DINDING PENAHAN TANAH
JENIS – JENIS DINDING PENAHAN TANAH

(a) Gravity wall
(b) Semigravity wall
(c) Cantilever wall
(d) Counterfort wall
DESAIN OF GRAVITY AND SEMIGRAVITY WALLS

Sumber: Bowles, Joseph E., Foundation analysis and design
DESAIN OF GRAVITY AND SEMIGRAVITY WALLS

Soil data:

<table>
<thead>
<tr>
<th>Backfill</th>
<th>Base soil</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\gamma = 17.5 \text{ kN/m}^3$</td>
<td>18.5</td>
</tr>
<tr>
<td>$\phi = 32^\circ$</td>
<td>0</td>
</tr>
<tr>
<td>$q_a$ (based on $q_a = 275 \text{ kPa}$)</td>
<td>275 kPa</td>
</tr>
</tbody>
</table>

$f_{c'} = 21 \text{ MPa}$  
$f_r = 0.42\phi\sqrt{f_{c'}}$  
$\phi = 0.65$

$\nu_r = 0.16\phi\sqrt{f_{c'}}$  
$\phi = 0.85$

Solution

Step 1  Find the lateral wall force using the Rankine $K_a$:

$K_a = 0.321$  (Table 11-3)

$P_a = \frac{1}{2}\gamma H^2 K_a = \frac{1}{2} \times 17.5 \times 7.1^2 \times 0.321 = 142 \text{ kN}$

$P_a = 142 \cos 10^\circ = 139$  
$P_x = 142 \sin 10^\circ = 25 \text{ kN}$

Sumber: Bowles, Joseph E. Foundation analysis and design
DESAIN OF GRAVITY AND SEMIGRAVITY WALLS

1. Compute wall stability. Neglect soil over toe and do not use passive pressure.

<table>
<thead>
<tr>
<th>Weight, kN</th>
<th>Arm, m</th>
<th>$M_s$, kN·m</th>
</tr>
</thead>
<tbody>
<tr>
<td>$17.5\left(\frac{0.15}{2} + \frac{2.27}{2} \times 5.8 + 2.27 \times 0.4 \times \frac{1}{2}\right)$ = 131</td>
<td>2.6*</td>
<td>341</td>
</tr>
<tr>
<td>$2.12 \times \frac{5.8}{2} \times 23.6$ = 145</td>
<td>1.8</td>
<td>261</td>
</tr>
<tr>
<td>$0.5 \times 5.58 \times 23.6$ = 68</td>
<td>0.9</td>
<td>61</td>
</tr>
<tr>
<td>$0.5 \times \frac{5.8}{2} \times 0.48 \times 23.6$ = 16</td>
<td>0.5</td>
<td>8</td>
</tr>
<tr>
<td>$0.9 \times 3.4 \times 23.6$ = 72</td>
<td>1.7</td>
<td>122</td>
</tr>
<tr>
<td>(vertical component of soil pressure)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>25</td>
<td>3.4</td>
<td>85</td>
</tr>
</tbody>
</table>

Total $W = 457$ kN  Total $M_s = 878$ kN·m

*Using approximation of centroid at 2.27/3 from shear plane.

2. The overturning safety factor is

$$M_s = P_h \tilde{y} = 139 \times \frac{7.1}{3} = 329 \text{ kN} \cdot \text{m}$$

$$F = \frac{M_s}{M_p} = \frac{878}{329} = 2.67 > 1.5 \quad \text{O.K.}$$

3. Calculate the factor of safety (and neglecting any passive pressure) is

Take cohesion $c = \frac{q}{2} = \frac{275}{2} = 137.5 \text{ kPa}$

Take cohesion $c_s = 0.6c = 0.6 \times 137.5 = 82.5 \text{ kPa}$

$$F_s = \frac{Bc_s}{E} = 3.4 \times 82.5 = 280.5 \text{ kN}$$

$$F = \frac{F_s}{F_h} = \frac{280.5}{139} = 2.02 > 1.5 \quad \text{also O.K.}$$

4. Locate resultant on base and eccentricity:

$$R \tilde{x} = \Sigma M_{\text{rot}}$$

$$\tilde{x} = \frac{M_s - M_x}{\Sigma W} = \frac{878 - 329}{457} = 1.20 \text{ m}$$

$$e = \frac{B}{2} - \tilde{x} = 1.7 - 1.20 = 0.50 \text{ m} < \frac{L}{6} \quad \text{O.K.}$$

