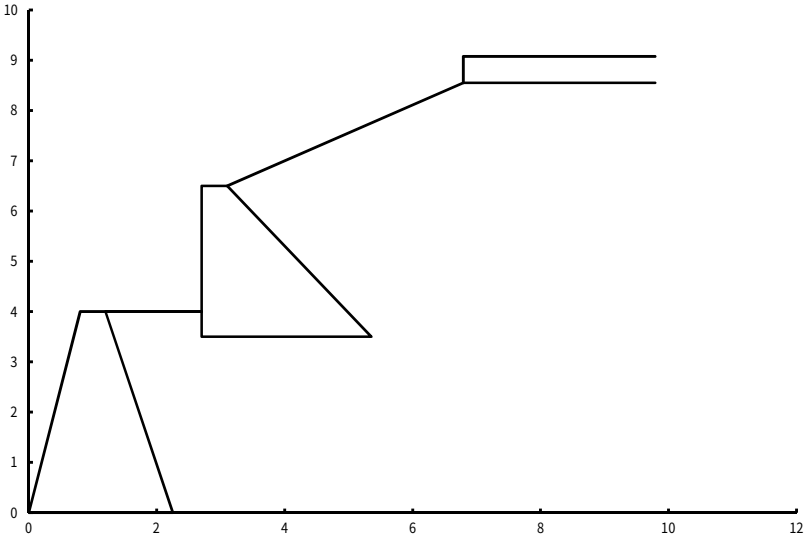
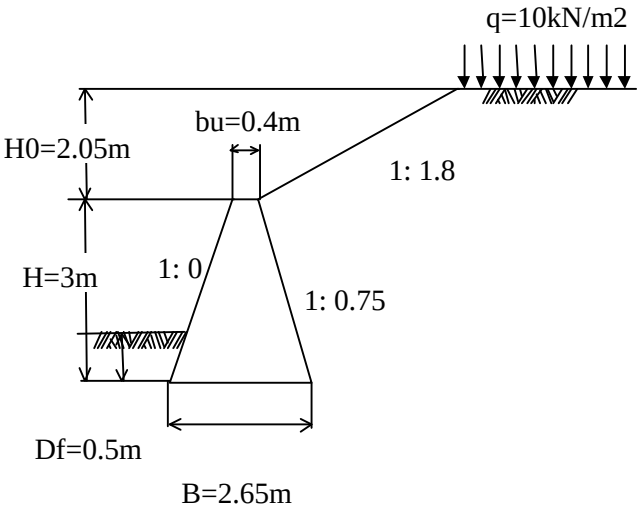
1. 擁壁形状

荷重条件		常時	
上段擁壁	H=	3.00	m
下段擁壁	H=	4.00	m
擁壁間隔	λ =	0.45	m



2. 上段擁壁



(1) 擁壁形状

擁壁高さ	H=	3.00	m
天端幅	b_u=	0.40	m
底面幅	B=	2.65	m
前面勾配	1:n_f=1:	0.00	
背面勾配	1:n_r=1:	0.75	

(2) 盛　　土

嵩上げ高	H_0=	2.05	m
盛土勾配	1:m =1:	1.80	
盛土傾斜角	β =	29.05	度
単位重量	γ =	19.00	kN/m^3
内部摩擦角	φ =	30.00	度
粘着力	c =	0.00	kN/m^2

(3) 地表面載荷重　　q = 10.00 kN/m^2

(4) 設計水平震度　　k_H = 0.00

(5) 支持地盤

摩擦係数	μ =	0.60	
極限支持力	q_d =	900.00	kN/m^2

(6) 根入地盤

根入深さ	D_f =	0.50	m

(7) コンクリート

単位体積重量	γ $_c$ =	23.00	kN/m^3

2．2　荷　重

⑴ 自　　重

重　量　$W_c = \frac{\gamma_c}{2}(b_u + B)H$　=　105.2　kN/m

重　心　$x_c = \frac{B}{2} + \frac{H}{6} \cdot \frac{2b_u + B}{b_u + B}(n_f - n_r)$　=　0.90　m

