
<td>11.77</td>
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<td>25.3</td>
<td>215.9</td>
<td>0.45</td>
<td>0.60</td>
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## Available Steel Grades

<table>
<thead>
<tr>
<th>ASTM</th>
<th>Yield Strength</th>
<th>ASTM</th>
<th>Yield Strength</th>
<th>Interlock Strength</th>
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<tr>
<td></td>
<td>(ksi)</td>
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<td>(ksi)</td>
<td>(ksi)</td>
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<tr>
<td>A323</td>
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<td>A323</td>
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<td>A572 Grade 50</td>
<td>50</td>
<td>A572 Grade 60</td>
<td>60</td>
<td>24</td>
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<tr>
<td>A572 Grade 65</td>
<td>65</td>
<td>A572 Grade 65</td>
<td>65</td>
<td>24</td>
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<tr>
<td>A588</td>
<td>50</td>
<td>A588</td>
<td>50</td>
<td>20</td>
</tr>
<tr>
<td>A690</td>
<td>50</td>
<td>A690</td>
<td>50</td>
<td>20</td>
</tr>
</tbody>
</table>

## Corner and Junction Piles

- **Female or Male Corner**
  - **PC-σ**: 90°
  - **MC-σ**: 120°

## Delivery Conditions & Tolerances

| ASTM A 6 |  |
|----------|  |
| Width: ± 3/8 in | Length: ± 5 in |

### Maximum Rolled Lengths

- **PZ**: 85 ft max, 70 ft max for pairs
- **PS**: 60 ft max

*Maximum lengths may be available upon request.*
## SKZ/SCZ Sections (Skyline)

### Section Dimensions

<table>
<thead>
<tr>
<th>SECTION</th>
<th>Width (w)</th>
<th>Weight (h)</th>
<th>Thickness (t)</th>
<th>Cross Sectional Area</th>
<th>Pile</th>
<th>Wall</th>
<th>Elastic Plastic</th>
<th>Moment of Inertia</th>
<th>Back's Sides</th>
<th>Coating Area</th>
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<tbody>
<tr>
<td>SKZ 20</td>
<td>28.50</td>
<td>72.49</td>
<td>0.315</td>
<td>7.00</td>
<td>56.2</td>
<td>40.1</td>
<td>6.30</td>
<td>80.6</td>
<td>7.60</td>
<td>1.60</td>
</tr>
<tr>
<td>SKZ 22</td>
<td>28.50</td>
<td>72.49</td>
<td>0.335</td>
<td>6.70</td>
<td>51.3</td>
<td>40.1</td>
<td>6.30</td>
<td>74.6</td>
<td>7.60</td>
<td>1.60</td>
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<tr>
<td>SKZ 23</td>
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<td>46.0</td>
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<td>6.30</td>
<td>69.0</td>
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<td>1.60</td>
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<td>39.9</td>
<td>30.2</td>
<td>6.30</td>
<td>63.4</td>
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<td>1.60</td>
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<td>SKZ 25</td>
<td>28.50</td>
<td>72.49</td>
<td>0.395</td>
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<td>33.8</td>
<td>25.3</td>
<td>6.30</td>
<td>58.1</td>
<td>7.60</td>
<td>1.60</td>
</tr>
</tbody>
</table>

### Corner Piles

**SCZ 14 - SCZ 16**
- A = 3.5 inches (89.9 mm)
- B = 2.5 inches (63.5 mm)

**SKZ 20 - SKZ 25**
- A = 3.5 inches (89.9 mm)
- B = 2.25 inches (57.2 mm)

**SCZ 22 - SCZ 30**
- A = 3.5 inches (89.9 mm)
- B = 2.0 inches (50.8 mm)

### Delivery Conditions & Tolerances

- **A106**: 0.25K
- **Length**: +5 inches, -0 inches
- **Tolerance**: +2.00 inches
- **Straightness**: 0.25% of the length
- **Twist**: 0.02% of the width

### Maximum Rolled Lengths
- **M, SCZ**: 70 feet (21.3 m)
- **Other**: 50 feet (15.2 m)

### Optional Accessories

- Sheet Pile Protector
- Cap
- Washer
Cantilever and Anchored Walls

- **Anchored Walls**
  - Walls which have additional supports buried in the soil (tiebacks)

- **Cantilever Walls**
  - Walls which have no additional supports, and which rely on the lateral earth pressures in the lower portion of the wall to support the earth in the upper portion
  - Limited in height and soil type
  - Almost exclusively done with steel piling
  - Generally restricted to temporary structures
Failure of Sheet Pile Walls

