5

Dinding Penahan Tanah

PENDAHULUAN

Dinding penahan tanah biasanya digunakan untuk menahan tanah dan struktur yang terletak pada dua level permukaan tanah yang berbeda akibat penggalian atau penimbunan. Ada banyak jenis dinding penahan tanah yang dapat dipilih berdasarkan beberapa faktor. Pertama, dinding penahan tanah digolongkan menjadi empat kelompok yaitu : (a) Gravity wall (dinding graviti); (b) Cantilever wall (dinding kantilever) (c) sheet pile dan (d) anchored wall (angkur). Termasuk dalam tipe anchored wall adalah counter-fort wall, soil nailing, dan reinforced soil wall.

Dinding tipe graviti adalah tipe yang paling sering digunakan apabila konstruksi dilaksanakan di atas muka tanah. Stabilitas didapatkan oleh berat sendiri atau kombinasi beray sendiri dinding dan berat tanah yang berada di atas bagian dasar dinding. Pilihan material untuk dinding penahan tanah adalah: beton, bata, atau batu kali. Gabions adalah salah satu jenis dinding penahan tanah tipe gravity yang umum digunakan pada sungai dan pantai.

Sheet pile disisipkan ke dalam tanah dan mengandalkan kekuatan jepitan tanah di depan dinding. Stabilitas dinding ini dapat ditambahkan dengan membuat system penunjang seperti angkur, props dan penggunaan bored piles.

Anchored wall mengandakan kekuatan angkur yang disisipkan secara horizontal maupun membentuk sudut dengan bidang horizontal.

Reinforced soil walls (dinding penahan tanah yang diperkuat) mengandalkan berat tanah dan ikatan yang terjadi antara tanah yang diperkuat dengan bahan perkuatan. Reinforced soil wall biasanya relatif ringan dan lentur serta mempunyai toleransi yang besar terhadap pergerakan; dengan demikian ideal untuk dibangun di atas tanah lunak atau yang mempunyai daya dukung rendah. Bahan perkuatan biasanya dibuat dari bahan logam dan geosintetik. Pengerjaan konstruski in cepat dan keperluan akan material cukup rendah dibandingkan dengan dinding tipe lainnya. Dinding jenis ini cocok untuk area dengan akses terbatas tapi tidak boleh dipilih bila daerah tersebut banyak memiliki penghalang dalam pemasangan bahan perkuatan.

Hal pertama yang dapat dipertimbangkan dalam memilih jenis dinding penahan tanah adalah tinggi tanah yang harus ditahan, topography, kondisi tanah serta kondisi proyek itu sendiri. Faktor lainnya adalah ketersediaan material, ketersediaan alat, ruangan yang tersedia, akses, umur rencana, persyaratan perawatan konstruksi, faktor lingkungan, penampilan dan tentu saja biaya.

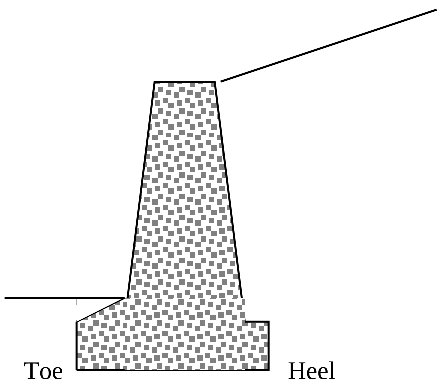
Setelah jenis dinding penahan tanah dipilih, maka langkah berikutnya adalah memperkirakan besarnya dan distribusi beban yang bekerja pada dinding tersebut. Dimensi dinding ditentukan berdasarkan evaluasi stabilitas internal dan eksternal.

GRAVITY WALLS (DINDING TIPE GRAVITI)

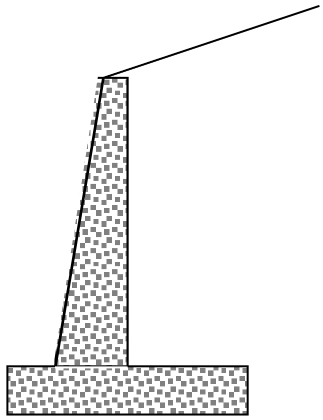
Dinding penahan tanah tipe ini biasanya berada di atas permukaan tanah (free standing). Stabilitasnya tergantung pada berat sendiri atau kombinasi berat sendiri dengan berat tanah diatas bagian dasar dinding. Gambar 1 memperlihatkan jenis jenis dinding penahan tanah tipe graviti. Dinding graviti atau semi graviti hanya sesuai untuk perbedaan tinggi sampai dengan 3 m, sedang dinding cantilever or counter fort wall dapat menahan galian atau timbunan sampat ketinggian 8 m.

Tekanan tanah aktif umumnya digunakan untuk disain dinding tipe ini karena kemungkinan pergerakan di bagian bawah pondasi (*sliding*). Langkah selanjutnya adalah mendisai ukuran dinding penahan tanah berdasarkan tinggi tanah yang harus ditahan (*H*). Sebagai estimasi awal ukuran dasar pondasi (*B*) adalah 2/3 of the height of the wall.

Analisis stabilitas lereng termasuk kestabilan terhadap guling, geser dan daya dukung tanah pondasi. Bila salah satu kondisi ini tidak terpenuhi maka ukuran dinding harus di modifikasi dan stabilitasnya dihitung lagi.



(a) gravity wall



(b) cantilever type wall

Gambar 1 Dinding penahan tanah tipe gravitasi

Untuk mengevaluasi stabilitas dinding terhadap guling, kita harus menentukan titik O pada bagian depan dasar dinding (toe). Kemudian menhitung momen guling (M_O) dan momen penahan (M_{Ro}) pada titik O. Faktor keamanan terhadap guling adalah:

$$FS = \frac{M_{RO}}{M_O} \geq 2.5 \qquad (3.33)$$

Pergeseran dinding mungkon terjadi karena dinding berada di atas muka tanah. Pergeseran terjadi akibat gaya dorong tanah dari bagian belakang dinding. Gaya ini ditahan oleh berat dinding itu sendiri dan gesekan pada dasar pondasi. Faktor keamanan terhadap geser adalah :

$$FS = \frac{\sum V \tan \delta}{\sum H} \geq 2.0 \qquad (3.34)$$

dimana δ adalah sudut geser antara pondasi dengan tanah. Tekanan (P)beban vertical dan momen yang bekerja pada *the center of gravity* dinding tersebut (titik C). Ini dihitung sebagai:

$$P = \frac{V}{B} \pm \frac{M_C\, y}{I} \qquad (3.35)$$

dimana $M_C = V\,e$, dalam hal ini e adalah eksentrisitas, I adalah momen inersia bidang dasar pondasi, y adalah eksentrisitas maximum = $\frac{B}{2}$. Dalam hal analisis 2 dimensi, pondasi dinding adalah pondasi menerus dengan lebar B dan panjang 1. Dengan menggunakan momen inersia persegi panjang $I = \frac{1}{12} l B^3$ and $y = \frac{B}{2}$, persamaan 3.35 dapat dituliskan sebagai berikut:

$$P = \frac{V}{B}\left(1 \pm \frac{6e}{B}\right) \tag{3.36}$$

Tekanan maximum (P_{max}) harus lebih kecil dari daya dukung tanah, sedangkan tekanan minimum (P_{min}) harus positif (> 0) karena tanah tidak dapat menahan tarik.

Dinding penahan tanah tipe graviti maupun kantilever tidak di disain untu menahan beban air. Dengan demikian harus dibuat drainase yang cukup untuk mengalirkan air secepat mungkin dari belakang dinding.

Example Problem 3.8

A gravity wall is designed to retain a cut along a highway project as shown in Figure P3.8. The wall is made of unreinforced concrete with γ_c = 22 kN/m^3. The backfill soil is free draining medium sand with ϕ' = 34^o, wall friction angle δ = 20^o, and unit weight 19.2 kN/m^3. The wall is resting on stiff clay deposit with bearing capacity of 550 kN/m^2. Evaluate the stability of the wall against overturning, sliding and bearing capacity failure.

Solution

The back of the wall is not smooth and δ = 20^o, then use Coulomb method
In this case θ = 90^o, β = 0.

$$K_a = \frac{\sin^2(90+34)}{\sin^2 90 \sin(90-20)\left[1+\sqrt{\frac{\sin(34+20)\sin(34-0)}{\sin(90-20)\sin(90+0)}}\right]} = 0.3767$$

To check the stability of the wall against overturning, sliding on the base, and bearing capacity failure, we can prepare Table P 3.8 by referring to Figure P3.8.