$y_c = \frac{H}{3} \cdot \frac{2b_u + B}{b_u + B}$　=　1.13　m

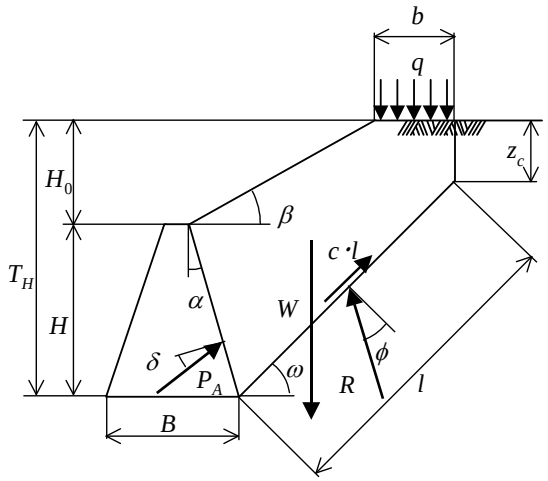
(2) 主働土圧

試行くさび法による.

$$P_A = \frac{W \sec\theta \sin(\omega - \phi + \theta) - cl\cos\phi}{\cos(\omega - \phi - \alpha - \delta)}$$

$$\theta = \tan^{-1} k_H$$

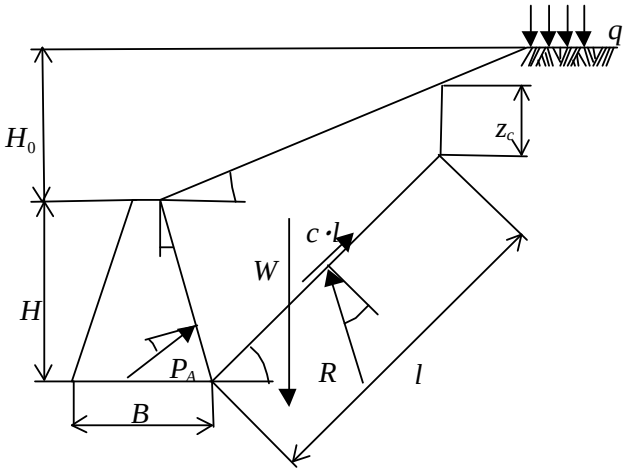
$$\omega \le \tan^{-1} \frac{T_H - z_c}{H_0 \cot\beta - H\tan\alpha}$$



$$W = \frac{\gamma}{2\cos\alpha}\left\{\frac{\cos(\omega-\alpha)}{\sin\omega}{T_H}^2 - \frac{\cos(\alpha-\beta)}{\sin\beta}{H_0}^2 - \frac{\cos\alpha}{\tan\omega}{z_c}^2\right\} + \left(\frac{T_H - z_c}{\tan\omega} + H\tan\alpha - \frac{H_0}{\tan\beta}\right)q$$

$$l = \frac{T_H - z_c}{\sin\omega}$$

$$\omega > \tan^{-1}\frac{T_H - z_c}{H_0\cot\beta - H\tan\alpha}$$



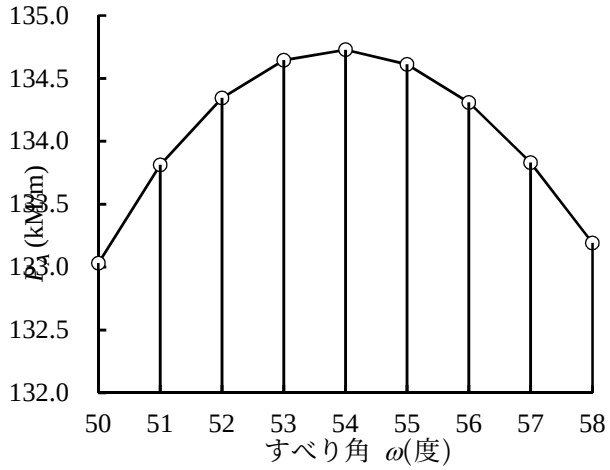
$$W = \frac{\gamma}{2\sin(\omega-\beta)}\left\{\frac{\cos(\omega-\alpha)\cos(\alpha-\beta)}{\cos^2\alpha}H^2 - \cos\omega\cos\beta z_c{}^2\right\}$$

$$l = \frac{1}{\sin(\omega-\beta)}\left\{\frac{\cos(\alpha-\beta)}{\cos\alpha}H - \cos\beta z_c\right\}$$