• Geotechnical Failure
  ○ Deep Seated Failure
  ○ Inadequate Penetration

• Structural Failure
  ○ Flexural Failure
  ○ Anchorage Failure (can be geotechnical)
Methods of Solution for Cantilever Walls

• Conventional Method
  - Involves analysing active and passive pressures on sheet pile wall per layer
  - Traditionally the most common method used
  - Versatile but requires some experience for proficiency
  - Complex soil profiles can be difficult

• Simplified Method
  - Variation of conventional method
  - Eliminates problems at wall toe
  - Still requires some proficiency

• Chart Method
  - Very straightforward to use
  - Only applicable to simplest cases
  - Good check on other methods

• Closed Form Solution
  - Math complex but more straightforward
  - Limited number of cases

• Computer Software
  - For complex soil profiles, only practical solution
Design of Cantilever Walls:

**Conventional Method**

**1.** Assume a trial depth of penetration, \( D \). This may be estimated from the following approximate correlation.

<table>
<thead>
<tr>
<th>Standard Penetration Resistance, ( N ) Blows/foot</th>
<th>Depth of Penetration*</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 4</td>
<td>2.0H</td>
</tr>
<tr>
<td>5 - 10</td>
<td>1.5H</td>
</tr>
<tr>
<td>11 - 30</td>
<td>1.25H</td>
</tr>
<tr>
<td>31 - 50</td>
<td>1.0H</td>
</tr>
<tr>
<td>≥ 50</td>
<td>0.75H</td>
</tr>
</tbody>
</table>

* \( H \) = height of piling above dredge line

2. Determine the active and passive lateral pressure using appropriate coefficients of lateral earth pressure. If the Coulomb method is used, it should be used conservatively for the passive pressure.

3. Satisfy the requirements of static equilibrium: the sum of the forces in the horizontal direction must be zero and the sum of the moments about any point must be zero. The sum of the horizontal forces may be written in terms of pressure areas:

\[ \Delta(FA) + \Delta(FB2) - \Delta(FC) = 0 \]

Solve the above equation for the distance, \( z \). For a uniform granular soil,

\[ z = \frac{K_p D^2 - K_a (H+D)^2}{(K_p - K_a) (H+2D)} \]

**FIGURE 23**
Analysis for Cantilever Wall

4. Take moments about point \( F \). If sum of moments is other than zero, readjust \( D \) and repeat calculations until sum of moments around \( F \) is zero.

5. Compute maximum moment at point of zero shear.

6. Increase \( D \) by 20% - 40% to result in approximate factor of safety of 1.5 to 2.

**Notes:**

A) For cantilever or anchored sheet pile walls, you may use either Rankine earth pressure coefficients for both \( K_a \) and \( K_p \) or the ASD/DS scheme (Coulomb \( K_a \)/log-spiral \( K_p \)).

B) Re item 6, a better way of including the factor of safety is to divide the \( K_p \) by 1.5-2. Once this is done, no further consideration of factor of safety is necessary. Murthy suggests this for anchored walls but this is also acceptable for cantilever walls as well, and is better practice in both cases.

C) Do not confuse the method in 6 with the \( D \) increase in the "simplified" method discussed in Murthy. They are two entirely different factors. The \( D \) increase for the simplified method is a result of the method, not a factor of safety.
Cantilever Method Closed Form Example: Cohesionless Soil, No Water, Simplified Method

- Simplified method eliminates pressure reversals at pile toe
- Active and passive forces on each side of wall shown, along with nomenclature
- Object is for moment at toe to be zero

\[ F_p = \frac{\gamma K_p d^2}{2} \quad \text{Passive Pressures} \]

\[ F_a = \frac{\gamma K_a (d + h)^2}{2} \quad \text{Active Pressures} \]
Cantilever Method Closed Form Example: Cohesionless Soil, No Water, Simplified Method

- Changes in variables
  \[ \kappa = \frac{K_p}{K_a} \quad z' = \frac{z}{h} \quad d' = \frac{d}{h} \]

- Dimensionless moment equation at any point below the dredge line:
  \[ F(z) = 1/6 \, z'^3 + 1/2 \, z'^2 + 1/2 \, z' + 1/6 - 1/6 \, z'^3 \kappa \]

- Real root for zero moment at pile toe (solution for \(d'\) and thus \(d\)):
  \[ d' = \frac{\kappa^{2/3} + \sqrt[3]{\kappa} + 1}{\kappa - 1} \]

Note: for simplified method, necessary to add 20-40% to \(d\)
Cantilever Method Closed Form Example: Cohesionless Soil, No Water, Simplified Method

