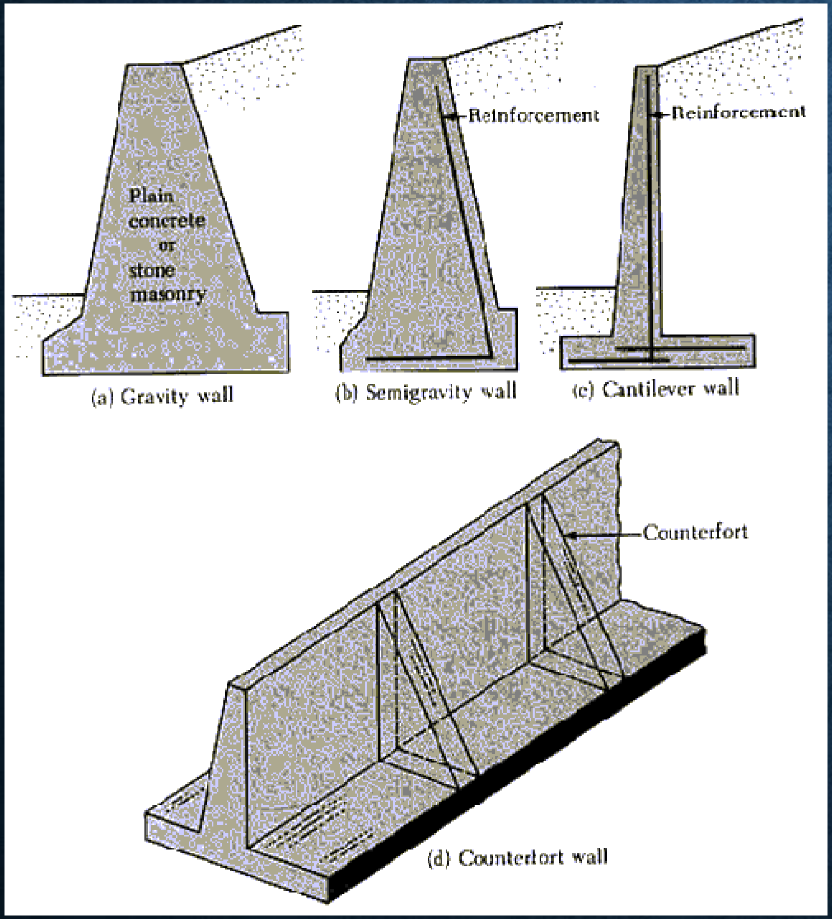
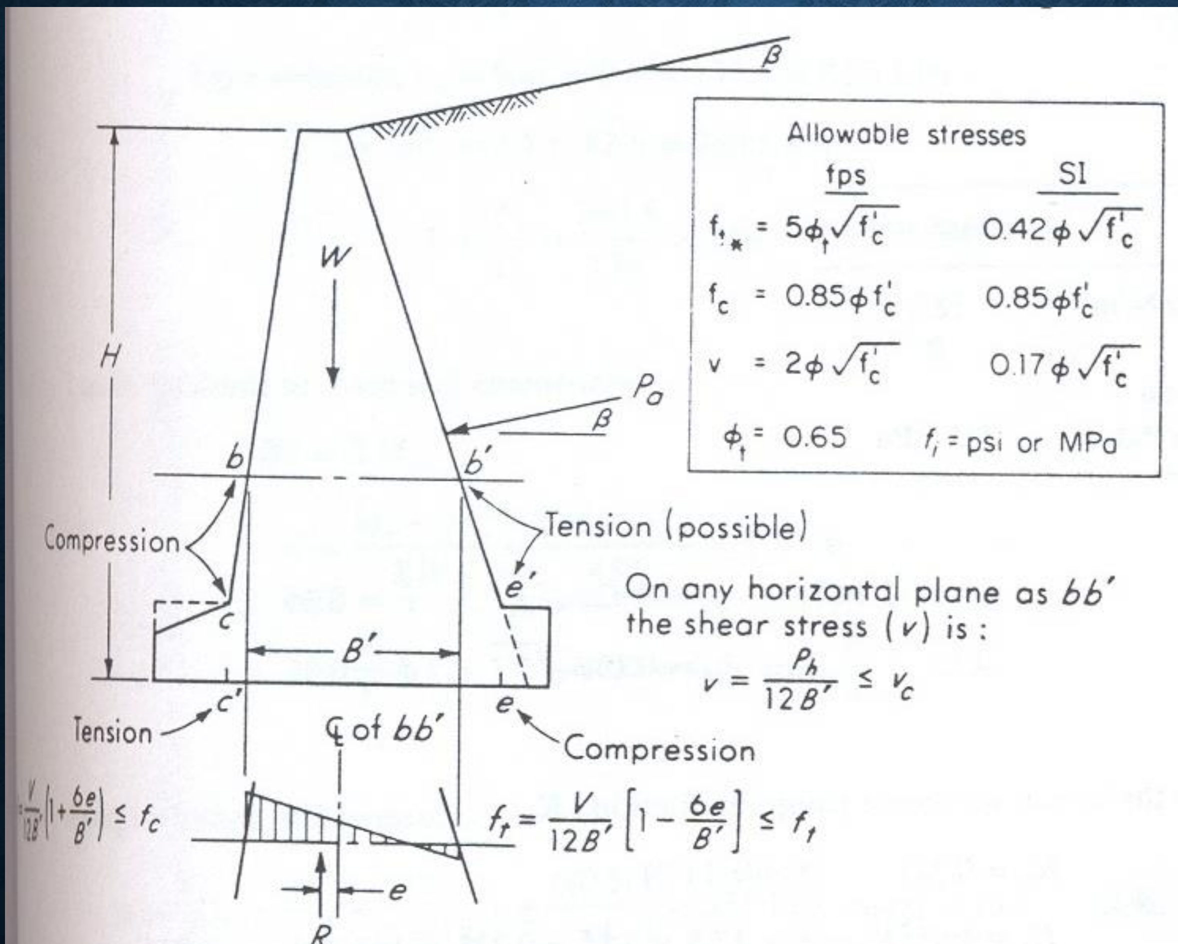
DINDING PENAHAN TANAH

JENIS –JENIS DINDING PENAHAN TANAH



DESAIN OF GRAVITY AND SEMIGRAVITY WALLS



Sumber : Bowles, Joseph E, Foundation analysis and design

DESAIN OF GRAVITY AND SEMIGRAVITY WALLS

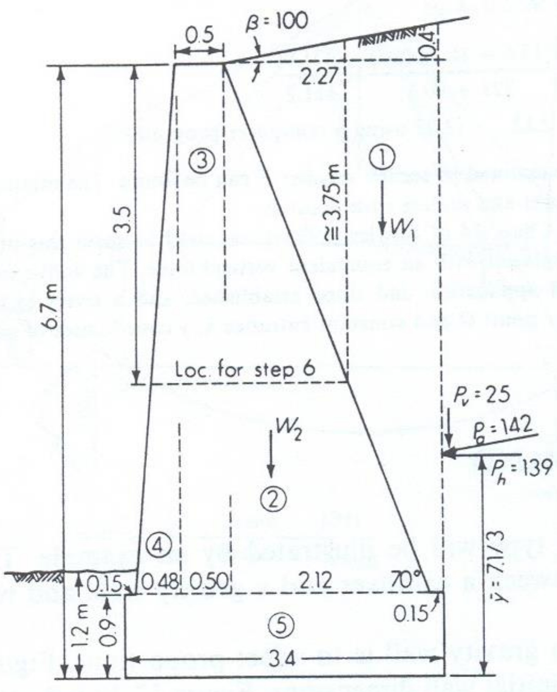


Figure E12-3a

Soil data:

Backfill	Base soil
$\gamma = 17.5$ kN/m^3	18.5
$\phi = 32°$	0
q_a (based on $q_u = 275$ kPa)	275 kPa

$f_c' = 21$ MPa $\quad f_t = 0.42\phi\sqrt{f_c'} \quad \phi = 0.65$

$v_c = 0.16\phi\sqrt{f_c'} \quad \phi = 0.85$

Solution

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

$$K_a = 0.321 \qquad \text{(Table 11-3)}$$

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

$$P_h = 142 \cos 10° = 139 \qquad P_v = 142 \sin 10° = 25 \text{ kN}$$

Sumber : Bowles, Joseph E, Foundation analysis and design

DESAIN OF GRAVITY AND SEMIGRAVITY WALLS

Step 2 Compute wall stability. Neglect soil over toe and do not use passive pressure.

Weight, kN	Arm, m	M_r, kN · m
$17.5\left(\frac{0.15}{2} + \frac{2.27}{2} \times 5.8 + 2.27 \times 0.4 \times \frac{1}{2}\right) = 131$	2.6*	341
$2.12 \times \frac{5.8}{2} \times 23.6 = 145$	1.8	261
$0.5 \times 5.58 \times 23.6 = 68$	0.9	61
$0.5 \times \frac{5.8}{2} \times 0.48 \times 23.6 = 16$	0.5	8
$0.9 \times 3.4 \times 23.6 = 72$	1.7	122
(vertical component of soil pressure) $= 25$	3.4	85
Total $W = 457$ kN		Total $M_r = 878$ kN · m

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

The overturning safety factor is

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

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

The sliding factor of safety (and neglecting any passive pressure) is

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

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

$$F_r = Bc_a = 3.4 \times 82.5 = 280.5 \text{ kN}$$

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

Step 3 Locate resultant on base and eccentricity:

$$R\bar{x} = \Sigma M_{\text{toe}}$$

$$\bar{x} = \frac{M_r - M_o}{\Sigma W} = \frac{878 - 329}{457} = 1.20 \text{ m}$$

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

Step 4 Compute actual soil pressure:

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

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

Sumber : Bowles, Joseph E, Foundation analysis and design

DESAIN OF GRAVITY AND SEMIGRAVITY WALLS

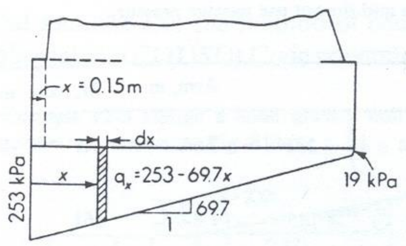


Figure E12-3b

Step 5 Check shear and tensile bending stresses in toe at 0.15 m from edge; E12-3*b*.

(*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

$$v_a = \frac{LF \times V}{bd} \qquad v_a = \frac{2 \times 37}{1 \times 0.9} = 82.2 \text{ kPa}$$

$$v_c = 0.16(0.85)\sqrt{21} \times 10^3 = 623 \text{ kPa} > 82 \qquad \text{O.K.}$$

(*b*) Tension check:

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

at $x = 0.15$ and $M = 2.81$ kN · m.

For $\qquad LF = 2 \qquad$ and $\qquad S_x = \frac{bh^2}{6}$

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

$$\text{Allowable } f_t = 0.42(0.65)\sqrt{21} \times 10^3 = 1250 \gg 42 \text{ kPa} \qquad \text{O.K.}$$

Step 6 Approximately check f_t at $\frac{1}{2}$ wall height (3.5 m from top; ≃3.75 at slope):

$$\text{Approx. } M = \tfrac{1}{2}\gamma H^2 K_a \bar{y} \cos\beta$$

$$M = \tfrac{1}{2}(17.5)(3.75)^2(0.321)\,\frac{3.75}{3}\cos 10 = 48.6 \text{ kN} \cdot \text{m}$$

Find wall h at 3.5 m by proportion:

$$\frac{h'}{3.5} = \frac{2.6}{5.8} \qquad h' = 1.57 \text{ m}$$

Sumber : Bowles, Joseph E, Foundation analysis and design

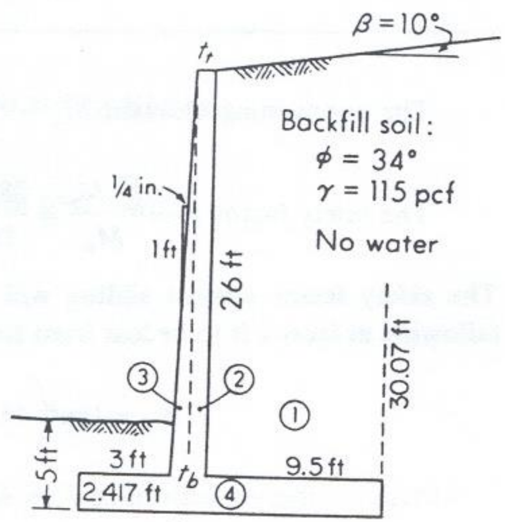
DESIGN OF CANTILEVER RETAINING WALL

Other data: $f'_c = 3$ ksi $\quad f_y = 60$ ksi

$\gamma_c = 0.15$ kcf $\quad LF = 1.8$

Batter on front face of wall = 1 : 48

Top = 16 in



Base soil: φ = 32°
γ = 112 pcf
c = 0.40 ksf

Figure E12-4a

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 3.5 in from cgs to soil interface to allow approximately 3.0 in of clear steel cover.