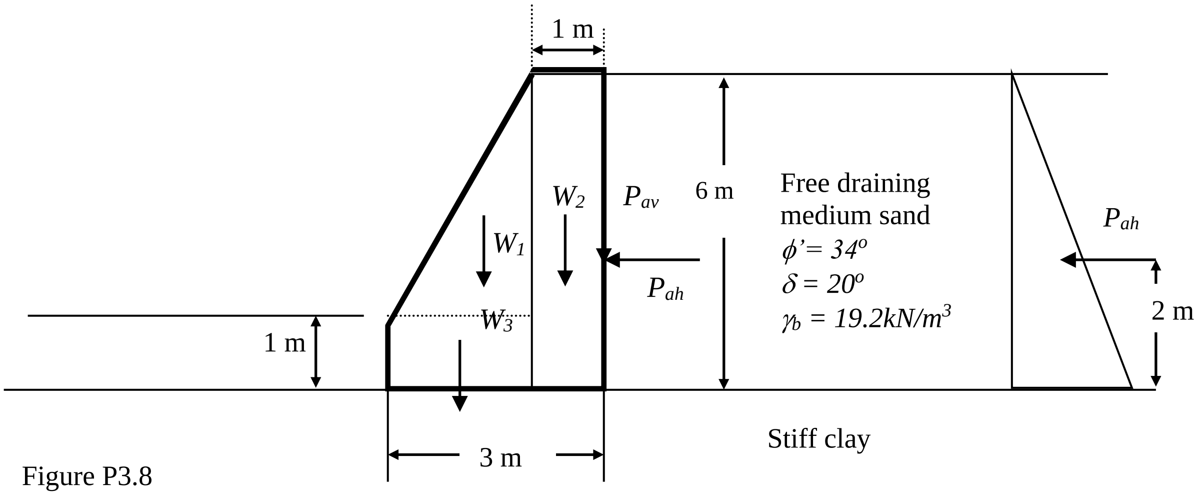


Figure P3.8

No	Force	Moment Arms (about O)	M_O
1	½×5×2×22 = 110	4/3	+147
2	1×6×22= 132	2.5	+330
3	2×1×22 = 44	1	+44
4	½ ×19.2× $(6)^2$ ×0.3767×cos20°= 122.30	2	-244.60
5	½ × 19.2 × 6^2 × 0.3767 × sin20°= 44.5	3	+133.50

Stability against overturning

$$FS = \frac{M_{RO}}{M_o} = \frac{147 + 330 + 44 + 133.5}{244.6} = 2.68 > 2.50$$

Stability against sliding on the base

$$FS = \frac{V \tan\delta}{H} = \frac{(110 + 132 + 44 + 44.5)\tan 20^o}{122.3} = 0.98 < 2$$

Bearing capacity failure

$$e = \frac{M_O}{V} - \frac{B}{2} = \frac{147 + 330 + 44 + 133.5 - 244.6}{(110 + 132 + 44 + 44.5).} - \frac{3}{2} = 1.24 - 1.50 = 0.26 \text{ m}$$

$B/6 = 3/6 = 1.5$ m → $e < B/6$

Check P_{max} and P_{min}

$$P = \frac{V}{B}\left(1 \pm \frac{6e}{B}\right) = \frac{330.5}{3}\left(1 \pm \frac{6 \times 0.26}{3}\right) = 110.17 \pm 57.29$$

$P_{max} = 167.5 < Q_{all}$ and $P_{min} = 53.1 > 0$

The wall is safe against overturning and bearing capacity failure but unsafe against sliding. We may need to provide heel to increase the passive resistance against sliding.

Example Problem 3.9

A gravity wall as shown in Figure P3.9 is retaining a backfill with unit weight of 18 kN/m^3 and shear strength parameters $c' = 0$, and $\phi' = 38^o$. The friction between the soil and wall is represented by an angle $\delta = 25^o$. Evaluate the stability of the wall against overturning, sliding and bearing capacity failure if the unit weight of wall material is 23.5 kN/m^3

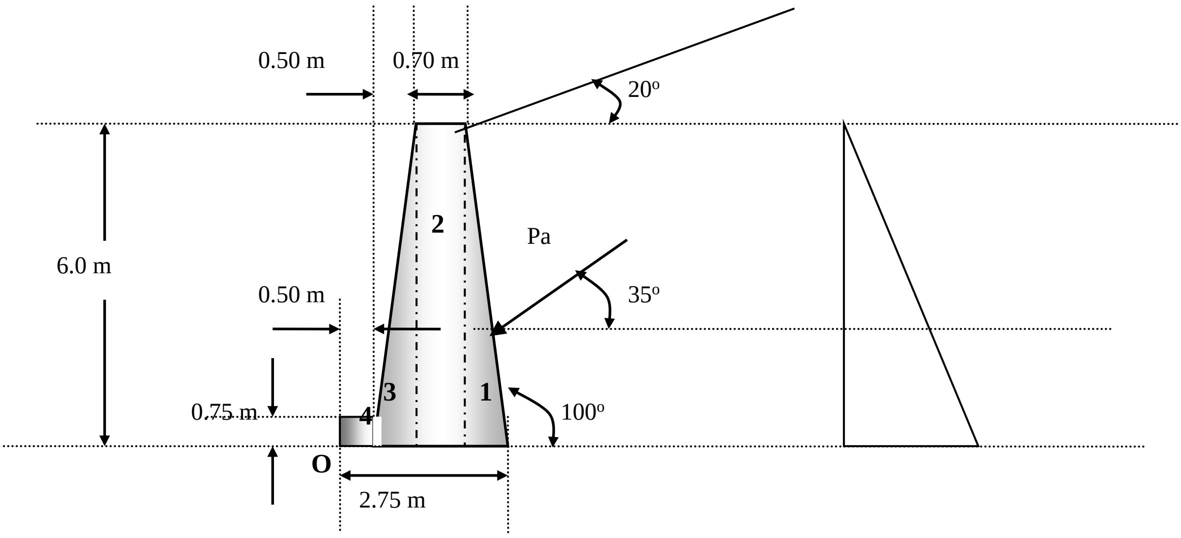


Figure P3.9

Solution

For $\phi' = 38^o$, $\delta = 25^o$, $\beta = 20^o$ and $\theta = 100^o$

$$K_a = \frac{\sin^2(100+38)}{\sin^2 100 \sin(100-25)\left[1+\sqrt{\dfrac{\sin(38+25)\sin(38-0)}{\sin(100-25)\sin(100+0)}}\right]} = 0.39$$

P_a = ½ × 0.39 × 18 × (6)^2 = 126 kN/m

acts at $^1/_3$ height and forming an angle of 25 + (100– 90) = 35^o to horizontal.

Calculation of force and pressure on wall

No	Force	Moment Arms (about O)	Momen
W_1	½ ×1.05 × 6 × 23.5 = 74	2.05	+151.7
W_2	0.7 ×6 × 23.5 = 98.7	1.35	+133.25
W_3	½ × 0.5 × 5.25 × 23.5 = 30.8	0.833	+25.66
W_4	1 ×0.75 ×23.5 = 17.6	0.50	+8.81
P_h	126 cos 35^o = 103.2	2	-206.40
P_v	126 sin 35^o = 72.3	2.40	+173.52
	ΣV = 293.4 ; ΣH = 103.2	ΣM_O = 206.40 ; ΣM_{RO} = 484.13	

Stability against overturning

$$FS = \frac{M_{Ro}}{M_o} = \frac{484.13}{206.4} = 2.34$$

Stability against sliding on the base

$$FS = \frac{V \tan\delta}{H} = \frac{293.4 \tan 25^o}{103.2} = 1.33 < 2.0$$

Bearing capacity failure

$$e = \frac{M}{V} - \frac{B}{2} = \frac{484.13 - 206.4}{293.4} - \frac{2.75}{2} = 0.43\,\text{m}$$

$B/6$ = 2.75/6 = 0.458 m → $e < B/6$

$$P = \frac{V}{B}\left(1 \pm \frac{6e}{B}\right) = \frac{293.4}{2.75}\left(1 \pm \frac{6.\times 0.43}{2.75}\right) = 106.7\,(1 \pm 0.06)$$