H =	3.00	m	H_0 =	2.05	m
β =	29.05	度	α =	36.87	度
γ =	19	kN/m^3	ϕ =	30	度
c =	0.0	kN/m^2	δ =	20.00	度
q =	10	kN/m^2	θ =	0.000	度

$$z_c = \frac{2c}{\gamma}\tan\left(\frac{\pi}{4}+\frac{\phi}{2}\right) \quad = \quad 0.00 \text{ m}$$

ω(度)	b(m)	l(m)	W(kN/m)	P_A(kN/m)
50	2.797	6.592	311.166	133.031
51	2.649	6.498	302.583	133.814
52	2.505	6.409	294.240	134.346
53	2.365	6.323	286.121	134.645
54	2.229	6.242	278.212	134.728
55	2.096	6.165	270.502	134.612
56	1.966	6.091	262.978	134.309
57	1.840	6.021	255.629	133.832
58	1.716	5.955	248.445	133.191



主働すべり角		ω_A =	54	度
主働土圧	合　　力	P_A =	134.7	kN/m
	鉛直成分	P_{AV} =	112.8	kN/m
	水平成分	P_{AH} =	73.6	kN/m
土圧係数		K_A =	1.576	(=2×P_A/(γ×H^2))
土圧合力の作用位置		y_A =	1.00	m
		x_A =	1.9	m

(3) 荷重の集計

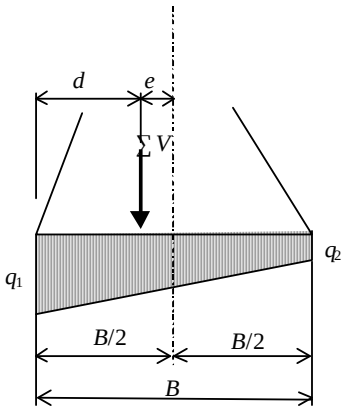
荷　重	V(kN/m)	H(kN/m)	x(m)	y(m)	$V\cdot x$	$H\cdot y$
自　重	105.2	0.0	0.90	***	94.7	0.0
土　圧	112.8	73.6	1.90	1.00	214.3	73.6
Σ	218.0	73.6	―	―	309.0	73.6

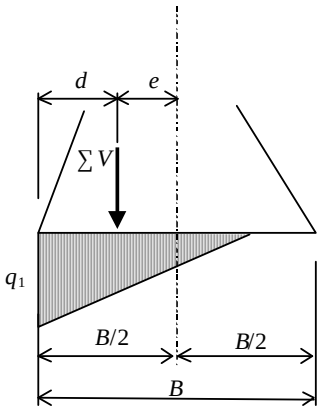
合力作用位置　$d=\dfrac{\Sigma Vx-\Sigma Hy}{\Sigma V}=$ 1.08 m

合力の偏心量　$e=\dfrac{B}{2}-d=$ 0.25 m

地盤反力度　$q_1=$ 128.8 kN/m^2　　$q_2=$ 35.7 kN/m^2

$q_{max}=$ 128.8 kN/m^2



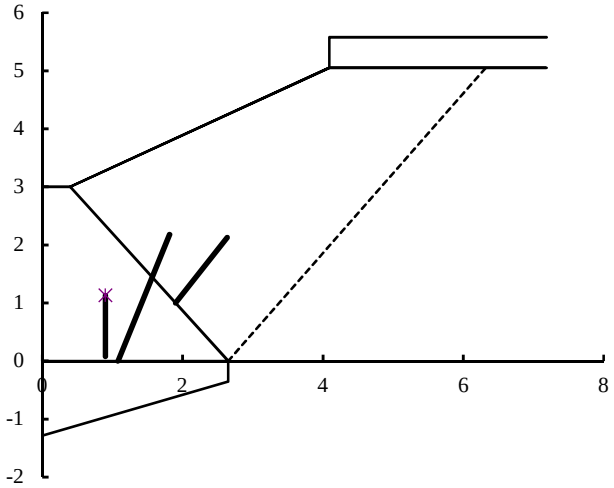


$e\le\dfrac{B}{6}$

$\left.\begin{matrix}q_1\\q_2\end{matrix}\right\}=\dfrac{\Sigma V}{B}\left(1\pm\dfrac{6e}{B}\right)$