- Dimensionless shear equation below the dredge line:
  \[
  \frac{\partial F(z')}{\partial z'} = \frac{1}{2} z'^2 + z' + \frac{1}{2} - \frac{1}{2} z'^2 \kappa
  \]

- Distance \( o' \) below dredge line for maximum moment
  \[
  o' = \frac{o}{h} = \frac{1 + \sqrt{\kappa}}{\kappa - 1}
  \]

- Value of maximum moment:
  \[
  M_{max} = \frac{-\kappa - 2 \kappa^{3/2} + 2 \kappa^{5/2} + \kappa^3}{6 (\kappa - 1)^3} \gamma K_a h^3
  \]
Cantilever Method Closed Form Example: Cohesionless Soil, No Water, Simplified Method—Example Problem

- **Given**
  - Dry cantilever wall, uniform cohesionless soil, level backfill
  - $\varphi = 30^\circ$, $y = 109.2$ pcf
  - Rankine conditions
  - $h = 10'$
  - Passive reduction factor of safety = 1.5

- **Find**
  - Penetration below dredge line
  - Maximum moment on sheeeting

- **Solution**
  - From Rankine theory, $K_a = 1/3$, $K_p = 3$
  - Reduce passive earth pressure coefficient, thus $= K_p \times \frac{3}{1.5} = 2$
  - From this $\kappa = \frac{2}{(1/3)} = 6$
  - Equation for $d'$:
    $$d' = \frac{\kappa^{2/3} + \sqrt[3]{\kappa} + 1}{\kappa - 1}$$
  - Substituting, $d' = 1.22$, thus $d = (1.22)(10') = 12.2$
  - We will increase $d$ for design for the simplified method, see next slide
Cantilever Method Closed Form Example: Cohesionless Soil, No Water, Simplified Method—Example Problem

- **Solution**
  - **Equation for maximum moment:**
    \[
    M_{\text{max}} = \frac{-\kappa - 2\kappa^{3/2} + 2\kappa^{5/2} + \kappa^3}{6(\kappa - 1)^3} \gamma K_0 h^3
    \]
  - Substituting for maximum moment, \( M_{\text{max}} = (.476)(109.2)(1/3)(10)^3 = 17,325 \text{ ft-lb/ft of wall} \)

- **Solution**
  - **Location of maximum moment below dredge line:**
    \[
    o' = \frac{o}{h} = \frac{1 + \sqrt{\kappa}}{\kappa - 1}
    \]
  - Substituting, \( o' = (.69)(10) = 6.9' \) below dredge line = 16.9 below top of wall
  - **Increase in working \( d \) for simplified method:** add 0.2\( d \) to \( d \), thus \( 12.2 + 0.2(12.2) = 14.6' \)
    - **This is actually conservative, traditionally simplified method used a shorter length, this done for simplicity**
Comparison with Sheet Pile Software
Cantilever Piles in Clay

- Two step analysis
  - Short term, where $\phi = 0$ and $c = s_u$ (analyse for cohesion only)
  - Long term, where $\phi > 0$ and $c > 0$ (use strengths from S (C-D) tests)
  - Include critical height considerations
Anchored Sheet Pile Walls

- Includes an anchor or tieback at or near the head of the wall
- More than one set of anchors or tiebacks can be used
- Increases wall stability and enables taller walls to be built and sustained
- Almost a necessity with vinyl, aluminum and fiberglass sheet piles
- Not exclusive to sheet piling; also used with other types of in situ wall systems
Anchored Sheet Pile Walls

a. Tie rods and dead man

b. Tie rods and anchor wall

c. Tiebacks with grout anchor

d. Tie rods and A-frame

e. Steel H-pile tension anchors

f. Steel H-pile anchors
Wales and Tiebacks in Sheet Pile Walls

a. Wales inside of wall
b. Wales outside of wall

All piles must be bolted to wale.
Analysis Methods for Anchored Walls

• Design Methods
  ● Free Earth Support Method
    • Assumes lower end of the pile incapable of producing negative bending moments
    • Converts problem into a statically determinate one
    • Rowe’s Moment Reduction Method used to take in to account flexibility of sheeting
      • Is unnecessary with SSI (soil-structure interaction) solutions such as SPW 2006
  ● Fixed Earth Support Method/Blum’s Method (Equivalent Beam Method)
    • Makes lower end has no angle like a cantilever beam, but no moment
    • Results in sheeting which is longer but has lower moment (and thus can be lighter)
    • Useful for some kinds of sheeting, more conservative
    • Also does not require Rowe’s Moment Reduction
  ● Beam on Elastic Foundation Method (generally w/plasticity considered)