P_{max} = 113.1 kN/m^2 < σ_s P_{min} = 100.3 kN/m^2 > 0

Example Problem 3.10

A cantilever type wall as shown in Figure P3.10 is required to retain an excavation. The shear strength parameters for the soil are $c' = 0$, $\phi' = 40^o$, $\gamma = 17$ kN/m^3 . Water table is located far below the base of the wall. The friction angle at the base of wall is 30^o. Evaluate the stability of the wall against overturning, sliding and bearing capacity failure. (Take the unit weight of concrete $\gamma_c = 23.5$ kN/m^3 and use Rankine theory to calculate lateral stress)

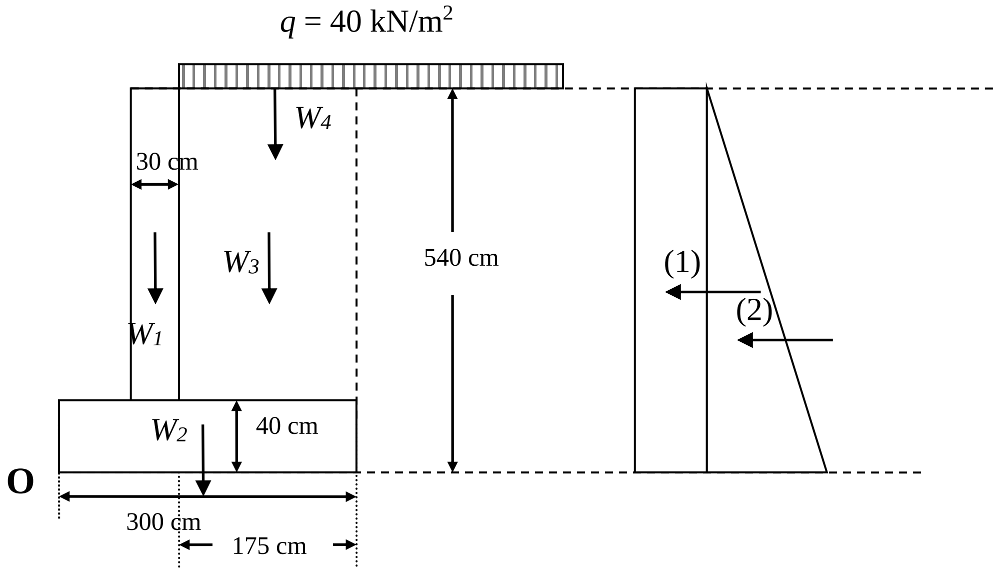


Figure P3.10

Solution:

For $\phi' = 40^o$; $\delta = 0$ → $K_a = \tan^2 (45 - \frac{20}{2}) = 0.22$

Calculation of force and pressure on wall is tabulated in table P3.10

Table P 3.10

No	Force	Arms (about O)	Moment
W_1	5 × 0.3 × 23.5 = 35.3	1.10	+38.83
W_2	3 × 0.4 × 23.5 = 28.2	1.50	+42.30

W_3	5 × 1.75 ×17 = 148.8	2.125	+316.20
W_4	1.75 × 40 = 70.0	2.125	+148.75
P_1	0.22 × 40 × 5.40 = 47.5	2.70	-128.20
P_2	½ × 0.22 × 17 ×5.40^2 = 54.6	1.80	-98.30

Stability against overturning

$$FS = \frac{MRo}{Mo} = \frac{38.83 + 42.30 + 316.20 + 148.75}{128.20 + 98.30.} = 2.40$$

Stability against sliding on the base

$$FS = \frac{V \tan \delta}{H} = \frac{(35.3 + 28.2 + 148.8 + 70) \tan 30^o}{47.5 + 54.6} = 1.60$$

Bearing capacity failure

$$e = \frac{M}{V} - \frac{B}{2} = \frac{38.83 + 42.30 + 316.2 + 148.75 - 128.20 - 98.30}{(35.3 + 28.2 + 148.8 + 70)} - \frac{3}{2} = 1.13 - 1.50 = 0.37\ \text{m}$$

$B/6 = 3/6 = 0.5$ m → $e < B/6$

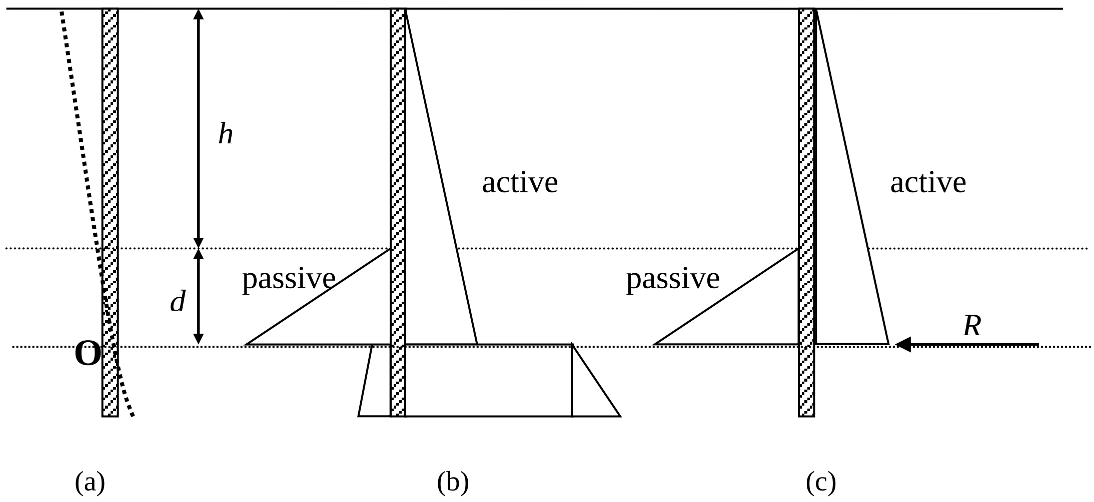
$$P = \frac{V}{B}\left(1 \pm \frac{6e}{B}\right) = \frac{282.3}{3}\left(1 \pm \frac{6.0.37}{3}\right) = 94.1\ (1 \pm 0.74)$$

$P_{max} = 163.7$ kN/m$^2 < \sigma_s$ $P_{min} = 24.5$ kN/m$^2 > 0$

SHEET-PILE WALL (DINDING SHEET PILE)

Dinding sheet pile direkomendasikan untuk menahan tanah dengan ketinggian 5 sampai 12 m. Defleksi dinding harus di awasi terutama pada bagian atas dinding. Dinding sheet pile berperan sebagai balok kantilever yang lebar di atas bidang jepit. Tujuan analisis stabilitas dinding sheet pile adalah untuk menentukan kedalaman pemasangan sheet pile dibawah garis pengerukan (*dredge line*). Konsep pengiraan tekanan tanah lateral pada dinding sheet pile dijelaskan pada Gambar 2.

Mode keruntuhan dengan rotasi pada titik O diasumsikan dekat dengan bagian bawah dinding (Gambar 2a). Tekanan tanah aktif yang bekerja di belakang dan tekanan tanah pasif yang bekerja di depan dinding diperlihatkan pada gambar 2b. Hanya diagram bagian atas (Gambar 2c) yang diperhitungkan untuk analisis. Faktor keamanan digunakan untuk pertimbangan tekanan pasif dan kedalaman penanaman sheet pile.



Gambar 2 Cantilever sheet pile wall

Dalam analisis sheet pile, biasanya dianggap bahwa muka air di belakang dan di depan sheet pile ada pada level yang sama sehingga gaya air di belakang dan di dapan dinding saling meniadakan.

Example Problem 3.11

A cantilever sheet pile wall is required to support 5 m depth of soil with strength parameters $c' = 0$, $\phi' = 35^o$, and $\gamma = 20.8$ kN/m^3. Determine the length of pile required for a safety factor of 1.5 applied to passive pressure. Use horizontal equilibrium only.