Step 1 Establish stem dimensions; round to even values:

$$K_a = 0.294 \qquad \text{[Eq. (11-8) and Rankine value]}$$

Stem uses $H = 26$ ft.

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

$$P_{ah} = 11.43 \cos 10° = 11.25 \text{ kips/ft}$$

$$v_c = 2\phi\sqrt{f'_c} = 0.09311 \text{ ksi}$$

$$t = \frac{11.25(1.8)}{(0.093)(12)} = 18.14 + 3.5 = 21.6$$

$$\text{Top} = 21.6 - 26(0.25) = 15.1 \qquad \text{Use 16 in}$$

To maintain even dimensions

$$t = 16 + 26(0.25) = 22.5 \qquad \text{Use 23 in}$$

Step 2 Compute overturning and sliding stability for wall:

$$H' = 26 + 2.42 + 9.5 \tan 10° = 30.1 \text{ ft}$$

$$P'_{ah} = \left(\frac{30.1}{26}\right)^2 11.25 = 15.1 \text{ kips/ft}$$

$$P'_{av} = 15.1 \sin 10° = 2.6 \text{ kips/ft}$$

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

Part	Weight, kips		Arm, ft	M_r, ft · kips
1	0.5(26 + 27.65) × 0.115 × 9.5	= 29.3	9.67	283.3
2	1.33 × 26 × 0.15	= 5.2	4.25	22.1
3	0.5 × 0.59 × 26 × 0.15	= 1.2	3.39	4.0
4	2.42 × 14.42 × 0.15	= 5.2	7.21	37.5
P'_{av}		= 2.6	14.42	37.5
		$\sum W = 43.5$		$\sum M_r = 384.4$

The overturning moment $M_o = P'_{ah}\bar{y} = \dfrac{15.1 \times 30.1}{3} = 151$ ft · kips/ft

The safety factor $F = \dfrac{\sum M_r}{M_o} = \dfrac{384.4}{151} = 2.54 > 1.5$ O.K.

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

$$K_p = \tan^2\left(45 + \frac{32}{2}\right) = 3.255 \qquad \sqrt{K_p} = 1.804$$

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

From integration of Eq. (2-45),

Sumber : Bowles, Joseph E, Foundation analysis and design

DESIGN OF CANTILEVER RETAINING WALL

$$P_p = \tfrac{1}{2}\gamma H^2 K_p + 2cH\sqrt{K_p}$$

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

$$\sum F_r = 31 + 6 = 37 \text{ kips}$$

The resulting $F = \dfrac{\sum F_r}{P'_{ah}} = \dfrac{37}{15.1} = 2.45 > 1.5$ O.K.

Now locate the resultant $= \sum W$ on base and the eccentricity. Find $\bar{x}$ with respect to t 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:

$$q_{ult} = cN_c d_c i_c + qN_q d_q i_q + \tfrac{1}{2}\gamma B N_\gamma d_\gamma i_\gamma \quad \text{(Tab}$$

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

$$N_c = 35.5 \qquad N_q = 23.2 \qquad N_\gamma = 20.8$$

$$i_c = 0.42 \qquad i_q = 0.44 \qquad i_\gamma = 0.309$$

$$d_c = 1.19 \qquad d_q = 1.13 \qquad d_\gamma = 1.0$$

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

$$= 7.1 + 6.5 + 3.9 = 17.5$$

$$q_a = \frac{17.5}{3} = 5.8 \text{ ksf}$$

Actual 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. E12-4*b*.

For toe at stem face $x = 3.00$:

$$q = 5.3 - 0.36 - 0.32x \qquad \text{(neglect soil over toe)}$$

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

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

For heel at approx. cg of tension steel

$$x = 9.5 + \frac{3.5}{12} = 9.79 \text{ ft for moment} \qquad \text{Use 9.5 for shear}$$

Sumber : Bowles, Joseph E, Foundation analysis and design

DESIGN OF CANTILEVER RETAINING WALL

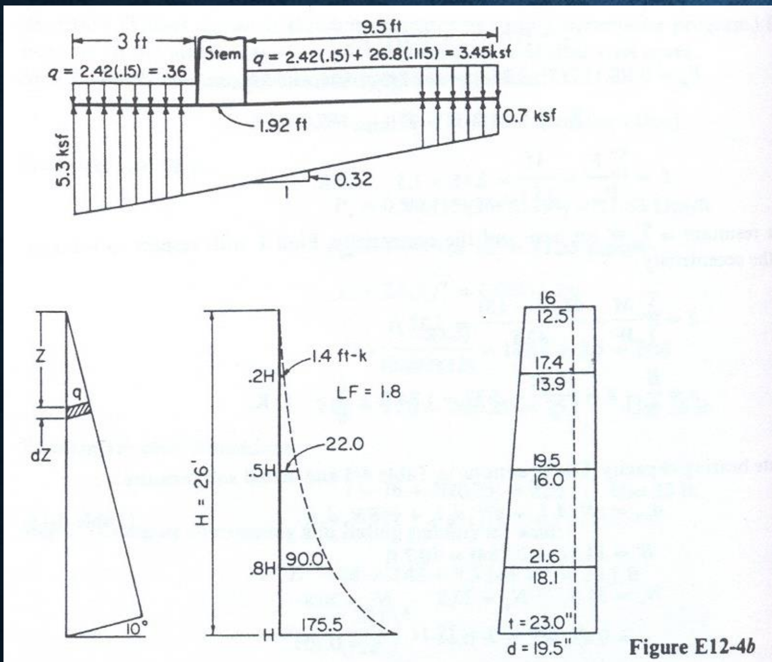


Figure E12-4b

Use average height of soil on heel for downward pressure; include $P_{av} = 2.6$ kips:

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

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

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

Step 5 Check base-slab shear stress using largest base V, $LF = 1.8$, $d = 2.417 - 0.29 = 2.13$ ft:

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

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:

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

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

$$0.5a = 0.5\frac{A_s f_y}{0.85 f'_c b} = 0.98\ A_s$$

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

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

Sumber : Bowles, Joseph E, Foundation analysis and design

DESIGN OF CANTILEVER RETAINING WALL

$$A_s^2 - 26.02A_s = -43.76$$

$$A_s = 1.81 \text{ in}^2/\text{ft} \qquad p = 0.006$$

For toe

$$A_s^2 - 26.02A_s = -8.49$$

$$A_s = 0.33 \text{ in}^2 \qquad p = 0.0011 < 0.0033$$

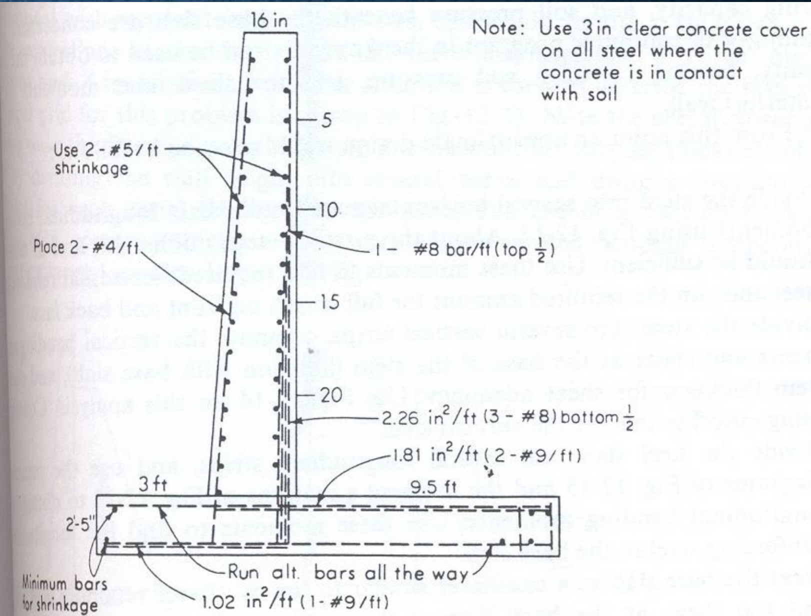
$$A_s = 0.0033(12)(25.5) = 1.02 \text{ in}^2/\text{ft}$$

Step 7 Compute stem steel at top, $0.5H$, $0.8H$, and H using $LF = 1.8$.

Point	M, ft·kips	Wall thickness	d	A_s^*	p (used)
0	0	16.0	12.5	0.5	0.0033
$0.5H$	22.0	19.5	16.0	0.64	0.0033
$0.8H$	90.0	21.60	18.1	1.18	0.005
H	175.8	23.0	19.5	2.26	0.010

* in²/ft

From this it is evident minimum $(200/f_y)$ requirements control top half of wall. Use 3 No. 8 bars/ft ($A_s = 2.37$ in²) in bottom $\frac{1}{2}$ of wall.

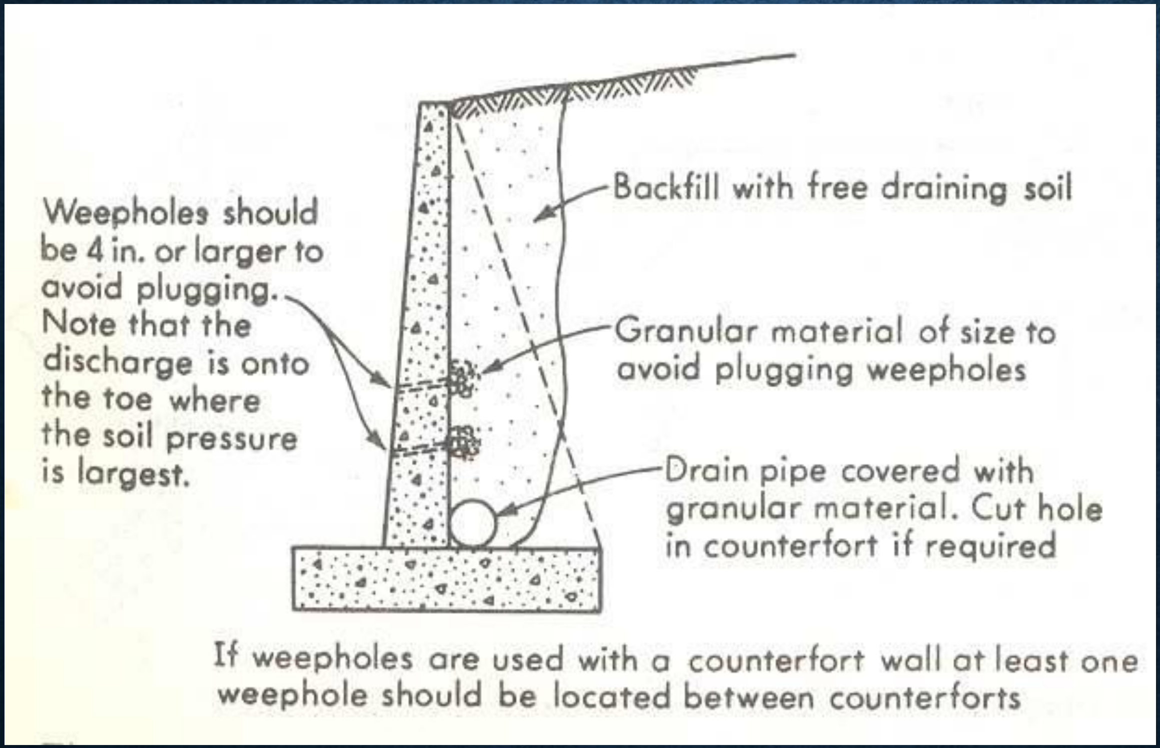


Final design sketch
(not to scale)

Figure E12-4c

Sumber : Bowles, Joseph E, Foundation analysis and design

DRAINAGE WALL

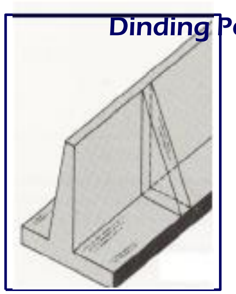


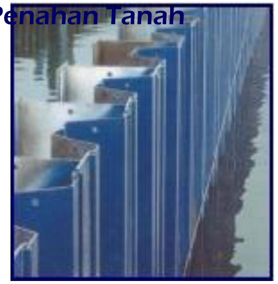
Sertifikasi HATTI:

Tegangan Lateral

dan

Dinding Penahan Tanah





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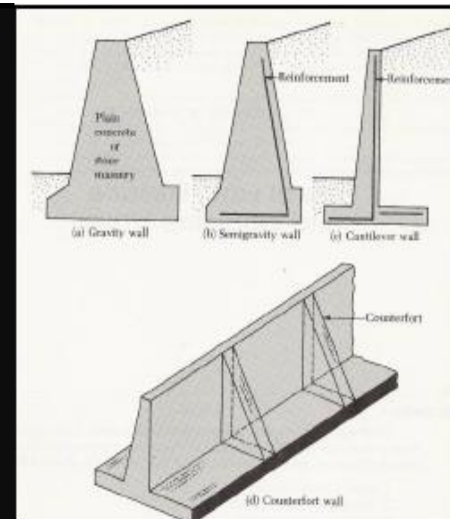
Teknik Sipil, Institut Teknologi Bandung



(Retaining wall Gabion)

Sheet Pile

Braced Cut Excavation

CONTOH STRUKTUR SHEET PILE

Contoh PelaksanaanDiapraghmWall

Clamshell Grabber

Clamshell Grabber

CWS (Cofrage Avec Waterstopper)

Stopper baja untuk mencetak bentuk ujung kiri-kanan panel

Rebar cage + anchor sleeve

Rebar Lifting

Rebar Lifting

Rebar Lifting

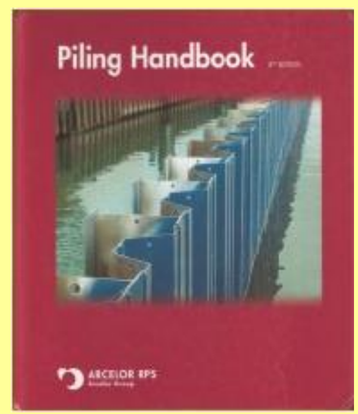
GEDUNG PERKANTORAN JAKARTA

Ground Anchor

GEDUNG KANTOR

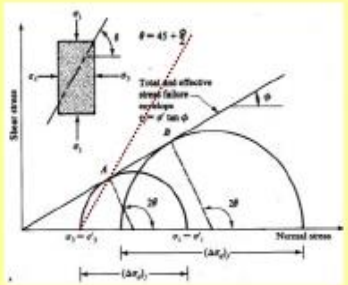
PLUIT SELATAN RAYA - JAKARTA

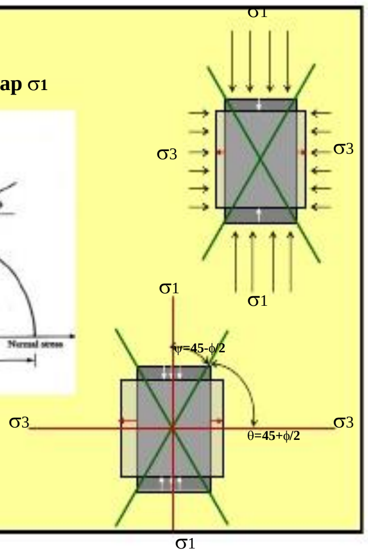
Rekomendasi Buku Untuk Sheet Pile



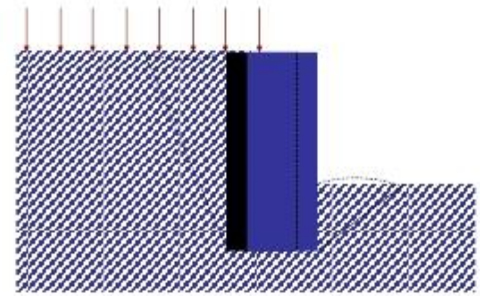
Test Triaxial:

Arah Bidang Runtuh Terhadap σ1



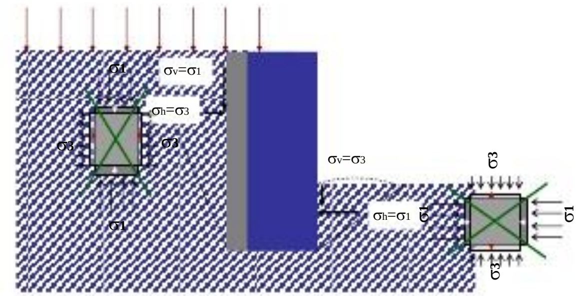


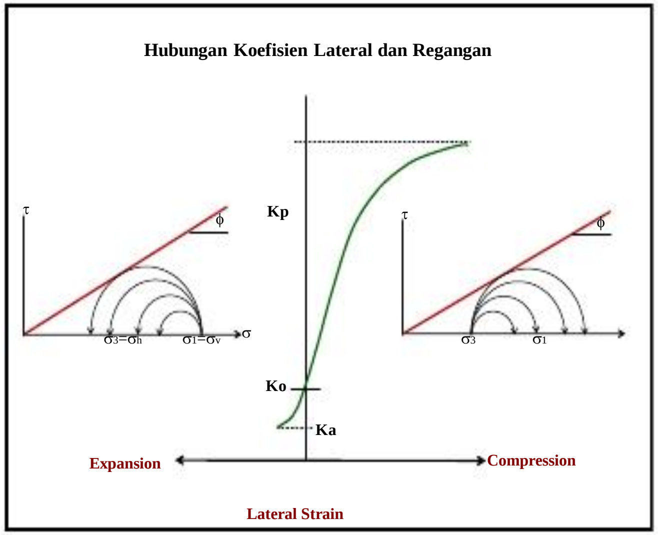
Mekanisme Aktif dan Passif

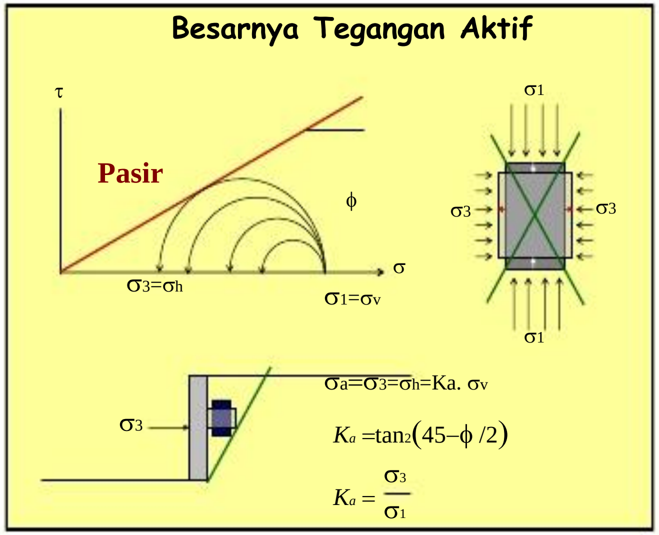


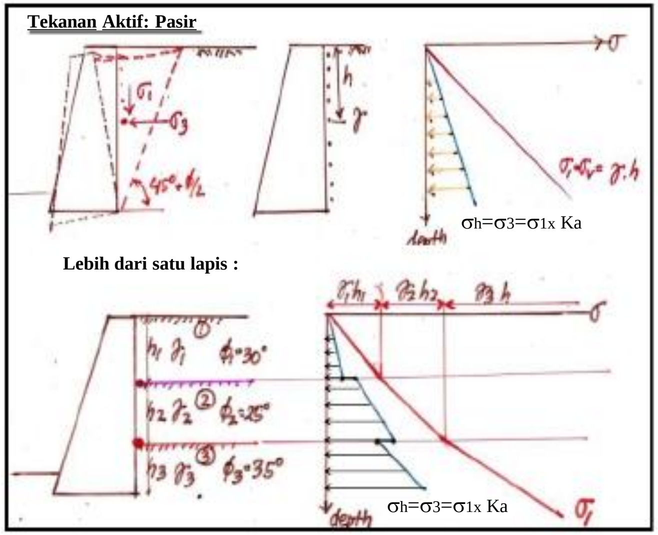
Mekanisme Aktif dan Passif

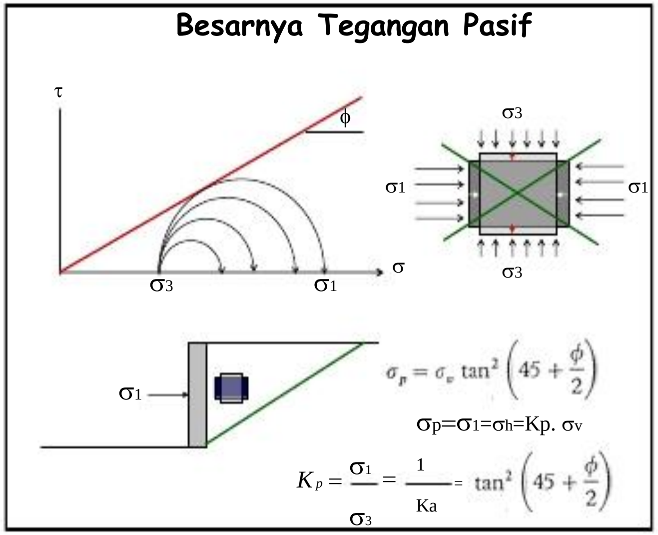
Mekanisme Aktif dan Passif



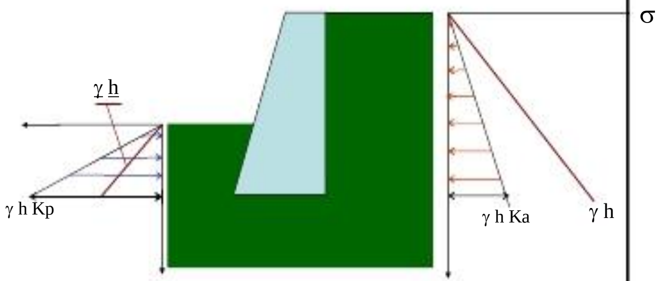

Hubungan Koefisien Lateral dan Regangan
τ
ϕ
σ3=σh
σ1=σv
σ
Kp
Ko
Ka
τ
ϕ
σ3
σ1
Expansion
Compression
Lateral Strain


Besarnya Tegangan Aktif
τ
Pasir
ϕ
σ
σ3=σh
σ1=σv
σ1
σ3
σ3
σ1
σ3
σa=σ3=σh=Ka. σv
$K_a = \tan^2(45 - \phi/2)$
$K_a = \frac{\sigma_3}{\sigma_1}$


Tekanan Aktif: Pasir
σh=σ3=σ1x Ka
Lebih dari satu lapis :
σh=σ3=σ1x Ka


Besarnya Tegangan Pasif
τ
ϕ
σ
σ3
σ1
σ3
σ1
σ1
σ3
σ1
$\sigma_p = \sigma_v \tan^2\left(45 + \frac{\phi}{2}\right)$
σp=σ1=σh=Kp. σv
$K_p = \frac{\sigma_1}{\sigma_3} = \frac{1}{Ka} = \tan^2\left(45 + \frac{\phi}{2}\right)$