• Implementation Methods
  ● All of the methods used in cantilever walls can be used in anchored walls as well, with same strengths and weaknesses
  ● Anchored walls are both simpler and more complex for hand solution than cantilever ones
  ● Key for solution is to take moment around anchor point, solve for sheeting length and then compute anchor force
  ● In this course, we will primarily use SPW2006 to analyze these walls
    • Stable wall is arrived at by varying the length of the wall
    • Solution doing this is basically free earth support method with SSI, obviating need for Rowe’s moment reduction
  ● Blum’s Method
    • Discussed in Veruijt for simple cases
    • A more comprehensive closed-form solution is available

Figure 37.1: Blum’s schematization.
Closed Form Example: No Water, Homogeneous Soil

- Similar to cantilever except now there is an anchor distance $a$ from the wall top
- New variable $a' = a/h$

Figure 36.1: Anchored sheet pile wall.
Closed Form Example: No Water, Homogeneous Soil

• Governing Equations

Anchored Wall, Penetration Distance Equation:

\[ \frac{1}{3} + d' - \frac{1}{2} a' + \frac{d'^2}{2} - d' a' + \frac{1}{3} d'^3 - \frac{1}{2} d'^2 a' - \frac{1}{2} \kappa d'^2 - \frac{1}{3} \kappa d'^3 + \frac{1}{2} \kappa d'^2 a' = 0 \]

Anchor Pull Ratio to Active Earth Pressure:

\[ \frac{T}{K_a \gamma h^2 (1 + d')^2} = \frac{1 + 2d' + d'^2 (1 - \kappa)}{(1 + d')^2} \]

Location of Maximum Moment:

\[ z' = \sqrt{1 + 2d' + d'^2 (1 - \kappa)} \]

Dimensionless Maximum Moment:

\[ \frac{M_{max}}{K_a \gamma z^3} = \left( \frac{1}{2} + d' + \frac{d'^2}{2} \frac{1 - \kappa}{2} \right) (z' - a') - \frac{z'^3}{6} \]
**Closed Form Example: No Water, Homogeneous Soil**

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## Closed Form Example: No Water, Homogeneous Soil

### Values of $z/h$ for maximum moment varying $a/h$ and kappa $\kappa$

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### Values of $M_{\text{max}}$ varying $a/h$ and kappa $\kappa$

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Closed Form Example: No Water, Homogeneous Soil

- Use the same basic variables as with cantilever case
  - Simply add an anchor at a depth of 2.5’ from the top, thus \( a' = 2.5/10 = 0.25 \)

- Ratios from equations:
  - \( d' = 0.516 \)
  - Anchor Pull Ratio = 0.305
  - \( z' \) for maximum moment = 0.837
  - Moment ratio = 0.108

- Determine actual quantities from ratios
  - \( d = (0.516)(10') = 5.16' \)
  - \( T = \text{(Anchor Pull Ratio)}K_a\gamma h^2(1+d')^2/2 = (0.305)(1/3)(109.2)(10)^2(1+0.516)^2 = 1274.6 \text{ lb/ft of wall} \)
  - \( z = (0.837)(10) = 8.37' \)
  - \( M_{\text{max}} = \text{(Moment Ratio)}K_a\gamma h^3 = (0.108)(1/3)(109.2)(10)^3 = 3924.5 \text{ ft-lb/ft} \)
Closed Form Example: No Water, Homogeneous Soil

<table>
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<tr>
<th>Maximum</th>
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<tbody>
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<tr>
<td>3823.5 ft/lb/ft</td>
<td>8.33</td>
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<tr>
<td>1139.1 lb/ft</td>
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Free Earth Analysis of Anchored Walls

Notes:
A) #1, "methods of Figures 2-7" refer to methods of computing Ka and Kp and lateral earth pressures previously discussed.
B) #5, same note as with cantilever walls; however, however, consider subsequent changes in level of "dredge line" when designing wall.
Rowe’s Moment Reduction Curves

- Used to take into consideration the flexibility of the pile and its effect on relieving the actual bending moment the wall experiences.
- Different set of curves for clay and sand.
Anchored Wall Example

SURCHARGE 10 kN/m²

Active Pressure $p_a = \gamma_h K_a - 2c \sqrt{K_a} + p_w$ kN/m²

Passive Pressure $p_p = \gamma_h K_p + 2c \sqrt{K_p} + p_w$ kN/m²

The coefficients of earth pressure are obtained from the tables in the section on earth and water pressures.