5. Compute actual soil pressure:

$$q = \frac{P}{A} \left(1 \pm \frac{6c}{L}\right) = \frac{457}{3.4(1)} \left[1 \pm \frac{6(0.5)}{3.4}\right] = 253 \text{ kPa} \text{ (max)} < 275 \quad \text{O.K.}$$

$$= 16 \text{ (min)}$$

Sumber: Bowles, Joseph E. Foundation analysis and design
DESAIN OF GRAVITY AND SEMIGRAVITY WALLS

Step 5 Check shear and tensile bending stresses in toe at 0.15 m from edge; E12-3b.
(a) Shear check:

\[ q = 253 - 69.7x \]

\[ V = \int_0^x (253 - 69.7x) \, dx \]

\[ = 253x - \frac{69.7x^2}{2} \]

at \( x = 0.15 \) and \( V = 37 \) kN.

For load factor = 2, \( d = D \) for no rebars, and

\[ v_a = \frac{LF \times V}{bd} \quad v_e = \frac{2 \times 37}{1 \times 0.9} = 82.2 \text{ kPa} \]

\[ v_e = 0.16(0.85)\sqrt{21} \times 10^3 = 623 \text{ kPa} > 82 \quad \text{O.K.} \]

(b) Tension check:

\[ M = \int_0^x V \, dx = \frac{253x^2}{2} - \frac{69.7x^3}{6} \]

at \( x = 0.15 \) and \( M = 2.81 \text{ kN} \cdot \text{m} \).

For \( LF = 2 \) and \( S_x = \frac{bh^2}{6} \)

Actual \( f_t = \frac{6(LF)M}{bh^2} = \frac{6(2)(2.81)}{(1)(0.9^2)} = 42 \text{ kPa} \)

Allowable \( f_t = 0.42(0.65)\sqrt{21 \times 10^3} = 250 \gg 42 \text{ kPa} \quad \text{O.K.} \)

Step 6 Approximately check \( f_t \) at \( \frac{1}{3} \) wall height (3.5 m from top; \( \simeq 3.75 \) at slope):

Approx. \( M = \frac{1}{2}yH^2K_s\gamma \cos \beta \)

\[ M = \frac{1}{2}(17.5)(3.75)^2(0.321) \times \frac{3.75}{3} \cos 10 = 48.6 \text{ kN} \cdot \text{m} \]

Find wall \( h \) at 3.5 m by proportion:

\[ \frac{h'}{3.5} = \frac{2.6}{5.8} \quad h' = 1.57 \text{ m} \]

Sumber: Bowles, Joseph E. Foundation analysis and design
DESIGN OF CANTILEVER RETAINING WALL

Other data: $f'_c = 3 \text{ ksi}$, $f_y = 60 \text{ ksi}$
$\gamma_e = 0.15 \text{ kcf}$, $LF = 1.8$
Batter on front face of wall = 1 : 48
Top = 16 in

Sumber: Bowles, Joseph E. Foundation analysis and design
DESIGN OF CANTILEVER RETAINING WALL

SOLUTION (Values shown in sketch from optimizing using a computer program) Estimate from cgs to soil interface to allow approximately 3.0 in of clear steel cover.

**Step 1** Establish stem dimensions; round to even values:

\[ K_s = 0.294 \quad \text{[Eq. (11-8) and Rankine value]} \]

Stem uses \( H = 26 \text{ ft} \)

\[ P_a = 0.5(0.115)(26)^2(0.294) = 11.43 \text{ kips/ft} \]

\[ P_{as} = 11.43 \cos 10^\circ = 11.25 \text{ kips/ft} \]

\[ v_c = 2\phi\sqrt{f'} = 0.09311 \text{ ksi} \]

\[ t = \frac{11.25(18)}{(0.093)(12)} = 18.14 + 3.5 = 21.6 \text{ in} \]