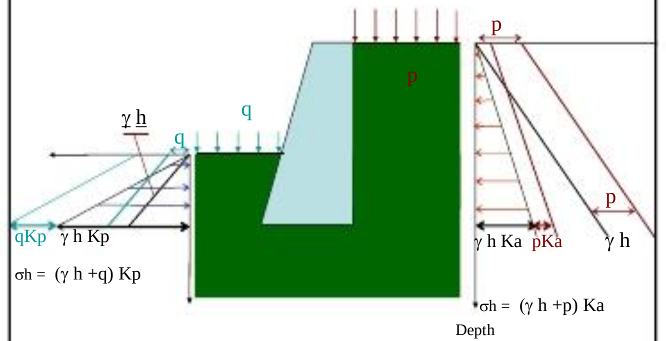
Tekanan Aktif dan Pasif: Pasir



Depth

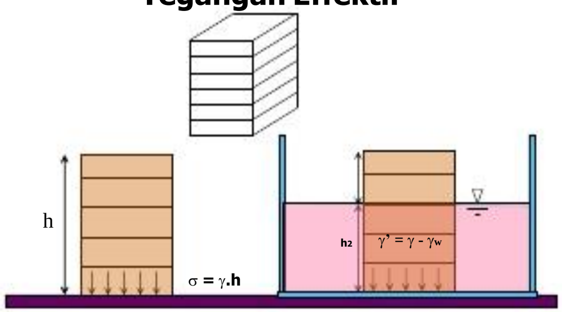
Tekanan Aktif dan Pasif: Pasir

Ada Beban Merata



Depth

Tegangan Effektif



σ' = γh1 + γ'h2

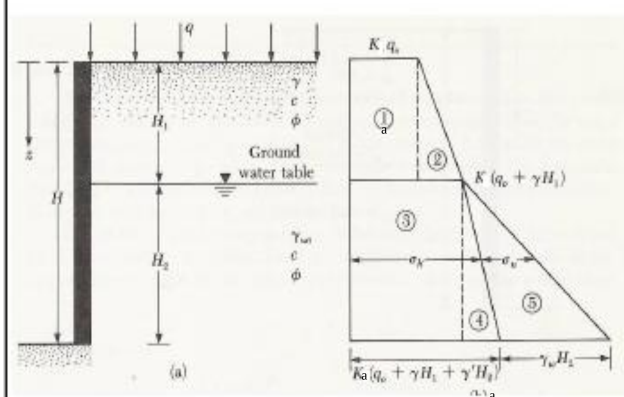
= γ (h1+ h2) - γw.h2

σ' = σ - μ

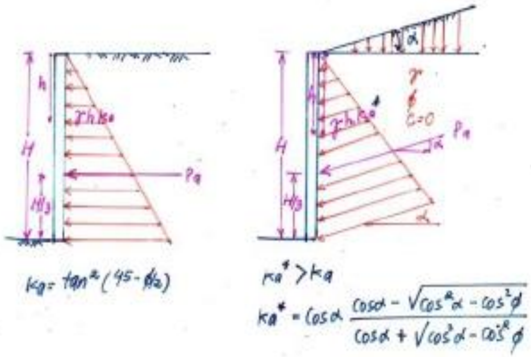
σ'= tegangan efektif

σ = tegangan total

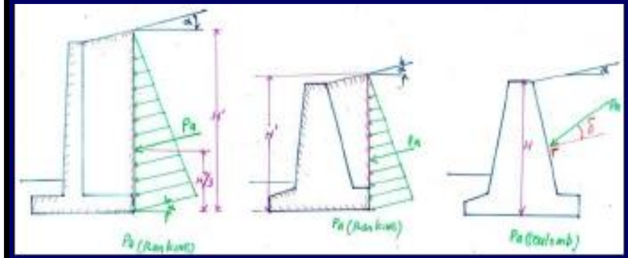
Pengaruh Beban dan Air Tanah



Pengaruh kemiringan tanah diatas:



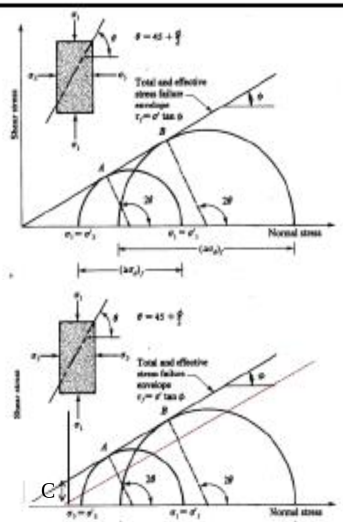
Cantilever & Gravity Wall



Cantilever

Gravity Wall

Pasir



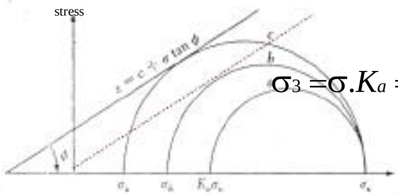
Lempung

Penaruh C-Kohesi Tanah

Tekanan Aktif Rankine

Shear

stress



$$\sigma_3 = \sigma . K_a = \sigma_v K_a - 2c\sqrt{K_a}$$

Normal

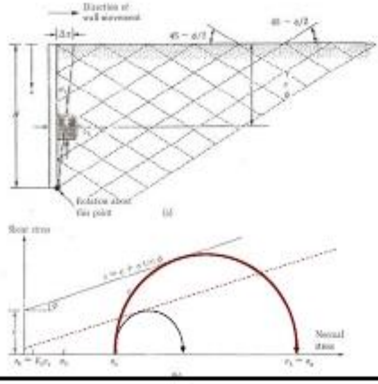
stress

(b)

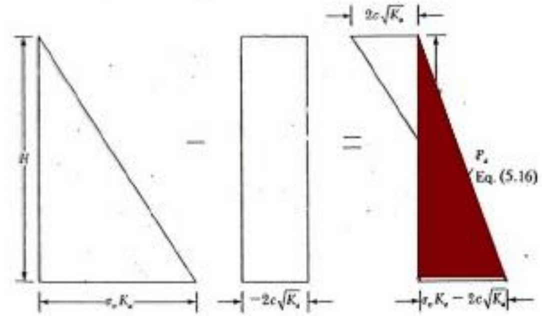
Tekanan Pasif Rankine

$\sigma_p = \sigma_v \tan^2\left(45+\frac{\phi}{2}\right) + 2c\tan\left(45+\frac{\phi}{2}\right)$

$\sigma_1 = \sigma_v . K_p = \sigma_v K_p + 2c\sqrt{K_p}$

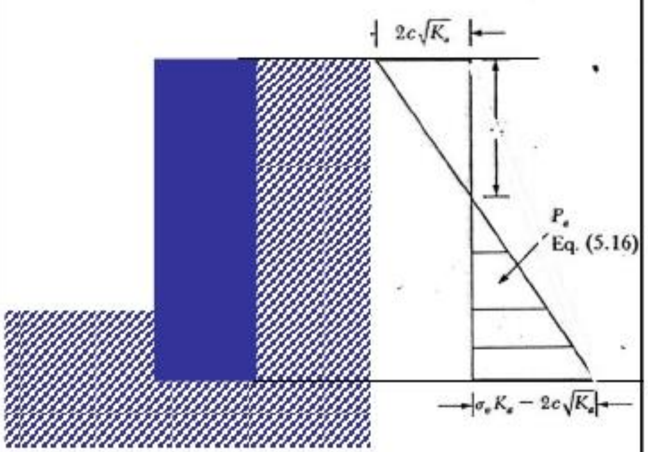


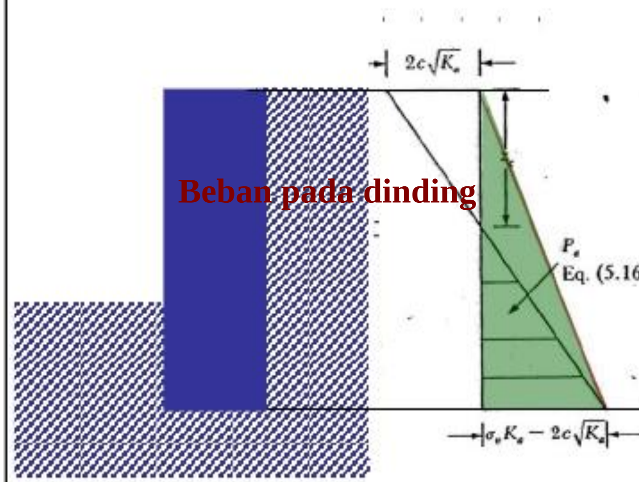
Tekanan Aktif Rankine untuk Perencanaan



(c)

Beban pada dinding





Disain Dinding Penahan Tanah:

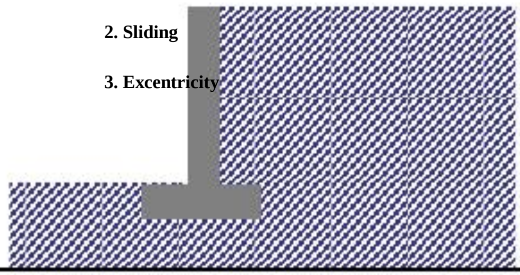
Stabilitas

4. Bearing Capacity

1. Overtuning

2. Sliding

3. Excentricity



Stabilitas Retaining Wall

1. Overtuning

2. Sliding

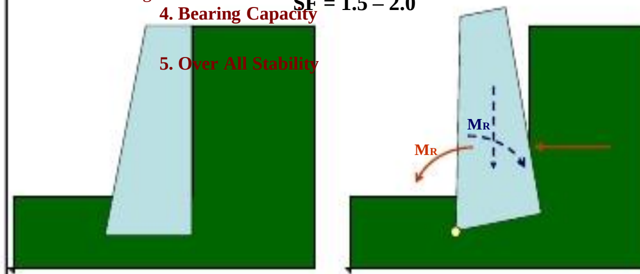
3. Excentricity

4. Bearing Capacity

5. Over All Stability

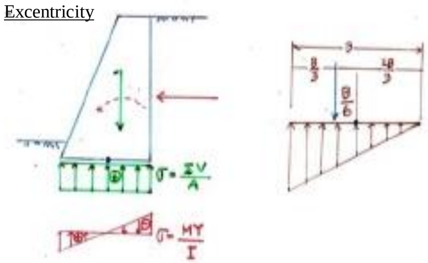
1. Overturning

SF = 1.5 – 2.0

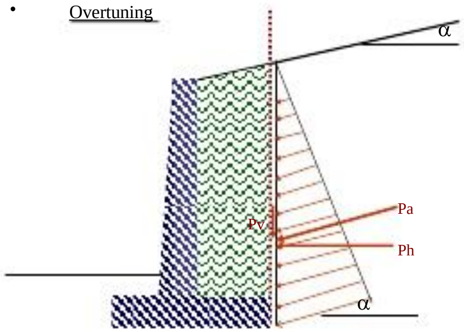


- 2. Sliding

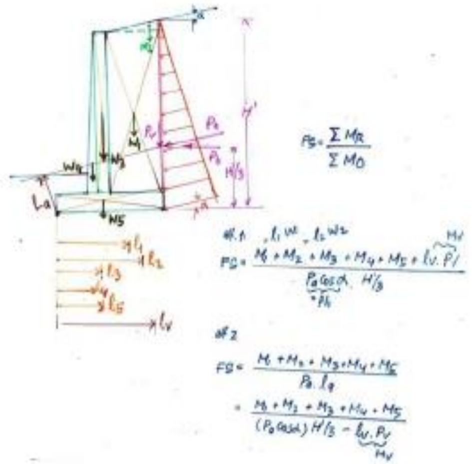
- 3. Excentricity

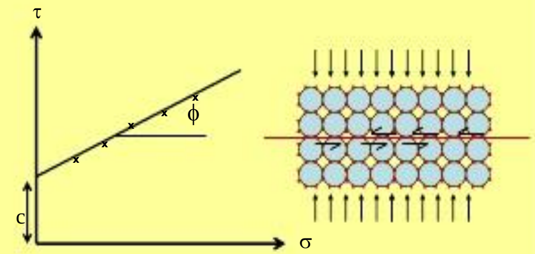


- Overtuning



- Overtuning





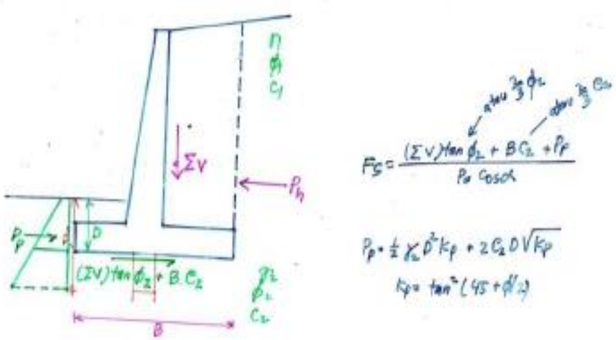
$s = c + \sigma' \tan \phi$

Where σ' = effective normal stress on plane of shearing

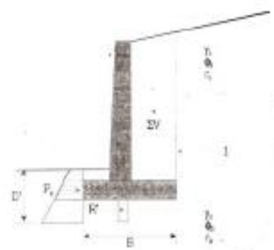
c = cohesion, or apparent cohesion

ϕ = angle of friction

- Sliding



- Sliding



$$FS_{sliding} = \frac{\sum F_{R'}}{\sum F_d}$$

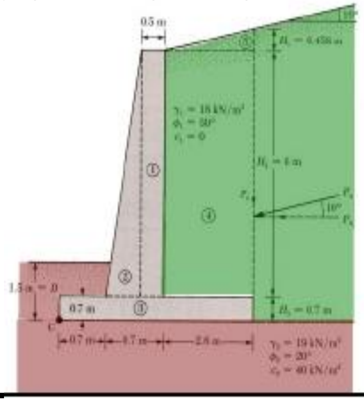
$$\sum F_{R'} = (\sum V)\tan(k_1 \phi_2) + B k_2 c_2 + P_p$$

$$\sum F_d = P_a \cos \alpha$$

$$FS_{sliding} = \frac{(\sum V)\tan(k_1 \phi_2) + B k_2 c_2 + P_p}{P_a \cos \alpha}$$