$e>\dfrac{B}{6}$

$q_1=\dfrac{2\Sigma V}{3d},\ q_2=0$



2. 3　安定計算

(1) 転倒に対する照査

底版幅　B= 2.65 m　　偏心量　e= 0.25 m

安定率　$F_t = \dfrac{B}{2e}$ = 5.30 > 3.00 SAFE

(2) 滑動に対する照査

鉛直力　$\sum V$= 218.00 kN/m

水平力　$\sum H$= 73.60 kN/m

摩擦係数　μ = 0.6

受働土圧

D_f = 0.50 m　　γ_1 = 19.00 kN/m^3

ϕ_1 = 30.00 度　　c_1 = 0.00 kN/m^2

$$K_P = \tan^2\left(\frac{\pi}{4} + \frac{\phi}{2}\right) = 3.00$$

$$P_P = \frac{1}{2}\gamma_1 {D_f}^2 K_P + 2c_1 D_f \sqrt{K_P} = 7.125 \text{ kN/m}$$

安全率　$F_s = \dfrac{\Sigma V\mu + 0.5P_P}{\Sigma H}$ = 1.83 > 1.50 SAFE

(3) 支持力に対する照査

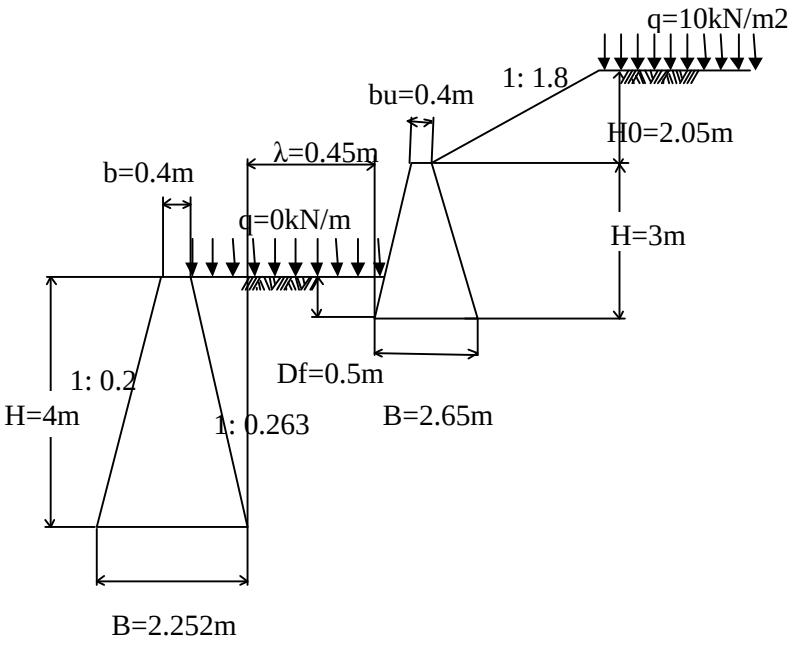
極限支持力度　q_d = 900.0 kN/m^2

最大地盤反力度　q_{max} = 128.8 kN/m^2

安全率　$F_s = \dfrac{q_d}{q_{max}}$ = 6.99 > 3.00 SAFE

3． 下段擁壁

3．1 設計条件



(1) 擁壁形状

擁壁高さ	H =	4.00	m
天端幅	b =	0.40	m
底面幅	B =	2.25	m
前面勾配	$1:n_f$ =	1: 0.2	
背面勾配	$1:n_r$ =	1: 0.263	

(2) 盛　土

嵩上げ高	H_0 =	0.00	m
盛土傾斜角	β =	0.00	度
単位重量	γ =	19.00	kN/m^3
内部摩擦角	φ =	30.00	度
粘着力	c =	0.00	kN/m^2