Solution

The problem is shown in Figure P3.11

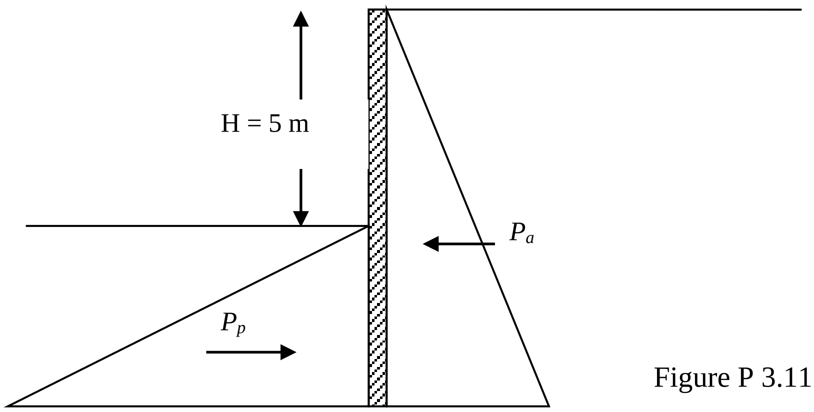


Figure P 3.11

Assume that the back of sheet pile is smooth, then use Rankine's formula to calculate K_a and K_p

$$K_a = \tan^2(45 - \frac{\phi}{2}) = 0.271$$

$$K_p = \tan^2(45 + \frac{\phi}{2}) = 3.690$$

Active force: $P_a = \frac{1}{2}\ \gamma\ (H + d)^2\ K_a = \frac{1}{2} \times 20.8 \times (H + d)^2 \times 0.271 =$
$P_a = 2.818\ (5 + d)^2$

Passive force $P_p = \frac{1}{2}\ \gamma\ z^2\ K_a = \frac{1}{2} \times 20.8 \times z^2 \times 3.69 =$
$P_p = 38.38\ z^2$

Horizontal equilibrium is reached when $P_a = P_p$

Apply factor of safety 1.5 with respect to passive resistance

$$P_a = 1.5\ P_p$$

$$2.818\ (5 + d)^2 = 1.5 \times 38.38\ d^2$$

Get $d = 3.85$ m.

Take the length of embedment = 4 m

Example Problem 3.12

The sides of an excavation 3.00 m in sand are to be supported by a cantilever sheet pile wall. The water table is 1.5 m below the bottom of the excavation. The sand has a bulk unit weight of 17 kN/m^3, and saturated unit weight of 20 kN/m^3. The effective friction angle of the sand is 36^o. Determine the depth of penetration of the piling below excavation to give a factor of safety of 2.0 with respect to passive resistance.

Solution:

For $\phi' = 36^o$ $\quad K_a = \tan^2(45 - \frac{\phi}{2}) = 0.26,$

and $\quad K_p = \tan^2(45 + \frac{\phi}{2}) = 3.85;$ $\quad$ with FS = 2, $K_p = 1.925$

$\gamma' = 20 - 9.8 = 10.2$ kN/m^3

The pressure distribution is shown in Figure P3.11. Hydrostatic pressure on the two sides of wall balances.

Consider moments about X (per m), assuming $d > 0$.

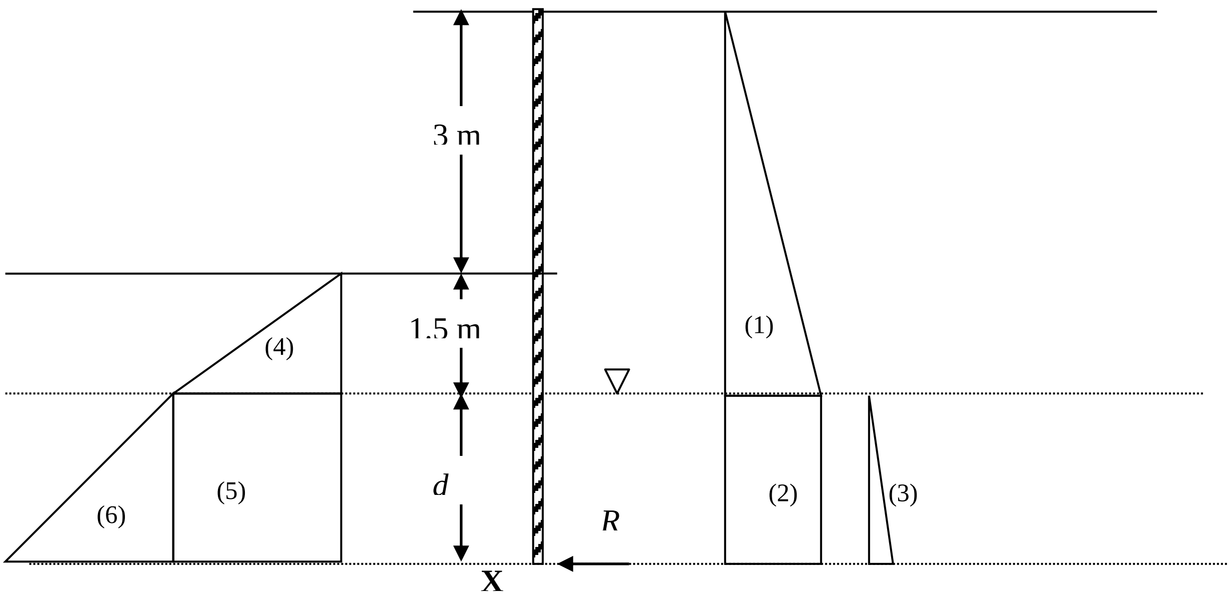


Figure P3.11

To simplify, develop a table for calculation of depth of embedment based on the free-body of the pressure diagram shown in Figure 3.11:

No	Forces	Arms (m)	Moment at X (kN m)
1.	$\frac{1}{2} \times 0.26 \times 17 \times 4.5^2 = 44.8$	1.5+d	67.2 + 44.8d
2	$0.26 \times 17 \times 4.5 \times d = 19.9\ d$	d/2	$9.95\ d^2$
3	$\frac{1}{2} \times 0.26 \times 10.2 \times d^2 = 1.33\ d^2$	d/3	$0.44\ d^3$
4	$- \frac{1}{2} \times \frac{1}{2} \times 3.85 \times 17 \times 1.5^2 = -36.8$	d+0.5	-36.8 d – 18.4
5	$- \frac{1}{2} \times 3.85 \times 17 \times 1.5 \times d = -49.1\ d$	d/2	$-24.55\ d^3$
6	$-\frac{1}{2} \times \frac{1}{2} \times 3.85 \times 10.2 \times d \times d = -9.82\ d^2$	d/3	$-3.27\ d^3$

Taking moment at X equal to 0, we get the equation

$$-2.83\ d^3 - 14.6\ d^2 + 8\ d + 48.8 = 0$$

By trial and error, we get d = 1.79 m
and applying FS = 1.2, depth of penetration = 1.2 x (1.79+1.50) = 3.95m

Substituting d = 1.79 m to the horizontal force equilibrium
$44.8 + 19.9\ d + 1.33\ d^2 – 36.8 – 49.1\ d – 9.82\ d^2 = R$
Get force resultant R = 71.5 kN

Over additional depth of 20% (d = 3.95 – 1.5 = 2.45 m)

$$P_p – P_a = (K_p\ \gamma\ 4.5) + (K_a\ \gamma\ 1.5) + (K_p\text{-}K_a)\gamma(2.45)$$

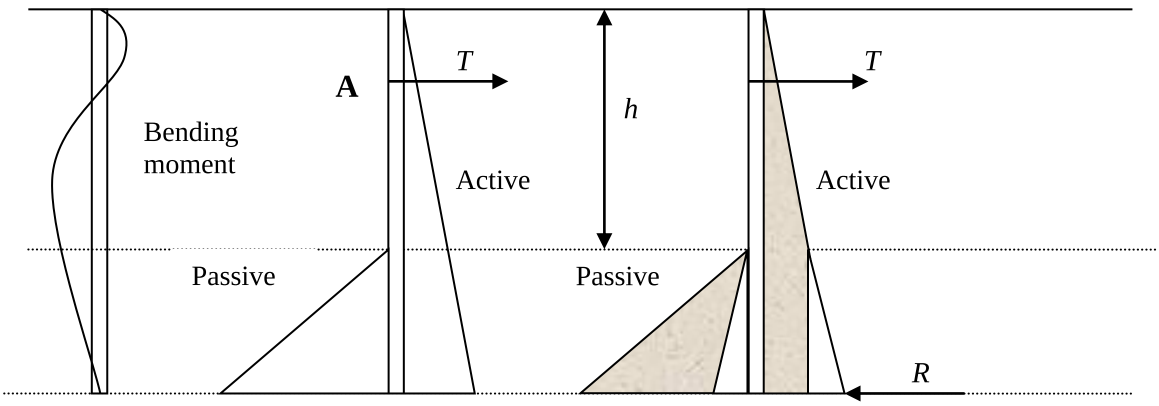
$$= (3.85 \times 17 \times 4.5) + (0.26 \times 17 \times 1.5) + (3.85–0.26)\ 10.2 \times 2.45$$