1. Contoh Soal

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



1. Solusi :

$$H' = H_1 + H_2 + H_3 = 2.6 \tan 10° + 6 + 0.7$$
$$= 0.458 + 6 + 0.7 = 7.158 \text{ m}$$

Rankine active force per unit length of wall $= P_a = \frac{1}{2}\gamma_1 H'^2 K_a$. For $\phi_1 = 30°$, $\alpha = 10°$, K_a is equal to 0.350 (Table 5.6). Thus,

$$P_a = \tfrac{1}{2}(18)(7.158)^2(0.35) = 161.4 \text{ kN/m}$$
$$P_v = P_a \sin 10° = 161.4(\sin 10°) = 28.03 \text{ kN/m}$$
$$P_h = P_a \cos 10° = 161.4(\cos 10°) = 158.95 \text{ kN/m}$$

Factor of Safety Against Overturning

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

Section* No.	Area (m²)	Weight/unit length (kN/m)	Moment arm from point C (m)	Moment (kN-m)
1	$6 \times 0.5 = 3$	70.74	1.15	81.35
2	$\frac{1}{2}(0.2)6 = 0.6$	14.15	0.833	11.79
3	$4 \times 0.7 = 2.8$	66.02	2.0	132.04
4	$6 \times 2.6 = 15.6$	280.80	2.7	758.16
5	$\frac{1}{2}(2.6)(0.458) = 0.595$	10.71	3.13	33.52
		$P_v = 28.03$	4.0	112.12
		$\sum V = 470.45$		$\sum$ 1128.98 $= \sum M_R$

*For section numbers, refer to Figure 5.34
$\gamma_{concrete} = 23.58 \text{ kN/m}^3$

The overturning moment, M_O

$$M_O = P_h\left(\frac{H'}{3}\right) = 158.95\left(\frac{7.158}{3}\right) = 379.25 \text{ kN-m}$$

$$FS_{(overturning)} = \frac{\Sigma M_R}{M_O} = \frac{1128.98}{379.25} = \underline{2.98 > 2\text{—O.K.}}$$

Factor of Safety Against Sliding

From Eq. (5.60)

$$FS_{(sliding)} = \frac{(\Sigma V)\tan(k_1\phi_2) + Bk_2c_2 + P_p}{P_a \cos\alpha}$$

Let $k_1 = k_2 = 2/3$
Also

$$P_p = \frac{1}{2}K_p\gamma_2 D^2 + 2c_2\sqrt{K_p}D$$

$$K_p = \tan^2\left(45 + \frac{\phi_2}{2}\right) = \tan^2(45 + 10) = 2.04$$

$$D = 1.5 \text{ m}$$

So

$$P_p = \tfrac{1}{2}(2.04)(19)(1.5)^2 + 2(40)(\sqrt{2.04})(1.5)$$
$$= 43.61 + 171.39 = 215 \text{ kN/m}$$

Hence

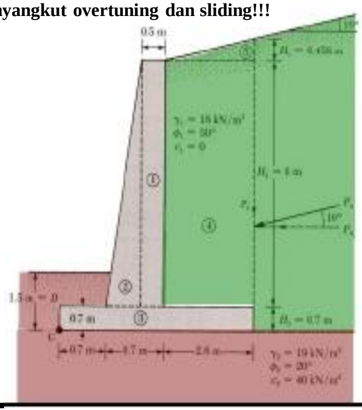
$$FS_{(sliding)} = \frac{(470.45)\tan\left(\frac{2 \times 20}{3}\right) + (4)\left(\frac{2}{3}\right)(40) + 215}{158.95}$$

$$= \frac{111.5 + 106.67 + 215}{158.95} = \underline{2.73 > 1.5\text{—O.K.}}$$

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 overtuning dan sliding!!!



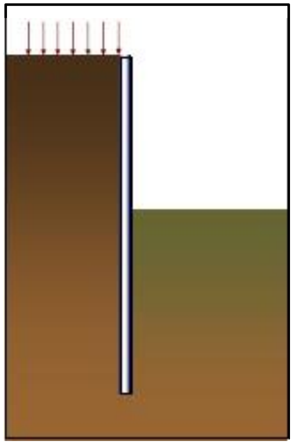
Sheet Pile

KLASIFIKASI STRUKTUR SHEET PILE

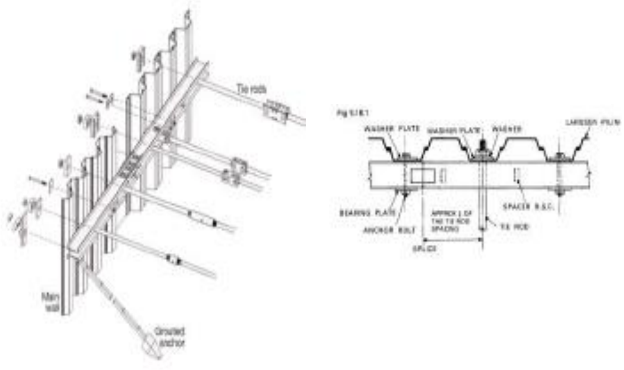
- Sheet Pile Structures
  - Cantilever Type
  - Anchored Type
    - Tie-Rod Type
      - Single Tie-Rod Type
      - Double Tie-Rod Type
      - Deadman Type
    - Batter-Pile Anchor Type
    - Relieving Platform Type
  - Cellular Type
  - Others
    - Double Sheet Pile Wall Type
    - Multi-Strut Type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