Loose fine sand: $K_a = 0.33$ $K_p = 4.9$; Compact fine sand: $K_a = 0.27$ $K_p = 6.0$
Anchored Wall Example

Free Earth Support—Example

Active Pressure $p_a = \gamma_h \cdot K_a - 2c \cdot \sqrt{K_a + p_w}$ kN/m²

Passive Pressure $p_p = \gamma_h \cdot K_p + 2c \cdot \sqrt{K_p + p_w}$ kN/m²

The coefficients of earth pressure are obtained from the tables in the section on earth and water pressures.

Loose fine sand: $K_a = 0.33$ $K_p = 4.0$; Compact fine sand: $K_a = 0.27$ $K_p = 6.0$

- $p_a$ at ground level: $10 \times 0.33 = 3.3$ kN/m²
- $p_a$ at 4 m below ground level in loose fine sand: $78.6 \times 0.33 = 25.9$ kN/m²
- $p_a$ at 4 m below ground level in dense fine sand: $78.6 \times 0.27 = 21.2$ kN/m²
- $p_a$ at 5 m below ground level in dense fine sand: $89.4 \times 0.27 + 9.8 = 33.9$ kN/m²
- $p_a$ at 7 m below ground level in dense fine sand: $111.0 \times 0.27 + 23.4 = 59.4$ kN/m²
- $p_a$ at 11 m below ground level in dense fine sand: $154.1 \times 0.27 + 68.6 = 110.2$ kN/m²

- $p_p$ at 7 m below ground level: $43.1 \times 6.0 + 58.8 = 19.6$ kN/m²
- $p_p$ at 11 m below ground level: $317.4 - 110.2 = 207.2$ kN/m²

Retaining Walls

Moments of Active Pressure about Tie Rod

For stability the moment of Passive Pressure about the tie rod must be equal to, or not less than, the moment of Active Pressure about the tie rod.

$\frac{1}{2} (6645 + 2y) \times 1 = 6364.4 \times \Delta x = 163$ m

Total Net Passive Pressure $= 6175.0 \times 1.63 = 820$ kN

Moments of Nett Passive Pressure about tie rod $= 820 \times 7.32 - 634.0$ kN/m check $M_a - M_r = 0$

Total Net Active Pressure $= 172.5$ kN

Zero Shear (maximum bending moment) occurs at 5.120 m below ground level.

B.M. on piles $= 202.3$ kN/m run of wall

Use Larsen No. 3/20 or Frodingham 3N

Penetration required to give a factor of safety of 2 against rotation of wall about the tie rod $= y + 0.645 = 7.000$

To find $y$: $\frac{1}{2} (6645 + 2y) \times 1 = 2 \times 6364.4 \times \Delta x = 2.24$ m

Moments of Net Passive Pressure with 2.24 + 0.645 = 2.885 m cut off $= 1260.7$ kN/m

Penetration reqd. $= 0.645 + 2.24 + 7 = 9.885$ m

Use Larsen No. 3/20 or Frodingham No. 3N x 10.0 m long in Grade 43A steel
SPW 2006

- Simple software to use, does not require installation
- Input according to instructions
- For active-passive stroke, use chart at right
  - Complete stroke $Y/H$ should be sum of active and passive rotation values, depending upon the soil
  - $Y/H$ sum is then multiplied by height $H$ (as shown) to determine stroke

Figure 10-4. Effect of wall movement on wall pressures (after Canadian Geotechnical Society, 1992).
### Anchored Wall Example

#### Left Side

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<th>Cap</th>
<th>q</th>
<th>c</th>
<th>Ka</th>
<th>Kp</th>
<th>Kn</th>
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<td>1.00</td>
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<td>10.00</td>
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<td>4.00</td>
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<td>0.00</td>
<td>0.270</td>
<td>4.00</td>
<td>0.426</td>
<td>0.018</td>
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#### Diagram
Anchored Wall Example
Estimation of Maximum Moment using SPW 2006