Top = 21.6 - 26(0.25) = 15.1 in  Use 16 in

To maintain even dimensions

\[ t = 16 + 26(0.25) = 22.5 \text{ in} \]

Use 23 in

**Step 2** Compute overturning and sliding stability for wall:

\[ H' = 26 + 2.42 + 9.5 \tan 10^\circ = 30.1 \text{ ft} \]

\[ P_{as} = \left( \frac{30.1}{26} \right)^2 11.25 = 15.1 \text{ kips/ft} \]

\[ P_{as} = 15.1 \sin 10^\circ = 2.6 \text{ kips/ft} \]

From these data and wall dimensions we can set up the following table:

<table>
<thead>
<tr>
<th>Part</th>
<th>Weight, kips</th>
<th>Arm, ft</th>
<th>( M ), ft-kips</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.5(26 + 27.65) \times 0.115 \times 9.5</td>
<td>29.3</td>
<td>9.67</td>
</tr>
<tr>
<td>2</td>
<td>1.33 \times 26 \times 0.15</td>
<td>5.2</td>
<td>4.25</td>
</tr>
<tr>
<td>3</td>
<td>0.5 \times 0.59 \times 26 \times 0.15</td>
<td>1.2</td>
<td>3.39</td>
</tr>
<tr>
<td>4</td>
<td>2.42 \times 14.42 \times 0.15</td>
<td>5.2</td>
<td>7.21</td>
</tr>
</tbody>
</table>

\[ \sum W = 43.5 \]

\[ \sum M = 384.4 \]

The overturning moment \( M_o = P_{as}y = \frac{15.1 \times 30.1}{3} = 151 \text{ ft} \cdot \text{kips/ft} \)

The safety factor \( F = \frac{\sum M}{M_o} = \frac{384.4}{151} = 2.54 > 1.5 \quad \text{O.K.} \)

The safety factor against sliding will be based on using 3 ft of the depth of soil at the top (allowing at least 2 ft to be lost from some means). Use 0.67c.

\[ K_s = \tan^2 \left( 45 + \frac{32}{2} \right) = 3.255 \quad \sqrt{K_s} = 1.804 \]

The friction-cohesion resistance \( F_r = 43.5 \tan 32 + 0.67(0.4)14.42 = 31 \text{ kips} \)

From integration of Eq. (2-45),

Sumber: Bowles, Joseph E. Foundation analysis and design
DESIGN OF CANTILEVER RETAINING WALL

\[ P_p = \frac{1}{2} \gamma H^2 K_p + 2cH\sqrt{K_p} \]

\[ P_p = 0.5(0.112)(3^2)(3.255) + 2(0.4)(3)(1.804) = 6 \text{ kips} \]

\[ \sum F_x = 31 + 6 = 37 \text{ kips} \]

The resulting

\[ F = \frac{\sum F_x}{P_{aw}} = \frac{37}{15.1} = 2.45 > 1.5 \quad \text{O.K.} \]

Now locate the resultant \( \sum W \) on base and the eccentricity. Find \( \bar{x} \) with respect to the back-compute the eccentricity:

\[ \bar{x} = \frac{\sum M}{\sum W} = \frac{384.4 - 151}{43.5} = 5.37 \text{ ft} \]

\[ e = \frac{B}{2} - \bar{x} = \frac{14.42}{2} - 5.37 = 1.84 \text{ ft} < \frac{L}{6} \quad \text{O.K.} \]

Step 3: Compute bearing capacity. Use equations in Table 4-3 and actual soil pressure:

\[ q_{ult} = cN_x d_x i_x + qN_y d_y i_y + \frac{1}{2} \gamma N_t d_t i_t \]

\( B' = 14.42 - 2(1.84) = 10.7 \text{ ft} \)

\( N_x = 35.5 \quad N_y = 23.2 \quad N_t = 20.8 \)

\( i_x = 0.42 \quad i_y = 0.44 \quad i_t = 0.309 \)

\( d_x = 1.19 \quad d_y = 1.13 \quad d_t = 1.0 \)

\[ q_{ult} = 0.4(35.5)(0.42)(1.19) + 5(0.112)(23.2)(1.13)(0.44) + 0.5(0.112)(10.7)(20.8)(0.309) \]

\[ = 7.1 + 6.5 + 3.9 = 17.5 \]

\[ q_s = 17.5 \frac{3}{3} = 5.8 \text{ ksf} \]