Steel Pipe Pile - Japan

Cantilever Pile/ Sheet Pile

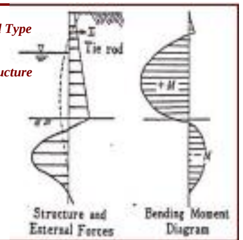


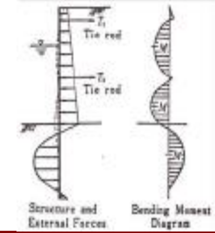
Anchored Type



Anchored Type

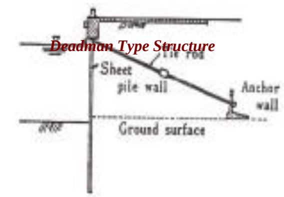
Tie Rod Type Structure



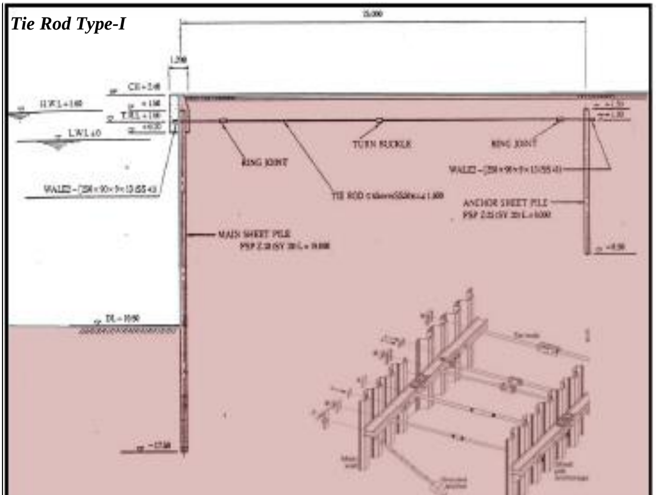


Double Tie Rod Type Structure

Deadman Type Structure

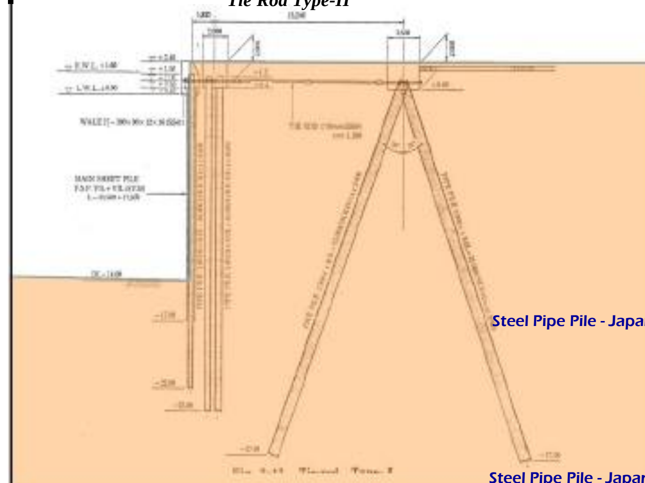


Tie Rod Type-I



Steel Pipe Pile - Japan

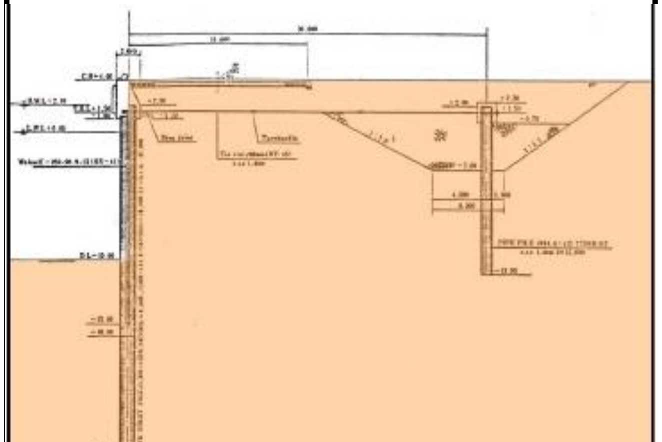
Tie Rod Type-II



Steel Pipe Pile - Japan

Steel Pipe Pile - Japan

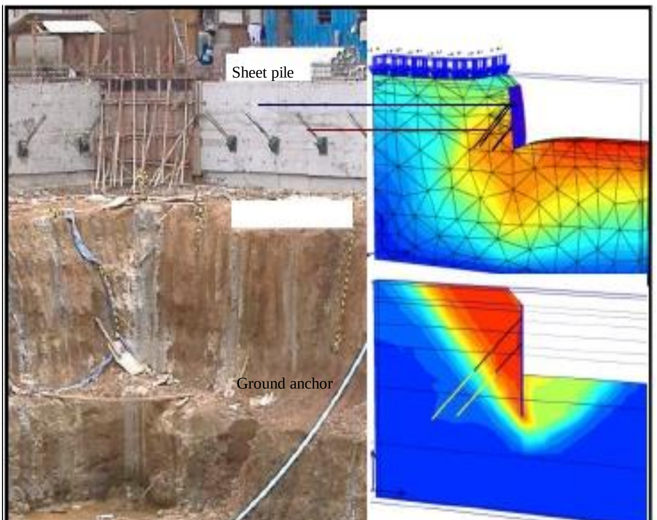
Tie Rod Type-III



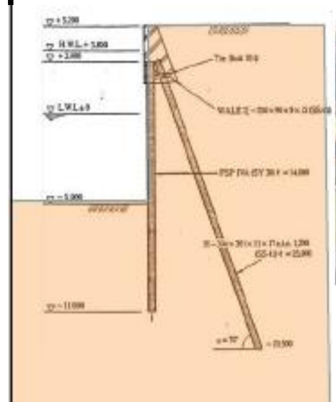
Steel Pipe Pile - Japan

Anchored Sheet Pile

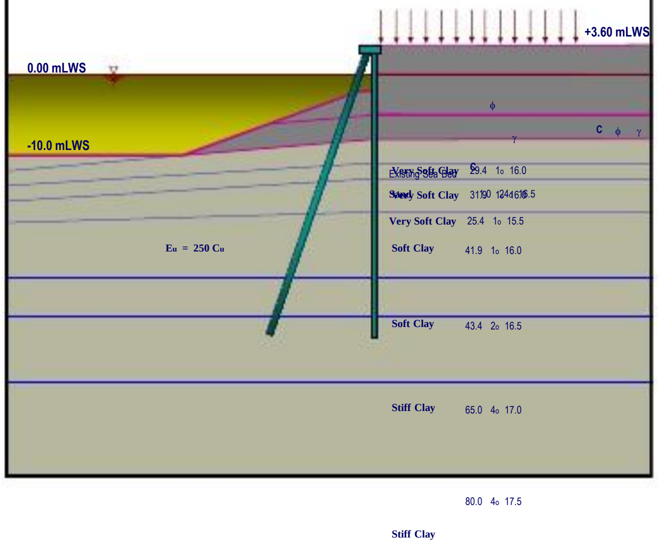
Piling Handbook - Arcelor



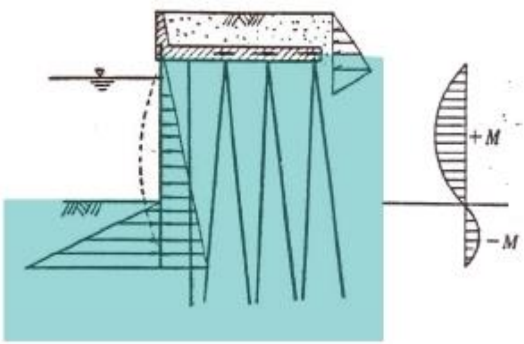
Batter-Pile Anchor Type



Dermaga Tanjung Emas

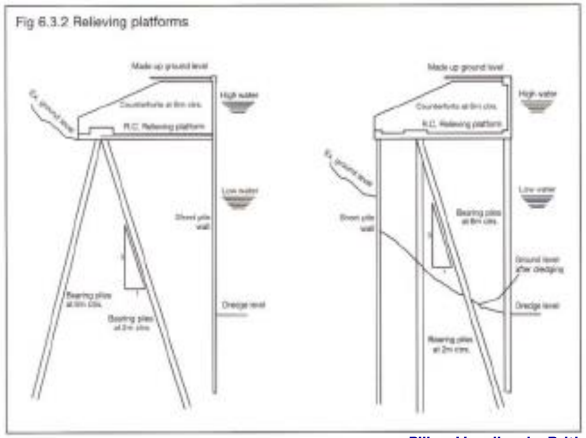


Relieving Platform Type Structure

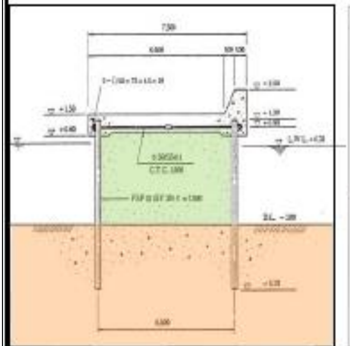


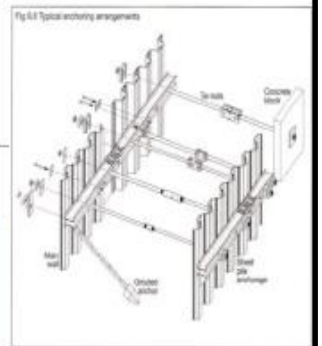
Steel Pipe Pile - Japan

Relieving Platforms Retaining Wall



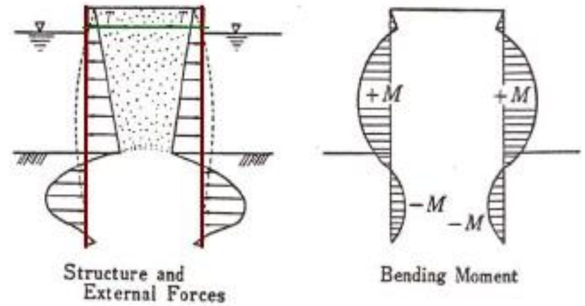
Piling Handbook - British





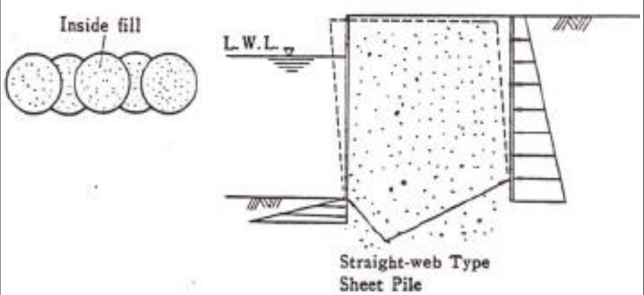
Double Sheet Pile Wall Type

Double Sheet Pile Wall Type Structure



Steel Pipe Pile - Japan

Cellular Type Structure

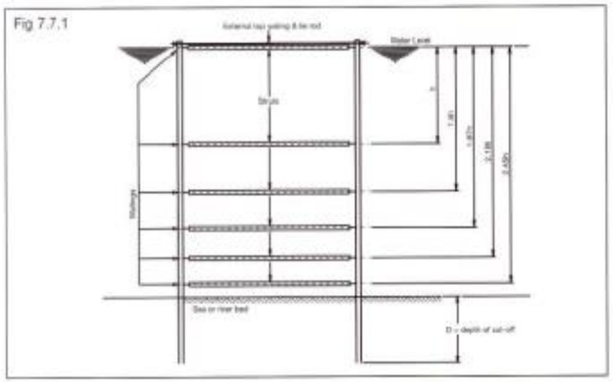


Piling Handbook - Arcelor

Cofferdams

Piling Handbook - Arcelor

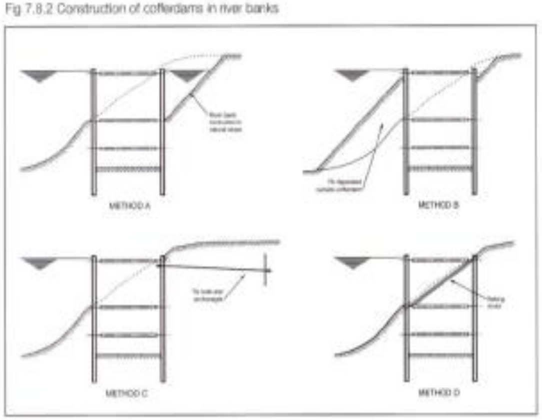
Cofferdams



Piling Handbook - Arcelor

Cofferdams

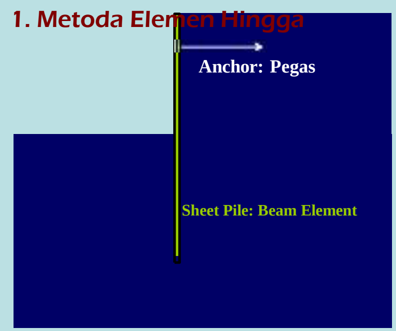
Fig 7.8.2 Construction of cofferdams in river banks



Piling Handbook - Arcelor

Metoda Perhitungan Sheet Pile

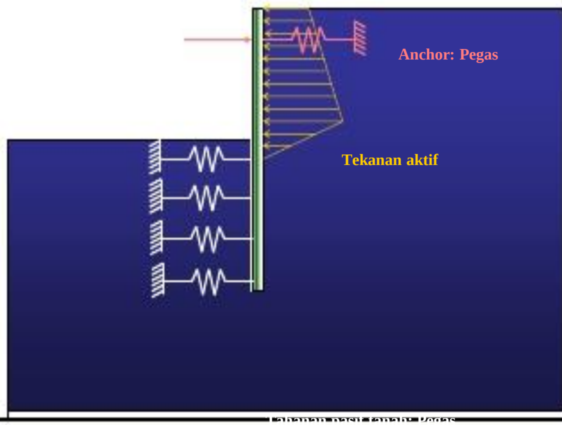
1. Metoda Elemen Hingga



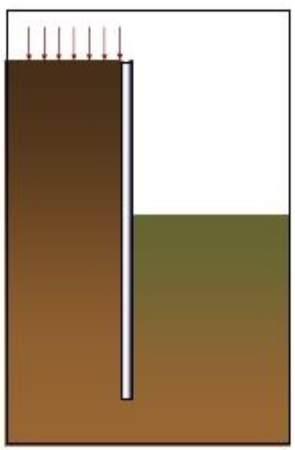
Parameter Tanah:
C (kohesi)
ϕ (Sudut geser dalam)
E (Young's modulus)
υ (Poisson's ratio)

M < M konvensional

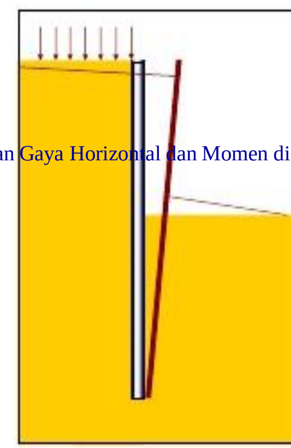
2. Beam on Elastic Foundation



3. Metoda Konvensional



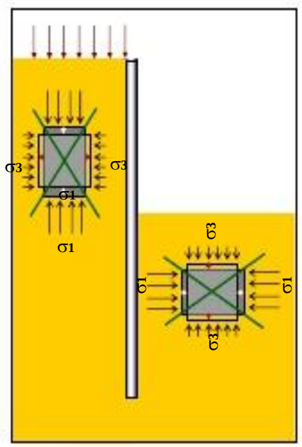
Cantilever Pile/ Sheet Pile



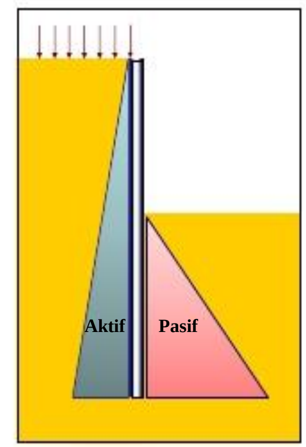
Keseimbangan Gaya Horizontal dan Momen di Titik O

Cantilever Pile/ Sheet Pile

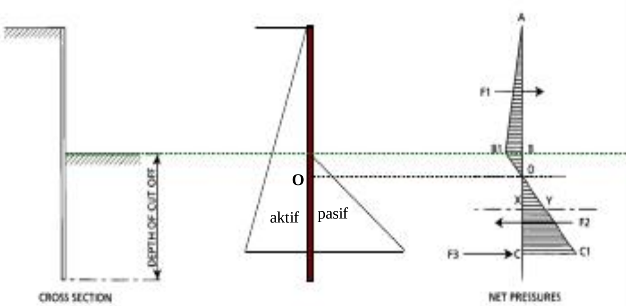
Cantilever Pile/ Sheet Pile



Tegangan Lateral Pada Cantilever Pile/ Sheet Pile



Cantilever Sheet Pile



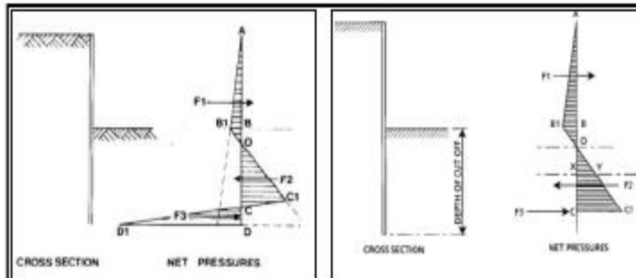
C

Σ Mc = 0

D

OD = 1.20 OC

Mmax pada zero shear



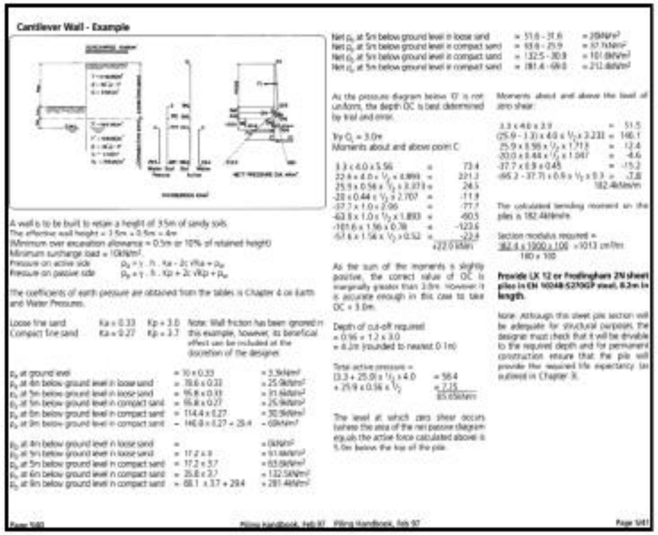
Hence, depth of cut off = B.O. + 1.2 x O.C.

F1 (Representing area A.O.B1) = total net active pressure
F2 (Representing area O.C.C1) = 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.