$$= 365.5 \text{ kN } (>R)$$

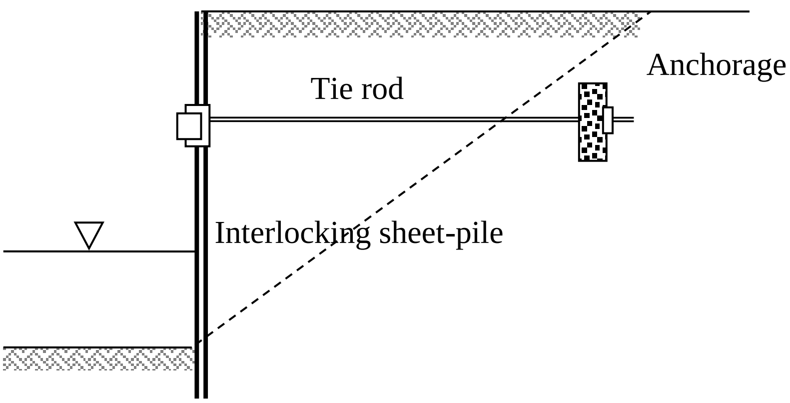
ANCHORED SHEET PILE WALL

Bila tinggi tanah yang harus di tahan cukup besar maka perlu di pasang tie back yang dihubungkan dengan angkur untuk mengurangi momen yang bekerja pada dinding. Angkur ditempatkan pada suatu jarak yang cukup dari dinding penahan tanah (Gambar 3). Dinding tipe ini biasa digunakan untuk konstruksi waterfront dan galian dalam.

Analisis stabilitas sheet pile yang di angkur dilakukan dengan asusmsi bahwa dinding berputar pada posisi sambungan angkur dengan dinding (titik A pada Gambar 3). Gaya per satuan panjang yang ditanah oleh tali angkur (T) adalah selisih gaya horizontal yang bekerja pada dinding.



Gambar 3.14 Dinding sheet pile yang di angkur (free earth support methods)



Gambar 4. Dinding sheet pile dengan angkur

Bila *T* adalah gaya pada tali angkur per satuan panjang dinding, *s* adalah jarak, and *FS* adalah faktor keamanan, maka berdasarkan tekanan aktif dan pasif, gaya yang bekerja pada tali angkur adalah:

$$T = \frac{\gamma d_a^2 l}{2 FS\ s}\left(K_p - K_a\right) \qquad (3.37)$$

dan kedalaman angkur adalah

$$d_a^2 = \frac{2 FS\ s T}{\gamma L\left(K_p - K_a\right)} \qquad (3.38)$$

bila s = *1*, maka

$$d_a = \sqrt{\frac{2 FS\ \ T}{\gamma\left(K_p - K_a\right)}} \qquad (3.39)$$

Example Problem 3.12

A sheet-pile wall is used to retain a cut to a depth of 6 m. An anchor was placed at depth of 1.5 m below the ground surface (see figure P3.12). Use the free earth support method to calculate (a) the depth of embedment of the sheet pile for a factor of safety with respect to passive pressure of 2, (b) the force on the anchor if the distance between two anchors is 1 m.

Solution

Since the wall is smooth, then we use Rankine's formula for calculation of earth pressure coefficient

For $\phi = 30^o$ $\quad K_a = \tan^2(45 - \frac{\phi}{2}) = 0.3333$

$K_p = \tan^2(45 + \frac{\phi}{2}) = 3$

For $\phi = 35^o$ $\quad K_a = \tan^2(45 - \frac{\phi}{2}) = 0.271$

$K_p = \tan^2(45 + \frac{\phi}{2}) = 3.69$

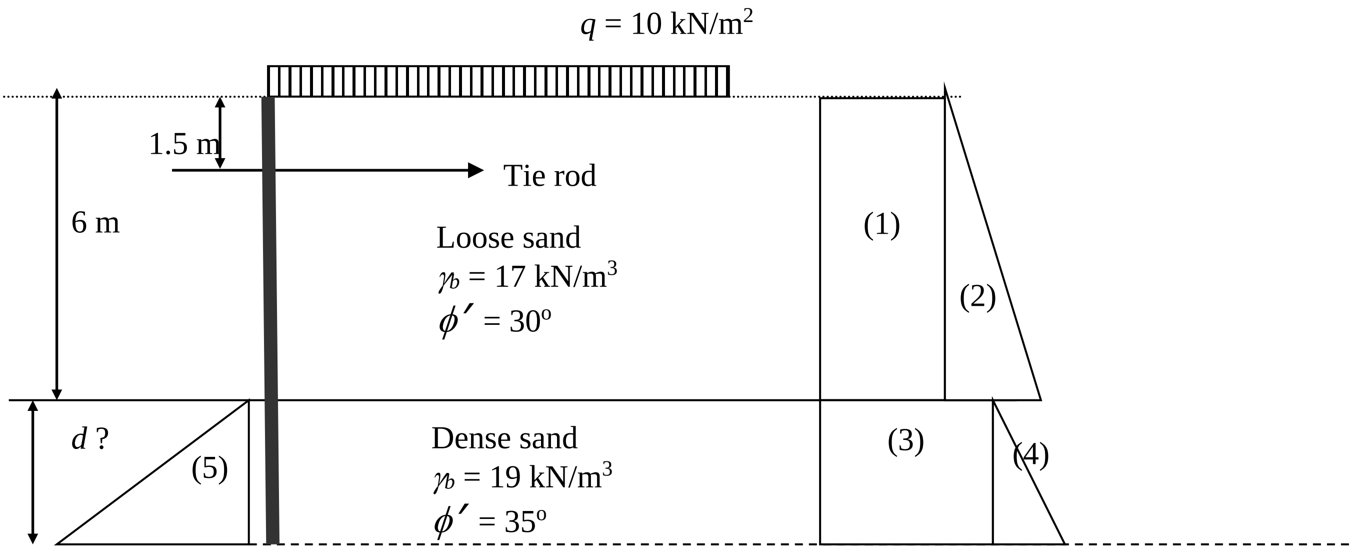


Figure P 3.12

Produce table to calculate depth of embedment *d*

	Force per m (kN)	Arms (m)	Moment (A)
(1)	10×6×0.33 = 19.8	1.5	+29.7
(2)	½ × 17 × 6^2×0.33 = 101	2.5	+252.5

(3)	(10 +17× 6) d ×0.271 = 30.35 d	4.5 + d/2	+136.575 d + 15.175
(4)	½×19 × d^2 × 0.271 = 2.574 d^2	4.5 + 2d/3	+11.583 d^2 + 1.716 d^3
(5)	–(½ × ½ × 19 × d^2×3.69) = – 17.524 d^2	4.5 + 2d/3	– 78.86 d^2 – 11.68 d^3
Tie	– T	0	0

From the table, taking $\Sigma M_A = 0$,

$$-9.96\, d^3 - 67.28\, d^2 + 136.575\, d + 297.38 = 0$$

By trial and error, get d =3 m → take d = 3 m.

Subsituting d into the force equilibrium

$$\sum \text{Force} = 0$$

$$19.8 + 101 + 30.35d + 2.574\, d^2 - 17.524\, d^2 - T = 0$$

$$T = 77.3 \text{ kN/m'}$$

The force in each tie = 1 m × 77.3 = 77.3 kN

For a continuous anchor and factor of safety = 1.5,

the required depth of $d_a = \sqrt{\dfrac{2\,FS\ \ T}{\gamma\left(K_p - K_a\right)}} = \sqrt{\dfrac{2\times1.5\times77.3}{17\,(3.69-0.271)}}$ = 2 m

The anchor is centered 1.5 m below the surface

Then width of the anchorage b = 2 × (2– 1.5) = 1 m

Example Problem 3.13

The soil in both sides of the anchored sheet pile wall detailed in Figure P3.13 has a bulk unit weight of 18 kN/m^3 and saturated unit weight of 21 kN/m^3. The shear strength parameters of the soil are c'= 0, and ϕ' = 36°. (a) Determine the factor of safety of the wall with respect to gross passive resistance and (b) calculate the force in each tie rod.