(3) 地表面載荷重　q = 0.00 kN/m^2

(4) 設計水平震度　k_H = 0.00

(5) 支持地盤

摩擦係数	μ =	0.60	
極限支持力	q_d =	900.00	kN/m^2

(6) 根入地盤

根入深さ	D_f =	0.50	m

(7) コンクリート

単位体積重量	γ$_c$ =	23.00	kN/m^3

3．2 荷　重

(1) 自　重

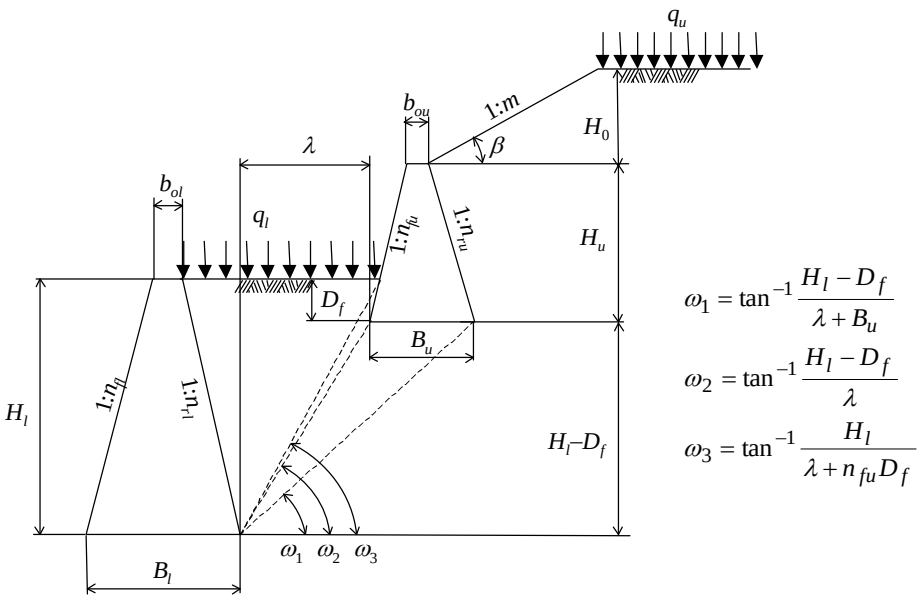
重　量　$W_c = \frac{\gamma_c}{2}(b_u + B)H$　=　122.0 kN/m

重　心　$x_c = \frac{B}{2} + \frac{H}{6} \cdot \frac{2b_u + B}{b_u + B}(n_f - n_r)$　=　1.08　m