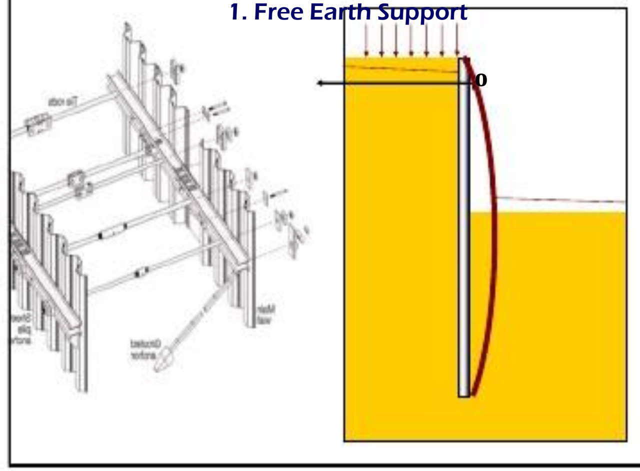
Calculations may be simplified by considering the line C1.C.D1. to be horizontal and to pass through point C. The area C. D. D1. is replaced by force F3 acting at C as shown in Fig 5.23.2 below.

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.


Cantilever Wall - Example

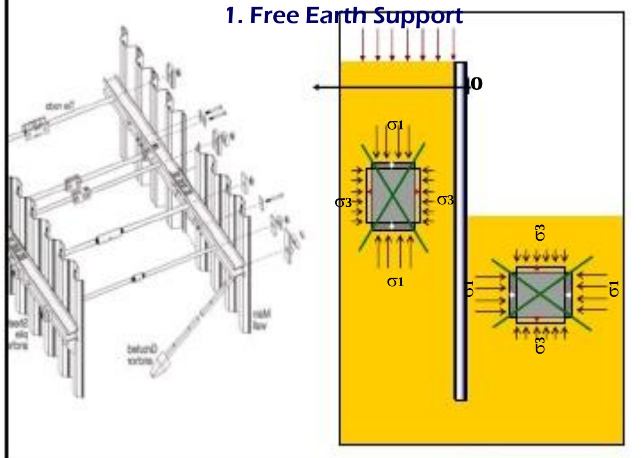
Metoda Konvensional:

1. Free Earth Support

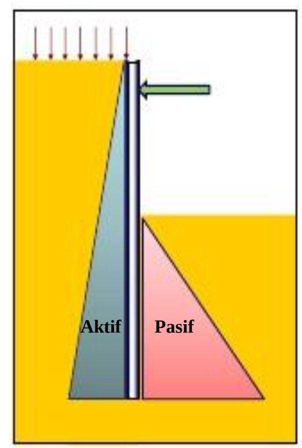


Metoda Konvensional:

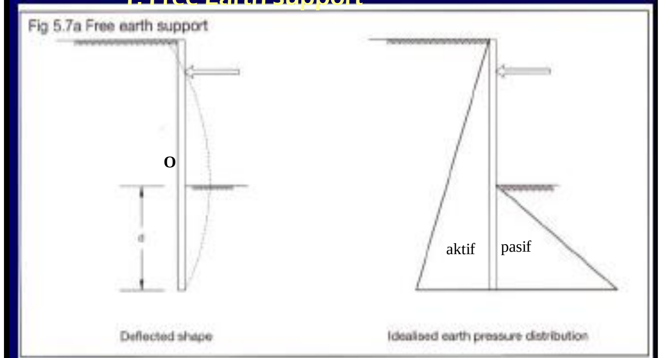
1. Free Earth Support



Tegangan Lateral Anchored Sheet Pile

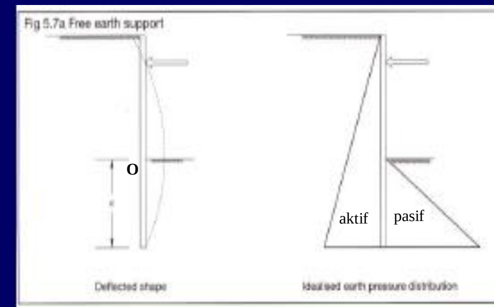


Metoda Konvensional: 1. Free Earth Support

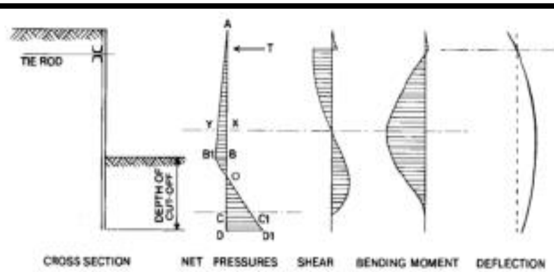


Panjang teoritis d: Keseimbangan momen di titik O
Gaya angkur : Keseimbangan gaya horizontal

Metoda Konvensional: 1. Free Earth Support



Untuk Disain Panjang d : SF momen di titik O = 2.0



The depth of cut off should be sufficient to give a factor of safety of 2 against rotation of the wall about the waling.

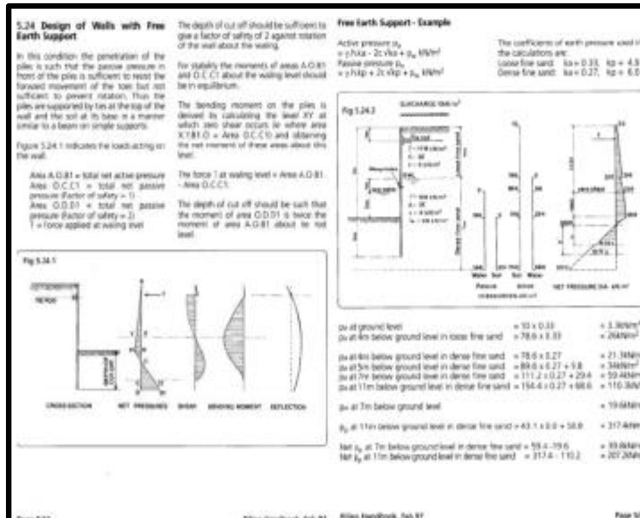
For stability the moments of areas A.O.B1 and O.C.C1 about the waling level should be in equilibrium.

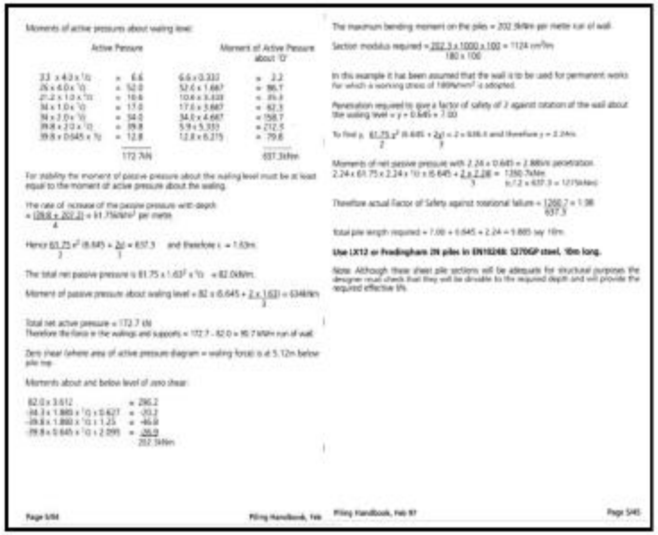
The depth of cut off should be such that the moment of area O.D.D1 is twice the moment of area A.O.B1 about tie rod level.

The bending moment on the piles is derived by calculating the level XY at which zero shear occurs (ie where area X.Y.B1.O = Area O.C.C1) and obtaining the net moment of these areas about this level.

The force T at waling level = Area A.O.B1. - Area O.C.C1.

Piling Handbook -British





Moments of active pressures about waling level:

Active Pressure		Moment of Active Pressure about 'O'	
3.3 x 4.0 x ½	= 6.6	6.6 x 0.333	= 2.2
26 x 4.0 x ½	= 52.0	52.0 x 1.667	= 86.7
21.2 x 1.0 x ½	= 10.6	10.6 x 3.333	= 35.3
34 x 1.0 x ½	= 17.0	17.0 x 3.667	= 62.3
34 x 2.0 x ½	= 34.0	34.0 x 4.667	= 158.7
39.8 x 2.0 x ½	= 39.8	39.8 x 5.333	= 212.5
39.8 x 0.645 x ½	= 12.8	12.8 x 6.215	= 79.8
	172.7kN		637.3kNm

For stability the moment of passive pressure about the waling level must be at least equal to the moment of active pressure about the waling.

The rate of increase of the passive pressure with depth $= \frac{(39.8 + 207.2)}{4} = 61.75 kN/m^2$ per metre

Hence $\frac{61.75}{2} x^2 (6.645 + \frac{2x}{3}) = 637.3$ and therefore x = 1.63m.

The total net passive pressure is $61.75 \times 1.63^2 \times ½ = 82.0 kN/m$.

Moment of passive pressure about waling level $= 82 \times (6.645 + \frac{2 \times 1.63}{3}) = 634 kNm$

Total net active pressure = 172.7 kN
Therefore the force in the walings and supports = 172.7 - 82.0 = 90.7 kN/m run of wall.

Zero shear (where area of active pressure diagram = waling force) is at 5.12m below pile top.

Moments about and below level of zero shear:

82.0 x 3.612	= 296.2
-34.3 x 1.880 x ½ x 0.627	= -20.2
-39.8 x 1.880 x ½ x 1.25	= -46.8
-39.8 x 0.645 x ½ x 2.095	= -26.9
	202.3kNm

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The maximum bending moment on the piles = 202.3kNm per metre run of wall.

Section modulus required $= \frac{202.3 \times 1000 \times 100}{180 \times 100} = 1124 cm^3/m$

In this example it has been assumed that the wall is to be used for permanent works for which a working stress of $180N/mm^2$ is adopted.

Penetration required to give a factor of safety of 2 against rotation of the wall about the waling level = y + 0.645 + 7.00

To find y, $\frac{61.75 y^2}{2} (6.645 + \frac{2y}{3}) = 2 \times 636.6$ and therefore y = 2.24m

Moments of net passive pressure with 2.24 + 0.645 = 2.885m penetration
$2.24 \times 61.75 \times 2.24 \times ½ \times (6.645 + \frac{2 \times 2.24}{3}) = 1350.7kNm$ (≥ 2 x 637.3 = 1275kNm)

Therefore actual Factor of Safety against rotational failure $= \frac{1260.7}{637.3} = 1.98$

Total pile length required = 7.00 + 0.645 + 2.24 = 9.885 say 10m

Use LX12 or Frodingham 2N piles in EN10248: S270GP steel, 10m long.

Note: Although these sheet pile sections will be adequate for structural purposes the designer must check that they will be drivable to the required depth and will provide the required effective life.

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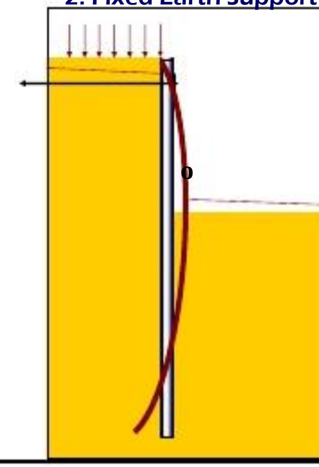
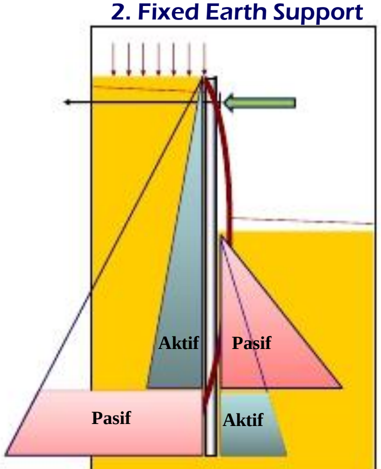
Metoda Konvensional:

2. Fixed Earth Support



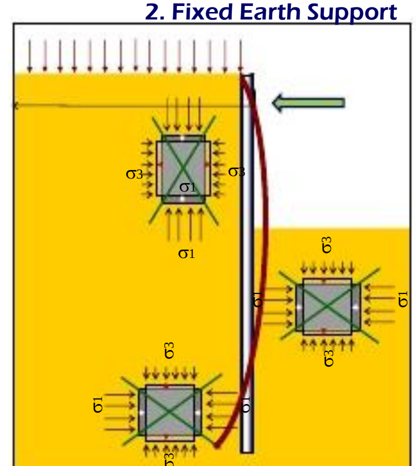
Metoda Konvensional:

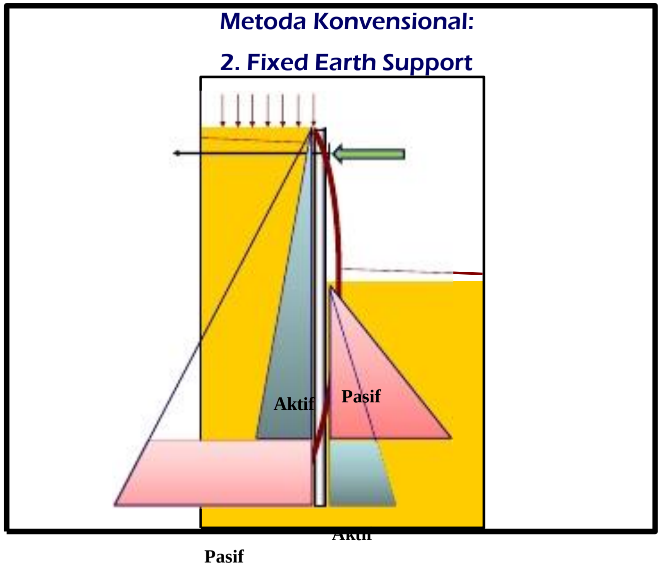
2. Fixed Earth Support



Metoda Konvensional:

2. Fixed Earth Support




Metoda Konvensional:
2. Fixed Earth Support
Aktif
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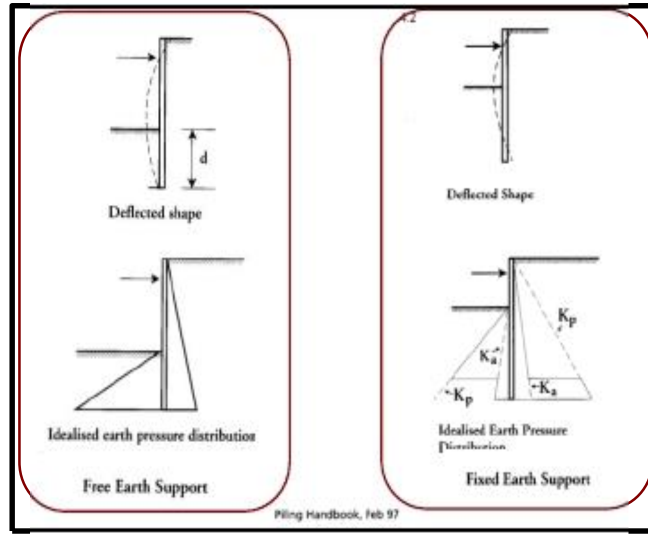
d
Deflected shape
Idealised earth pressure distribution
Free Earth Support
Deflected Shape
Kp
Ka
Kp
Ka
Idealised Earth Pressure Distribution
Fixed Earth Support
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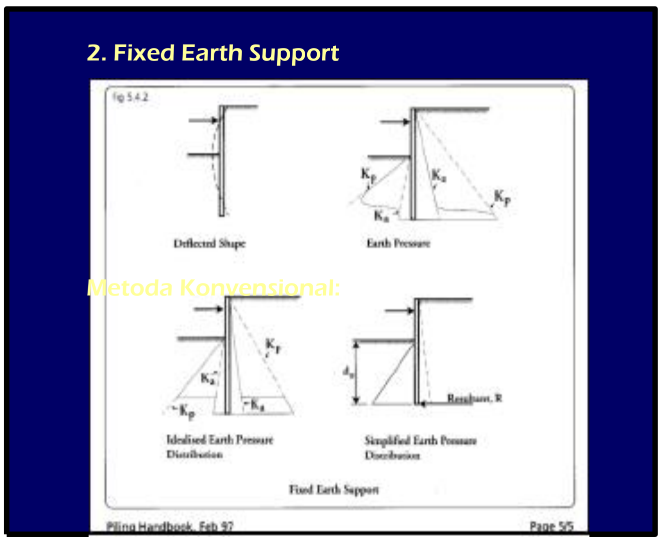
2. Fixed Earth Support
Deflected Shape
Earth Pressure
Metoda Konvensional:
Idealised Earth Pressure Distribution
Simplified Earth Pressure Distribution
Resultant, R
Fixed Earth Support
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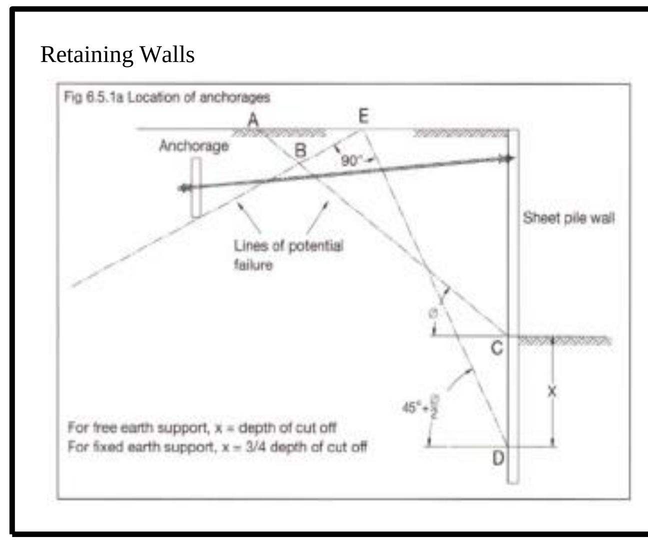
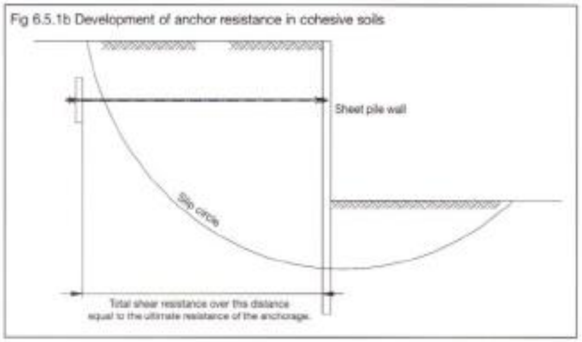
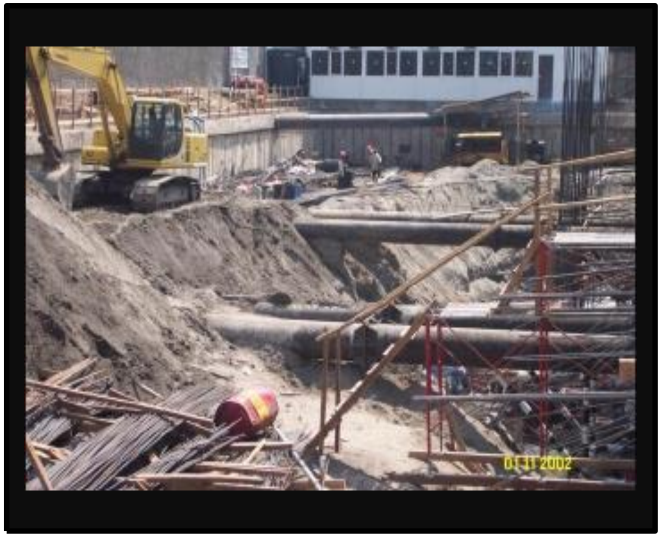

Retaining Walls
Fig 6.5.1a Location of anchorages
A
E
B
Anchorage
90°
Sheet pile wall
Lines of potential failure
C
X
D
For free earth support, x = depth of cut off
For fixed earth support, x = 3/4 depth of cut off

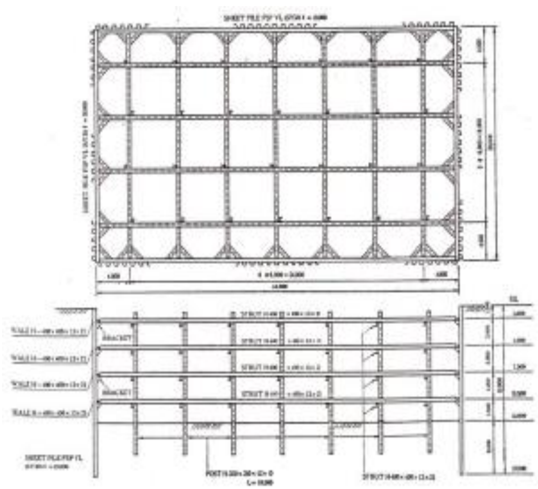
Retaining Walls



CONTOH STRUKTUR SHEET PILE

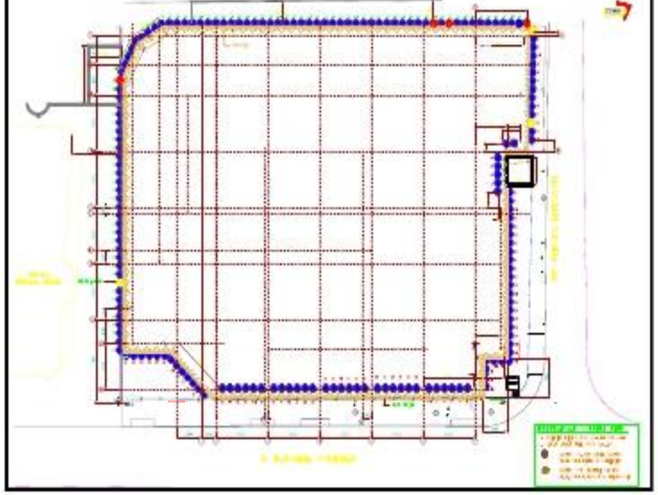
Braced Cut Excavation

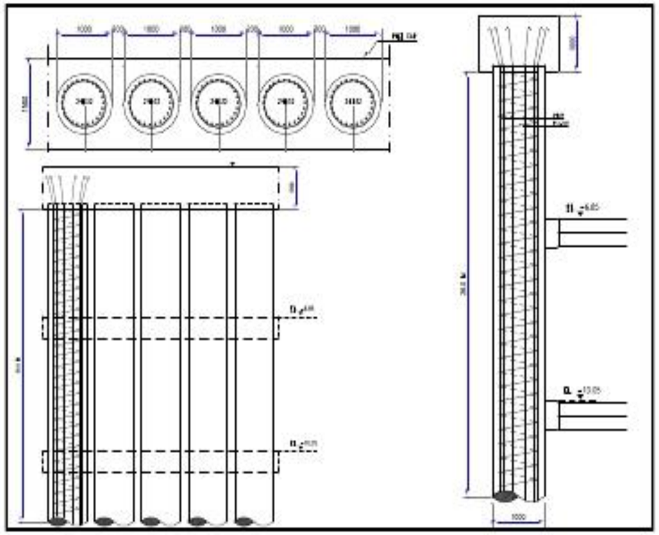




Multi-Strut Type

BORED PILE DENGAN DISKONTINUITAS DARI HASIL PENGUJIAN





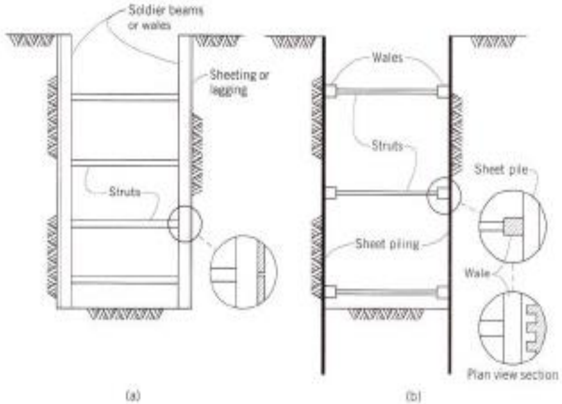
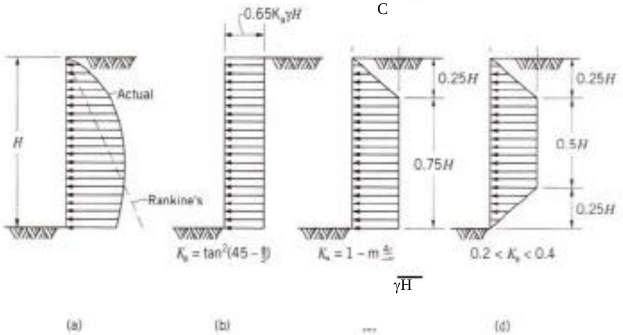


Figure 10.39 Common methods of bracing cuts. (a) Lagging. (b) Sheet piles.

Sand **Clay**

Soft to Medium **Stiff**

$\frac{\gamma H}{C} > 4$ $\frac{\gamma H}{C} < 4$



Thank You!