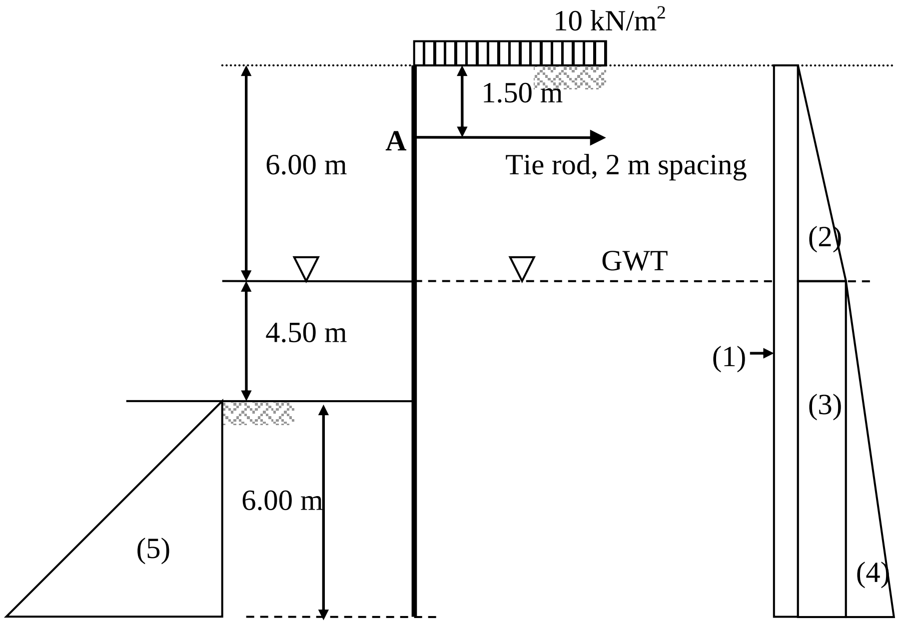


Figure P3.13

Solution

Use Rankine's formula for calculation of earth pressure coefficient

For $\phi' = 36^o$ $\quad K_a = \tan^2(45 - \frac{\phi}{2}) = 0.26$

$$K_p = \tan^2(45 + \frac{\phi}{2}) = 3.85$$

$$\gamma' = \gamma_{sat} = 21 - 9.8 = 11.2 \text{ kN/m}^3$$

Pressure due to water can be ignored because water table at both sides of the wall is at the same level.

Consider moment about anchor point A and tabulate the calculation

	Force per m (kN)	**Arms (m)**	**Moment (A)**
(1)	10 x 0.26 x 15 = 39	6.0	+234
(2)	½ x 18 x 6^2 x 0.26 = 84.24	2.5	+210.6
(3)	18 x 6 x 10.5 x 0.26 = 294.84	9.75	+2874.69
(4)	½ x 11.2 x 10.5^2 x 0.26 = 160.52	11.5	+1846
(5)	-½ x 11.2 x 6^2 x 3.85 = -776	13	-10090
Tie	-T	0	0
	Σ F = 578.6-776		Σ M = 5165.3-10090

Available Factor of safety for gross passive resistance ($\Sigma M_A = 0$)

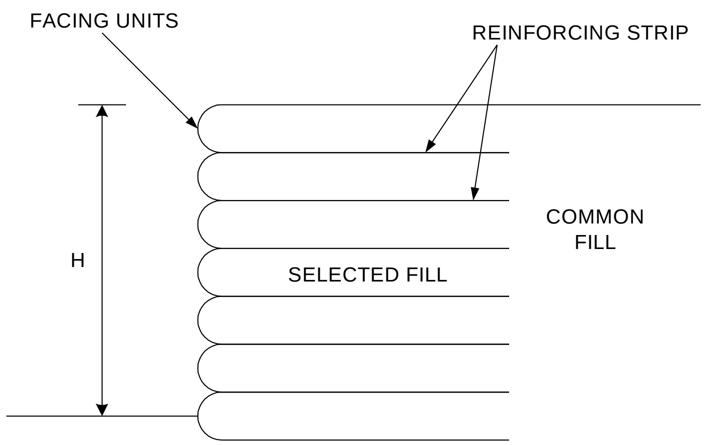
$$FS = \frac{P_p}{P_{pm}} = \frac{776}{(5165.3/13)} = \frac{776}{397} = 1.95$$

Force in each tie rod

$$2\,T = 2\,(578.6 - 776) = 395 \text{ kN (distance 2 m)}$$

REINFORCED SOIL WALL
(DINDING PENAHAN TANAH YANG DIPERKUAT)

Reinforced soil wall, pertama kali diperkenalkan oleh Vidal, merupakan alternative terhadap dinding penahan tanah konvensional. Konstruksi ini trediri dari tanah yang diapadatkan, bahan perkuatan biasanya terbuat dari logam atau geosintetik, dan bahan penutup bagian depan dinding (facing unit). Konstruksi ini bersifat lentur (flexible structure) sehingga mempunyai toleransi terhadap deformasi. Konstruksi ini memerlukan relatif lebih sedikit bahan dan peralatan. Dengan demikian sangat berguna untuk lokasi dengan tanah yang tudak cikup kuat, atau lokasi yang memiliki keterbatasan ruang dan akses. Bentuk dinding penahan tanah yang diperkuat diperlihatkan pada Gambar 5.



Gambar 5 Bagian bagian dinding penahan tanah yang diperkuat

Analisis stabilitas dinding yang diperkuat dilakukan dalam dua bagian yaitu : stabilitas secara keseluruhan (*external stability*) dan stabilitas dalam (*internal stability*).

Analisis stabilitas secara keseluruhan (*overall*) dilakukan dengan menganggap bahwa dinding bergerak sebagai satu kesatuan seperti gravity wall. Stabilitas didapat dari berat tanah yang diperkuat. Evaluasi stabilitas meliputu kestabilan terhadap geser dibawah pondasi, guling, dan daya dukung tanah pondasi, srta stabilitas lereng.

Stabilitas internal dihasilkan dari gesekan antara tanah dan bahan perkuatan. Analisis meliputi kekuatan bahan perkuatan dan gesekan antara tanah dengan bahan perkuatan untuk menahan tarikan yang diakibatkan pergerakan dinding kearah depan; untuk menahan tarikan Analisis ini memerlukan asumsi bidang keruntuhan, sebaran tekanan tanah lateral, dan rumus empiris untuk gesekan. Metode analisis telah dikembangkan oleh FHWA untuk analisis stabilitas internal.

Example Problem 3.14

A 10 m high retaining wall reinforced with galvanized steel strips in a granular backfill is to be constructed. The unit weight of the granular backfill is 16.5 kN/m^3, while its internal friction angle is 36^o. The shear strength parameters of the foundation soil are c' kPa, and ϕ' = 28^o. The width of galvanized strip is 7 cm while its thickness is 5 mm, and the vertical and horizontal center to center distance between the strips (S_V = S_H) are equal to 1 m. The tensile strength of the steel (f_y) is 250 MPa.   (a) Check the external stability of the wall assuming the bearing capacity of the foundation soil is 300 kPa and the base friction angle is 24^o, (b) Evaluate the internal stability of the wall if the lifespan of the structure is 50 years and the corrosion rate is 0.02 mm/year.