$y_c = \frac{H}{3} \cdot \frac{2b_u + B}{b_u + B}$　=　1.53　m

(2) 主働土圧

上段擁壁の影響を考慮した試行くさび法による．

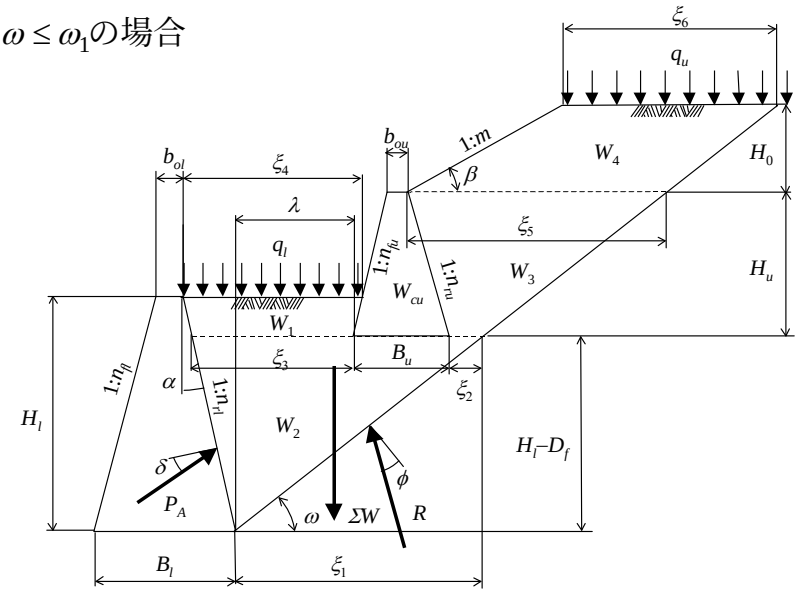


$$\omega_1 = \tan^{-1}\frac{H_l - D_f}{\lambda + B_u}$$

$$\omega_2 = \tan^{-1}\frac{H_l - D_f}{\lambda}$$

$$\omega_3 = \tan^{-1}\frac{H_l}{\lambda + n_{fu}D_f}$$

$\omega \le \omega_1$の場合



$$W = W_1 + W_2 + W_3 + W_4 + W_{cu}$$

$$W_1 = \frac{\gamma}{2}(\xi_3 + \xi_4)D_f + \xi_4 q_l$$

$$W_2 = \frac{\gamma}{2}(H_l - D_f)^2(\tan\alpha + \cot\omega)$$

$$W_3 = \frac{\gamma}{2}(\xi_2 + \xi_5)H_u$$

$$W_4 = \frac{\gamma}{2}(\xi_5 + \xi_6)X + \xi_6 q_u \left[\because X = \min(H_0, \eta) \rightarrow\right]$$

$$\xi_1 = \frac{H_l - D_f}{\tan\omega}$$

$$\xi_2 = \xi_1 - \lambda - B_u$$

$$\xi_3 = \lambda + (H_l - D_f)n_{rl}$$

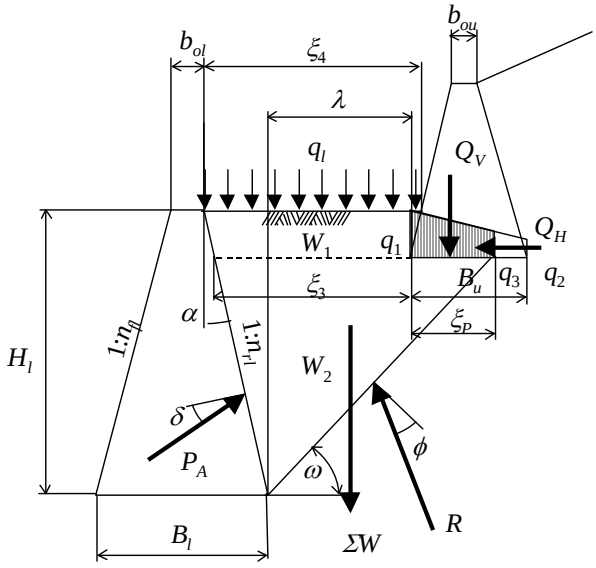
$$\xi_4 = \xi_3 + (n_{rl} + n_{fu})D_f$$

$$\xi_5 = \xi_2 + \left(n_{ru} + \frac{1}{\tan\omega}\right)H_u$$

$$\xi_6 = \xi_5 + \left(\frac{1}{\tan\omega} - m\right)H_o \left[\because \eta < H_o \rightarrow \xi_6 = 0\right]$$