Total soil pressure

\[ q = \frac{V}{L} \left( 1 \pm \frac{6e}{L} \right) = \frac{43.5}{14.42} \left[ 1 \pm \frac{6(1.84)}{14.42} \right] = 3.02(1 \pm 0.76) \]

\( = 5.3 \text{ ksf max at toe} \)

\( = 0.7 \text{ ksf min at heel} \)

Step 4: Compute base-slab shear and bending moments. Refer to Fig. E12-4b. For toe at stem face \( x = 3.00 \):

\[ q = 5.3 - 0.36 - 0.32x \quad \text{(neglect soil over toe)} \]

\[ V = 4.94x - \frac{0.32x^2}{2} = 13.4 \text{ kips} \]

\[ M = \frac{4.94x^2}{2} - \frac{0.32x^3}{6} = 20.8 \text{ ft-kips} \]

Steel at approx. cg of tension steel

\[ x = 9.5 + \frac{3.5}{12} = 9.79 \text{ ft for moment} \quad \text{Use 9.5 for shear} \]
DESIGN OF CANTILEVER RETAINING WALL

Use average height of soil on heel for downward pressure; include $P_r = 2.6$ kips:

$$q = 3.45 - 0.70 - 0.32x$$

$$V = 2.75x - \frac{0.32x^2}{2} + P_r = 14.3 \text{ kips}$$

$$M = \frac{2.75x^2}{2} - \frac{0.32x^3}{6} + P_r x = 107.2 \text{ ft} \cdot \text{kips}$$

**Step 5** Check base-slab shear stress using largest base $V$, $LF = 1.8$, $d = 2.417 - 0.29 = 2.12$ ft:

$$\nu = \frac{14.3(1.8)}{12(25.5)} = 0.084 < 0.093 \quad \text{O.K.}$$

Note we could reduce the base slab by about 1 to 1.5 in. Leave at 2 ft 5 in.

**Step 6** Compute toe (bottom of footing) and heel (top of footing) reinforcing-steel requirements:

$$p_{\text{max}} = 0.016 \quad \text{(Table 8-1)}$$

$$p_{\text{min}} = \frac{200}{f_y} = 0.0033$$

$$0.5a = 0.5 \cdot \frac{A_s f_s}{0.85 f_y b} = 0.98 A_s$$

For heel and $d = 29 - 3.5 = 25.5$ in

$$A_s(25.5 - 0.98 A_s) = \frac{107.2(12)(1.8)}{0.9(60)}$$

Sumber: Bowles, Joseph E. Foundation analysis and design
DESIGN OF CANTILEVER RETAINING WALL

\[ A_r^2 - 26.02A_r = -43.76 \]
\[ A_r = 1.81 \text{ in}^2/\text{ft} \quad p = 0.006 \]

For toe

\[ A_r^2 - 26.02A_r = -8.49 \]
\[ A_r = 0.33 \text{ in}^2 \quad p = 0.0011 < 0.0033 \]
\[ A_r = 0.0033(12)(25.5) = 1.02 \text{ in}^2/\text{ft} \]

Step 7: Compute stem steel at top, 0.5\(H\), 0.8\(H\), and \(H\) using \(LF = 1.8\).

<table>
<thead>
<tr>
<th>Point</th>
<th>(M), ft·kips</th>
<th>Wall thickness</th>
<th>(d)</th>
<th>(A_r^*)</th>
<th>(p) (used)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
<td>16.0</td>
<td>12.5</td>
<td>0.5</td>
<td>0.0033</td>
</tr>
<tr>
<td>0.5(H)</td>
<td>22.0</td>
<td>19.5</td>
<td>16.0</td>
<td>0.64</td>
<td>0.0033</td>
</tr>
<tr>
<td>0.8(H)</td>
<td>90.0</td>
<td>21.60</td>
<td>18.1</td>
<td>1.18</td>
<td>0.005</td>
</tr>
<tr>
<td>(H)</td>
<td>175.8</td>
<td>23.0</td>
<td>19.5</td>
<td>2.26</td>
<td>0.010</td>
</tr>
</tbody>
</table>

* in\(^2/\text{ft}\)

From this it is evident minimum (200/\(f_c\)) requirements control top half of wall.
Use 3 No. 8 bars/ft \((A_r = 2.37 \text{ in}^2)\) in bottom \(\frac{1}{2}\) of wall.