- Sheet pile database has an unusual method of input for sheet pile properties
  - Modulus of elasticity “E”
  - “EI” product of modulus of elasticity and moment of inertia per meter of wall
  - “h” distance from neutral axis to face of sheeting = half of depth (this is different from way it’s presented in tables)
- Conventional beam theory
  - \( \sigma_{\text{max}} = \frac{(Mc)}{I} \)
  - \( \sigma_{\text{max}} \) based on failure criterion, usually = \( \sigma_{\text{yield}}/FS \)
  - Maximum permissible stresses given for a variety of materials in previous slide set
- Substitutions using values in SPW 2006
  - I = “EI”/E
  - c = h
  - \( \sigma_{\text{max}} = \frac{(MEh)}{("EI")} \)
  - \( M_{\text{max}} = \sigma_{\text{max}} \frac{"EI"}{Eh} \)
- To determine whether sheeting meets structural requirements:
  - Determine \( \sigma_{\text{max}} \) based on alloy
  - Select section, which gives you “EI,” E, h.
  - Determine maximum permissible moment for section using formula above
  - If moment from SPW2006 is greater, use heavier (greater I, h) section
- Valid for both cantilever and anchored walls
- For example:
  - Maximum moment = 520 kN-m/m
  - PZ-22 used in the example
    - h=.1145 m
    - EI = 24150.0 kN-m²/m
    - E = 210 GPa
    - \( \sigma_{\text{max}} = 220 \) MPa (ASTM A 572 Gr. 50)
    - \( M_{\text{max}} = \sigma_{\text{max}} \frac{"EI"}{Eh} = (24150)(220)(1000)/(210)(1000000)(.1145) = 221 \) kN-m/m
    - This section is obviously too small; you will need to input a heavier section until you exceed the maximum moment
Blum’s method assumes that the pile toe has no moment, deflection or angle, but does have a reaction. The result is generally sheeting which is lighter than free earth support, but has more penetration. Verruijt has solutions for simple problems but a more comprehensive (and simpler) solution is online.
Tiebacks and Wales

Tieback Formulae:

\[ T_{ah} = TS \]

where

- \( T_{ah} \) = load per tieback
- \( T \) = tieback load per foot of wall
- \( S \) = spacing between tiebacks

\[ K_{ah} = \frac{1}{4} \frac{E_{ah} \pi d_{ah}^2}{L_{ah}} \]

where

- \( K_{ah} \) = tieback stiffness per tieback
- \( E_{ah} \) = tieback modulus of elasticity
- \( d_{ah} \) = tieback diameter
- \( L_{ah} \) = tieback length

\[ M_{\text{max}} = \frac{1}{10} TS \]

where

- \( M_{\text{max}} \) = maximum moment on wale

\[ K = \frac{K_{ah}}{S} \]

where

- \( K \) = combined stiffness per unit length of wall of tieback

Can be considered either as a beam with rigid or flexible supports

a. Wales inside of wall

b. Wales outside of wall
Anchor Design (NAVFAC DM 7.02)

**Effect of Anchor Location Relative to the Wall**

- Anchor block left of bc provides no resistance.
- Anchor block right of bf provides full resistance with no load transferred to wall.
- Anchor block between bc and bf provides partial resistance and transfers load $AP_d$ to base of wall.

VECTOR DIAGRAM FOR FREE BODY $\bar{Q}_d$:

\[ \text{WHERE} \quad P_a = \text{active force on back of} \quad \text{de at anchor block.} \]

**Continuous Anchor Wall Located Between Rupture Surface and Slope at Friction Angle**

**Effect of Depth and Spacing of Anchor Blocks**

ANCHOR RESISTANCE FOR $h_1 < \frac{h}{2}$

1. Continuous Wall:
   - Ultimate $Ap_c/d = Ap_c/d - \frac{P_y}{d}$, where $Ap_c/d$ is anchor resistance and $P_y$ is resultant force of soil at rest on vertical area $cde$ or $c'de$.

2. Individual Anchors:
   - If $d = V + h$, ultimate $Ap_c/d = b(P_y - P_x) + \frac{P_y}{d} \tan \phi$, where $P_y$ is resultant force of soil at rest on vertical area $cde$ or $c'de$.
   - $d = h + b$, $Ap_c/d$ is $70\%$ of $Ap_c/d$ for continuous wall.
   - $P_y$ for this condition is $L'$ and $L' = h$.
   - If $d < V + b$, $Ap_c/d = Ap_c/d - \frac{L'}{d} (3.3 \frac{Ap_c/d}{d})$, $L' = h$.

ANCHOR RESISTANCE FOR $h_1 < \frac{h}{2}$

Ultimate $Ap_c/d$ or $Ap_c/d$ equals bearing capacity of strip footing of width $h_1$ and surcharge load $y(h - \frac{h_1}{2})$, see Figure 1, Chapter 4.

USE FRICTION ANGLE $\psi' = \text{where} \tan \psi' = 0.6 \tan \psi$.