$$\eta = \frac{\sin\beta\sin\omega}{\sin(\omega - \beta)}\xi_5$$

$\omega_1 < \omega < \omega_2$の場合



$$W = W_1 + W_2 + Q_V$$

$W_1, W_2 : \omega \le \omega_1$の場合と同じ

$3d \ge \xi_P$の場合

$$q_3 = q_1 - \frac{q_1 - q_2}{B_u}\xi_P \left[\because 3d \ge B_u\right]$$

$$q_3 = \frac{q_1}{3d}(3d - \xi_P) \left[\because \xi_P \le 3d < B_u\right]$$

$$Q_V = \frac{1}{2}(q_1 + q_3)\xi_P$$

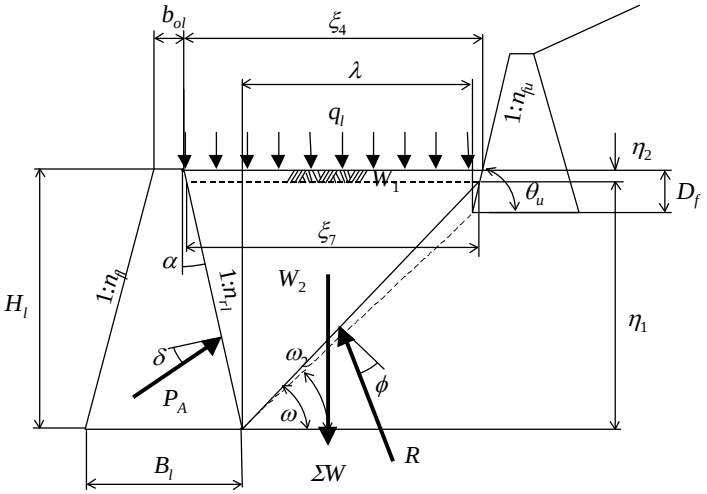
$$Q_H = \frac{\Sigma H}{B_u}\xi_P$$

$3d \le \xi_P$の場合

$$Q_V = \Sigma V, Q_H = \Sigma H$$

$$\xi_P = (H_l - D_f)\cot\omega - \lambda$$

$\omega_2 \le \omega < \omega_3$の場合



$$W = W1 + W2$$

$$W_1 = \frac{\gamma}{2}(\xi_4 + \xi_7)\eta_2 + \xi_4 q_l$$

$$W_2 = \frac{\gamma}{2}\eta_1 \xi_7$$

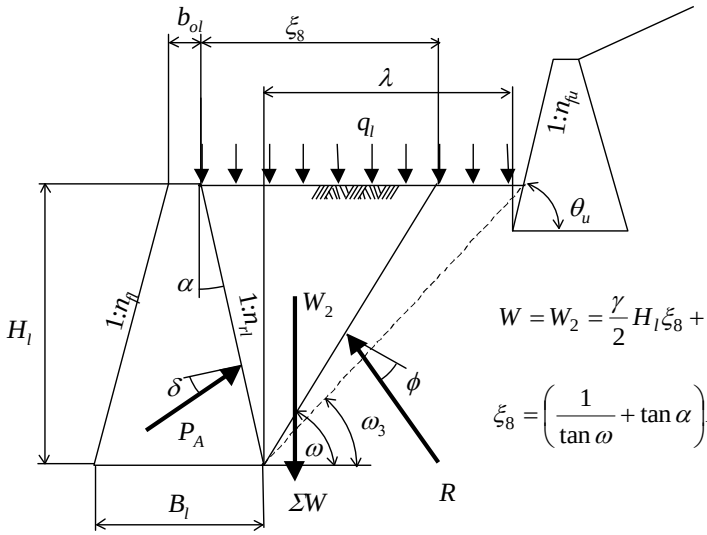
$$\theta_u = \frac{\pi}{2} - \tan^{-1} n_{fu}$$

$$\eta_1 = \frac{\sin(\theta_u - \omega_2)\sin\omega}{\cos\omega_2 \sin(\theta_u - \omega)}\lambda$$

$$\eta_2 = H_l - \eta_1$$

$$\xi_7 = \xi_4 - (n_{fu} + n_{rl})\eta_2$$

$\omega \ge \omega_3$の場合



$$W = W_2 = \frac{\gamma}{2} H_l \xi_8 + \xi_8 q_l$$

$$\xi_8 = \left(\frac{1}{\tan\omega} + \tan\alpha\right) H_l$$

主働土圧

$$P_A = \frac{\sin(\omega - \phi + \theta_0)}{\cos(\omega - \phi - \delta - \alpha)\cos\theta_0}(W + Q_V)$$

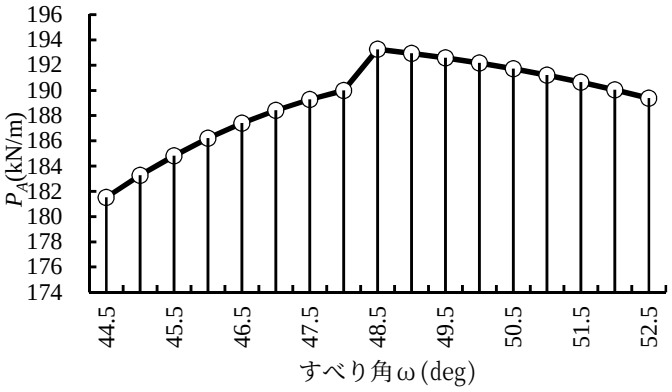
$$\theta_0 = \tan^{-1}\frac{Q_H}{W + Q_V}$$

計算条件

H_u =	3.00	m	H_l =	4.000	m
B_u =	2.65	m	B_l =	2.252	m
n_{fu} =	0.00		n_{fl} =	0.200	
n_{ru} =	0.75		n_{rl} =	0.263	
H_0 =	2.05	m	q_l =	0.00	kN/m^2
m =	1.80		λ =	0.450	m
D_f =	0.50	m	γ =	19.0	kN/m^3
q_u =	10.00	kN/m^2	ϕ =	30.0	度
W_{cu} =	105.20	kN/m			