Sumber: Bowles, Joseph E. Foundation analysis and design
DRAINAGE WALL

Weepholes should be 4 in. or larger to avoid plugging. Note that the discharge is onto the toe where the soil pressure is largest.

Granular material of size to avoid plugging weepholes

Backfill with free draining soil

Drain pipe covered with granular material. Cut hole in counterfort if required

If weepholes are used with a counterfort wall at least one weephole should be located between counterforts.
Sertifikasi HATTI:
Tegangan Lateral
dan
Dinding Penahan Tanah

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Teknik Sipil, Institut Teknologi Bandung

(Retaining wall Gabion)

Sheet Pile
Clamshell Grabber

CWS (Cofrage Avec Waterstopper)
Stopper baja untuk mencetak bentuk ujung kiri-kanan panel

Rebar cage + anchor sleeve
GEDUNG KANTOR
PLUIT SELATAN RAYA - JAKARTA

Ground Anchor

Rekomendasi Buku Untuk Sheet Pile

Piling Handbook

PT. Berlogo Internasional
Test Triaxial:
Arah Bidang Runtuh Terhadap $\sigma_1$

Mekanisme Aktif dan Passif

$\sigma_v = \sigma_1$  
$\sigma_h = \sigma_3$
Hubungan Koefisien Lateral dan Regangan

Kp
Ko
Ka

Expansion
Lateral Strain
Compression

Besarnya Tegangan Aktif

Pasir

$K_a = \tan(45 - \phi / 2)$

Besarnya Tegangan Pasif

$K_p = \frac{1}{K_a} \tan^2 \left(45 + \frac{\phi}{2}\right)$

Tekanan Aktif: Pasir

Lebih dari satu lapis:

$\sigma_h = \sigma_3 = \sigma_1 \times K_a$
Tekanan Aktif dan Pasif: Pasir

Depth

Tekanan Aktif dan Pasif: Pasir

Ada Beban Merata

Depth

Tegangan Effektif

\[ \sigma' = \sigma - \mu \]

\[ \sigma' = \text{tegangan efektif} \]

\[ \sigma = \text{tegangan total} \]

\[ \sigma_1 = \gamma_h \]

\[ \sigma_2 = \gamma h + \gamma' h_2 \]

\[ \sigma_3 = \gamma (h_1+ h_2) - \gamma w h_2 \]
Pengaruh kemiringan tanah diatas:

\[
\theta = \tan^{-1}\left(\frac{45 - \phi}{n}\right)
\]

\[
K_a = \frac{\cos \phi - \sqrt{\cos^2 \phi - \cos^2 \theta}}{\cos \phi + \sqrt{\cos^2 \phi - \cos^2 \theta}}
\]

Pasir

Lempung

Cantilever & Gravity Wall

Cantilever Gravity Wall

Penaruh C-Kohesi Tanah

Tekanan Aktif Rankine

\[
\Delta \sigma = \sigma_a K_a = \sigma_a K_a - 2c K_a
\]
Tekanan Pasif Rankine

\[ \sigma_1 = \sigma_v \cdot K_p = \sigma_v K_p + 2c \sqrt{K_p} \]

Tekanan Aktif Rankine untuk Perencanaan

Beban pada dinding
Disain Dinding Penahan Tanah:

Stabilitas

1. Overtuning
2. Sliding
3. Excentricity
4. Bearing Capacity

Stabilitas Retaining Wall

1. Overtuning
2. Sliding
3. Excentricity
4. Bearing Capacity
5. Overall Stability

\[ SF = 1.5 - 2.0 \]
Overtuning

\[ s = c + \sigma' \tan \phi \]

Where \( \sigma' \) = effective normal stress on plane of shearing

\[ c = \text{cohesion, or apparent cohesion} \]

\[ \phi = \text{angle of friction} \]