**General Requirements:**
1. Allowable value of $Ap$ and $Ap_c = \text{Ultimate value/2, factor of safety of 2 against failure.}$
2. Values of $K_a$ and $K_p$ are for cohesionless materials. If backfill has both $k_d$ and $k_c$ strengths, compute active and passive forces according to Figures 7 and 9. Fine-grained soils of medium to high plasticity should not be used at the anchorages.
3. Soils within passive wedge of anchor wall shall be compacted to no less than 90% of max unit weight (ASTM D5998 test).
4. Tie rod is designed for allowable $Ap$ or $Ap_c$. Tie rod connections to wall and anchorages are designed for 1.2 (allowable $Ap$ or $Ap_c$).
5. Tie rod connection to anchorage is made at the location of the resultant earth pressures acting on the vertical face of the anchorage.

**Figure 20 (continued)**

Design Criteria for Deadman Anchorage
Soldier Beams
Overview of Braced Cuts and Tied Back Excavations

- All of the walls we have looked at up until now have been a single wall with no more than one anchor.
- There are situations where either a) we need two facing walls or b) we need more than one anchor or c) both.
- Braced cuts are sets of walls (usually sheet pile, but can be slurry) which are used to create a “mechanical ditch” inside of which construction can take place.
- Tied back walls have multiple anchors, but the braces are outside of the cut (if any).
History of Braced Excavation Design

• Original work by Terzaghi and Peck on subway braced excavations in 1940’s
• Discovered that progressive excavation and placement of braces altered lateral earth pressure profile
• Toe design entirely different from conventional sheet pile walls

Important considerations in braced excavation design
  ○ Suitable for walls constructed “progressively”
  ○ Strictly speaking, brace loads are statically indeterminate
  ○ Terzaghi and Peck profiles assume hinges at braces for wall beam loading
  ○ Must consider bottom heave (esp. cohesionless soils, see ENCE 3610) and bearing capacity failure (esp. cohesive soils)
Pressure Distribution for Braced Loads

(a) SAND
\[ \sigma_h = 0.65 K_A \gamma H \]
WHERE \( K_A = \tan^2 (45 - \phi/2) \)

(b) SOFT TO MEDIUM CLAY
\( (N_o>6) \)
For clays base the selection on
\[ N_o = \gamma H/c \]
\[ \sigma_h = K_A \gamma H \]
\[ K_A = 1 - m \frac{4c}{\gamma H} ; \]
m = 1 except where cut is
underlain by deep soft
normally consolidated
clay, then \( m = 0.4 F_{SB} \)

ASSUME HINGES AT STRUT
LOCATIONS FOR CALCULATING
STRUT FORCES

See Figure 28 for Factor of Safety
against bottom instability,
\( (F_{SB}) : 1 \leq F_{SB} \leq 1.5 \)

(c) STIFF CLAY
\( (N_o<4) \)
For \( 4<N_o<6 \), use larger of
diagrams (b) and (c).
\[ \sigma_h = 0.2 \gamma H ; \sigma_2 = 0.4 \gamma H \]
Use lower value when movements
are minimal and short
construction period.
Design Criteria for Braced Cuts

1. Compute pressures on wall above base of cut by methods of Figure 26. For water at backfill surfaces use $\gamma'' \gamma_{sub}$ and add pressures for unbalanced water level. For water at base of cut use $\gamma'' \gamma_{int}$, interpolate between these pressure diagrams for an intermediate water level.

2. Determine stability of base of cut by methods of Figure 28. If base is stable, sheeting toes in several feet and no force acts on buried length. If base is unstable, sheeting penetrates as shown in Figure 29 and unbalanced force $P_H$ acts on buried length. In any case, penetration may be controlled by requirement for cut-off of underwaterpage.

3. Moments in sheeting between braces = 0.8 x (simple span moments), except for upper span where moment = 1.0 x (simple span moment). Moments in sheeting at point A is computed for cantilever span below A, including unbalanced force $P_H$.

4. Reaction at braces computed assuming simple span between braces.

5. Check positive moments in span below point A for this final loading condition.

---

FIGURE 27
Design Criteria for Braced Flexible Walls

FIGURE 27 (continued)
Design Criteria for Braced Flexible Walls

DEFLECTED POSITION

FLEXIBLE WALL WITH RAKING BRACES

PA = RESULTANT ACTIVE PRESSURE
PAI = RESULTANT ACTIVE BELOW POINT A

1. Compute active and passive pressures by methods in section 2. Passive pressures for clean, coarse-grained soils include wall friction ($\phi$), Table 1. Ignore wall friction for passive pressures in other soil types and for active pressures in all soils.

2. Maximum moments in sheeting and maximum loads in braces are usually obtained at a construction stage when excavation for a brace and wale is complete and just prior to placing the brace. For each successive stage of excavation compute sheeting moments and brace loads by assuming simple span between lowest brace then in place and point of zero net pressure below excavation.