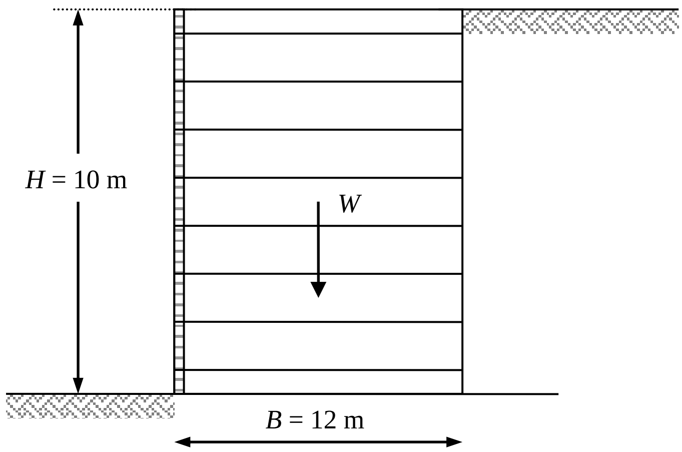


Figure P3.14

Solution

External stability analysis (for B = 12 m) assuming active condition prevails along the height of wall (flexibel reinforcement)

$$K_a = \tan^2(45 - \frac{\phi}{2}) = 0.26$$

$$P_a = ½\ \gamma_1\ K_a\ H^2 = ½ \times 16.5 \times 0.26 \times (10)^2 = 214.5 \text{ kN/m}^2$$

$$W = \gamma B\,H = 16.5\times12\times10 = 1980 \text{ kN/m}'$$

Check for overturning

$$M_{RO} = \tfrac{1}{2}\,W\;\;B = \tfrac{1}{2}\times 1980\times12 = 11880 \text{ kN-m/m}'$$

$$M_O = P_a\,H/3 = 214.5\times 10/3 = 715 \text{ kN-m/m}'$$

$$FS_{\text{overturning}} = \frac{M_{RO}}{M_O} = \frac{11880}{715} = 16.6 > 3 \rightarrow \text{OK}$$

Check for sliding along the base
Assume the friction angle at the base is 2/3 $\phi = 24^o$

$$FS_{\text{sliding}} = \frac{W\tan\delta_b}{P_a} = \frac{1980\tan 24^o}{214.5} = 4.1 > 3 \rightarrow \text{OK}$$

Check for bearing capacity

$$e = \frac{B}{2} - \frac{M_{RO} - M_O}{V} = \frac{12}{2} - \frac{11880 - 715}{1980} = 0.36$$

$$e < \frac{B}{6} \qquad\qquad 0.36 < 2 \text{ m.} \rightarrow\text{OK}$$

Assume ultimate bearing capacity is 300 kPa

$$P = \frac{V}{B}\left(1 \pm \frac{6e}{B}\right) = \frac{1980}{12}\left(1 \pm \frac{6\times0.36}{12}\right) = 81.67\ (1\pm0.18)$$

P_{max} = 96.37 kPa < BC soil $\qquad\qquad$ P_{min} = 67 kPa > 0 → OK

Internal stability check

The check should be done for each reinforcement layer. To simplify the calculation we will have to develop a table (Table P 3.14)

Table P3.14

No	***z* (m)**	***K***	***f****	**L_e (m)**	***T* (kN)**	***R* (kN)** $2bL_e\gamma z f^*$	T_R $b\,t\,f_y$	***FS* tensile**	***FS* pullout**
1	0.5	0.3975	1.405	9	3.2794	3.7422	70.0	18.70	1.1411

2	1.5	0.3725	1.215	9	9.2194	11.2266	70.0	7.59	1.2177
3	2.5	0.3475	1.025	9	14.3344	18.711	70.0	4.88	1.3053
4	3.5	0.3225	0.835	9	18.6244	26.1954	70.0	3.75	1.4065
5	4.5	0.2975	0.645	9	22.0894	33.6798	70.0	3.17	1.5247
6	5.5	0.2725	0.455	9.3	24.7294	42.5363	70.0	2.81	1.7200
7	6.5	0.26	0.36	9.9	27.885	53.5135	70.0	2.51	1.9190
8	7.5	0.26	0.36	10.5	32.175	65.4885	70.0	2.18	2.0353
9	8.5	0.26	0.36	11.1	36.465	78.4615	70.0	1.92	2.1516
10	9.5	0.26	0.36	11.7	40.755	92.4323	70.0	1.72	2.2680

Note: For metal strip reinforcement,
K varies from K_o at the surface to K_a at depth of 6m
L_e will follow the coherent gravity methods
f^* varies from 1.5 at the surface to $\tan\delta$ at depth of 6m

$K_o = 1\text{-sin } 36 = 0.41$ $\quad$ $K_a = K_a = \tan^2(45 - \frac{\phi}{2}) = 0.26$

$\tan\delta = \tan 20 = 0.36$
Thickness of metal strip after 50 years = 5 – (0.02 x 50) = 4 mm

From Table 3.14

The minimum factor of safety against tensile breakage = 1.72 (at z = 9.5 m)
The minimum factor of safety against pullout = 1.14 (at z = 0.5m)

PROBLEMS

3.1. A wall is required to retain a cut on a cohesionless soil which has a unit weight of 18.4 kN/m^3 and effective internal friction angle ϕ' of 25^o. The height of the cut is 3 m. If the wall is very rigid and the soil is in “at rest” condition (a) Draw the diagram of lateral pressure behind the wall (b) Calculate the magnitude and point of application of the thrust force on a wall

3.2. An excavation was to be made on a cohesive soil with unit weight γ_b = 19.2 kN/m^3; c' = 10 kPa, and φ' = 10^o. Calculate the depth of unsupported excavation. Plot the active earth pressure diagram if the excavation was to be made up to 9 m deep, and calculate the resulting force and point of application.

3.3. A vertical wall retaining a 3 m high dry sand with unit weight 18 kN/m^3, and internal friction angle ϕ' = 36^o. What is the thrust force working on the wall if uniform surcharge load of 10 kN/m^2 was placed on the surface of the soil behind the wall. Active condition prevails.

3.4. A retaining wall as shown in figure Q3.4 retains soil for a depth of 12 m. The soil mass consists of two layers with soil properties given in the figure. Water table coincide the interface between layer 1 and 2. (a) determine the magnitude and point of application of the active thrust; (b) determine also the hydrostatic pressure working on the back of the wall.

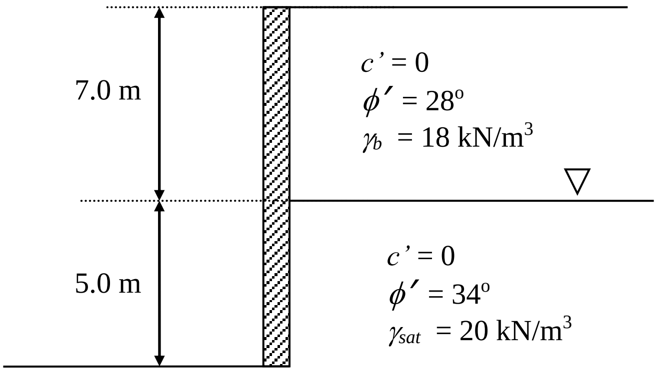


Figure Q3.4

3.5. A concrete wall is designed to retain a cohesionless soil with the following properties: $\gamma = 20.4$ kN/m^3; $\phi' = 35^o$; c' = 0, $\delta = 20^o$. Determine the active thrust force acting on the wall if the height of the wall is 4.6 m and the backfill soil form an angle of 10^o to the horizontal.

3.6. A vertical wall is retaining a 5 m high soil fill with unit weight of 20 kN/m^3, $\phi' = 32^o$, $\delta = 20^o$. A line load P = 75 kN/m is working on the soil surface parallel to the wall. What is (a) the distance between the line load and the back of the wall so the load will not give additional stress to the wall, (b) the thrust force on the wall if the line load is working 2 m behind the wall.

3.7. A gravity wall 8 m high is retaining a soil with horizontal surface. The saturated unit weight of the sand is18.5 kN/m^3 and the internal friction angle $\phi' = 30^o$. The wall friction angle $\delta = 20^o$. Ground water table is at the surface and there is no drainage to dry the soil quickly. Calculate the total force applied to the back of the wall.

3.8. For the retaining wall shown in Figure Q3.8, calculate the factor of safety against overturning, sliding, and bearing capacity failure. Use the Rankine equation to compute the earth pressures.

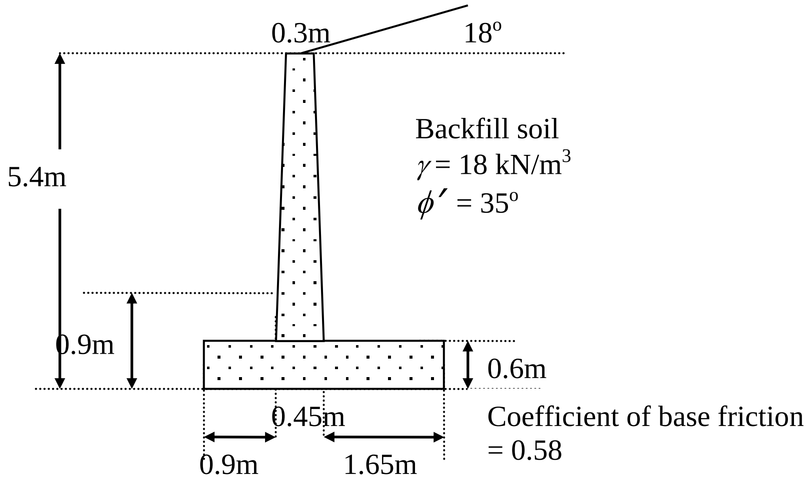


Figure Q3.8

3.9. A retaining wall shown in Figure Q3.10 is to be constructed of concrete with unit weight of 23.5 kN/m^3. The backfill soil has the following properties: γ = 18 kN/m^3, c' = 0, and ϕ' = 30°. The friction between the soil and the concrete wall is given by δ = 20°. The coefficient of base friction is 0.55 and the soil's ultimate bearing capacity is 620 kN/m^2.