Sliding

\[ F_{\text{sliding}} = \frac{\Sigma P_x}{\Sigma F_y} \]

\[ \Sigma F_z = (\Sigma V)\tan(\theta_1 \phi_2) + F k_2 c_2 + P_p \]

\[ \Sigma F_d = F_z \cos \alpha \]

\[ F_{\text{sliding}} = \frac{(\Sigma V)\tan(\theta_1 \phi_2) + F k_2 c_2 + P_p}{P_n \cos \alpha} \]
1. **Contoh Soal**

Berikut ini gambar cross section dari cantilever retaining wall. Hitung angka keamanan (factor of safety) yang menyangkut overturning dan sliding!!

![Cross Section Diagram](image)

The overturning moment, $M_o$

$$M_o = R_1 \left( \frac{H}{3} \right) - 158.05 \left( \frac{7.158}{3} \right) = 379.25 \text{ kNm}$$

$$FS_{overturning} = \frac{\sum M_4}{M_o} = \frac{1128.06}{379.25} = 2.98 > 2 — \text{O.K.}$$

**Factor of Safety Against Sliding**

From Eq. (5.60)

$$FS_{sliding} = \left( \sum V \right) \tan(\theta_1 + \theta_2) + \frac{k_2 \gamma D^2 + P_v}{P_i \cos \alpha}$$

Let $k_1 = k_2 = 2/3$

Also

$$P_v = \frac{1}{2} k_1 \gamma D^2 + 2 c_2 \sqrt{K_o} D$$

The following table can now be prepared for determination of the resisting moment.

<table>
<thead>
<tr>
<th>Section</th>
<th>Area (m$^2$)</th>
<th>Weight/unit length (kN/m)</th>
<th>Moment arm from point C (m)</th>
<th>Moment arm (kNm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>6 x 0.5</td>
<td>76.74</td>
<td>1.15</td>
<td>85.63</td>
</tr>
<tr>
<td>2</td>
<td>10.08 x 4.6</td>
<td>11.13</td>
<td>0.63</td>
<td>11.70</td>
</tr>
<tr>
<td>3</td>
<td>4 x 0.67</td>
<td>16.02</td>
<td>2.0</td>
<td>32.04</td>
</tr>
<tr>
<td>4</td>
<td>9 x 2.5</td>
<td>299.96</td>
<td>3.7</td>
<td>179.01</td>
</tr>
<tr>
<td>5</td>
<td>12.09 x 4.9</td>
<td>16.71</td>
<td>3.13</td>
<td>53.52</td>
</tr>
</tbody>
</table>

E = 28.00

\[ \sum V = 470.45 \]

\[ \sum M_i = 1128.06 \]

So

$$\varphi = \frac{1}{2} [2.04(1.5)^2 + 2(40)\sqrt{2.04}(1.5)] = 43.61 + 171.39 = 215 \text{ kN/m}$$

Hence

$$FS_{sliding} = \frac{(470.45) \tan(\frac{2 \times 2.0}{3}) + (4) \frac{2}{3} (40) + 215}{111.5 + 106.67 + 215} = 2.73 > 1.5 — \text{O.K.}$$

Note: For some designs, the depth $D$ for passive pressure calculation may be taken to be equal to the thickness of the base slab.
1. **Contoh Soal**

Berikut ini gambar cross section dari cantilever retaining wall. Hitung angka keamanan (factor of safety) yang menyangkut overturning dan sliding!!!

KLASIFIKASI STRUKTUR SHEET PILE

- Cantilever Type
  - Single Tie-Rod Type
  - Double Tie-Rod Type
  - Deadman Type
- Anchored Type
  - Batter-Pile Anchor Type
  - Relieving Platform Type
- Cellular Type
  - Double Sheet Pile Wall Type
  - Multi-Strut Type
  - Dolphin Type

Sheet Pile

Cantilever Pile/ Sheet Pile
Dermaga Tanjung Emas

**Existing Sea Bed**
- Sand: 1.0, 24
- Very Soft Clay: 25.4, 1, 16.5
- Very Soft Clay: 29.4, 1, 16.0

**Stiff Clay**
- 22.0, 4, 17.0

**Soft Clay**
- 0.00 mLWS
- E_u = 250
- f_Cu = 10.0 mLWS

Relieving Platforms Retaining Wall

Fig 6.3.2 Relieving platforms:
- Steel Pipe Pile - Japan
- Piling Handbook - British
Double Sheet Pile Wall Type