計算結果

ω (度)	W (kN/m)	Q_V (kN/m)	Q_H (kN/m)	P_A (kN/m)
44.5	680.213	0.000	0.000	181.515
45	666.480	0.000	0.000	183.262
45.5	652.984	0.000	0.000	184.820
46	639.718	0.000	0.000	186.196
46.5	626.673	0.000	0.000	187.396
47	613.843	0.000	0.000	188.425
47.5	601.220	0.000	0.000	189.290
48	588.797	0.000	0.000	189.995
48.5	365.087	217.876	73.504	193.256
49	361.304	215.889	72.003	192.939
49.5	357.477	213.832	70.525	192.576
50	353.609	211.708	69.069	192.165
50.5	349.703	209.520	67.634	191.705
51	345.761	207.272	66.219	191.198
51.5	341.785	204.966	64.824	190.642
52	337.778	202.604	63.449	190.037
52.5	333.740	200.191	62.092	189.384



主働すべり角		ω_A =	48.5 度
主働土圧	合　　力	P_A =	193.3 kN/m
	鉛直成分	P_{AV} =	110.1 kN/m
	水平成分	P_{AH} =	158.8 kN/m
土圧係数		K_A =	1.271 (=$2P_A/(\gamma H^2)$)
土圧合力の作用位置		y_A =	1.33 m
		x_A =	1.9 m

(3) 荷重の集計

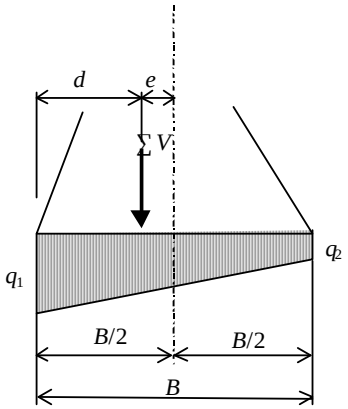
荷　重	V(kN/m)	H(kN/m)	x(m)	y(m)	$V \cdot x$	$H \cdot y$
自　重	122.0	0.0	1.08	1.53	131.8	0.0
土　圧	110.1	158.8	1.90	1.33	209.2	211.2
Σ	232.1	158.8	—	—	341.0	211.2

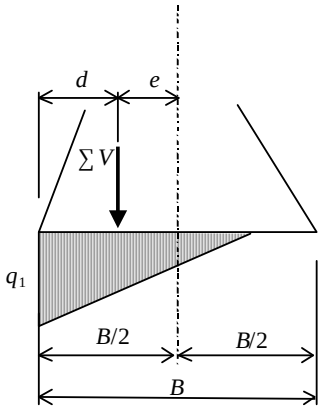
合力作用位置　$d = \dfrac{\Sigma Vx - \Sigma Hy}{\Sigma V}$ = 0.56 m

合力の偏心量　$e = \dfrac{B}{2} - d$ = 0.57 m

地盤反力度　q_1= 276.3 kN/m^2　　q_2= 0.0 kN/m^2

q_{max}= 276.3 kN/m^2





$e \le \dfrac{B}{6}$

$\left.\begin{matrix} q_1 \\ q_2 \end{matrix}\right\} = \dfrac{\Sigma V}{B}\left(1 \pm \dfrac{6e}{B}\right)$

$e > \dfrac{B}{6}$

$q_1 = \dfrac{2\Sigma V}{3d},\ q_2 = 0$

3．3 安定計算

(1) 転倒に対する照査

底版幅　B= 2.25 m　　偏心量　e= 0.57 m

安定率　$F_t = \dfrac{B}{2e}$ = 1.98 < 3.00 OUT

(2) 滑動に対する照査

鉛直力	$\sum V$=	232.10	kN/m
水平力	$\sum H$=	158.80	kN/m
摩擦係数	μ =	0.6	

安全率	$Fs = \frac{\Sigma V}{\Sigma H}\mu =$	0.88	<	1.50	OUT

(3) 支持力に対する照査

極限支持力度	q_d =	900.0	kN/m^2
最大地盤反力度	q_{max}=	276.3	kN/m^2

安全率	$F_s = \frac{q_d}{q_{max}} =$	3.26	>	3.00	SAFE