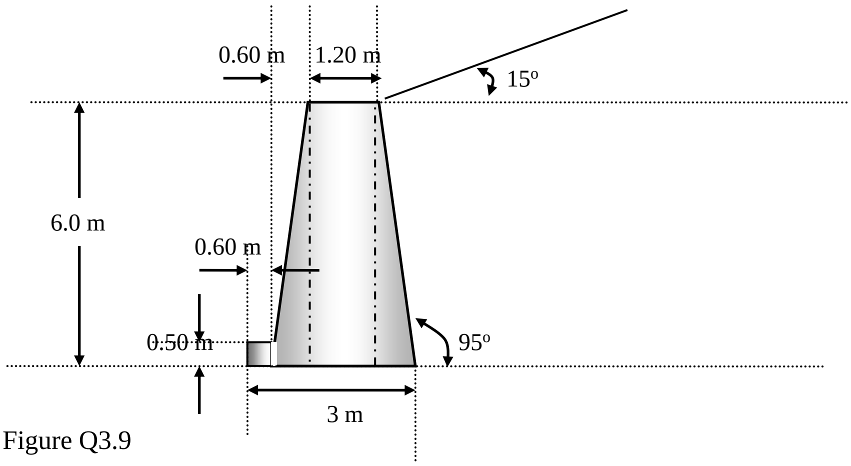


Figure Q3.9

3.10. A sheet-pile wall is used to retain a cut to a depth of 6.5 m. An anchor was placed at depth of 1.65 m below the ground surface (see figure Q3.10). Use the free earth support method to calculate the depth of embedment of the sheet pile for a factor of safety with respect to passive pressure of 2.5. Estimate the force on the anchor.

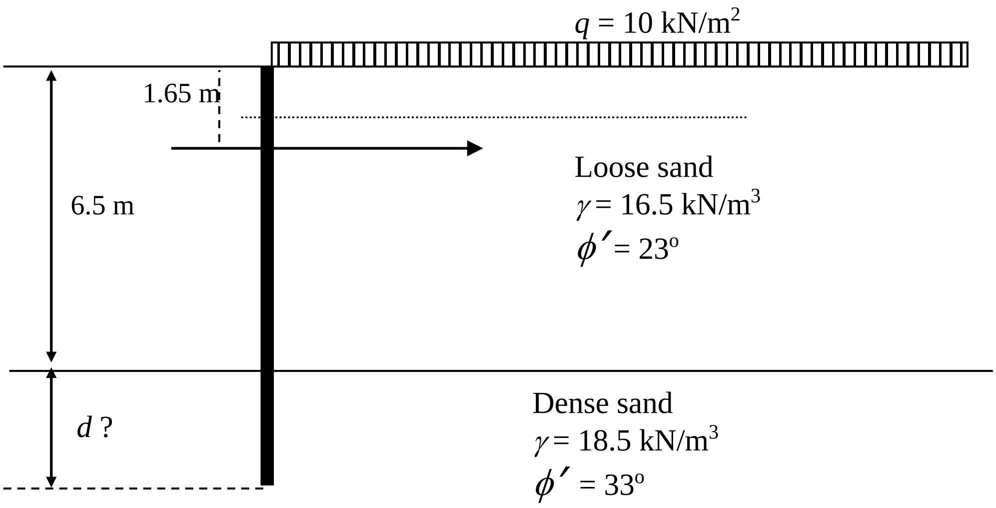


Figure Q3.10

3.11 A 6-m high reinforced earth wall is to be constructed using granular soil with unit weight of 17.2 kN/m^3, and angle of internal friction of 34^o as backfill material. The steel strip of width 75 mm will be used for reinforcement. Following the size of facing unit, the vertical and horizontal spacing of the strips are 0.3 m and 1.0 m respectively. The allowable stress of the steel is 140,000 kN/m^2. Determine (a) the total length of strip required for a factor of safety against pullout 1.5, and (b) the required thickness of strip.

3.12 A quay wall as shown in Figure Q3.12 is to be constructed using anchored sheet piling as shown in figure below. The unit weight of the soil is 17kN/m^3 above ground water table, and the saturated unit weight is 20kN/m^3 below. The shear strength parameters are c'= 0: and φ'= 36^o. For a factor of safety of 2.0 with respect to passive resistance, determine the required depth of embedment and the force in each tie if they are spaced at 2 m. Design a continuous anchor to support the ties.

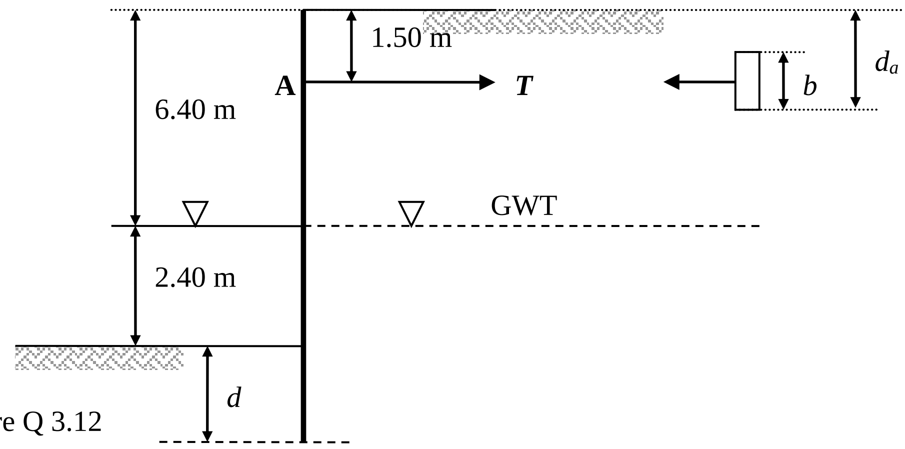


Figure Q 3.12

3.13 A sheet pile wall as shown in Figure Q3.13 is retaining a soil with $\gamma = 18$ kN/m^3, $\gamma_{sat} = 21$ kN/m^3, $c' = 0$, and $\phi' = 33^o$. Using a factor of safety 2.0 for passive resistance, (a) determine the required depth of embedment and the force in the tie rod, and (b) design the size and the length of tie rod if the strength of the steel rod is 140,000 kN/m^2.

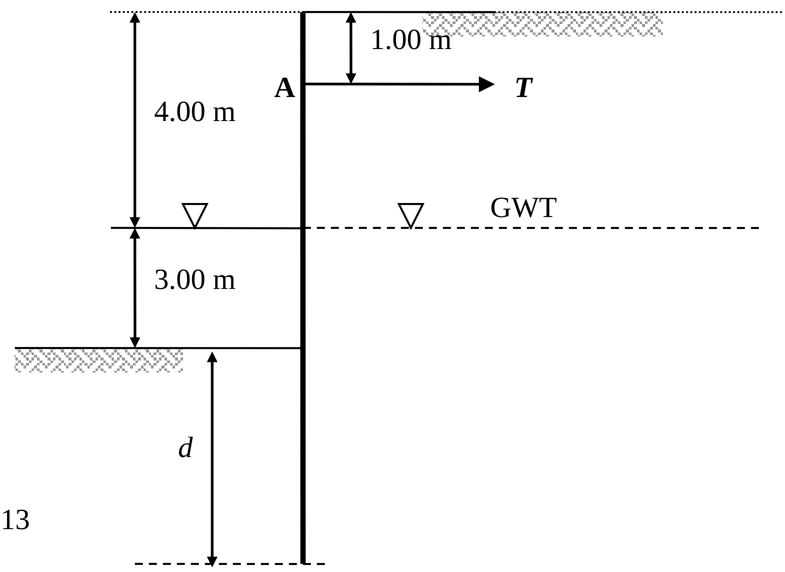


Figure Q 3.13