Cellular Type Structure

Cofferdams
1. Metoda Elemen Hingga

**Parameter Tanah:**
- C (kohesi)
- $\phi$ (Sudut geser dalam)
- $E$ (Young's modulus)
- $\nu$ (Poisson's ratio)

**Sheet Pile: Beam Element**

M < M konvensional
2. Beam on Elastic Foundation

3. Metoda Konvensional

Cantilever Pile/ Sheet Pile
Cantilever Pile/ Sheet Pile

Tegangan Lateral Pada Cantilever Pile/ Sheet Pile

Cantilever Sheet Pile

Hence, depth of cut off = B.D + 1.2 x D.C.

F1 (Representing area A.O.F1) = total net active pressure
F2 (Representing area O.C.F1) = total net passive pressure
F3 (Representing area C.D.D1) = total net passive pressure required to fix the toe of the wall

Forces F1, F2 and F3 act through the centres of gravity of their respective areas.

Calculations may be simplified by considering the line C1.C.D1, to be horizontal and to pass through point C. The area C. D. D1 is replaced by force F3 acting at C as shown in Fig. 5.23.2 below.

The depth O.C. should be such that the moments of forces F1 and F2 about F3 are in equilibrium. The value of force F3 is such that the algebraic sum of forces F1, F2 and F3 is zero.

Moments pada area shear.
Metoda Konvensional:

1. Free Earth Support

Tegangan Lateral Anchored Sheet Pile
Panjang teoritis \( d \): Keseimbangan momen di titik \( O \)

Gaya angkur : Keseimbangan gaya horizontal

Untuk Desain Panjang \( d \): SF momen di titik \( O = 2.0 \)

**Metoda Konvensional:**

1. **Free Earth Support**

   - Aktif
   - Pasif

   - Idealized earth pressure distribution

   Deflected shape

   \[ SF \text{ moment at point } O = 2.0 \]
### Metoda Konvensional:

#### 2. Fixed Earth Support

<table>
<thead>
<tr>
<th>Type</th>
<th>Active Pressure</th>
<th>Moment of Active Pressure about a Vertical Line</th>
</tr>
</thead>
<tbody>
<tr>
<td>Aktif</td>
<td>2.0 kN/m</td>
<td>0.6 kN·m</td>
</tr>
<tr>
<td>Pasif</td>
<td>2.0 kN/m</td>
<td>0.6 kN·m</td>
</tr>
<tr>
<td>Aktif</td>
<td>2.0 kN/m</td>
<td>0.6 kN·m</td>
</tr>
</tbody>
</table>

The maximum bending moment on the piles = 3.6 kN·m per meter of wall.

Stair modulus = 200 kN·m.

For the stability of the moment of active pressure about the vertical line must be at least equal to the moment of active pressure about the soil.

The total moment of active pressure with depth = 3.6 kN·m per meter.

Hence the moment of active pressure at soil level = 1.8 kN·m per meter.

The total active pressure = 1.8 kN/m.

Total moment of active pressure = 2.0 kN/m.

Therefore the moment about the vertical line = 0.5 kN·m per meter.

Zero shear stress area of active pressure diagram = soil level is 0.2 m below soil.

### Notes:

- Although these results may be adequate for structural purposes, the shallower depth may require additional provisions, such as piles.
2. Fixed Earth Support

Retaining Walls

For free earth support, $x = \text{depth of cut off}$
For fixed earth support, $x = 3/4 \text{ depth of cut off}$
Retaining Walls

Braced Cut Excavation

CONTOH STRUKTUR SHEET PILE
Multi-Strut Type
BORED PILE DENGAN DISKONTINUITAS DARI HASIL PENGUJIAN

BM.1
Sand

Clay

Soft to Medium

\[ \frac{\gamma H}{c} > 4 \]

Stiff

\[ \frac{\gamma H}{c} \leq 4 \]

Thank You!