3. For temporary construction conditions, apply factor of safety of 1.5 to compute passive pressures. To allow for possible construction surcharge and rigidity of upper brace point, increase load on upper wale and brace by 15% of computed value.

4. Required penetration of sheeting below final subgrade generally is controlled by conditions at completion of excavation. Penetration required is determined by equilibrium of free ended span below point A. Assuming fixity at point A:

$$PA \cdot L_1 - P_D \cdot L_2 - M_S = 0$$

$M_S$ = allowable moment in sheeting

5. Check positive moments in span below point A for this final loading condition.
Base Stability for Braced Cuts
Braced Cuts

Example

**Example**

**GIVEN CONDITIONS:**
- EXCAVATION IN SILTY CLAY.
  - C = 400 PSF, \( \phi = 0, \gamma_T = 120 \text{ PCF} \)
  - LENGTH OF EXCAVATION, \( L = 80' \)
- DETERMINE: PRESSURES ON WALL, FORCE ON BURIED LENGTH OF SHEETING AND STABILITY OF BASE OF CUT.

**STABILITY OF BASE OF CUT:** (SEE FIGURE 26)

\[
F_{SB} = \frac{\gamma_T + q}{H} 
\]

**FOR** \( N_c \): (FIGURE 2, CHAPTER 5)

\[
\frac{H}{B} = 5.02 \quad \frac{20}{12} = 1.67,
\]

\[
B = 12, \quad L = 80 = 0.15, N_c = 6.9
\]

\[
N_{CR} = N_c (1 + 0.2 B/L) = 6.9 (1 + 0.2 (0.15)) = 7.1
\]

\[
F_s = \frac{T \times 400}{120 \times 20} = 1.18 (1.5)
\]

**DRIVE SHEETING BELOW BOTTOM OF EXCAVATION**

**PRESSURE ON WALL FROM SURROUNDING SOIL:** (SEE FIGURE 26)

\[
K_A = 1 - m \frac{4C}{H} 
\]

\[
m = 0.4 \quad F_{SB} = 0.4 \times 1.18 = 0.47
\]

\[
\sigma_h = K_A \gamma_H = 0.69 \times 0.12 \times 20 = 166 \text{ KSF}
\]

\[
P_{H1} = \frac{(15 + 20)(1.66)}{2} = 29.05 \text{ KIPS}
\]

**LOCATION OF RESULTANT:**

\[
R_1 = 166 \times 5/2 \times (15 + 5/3) + 166 \times 15/2 = 8.81
\]

**PRESSURES ON WALL FROM SURCHARGE:** (SEE FIGURE 11)

\[
m = \frac{X}{H} = \frac{4}{20} = 0.2
\]

\[
P_{H2} = 0.78 \frac{Q_D}{H} = 0.78 \frac{10}{20} = 0.39 \text{ KIP}
\]

**LOCATION OF RESULTANT:**

\[
R_2 = 50 \times 0.59 \times 20 = 11.8
\]

**FORCE ON BURIED LENGTH OF SHEETING:** (SEE FIGURE 28)

Assume \( H_1 = 5 \frac{2}{3} \sqrt{2} \), for \( T > 0.78 \) RESULTANT FORCE \( P_{H2} \)

\[
P_{H3} = 1.5 \frac{H}{B} (\gamma_T + \frac{14CH}{B} - \pi C)
\]

\[
P_{H3} = 1.5 \times 5(0.2 \times 20 - \frac{14 \times 4 \times 20 - 3.14 \times 4}{12}) = 1.6 \text{ KIP}
\]

NOTE: ALL COMPUTATIONS ARE PER LINEAR FOOT OF WALL.

**FIGURE 30**

Example of Analysis of Pressures on Flexible Wall of Narrow Cut
In Clay - Undrained Conditions
Tied Back Walls
Pressure Distribution

SOFT TO MEDIUM CLAY

Compute pressure based on at-rest conditions with $K_0$ from 0.5 to 0.6. In normally consolidated clays excessive prestressing should not be permitted because of the potential for induced consolidation. Use design procedure as in Figure 26.

SANDS

Where deformations are critical and tie-backs are prestressed to 100% of design load, compute pressure based on at-rest conditions. Use $K_0 = 0.4$ for dense sand, and $K_0 = 0.5$ for loose sand.

STIFF TO VERY STIFF CLAY

Use pressure ordinate to produce the same force as for braced excavation. 0.3 is applicable for stability number of about 4, and 0.15 is applicable when stability number is less than 4. Use design procedure as in Figure 26.
Questions